SOIL MIX WALLS as retaining structures – mechanical characterization

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ABSTRACT

Since several decennia, the deep soil mix (DSM) technique has been used for ground improvement (GI) applications. In recent years, soil mix walls (SMW) have become an economical alternative to traditional excavation support systems. The Belgian building market has also witnessed such development with the growing use of the Cutter Soil Mix (CSM), the Tubular Soil Mix (TSM) and the CVR C-mix® systems.
Unfortunately, standardized guidelines for SMW are not currently available. For the purpose of developing such guidelines, mechanical characteristics of DSM material must be investigated. Within the framework of a Flemish regional research program (IWT 080736), DSM material from 38 Belgian construction sites, with various soil conditions and for different execution processes, has been tested.
In the present paper, results of various tests, performed to determine characteristics of DSM material are firstly described. Porosity, permeability, Unconfined Compressive Strength (UCS) and tensile strength, as well as the modulus of elasticity, the ultrasonic pulse velocity and the adherence between DSM material and steel reinforcement are investigated. In addition, results of petrographic analysis performed using thin section technology are also presented in order to obtain a microscopic view of the material.
On the basis of UCS tests performed on core samples, the determination of the 5% quantile characteristic UCS value of the DSM material is then discussed with regard to the influence of unmixed soil inclusions in the material and considering the scale effect.

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1. INTRODUCTION

Since several decennia, the deep soil mix method (DMM) has been used for GI applications. In recent years, SMW have increasingly been used – in Belgium and in several other countries – for the retaining of soil and water in the case of excavations. Indeed, SMW represent a more economical alternative to concrete secant pile walls and even in several cases to king post walls.
In the DSM process, the ground is in situ mechanically mixed, while a binder, based on cement, is injected. For SMW applications, the DSM cylindrical columns or the rectangular panels are placed next to each other, in a secant way. By overlapping the different soil mix elements, a continuous SMW is realized. Steel profiles are inserted into the DSM fresh material to resist the shear forces and bending moments. The main structural difference between SMW and the more traditional secant pile walls is the constitutive DSM material which consists of a soil – cement mixture instead of concrete.
Unfortunately, up to now, guidance rules and recommendations concerning the realization of SMW with a soil and/or water retaining function are lacking while various DSM systems are active on the Belgian market such as the CVR C-mix®, the Tubular Soil Mix (TSM) and the Cutter Soil Mixing (CSM). Moreover, the number of applications is fast increasing. For QA/QC development and in the context of the European standardization, basic rules are required with regard to design, execution and control of these different DSM execution processes.
These issues have encouraged the Belgian Building Research Institute (BBRI) to initiate research actions that address the execution, design and testing of DSM systems in Belgium. For the purpose of investigating the DSM technology and its applicability in the various Belgian soils, the 'Soil Mix' project was initiated in 2009 in collaboration with the KU Leuven and the Belgian Association of Foundation Contractors (ABEF). Financial support has been obtained from IWT, the Flemish government agency for Innovation by Science and Technology (http://www.iwt.be/).
If DSM technique is currently used for retaining structures with a temporary character, permanent and bearing applications are fast increasing in spite of the lack of knowledge concerning the strength, the stiffness, the permeability, the durability of DSM material and its adherence with steel. Hence, within the framework of the BBRI 'Soil Mix' project, numerous tests on in situ DSM material have been performed.

A good insight has been acquired with regard to mechanical characteristics that can be obtained with the CVR C-mix®, the TSM and the CSM systems in several Belgian soils. This article gives an overview of the test results. The CVR C-mix®, the TSM and the CSM systems are presented in Denies et al. (2012a).

2. MECHANICAL CHARACTERIZATION OF DSM MATERIAL

During the first part of the experimental campaign, cores of DSM material have been drilled at 38 Belgian construction sites, with different soil conditions and for various DSM systems. The cylindrical core samples have a diameter ranging between 85 mm and 115 mm. The measurement accuracy of the core diameter is 0.3 mm. Before testing, they are preserved in an acclimatized chamber with a relative humidity larger than 95% and a temperature equal to 20 ± 2°C.
The following paragraphs illustrate the mechanical characterization of the DSM material with the help of UCS and tensile splitting strength tests, as well as tests to determine its modulus of elasticity and its ultrasonic pulse velocity.

2.1. Unconfined Compressive Strength (UCS)

2.1.1. UCS tests on core samples

UCS tests are performed by a MFL 250 kN loading machine according to NBN EN 12390-3. The loading rate amounts to 2.5 kN/s. The height to diameter ratio is 1. This choice was based on the necessity to collect a maximum of cores and was made in order to compare the UCS test results on cylindrical cores with cube strength (NBN EN 12504-1). It can be noted that the height to diameter ratio will have an influence on the failure pattern and on the UCS test results. In such a way to investigate this question, a laboratory study is currently performed in collaboration with KU Leuven.
Figures 1 to 6 give the histograms of the UCS test results in function of the soil type and with regard to the execution technique. The age of the samples ranges between 7 and 200 days. Currently, no correction regarding to the influence of the sample age on the UCS results has been applied. Based on on-going laboratory experiments, this parameter will be introduced in the near future in order to generate more precise information, notably on the average UCS value and on the standard deviations for the different DSM systems in typical Belgian soils. Table 1 gives the minimal and maximal UCS values in function of the soil type and with regard to the execution technique.
On the basis of the previous results, several tendencies can be drawn:
- the UCS values in sandy soils are larger than UCS values in silty and clayey soils;
- the UCS values of CSM systems are generally smaller than UCS values of DSM columns;
- the variability of the UCS results is smaller for the CSM systems than for DSM columns.

The density of samples, measured according to NBN EN 12390-7, varies between 1372 and 2176 kg/m³. No specific correlation was observed between the density and the UCS. The depth of coring is always larger than 1 m. Indeed, Ganne et al. (2010) have observed on different sites that the strength of the DSM material over the first meter is strongly influenced by the execution process (e.g. infiltration of rinsing water), as illustrated in Fig. 7 for a CSM panel in quaternary sand. Hence, the top of the SMW is not representative for the deeper part with regard to its strength.

2.1.2. UCS tests on wet grab samples

At two construction sites (for CSM technique in sandy soils), wet grab sampling was conducted in the first half hour after execution. For wet grab sampling a cylindrical sampler is pushed in the fresh DSM material. It stays closed until the sampling depth is reached (about 2 m in the present case). At this moment, the sampler opens with a 0.2 m gap. After filling, it is locked and pulled up. The DSM material is preserved in a cylindrical mould – 113 mm diameter and 220 mm height – in the acclimatized chamber. Two weeks later, DSM material is in situ cored at the same location and similar depth. The cores and the wet grab samples are tested on the same day (age = 14 days). For all the UCS tests, the height to diameter ratio is 1. Table 2 illustrates the UCS test results.

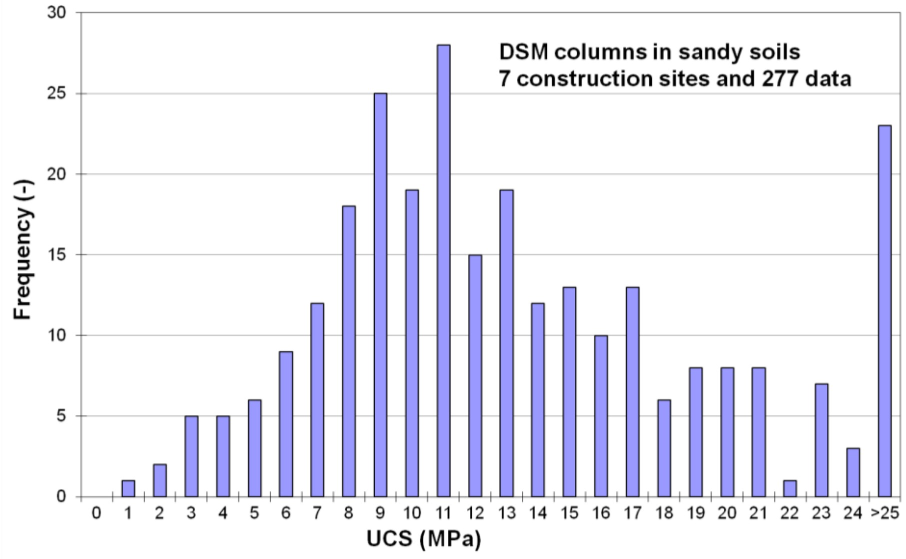

Figure 1: Histogram of the UCS test results for DSM column systems in sandy soils

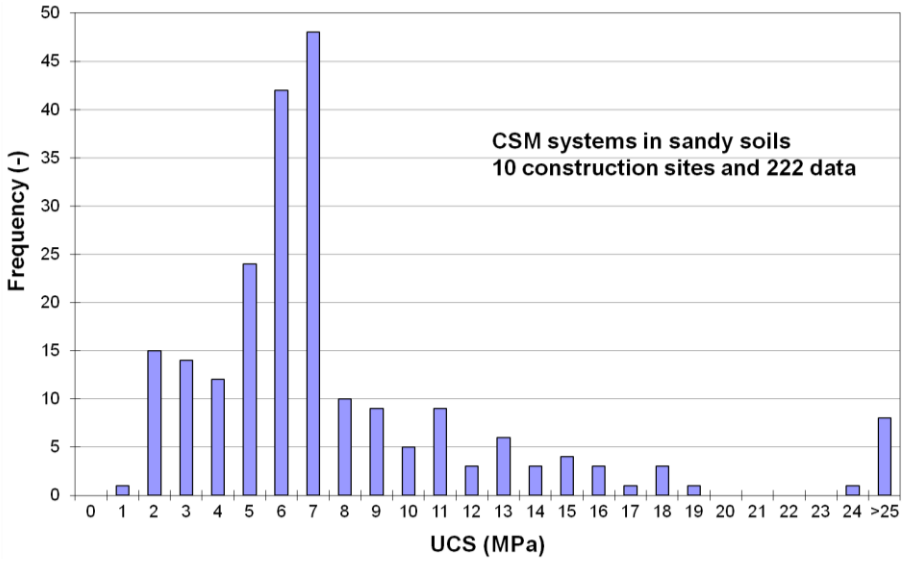

Figure 2: Histogram of the UCS test results for CSM systems in sandy soils

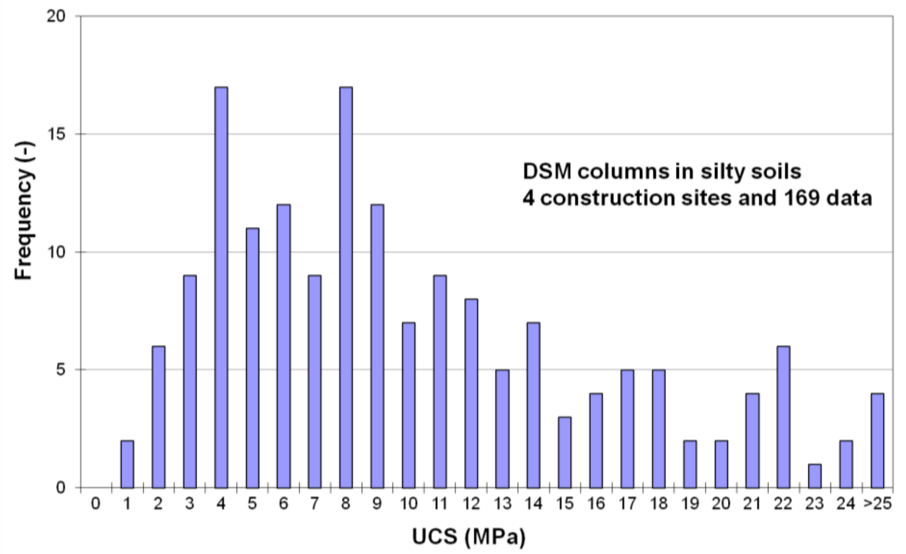

Figure 3: Histogram of the UCS test results for DSM column systems in silty soils

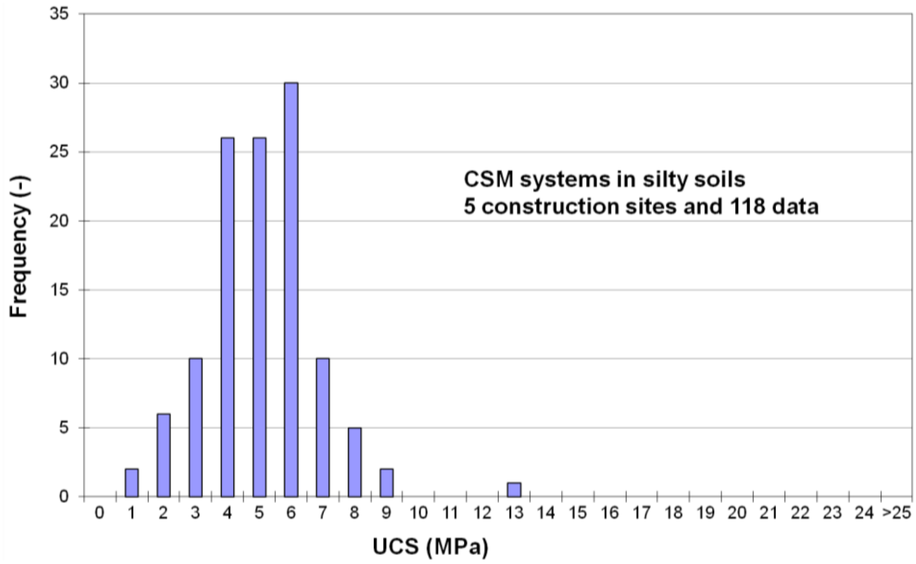

Figure 4: Histogram of the UCS test results for CSM systems in silty soils

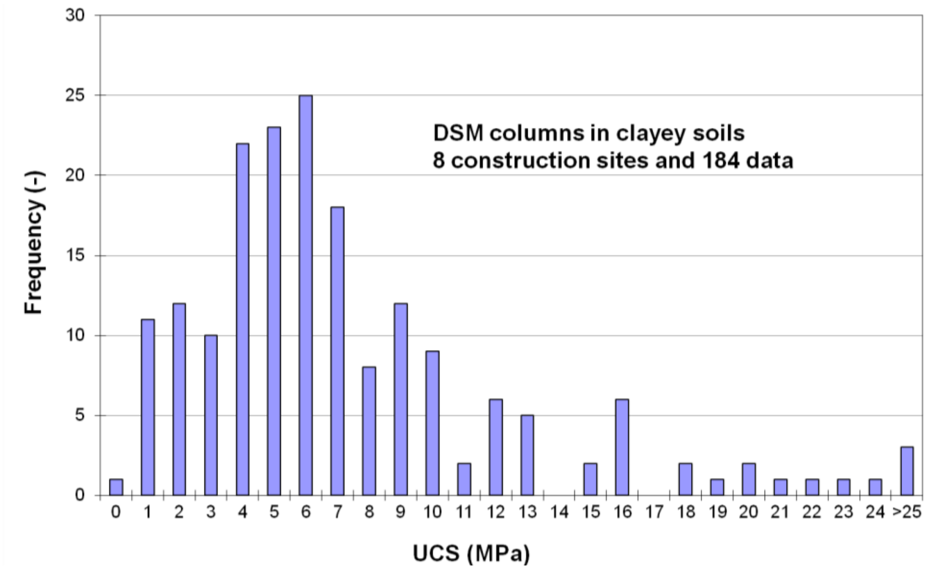

Figure 5: Histogram of the UCS test results for DSM column systems in clayey soils

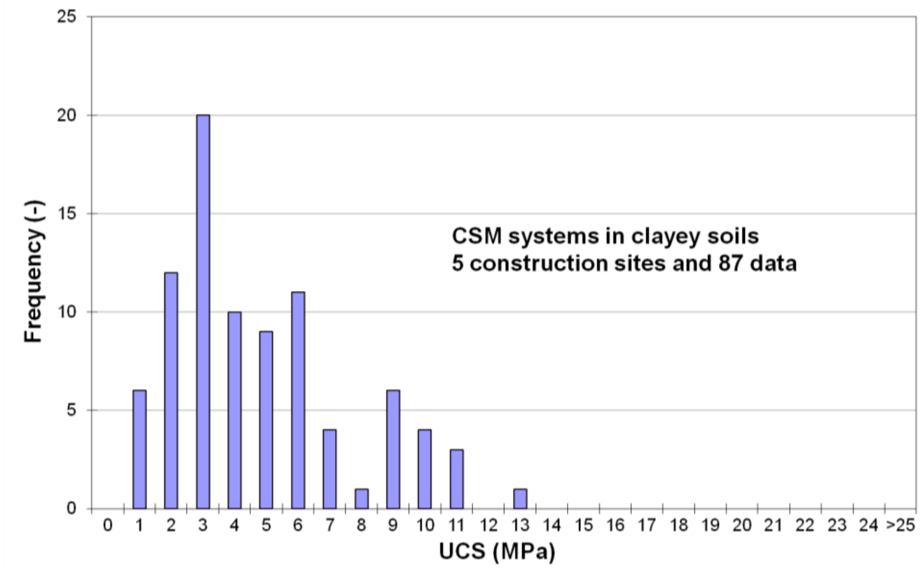

Figure 6: Histogram of the UCS test results for CSM systems in clayey soils

Table 1: Minimal and maximal UCS values in function of the soil type and with regard to the execution technique

	Sandy soils		**Silty soils**		**Clayey soils**	
	DSM columns	**CSM systems**	**DSM columns**	**CSM systems**	**DSM columns**	**CSM systems**
Minimal UCS values	1.32 MPa	1.28 MPa	0.93 MPa	0.66 MPa	0.44 MPa	0.65 MPa
Maximal UCS values	39.90 MPa	32.07 MPa	31.17 MPa	12.63 MPa	33.23 MPa	12.69 MPa

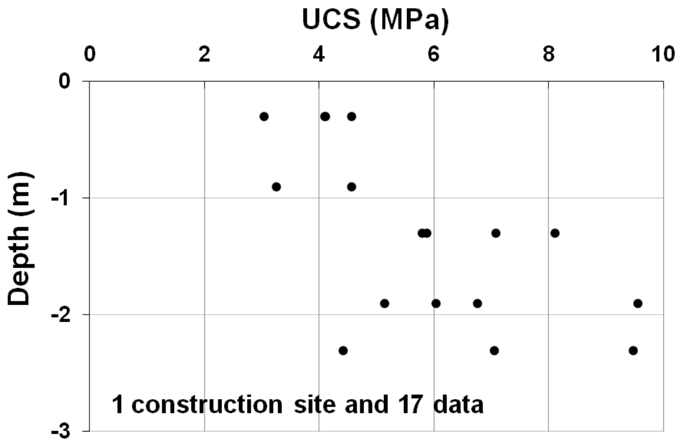

Figure 7: UCS test results of samples, cored at different depths (CSM panel in quaternary sand), after Ganne et al. (2010)

Table 2: UCS results of tests on core and wet grab samples (μ is the average UCS value), after Ganne et al. (2010)

	Core samples		**Wet grab samples**	
Site I – CSM element 1	UCS_1=2.33 UCS_3=2.27 UCS_5=2.85 **μ = 2.47 (MPa)**	UCS_2=2.04 UCS_4=2.85	UCS_1=2.94 UCS_3=2.44 **μ = 2.61 (MPa)**	UCS_2=2.46 UCS_4=2.59
Site I – CSM element 2	UCS_1=1.62 UCS_3=1.28 UCS_5=1.90 **μ = 1.66 (MPa)**	UCS_2=1.63 UCS_4=1.88	UCS_1=1.82 UCS_3=1.78 **μ = 1.85 (MPa)**	UCS_2=2.00 UCS_4=1.80
Site II – CSM element 3	UCS_1=2.95 UCS_3=4.64 **μ = 3.98 (MPa)**	UCS_2=4.53 UCS_4=3.79	UCS_1=3.80 UCS_3=3.66 **μ = 3.68 (MPa)**	UCS_2=3.40 UCS_4=3.89
Site II – CSM element 4	UCS_1=5.27 UCS_3=4.12 **μ = 4.99 (MPa)**	UCS_2=5.03 UCS_4=5.54	UCS_1=4.18 UCS_3=3.64 **μ = 3.64 (MPa)**	UCS_2=3.07 UCS_4=3.69

The differences between drilled cores and wet grab samples can be explained by the limited number of samples and the lack of uniformity of the samples on the one hand and by the different curing conditions on the other hand. In the following, only tests on core samples are discussed.

2.2. Modulus of elasticity (E)

The laboratory test to determine the modulus of elasticity (E) is performed in an unconfined way with the help of a MFL 250 kN loading machine according to NBN B 15-203.
The loading rate amounts once again to 2.5 kN/s. The height to diameter ratio is 2, according to NBN B 15-203. A selection is made of cores that are visually of a better quality in order to preserve the uniaxial behavior of the tested samples. E is determined in a tangent way varying the applied load between 10% ($\sigma_{10\%UCS}$) and 30% ($\sigma_{30\%UCS}$) of the estimated UCS. The sample deformations (ε) are measured along

three axes using DEMEC mechanical strain gauges. Once the difference between two cycles is smaller than 10 µstrains, E is calculated as the ratio:

$$E = \frac{\sigma_{30\%UCS} - \sigma_{10\%UCS}}{\varepsilon_{30\%UCS} - \varepsilon_{10\%UCS}} \quad (1)$$

The loading is then continued to determine the UCS.
Figure 8 presents the correlation between E and the density of the core samples without distinction of the soil type. Figure 9 illustrates the relationship between E and the determined UCS of the core samples regardless of the soil type. The age of the samples varies between 30 and 200 days. Since the aim is to determine the correlation between E and the UCS, the test results are not corrected for the age of the samples.

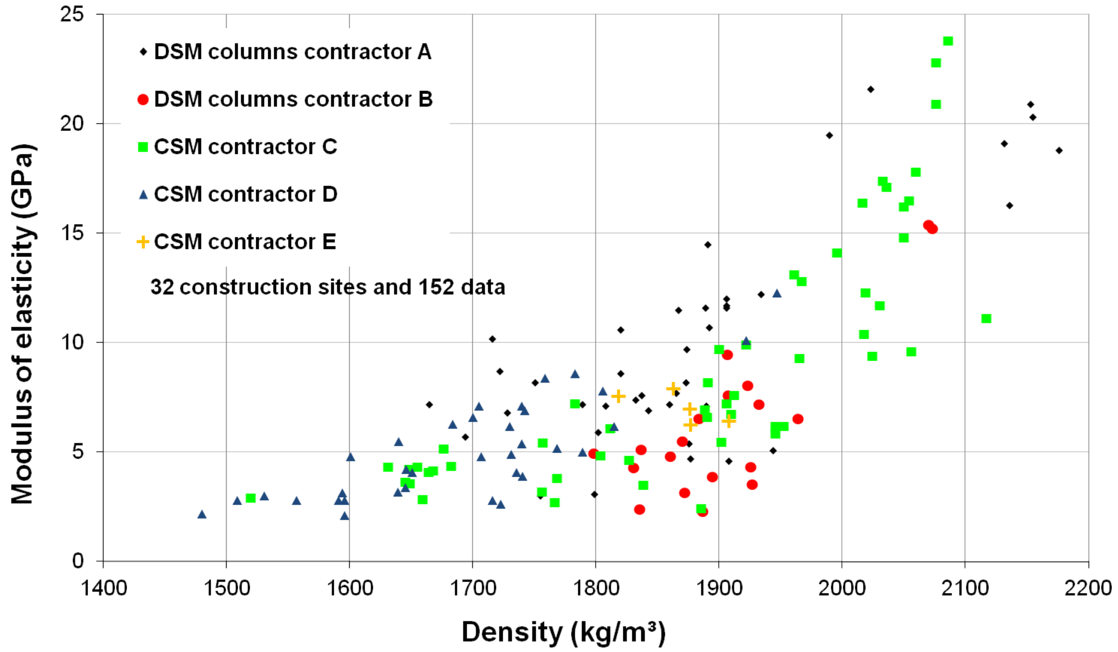

Figure 8: Relationship between the modulus of elasticity and the density of DSM material

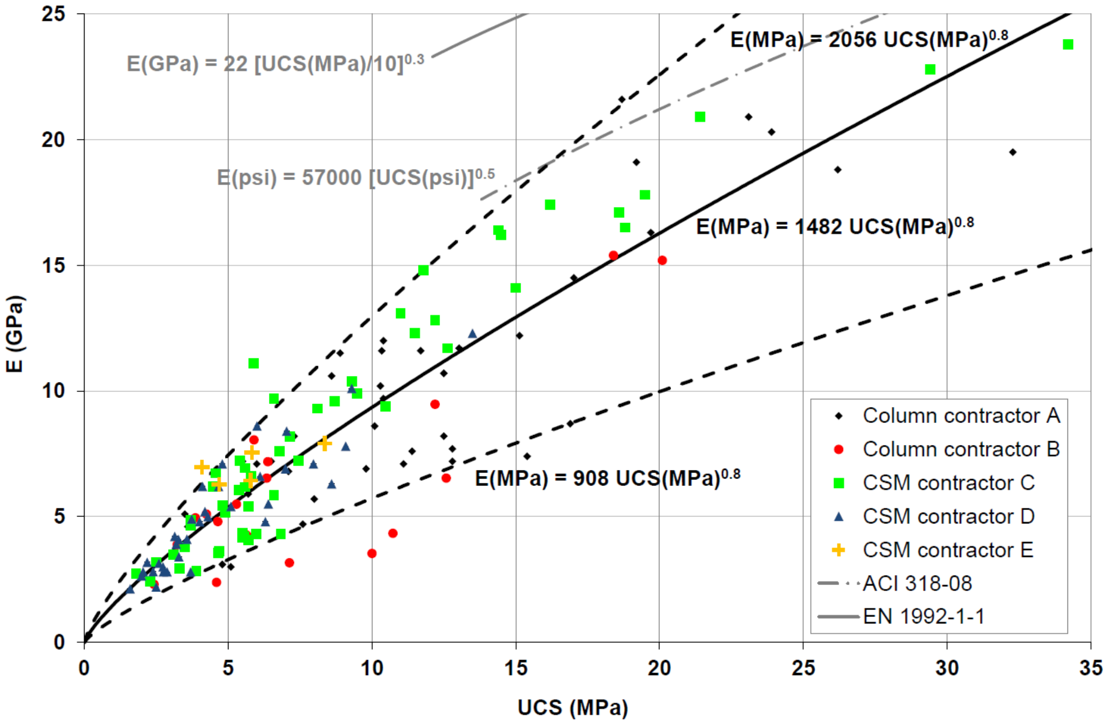

Figure 9: Relationship between the modulus of elasticity and the UCS of DSM material

The best fit corresponds to:

$$E = 1482\, UCS^{0.8} \quad (2)$$

with a coefficient of determination close to 0.80 (-). Lower and higher 5% quantile estimations of E respectively correspond with:

$$E = 908\, UCS^{0.8} \quad (3)$$

and

$$E = 2056\, UCS^{0.8} \quad (4)$$

These estimations are only valid for the range 1.5 MPa < UCS < 35 MPa.
In Fig. 9, relationships for normal concrete, from ACI 318-08 and EN 1992-1-1, are given for comparisons. According to ACI 318-08, the modulus of elasticity for normalweight concrete can be defined with regard to the UCS with the help of the following equation:

$$E\,(psi) = 57000\sqrt{UCS\,(psi)} \quad (5)$$

where E is defined as the secant modulus of elasticity between 0 and 45% ($\sigma_{40\%UCS}$) of the UCS. Based on previous research of Pauw (1960), equation (5) is valid for UCS values larger than 2000 psi (or 13.8MPa).
Eurocode 2 (EN 1992-1-1) provides the following relationship for concrete:

$$E(GPa) = 22\left[UCS(MPa)/10\right]^{0.3} \quad (6)$$

where E is the secant modulus of elasticity between 0 and 40% ($\sigma_{40\%UCS}$) of the UCS. Equation (6) is only valid for concrete samples containing quartzite aggregates and for a range of UCS varying between 12 and 90 MPa.

2.3. Tensile splitting strength (T)

For the determination of the tensile splitting strength (T), sometimes called Brazilian tensile strength, samples with H/D close to 1 have been tested with the help of a MFL 250 kN loading machine (2.5 kN/s of loading rate), according to NBN EN 12390-6. Figure 10 gives the relationship between T and the UCS, without distinction of the soil type. The samples were tested after a period varying between 32 and 200 days. Test results are not corrected for the age of the sample. In Fig. 10, experimental results for DSM cores are compared with well-established empirical relationships for concrete.

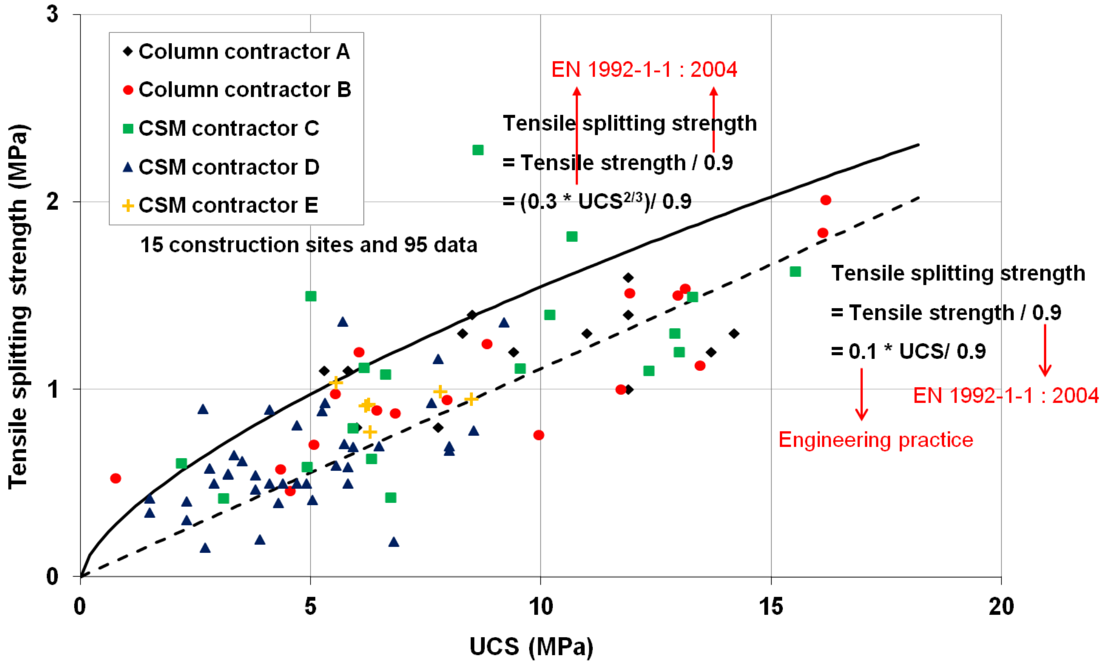

Figure 10: Relationship between the tensile splitting strength and the UCS of DSM material

According to Eurocode 2 (EN 1992-1-1), when the tensile strength is determined as the splitting tensile strength, an approximate value of the axial tensile strength, T_a, may be determined as:

$$T_a = 0.9T \tag{7}$$

Eurocode 2 also provides a correlation with the UCS:

$$T_a = 0.30UCS^{2/3} \tag{8}$$

which is only valid for concrete with UCS values less than the UCS of the C50/60 concrete type.
It is to note that in the engineering practice, the axial tensile strength of concrete is often related to the UCS by the following relationship:

$$T_a = 0.1UCS \tag{9}$$

2.4. Ultrasonic pulse velocity

The ultrasonic pulse velocity, V_p, of longitudinal stress waves in DSM cores was measured according to ASTM C597-09. Theoretically, V_p can be related to the elastic properties and density with the help of the following relationship:

$$V_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \tag{10}$$

where E is the dynamic modulus of elasticity, ν the Poisson's ratio and ρ the density. Nevertheless, as illustrated in Fig. 11a, most of the measured values of V_p are less than their theoretical values assuming ν = 0.35 (-). This could be due to the non-homogeneous and discontinuous character of the DSM material. Figure 11b presents the relationship between V_p and the UCS for DSM cores of 6 different construction sites, regardless of the soil conditions. For one particular site, the measurement of the ultrasonic pulse velocity, as well as the UCS test results, can be considered as an indicator of the homogeneity of the DSM material. Indeed, ultrasonic tests performed on laboratory mix homogeneous samples with the same age present limited variability (Hird and Chan, 2005).

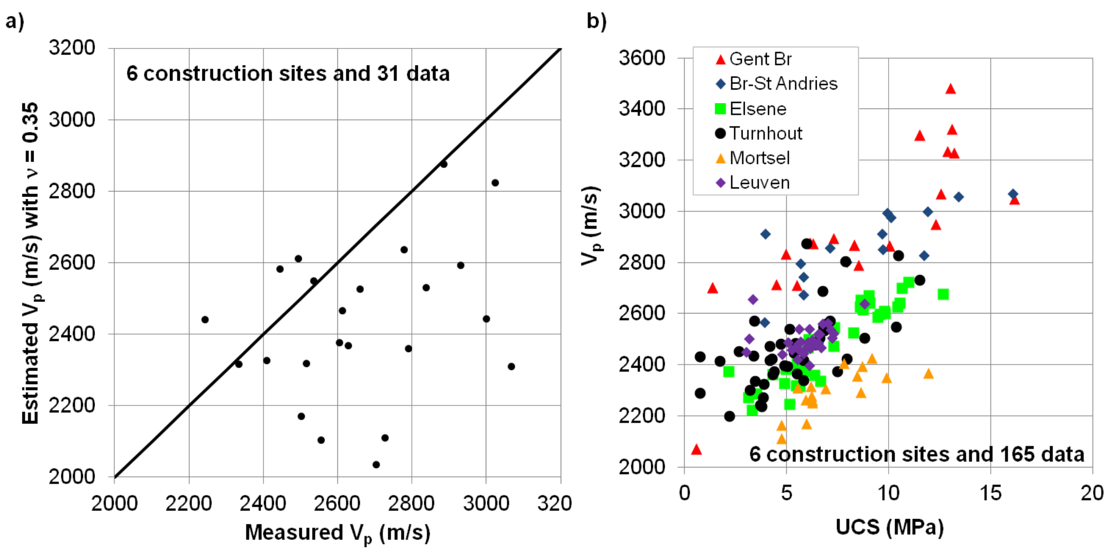

Figure 11: Correlation between the ultrasonic pulse velocity and a) the elastic properties and density of the DSM material; b) the UCS of DSM material.

3. POROSITY OF DSM MATERIAL AND PETROGRAPHIC ANALYSIS

In parallel to mechanical investigations, the porosity of DSM cores is determined according to NBN B15-215. As illustrated in Fig. 12, porosity varies between 25 and 65% for all soil types. In order to explain these high values, a petrographic analysis is conducted on samples from two construction sites (in silty and sandy soils) with the help of image processing techniques (IPT) and thin section technology.

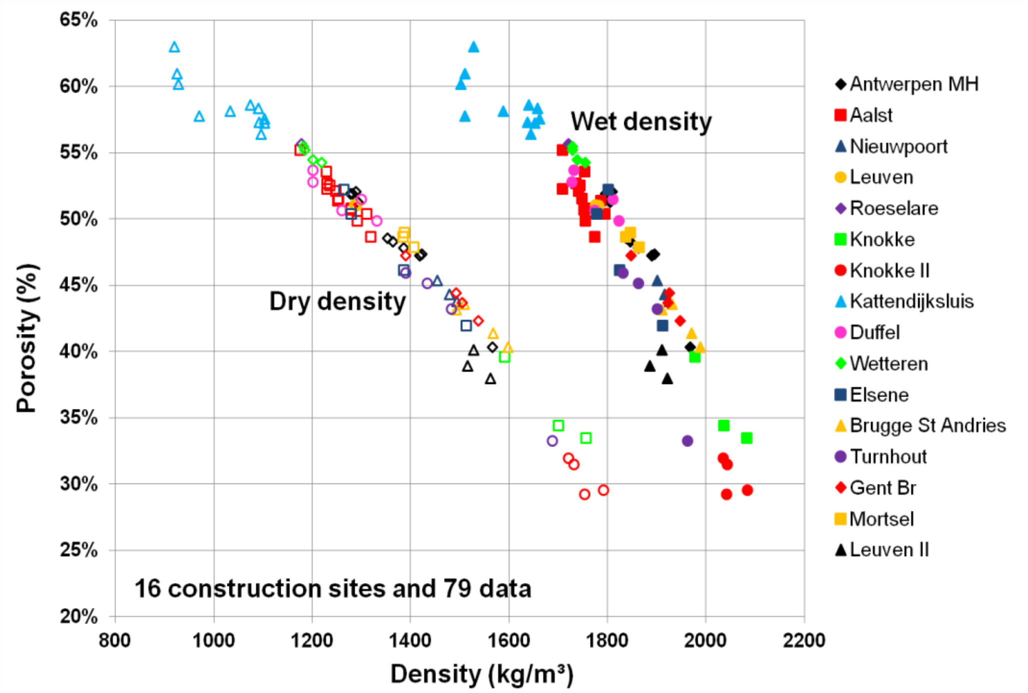

Figure 12: Relationship between dry and wet density and porosity for DSM cores

Figure 13 gives an example of microscopic analysis of a thin section cut from a DSM core, where P represents the pores, S the sand grains and C the cement stone. If open cracks, without specific orientation, are observed, they have a limited width varying between 10 and 200 µm.

The pores in the DSM sample are coloured by the resin used in the production of the thin section. All the pores with a surface area higher than 10 µm² are indicated as macropores. They represent around 2.4% of the total surface area. As a result, high porosity values, illustrated in Fig. 12, can only be related to the high and homogeneous capillary porosity. The high capillary porosity could result from the high water/cement ratio, W/C, used for the execution of the SMW. The high hydration level and the presence of portlandite $Ca(OH)_2$ in the DSM samples consolidate this assumption.

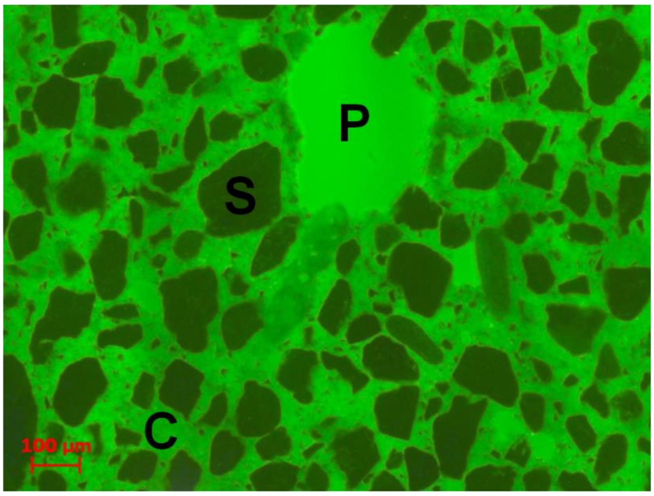

Figure 13: Microscopic analysis of DSM thin section with fluorescent light

4. HYDRAULIC CONDUCTIVITY OF DSM MATERIAL

Permeability tests are performed on DSM samples according to DIN 18130-1. As presented in Fig. 14, the coefficient of hydraulic conductivity of DSM material varies between 10^{-8} and 10^{-12} m/s, regardless of the soil conditions. No correlation is observed between porosity and permeability. Hence, assuming adequate positioning of the DSM columns/panels, the DSM material ensures the sealing of the SMW with water retaining function.

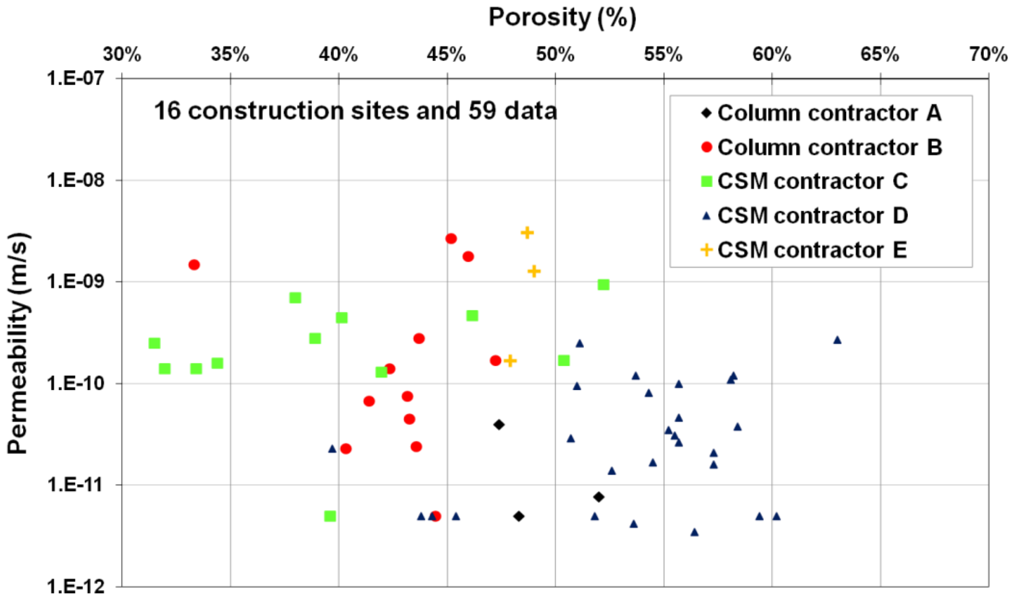

Figure 14: Relationship between permeability and porosity for DSM cores

5. 'STEEL – SOIL MIX' ADHERENCE

To investigate the adhesion between DSM material and various steel profiles, in situ pull-out tests were conducted on the basis of NBN EN 12504-3. Figure 15a presents the test setup. After the execution of the DSM element, steel reinforcement was suspended from the guidance device and vertically installed into the soil mix. As illustrated in Fig. 15b, the top part of the steel profile (over 1 m) is made frictionless using a flexible protection tube in order to eliminate the influence of the first non-representative meter on the results.
Figure 16 presents the peak extraction resistance in function of the UCS of DSM cores, for different types of steel reinforcements.

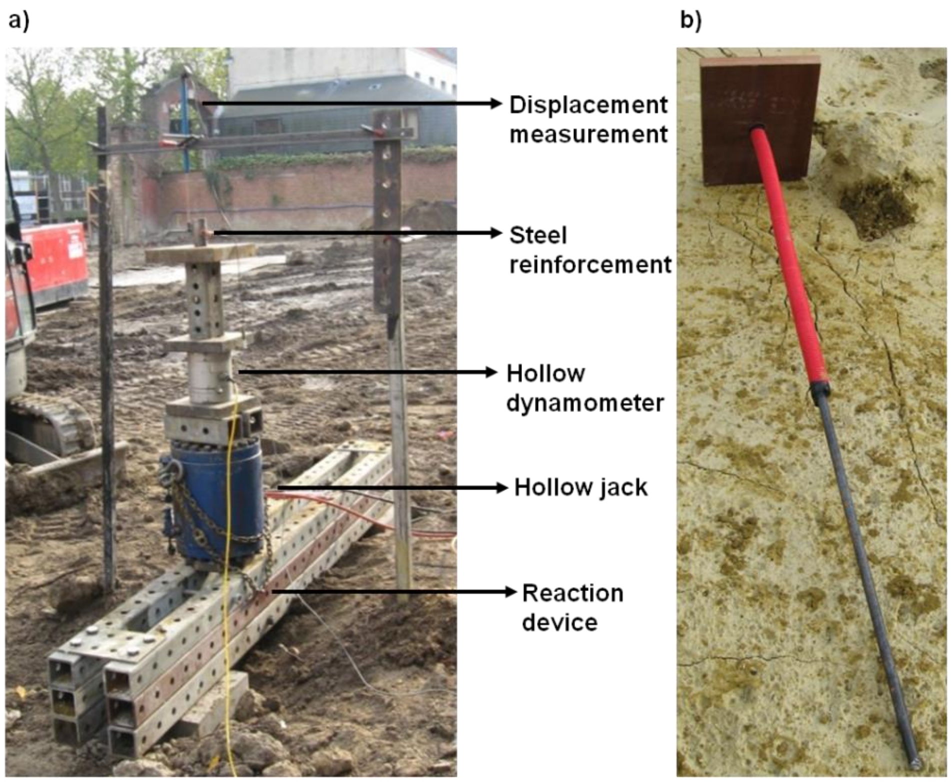

Figure 15: a) Pull-out test set-up and b) steel profile with protecting tube

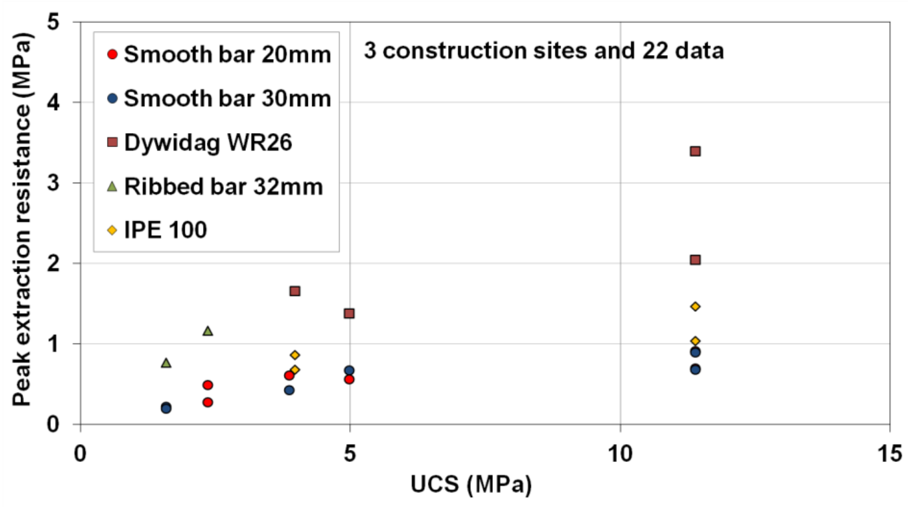

Figure 16: Peak pull-out resistance in function of the UCS of cored DSM material

6. UCS CHARACTERISTIC VALUE OF DSM MATERIAL

For engineering purposes and in particular with regard to the semi-probabilistic design approach in Eurocode 7, it is important to define a 'characteristic value' of the UCS that can be taken into account in the design of DSM structures. The definition of this value still remains a subject of discussion and further research is needed. In general terms, the determination of the characteristic value can be divided into two categories. The first category uses the average value of the population combined with a safety factor (methodology A), while the second category defines the characteristic value as a lower limit, e.g. 5% quantile (methodologies B and C). The (dis)advantages of both categories are discussed below.

6.1. Methodology A: average value with safety factor

If the definition of the characteristic value is based on the average value of a population combined with a safety factor, one should note that the determination of the average is not straightforward. First, one can argue which definition is most suitable (arithmetic mean, median, geometric mean, etc.). Second, problems may arise when estimating these values. For a Gaussian distribution, everything is relatively straightforward and the main problem is linked to a limited number of samples. However, when a population is composed of different subpopulations or when the population is skewed, the complexity increases and there is no single methodology for the different types of dataset. For example, when the original dataset is skewed, the value of the arithmetic mean is affected by the way it is estimated (i.e. applying e.g. the lognormal theory or not, and which method is considered). For example, for a lognormal distribution the geometric mean is considered to be a more efficient (with a larger reliability) estimator than the arithmetic mean (Rendu, 1981).

6.2. Methodology B: X% lower limit on the basis of a distribution function

As already mentioned before, a characteristic value can also be a reasonable choice of a minimum value (with or without considering an additional safety factor). This can be done by fitting one of the standard distribution functions to the dataset and working further with this theoretical function. In this way, an X% lower limit value can be determined (i.e. any percentage that is considered appropriate). This way of working takes (apart from the mean) also the spread of the dataset into account. Note that the easiest definition of the spread is the minimum and maximum value recorded. The advantage of this method is that it overcomes problems such as the fact that the minimum and maximum values of a dataset are normally not the proper extreme values (if one increases the number of data points, there is always the chance that an additional data point is situated outside the first recorded range). Besides, by assuming a theoretical distribution function one avoids to base the limit value on the information of one single data point.

Nevertheless, it is not always easy to fit a standard distribution to a given dataset. The most appropriate distribution should be determined, followed by the estimation of its properties. This distribution function can be different for each site and it is even not guaranteed that a suitable standard distribution exists. Apart from these remarks, it should also be noted that the properties of the theoretical distribution face similar problems as discussed for the average, i.e. the effect of the limited number of data points, but additional uncertainties introduced by the assumed statistical theory are also possible.

This is illustrated for the lognormal theory applied on a dataset of 41 UCS values with an arithmetic mean, μ, of 8.63 MPa and a standard deviation, σ, of 6.99 MPa (see Figure 17a for its distribution). This distribution is clearly not Gaussian; the red curve in Figure 17a is the theoretical Gaussian curve for μ and σ calculated. If a dataset is lognormally distributed, then the logarithm of the dataset is normally distributed. Possibly, a factor β has to be added to the values to obtain an optimal fit with a normal distribution after transformation (see Figure 17b, where β = 0.6).
Based on these logarithmic values, a confidence limit is then determined by the theory for Gaussian distributions, as illustrated in Figure 17b by the vertical red line for a 5% lower limit. The actual characteristic value is then obtained by back-transformation, resulting, in a value of 1.46 MPa.
Problems of this approach are the choice of a correct value for β, the fact if the distribution is really lognormal or not and the effect of the limited number of data points.

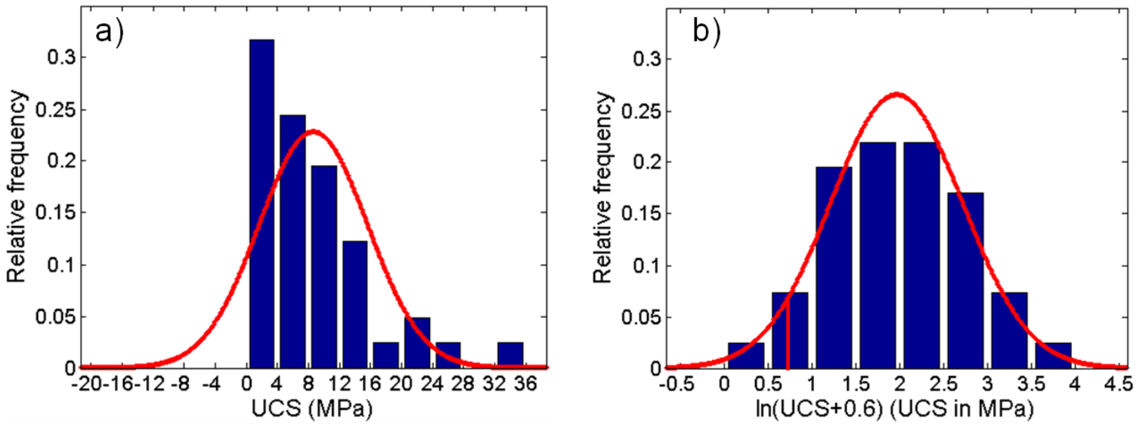

Figure 17: a) Distribution of the UCS values of 41 cores of DSM material from a site in Gent and the corresponding theoretical Gaussian curve. b) Distribution of the logarithm of the UCS values of the same site but increased with β = 0.6 and the corresponding Gaussian curve. The vertical red line indicates the 5% lower limit value

6.3. Methodology C: X% lower limit on the basis of the cumulative curve

Therefore, one should maybe consider determining the X% quantile directly on the original experimental dataset and independent of any distribution function. For the mentioned dataset this results in a 5% quantile UCS value of 1.36 MPa.
Of course, when one disposes of less than 20 data points, this method (i.e. direct estimation on the cumulative curve) cannot be applied. However, any other method probably results in a large uncertainty.

6.4. Determination of the percentage X

Note that for both approaches X needs to be defined. In Eurocode 7 design X is often stated at 5%, but in the case of the UCS of DSM material a more detailed analysis of all the experimental test data is necessary in order to determine if a 5% lower limit is a representative characteristic value, in particular with regard to the treatment (elimination rule or not) of the samples with large inclusions (see Section 7).

It is clear that more extensive investigation is necessary in order to formulate clear directives. Some points to be further investigated are the statistical methodology, the scale effect, the effect of the execution method and the difference between soil types. However, it is recommended to (i) plot the distribution, (ii) look at the range (minimum and maximum) and (iii) compare mode, median and mean before applying any statistical approach.

Table 3 presents an overview of some of these statistical values for DSM cores originating from several Belgian sites.

Table 3: Some important statistical values for DSM cores originating from several Belgian sites

Site	Soil type	Age (days)	Nǂ (-)	Min. (MPa)	Max. (MPa)	Median (MPa)	μ† (MPa)	σ‡ (MPa)
Alsemberg	Clayey loam	120	15	1.49	3.54	2.75	2.72	0.52
Brugge	Sand	55	51	2.33	9.56	5.80	5.65	1.88
Zeebrugge	Sand	86	40	4.14	7.79	6.65	6.44	0.9
Gent I	Alluvial clay	60	41	0.96	33.23	6.44	8.63	6.99
Borgloon	Soft clay	130	26	0.53	15.44	5.36	5.65	3.1
Antwerpen J	Sand	30	29	2.30	18.74	11.76	11.00	3.72
Antwerpen K	Sandy clay	30	12	2.20	24.80	10.80	12.79	6.75
Knokke I	Peaty clay	30	25	0.44	30.81	5.45	7.33	7.19
St-Lievens Houtem	Loam	30	31	3.56	23.03	9.34	10.78	4.93
Limelette	Loam	150	45	3.76	31.04	13.57	14.05	6.2
Antwerpen MH	Sand	30	50	4.88	23.77	10.91	11.93	4.09
Duffel	Alluvial clay	85	6	0.65	1.52	1.27	1.18	0.36
Gent II	Remolded and sandy soil	14	18	1.28	2.94	2.02	2.14	0.48
Knokke II	Sand	91	16	5.05	13.60	10.95	10.71	2.15
		523	17	11.78	19.98	13.61	14.16	2.14
Wetteren	Remolded and loamy/sandy soil	49	25	0.66	6.80	4.27	4.03	1.33
		91	24	2.07	8.84	4.63	4.72	1.61
Elsene	Clay	117	10	2.19	12.69	9.71	9.38	2.77
Turnhout	Sand with bricks	57	11	3.40	11.54	6.79	6.74	2.75

ǂN is the number of UCS test data
†μ is the arithmetic mean of the UCS data
‡σ is the standard deviation of the UCS data

7. PRELIMINARY CONCLUSIONS ABOUT THE INFLUENCE OF SOIL INCLUSIONS

Within the framework of the BBRI 'Soil Mix' research, all inclusions in DSM material are considered as unmixed soft soil inclusions. A methodology taking into account these inclusions was developed and illustrated with case studies of DSM material executed in several Belgian soils (Ganne et al., 2011 and 2012). Figure 18 gives an overview of the results for 27 Belgian construction sites.
The amount of soil inclusions in DSM material mainly depends on the nature of the soil:
- in quaternary or tertiary sands, it is less than 3.5%,
- in silty (or loamy) soils and alluvial clays, it ranges between 3 and 10%,
- in clayey soils with high organic content (such as peat) or in tertiary (overconsolidated) stiff clays, it can amount up to 35% and higher.

One major issue concerns the representativeness of the core samples with regard to the in situ executed DSM material. On the one hand, there is the question of the scale effect and on the other hand, the question of the influence of soil inclusions. Both have an influence on the UCS test results. To investigate these topics, an experimental, as well as a numerical simulation research programme has been initiated at KU Leuven (Vervoort et al., 2012). This research programme studies the behaviour before and after failure of the DSM material.
The experimental part focuses on laboratory experiments with the study of the scale effect. The behaviour in laboratory is certainly affected by the scale and the dimensions of the test samples. Apart from traditional cores (with a diameter around 10 cm), large scale tests are conducted on rectangular blocks with approximately a square section, with a width corresponding to the width of the in situ SMW (about half a meter) and with a height approximately twice the width.
In parallel, numerical simulations (2D) were performed to quantify the effect of soil inclusions on the strength and stiffness of the DSM material. The following parameters are being considered: size, number, relative position and percentage of soil inclusions. Three approaches were followed with the help of:
- an elastic model only focusing on the DSM stiffness,
- an elasto-plastic model whereby, apart from the stiffness, the strength is analysed,
- a discontinuous model concentrating on the initiation and growth of individual fractures.

That research programme should quantify the maximum acceptable limits of volume percentages of inclusions and optimize the test procedure with regard to the tests on samples with soil inclusions larger than 1/6 of the specimen diameter. In engineering practice test specimens with inclusions/particles larger than one sixth of the specimen diameter must be cautiously regarded, as highlighted in the following standards.

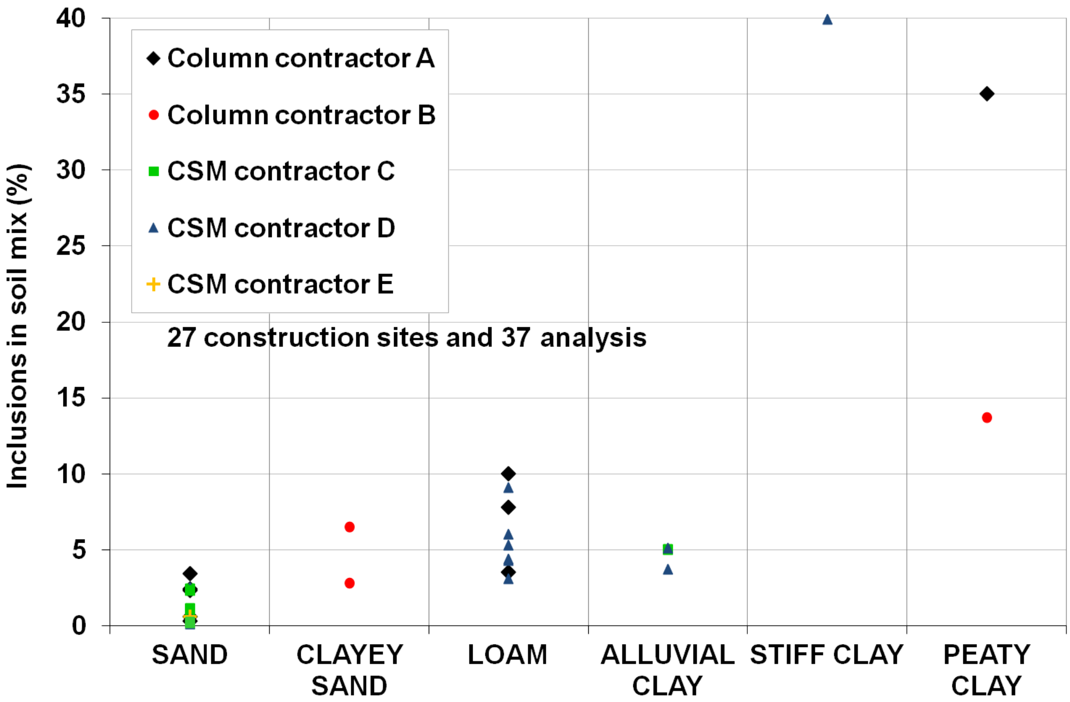

Figure 18: Percentage of soil inclusions in DSM material

As reported in ASTM D 2166-91, describing the standard test method for **UCS of cohesive soil**: *"For specimens having a diameter of 72 mm (2.8 in.) or larger, the largest particle size shall be smaller than one sixth of the specimen diameter. If, after completion of a test on an undisturbed specimen, it is found, based on a visual observation, that larger particles than permitted are present, indicate this information in the remarks section of the report of test data"*.
The procedure for **UCS tests on fine grained soils** is described in NBN CEN ISO/TS 17892-7: 2005, which specifies that: *"The largest particle in the specimen should not exceed 1/6 of the specimen diameter for cylindrical specimens and not exceed 1/6 of the side length for square specimens"*.
ATSM D 5102-90, dedicated to **UCS tests of compacted soil-lime mixtures**, reports that: *"For specimens having a diameter of 72 mm (2.8in.) or larger, the largest particle size shall be smaller than 1/6 of the specimen diameter"*.
For **testing hardened concrete** samples, the requirements concerning the dimensions of the specimens intended for UCS tests are described in NBN EN 12390-1. According to this standard: *"for each shape of test specimen, cube, cylinder and prism, the basic dimension should be chosen to be at least three and a half times the nominal size of the aggregate in the concrete"*.
The philosophy behind these standards is that samples with inclusions or particles larger than 1/6 of the sample dimension are not representative for the material as such. The practical question is what to do when in the core material with a fixed diameter such inclusions or particles are observed. The theoretical answer to this question is that one should drill again, but now with a larger diameter. For various reasons, this is not necessary feasible: often the wall is not easily accessible anymore, one cannot drill too many boreholes and certainly with large diameter without weakening the wall, etc. In other applications for concrete or rock masses, one has often a good idea beforehand of the size of inclusions or particles. This is for soil mix material not really the case. The BBRI 'Soil Mix' project aims to quantify the effect of such large inclusions on the behaviour of core samples, of large blocks and on in situ walls (Vervoort et al., 2012). For example, for a core sample with a diameter of 90 mm, an inclusion larger than 15 mm affects the strength in most cases significantly; however such an inclusion of 15 mm diameter within an in-situ wall is most likely acceptable. This is the main reason why Ganne et al. (2010) proposed to reject all test samples with soil inclusions larger than 1/6 of the specimen diameter, on the condition that no more than 15% of the test samples from one particular site are rejected. If more than 15% of the test samples are rejected, it is obvious that the problem is not a local one and that further investigation and evaluation are needed, combined with a good engineering judgement. For the calculation of the values of Table 3 this elimination rule was not applied.
In Fig. 19, two examples are given for the distribution of UCS values whereby the samples with inclusion(s) larger than 1/6 of the diameter are indicated. For the first site (Fig. 19a), these samples (6 on a total of 51 samples) have a UCS value less than 3.95 MPa and are clearly situated in the left part of the distribution, as one would normally expect. For the second site (Fig. 19b), some of these samples are situated in the left part of the distribution (6 of the 9 samples with inclusion(s) larger than 1/6 of the diameter on a total of 26 samples), but the other 3 samples have a UCS value of more than 7.37 MPa. The reason why such samples still result in a relatively large strength value is probably linked to the number and relative position of the inclusion(s) (among themselves and within the sample). This forms also part of the BBRI 'Soil Mix' project, where numerical simulations are conducted to quantify the effect of various positions, sizes, shapes, number, etc. of inclusions and this for different scales (Vervoort et al., 2012). These examples show that the way how is dealt with samples containing large inclusions, can have a considerable influence on the deduction of engineering parameters (cfr. Section 6)

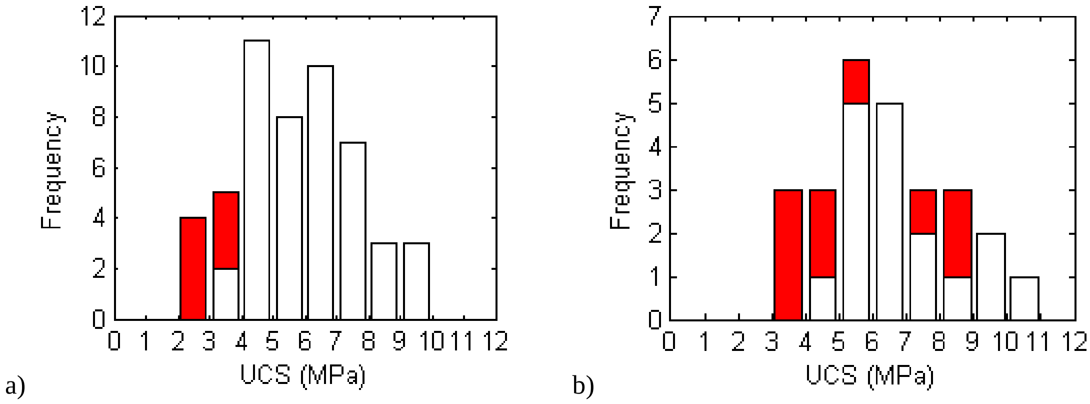

Figure 19: Distribution of the UCS values from 2 different sites. The samples with inclusions larger than 1/6 of the specimen diameter are indicated in red: a) 6 on a total of 51 samples; b) 9 on a total of 26 samples

8. CONCLUSIONS AND PERSPECTIVES

Within the framework of a Flemish regional research program "Soil Mix" (IWT 080736), DSM materials from 38 Belgian construction sites, with various soil conditions and for different execution processes, have been tested. The present paper describes results of mechanical tests performed on DSM material executed with the Cutter Soil Mix (CSM), the Tubular Soil Mix (TSM) and the CVR C-mix® systems (Denies et al., 2012a). The UCS, the modulus of elasticity and the tensile strength are determined, as well as the porosity, the permeability and the ultrasonic pulse velocity of core samples. As illustrated in Fig. 9 and 10, UCS, modulus of Elasticity and tensile strength are correlated with the help of equations (2) to (4) and (7) to (9). They correspond to previous correlations proposed by Topolnicki and Trunk (2006), as presented in Table 4.

Table 4: Correlations between the UCS, the modulus of elasticity, the shear and tensile strengths, after Topolnicki and Trunk (2006)

Parameter	**Empirical relationships**	
Age of the specimen (days)	$UCS_{28\ days}$	$= 2.0\ UCS_{4\ days}$
		$= 1.4 – 1.5\ UCS_{7\ days}$ (silt and clay)
		$= 1.5 – 2.0\ UCS_{7\ days}$ (sand)
	$UCS_{56\ days}$	$= 1.4 – 1.5\ UCS_{28\ days}$ (silt and clay)
Coefficient of variation (%)	COV	= 20 – 60
		= 30 – 50 (usually)
Shear strength (MPa)	τ	= 0.40 – 0.50 UCS for UCS < 1 MPa
		= 0.30 – 0.35 UCS for 1< UCS < 4 MPa
		= 0.20 UCS for UCS > 4 MPa
Tensile strength (MPa)	T_a	= 0.08 – 0.15 UCS with a maximal value of 0.2 MPa
Secant modulus of elasticity (50% UCS) (MPa)	E_{50}	= 50 – 300 UCS for UCS < 2 MPa
		= 300 – 1000 UCS for UCS > 2 MPa
Elongation at maximal force (%)	ε_u	= 0.5 – 1.0 for UCS > 1 MPa
		= 1.0 – 3.0 for UCS < 1 MPa
Poisson ratio (-)	ν	= 0.25 – 0.45
		= 0.30 – 0.40 (usually)

Some of the findings so far are:

- As a result of the petrographic analysis, high porosity values of the DSM material can be related to the high and homogeneous capillary porosity of cement stone resulting from high W/C ratio used for the execution of the SMW. From permeability tests, the coefficient of hydraulic conductivity of DSM material varies between 10^{-8} and 10^{-12} m/s.
- The sampling, the transportation, the storage, the handling and the preparation of the test specimens are detailed in Denies et al. (2012b).
- In situ pull-out test results are also described in order to verify the adherence between DSM material and various steel profiles ensuring the SMW stiffness.
- On the basis of UCS tests performed on core samples, the determination of the 5% quantile characteristic UCS value is then discussed.
- Finally, the results of a methodology quantifying unmixed soil inclusion in the material are presented for various Belgian soils. The representativeness of the core samples is questioned with regard to the influence of unmixed soil inclusions in the material and considering the scale effect.For the purpose of investigating this engineering issue, reference is made to the experimental and numerical research program performed in KU Leuven with the help of large scale tests and numerical developments (Vervoort et al., 2012).

While SMW was previously used only for temporary excavation support, permanent retaining and foundation applications with DMM are increasingly applied in Belgium. Hence, the durability aspects of the DSM material have to be considered. In the second period of the BBRI 'Soil Mix' project, the DSM material shall be investigated in terms of its alkalinity properties, with the help of pH long term measurements, in order to control its level of corrosion protection. The viability of the process in the presence of polluted soils shall also be considered.

Based on the results of the BBRI 'Soil Mix' project, a design method for DSM structures, accounting for the presence of the heterogeneities and unmixed soil inclusions, the scale effects and the time effects such as curing time and creep shall be developed.

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REFERENCES

Denies, N., Huybrechts, N., De Cock, F., Lameire, B., Maertens, J. and Vervoort, A. 2012a. Soil Mix walls as retaining structures – Belgian practice. International symposium & short courses of TC211. Recent research, advances & execution aspects of ground improvement works. 30 May-1 June 2012, Brussels, Belgium.

Denies, N., Huybrechts, N., De Cock, F., Lameire, B., Vervoort, A. and Maertens, J. 2012b. Mechanical characterization of deep soil mix material – procedure description. International symposium & short courses of TC211. Recent research, advances & execution aspects of ground improvement works. 30 May-1 June 2012, Brussels, Belgium.

Ganne, P., Huybrechts, N., De Cock, F., Lameire, B. and Maertens, J. 2010. Soil mix walls as retaining structures – critical analysis of the material design parameters, International conference on geotechnical challenges in megacities, June 07-10, 2010, Moscow, Russia, pp. 991-998.

Ganne, P., Denies, N., Huybrechts, N., Vervoort, A., Tavallali, A., Maertens, J, Lameire, B. and De Cock, F. 2011. Soil mix: influence of soil inclusions on structural behaviour. Proceedings of the XV European conference on soil mechanics and geotechnical engineering, Sept. 12-15, 2011, Athens, Greece, pp. 977-982.

Ganne, P., Denies, N., Huybrechts, N., Vervoort, A., Tavallali, A., Maertens, J, Lameire, B. and De Cock F. 2012. Deep Soil Mix technology in Belgium: Effect of inclusions on design properties. Proceedings of the 4th International conference on grouting and deep mixing, Feb. 15-18, 2012, New Orleans, Louisiana, USA.

Hird, C. C. and Chan, C. M. 2005. Correlation of shear wave velocity with unconfined compressive strength of cement-stabilised clay. Proceedings of the International Conference on Deep Mixing: Best Practice and Recent Advances, May 23-25, 2005, Stockholm, Sweden, pp. 79-85.

Pauw, A. 1960. Static modulus of elasticity of concrete as affected by density. Journal of the American Concrete Institute, Vol. 32, N°6, pp. 679-687.

Rendu, J.-M, 1981. An Introduction to Geostatistical Methods of Mineral Evaluation, Geostatistics 2. South African Institute of Mining and Metallurgy.

Topolnicki, M. and Trunk, U. 2006. Tiefreichende bodenstabilisierung. Einsatz im Verkehrswegebau für Baugrundverbesserung und Gründungen. Tiefbau, Vol. 118, N°6, pp. 319 – 330 (in German).

Vervoort, A., Tavallali, A., Van Lysebetten, G., Maertens, J., Denies, N., Huybrechts, N., De Cock, F. and Lameire, B. 2012. Mechanical characterization of large scale soil mix samples and the analysis of the influence of soil inclusions. International symposium & short courses of TC211. Recent research, advances & execution aspects of ground improvement works. 30 May-1 June 2012, Brussels, Belgium.