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Evaluation of the State of Sandy Soils on a Sinkhole Area with the use of Noninvasive (MASW) and Invasive (SDMT) Tests

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The subject of the study is to assess the state of soil in the area of a former shallow underground mining site which presents a potential hazard of sinkholes. Two kinds of tests were performed i.e. noninvasive seismic tests MASW and invasive seismic dilatometer tests SDMT. The test procedures and the used equipment are described in the paper. The MASW tests allowed the detection of shallow and deep seismic anomalies – places with reduced mechanical parameters. Shallow anomalies were subjected to SDMT tests. Stiffness parameters were adopted as a measure of the state of soil. The applied consistent methodology allowed for the assessment of soil stiffness for intermediate and very small strains. It has been shown that the shallow anomalies were not caused by sinkhole processes. The values of stiffness for intermediate and very small strains in the zone above shallow anomalies occurred to be inconsistent, suggesting the influence of cementation or desiccation processes.

Key words: in-situ tests, MASW, SDMT, soil stiffness, sinkhole detection.

1. Introduction

Construction on brownfields is associated with various types of threats. Areas where previously underground mining was carried out are vulnerable to soil subsidence. Whittaker and Reddish [25] characterised the phenomenon of soil surface deformation caused by mining activities, described the conditions that are conducive for the development of deformations and linked them to the geology of the area and methods of mining operation.

Mining-caused subsidences are divided into continuous (troughs or sags) and discontinuous (sinkholes). The latter ones occur in the analyzed area and are considered in this paper. Discontinuous deformations are described in detail in Chudek *et al.* [6] and Singh and Dhar [19], among others. According to these authors, the risk of sinkholes is high if exploitation is carried out at shallow depths and overburden is weak.

Methods for sinkhole prediction may be divided into:

- empirical (e.g. Chudek *et al.* [6]), where basic parameters include geometric data regarding exploitation depth, extracted seam height or the thickness of hard rock above the seam,
- numerical (e.g. Augarde *et al.* [2]), where geometric information is supplemented by strength and stiffness parameters of soil which are necessary to model its mechanical behaviour.

Both empirical and numerical methods present a number of advantages and disadvantages. The main drawback of the former ones is the lack of extrapolative abilities of mathematical formulas or charts developed on the basis of observations in one region, to areas of a different geological structure or other extraction methods. On the other hand, numerical methods lack a universal constitutive model to effectively predict the behaviour of heterogeneous soil. Additionally, for advanced constitutive models, there is a large number of mechanical parameters, values of which should be determined in tests in numerous points of soil mass.

Observations of sinkholes and their parametric analyses, performed mainly with the use of empirical methods, allowed categorisation of areas potentially jeopardised by sinkholes. Examples of such classifications can be found in Chudek *et al.* [6] or Ruegsegger [17]. According to the criteria presented in the mentioned works, the area analysed in this study belongs to the category with the highest risk of sinkhole formation. The recognition of the category that the area belong to may be very helpful when making future investment decisions, but is completely insufficient in the assessment of the state of existing buildings in the threatened area.

Before World War II, lignite was mined in the analysed area by the underground method, at shallow depths. The lignite seams are within the young Cenozoic formations. The geological structure is complicated, because of glacitectonic deformations. In addition, the mining documentation of the area is incomplete due to the war and post-war territorial changes. The area is urbanized and sinkholes which develop every few years threaten people and existing buildings.

The geological structure together with the recently developed sinkholes is described by Szajna and Gontaszewska [23]. They also present archival records of mining operations before the war. Lignite mining did not exceed the depth of 50 m. The area is heavily folded and the several-meter superficial zone consists of non-cohesive glacial cover. Ground water level is low, approximately 20 m below the surface.

The aim of this study is to assess the parameters of the state of soil in a built-up area adjacent to sinkholes and to find possible locations where such processes are developing. Two research methods were used: a noninvasive method – MASW (Multichannel Analysis of Surface Waves) and an invasive method – SDMT (Seismic Dilatometer Test). The use of two different methods allowed for cross verification of results.

The following key issues are discussed in this paper:

- the selection of parameters that would adequately describe the state of soil and, at the same time, could be useful in numerical analyses,
- the description of the used research methods and the applied methodology,
- the description and the evaluation of the obtained results.

2. Selection of parameters for the description of the state of soil

Soil is a porous multiphase medium, consisting of grains and particles forming its skeleton (i.e. solid phase of volume V_p) and pores of volume V_v, part of which is filled with water (liquid phase of volume V_w). The remaining part is filled with compressible air (gas phase of volume V_a). Strength and stiffness parameters of soil depend on its stress and strain state. Due to the high stiffness of soil particles, major deformations are associated with the change of porosity. Thus, in the water-saturated medium, both stiffness and strength depend on the coefficient of permeability. Since it is difficult to consider permeability in calculations, simplified soil analyses are carried out separately for drained and undrained load conditions. In this paper, drained load conditions are considered.

Only some mechanical parameters, e.g. the critical angle of internal friction, depend solely on the type of soil and can be regarded as material constants. Most parameters also depend on the current state and the current structure of soil (see, e.g. Mitchell [16]).

In classical soil mechanics, the current state of soil is most frequently described by: state parameters (e.g., void ratio e and the degree of saturation S_r), state variables (e.g., effective stress tensor composed of normal stress σ' and shear stress τ' components) and a parameter relating to the history of stress changes (e.g. overconsolidation ratio OCR). The definitions of the particular parameters and state variables are as follows:

$$\begin{gathered} e = V_v/V_p, \qquad S_r = V_w/V_v, \qquad \sigma' = \sigma - u, \\ \tau' = \tau, \qquad OCR = \sigma'_v/\sigma'_p, \end{gathered} \tag{2.1}$$

where σ and τ are respectively a normal and a shear component of the total stress tensor, u is pore water pressure, σ'_v is the current value of the vertical component of effective normal stresses, and σ'_p is the effective stress of overconsolidation.

A soil structure is understood as the combination of particle arrangement (fabric) and interparticle forces (bonding), excluding friction and dilation effects, Mitchell [16]. Bonding results mainly from cementation. In terms of geological processes, fabric depends mainly on sedimentation conditions, whereas bonding is the result of soil diagenesis.

The essessment of the current state of non-cohesive soils, which are subject of this paper, is a difficult task. The determination of the void ratio in a laboratory requires undisturbed samples. However, during sampling, the porosity, pore pressure, the state of stresses and the humidity distribution alter in the soil sample. It is particularly difficult to obtain a sample of sand when it is in a loose state which frequently prevents testing the state parameters and also other mechanical parameters, values of which depend on the state. In a highly inhomogeneous soil medium, the representativeness of the sample taken for laboratory tests is also questionable.

In-situ tests of soil carried out in its natural state and environment provide an alternative to soil laboratory tests. Still, major difficulties occur since most research tools do not measure directly mechanical parameters of stiffness and strength but only the reaction of soil induced by the movement of a testing device in the soil medium. The determination of the required mechanical parameters involves therefore resolving the inverse boundary problem. In practice, the correlations between the measured reaction and the required parameter (stiffness or strength) are used most frequently. However, the correlations are not general, since the soil as a natural material is inhomogeneous and its reactions to load may also vary. The above discussion suggests that, in the case of granular soil, the selection of parameters describing its state, values of which could be reliably determined at numerous points of a heterogeneous soil medium, is an important issue.

In the above context, the studies made by Atkinson and Sallfors [1] and confirmed by subsequent authors are of essential importance. An overview of these works was presented by Clayton [7]. The authors reported linear-elastic behaviour of soil for very small strains, below 0.001%. It means that within this range, such parameters as Young's modulus or shear modulus are constant. They can be regarded as material constants for a given type of soil, in a particular state, and with a specific structure.

With an increase in strains, stiffness parameters, shear modulus G including, are decreasing. Modulus G determined for very small strains reaches its maximum value and is usually denoted as G_0. Figure 1 presents exemplary test

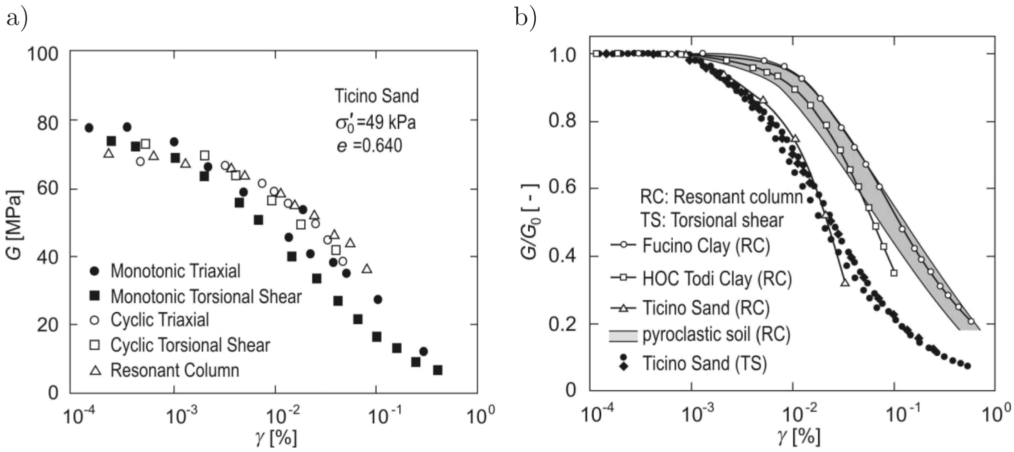

FIG. 1. Secant shear modulus degradation with strain shear increase: a) for sand [24], b) for several different soils [11].

results for secant shear modulus as a function of shear strain γ, made by TATSUOKA *et al.* [24] for Ticino sand, with the use of five different measurement methods and by LANCELLOTTA [11] for several types of soil.

CLAYTON [7] states that the value of shear modulus G_0 for sands in the range of very small strains depends on:

- deformability of individual grains which is characteristic for a given type of soil,
- void ratio e which determines the state of soil,
- stiffness of the contact between soil grains, which is influenced by the type of mineral, the degree of abrasion of the grain surface and the level of effective stresses.

The first factor is particularly important in the case of highly deformable sands containing mica. The second factor relates to all types of sand. The third factor includes also the effect of cementation and matrix suction of unsaturated soils.

Figure 2a shows the effect of void ratio and cementation on shear stiffness of cohesive and non-cohesive soils according to JAMIOLKOWSKI *et al.* [9]. Value G_0 increases with decrease in e, and the increase is dramatic (by almost one order) in the case of soil cementation.

Figure 2b shows the test results for natural non-cemented sands (coarse – CSa and fine – FSa), their mixtures, and mixtures of coarse sand and mica platelets, obtained by CLAYTON et al. [8]. The figure shows that the growth of normal effective stresses and the growth of the quantity of coarse fraction, results in the increase of stiffness G_0.

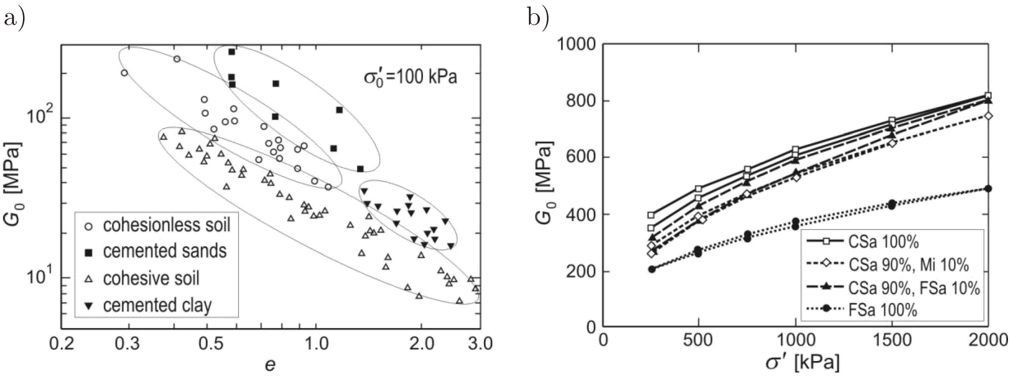

FIG. 2. Relation between stiffness G_0 and: a) void ratio and cementation by JAMIOLKOWSKI *et al.* [9], b) effective stresses and soil composition by CLAYTON *et al.* [8].

For a linear-elastic medium, shear modulus is proportional to the product of the mass density (ρ) and the square of the velocity of shear waves (V_s) according to the following relation:

$$G_0 = \rho V_s^2. \tag{2.2}$$

This means that the soil shear modulus can be easily determined in the range of very small strains by measuring the velocity of wave propagation in the ground and estimating the mass density.

The examination of the velocity of shear waves in silts and sands carried out by CHO and SANTAMARINA [5] showed that the velocity and the modulus depend on the degree of saturation (S_r), Fig. 3. Low degree of saturation

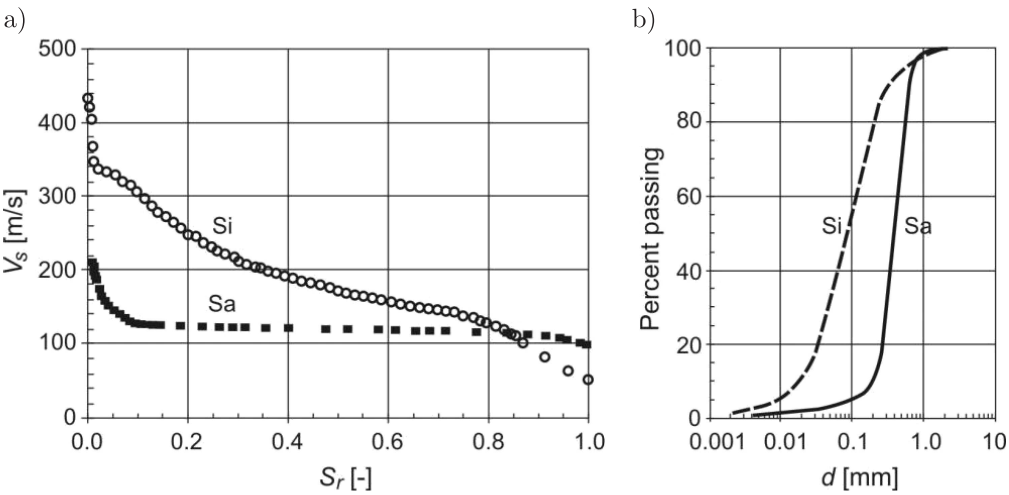

FIG. 3. Results of the granite silt (Si) and natural sand (Sa): a) the impact of saturation degree S_r on shear wave velocity V_s, b) soil sieving curves, [5].

is associated with considerable impacts of capillary forces. The forces are inversely proportional to the radius of the capillary hence their values in silts are much larger than in sands. Considerable capillary forces contribute to the high stiffness of the contact between ground particles, which is consistent with the observations of Clayton *et al.* [8].

It can be concluded that, for very small strains, the shear modulus may be a convenient parameter in indirect estimation of the state and structure of soil in the area potentially threatened with sinkholes. Significant values G_0 will indicate dense material with a low void ratio, cemented soil or soil subjected to large capillary forces while drying in its surface zone. For homogeneous soil, the modulus values should increase slightly with depth due to the increase in geostatic stresses. Small values of the modulus may suggest a very loose material, where sinkholes develop.

Assuming stiffness as a substitute measure of the state of soil, it is advisable to check changes of the stiffness while strains increase (compare Fig. 1). A dilatometer, described in the following chapter, is a useful device which allows effective measurement of stiffness within the range of intermediate strains. However, the device enables the determination of constrained modulus M, (measure of stiffness for zero lateral strain), instead of shear modulus.

In order to directly compare shear stiffness for very small strains, where the behaviour of soil is linear, with the constrained stiffness for intermediate strains, G_0 will be converted into M from the equations of the linear theory of elasticity:

$$M = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}, \qquad E = 2(1+\nu)G, \tag{2.3}$$

where E is Young's modulus and ν is Poisson's ratio. Constrained modulus calculated for very small strains will be denoted as $M_{\max}$.

3. Research methods

According to the objective of the study, i.e. the assessment of the soil state and the search for possible locations of sinkholes developing in the vicinity of existing buildings, an area measuring approximately 100 by 100 m, Fig. 4, is subjected to tests and analyses. A sinkhole, formed in February 2012 is marked in the northern part of the area. This small, cylindrical hollow, with a diameter of 2.5 m and a similar depth, developed only in a few hours, reflecting the dynamics of the process and hazards to people and existing buildings. The location of the sinkhole is difficult to be associated unambiguously with a particular object of the mining infrastructure shown on archival maps. Considering the size of the studied area, the average exploitation depth (20–40 m) and incomplete mining

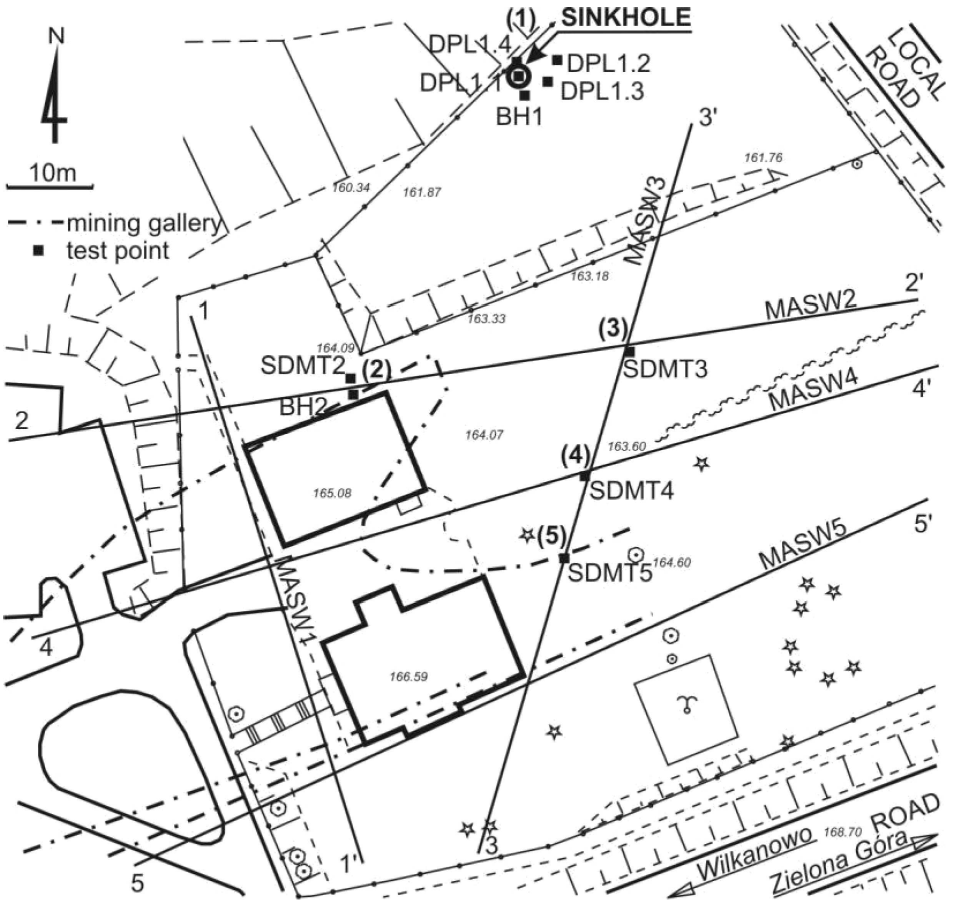

Fig. 4. Map of the studied area (abbreviations are explained in Sec. 4), modified after [3].

archival data, traditional research methods i.e. soundings and drillings might be inefficient.

The values of the velocity of shear waves V_s were determined in seismic surveys, performed by a commercial company GeoSpectrum [3] with the use of a noninvasive method, i.e. MASW, and in additional tests performed with an invasive probe, i.e. SDMT. The use of MASW was very beneficial since it allowed the estimation of V_s in a large volume of soil on the basis of surface tests. A 2-D picture of the analysed medium in a vertical cross-section below the measurement line was received. The locations of particular lines, marked as MASW1 to MASW5, are shown in Fig. 4. The applied methodology is well suited to the detection of seismic anomalies in the area of relatively contrasting features, but the absolute accuracy of the designated velocity of waves V_s is moderate. For these reasons, control tests were performed with SDMT method in selected points. This invasive method allows direct measuring of the velocity of shear waves as well as some additional characteristics of the soil, but the obtained data is only 1-D. Since the methods are still a novelty and are not popular, their brief description is presented below.

MASW method belongs to the group of geophysical seismic tests which use surface waves. Physical basis of this group of methods are presented by Stokoe and Santamarina [21]. The general scheme is shown in Fig. 5. Tests are carried out in two stages.

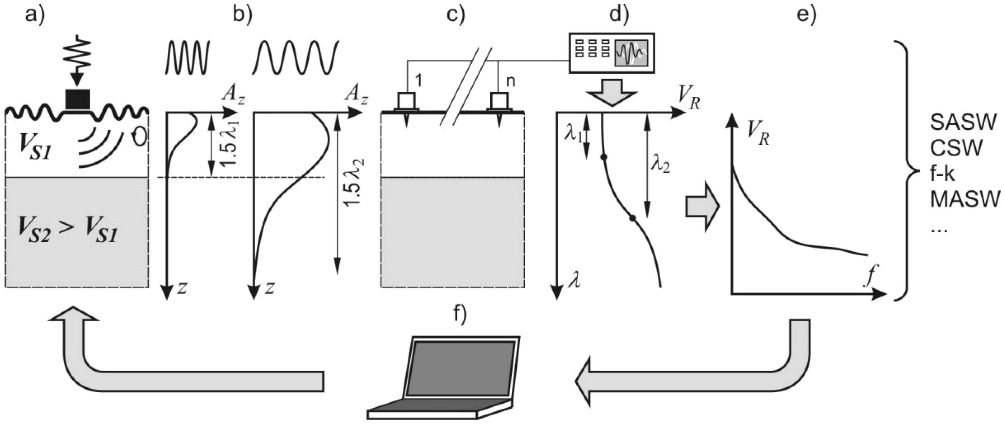

Fig. 5. Diagram of noninvasive seismic methods using surface waves: a)–d) measurement of wave velocities, e) dispersion curve, f) inverse analysis (determination of soil profile), modified after [26].

The first stage – in situ tests:

a) In layered soil, shear wave velocity V_s, and thus shear modulus, is searched for each individual layer. An exemplary diagram, shown in Fig. 5a, assumes that the stiffness of subsequent layers increases with depth (so $V_{s2} > V_{s1}$). The test starts with vibrations generated on the surface. They propagate as longitudinal waves (P), shear waves (S), as well as Rayleigh surface waves (R) essential for the test. The cylindrical front of the Rayleigh waves propagates radially along the surface. Vibrating soil particles oscillate along elliptical trajectories. A vertical vibration amplitude A_z fades with depth. Oscillations disappear totally at a depth of approx. 1.5 λ, Fig. 5b. The wavelength λ depends on excitation frequency. At sufficiently high frequencies, the formed waves are so short that they propagate only in the first layer, therefore – given the assumption – the velocity is low. When the excitation frequency is sufficiently low, the long waves propagate also in the lower layer of greater stiffness – and thus with a higher velocity. The waves are received by a system of n geophones located on the surface along the measurement line, Fig. 5c.

b) Waves of different lengths reach a given receiver after different times. Incoming waves are recorded and then analysed, the results of which are presented in a form of a graph $\lambda - V_R$, where V_R is Rayleigh wave velocity, Fig. 5d.

c) The resulting graphs $\lambda - V_R$ are converted into a dispersion curve in $f - V_R$ system, where f is the frequency, Fig. 5e. The $f - V_R$ graph is a characteristic of a given soil medium.

The second stage – numerical modelling:

d) An inverse analysis is performed. It involves the determination of the required stiffness profile (velocity V_s) of the soil, Fig. 5f.

The above steps can be carried out with one of the methods: spectral analysis of surface waves (SASW) method, the continuous surface wave (CSW) method, the frequency wave number (f-k) spectrum method or the multi-channel analysis of surface wave (MASW) method. The particular methods differ in the way of wave excitation (e.g. f-k: a pulse, CSW: a source of harmonic waves), the amount of geophones (e.g. SASW: $n = 2$–4, f-k: $n = 128$, 256, etc.), the way of phase velocity calculation. There are different inverse procedures for the determination of the stiffness profile. The comparison of particular methods and discussion of their advantages and disadvantages in the context of geotechnical applications are presented by Stokoe, Joh and Woods [20].

The SDMT probe is a combination of a downhole true interval seismic method (D-H) and the traditional dilatometer probing (DMT). In the D-H method, the time for shear waves to travel through the soil mass from a signal source on the surface to a geophone receiver is measured according to the procedure shown in Fig. 6a, Marchetti *et al.* [14]. The figure presents the source of waves (in the form of a hammer and anvil), two geophones at depths z_1 and z_2 and a graph of recorded signals.

A rig of 100 kN maximum allowable trust was used for SDMT penetration. The rig was mounted on a track of 1 tonne total deadweight. Due to the existence of non-cohesive soil and industrial waste – slag with large lumps of glass – in the superficial zone, helical anchors were used for track stabilization.

Mechanical flat dilatometer, Marchetti [12], Fig. 6b, is composed of a steel blade (1) inserted to tested soil by rods (2), A flexible circular steel membrane (3) is expanded in soil by gas pressure supplied through a pneumatic tube (4) from gas tank. An electrical cable (5) connects the blade and a control unit. The gas tank and the control unit are on the ground surface. The unit controls gas pressure p_0, p_1, p_2 and electrical signals assigned to particular stages of the membrane (A and C – flat membrane, B – expanded membrane). Pressure p_0, corresponding to the flat position of the membrane, balances the horizontal geostatic stress and allows assessing its value. Pressure p_1 illustrates the resistance

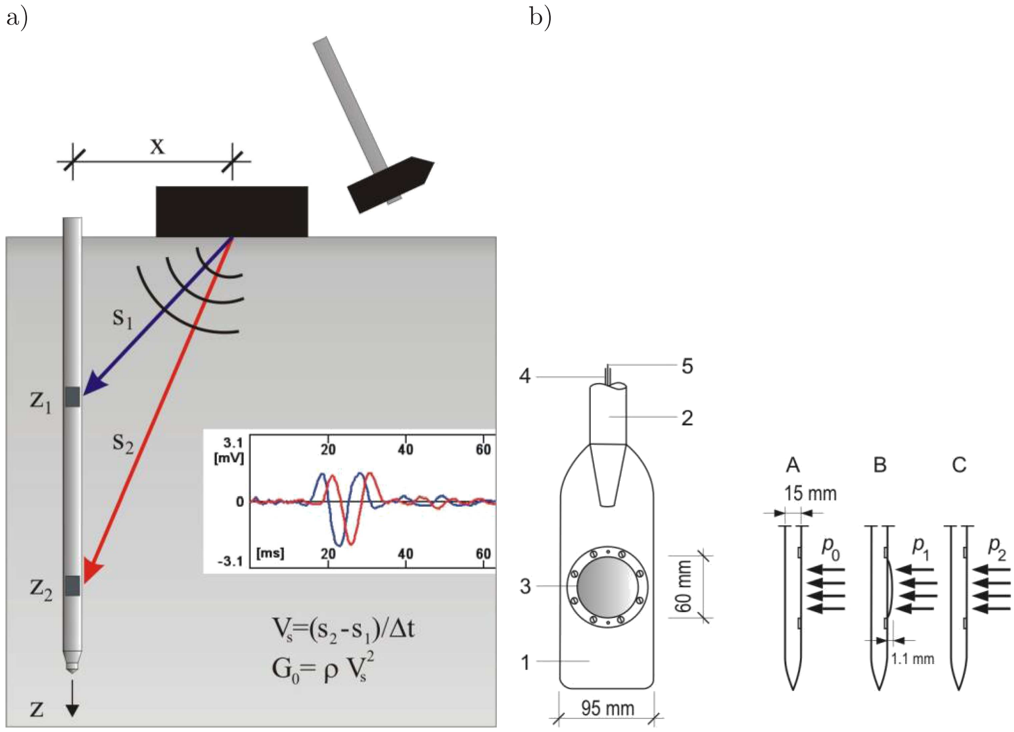

FIG. 6. Seismic dilatometer (SDMT): a) seismic module (D-H), b) blade of mechanical dilatometer (DMT) and test phases, modified after [14].

of soil caused by expanding membrane. Measurement p_2 may be performed in order to evaluate hydrostatic groundwater pressure u_0. In this study, due to the low position of the water table, the measurement was not performed.

The values of the measured pressures p_0 and p_1 and the known value of hydrostatic pressure u_0, allow the determination of the so-called material index I_{DMT}. Soil stiffness parameter, called dilatometer modulus E_{DMT}, may be determined on the basis of DMT tests by solving the task of circular load acting on an elastic half-space, with the known values of membrane diameter and its displacement. Formulas defining the parameters are as follows:

$$I_{DMT} = (p_1 - p_0)/(p_0 - u_0), \qquad E_{DMT} = 34.7(p_1 - p_0). \tag{3.1}$$

DMT is predestined for stiffness measurements for intermediate strains (0.05–0.1%). MARCHETTI *et al.* [13] presented a formula allowing the determination of constrained modulus M:

$$M = R_M \cdot E_{DMT}, \tag{3.2}$$

where R_M is the empirical correction factor. The factor depends on the type and state of soil and varies mostly in the range 1 to 3. A detailed description of data reduction procedure is presented in [13].

Mayne *et al.* [15] proposed the correlation allowing the determination of bulk density of soil:

$$\rho = 1.12 \cdot \rho_w \cdot (E_{DMT}/p_a)^{0.1} I_{DMT}^{-0.05}, \tag{3.3}$$

where ρ_w is water density and p_a is atmospheric pressure.

In this paper, reference is also made to tests performed with the use of a light dynamic probing (DPL). These tests were used to examine the interior of the sinkhole shown in Fig. 4. The discussion of the testing device, the measured parameters and measurement results are presented by Szajna and Gontaszewska [23].

Summarising, the used research methods allow obtaining mutually complementary test results and their cross control. Using the MASW surface test, images of velocities of shear waves in a large volume of soil are obtained and thereby it is possible to locate zones of relatively low elastic characteristics (zones of loose medium). The received wave velocities V_s are not obtained in direct measurements, but are the result of inverse analysis, not always unambiguous.

Invasive tests SDMT provide control and complementary data. It is inefficient to perform them in random locations, but in zones of loose medium determined in the MASW tests. SDMTs provide direct values of wave velocities V_s and allow the control of values estimated at the stage of the inversed analysis of MASW. Additional measurements performed in the SDMT enable the assessment of the stiffness at the intermediate strains, the estimation of the type of the tested soil as well as its density. The last parameter is especially important for the assessment of shear modulus G_0 in Eq. (2.2).

4. Results and interpretation

To present the results of the measurements a following notation, corresponding to Fig. 4, was assumed. The results achieved with the MASW method in subsequent five measurement lines are denoted as MASW1 to MASW5. The subsequent test points are reflected by numbers in brackets from (1) to (5). The invasive tests (SDMT or DPL) or borehole tests (BH) are marked by an abbreviation of the test and a number of the test point (e.g. SDMT2, BH2). Seismic dilatometer test, as mentioned before, is a combination of seismic downhole and classical dilatometer tests. In order to distinguish the results obtained in seismic dilatometer measurements from the ones obtained in dilatometer, they are denoted as SDMT and DMT respectively.

The research started with the seismic MASW tests, which were performed along lines shown in Fig. 4. For each line a vertical cross-section was performed showing the field of the shear wave velocity V_s in the form of contour lines. Places of any disturbances in propagation of seismic waves were interpreted as seismic anomalies. In these areas the wave velocities are small, which may be caused – as mentioned in Sec. 2 – by loosening, saturation or destructuring of the soil medium. The studies revealed the occurrence of shallow seismic anomalies in the surface zone at a depth of 2 to 7 m, in almost all the cross-sections. Also, deep seismic anomalies were registered in two sections.

Figures 7a and 7b illustrate shear wave velocity V_s in cross-sections 2-2' and 3-3' respectively (Fig. 4). A general tendency, i.e. the increase in the wave velocity with depth, was observed in all the investigated cross-sections. Also shallow and deep seismic anomalies are shown in Fig. 7. The deep anomaly, shown in the lower part of Fig. 7b, coincides with the mine transporting gallery. Shallow seismic anomalies are subjected to invasive SDMT tests. Their locations are shown in Figs. 7 and 4. The deep anomalies, located well below 20 m, are beyond the reach of the rig used in the investigation.

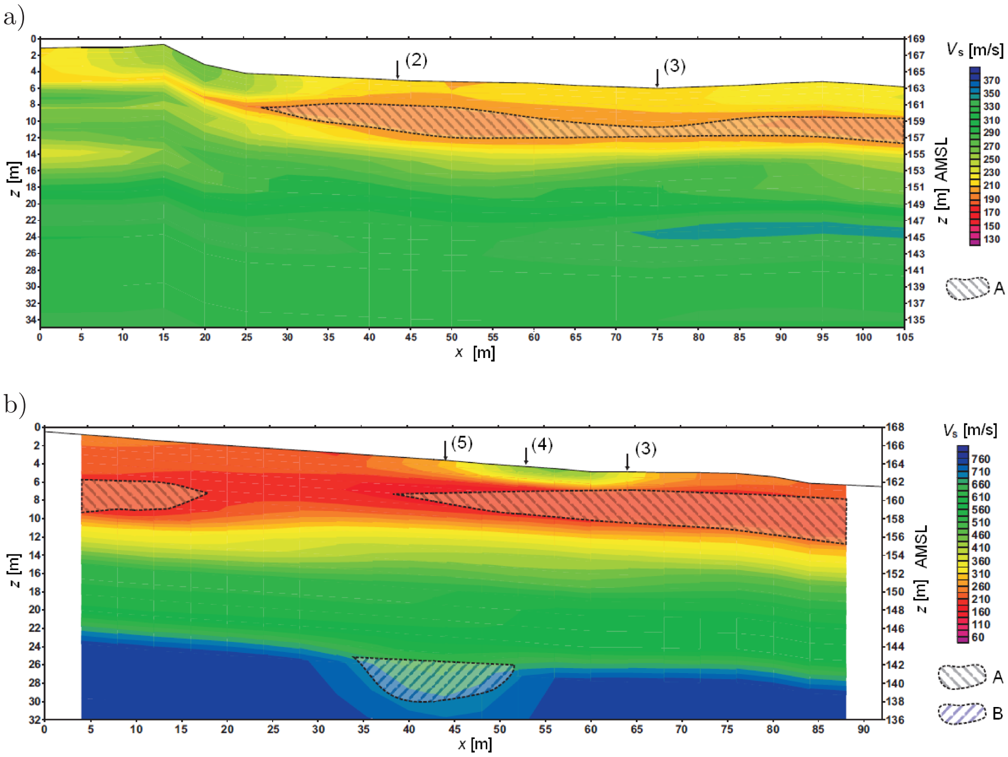

Fig. 7. Velocities V_s of shear waves obtained with the use of MASW method in the following cross-sections: a) 2-2', b) 3-3', modified after [3]. Notations: A – shallow seismic anomalies; B – deep seismic anomalies; (2), (3), (4), (5) – places where invasive SDMT tests were performed.

The points for the invasive tests (Fig. 4) were selected on the basis of the following premises. Testing point (2) was selected in the immediate vicinity of the existing building, over a mine transporting gallery. In this place, the thickness of the shallow anomaly is significant, and the people working in the building reported the lowering of the land. Points (3) and (4) are situated at the junctions of measuring lines respectively: MASW2 – MASW3 and MASW3 – MASW4. These places were convenient for checking the correctness of the determined wave velocities V_s. Point (5) was selected at the junction of a measurement line MASW3 and transporting gallery over a revealed deep seismic anomaly. Also an attempt was made to perform an SDMT test in the developed sinkhole – point (1), Fig. 4. However, no results were obtained to a depth of 4 m due to a considerably loose state of soil (the values of soil resistances were too low for p_0 to be read). Additionally, the vibrations of the rig generated during the penetration threatened a collapse of the track and caused the compaction of the examined soil changing its state, so the test was abandoned.

Let us focus now on the results of the SDMT tests. Figure 8a presents the results of a standard dilatometer test, which recorded pressures p_0 and p_1. Figure 8b presents the results of a seismic test which recorded the values of wave velocity V_s directly. To assess the desired soil stiffness, the type of soil and its density need to be evaluated.

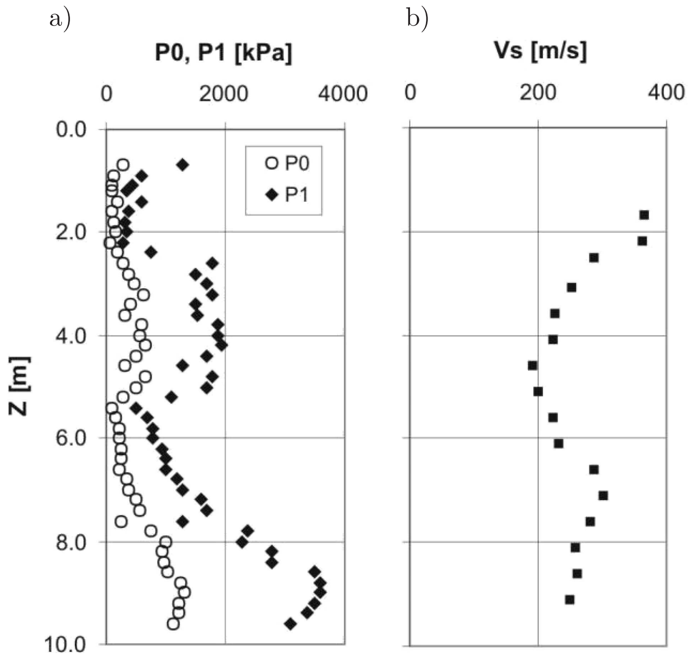

FIG. 8. Standard results of SDMT tests in point (2): a) pressures p_0 and p_1, b) velocities V_s.

Material index I_{DMT} is calculated from the measured values p_0 and p_1 from Eq. (3.1), which indirectly allows specifying the type of the tested soil. Figure 9a shows the obtained soil type profile in test point (2). According to the applied interpretation of the measurements, the profile reveals sands, i.e. from silty sands (siSa) and fine sands (FSa) to thin layers of medium sands (MSa).

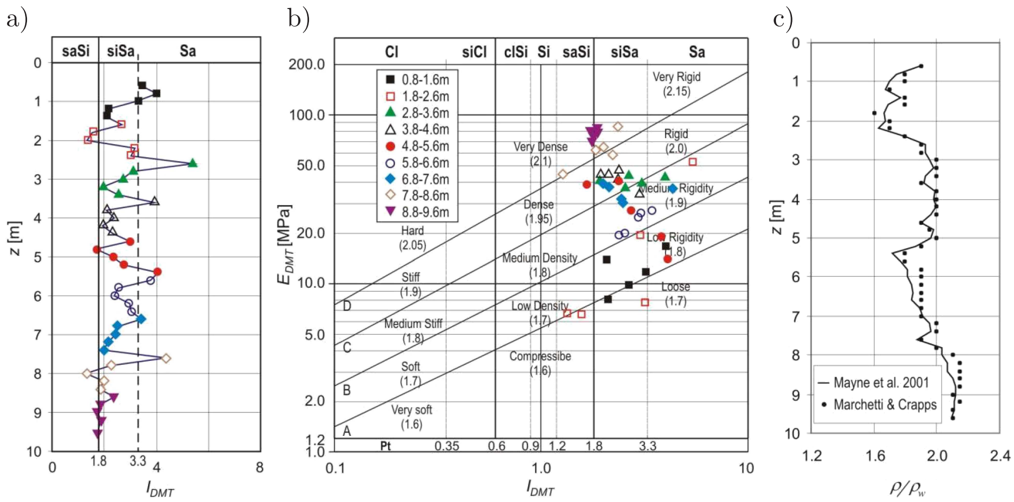

FIG. 9. Interpretation of DMT2 test: a) soil type classification, b) the classification of soil due to the type and state, c) the relative density of soil.

To check the correctness of the type of soil determined indirectly with the use of DMT, two control boreholes BH1 and BH2 were drilled to depths of 10 m and 6 m respectively. In both boreholes the soil profiles consisted of a layer of anthropogenic soils (slag) occurring at depths from 0 to approx. 1.3 m, overlapping Quaternary sands (from silty sands to sands with gravel). Figure 10 shows the results of laboratory sieve analysis of two samples of sand taken from borehole

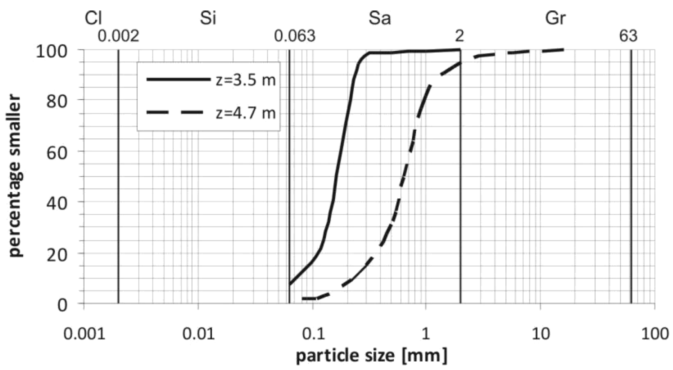

FIG. 10. Particle size distribution in BH2.

BH2 at depths of 3.5 and 4.7 m. The comparison of the results obtained in direct measurements and the ones obtained through the interpretation demonstrates acceptable consistency in the determined type of soil.

In a classical approach of DMT data reduction, it is possible to determine the soil state and the relative density of soil from material index I_{DMT} and dilatometer modulus E_{DMT} with the use of a special diagram developed by Marchetti & Crapps in 1981 (see e.g. [13]). The diagram presented for data obtained in test point (2) is shown in Fig. 9b. Numbers in brackets refer to relative densities of soil, i.e. soil bulk densities related to the density of water. The diagram is not convenient for computer processing of the measurements (e.g. test data reduction by spreadsheet or computer program). It is much more convenient to use Eq. (3.3). The comparison of both methods is shown in Fig. 9c. High consistency of the obtained results justifies the use of a simpler methodology proposed by Mayne *et al.* [15] to assess the soil density.

Let us now focus on a comparison between the results obtained from seismic noninvasive and invasive methods. In order to compare the velocities of waves obtained in two independent MASW interpretations with the values obtained from direct measurements in the SDMT tests, the results for the vertical soil profile obtained in point (3) are presented in Fig. 11a. At this point, the MASW measurement lines intersect, and all the three measurement results should be identical.

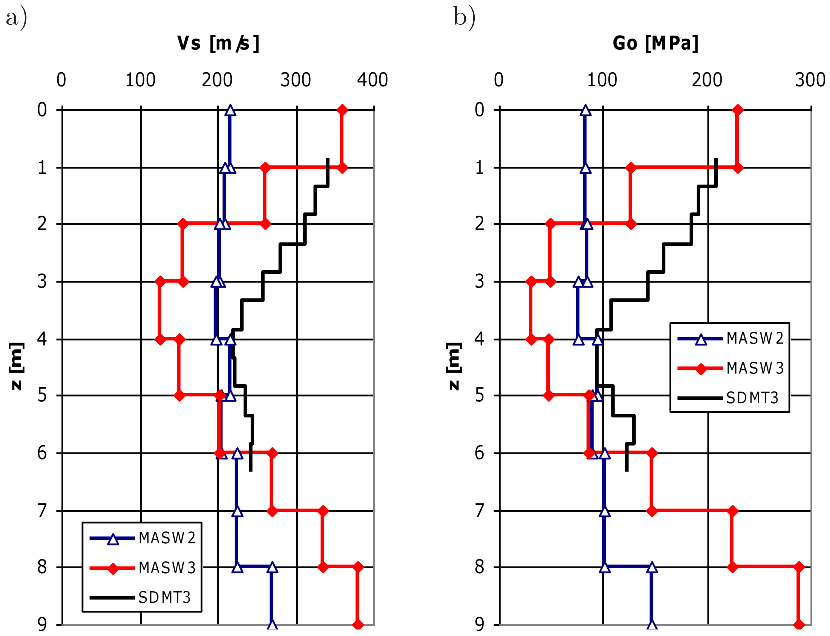

Fig. 11. Comparison between test results obtained in MASW and SDMT test in point (3): a) shear wave velocity V_s, b) shear modulus G_0.

In Fig. 11, results obtained in MASW2 and MASW3 are denoted with triangular and diamond markers, and results obtained in SDMT3 – with a line with no additional markers. A significant quantitative discrepancy of results is clearly visible when comparing the values of wave velocity V_s, Fig. 11a. In the subsurface zone, the results obtained in MASW3 are greater than the ones obtained in MASW2 by nearly 70%, while, at the same time, they are lower by 35% at a depth of 3–4 m. In addition, the MASW results are not consistent, in terms of quantity, with direct measurements obtained in SDMT3. As far as stiffness G_0 is concerned, the results vary even more (Fig. 11b). It obviously stems from Eq. (2.2). However, it should be stated that there is qualitative consistency in the examined points, which means that the subsurface noninvasive MASW tests are well suited for the preliminary assessment of soil stiffness variation. At the same time, the absolute value of stiffness must be calibrated by point-invasive direct measurements.

Both used seismic methods (MASW and SDMT) showed a reduced stiffness of soil at a depth of several meters. The stiffness of soil in these places may even be several times lower than the one on the surface. Due to the risk of sinkholes, the following questions arise:

- whether the reduced stiffness results from natural variability of glacial soil (glacier acted like a huge bulldozer at terminal moraines)?
- whether they are caused by other natural geological processes?
- or, finally, whether the cause is anthropogenic and results from mining activities?

Let us start by trying to answer the last question. To this end, we compare the scope of changes in soil stiffness in the developed sinkhole and the neighbouring area. For the reasons mentioned at the beginning of the chapter, it seems that uniform research methods cannot be applied to compare the investigated area (the soil in the sinkhole consists of too loose sands to obtain the DMT readings). Significantly less accurate tests DPL were performed in the sinkhole. These tests, however, allow for the determination of constrained modulus M. The same parameter may be determined in standard DMT tests, which were performed in the remaining points.

Modulus M was calculated on the basis of the DPL test with the use of a correlation presented by Stenzel and Melzer [22]. Figure 12a shows the value of modulus M inside the sinkhole, in places marked as (1–1) and (1–4). The obtained values of M are very low. The results of the DPL1.1 test, which was performed in the central part of the sinkhole, indicate almost zero stiffness of soil at depths 4–5.5 m. In this zone, it is likely to find unstable soil structures or even voids, as a single dynamic probe impulse caused the device to go downward into the soil by over a dozen centimetres (normally several blows are needed for 10 cm penetration).

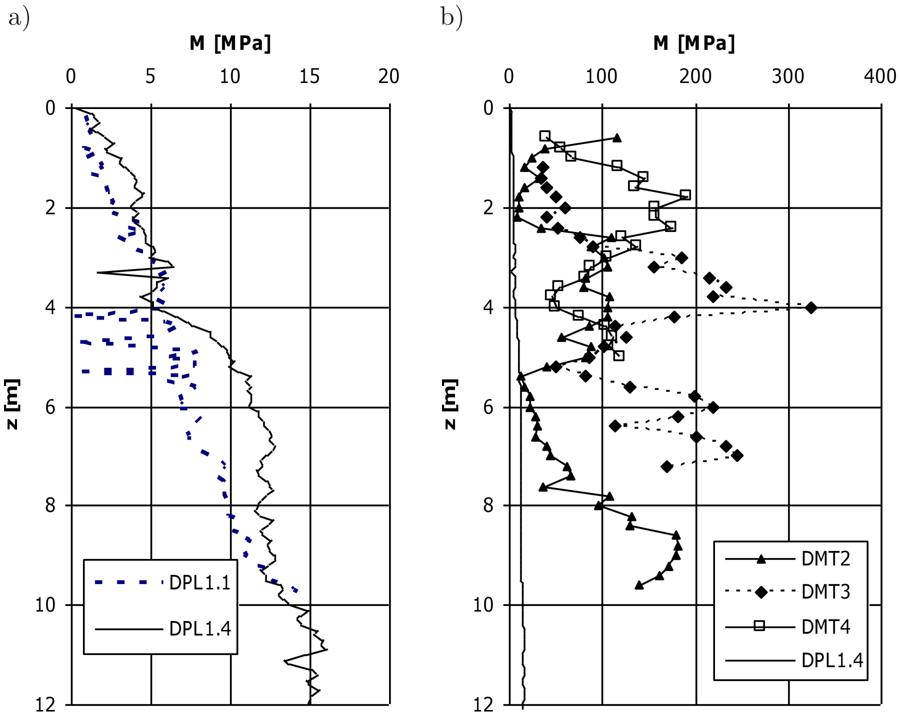

Fig. 12. Constrained modulus M: a) inside the sinkhole in points (1.1) and (1.4), b) in undisturbed test points (2), (3), (4) and inside the sinkhole in point (1.4).

Figure 12b presents the values of M obtained in tests: DMT2, DMT3 and DMT4 in relation to the DPL1.4 results. The values of stiffness obtained from dilatometer tests were calculated from formula (3.2). A comparison of the results of DMT performed outside the sinkhole with the results of DPL made inside the sinkhole allow for a conclusion that the process of sinkhole development is associated with a reduction in stiffness M by one order for the analysed soil. Although diagram DMT2 is tangent to the graph DPL1.4 at two points, which results from natural variability of glacial sands, the values outside these points differ significantly. Thus, it is not the sinkhole formation processes which are responsible for the shallow anomalies.

At this point MASW tests are very advantageous. The reduced values of velocity of shear waves in the subsurface zone observed in a single profile achieved in SDMT (Fig. 8b) do not stand out too much against the occurring fluctuations and their values remain within the range resulting from the natural variability of soil. However, MASW images of wave velocities fluctuation seen in the entire cross-section (Fig. 7) show some regularities. Horizontal distribution of zones of reduced stiffness indicates that it may be caused by the influence of either climate or groundwater. It is also an additional argument that the shallow anomalies do not result from mining activities.

Let us return to the results in Fig. 8. We can see an inconsistency between the pressure values obtained in the mechanical tests (Fig. 8a) and the values of wave velocity obtained in the seismic tests. In areas where resistances caused by the expansion of the membrane are large (depth from 2.5 to 5.0 and below 7 m), the observed values of wave velocity were low – indicating a poor initial stiffness of soil. According to the intuition, large values of resistance of the membrane should be accompanied by large values of V_s. It is worth noting that Fig. 8 shows the pure readings of two types of measurements performed during one test and they are not based on interpretation or approximate correlation functions. The results relate to test point (2). Let us examine the results for other points.

Figure 13 shows a comparison between the calculated values of constrained modulus M in test points (2) and (3). The values of the modulus for intermediate strains were determined from Eq. (3.2), on the basis of classical DMT test, with the assumption that $\nu = 0.2$ for very small strains (Kumar and Madhusudhan [10]). The results are marked by a dotted line with open diamond marks. The calculated values based on the seismic tests within very small strains are denoted as $M_{\max}$ and presented by a continuous line with filled square marks. The values of stiffness obtained in classical DMT tests are much lower than those obtained in seismic SDMT tests. This is understandable, given the range of strains occurring in both tests – compare the degradation of stiffness G with increasing strains, Fig. 1. High value of M determined at a depth of 4 m (Fig. 13b) is probably due

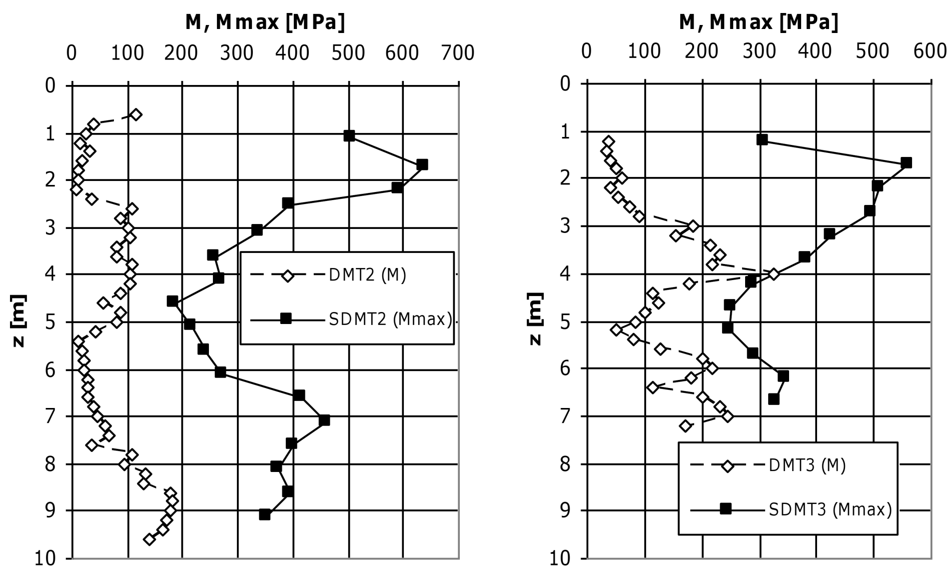

Fig. 13. Comparison between moduli M and $M_{\max}$ in test points (2) and (3).

to resistance caused by a large grain of gravel or a stone. However, the nature of changes of modules M and $M_{\max}$ is inconsistent with the intuition, in particular for depths within 1–2.5 m, where the minimum values of M correspond to the maximum values of $M_{\max}$. This inconsistency repeats in points (2) and (3), as well as in other points which are not shown in the drawings.

Very large values of $M_{\max}$ (large values of V_s) registered in the subsurface zone may be caused by natural geological processes. It may be caused by a low degree of saturation of the sand directly under the surface of the ground (see Fig. 3) or cementation of sand grains present at this depth range which also results in the increase in stiffness (Fig. 2a). In principle, sand that is stiffer is also stronger. However, according to Schnaid [18], cementation influences stiffness more then strength. Additional research and analysis would be required to resolve the causes of inconsistencies occurring in Fig. 13.

5. Summary and conclusions

The paper presents the assessment of the state of sandy soil in the area threatened with sinkhole formation. The aim of the work was to determine the state of soil and to search for soil loosening zones where sinkhole processes may develop. The investigated area reveals a complex geological structure. It is a terminal moraine zone with significant glacitectonic deformations.

Since the surface of the soil (to a depth of several meters) consists of glacial sands of varying index of density, obtaining a representative soil sample for testing its state in a laboratory would be technically very difficult and expensive. It was decided to perform *in-situ* tests. The stiffness parameters were adopted as a substitute measure of the state of soil: shear modulus G, and alternatively constrained modulus M.

Stiffness was evaluated in noninvasive seismic MASW tests and invasive SDMT tests. MASW tests allow the determination of the velocity of shear waves basing on an inverse analysis, hence the absolute value of the determined wave velocity may be considerably inaccurate. After determining zones that are likely to contain loosened soil, control invasive SDMT tests were performed in these places. SDMT consists of a classical dilatometer test and a seismic test which directly measures the velocity of shear waves. A classical dilatometer test allows the determination of modulus M for intermediate strain, from correlation functions, and soil bulk density ρ. Knowing the density and wave velocity, it is possible to calculate the initial shear modulus G_0 for very small strains, for which soil behaves as a linear-elastic material. To compare stiffness for intermediate and very small strains, G_0 is converted into constrained modulus $M_{\max}$.

MASW tests provide a kind of a tomography of soil. Each single test allowed the evaluation of a large volume of soil. The possibility of viewing the results in the entire cross-section was its great advantage, despite the fact that the accuracy of the determined wave velocity was not the highest. Shallow and deep seismic anomalies were identified. Subsequent invasive tests were carried out only for shallow anomalies to improve the accuracy.

The applied methodology of research proved to be efficient. Adopting stiffness as a measure of the state of soil resulted in the fact that the MASW and SDMT tests complemented each other. A major limitation was the use of a light rig (100 kN capacity and own weight below 1 t). On the one hand, the high local degree of density of the examined sands caused large resistances while inserting the probe, but on the other hand, the anthropogenic soil lying on the surface prevented the effective anchoring of the rig.

By comparing the values of modulus M obtained for shallow anomalies to the ones measured inside the sinkhole, it can be concluded that shallow anomalies do not result from sinkhole processes. The regular, horizontal distribution of the anomalies visible on the MASW cross-sections confirms this conclusion. The anomalies are likely to be caused by surface phenomena (climate influences).

A distinct inconsistency was revealed between stiffness for intermediate and very small strains within depths 1 to 3.5 m. At these depths, large values of $M_{\max}$ are accompanied by low values of M and the results seemed to be uncorrelated. A possible reason for this inconsistency can be cementation processes in the subsurface zone or capillarity forces resulting from low saturation.

The above conclusion calls into question the accuracy of methods for estimating G_0 on the basis of classical DMT test, which extrapolate the results within the strain range of 0.05–0.1% onto the range of very small strains – less than 0.001%. A review of these methods was presented by Briaud and Miran [4]. The methods should be complemented by a restriction that they may be invalid for cemented or unsaturated soils.

The observed discrepancies allow drawing a more general conclusion, not only regarding sinkholes. The stiffness of soil in a subsurface zone is extremely important while designing foundations for engineering objects. The picture of stiffness obtained with the use of only one device may be incomplete or even misleading. In the analysed case, the constrained modulus obtained from measurements carried out with the use of a classical dilatometer demonstrates small values, whereas the one obtained on the basis of seismic tests presents large values. This is, obviously, connected with a different range of strains produced in each test. The final recommendation is as follows: due to the complex behaviour of natural soils, it is advisable to use various tests, enabling the cross-correlation of measured results. When stiffness is tested, it is important to get data for both very small strains and intermediate strains of the soil.

A more detailed explanation of the discrepancies observed in the subsurface zone would require additional tests and analyses.

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