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The modification of the dynamic properties of cohesive soil resulting from triaxial consolidation

Zmiana parametrów dynamicznych gruntu poddanego konsolidacji w aparacie trójosiowego ściskania

Abstract

This paper describes the laboratory examination of the dynamic parameters of cohesive soil together with an analysis of these parameters using artificial intelligence. The analysis yielded the propagation velocity of shear wave V_s and the dynamic Kirchhoff modulus G obtained during the soil tests in the triaxial stress apparatus. The investigation was conducted using ***bender*** elements. The artificial neural networks trained on data obtained from the test were used for the further analysis.

Keywords: dynamics, soil parameters, wave propagation

Streszczenie

Artykuł łączy ze sobą badania laboratoryjne parametrów dynamicznych gruntu spoistego wraz z ich analizą z użyciem metod sztucznej inteligencji. Rezultatami analizy są wartości prędkości propagacji fali ścinającej V_s oraz dynamicznego moduł Kirchhoffa G uzyskane podczas badania gruntu w aparacie trójosiowego ściskania. Do otrzymania tych parametrów posłużyło wykorzystanie elementów ***bender***. Do analizy użyto sztucznych sieci neuronowych uczonych na danych pozyskanych z badań.

Słowa kluczowe: dynamika, parametry gruntu, propagacja fal

1. Introduction

This paper presents a project aimed at verifying the effect of various parameters on the state and quality of soil under complex states of stresses. In the study, soil samples dug from a depth of approximately 5–6 m were utilized. They were underlying directly on a layer of clay, which, in the past, was a source of material for a brickyard in Zesławice, near Nowa Huta. Triaxial stress apparatus was used in this experiment.

By exploiting the porous quality of the soil, one may study its physical properties. When using the soil material for construction purposes, the parameters relating to strength are a significant issue. By studying the stresses in the soil, one may assess the friction between the soil particles and the liquid that make up the soil. This friction is an internal friction and in a number of soil constitutive models, it is defined as the internal friction angle. This level of friction is a function of number of soil parameters (e.g. size, shape, degree of roundness or angularity of particles, mineral composition, and the geological origin or history of the loading in the soil deposits).

As far as modelling of the soil behaviour is concerned, two parameters are particularly important: the internal friction angle and the cohesion. The internal friction angle depends on the porosity, water content and the water pressure in the pores. The cohesion may be in the form of soil resistance to external forces dependent on the forces attracting the particles and the liquid component accumulated in the soil. The quantity of liquid is important mainly in cohesive soils, and may depend on particle size, water content, and the origin of mineral composites. If the water content of soil decreases, the cohesive forces in the soil become larger; the strongest cohesive forces are found in soil that is in a dry and hard state. However, the degree of internal friction and cohesion also depends on the state of soil and how the soil structure acts when the pores are filled with water. It is commonly known that the in-situ conditions are different depending on whether the zones are aerated or saturated. In the aeration zone, soil is a mixture of particles (mineral and/or organic), gases (air) and liquid (mostly water) confined in the pores. It is therefore a three-phase mixture which may be partially saturated – this occurs in the case of soil located above the water table. It is natural to assume that soil below the water table would be completely saturated. In such conditions, one assumes that the soil becomes a two-phase mixture, i.e. it is composed of mineral and/or organic particles and liquid which is primarily water, completely filling the pores of the soil.

2. Laboratory tests of soil

Laboratory test methods for basic static soil parameters, where the relationship between the stress and strain is determined, are based on the equipment and procedures recommended by European and ISO standards e.g. PN EN ISO TS 17892:

- direct and residual shear machine (total value of Φ_c and c_c);
- oedometer (oedemetric modulus of compressibility);
- cone penetrometer (shear strength – C_u);

- triaxial stress apparatus (depending on the methodology: total value of Φ_c and c_c or effective value of Φ' and *c'*).

Dynamic testing may be conducted with appropriately equipped triaxial stress apparatus. This type of apparatus may be supplemented with additional elements such as a resonant column or, as was used in this study, bender element test equipment.

The research conducted with the use of the triaxial stress apparatus may be performed in various ways. In fact, one may distinguish between three basic test methods depending upon how excess water is drained from the soil sample:

- unconsolidated undrained (UU) method – the shear strength of the sample is received with respect to total stress without a preliminary soil consolidation. During basic research, the pore pressure of water in the sample is not usually measured and the test is performed without draining the water from the sample. Tests using this method are performed when the investigated soil will carry the loading from the building for which the live load has to account for over 70% of the total loading. From these studies, the strength parameters are determined on the basis of the total stress (Φ_{uu} and c_{uu}).
- consolidated (isotropic) undrained (*CU, CIU*) method – soil samples are initially consolidated. During fundamental testing, the measurement of pore pressure of water in the sample is carried out and the test is performed without draining the water from the sample. The effective stress is then calculated as the difference between the total stress and the pore pressure. This method is used when the loading imposed on the building varies from 30% to 70% of the total loading; such conditions may occur when after the construction of a building, the operational loading is anticipated in a relatively short period of time. Therefore, on the basis of this test, one may determine total and effective strength parameters (Φ_{cu} and c_{cu} or Φ'_{cu} and c'_{cu}).
- consolidated (isotropic) drained (*CD, CID*) method – this test method is similar to the *CU* test; it is performed very slowly and shear samples are subjected to a pre-consolidation procedure. During basic investigation, followed by a continuous flow of water from the sample, the test speed is adjusted so that the value of pore pressure measured in the sample is at a constant level. This method is used in cases where the anticipated operational load of the building does not exceed 30% of the total load, and the construction time is long enough to develop full consolidation of the substrate. Therefore, on the basis of this test, one may determine the effective strength parameters (Φ'_{CD} and c'_{CD}).

The significant difference between the parameters achieved by diverse testing methods is shown in Fig. 1 [1, 2]. You can see a distinct change in the position of the Mohr's circles, when considering different types of stress.

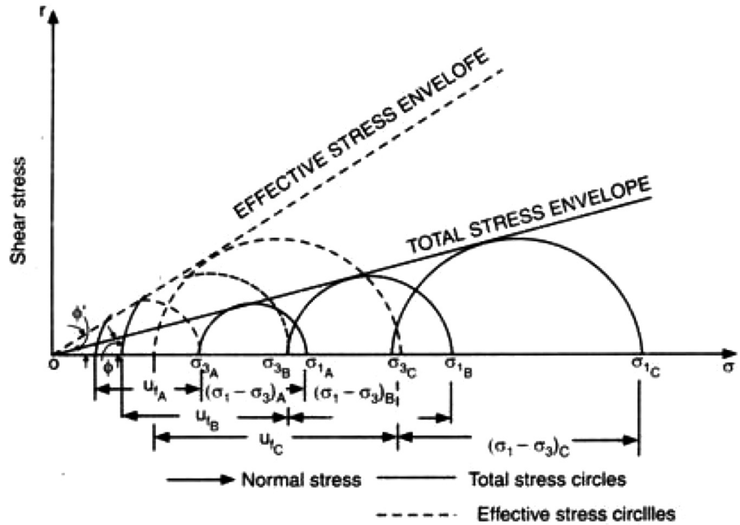


Fig. 1. Mohr's circles obtained for various types of tests in the triaxial stress apparatus for the cohesionless soil

3. The application of bender elements in the chamber of the triaxial stress apparatus

The study of dynamic stiffness or wave propagation in soil by bender elements has been in use since the 1970s. This method is carried out due to its ease of execution and accurate results for small deformations in the soil. Bender elements are made of piezoelectric ceramic material and are used to measure the propagation of the longitudinal wave and the transverse (shear) wave of a soil sample under laboratory conditions. This test involves inserting two piezoelectric elements, i.e. transmitter and receiver, into the soil and applying tension of the excitation wave in transmitter and measurement of wave propagation by the receiver. Piezoelectric bender elements are shown in Fig. 2.

The methodology for bender element tests is based on several simplifying assumptions that underlie all laboratory testing methods of wave propagation in porous medium [6]:

- the deformation induced in the soil by the transmitter is very small; therefore, the response to dynamic excitation of soil is in the elastic range;
- the wave induced by the transmitter in the soil sample is a shear wave – this means that only the transverse wave travels at shear wave velocity and the course of the shear wave is equal to the distance between the transmitter and receiver;
- the soil sample is an infinite body in a given configuration, i.e. all the reflected waves in the sample arrive later than the wave coming directly from the transmitter; the soil sample is treated as a homogeneous and isotropic medium.

Fig. 3a shows the bender elements mounted to the soil sample and the set located on the base of the triaxial stress apparatus; Fig. 3b shows the polycarbonate chamber of the triaxial stress apparatus and instrumentation during the test. The pore pressure gauge is visible in the front.

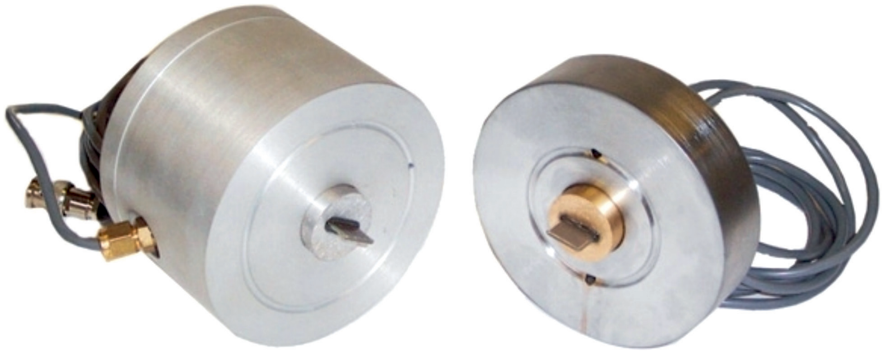


Fig. 2. Cylinders with visible piezoelectric elements used to study wave propagation under laboratory conditions in triaxial stress apparatus

a) b)

Fig. 3. Bender elements, mounted on the base of the apparatus (a) and located directly in the polycarbonate chamber during triaxial stress testing (b), used for the testing of wave propagation under laboratory conditions

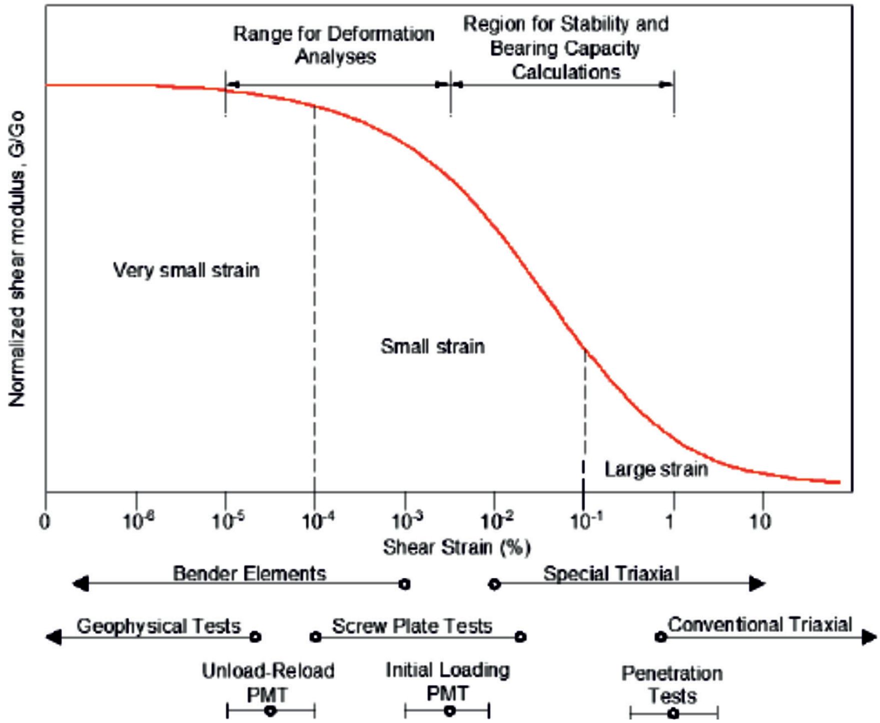


Fig. 4. The ranges of the Kirchhoff modulus obtained using different testing methods [7]

Fig. 4 shows dynamic Kirchhoff module range obtained with the application of various laboratory test methods. Bender elements allow one to obtain reliable results in the range of deformations less than 0.001%; therefore, the method is particularly appropriate for small elastic deformation of soil. In Fig. 4, laboratory methods are supplemented by applying other methods for the field investigation of dynamic soil parameters. These types of tests take place in perspective investment areas or areas of seismic or paraseismic wave action [3–5].

As far as the propagation of the transverse wave is concerned, a *peak to peak* method is applied in which running times of vibration displacements are obtained from the excitation waveforms and the waveform registered at the receiver. Assuming that the medium is an elastic body, one may use the basic relationship between the velocity of propagation of shear wave and shear module G as:

$$G = \rho\, V_s^2 \tag{1}$$

where:

ρ – soil density [kg/m^3],

V_s – shear wave velocity [m/s].

Fig. 5 shows what the waveforms induced in the source and recorded by the receiver look like. The *peak to peak* method allows you to find the duration of the waveform and the known height of the sample allows us to obtain the value of velocity of the propagation of transverse wave V_s.

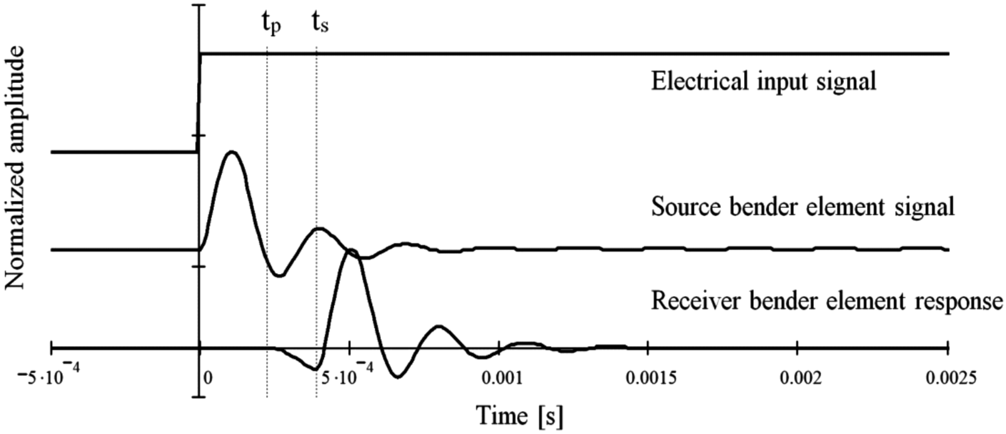


Fig. 5. Example of time history of wave propagation registered during bender element test

4. Results of the laboratory tests

Under laboratory conditions, a cohesive soil sample obtained *in-situ* from the former brickyard in Zesławice, near Nowa Huta was analysed with triaxial stress apparatus using bender elements. The soil bulk density was $\rho = 2300$ kg/m^3, and the average water content was $w_n = 15.5\%$. The sample was subjected to a consolidation process in the triaxial stress apparatus with different paths of consolidating isotropic stress values in the range of [0, 300] kPa. The studies were carried out at different soil consolidation times. During the tests, neither drainage water from the sample nor pore water pressure measurement was possible.

There were three phases of action that could be distinguished as follows:

Phase 1 – type UU conditions: during this phase, the soil underwent a regular isotropic consolidation process from 0 to 150 kPa. Over a short period of time, tension increased with an increment of $\delta = 25$ kPa. Simultaneously, shear wave propagation V_s and Kirchhoff modulus G were determined. After reaching a consolidation stress level equal to 150 kPa, the soil was left for 96 hours to consolidate.

Phase 2 – it was assumed that CU (CIU) conditions were present. Therefore, it made further study of shear wave propagation V_s and Kirchhoff modulus G with value of consolidation stresses increasing by $\delta = 50$ kPa up to 300 kPa. Again, the soil was allowed to fully consolidate, this time for another 48 hours.

Phase 3 – unloading phase, during this phase, the rate of the over-consolidation ratio (OCR) changes, isotropic stress consolidation was reduced to 150 kPa. Thus, the OCR changed from 1 to 2.

Fig. 6 presents the actual time history of wave propagation. It is obtained by the propagation in the soil sinusoidal wave with a frequency of 1 kHz at voltage of 5 V. The soil was consolidated with the stress of 25 kPa. The results obtained during the experiment are shown in Table 1.

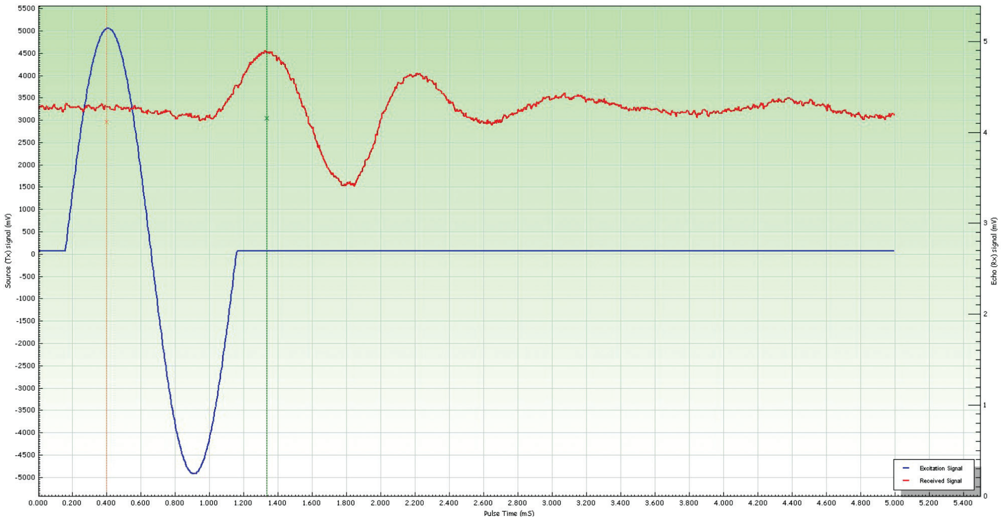


Fig. 6. Actual time history of propagation of transversal waves received during the measurements of the soil with the isotropic consolidation of 25 kPa

Table 1. The velocity of transversal waves and shear module G obtained in laboratory tests of soil

Time of consolidation [hrs.]	Consolidation stress [kPa]	Vs [m/s]	OCR [–]	G [MPa]
0	0	80	1	14.7
1	25	100	1	23.0
1	50	120	1	33.1
1	75	140	1	45.1
1	100	152	1	53.1
1	125	170	1	66.5
1	150	185	1	78.8
96	150	220	1	111.3
96	200	240	1	132.5
96	250	250	1	143.8
96	300	255	1	149.6
144	300	270	1	167.7
144	250	255	1.2	149.6
144	150	245	2	138.1

Fig. 7 presents a graph of modulus G as a function of the consolidation stress in phase 1, i.e. under normal consolidation. The increase of the consolidation stress follows a significant rise of the dynamic Kirchhoff modulus G. However, this increase is non-linear. With a six-fold increase of consolidating stresses (from 25 kPa to 150 kPa), modulus G increased by a factor of approximately 3.5.

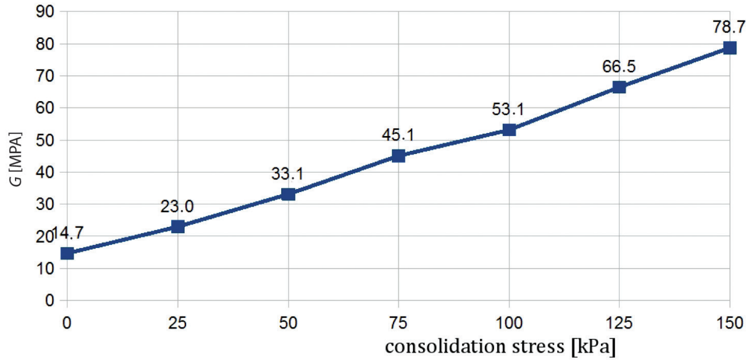


Fig. 7. Modification of the dynamic Kirchhoff modulus during normal soil consolidation (*UU*)

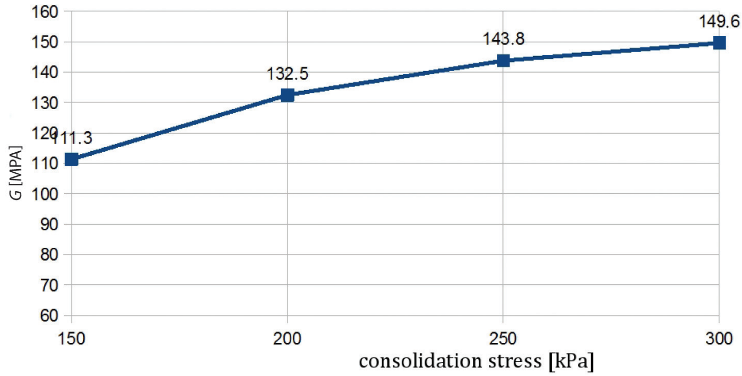


Fig. 8. Modification of the dynamic Kirchhoff modulus during normal soil consolidation (CU)

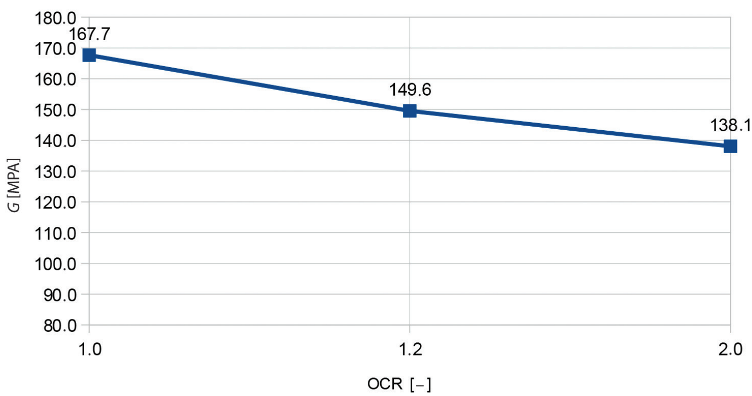


Fig. 9. Modification of the dynamic Kirchhoff modulus of over-consolidated soil during the unloading phase

Fig. 8 presents the behaviour of the Kirchhoff modulus after 96 hours of isotropic soil consolidation under 150 kPa of stress. In the remaining steps, the consolidation stress increased and both the velocity of the transverse wave and the dynamic modulus G were measured until the level of the stress reached 300 kPa. In this phase of the experiment, the consolidation stress increased by a factor of two, while the shear modulus G only increased by about 35%. A clear change that is non-linear is visible from the figure. Fig. 9 shows the change in the dynamic modulus G during the unloading phase when the soil was over-consolidated with a stress level of 300 kPa. OCR changed twice, while the shear modulus was reduced by approximately 18 percent. Fig. 10 shows the results obtained under different consolidation conditions for a stress level of 150 kPa. One may notice from the figure that the history of the load affects the dynamic Kirchhoff modulus. It may vary, according to the values obtained in this study, it even nearly doubles. This is particularly important when dealing with shallow layers of soil. In such situations, the process of modelling dynamic behaviour results from, for example, the interaction between a vehicle and the road, the substrate becomes very complex.

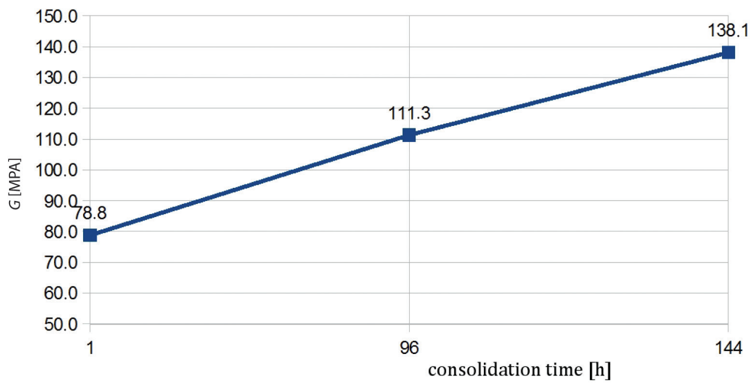
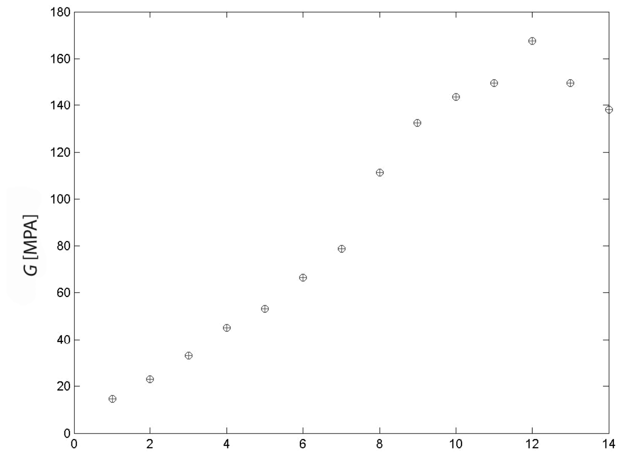


Fig. 10. Modification of the dynamic Kirchhoff modulus for soil consolidated with a stress level of 150 kPa

5. Analysing the data using artificial neural networks

Artificial neural networks (ANNs) were applied for the analysis of experimental results. During the study, different networks have been tested, that are learned on the data by the method of error backpropagation [8, 9]. The main objective of the application of this tool was to obtain the possibility of complete fit of the learning outcomes of the research results. Unfortunately, this objective was not met using the methods of convergence of results using several different gradient methods. Both, the method of Polak-Ribiére with algorithm of conjugate gradient and quasi-Newton BFGS method, which requires storage of Hessian matrix [10] did not give proper convergence. Briefly, in both cases, the solution got stuck in a local minimum of error despite the use of different variants of the network topology and number of neurons. The best method proved to be the resilient backpropagation (rpop) method [11] as it managed to obtain an expected convergence (Fig. 11). The data used for analysis is shown in Table 1.



Sample number

Fig. 11. Comparison of the results of network training (rprop) with the experimental data ('o' experimental results, '+' ANN results)

There are three parameters which are inputs to the network: the time of consolidation; the level of consolidation; the OCR ratio. The output data is the result describing the propagation velocity of shear wave V_s and the dynamic Kirchhoff modulus G. Both values are linked; therefore, the network also has the ability to assess the correlation between them according to formula (1). The network consisted of two layers of hidden neurons. In the present case, namely 61 and 31 neurons with arc-tangent activation function were applied.

6. Tests and ANN simulations results

Subsequently, ANN is applied in order to simulate the behaviour of the soil under different load conditions and variable coefficients of OCR. Table 2 shows the results obtained for the simulations of the cohesive soil behaviour under isotropic stress consolidation of 150 kPa. The results of further simulations are also shown in a graph of shear modulus as a function of the OCR coefficient. Fig. 12 presents a change in the dynamic modulus G during the unloading phase in cases where the soil is over-consolidated. For stress levels of 150 kPa, when the OCR coefficient increases, the dynamic Kirchhoff module at first increases and then decreases. The difference is minimal.

Table 2. SSN test results for soil consolidated stress of 150 kPa and various parameters of OCR

Time of consolidation [hrs.]	**Consolidation stress [kPa]**	**Vs [m/s]**	**OCR [–]**	**G [MPa]**
144	150	247.3	1.0	140.8
144	150	247.9	1.2	141.7
144	150	249.9	1.7	143.1
144	150	246.4	1.9	139.8
144	150	245.0	2.0*	138.1

* refers to outputs obtained from measurements and reproduced by ANN (Fig. 11)

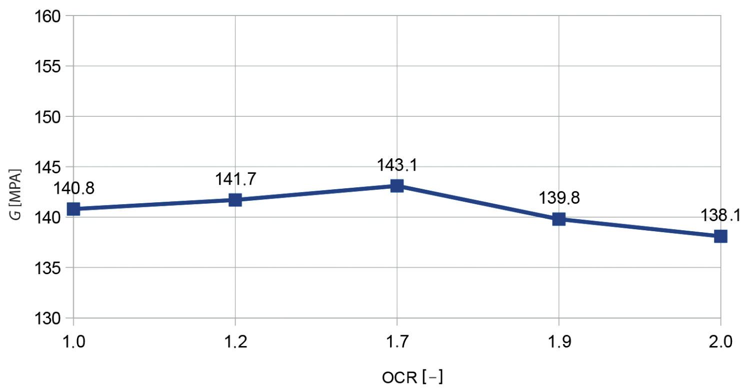


Fig. 12. Modification of the Kirchhoff dynamic modulus for soil consolidated stress of 150 kPa and various OCR

Table 3. The results of SSN for soil consolidated stress of 250 kPa and various OCR

Time of consolidation [hrs.]	**Consolidation stress [kPa]**	**Vs [m/s]**	**OCR [–]**	**G [MPa]**
144	250	256.3	1.0	151.2
144	250	255.0	1.2*	149.6
144	250	244.2	1.7	136.5
144	250	243.0	1.9	135.0

* refers to outputs obtained from measurements and reproduced by ANN (Fig. 11)

Table 3 presents the results obtained for the stress consolidation of 250 kPa. Fig. 13 exhibits a change in the dynamic modulus *G* during the unloading phase, in cases where the soil is over-consolidated. For stress levels of 250 kPa when the OCR coefficient increases, Kirchhoff modulus *G* decreases significantly. The difference of Kirchhoff modulus *G* in this range of OCR is about 12% in comparison to the initial value.

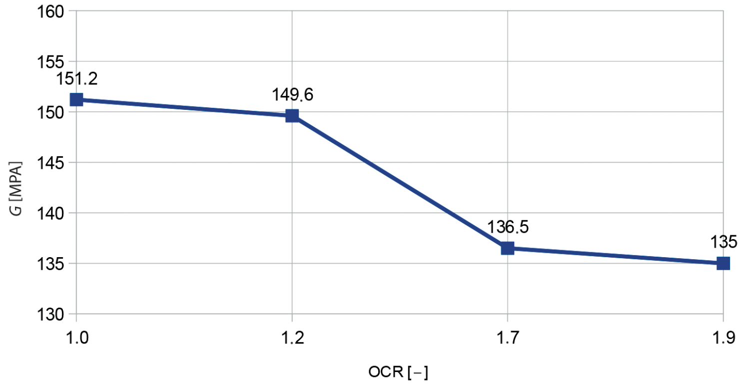


Fig. 13. Modification of Kirchhoff dynamic modulus for soil consolidated stress of 250 kPa and various OCR

Table 4 shows the results obtained for the consolidation stress at 180 kPa. Such stress is not taken into account in training samples; therefore, the results are entirely dependent on the quality of the generalisation of the neural network.

Table 4. The results of ANN generalisation for soil consolidated stress of 180 kPa and various OCR ratios

Time of consolidation [hrs.]	**Consolidation stress [kPa]**	**Vs [m/s]**	**OCR [–]**	**G [MPa]**
144	180	247.7	1.0	141.3
144	180	248.3	1.2	142.0
144	180	251.5	1.5	145.9
144	180	249.2	1.7	142.6
144	180	245.9	1.9	138.3

Fig. 14 presents changes in the dynamic modulus G during the unloading phase when the soil is over-consolidated with a stress level of 180 kPa. In this case, the OCR increases, while the Kirchhoff modulus first increases and then decreases. Fluctuations are limited to a very short range. The process of the function describing the nature of the changes is similar to that which was previously shown for stress levels of 150 kPa.

Table 5 shows the behaviour of the network for completely unknown values of consolidation stress and OCR ratios. The results of ANN generalisation are very good – in this case, the neural network, despite the low amount of the training data, does not display unrealistic values of shear wave propagation velocity V_s and Kirchhoff dynamic modulus G.

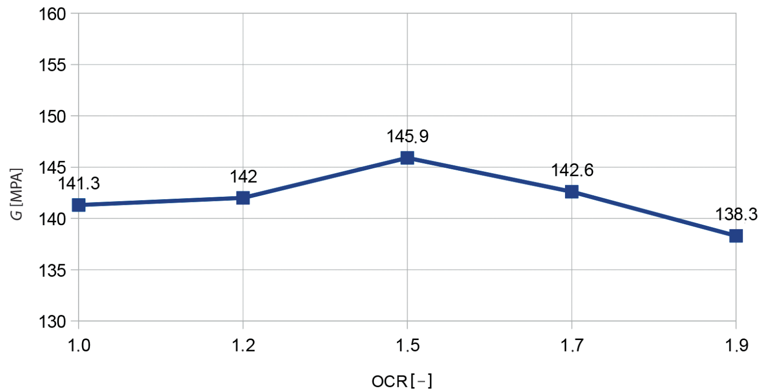


Fig. 14. Modification of Kirchhoff dynamic modulus for soil consolidated stress of 180 kPa and various OCR

Table 5. The results of ANN generalisation for the cohesive soil consolidated with different stresses and different OCR

Time of consolidation [hrs.]	Consolidation stress [kPa]	Vs [m/s]	OCR [–]	G [MPa]
144	180	251.5	1.5	145.9
144	220	245.9	1.7	138.5
144	240	243.2	1.9	135.2

7. Final remarks

This work brings together the laboratory examination of the dynamic parameters of cohesive soil together with an analysis using an artificial intelligence method. The results of the analysis are the propagation velocity of shear wave V_s and the dynamic Kirchhoff modulus G obtained during the soil tests in the triaxial stress apparatus. These results are obtained using bender elements. The artificial neural network trained on data obtained from the test was used for further analysis. The ANN is very well trained and it reproduced the output results accurately. Then the network is used to simulate other states of soil. Despite the small database of results collected from the experiment, ANN simulates appropriately dynamic soil parameters. The *in-situ* tests (i.e. SCPT, SASW, CSWS) complement and verify the dynamic parameters obtained during the laboratory tests and broaden their range in various states of stress and strain. In the future, it is planned to further study soil behaviour in terms of undrained and drained consolidation and take into account other soil parameters from the measurements (e.g. pore pressure) in the other soil types.

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