

SHEAR STRENGTH OF PTOLEMAIDA LIGNITES

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Abstract

This paper aims to investigate the shear resistance behavior of the Ptolemaida lignite. An attempt is made to determine the laws governing the development of the shear strength of lignite material and to compare it with results of research concerning the same material in Greece.

This work refers initially to the geology of the region, to the natural and chemical properties of its lignites and to its quality compared to that of the Megalopolis lignite.

The results of this research are listed below. Initially, the physical characteristics were determined: water content, unit (apparent) weight, unit weight, organic content. The preload tension (O.C.R) was found to be of the order of 20 Kg/m³ according to Casagrade. All of the samples examined were from the North Field of the Ptolemaida Main Lignite Ground Field.

In order to determine the laws applicable to the shear strength of lignite under consideration, triaxial compression trials were conducted on isotropic consolidation specimens with undrained conditions. Resistance parameters were investigated in a wide range of pressures due to the history of loads. Systematic measurement of the rising pore pressure, deformations and proper load speed selection was performed. We outlined Mohr circles, deviatoric stress diagrams, pore pressure, and other useful diagrams to draw conclusions.

In conclusion, we can deduce that the lignite material examined can generally be treated as a preloaded material in order to solve the problems of Soil Mechanics and foundations, but also at the same time take into account certain differences, which are explained on a case-by-case basis.

Key words: lignites, preload stress, deviator of stresses, pore pressure, deformations, over consolidation ratio

1. INTRODUCTION: LOCATION AND METHOD OF SAMPLING

Lignite samples from the North Sector of the Main Field of Ptolemaida Lignite were used to conduct this survey. The exact place of sampling was the location (tile) 1980 in Excavator E₃. The specimens were generally cut from the sloping side in cubic-shaped pieces, 30-40 cm per side and were immediately placed in double plastic bags suitably sealed to maintain their natural moisture during transport to the laboratory.

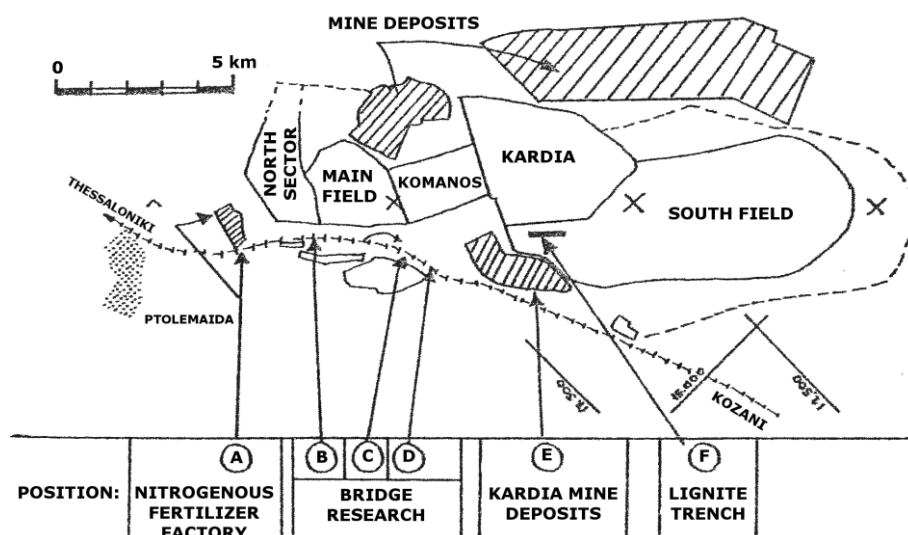


Figure 1. Area of lignite fields of Ptolemais.

	BROW	
Clay	2 meters	
Type of material	Thickness	
Lignite	15cm	20
Marl	35cm	
Carbonated marl	30-35cm	19
Lignite	30cm	
Carbonated marl	35cm	
Lignite	110cm	
Clay	20cm	18
Lignite	50cm	
Clay	35cm	
Lignite	15cm	
Carbonated marl	10cm	
Lignite	15cm	
Carbonated clay	25cm	
Lignite	15cm	
Carbonated marl	15cm	17
Lignite	30cm	
Carbonated marl	15cm	
Lignite	130cm	
Clay	15cm	
Lignite	10cm	
Clay	15cm	
Lignite	30cm	16
Clay	15cm	
Lignite	70cm	
Carbonated marl	15cm	
Lignite	50cm	
Carbonated marl	20cm	
Compacted lignite	75cm	15
Lignite	45cm	
Carbonated marl	15cm	14
Lignite	90cm	
Lenticular marl	15cm	
Lignite	35cm	13
	FLOOR	+662

Figure 2. Sample slopes cut in the North Sector.

During the tests, the vertical load imposed, the deformation of the specimen, and the increasing pore pressure were monitored. Readings were taken beyond the "maximum value" of the deviating tension (σ_1 - σ_3). After the fracture of the specimen, the type of fracture as well as the formed breakage plane were checked.

Table 1. Main stresses and over consolidation ratio in triaxial trials performed

Num	σ_3	O.C.R.
1	2.00	10.00
2	5.00	4.00
3	8.15	2.45
4	16.31	1.23
5	20.39	1.00
6	24.69	1.00
7	28.54	1.00
8	30.58	1.00
9	32.62	1.00

Triaxial compression devices, pore pressure measurement, sample volume change, etc. for both standard pressures (up to 10 Kg/cm²) and for high pressures (up to 32 Kg /cm²), which were used for this research in the laboratory, were manufactured by Wykeham Farrance.

2. CONCLUSIONS

Ptolemaida's earth lignite is an organic preloaded sediment with high water content. Its physical characteristics change at different locations in the basin, depending on the various mineral impurities and mainly depending on its organic content.

Triaxial trials with undrained conditions (CU) were performed to investigate shear strength. The research has shown that the material under the influence of various charges - behaves in almost the same way as clays with some discrepancies as indicated on a case-by-case basis.

The average value of natural moisture is in the range of 150° C when drying at 60° C while when drying at 105° C the figures are in the order of 159.50%. Achieving stable (dry) weight required four to five days at 60° C and one to two days at 105° C.

Its liquid apparent weight is of the order of 1,165 gr/cm³.

For the determination of the unit (specific) weight the leech method was applied, while the mean value determined was about 1,56 gr/cm³. This value is of the same order with corresponding values of other known lignites. The small differences observed are due to the different degree of carbonation or even to the mineral content. So, the more xylithic is the lignite, the lower the value of its specific weight is.

The determination of the organic content of the lignite material was made by use of the loss method by burning at 550° C and 775° C according to DIN). The percentage in organics by burning at 550° C is 86% and by burning at 775° C is 88.5%. By examining the empirical relationship of Skempton and Petley (1970) with that of Mac Farlane (1969) - which associates the organic content with the specific gravity of mainly organic soils - the relationship between Skempton and Petley was found that for the lignite under consideration gives satisfactory results towards a quick determination of the specific gravity, but with the percentage of organics as determined by burning at 550° C.

One of the main factors that influence the strength of a terrestrial material is its load history (preload). To determine the preloading pressure (P_c), routine compressibility tests with $\Delta p/p=1$ and $T=1$, were carried out on both the lignite material and adjacent rocks with horizontal stratification. The pre-load pressure determined by Casangrade was of the order of 20 Kg/cm^2 .

Table 2. Cumulative triaxial test results.

σ_c	OCR	$(\sigma_1 - \sigma_3)_f$	q_f	p_{1f}	u_f	σ_{1f}	σ_{3f}	u_f/σ_c	σ_{1f}/σ_c	σ_{3f}/σ_c	A_f	$(\sigma_1 - \sigma_3/\sigma_c)_f$	$\varepsilon_f \%$
2,00	10,00	22,17	11,08	11,26	1,83	22,34	0,18	0,92	11,17	0,09	0,082	11,48	5,13
5,00	4,00	24,07	12,03	12,44	4,60	24,47	0,41	0,92	4,90	0,08	0,19	4,81	5,99
8,15	2,45	21,09	10,52	11,71	6,96	22,22	1,19	0,83	2,72	0,14	0,33	2,58	5,73
16,31	1,23	20,79	10,40	15,11	11,60	25,51	4,71	0,70	1,56	0,29	0,55	1,27	9,82
20,39	1	22,51	11,26	15,69	15,97	26,95	4,44	0,78	1,32	0,22	0,71	1,10	9,97
24,67	1	29,69	14,85	19,34	20,18	34,11	4,49	0,82	0,38	0,18	0,67	1,23	11,19
28,54	1	33,04	16,52	22,49	22,57	39,01	5,97	0,80	1,37	0,21	0,67	1,18	13,77
30,58	1	33,27	16,64	20,39	26,82	37,03	3,75	0,88	1,21	0,12	0,81	1,08	14,56
32,62	1	36,92	18,46	23,80	27,28	42,26	5,34	0,84	1,30	0,16	0,71	1,07	13,34

The laws governing shear resistance and its growth pattern were investigated with samples from the North Sector of the Main Field of the Ptolemaida Lignite mine. The material was saturated with high natural moisture, low apparent weight and highly preloaded. Despite these characteristics, it develops relatively high shear strength. From triaxial stress trials in undrained conditions (CU), the envelopes of the Mohr circles were determined. For active stresses, the values of the resistance parameter from the modified Mohr envelope (active stress paths) were determined because it was virtually impossible to engrave envelopes in the corresponding Mohr circles. For the active stresses, there are two distinct branches of the envelope with a resistance parameter $\varphi'=21,8^\circ$, $c'=5.8 \text{ kg/cm}^2$ for the preloaded segment and $\varphi'=36,9^\circ$, $c'=0$ for the smoothly charged (normal consolidated) segment (Figure 4). For the preloaded segment, the c , φ values are completely calculated because many possible envelopes can be charted. The points are not clearly arranged around a straight line, which may be due to the heterogeneity of the material. This shape of the envelope is similar to that of clay.

For total stresses, the preloaded segment is linear with zero practical slope ($\varphi=0^\circ$) to the preload pressure, while after the preload pressure it remains straight but with a higher inclination. Contrastingly, in clays, the curvature of the preloaded portion is observed in the low-pressure range. The intersection of the two straight sections corresponds to the preloading area as it does with the clays. In the preloaded section $c=11.2 \text{ kg/cm}^2$, $\varphi=0$, while in the smoothly charged (normal consolidated) section $\varphi=18,43^\circ$ and $c=2.6 \text{ kg/cm}^2$.

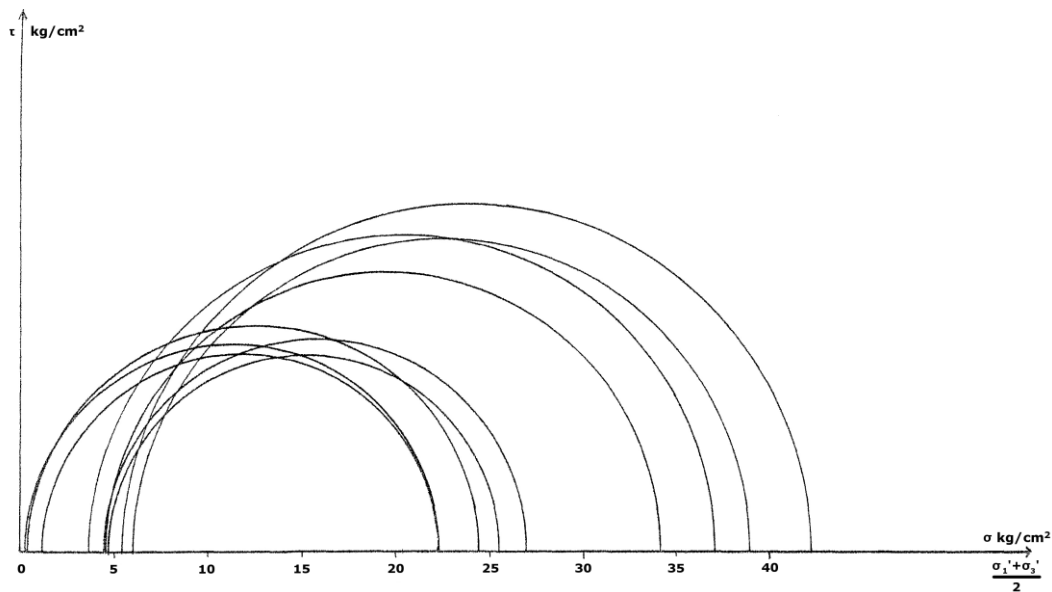


Figure 3. Circles of effective stresses by Mohr.

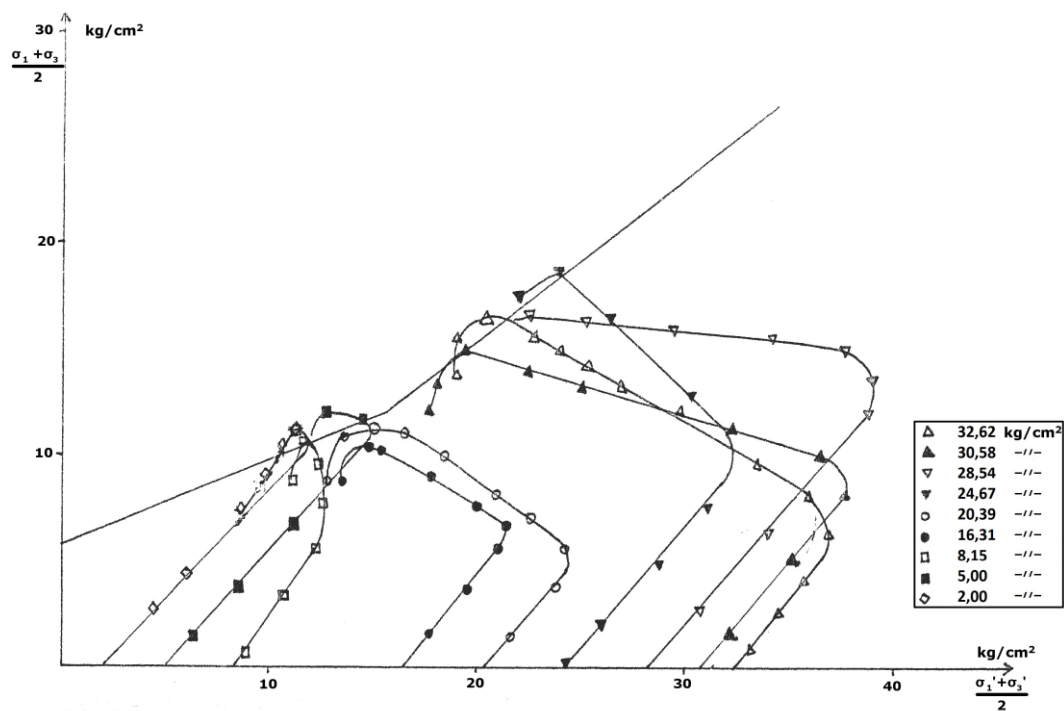


Figure 4. Envelopes and effective stress paths

From the stress-strain diagrams it was found that the deviator stress increases with lateral pressure for the same reduced deformation, along with the increased deformation at break-point with the increase in lateral pressure. This observation is amplified by the Ladd diagrams, and indeed, it can be said that the additional vertical pressure (stresses divergence) is proportional to lateral pressure for the smoothly loaded (normal consolidated) specimens. The charts $\{(\sigma_1 - \sigma_3)/\sigma_c - \epsilon\}$ for the pressure region beyond the preload, converge in a beam while the preloading effect is strong in the area before the preload, as shown by the graphs $\{(\sigma_1 - \sigma_3)/\sigma_c - \epsilon\}$. The behavior of the lignite material compared with the clays is obvious, as is evident from the following figures:

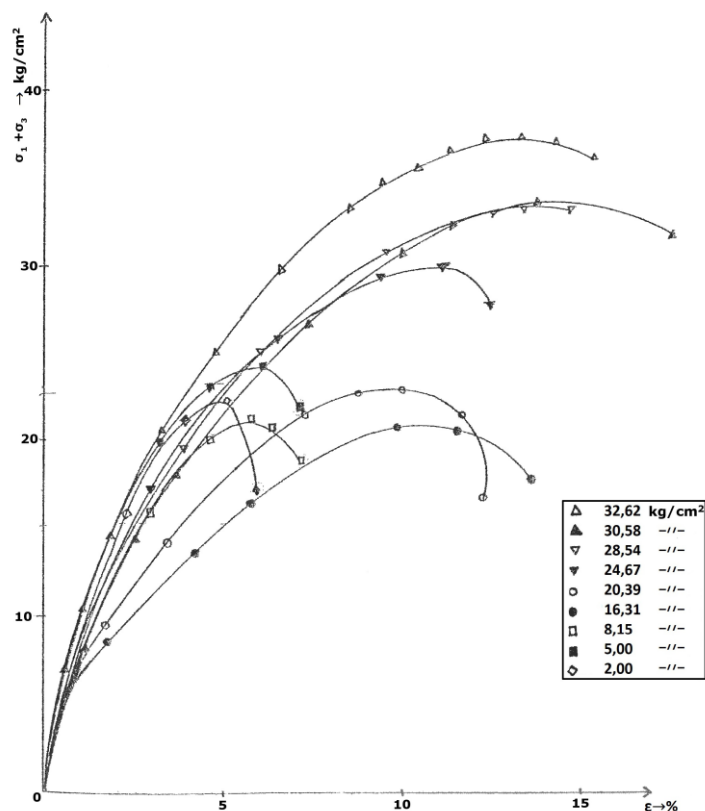


Figure 5. Variation of additional vertical pressures (deviatoric stresses) with deformation.

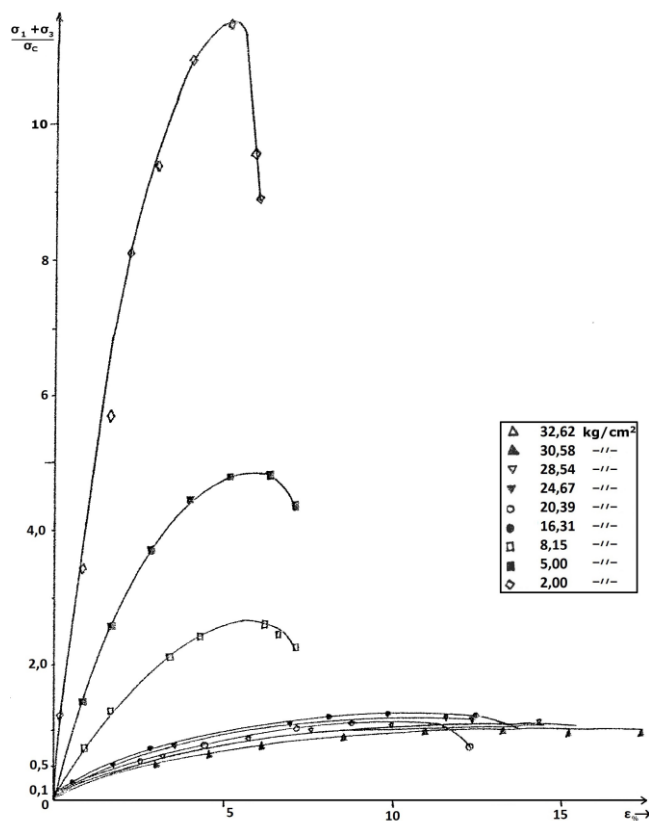


Figure 6. Variation of "undimensional" additional vertical pressures with deformation.

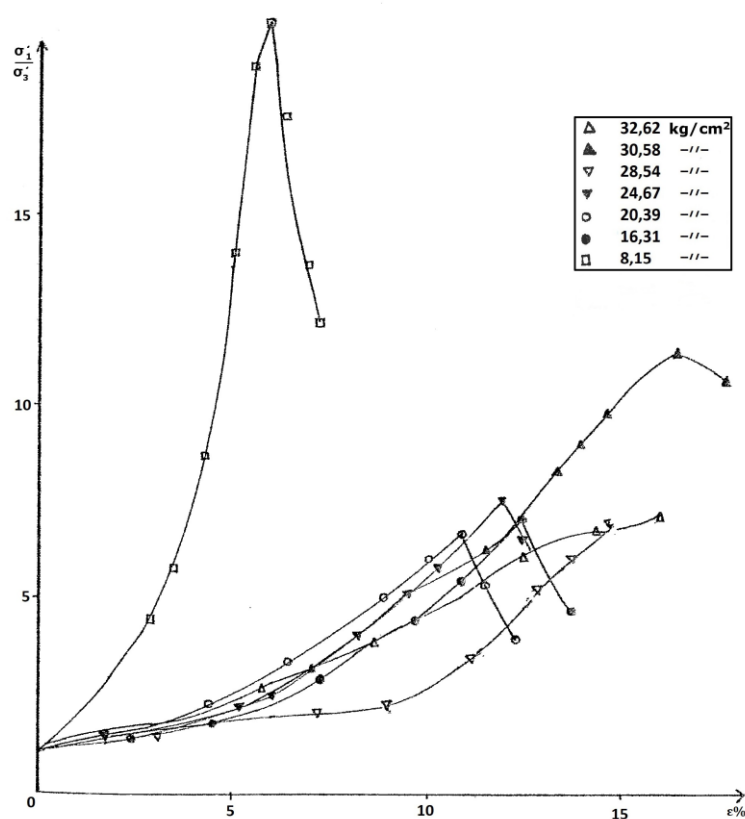


Figure 7. Variation of the ratio of effective main stresses with deformation.

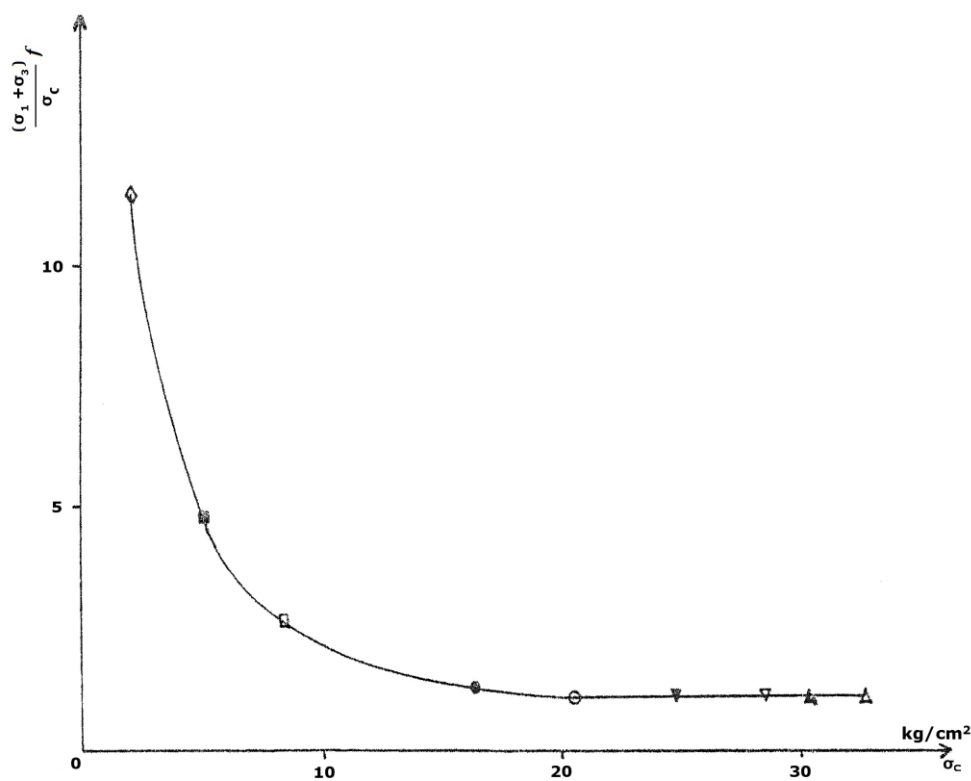


Figure 8. Development of "undimensional" additional vertical pressure at break-point with lateral pressure

It is remarkable that two forms of fracture were observed. One in a vertical plane and one in an inclined plane. Different forms must be attributed to the structure of the lignite material where the presence of wood materials is evident. We observed lag in the development of pores pressure in the early stages of the test, which is attributable to the low permeability of the material. However, it has not been possible to relate the development of pressure of pores to the way the specimens fractured.

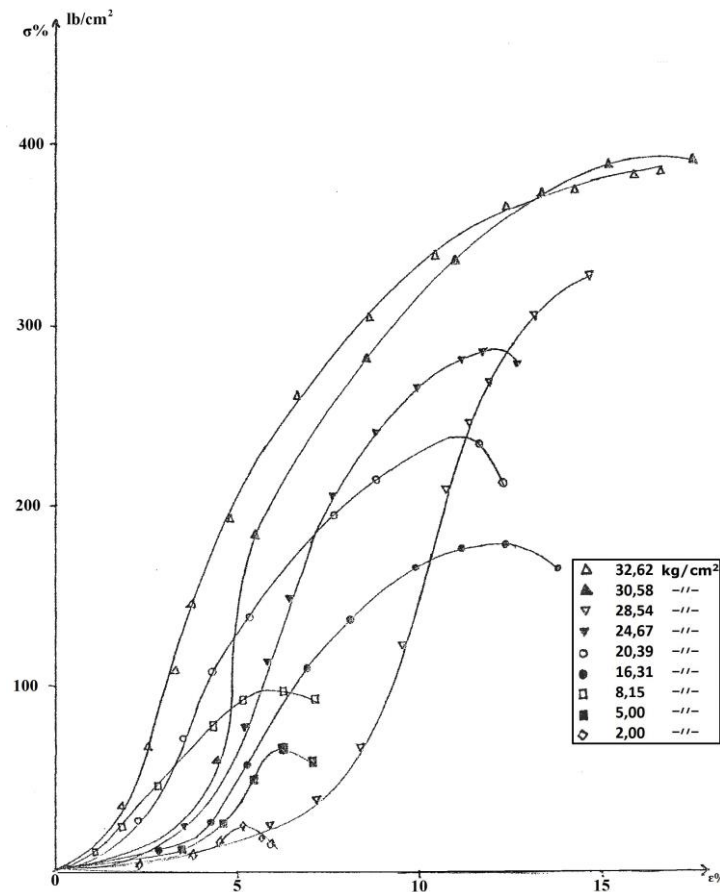


Figure 9. Development of pore pressure with deformation.

After crushing, we observe a continuous increase of the pores pressure in the smoothly charged (normal consolidated) specimens, while in the preloaded ones there is a decrease, albeit not significant, i.e. we have a behavior analogous to that of the clays. From the undimensional diagrams of pore pressure- deformation, the Ladd principle for clays was found to be valid. The patterns of smoothly charged (N.C) specimens tend to coincide, and the preloaded diagrams are removed from the previous ones as the preload ratio (O.C.R) increases. Ultimately, the resemblance to clays is greater for smoothly loaded (N.C) than for preloaded (O.C.R) specimens. The values at break point A_b for the preloaded specimens are larger and are all positive (as opposed to the clays) whereas for the smoothly charged (N.C) lignite specimens the values of A_b are of the same order as the ones for smoothly charged (N.C) clays. By increasing lateral pressure, the value of break point increases by approaching 0,8. The patterns of effective stress paths, for all smoothly charged (N.C) and preloaded (O.C.R) samples are similar to those of smoothly charged clays (type III according to the well-known Wissa and Ladd suffix) and this is due to the way pore pressure develops.

In summary, we emphasize that because the shear resistance of lignite is not expressed unambiguously with two parameters, it is preferable for the researcher to refer to the field of trends of interest. We still think that this work on lignite material should continue with a series of CD (consolidated drained) tests which although somewhat time-consuming, we believe will help to better investigate the issue.

3. CONCLUSIONS OF EARLIER RESEARCH

Professor A. Anagnostopoulos (1980), conducting three-direction unbalanced crushing tests in saturated lignite specimens from the lignite material of Megalopolis, reports that the change in the undrained shear strength C_u , followed an elliptical change, depending on the orientation of the sample.

Hasapis, Kazantzis (1985) in their diploma thesis on Ptolemaida lignite found that the undrained shear resistance C_u of lignite does not follow an elliptical change. It can be considered in practice that the change of undrained cohesion C_u is cyclical and, without great error, we can accept that the lignite material examined is isotropic.

N. Charalambidou (1986) in her thesis studied the anisotropy of shear resistance of Ptolemaida lignite by triaxial unconsolidated undrained (UU) tests. Test specimens obtained from cubic-shaped samples were used in 4 different directions (vertical, horizontal, and inclined at 45° clockwise and counterclockwise). Shear strength was found to be following an elliptical law of shear strength change, so the material was found to behave similarly to the lignite material of Megalopolis.

K.J. Rosengren (1961) studied the compressibility and shear strength of Yallourn and Morwell's lignite in Australia. From the tests that were made, he observed that the natural humidity - logarithm of the pressure diagram is similar to the corresponding diagram of a preloaded (O.C.R) saturated clay.

Rosengren (1963) then investigated the lignites of the Victoria region of Australia (Yallourn mines, Yallourn North and Morwell mines), concluding that lignites consolidation in the same way as clay and that Terzaghi's consolidation theory can be applied to them.

Trollope and K.J. Rosengren (1963), exploring the lignite breakdown, investigated their structure and strength (lignites in Yallourn and Morwell, Australia). They found that the behavior of the material under compression is similar to clay soils and that the principle of active stresses in Terzaghi applies.

E.T. Brown (1963) supplemented the study of the "Soil Mechanical Behavior of Lignites" in the Morwell Australia area, by expanding and completing the previous work of Rosengren and Trollope. To this end, triaxial stress trials were conducted in various places of lignite such as terrestrial, pollen, etc., as well as tests to determine the physical characteristics. In summary, the results of his work were as follows:

Natural humidity: from 157-179%

Apparent unit weight: from 1.10-1.15gr / cm³

Angle of internal friction

(A) for preloaded material, undrained test: $\phi' = 40^\circ - 48^\circ$

(B) for smoothly cured material, undrained test: $\phi' = 46^\circ$

Cohesion: Depending on the preload load, from 2,8-4,9 Kg/cm²

PH Trollope - KJ Rosengren - ET Brown (1965) completing the Latrobe Valley study of the State of Victoria in Australia, carried out compressibility and triaxial stress tests on new lignite samples of Yallourn, Yallourn North and Morwell. Concerning the lignite behavior during compression, it was found that this is ultimately analogous to that of clay.

However, it should be noted that there is a reservation on how to modify strength with the direction, due to the small differences in strength in different directions that are comparable to the uncertainties that exist due to the heterogeneity of the material in estimating strength tolerance in a direction. Therefore, it is recommended to continue this research and carry out a large series of tests in each direction, taking care that the orientation of the specimens is exactly ascertained as the direction of stress is likely to affect the directional strength estimation.

There are, however, two ways in which we can assess the parameters that contribute to the shear resistance of the coherent materials as well as the imaging of its problems in the laboratory.

The first and most well-known and widespread is to attempt physical measurements using appropriate laboratory tests. Then, in combination with an appropriate criterion for failure, we can obtain the analytical endurance parameters (c , ϕ).

Finally, a second way of estimating the parameters that contribute to the shear strength of coherent materials is the following: We examine the mechanisms involved in the deformation of the soil under the influence of applied load and we deduce the physical parameters responsible for the resistance. This is also the most interesting way, since, by considering the mechanisms that are responsible for the development of shear resistance, we can come up with very useful conclusions about resistance in different directions.

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