

**UNIVERSITY OF CYPRUS
UNIVERSITY CAMPUS
GEOTECHNICAL INVESTIGATION REPORT**

For

SITE 3

POTENTIAL SITE FOR FUTURE DEVELOPMENT

Client: UNIVERSITY OF CYPRUS

***Nicosia, January 2004
SH Soil Engineering Ltd***

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1. INTRODUCTION

This report gives account of the geotechnical investigations carried out in the area of Site '3' of the Campus of the University of Cyprus. The report describes the field and laboratory work carried out and includes all the results of the in-situ and laboratory tests performed. It gives an account of the engineering properties of the strata encountered and typical values of allowable bearing capacity and settlement of foundations.

The work was undertaken on behalf of the Office for the Development of the University Campus of the University of Cyprus, after the award of the relevant Contract to SH Soil Engineering Ltd by the University of Cyprus, letter reference Π.K. 115.1.6.28/CDO 028/3.

The University Campus is located 5.5 km South-East of Nicosia center in the broader area of Athalassa, Figure 1. The location of Site '3' area investigated is shown in Figure 2.

The Geological Survey Department and others previously carried out geological investigations, in the broader area of the University Campus and in specific areas where various structures and works have been constructed. The work carried out previously is described in the following reports:

1. University of Cyprus, Geological – Geotechnical Report, July 1994. (Geological Survey Department)
2. Supplementary exploratory boreholes drilled at the proposed site of the University of Cyprus, April 1995 (Geological Survey Department).
3. University of Cyprus- Detailed Geological Investigation, June 1998 (Geological Survey Department)
4. Site Investigations for the proposed buildings of the University of Cyprus- Volumes I and II, September 1998 (Geoinvest Ltd)

The purpose of this geotechnical investigation was to establish the geological profile of the site investigated, obtain the geotechnical characteristics and engineering properties of the strata encountered, and record the water table profile at the site.

The site investigated is a potential site for future development and will be used for the construction of various buildings. There are no details of the proposed buildings available at this stage.

2. FIELD WORK

The Field Work carried out consisted of the drilling of five boreholes having a total depth of 47.20m, the recovery of disturbed and undisturbed soil samples, in-situ testing, measurement of the water table and taking water samples for chemical analysis.

Drilling was carried out using the rotary 'crawler' type rig, Casagrande C6 type, and a high capacity air compressor for removing the drilled material.

2.1 Boreholes

The five boreholes drilled in this area were numbered N9 to N13 and their respective depths were 7.50, 7.50, 10.50, 10.50 and 11.20m. Their approximate position is shown on the Borehole Location Plan, Fig 2. The boreholes had a diameter of 135 mm and the method of drilling used was a combination of rotary and percussive drilling using the 'down-the-hole hammer', with air and sometimes air and water flushing to remove the cuttings of the soil. With this method all boreholes are cased during the drilling process.

The soil layers encountered in the boreholes are described in the Borehole Records, Figs. 4 to 8, Appendix A. Photographs of the samples of the soil layers encountered are presented in Figs. 9 to 11, Appendix A.

2.2. Sampling

Four types of soil samples were recovered from the boreholes during drilling:

- (a) **Disturbed** representative bulk samples were recovered from the soil cuttings brought to the surface by air flushing. These samples are suitable for identifying and describing the soil layers encountered and carrying out classification tests, such as particle size distribution, Atterberg Limits etc.
- (b) **Disturbed** samples recovered from the split spoon sampler of the Standard Penetration Tests. These are suitable for identifying and describing more accurately the soil layers encountered and performing classification tests as above.
- (c) **Undisturbed** cohesive samples recovered from the split spoon sampler of the Standard Penetration Test. Apart from the above classification tests, these samples are suitable for performing Unconfined Compression Tests and in some cases Quick Undrained Triaxial Tests. In addition they can be used for carrying out natural moisture content tests and natural density tests.
- (d) Two **Undisturbed (U100)** samples were recovered from the two types of khaki Marl and grey Marl. The samples were taken using 100mm diameter by 460mm long U100 sampling tubes. Penetration of the tubes was effected by pushing them into the soil layers using the drilling rig equipment.

2.3 In-situ Testing

In-situ testing performed in the boreholes consisted of the Standard Penetration Test (SPT). The tests were performed at intervals of 1.50m. The total number of tests performed was 29 and the results are recorded in the Borehole Records. The SPT results are also presented graphically in Figures 12 and 12A. The tests were performed in accordance with BS 1377:90 titled 'Methods of Testing Soils for Civil Engineering Purposes'. A standard split spoon sampler is driven into the soil to a depth of 450 mm by the repeated blows of a 63.5 kg standard penetration monkey trip hammer. The number of blows for every 150mm penetration is recorded. The penetration

resistance 'N' is defined as the number of blows required to drive the sampler into the soil the last 300 mm.

In the sandy coarse gravel layers the open split sampler is substituted by a closed cone sampler. In this case no sample is recovered.

3. LABORATORY TESTING

A program of laboratory testing was carried out on a selection of the disturbed and undisturbed samples recovered. The tests included classification tests (natural moisture content, particle size distribution, Atterberg limits, specific gravity, natural density), shear strength tests (unconfined compression and triaxial tests) swelling and consolidation tests on samples of the Marl and chemical tests on soils. The number and type of tests carried out are outlined in the following sections and the results are presented in the relevant Figures in Appendix B and in tabular form in Tables 1 and 2.

The purpose of the laboratory tests was to establish the mechanical characteristics of the soil layers and deduce their engineering properties, which are required for the proper design of the foundations of the proposed structures.

3.1 Classification Tests

Classification tests carried out on selected samples included natural moisture content, liquid and plastic limit, particle size distribution by wet sieving and sedimentation (hydrometer) tests, specific gravity and density tests. The results of these tests enable the classification and correlation of the soil strata and their comparison with other tests such as SPT, shear strength and consolidation tests. They are also useful for a better understanding of the behavior of the soil layers encountered.

3.1.1 Natural Moisture Content

A total of 23 natural moisture content tests were performed mainly on cohesive samples -most of them on samples of Marl- recovered from the split spoon sampler and from the undisturbed U100 samples. The results are presented in a graphical form, Fig.13, where the moisture content was plotted against depth. They are presented in the Summary of Test Results Table 1 and recorded on Figures presenting the Unconfined Compression and Triaxial Tests.

The variation with depth of the moisture content of the Marl, Fig. 13, shows, as expected, lower moisture content at shallow depths which increases with depth. Over the depth range of 1.0 to 4.0m, the moisture content of the Marl recorded varies from 15% to 24% with a higher value of 28%. For depths greater than 6.0m, the moisture content of the Marl ranges from 27% to 32%.

3.1.2. Atterberg Limits

The Liquid Limit of five cohesive samples was found using the Cone Penetration Method. Their Plastic Limits were also determined and hence their Plasticity Index obtained. The results are presented in Figures 14 to 18, and in the Summary of Test Results, Table 1. Four of the samples were from the khaki silty Marl and gave liquid limits of 51.2% to 56.0% with plastic limits ranging from 32.6% to 36.1% and plasticity index ranging from 18.0% to 21.7 %. The fifth sample was from the grey silty Marl giving a lower Liquid Limit of 49.5%, plastic limit of 29.2% and plasticity index 20.4%.

The above results have also been plotted on the Plasticity Classification Chart, Figure 19. The results plot below the A-line for all Marl samples. According to this Chart the khaki Marl may be characterized as highly elastic silt of high plasticity, and the grey Marl as inorganic Silt of intermediate plasticity.

3.1.3. Specific Gravity

The specific gravity of five cohesive or fine-grained samples tested also for their particle size distribution (hydrometer), was found. Four samples of the khaki Marl gave specific gravity values of 2.74 to 2.78 whereas one sample of the grey Marl, gave a value of 2.73.

3.1.4 Natural Density Tests

Fifteen natural density tests were carried out on silty Marl samples and one on the brownish-khaki silty Clay. The density of the khaki Marl was found to vary from 19.0 to 20.3 kN/m³ with an average value of 19.8 kN/m³ and that of the grey silty Marl ranged between 19.1 and 19.5 kN/m³ with an average value of 19.4 kN/m³. The density of brownish-khaki Clay was found to be 21.8 kN/m³. The results are presented in the Summary of Test Results, Table 1 and on the Figures of the Unconfined Compression and Triaxial tests.

3.1.5 Particle Size Distribution

The particle size distribution of 11 samples was found. Five of the samples were cohesive and the hydrometer (sedimentation) method was used for the finer particles and wet sieving for the coarser ones. The remaining 6 samples were non-cohesive, silty sands and gravels and the wet sieving method was adopted. The results are presented in Figures 20 to 30.

Four samples of the khaki Marl gave 7.5 to 28% clay size particles (>2m), 52 and 75.5% Silt and 12 and 24% fine sand. The sample of grey Marl gave 21% clay, 67% silt and 12% fine sand.

The gradings of the alluvium deposits show a variation of Sands with Gravels and sandy Gravels. Two gradings of predominantly sand samples gave sand content of 54 and 68% with gravel content of 19% and silt 13 to 27%. Four gravel samples gave gravel content ranging from 64 to 79% with sand content of 21 to 35% and small amount of silt.

It must be mentioned that large size gravels could not be included in the samples recovered from the boreholes since these were broken down by the drilling operations. Therefore this should be born in mind when examining the particle size distribution curves of the sandy gravels.

3.2 Unconfined Compression Tests

Eight unconfined compression tests were performed on selected cohesive samples. Six were selected from the khaki silty Marl, one from the brownish-khaki Clay and one was a khaki sandy silty Marl sample. The samples were recovered from the split spoon sampler and were considered suitable for these tests. The tests were performed using the triaxial compression machine with suitable attachments designed for such tests.

The stress-strain curves obtained are presented in Figures 31 to 34. The undrained cohesion c_u obtained from the samples of the khaki silty Marl ranged from 180 to 375 kN/m² with an average of 260 kN/m². The sample of the brownish-khaki Clay gave a c_u value of 120 kN/m² and that of the sandy silty Marl only 83 kN/m².

3.3 Triaxial Tests

Two sets of quick undrained triaxial tests were carried out on undisturbed samples of the khaki silty Marl and of the grey silty Marl. The tests were performed on samples recovered from the U100 sampler. The samples tested had a diameter of 35mm and a height of 70mm. They were tested in the triaxial machine using cell pressures of 100, 200 and 300 kN/m².

The stress-strain curves obtained are presented in Figs. 35 & 36 and the Mohr circle of stresses in Figs. 37 & 38. The undrained cohesion c_u obtained from the Mohr circle of stresses was 275 kN/m² for the khaki Marl with zero angle of shearing resistance ϕ_u , whereas for the grey Marl, c_u was found to be 150kN/m² with ϕ_u value of 2.5°.

3.4 Swelling Pressure Tests

Swelling Pressure Tests were performed on undisturbed specimens which were also tested for their consolidation characteristics. The tests were performed in the Oedometer front loading machine on two Marl samples, one from the khaki Marl and the other from the grey Marl. The specimens had a diameter of 50 mm and a thickness of 19.05mm.

After placing the specimen in the Oedometer machine, it was first loaded with a load approximately equal to its effective overburden pressure. Water was then added to the consolidation cell and the specimen observed for any swelling tendency. In case a tendency for swelling was observed, this was prevented by loading the specimen accordingly. The loads on the specimen and the corresponding time were recorded.

No swelling pressure was observed for any of the two samples tested.

3.5 Consolidation Tests

After completion of the swelling pressure test, the consolidation test was started by adding load on the specimen and recording its compression at frequent time intervals. The load on the specimen was doubled every 24 hours until the maximum pressure of 1600 kN/m² was reached. The specimen was then unloaded in stages every 24 hours, allowing the specimen to swell and recording the swelling of the specimen at each load. From the compression measurements, the void ratio 'e' vs log Pressure was produced and the modulus of volume change m_v and the coefficient of consolidation c_v were calculated.

Two consolidation tests were performed, on samples of the khaki and grey Marl. The results are presented in Figures 39 to 50, and in Table 2. For the khaki Marl m_v values varied from 3.97 to 7.74×10^{-5} m²/kN with an average of 6.02×10^{-5} m²/kN and c_v from 3.04 to 3.27 m²/year with an average of 3.13 m²/year. For the grey Marl, the corresponding m_v values varied from 4.2 to 8.36×10^{-5} m²/kN with an average of 5.83×10^{-5} m²/kN and c_v from 3.70 to 4.38 m²/year with an average of 4.01 m²/year.

3.6 Montmorillonite and Chemical Tests

One montmorillonite test was carried out to determine the montmorillonite content of a sample of the brown silty Clay. Montmorillonite tests are not included in the British Standard Methods of test for Soils for Civil Engineering Purposes BS 1337:1990, but this was a requirement from the Client.

The montmorillonite content of the brown silty Clay was found to be 20%.

The chemical tests carried as required by the Contract, included the determination of sulphate content of SO₄ and Chloride content Cl. The same soil sample of the brown silty Clay was analyzed and the results are the following:

BH/depth	Soil type	SO ₄ (%)	SO ₃ (%)	Cl (%)	Montmorillonite (%)
N12/4.00-4.45m	Brown silty Clay	0,038	0,032	0,013	20.0

4. SITE GEOLOGY

4.1 The Site

Site No. '3' as it is designated on the Location Plan in Fig. 2, has an irregular shape and covers an area having an average length of about 280m and an average width of about 200m. Most part of the site slopes towards the Southeast with its Northwestern part having a steeper slope. The only vegetation existing on the site was dry grass.

4.2 General Geology

A stream, named Kaloyeros, flows from Southwest to Northeast, separating the Campus area in two. A valley of about 400 to 500 meters wide extends on both sides of the stream.

The main bedrock in the broader area of the University Campus is the Marl of the Nicosia-Athalassa Formation. The Marl is stiff to hard and has a yellowish-khaki colour. The khaki Marl is underlain by the grey Marl which is usually more silty and sandy.

The Marl is overlain by younger deposits, such as the Fanglomerates, composed mainly of gravels in a calcareous silty sand matrix with a variable degree of cementation. This layer is found on the hillsides or on top of the hills on either side of the valley. Talus, or hill wash material, usually composed of brown clayey sandy silty or sandy silty clay is found on the lower part of the hill slopes and in the stream valley.

Finally, the stream valley is covered by Alluvium deposits composed of sands and sandy gravels. The overall depth of the Alluvium deposits reaches a maximum of 9.0 metres.

4.3 Information from Boreholes

The strata encountered by the boreholes drilled within Site 3 are shown in the Geological Section Fig. 3 and may be distinguished into four layers:

Layer 1 is the uppermost layer and is composed of brown clayey sandy silty with some gravel and/or havara in places. In Borehole N12, a thickness of 3.25m of this layer is made of brownish-khaki silty Clay. This layer was most probably formed from slope-wash material. Its maximum thickness found in Borehole N12, was 5.50 metres.

Layer 2, underlying Layer 1, is made of beds of Sands, and sandy Gravels, and has a maximum thickness of 5.50 metres as found in Borehole N13. This layer forms the Alluvium river

deposits. Hence the maximum total thickness of the superficial deposits (Layers 1 and 2) in this area, covering the khaki Marl is about 9.0 meters.

Layer 3 is the khaki silty Marl which is stiff to hard, with a considerable variation in thickness. Its thickness was determined by boreholes N12 and N13 to be 2.35 and 1.40 metres respectively. Its thickness increases in the Northwestern part of the site where the Alluvium deposits are absent.

Layer 4 is the stiff to hard grey silty sandy Marl below the khaki Marl, which has been encountered in Boreholes N12 and N13.

4.4 Ground Water

Ground water was encountered during drilling in Boreholes N11, N12 and N13. The depth of the water was measured in these boreholes and the water table was found to be almost at the same elevation in all Boreholes. The recorded water depths are given on the Borehole Records and have been plotted on the Geological Section, Figure 3.

5. ENGINEERING PROPERTIES OF STRATA

The laboratory test results have been presented in Section 3. The engineering properties of the layers encountered presented in this section, are based on the laboratory test results and the in-situ testing.

5.1 Layer 1

Visual examination of samples of Layer 1 shows a variation from clayey Silt to silty Clay with the presence of havara and gravels in places. This layer is quite variable both in texture and engineering characteristics. This is reflected in the results of the Standard Penetration Tests.

Seven Standard Penetration Tests carried out in this layer, gave values of Standard Penetration Resistance N varying from 18 to 70, with three high values of 54, 54 and 70. These high values are due to the existence of gravel at the depth where they were performed. Neglecting these three high values, the remaining four N values ranged from 18 to 42 with an average of 32.

One unconfined compression test performed on a sample of brownish-khaki silty Clay similar in appearance with the Marl, gave a cohesion value of 120 kN/m^2 .

5.2 Layer 2

This layer consists of sands and sandy gravels and laboratory testing was restricted to wet sieving to obtain the particle size distribution of various samples. The results are discussed in Section 3.1.

The most important tests performed in this layer were the in-situ Standard Penetration Tests. The results obtained have been plotted in Figure 12A. The Standard Penetration Resistance ‘ N ’ is lower for the Sand layers and higher for the more gravelly layers. The existence of large gravels gave two high N values of 64 and 47. On the other hand, the existence of silt lenses or of not so dense fine sand, gave three low N values of 10, 15 and 17. The remaining four N values varied from 32 to 37, with an average value of 34.

The modulus of compressibility E_s for the sandy gravel deposits can be obtained from empirical relationships using the average value of N . Taking $N = 34$ for sandy gravel deposits, the value of E_s is given as $42,000 \text{ kN/m}^2$. For the lower N values of the silt and fine sand lenses, taking an average $N=14$, E_s is given as $14\,000 \text{ kN/m}^2$.

From empirical relationships also, we can obtain an average value of the angle of shearing resistance ϕ . For an average $N=34$, ϕ value can be taken conservatively as 37° for the sandy gravels. For the weaker fine sand and silt layers, ϕ may be taken as 30° .

5.3 Layer 3

This is the khaki silty Marl. A considerable number of tests were performed on samples of this layer, i.e. classification tests, shear strength tests and compressibility tests. Also in-situ SPT tests were performed.

5.3.1. Classification Tests

For the natural moisture content, Atterberg Limits, natural density and particle size distribution, please refer to Section 3.

5.3.2. Standard Penetration Resistance

The Standard Penetration Resistance values measured for the Marl are presented in Figure 12. Eleven SPTs were performed. With the exemption of two high N values equal to 47 recorded in Borehole N10, the remaining nine values ranged from 24 to 37 with an average of 30. Most of the N values in Boreholes N9 and N12 are the same and around 28.

5.3.3. Shear Strength

In addition to the Standard Penetration tests, in order to assess the shear strength of the khaki Marl, one set of quick undrained triaxial tests and seven tests of unconfined compression have been carried out. The undrained cohesion c_u obtained from 6 of the unconfined compression tests ranged from 180 kN/m² to 375 kN/m² with an average of 260 kN/m². One sample of the khaki Marl gave a c_u value of 83 kN/m².

The triaxial test performed on samples of the khaki Marl from the depth of 10.0m, gave a c_u value of 275 kN/m² and ϕ_u of zero.

5.3.4 Modulus of Elasticity E_s

The modulus of elasticity E_s of the khaki Marl can be estimated from the stress-strain curves of the unconfined compression and triaxial test. This was estimated to have an average value of 30,000 kN/m². The in-situ E_s , however, is almost 2 to 5 times greater than the values estimated from the stress-strain curves. Therefore, an approximate value for E_s for the khaki Marl of 70,000 kN/m² may be taken when calculating the immediate elastic settlement of the foundations.

5.3.5. Swelling and Compressibility of the Marl

One specimen of the Khaki Marl tested for its swelling potential developed no swelling pressure.

By plotting the plasticity index and clay fraction of the Marl on the Chart for Expansiveness of Soils, Fig. 51, the expansiveness of the Marl is shown to be 'medium'.

The values of the coefficient of volume change m_v of the khaki Marl were found to range between 3.97 to 7.74 x10⁻⁵ m²/kN with an average value of 6.02 x10⁻⁵ m²/kN. The values of m_v are used to estimate the amount of consolidation settlement of foundations. Consolidation settlement can also be estimated using the e-log p curve obtained, Fig 39.

The coefficient of consolidation c_v was found to vary from 3.04 to 3.27 m²/year with an average of 3.13 m²/year. The c_v values are used to estimate the time (number of years) the consolidation settlement will be completed.

5.4 Layer 4

The engineering properties of the grey Marl layer differ only slightly from those of the khaki Marl. The grey Marl was found to have a lower Liquid Limit and plasticity index but a similar amount of clay size particles.

5.4.1 Standard Penetration Resistance

No Standard Penetration Tests were performed in this layer.

5.4.2 Shear Strength

One set of triaxial tests performed gave undrained cohesion c_u of 150 kN/m² and angle of shearing resistance ϕ_u of 2.5°.

5.4.3 Swelling and Consolidation

The sample tested exhibited no swelling pressure. The plot in Figure 51 of the plasticity index and clay fraction of the sample, shows that the grey Marl has a medium expansiveness potential.

The coefficient of volume change m_v was found to vary from 4.20 to 8.36×10^{-5} m²/kN with an average of 5.83×10^{-5} m²/kN and the coefficient of consolidation c_v , varied from 3.70 to 4.38 m²/year with an average of 4.01 m²/year.

6. BEARING CAPACITY AND SETTLEMENT OF FOUNDATIONS

The Bearing Capacity of the strata encountered and the settlement of the foundations depend not only on the engineering properties of the strata but on the type, shape and depth of the foundations to be adopted. Hence, the allowable bearing capacity values and estimated expected settlement given below are only indicative. The bearing capacity and settlement of foundations can be found when the foundation loads, their depth and type are known.

6.1 Bearing Capacity

6.1.1 Layer 1-Top silty clayey Soil

Assuming a $\phi_u=0$ soil and taking $c_u=120$ kN/m² (see Sect. 5.1) the safe Bearing Capacity of isolated foundations may be estimated using Skempton's bearing capacity formula

$$q_{\text{safe}} = (c_u N_c) / F + \gamma' D$$

where: c_u = undrained cohesion, kN/m²

N_c = Bearing capacity factor

F = factor of safety

$\gamma' D$ = effective overburden at foundation level

For a square foundation of 1.5 m at a depth of 2.5m, $N_c=8.3$ and assuming $F=3.0$, q_{safe} is calculated to be 380 kN/m². In order to minimize settlement, an Allowable Bearing Capacity of 300 kN/m² can be adopted.

6.1.2 Layer 2- Sands and Sandy Gravels

The allowable bearing for this layer can be estimated using the empirical relationship between standard penetration resistance and allowable bearing pressure proposed by Terzaghi and Peck. This correlation was intended to limit the foundation settlement to a maximum of 25 mm.

Using an average value of Standard Penetration Resistance of 30, for a foundation width of 1.50m, the allowable bearing capacity obtained from the relevant chart is 350 kN/m². However, consideration should be given to the existence of weaker lenses of silt or fine sand and the foundations designed accordingly.

6.1.3 Khaki Marl

For foundations resting on the upper part of the khaki Marl layer, the bearing capacity can be estimated assuming $\phi_u=0$ and $c_u=190 \text{ kN/m}^2$ (conservative assumption) and a factor of safety of 3.0, the safe bearing capacity is found to be 575 kN/m^2 . To minimize settlement, an allowable bearing pressure of 500 kN/m^2 may be used.

6.2 Settlement

Foundation settlement would not exceed 25 mm for foundations constructed on Layers 1 and 2 under the assumptions made in sections 6.1.1 and 6.1.2 above, and using the allowable pressures stated. In view of the variability of these layers, this should be checked by carrying out settlement calculations during the design of the foundations.

For foundations constructed on the khaki Marl, foundation settlement 's' is made up of the immediate elastic 's_i' and consolidation settlement 's_c'.

$$s = s_i + s_c$$

Now $s_i = \mu_o \mu_i \times q \text{ B}/E_s$

Where:

μ_o and μ_i depend on the depth and size of the foundation and are obtained from Charts published by Janbu and Bjerrum

q is the effective bearing pressure

B is the foundation width

E_s is the elastic or compressibility modulus

For a 1.5 m square foundation, loaded with maximum pressure of 500 kN/m^2 (effective = 367.5 kN/m^2), at a depth of 2.5 m and taking $E_s = 70,000 \text{ kN/m}^2$

The immediate elastic settlement was found = 5.5 mm

The consolidation settlement $s_c = \mu \sum m_v \Delta\sigma h$

Where:

μ = coefficient of settlement (0.5 for overconsolidated clays)

m_v = coefficient of volume change ($6.02 \times 10^{-5} \text{ m}^2/\text{kN}$)

$\Delta\sigma$ = change in stress due to foundation load

h = thickness of clay layer considered

Values of $\Delta\sigma$ for the various clay layers considered are calculated using the appropriate 'Influence coefficients' found from suitable charts.

For the same foundation and load considered above, the consolidation settlement was estimated to about 30 mm.

Hence total settlement = $5.5+30 = \underline{35.5 \text{ mm}}$

This settlement is considered excessive and in order to reduce it to acceptable limits (of say 25mm) the allowable bearing capacity should be reduced to 400 kN/m².

CONCLUSIONS AND RECOMMENDATIONS

The strata encountered by the boreholes drilled may be differentiated into at least four layers:

Layer 1 which is composed of brown clayey silt or silty clayey Soil, with the presence of some gravel and havara in places, is a layer of variable shear strength as indicated by the SPT tests results. The estimated allowable bearing capacity for this layer, calculated for a 1.5 m square footing placed at 2.5 m depth, is 300 kN/m². Foundation settlement for such footing is expected to be not greater than 25 mm.

Layer 2, consists of sands and gravels with varying degree of compaction but with a relatively high average 'N' value. The estimated allowable bearing capacity for this layer for a 1.5 m square footing is 350 kN/m². Settlement is expected to be not more than 25 mm. The existence of weaker lenses of silt or fine sand should be taken carefully into consideration during the design of the foundations.

Layer 3, is the khaki Marl which has relatively high shear strength. The allowable bearing capacity for a 1.5m square foundation at 2.5m depth was calculated to 400 kN/m² in order to keep the settlement to acceptable limits. Settlement for the above foundation was estimated to only 25 mm.

Layer 4, the grey Marl is more silty with shear strength values of the same order as for the khaki Marl. This Marl has shown no swelling potential.

Any type of foundation can be used on the above layers, such as isolated pad footings, strip footings, raft and piles. However, in view of the different nature of the above layers, and the possibility of the foundations of the same building to bear on different layers, the foundation type, bearing capacity and foundation settlement must be reconsidered carefully and relevant calculations performed using the engineering parameters available in this report.

Consultation with a Geotechnical Engineer during the design of the foundations is strongly recommended. Professional consideration of the engineering properties and extensive calculations for the bearing capacity and settlement of the foundations should be made in order to achieve both safe and economic foundations.

Due to rapid deterioration of the Marl when exposed, it is very important to keep such exposure to minimum. As soon as the excavation for the foundation is completed and the bottom of the excavation cleaned carefully, it should be covered with a blinding concrete of about 80mm

thick. The construction of the foundations should proceed and completed as soon as possible. Drying and wetting of the bottom of excavations in the Marl should be avoided.

Sulphates in soils or groundwater may attack concrete depending on their concentration. For sulphate concentration in soils less than 0.2% of SO_3 , ordinary Portland cement may be used with maximum water/ cement ratio of 0.55. For SO_3 of 0.2% - 0.5%, the minimum cement content should be 330 kg/m^3 and the water cement ratio less than 0.50.

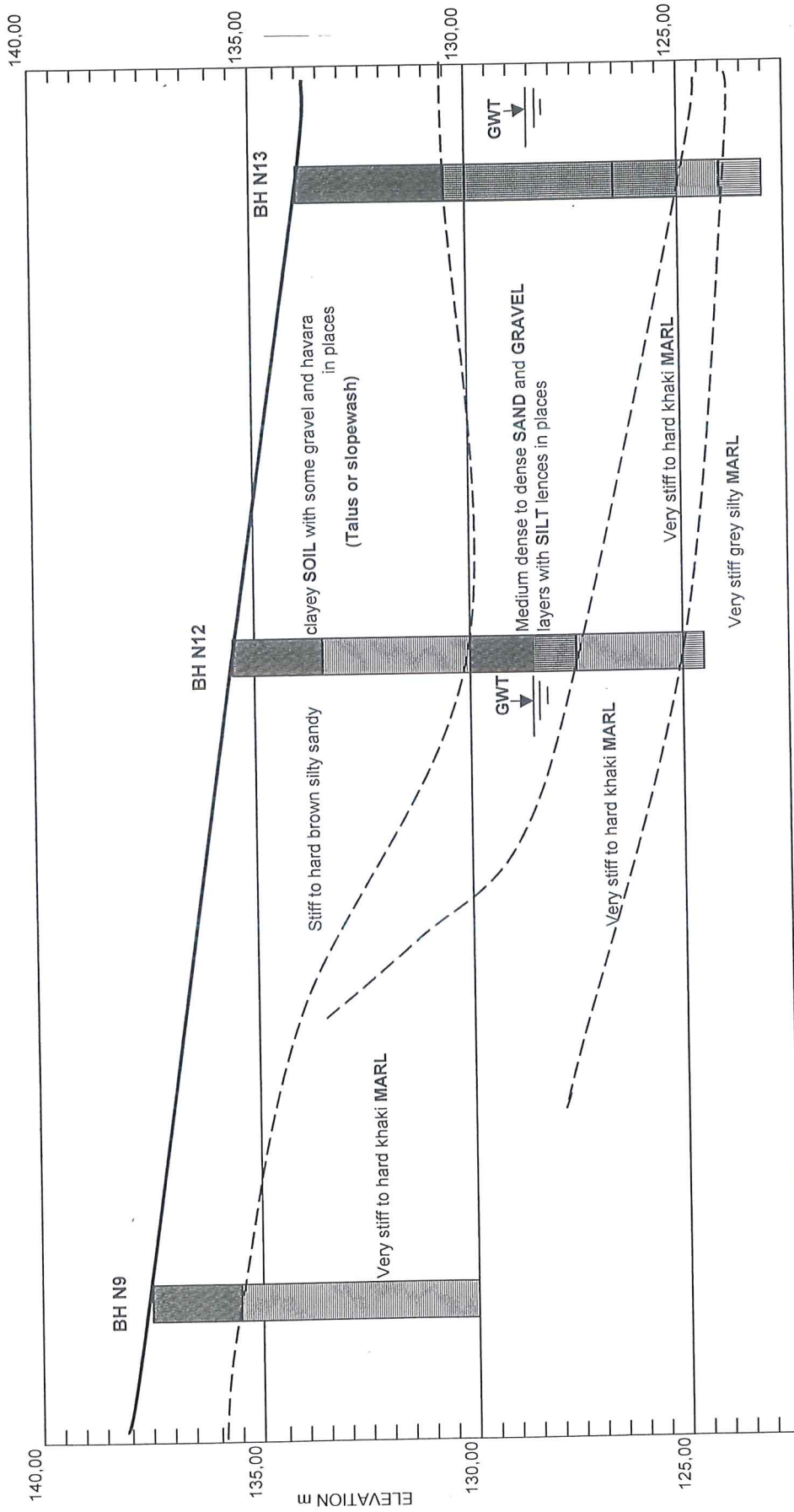
APPENDIX A

- Fig.1: Location Plan
- Fig.2: University Campus, Site Plan
- Fig.3: Geological Section
- Fig. 4 to 8: Borehole Records
- Fig. 9 to 11: Photos of Samples
- Fig.12 & 12A: Standard Penetration Results



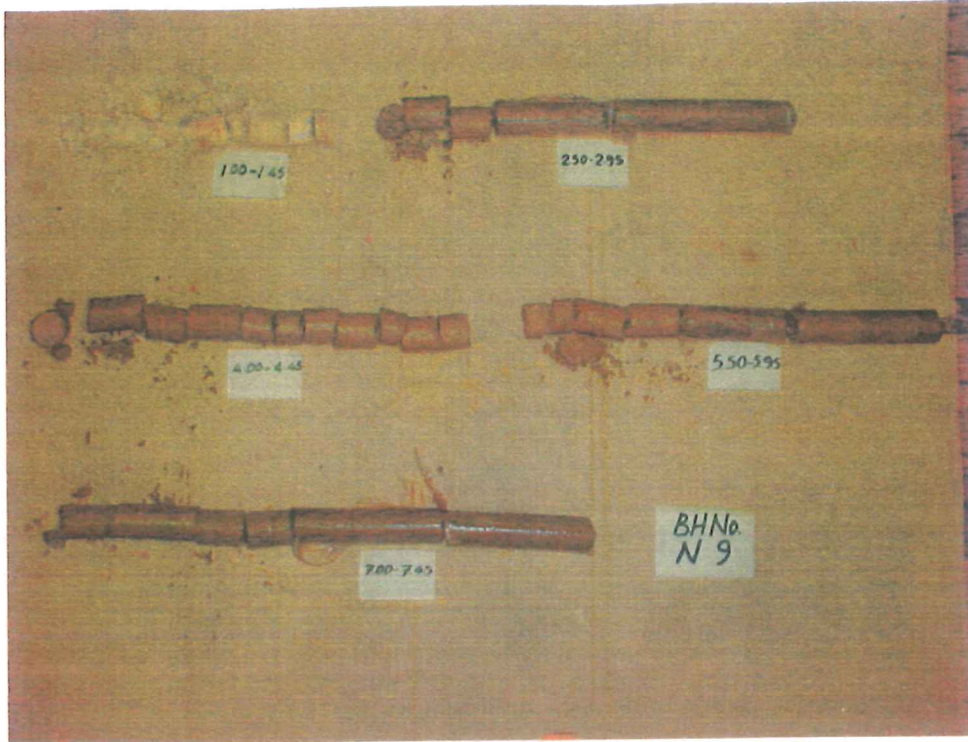
UNIVERSITY OF CYPRUS
UNIVERSITY CAMPUS
BOREHOLE LOCATION PLAN
 Scale: 1:5000
FIG. 2

ΠΑΝΜΟΤΕΡΗ ΕΡΕΥΝΗΤΙΚΗ ΓΕΩΡΓΙΑ
 ΠΑΡΑΡΤΗΜΑΤΑ
 ΠΑΡΑΡΤΗΜΑΤΑ

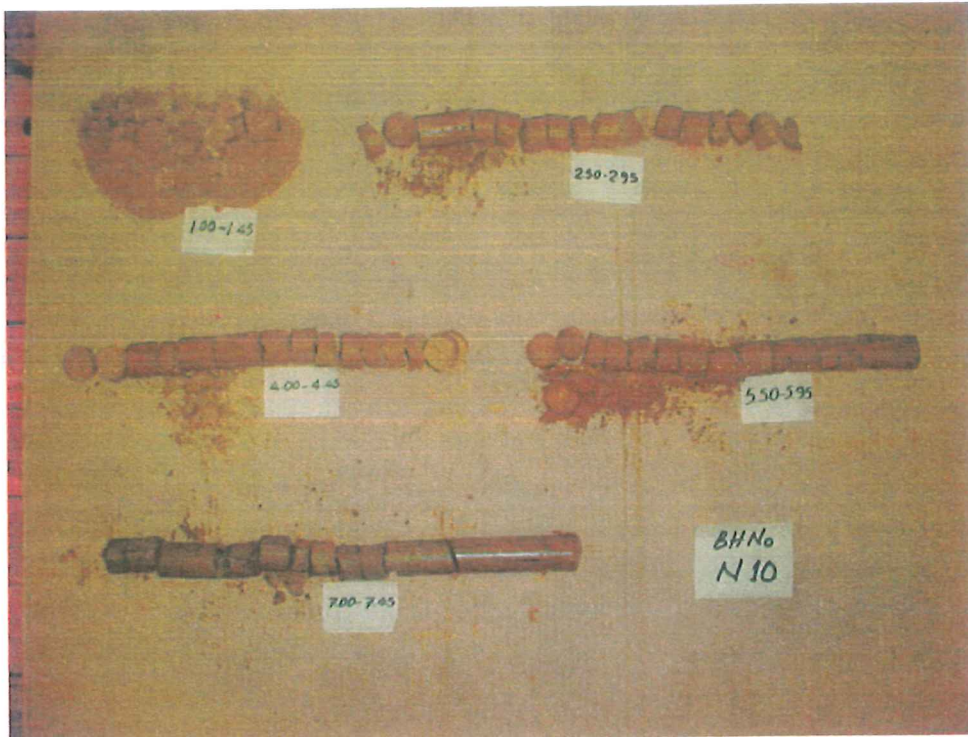


NOTE:
 Drawn boundaries between layers are inferred, based on the borehole findings and are not exact

CAMPUS OF UNIVERSITY OF CYPRUS
 SITE 3
GEOLOGICAL SECTION 1
FIG. 3

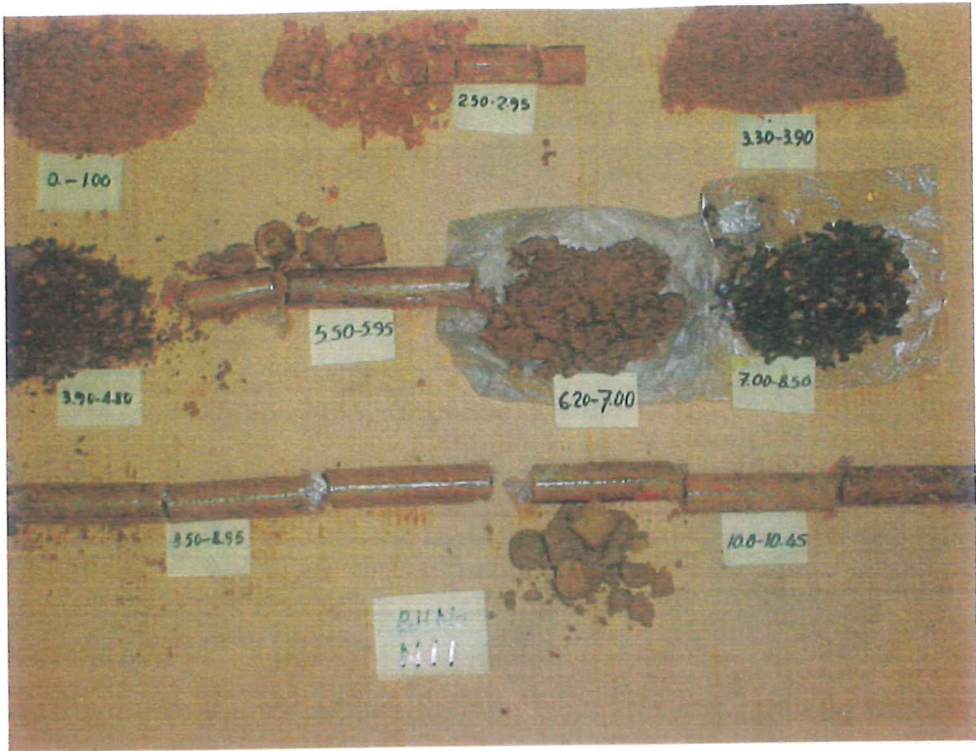


Samples from BH N9

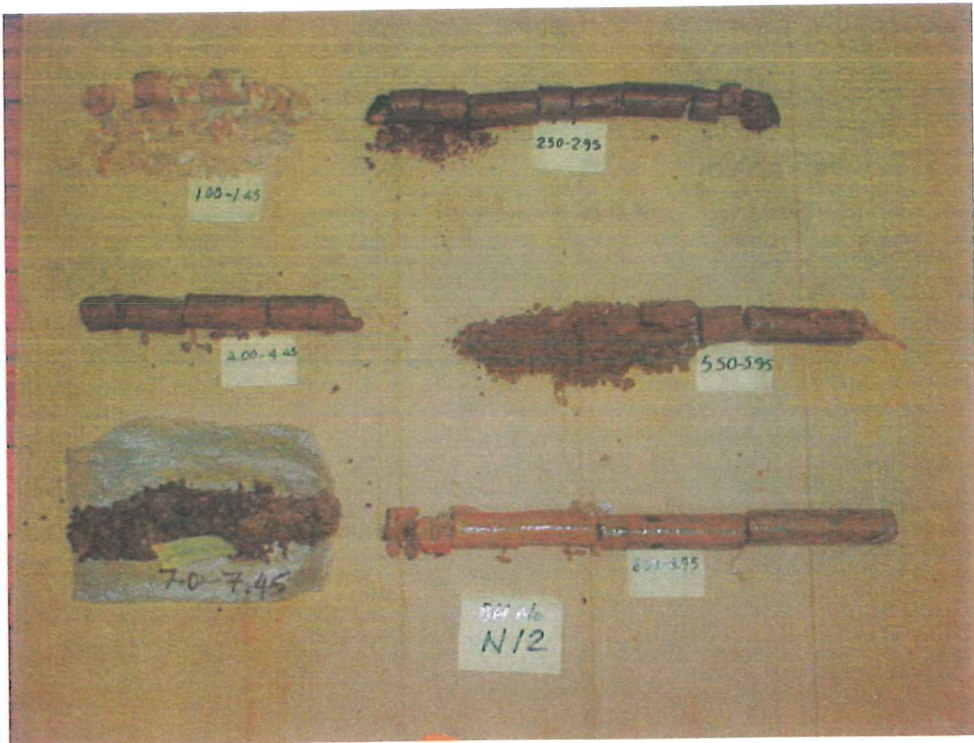


Samples from BH N10

FIG. 9

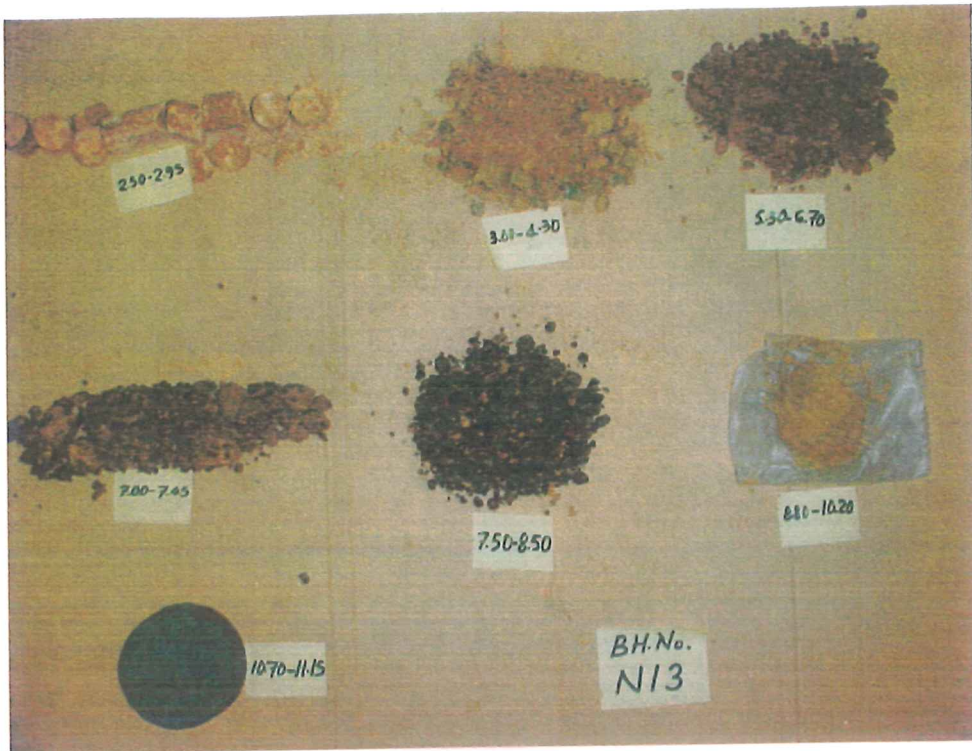


Samples from BH N11



Samples from BH N12

FIG. 10



Samples from BH N13

FIG. 11

STANDARD PENETRATION RESISTANCE

Project: University of Cyprus

Site Location: University Campus - Site "3"

Client: University of Cyprus

Soil: Silty Marl

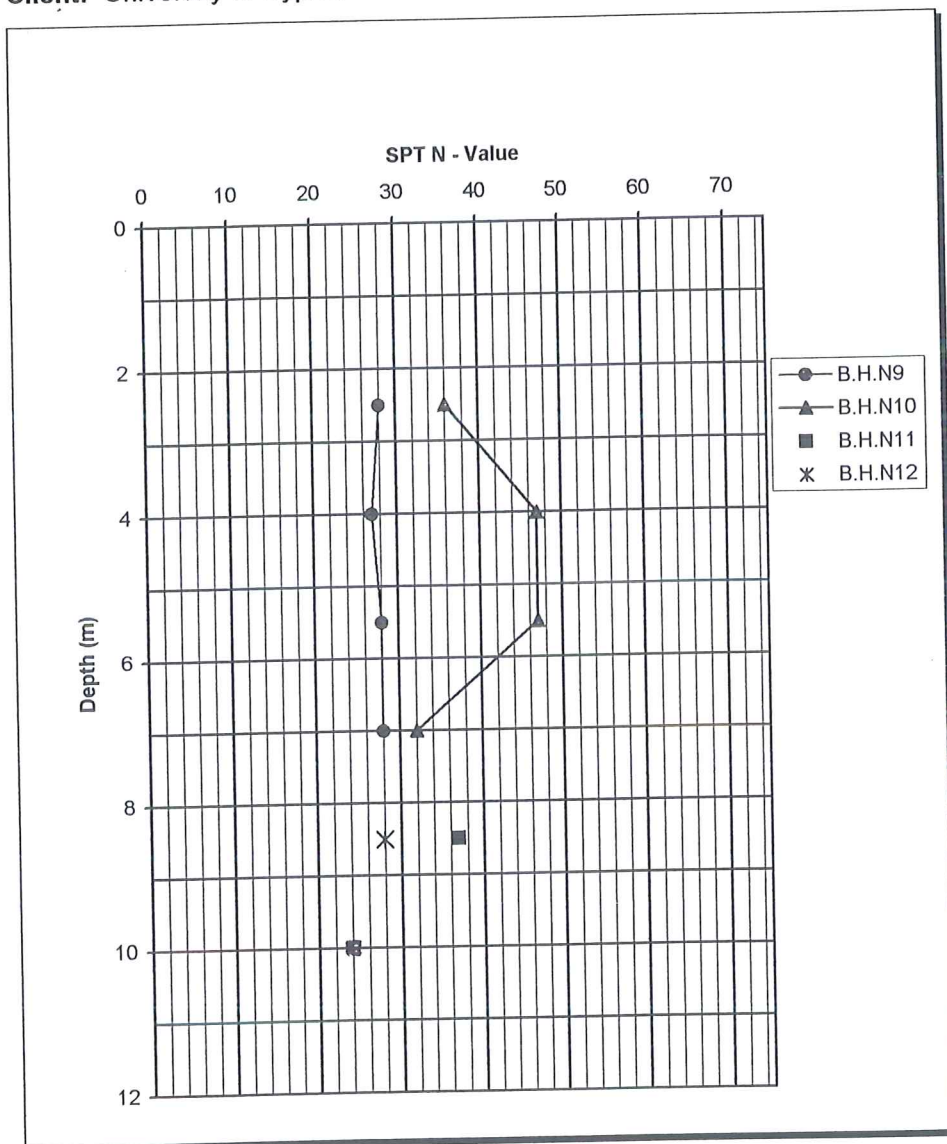


FIG.12

STANDARD PENETRATION RESISTANCE

Project: University of Cyprus

Site Location: University Campus - Site "3"

Client: University of Cyprus

Soil: Alluvium deposits

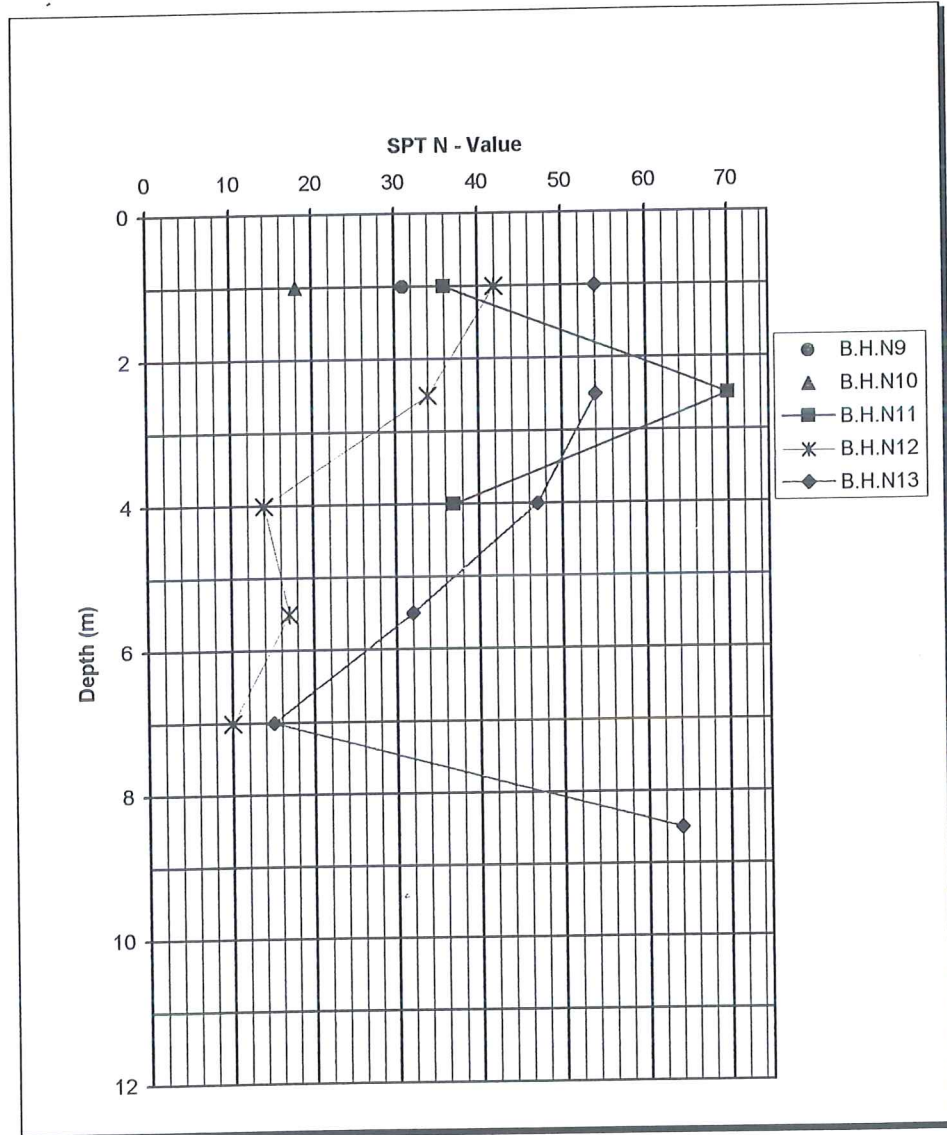


FIG.12A

APPENDIX B

LABORATORY TEST RESULTS

- Tables 1, 2
- Figs. 13 to 51: Laboratory Test Results

TABLE 2
SUMMARY OF CONSOLIDATION TEST RESULTS

B.H. NO.	Depth m	Soil Type	Initial Moisture Content %	Final Moisture Content %	Initial Bulk Density kN/m ³	Initial Void Ratio	Final Void Ratio	Av. Coeff. of Vol. Change m _v m ² /kN×10 ⁻⁵	Av. Coeff. of Consolidation c _v m ² /year	Initial Saturation Sr %
N12	10,00	Khaki silty MARL	32,5	30,1	19,3	0,892	0,793	6,02	3,13	100
N13	10,70	Grey silty MARL	31,9	29,4	19,1	0,890	0,784	5,83	4,01	98

NATURAL MOISTURE CONTENT

Project: University of Cyprus

Site Location: University Campus - Site "3"

Client: University of Cyprus

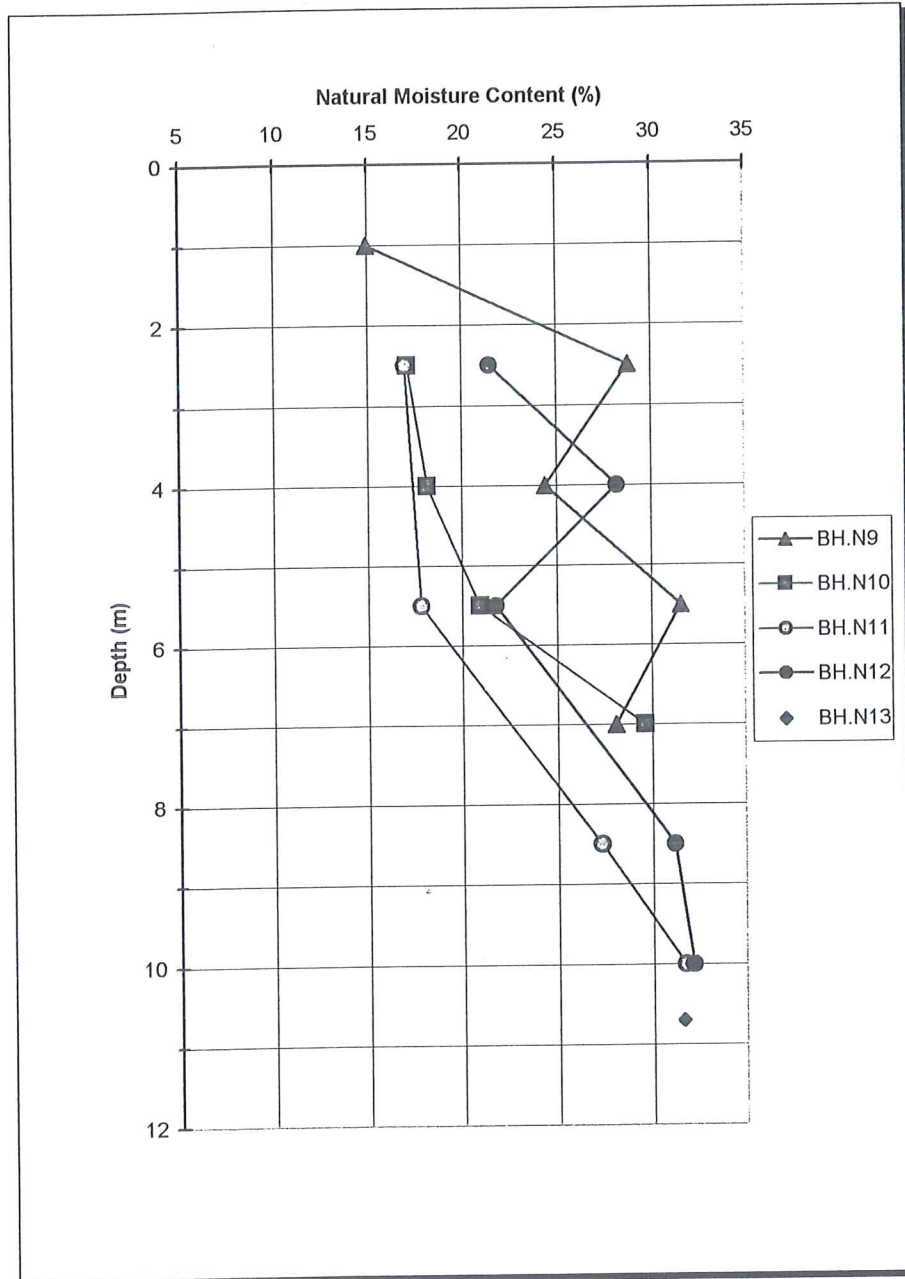


FIG.13

Liquid Limit Test

Cone Penetration Method

Project: University of Cyprus

BH No.: N9

Date: 2/1/2004

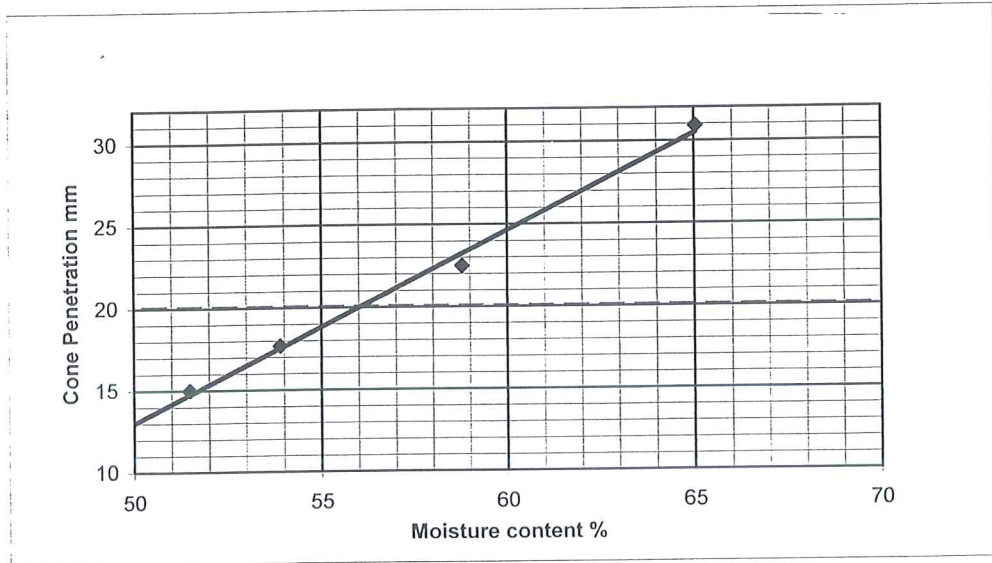
Site Location: University Campus - Site "3"

Depth: 2,50m

Operator:

Client: University of Cyprus

Soil: Khaki silty MARL



Liquid limit	56,0%
Plastic limit	34,5%
Plasticity Index	21,5%

FIG. 14

Liquid Limit Test

Cone Penetration Method

Project: University of Cyprus

BH No.: N9

Date: 2/1/2004

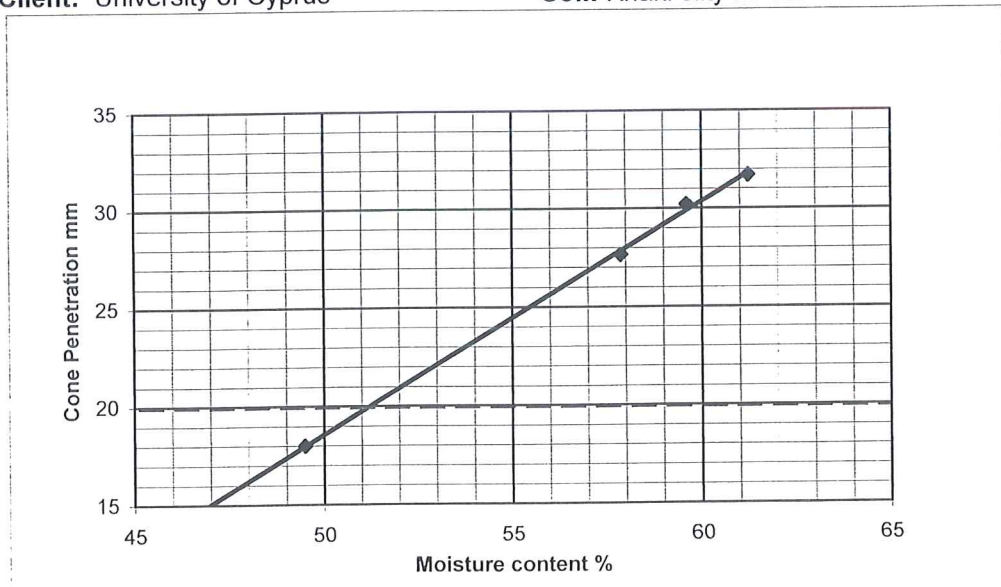
Site Location: University Campus - Site "3"

Depth: 7,00m

Operator:

Client: University of Cyprus

Soil: Khaki silty MARL



Liquid limit	51,2%
Plastic limit	33,2%
Plasticity Index	18,0%

FIG. 15

Liquid Limit Test

Cone Penetration Method

Project: University of Cyprus

BH No.: N10

Date: 2/1/2004

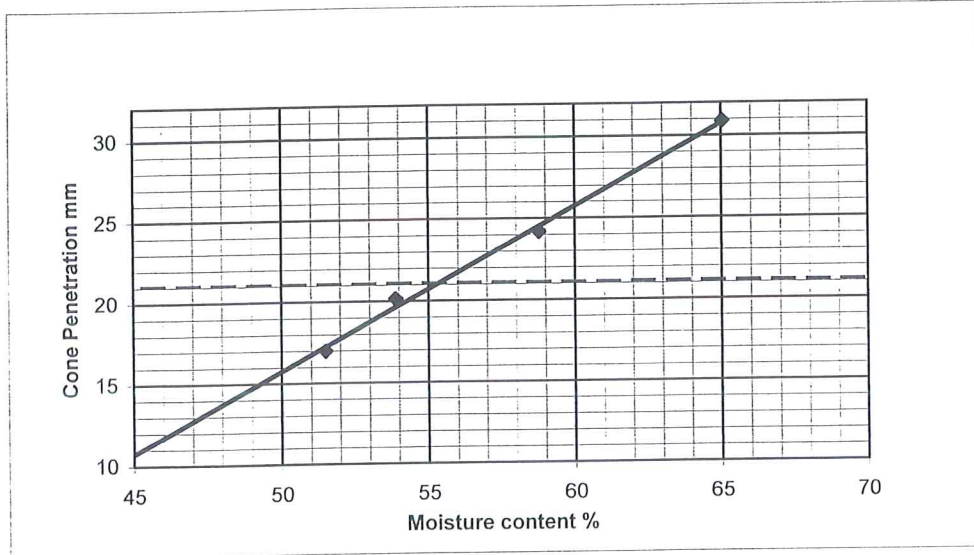
Site Location: University Campus - Site "3"

Depth: 7,0m

Operator:

Client: University of Cyprus

Soil: Khaki silty MARL



Liquid limit	54,3%
Plastic limit	32,6%
Plasticity Index	21,7%

FIG. 16

Liquid Limit Test

Cone Penetration Method

Project: University of Cyprus

Site Location: University Campus - Site "3"

Client: University of Cyprus

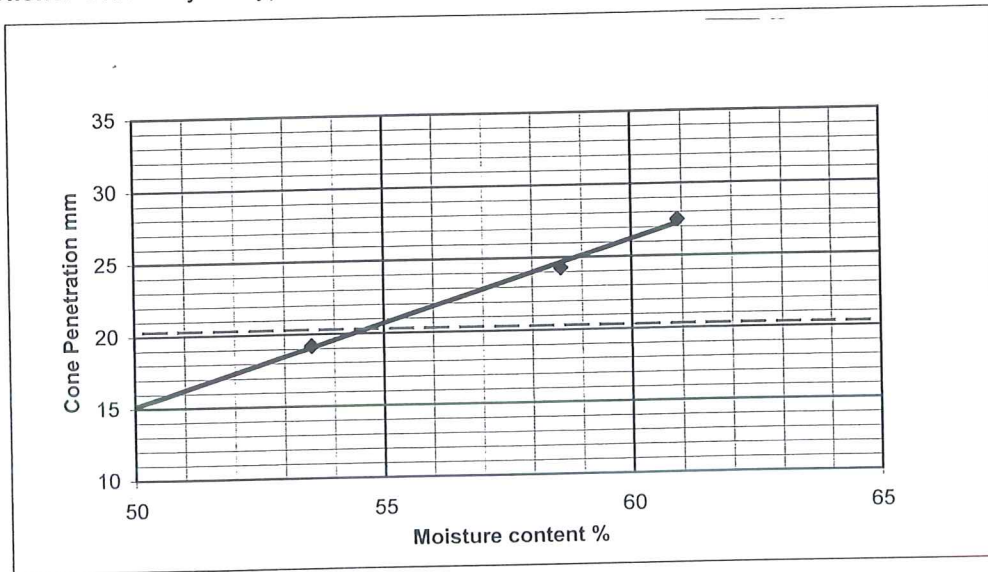
BH No.: N12

Depth: 10,0m

Soil: Khaki silty Marl

Date: 2/1/2004

Operator:



Liquid limit
Plastic limit
Plasticity Index

54,5%
36,1%
18,4%

FIG. 17

Liquid Limit Test

Cone Penetration Method

Project: University of Cyprus

Site Location: University Campus - Site "3"

Client: University of Cyprus

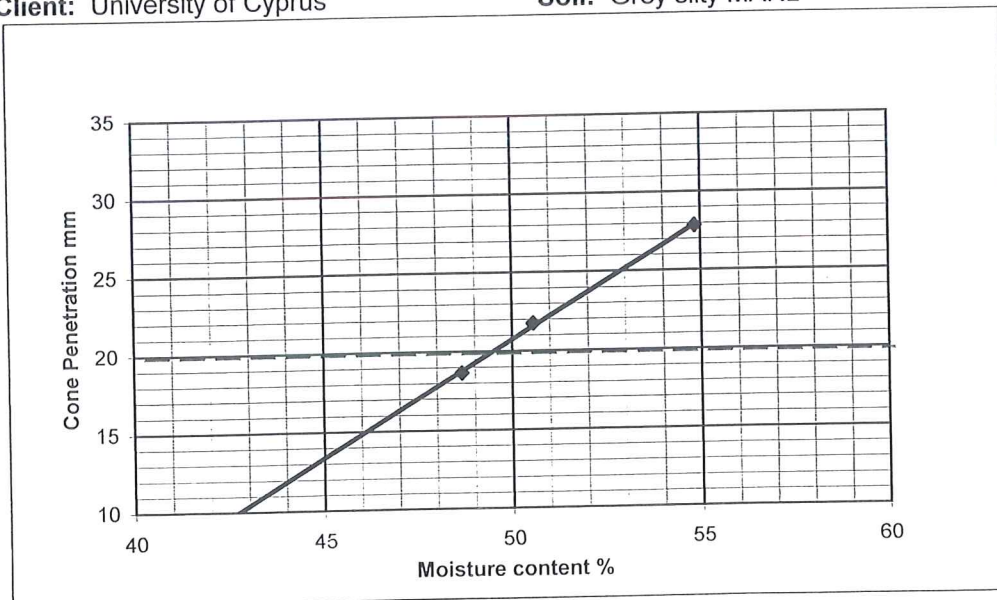
BH No.: N13

Depth: 10,70m

Soil: Grey silty MARL

Date: 2/1/2004

Operator:



Liquid limit
Plastic limit
Plasticity Index

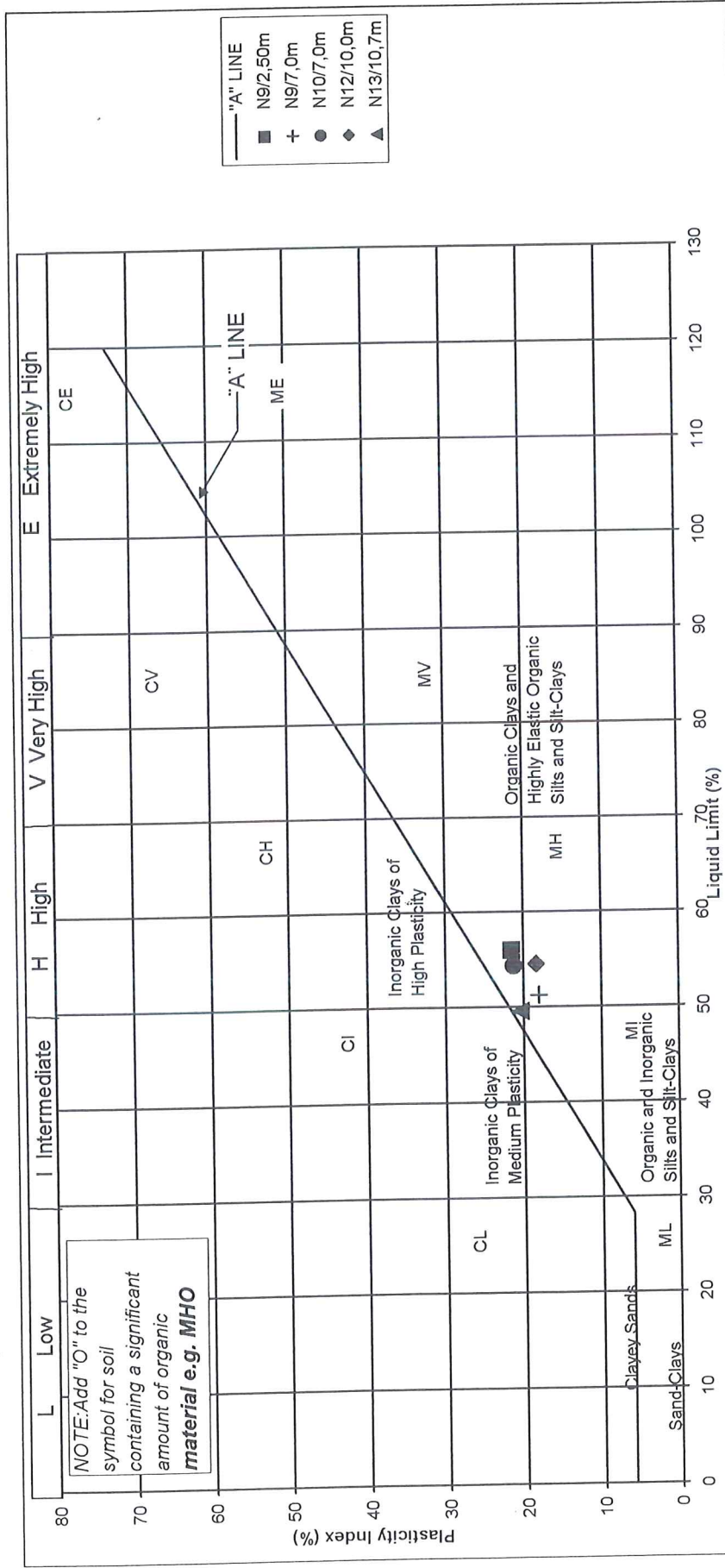
49,5%
29,2%
20,4%

FIG. 18

PLASTICITY CLASSIFICATION CHART

Project: University of Cyprus
 Site Location: University Campus - Site "3"
 Client: University of Cyprus

Date: January 2004
 Operator:



SILT(M-SOIL), M, plots below "A" Line
 CLAY, C, plots above "A" Line
 M and C may be combined as FINE SOIL, F

FIG. 19

PARTICLE SIZE DISTRIBUTION

Wet Sieving and Hydrometer

Date: 29-31/12/03
 Operator:
 Sieving: Wet

BH No.: N9
 Depth: 2,50-2,95m
 Soil: Khaki silty MARL

Project: University of Cyprus
 Site Location: University Campus - Site "3"
 Client: University of Cyprus

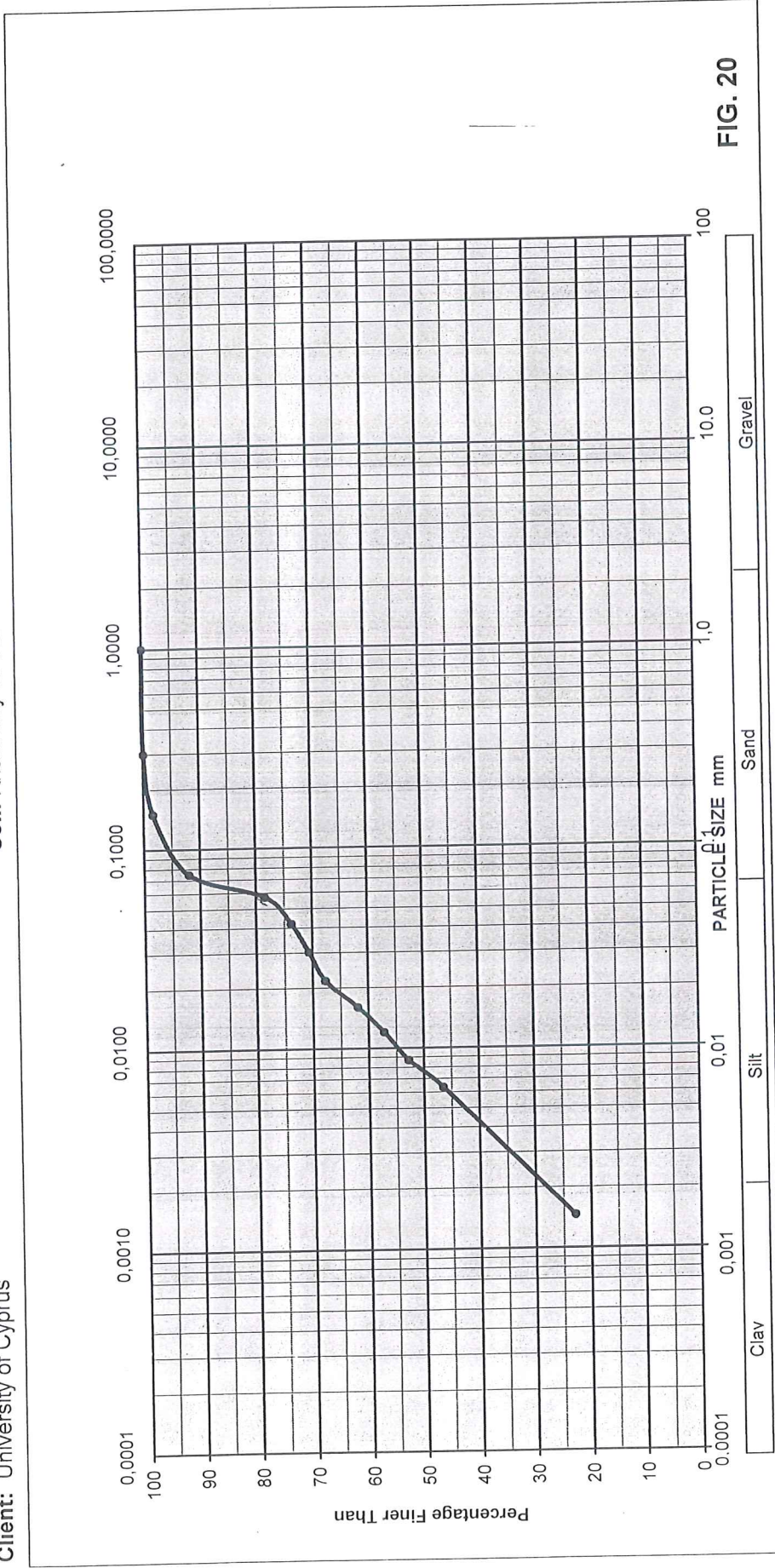


FIG. 20

SH Soil Engineering Ltd
 P.O.Box 26457, 1310 Nicosia
 Tel. 22663191, Fax 22663192

UC3 HYDROMETER G N9-2,5

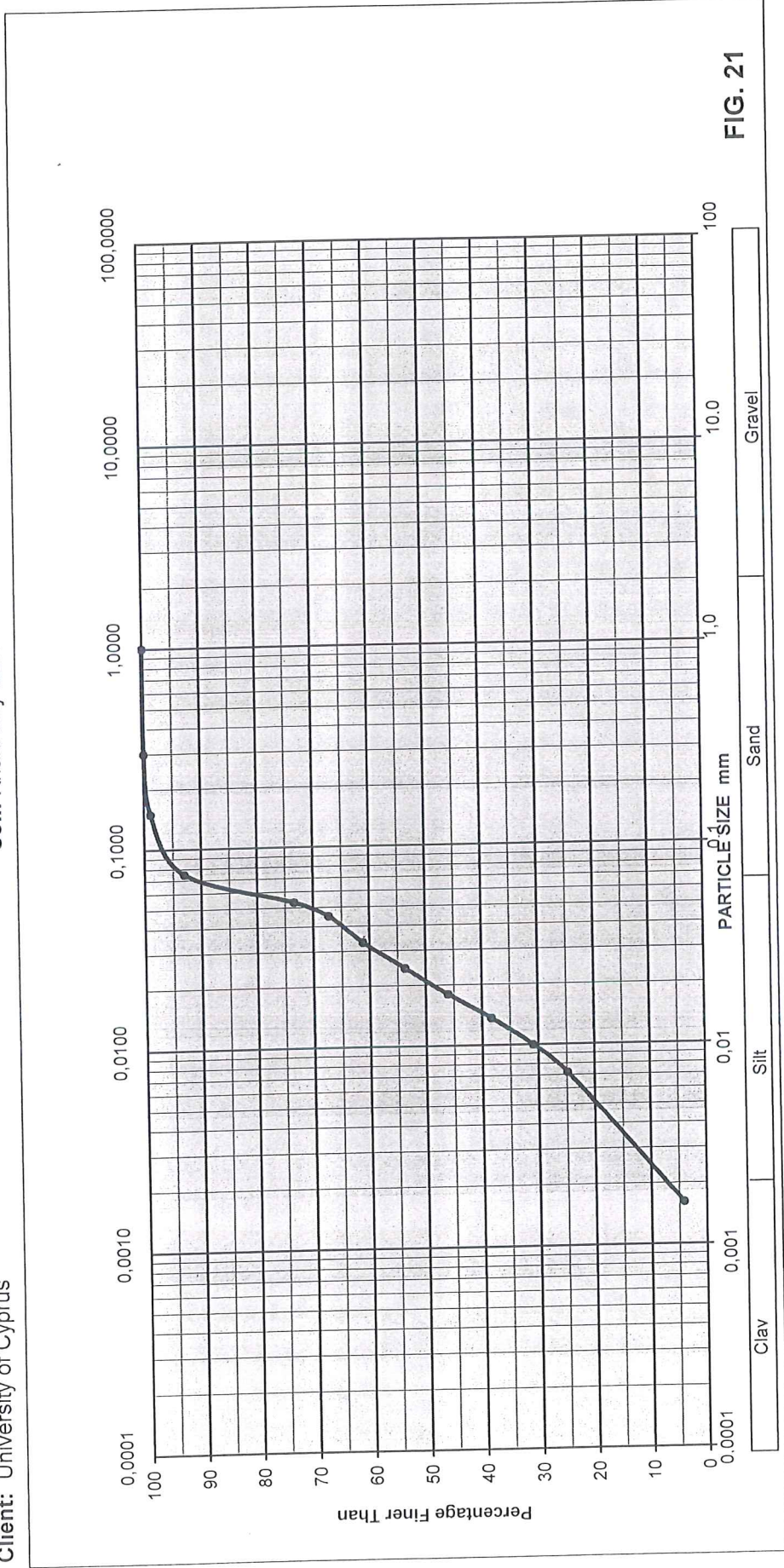
PARTICLE SIZE DISTRIBUTION

Wet Sieving and Hydrometer

Date: 29-31/12/03
Operator:
Sieving: Wet

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

BH No.: N9
Depth: 7,0-7,45m
Soil: Khaki silty Marl



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Tel. 22663191, Fax 22663192

UC3 HYDROMETER G N9-7,0

PARTICLE SIZE DISTRIBUTION

Wet Sieving and Hydrometer

Project: University of Cyprus
 Site Location: University Campus - Site "3"
 Client: University of Cyprus

Date: 29-31/12/03
 Operator:
 Sieving: Wet

BH No.: N10
 Depth: 7,0-7,45m
 Soil: Khaki silty Marl

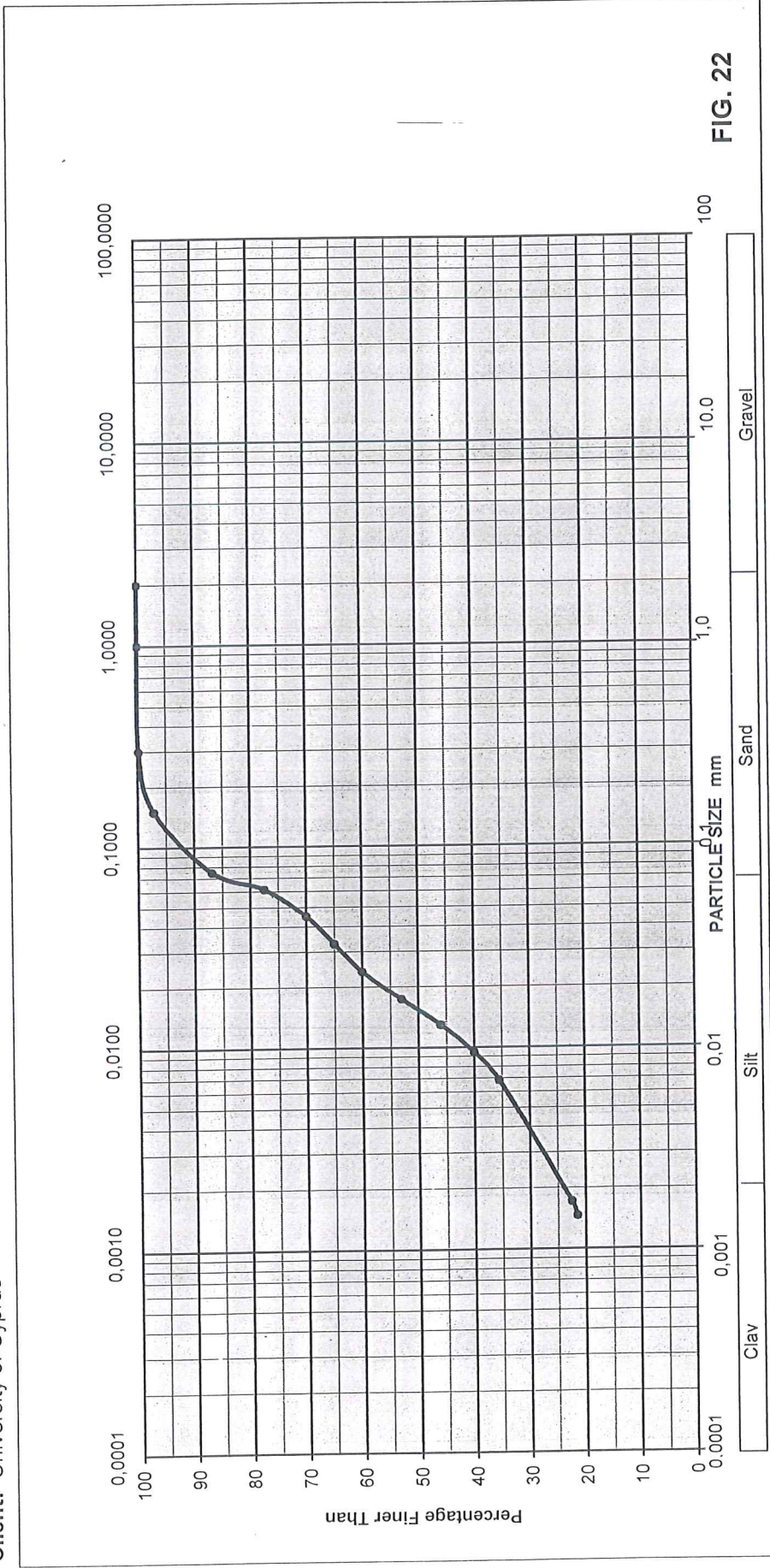


FIG. 22

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 P.O.Box 25457, 1310 Nicosia
 Tel. 22663191, Fax 22663192

UC3 HYDROMETER G N10-7,0

PARTICLE SIZE DISTRIBUTION

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

BH No.: N11
Depth: 3,3-3,9m
Soil: Brown silty fine Sand with traces of gravel

Date: 19/12/03
Operator:
Sieving: Wet

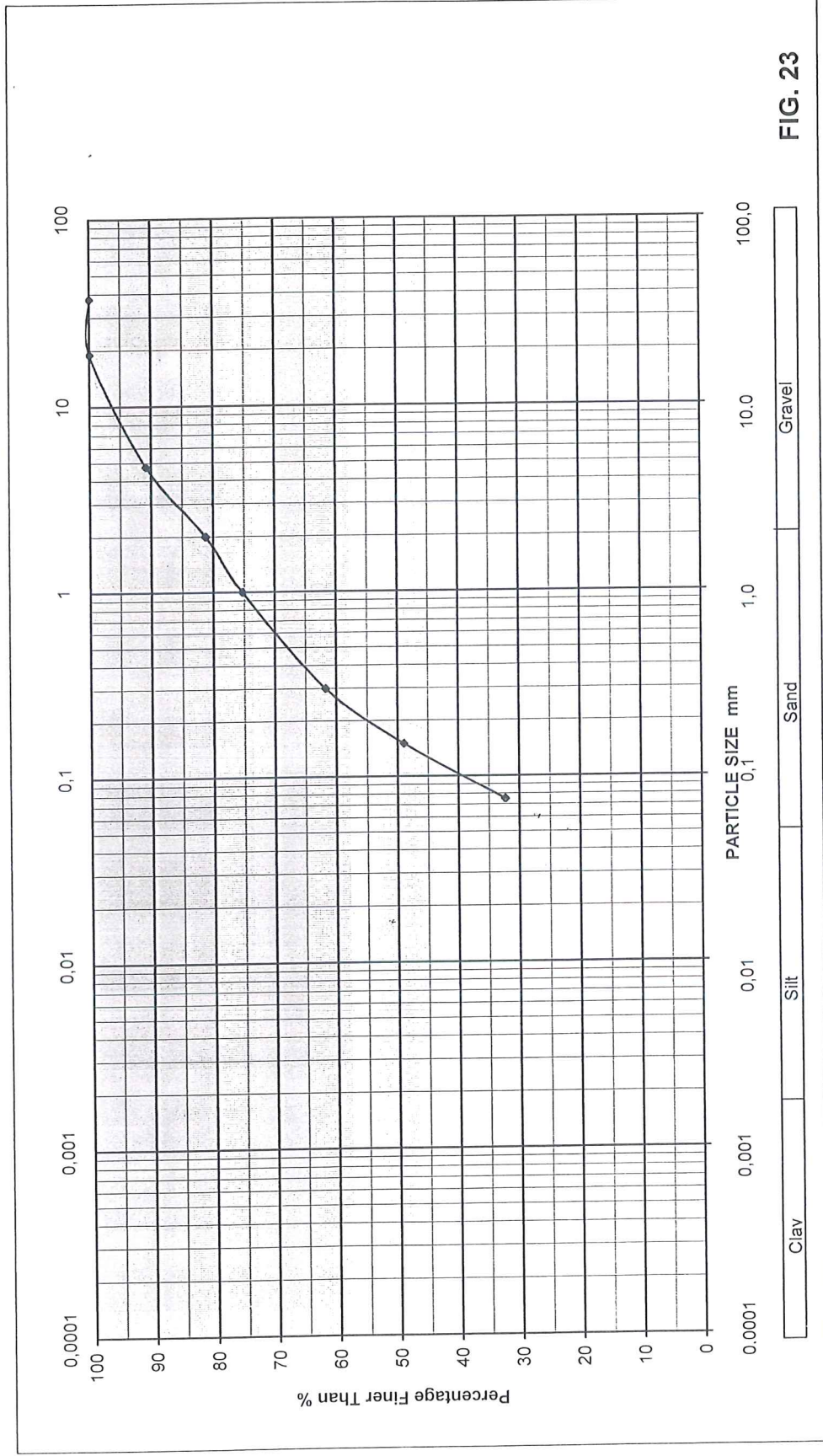


FIG. 23

SH Soil Engineering Ltd
P.O.Box 25457, 1310 Nicosia
Tel.No.22663191, Fax22663192

PARTICLE SIZE DISTRIBUTION

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

BH No.: N11
Depth: 3,9-4,8m
Soil: Grey sandy Gravel

Date: 19/12/03
Operator:
Sieving: Wet

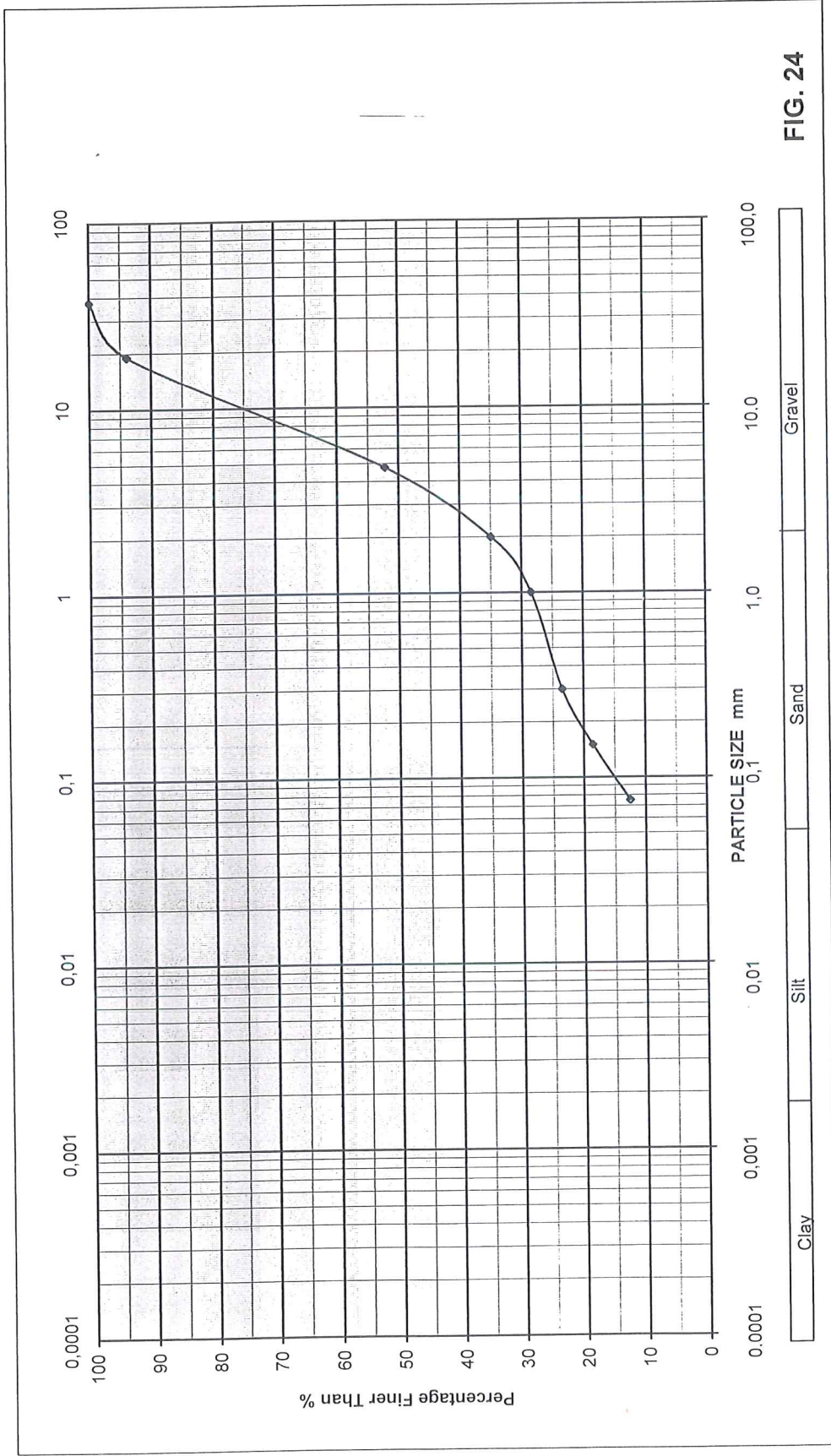


FIG. 24

SH Soil Engineering Ltd
P.O.Box 25457, 1310 Nicosia
Tel.No.22663191, Fax22663192

PARTICLE SIZE DISTRIBUTION

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

BH No.: N12
Date: 19/12/03
Operator:
Sieving: Wet

Depth: 7,0-7,45m
Soil: Brownish-grey silty Sand with gravel

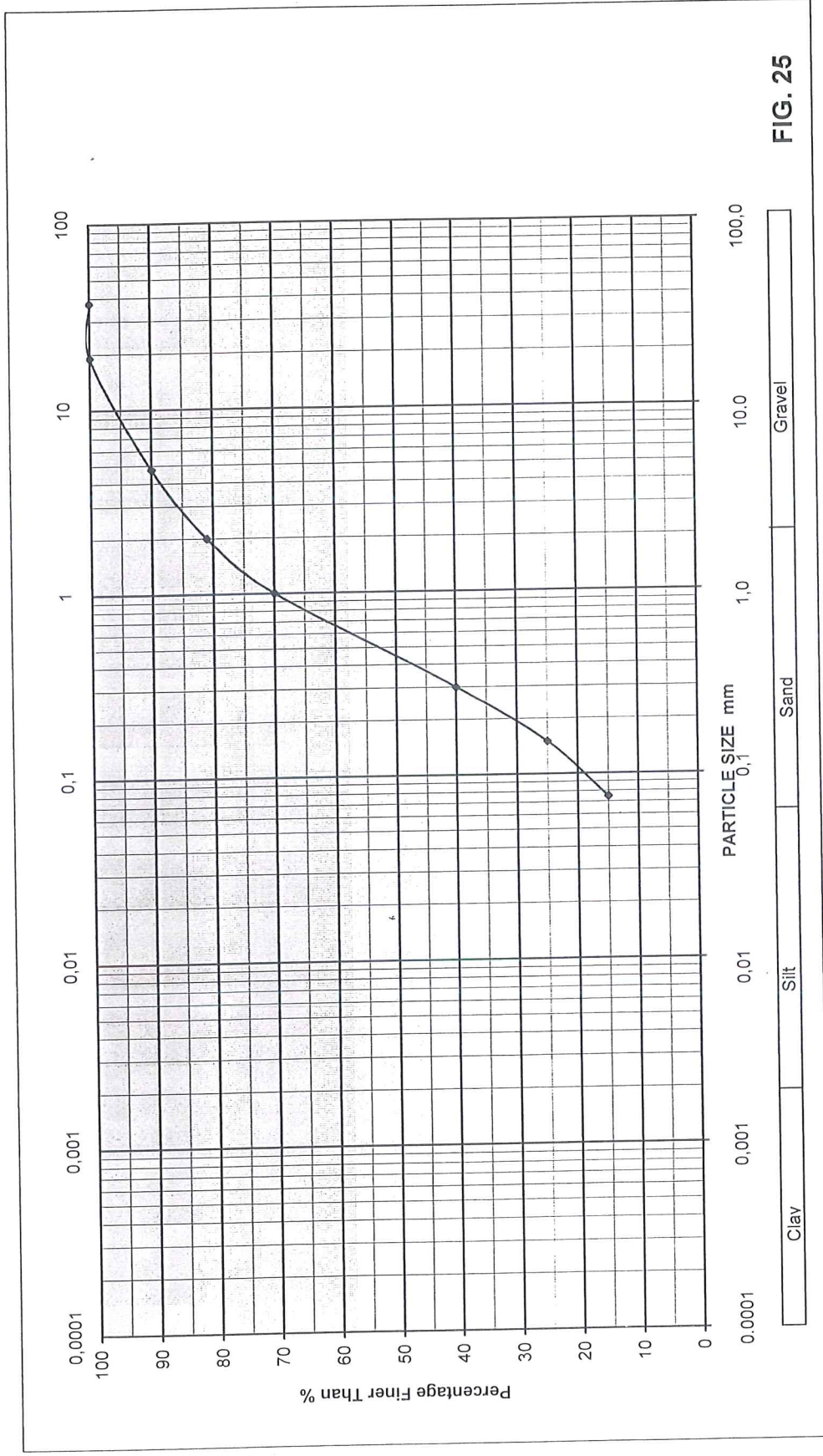


FIG. 25

SH Soil Engineering Ltd
P.O.Box 25457, 1310 Nicosia
Tel.No.22663191, Fax:22663192

PARTICLE SIZE DISTRIBUTION

Wet Sieving and Hydrometer

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

Date: 29-31/12/03
Operator:
Sieving: Wet

BH No.: N12
Depth: 10,0-10,45m
Soil: Khaki silty MARL

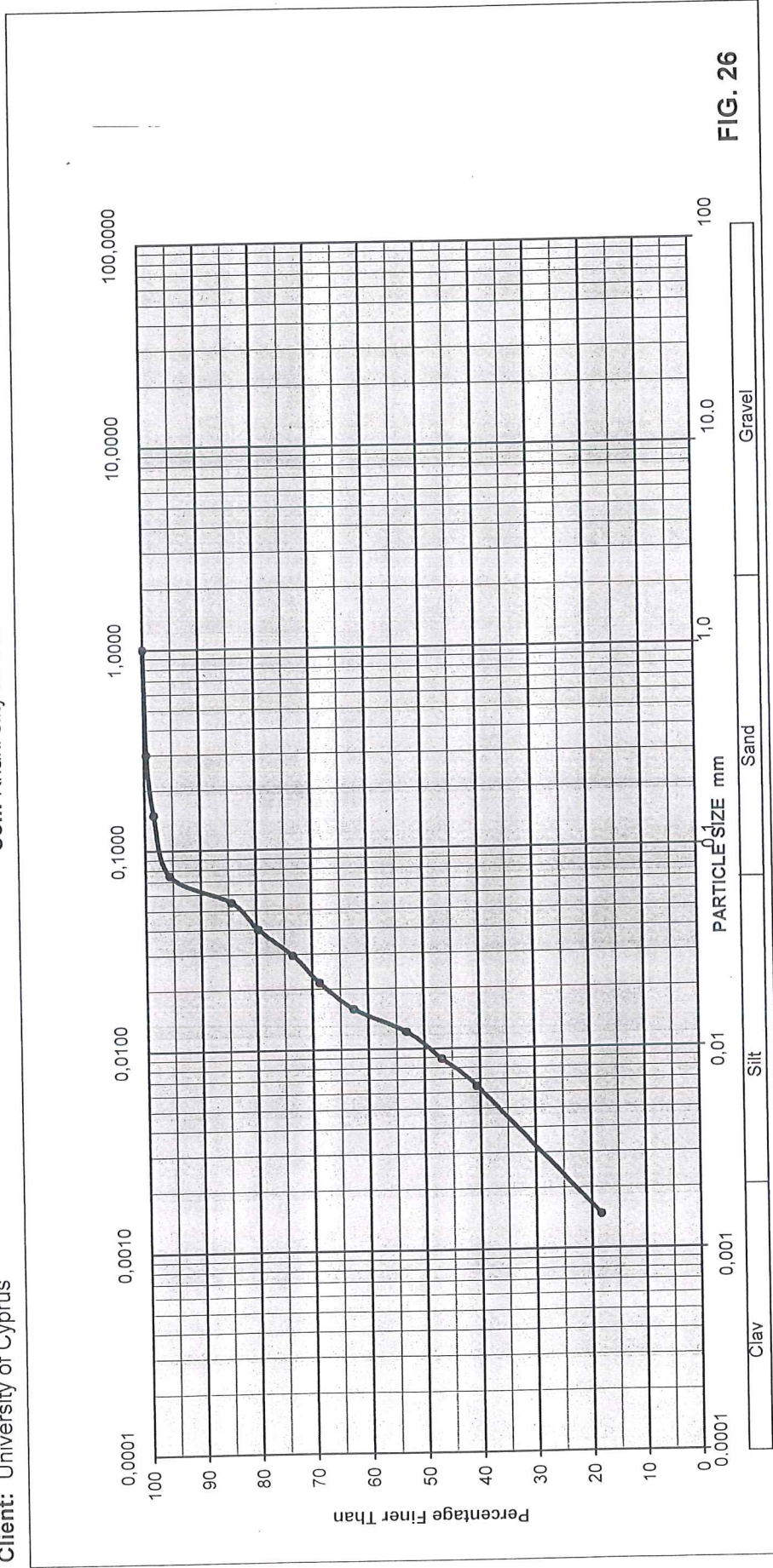


FIG. 26

SH Soil Engineering Ltd
P.O.Box 25457, 1310 Nicosia
Tel. 22663191, Fax 22663192

PARTICLE SIZE DISTRIBUTION

Date: 19/12/03
Operator:
Sieving: Wet

BH No.: N13
Depth: 5,3-6,7m
Soil: Grey sandy Gravel

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

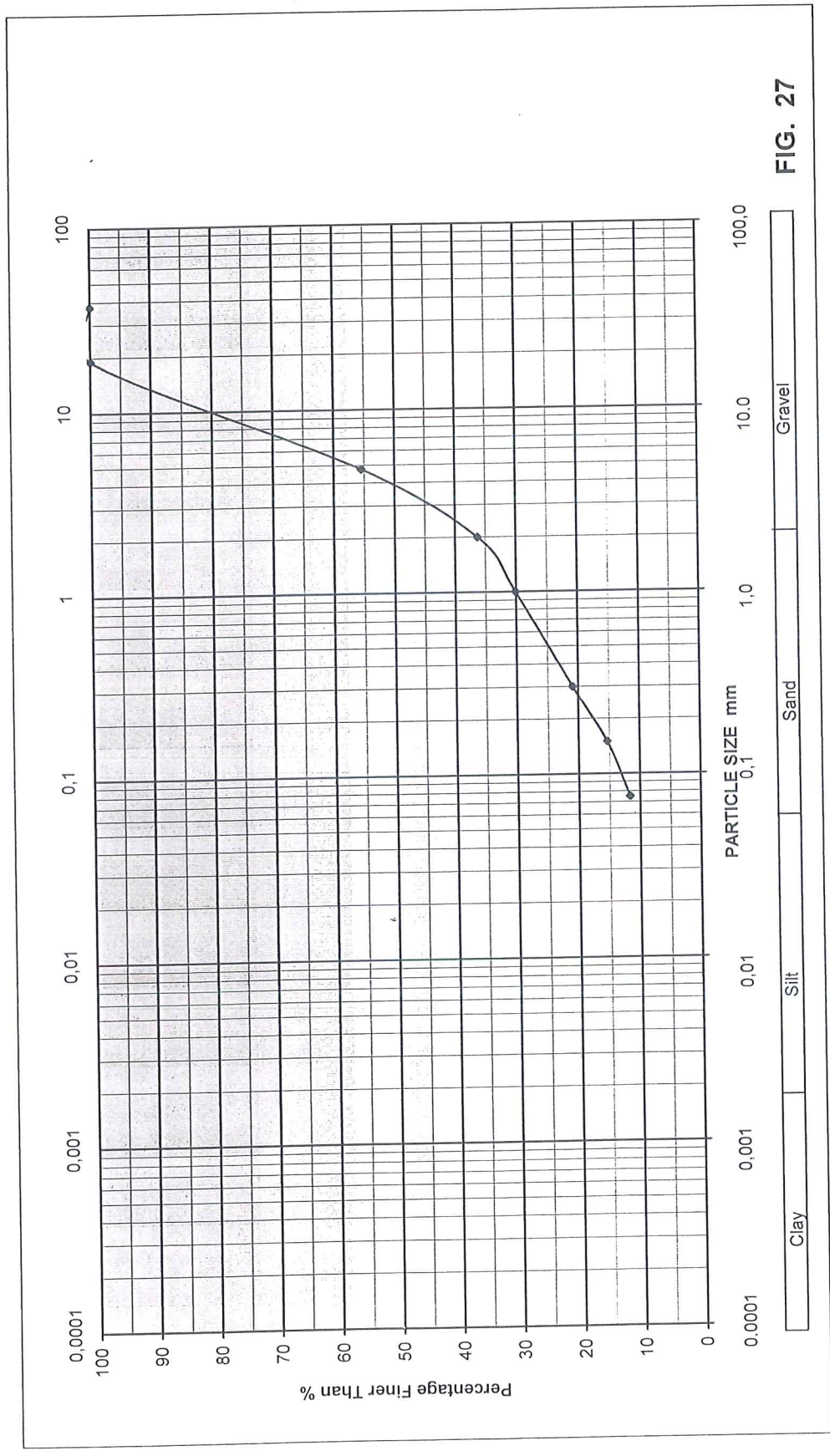


FIG. 27

SH Soil Engineering Ltd
P.O.Box 25457, 1310 Nicosia
Tel.No.22663191, Fax22663192

PARTICLE SIZE DISTRIBUTION

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

BH No.: N13
Depth: 7,0-7,45m
Soil: Grey sandy Gravel

Date: 19/12/03
Operator:
Sieving: Wet

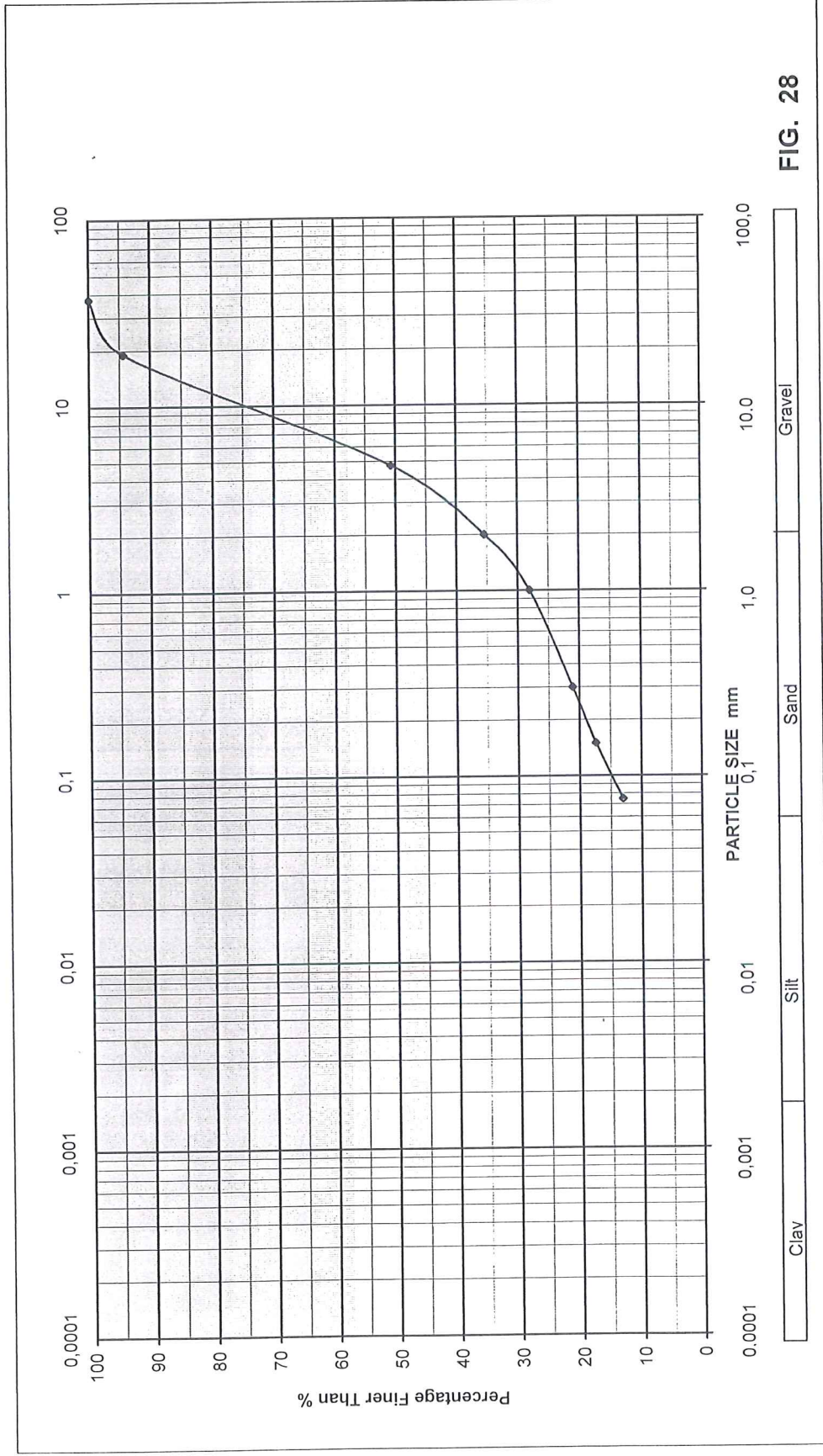


FIG. 28

SH Soil Engineering Ltd
P.O.Box 25457, 1310 Nicosia
Tel.No. 22663191, Fax22663192

PARTICLE SIZE DISTRIBUTION

Date: 19/12/03
Operator:
Sieving: Wet

BH No.: N13
Depth: 7,5-7,8m
Soil: Grey sandy Gravel

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

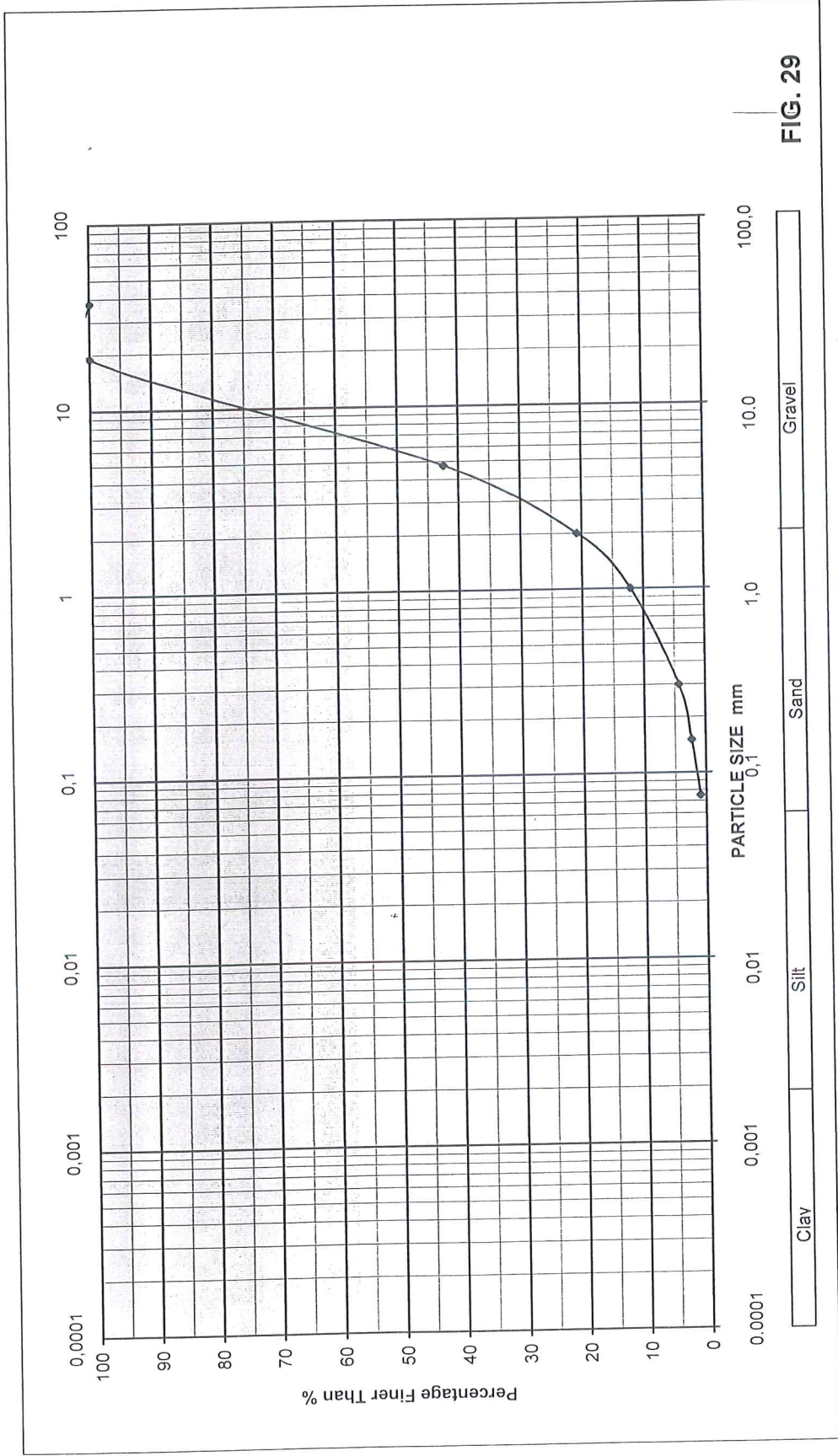


FIG. 29

SH Soil Engineering Ltd
P.O.Box 25457, 1310 Nicosia
Tel.No.22663191, Fax22663192

PARTICLE SIZE DISTRIBUTION

Wet Sieving and Hydrometer

Date: 29-31/12/03
Operator:
Steiving: Wet

BH No.: N13
Depth: 10,7-11,15m
Soil: Grey silty MARL

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

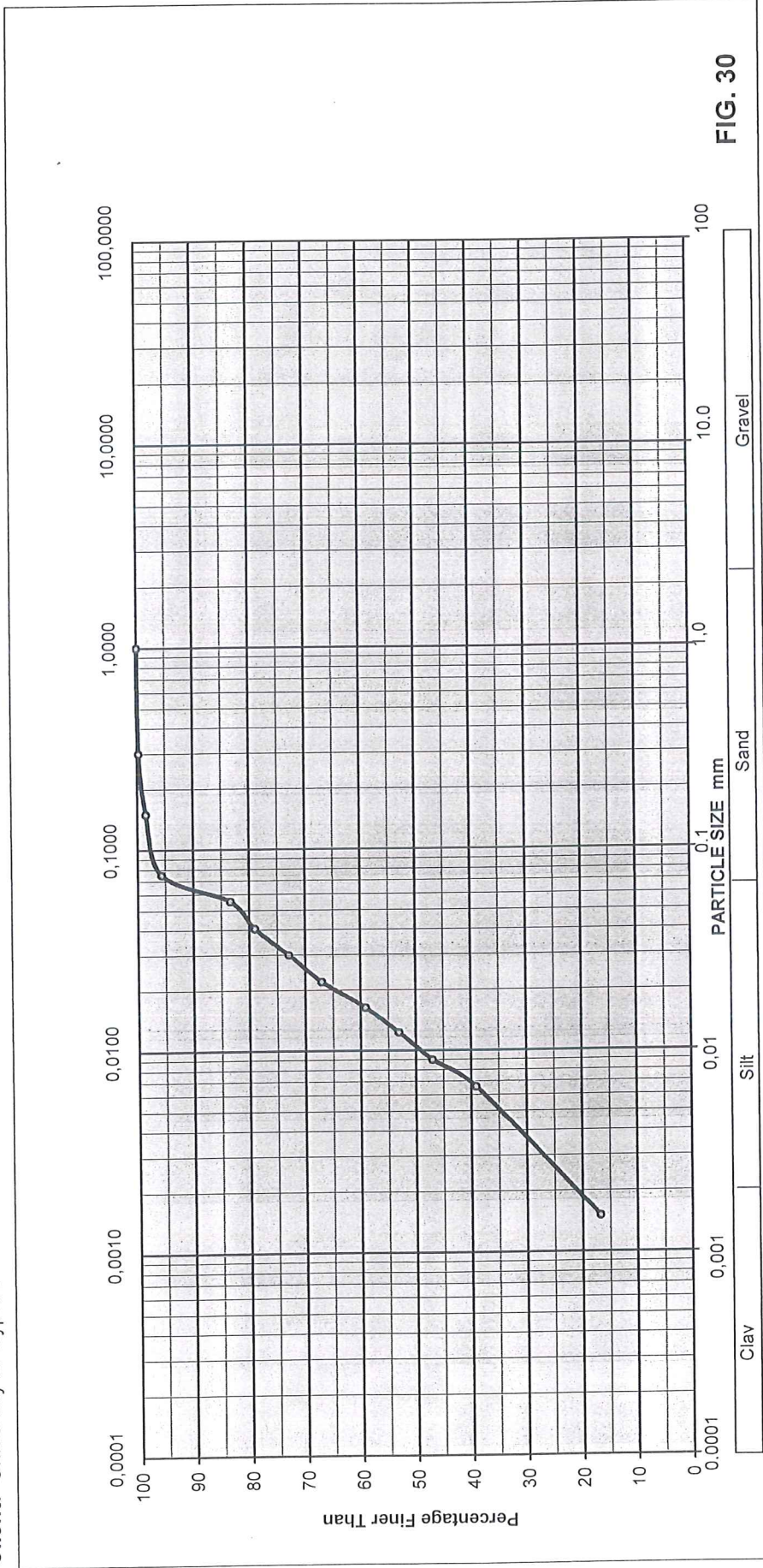


FIG. 30

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Tel. 22663191, Fax 22663192

UC3 HYDROMETER G N13-10,7

UNCONFINED COMPRESSION TEST

Project: University of Cyprus

BH No.: N9

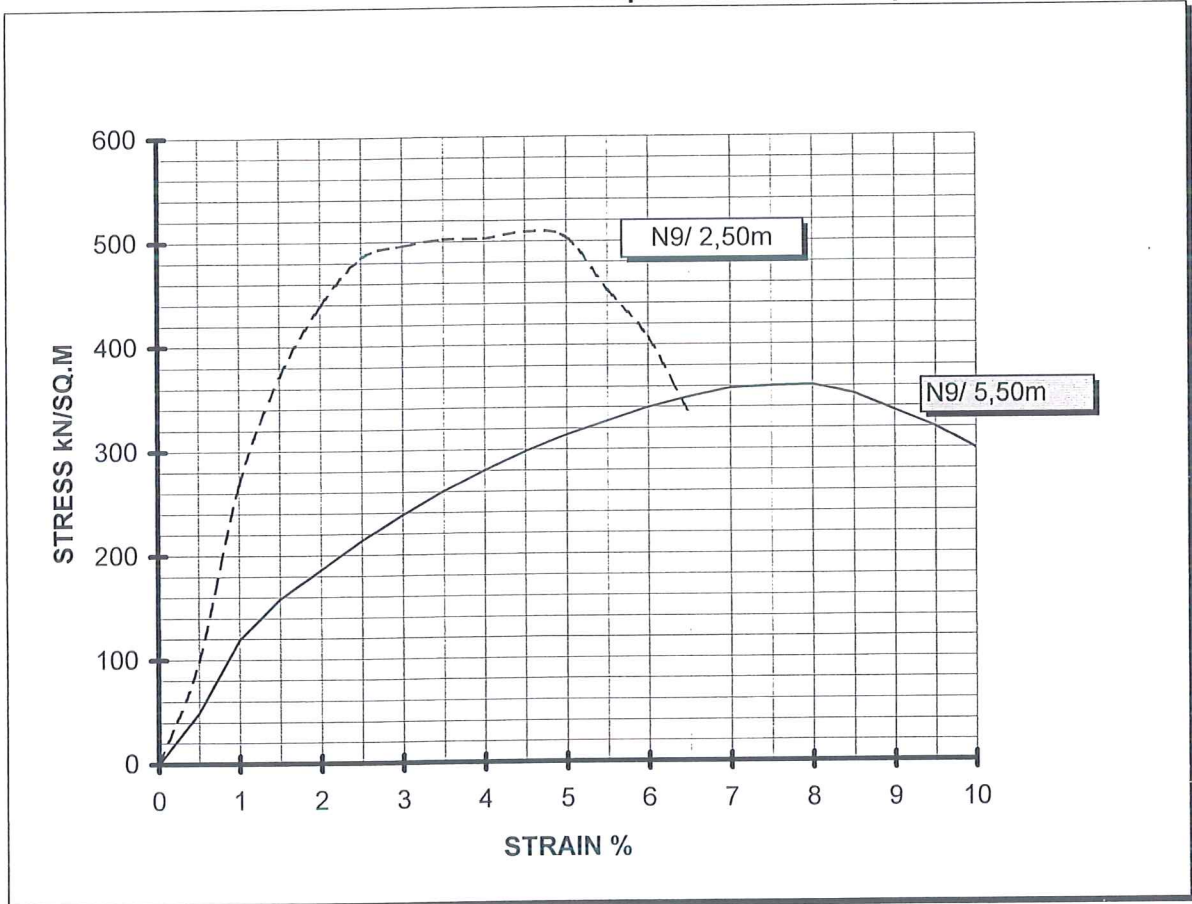
Date: 22/12/03

Site Location: University Campus - Site "3" **Depth:** 2,5&5,50m **Operator:**

Client: University of Cyprus

Soil: Khaki silty MARL

Sample Dia:34.5mm **Sample height:** 70mm



Specimen No.	BH No.	Depth m	Bulk Density kN/m ³	Natural Moisture Content %	Undrained Cohesion Cu kN/m ²
1	N9	2,50	19,30	28,78	255
2	N9	5,50	19,00	31,56	180

FIG. 31

UNCONFINED COMPRESSION TEST

Project: University of Cyprus

BH No.: N9& N10

Date: 22/12/03

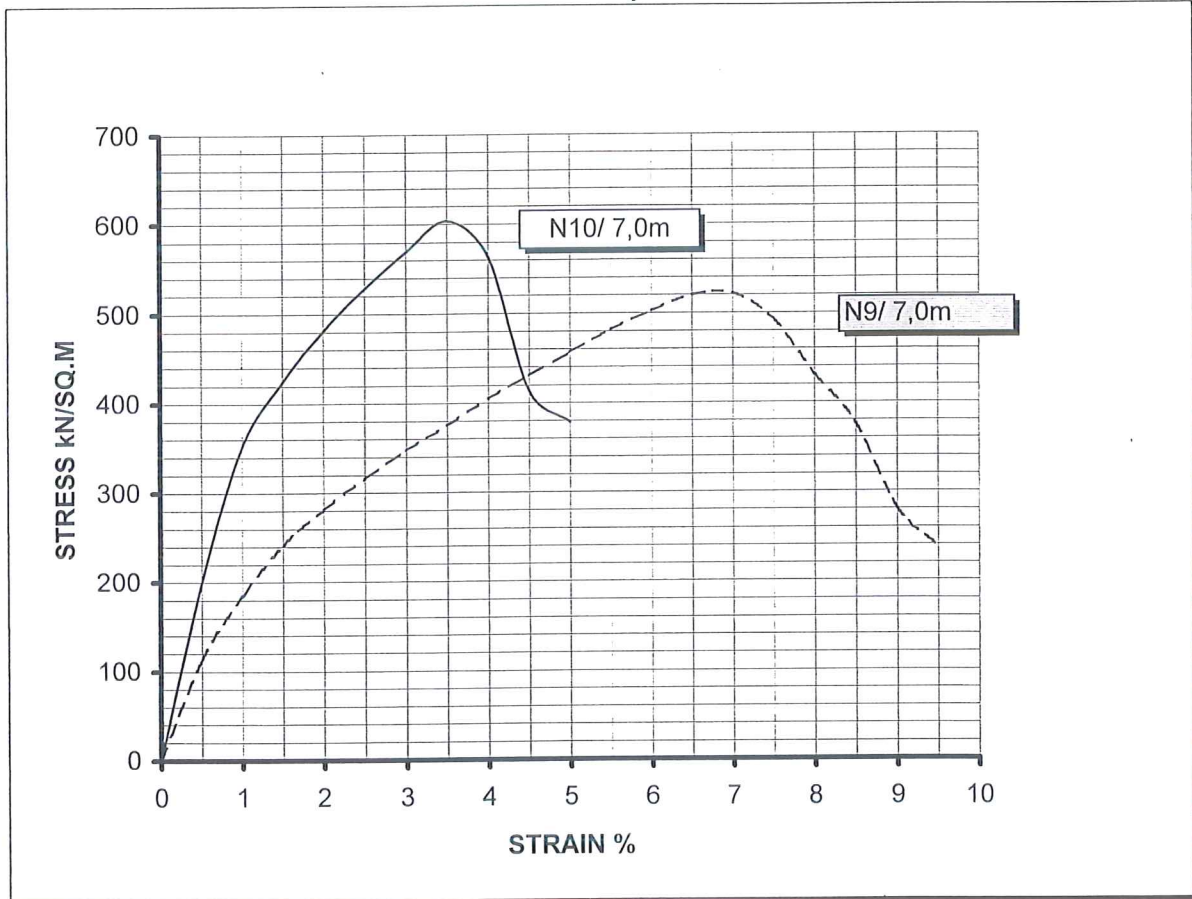
Site Location: University Campus - Site "3" **Depth:** 7,0&7,0m

Operator:

Client: University of Cyprus

Soil: Khaki silty MARL

Sample Dia:34.5mm **Sample height:** 70mm



Specimen No.	BH No.	Depth m	Bulk Density kN/m ³	Natural Moisture Content %	Undrained Cohesion Cu kN/m ²
1	N9	7,00	19,90	28,08	260
2	N10	7,00	19,50	29,62	300

FIG. 32

UNCONFINED COMPRESSION TEST

Project: University of Cyprus

BH No.: N11

Date: 23/12/03

Site Location: University Campus - Site "3"

Depth: 5,5&8,50m

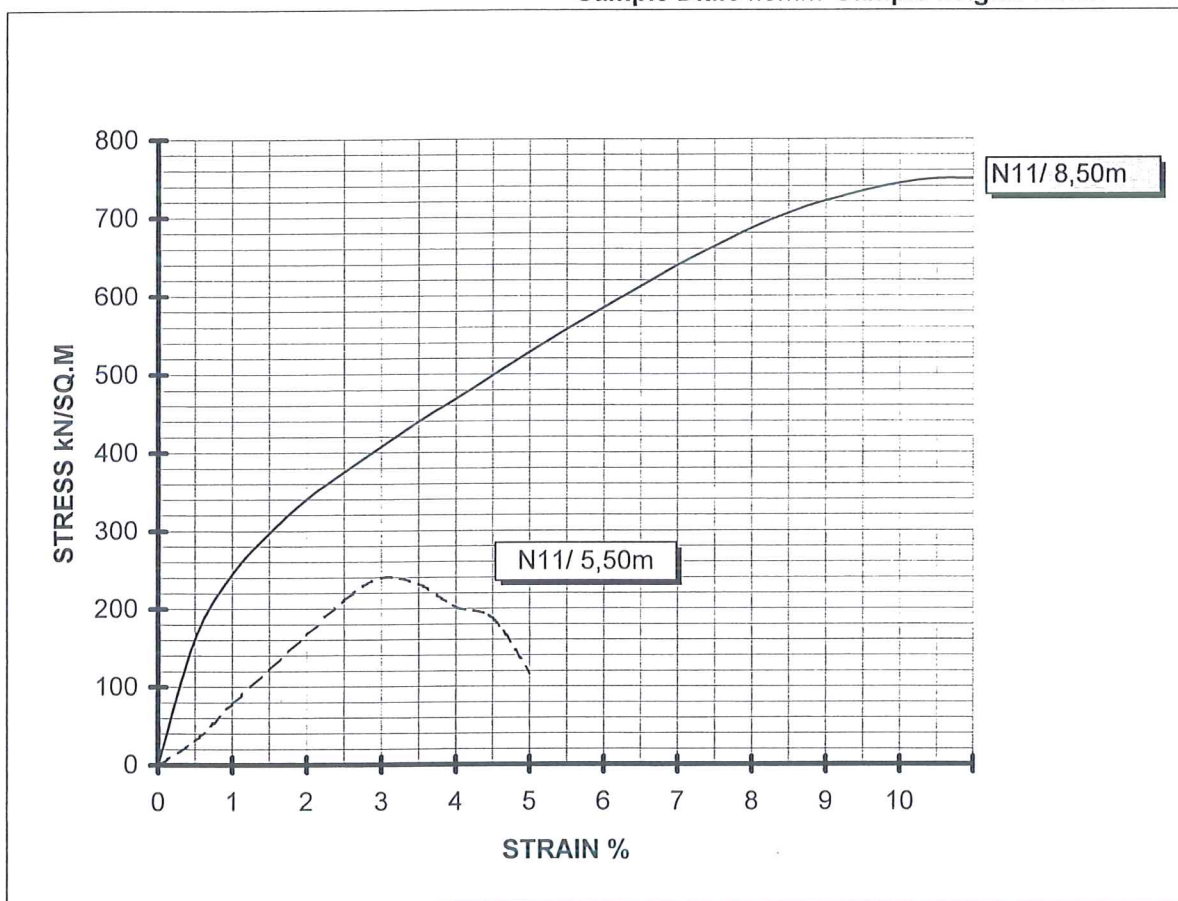
Operator:

Client: University of Cyprus

Soil: 5,5m: Brown/khaki silty Clay

8,5m: Khaki silty MARL

Sample Dia:34.5mm **Sample height:** 70mm



Specimen No.	BH No.	Depth m	Bulk Density kN/m ³	Natural Moisture Content %	Undrained Cohesion Cu kN/m ²
1	N11	5,50	21,8	17,81	120
2	N11	8,50	20,3	27,30	375

FIG. 33

UNCONFINED COMPRESSION TEST

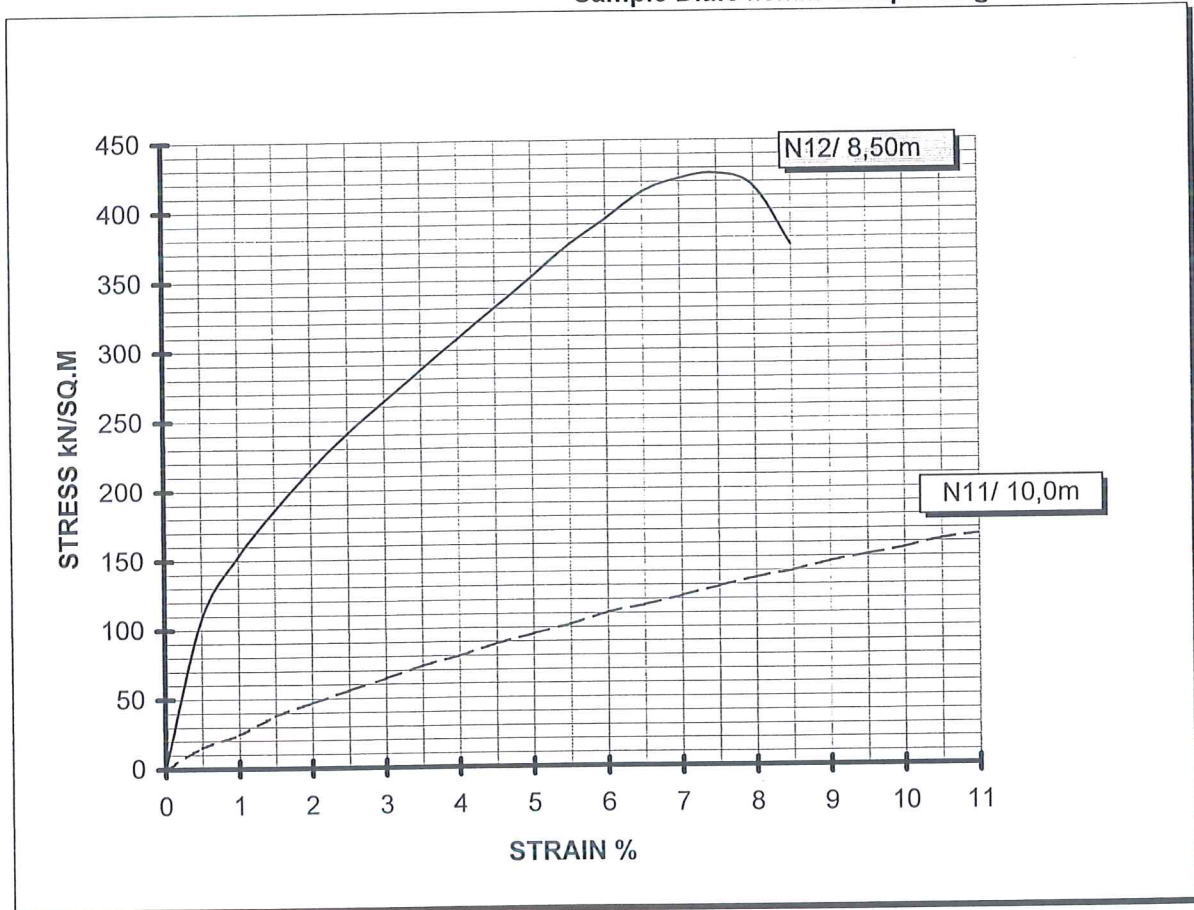
Project: University of Cyprus

BH No.: N11&N12 **Date:** 23/12/03

Site Location: University Campus - Site "3" **Depth:** 10,0&8,50m **Operator:**

Client: University of Cyprus

Soil: 10,0m: Khaki sandy silty MARL
8,5m: Khaki silty MARL slightly sandy
Sample Dia:34.5mm **Sample height:** 70mm



Specimen No.	BH No.	Depth m	Bulk Density kN/m ³	Natural Moisture Content %	Undrained Cohesion Cu kN/m ²
1	N11	10,00	20,1	31,72	83
2	N12	8,50	19,6	31,18	210

FIG. 34

UNDRAINED TRIAXIAL TEST

STRESS Vs STRAIN CURVES

Project: University of Cyprus

BH No.: N12

Date: 29/12/03

Site Location: University Campus - Site "3"

Depth: 10,0-10,45m

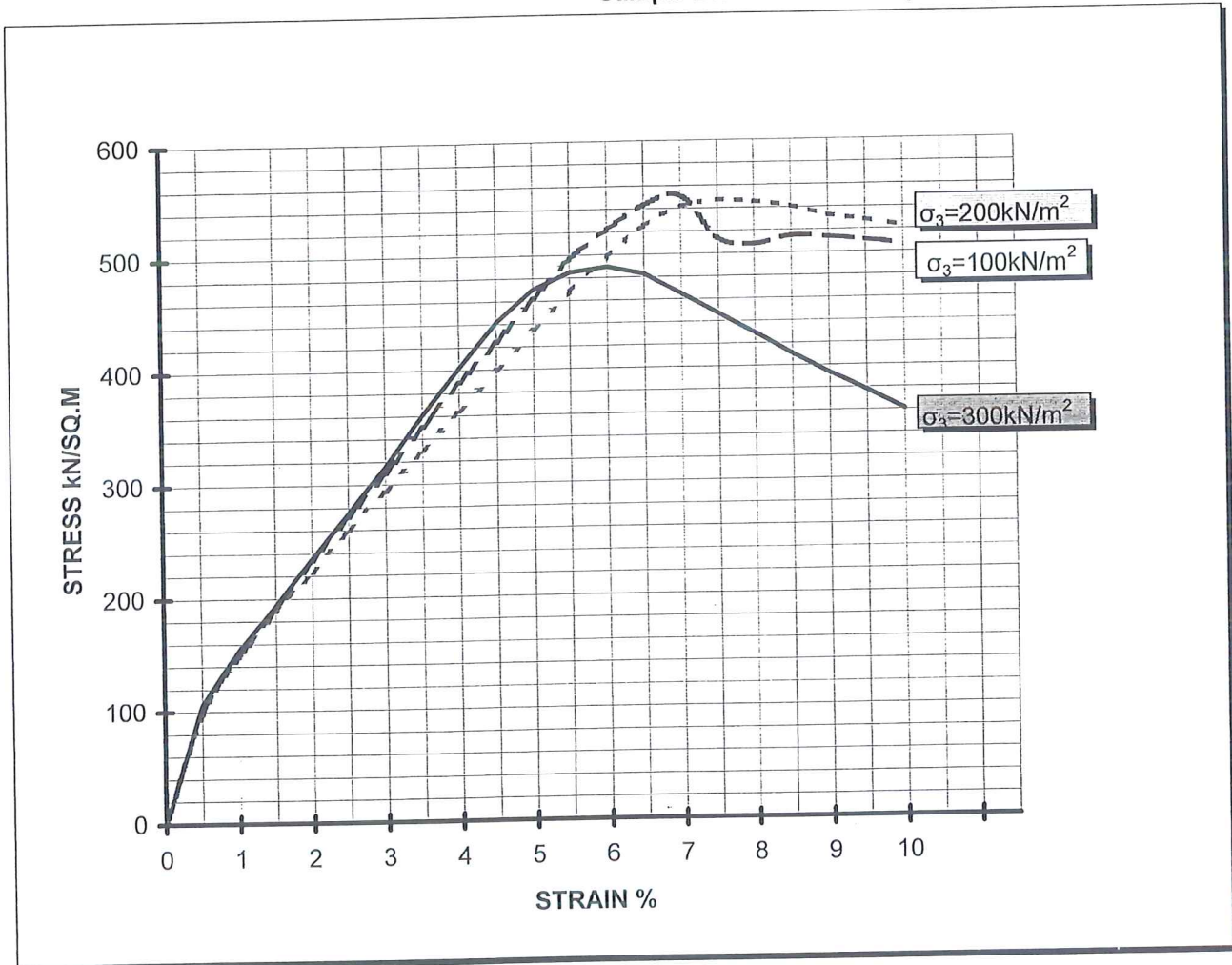
Operator:

Client: University of Cyprus

Soil: Khaki silty MARL

Sample Dia: 35mm

Sample height: 70mm



Specimen No.	Cell Pres. σ_3 kN/m ²	Bulk Dens. kN/m ³	Moist. Cont. %
1	100	19,4	32,33
2	200	19,5	31,61
3	300	19,5	32,54

FIG. 35

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Tel.No. 22663191, Fax 22663192

UNDRAINED TRIAXIAL TEST

STRESS Vs STRAIN CURVES

Project: University of Cyprus

BH No.: N13

Date: 29/12/03

Site Location: University Campus - Site "3"

Depth: 10,70-11,15m

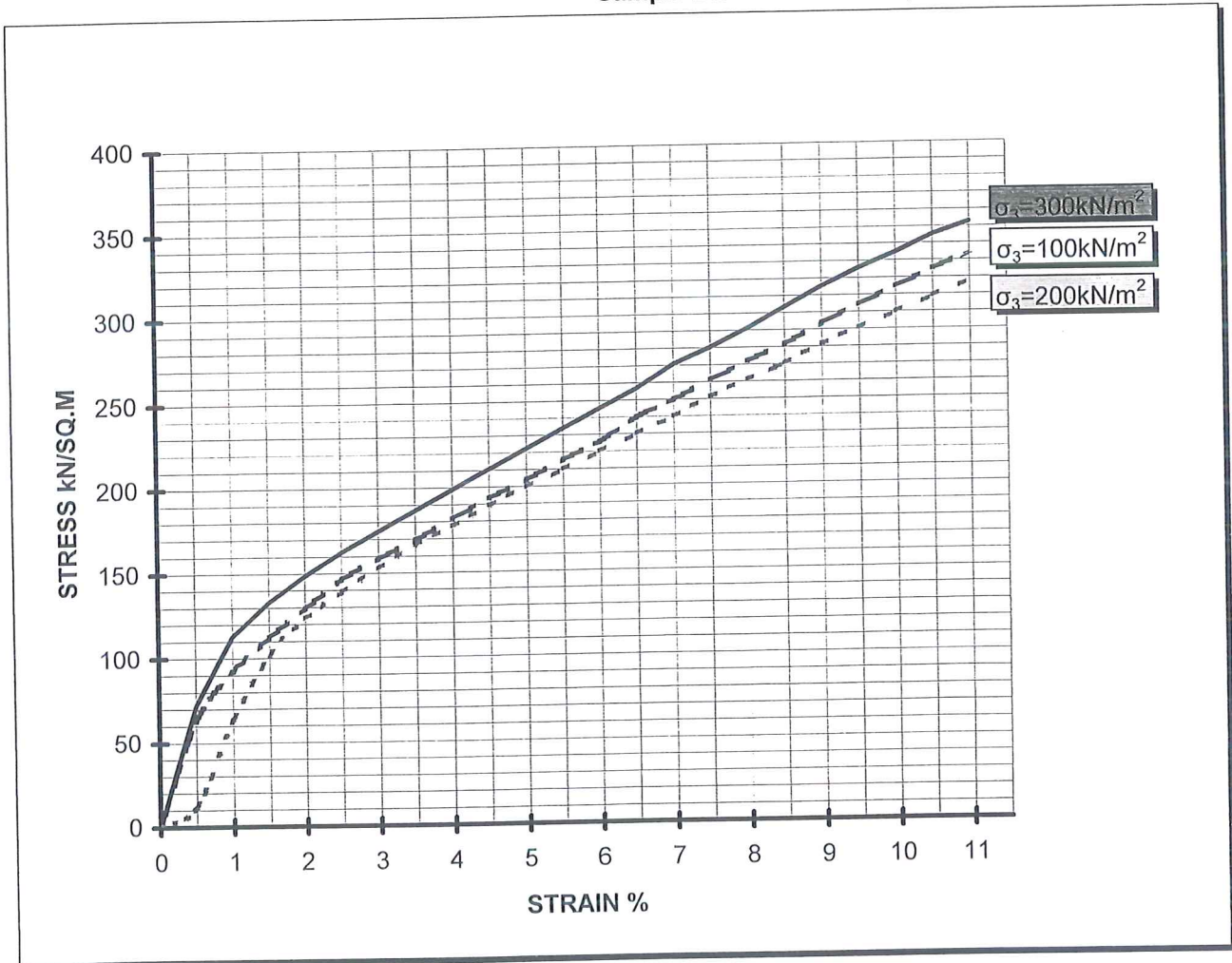
Operator:

Client: University of Cyprus

Soil: Grey silty MARL

Sample Dia: 35mm

Sample height: 70mm



Specimen No.	Cell Pres. σ_3 kN/m ²	Bulk Dens. kN/m ³	Moist. Cont. %
1	100	19,5	31,58
2	200	19,3	31,92
3	300	19,5	31,38

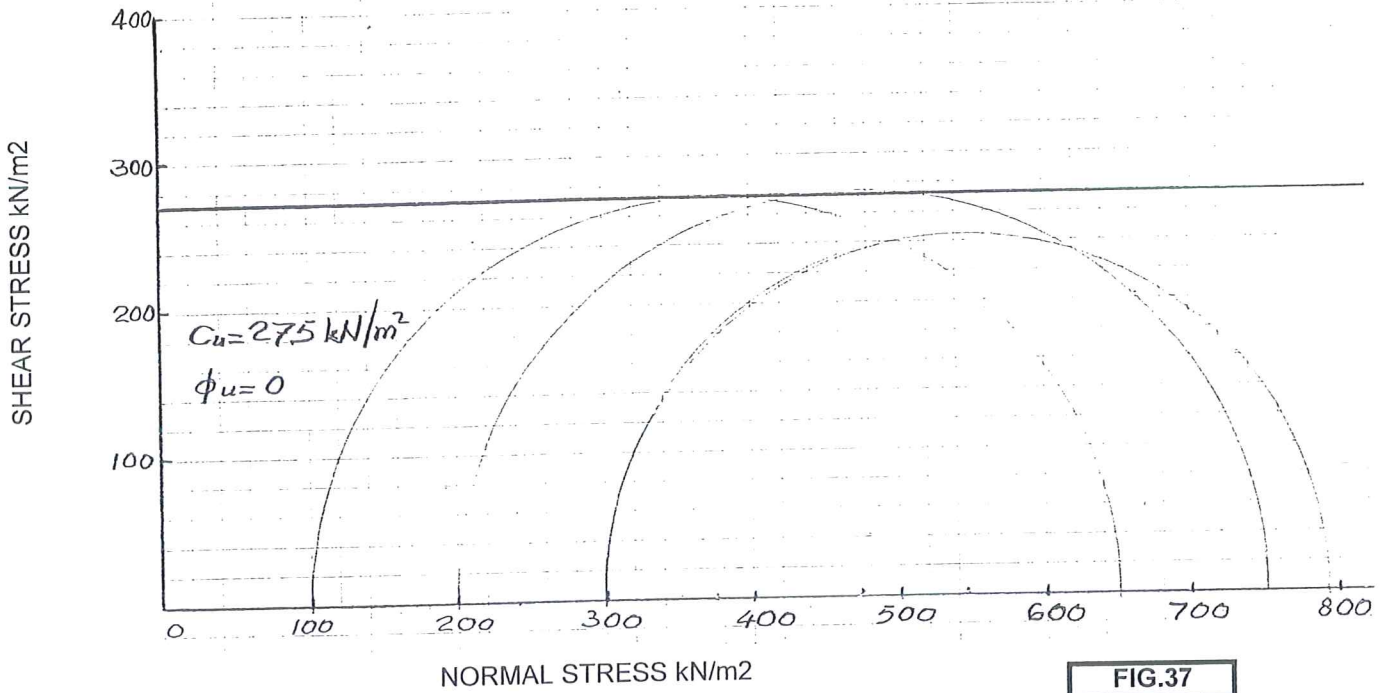
FIG. 36

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P.O.Box 25457, 1310 Nicosia
Tel.No. 22663191, Fax 22663192

UNDRAINED TRIAXIAL TEST
MOHR CIRCLE OF STRESSES

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

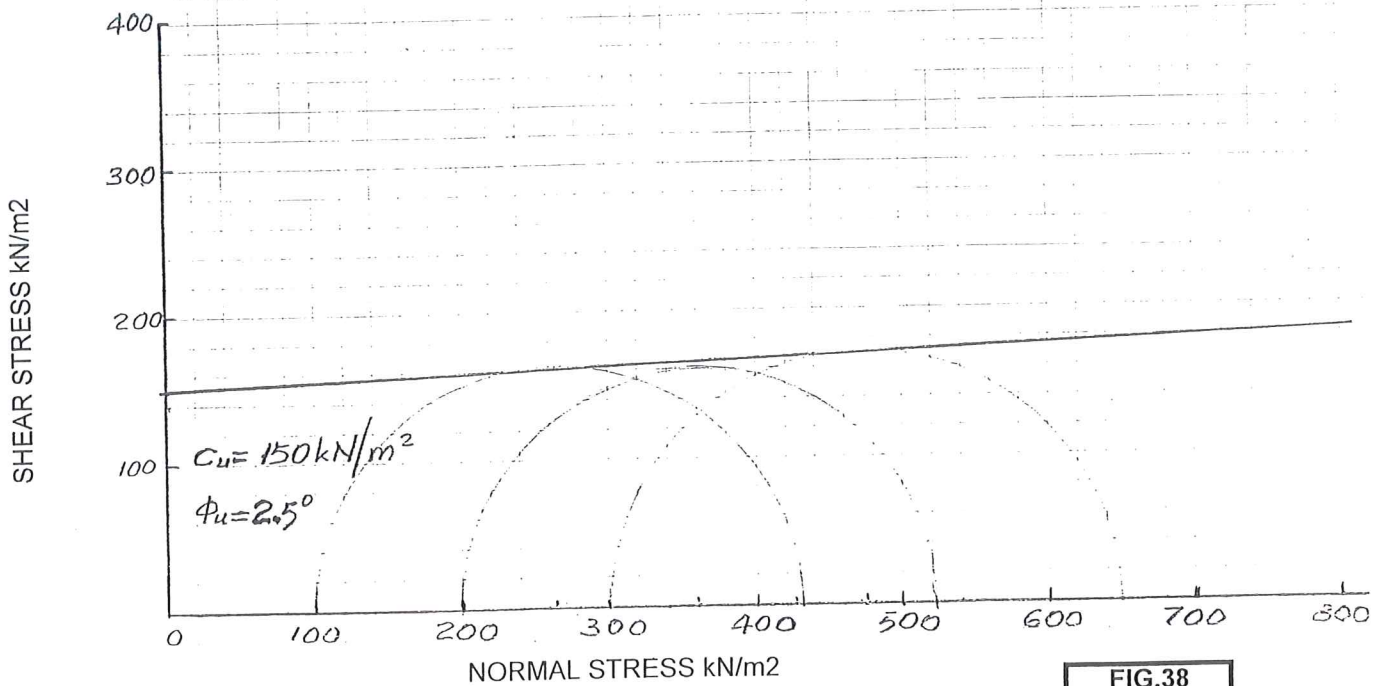
BH No.: N12 **Date:** 29/12/03
Depth: 10,0-10,45m **Operator:**
Soil: Khaki silty MARL
Sample Dia: 35mm **Sample height:** 70mm



UNDRAINED TRIAXIAL TEST
MOHR CIRCLE OF STRESSES

Project: University of Cyprus
Site Location: University Campus - Site "3"
Client: University of Cyprus

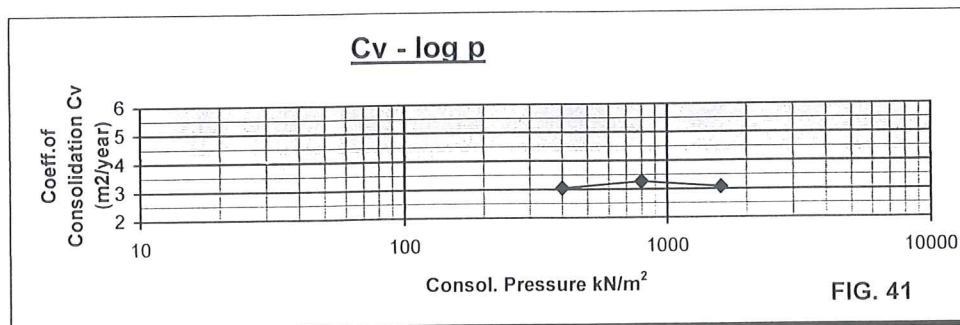
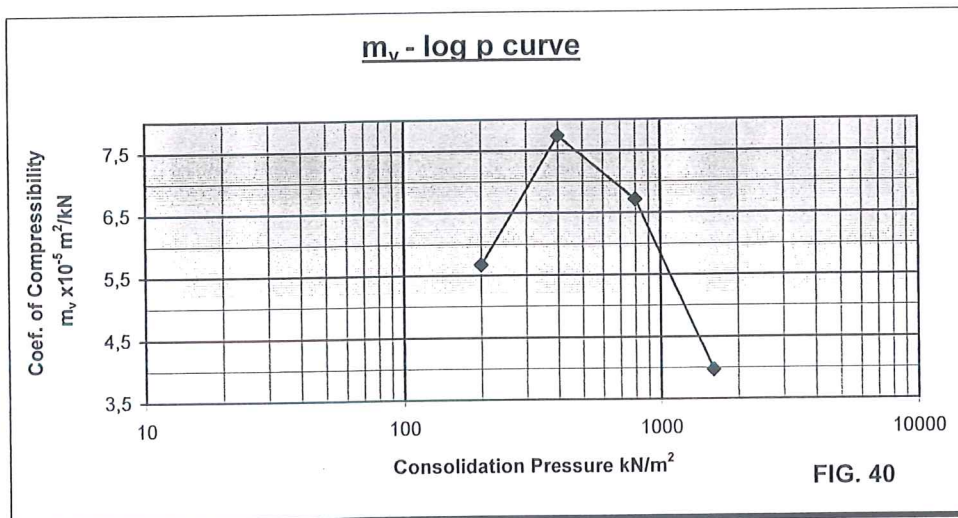
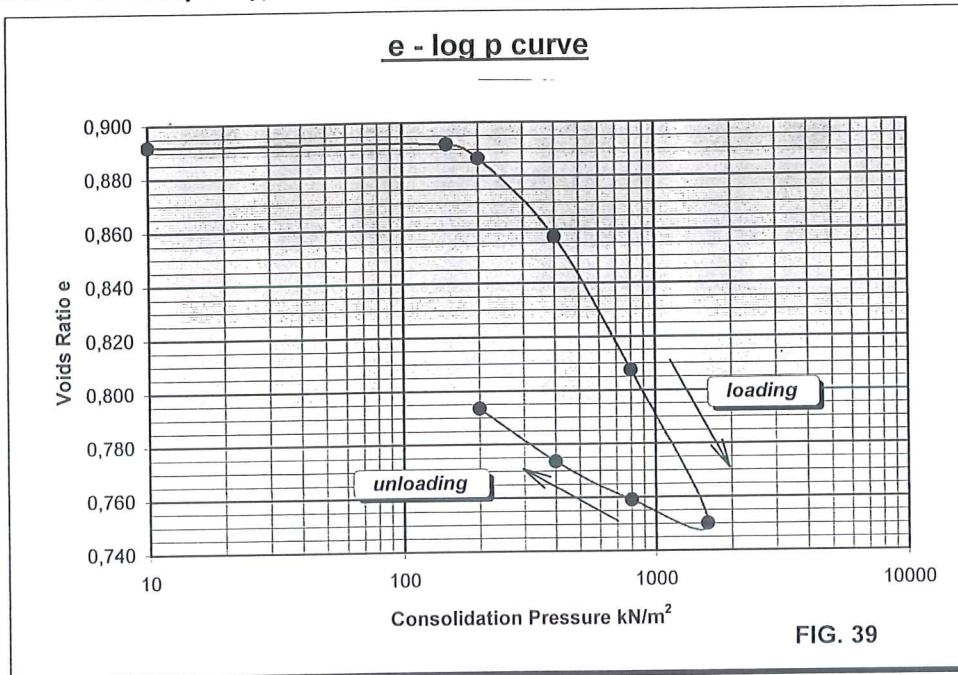
BH No.: N13 **Date:** 29/12/03
Depth: 10,70-11,15m **Operator:**
Soil: Grey silty MARL
Sample Dia: 35 mm **Sample height:** 70mm



CONSOLIDATION TEST

Project: University of Cyprus
 Site Location: University Campus - Site "3"
 Client: University of Cyprus

BH No.: N12 Date: 23 to 31/12/03
 Depth: 10,0-10,45m Operator:
 Soil: Khaki silty MARL



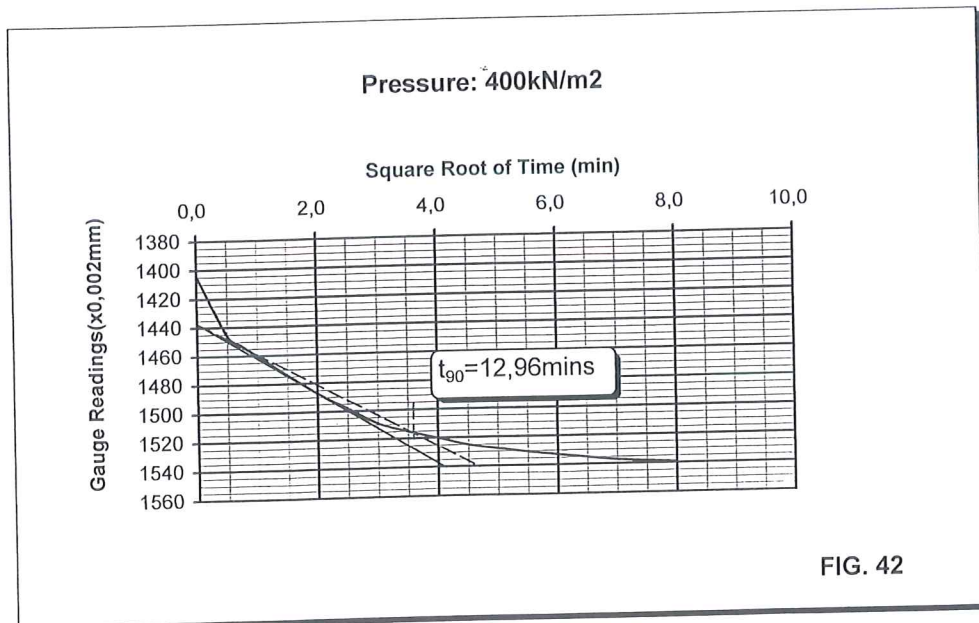
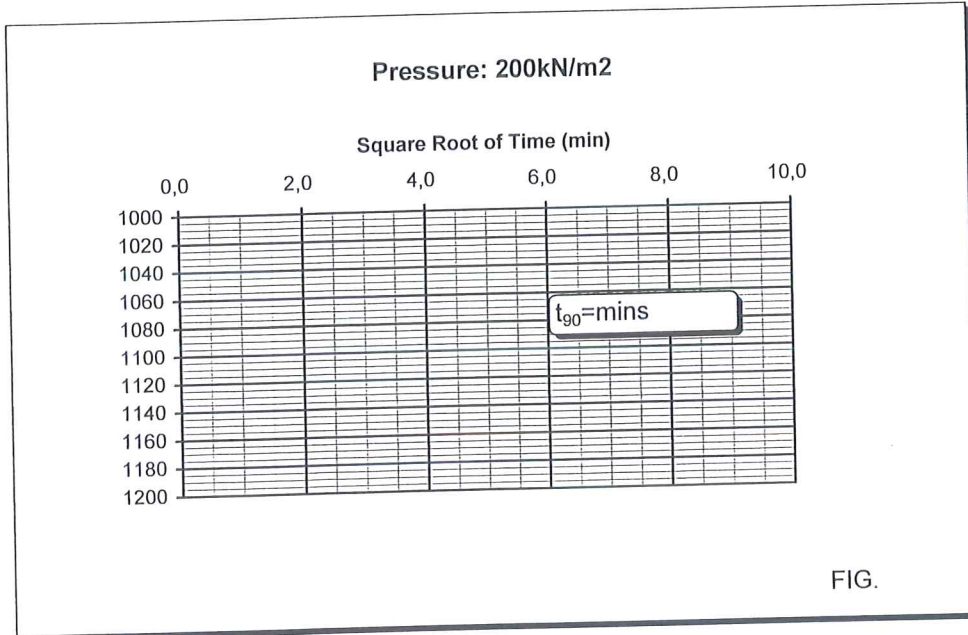
CONSOLIDATION TEST

Consolidation Vs Sq.Root Time Curves

Project: University of Cyprus
 Site Location: University Campus - Site "3"
 Client: University of Cyprus

BH No.: N12
 Depth: 10,0-10,45m
 Soil: Khaki silty MARL

Date: 23 to 31/12/03
 Operator:



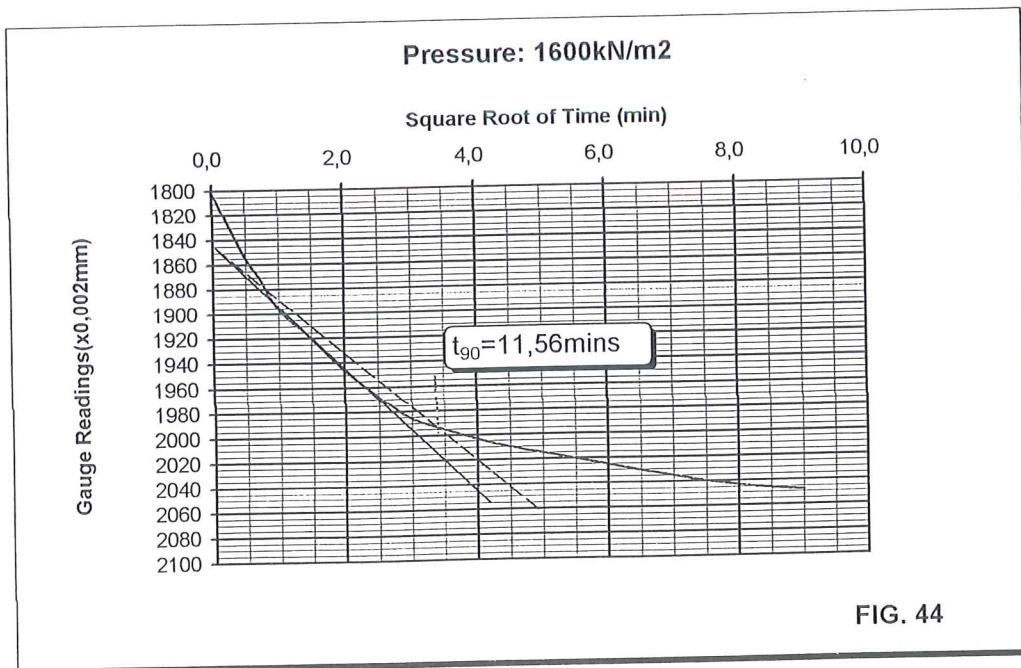
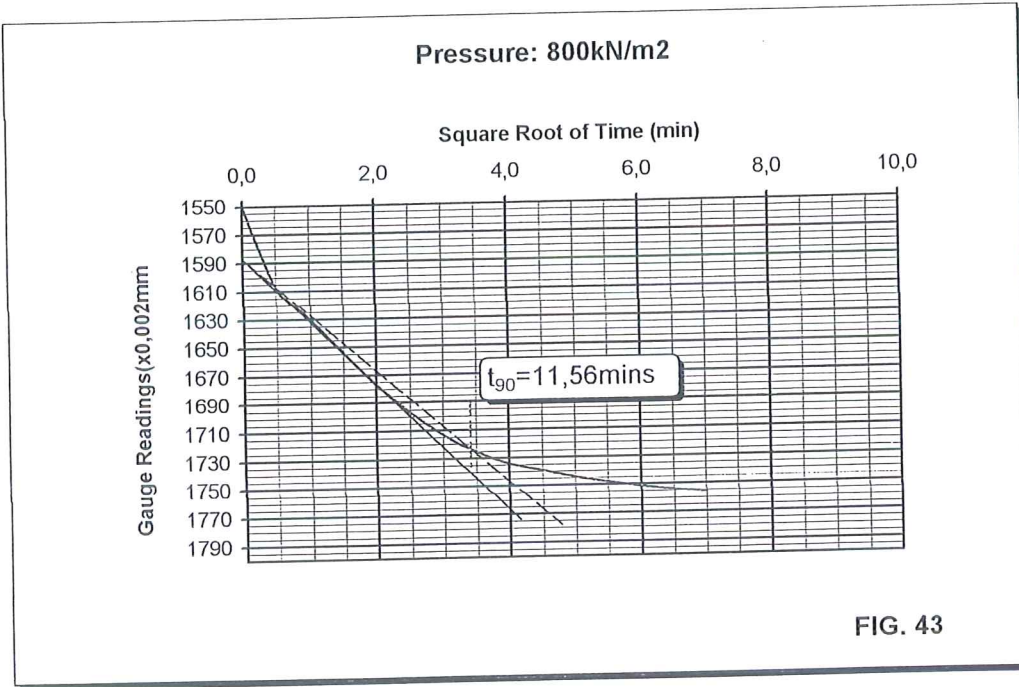
CONSOLIDATION TEST

Consolidation Vs Sq.Root Time Curves

Project: University of Cyprus
 Site Location: University Campus - Site "3"
 Client: University of Cyprus

BH No.: N12
 Depth: 10,0-10,45m
 Soil: Khaki silty MARL

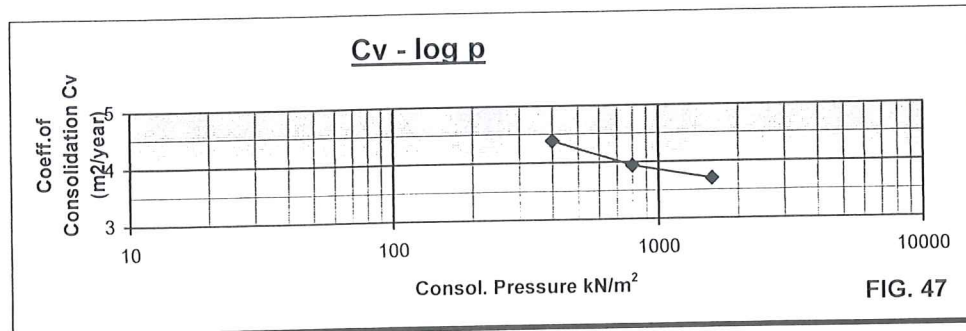
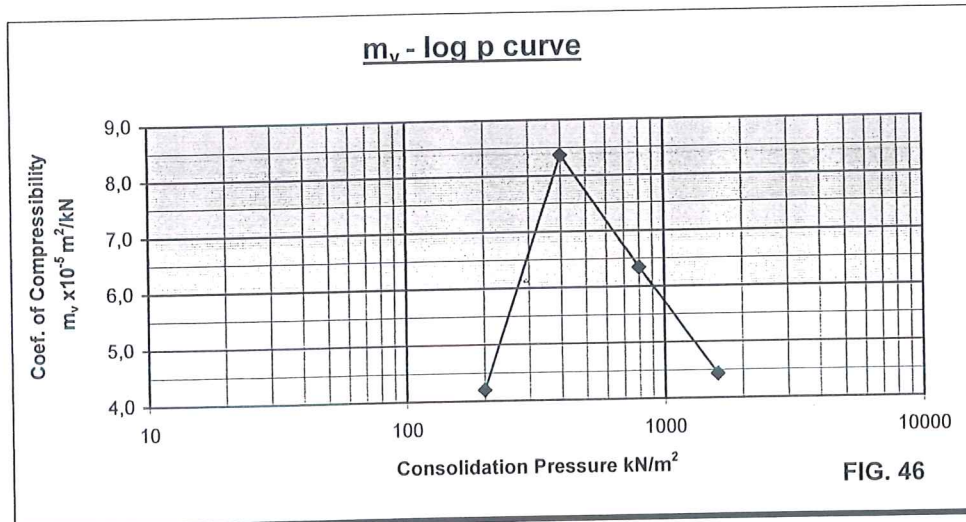
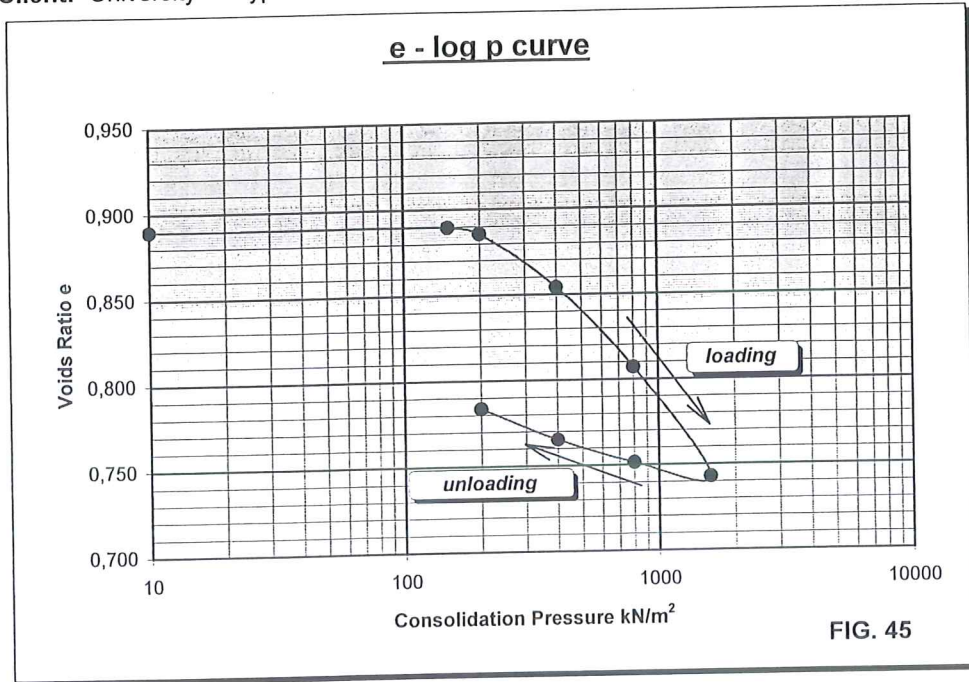
Date: 23 to 31/12/03
 Operator:



CONSOLIDATION TEST

Project: University of Cyprus
 Site Location: University Campus - Site "3"
 Client: University of Cyprus

BH No.: N13 Date: 23 to 31/12/03
 Depth: 10,7-11,15m Operator:
 Soil: Grey silty MARL



CONSOLIDATION TEST

Consolidation Vs Sq.Root Time Curves

Project: University of Cyprus

BH No.: N13

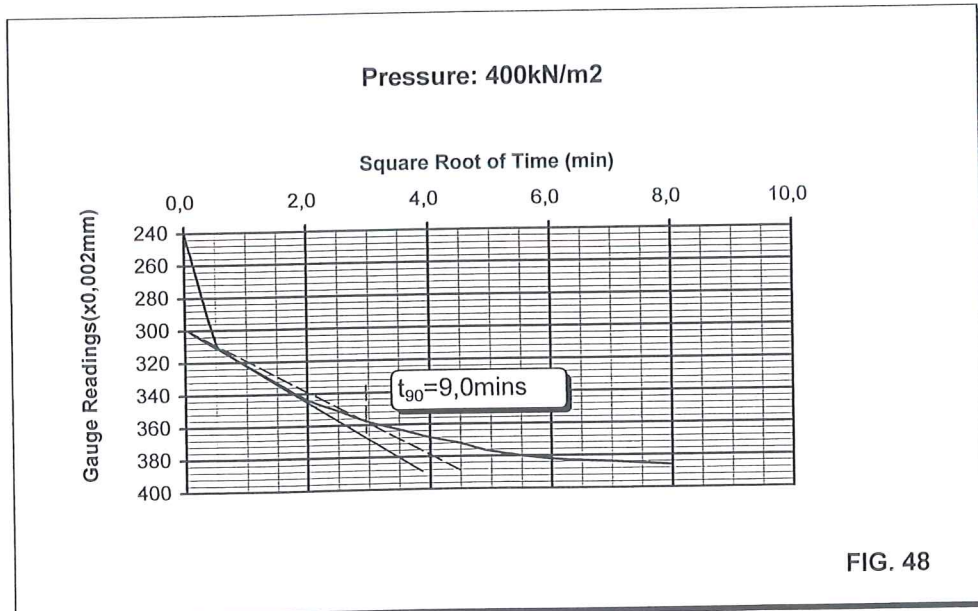
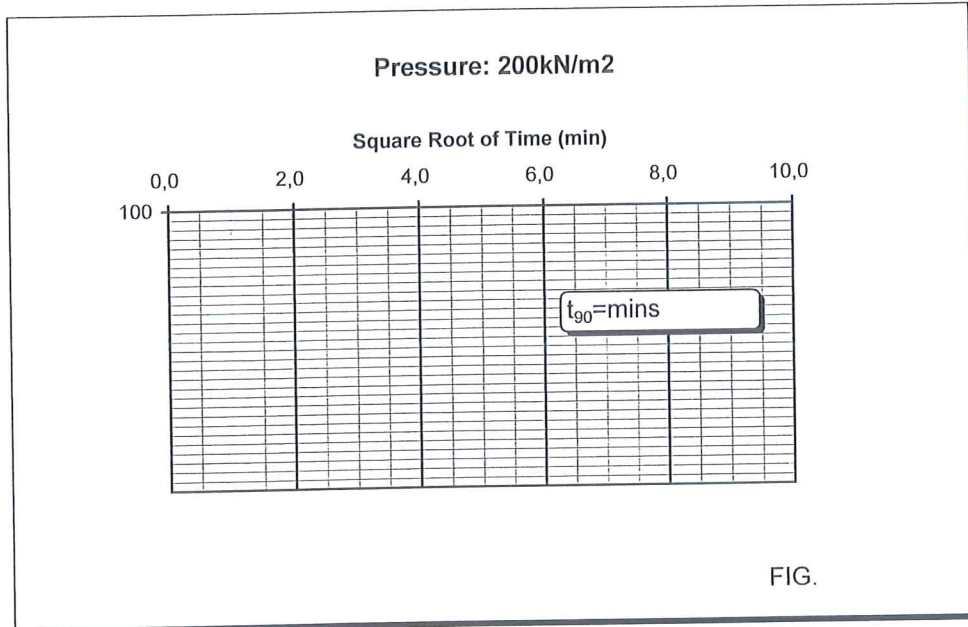
Date: 23 to 31/12/03

Site Location: University Campus - Site "3"

Depth: 10,70-11,15m Operator:

Client: University of Cyprus

Soil: Grey silty MARL

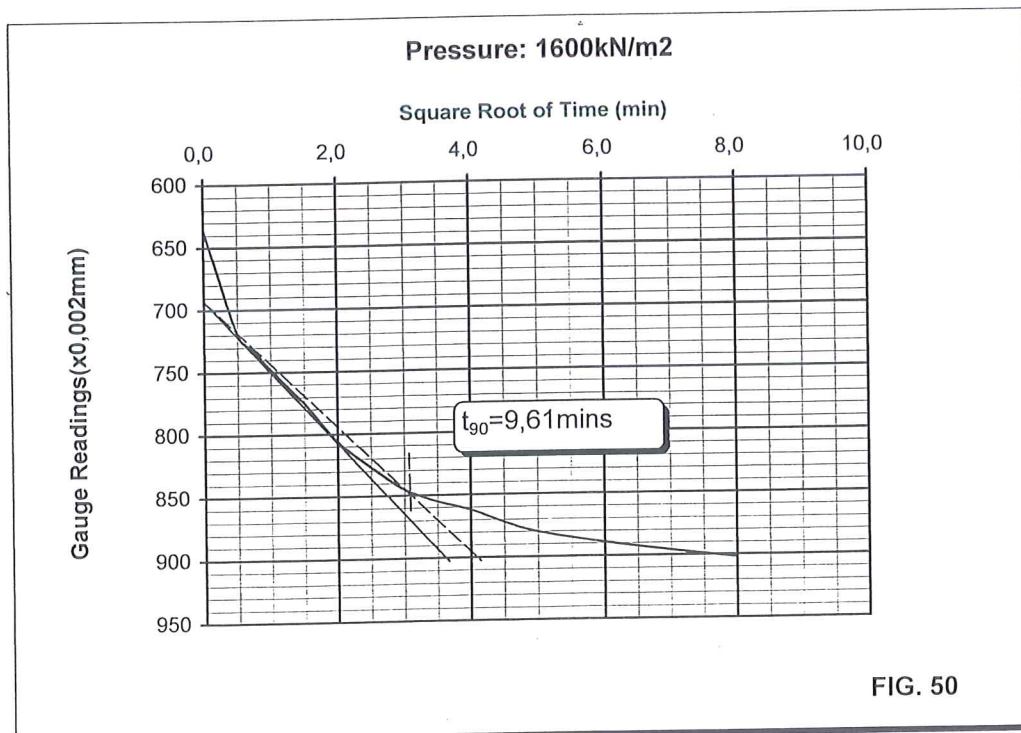
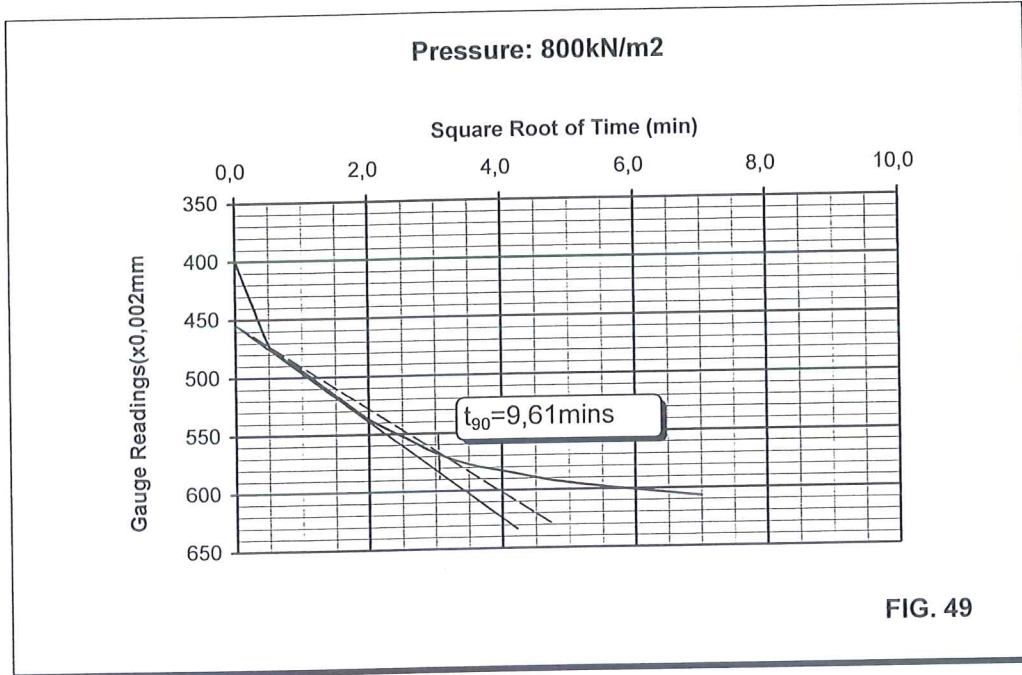


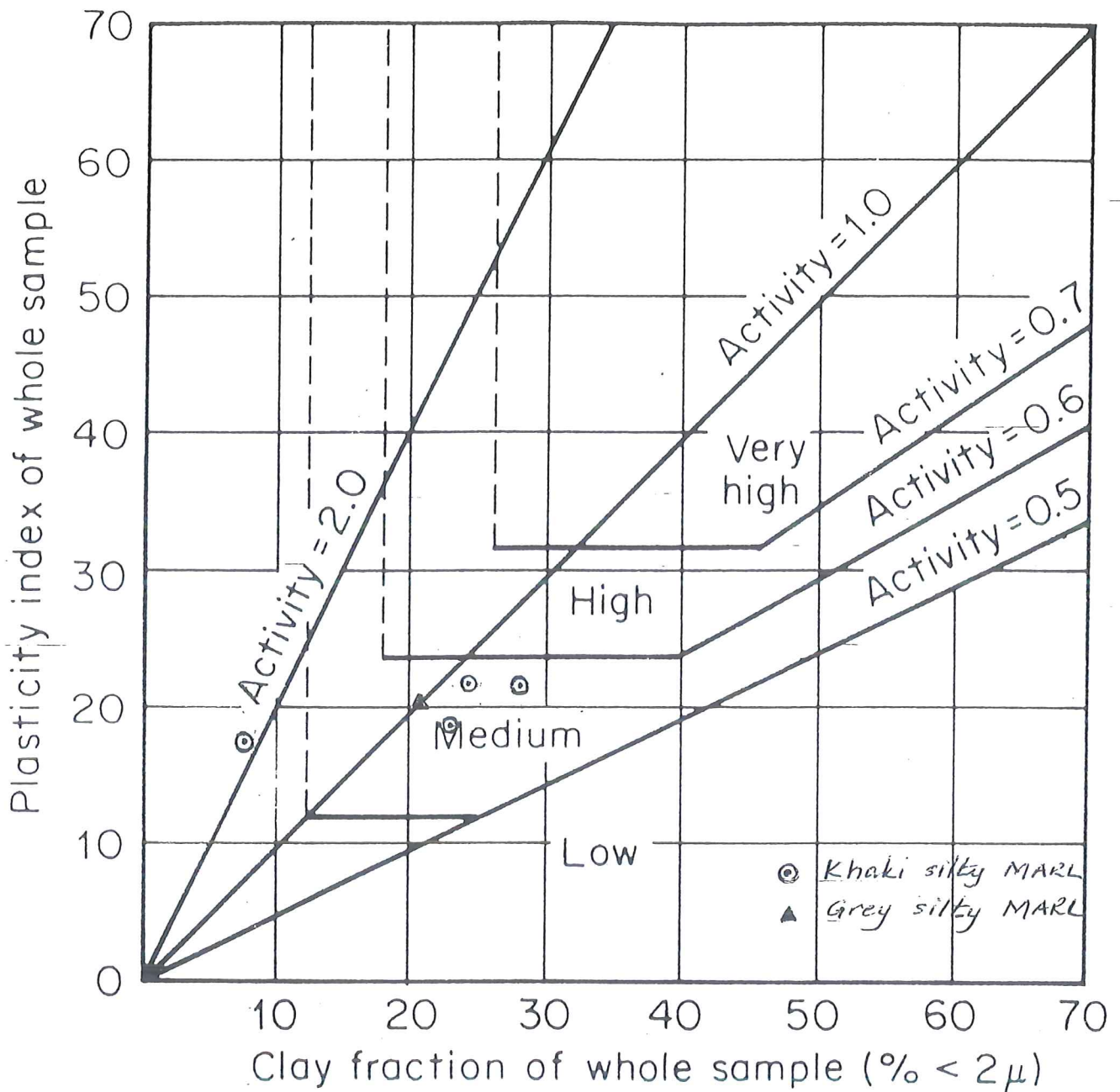
CONSOLIDATION TEST

Consolidation Vs Sq.Root Time Curves

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Potential expansiveness

Inch per foot of soil*

Very high

1.0

High

0.5

Medium

0.25

Low

0

*After Van der Merwe (1975).^{65b}

FIG. 10.40 Proposed modified chart for determining expansiveness of soils. [From Williams and Donaldson (1980)^{65a}; after Van der Merwe (1975)^{65b}.]