AL-Shams Laboratory
For Structural Tests

Soil Investigation Report for Turkish Embassy Rehabilitation / Baghdad Province

AL-SHAMS LABORATORY FOR STRUCTURAL TESTS
Contact:
Address: Buil.14/1, Str.12, Sec.607,Al-Mansour District, Baghdad, Iraq
Phone: +9647705033350 +9647901827013
Email: alialshamoosi@alshamslaboratory.com
Website: www.alshamslaboratory.com

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Client: Resident Engineer in the Turkish Embassy

AL-Shams Laboratory

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Contents:

Abbreviations:

ASTM	American Society for Testing and Materials
BS	British Standards
$CaSO_4.2H_2O$	Gypsum content, %
c	Cohesion, kPa
D.S.	Disturbed Samples
e	Void ratio
G_s	Specific gravity
k	Permeability coefficient, m/s
L.L.	Liquid limit, %
N	No. of blows of SPT
N.G.L	Natural ground level
P.I.	Plasticity index, %
P.L.	The plastic limit, %
P_c	Pre-consolidation pressure, kPa
P_o	Overburden pressure, kPa
S.S.	The Split Spoon Samples
SO_3	Sulfate content, %
SPT	Standard penetration test
T.S.S.	Total Soluble Salts, %
U.S.	Undisturbed Samples
φ	The angle of shearing resistance, degree

Soil Investigation Report for Turkish Embassy Rehabilitation / Baghdad Province

1. Introduction:

This report summarizes the findings from a soil investigation done by AL-Shams laboratory for structural testing to carry out the soil investigation report for the Turkish Embassy site in Baghdad Province. The objective of this report is to identify and provide an assessment of the variability of the subsoil as required by the client.

The scope of work included the following:

Review of obtainable confirmed data to the site.

- Conduct a soil investigation that consists of drilling, and securing representative samples.

- Field Standard Penetration Tests (SPT).

- Collecting disturbed and undisturbed soil samples if applicable for visual inspection and for conducting the basic laboratory testing of select soils.

- Chemical analysis of soil samples.

- Perform a geotechnical engineering analysis regarding the proposed construction, using the information obtained from the subsurface investigation and laboratory testing.

- Preparing this report of our investigation, including conclusions, and recommendations for the geotechnical engineering aspects of the proposed construction in the project.

2. Authorization:

AL-Shams laboratory for structural testing is authorized by the Resident Engineer in the Turkish Embassy to carry out the geotechnical investigation & laboratory testing of the project.

3. Site Location and Geological Description:

3.1 Site location

This subsoil investigation was carried out in the site within the soil investigation report for the Turkish Embassy Rehabilitation site / Baghdad Province). In general, the site is almost a regular area within the Al-Waziriyah area, as shown in Figures (1-2).

Fig. (1): The drilling machine which was used in the site

Fig. (2): Secured samples extracted from borehole No.2 in the site

3.2 The Geological History of Baghdad

Baghdad city is located within the Mesopotamian Delta plan, which is an unfolded zone in general and extends into the middle of the Iraq area. This area is covered with recent Tigris River sediments, which were carried by the sequence of floods of the river. The thickness of these sediment deposits is changing towards the south and southern, west, that's because of the effect of the tectonic faults. The other topography disappeared because of erosion and weathering. The geological age of this area in the recent period during the Tertiary-Quaternary is about ten thousand years to 63 million years.

Because of the humidity active and the change in the river path, all-cause many influences and differences in the soil contents, which causes variations in both vertical and horizontal directions. The sequence of soil layers in Baghdad city declares three major horizons. The upper layer consists of fill material followed by a cohesive layer and the third one consists of non-cohesive soil.

4. Site Exploration:

4.1 Drilling and Sampling:

Drilling was done by using a drilling machine provided with a wash rotary drilling method according to the requirements of the specification (ASTM D 1452-03) for boreholes. The diameter of drilled boreholes is (10 cm). The disturbed samples (D.S.) were collected from the cutting of the auger at any depth. The undisturbed samples (U.S.) were obtained by Shelby tubes due to the nature of the soil. The split spoon samples were obtained from the standard split spoon used in the standard penetration test which was performed at different intervals depending on the stratifications of soil.

4.2 Number of Boreholes:

The three boring points were assigned and located by the concerned authority represented by the Turkish Embassy site, as shown in Table No. (1).

B.H. No.	***Depth from Ground Level***	***W.T.L. m***
B.H.1	**25**	**2.0**
B.H.2	**20**	**2.0**

4.3 In-Situ Testing (Standard Penetration Test):

To obtain the penetration resistance of the underground strata in boreholes, the standard penetration test was carried out. The test consists of driving the standard split spoon sampler in the soil and counting the number of blows required to drive the sampler at a distance of 30 cm by dropping a 63.5 kg hammer falling freely 76 cm. The corrected blows can be estimated by using (McGregor and Duncan 1998) and are referred to (0.7) value of N recorded and represents the standard penetration resistance N60 according to the following formula:

$$N60 = (N * \eta_H * \eta_B * \eta_S * \eta_R) / 60$$

Where: *N* = measured SPT blow countered,

η_H = hammer efficiency (%),

η_B = correction for borehole diameter,

η_S = sampler correction, η_R = correction for rod length.

Table No. (2): the correction factors for the standard penetration test

2. Variation of η_B

Diameter mm	Diameter in.	η_B
60–120	2.4–4.7	1
150	6	1.05
200	8	1.15

4. Variation of η_R

Rod length m	Rod length ft	η_R
>10	>30	1.0
6–10	20–30	0.95
4–6	12–20	0.85
0–4	0–12	0.75

1. Variation of η_H

Country	Hammer type	Hammer release	η_H (%)
Japan	Donut	Free fall	78
	Donut	Rope and pulley	67
United States	Safety	Rope and pulley	60
	Donut	Rope and pulley	45
Argentina	Donut	Rope and pulley	45
China	Donut	Free fall	60
	Donut	Rope and pulley	50

3. Variation of η_S

Variable	η_S
Standard sampler	1.0
With liner for dense sand and clay	0.8
With liner for loose sand	0.9

Table No. (3): Relative density, consistency & strength according to results of S.P.T

Sandy Soil		Clayey Soil			
N value (per 30 cm)	*Relative density*	*N value (per 30 cm)*	*Consistency*	*Consistency index =(L.L-W.C)/P.I*	*Undrained strength qu=2Cu KPa*
0 - 4	*Very Loose*	*0 -2*	*Very soft*	*< 0.5*	*<25*
4 - 10	*Loose*	*2 -4*	*Soft*	*0.5 - 0.75*	*25 - 50*
10 - 30	*Medium*	*4 -8*	*Medium*	*0.5 - 0.75*	*50 - 100*
30 - 50	*Dense*	*8 -16*	*Stiff*	*0.75 - 1.0*	*100 - 200*
>50	*Very Dense*	*16 -32*	*Very stiff*	*1.0 - 1.5*	*200 - 400*
		>32	*Hard*	*>1.5*	*> 400*
		*Undrained shear strength (qu) for clay equal to (12.5*SPT value) And (qu)equal to (10*SPT value)for clay with P.I. >30*			

4.4 Laboratory Works:

In general, a series of laboratory tests are performed on selected soil samples as listed below in Table No. (3): Summary of laboratory tests.

Type	Test	Standard Specification
Classification & Physical Properties	Natural water content and density	ASTM D2216
	Liquid and Plastic Limits	ASTM D 4318
	Sieve Analysis	ASTM D 422
	Sieve Analysis and Hydrometer	
	Specific Gravity	ASTM D 854
Strength Tests	Direct Shear Test	ASTM D 3080
	Unconfined Test	ASTM D 2166
Compressibility Test	Consolidation Test	ASTM D 2435
Chemical Tests	Sulfate content, gypsum content, organic matter, and Total Soluble Salts(T.S.S.)	BS 1377:1990 part 3 and Earth Manual

5. Discussion of Tests Results:

5.1 Field Tests (Standard Penetration Test):

Standard penetration tests were conducted at different depths for soil samples. From SPT test results obtained for all boreholes, as shown in Appendix(A), N recorded values ranged between (14- 46) blow with the average value for SPT blows top (20) meters to evaluate the Site Seismic Parameters, equal to (31.6) blow, which indicated in Regarding SPT-values results in cohesive and non -cohesive layers, it is obvious that the shear strength of the cohesive soil is medium stiff, grading to very stiff in clayey soil layer and medium dense grading to very dense in the sandy layers as shown in Fig.(4).

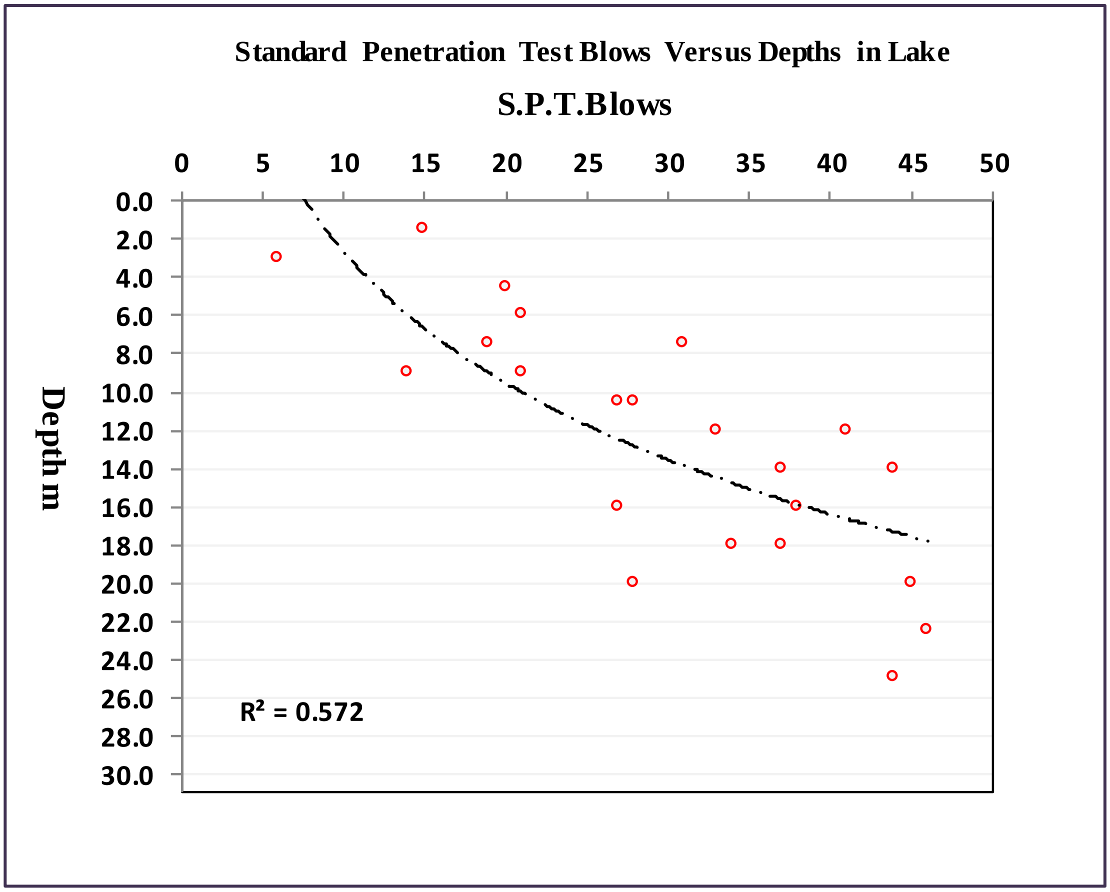

Fig.(3) S.P.T. blows versus depths for all boreholes

5.2 Laboratory Tests:

5.2.1 Subsurface stratification:

According to the test results and soil profiles, as shown APPENDIX-B-and by adopting the Unified Soil Classification System and textural classification obtained from all boreholes, The stratification of layers was described independently for each borehole in appendix A&B, which was characterized as **erratic sedimentation** and according to the test results and summarized:

- The first soil layer is cohesive soil was appeared in boreholes BH.1 & B.H.2, which consists of medium stiff to very stiff brown fat silty CLAY with more broken bricks, some of organic materials and rusty areas. This layer extends from the natural ground surface (N.G.S) down to (7.5 – 12.0) m. depths.

- The second soil layer is cohesion-less soil, which consists of medium dense grading to dense grey silty sand or sand with clayey lenses. This layer extends from (7.5 – 12.0) m down to the end of boring at (20-25) m. depths in two boreholes. Details of soil stratification for each borehole are shown in the "Bore logs" appended and the subsoil profile in Figure (4) is shown below.

5.2.2 Underground Water Table:

The underground water table was (2.0) m below the existing ground surface after the drilling termination at the time of in situ investigation in January 2024, due to variations in the existing ground level and may fluctuate due to effects of construction in the future and seasons.

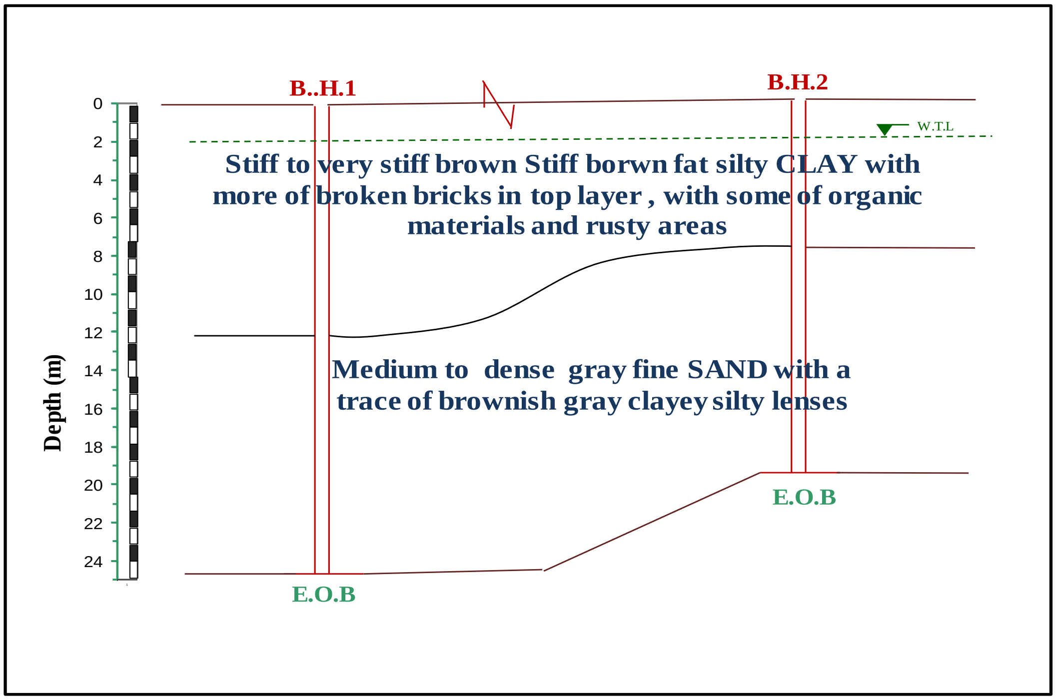

Fig.(4) Subsoil profile through two boreholes

Fig.(5&6) samples testing in lab. during unconfined and direct shear test

5.2.3 Atterberg Limits:

Atterberg limits tests were conducted on soil fractions passing sieve No. 40 according to ASTM D 4318. The values of liquid limit (LL), plastic limit (PL), and plasticity index (PI) at different locations of the topsoil layer are summarized. The plasticity index indicates the plasticity of cohesion soil, which has a liquid limit range between (40 - 60), while the plasticity index ranges between (20 – 33), as shown in Fig. (7). The values indicate that the soil can be classified in general low to high plastic soil and low plastic silt(ML or ML). Plasticity indices indicate that the soil is inorganic with low to high compressibility.

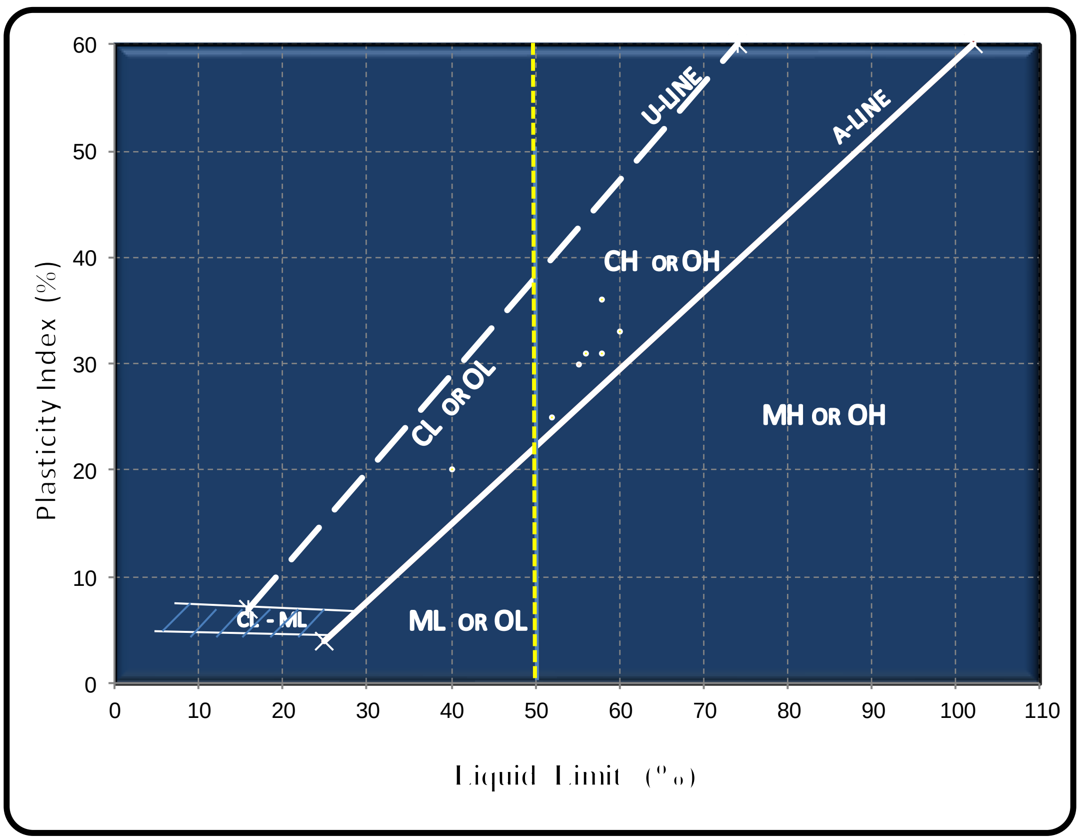

Fig. (7) Explain the consistency tests on the plasticity chart

5.2.4 Unconfined Compression Test

Unconfined compression shear tests have been carried out on undisturbed samples, according to specification ASTM-D2166, on different undisturbed soil clayey samples at different depths of boreholes derived. The test results are shown in Table (5) and Appendix -B-. All tested soil sample was carried out in a natural state during the test. The estimated modulus of subgrade reaction (Ks) can be calculated from unconfined compression tests As per Joseph Bowles the modulus of subgrade reaction, which is a conceptual relationship between soil pressure and deflection, as indirect method (Ks = 120*qu), which is explained with ranged between (13.56 – 41.40) M4N/m^3 for depths (1.5- 7.5) m below existing ground surface.

Table No. (5): Summary of Estimated Modulus of Subgrade Reaction (Ks).

Borehole No.	Depth (m)	Average unconfined compression strength (KPa)	Estimated Modulus of subgrade reaction (Ks) (MN/m3)
BH1	1.5	113	13.56
BH1	4.5	219	26.28
BH1	7.5	345	41.4
BH2	3.0	120	14.4
BH2	4.5	189	22.68
BH2	6.0	120	14.4
BH2	7.5	221	26.52
Maximum Value		345	41.4
Minimum Value		113	13.56

5.2.5 Direct Shear Test

Drained shear box tests were carried out on disturbed cohesion-less soil samples. Appendix –B- and Table (6) of the direct shear test for tested samples, cohesion values (c), and angle of friction (ϕ). It is noticed that the values of (c) are (3-12) kN/m^2 and those of (ϕ) are in the range of (28 - 40) degrees. These results in general indicate that the cohesion-less soil layer is medium to very dense.

Table No. (6): Cohesion values (c) and angle of internal friction (ϕ)

Borehole No.	Depth (m)	C (kPa)	Ø (deg.)
B.H.1	**14.0**	**12**	**28**
	22.5	**3**	**30**
B.H.2	**14.0**	**4**	**33**
	20.0	**3**	**40**
Maximum Value		**12**	**40**
Minimum Value		**3**	**28**

5.2.6 Consolidation Test Results

The variations of overburden (p_o), pre-consolidation (p_c) &swelling (p_s) pressures with depths are presented in Table (7) & APPENDIX- C-. In general, these results indicate that the clayey soil layer, in general, is over-consolidated and normal-consolidated with increasing depths as shown, and swelling potential appeared in tests with (7.0-42.0) KN/m^2, as swelling pressure potential.

Table (7) Consolidation parameters with depth

B.H. No.	Depth (m)	e_o	mv (m^2/kN)	Cv (m^2/min)	Pc (kPa)	Cc	Cr	P_o (kPa)	OCR	K (m/min)	Swelling Pressure (kPa)
B.H.1	1.5	0.959	1.4E-04	5.90E-06	108	0.15	0.020	31.9	3.4	8.1E-08	10
B.H.1	4.5	0.913	1.8E-04	5.30E-06	90	0.12	0.020	63.7	4.4	9.2E-09	42
B.H.2	3.0	0.783	2.7E-04	1.30E-05	105	0.19	0.040	49.8	2.1	3.3E-08	7.0
B.H.2	6.0	1.319	4.3E-04	9.40E-06	90	0.33	0.060	60.7	1.5	4.0E-08	21.0

5.2.7 Site Seismic Parameters

According to (IBC / 2018), depending on the type of foundation soil, the recommended value can be considered as (the maximum ground acceleration PGA = 0.2). According to the Iraqi seismic code (2017), the ordinary structure may be designed by the equivalent static method using conventional liner elastic analysis. The seismic analysis of structures shall consider the dynamic properties of the structure by equivalent static analysis. The Seismic Coefficients (Ss) & (S1) are presented in Figures (8,9,10& 11) respectively. According to the Iraqi Seismic Code 2017.

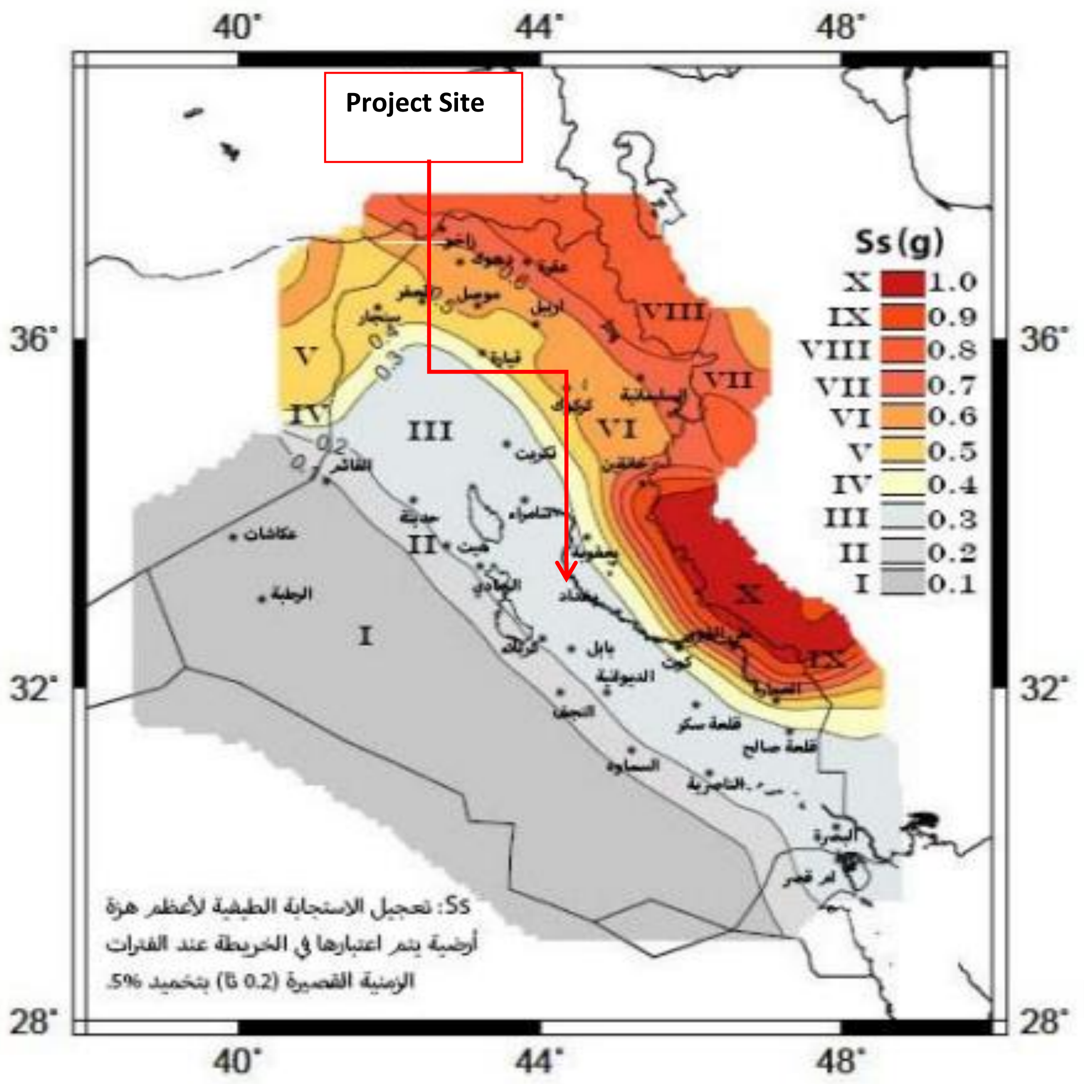

Fig. (8) Seismic zoning map of Iraq showing spectral response acceleration parameter (Ss) for time (0.2 Sec.)

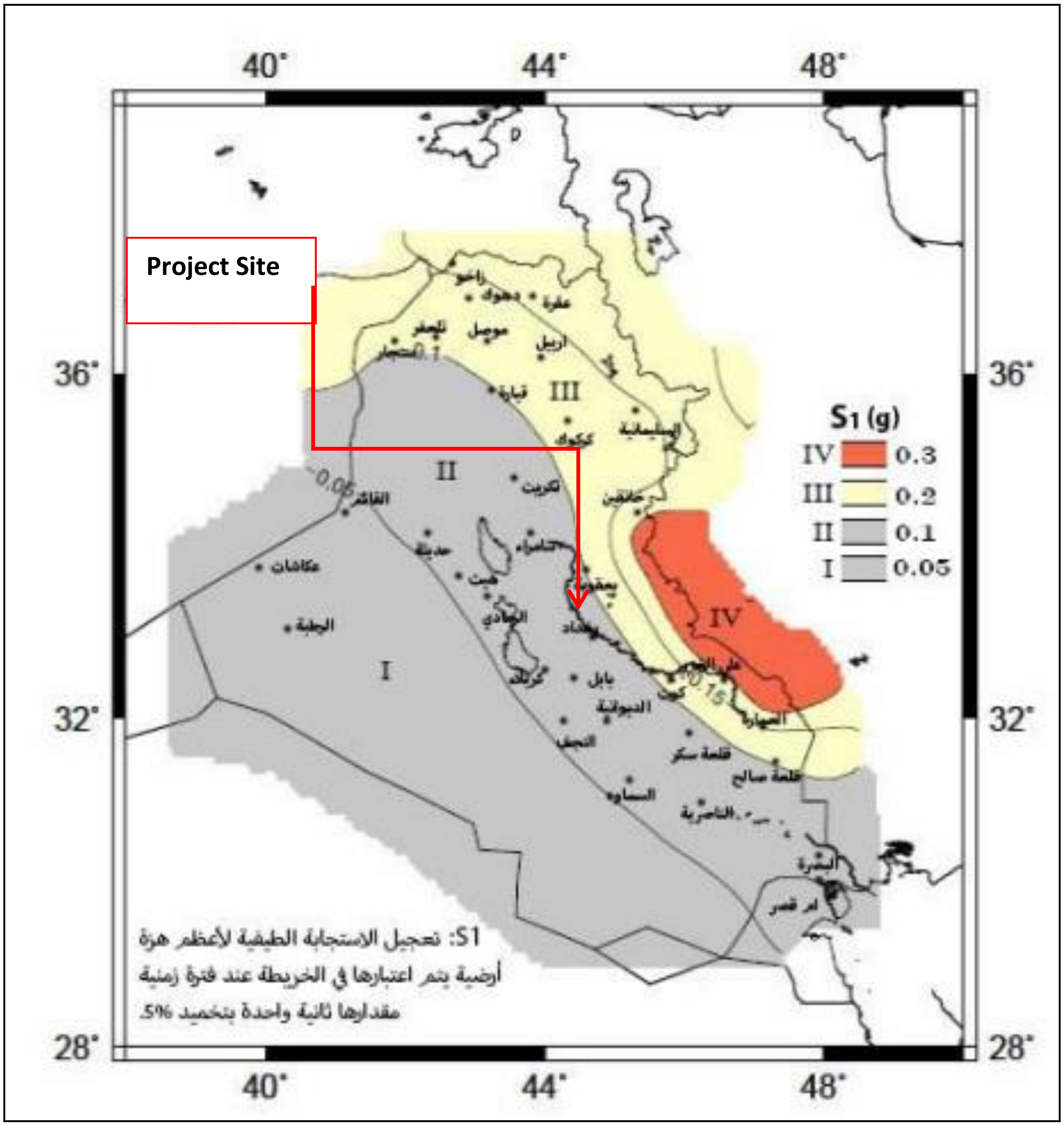

Fig. (9) Seismic zoning map of Iraq showing spectral response acceleration parameter (S1) for time (1 Sec.)

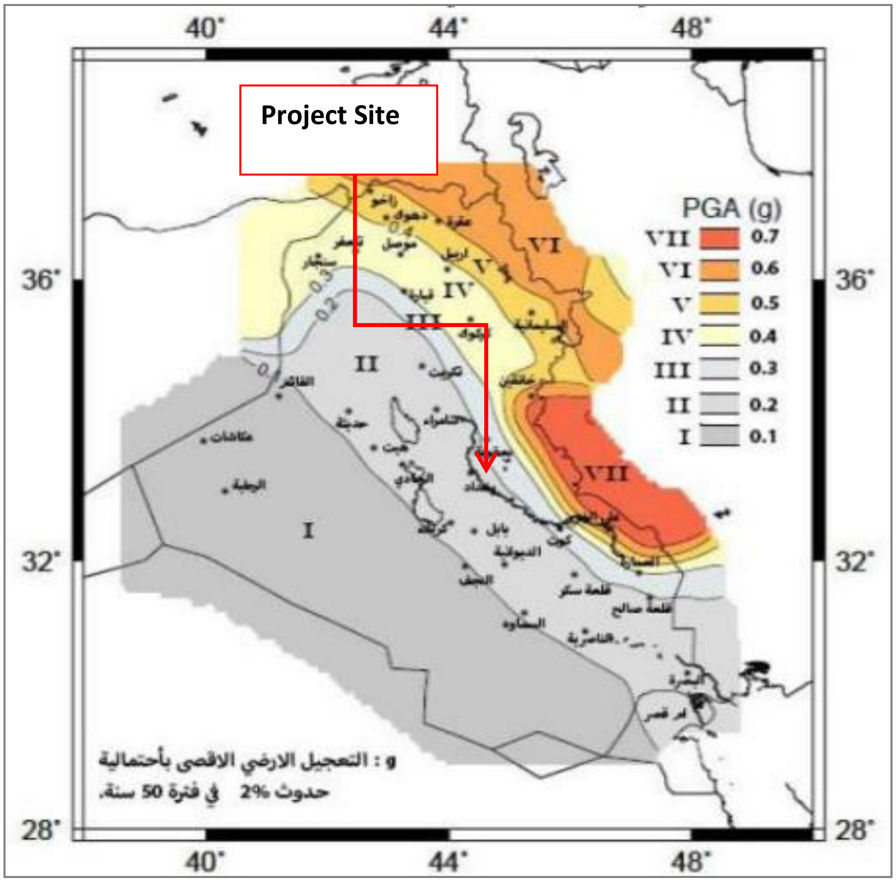

Figure (10) Iraq map showing the values of maximum ground acceleration of an earthquake. The value of this acceleration exceeds 2% for 50 years

The following factors and coefficients can be used in the design of the building:

With regards to shear wave velocities (Vs), various authors have considered different empirical correlations between N_{spt} and Vs based on soil type and geological age of the deposits. A correlation by Seed (1983) can be considered as follows **$Vs = 56 N_{spt}^{0.5}$ (m/sec.)**

N_{spt} average blows for 25 meters' depth= 31.6 blow

$Vs = 56*(31.6)^{0.5}$ $Vs = 314.8$ m/sec.

Based on seismic activity according to Iraqi seismic code requirements for building code (2017) Baghdad city under zone II with Iraq map, the values of maximum ground acceleration of an earthquake. The value of this acceleration exceeds 2% for 50 years equal to (0.2). Considering the shear wave velocity equal to **314.8 m/sec.** the soil profile type can be defined as S_D in Table (8).

Soil profile type	Soil profile name/ generic description	Average soil properties for top 25 m of soil profile		
		Shear wave velocity m/s.	Standard penetration test N_{60}	Undrained shear strength KPa
S_A	Hard rock	>1500	-	-
S_B	Rock	760 -1500		
S_C	V. dense soil of soft rock	370 - 760	>50	>100
S_D	Stiff soil profile	180 - 370	15 -50	50 -100
S_E	Soft soil profile	< 180	< 15	< 50
S_F	Soil requiring site specific evaluation			

S_D: stiff soil profile (wave velocity within the range 180-370 m/sec.) according to UBC1997.

- The soil profile type (D) can be used for the cohesive soil layers (15> N > 50).
- The spectral response acceleration parameters Ss = 0.3 and S1 = 0.1

TABLE 1613.2.3(1)
VALUES OF SITE COEFFICIENT F_a [a]

SITE CLASS	MAPPED RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) SPECTRAL RESPONSE ACCELERATION PARAMETER AT SHORT PERIOD					
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s = 1.25$	$S_s \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	Note b	Note b	Note b
F	Note b	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Values shall be determined in accordance with Section 11.4.8 of ASCE 7.

TABLE 1613.2.3(2)
VALUES OF SITE COEFFICIENT F_V [a]

SITE CLASS	MAPPED RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) SPECTRAL RESPONSE ACCELERATION PARAMETER AT 1-SECOND PERIOD					
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[c]	2.0[c]	1.9[c]	1.8[c]	1.7[c]
E	4.2	3.3[c]	2.8[c]	2.4[c]	2.2[c]	2.0[c]
F	Note b	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
b. Values shall be determined in accordance with Section 11.4.8 of ASCE 7.
c. See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7.

- Then the site coefficients Fa for the soil type of (D) are equal to 1.56 & and Fv = 2.4.

[From Iraqi Seismic Code (2017), pages (28-29), 5/2 -6/2]

- Modified spectral acceleration value :

  $\mathbf{S_{MS}}$= Fa $\mathbf{S_S}$ **=** 1.56* 0.3= 0.468

  $\mathbf{S_{M1}}$= Fv S_1 = 2.40* 0.1= 0.24

  Design value for the spectral acceleration of seismic ground motion:

  $\mathbf{S_{DS}}$= 2/3 S_{MS} = 0.666* 0.468= **0.312**

  $\mathbf{S_{D1}}$= 2/3 S_{M1} = 0.666* 0.24= **0.160**

- **Design Response Spectrum**

  Where the required structural analysis and design, depend on the spectral response diagram for acceleration as explained in Fig (11):

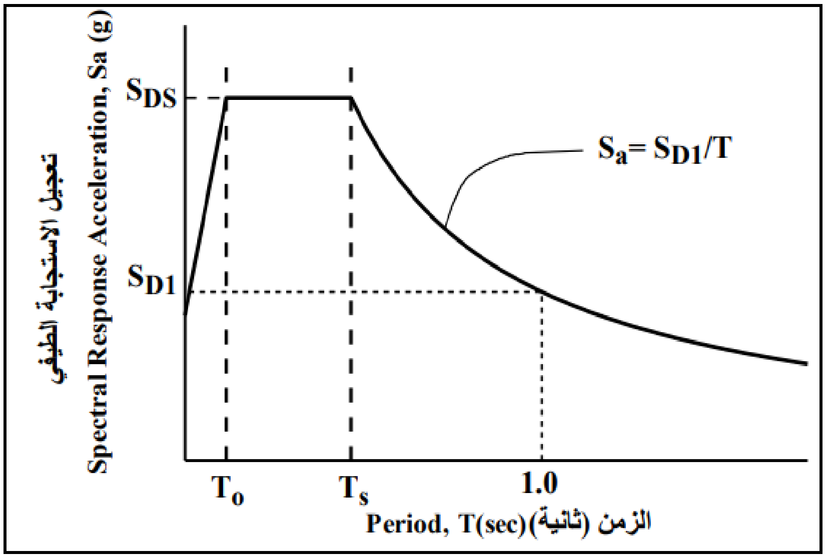

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Fig. (11) Spectral response acceleration and determine the T_o & T_s

This can be calculated according to the following:

$T_o = 0.2(S_{D1}/S_{Ds})$ $T_o = 0.2(0.160/0.312) =$ **0.103** sec

$T_s = S_{D1}/S_{Ds}$ $T_s = 0.160/0.312=$ **0.513** sec

Table (9) Nature of occupancy

Occupation category	Occupancy Importance Factor
II or I	1.0
III	1.25
IV	1.50

6. Allowable Bearing Capacity Discussion (Method of Calculations)

6.1 The Shallow Foundations

Since damage may result from foundation failure (collapse) as well as from excessive settlement. The following criteria must always be used in evaluating the bearing capacity:

1. Adequate factor of safety against failure.

2. Adequate margin against excessive settlement.

The bearing capacity could be evaluated from one of the following methods.

1. The bearing capacity is calculated according to the Terzaghi equation with modification suggested by Meyerhof (1963)

$q_{ult} = CN_c + qN_q + 0.5\, B_\gamma\, N_\gamma$ continuous footing

$q_{ult} = 1.3\, CN_c + q\, N_q + 0.4\, \gamma\, B\, N_\gamma$ square footing

$q_{ult} = 1.3\, CN_c + q\, N_q + 0.3\, \gamma\, B\, N_\gamma$ round footing

$q_{ult} = CN_c\, S_c\, d_c + q\, N_q\, S_q\, d_q + 0.5\, \gamma\, B\, N_\gamma\, S_\gamma\, d_\gamma$ Meyerhof

N_c, N_q, N_γ Bearing capacity factor

S_c, S_q, S_γ Shape factors

d_c, d_q, d_γ Depth factors

$$S_c = 1 + \frac{N_q}{N_c}\frac{B}{L} \;, \quad S_q = 1 + \frac{B}{L}\tan\phi \;, \quad S_\gamma = 1 - 0.4\frac{B}{L}$$

$$d_c = 1 + 0.4\frac{D_f}{B} \;, \quad d_q = 1 + 2\tan\phi(1-\sin\phi)^2\,\frac{D}{B}$$

2. Bearing capacity for the foundation on undrained saturated clay for φ =0, so the general expression will be :

$q_{ult} = CN_c + \gamma\, D_f$ (i.e. $N_q = 1$, $N\gamma = 0$)

$$(N_c)_{rectangular} = \left(1 + 0.2\frac{B}{L}\right)(N_c)_{Strip}$$ (Skempton formula)

3. The net allowable bearing capacity of clay or plastic silt is approximately equal to the unconfined compressive strength

Where $q_{ult} = CN_c + \gamma D_f$ for $\Phi=0$

The net ultimate bearing capacity (qult) is defined as the pressure that can be supported at the base of the footing in excess of that at the same level due to the surrounding surcharge.

$$q_{ult} = q_{ult} - \gamma D_f = CN_c + \gamma D_f - \gamma D_f$$

$q_{ult} = CN_c$ take F.O.S=3

$$q_{all} = \frac{CN_c}{3}$$

$C = \frac{q_{unconfined}}{2}$, usually $N_c \approx 6$, so

$$q_{all} = \frac{q_{unconfined} \times 6}{2 \times 3}, \quad \text{so}$$

$$\boxed{q_{all} = q_{unconfined}}$$

Thus the allowable bearing capacity of clay or plastic silt is approximately equal to the unconfined compression strength.

4. The bearing capacity calculated from SPT result using the following equation: This is suitable for cohesion-less soil for (25 mm) of settlement.

qall = (N/4)/K for footing width 4 feet or less (Meyerhof) and
qall = $(N/6)[(B+1)/B]^2/K$ for footing width greater than 4 feet

where: K= 1+0.33(D/B) Df= Depth of foundation,

B =Width of foundation, N = No. of blows for SPT

The most reliable values of allowable bearing capacity adopted in this report were those values evaluated from paragraphs 1, 2,3and 4, then taking the most critical value (minimum) to be the convenient allowable bearing capacity.

The above allowable net soil bearing capacity was evaluated using a factor of safety of (3.0) against bearing capacity failure which means the contact pressure will be sufficiently low in magnitude to keep load-induced deformation within the elastic range of the bearing soils.

6.2 Deep foundation

The ultimate bearing capacity of the pile (Qu) is made up of adhesion (Qs) and end bearing (Qb); (adhesion often called skin friction) is usually much greater than end bearing in clay

Qu = Qs + Qb

Qu = αCuAs + Nc Cub Ab

Where Cu = shear strength of soil adjacent to the shaft

α = shaft adhesion factor taken as 2/3 =0.67 for uncased piles

As = surface area of pile shaft

Nc= bearing capacity factor (usually taken as = 9)

Cub = shear strength of soil (2/3)d below base where b= base diameter Ab = area of pile base.

The ultimate bearing capacity of the pile (Qu) has values C & Φ, then should use the following conservative Terzaghi bearing capacity factors formula

$$Q_b = A_P\left[cN_c + \sigma_{vb}N_q + 0.5\gamma DN_\gamma\right] + \sum A_s(\alpha c + K\sigma_v \tan\phi)$$

where

N_c, N_q, N_γ = Terzaghi's bearing capacity factors

σ_{vb}, σ_v = effective overburden pressure of base and pile shaft, irrespective of the critical depth.

As well, there are many methods, suggested by Meyerhof (1965-1976) were used in calculating the required length of driven and bored piles depending on the results obtained from standard penetration tests.

1. Qb = (4N) (Lb / B) Ab (ton) for the driven pile.

2. Qs = (0.1N) As = ∑ fs (ton) for driven pile.

3. Qb = (1.4N) (Lb / B) Ab (ton) for the bored pile.

4. Qs = (0.067N) As = ∑ fs (ton) for bored pile.

5. Qs = (0.5N) As = ∑ fs (ton) for bored pile for clay.

Where Qb = pile bearing resistance (ton)

Qs = pile skin friction Where (ton) As = pile surface area = 0.275 *4*D

Lb = length of the part of the pile that penetrated in bearing layer.

B = width or diameter of pile.

N = the average number of blows of standard penetration.

Depending on the above equations and the results of the standard penetration test for the different soil layers in the project site which are shown in the following recommendations.

AASHTO 2020 in article 10.8.3.5 determined the predicating of Nominal Axial Compression Resistance of Single Drilled Shafts to making the simulation The factored resistance of drilled shafts, RR, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad (10.8.3.5\text{-}1)$$

ϕ qp = Resistance factor for tip resistance Specified in Table 10.5.5.2.4-1

ϕ qs = Resistance factor for shaft side resistance Specified in Table 10.5.5.2.4-1

Table (10) for Resistance factor for tip and shaft side resistance for drilled shaft

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand	β-method (Brown et al., 2010)	0.55
	Tip resistance in sand	Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010)	0.60
	Tip resistance in cohesive IGMs	Brown et al. (2010)	0.55
	Side resistance in rock	Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	0.50
Block Failure, φ_{bl}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (Brown et al., 2010)	0.35
	Sand	β-method (Brown et al., 2010)	0.45
	Rock	Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

$$Rp = qp * Ap$$

$$Rs = qs * As$$

Where:

Rs : nominal shaft side resistance

A- For top clayey soil layers

$$q_s = \alpha S_u \qquad (10.8.3.5.1b\text{-}1)$$

S_u = undrained shear strength (MPa)

α = adhesion factor (dim.)

p_a = atmospheric pressure (= 0.101 MPa)

$$\alpha = 0.55 \text{ for } \frac{S_u}{p_a} \le 1.5 \qquad (10.8.3.5.1b\text{-}2)$$

The nominal axial resistance of drilled shafts in cohesionless soils by the β-method shall be taken as:

$$q_s = \beta\sigma'_v \le 0.19 \text{ for } 0.25 \le \beta \le 1.2 \qquad (10.8.3.5.2b\text{-}1)$$

in which, for sandy soils:

- for $N_{60} \ge 15$:

$$\beta = 1.5 - \left(7.7 \times 10^{-3}\sqrt{z}\right) \qquad (10.8.3.5.2b\text{-}2)$$

Rp : nominal shaft tip resistance

$$\text{for } 0.057N_{60} \le 50,\ q_p = 1.2N_{60} \qquad (10.8.3.5.2c\text{-}1)$$

When the N_{60} =50 blows 0.057*50 =2.85 < or = 50

Then q_p = unit tip resistance (Mpa) = 1.2* 50= 60 Mpa

The value of q_p in Eq. 1 should be limited to 3.0 MPa, unless greater values can be justified though load test data.

Cohesionless soils with $SPT\text{-}N_{60}$ blow counts greater than 50 shall be treated as intermediate geomaterial (IGM) and the tip resistance, in MPa, taken as:

$$q_p = 0.59\left[N_{60}\left(\frac{p_a}{\sigma'_v}\right)\right]^{0.8}\sigma'_v \qquad (10.8.3.5.2c\text{-}2)$$

7. The Design Data

Since the design data for lightweight or heavy structures in the site project is not specified yet and will be available in the details design stage, then in this report includes the general conclusions and recommendations with multiple options.

8. The Conclusions and Recommendations

8.1 Conclusions

A program of laboratory tests was carried out on samples of soil taken from the site of the project; the following conclusions are fixed:

8.1. The stratification of layers was characterized as **erratic sedimentation** and affected and according to the test results and summarized:

- The first soil layer is cohesive soil was appeared in boreholes BH.1 & B.H.2, which consists of medium stiff to very stiff brown Stiff brown fat silty CLAY with more broken bricks, some organic materials, and rusty areas. This layer extends from the natural ground surface (N.G.S) down to (7.5 – 12.0) m. depths.

- The second soil layer is cohesion-less soil, which consists of medium dense grading to dense grey silty sand or sand with clayey lenses. This layer extends from (7.5 – 12.0) m down to the end of boring at (20-25) m. depths in two boreholes

8.3. The underground water table was (2.0) m below the existing ground surface after the drilling termination at the time of in situ investigation in January 2024, due to variations in existing ground level may fluctuate due to the effects of construction in the future and seasons.

8.4. In general, the site is almost a regular area.

8.2 Bearing Capacity for Shallow Foundation and Type of Deep Foundation

8.2.1 Bearing Capacity for Shallow Foundation for Abutment

- According to test results and available design data, the allowable bearing capacity should be no more than **(8.0 – 9.00) ton/m^2 (80 - 90 KN/m^2)** can be used for shallow foundations, at depths of **(-1.00 - 2.00)** m respectively, below existing ground surface (E.G.S).

8-3 Type of Cement

- Sulfate-resisting cement should be used for concrete works that, are in touch with the soil.
- Minimum cement content and maximum free water/cement ratio within requirements of the specifications.

About this report:-

1- The recommendations services have been clarified & approved by ***Taha Yaseen Alkaabi***, a Geotechnical Expert.

2- The testing, supervision, and logistic support were done by consultant engineer ***Ali Abdulkhadhim Al-Shamoosi***, director of Al-Shams Laboratory.

9. References:

- ASTM(2003): American Society for Testing and Materials, USA.
- BS 1377,part1,1990.British Standard Method of Test of Soil for Civil Engineering.
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- Head, K.H.(1982): 'Manual of Soil Laboratory'. Vol.2, London.
- Murthy, V.N.S.(2009): 'Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering.New York.
- Terzaghi, K. and Peck, RB. (1948) Soil Mechanics in Engineering Practice, 1st Edition, John Wiley, New York.

APPENDICES

APPENDIX-A-

BOREHOLE LOG

BOREHOLE LOG

Turkish Embassy

Baghdad Province

AL-SHAMS LABORATORY
FOR STRUCTURAL TESTS

B.H No:	1		
B.H Diam.	100 mm	Date of Drilling	13/01/2024
B.H Depth	25 m	Method of Drilling	Flight Auger
Coordinate		Weather	Sunny 22 °C

Depth (m)	Sample Type	Legend	SPT Blows				Visual Description of Soil	Symbol
			15cm	15cm	15cm	N60		
0.5	BS		-	-	-	-	Medium brown lean silty CLAY with a lot of gray fine sandy spots and a trace of roots of plants.	CL
1.5	US		-	-	-	-	Stiff brown fat silty CLAY with a little of gypsum pockets, a trace of organic materials, and rusty areas.	CH
3.0	DS		4	3	3	6	Medium bown fat silty CLAY with a little of egg shells.	CH
4.5	US		-	-	-	-	Very stiff brown fat silty CLAY with a trace of lines of salt, organic materials, and crystal shiny spots.	CH
6.0	DS		5	9	12	21	Very stiff brown fat silty CLAY with more of rusty areas, a trace of organic materials, and egg shills.	CH
7.5	DS		9	12	19	31	Very stiff grayish borwn sandy fat silty CLAY with a trace of organic materials and rusty areas.	CS
9.0	DS		5	5	9	14	Stiff brownish gray fat silty CLAY.	CH
10.5	DS		9	12	15	27	Very stiff grayish brown fat silty CLAY with a trace of organic materials and gray fine sandy thin layers.	CH
12.0	DS		11	16	17	33	Hard grayish brown sandy silty CLAY with a trace of organic materials, and rusty areas.	CS
14.0	DS		7	12	25	37	Dense brownish gray silty clayey fine SAND	SC
16.0	DS		10	12	15	27	Medium brown silty clayey fine SAND.	SC
18.0	DS		10	16	21	37	Dense brownish gray fine SAND with more of silty clayey layers and fine gravel.	SP
20.0	DS		5	10	18	28	Medium brownish gray fine SAND with more of silty clayey layers.	SP
22.5	DS		9	20	26	46	Dense brownish gray fine SAND with a trace of silty clayey layers.	SP
25.0	DS		8	17	27	44	Dense brownish gray gravelly fine SAND with more of silty clayey blocks.	SG

Drilling conducted according to requirements of ASTM D-1452

Effeciency of S.P.T hammer	%	60
Final Water Table Level	m	2.0
Final Water Elevation Level	m	2.0

BOREHOLE LOG

Turkish Embassy

Baghdad Province

AL-SHAMS LABORATORY
FOR STRUCTURAL TESTS

B.H No:	2		
B.H Diam.	100 mm	Date of Drilling	13/01/2024
B.H Depth	20 m	Method of Drilling	Flight Auger
Coordinate		Weather	Sunny 22 °C

Depth (m)	Sample Type	Legend	SPT Blows				Visual Description of Soil	Symbol
			15cm	15cm	15cm	N60		
0.5	BS		-	-	-	-	Medium dark brown lean clayey SILT with more of roots of plants.	MC
1.5	DS		4	6	9	15	Stiff brown fat silty CLAY with some of organic materials and rusty areas.	CH
3.0	US		-	-	-	-	Stiff borwn fat silty CLAY with more of broken bricks, some of organic materials and rusty areas.	CH
4.5	DS		5	8	12	20	Very stiff brown fat silty CLAY with more of rusty areas and some of organic materials.	CH
6.0	US		-	-	-	-	Very stiff brown fat silty CLAY with more of rusty areas.	CH
7.5	DS		5	7	12	19	Very stiff grayish bown fat silty CLAY with more of rusty areas.	CH
9.0	DS		4	7	14	21	Medium grayish brown silty clayey fine SAND.	SC
10.5	DS		8	11	17	28	Medium brownish gray fine SAND with more of silty clayey pockets.	SP
12.0	DS		12	17	24	41	Dense brownish gray fine SAND with more of silty clayey pockets and fine gravel.	SP
14.0	DS		16	20	24	44	Dense brownish gray fine SAND with a trace of silty clayey layers.	SP
16.0	DS		11	16	22	38	Ditto	SP
18.0	DS		13	16	18	34	Dense brownish graded fine SAND with a trace of silty clayey pockets and more of fine gravel.	SP
20.0	DS		12	17	28	45	Dense brownish gray fine SAND with a trace of silty clayey layers.	SP

Drilling conducted according to requirements of ASTM D-1452

Effeciency of S.P.T hammer	%	60
Final Water Table Level	m	2.0
Final Water Elevation Level	m	2.0

APPENDIX-B-

SUMMARY OF TEST RESULTS

SUMMARY OF LABORATORY TESTS RESULTS

Site Name: Turkish Embassy

Borehole No. 1

Date: 18/01/2024

Sample Definition				Atterberg Limits			Natural Water Content %	Dry Density g/cm^3	Specific Gravity	Strength Tests			Consolidation Test										GRAIN SIZE ANALYSIS			Chemical Test			
										Unconfined Compression (kPa)	Direct Shear																		
B.H No.	SN	Depth (m)	Sam. Type	LL %	PL %	PI %					C (kPa)	Ø (deg.)	e_o	mv (m^2/kN)	Cv (m^2/min)	Pc (kPa)	Cc	Cr	P_o (kPa)	OCR	K (m/min)	Swelling Pressure (kPa)	Fines %	Sand %	Gravel %	SO_3 %	TSS %	PH	OMC %
1	1.0	0.5	BS	40	20	20																	82.0	18.0	0.0	0.20	1.2	7.2	2.10
	2.0	1.5	US				29.0	1.39	2.72	113			0.959	1.4E-04	5.90E-06	108	0.15	0.020	31.9	3.4	8.1E-08	10	87.0	3.0	0.0				
	3.0	3.0	DS	58	22	36																	95.0	5.0	0.0				
	4.0	4.5	US				31.4	1.50	2.72	219			0.913	1.8E-04	5.30E-06	90	0.12	0.020	63.7	4.4	9.2E-09	42.0	97.0	3.0	0.0	0.11	1.00	7.0	1.43
	5.0	6.0	DS				30.5																96.0	4.0	0.0				
	6.0	7.5	DS	58	27	31	26.4			345													76.0	24.0	0.0				
	7.0	9.0	DS																				94.0	6.0	0.0				
	8.0	10.5	DS	55	25	30																	91.0	9.0	0.0				
	9.0	12.0	DS				29.4																63.0	37.0	0.0				
	10.0	14.0	DS								12	28											30.0	70.0	0.0				
	11.0	16.0	DS																				7.0	93.0	0.0				
	12.0	18.0	DS	No-plasticity																			13.0	74.0	13.0				
	13.0	20.0	DS								3	30											11.0	89.0	0.0				
	14.0	22.5	DS																				6.0	94.0	0.0				
	15.0	25.0	DS				26.5																12.0	51.0	37.0				

SUMMARY OF LABORATORY TESTS RESULTS

Site Name: Turkish Embassy

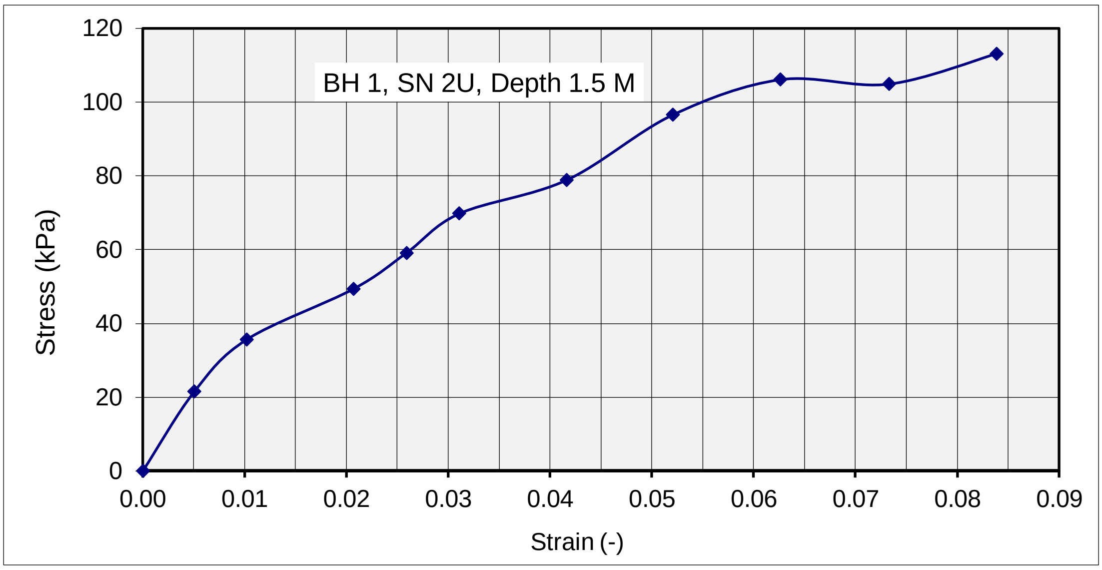
Borehole No. 2

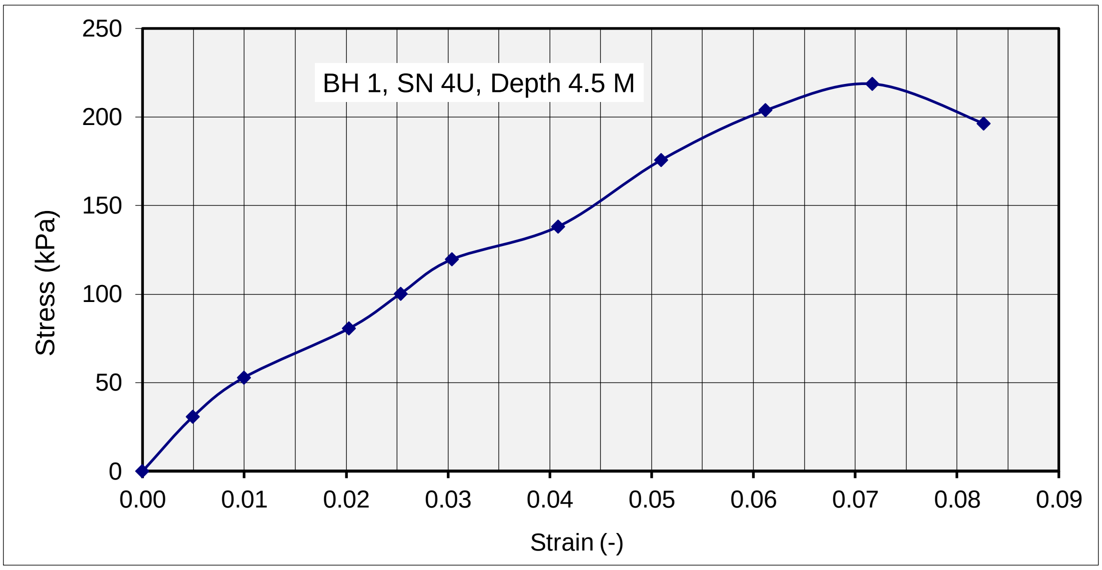
Date: 18/01/2024

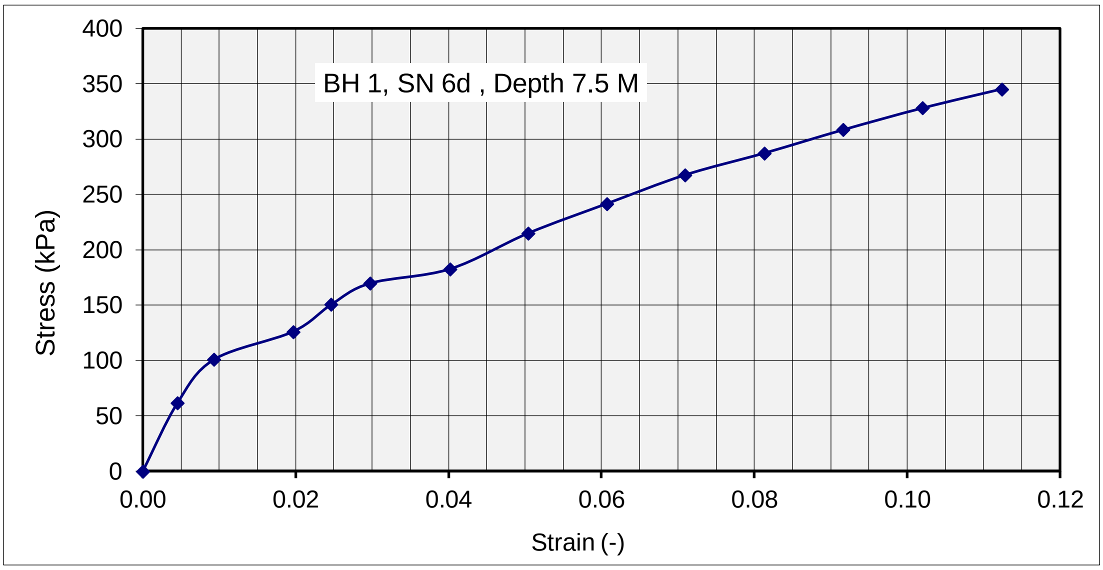
Sample Definition				Atterberg Limits			Natural Water Content %	Dry Density g/cm^3	Specific Gravity	Strength Tests			Consolidation Test										GRAIN SIZE ANALYSIS			Chemical Test			
										Unconfined Compression (kPa)	Direct Shear																		
B.H No.	SN	Depth (m)	Sam. Type	LL %	PL %	PI %					C (kPa)	Ø (deg.)	e_o	mv (m^2/kN)	Cv (m^2/min)	Pc (kPa)	Cc	Cr	P_o (kPa)	OCR	K (m/min)	Swelling Pressure (kPa)	Fines %	Sand %	Gravel %	SO_3 %	TSS %	PH	OMC %
2	1.0	0.5	BS																				95.0	5.0	0.0				
	2.0	1.5	DS	52	27	25																	96.0	4.0	0.0	0.24	3.21	7.1	2.95
	3.0	3.0	US				30.1	1.52	2.73	120			0.783	2.7E-04	1.30E-05	105	0.19	0.040	49.8	2.1	3.3E-08	7.0	93.0	0.0	0.0				
	4.0	4.5	DS	60	27	33	27.2			189													95.0	5.0	0.0				
	5.0	6.0	US				25.3	0.18	2.73	120			1.319	4.3E-04	9.40E-06	90	0.33	0.060	60.7	1.5	4.0E-08	21.0	95.0	5.0	0.0	0.21	2.78	7.3	2.00
	6.0	7.5	DS	56	25	31	25.4			221													96.0	4.0	0.0				
	7.0	9.0	DS																				27.0	73.0	0.0				
	8.0	10.5	DS				22.4																13.0	87.0	0.0				
	9.0	12.0	DS																				12.0	78.0	10.0				
	10.0	14.0	DS	No-plasticity							4	33											8.0	92.0	0.0				
	11.0	16.0	DS																				9.0	91.0	0.0				
	12.0	18.0	DS																				7.0	80.0	13.0				
	13.0	20.0	DS				25.9				3	40											8.0	92.0	0.0				

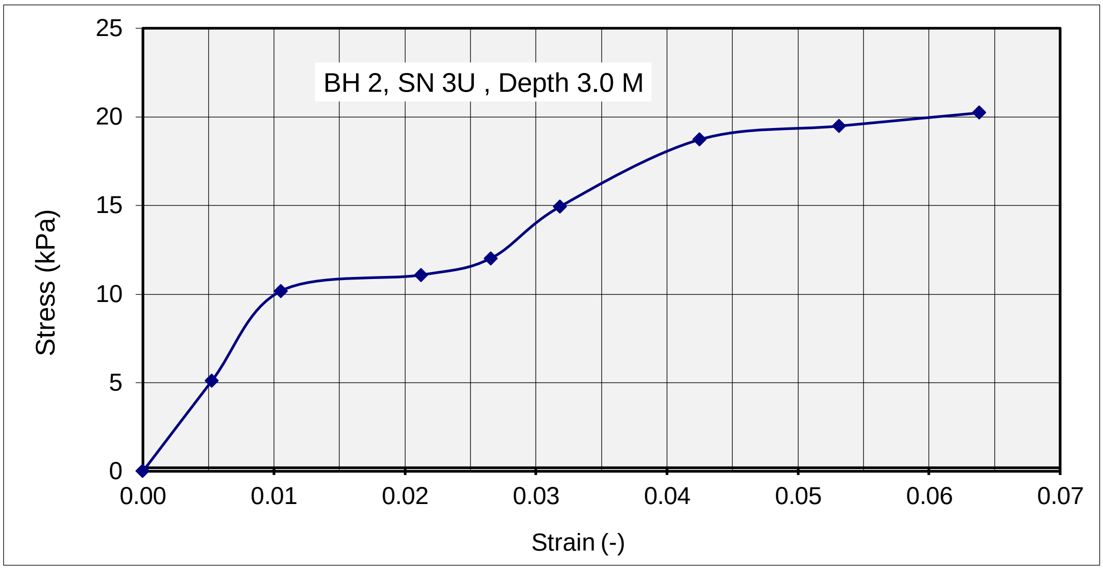
APPENDIX-C-

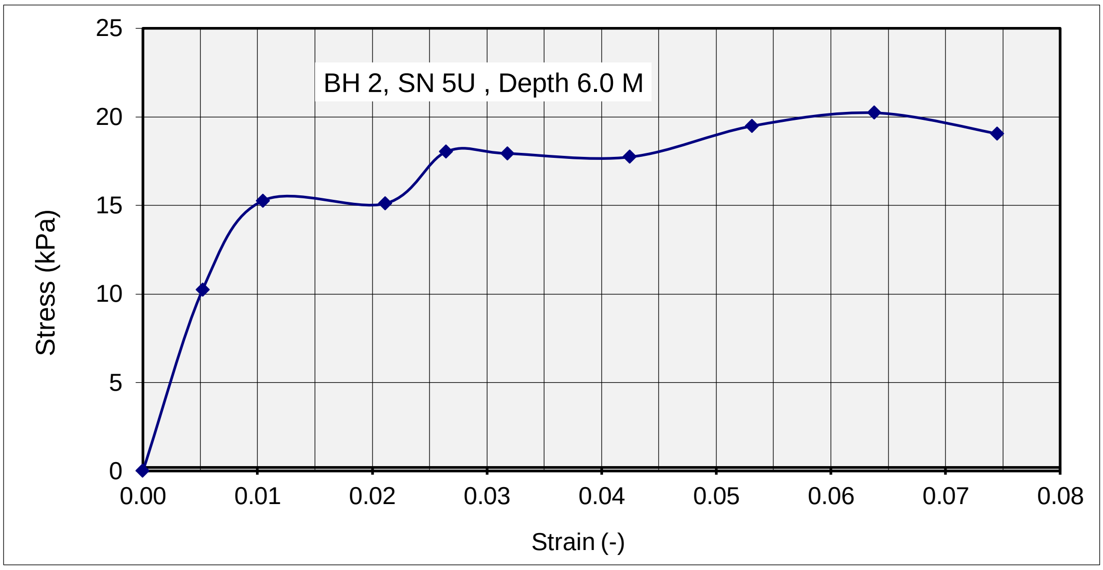
Test Results Figures

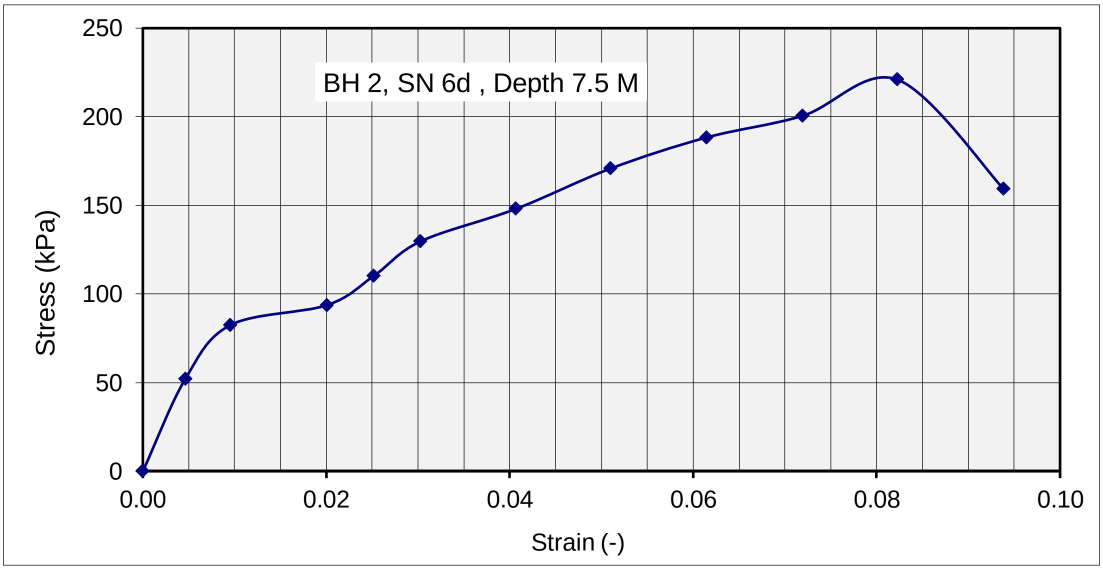

BH 1, SN 2U, Depth 1.5 M
Stress (kPa)
0
20
40
60
80
100
120
Strain (-)
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09

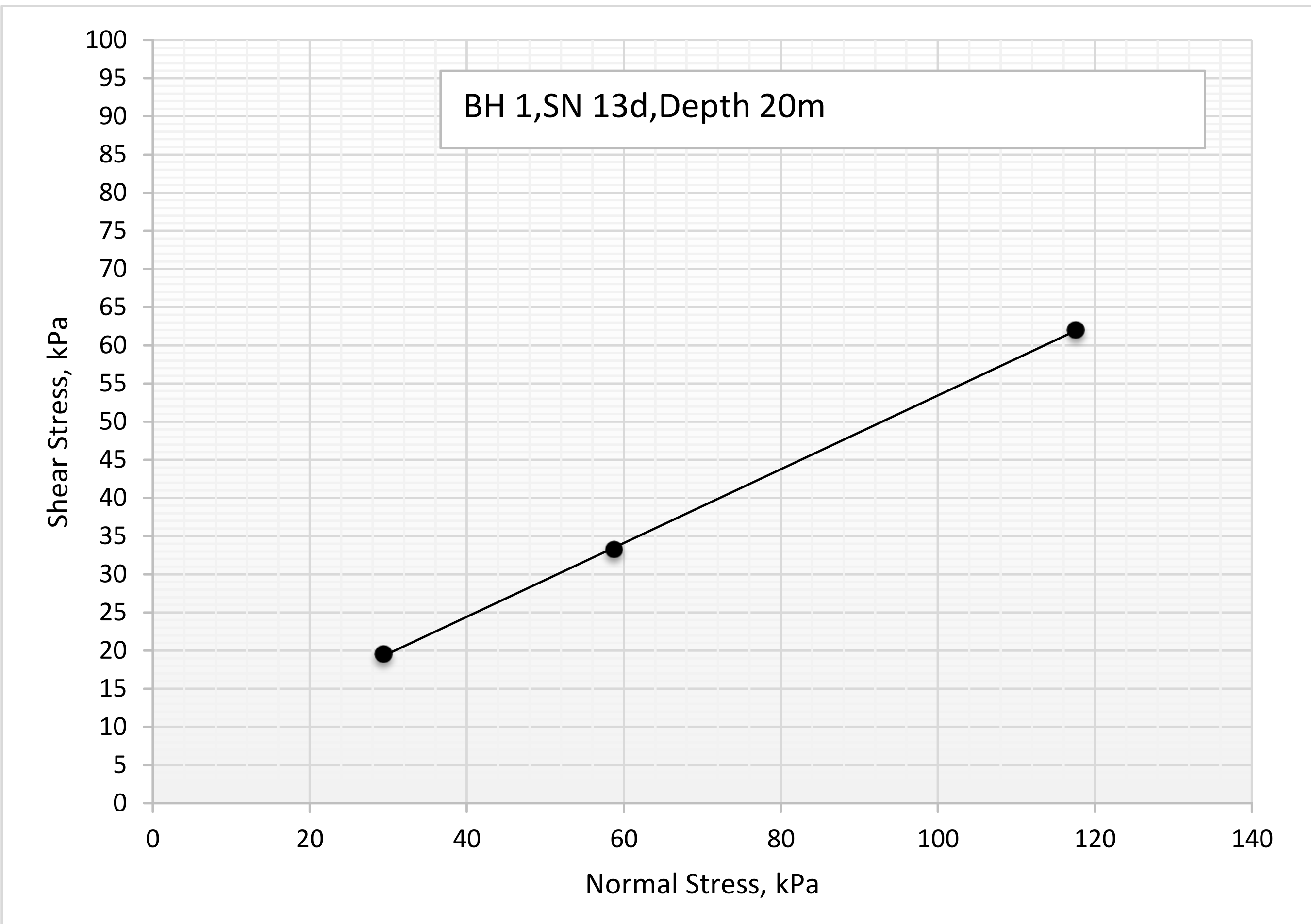

BH 1, SN 4U, Depth 4.5 M
Stress (kPa)
0
50
100
150
200
250
Strain (-)
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09

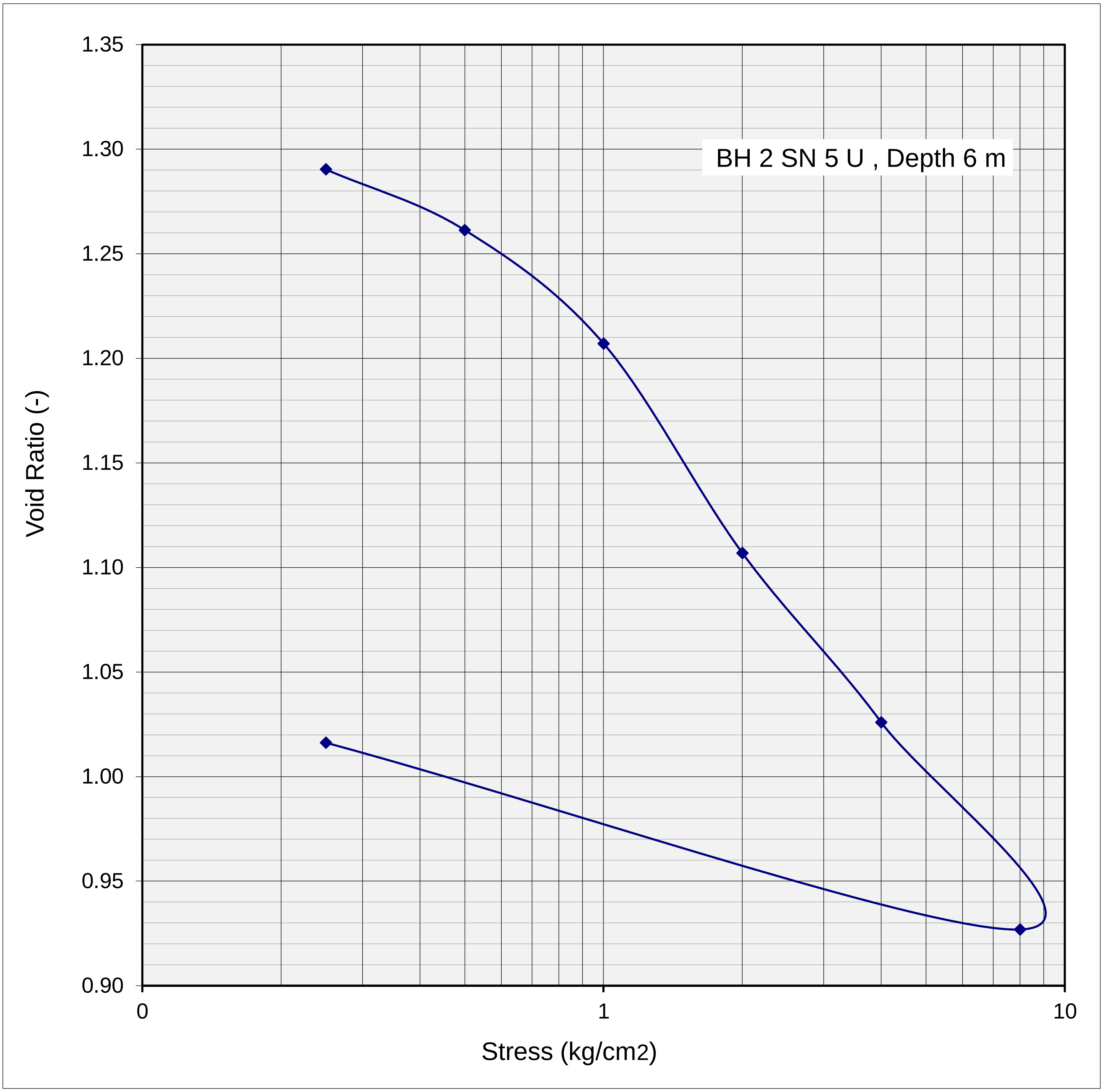

BH 1, SN 6d , Depth 7.5 M
Stress (kPa)
0
50
100
150
200
250
300
350
400
Strain (-)
0.00
0.02
0.04
0.06
0.08
0.10
0.12


BH 2, SN 3U , Depth 3.0 M
Stress (kPa)
Strain (-)
0
5
10
15
20
25
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07

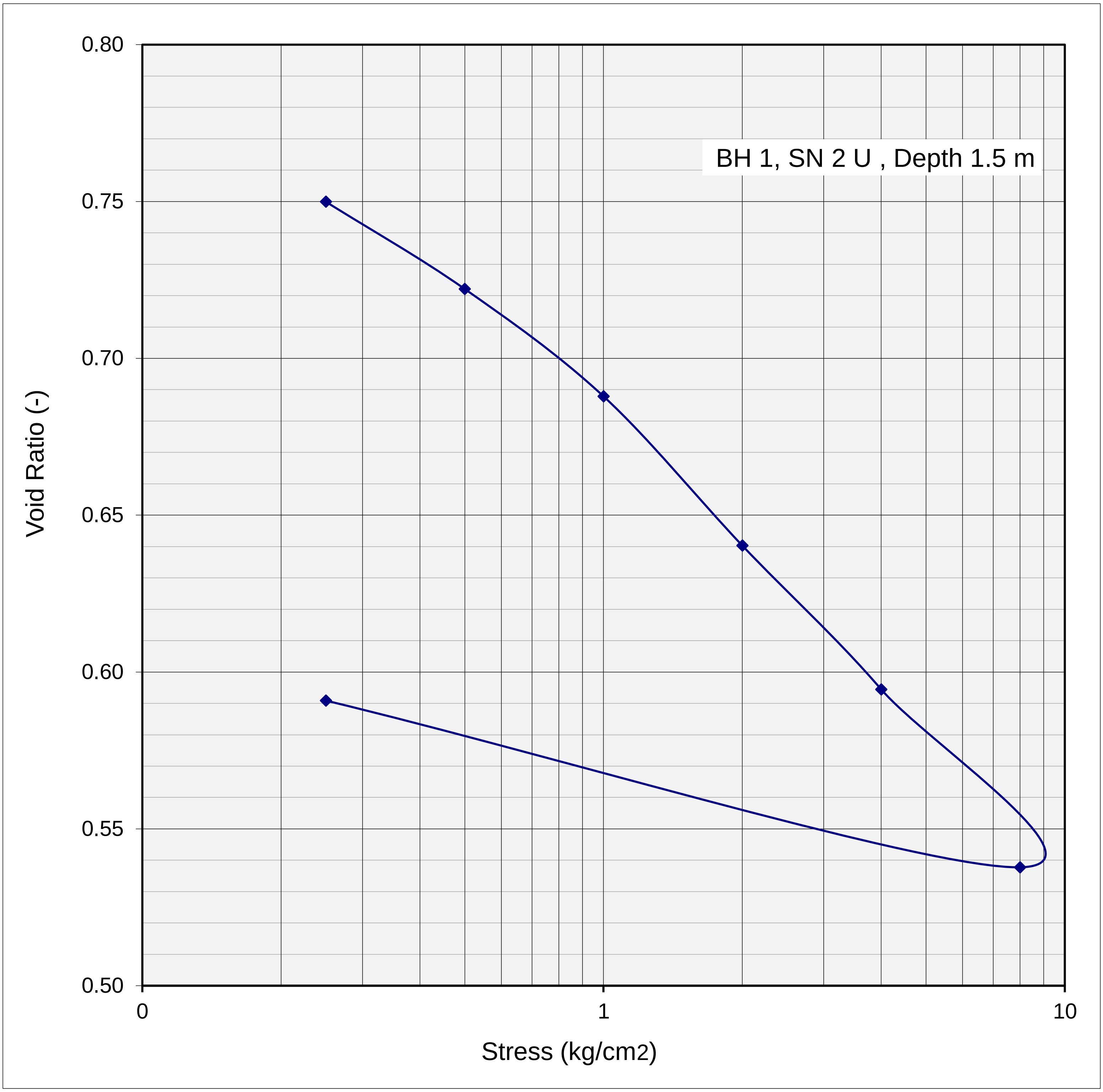

BH 2, SN 5U , Depth 6.0 M
Stress (kPa)
Strain (-)
0
5
10
15
20
25
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08


BH 2, SN 6d , Depth 7.5 M
Stress (kPa)
Strain (-)
0
50
100
150
200
250
0.00
0.02
0.04
0.06
0.08
0.10

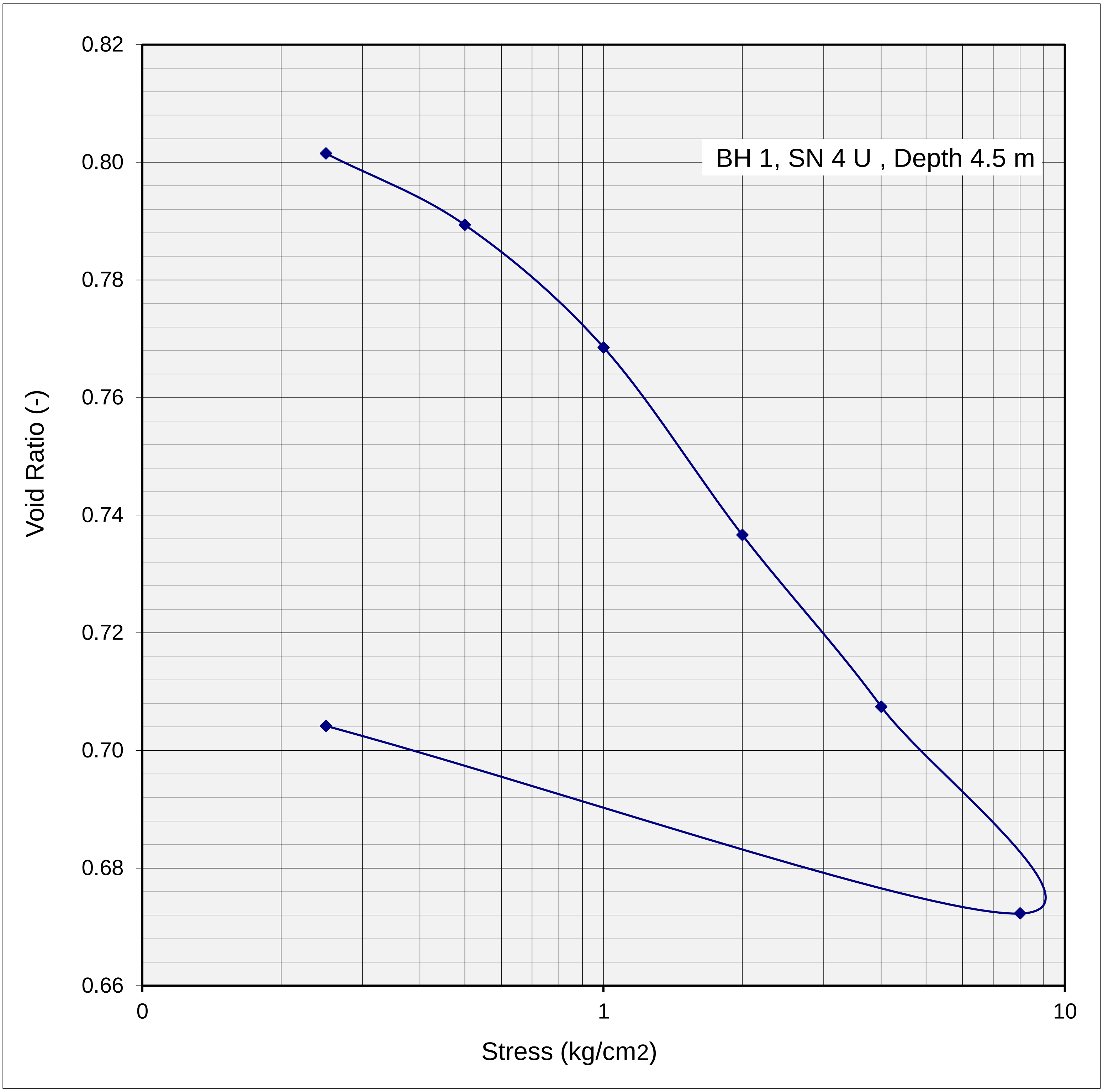

BH 1,SN 13d,Depth 20m
Shear Stress, kPa
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0
20
40
60
80
100
120
140
Normal Stress, kPa



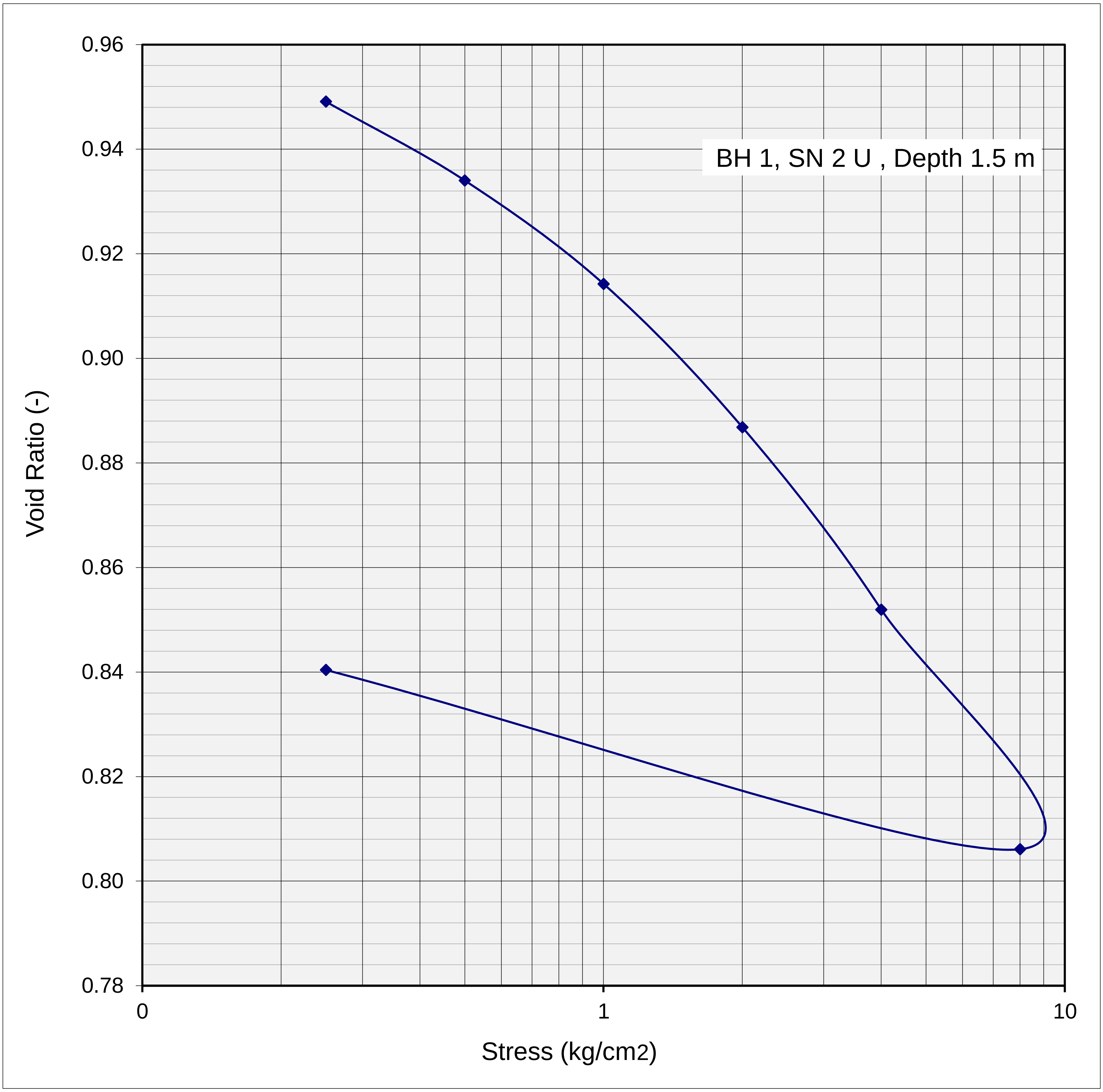
BH 2 SN 5 U , Depth 6 m
Void Ratio (-)
1.35
1.30
1.25
1.20
1.15
1.10
1.05
1.00
0.95
0.90
0
1
10
Stress (kg/cm2)



BH 1, SN 2 U , Depth 1.5 m
Void Ratio (-)
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0
1
10
Stress (kg/cm2)



BH 1, SN 4 U , Depth 4.5 m
Void Ratio (-)
Stress (kg/cm2)
0.82
0.80
0.78
0.76
0.74
0.72
0.70
0.68
0.66
0
1
10



BH 1, SN 2 U , Depth 1.5 m
Void Ratio (-)
0.96
0.94
0.92
0.90
0.88
0.86
0.84
0.82
0.80
0.78
0
1
10
Stress (kg/cm2)

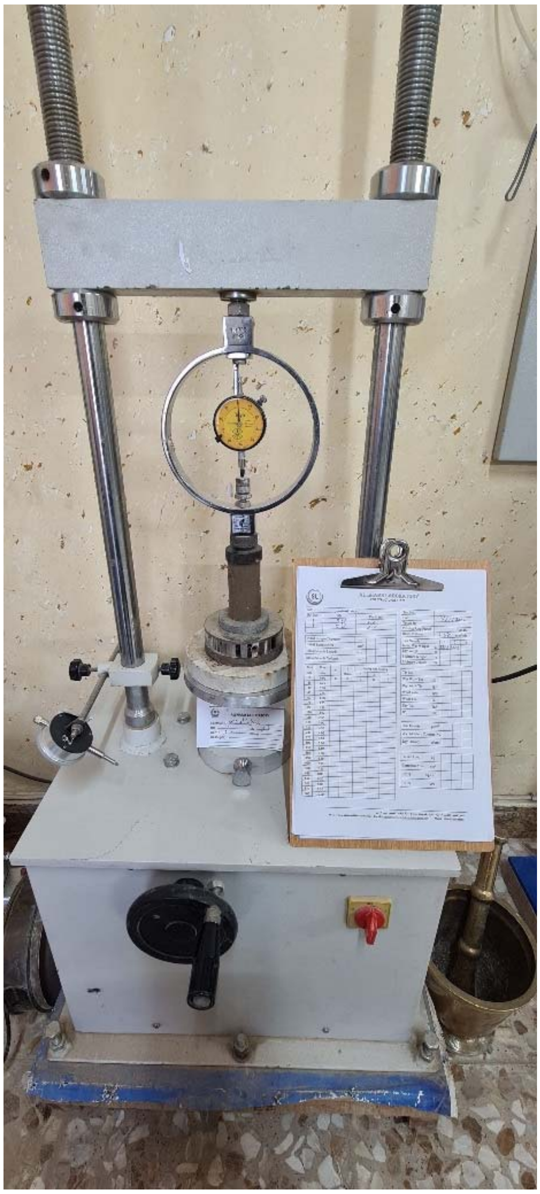
APPENDIX-D-

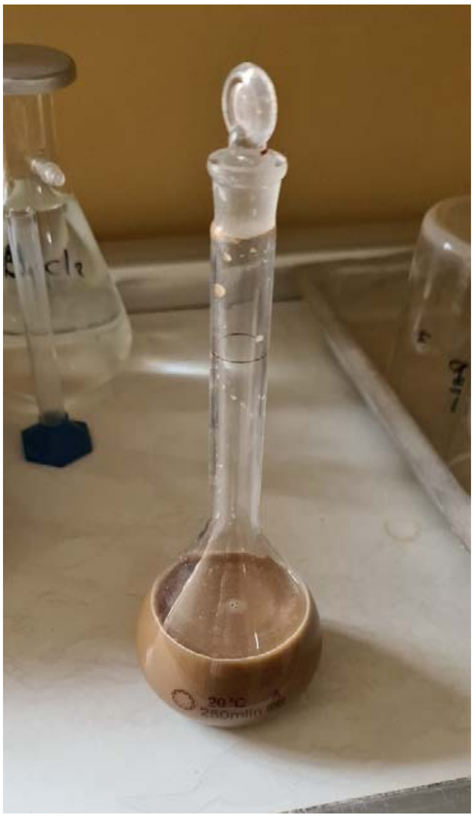
Photography





C1
C2
C3
C4
C5
C6
C7






sartorius
86.3654 g
M-power