ELECTROGAS MALTA CONSORTIUM

Delimara LNG Regasification Terminal

Geotechnical Investigation for Onshore Regasification Facilities – Evaluation Report

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1. INTRODUCTION

1.1 Purpose and Scope

This report contains presentation and evaluation of the geotechnical investigations which were carried out for the onshore Regasification Facilities of the LNG Regasification Terminal at Delimara, Malta for the purpose of deriving geotechnical design parameters.

The results of the field investigations and laboratory tests are contained in the report No "2779-77-CI-RE-00006- Geotechnical Investigation for Regasification Plant Area- Factual Report" (25 Feb. 2015). This report will be referred to hereafter as Factual Report.

1.2 Work Performed

Field work was carried out between 28.10.2014 and 04.11.2014.

Planning and coordination of the field investigations and the laboratory testing was made by Mr. Sp. Nikoloudakos, Senior Geologist of Castor Ltd. Mr Nikoloudakos supervised all the on land boreholes and partly the laboratory tests.

Execution of the boreholes and the laboratory tests was done by Messrs Terracore Ltd, Malta.

2. BACKGROUND INFORMATION

2.1 General

Detailed description of the facilities of the Regasification plant , general arrangement, loadings and foundations will be given in the Design Reports and drawings.

2.2 Geotechnical Reports Related to the Project

The following reports contain geotechnical data at the area of the Delimara LNG Regasification facilities:

a. "2779-77-CI-RE-00010- Excavation and Disposal for Regasification Plant-Geotechnical Slope Stability Study Report" (19 Nov. 2014)

b. "2779-77-CI-RE-00001- Excavation and Disposal for Regasification Plant-Geotechnical Investigation -Factual Report" (19 Nov. 2014)

3. FIELD INVESTIGATIONS AND LABORATORY TESTS

3.1 Field Investigations

Four (4) boreholes, namely GL1 to GL4, were rotary drilled at the locations shown in figure

1.

Boreholes GL1, GL2 and GL4 were drilled from elevation +6.2m after removal of a previous fill to create a graded level at the area of the Regasification Facilities (not shown in figure 1). Borehole GL3 was drilled at the top of the existing deposits at elevation +24.0m near the recently excavated permanent slope.

All boreholes were drilled into the natural ground to the depths shown in the subsurface profile of figure 2. Standard penetration tests were executed in general at 2m to 3m intervals and undisturbed samples were obtained in Shelby tubes.

All samples were properly labeled and transferred to the testing laboratory in Malta.

The results of the field investigations and the laboratory tests on samples from the boreholes are presented in the Factual Report ("2779-77-CI-RE-00001- Excavation and Disposal for Regasification Plant-Geotechnical Investigation -Factual Report" (19 Nov. 2014).

3.2 Piezometers

One standpipe piezometer was installed in borehole GL1 in order to be able to monitor fluctuations of ground water table (if any) with time. Piezometer tip was sealed with betonite above sea level.

3.3 Laboratory Tests

Laboratory tests on samples from the boreholes were carried out in Malta in according to the British Standards BS1377:1990. Tables 1 and 2 summarize the results of soil classification tests, and unconsolidated undrained (UU) Triaxial strength tests respectively. Both tables were compiled on the basis of relevant data presented in the Factual Report.

Soil descriptions are according to the Unified Soil Clarification System (USCS).

The results of the UU triaxial test results on samples from the boreholes are presented in table 2 for info only, as they were not used in this report for the evaluation of the geotechnical investigations for the reasons given in the Note of table 2.

In addition to the above, the Geotechnical Consultant, Castor Ltd, requested the execution of three UU triaxial tests on reconstituted specimens on materials from the deposits at 95% of maximum modified Proctor density with compaction water content equal to the optimum increased by 4 units. The results of these tests were never reported.

4. PRESENTATION OF THE GEOTECHNICAL INVESTIGATION DATA

4.1 General

The results of the geotechnical investigations are summarized in the subsurface profile of figure 2. It can be seen that subsoil can be broadly differentiated in the deposits of excavation materials (formations Da & Db), the upper soil formations (layers II and I Ia) and the underlying very low and low strength marly rock formations (layers III and IV).

The subsurface conditions at the flat area at elevation +6.2m for the Regasification Facilities of the complex are presented in the following paragraph 4.2, while the subsurface conditions near the excavated slope at the deposits are presented in paragraph 4.3.

Characteristics of the soil formations (deposits Da & Db plus layers I, II and I Ia) considered as one entity are presented in paragraph 4.4. Strength characteristics of the underlying very low and low strength marly rock formations (layers III and IV) are given in paragraphs 4.5 and 4.6 respectively.

Evaluation of the geotechnical data and selection of design parameters are given in chapter 5.

Important Note: The main characteristic of the soil formations is that they have low values of specific gravity of grains (G_s), which were found to vary between 2.47 and 2.57 (see table 1). This observation was reported also in both previous geotechnical reports of paragraph 2.2. Similarly, the very low and low strength marly rock formations have low values of bulk unit weight of the order of 2.13 Mg/m^3 (see table 5).

4.2 Area of the Regasification Facilities

Based on the results of boreholes GL1, GL2 and GL4 which were drilled from elevation +6.2m, subsurface consists of the following formations:

- Deposits Db: Medium dense clayey gravel (GC) which extends from the graded level at elevation +6.2m to depths of 6m to 7m.

- Layer II: Medium dense clayey gravel (GC) and sandy clay (SP-SC) with thickness 3m to 4m. This layer was encountered only in borehole GL1
- Layer IIa: Dense clayey gravel (GC) and silty sand (SM) with thickness 2m to 3m in boreholes GL2 and GL4.
- Layer III : Very low strength clay marl of light brown to light grey colour with thickness of 0.5m to 2m. This layer was encountered in all boreholes.
- Layer IV: Low strength limestone of grey colour which was encountered in boreholes GL1, GL2 and GL4 at elevations -2.0m to -5.0m, underlying all previous layers.

Layers II, IIa, III and IV are similar to those presented in the two geotechnical reports listed in paragraph 2.2.

Ground Water Table : Water level in the surface piezometers of borehole GL1 dropped within one month after completion of the borehole, from 2.4m to 4.6m from ground at elevation of +6.2m. It is believed that this is entrapped water from the borehole drilling fluid and that ground water table will eventually reach the sea level at elevation ±0.0.

4.3 Excavated Permanent Slope (Borehole GL3) - Subsurface Conditions

Based on the results of borehole GL3 which was drilled at the top of the slope, subsurface conditions consist of the following formations:

. Deposits Da	Loose to medium dense clayey sandy gravel (GC) and gravelly sandy clay (CL), with thickness of the order of 28m.
• Layer I	Very loose to loose silty sand (SM) underlying formation Da with thickness of the order of 9m.
• Layer III	Very low strength clay marl of light brown to grey colour with thickness of about 8m.
. Layer IV	Low strength marly limestone of grey colour with thickness of about 6m.

Layers I, III and IV are similar to those presented in the two geotechnical reports listed in paragraph 2.2.

4.4 Characteristics of the Soil Formations (DepositsDa & Db, Layers I, II and IIa)

a. <u>Classification Properties</u>

Classification properties of the above soil formations are given in figure 3. Original data are given in table 1.

Water contents vary between 15 and 40 percent and dry unit weights between a minimum of 1.30 Mg/m^3 and a maximum of 1.88 Mg/m^3. Water contents are higher than plastic limits (figure 3c).

b. <u>Organics Content</u>

Loss on ignition on one sample from borehole GL2 yielded a value of 7.1 percent (table 4).

c. Compaction Characteristics

One modified Proctor compaction test was performed according to BS1377-4 : 1990 on a composite sample of excavation materials which was taken in the area of boreholes GL1, GL3 and GL4. It yielded the following results:

- Maximum dry unit weight: Yd_{max} = 1.81 Mg/m^3
  Optimum water content: w_{opt}=13%

Original data are given in the Factual Report.

d. Compressibility

Two one-dimensional consolidation tests were carried out on Shelby tube samples of the Deposits Db and of layer I in boreholes GL1 and GL3 respectively.

Figure 4 shows the results of these tests plotted in terms of oedometer modulus E_{oe}d versus vertical pressure ($E_O$ed=1/Mv). Analytical data are given in Factual Report and summarized in table 3. Oedometer modulus values are low at vertical pressures equal to the overburden plus 200kPa, E_{oe}d was found to be equal to 4750kPa for the deposits Db and equal to 6100kPa for layer I.

In addition to these, one oedometer test was performed on a reconstituted specimen of deposited materials with the following compaction characteristics:

Dry unit weight: 95% Yd_{max} = 95% 1.81 Mg/m^3 = 1.72Mg/m^3
Compaction water content: w_{opt} + 4% = 13% + 4% =17%

The results of this test are given in figure 4 plotted in terms of oedometer modulus E_{oe}d versus vertical pressure. Analytical data are given in the Factual Report and summarized in Table 3. Oedometer modulus values are low: at vertical pressure equal to 300kPa the oedometer modulus E_{oe}d was found to be equal to 5200kPa.

4.5 Characteristics of the Clay Marl (Layer III)

Layer III consists of very low strength clay marl. As mentioned in paragraphs 4.2 and 4.3, this layer was encountered in boreholes GL1, GL2 and GL4 with thickness of 0.5m to 2m, and in borehole GL3 with thickness of about 8m.

Strength and compressibility characteristics from one sample are given in table 5. Similar data were also obtained from the geotechnical reports of the offshore investigations and the investigations at the deposits (see paragraph 2.2) and are included in figures 7 and 8.

4.6 Characteristics of the Underlying Marly Limestone Formation (Layer IV)

Layer IV consists of low strength marly limestone of grey colour. Statistical presentations of strength, bulk unit weight and Young's modulus are given in figures 5 and 6. Original data are given in the Factual Report and summarized in table 5.

Uniaxial compressive strengths vary between a minimum of 8.2 MPa and a maximum of 14.6MPa. Youngs modulus values vary between a minimum of about 1400kPa and a maximum of about 9100kPa. The majority of the reported Poisson ratio values are high and therefore not realistic.

Point load strength Is_{50} axial varies between 1.9MPa and 2.8MPa, and Is_{50} oblique between 0.8MPaand 1.3MPa.

Bulk unit weight is low varying between a minimum of 2.10 Mg/m^3 and a maximum of 2.23Mg/m^3.

4.7 Seismicity

According to EN 1998-1: 2004, the subsurface formations are classified to the following ground types:

- Deposits Da: Ground type D
  Deposits Db: Ground type D
- Layer II: Ground type C
  Layer I Ia: Ground type C
- Layer III: Ground type A
  Layer IV: Ground type A

5. EVALUATION AND SELECTION OF GEOTECHNICAL DESIGN PARAMETERS

5.1 Evaluation of the Geotechnical Investigation Data

Evaluation of the field investigations and the laboratory tests, leading to selection of geotechnical design parameters is given in Appendix A. Sheets A-1 to A-9 present the evaluation of strength and compressibility parameters of the upper soil formations, while sheets A-10 to A-18 present the evaluation of strength and compressibility parameters of the underlying marly rock formations.

Cautious estimates of design parameters for each borehole are given in figures 7 and 8.

Selection of geotechnical design parameters for the Regasification Facilities will be based on these data when the loads and the layout of these installations will be finalized, depending on the method(s) of foundation which will be decided to be applied.

TABLES

TABLE 1 SUMMARY OF LABORATORY TEST RESULTS

Borehole/ Sample	Depth (m)	Water content (%)	Dry unit weight (t/m³)	Liquid Limit LL (%)	Plastic Limit PL (%)	Plasticity Index PI	GRADATION ANALYSIS			Specific Gravity (Gs)	Soil type	Layer
							Gravel (%)	Sand (%)	Fines (%)			
GL1/2	1.8	17		35	11	24	41	19	40		GC	Db
GL1/5	4.3	20	1.71	34	10	24	39	15	46	2.47	GC	Db
GL1/-	6.5	18	1.72	33	11	22						Db
GL1/8	8.8	18	1.80	33	11	22	38	18	43	2.53	GC	II
GL1/11	9.8	31		27	9	18	35	54	11		SP-SC	II
GL2/2	1.8	22		36	11	25	45	24	31		GC	Db
GL2/5	4.3	20	1.72	34	11	23	49	22	29		GC	Db
GL2/8	7.8	21		32	10	22	67	13	20		GC	IIa
GL2/10	8.4	26	1.75	NP	NP	NP	23	61	16		SM	IIa
GL3/8	8.3	17	1.72	36	11	25	64	14	22		GC	Da
GL3/17	21.3	17	1.82	36	11	25	35	13	52		CL	Da
GL3/23	29.3	39		NP	NP	NP	5	73	22		SM	I
GL3/26a	32.4	40	1.30	NP	NP	NP	5	67	28	2.57	SM	I
GL4/2	2.3	11		37	11	26	36	28	36		GC	Db
GL4/4	4.9	15	1.88	31	9	22	53	15	32		GC	Db
GL4/7	8.9	18		31	10	21	70	12	18		GC	IV

TABLE 2 SUMMARY OF TRIAXIAL TEST RESULTS UU

Borehole	Sample	Sample Depth (m)	Water content (%)	Dry unit weight (t/m^3)	Cell pressure σ_3 (kPa)	Maximum axial stress (σ_1-σ_3) (kPa)	Soil type	Undrained shear strength c_u (kPa)	Layer	N_{SPT} (blows)
GL1	2	1.5	23	1.32	40	128	GC	64	Db	14
	11	9.5	25	1.59	160	41	GC,SC	20	II	19
GL2	2	1.5	25	1.39	50	40	GC	20	Db	28
	8	7.5	24	1.59	140	185	SM	93	IIa	37
GL3	8	8.5	28	1.31	160	91	GC	46	Da	8
	17	21.0	27	1.44	420	244	CL	122	Da	6
	23	29.0	35	1.37	540	170	SM	85	II	3
GL4	2	2.0	20	1.34	40	108	GC	54	Db	15
	6	7.3	12	1.84	150	728	GC	364	IIa	15

UU: Triaxial test unconsolidated, undrained.

N_{SPT} values were taken from closest depths.

NOTE: Above tests were carried out on remoulded specimens from the boreholes at considerably lower dry unit weigths, of the order of 82%, of the original values in table 1, and at higher water contents. For this reason it was not possible to consider them for the evaluation of the geotechnical investigatons.

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TABLE 3 **SUMMARY OF ONE-DIMENSIONAL CONSOLIDATION TEST RESULTS**

BH	Sample	Depth (m)	Overburden Stress (kPa)	Water Content (%)	Liquid Limit (%)	Plasticity Index	Dry Unit Weight (t/m^3)	Specific Gravity	Void Ratio e_o	Degree Saturation (%)	Oedometer Modulus E_{oed} (kPa)	Soil Type	Layer	N_{SPT}
GL1	8	6.5	130	18	33.3	22.7	1.72	2.53	0.510	91	4750	GC	Db	23
GL3	26	32.2	560	19	NP	NP	1.41	2.56	0.833	60	6100	SM	I	7
Reconstituted specimen at 95% $\gamma d_{max}=1.72 Mg/m^3$ and $w_{opt}+4\%=17\%$				17	NR	NR	1.72	Assumed 2.52	0.510	87	At 300kPa $E_{oed}=5200kPa$	-	-	-

NOTES:

Compression Index Cc is taken from the virgin compression curve.
The oedometer modulus (E_{oed}) is taken at vertical pressure equal to the overburden pressure γH + 200kPa.
Coefficient of consolidation is given as average value.

N_{SPT} values were taken from closest depths.

NP: Non Plastic

$\gamma d_{max}=1.81 Mg/m^3$: Maximum dry unit weight of modified proctor compaction test

$w_{opt}=13\%$: Optimum water content of modified proctor compaction test

TABLE 4 SUMMARY OF ORGANICS CONTENT DETERMINATIONS

Borehole	Sample	Depth (m)	Loss on Ignition (%)	Soil Type	Layer
GL2	9	7.0	7.10	SM	IIa

TABLE 5 SUMMARY OF UNIAXIAL TEST RESULTS ON ROCK SPECIMENS

Borehole	Depth (m)	Specimen diameter D (cm)	Specimen height H (cm)	Ratio H/D	Specimen weight (gr)	Unit weight γ (t/m^3)	Load at failure F (N)	Stress at failure σ_c (MPa)	Stress at failure σ_{c50} (MPa)	Young's modulus E_m (MPa)	Poison ratio	Layer	Lithology
GL1	11.6	5.96	13.30	2.2	787	2.12	33600	12.10	12.49	9130	0.44	IV	Marly limestone
GL1	14.8	5.96	13.30	2.2	792	2.13	27400	9.80	NR	3800	0.73	IV	Marly limestone
GL1	20.2	5.95	13.30	2.2	824	2.23	29700	10.70	11.04	5490	0.11	IV	Marly limestone
GL1	23.1	5.44	13.30	2.4	NR	2.16	33800	14.60	14.82	7070	0.45	IV	Marly limestone
GL2	12.9	5.88	12.90	2.2	761	2.16	36200	13.30	13.69	5640	0.73	IV	Marly limestone
GL2	17.2	5.94	13.10	2.2	776	2.15	26800	9.70	10.01	4200	0.57	IV	Marly limestone
GL2	22.2	5.95	13.00	2.2	777	2.14	22900	8.30	8.56	2920	0.27	IV	Marly limestone
GL3	42.6	5.86	12.80	2.2	712	2.08	12600	4.70	4.84	670	0.21	III	Clay marl
GL3	47.4	5.85	12.70	2.2	725	2.12	31300	11.60	11.93	6210	1.01	IV	Marly limestone
GL3	50.4	5.85	12.80	2.2	730	2.12	29400	10.90	11.21	6970	0.78	IV	Marly limestone
GL4	13.2	5.95	13.10	2.2	778	2.14	35400	12.70	13.10	6040	0.64	IV	Marly limestone
GL4	16.3	5.94	13.20	2.2	767	2.10	22700	8.20	8.46	1440	0.29	IV	Marly limestone
GL4	19.9	5.94	13.20	2.2	784	2.14	38100	13.70	14.13	3570	0.31	IV	Marly limestone

NR: Not Reported

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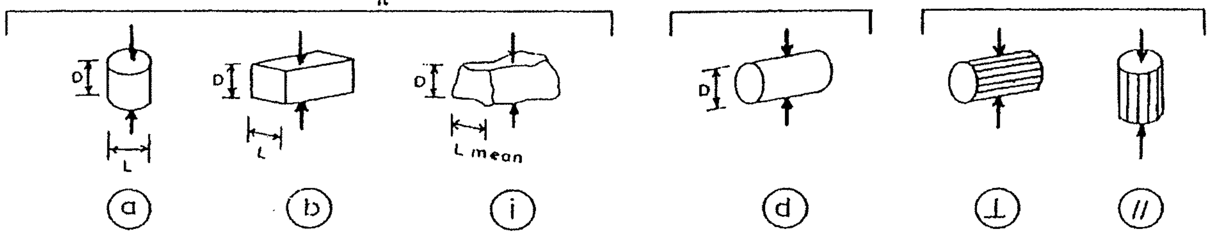
TABLE 6 SUMMARY OF POINT LOAD TEST RESULTS (continued)

Borehole	Depth (m)	Test type	Pick distance D (mm)	Character. dimension L (mm)	Equivalent dimension De (mm)	Failure Load F (N)	Index Is = F/De² (MPa)	Correction coefficient	Corrected index Is(50) (MPa)	Anisotropy index Ia<=1	Layer	Lithology
GL1	15.00	d	63.91	-	63.91	1560	2.10	1.19	2.50	2.50	IV	Marly limestone
		a	63.83	32.71	51.55	1560	1.00	1.01	1.00			
GL1	23.00	d	63.78	-	63.78	1750	2.40	1.20	2.80	2.15	IV	Marly limestone
		a	63.86	35.50	53.72	1750	1.20	1.03	1.30			
GL2	10.20	d	66.91	-	66.91	1140	1.60	1.23	1.90	0.33	III	Clay marl
		a	70.18	34.64	55.63	8820	5.50	1.05	5.80			
GL2	22.50	d	66.91	-	66.91	1140	1.60	1.23	1.90	2.38	IV	Marly limestone
		a	67.03	35.58	55.10	1140	0.80	1.04	0.80			
GL4	9.80	d	67.00	-	67.00	1420	1.70	1.19	2.10	1.91	IV	Marly limestone
		a	67.10	37.24	56.40	1420	1.00	1.06	1.10			

ΤΥΠΟΙ ΔΟΚΙΜΩΝ

$D_e^2 = \frac{4 \cdot D \cdot L}{\pi}$ $D_e = D$ ΑΝΙΣΟΤΡΟΠΟ



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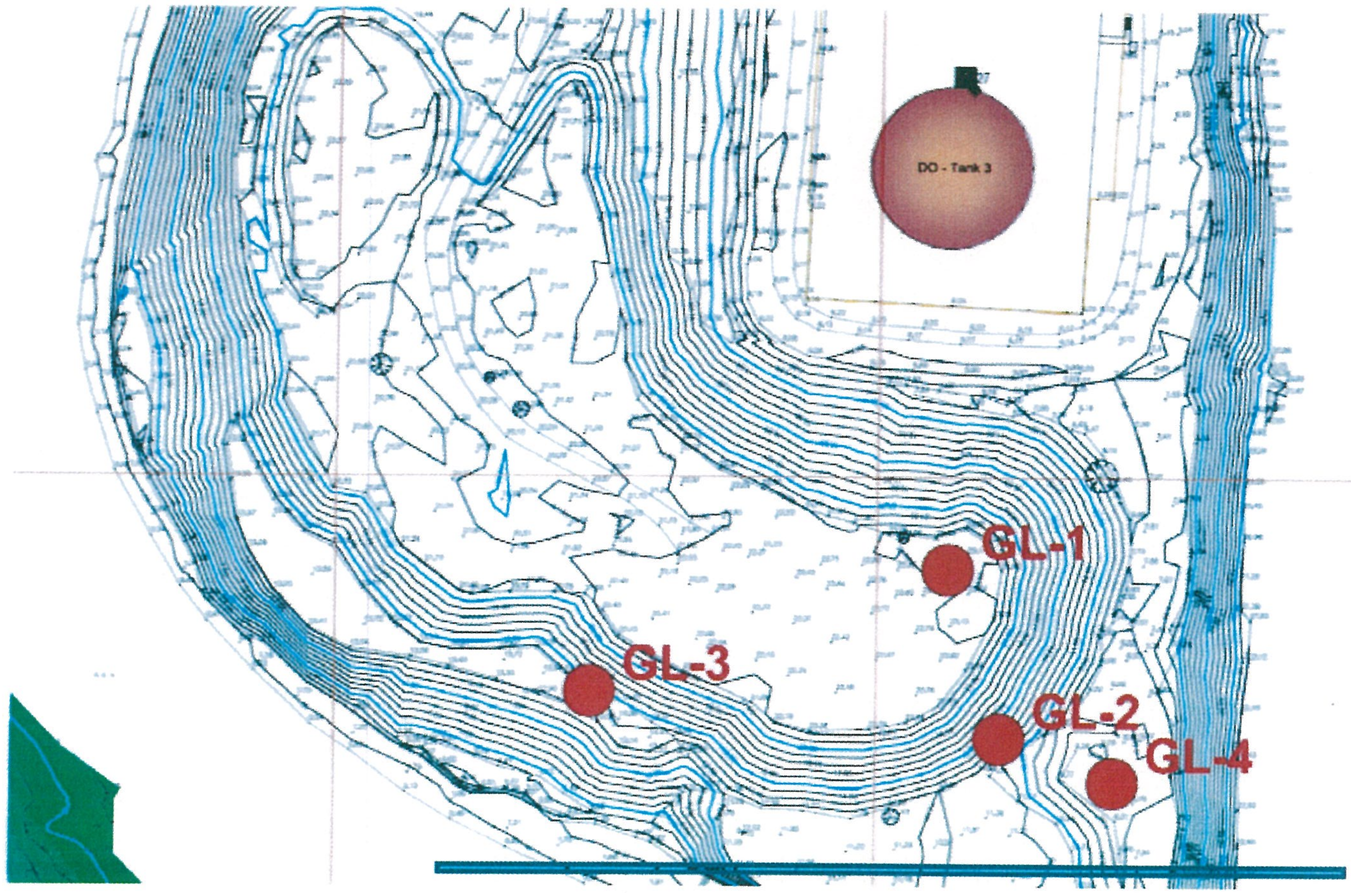
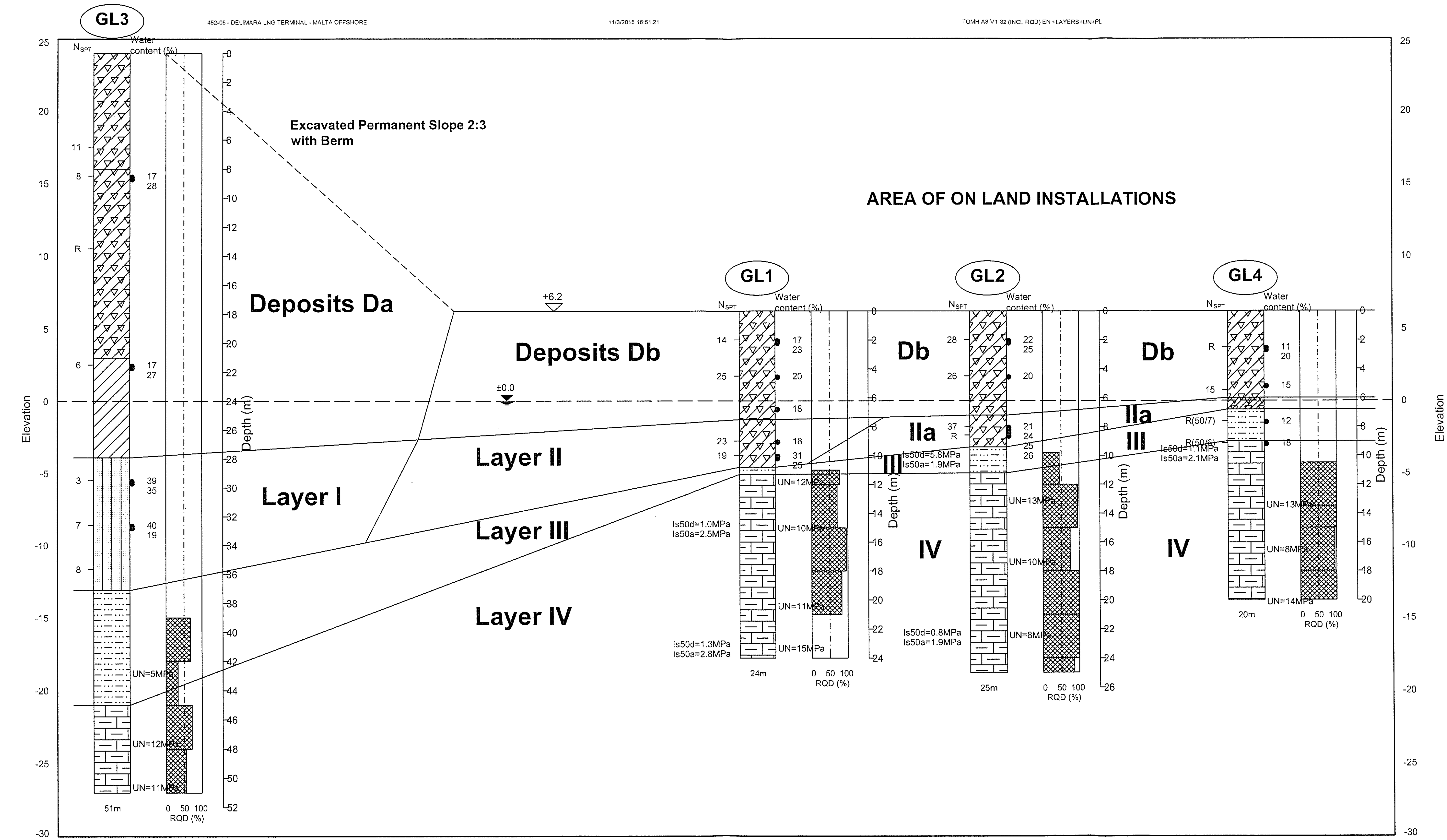
FIGURES



BOREHOLE LOCATIONS
FIG 1



LEGEND

10 ⊣	Standard Penetration Test NSPT (blows/30cm)
R ⊣	Refusal to SPT
R(50/7) ⊣	Penetration of 7 cm for 50 blows
● 17	Water Content (%)
	Ground Water Level

UN=	Uniaxial Compression Strength (MPa)
Is50d=	Point Load Strength Index - Oblique (MPa)
Is50a=	Point Load Strength Index - Axial (MPa)

GRAVEL (GC)

MARL

LIMESTONE MARLY

CLAY (CL)

SAND (SM)

Vertical Scale 1:250

SUBSURFACE PROFILE
FIG 2

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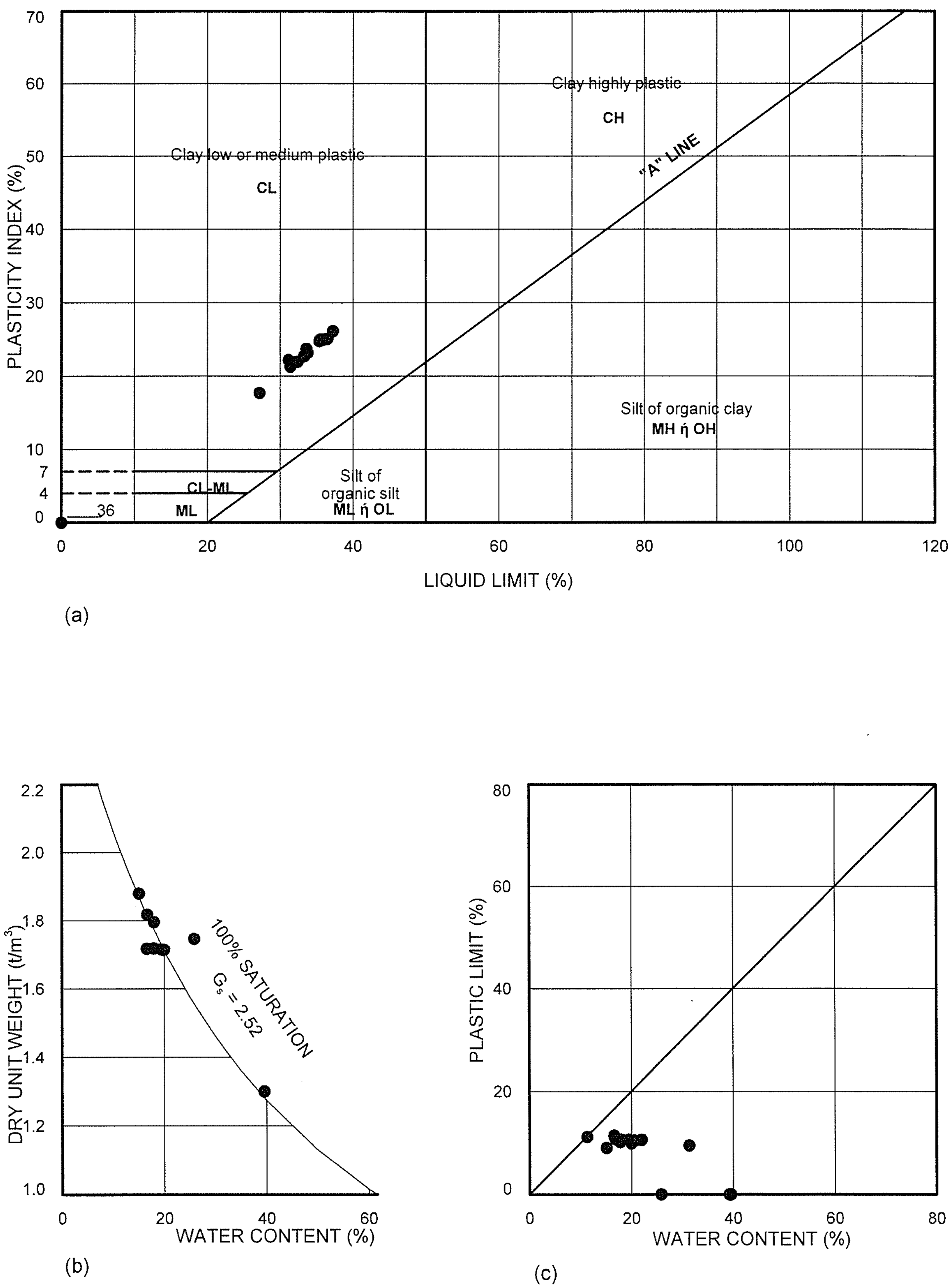

Soil formations: Deposits Da & Db, Layers I, II and IIb

SOIL FORMATIONS CLASSIFICATION CHART FIG 3

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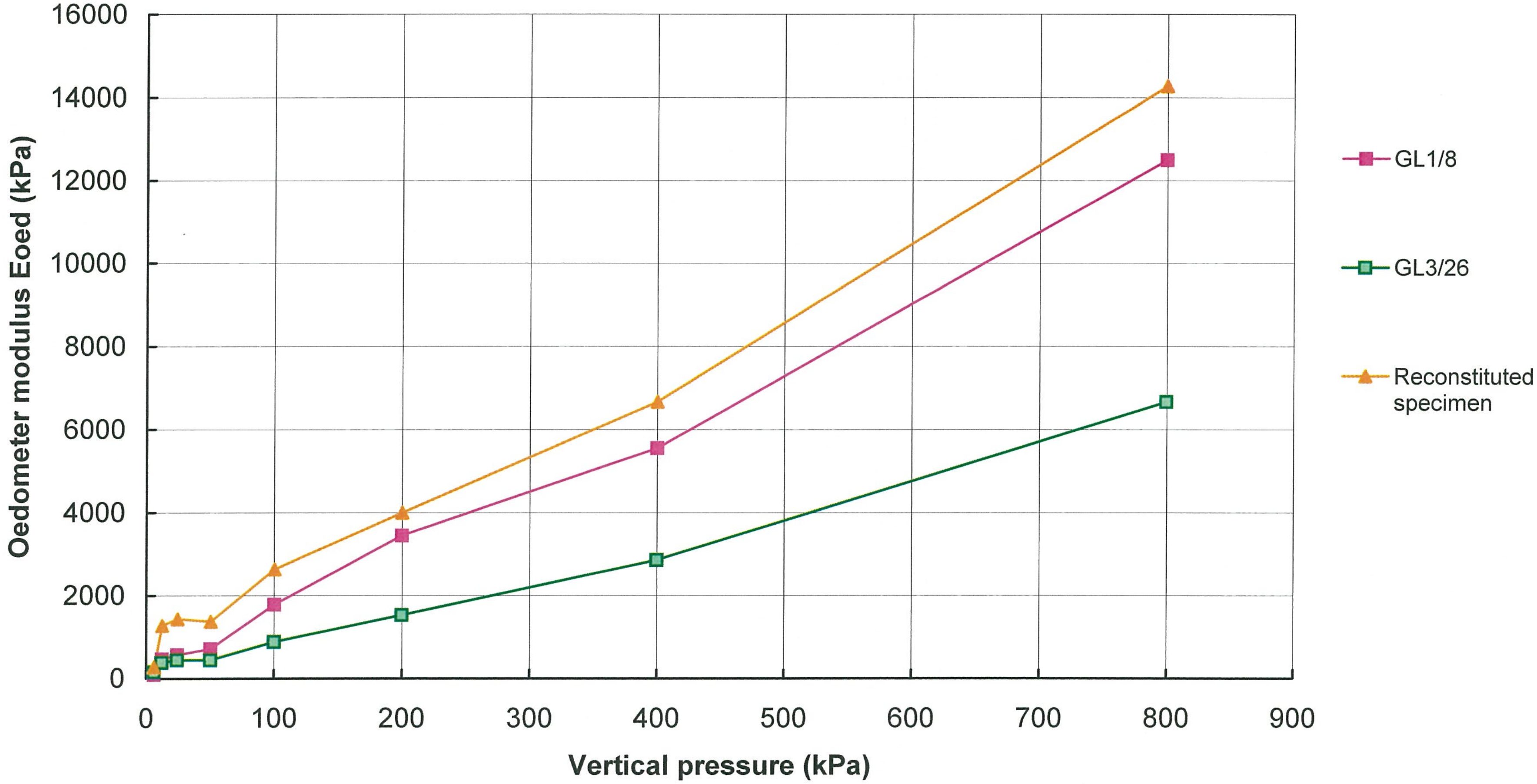
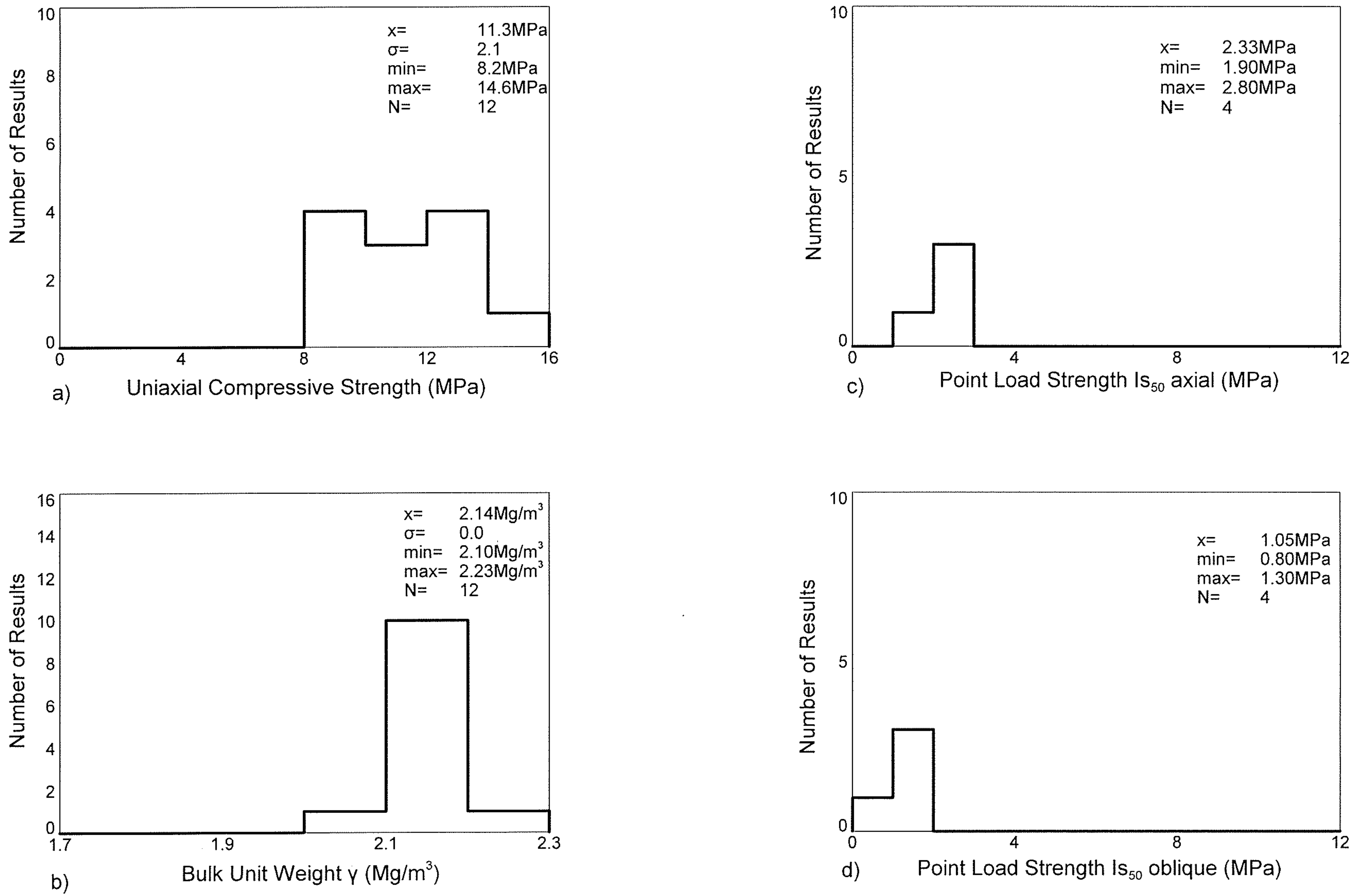


ONE-DIMENSIONAL CONSOLIDATION TESTS
FIG 4

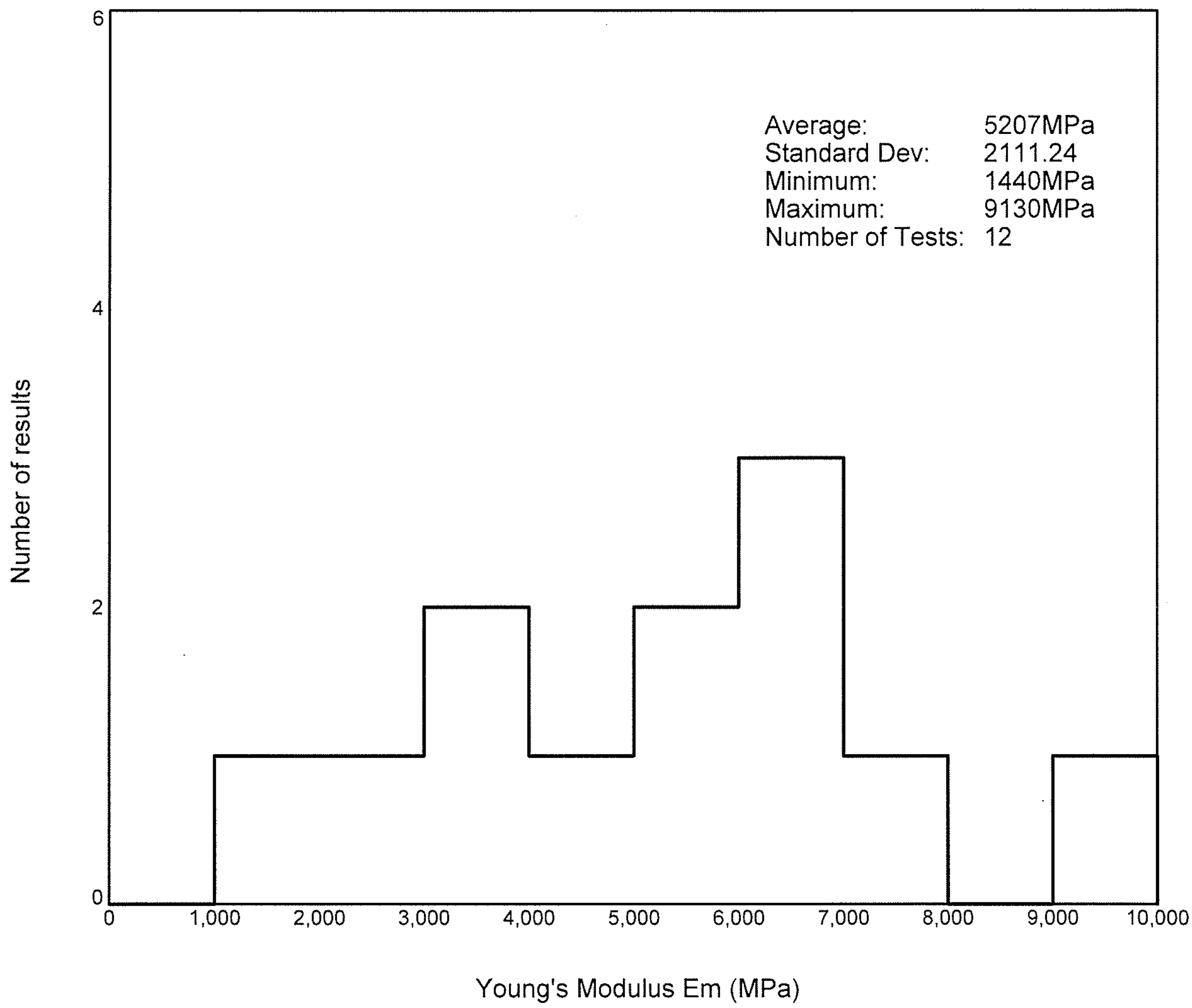
LAYER IV

Soft marly limestone of grey colour



STRENGTH AND BULK UNIT WEIGHT

FIG 5



Layer IV: Soft marly limestone of grey colour

LAYER IV
YOUNG'S MODULUS Em
STATISTICAL PRESENTATION
FIG 6

BOREHOLES GL1, GL2, GL4

(At Graded Level)

Elevation

+6.20 Soil surface

Existing Deposits Db (GC), (CL) Clayey-Sandy Gravel, Clay - Medium Dense

N_{SPT}= 25

c' = 10kPa c_u = 70kPa

φ' = 32° φ_u = 5°

γ = 19.5kN/m^3 E_{oed} = 10000kPa

6.0m to 7.5m

+0.2 to -1.3

Layer II (GC) Clayey Gravel Only in GL1 (-1.3 to -4.6) - Medium Dense

N_{SPT}= 20

c' = 10kPa c_u = 70kPa

φ' = 32° φ_u = 10°

γ = 19.0kN/m^3 E_{oed} = 12000kPa

Layer IIa (GC) Clayey Gravel - Dense

N_{SPT}= 40

c' = 10kPa c_u = 150kPa

φ' = 37° φ_u = 5°

γ = 19.5kN/m^3 E_{oed} = 18000kPa

0.5m to 2.2m

-0.8 to -5.0

Layer III CLAY MARL

RQD= 50% σ_{ci} = 5MPa

c' = 50kPa γ = 22kN/m^3

φ' = 28° E_m = 400MPa

0.5 to 2.2m

-2.8 to -5.1

Layer IV MARLY LIMESTONE

RQD= 80% σ_{ci} = 12MPa

c' = 100kPa γ = 22.5kN/m^3

φ' = 30° E_m = 1200MPa

11.0m to 13.8m

End of Borehole

Fig. A1 (no scale)

Legend

c_u= Undrained shear strength

φ_u= Undrained friction angle

c' = Effective cohesion

φ' = Effective internal friction angle

σ_{ci} = Uniaxial Compressive Strength

γ = Wet unit weight

E_{oed} = Oedometer modulus

RQD= Rock quality index

E_m= Rock deformation modulus

N_{SPT} = SPT , blow counts/30cm

BOREHOLES GL1, GL2, GL4
DESIGN PARAMETERS
FIG 7

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BOREHOLE GL3

(Top of Deposits)

Elevation
+24.0 Soil surface

Existing Deposits Da (GC), (CL) Clayey-Sandy Gravel, Clay - Loose — 27.9m

N_{SPT}= 8

c' = 5 to 10 kPa | c_u = 60kPa
φ' = 30° to 32° | φ_u = 5°
γ = 18.5kN/m^3 | E_{oed} = 7000kPa

-3.9

Layer I (SM) , Silty Sand - Loose — 9.2m

N_{SPT}= 6

c' = 5kPa | c_u = 40kPa
φ' = 30° | φ_u = 5°
γ = 18.5kN/m^3 | E_{oed} = 6000kPa

-13.1

Layer III CLAY MARL — 7.9m

RQD= 50% | σ_{ci} = 5MPa
c' = 50kPa | γ = 22kN/m^3
φ' = 28° | E_m = 400MPa

-21.0

Layer IV MARLY LIMESTONE — 6.0m

RQD= 60% | σ_{ci} = 12MPa
c' = 100kPa | γ = 22.5kN/m^3
φ' = 30° | E_m = 1000MPa

-27.0

End of Borehole

Fig. A2 (no scale)

Legend

c_u= Undrained shear strength
φ_u= Undrained friction angle
c' = Effective cohesion
φ' = Effective internal friction angle
σ_{ci} = Uniaxial Compressive Strength
γ = Wet unit weight
E_{oed} = Oedometer modulus
RQD= Rock quality index
E_m= Rock deformation modulus
N_{SPT} = SPT , blow counts/30cm

BOREHOLES GL3
DESIGN PARAMETERS
FIG 8

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APPENDIX A

EVALUATION AND SELECTION OF GEOTECHNICAL DESIGN PARAMETERS

A1. DESIGN PARAMETERS FOR SOIL FORMATIONS

A1.1 Effective Strength Parameters c' & φ'

a) General Relationships

Table 22. Correlations of internal friction angle and SPT N-values (data from Hatanaka and Uchida (1996) and Broms and Flodin (1988))

Soil Type	φ (degrees)	Reference
Angular and well-grained soil particles	$\phi = (12N)^{0.5} + 25$ (See Note)	Dunham (1954) (#1)
Round and well-grained or angular and uniform-grained soil particles	$\phi = (12N)^{0.5} + 20$ (See Note)	Dunham (1954) (#2)
Round and uniform-grained soil particles	$\phi = (12N)^{0.5} + 15$ (See Note)	Dunham (1954) (#3)
Sandy	$\phi = (20N)^{0.5} + 15$ (See Note)	Ohsaki et al. (1959)
Granular	$\phi = 20 + 3.5(N)^{0.5}$ (See Note)	Muromachi et al. (1974)
Sandy	$\phi = (15N)^{0.5} + 15 \leq 45$ $(N > 5)$ (See Note)	Japan Road Association (1990)
Sandy	$\phi = (20N_1)^{0.5} + 20$ N_1 = N-value normalized to 1 tsf of overburden pressure using the Liao and Whitman (1986) equation. It is the recommendation of this report to use $N_{1,60}$ with this correlation..	Hatanaka and Uchida (1996)

Equation 1

Note: As originally proposed, these correlations used the uncorrected SPT blowcount, N. However, hammers delivering 60% of the theoretical energy have been the most commonly used hammers for SPT tests, and it seems likely that the data on which these correlations were based was obtained primarily from tests with such hammers. It therefore seems logical to use N_{60} with these correlations, and it is the recommendation of this report that this be done.

(McGregor and Duncan, 1998)

Table A1

Representative values for angle of internal friction ϕ

	Type of test*		
Soil	**Unconsolidated-undrained, U**	**Consolidated-undrained, CU**	**Consolidated-drained, CD**
Gravel			
Medium size	40–55°		40–55°
Sandy	35–50°		35–50°
Sand			
Loose dry	28–34°		
Loose saturated	28–34°		
Dense dry	35–46°		43–50°
Dense saturated	1–2° less than dense dry		43–50°
Silt or silty sand			
Loose	20–22°		27–30°
Dense	25–30°		30–35°
Clay	0° if saturated	3–20°	20–42°

*See a laboratory manual on soil testing for a complete description of these tests, e.g., Bowles (1992).

Notes:

1. Use larger values as γ increases.
2. Use larger values for more angular particles.
3. Use larger values for well-graded sand and gravel mixtures (GW, SW).
4. Average values for gravels, 35–38°; sands, 32–34°.

Figure 2-35 Correlation between ϕ' and plasticity index I_p for normally consolidated (including marine) clays. Approximately 80 percent of data falls within one standard deviation. Only a few extreme scatter values are shown [*Data from several sources: Ladd et al. (1977), Bjerrum and Simons (1960), Kanja and Wolle (1977), Olsen et al. (1986).*]

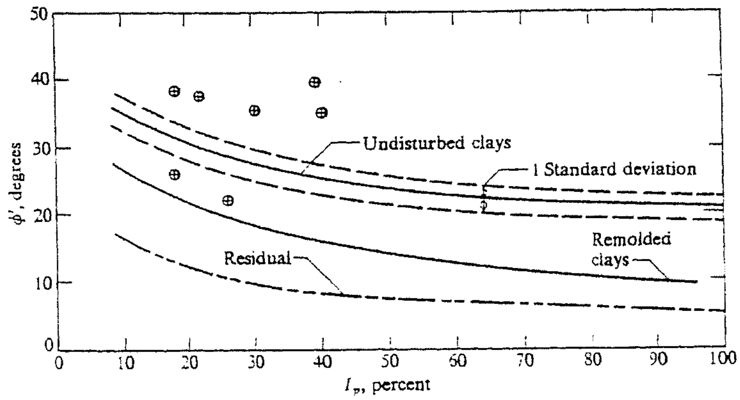

Fig. 2

(Bowles, 1996)

Table A2

TABLE 3.5
COMMON PROPERTIES OF COHESIONLESS SOILS**

Material	Compactness	D_R, %	N*	γ dry,† g/cm³	Void ratio e	Strength‡ ϕ
GW: well-graded gravels, gravel-sand mixtures	Dense	75	90	2.21	0.22	40
	Medium dense	50	55	2.08	0.28	36
	Loose	25	<28	1.97	0.36	32
GP: poorly graded gravels, gravel-sand mixtures	Dense	75	70	2.04	0.33	38
	Medium dense	50	50	1.92	0.39	35
	Loose	25	<20	1.83	0.47	32
SW: well-graded sands, gravelly sands	Dense	75	65	1.89	0.43	37
	Medium dense	50	35	1.79	0.49	34
	Loose	25	<15	1.70	0.57	30
SP: poorly graded sands, gravelly sands	Dense	75	50	1.76	0.52	36
	Medium dense	50	30	1.67	0.60	33
	Loose	25	<10	1.59	0.65	29
SM: silty sands	Dense	75	45	1.65	0.62	35
	Medium dense	50	25	1.55	0.74	32
	Loose	25	<8	1.49	0.80	29
ML: inorganic silts, very fine sands	Dense	75	35	1.49	0.80	33
	Medium dense	50	20	1.41	0.90	31
	Loose	25	<4	1.35	1.0	27

*N is blows per foot of penetration in the SPT. Adjustments for gradation are after Burmister (1962).[24] See Table 6.4 for general relationships of D_R vs. N.

†Density given is for G_s = 2.68 (quartz grains).

‡Friction angle ϕ depends on mineral type, normal stress, and grain angularity as well as D_R and gradation (see Fig. 3.29).

**From Hunt (1984).[1] Reprinted with permission of McGraw-Hill Book Company.

(Hunt, 1983)

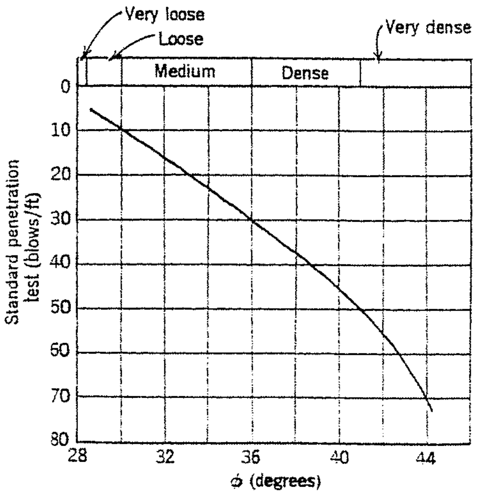

Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Fig. A1

Table A3

1 Soil Exploratory Work, Soil Investigations and Theoretical Soil Properties

1.1 Mean Soil Properties for Preliminary Designs (R 9)

1.1.1 General

The values in Table R 9-1, designated with the secondary sign cal, are theoretical values. They can be employed in the statical calculations for preliminary designs without further reductions. Mean values of a larger area are concerned here, whereby the values determined later on after the assessment of the various soil investigations for the relevant structure, for example according to R 96, section 1.12, can lie both above, as well as below.

1.1.2 Theoretical Values (Table R 9-1)

Type of Soil	Density: Above Water cal γ	Density: Sub-merged cal γ'	Final Strength: Angle of Internal Friction cal φ'	Final Strength: Co-hesion cal c'	Initial Strength[1]) Shear Strength cal c_u	Coefficient of Compres-sibility cal E_s
	kN/m³	kN/m³	degree	kN/m²	kN/m²	MN/m²
Non-cohesive Soils						
Sand, loose, round	18	10	30	–	–	20 – 50
Sand, loose, angular	18	10	32.5	–	–	40 – 80
Sand, medium dense, round	19	11	32.5	–	–	50 – 100
Sand, medium dense, angular	19	11	35	–	–	80 – 150
Gravel without sand	16	10	37.5	–	–	100 – 200
Coarse gravel, sharp edged	18	11	40	–	–	150 – 300
Sand, dense, angular	19	11	37.5	–	–	150 – 250
Cohesive Soils	(Empirical Values for Undisturbed Samples from the North German Area)					
Clay, semi-firm	19	9	25	25	50 – 100	5 – 10
Clay, difficult to knead, stiff	18	8	20	20	25 – 50	2.5 – 5
Clay, easy to knead, soft	17	7	17.5	10	10 – 25	1 – 2.5
Boulder clay, solid	22	12	30	25	200 – 700	30 – 100
Loam, semi-firm	21	11	27.5	10	50 – 100	5 – 20
Loam, soft	19	9	27.5	–	10 – 25	4 – 8

(Table R 9-1 cont)

Type of Soil	Density: Above Water cal γ	Density: Sub-merged cal γ'	Final Strength: Angle of Internal Friction cal φ'	Final Strength: Co-hesion cal c'	Initial Strength[1]) Shear Strength cal c_u	Coefficient of Compres-sibility cal E_s
	kN/m³	kN/m³	degree	kN/m²	kN/m²	MN/m²
Silt	18	8	27.5	–	10 – 50	3 – 10
Soft, org. slightly clayey sea silt	17	7	20	10	10 – 25	2 – 5
Soft, very org. strongly clayey sea silt	14	4	15	15	10 – 20	0.5 – 3
Peat	11	1	15	5	–	0.4 – 1
Peat under moderate initial loading	13	3	15	10	–	0.8 – 2

cal φ' = Theoretical value of the angle of internal friction in cohesive and non-cohesive soils
cal c' = Theoretical value of the cohesion, corresponding to cal φ'
cal c_u = Theoretical value of the shear strength from undrained tests in saturated cohesive soils

[1]) The appertaining angle of internal friction is to be assumed at cal $\varphi'_u = 0$

1.1.3

In the absence of other information, loose deposit is to be assumed for undisturbed sandy soil. Except in older geological stratifications, medium dense compaction is to be expected only after compaction by vibration or tamping. The values for gravelly sand are the same as for sand. The density given for coarse gravel is a rough average value. The actual density depends on the type of rock.

1.1.4

The angles of internal friction cal φ' and the cohesion cal c' for cohesive soils are rough average values for calculating the final stability (consolidated state = final strength). If soft to stiff clay and silty clay layers of considerable depth will receive a surcharge such as backfill, structures, etc., the influence of pore pressure is to be considered in the determination of the active earth pressure (initial strength). In some cases, the initial strength may also be considered in the determination of the passive earth pressure.

1.1.5

As static penetrometer tests in loosely deposited soils can frequently be executed economically and quickly in many cases, it is frequently justifiable to already carry out such tests for preliminary designs. As a result, the correct classification in the table, section 1.1.2 is enabled through the thereby determined approximate degree of density of sand. Reference is made to DIN 4094 and to [1], as well as [2] regarding the evaluation for cal c_u and the expected modulus of volume change.

(EAU, 1985)

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b) Selection of Soil Design Parameters c' & φ'

Borehole GL3

Layer Da:

(GC), (CL)	From empirical equations	$\varphi'=(12N)^{0.5}+20=30°$ c'=5 to 10kPa, φ' = 30°
$N_{SPT}=8$	From laboratory test results	c'=-, φ' = -
	Selected values	c'=5 to 10kPa, φ' = 30° to 32°

Layer I:

(SM)	From empirical equations	$\varphi'=(12N)^{0.5}+20=29°$ φ'=28° to 34° c'=5kPa, φ' = 30°
$N_{SPT}=6$	From laboratory test results	c'=-, φ' = -
	Selected values	c'=5kPa, φ' = 30°

Borehole GL1, GL2, GL4

Layer Db:

(GC), (CL)	From empirical equations	$\varphi'=(12N)^{0.5}+20=35°$ c'=10kPa, φ' = 35°
$N_{SPT}=25$	From laboratory test results	c'= - , φ' = -
	Selected values	c'=10kPa, φ' = 32°

Layer II:

(GC)	From empirical equations	$\varphi'=(15N)^{0.5}+15=32°$ c'=10kPa, φ' = 32°
$N_{SPT}=20$	From laboratory test results	c'= - , φ' = -°
	Selected values	c'=10kPa, φ' = 32°

Layer IIa:

(GC), (SM)	From empirical equations	$\varphi'=(12N)^{0.5}+15=37°$ c'=10kPa, φ' = 37°
$N_{SPT}=40$	From laboratory test results	c'= - , φ' = -°
	Selected values	c'=10kPa, φ' = 37°

A1.2 Undrained Shear Strength c_u

a) General Relationships

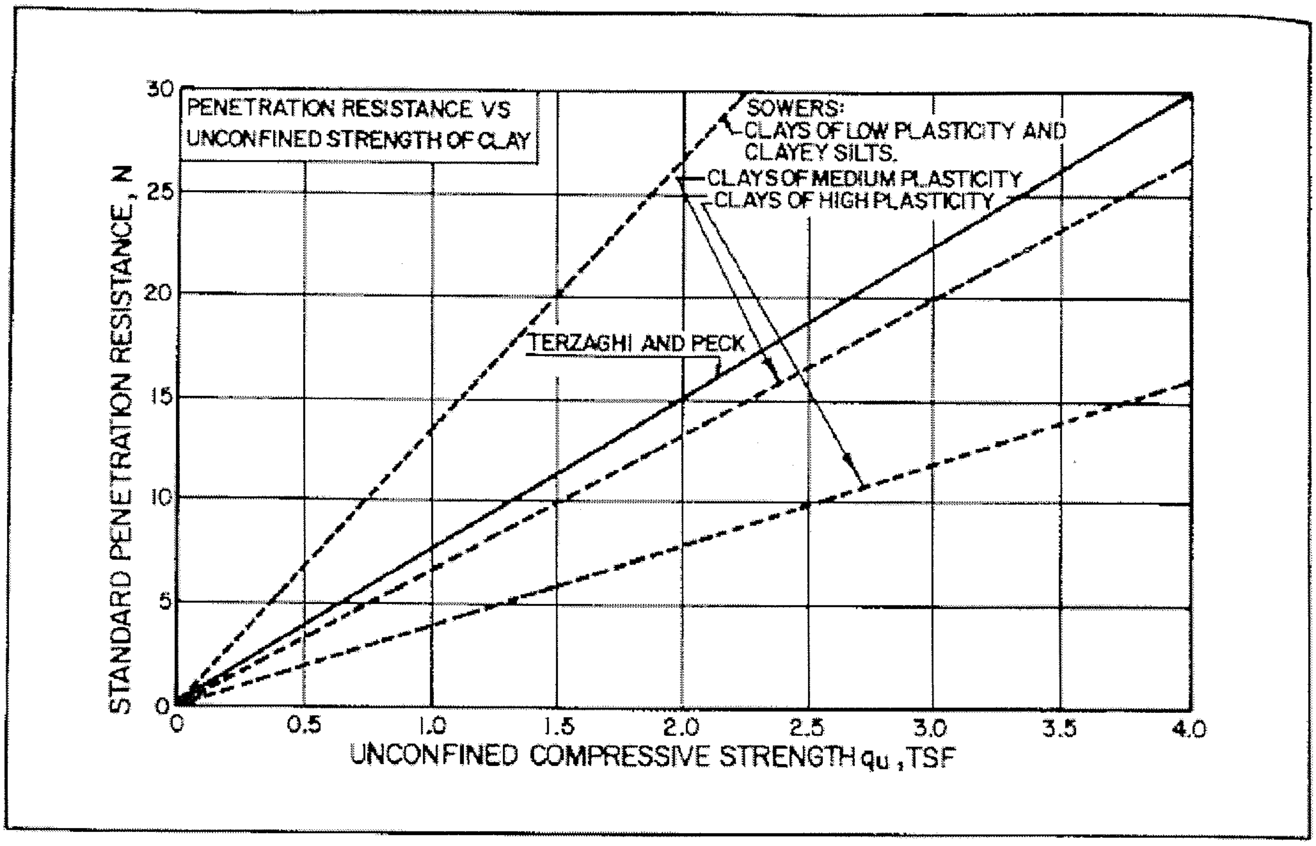


FIGURE 4
Correlations of Standard Penetration Resistance

Fig.1 Relationship between blow count N and unconfined compressive strength q_u cohesive soils. (NAVFAC DM 7.1, 1982) → $c_u = q_u/2$

1 TSF = 107.3kPa

b) Selection of Soil Design Parameters - c_u

Borehole GL3

Layer Da:

(GC), (CL)	From chart NAVFAC	c_u =60kPa
N_{SPT}=8	From laboratory test results	c_u =46kPa, 122kPa (not considered)
	Selected values	c_u =60kPa, $\varphi_u = 5^o$

Layer I:

(SM)	From chart NAVFAC	c_u = 40kPa
N_{SPT}=6	From laboratory test results	c_u = 85kPa(not considered)
	Selected values	c_u = 40kPa, $\varphi_u = 5^o$

Borehole GL1, GL2, GL4

Layer Db:

(GC)	From chart NAVFAC	c_u = 70kPa
N_{SPT}=25	From laboratory test results	c_u =64kPa, 20kPa, 54kPa(not considered)
	Selected values	c_u = 70kPa, $\varphi_u = 5^o$

Layer II:

(GC)	From chart NAVFAC	c_u = 75kPa
N_{SPT}=20	From laboratory test results	c_u = 20kPa (not considered)
	Selected values	c_u = 70kPa, $\varphi_u = 10^o$

Layer IIa:

(GC)	From chart NAVFAC	c_u = 150 kPa
N_{SPT}=40	From laboratory test results	c_u = 93kPa, 364kPa(not considered)
	Selected values	c_u = 150kPa, $\varphi_u = 5^o$

A1.3 Oedometer Modulus E_{oed}

a) General Relationships

Table 4

TABLE 5-6

Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — — ‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ *$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$ $E_{s(OCR)} \approx E_{s,nc}\sqrt{OCR}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$ If $q_c < 2500$ kPa use $\quad$ §$E_s' = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E_s' = 4q_c + 5000$ where $E_s' = \text{constrained modulus} = \frac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \frac{1}{m_v}$	$E_s = (1 \text{ to } 2)q_c$
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

(Bowles, 1996)

b) Selection of Soil Design Parameters - E_{oed}

Borehole GL3

Layer Da:

(GC), (CL)	From empirical equations	E_{oed}= 320*(N+15)=7360kPa
N_{SPT}=8	From laboratory test results	E_{oed}=-
	Selected values	E_{oed}=7000kPa

Layer I:

(SM)	From empirical equations	E_{oed}= 250*(N+15)=5250kPa
N_{SPT}=6	From laboratory test results	E_{oed}=6100kPa
	Selected values	E_{oed}=6000kPa

Borehole GL1, GL2, GL4

Layer Db:

(GC)	From empirical equations	E_{oed}= 320*(N+15)=11200kPa
N_{SPT}=25	From laboratory test results	E_{oed}=4750kPa
	Selected values	E_{oed}=10000kPa

Layer II:

(GC)	From empirical equations	E_{oed}= 320*(N+15)=11200kPa
N_{SPT}=20	From laboratory test results	E_{oed}=-
	Selected values	E_{oed}=12000kPa

Layer IIa:

(GC)	From empirical equations	E_{oed}= 320*(N+15)=17600kPa
N_{SPT}=40	From laboratory test results	E_{oed}=-
	Selected values	E_{oed}=18000kPa

A2. DESIGN PARAMETERS FOR ROCK FORMATIONS

A2.1 General Relationships

a) Generalised Hoek & Brown criterion (2002)- GSI

Sheets A-11 to A-15

Summary of Selected Values

- **Clay Marl Layer III**

c'=50kPa
φ'=28°
γ=22kN/m^3
E_m=400MPa

- **Marly Limestone Layer IV**

c'=100kPa
φ'=30°
γ=22.5kN/m^3
E_m=1200 MPa

GENERALISED HOEK-BROWN CRITERION AND EQUIVALENT MOHR-COULOMB

MALTA - DELIMARA OFFSHORE

Formation: **Marl Layer III**

Facts

σ_{ci}= 5 MPa	Weathering degree= W3	GSI= 20
γ= 0,0210 kN/m^3	m_i= 6	GSI=RMR$_{89}$-5 για RMR>23 με Υπεδαφικό Νερό=15 και Προσανανατολισμό Ασυνεχειών=0
Depth 7 m	JRC= 3 Επιπεδη Τραχεία	Angle of Discon= 45°
D= 0,2	degree of disruption (due to surfacial weathering-excvation)	

Results

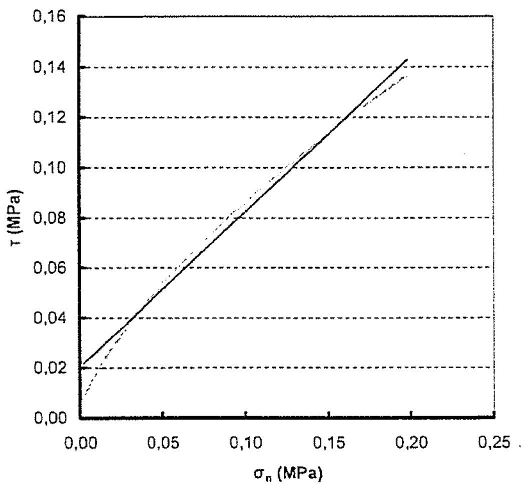
m_b= 0,251	s= 0,0001	α= 0,544
$\sigma_{tensile\ m}$= -0,001 MPa	σ_n= 0,147 MPa	ρ= 2140,67 kg/m^3
σ_c= 0,028 MPa	σ_{cm}= 0,259 MPa	σ_{ci}/σ_{cm}= 19,30

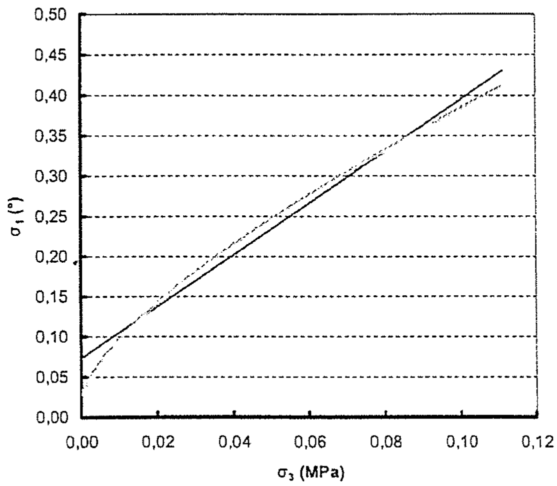
Generalised (foundations)

σ'_{3max}= 1,2500 MPa	σ_{3n}= 0,25	c'= 0,098 MPa φ'= 15,62°

Slopes

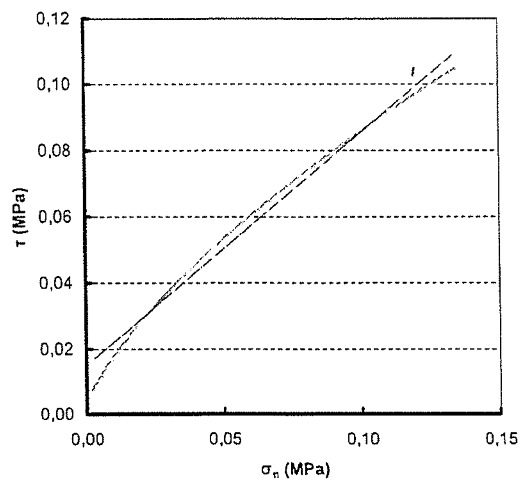
σ'_{3max}= 0,1114 MPa	σ_{3n}= 0,022	c'= 0,020 MPa φ'= 31,66°

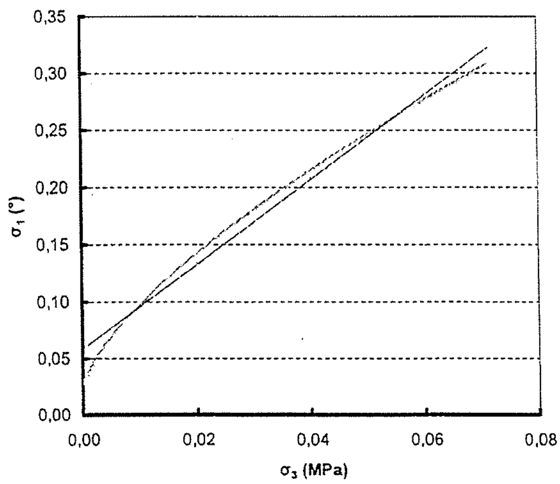




Tunnels

σ'_{3max}= 0,0715 MPa	σ_{3n}= 0,014	c'= 0,015 MPa φ'= 35,04°





Modulus of compressibillity

Jacky's Formula		*Sheorey (1994)*		
ν= 0,32		ν= 0,38		
k_0= 0,48		k_0= 0,61		E_m= 357,87 MPa
G= 135 MPa		G= 130 MPa		
E_{oed}= 516 MPa		E_{oed}= 666 MPa		

Summary of Results

Project:	MALTA - DELIMARA OFFSHORE
Formation:	**Marl Layer III**

Unconfined compressive strength σ_{ci}=	5,000	MPa
Strength of intact rock σ_c=	0,028	MPa
Compressive strength σ_{cm}=	0,259	MPa
Unit weight γ=	0,0210	MN/m^3
Density ρ=	2140,67	kg/m^3
Factor m_i=	6	
Degree of weathering=	W3	
GSI=	20	
JRC=	3	
Angle of discontin=	45°	
Depth of computation=	7	m
Degree of disruption D=	0,2	

Indexes and Moduli

Modulus of elasticity E_m=	358 MPa		Recommended value	**E_m=400kPa**
	Jacky's Formula	*Sheorey (1994)*		
Shear Modulus G=	135 MPa	130 MPa		
Stress-strain modulus E_{oed}=	516 MPa	666 MPa		
Poisson's ratio v=	0,32	0,38		
k_0=	0,48	0,61		

Mohr-Coulomb parameters

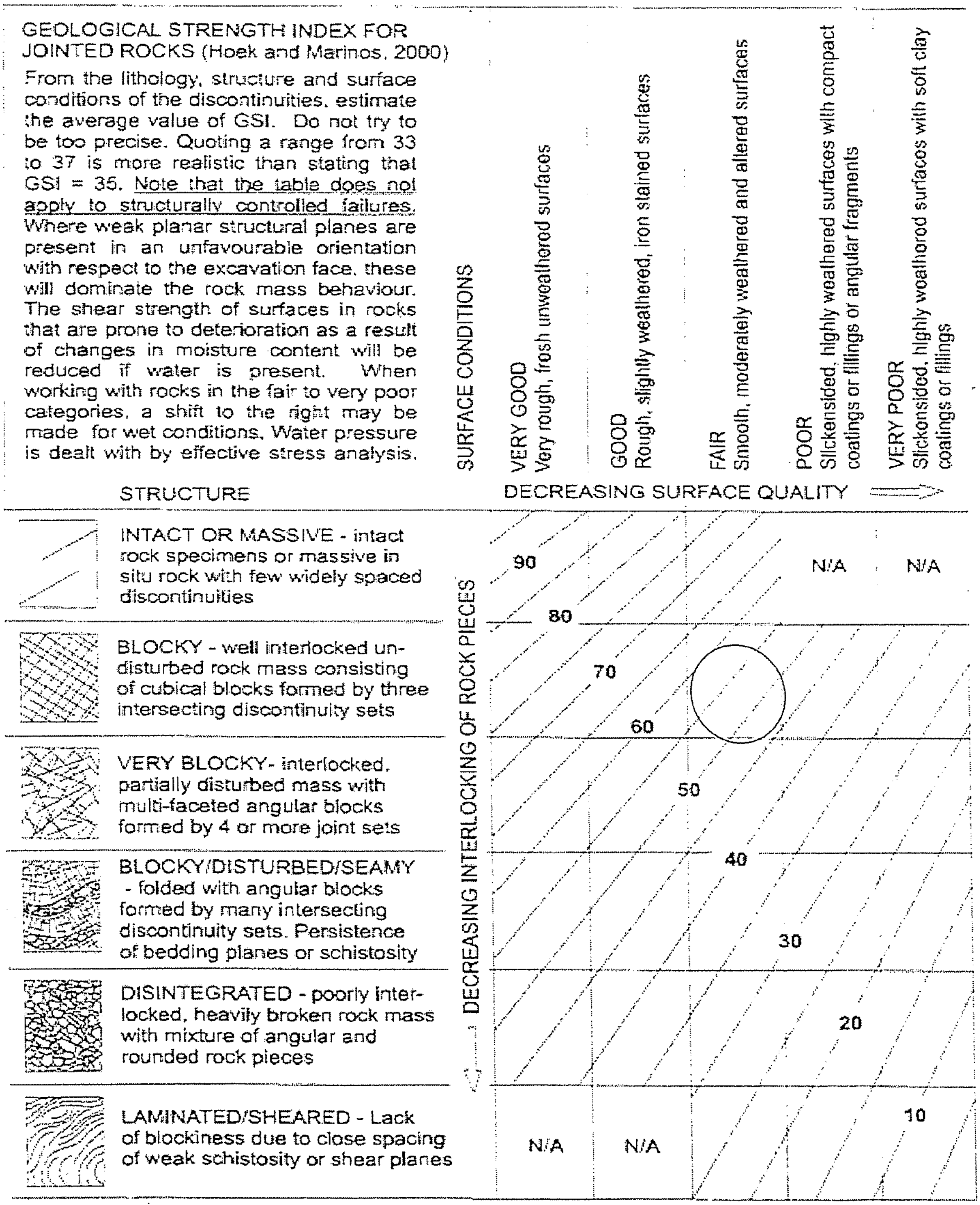
	Foundations	Slopes	Tunnels	Recommended value
Cohesion c'=	98 kPa	20 kPa	15 kPa	**c'=50kPa**
Angle of internal friction φ'=	16°	32°	35°	**φ'=28°**

Mohr-Coulomb parameters for Discontinuities

	Depth of computation		
	7,0 m	**3,0 m**	**0 m**
Cohesion c'=	3 kPa	1 kPa	0,13 kPa
Angle of internal friction φ'=	18°	19°	22°

Geological Strength Index

Project : Malta - Delimara Offshore		**Rock Description** : Marly Limestone **Layer IV**
Geologist:	**Date :**	**Geological Strength Index (GSI):** 45-60 Chosen value 55



GENERALISED HOEK-BROWN CRITERION AND EQUIVALENT MOHR-COULOMB

MALTA - DELIMARA OFFSHORE

Formation: **Marly Limestone Layer IV**

Facts

	σ_{ci}= 10 MPa	**Weathering degree=** W2	**GSI=** 50
	γ= 0,0240 kN/m³	m_i= 8	GSI=RMR₈₉-5 για RMR>23 με Υπεδαφικό Νερό=15 και Προσανατολισμό Ασυνεχειών=0
	Depth 44 m	**JRC=** 3 Επιπεδη Τραχεία	**Angle of Discon=** 45°
	D= 0,6	degree of disruption (due to surfacial weathering-excvation)	

Results

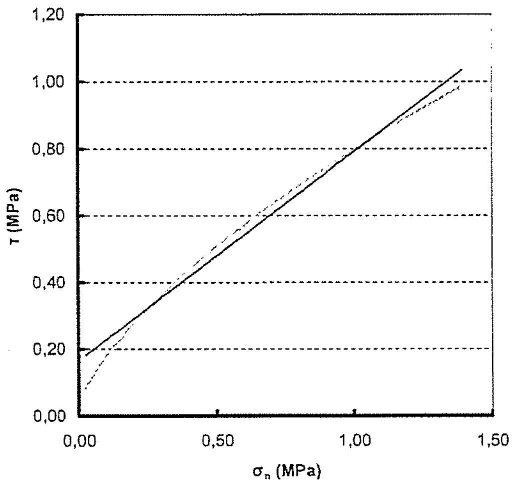
m_b= 0,624	s= 0,0010	a= 0,506
$\sigma_{tensile\ m}$= -0,015 MPa	σ_n= 1,056 MPa	ρ= 2446,48 kg/m³
σ_c= 0,298 MPa	σ_{cm}= 1,046 MPa	σ_{ci}/σ_{cm}= 9,56

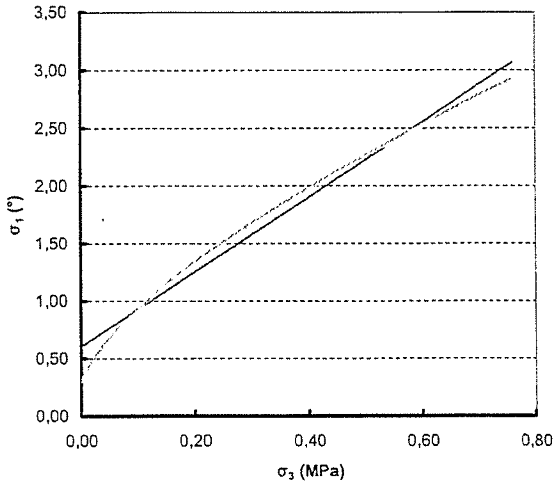
Generalised (foundations)

σ'_{3max}= 2,5000 MPa	σ_{3n}= 0,25	c'= 0,349 MPa
		φ'= 22,66°

Slopes

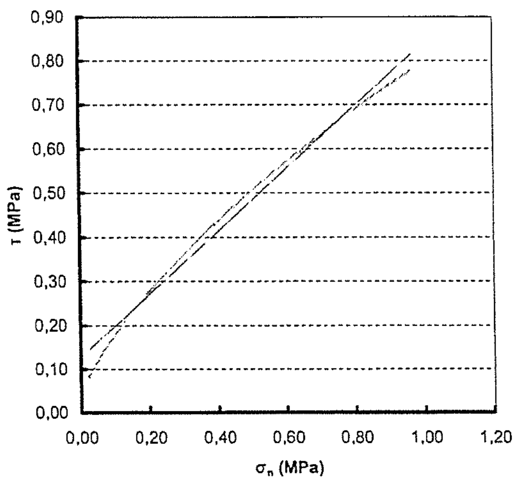
σ'_{3max}= 0,7597 MPa	σ_{3n}= 0,076	c'= 0,167 MPa
		φ'= 31,91°

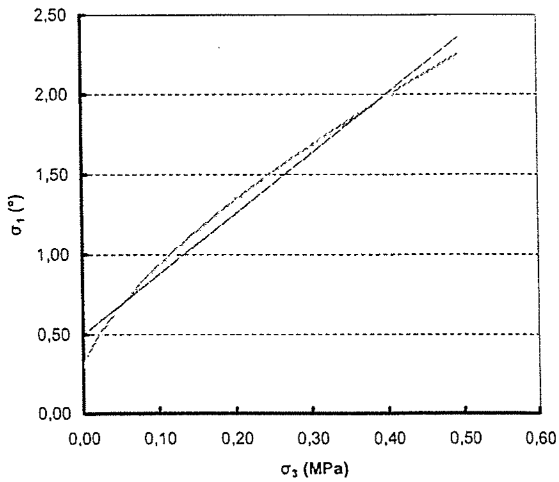




Tunnels

σ'_{3max}= 0,4960 MPa	σ_{3n}= 0,050	c'= 0,129 MPa
		φ'= 35,40°





Modulus of compressibillity

Jacky's Formula	*Sheorey (1994)*	
ν= 0,32	ν= 0,38	
k_0= 0,47	k_0= 0,62	E_m= 2213,59 MPa
G= 838 MPa	G= 801 MPa	
E_{oed}= 3172 MPa	E_{oed}= 4190 MPa	

Summary of Results

Project: MALTA - DELIMARA OFFSHORE

Formation: **Marly Limestone Layer IV**

Unconfined compressive strength σ_{ci}=	10,000	MPa
Strength of intact rock σ_c=	0,298	MPa
Compressive strength σ_{cm}=	1,046	MPa
Unit weight γ=	0,0240	MN/m^3
Density ρ=	2446,48	kg/m^3
Factor m_i=	8	
Degree of weathering=	W2	
GSI=	50	
JRC=	3	
Angle of discontin=	45°	
Depth of computation=	44	m
Degree of disruption D=	0,6	

Indexes and Moduli

Modulus of elasticity E_m=	2214 MPa		Recommended value **E_m =1200**
	Jacky's Formula	*Sheorey (1994)*	
Shear Modulus G=	838 MPa	801 MPa	
Stress-strain modulus E_{oed}=	3172 MPa	4190 MPa	
Poisson's ratio v=	0,32	0,38	
k_0=	0,47	0,62	

Mohr-Coulomb parameters

	Foundations	Slopes	Tunnels	
Cohesion c'=	349 kPa	167 kPa	129 kPa	Recommended value **c'=100kPa**
Angle of internal friction φ'=	23°	32°	35°	**φ'=30°**

Mohr-Coulomb parameters for Discontinuities

	Depth of computation		
	44,0 m	**17,8 m**	**0 m**
Cohesion c'=	21 kPa	8 kPa	0,17 kPa
Angle of internal friction φ'=	23°	24°	30°

b) Relationship between RQD and modulus reduction ratio

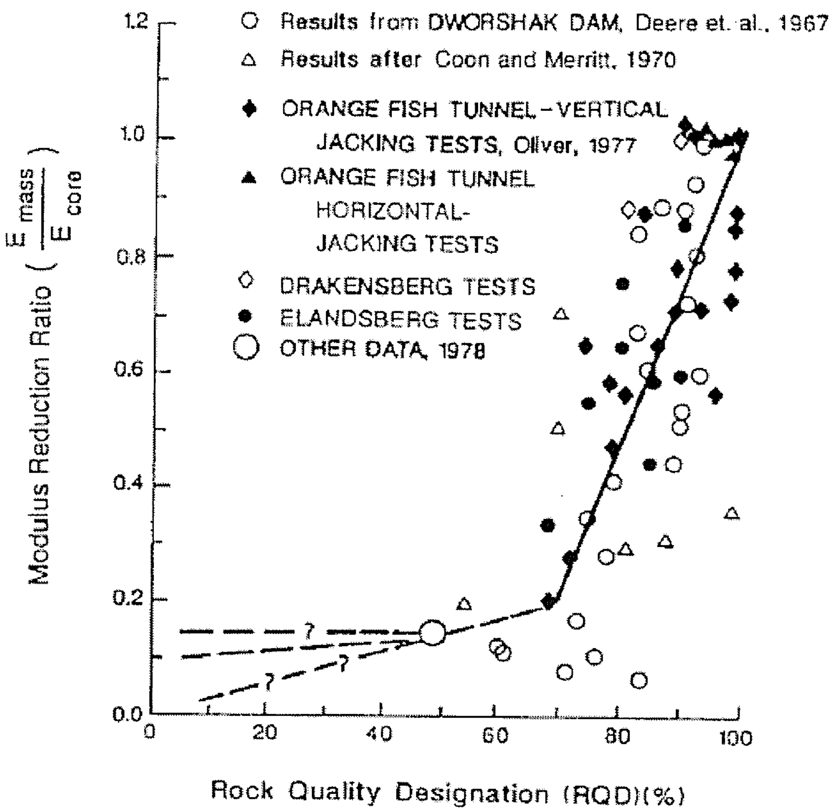


Fig. 3.12. Modulus Reduction Ratio as a Function of RQD (From Bieniawski, 1984)

Fig.1 Relationship between Rock Quality Designation and Modulus Reduction Ratio. (Bieniawski, 1984).

c) Summary of Laboratory Test Results – Selected σ_c & E_m Values

BOREHOLES	Formations	RQD (%)	Laboratory Values Is_{50} (MPa)	Laboratory Values σ_c (MPa)	Mean Values σ_c (MPa)	Selected Values σ_c (MPa)	Laboratory Values E_{core} (MPa)	Mean Values E_{core} (MPa)	E_m/E_{core}	Mean Values E_m (MPa)	Selected Values E_m (MPa)
GL1	Clay Marl	-	-	-	-	-	-	-	-	-	-
	Marly Limestone	85	1.0-2.8	10-15	12	12	3800-9130	6350	0.6	3810	1200
GL2	Clay Marl	-	-	-	-	-	-	-	-	-	-
	Marly Limestone	95	0.8-1.9	8-13	11	11	2920-5640	4250	0.8	3400	1200
GL3	Clay Marl	50	-	5	5	5	670	670	0.3	201	400
	Marly Limestone	60	-	11-12	12	12	6210-6970	6600	0.35	2310	1200
GL4	Clay Marl	-	-	-	-	-	-	-	-	-	-
	Marly Limestone	98	1.1-2.1	8-14	12	12	1440-6040	3680	0.95	3496	1200

E_m=Rock mass elasticity modulus
E_{core}=Uniaxial test elasticity modulus

A3.2 Selected Rock Parameters

a) Layer III, Clay Marl

Boreholes GL3, GL2

RQD=	50%	σ_{ci} =	5MPa
c′ =	50kPa	γ =	22kN/m^3
Φ′ =	28°	E_m =	400MPa

b) Layer IV, Marly Limestone

Borehole GL3

RQD=	60%	σ_{ci} =	12MPa
c′ =	100kPa	γ =	22.5kN/m^3
Φ′ =	30°	E_m =	1000MPa

Boreholes GL1, GL2, L4

RQD=	80%	σ_{ci} =	12MPa
c′ =	100kPa	γ =	22.5kN/m^3
φ′ =	30°	E_m =	1200MPa

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