CONFIDENTIAL

ROTTERDAM SITE DEVELOPMENT – FEASIBILITY PHASE
NESTE

JOB DESIGN SPECIFICATION FOR GEOTECHNICAL RECOMMENDATIONS

FEASIBILITY PHASE

RDCG REFINERY MNA AREA

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1. FOREWORD

The present report deals with the geotechnical issues related to the Feasibility Phase of the proposed Expansion Project of the Neste Plant located in the Maasvlakte industrial site in harbour area of Rotterdam, Netherlands (refer to Figure 1.1).

In particular, at this stage TechnipFMC is responsible for three different areas:

- RDCG REFINERY MNA AREA
- CORRIDOR AREA
- REFINERY RDCG PLANT AREA

Figure 1.2 shows the areas listed above, which are within the scope of work of TechinpFCM.

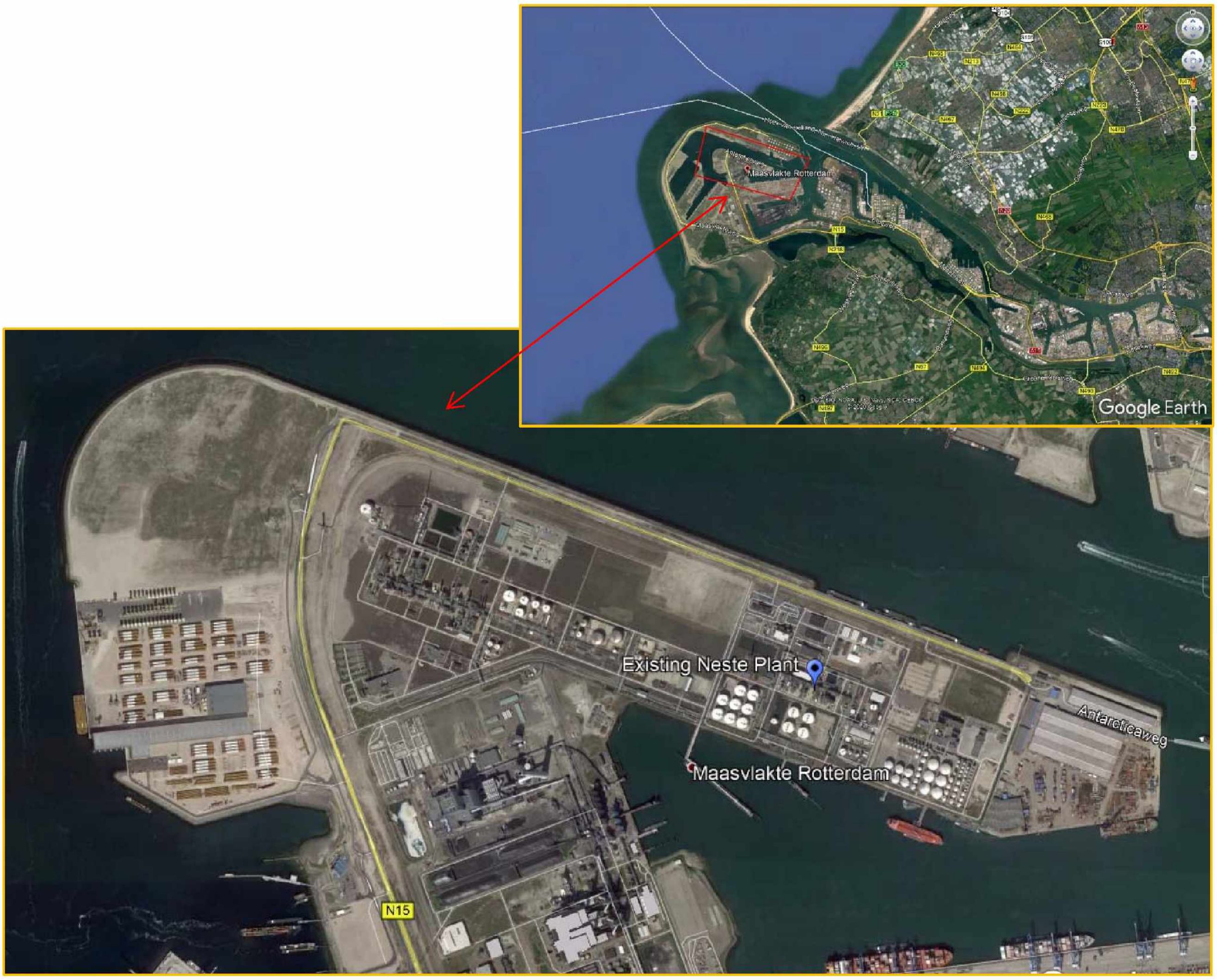

Figure 1.1: Maasvlakte industrial site (from Google earth).

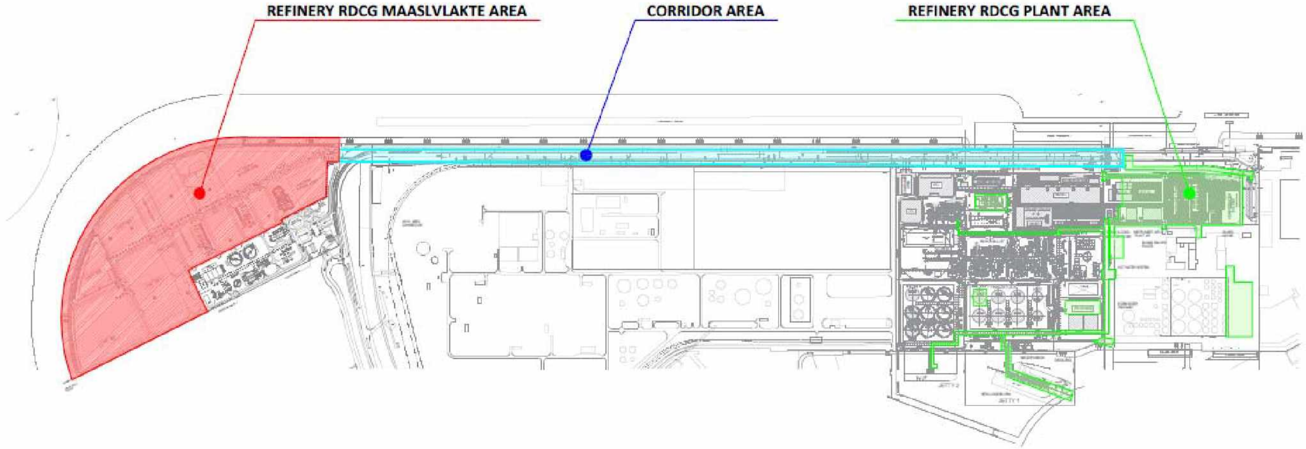

Figure 1.2: Locations of Areas scope of TechnipFCM work

All evaluation and recommendations reported in the present document are based on the results of ground investigation campaign carried out by RSK Netherlands in January-March 2020. The investigation was carried out in the four different areas shown in Figure 1.3.

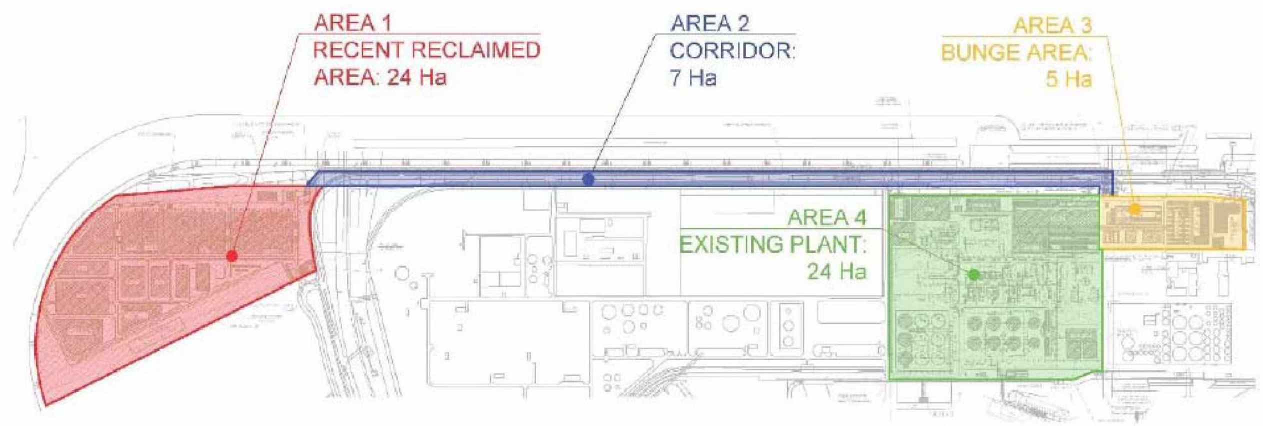

Figure 1.3: Locations of Areas scope of RSK campaign 2020

Concerning Refinery RDCG MNA Area the results of ground investigations carried out by Port of Rotterdam Authority for General Research (Feasibility), have been taken into account.

In particular, the present report is focuses on **Refinery RDCG MNA Area,** placed on the recent reclaimed area. Separate reports have been developed for the other areas.

Currently the **Refinery RDCG MNA Area** development involves the construction of the following main units:

- Unit 011 – Hydotreating unit (HTU)
- Unit 057 – Hot Oil
- Unit 021 – Process unit NexBTL
- Unit 012 – Pretreatment Unit (PTU)
- Unit 042 – Intermediate Tanks
- Unit 067 – Flare
- Unit 053 – Utilities
- Unit 070 – Building (operator Build. & Lab.)
- Unit 076 – Technical Buildings
- Unit 081 – Interconnection pipe racks
- Unit 086 – Fire Water Tanks

These currently planned units are highlighted in the plot plan reported in ATTACHMENT 1.

This document is a preliminary geotechnical interpretative and recommendation report for the design of the main works. The following topics are examined:

- geological setting (brief description);
- summary and interpretation of the available ground investigation;
- description of the ground and groundwater condition at the site;
- definition of the geotechnical ground model and relevant parameters;
- recommendations on the type of foundations most suitable for the site conditions;
- criteria and formulae for the design of shallow foundations, with design charts for type foundations;
- criteria and formulae for the design of deep foundations; for each pile type and soil profile considered, pile bearing capacity of single pile, including estimate of negative skin friction, pile driveability analysis (where appropriate), resistance to lateral loads;
- recommendations for design of foundations of main vibrating machines (pumps, compressors, turbine etc.), including design value of dynamic shear modulus, Poisson ratio and damping, and associated range of variability to be considered in design
- recommendations for excavations and stability of temporary slopes;

It falls outside the scope of this document to:

- determine the loads and the load combinations applied to specific foundations;
- verify that specific foundations satisfy all applicable geotechnical limit states;
- define the pile group geometry;
- determine the load distribution between the different piles within each pile group;
- verify the structural design of foundation slab, of the piles and pile cap in relation to the applied loads.

These activities shall be the subject of specific verification reports relevant to the foundations of each structure to be constructed for the Project.

Moreover, it is outside the scope of this report to consider environmental aspects of the Project.

2. REFERENCES

2.1. Project documents

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[2] 082755C-000-JSD-1422-001 – Feasibility Phase – Job Design Specification for Soil Improvement

[3] 082755C-000-CN1410-01 - Calculation Report for Platform Settlements Evaluation for Recent Reclaimed Area (Maasvlakte2)

2.2. Reference documents

[4] 080871C 000 RT 1410 001- Soil investigation Final Factual Report - Maasvlakte Industrial Site in Rotterdam, the Netherlands - RSK Netherlands, 20 June 2020

[5] R1902305-02 Uitgevoerd grondonderzoek en laboratoriumonderzoek - I.001330 Neste Biofuels te ROTTERDAM MAASVLAKTE

2.3. Reference standards

[6] EN 1990. Eurocode: Basis of Structural Design

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3. SITE LOCATION AND DESCRIPTION

The site is located in the Maasvlakte area of Port of Rotterdam. The area was originally located offshore and it was part of the North Sea, but in 1960 – 1970's it was reclaimed by the Dutch government for industrial activities. For the reclamation, sand originating from the Oostvoorne Lake and nearby harbors (Hartelhaven, Mississippihaven, Europahaven and 8th Petroleumhaven) was used.

Successively, the Port of Rotterdam Authority developed the Maasvlakte2 project to create a new port and supporting infrastructure on reclaimed land adjoining the existing Maasvlakte area. Approximately 2000 hectares will be reclaimed, behind a 4 km dike. Construction of this land reclamation project, programmed in 2008, started in 2010 and was completed in 2013.

Refinery RDCG MNA Area, object of the present document, is located on Maasvlakte2.

The original water depth elevation before the above mentioned recent reclamation was between 0 NAP (coastline) to about -18 m NAP (see Figure 3.1). Because of the reclamation and landfill, surface was raised to about +4.5 ÷ +5 m NAP.

It has to be noted that an historic sea barrier/breakwater was identified to be present in the past within the area in object. According to the competent authority (Port of Rotterdam) the barrier was disassembled and removed during the reclamation of the Maasvlakte2. It is anticipated, that from the CPTU results a number refusals were identified in the approximate location of this sea barrier. This might indicate that some residual boulders/stones/sections were not completely removed during the disassembly.

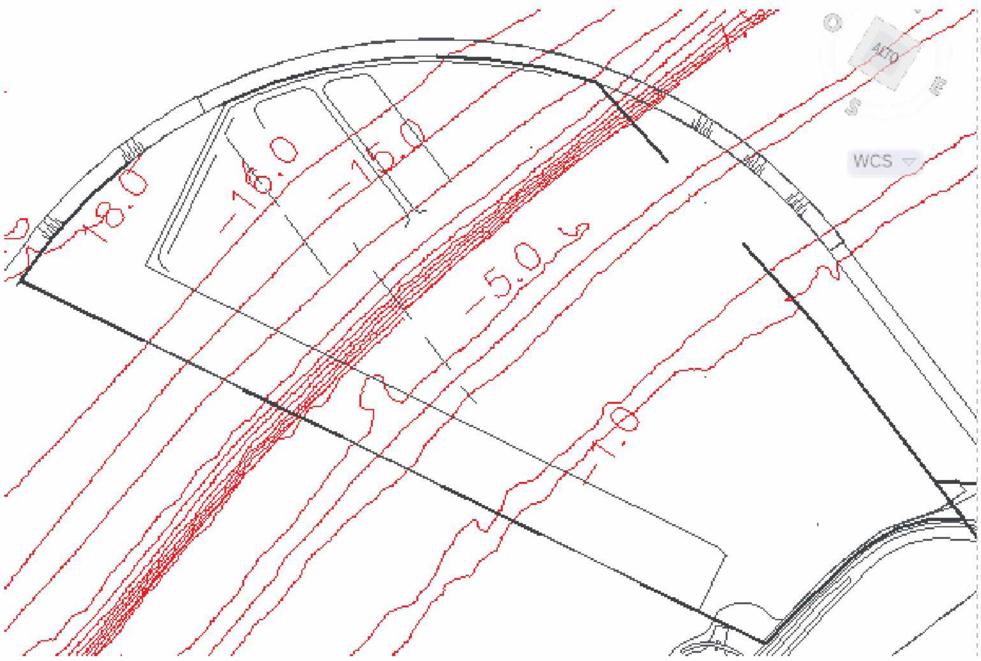


Figure 3.1: Bathymetric survey 2008 (based on information reported in Annex 14 of Ref.Doc.[4]).

Figure 3.2: Google Earth 2011 and bathymetric survey 2008

4. GROUND INVESTIGATION

4.1. Soil investigation RSK Netherlands 2020 for Neste Expansion Project

FEED-level ground investigation has been performed by RSK Netherlands in January-March 2020 in support of the design of the planned Expansion Project. The investigation included geotechnical investigation, a geophysical survey for the mapping of utility cables or other obstacles and a preliminary screening of potential contaminants in areas.

As detailed in the final factual report (Ref.Doc.[4]), for the recent reclaimed area the geotechnical investigation comprised:

<u>AREA 1 – Recent Reclaimed Area</u>

- N°135 cone penetration tests with pore pressure measure (CPTU tests) performed up to a depth of 35 m below surface level, of which N°110 in the Refinery RDCG MNA Area;
- N°9 boreholes of which N°7 in the Refinery RDCG MNA Area. N°7 of them drilled to a depth of about from about 30 m below ground level, N°2 drilled to a depth of about 6 m below ground level. Undisturbed samples have been collected during boring;
- N°4 boreholes were equipped each with one standpipe piezometer for the groundwater level measurement, of which N°2 in the Refinery RDCG MNA Area. Water samples were taken for chemical analyses;
- N°3 in situ electrical resistivity tests;
- N°3 in situ thermal conductivity tests.
- Geotechnical laboratory tests on selected remoulded and undisturbed samples (currently on going)

4.2. Port of Rotterdam Authority campaign for General Research (Feasibility)

The following geotechnical soil investigation has been performed by Port of Rotterdam Authority for General Research (Feasibility) of Port development (see Ref.Doc.[5]).

- N° 64 cone penetration tests with pore pressure measure (CPTU tests) and dissipation tests performed up to a depth of 45 meters below surface level, of which N°57 in the Refinery RDCG MNA Area;
- N° 11 boreholes, all drilled to a depth of about from about 30 m below ground level, of which N°9 in the Refinery RDCG MNA Area. Undisturbed samples have been collected during boring;
- N° 4 water standpipes in 4 boreholes for groundwater level measurements

The location of all boreholes, CPT tests, trial pits and associated plate load tests performed in 2020 and in previous investigation is shown in ATTACHMENT 1.

5. GROUND AND GROUNDWATER CONDITIONS

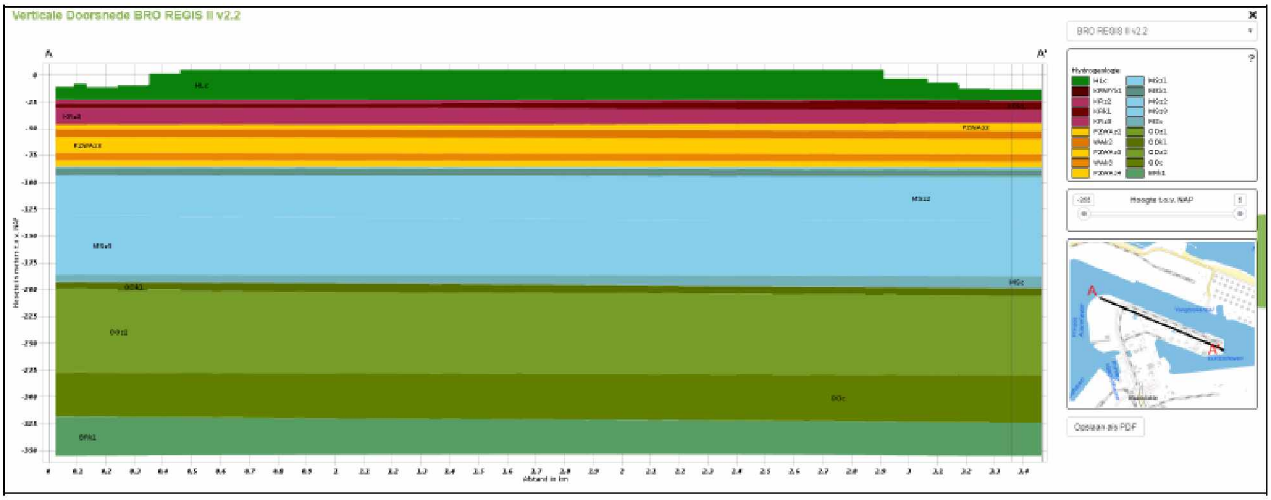
5.1. Geology

Large parts of the Netherlands today are below sea level and have in the past been covered by the sea or flooded at regular intervals. The modern Netherlands formed as a result of the interplay of the four main rivers (Rhine, Meuse, Schelde and IJssel) and the influence of the North Sea. The Netherlands is mostly composed of deltaic, coastal and eolian derived sediments during the Pleistocene glacial and interglacial periods.

Nearly all of the west Netherlands is composed of the Rhine-Meuse river estuary, but human intervention greatly modified the natural processes at work. Most of the western Netherlands is below sea level due to the human process of turning standing bodies of water into usable land, “polder”. In eastern Holland, remains are found of the last ice age, which took place approximately ten thousand years ago. As the continental ice sheet moved in from the north, it pushed moraine forward. The ice sheet halted as it covered the eastern half of the Netherlands. After the ice age ended, the moraine remained in the form a long hill-line.

In particular, in the area of interest for the Expansion Project of the Neste Plant, the indicative geological model is shown in the figure below. According to it the following formations are found:

- a superficial layer of reclaimed Holocene materials characterized by alterations between sandy clay with fine/middle sand and loam from G.L up to about -22 m NAP;
- Kreftenheye formation (NAP -22.3 to -46.0 m), consisting of middle to coarse sand with a sandy clay layer (NAP -25.0 to -30.0 m);
- an alteration of material from Peize formation (middle to coarse sand) and Waalre formation (sandy clay) from NAP -46.0 to -85.0 m;
- below the Maassluis formation (NAP -85.0 to -192.7 m) is found, mainly composed of a middle to coarse sandy layer and contains two sandy clay layers (NAP -85.0 to -93.4 m and NAP -185.7 to -192.7 m)
- Oosterhout formation (NAP -192.7 to -318.1 m) consisting in two sandy clay units (NAP -192.9 to -199.09 m and NAP -276.9 to -318.12 m) with in between a fine to middle sand layer with seashells.
- Breda formation consisting in sandy clay (NAP -318.1 to -354.3 m).



5.2. Soil Conditions

The general soil profile at the site consists mainly of:

- **Recent sand** unit consists of Holocene deposits. It is encountered up to a depth from about -21 to -22 m NAP, locally up to -24 m NAP. Fill present at the site derives from material dredged in the Holocene sand deposits and the two have been considered as a single unit for the porpoises of the geotechnical characterization. Recent sands are generally sand and silty sand, with relative density $D_r \geq 60\%$. Locally looser levels are found above +1 m NAP.
- **Clay to clayey silt** levels/lenses of variable thickness within the above deposits. These levels/lenses are found frequently at about -8 m ÷ -14 m NAP, with a maximum registered thickness of about 3 ÷ 3.5 m. A thicker layer, of about 1.5 m to 3 m thick, is often encountered beneath -20.5 ÷ -23 m NAP. Possible random presence of localized thin levels of fine-grained material can be found even in the first meters from the ground level above +0.5 m NAP. Minor lenses of peat have been singled out during boring at various depths.
- **Mixed soil** levels**,** consisting in clays to clayey silt mixed with silty sands/sandy silts, are recovered randomly , but generally below -1.5/- 2 m NAP.
- **Pleistocene sands,** medium to coarse, densely packed, placed generally beneath -22 ÷ -24 m NAP, but locally the top of this layer deepens up to -26 ÷ -29 m NAP.

Figure 5.1 and Figure 5.2 show the contours of the elevation of the top of clay layer that overlies the Pleistocene sands and the top of the Pleistocene sands, respectively.

The subsoil condition across the investigated recent reclaimed Area 1 is highlighted by the geotechnical sections encloses in ATTACHMENT 2**.** Section position is shown on the site plan reported in ATTACHMENT 1.

Figure 5.1: Contours of the top of the Clay Layer that overlies the Pleistocene sands.

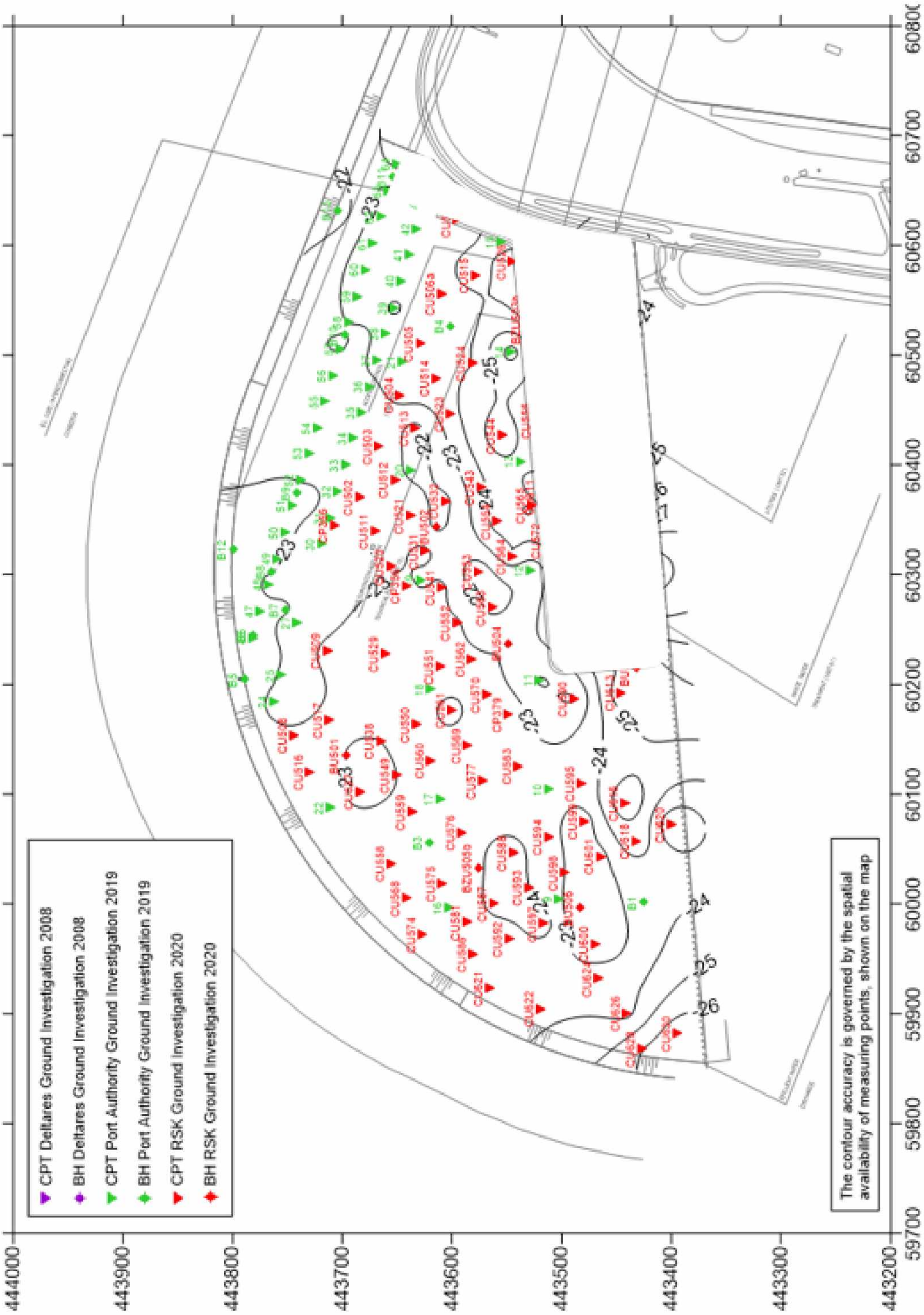


Figure 5.2: Contours of the Top of the Pleistocene sands.

5.2.1. Design Soil Profiles

In accordance with the available ground investigation data, representative soil profiles at the area of interest assumed in the current design phase are summarizes in Table 5.1 (see key plan in ATTACHMENT 1).

Table 5.1:Design soil profiles

UNIT	Reference CPTs/boreholes	Loose silty sands/sandy silts (m NAP)	Mixed soil[1] (m NAP)	Clay to clayey silt level (m NAP)	Top of Pleistocene sand (m NAP)
067	CU588, CU593 CU597, CU598 CU600, CU601 CU618, CU622 CU623, CU624 CU625, CU626 CU627, CU628 CU629, CU630 1, 2, 8, 9, B1, BU506	-	-21 ÷ -24	-8.5 ÷ -11.5 -12.5 ÷ -13.5	-23/-25
062	CU558, CU568 CU574, CU575 CU581, CU586 CU587, CU592 16, BZU505a,b	-	-16 ÷ -17.5	-8 ÷ -11 -13 ÷ -14 -21.5 ÷ -24	-24/-30
057	CU536, CU537 CU548, CU559 22, B3	+3 ÷ +1	-	-6 ÷ -9.5 -12.5 ÷ -13.5 -15 ÷ -16 -21 ÷ -23	-23/-27
011	CU508, CU516 CU538, CU549 BU501	-	-9 ÷ -10.5 -12.5 ÷ -15	-21 ÷ -23	-23/-25
042	CU509, CU510 CU518, CU519 CU527, CU529 25, 26, 27 28, 46, 47 B5, B6, B7	-	-	-6.5 ÷ -8 -12 ÷ -13 -21 ÷ -23	-23/-24
012	CU366, CU502 CU511, CU520 CU602 29, 30, 31, 32 49, 50, 51, 52 B8, B9	-	-	-9 ÷ -12.5 -21.5 ÷ -23	-23/-24

UNIT	Reference CPTs/boreholes	Loose silty sands/sandy silts (m NAP)	Mixed soil[1] (m NAP)	Clay to clayey silt level (m NAP)	Top of Pleistocene sand (m NAP)
021	CU569 CU576, CU577 CU578, CU582 CU583, CU584 CU589, CU594 CU595, CU596a,b CU599, CU615 CU616, CU617 CU619, CU620 3, 7, 10	-	-4 ÷ -4.5 -7 ÷ -15	-21.5 ÷ -24	-23/-24
021A	CU561, CU570 11, 18	-	-4 ÷ -6 -7 ÷ -10	-11.5 ÷ -12.5 -21 ÷ -23	-23/-24
021B	CU590, CU613 4 BU507	-	-	-7 ÷ -10 -22.5 ÷ -25.5	-25/-26
076 (SS12)	CU541, CU553 CU563, CU571 12	-	-6.5 ÷ -10.5	-2.5 ÷ -3.5 -11 ÷ -12.5 -21.5 ÷ -23	-23/-24
076 (SS13)	CU522, CU533 CU544 13, 20	+3.5 ÷ +2	-11 ÷ -12	-6.5 ÷ -8 -21 ÷ -22	-22/-25
053	CU531, CU532 CU542, CU543 CU554, CU564 BU502	-	-7 ÷ -9	-2.5 ÷ -3 -9.5 ÷ -12 -23.5 ÷ -24	-23/-25
086	CU504, CU513 CU514, CU523 CU534	+3 ÷ 0	-	-11 ÷ -12 -21 ÷ -22 -23.5 ÷ -24	-23/-25
070	CU505, CU524 CU525, 21 B4	+3 ÷ +0.5	-	-9 ÷ -11 -22 ÷ -23	-23/-25
081	37, 38, 39, 60 61, 62	+4 ÷ +1	-	-10 ÷ -11.5 -21 ÷ -23	-23/-24
	CU356, CU503 CU512, CU521 CU530, CU539 CU550, CU560	-	-11 ÷ -14	-7 ÷ -8 -9.5 ÷ -12 -21 ÷ -23	-23/-24
Laydown (1)	CU517, CU528 23, 24	-	-11.5 ÷ -13 -14 ÷ -16	-6 ÷ -10.5 -21 ÷ -23	-23/-24
Laydown (2)	CU551, CU552 CU562, BU504	-	-1.5 ÷ -2.5 -5 ÷ -12	-21 ÷ -23	-23/-24
Laydown future (1)	33, 34, 35, 36 53, 54, 55, 56 57, 58, 59	-	-	-6 ÷ -6.5 -9 ÷ -12 -21.5 ÷ -23	-23/-24

UNIT	Reference CPTs/boreholes	Loose silty sands/sandy silts (m NAP)	Mixed soil[1] (m NAP)	Clay to clayey silt level (m NAP)	Top of Pleistocene sand (m NAP)
	B10				
Laydown future (2)	CU506, CU506a CU507, CU515 40, 41, 42, 43	+3.5 ÷ +1.5	-	-8.5 ÷ -11 -23 ÷ -24	-23/-24
[1] Clays to clayey silts mixed with silty sands/sandy silts					

It is anticipated that, at the present design stage:

- for shallow foundations one calculation soil profile has been identified for relevant ULS and SLS checks (see Section 16)
- for piles design, bearing capacity evaluations are performed for each available CPT (see Section 17)
- for the tanks the specific profile at their location has been considered in calculations (see Section 18).

5.3. Groundwater conditions

The available groundwater level investigation include:

- standpipes piezometers, to measure the piezometric level in the lower sand and in the top fill layer.
- standpipes piezometers of the environmental investigation, installed to sample water quality and to measure the piezometric level in the top fill layer.

Moreover, the measures of piezometric pressure in the CPTU tests carried out during the geotechnical soil investigation 2020 have been taken into account.

Table 5.2 summarise all the available data in terms of elevation (in m NAP) of measured ground water level. With regard to the measures relevant to the geotechnical investigation the maximum elevation recorded to date in each piezometer has been reported; in case of ground water level in the lower sand this correspond to the maximum spring high tide event.

Figure 5.3 shows the groundwater contours based on the whole available ground investigation data.

According to available information groundwater table in the zones object in the scope of the works of the **Refinery RDCG MNA Area**, is localized at minimum elevation of +0 m NAP (about 4.5÷5 m below present GL) to a maximum elevation of +2 m NAP (about 2.5÷3 m below present G.L.). A representative value of +1.0 m NAP is assumed in the design.

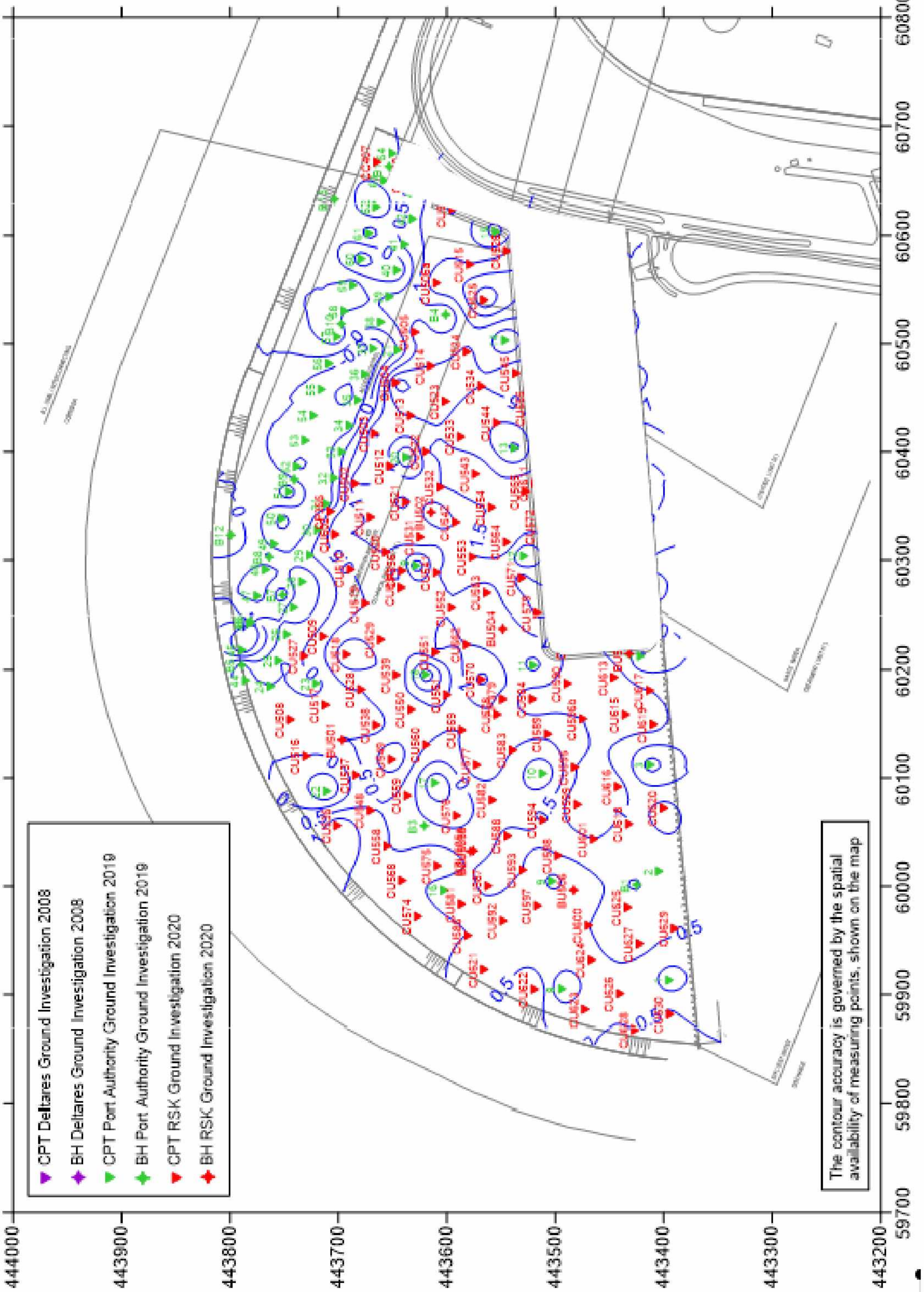


Figure 5.3: Contours of the groundwater level, based on available ground investigation.

Table 5.2: Piezometric data

ID	E	N	GL (m NAP)	TYPE	BH/CPT TOE (m NAP)	GWL (m NAP)
BU501	60135.25	443696.70	5.02	BH	-25.00	0.52
BU502	60344.10	443614.61	5.23	BH	-25.00	0.63
BU504	60237.75	443548.56	4.97	BH	-30.00	0.67
BU506	59996.53	443483.10	5.02	BH	-25.00	0.22
BZU505a	60032.18	443578.07	4.89	BH	-0.51	0.38
BZU505b	60032.87	443575.62	4.89	BH	-25.00	0.19
CC407	60666.36	443664.22	5.14	CPTU	-4.86	0.20
CC408	60654.78	443635.94	5.04	CPTU	-4.96	0.80
CP356	60290.07	443640.98	5.21	CPTU	-25.23	1.00
CP366	60344.90	443706.95	4.70	CPTU	-25.78	0.20
CP379	60172.79	443548.09	4.82	CPTU	-23.16	0.80
CU502	60370.71	443684.12	5.18	CPTU	-24.82	1.50
CU503	60416.92	443666.26	5.08	CPTU	-25.32	1.60
CU504	60463.82	443646.92	5.22	CPTU	-24.22	2.25
CU505	60510.63	443628.87	5.04	CPTU	-24.42	1.25
CU506a	60555.99	443609.23	5.03	CPTU	-24.53	2.00
CU507	60623.89	443594.59	5.05	CPTU	-24.19	1.60
CU508	60153.95	443743.84	4.95	CPTU	-25.05	0.25
CU509	60230.92	443713.57	4.98	CPTU	-29.80	0.25
CU510	60291.42	443688.64	4.81	CPTU	-10.81	0.50
CU511	60339.59	443670.01	5.06	CPTU	-24.44	1.00
CU512	60386.74	443652.04	5.12	CPTU	-25.24	1.25
CU513	60433.56	443633.07	5.18	CPTU	-22.44	1.50
CU514	60479.21	443614.46	5.16	CPTU	-24.76	1.50
CU515	60573.01	443578.37	5.11	CPTU	-24.87	1.50
CU516	60120.35	443729.31	5.00	CPTU	-25.02	0.00
CU517	60167.66	443711.59	5.07	CPTU	-24.83	0.25
CU518	60214.26	443692.31	5.08	CPTU	-15.70	1.50
CU519	60260.17	443674.84	4.94	CPTU	-12.02	0.50
CU520	60307.40	443655.60	5.27	CPTU	-25.29	1.00
CU521	60353.54	443637.63	5.26	CPTU	-24.76	1.60
CU522	60400.21	443618.92	5.28	CPTU	-24.72	1.50
CU523	60446.57	443600.94	5.18	CPTU	-24.40	1.60
CU524	60492.55	443581.28	5.06	CPTU	-24.30	1.60

ID	E	N	GL (m NAP)	TYPE	BH/CPT TOE (m NAP)	GWL (m NAP)
CU525	60540.05	443565.47	5.06	CPTU	-24.92	3.00
CU527	60212.86	443731.34	5.21	CPTU	-17.19	0.00
CU528	60181.66	443678.71	5.10	CPTU	-15.74	0.50
CU529	60228.00	443660.58	4.92	CPTU	-25.46	0.75
CU530	60274.63	443641.87	5.07	CPTU	-14.01	1.00
CU531	60321.37	443624.18	5.15	CPTU	-24.73	1.50
CU532	60367.45	443605.61	5.33	CPTU	-24.67	1.50
CU533	60413.96	443587.04	5.18	CPTU	-15.66	1.30
CU534	60460.50	443567.85	5.03	CPTU	-26.49	1.50
CU536	60056.08	443700.73	4.93	CPTU	-17.09	1.50
CU537	60102.67	443683.43	4.82	CPTU	-27.50	0.25
CU538	60148.95	443664.88	5.01	CPTU	-24.99	0.25
CU539	60195.10	443646.13	5.09	CPTU	-13.93	1.00
CU541	60288.51	443609.24	5.09	CPTU	-24.89	1.00
CU542	60334.90	443591.27	5.12	CPTU	-22.74	1.00
CU543	60379.54	443573.23	5.15	CPTU	-24.55	2.00
CU544	60426.78	443553.81	5.21	CPTU	-26.25	1.00
CU545	60472.51	443535.88	5.04	CPTU	-26.22	2.00
CU548	60070.07	443669.89	4.98	CPTU	-16.22	0.50
CU549	60117.29	443650.57	4.93	CPTU	-25.05	1.50
CU550	60163.32	443632.34	4.96	CPTU	-25.04	1.00
CU551	60216.54	443610.14	4.97	CPTU	-25.45	1.00
CU552	60256.34	443595.33	5.10	CPTU	-24.90	1.00
CU553	60302.83	443575.89	5.06	CPTU	-24.94	1.50
CU554	60348.69	443558.50	5.13	CPTU	-25.31	1.50
CU555	60440.48	443521.70	5.03	CPTU	-24.45	1.50
CU558	60037.26	443654.71	4.93	CPTU	-25.87	0.50
CU559	60083.80	443636.16	4.74	CPTU	-25.74	0.30
CU560	60130.85	443618.74	4.97	CPTU	-25.05	0.50
CU561	60176.79	443599.71	4.96	CPTU	-25.04	0.50
CU562	60223.39	443581.52	4.80	CPTU	-25.20	1.00
CU563	60269.98	443562.92	5.06	CPTU	-25.44	0.75
CU564	60316.69	443544.70	5.15	CPTU	-24.91	2.00
CU568	60005.42	443640.98	5.00	CPTU	-25.50	0.50
CU569	60144.31	443585.56	4.99	CPTU	-25.47	0.50
CU570	60190.72	443567.77	4.75	CPTU	-25.23	1.70

ID	E	N	GL (m NAP)	TYPE	BH/CPT TOE (m NAP)	GWL (m NAP)
CU571	60283.48	443530.51	5.11	CPTU	-24.87	1.70
CU574	59972.28	443626.68	4.92	CPTU	-25.56	1.00
CU575	60018.91	443608.41	4.95	CPTU	-25.03	1.00
CU576	60065.32	443590.53	4.86	CPTU	-25.12	-0.30
CU577	60112.25	443571.99	5.10	CPTU	-25.40	1.00
CU578	60158.66	443553.25	4.98	CPTU	-14.50	1.00
CU581	59983.58	443586.14	4.96	CPTU	-26.02	0.40
CU582	60079.76	443557.68	5.12	CPTU	-13.32	0.30
CU583	60125.84	443539.06	4.87	CPTU	-25.63	0.50
CU584	60172.88	443520.64	4.97	CPTU	-14.19	1.30
CU586	59954.51	443580.22	5.07	CPTU	-25.91	0.50
CU587	60000.34	443561.74	4.93	CPTU	-30.15	0.60
CU588	60046.54	443543.71	4.94	CPTU	-25.08	0.60
CU589	60140.69	443506.65	4.86	CPTU	-16.24	0.60
CU590	60187.07	443488.52	5.02	CPTU	-28.66	1.20
CU592	59968.27	443547.77	4.91	CPTU	-23.99	0.20
CU593	60014.99	443529.71	4.94	CPTU	-25.14	0.50
CU594	60061.29	443511.08	4.97	CPTU	-25.03	0.50
CU595	60109.97	443481.60	4.70	CPTU	-26.78	1.00
CU596b	60154.65	443474.29	4.96	CPTU	-15.10	1.00
CU597	59982.11	443516.11	4.93	CPTU	-25.37	0.50
CU598	60028.39	443497.14	4.97	CPTU	-24.55	0.50
CU599	60075.63	443479.16	4.76	CPTU	-25.54	1.50
CU600	59964.02	443469.12	4.98	CPTU	-24.48	0.50
CU601	60043.71	443464.15	4.96	CPTU	-25.04	1.00
CU602	60323.63	443702.73	4.81	CPTU	-16.29	1.00
CU603	60497.69	443478.10	5.04	CPTU	-24.80	1.50
CU613	60192.65	443445.47	5.16	CPTU	-26.84	1.25
CU615	60158.58	443434.84	5.16	CPTU	-12.54	1.40
CU616	60092.05	443441.99	4.79	CPTU	-25.93	1.20
CU617	60180.80	443412.52	5.21	CPTU	-12.43	1.00
CU618	60057.53	443431.82	4.32	CPTU	-26.26	0.80
CU619	60149.54	443409.18	5.27	CPTU	-11.63	1.25
CU620	60072.33	443399.65	3.81	CPTU	-26.55	1.20
CU621	59923.27	443565.36	5.06	CPTU	-25.16	0.00
CU622	59904.89	443518.93	5.08	CPTU	-24.86	1.00

ID	E	N	GL (m NAP)	TYPE	BH/CPT TOE (m NAP)	GWL (m NAP)
CU623	59886.43	443472.26	5.02	CPTU	-17.22	0.40
CU624	59931.95	443466.15	5.09	CPTU	-24.83	0.40
CU625	59980.60	443432.85	4.96	CPTU	-14.78	0.70
CU626	59900.80	443440.04	5.04	CPTU	-24.20	0.50
CU627	59946.86	443421.94	4.80	CPTU	-15.66	0.70
CU628	59868.21	443426.38	5.06	CPTU	-26.78	0.50
CU629	59961.17	443389.62	4.59	CPTU	-16.15	0.60
CU630	59882.19	443394.03	4.96	CPTU	-27.54	0.60

6. SEISMIC CONDITIONS

The site at the Maasvlakte is not located in a seismic active region and does not include any active faults. Earthquakes in the Netherlands are limited to the seismic active region in Limburg (Faults) and in Groningen (Gasfield extractions). However, the effect of these earthquakes on the site are minimal to none. Consequently, also the risk of liquefaction ranges from negligible to zero.

7. GEOTECHNICAL CHARACTERIZATION

7.1. Foreword

The geotechnical properties of the soils encountered at the site have been estimated mainly on the basis of the CPT tests and measures from laboratory tests.

Concerning laboratory tests, due to the small quantity of available data information coming from boreholes performed on the whole recent reclaimed Area 1 are reported in figures of ATTACHMENT 3. Data for the specific **Refinery RDCG MNA Area** are appropriately highlighted.

7.2. Coarse grained soils

7.2.1. Recent sands and fill

The geotechnical properties of the Recent sands encountered at the site have been estimated mainly on the basis of the CPT tests and then some measures from laboratory tests.

The q_c values are generally greater than 10 MPa, locally value between 5 and 10 MPa are measured in correspondence of layers with an higher fine content. Values up to 20 ÷ 30 MPa, and more, are generally measured in the top fill layer above the +2 NAP and between -14 m NAP and -18 ÷ -20 m NAP.

7.2.1.1. Grain size distribution

According to the results of grain size analyses summarized in the Attach.3 Figure 1 to Attach.3 Figure 4, typical values of the significant parameters are:

- D_{50} generally in the range 0.1 to 0.4 mm with occasional values up to 0.5 mm;
- D_{10} in the range 0.06 to 0.2 mm, except between -6 m and -14 m NAP where it is in the range 0.05 to 0.1 mm;
- uniformity coefficient generally in the range 1.5 to 2.5 with occasional higher values up to 0.3;
- fines content is less than 10% , with some higher values up to 20% and occasional value in the range of 20-50%.

Therefore, the sand at the site can be generally classified as silty occasionally very silty fine to medium, poorly graded to uniform sand

7.2.1.2. Unit Weight

According to the available laboratory tests results the bulk unit weight ranges between 18.5 ÷ 20.5 kN/m^3 with occasional lower values (see Attach.3 Figure 5).

7.2.1.3. Natural water content

The laboratory results are reported in Attach.3 Figure 6.

7.2.1.4. Relative density

Values of relative density can be inferred from CPTs according to the following relationship with the effective pressure p' (see Garizio, 1997; [redacted] et al., 2001):

$$D_r = \frac{1}{C_2} \cdot \ln\left[\frac{q_c}{C_0 \cdot (p')^{C_1}}\right]$$

With the empirical coefficients C_0, C_1 and C_2 reported in Table 7.1, according to [redacted] (1976), Villet & [redacted] (1981), Baldi et al. (1986) and [redacted] et al. (1988):

Table 7.1: Coefficients C_0, C_1 and C_2

sand	D_{50} (mm)	C_0 (-)	C_1 (-)	C_2 (-)
fine	0.20÷0.25	110	0.59	2.95
medium	0.40÷0.55	205	0.51	2.91
coarse	1.00÷2.00	234	0.48	3.30

The values of Dr in the different areas of the site are reported in Attach.3 Figure 15 to Attach.3 Figure 41. The estimated relative density of sands is generally greater than 50 ÷ 60 %; lower values (30 ÷ 40%) are locally measured in layers with an higher fine content. Values up to 80 ÷ 90 %, and more are generally evaluated in the top fill layer above the +2 m NAP and between -14 m NAP and -18 ÷ -20 m NAP. These high values are estimated between 0 and -4 m NAP in the Buildings area (UNIT 070) and Pre-Treatment area (UNIT 012) and between -6 m and -8 m NAP in the Process Area (UNIT 021).

On the whole the recent sands at the site can be classified as dense to very dense, with local strata characterized by a medium density.

7.2.1.5. Deformability

The deformability of the sand has been estimated from the CPT tests reported in Attach.3 Figure 15 to Attach.3 Figure 41 with the Rix and Stokoe (1991) approach.

The following design values can be generally adopted; however in case of specific settlement calculation necessary for example for large rafts, slabs or tanks, reference to the specific results of the available CPT tests close to the structures of interest shall be made:

- From present g.l. to the top of the Pleistocene:

  $G_0 = 30 + 3z$ (MPa) lower bound

  $G_0 = 50 + 5z$ (MPa) upper bound

  with z = depth from the present ground level.

In case of loose levels of silty sands/sandy silts $G_0 = 20 \div 30$ MPa

The correspondent initial Young's modulus can be calculated as $E_0 = 2\,(1+\nu)\,G_0$ with ν=0.20:

- From present g.l. to the top of the Pleistocene sand layer:

  $E_0 = 72 + 7.2\,z$ (MPa) lower bound

  $E_0 = 120 + 12\,z$ (MPa) upper bound

  with z = depth from the present ground level.

  In case of loose levels of silty sands/sandy silts $E_0 = 48 \div 72$ MPa

Values of operational Young's modulus can be taken, for different geotechnical problems, as a fraction of the initial one on the basis of typical degradation curve published in the literature as a function of induced shear strains. A strain of 0.1% is usually taken as a reference upper limit for sandy soils deformations under shallow foundation (Atkinson and Sallfors, 1991, and Mair, 1993). Considering the degradation curve proposed by Ishihara (1996) for shallow foundation, the operational Young's modulus in sand can be in the order of 1/6 of the initial Young's modulus.

For the recent sands encountered at the site the following conservative operational Young's modulus E' can be assumed for shallow foundation design:

- $E' = 1/6 \cdot E_0$ for small foundations of light structures, isolated equipment and ancillary structures
- $E' = 1/8 \cdot E_0$ for large rafts or slabs and tanks

Therefore:

For small foundations of light structures, isolated equipment and ancillary structures:

- From present g.l. to the top of the Pleistocene sand layer:

  $E' = 12 + 1.2\,z$ (MPa) lower bound

  $E' = 20 + 2\,z$ (MPa) upper bound

  with z = depth from the present ground level.

  In case of loose levels of silty sands/sandy silts $E_0 = 8 \div 12$ MPa

For large rafts or slabs and tanks:

- From + 2 m to the top of the Pleistocene sand layer:

  $E' = 9 + 0.9\,z$ (MPa) lower bound

  $E' = 15 + 1.5\,z$ (MPa) upper bound

  with z = depth from the present ground level.

  In case of loose levels of silty sands/sandy silts $E_0 = 6 \div 9$ MPa

The recommended design operational values E' are reasonable in accordance with the suggested representative values of Young modulus indicated in *Table 1* and relevant partial factors of *Table 3* of the NEN 6740.

7.2.1.6. Effective strength parameter

Shear strength of sand also can be evaluated using the correlation proposed by Bolton (1986):

$\phi' \quad = \quad \phi'_{cv} + m \cdot DI$

$DI \quad = \quad D_r \cdot [Q - \ln(p_f')] - 1$

with:

ϕ'_{cv} = fiction angle at constant volume (see Table 7.2 according to Stroud, 1988 and Youd, 1972)

Q = 10

p_f' = $1.4 \cdot \sigma_{ff}'$ (see Jamiokowski et al. 1988) (kPa)

m = empirical coefficient due to stress conditions assumed equal to 3 (see Bolton, 1986)

D_r = relative density (-)

Table 7.2: Values of ϕ'_{cv} for silica sands according to Stroud (1988) and Youd (1972)

	Well graded sands	Uniform sands
Angular sands	$\phi_{cv}' = 38^{\circ}$	$\phi_{cv}' = 34^{\circ}$
Rounded sands	$\phi_{cv}' = 33^{\circ}$	$\phi_{cv}' = 30^{\circ}$

Based on the relative density data estimated from CPT tests and assuming $\phi_{cv}' = 30^{\circ}$, the values of ϕ' for the different areas of the site are reported in ATTACHMENT 3. The estimated values range generally between 35° and 38°. Locally, in presence of loose levels of silty sands/sandy silts values from 30° to 33° are estimated.

For practical purpose, in accordance with *Table 1* of the NEN 6740 the following conservative representative values shall be adopted:

- for bearing capacity of shallow foundation and stability of excavations $\phi' = 35^{\circ}$;
- for the evaluation of negative skin friction on piles $\phi' = 38^{\circ}$

In case of loose levels of silty sands/sandy silts $\phi' = 30^{\circ}$ is adopted

7.2.1.7. Permeability coefficient

Based on correlations with D_{10} size of the material, $k = C/10^4 \cdot (D_{10})^2$ in m/s with C = 100 for single size material (Hazen, 1892), the permeability of the sands is estimated to fall within the range of 3×10^{-5} to 3×10^{-4} m/ s (see Attach.3 Figure 7).

7.2.2. Pleistocene sands

The geotechnical properties of the Pleistocene sands encountered at the site have been estimated on the basis of some measures from laboratory tests and limited data from CPT tests. The available CPTs generally refuse in the first meters of this layer, where the q_c is greater than 30 MPa. The six CPTs of the additional investigation, where a special effort has been made to penetrate into the Pleistocene deposit, show q_c values up to 50 ÷ 60 MPa.

7.2.2.1. Grain size distribution

According to the results of grain size analyses typical values of the significant parameters are:

- D_{50} generally in the range 0.1 to 0.4 mm;
- D_{10} in the range 0.1 to 0.2 mm;
- uniformity coefficient generally in the range 1.4 to 2.7 with local higher values up to 0.4;
- fines content is less than 12% , with some higher values up to 14% and local values of about 40%.

According to this Pleistocene sands at the site can be generally classified as silty, occasionally very silty fine to medium, poorly graded to uniform sand.

7.2.2.2. Unit weight

According to the available laboratory tests results the bulk unit weight ranges between 19 ÷ 20 kN/m^3 with occasional lower values (see Attach.3 Figure 5).

7.2.2.3. Relative density

Values of relative density have been inferred from the available CPT tests results according to the relationship indicated at point 7.2.1.4.

The values of Dr in the different areas of the site are reported in Attach.3 Figure 15 and Attach.3 Figure 41). The estimated values are generally between 60÷ 80%.

The Pleistocene sands at the site can be classified as very dense.

7.2.2.4. Deformability

Deformability of the Pleistocene sand estimated from CPT values (Rix and Stokoe, 1991) is verified to be underestimated if compared with the values derived from V_S measures obtained from seismic cone tests of 2008 investigation in the Existing Plant area.

The estimated G_0 values range from 250 MPa to 350 MPa, increasing with depth.

The correspondent initial Young's modulus can be calculated as $E_0 = 2\ (1+\nu)\ G_0$ with ν=0.20.

Values of operational Young's modulus can be taken as a fraction of the initial one. In particular the following ratio can be assumed:

$E' = 1/4 \cdot E_0$

7.2.2.5. Effective strength parameters

The shear strength in terms of angle of internal friction ϕ' has been evaluated according to the correlation indicated at point 7.2.1.6

The values of ϕ' in the different areas of the site are reported Attach.3 Figure 15 to Attach.3 Figure 41. The estimated values range between 35° and 40°.

7.3. Fine grained soils

Fine grained soil includes clay to clayey silts and clays to clayey silts mixed with silty sands/sandy silts recovered randomly at different depths.

7.3.1. Grain size distribution

According to the results of grain size analyses the fine grained soils encountered at the site can be generally classified as clay with silt and clayey silts, generally slightly sandy and occasionally with sand.

7.3.2. Unit weight

The bulk unit weight ranges between about 15 kN/m^3 and 19 kN/m^3 (see Attach.3 Figure 9) with occasional values up to 20 kN/m^3. Same lenses of peat are encountered with bulk unit weight of about 12 kN/m^3

7.3.3. Natural water content

According to the laboratory results the natural water content is in the range of 30 ÷ 60% (Attach.3 Figure 10), with occasional lower values of about 20% and higher values between 70 % and 88% in the peat lenses.

7.3.4. Plasticity characteristics

The mean values of Atterberg Limits are:

- Liquid Limit ranging from 40 % to 60%, with some values up to 90% in the peat lenses;
- Plastic limit ranging from 20 % to 40% ;
- Plasticity Index ranging from 12 % to 45%;

The soils' classification on the Casagrande Plasticity Chart indicates that soils are generally inorganic clays of medium to high plasticity and locally silts of medium to high compressibility (Attach.3 Figure 12).

7.3.5. Void ratio

The measured void index ranges between 0.8 and 1.4 with occasional values up to 1.7 in the peat lenses.

7.3.6. Pre-consolidation pressure

The values of pre-consolidation pressure $\sigma'_{v,max}$ have been estimated from results of CPT tests according to the relationship proposed by Chen e Mayne (1996). The values of $\sigma'_{v,max}$ for the different areas of the site are reported in Attach.3 Figure 15 and Attach.3 Figure 41. The estimated values are highly variable; representative values for the most significant layers can be taken to range from 50 to 400 kPa, increasing with depth.

The experimental results obtained from the oedometer tests are in fair agreement with those estimated from CPT tests.

Values of corresponding overconsolidation ratio (OCR) can be taken in the range of 1.1 ÷ 1.4, with occasionally lower values close to 1 and some other higher values up to 2.

7.3.7. Compressibility

The initial shear modulus G_o has been estimated from CPT tests according to the empirical relationship proposed by Mayne & Rix (1993). The values of G_o, reported in Attach.3 Figure 15 and Attach.3 Figure 41, range from a minimum of 20 to 70 MPa, increasing with depth, and a maximum of 80 to 100 MPa, increasing with depth.

The behaviour under loading conditions of these materials may be appropriately evaluated by means of elasto-plastic constitutive models like the well known modified Cam-Clay model.

Compression (CR) and recompression (RR) indexes determined from oedometer tests are shown in Attach.3 Figure 14. The following can be inferred:

- measured values of compression index generally range between 0.1 and 0.25, with occasional higher values up to 0.5 encountered especially at about -20 ÷ -24 m NAP;
- measured values of recompression index generally range between 0.02 and 0.075;
- the CR/RR ratio ranges between 3 and 8; these quite low values of the recompression to virgin compression ratio is probably due to samples disturbance.

For practical purpose the following design values shall be adopted:

- for clay to clayey silt levels: CR = 0.22 and RR = 0.062;
- for mixed soil levels: CR = 0.14 and RR = 0.044.

The recommended values are reasonable in accordance with the suggested representative values of compression index indicated in *Table 1* and the relevant partial factor of *Table3* of the NEN 6740.

7.3.8. Undrained shear strength

The undrained shear strength c_u values estimated from CPT tests (Lunne et al.,1997) are reported in in Attach.3 Figure 15 and Attach.3 Figure 41 for different areas of the site. The estimated values are strongly variable; typical conservative representative values of the undrained shear strength for the most significant layer can be taken ranging from a minimum of 20 to 50 kPa, increasing with depth, and a maximum of 50 to 150 kPa, increasing with depth, where less plastic material are encountered.

7.3.8.1. Effective strength parameters

According to the available laboratory results from TXCIU tests the following effective strength parameters can be evaluated (see Attach.3 Figure 13):

- angle of internal friction ϕ' ranging from 20° to 30°, depending on the different clay and silt content;
- drained cohesion c' varying from 10 to 25 kPa.

7.3.8.2. Permeability coefficients

The coefficient of vertical permeability k_v assessed interpreting the oedometer tests with Terzaghi's consolidation theory may be taken in the range $8{\cdot}10^{-12}$ m/sec and $1{\cdot}10^{-10}$ m/sec, with occasional higher values up to $7{\cdot}10^{-10}$ m/s (see Attach.3 Figure 11)

The coefficient of vertical consolidation c_v evaluated interpreting the results of the oedometer tests is found to be in the range of 0.025 ÷ 0.25 m²/year with sporadic higher values.

The interpretation of the results of the dissipation tests nominally carried out in fine grained soils at different depths, shows c_h values estimated according to the Houlsby & Teh (1988) theoretical solution considerably higher (up to 100 times) than the c_v evaluated from oedometer tests. The values of c_h obtained from the Houlsby & Teh procedure is applicable to horizontal flow in the overconsolidated range. The coefficient of consolidation c_h values in the normally consolidated range can be estimated from the empirical rule proposed by 2E [redacted] et al. (1985):

$c_h(NC) = RR/CR\ c_{h\_piezocone}$

with a RR/CR ratio ranging from 0.1 to 0.15, for undisturbed samples.

The coefficient of consolidation for vertical flow c_v , which is significant for vertical drainage paths as applicable to consolidation in absence of vertical drains, can be estimated considering the anisotropy of permeability (k_h/k_v) which, for highly stratified deposits can ranges from 5 to 15 (see 2E [redacted] et al. ,1985).

Taking into account these considerations, the interpretation of the dissipation tests and the results of laboratory tests seems to be in reasonable agreement.

7.4. **Characteristic geotechnical parameters**

Waiting for the results of laboratory tests of soil investigation 2020, in the following Table 7.3 the preliminary evaluation of characteristic values (to be factorized according to NEN) of the major geotechnical parameters for different soil at the Site.

Table 7.3: Characteristic values of soil parameters (preliminary)

Parameters	Recent sands and fill	Mixed soil	Clay to clayey silt levels	Pleistocene sand
γ (kN/m^3)	18.5 ÷ 20.5	18	16 ÷ 18	19 ÷ 20
D_r (%)	≥ 60 20 ÷ 40$^{(2)}$	-	-	> 70 Locally 45 ÷ 70
ϕ (°)	35 ÷ 38 30 ÷ 33$^{(2)}$	30	25	35 ÷ 40
c' (kPa)	-	0 ÷ 10	10 ÷ 25	-
Cu (kPa)	-	20 ÷ 30	20 ÷ 50 lower bound 50 ÷ 100 upper bound	-
OCR (-)	1	2	1.05 ÷ 1.2	1
CR (-)	-	0.14	0.22	-
RR (-)	-	0.044	0.062	-
k (m/s)	$3 \cdot 10^{-5} \div 3 \cdot 10^{-4}$	$1 \cdot 10^{-8} \div 1 \cdot 10^{-10}$	$1 \cdot 10^{-11} \div 1 \cdot 10^{-9}$	-
$c_{\alpha\varepsilon}$ (%)	-	0.6	0.9	-
G_0 (MPa)	20 ÷ 30$^{(2)}$ $30 + 3 \cdot z^{(1)}$ lower bound $50 + 5 \cdot z^{(1)}$ upper bound	20 ÷ 30	20 ÷ 50 lower bound 50 ÷ 100 upper bound	250 ÷ 350
E_0 (MPa)	$2.4 \cdot G_0$	$2.4 \cdot G_0$	$2.4 \cdot G_0$	$2.4 \cdot G_0$
$E'_{operational}$ (MPa)	$E_0/6$ $^{(3)}$ $E_0/8$ $^{(4)}$	-	-	$E_0/4$

(1) z = m from FGL
(2) loose levels of silty sands/sandy silts
(3) for small foundations of light structures, isolated equipment and ancillary structures
(4) for large rafts or slabs and tanks

8. PARAMETERS FOR THE DESIGN OF DYNAMIC EQUIPMENT FOUNDATIONS

For the dynamic analysis of the foundations of compressors, pumps and other vibrating machinery, reference shall be made to the profile of shear modulus at small strain G_o as reported in Table 7.3.

In order to account for possible variability in soil conditions across the site and for critical uncertainties in parameter estimation and dynamic analyses, sensibility analyses shall be carried out considering possible variations of G_o in the range of -30% to +50% compared to the values estimated from available soil investigation.

9. ELECTRICAL AND THERMAL RESISTIVITY RESULTS

Electrical and thermal resistivity tests have been planned within the plant area in order to investigate the corrosion potential and rate of heat transfer through soil.

Electrical resistivity data are summarized in Table 9.1. The corrosivity index was established on the basis of the threshold vales reported in Table 9.2.

Table 9.1 – Results of electrical resistivity tests

			Electrode Spacing [m]				
Test No.			1.0	2.0	4.0	8.0	16.0
EP501	Apparent Resistivity [Ωm]	North-South	1123	1004	648	270	31
		East-West	991	1072	554	261	24
	Corrosivity Index		Sl. C	Sl. C	Sl. C	Sl. C	C
EP502	Apparent Resistivity [Ωm]	North-South	839	844	574	209	28
		East-West	909	735	595	215	24
	Corrosivity Index		Sl. C	Sl. C	Sl. C	Sl. C	C
EP503	Apparent Resistivity [Ωm]	North-South	851	786	533	259	24
		East-West	921	791	554	253	24
	Corrosivity Index		Sl. C	Sl. C	Sl. C	Sl. C	C
MC → Moderately Corrosive Sl. C → Slightly Corrosive C → Corrosive Se. C → Severely Corrosive							

Table 9.2 – Correlation of Resistivity from ERT to Corrosivity

Apparent Resistivity	Corrosivity Index
up to 10 Ωm	Severely corrosive
10 Ωm to 50 Ωm	Corrosive
50 Ωm to 100 Ωm	Moderately corrosive
100 Ωm and above	Slightly corrosive

10. GROUND AND GROUNDAWATER CHEMICAL PROPERTIES

The following chemical tests are planned on selected soil and water samples to establish criteria for the protection of buried reinforced concrete and against possible corrosion of steel structures:
- Organic content of soils
- PH of soil and water
- Calcium Carbonate content of soils
- Particulate and dissolved matter in water
- Soluble salt content of soils by refractometer
- Chloride content in soil and water
- Sulphate Ion in soil and water

The results of tests on soils and water are summarized in Table 10.1 to Table 10.2. The maximum, minimum and average measured values for each kind of test on soil are reported in Table 10.3 and Table 10.4.

Table 10.1 – Results of soil chemical analyses

BH	Depth from GL	Org.Cont	CaCo₃	pH	SO_3	Cl^-
[n°]	[m]	[%]	[%]	[-]	[mg/l]	[mg/l]
BZU505	27.74	3.1				
BU501	2.00		3.8	8.81	<8.3	<7
BU501	11.50	1.8				
BU501	27.00	2.7				
BU502	16.40	1.6				
BU502	25.96	34.2				
BU504	4.00		3.2	9.02	<8.3	<7
BU506	25.80	2.4				
BU507	2.00	0.3	5.4	8.94	<10.8	<7
BZU503b(*)	*1.50*		*<0.5*	*8.76*	*55*	*<7*
BZU503b(*)	*14.90*	*3.3*				
(*) falling outside the specific area of Refinery RDCG MNA, but included in the Area 1 site investigation campaign						

Table 10.2 – Results of water chemical analyses

BPZ	pH	SO_3	Cl-	Mg
[n°]	[-]	[mg/l]	[mg/l]	[µg/l]
BZU503 S(*)	*7.6*	*70*	*33*	*8900*
BZU503 D(*)	*7.6*	*833*	*14000*	*1000000*
BZU505 S	7.6	208	31	61000
BZU505 D	7.3	1916	18000	1200000
(*) falling outside the specific area of Refinery RDCG MNA, but included in the Area 1 site investigation campaign				

Table 10.3 –Summary of soil chemical test results in soil (considering all data for Area 1)

Test	Maximum measured value	Minimum measured value	Average measured value
Water Soluble Sulphate as SO_3 (%)	0.0055	0.00083	0.00206
Water Soluble Chloride Ion Cl^- (%)	<7*10^{-4}	<7*10^{-4}	<7*10^{-4}
Organic Matter Content % (%)	34.2	0.3	6.18
pH	9.02	8.76	8.88
Calcium Carbonate $CaCo_3$ % (%)	5.40	3.20	4.13

Table 10.4 –Summary of soil chemical test results in water (considering all data for Area 1)

Test	**Maximum measured value**	**Minimum measured value**	**Average measured value**
pH	7.60	7.30	7.53
Water Soluble Sulphate as SO_3 (g/l)	1.92	0.07	0.76
Water Soluble Chloride Ion Cl^- (%)	0.02	<0.01	<0.02

11. ESTIMATED PLATFORM SETTLEMENTS

The platform settlements of the recent reclaimed area at Maasvlakte, that can be expected during the lifetime of the Plant and that have to be considered for the design purposes, have been studied in details in Ref.Doc.[3]. In particular the document analyses:

- the reclamation phase (from 2010 to 2013) up to the present ground level, determining the expected residual settlements over the lifetime of the Plant in absence of any other increase in site elevation;
- the currently planned site preparation phase, determining the expected residual settlements over the lifetime of the Plant for the site rising to the Final Ground Level elevation +5.80 NAP (corresponding to HPP).

For the purposes of this report, residual settlement due to site preparation activities up to 30 years are considered relevant.

In particular, residual settlements (both total and differential) from the most significant construction stages (piping connection and mechanical completion) of the **Refinery RDCG MNA Area** are shown in **Error! Reference source not found.** and Figure 11.2. A summary is reported in Table 11.1.

Table 11.1: Residual settlements due to site preparation activities on Refinery RDCG MNA Area

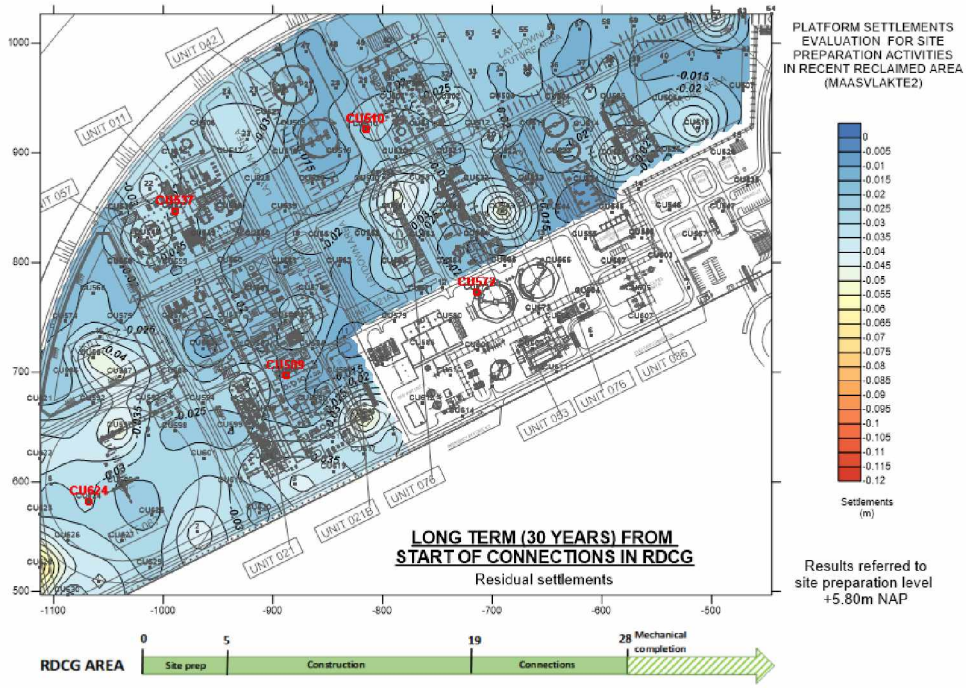
Residual settlements at s.p.l. at 30 years from start of SP	**Total settlements**	**Differential settlements**
	[cm]	**[$^0/_{00}$]**
From piping connections RDCG (19 months after start of SP)	1.5 ÷ 4.5	< 0.5
From mechanical completion RDCG (28 months after start of SP)	1 ÷ 4	< 0.5

In relation to the performance criteria adopted for the **Refinery RDCG MNA Area** Project, the compliance of the estimated residual settlements (total and differential) of Table 11.1 with the tolerable ones will be verified.

Depending on the results of this comparison and the extension of possible critical situations, the design strategy could be aimed at improving large areas through extensive works or rather acting on specific critical items through localized interventions (see Section 12 and Section 13).

<u>In any case, the sooner the site preparation activities begin, the greater the benefits in terms of the schedule and residual settlement.</u>

Moreover, it must be taken into account that any activity of raising of the areas adjacent to the Project ones can impact on the settlements of the structures currently being planned. Therefore, a constraint must be identified for future filling activities that can be performed by the Port Authority in the neighboring areas. Today we can identify this constraint at 1 m from of fill respect to the current G.L..



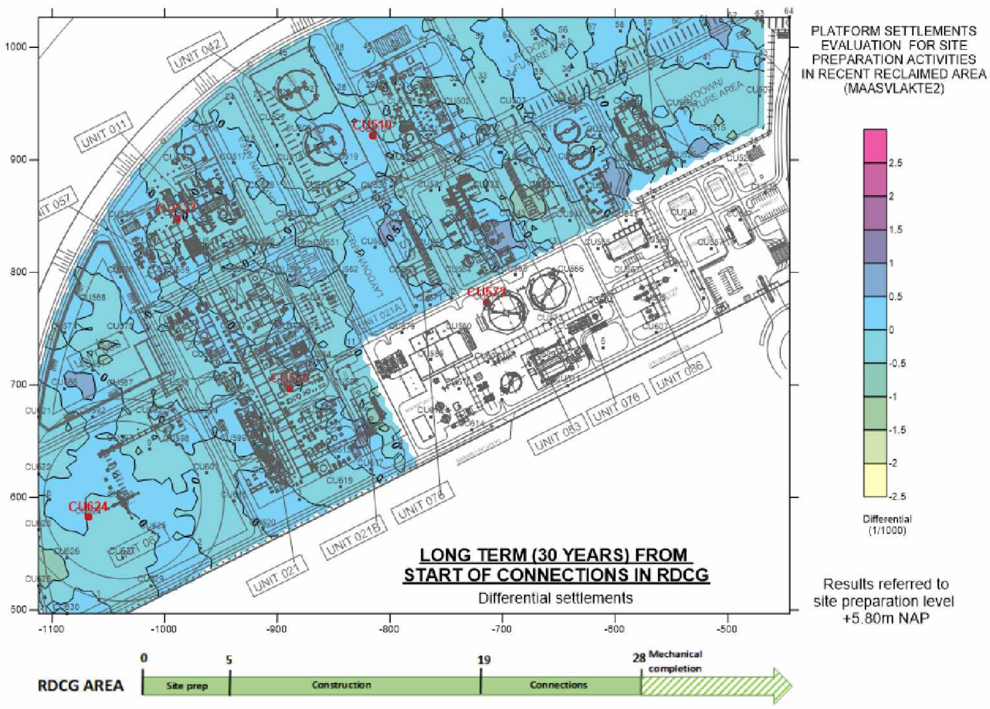
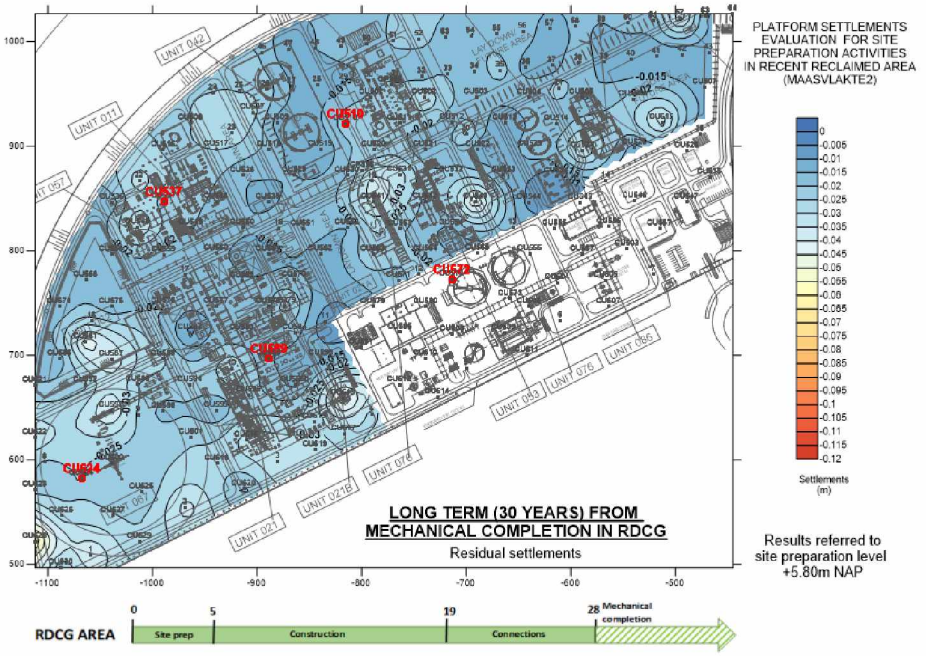


Figure 11.1: Residual total and differential settlements from piping connection



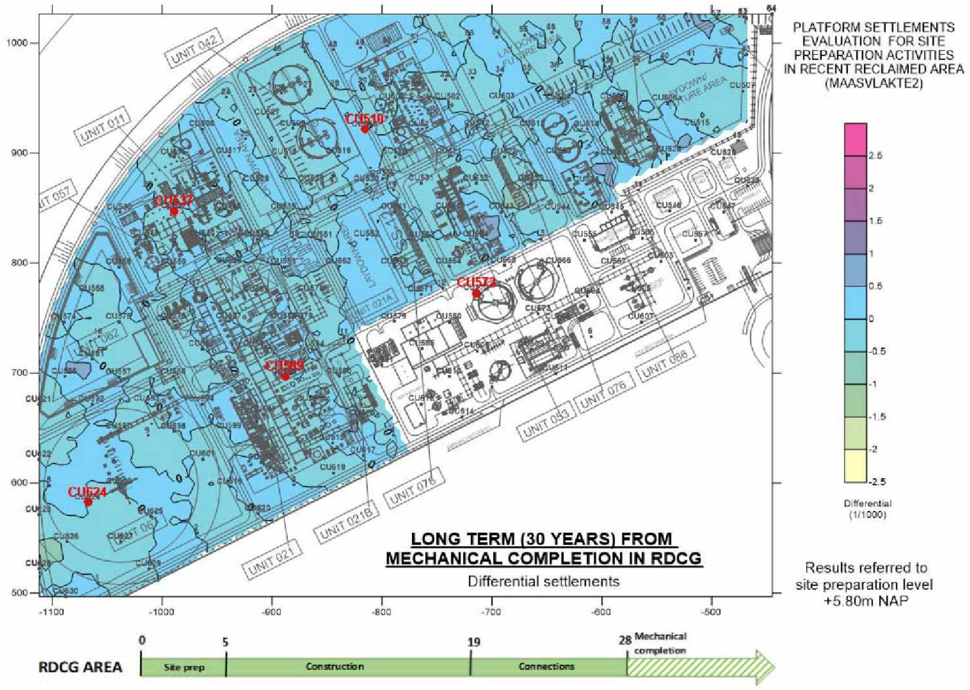


Figure 11.2: Residual total and differential settlements from mechanical completion

12. GEOTECHNICAL RECOMMENDATIONS FOR FOUNDATION DESIGN

12.1. Deep foundations

Deep foundation shall be foreseen for all buildings and heavy structures or structures which are sensitive to differential settlements will be supported on piles.

In particular, considering the type of structures, the soil characteristics and the residual settlements due to the previous activities on the area (reclamation and site preparation – see Section 11) piles are recommended for the Main structures in the Process areas (Units 21, 11, 57) and in the Pre-Treatment building area (Unit 12).

Piles may be taken into consideration also for other items in case the expected total and differential settlements of shallow foundations (see Section 16) are greater than the acceptable limits. The convenience of the use of deep foundation should be evaluated with respect to shallow foundation on improved soil.

For the geotechnical recommendations for deep foundation reference shall be made to JSD – Feasibility Phase – Job Design Specification for Piles (Ref.Doc.[1]).

12.2. Shallow Foundations

Direct shallow foundations may be adopted for light minor structures, isolated equipment and ancillary structures not sensitive to differential settlements.

For isolated foundations with width B ≤ 4 m, design bearing resistance and settlements have been evaluated for ULS and SLS checks respectively (see point 16.1).

For large rafts or slabs the evaluations shall be done case by case, taking into account the tolerances on total and differential settlements of each single structure (see point 14.4).

Shallow foundations could also be adopted for tanks constructed with usual annular concrete ring, provided that the estimated settlements (total and differential), which have been used for the geometric and structural design of the tanks themselves and of the connections, are compatible with the tolerable values. The detailed analyses of tanks settlements are reported at Section 18.

From a practical point of view, the depth of foundations should be limited to 2.5 ÷ 3.0 m below Final Ground Level, to minimize interference with groundwater both during construction (to limit dewatering requirements) and in service. A minimum foundation depth of 1 m has been assumed.

Criteria for the design of shallow foundations (bearing pressures for failure and serviceability checks) are given in Section 14. Results of the performed evaluations are reported in Section 16.

12.3. Shallow foundations on improved soil

The local presence natural material with poor to medium properties could require a soil improvement to reduce the expected total and differential settlements of structures placed on shallow foundations due to the loads acting on the specific examined items (see Section 13).

The convenience of the treatments should be evaluated with respect to the use of deep foundations.

13. SOIL IMPROVEMENT

Soil improvement should be evaluated as a suitable solution, can be taken into account in place of structural piles, to improve foundation subgrade (bearing capacity) and reduce within acceptable limits the expected long term settlements of shallow isolated foundations, the total and differential settlements of large raft/slab or tanks in specific Plant areas characterized by local poor to medium properties of the natural materials.

For the general geotechnical recommendations for soil improvement reference shall be made to JSD – Feasibility Phase – Job Design Specification for Soil Improvement (Ref.Doc.[2]).

It is anticipated that at the present design phase rigid inclusions are the solution most adequate to the scope.

Rigid inclusions represent the technique in which rigid columns, which remain stable without any lateral confinement, with relatively small diameter and usually without steel reinforcement, are connected to the structure by means of a load transfer platform (LTP) or a concrete raft.

Considering the characteristics of the site, the best solution for rigid inclusions is by means **unreinforced driven cast in situ concrete piles** (the same technology that is considered the most suitable for the structural piles of the **Refinery RDCG MNA Area)** connected to the structure by means a concrete slab on which a layer of structural fill is realized. Typological schemes of the solution is reposed in Figure 13.1.

This solution has been already successfully adopted in the Existing Plant.

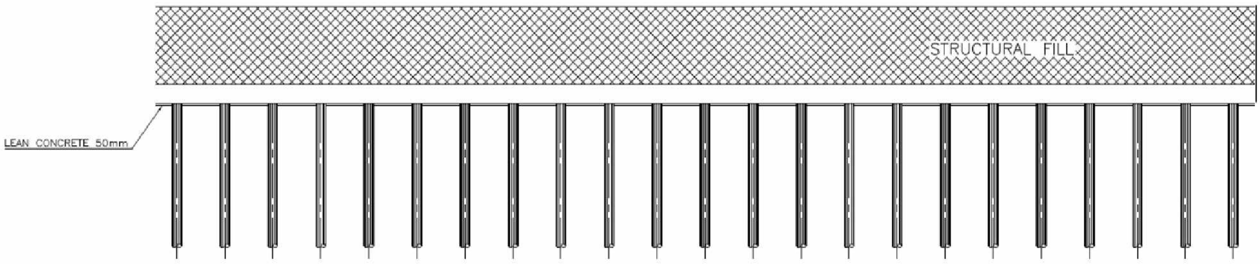


Figure 13.1: Residual total and differential settlements from mechanical completion

13.1. Intensity of treatment

The intensity of treatment indicated in the following can be considered as preliminary evaluation:

- rigid inclusions diameter D = 457/510 mm
- spacing ~(4 ÷ 5)·D on a square grid or an equivalent triangular grid.

Concerning to the rigid inclusions length the following preliminary indication are provided, assuming a pile head at ~3.5 m below F.G.L.:

- process areas (Unit 21, 11, 57) → L ~ 18.5 m (pile tip at ~ -16 m NAP)
- elsewhere → L ~ 15.5 m (up to ~ -13 m NAP)

A preliminary assessment in terms of beneficial effects of a soil treatments by means rigid inclusions has been provided at point 16.2.

14. BASIS OF GEOTECHNICAL DESIGN FOR SHALLOW FOUNDATIONS

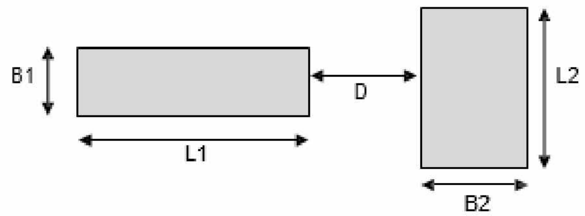
14.1. Premise

With the term "*shallow foundations*" are herein intended all direct (not piled) foundations having a maximum dimension of the minor side (B, being B ≤ L) in the order of 4÷6 m. For the design of these foundations reference shall be made to paragraphs 14.2 and 14.3. Larger foundations are herein considered as "*slab/raft foundations*". For the design of these foundations reference shall be made to paragraph 14.4.

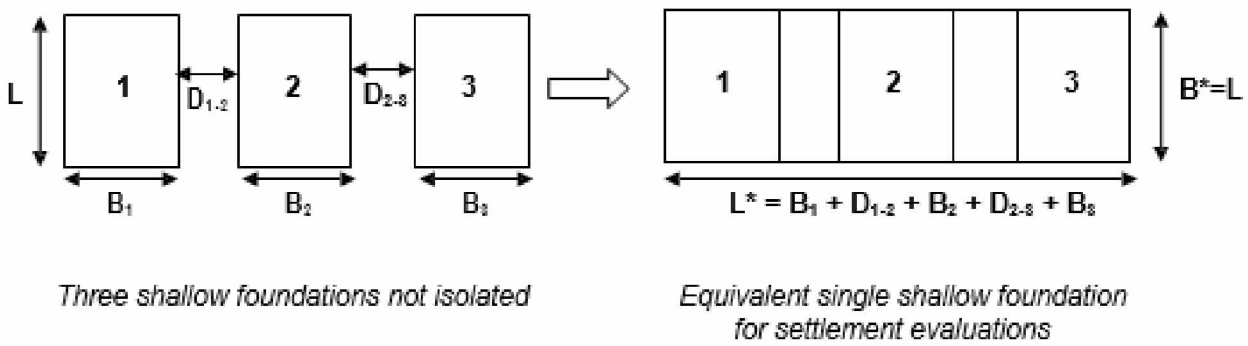
14.1.1. Definition of isolated foundation

Unless otherwise indicated, the recommendations provided in this report refer to isolated shallow foundations.

With the term isolated foundations is meant foundations of the same structure or of different structures distant each other at least as much as the maximum between the minimum foundations sizes (B1, B2, . . , Bn). For example, two foundations, having a minimum foundations size of B1 and B2 respectively, are considered isolated if the distance between them is D ≥ max {B1; B2} (see scheme below).



Should two or more foundations be not considered isolated, then the Design shall take into account, by means of specific analyses (not included in this report unless otherwise indicated for specifics foundations), the mutual interaction between adjacent foundations and the relevant effects in terms of total and differential settlements. For a preliminary evaluation of settlements in case that the distance between two or more shallow foundations is less than the clear distance abovementioned, the foundations shall be considered as a single foundation having dimensions that cover the footprints of all single foundations (see for example the scheme below).



14.1.2. Rigid foundation

Foundations are considered "rigid" if they have a flexibility factor K_F as defined below not less than 10 (Mayne and Poulos, 1999):

$K_F \sim (E_{found} / E_{op,av}) \cdot (t/a)^3$

where:

E_{found} = elastic modulus of foundation material (i.e., reinforced concrete)

$E_{op,av}$ = representative operative elastic modulus of soil (natural or treated) located beneath the foundation base (i.e., value of E_{op} at depth z = a)

a = foundation size

t = foundation thickness

Foundations with K_F less than 10 are admissible, but they will require separate evaluation.

14.1.3. Foundation dimensions and loads

Unless otherwise indicated, the recommendations on bearing pressures provided in this report refer to the effective dimensions of a shallow foundation according to Meyerhof theory.

According to the above, the following applies in this report:

- Real minor and major dimensions of a shallow foundation are denoted respectively with terms B and L (i.e. B ≤ L).
- Effective dimensions of a shallow foundation are denoted respectively with terms B′ and L′ and they are calculated according to Meyerhof theory (see Figure 10.1 and Figure 10.2).
- Stated that it is always considered in this document B ≤ L and that it would be possible, for some load combinations, to obtain B' > L' (depending on the actual values of the eccentricities along B and L sides), for sake of clarity all geotechnical verifications based on Meyerhof area shall be carried out considering the following effective dimensions: B* = min {B' ; L'} and L* = max {B' ; L'}, having considered B* ≤ L*.

In the case of rectangular foundation, the following applies (see Figure 10.1):

$B' = B - 2 \cdot e_x$

$L' = L - 2 \cdot e_y$

where

B	real minor dimension of the foundation along direction x (B side)
L	real major dimension of the foundation along direction y (L side, L ≥ B)
e_x	eccentricity along x direction (B side)
e_y	eccentricity along y direction (L side)
B'	effective dimension of the foundation along x direction (B side)
L'	effective dimension of the foundation along y direction (L side)

B* effective minor dimension of the foundation, B* = min {B' ; L'}

L* effective major dimension of the foundation, L* = max {B' ; L'}

In the case of circular foundations reference can be made directly to the equations provided in Figure 10.2.

Considering this approach, the eccentricity of vertical load N shall be accounted by the Structural Designer that shall evaluated the correct effective area B'xL' for the given eccentricities.

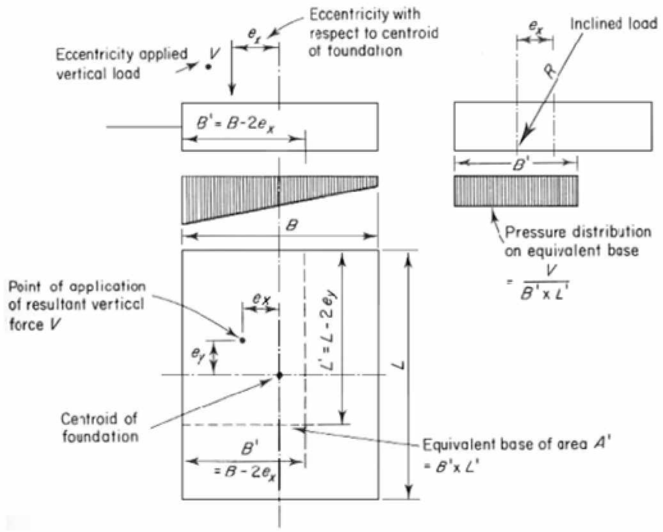


Figure 14.1: Effective footing dimensions for rectangular foundations (Meyerhof 1953).

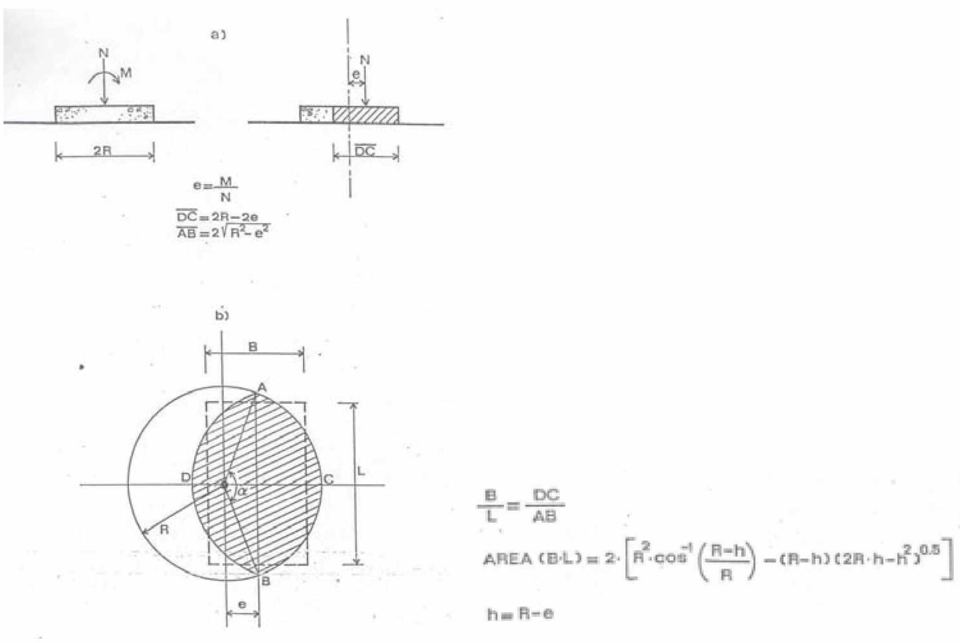


Figure 14.2: Effective footing dimensions for circular foundations (Meyerhof 1953).

14.1.4. Ratio between horizontal and vertical loads

The recommendations provided in this report refer to specific maximum ratios between horizontal and vertical loads. Should be the actual ratio between horizontal and vertical loads greater than indicated in Section 16 then the analyses need to be revised accordingly.

14.1.5. Gross and net pressures at the bottom of foundation

Unless otherwise indicated, the recommendations provided in this report refer to gross pressures.

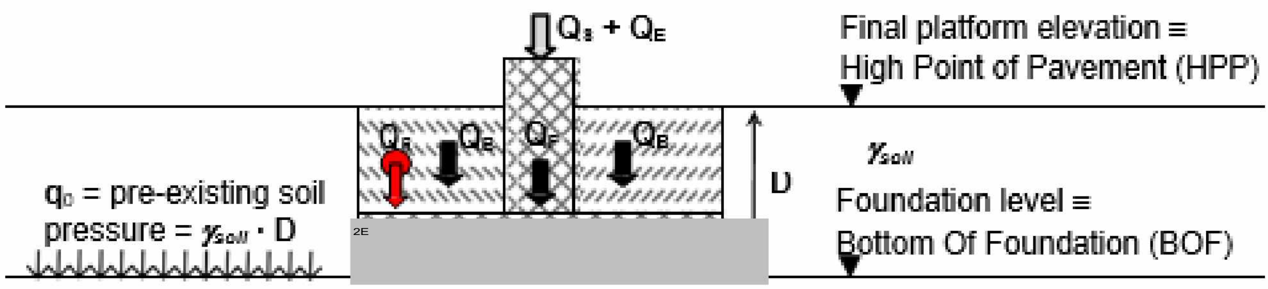
The gross pressure at the Bottom Of the Foundation (BOF) is defined as the total pressure acting at the foundation level due to loads from structures aboveground (Q_S), equipment aboveground (Q_E), foundation (Q_F), backfilling (Q_B) resting above the foundation and possible further loads (Q_R) acting above the foundation that have not taken into account in Q_S, Q_E and Q_B. All these loads shall be factorised and accounted by the Structural Designer according to the design approach adopted.

The net pressure (q_{net}), which represents the increase of the pressure at the foundation level with respect to the pre-existing pressure before the excavation for the construction of the foundation, is calculated as the difference between the gross pressure (q_{gross}) at the level of the bottom of the foundation and the pre-existing soil pressure at the same level. It is:

$q_{net} = q_{gross} - q_0$

where

$q_0 = \gamma_{soil} \cdot D$	pre-existing soil pressure at the foundation level
D	depth of bottom of foundation from final platform elevation



14.2. Ultimate Limit State (ULS)

With reference to shallow foundations this document provides information regarding the verifications that the following GEO ultimate limit states are not exceeded:

- bearing capacity
- sliding

Moreover, indications of actual groundwater levels and possible future rising are provided in this document for their use in UPL verifications (loss of equilibrium of the structure or the ground due to uplift by water pressure – buoyancy - or other vertical actions), to be accounted by the Structural Designer.

According to Ref.Doc.[8], the design of shallow foundations at the Ultimate Limit States (ULS) shall be carried out according to Design Approach 3 (herein after DA3), whereby it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

- (A1* or A2†) "+" M2 "+" R3

where "+" implies "to be combined with".

Values of sets A1 are given in Table 14.1.

Values of sets M1 are given in Table 14.2.

Partial factors on soil resistances (R3) are given in Table 14.3.

<u>Partial factors on actions according to sets A1* shall be applied by the Structural Designer to the loads coming from the superstructure, from the weight of foundation, from the weight of backfilled soil remaining above the foundation and from any further possible load acting above the foundation. Partial factors on actions according to sets A2† shall be applied to the loads coming from geotechnical actions.</u>

Table 14.1: Partial factors on actions or the effects of actions (table A.3 – Ref.Doc.[8])

Action	Symbol	Set	
		A1	A2
Permanent (Unfavourable)	γ_G	1.35(*)	1.00
Permanent (Favourable)		0.90	1.00
Variable (Unfavourable)	γ_Q	1.50(*)	1.30(*)
Variable (Favourable)		0.00	0.00
(*) Values apply to Reliability Class RC2. A multiplication factor applies equal to 1.1 for RC3 and 0.9 for RC1			

Table 14.2: Partial factors on actions or soil parameter (table A.4a – Ref.Doc.[8])

Soil parameter	Symbol	Set	
		M1	M2
Angle of shearing resistance [a]	$\gamma_{\varphi'}$	N/A	1.15
Effective cohesion	$\gamma_{c'}$	N/A	1.60
Undrained shear strength	γ_{cu}	N/A	1.35
Unconfined strength	γ_{qu}	N/A	1.35
Weight density	γ_{γ}	N/A	1.10
[a] This factor is applied to tan φ'			

Table 14.3: Partial factors for spread foundations (EN 1997-1, table A.5 – Ref.Doc.[8])

Resistance	Symbol	Set		
		R1	R2	R3
Bearing capacity	$\gamma_{R;v}$	N/A	N/A	1.00
Sliding	$\gamma_{R,h}$	N/A	N/A	1.00

14.2.1. Design bearing capacity

According to Ref.Doc. [8], the following inequality shall be satisfied at ULS:

$V_d \leq R_d$

where

V_d design action at the base of foundation: shall include the weight of the structures and equipment, the weight of the foundation, the weight of any backfill material, any further load acting above the foundation and all earth pressures, either favourable or unfavourable, and water pressures not caused by the foundation load. All loads acting at the base of the foundation, accounted by the Structural Designer, shall be factorised according to the partial factors provided in Table 14.1.

R_d design resistance to be calculated as follows

$$R_d = (B' \cdot L' \cdot q_{lim}) / \gamma_{R,v}$$

According to Ref.Doc.[8], the q_{lim} shall be evaluated as per method described in the following, considering the soil partial factor γ_m given in Table 14.2. Values of partial factor $\gamma_{R,v}$ are provided in Table 14.3 according to the selected Design Approach and Combination (where applicable).

14.2.1.1. Calculated bearing capacity in drained conditions

The calculated (ultimate, q_{lim}) bearing capacity of shallow foundations in drained (effective stress) conditions can be determined, for different sizes and shapes of foundation according to according to Eurocode 7. In the following the q_{lim} is calculated as follows:

$$q_{lim} = c' \cdot N_c \cdot b_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot b_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B' \cdot N_\gamma \cdot b_\gamma \cdot s_\gamma \cdot i_\gamma$$

where:

c', ϕ' = shear strength parameters in drained conditions factorized according to partial factor γ_m given in Table 14.2;

$q' = \gamma'_{t} \cdot D$ = vertical effective overburden pressure at footing base;

γ'_{t} = effective unit weight of the soil above the foundation level;

D = foundation embedment, Figure 14.3;

γ' = effective unit weight of the soil below the foundation level;

$B' = B - 2 \cdot e_x$	= effective foundation width, function of load eccentricity;
$L' = L - 2 \cdot e_y$	= effective foundation length, function of load eccentricity;
e_x , e_y	= load eccentricity along B and L sides, respectively;
$N_c = (N_q - 1) \cdot \cot\varphi$	= bearing capacity coefficient
$N_q = e^{\pi \tan\varphi} \cdot \tan^2(45 + \varphi/2)$	= bearing capacity coefficient
$N_\gamma = 2.0\,(N_q - 1) \cdot \tan\varphi$	= bearing capacity coefficient
$b_c = b_q - \frac{1-b_q}{N_c \cdot \tan\varphi}$	= footing inclination factor
$b_q = (1 - \alpha \cdot \tan\varphi)^2$	= footing inclination factor
$b_\gamma = (1 - \alpha \cdot \tan\varphi)^2$	= footing inclination factor
α'	= inclination of foundation, Figure 14.3;
$s_c = \frac{s_q \cdot N_q - 1}{N_q - 1}$	= shape factor for rectangular, square and circular shape
$s_q = 1 + \frac{B'}{L'} \cdot \sin\varphi$	= shape factor for rectangular, square and circular shape
$s_\gamma = 1 - 0.3 \cdot \frac{B'}{L'}$	= shape factor for rectangular, square and circular shape
$i_c = \frac{i_q \cdot N_q - 1}{N_q - 1}$	= coefficient of inclined load
$i_q = i_\gamma = 1 - \frac{H_d}{V_d + A' \cdot c' \cdot \cot\varphi}$	= coefficient of inclined load, when H acts along L and L'/B' ≥ 2 (Case 1)
$i_q = \left(1 - \frac{0.7 \cdot H_d}{V_d + A' \cdot c' \cdot \cot\varphi}\right)^3$	= coefficient of inclined load, when H acts along B (Case 2)
$i_\gamma = \left(1 - \frac{H_d}{V_d + A' \cdot c' \cdot \cot\varphi}\right)^3$	= coefficient of inclined load, when H acts along B (Case 2)

Case 3: when H acts along L and L'/B' < 2, the coefficients of inclined load are determined by interpolation of Case 1 and Case 2

Case 4: for H acting in any direction and L'/B' ≥ 2, the coefficients of inclined load are determined by interpolation of Case 1 and Case 2

Case 5: for H acting in any direction and L'/B' < 2, the coefficients of inclined load are determined by interpolation of Case 2 and Case 3

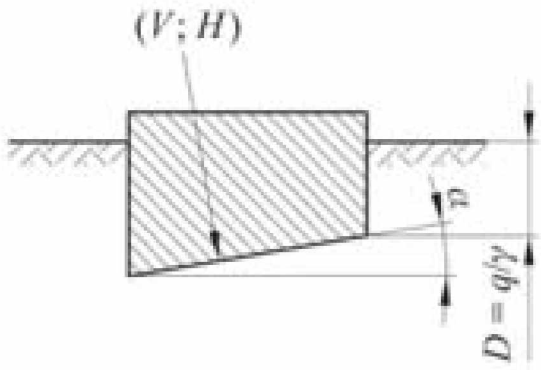


Figure 14.3: Foundation embedment level and foundation inclination.

14.2.2. Design sliding resistance

According to Ref.Doc.[8], the following inequality shall be satisfied at ULS:

$H_d \leq R_d$

where

H_d design action: shall include earth forces imposed on the foundation. All loads acting at the base of the foundation shall be factorised according to partial factors given in Table 14.1.

R_d design resistance to be calculated as follows:

$R_d = R_c / \gamma_{R,h}$

According to Ref.Doc.[8], the R_c shall be evaluated as per methods described in the following paragraphs, considering the soil partial factor γ_m given in Table 14.2. Values of partial factor $\gamma_{R,h}$ are provided in Table 14.3 according to the selected Design Approach and Combination (where applicable).

14.2.2.1. Calculated sliding resistance in drained conditions

The calculated sliding resistance of shallow foundations in drained (effective stress) conditions is determined according to Ref.Doc.[6]:

$R_c = V \cdot \tan(\delta)$

where:

V vertical factorised load or component of the total load acting normal to the foundation base, reduced for possible water underpressure

δ design value of structure-ground interface friction angle, being:

$\delta = 1.0 \cdot \phi'$ in case of cast-in situ concrete foundation

$\delta = 2/3 \cdot \phi'$ in case of smooth precast foundation where ϕ' is the soil friction angle in drained conditions factorized according to partial factor γ_m given in Table 14.2.

Any effective cohesion c' should be neglected in the calculations.

14.3. Serviceability Limit State (SLS)

Serviceability Limit State conditions are reached in case that the deformations of the foundation and/or structure exceed the required serviceability limits in terms of vertical movements, horizontal movements, rotations and/or deflections, etc., compromising the correct functionality and durability of the structures.

Concerning this document, calculations of the following deformations of shallow foundations are presented:

- vertical settlements;

Values of partial factors for serviceability limit states are taken equal to 1.0.

For rigid footings, the allowable total settlement under permanent and quasi-permanent static loads is typically 1" (25.4 mm). For larger foundations, the allowable total settlement under permanent and quasi-permanent static loads is 1" (25.4 mm) to 2" (50.8 mm).

The exact load combinations to be assumed in the design verifications and the corresponding reference required serviceability limits shall be defined by the General Designer of the plant and/or by the Structural Designer.

The settlements of foundation blocks and partially completed structures which accrues before any rigid connections are made, and which can be compensated in subsequent construction, are disregarded for the purposes of serviceability evaluations.

14.3.1. Vertical settlements

Settlement calculations are carried out based on the operational values of Young's modulus E'_{op} (typically for gravelly/sandy deposits) and/or compression (c_c) / recompression (c_r) indexes (typically for silty/clayey deposits) and considering the stress increments in soil calculated according to the elastic theory by Boussinesq. (1885) Such theory models the soil as a linear-elastic homogeneous and isotropic half-space and assumes that the loaded area is perfectly flexible (as implemented by Florin, 1959).

The total settlement at the foundation base is given by the sum of the contributions of each sub-layer where applied loads induce a vertical stress increment typically higher than 10% of the initial vertical effective stress σ'_{vo}.

In the absence of more precise calculations that take explicitly into account the rigidity of the foundation, the vertical displacement of a vertically loaded rigid area can be approximated by the mean vertical displacement of a uniformly loaded flexible area of the same shape.

For rectangular foundations the following formula can be used (Davis & Taylor 1962):

$$s = \frac{2 \cdot s_{centre} + s_{corner}}{3}$$

where:

s_{centre} = settlement calculated at the centre of the flexible foundation

s_{corner} = settlement calculated at the corner of the flexible foundation

For circular foundations the following formula can be used (Davis & Taylor 1962):

$$s = \frac{2 \cdot s_{centre} + s_{edge}}{2}$$

where:

s_{centre} = settlement calculated at the centre of the flexible foundation

s_{edge} = settlement calculated at the edge of the flexible foundation

Strip foundation are considered flexible, thus:

$$s = s_{centre}$$

where:

s_{centre} = settlement calculated at the centre of the flexible foundation

For rigid foundations the settlements may be taken as the 90% of the maximum settlement of the centre of flexible foundations.

14.3.2. Foundations with high eccentric loads

In order to limit rotation, foundations of free-standing structures should be designed such that eccentricity of loads under persistent loads (self weight, operating loads, backfill, earth pressures etc.) does not exceed 1/6 of the width (B) of rectangular footings or 1/4 of the radius (R) of circular footings. Careful consideration will be given to the rigid rotation of foundations under eccentric loads, to verify that it will not exceed acceptable limits, at all heights of interest, say where piping connections occur.
Under special load case, as seismic load, the eccentricity of loads should not exceed 1/4 of the width of rectangular footings or 0.5 of the radius of circular footings; if in special cases it is necessary to exceed these limits, special precautions shall be taken including:

- careful review of the design values of actions;
- increase the half-width or radius of the foundation by 0.1 m compared with nominal design.

The horizontal displacement due to the rotation of the foundation shall be added to the horizontal deflection of the structure due to bending moment and to the horizontal displacement due to foundation sliding; the calculated total displacement shall be checked according to the prescribed tolerances.

14.3.3. Stiffness parameter for foundation structural design

The subgrade reaction moduli to be adopted for the structural design of foundations are defined below, on the basis of the following criteria:

- for footings with the main loads applied near the edges of the foundation itself, a constant value of reaction modulus (k_v) can be adopted for the whole foundation. For footings with width up to

4 m, the value of the reaction modulus can be derived from the gross pressures curves, as a ratio between gross pressure (q_{gross}) and settlement;

- for footings with the main loads applied at the centre of the foundation itself, the reaction modulus can be taken to vary linearly from a peak value near the edges (k_{v1}) to a minimum value at the centre (k_{v2}). The values can be obtained as described at the previous point but with an increase of 20% for the edges (k_{v1}) and a decrease of 10% at the centre (k_{v2}).

14.4. Design of slab-raft foundations

For wider foundations, such as slab/raft foundations, it shall be considered that:

- Slab foundations and raft foundations are usually adopted to support complex structures, where loads are transmitted at various point of the foundation through columns and/or walls. These complex load conditions, and the finite stiffness of the foundation slab, are such that variable settlements and variable contact pressures occur. This is for example the case of slab foundations loaded by columns/walls carrying different vertical loads.
- Differential settlement usually is a verification criteria for the adequacy of the foundations and the limitation to the settlement to a unique fixed value obtained assuming uniform applied pressures is meaningless.
- Strictly speaking it is not correct to give unique values of “allowable pressure” for slab foundations. Guideline values given in this report are for preliminary evaluations only.
- Similarly, strictly speaking, it is not correct to give a unique value of the Winkler coefficient valid for the whole slab, independently of foundations stiffness and load conditions. A dedicated analysis should be carried out as recommended below. Guideline values given in this report are for preliminary evaluations only.

The procedure described below is recommended for slab foundations design and verifications

1. Estimate of Winkler reaction coefficient(s)
   - The foundations soil shall be modelled as a continuum elastic medium, having soil deformation parameters taken from in situ and laboratory investigations in the relevant area;
   - The foundation slab shall be modelled taking into account its stiffness, that plays an important role in the contact pressure distribution;
   - Selected load conditions shall then be studied, examining the most representative. In selecting load conditions, the different range of possible results, in terms of absolute values and homogeneity of the Winkler reaction spring shall be taken into account. Load location shall be modelled accurately, in order to represent at best the load distribution through the slab.
2. Geotechnical analysis and verifications of the obtained results
   - Total settlement shall be maintained below limits that shall be fixed by the Designer of the plant and/or the structural Designer or established in relation to serviceability/safety limits of the equipment. Usually, values of the order of say 2” (50.8 mm) are accepted for slab foundations.
   - Usually, the critical issue is to maintain the differential settlements between elements below tolerable values. Limits shall be provided by the structural Designer or by the Vendor of given equipment. As a guideline, for concrete frames, values of the order of 1/300 to 1/500 (of the distance existing between two close elements) are frequently reported by the international literature as

limit for architectural damage, and 1/150 to 1/200 for structural damage. Limits for steel structures are expected to be higher (higher differential settlements are allowed).

- Locally, calculated pressures on ground can attain high values. In this light, it is here considered that, according to the soil model adopted (perfectly elastic medium), the contact pressure can attain theoretically infinite values at the edge of the foundations. <u>These values shall not be considered in the verification.</u>
- It is instead suggested that average contact pressure below the most loaded areas shall not exceed limits for which the assumption of soil elasticity is no longer valid. As a guideline, it is suggested to maintain the average contact pressures below the most loaded areas below the design bearing pressure indicated in the in this report for the maximum analysed foundation dimension.

From the above calculations is possible to obtain values of Winkler coefficient through the foundation area as a ratio between contact pressure and settlements. The obtained values can be uniform or far from uniform according to load conditions, foundation stiffness and load intensity. The results obtained, as well as the possibility of carrying out structural analyses under the whole set of load conditions with constant values of Winkler coefficient, shall be evaluated and examined by the structural Designer.

It is recommended to utilize the above procedure for most important structures. Results of early analyses, in terms of Winkler reaction modulus, can in principle be extrapolated to others slab having similar dimensions/load conditions, also taking into account that, in general, relatively small variations in the Winkler spring constant do not usually yield to significant differences in foundation design.

In principle the same applies also to strip foundations, which are usually constituted by “T” shaped beam on which different columns transfer their vertical load. Also, in this case their design is usually governed by limiting differential settlements between columns and/or dictated by bending moments in the foundation beams.

15. BASIS OF GEOTECHNICAL DESIGN FOR PILED FOUNDATIONS

The basis of geotechnical design of piled foundation are given in JSD – Feasibility Phase – Job Design Specification for Piles (Ref.Doc.[1]).

16. DESIGN OF SHALLOW FOUNDATIONS

16.1. Shallow foundation on natural soil

In the following the design bearing resistance R_d evaluation for ULS checks and settlements calculation for SLS checks of shallow foundations in the **Refinery RDCG MNA Area**

It is anticipated that for the current design phase, shallow foundations have been dimensioned considering no improvement is carried out, except for local soil substitution with selected and compacted granular material, in case fine grained material is found at bottom of foundation.

16.1.1. Design bearing pressure (ULS checks)

Design bearing resistance R_d for ULS checks has been evaluated according to point 14.2.1.1.

The following assumptions have been considered in calculations:

- foundation level at 11.0, 1.5, 2, 2.5 and 3 m below FGL (assumed +5.8 m NAP);
- B' size equal to 1 m, 1.5 m, 2 m, 2.5 m, 3 m and 4 m;
- B'/L' ratios from 1 to 10 (strip foundation);
- horizontal loads equal to 10%, 20% and 30% of vertical load (including foundation and backfill), in both directions. <u>Specific calculations will need to be performed if load inclination exceeds those indicated above.</u>

Considering the general ground conditions encountered in the areas object of the present document, on the safe side the following calculation parameters for bearing capacity evaluations has been assumed, considering possible presence of loose silty sands/sandy silts at shallow depth:

- $\phi' = 30°$
- $\gamma = 18.5$ kN/m^3

The ground water level is taken at +1 m NAP.

The results of calculations in terms of design gross bearing capacity ($q_{d,gross}$) on the effective Meyerhof area (B' x L') <u>for failure checks</u> are reported in **Annex A.**

Values have to be compared with the design gross pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019.

16.1.2. Sliding checks (ULS checks)

Sliding checks shall be carried out assuming the friction angle between soil and concrete equal to:

$\delta = 2/3 \cdot \phi' = 20°$

to be factorized with the appropriate partial factors γ_{M2} according to Table 14.2. This leads to a design friction coefficient:

$\mu = 0.32$

The beneficial effect provided by lateral earth pressure on the passive side of foundation shall be neglected.

16.1.3. Settlements evaluation (SLS checks)

Settlements evaluations have been carried out in accordance with point 14.3.1.

The following assumptions have been considered in calculations:

- foundation level at 1.0, 1.5, 2, 2.5 and 3 m below FGL (assumed +5.8 m NAP);
- B' size equal to 1 m, 1.5 m, 2 m, 2.5 m, 3 m and 4 m;
- B'/L' ratios from 1 to 10 (strip foundation);
- Applied q_{gross} from 50 kPa to 250 kPa

The soil profile indicated in Table 16.1, which represent, on the safe side, typical ground conditions in the areas object of the present document, has been assumed in calculations. In the same Table 16.1 the geotechnical parameters used in the calculations are summarized.

The ground water level is taken at +1 m NAP.

The results of calculations in terms of expected settlements for each examined applied load (q_{gross}) for serviceability check are reported in **Annex A.**
It has to be noted that values in Annex A do not include areal displacements due to residual settlements for recent reclamation and site preparation activities (see Section 11).

Values have to be referred to the gross pressure for SLS checks

Table 16.1: Settlements evaluations – Reference calculation soil profile and parameters

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m^3)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	+0.0	Loose recent sand	18.5	-	-	-	8
+0.0	-9	Recent sand	19	-	-	-	$12+1.2\cdot z^{(1)}$
-9	-11	Clay to clayey silt	16	1.05	0.062	0.22	-
-11	-20.5	Recent sand	19	-	-	-	$12+1.2\cdot z^{(1)}$
-22	-23	Clay to clayey silt	16	1.05	0.062	0.22	-
-23	→	Pleistocene sand	19.5	-	-	-	150
(1) z = m from FGL							

16.2. Shallow foundation on soil improvement

As anticipated, soil improvement by means rigid inclusions may be a suitable solution, in place of structural piles, to improve foundation subgrade (bearing capacity) and reduce within acceptable limits the expected long term settlements of shallow foundations.

Considering the proposed soil improvement solution and intensity indicated at point 13.1, as a very preliminary estimate:

- values of pressures coming from soil failure criterion (ULS checks) are very high → 250 kPa can be assumed in the design

- expected settlements are limited to about 2 ÷ 2.5 cm, considering also the areal displacements due to residual settlements for recent reclamation and site preparation activities involving the layer below the rigid inclusions tip.

Any assessment of greater detail needs to know the actual characteristics of structures in the project and their exact location to evaluate the adequacy and the opportunity of using the proposed methods.

17. DESIGN OF PILED FOUNDATIONS

For the geotechnical design of piled foundations reference shall be made to JSD – Feasibility Phase – Job Design Specification for Piles – Refinery RDCG MNA (Ref.Doc. [1]).

18. TANK FOUNDATIONS

18.1. General

The following main tanks are foreseen in the area object of the present document, having the following geometrical characteristics:

- N° 2 Fire Water tanks D = 20.5 m and height H ~ 15 m
- N° 2 Intermediate Product tanks D = 34.3 m and height H ~ 16 m

Provided that possible fine grained levels that could be found at foundation level shall be excavated and substituted with a good quality well compacted fill, tanks may to be founded on direct foundations, with usual annular concrete ring filled with a good quality structural fill (refer to results of **Annex A** for ULS checks of ring annulus).

In case total and differential settlements estimated in the following are not compatible with the tolerable values, the opportunity of using a soil improvement (see Section **Error! Reference source not found.**) to improve foundation subgrade and limit the expected total and differential settlements of specific items, can be taken into account in place of structural piles.

Geotechnical evaluations reported in the following are carried out on the basis of specific soil profile considering the reference CPT listed below:

- Fire Water tanks D = 20.5 m → CU504, CU513, CU514 and CU523
- Intermediate Product tanks D = 34.3 m → CPT26, CPT 27, CU509 and CU5019

<u>Please note that currently available CPTs do not comply in terms of location (centre and perimeter) and number (min 5) to the National Code requirements for tanks settlements analyses. All the calculations presented in the following must be considered preliminary and they will be performed again for the next design phase, when the required investigation points will be executed.</u>

Soil profiles and relevant geotechnical parameters are reported in Table 18.1 to Table 18.8.

The groundwater has been assumed at +1.0 m NAP.

Table 18.1: Settlements evaluations – Reference calculation soil profile and parameters
Fire Water tanks – CU504

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m^3)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-10.5	Recent sand	19	-	-	-	$12+1.2 \cdot z^{(1)}$
-10.5	-11.5	Clay to clayey silt	16	1.05	0.062	0.22	-
-11.5	-21	Recent sand	19	-	-	-	$12+1.2 \cdot z^{(1)}$
-21	-23.5	Clay to clayey silt	16	1.05	0.062	0.22	-
-23.5	→	Pleistocene sand	19.5	-	-	-	150
(1) z = m from FGL							

Table 18.2: Settlements evaluations – Reference calculation soil profile and parameters
Fire Water tanks – CU513

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m^3)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-20.5	Recent sand	19	-	-	-	12+1.2·z[1]
-20.5	-22	Clay to clayey silt	16	1.05	0.062	0.22	-
-22	→	Pleistocene sand	19.5	-	-	-	150
[1] z = m from FGL							

Table 18.3: Settlements evaluations – Reference calculation soil profile and parameters
Fire Water tanks – CU514

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m^3)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-21	Recent sand	19	-	-	-	12+1.2·z[1]
-21	-23.5	Clay to clayey silt	16	1.05	0.062	0.22	-
-23.5	→	Pleistocene sand	19.5	-	-	-	150
[1] z = m from FGL							

Table 18.4: Settlements evaluations – Reference calculation soil profile and parameters
Fire Water tanks – CU523

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m^3)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	+2.5	Recent sand	19	-	-	-	12+1.2·z[1]
+2.5	0	Mixed soil	18	2	0.044	0.14	-
0	-11	Recent sand	19	-	-	-	12+1.2·z[1]
-11	-12	Clay to clayey silt	16	1.05	0.062	0.22	-
-12	-21.5	Recent sand	19	-	-	-	12+1.2·z[1]
-21.5	-24	Clay to clayey silt	16	1.05	0.062	0.22	-
-24	→	Pleistocene sand	19.5	-	-	-	150
[1] z = m from FGL							

Table 18.5: Settlements evaluations – Reference calculation soil profile and parameters
Intermediate Product tanks – CPT26

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-6.5	Recent sand	19	-	-	-	12+1.2·z(1)
-6.5	-8	Mixed soil	18	2	0.044	0.14	-
-8	-12	Recent sand	19	-	-	-	12+1.2·z(1)
-12	-13	Clay to clayey silt	16	1.05	0.062	0.22	-
-13	-14	Recent sand	19	-	-	-	12+1.2·z(1)
-14	-15	Clay to clayey silt	16	1.05	0.062	0.22	-
-15	-21	Recent sand	19	-	-	-	12+1.2·z(1)
-21	-23	Clay to clayey silt	16	1.05	0.062	0.22	-
-23	→	Pleistocene sand	19.5	-	-	-	150
(1) z = m from FGL							

Table 18.6: Settlements evaluations – Reference calculation soil profile and parameters
Intermediate Product tanks – CPT27

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-7.5	Recent sand	19	-	-	-	12+1.2·z(1)
-7.5	-13	Mixed soil	18	2	0.044	0.14	-
-13	-21	Recent sand	19	-	-	-	12+1.2·z(1)
-21	-23	Clay to clayey silt	16	1.05	0.062	0.22	-
-23	→	Pleistocene sand	19.5	-	-	-	150
(1) z = m from FGL							

Table 18.7: Settlements evaluations – Reference calculation soil profile and parameters
Intermediate Product tanks – CU509

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	21	Recent sand	19	-	-	-	12+1.2·z(1)
-21	-23	Clay to clayey silt	16	1.05	0.062	0.22	-
-23	→	Pleistocene sand	19.5	-	-	-	150
(1) z = m from FGL							

Table 18.8: Settlements evaluations – Reference calculation soil profile and parameters Intermediate Product tanks – CU519

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m^3)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-9.5	Recent sand	19	-	-	-	$12+1.2 \cdot z^{(1)}$
-9.5	-10	Clay to clayey silt	16	1.05	0.062	0.22	-
-10	-21.5	Recent sand	19	-	-	-	$12+1.2 \cdot z^{(1)}$
-21.5	-23	Clay to clayey silt	16	1.05	0.062	0.22	-
-23	→	Pleistocene sand	19.5	-	-	-	150
(1) z = m from FGL							

18.2. Settlements evaluation

The long term settlements of tanks under water test loads and their spatial distribution have been estimated considering the operational deformability parameters as discussed above and the elastic approach as formulated by Ahlvin and Ulery (1962) for the calculations of stress increment in the soil.

It has to be highlighted that:

- the soil profiles has been extended to a depth where the increment of stress due to the nominal average applied loads becomes lower than $0.10 \cdot \sigma_{vo}'$; below this depth it is considered that the induced settlements are negligible if compared to those that may develop in the overlaying layers;
- the maximum explored depths in the site investigations have been about 40 m; on the basis of the geology of the area it may be reasonably assumed that layers below 40 m mainly consist of very dense sand of similar characteristics as the ones found down to 40 m depth.

Loads

Conservatively, calculations have been carried out considering the loads applied during the hydro-test.

In this phase the expected total load at the base of the tank (bottom plate) is around 150 kPa applied uniformly to the whole tank area.

Results

The calculated long term settlements of tanks under water test loads are summarized in Table 18.9.

It has to be noted that values in Table 18.9 include areal displacements due to residual settlements for recent reclamation and site preparation activities.

The maximum differential settlements are also given in Table 18.9, according to the following relationship (see Figure 18.1):

- edge-to-centre = differential settlement between centre and edge/tank diameter;
- out-of-plane = differential settlement between two points on the edge/arc length;
- out-of-verticality = differential settlement between two diametrically opposed points on the edge/ tank diameter.

Table 18.9: Maximum estimated settlements of tanks under water test loads

Tank	CPTU	D	Total settlements		Centre-to-edge	Out-of-plane	Out-of-verticality
		(m)	centre	(%)	(%)	(%)	(%)
Fire Water	CU514	20.5	14.6	9.5	0.25	Not valuable	Not valuable
	CU523	20.5	27.5	16.9	0.51	Not valuable	Not valuable
	CU504	20.5	19.3	12.9	0.31	Not valuable	Not valuable
	CU513	20.5	13.9	8.7	0.25	Not valuable	Not valuable
Interm. Product	26	34.3	33.0	21.5	0.34	Not valuable	Not valuable
	27	34.3	26.7	18.0	0.25	Not valuable	Not valuable
	CU509	34.3	19.9	11.7	0.24	Not valuable	Not valuable
	CU519	34.3	23.2	13.7	0.28	Not valuable	Not valuable

The estimated total and differential settlements were compared with the limits generally recommended by the literature for tank. In particular, with reference to the tank diameter D and height H results:

- centre to edge
  according to PIP STE03020 δ_{MAX} =(1/100~1/50)D = (1%-2%)D
- out of plane
  according to PIP STE03020 δ_{MAX} = L/(350~450)
  where L indicates the circumference arch length between points considered;
  considering L1=32ft (~10m) δ_{MAX} = (0.2%-0.3%)L1;
  considering L2=1/8 (Dπ) δ_{MAX} = (0.2%-0.3%)L2.
- Out of verticality
  according to PIP STE03020 δ_{MAX}=(1/50)D = 2%D;
  according to API650 δ_{MAX}=(1/200)D = 0.5%D;
  according to API653 δ_{MAX}=(1/100)H = 1%H.

The predicted settlements calculated above have to be compared with the tolerable values which have been used for the geometric and structural design of the tanks themselves and of the connections.

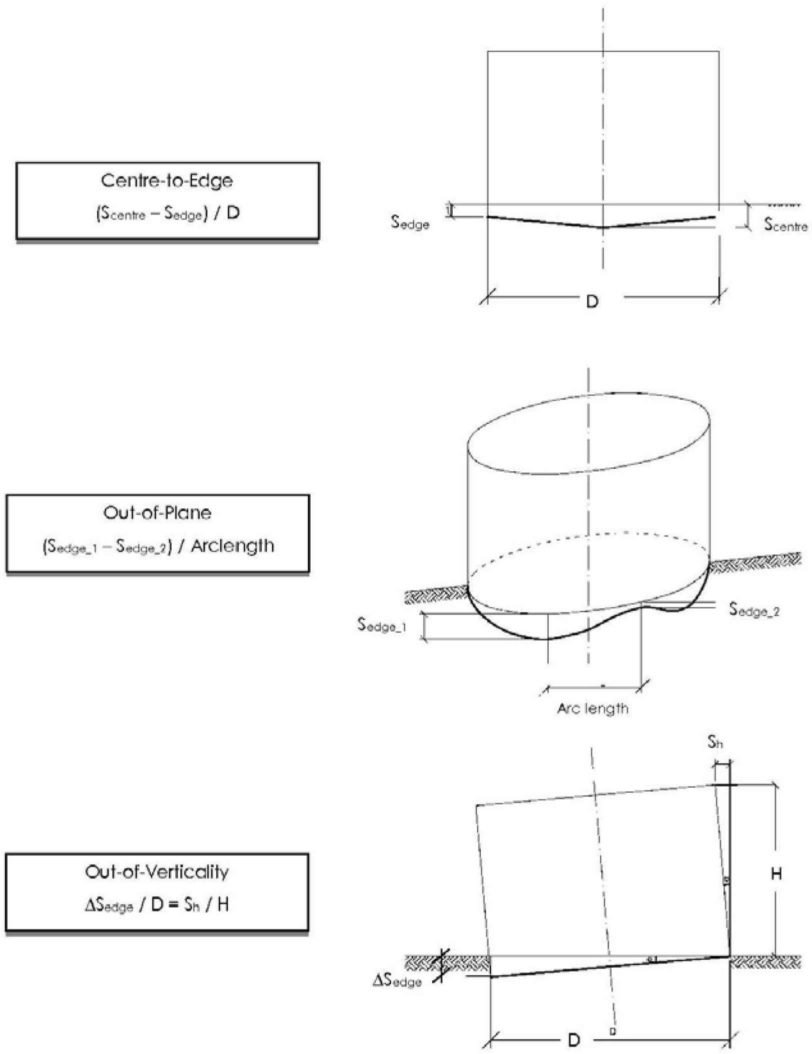


Figure 18.1

Concerning hydraulic tests, it is recommended that each tank is kept under full loading for a minimum of 30 days or such longer period as necessary that the daily increase in measured settlement does not exceed 2% of the total settlement from the start of test.

Table 18.10 and Table 18.11 show the estimated settlements during the hydraulic test and the residual ones after the hydraulic test in the hypothesis of maintaining the load 0 days, 30 days, 45 days and 60 days. The values refer to the expected settlements at the edge which will be those measured during the test and of interest for the evaluation of the connections.

It has to be noted that values in Table 18.10 and Table 18.11 include areal displacements due to residual settlements for recent reclamation and site preparation activities

If necessary, final piping connections could be made after the completion of the hydraulic test, to mitigate the effects of settlements of the perimeter during hydraulic test on the in-service performance of the connections.

Table 18.10: Expected total settlements at tank edge during hydraulic test

Tank	CPT	End of tank filling	Hydro test 30gg	Hydro test 45gg	Hydro test 60gg
		cm	cm	cm	cm
Fire Water	CU514	5.37	5.62	5.69	5.75
	CU523	7.26	9.16	9.73	10.13
	CU504	5.54	6.36	6.62	6.81
	CU513	5.49	5.76	5.84	5.90
Interm. Product	26	6.65	9.18	9.87	10.35
	27	7.35	8.08	8.24	8.36
	CU509	7.13	7.65	7.81	7.93
	CU519	7.25	8.83	9.16	9.36

Table 18.11: Expected residual settlements at tank edge after hydraulic test

Tank	CPTU	Hydro test 0gg	Hydro test 30gg	Hydro test 45gg	Hydro test 60gg	Unload/reload settlements in operative
		cm	cm	cm	cm	cm
Fire Water	CU514	4.1	3.9	3.8	3.7	0.9
	CU523	9.6	7.8	7.2	6.8	1.2
	CU504	7.4	6.6	6.3	6.1	0.9
	CU513	3.2	2.9	2.8	2.8	0.9
Interm. Product	26	14.9	12.3	11.7	11.2	1.1
	27	10.6	9.9	9.7	9.6	1.2
	CU509	4.5	4.0	3.9	3.7	1.2
	CU519	6.4	4.9	4.5	4.3	1.2

19. RECOMMENDATIONS FOR FILL MATERIAL AND PLACEMENT

19.1. General

Areas to receive fill should be scarified to an average depth of 500 mm, moisture conditioned to near the optimum water content and compacted to at least 95% of maximum dry density according to ASTM D 1557 (Modified Proctor test).

All fill material (included the fill placed below pavements) shall be spread in layers 150 to 200 mm thickness (loose material) and compacted using static roller and/or vibrating plates, reaching an in situ density of at least 95% of the maximum dry density as determined in Modified Proctor test according to ASTM D 1557. Special provisions shall apply to fill placed below foundations if any, to be determined on the basis of the specific case to be considered.

Positive drainage should be provided so that water and runoff will be diverted from the structures and no ponding occurs. All final grades in parking areas and adjacent to structures should be sloped to drain.

19.2. Material for fill

All fill and backfill should be free of organic matter, trash, chunks of high plasticity soils or other unsatisfactory materials.

In particular ordinary backfilling material shall be well graded granular material (sand and gravel) as class A1, A3, A2-4 and A2-5 of the AASHTO M 145, Classification System for Soil.

According to the available information excavated material will be generally classified as uniform (uniformity coefficient generally in the range 1.5 to 2.5) granular material as class A1-b of the AASHTO M 145, Classification System for Soil.

As a results of its uniform grading, the use as ordinary or structural fill of the material excavated from the site can give rise to the difficulties summarised below:

- Difficulty with trafficability. The uniform grading results in very low bearing capacity of the soil, even if compacted, when unconfined. As a result, normal wheeled construction equipment is not suitable to travel directly on this material. Access roads shall be constructed with well graded gravel at an early stage of the works;
- Difficulty with compaction. The uniform grading means that smaller soil particles are not available to fill the voids between larger ones. The absence of fines means that cohesion does not develop. As a result, the material is not very sensitive to unconfined compaction.
- The lack of cohesion and of particle interlock means that the soil could be lifted by the wind. However, it would be impractical to maintain wind barriers during the construction period. If high mobility due to wind erosion occurs, hindering construction activities, a well graded or cement stabilized working platform may be need to placed across the whole site;
- This type of material provides poor subgrade for pavements. A capping layer (say 50 – 60 cm thick) of stable material would need to be provided below the normal subbase.

Mechanical stabilization can be adopted for to correct the grading curve

19.2.1.1. Stabilization by correction of grain size distribution

The objective of mechanical stabilisation is to correct the grading curve of the soil so that a correct blend of particle sizes exists, the finer particles occupying the voids between coarser ones and the whole assembly benefiting from particle interlock, giving compacted soil apparent cohesion.

The amount of correcting fill to be used will need to be determined taking into account its grading and the grading of the original soil. The uniformity coefficient of the resulting “corrected” soil should be not less than 6 and preferably not less than 10.

Stabilisation by correction of grain size distribution can be carried out in all weather, subject to the normal constraints applicable to all earthworks operations, in particular control of moisture content in the soil to be compacted.

For site mixing, which typically proves more economic though it requires more control and supervision, construction would typically proceed as follows:

- Simultaneous supply of original material from site stockpile and of additional fill of specified grading; in practice, lorries will unload the fill in the right proportion in a chequered fashion;
- Simultaneous spreading of original soil and correction fill in layer of specified thickness (say 200 mm), paying special attention to reworking the material to obtain good mixing;
- Soil mixing by harrowing and turning if necessary, to achieve homogeneous composition;
- Wetting to moisture content suitable for optimum compaction;
- Final grading;
- Compaction.

There is no particular time constraint in which to complete these operations.

The exact construction sequence and the amount of correcting fill used should be confirmed by site trial. The resulting fill shall be compacted as specified.

Considering the large amount of corrective fill required, it is questionable whether such approach is technically justifiable.

20. RECOMMENDATIONS FOR TEMPORARY EXCAVATIONS

According to available information, the maximum perched groundwater level is encountered at a minimum depth of about 4 ÷ 5 m below final grade (+5.8 m NAP). Ground support will be necessary for excavations extending below this depth, together with engineered dewatering to prevent soil flow towards the excavation.

Subject to any more string requirement of applicable safety codes and regulations, considering on the safe side the presence of shallow loose silty sands/sandy silts/silts, a maximum temporary slopes inclination of 2H/1V can be adopted for unsupported excavations, to a maximum depth of 3 m, with the base of the excavation above the groundwater table.

For deeper and/or supported excavations specific analyses shall be carried out case by case taking into account the lateral earth pressures from the surrounding soil, groundwater level and relevant dewatering system, if needed, and possible surcharge loads.

In all cases excavated material shall be stockpiled at a sufficient distance from the edge of the excavation to prevent any overloading of the excavation wall. In practice the area around excavations shall be kept free for a distance not less than 1.5 times the depth of excavation.

ANNEX A
SHALLOW FOUNDATIONS

GROSS BEARING PRESSURE FOR ULS VERIFICATIONS

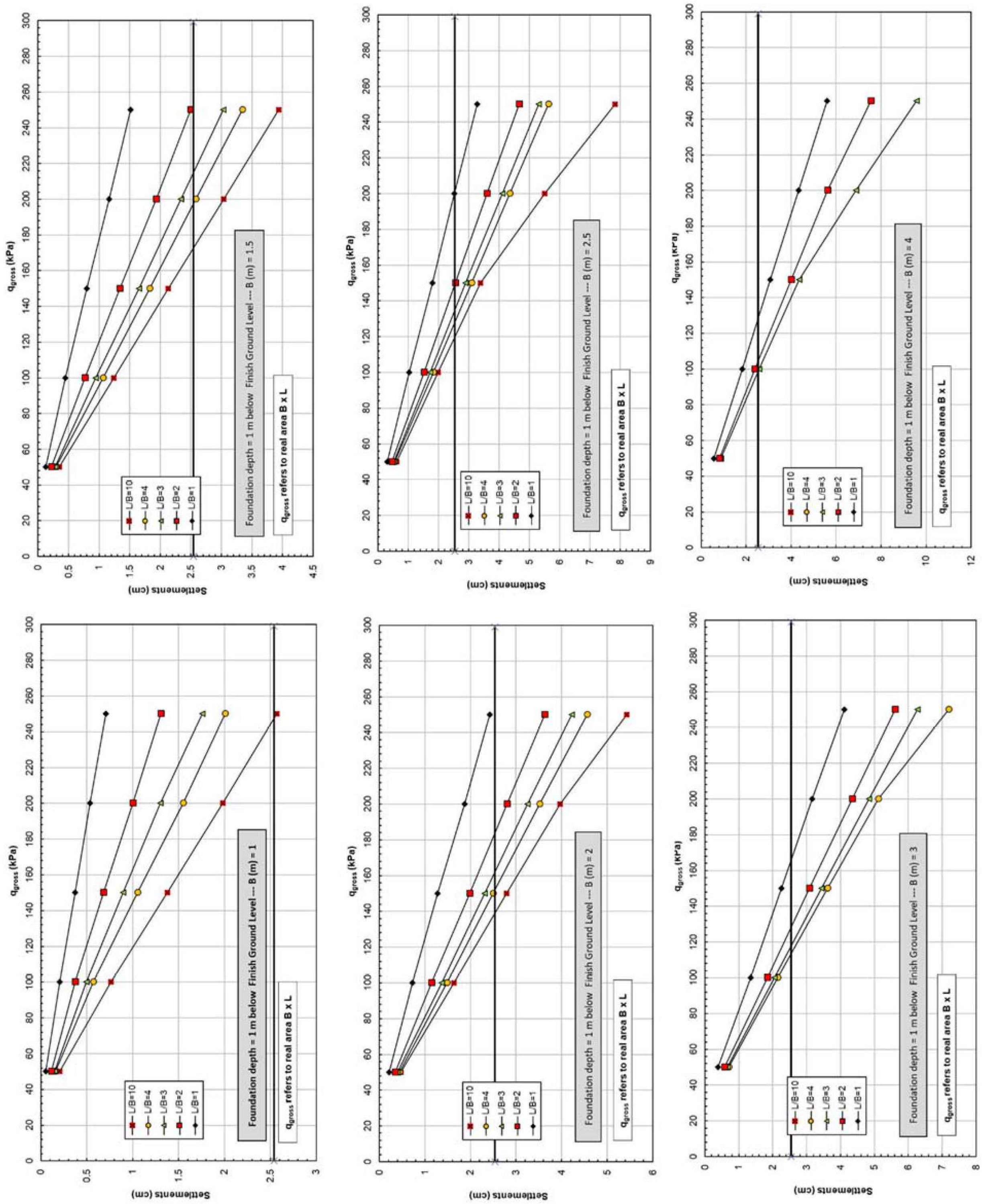
$q_{d,gross}$ (kPa)					
Foundation depth = 1 m below Finish Ground Level - H/N =0.1					
B’ (m)	**L’/B’=1**	**L’/B’=2**	**L’/B’=3**	**L’/B’=4**	**L’/B’=10**
1	295	270	260	255	245
1.5	325	300	295	290	280
2	350	330	325	320	315
2.5	375	360	360	355	350
3	400	390	390	390	
4	450	455	455		
Foundation depth = 1 m below Finish Ground Level - H/N =0.2					
B’ (m)	**L’/B’=1**	**L’/B’=2**	**L’/B’=3**	**L’/B’=4**	**L’/B’=10**
1	230	205	200	195	190
1.5	250	230	220	220	215
2	265	250	245	245	240
2.5	285	270	270	265	265
3	300	295	290	290	
4	335	335	335		
Foundation depth = 1 m below Finish Ground Level - H/N =0.3					
B’ (m)	**L’/B’=1**	**L’/B’=2**	**L’/B’=3**	**L’/B’=4**	**L’/B’=10**
1	175	155	150	145	140
1.5	185	170	165	160	155
2	200	185	180	180	175
2.5	210	200	195	195	190
3	220	215	210	210	
4	245	240	240		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2\cdot e_x$ and $L'=L-2\cdot e_y$					

Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

Values of pressures coming from soil failure criterion show very high values, the limit values of applied pressure will be controlled by the settlements criterion (see SLS verification in the following)

SETTLEMENTS EVALUATION FOR SLS VERIFICATIONS

Settlements (cm)					
Foundation depth = 1 m below Finish Ground Level - B = 1 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.05	0.11	0.15	0.16	0.21
100	0.20	0.37	0.50	0.57	0.77
150	0.37	0.68	0.89	1.05	1.38
200	0.53	1.00	1.30	1.55	1.98
250	0.71	1.31	1.76	2.01	2.57
Foundation depth = 1 m below Finish Ground Level - B = 1.5 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.13	0.22	0.28	0.29	0.35
100	0.44	0.76	0.94	1.07	1.24
150	0.80	1.34	1.65	1.82	2.13
200	1.16	1.93	2.34	2.58	3.04
250	1.52	2.50	3.03	3.34	3.94
Foundation depth = 1 m below Finish Ground Level - B = 2 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.22	0.35	0.40	0.44	0.47
100	0.73	1.14	1.37	1.48	1.63
150	1.28	1.98	2.31	2.49	2.79
200	1.87	2.81	3.26	3.52	3.97
250	2.42	3.63	4.22	4.56	5.43
Foundation depth = 1 m below Finish Ground Level - B = 2.5 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.31	0.47	0.52	0.56	0.62
100	1.03	1.52	1.73	1.83	1.98
150	1.80	2.56	2.90	3.09	3.39
200	2.53	3.61	4.10	4.35	5.51
250	3.28	4.67	5.30	5.64	7.82
Foundation depth = 1 m below Finish Ground Level - B = 3 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.39	0.58	0.67	0.71	-
100	1.35	1.84	2.05	2.14	-
150	2.26	3.08	3.43	3.62	-
200	3.17	4.34	4.84	5.11	-
250	4.11	5.61	6.26	7.19	-
Foundation depth = 1 m below Finish Ground Level - B = 4 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.57	0.81	0.87	-	-
100	1.84	2.38	2.58	-	-
150	3.08	3.99	4.35	-	-
200	4.33	5.62	6.88	-	-
250	5.59	7.55	9.58	-	-
q_{gross} refers to the real area B x L					



<u>Values do not include areal displacements due to residual settlements for recent reclamation and site preparation activities (see Section 11).</u>
Values have to be referred to the gross pressure for SLS checks.

GROSS BEARING PRESSURE FOR ULS VERIFICATIONS

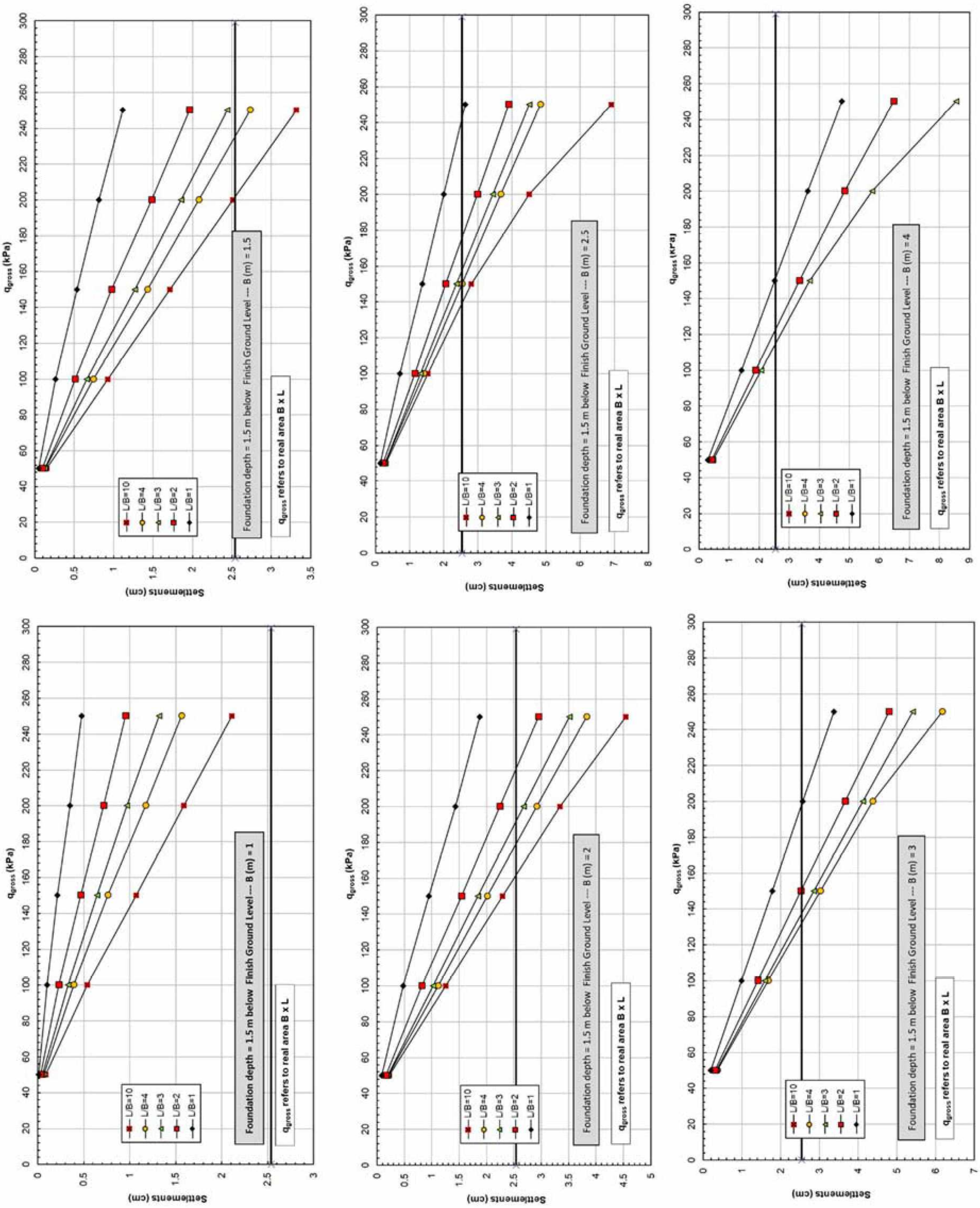
$q_{d,gross}$ (kPa)					
Foundation depth = 1.5 m below Finish Ground Level - H/N =0.1					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	420	375	360	350	335
1.5	445	405	390	385	370
2	470	435	425	420	405
2.5	500	465	455	450	440
3	525	500	490	485	
4	575	560	555		
Foundation depth = 1.5 m below Finish Ground Level - H/N =0.2					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	330	290	280	270	260
1.5	345	310	300	295	285
2	365	335	325	320	310
2.5	380	355	345	340	335
3	400	375	370	365	
4	435	420	415		
Foundation depth = 1.5 m below Finish Ground Level - H/N =0.3					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	250	220	210	205	195
1.5	260	235	225	220	210
2	275	250	240	235	230
2.5	285	265	255	250	245
3	300	280	270	270	
4	320	305	300		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2\cdot e_x$ and $L'=L-2\cdot e_y$					

Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

Values of pressures coming from soil failure criterion show very high values, the limit values of applied pressure will be controlled by the settlements criterion (see SLS verification in the following)

SETTLEMENTS EVALUATION FOR SLS VERIFICATIONS

Settlements (cm)					
Foundation depth = 1.5 m below Finish Ground Level - B = 1 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.01	0.03	0.05	0.06	0.08
100	0.10	0.23	0.33	0.39	0.54
150	0.21	0.47	0.64	0.76	1.07
200	0.35	0.71	0.97	1.17	1.59
250	0.47	0.95	1.32	1.56	2.11
Foundation depth = 1.5 m below Finish Ground Level - B = 1.5 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.05	0.10	0.12	0.13	0.15
100	0.26	0.51	0.66	0.74	0.92
150	0.53	0.97	1.27	1.43	1.71
200	0.81	1.48	1.85	2.08	2.51
250	1.11	1.96	2.44	2.73	3.32
Foundation depth = 1.5 m below Finish Ground Level - B = 2 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.08	0.16	0.20	0.20	0.23
100	0.48	0.82	1.02	1.11	1.25
150	0.94	1.54	1.84	2.01	2.29
200	1.43	2.24	2.68	2.92	3.34
250	1.88	2.95	3.51	3.83	4.54
Foundation depth = 1.5 m below Finish Ground Level - B = 2.5 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.14	0.22	0.27	0.28	0.30
100	0.72	1.15	1.32	1.41	1.53
150	1.37	2.05	2.37	2.53	2.81
200	2.00	2.98	3.44	3.67	4.50
250	2.63	3.90	4.50	4.83	6.92
Foundation depth = 1.5 m below Finish Ground Level - B = 3 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.18	0.29	0.33	0.36	-
100	0.98	1.41	1.59	1.67	-
150	1.77	2.52	2.85	3.02	-
200	2.57	3.66	4.12	4.36	-
250	3.36	4.79	5.40	6.16	-
Foundation depth = 1.5 m below Finish Ground Level - B = 4 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.30	0.42	0.46	-	-
100	1.41	1.88	2.06	-	-
150	2.51	3.34	3.68	-	-
200	3.62	4.84	5.76	-	-
250	4.75	6.47	8.55	-	-
q_{gross} refers to the real area B x L					



<u>Values do not include areal displacements due to residual settlements for recent reclamation and site preparation activities (see Section 11).</u>
Values have to be referred to the gross pressure for SLS checks.

GROSS BEARING PRESSURE FOR ULS VERIFICATIONS

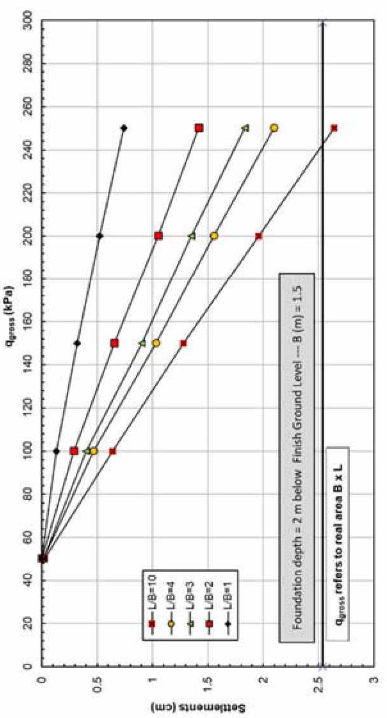
$q_{d,gross}$ (kPa)					
Foundation depth = 2 m below Finish Ground Level - H/N =0.1					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	545	480	460	445	425
1.5	570	510	490	480	460
2	595	540	525	515	495
2.5	620	570	555	545	530
3	650	605	590	580	
4	690	655	645		
Foundation depth = 2 m below Finish Ground Level - H/N =0.2					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	425	375	355	345	330
1.5	445	395	380	370	355
2	465	415	400	395	380
2.5	480	440	425	415	405
3	500	460	445	440	
4	530	495	485		
Foundation depth = 2 m below Finish Ground Level - H/N =0.3					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	325	285	270	265	250
1.5	340	300	285	280	265
2	350	315	300	295	285
2.5	365	330	315	310	300
3	375	340	330	325	
4	395	365	360		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2\cdot e_x$ and $L'=L-2\cdot e_y$					

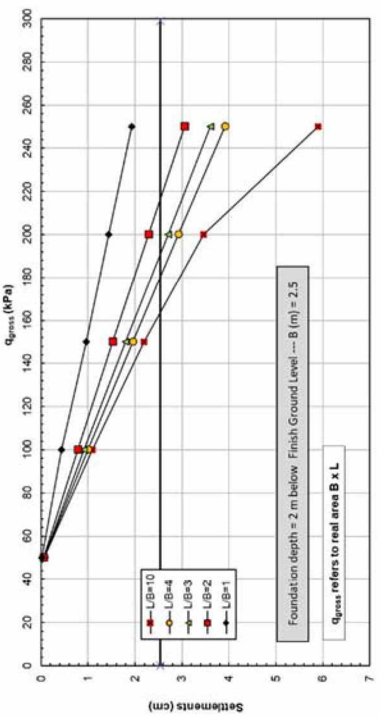
Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

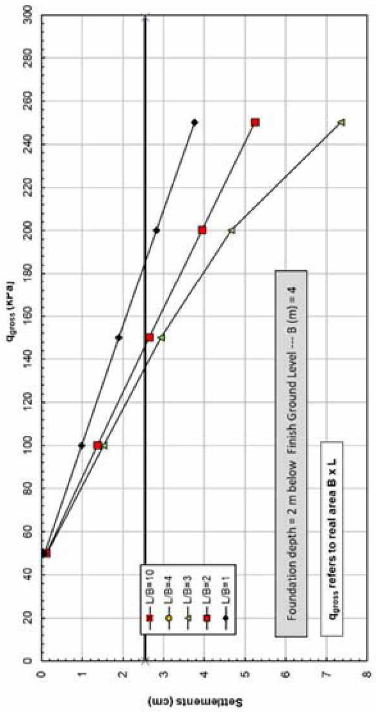
Values of pressures coming from soil failure criterion show very high values, the limit values of applied pressure will be controlled by the settlements criterion (see SLS verification in the following)

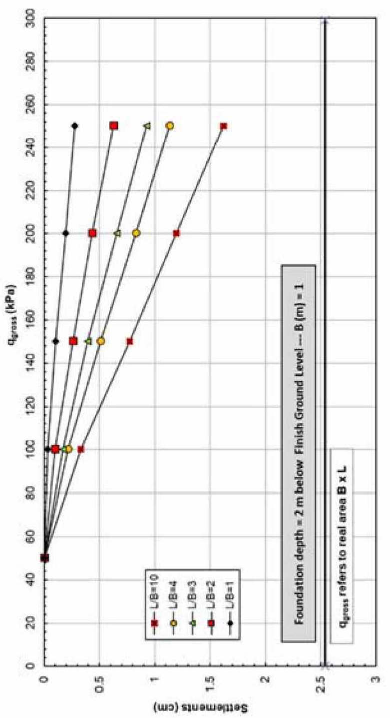
SETTLEMENTS EVALUATION FOR SLS VERIFICATIONS

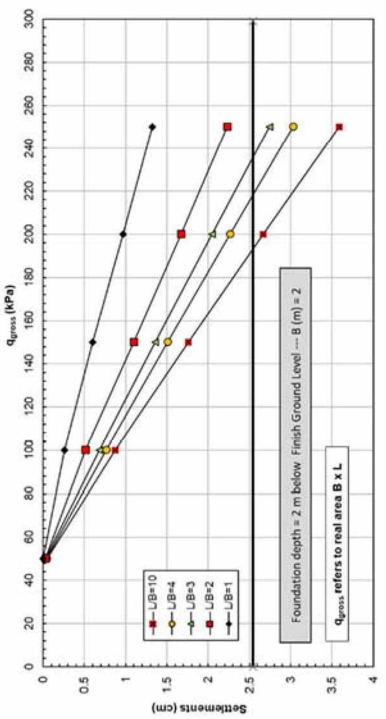
Settlements (cm)					
Foundation depth = 2 m below Finish Ground Level - B = 1 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.00	0.00	0.00	0.00	0.00
100	0.03	0.10	0.17	0.21	0.33
150	0.10	0.26	0.39	0.51	0.77
200	0.19	0.43	0.66	0.83	1.19
250	0.28	0.63	0.93	1.13	1.62
Foundation depth = 2 m below Finish Ground Level - B = 1.5 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.00	0.00	0.01	0.01	0.02
100	0.13	0.29	0.40	0.47	0.64
150	0.32	0.65	0.90	1.03	1.28
200	0.52	1.05	1.35	1.55	1.96
250	0.74	1.42	1.83	2.10	2.64
Foundation depth = 2 m below Finish Ground Level - B = 2 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.00	0.03	0.03	0.04	0.04
100	0.26	0.51	0.68	0.76	0.87
150	0.60	1.10	1.36	1.51	1.76
200	0.97	1.67	2.04	2.26	2.67
250	1.32	2.23	2.74	3.03	3.58
Foundation depth = 2 m below Finish Ground Level - B = 2.5 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.01	0.05	0.06	0.07	0.07
100	0.43	0.79	0.92	0.99	1.09
150	0.96	1.52	1.80	1.95	2.20
200	1.44	2.28	2.70	2.92	3.45
250	1.93	3.05	3.60	3.91	5.90
Foundation depth = 2 m below Finish Ground Level - B = 3 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.03	0.07	0.09	0.09	
100	0.61	0.99	1.14	1.21	
150	1.28	1.92	2.22	2.36	
200	1.91	2.87	3.31	3.54	
250	2.55	3.82	4.41	4.87	
Foundation depth = 2 m below Finish Ground Level - B = 4 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.07	0.12	0.14		
100	0.98	1.37	1.53		
150	1.89	2.64	2.94		
200	2.82	3.94	4.66		
250	3.76	5.24	7.35		
q_{gross} refers to the real area B x L					

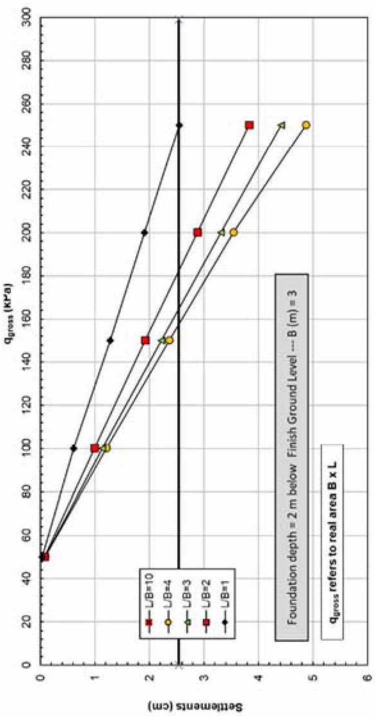












<u>Values do not include areal displacements due to residual settlements for recent reclamation and site preparation activities (see Section 11).</u>
Values have to be referred to the gross pressure for SLS checks.

GROSS BEARING PRESSURE FOR ULS VERIFICATIONS

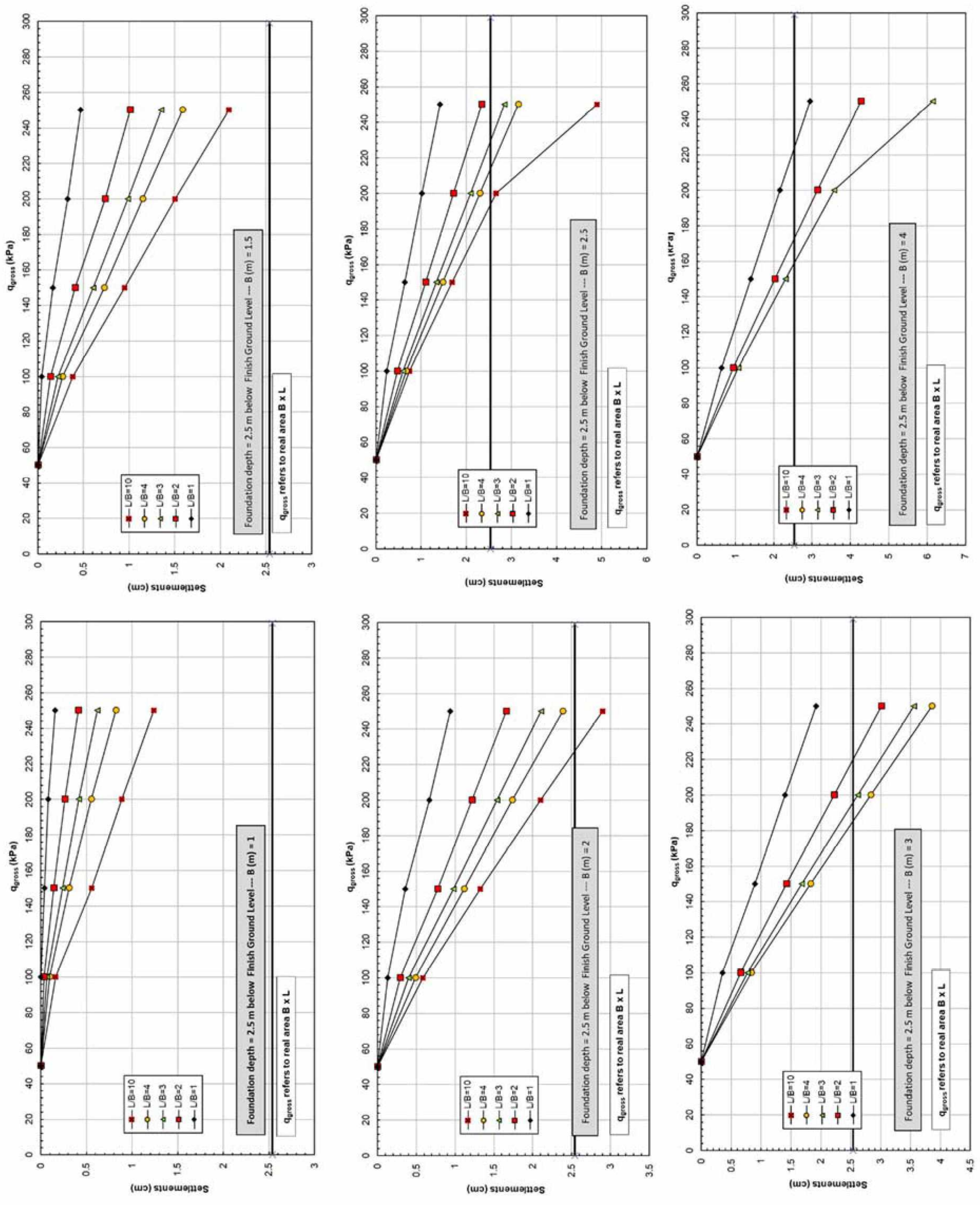
$q_{d,gross}$ (kPa)					
Foundation depth = 2.5 m below Finish Ground Level - H/N =0.1					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	670	585	555	545	515
1.5	695	615	590	575	550
2	720	645	620	610	585
2.5	745	680	655	645	620
3	770	705	685	675	
4	780	730	715		
Foundation depth =2.5 m below Finish Ground Level - H/N =0.2					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	525	455	435	425	400
1.5	545	480	455	445	425
2	560	500	480	470	450
2.5	580	520	505	495	475
3	595	540	525	515	
4	600	555	540		
Foundation depth = 2.5 m below Finish Ground Level - H/N =0.3					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	405	350	330	320	305
1.5	415	365	345	340	320
2	425	380	360	355	340
2.5	440	395	375	370	355
3	450	405	390	385	
4	450	415	400		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2 \cdot e_x$ and $L'=L-2 \cdot e_y$					

Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

Values of pressures coming from soil failure criterion show very high values, the limit values of applied pressure will be controlled by the settlements criterion (see SLS verification in the following)

SETTLEMENTS EVALUATION FOR SLS VERIFICATIONS

Settlements (cm)					
Foundation depth = 2.5 m below Finish Ground Level - B = 1 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.00	0.04	0.07	0.10	0.16
150	0.04	0.14	0.24	0.31	0.56
200	0.08	0.27	0.42	0.55	0.89
250	0.15	0.41	0.62	0.82	1.24
Foundation depth = 2.5 m below Finish Ground Level - B = 1.5 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.04	0.14	0.22	0.27	0.38
150	0.16	0.40	0.60	0.73	0.95
200	0.33	0.74	0.98	1.15	1.50
250	0.47	1.01	1.35	1.59	2.09
Foundation depth = 2.5 m below Finish Ground Level - B = 2 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.13	0.29	0.40	0.49	0.58
150	0.36	0.77	0.98	1.11	1.32
200	0.67	1.21	1.54	1.74	2.10
250	0.94	1.66	2.11	2.38	2.90
Foundation depth = 2.5 m below Finish Ground Level - B = 2.5 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.23	0.47	0.60	0.66	0.74
150	0.64	1.10	1.34	1.48	1.69
200	1.02	1.72	2.09	2.30	2.66
250	1.42	2.34	2.84	3.15	4.90
Foundation depth = 2.5 m below Finish Ground Level - B = 3 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	
100	0.36	0.66	0.77	0.83	
150	0.89	1.42	1.68	1.82	
200	1.40	2.22	2.61	2.83	
250	1.92	3.01	3.55	3.86	
Foundation depth = 2.5 m below Finish Ground Level - B = 4 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00		
100	0.64	0.94	1.07		
150	1.39	2.03	2.31		
200	2.16	3.14	3.57		
250	2.95	4.27	6.15		
q_{gross} refers to the real area B x L					



<u>Values do not include areal displacements due to residual settlements for recent reclamation and site preparation activities (see Section 11).</u>
Values have to be referred to the gross pressure for SLS checks.

GROSS BEARING PRESSURE FOR ULS VERIFICATIONS

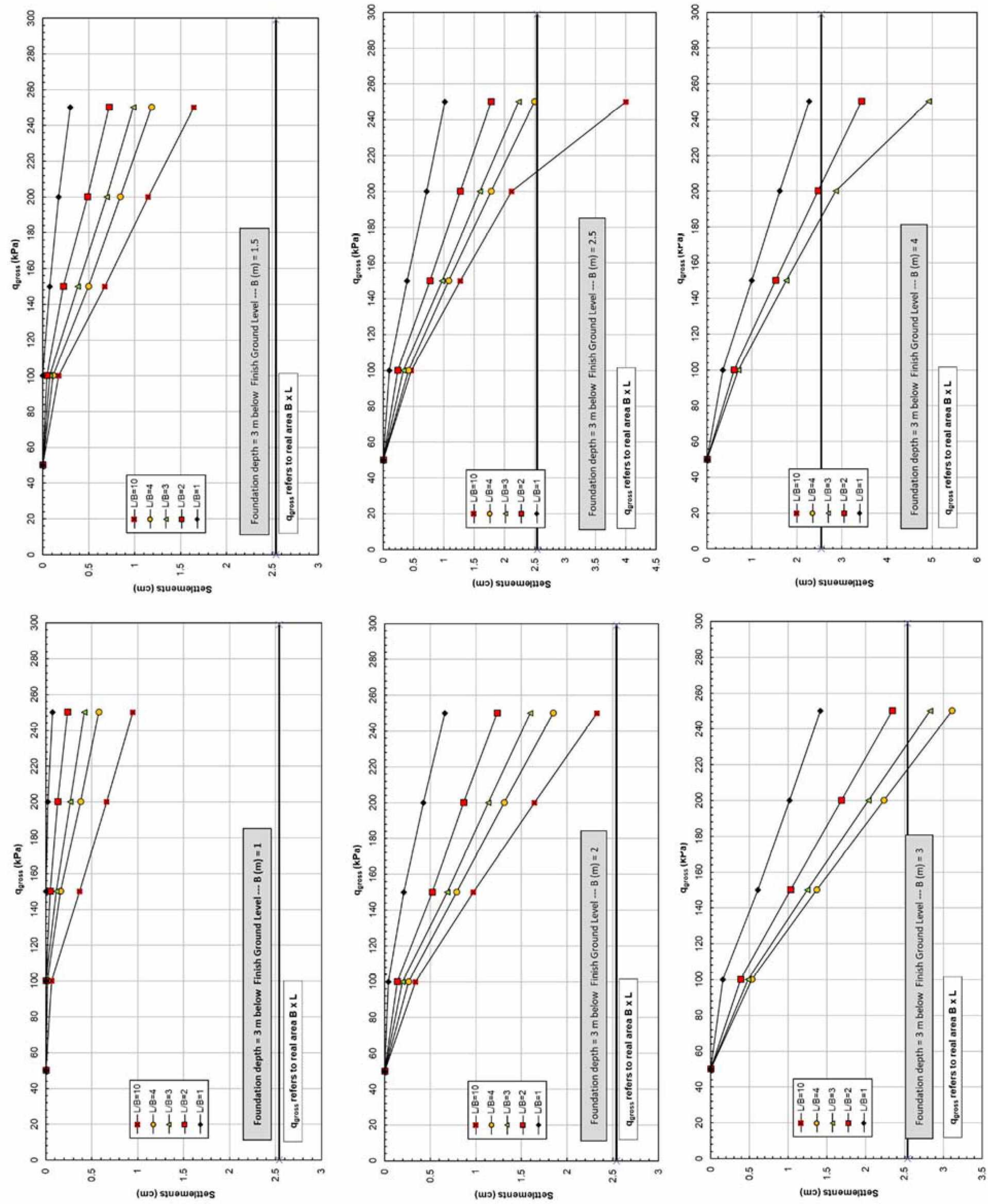
$q_{d,gross}$ (kPa)					
Foundation depth = 3 m below Finish Ground Level - H/N =0.1					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	795	690	655	640	605
1.5	820	720	690	670	640
2	845	750	720	705	675
2.5	865	775	750	735	705
3	860	780	755	740	
4	835	770	750		
Foundation depth = 3 m below Finish Ground Level - H/N =0.2					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	625	540	510	500	475
1.5	640	560	535	520	500
2	660	585	560	545	520
2.5	675	600	575	565	545
3	665	600	580	570	
4	645	590	575		
Foundation depth = 3 m below Finish Ground Level - H/N =0.3					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	480	415	390	380	360
1.5	490	430	405	395	380
2	505	445	420	410	395
2.5	510	455	435	425	405
3	505	455	435	425	
4	485	440	425		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2 \cdot e_x$ and $L'=L-2 \cdot e_y$					

Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

Values of pressures coming from soil failure criterion show very high values, the limit values of applied pressure will be controlled by the settlements criterion (see SLS verification in the following)

SETTLEMENTS EVALUATION FOR SLS VERIFICATIONS

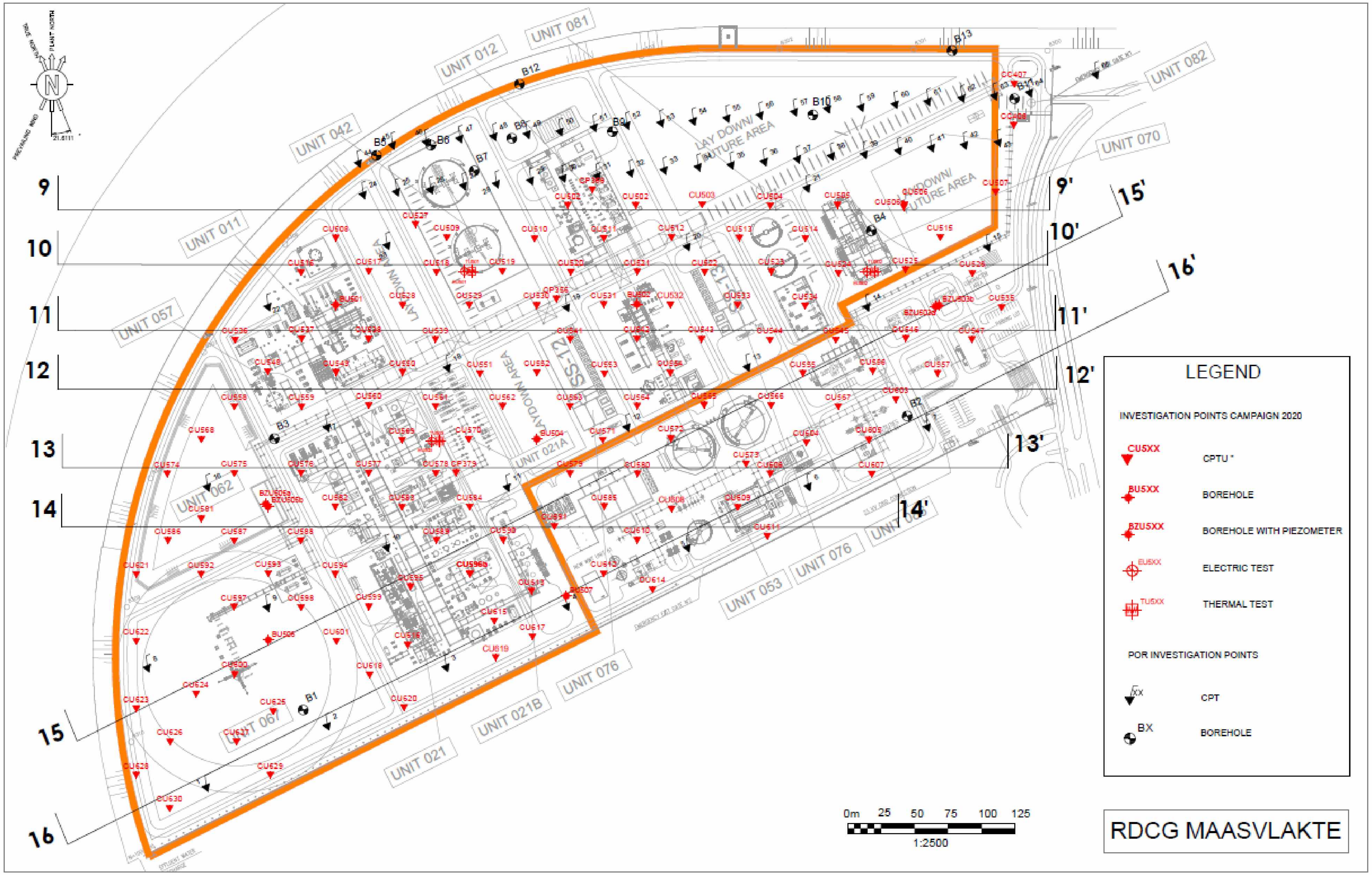
Settlements (cm)					
Foundation depth = 3 m below Finish Ground Level - B = 1 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.00	0.00	0.00	0.02	0.06
150	0.00	0.05	0.11	0.16	0.37
200	0.02	0.13	0.26	0.38	0.66
250	0.07	0.23	0.42	0.57	0.95
Foundation depth = 3 m below Finish Ground Level - B = 1.5 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.00	0.04	0.08	0.11	0.17
150	0.08	0.22	0.38	0.50	0.67
200	0.17	0.48	0.69	0.84	1.15
250	0.30	0.72	0.98	1.18	1.64
Foundation depth = 3 m below Finish Ground Level - B = 2 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.04	0.14	0.19	0.26	0.34
150	0.21	0.52	0.68	0.79	0.97
200	0.42	0.87	1.14	1.31	1.64
250	0.66	1.23	1.60	1.85	2.33
Foundation depth = 3 m below Finish Ground Level - B = 2.5 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	0.00
100	0.10	0.24	0.34	0.41	0.46
150	0.40	0.77	0.97	1.08	1.27
200	0.72	1.27	1.59	1.78	2.11
250	1.02	1.78	2.23	2.49	4.00
Foundation depth = 3 m below Finish Ground Level - B = 3 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00	0.00	
100	0.16	0.39	0.48	0.53	
150	0.61	1.03	1.24	1.36	
200	1.01	1.68	2.03	2.23	
250	1.42	2.34	2.83	3.11	
Foundation depth = 3 m below Finish Ground Level - B = 4 (m)					
$q_{d,gross}$ (kPa)	L/B=1	L/B=2	L/B=3	L/B=4	L/B=10
50	0.00	0.00	0.00		
100	0.35	0.61	0.70		
150	1.00	1.52	1.76		
200	1.62	2.47	2.86		
250	2.27	3.43	4.92		
q_{gross} refers to the real area B x L					



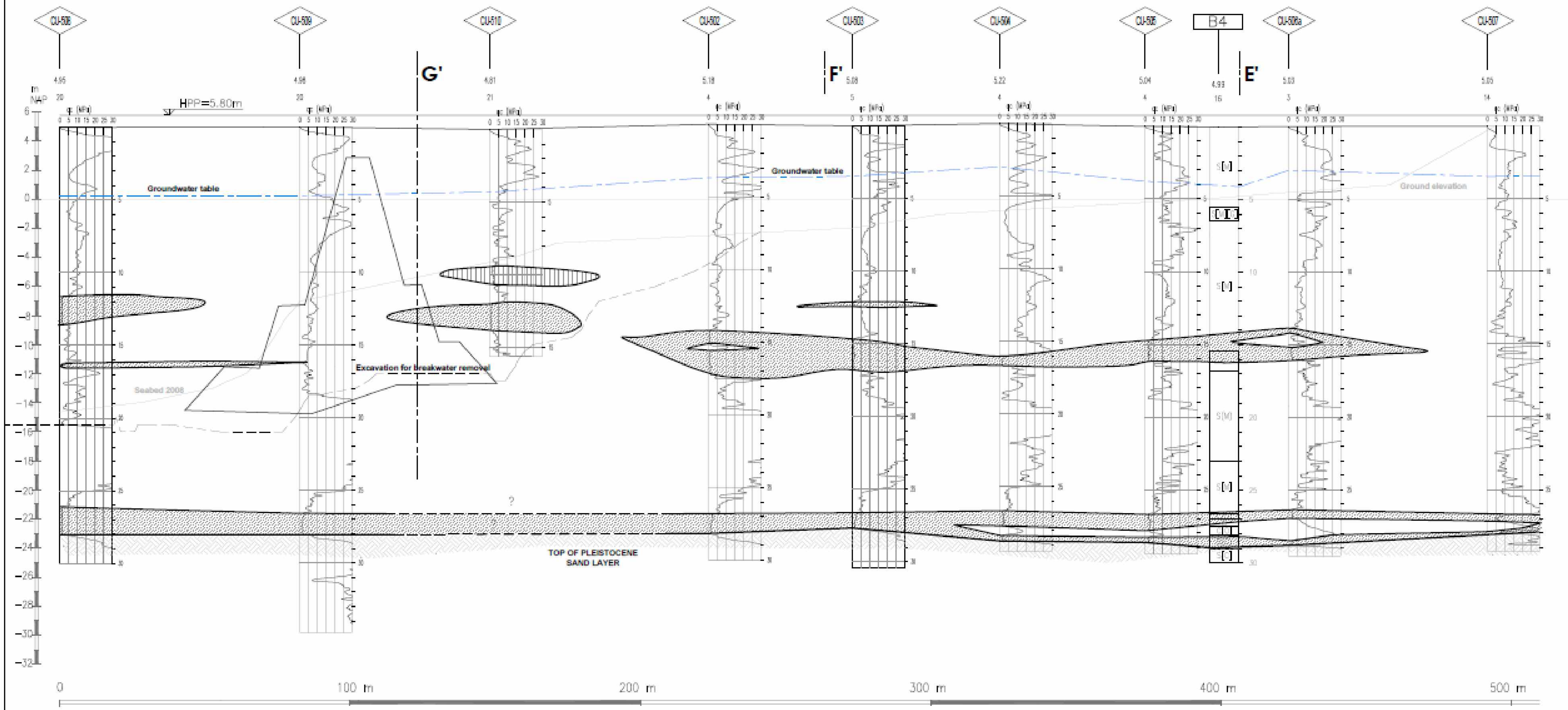
<u>**Values do not include areal displacements due to residual settlements for recent reclamation and site preparation activities (see Section 11).**</u>

Values have to be referred to the gross pressure for SLS checks.

ATTACHMENT 1
Ground Investigation Location Map

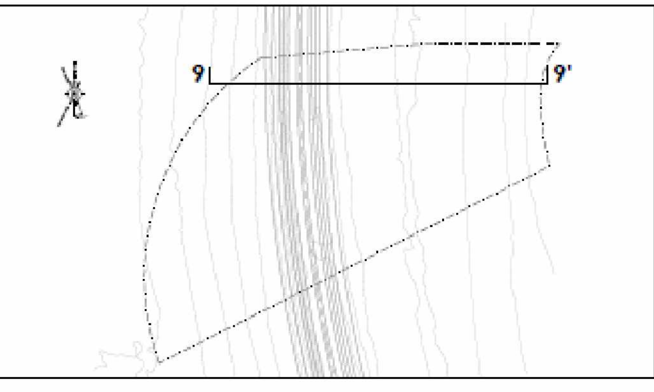

LEGEND
INVESTIGATION POINTS CAMPAIGN 2020
CU5XX CPTU *
BU5XX BOREHOLE
BZU5XX BOREHOLE WITH PIEZOMETER
EU5XX ELECTRIC TEST
TU5XX THERMAL TEST
POR INVESTIGATION POINTS
XX CPT
BX BOREHOLE
0m 25 50 75 100 125
1:2500
RDCG MAASVLAKTE
UNIT 081
UNIT 012
UNIT 082
UNIT 042
UNIT 070
UNIT 011
UNIT 057
UNIT 062
UNIT 021A
UNIT 067
UNIT 021
UNIT 021B
UNIT 076
UNIT 053
LAY DOWN/ FUTURE AREA
LAYDOWN AREA
9
9'
10
10'
11
11'
12
12'
13
13'
14
14'
15
15'
16
16'

ATTACHMENT 2
Geotechnical Sections



LEGEND

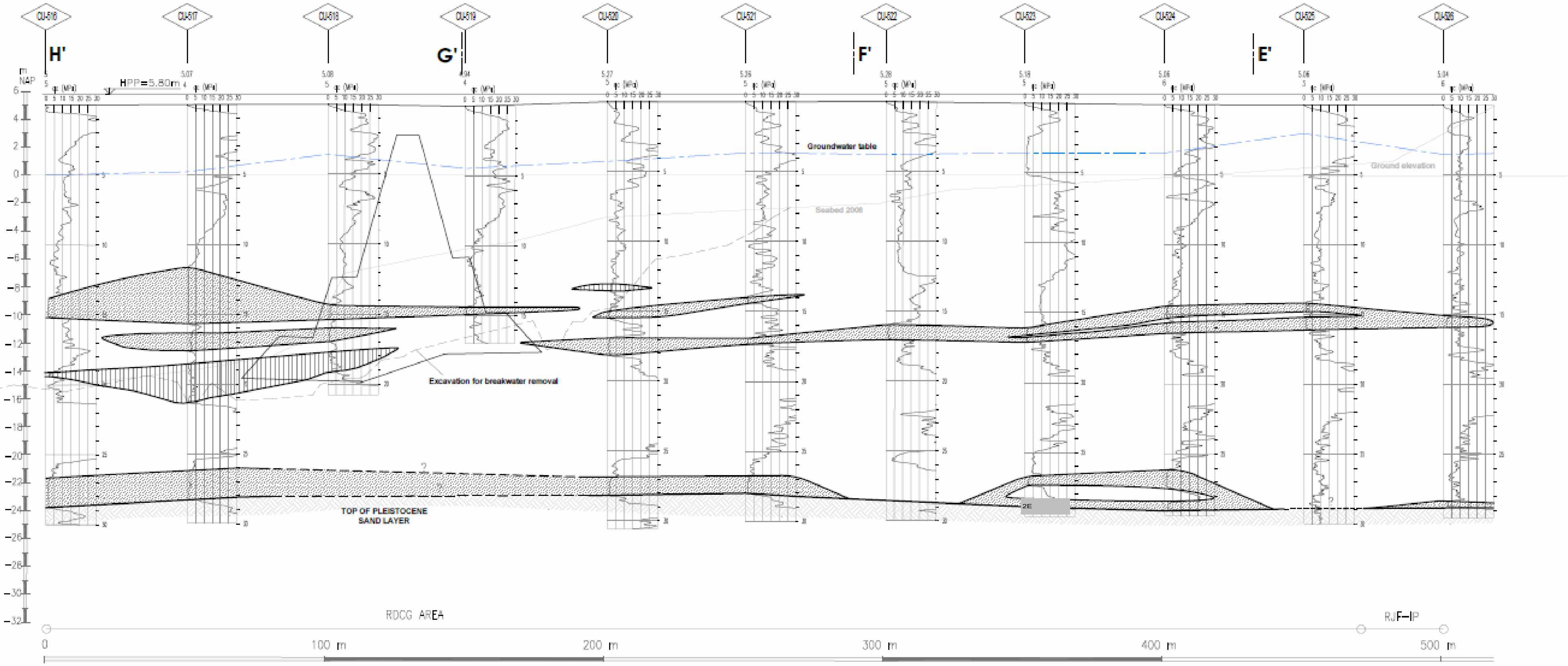
	Recent sand		Pleistocene sand
	Clay		Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

SECTION 9-9'

HORIZ. 1:1250 VERT. 1:250



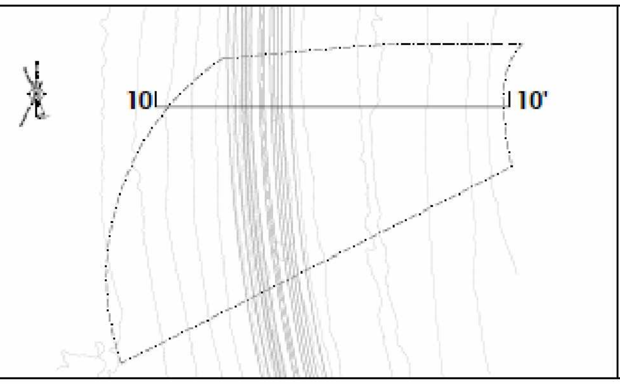
LEGEND

Recent sand

Clay

Pleistocene sand

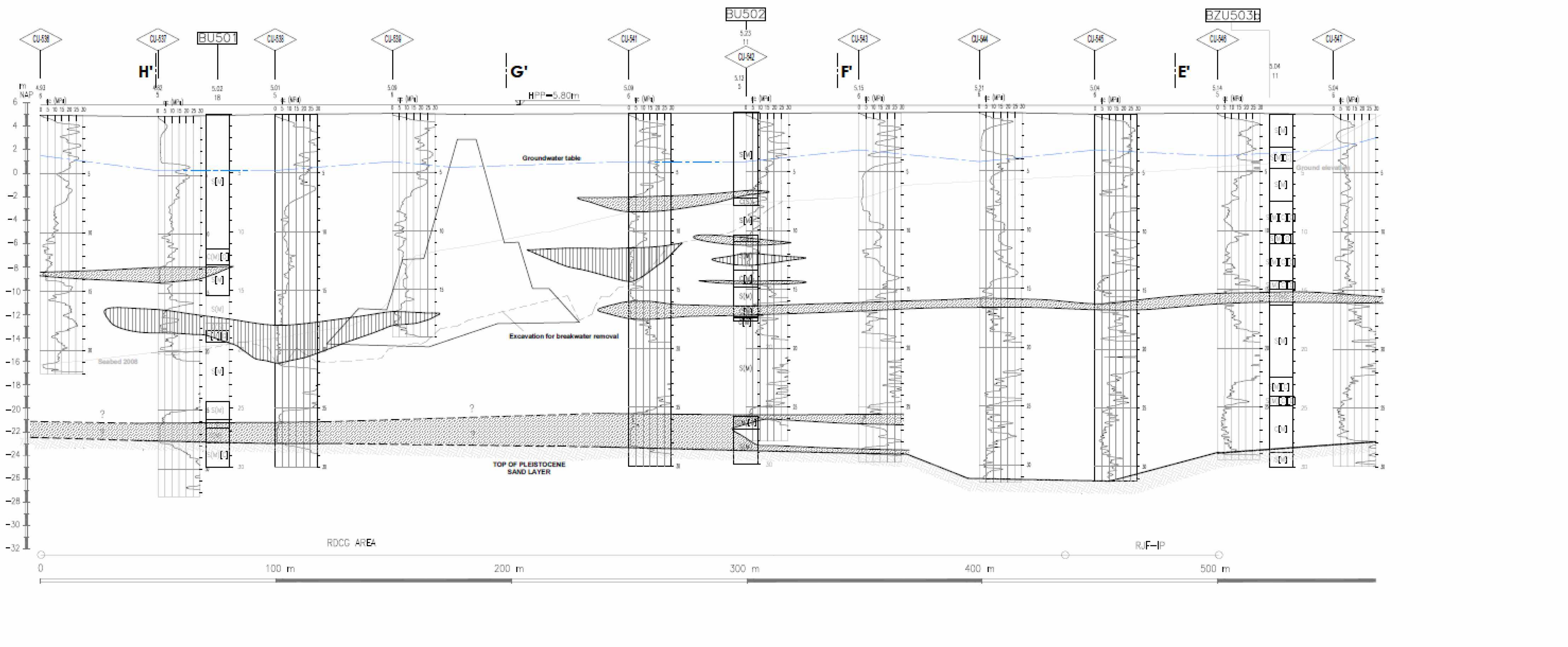
Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

SECTION 10-10'

HORIZ. 1:1250 VERT. 1:250



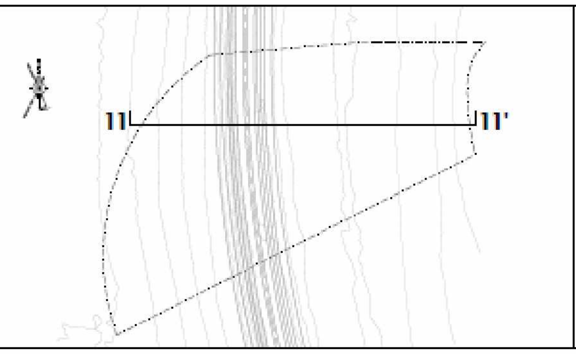
LEGEND

Recent sand

Clay

Pleistocene sand

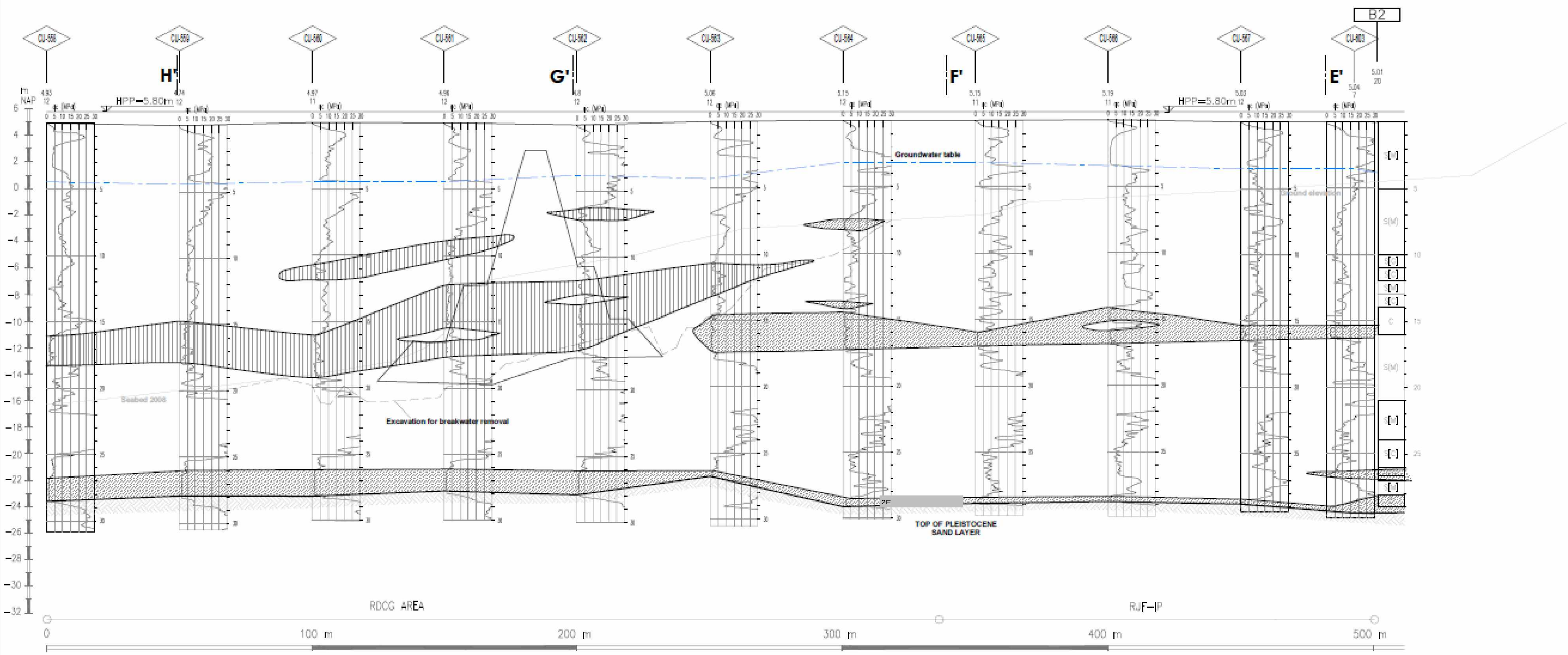
Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

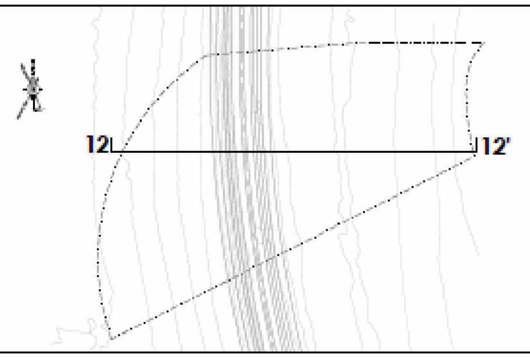
SECTION 11-11'

HORIZ. 1:1250 VERT. 1:250



LEGEND

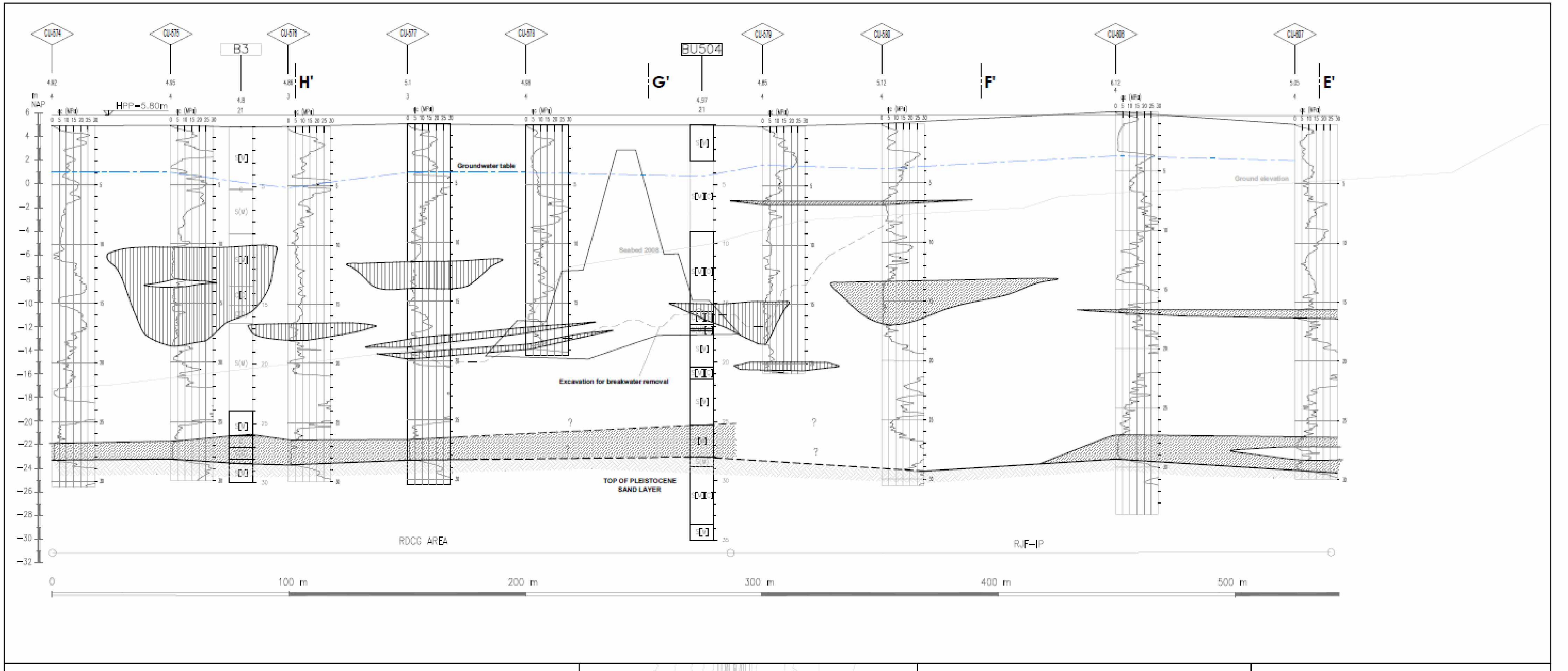
- Recent sand
- Pleistocene sand
- Clay
- Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

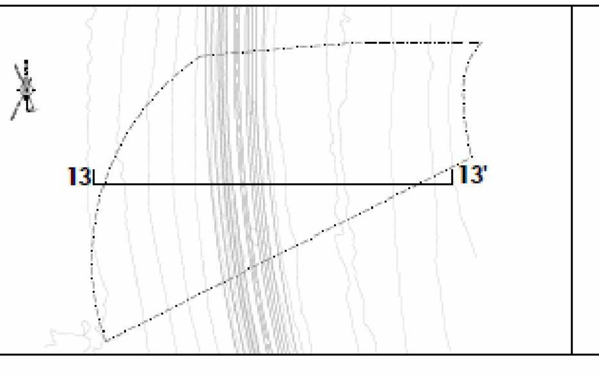
SECTION 12-12'

HORIZ. 1:1250 VERT. 1:250



LEGEND

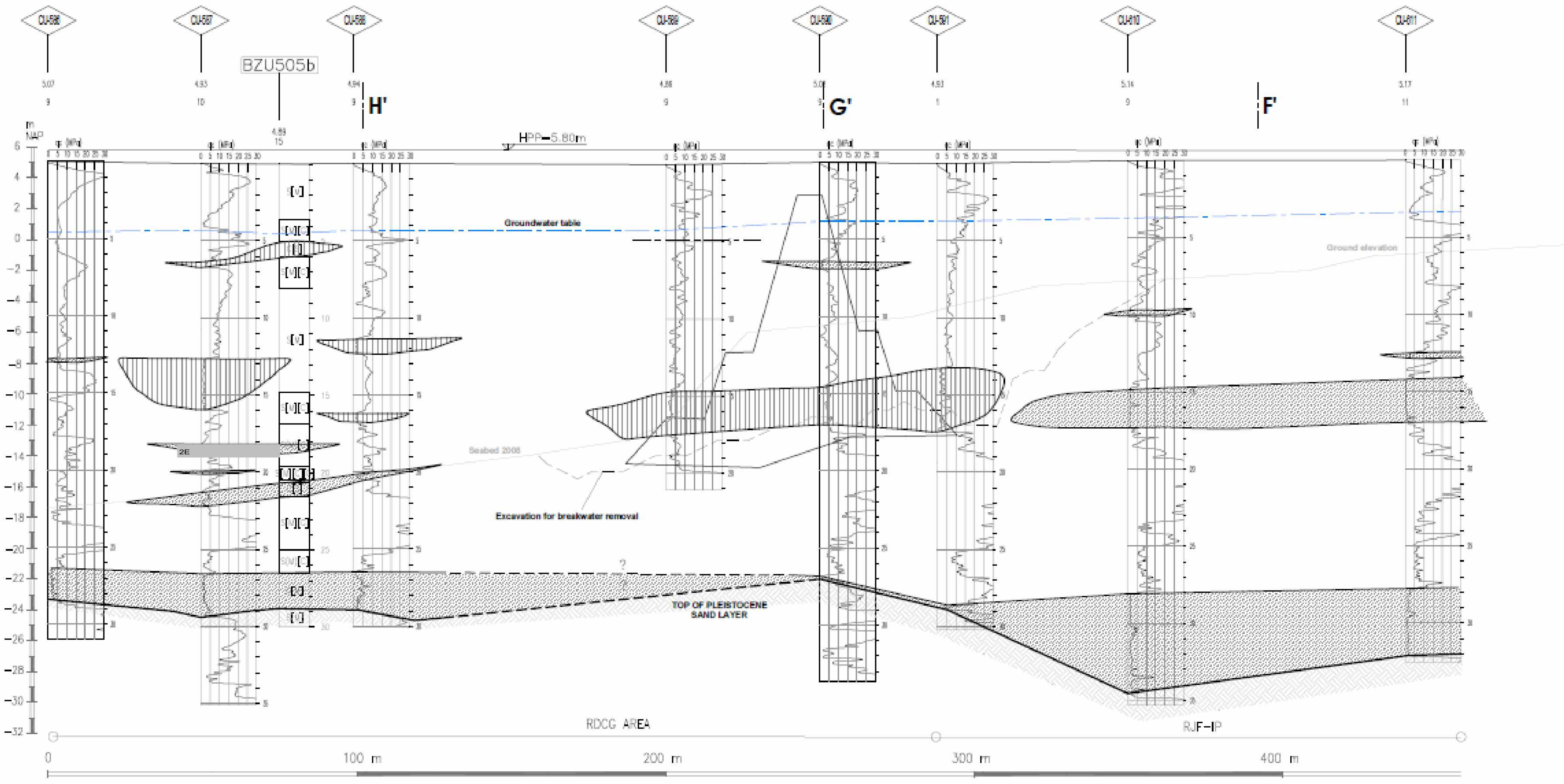
Recent sand
Pleistocene sand
Clay
Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

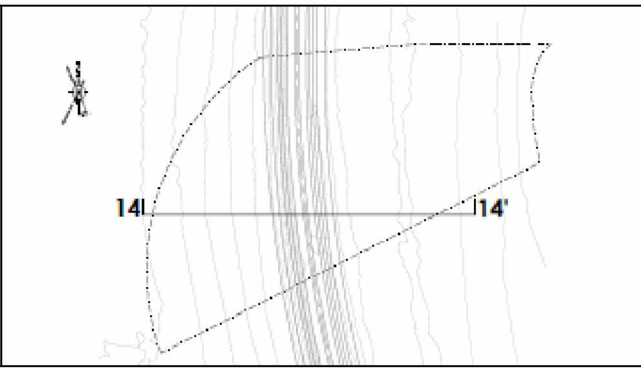
SECTION 13-13'

HORIZ. 1:1250 VERT. 1:250



LEGEND

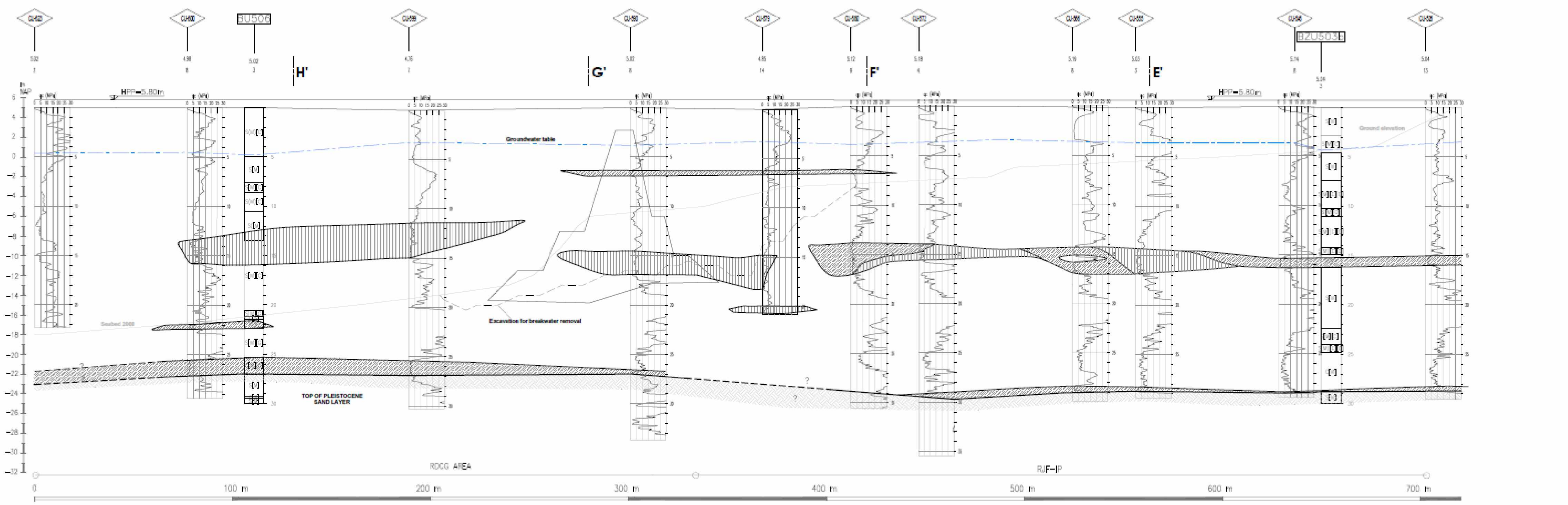
- Recent sand
- Clay
- Pleistocene sand
- Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

SECTION 14-14'

HORIZ. 1:1250 VERT. 1:250



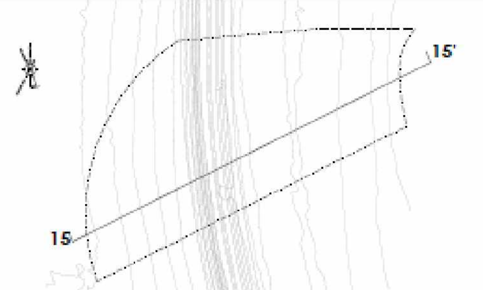
LEGEND

Recent sand

Clay

Pleistocene sand

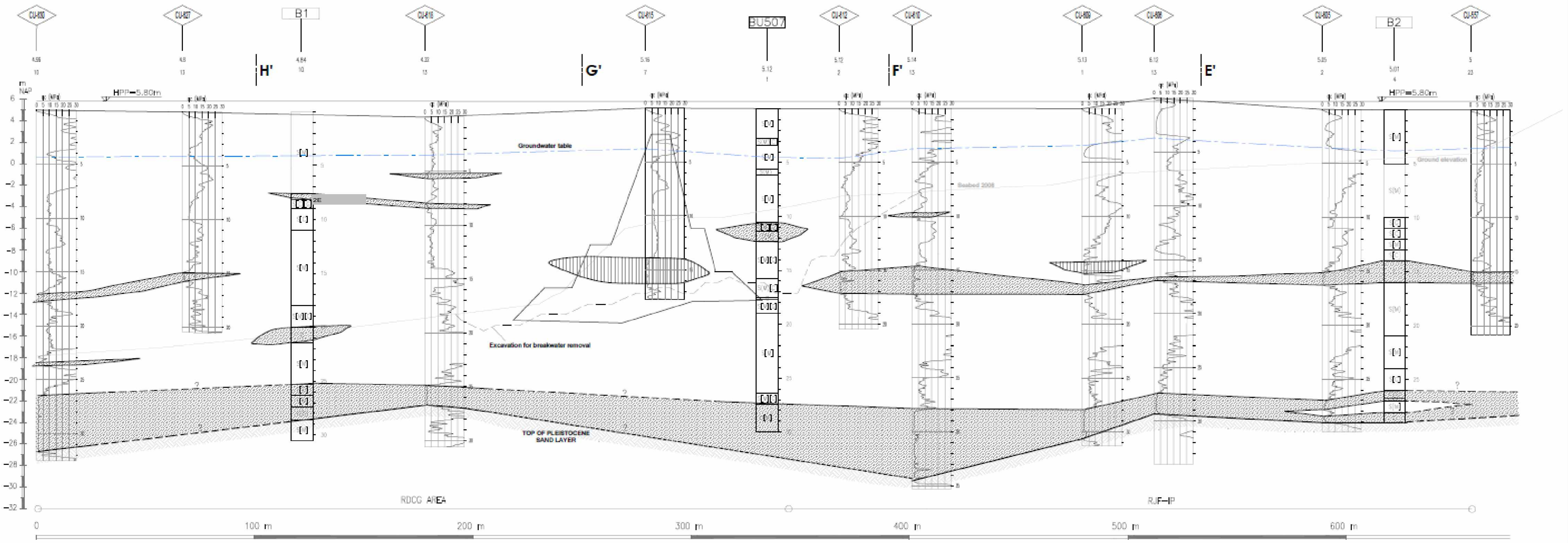
Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

SECTION 15-15'

HORIZ. 1:1250 VERT. 1:250



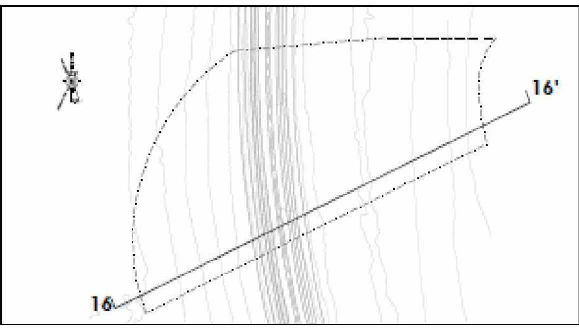
LEGEND

Recent sand

Pleistocene sand

Clay

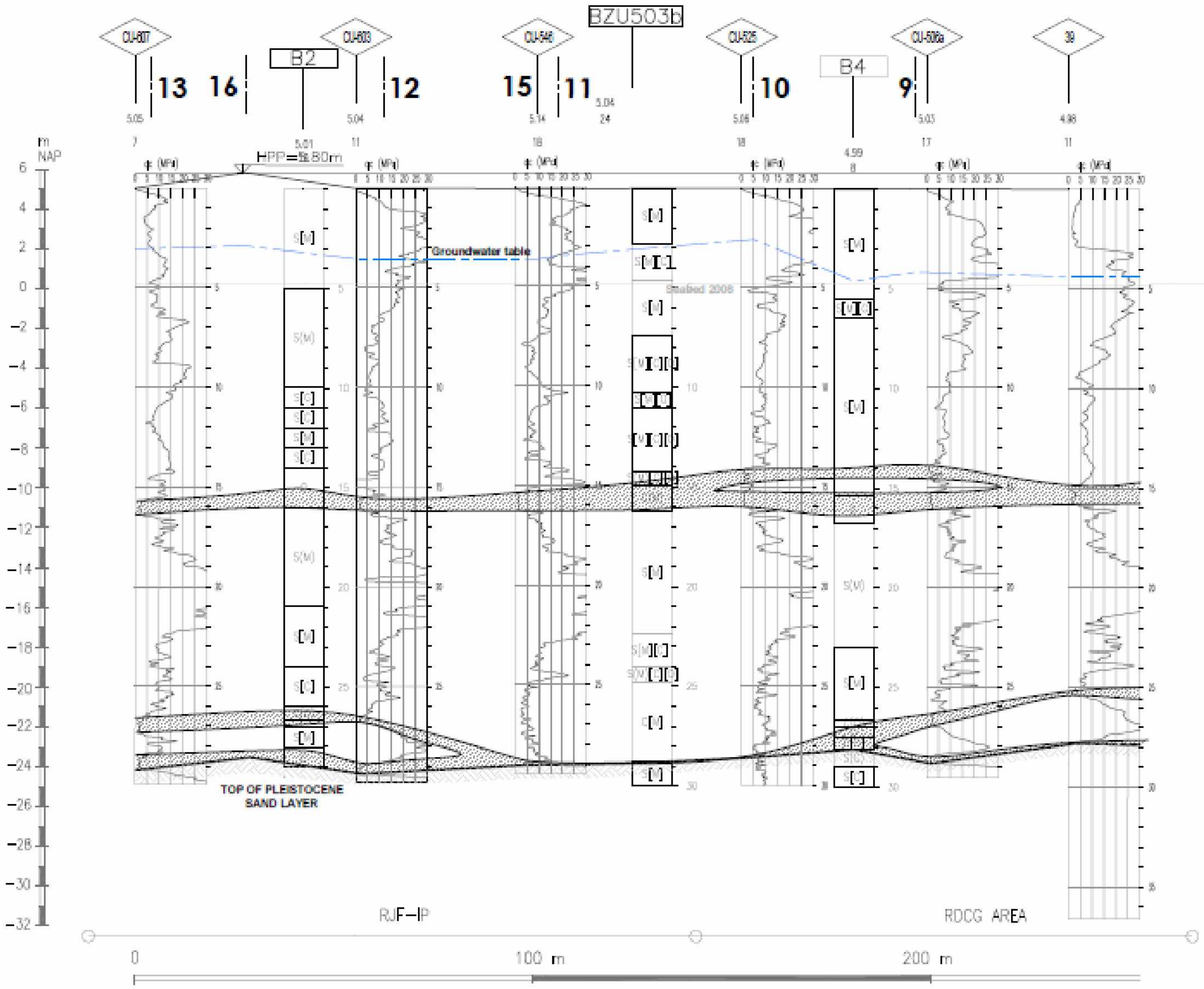
Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

SECTION 16-16'

HORIZ. 1:1250 VERT. 1:250



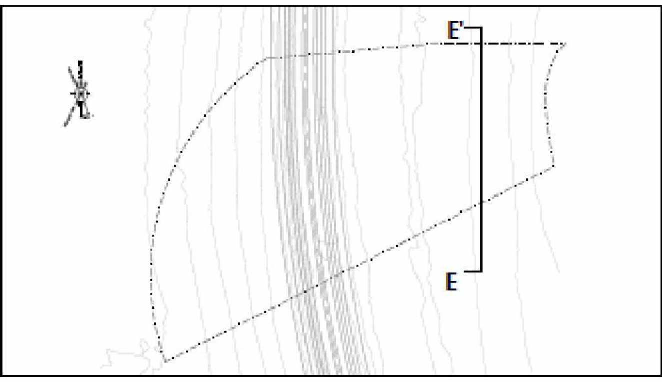
LEGEND

Recent sand

Pleistocene sand

Clay

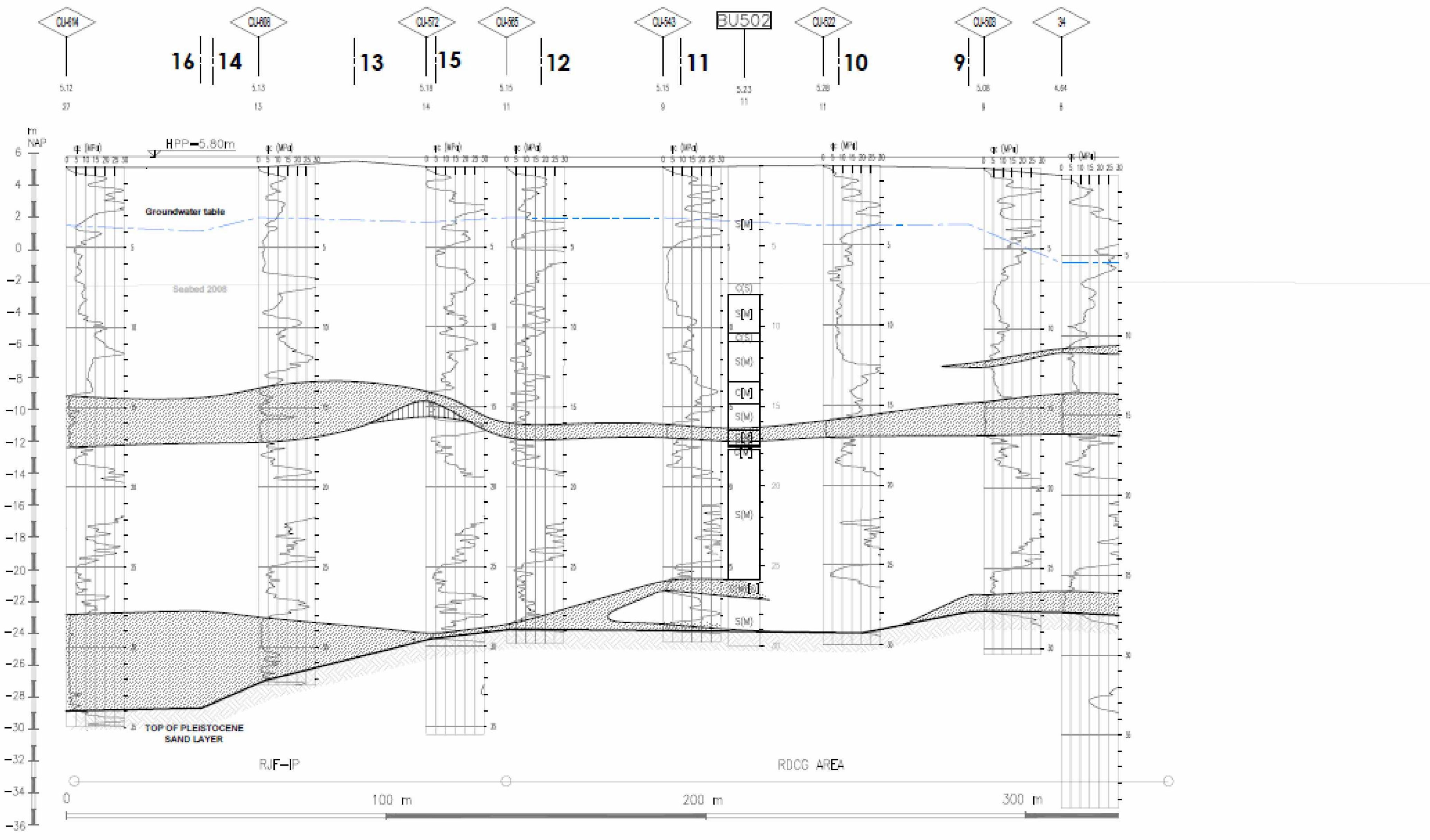
Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

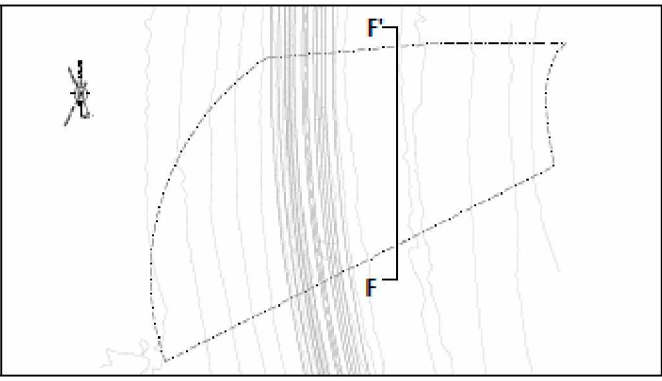
SECTION E-E'

HORIZ. 1:1250 VERT. 1:250



LEGEND

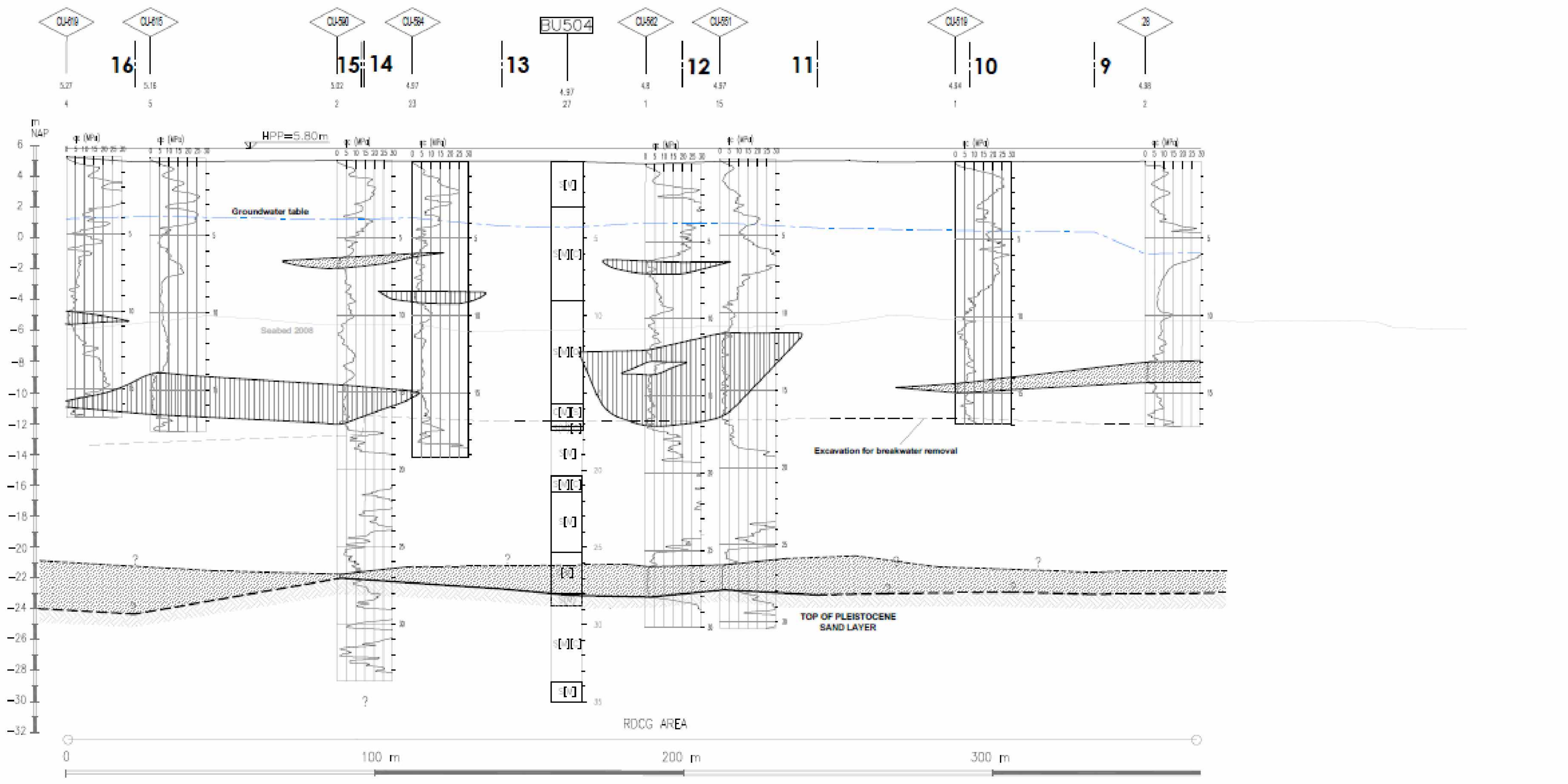
Recent sand	Pleistocene sand
Clay	Mixed soil: sandy, silts/silty sands and/or clayey silts



Recent Reclaimed Area at Maasvlakte

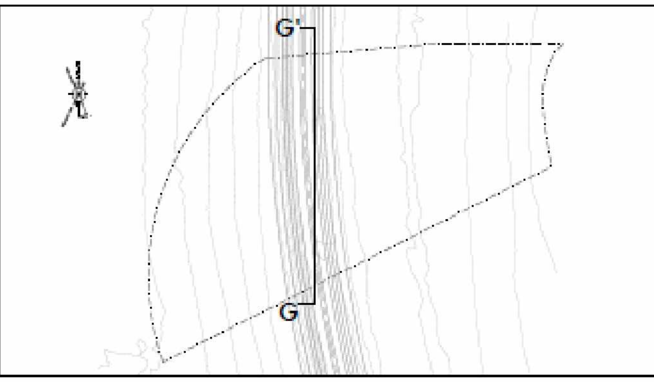
SECTION F-F'

HORIZ. 1:1250 VERT. 1:250



LEGEND

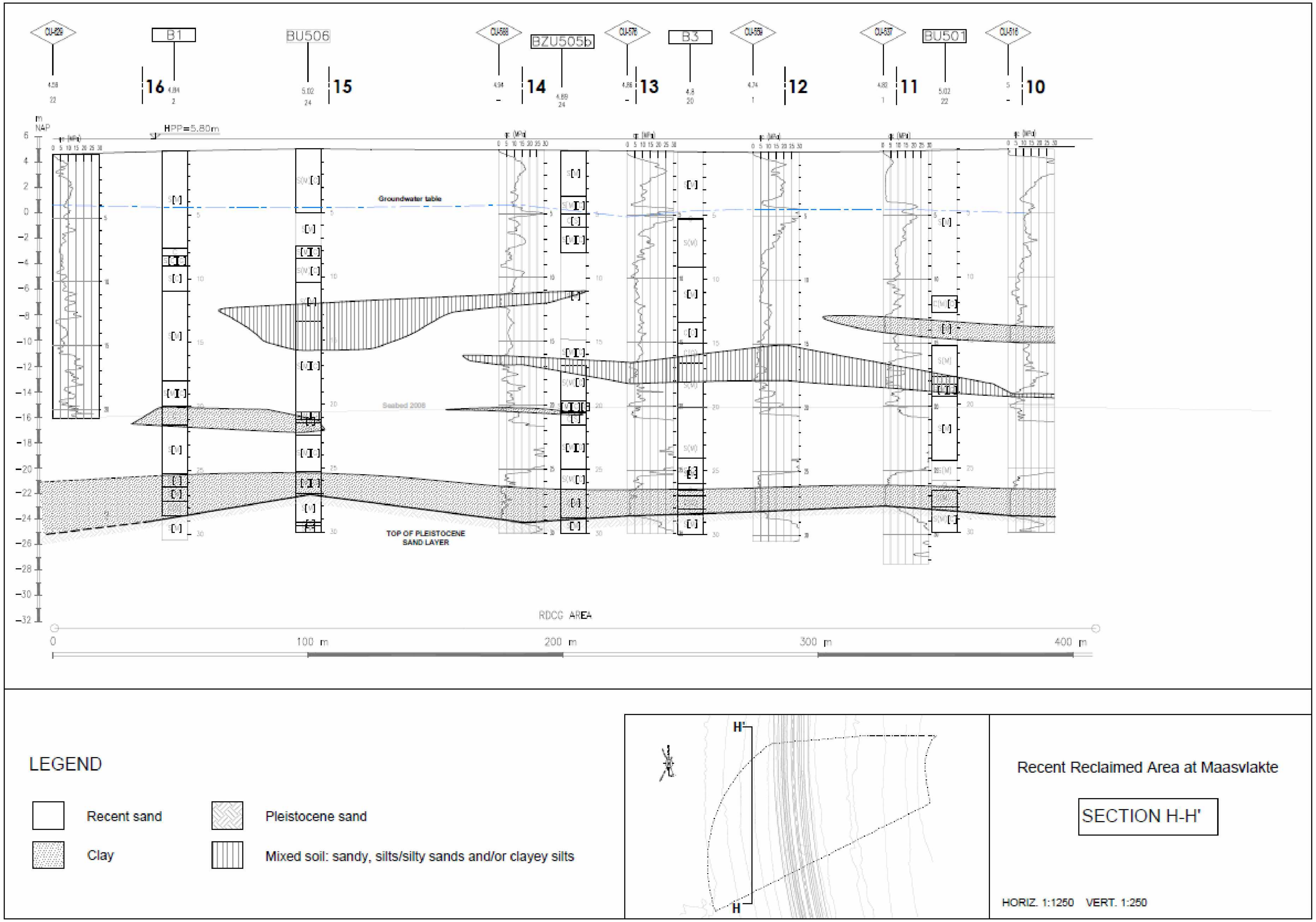
- Recent sand
- Clay
- Pleistocene sand
- Mixed soil: sandy, silts/silty sands and/or clayey silts



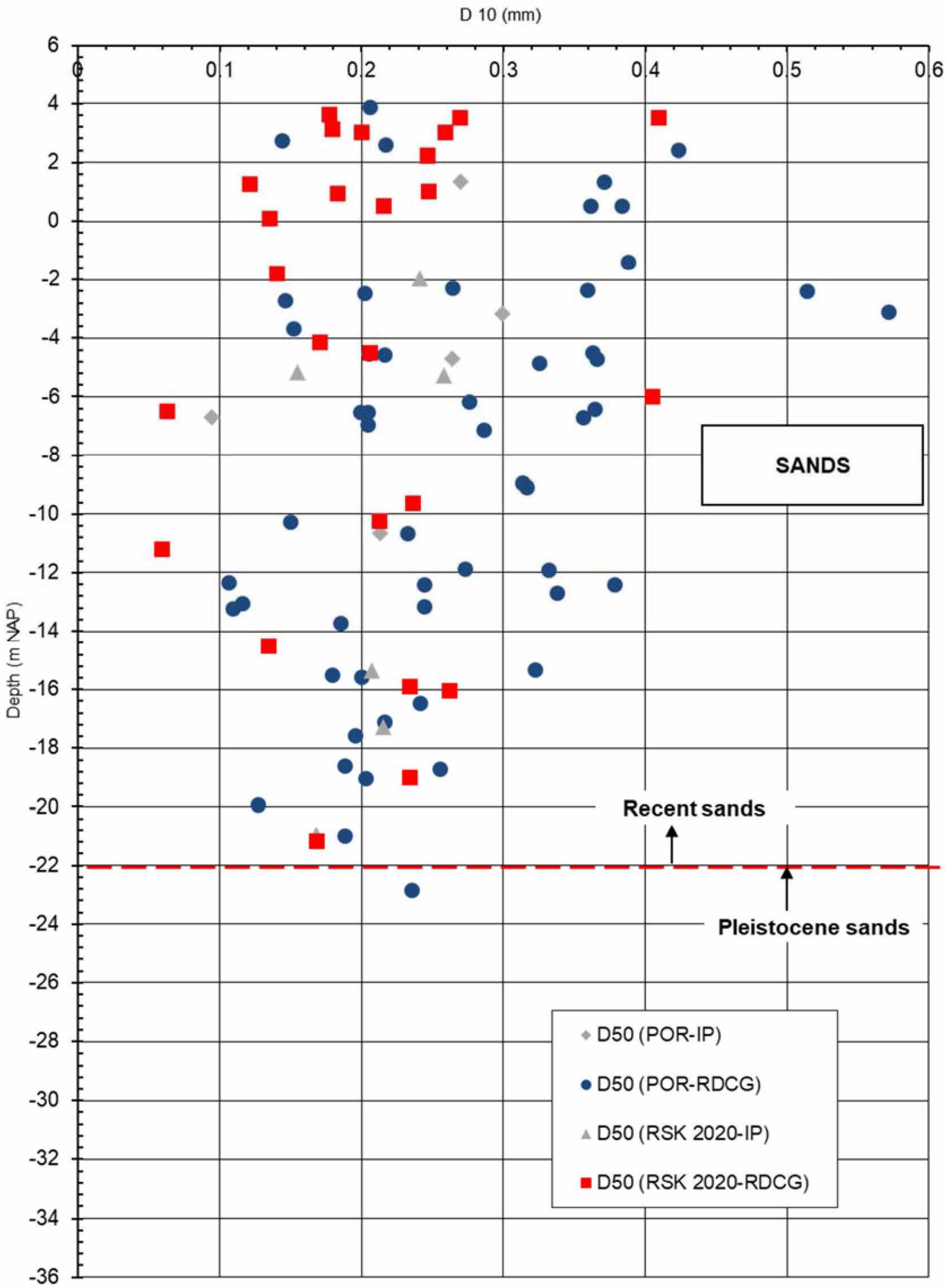
Recent Reclaimed Area at Maasvlakte

SECTION G-G'

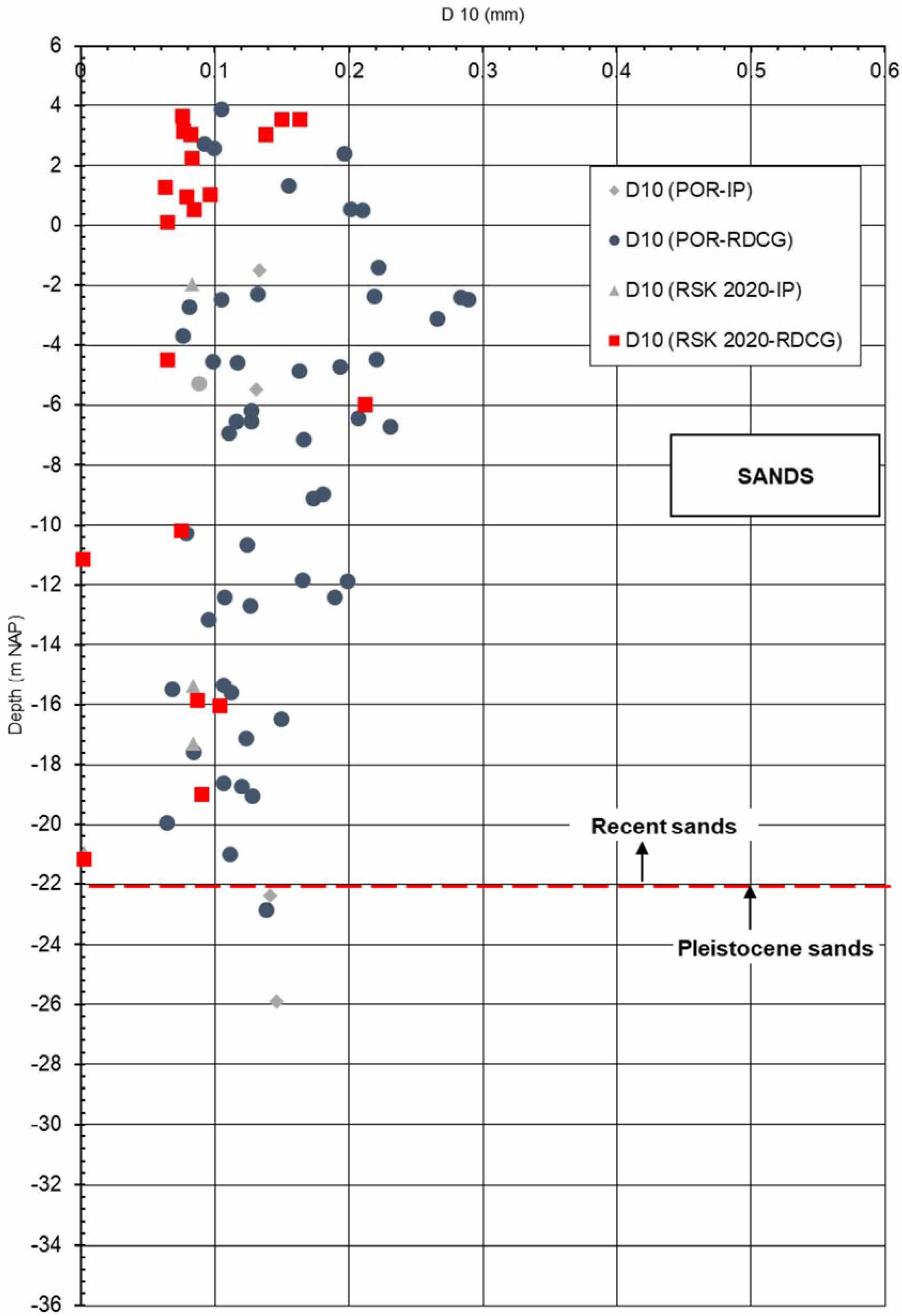
HORIZ. 1:1250 VERT. 1:250


CU-829
B1
BU506
CU-568
BZU505b
CU-576
B3
CU-559
CU-537
BU501
CU-516
16
15
14
13
12
11
10
HPP=5.80m
Groundwater table
Seabed 2008
TOP OF PLEISTOCENE SAND LAYER
RDCG AREA
0
100 m
200 m
300 m
400 m
LEGEND
Recent sand
Pleistocene sand
Clay
Mixed soil: sandy, silts/silty sands and/or clayey silts
H'
H
Recent Reclaimed Area at Maasvlakte
SECTION H-H'
HORIZ. 1:1250 VERT. 1:250

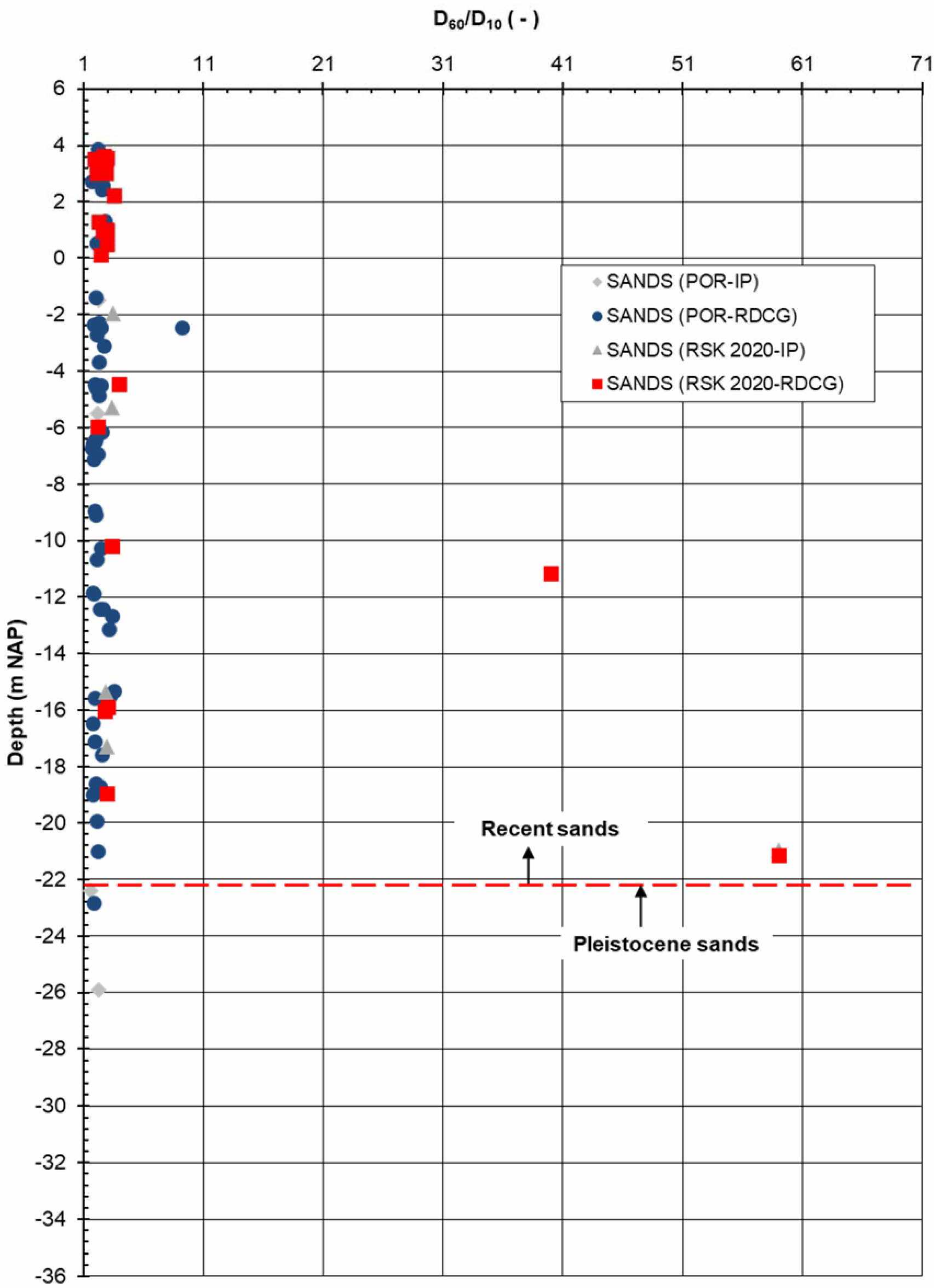
ATTACHMENT 3
Laboratory test
CPT results and interpretation



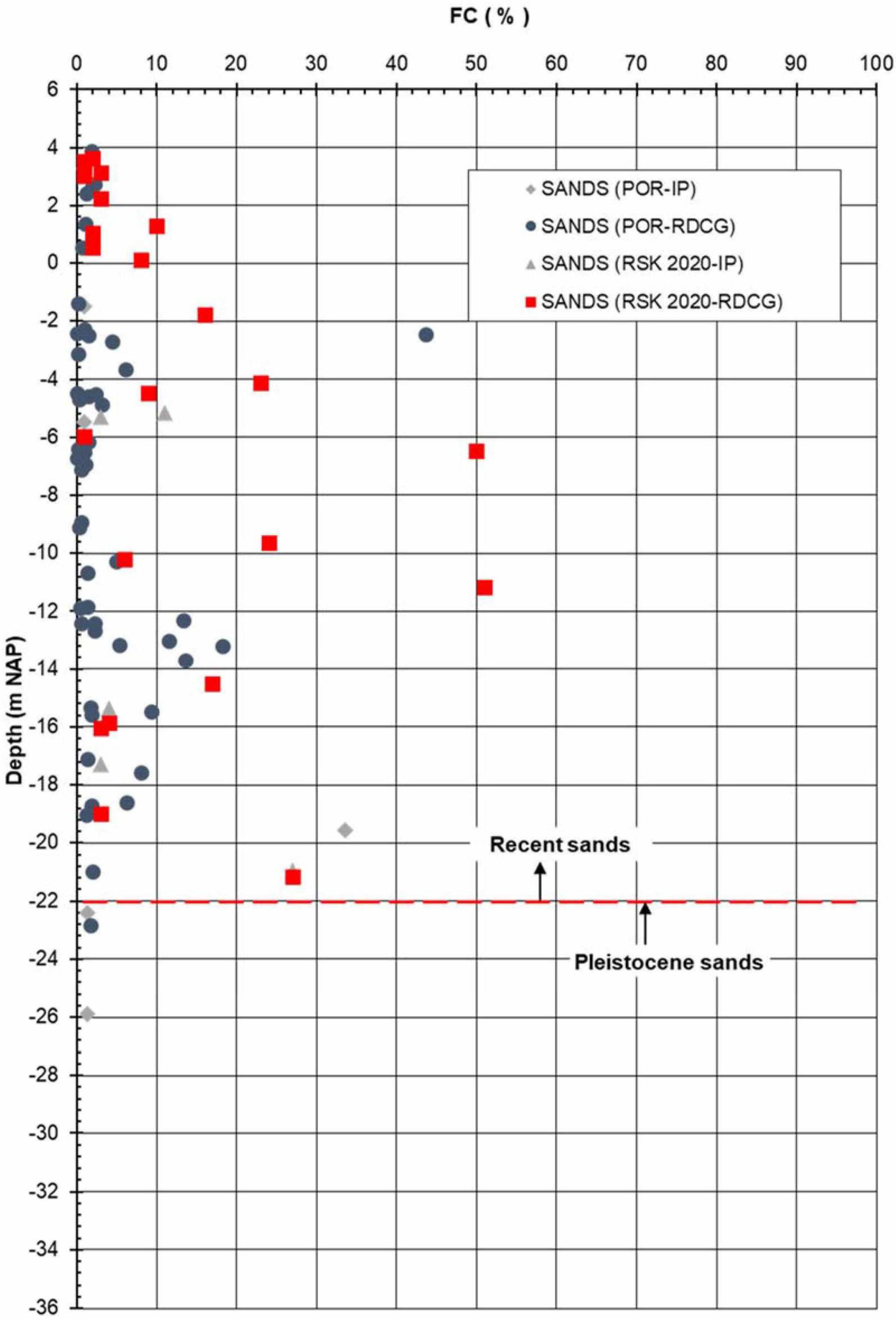
Attach.3 Figure 1- D50



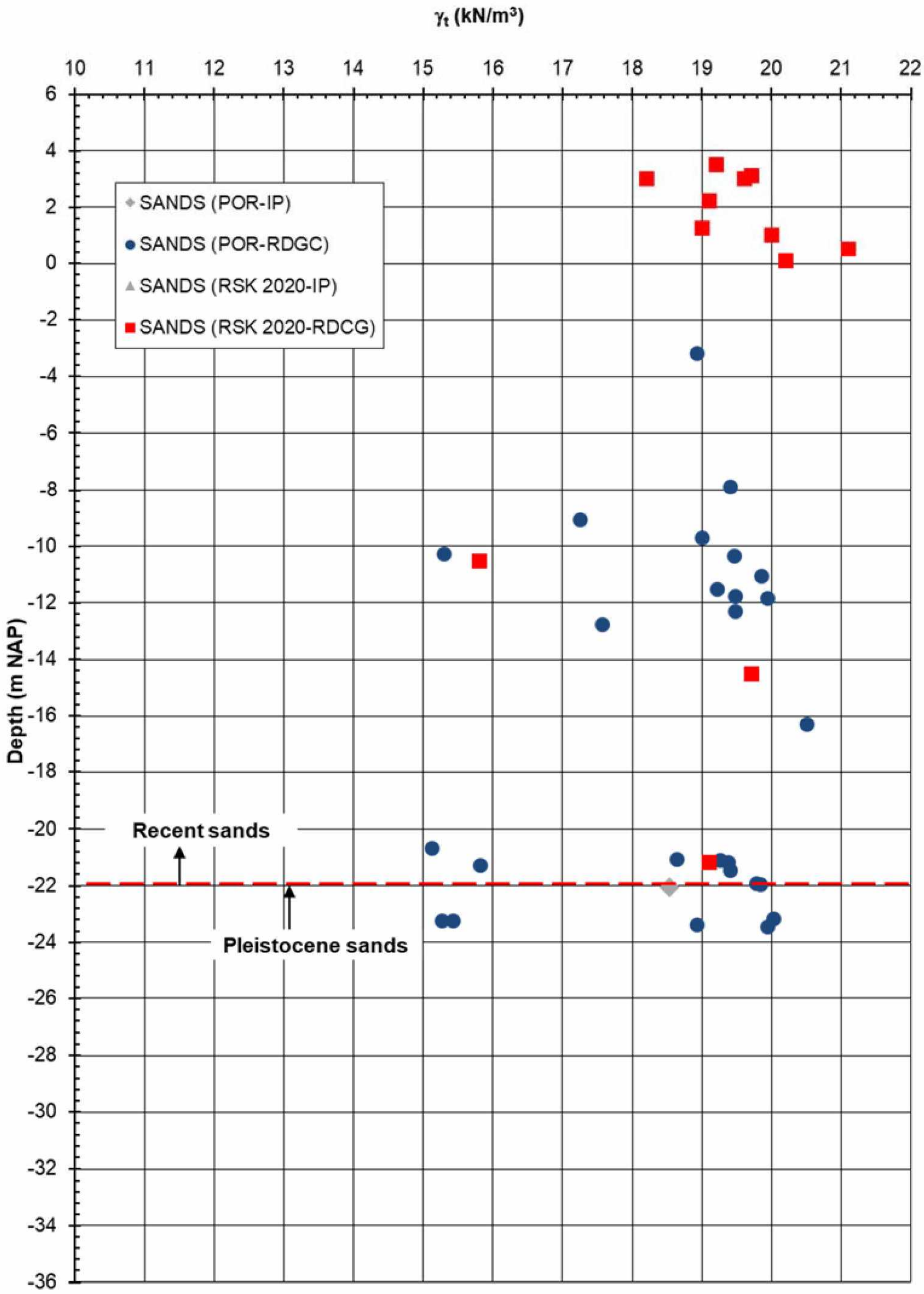
Attach.3 Figure 2 – D10



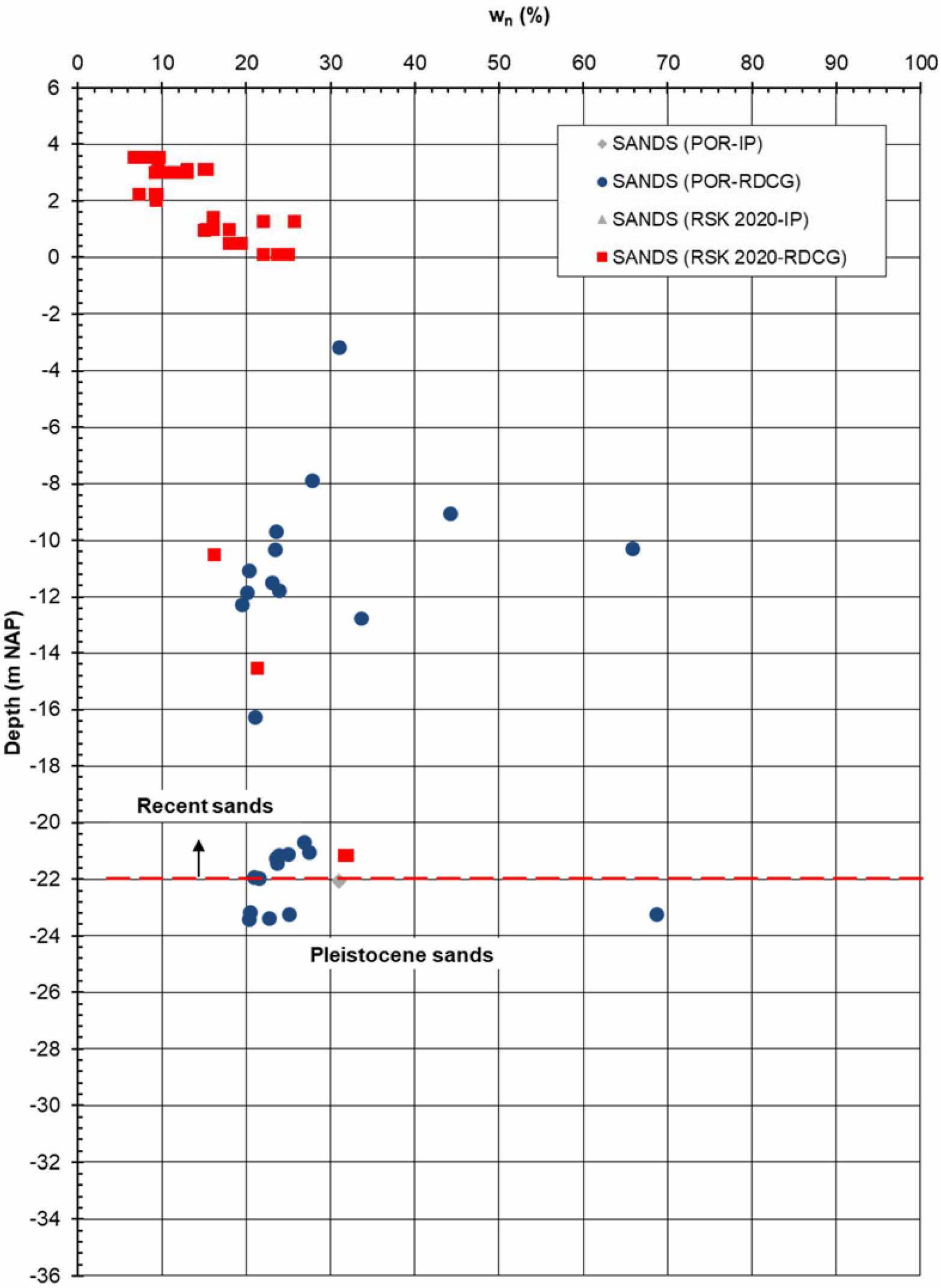
Attach.3 Figure 3 – Sands- Uniformity Coefficient



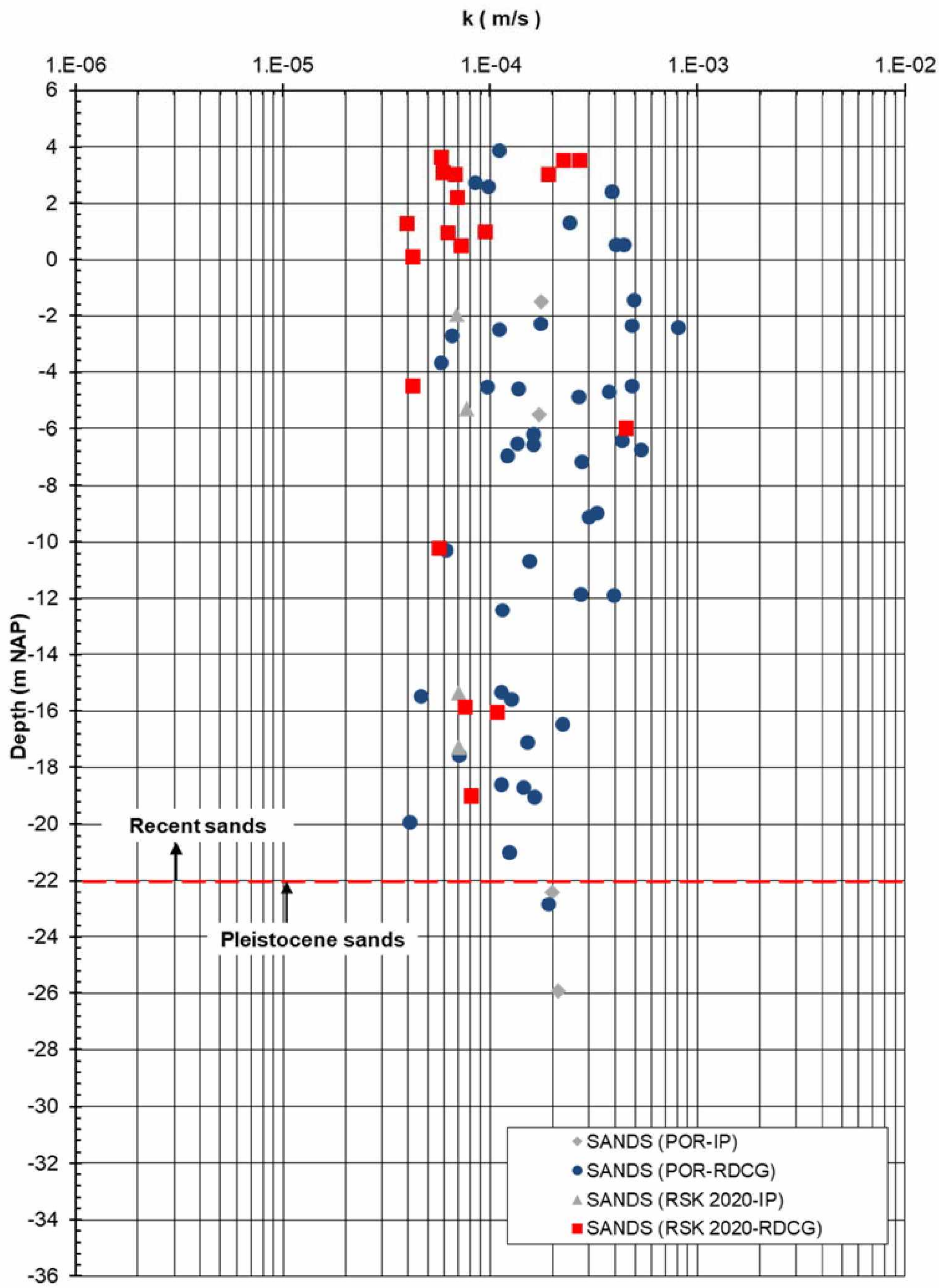
Attach.3 Figure 4 – Sands- Fine content



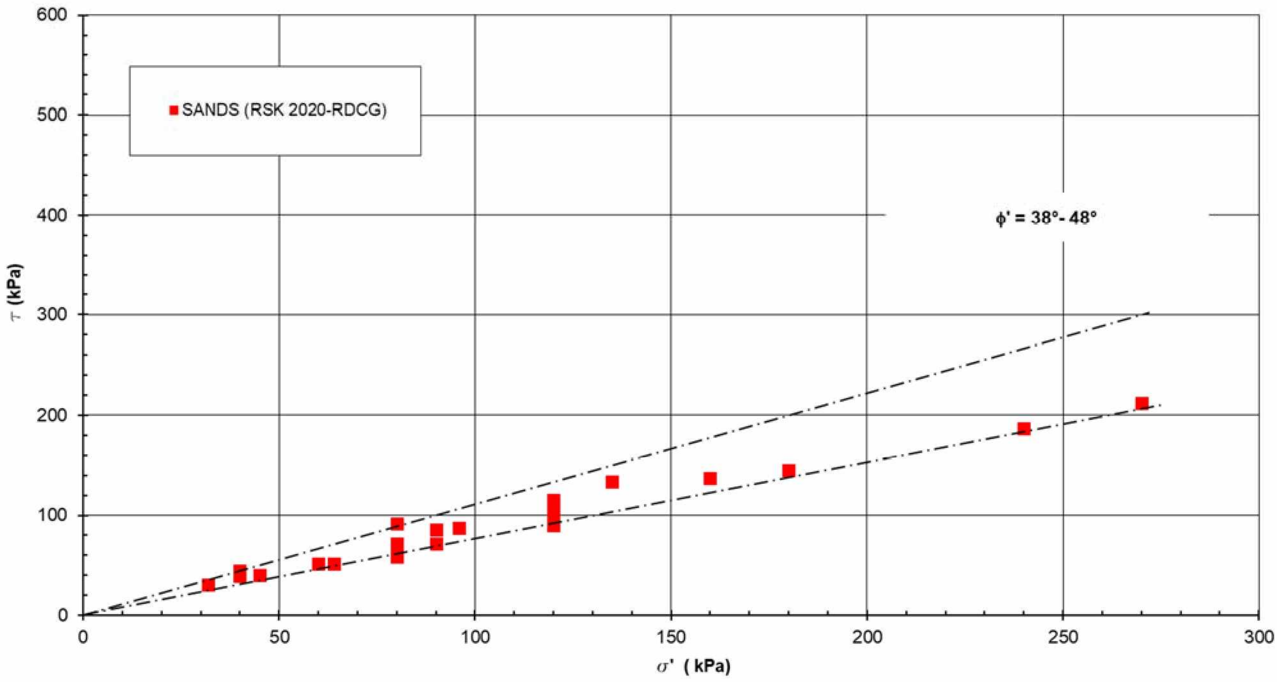
Attach.3 Figure 5- Sands - Total unit weight

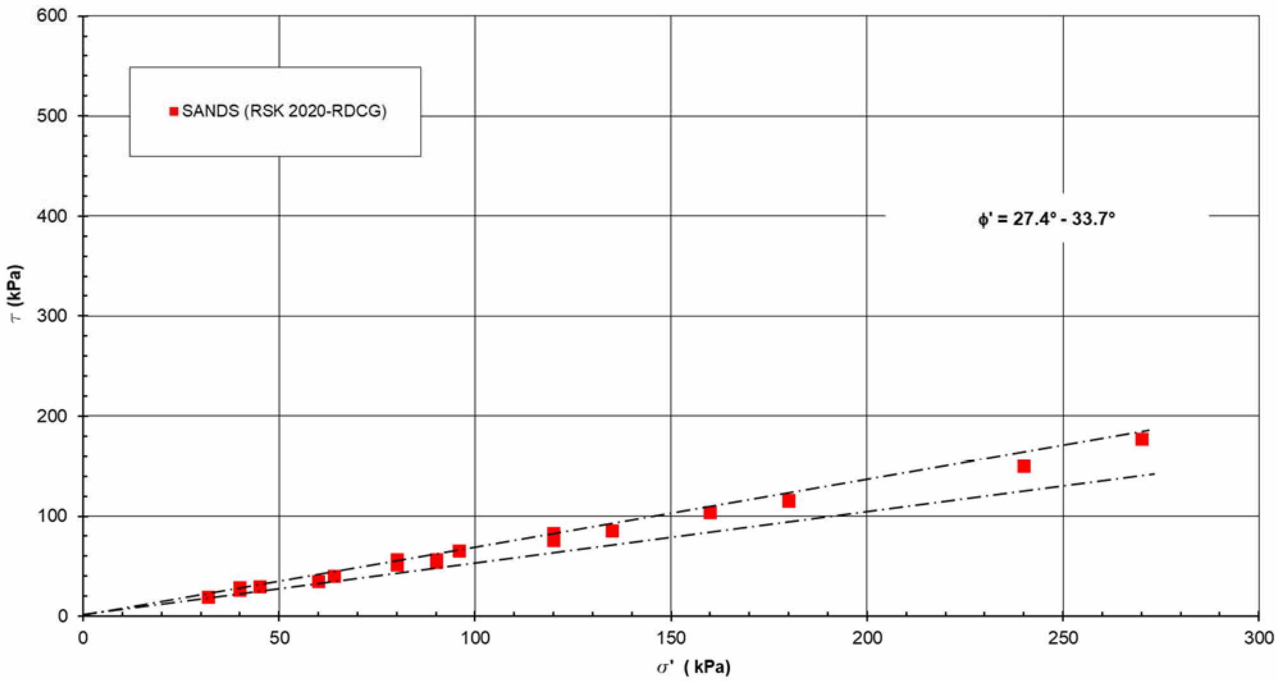


Attach.3 Figure 6- Sands - Water content

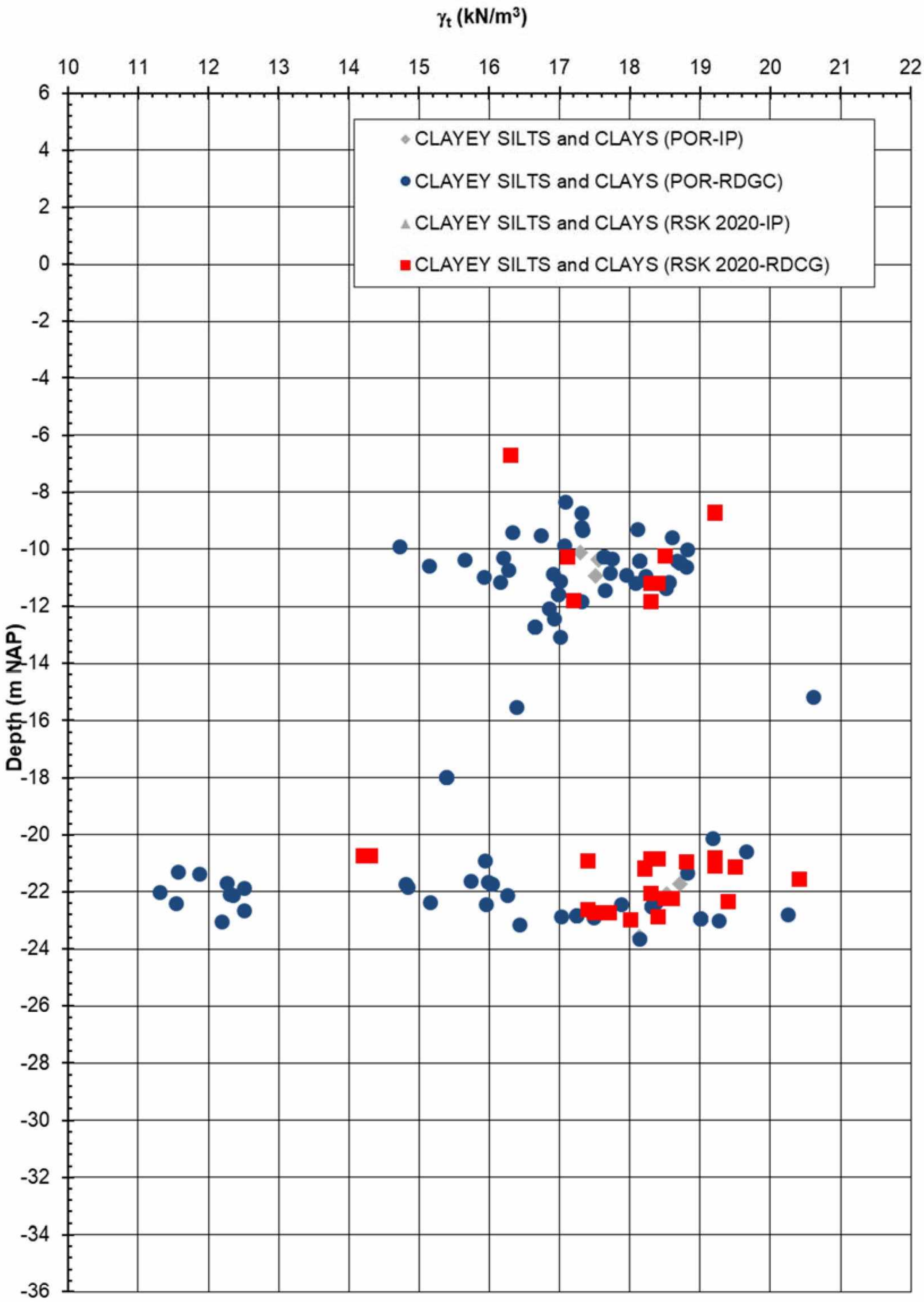


Attach.3 Figure 7- Sands - Coefficient of permeability (estimated from D10)

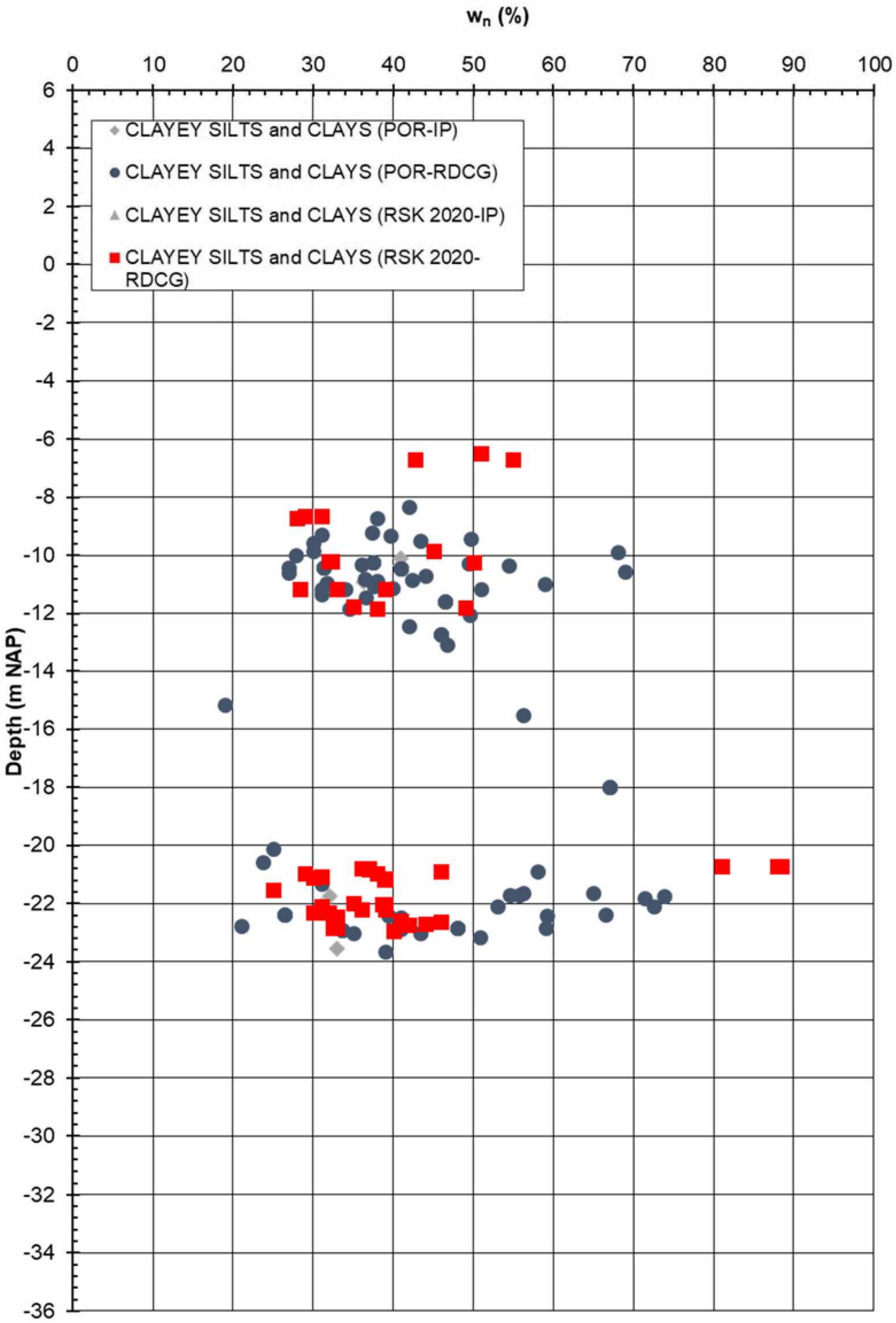




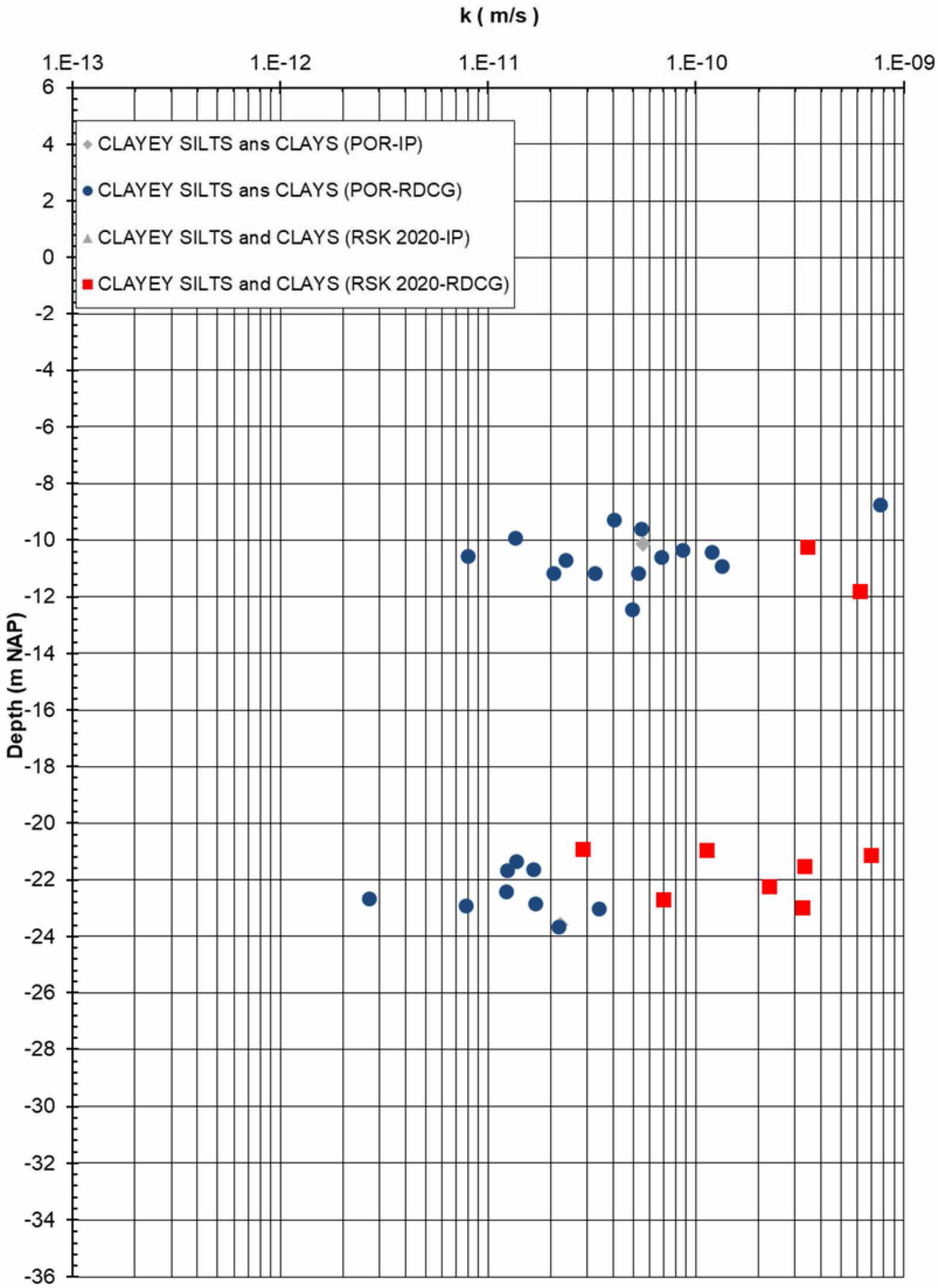
Attach.3 Figure 8- Direct shear test results on sand samples



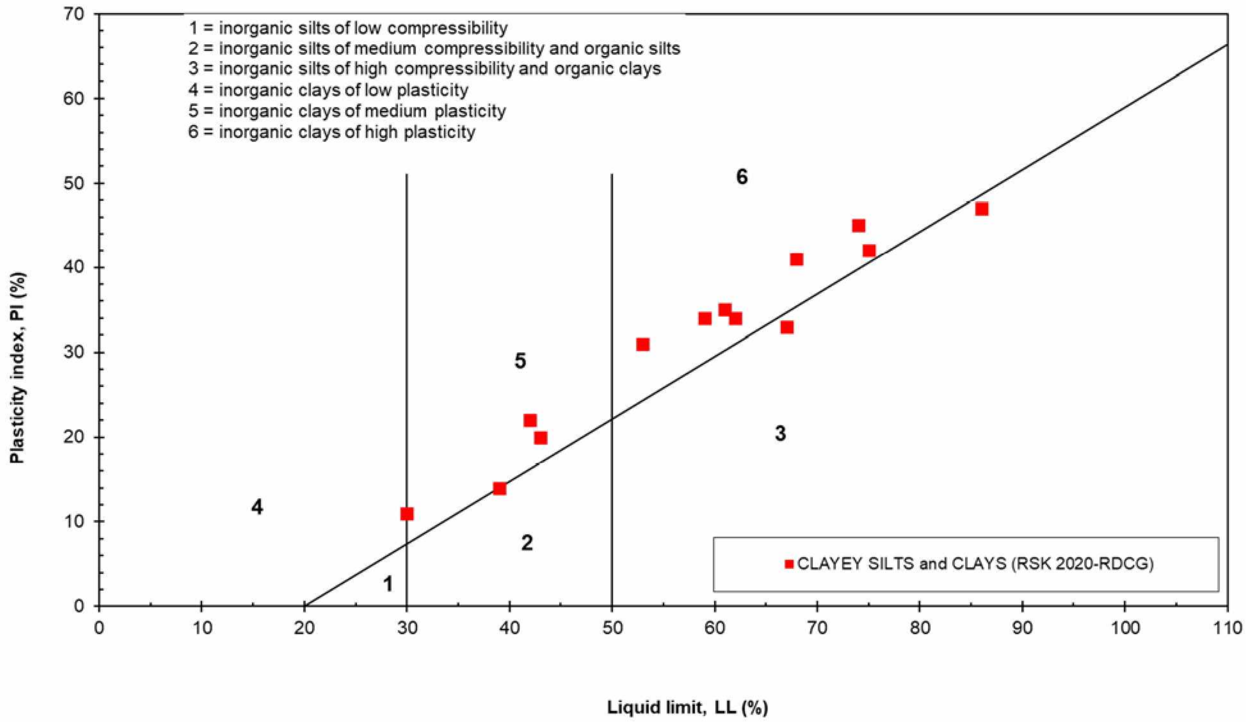
Attach.3 Figure 9 - Fine grained soils - Total unit weight



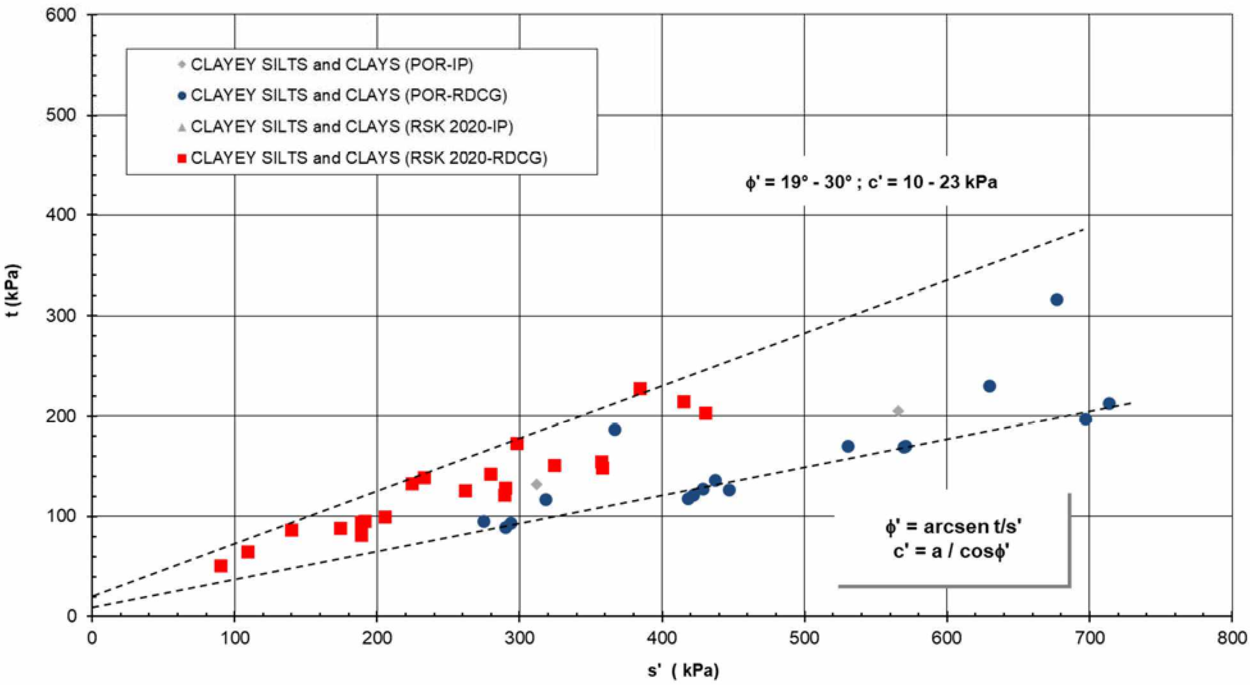
Attach.3 Figure 10 - Fine grained soils - Water content



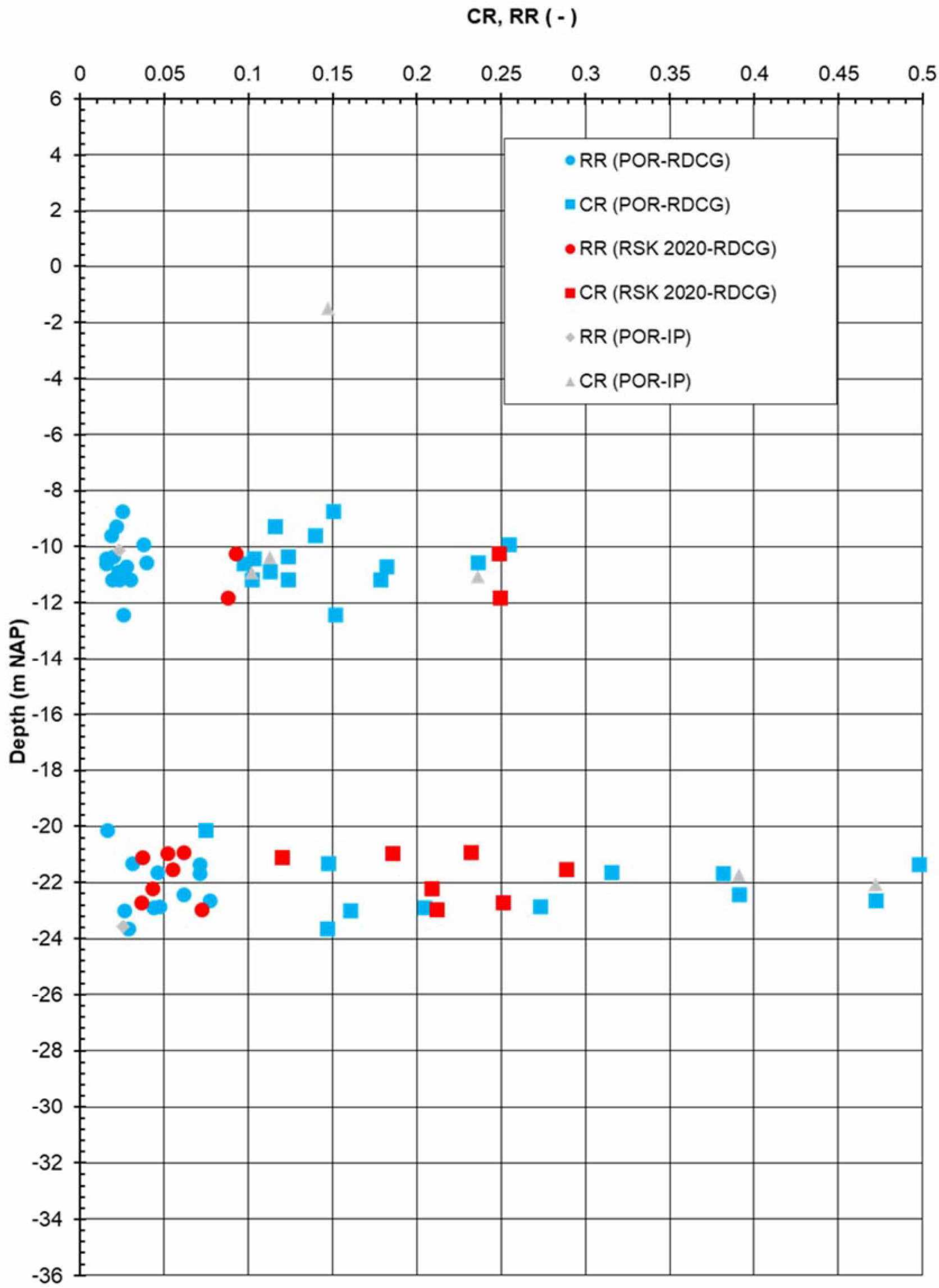
Attach.3 Figure 11- Clay - Coefficient of permeability (estimated from D10)



Attach.3 Figure 12- Casagrande plasticity chart for clays and silts



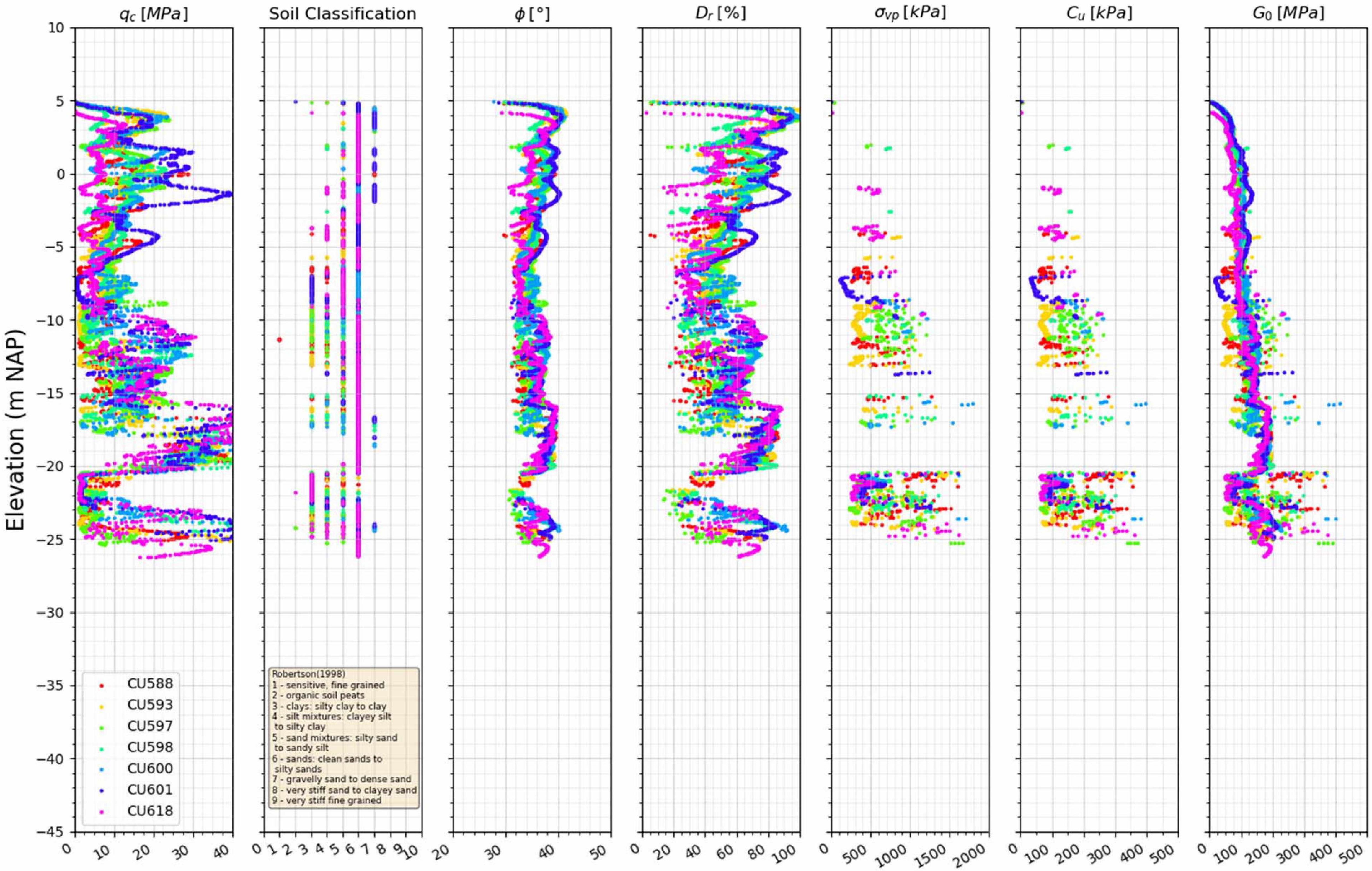
Attach.3 Figure 13- TXCIU results on clay samples



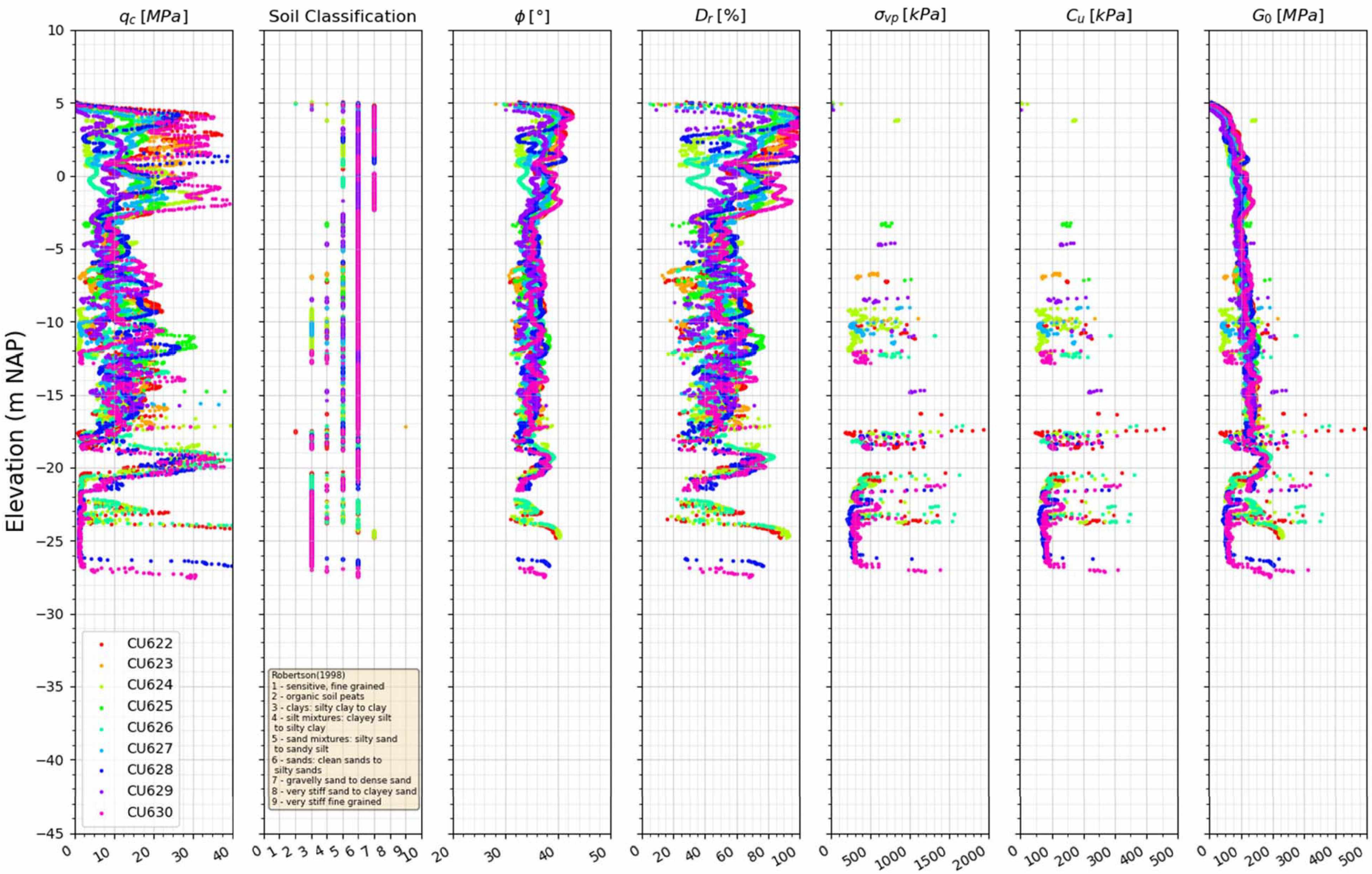
Attach.3 Figure 14 - CR,RR values from oedometer tests

CONFIDENTIAL

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE

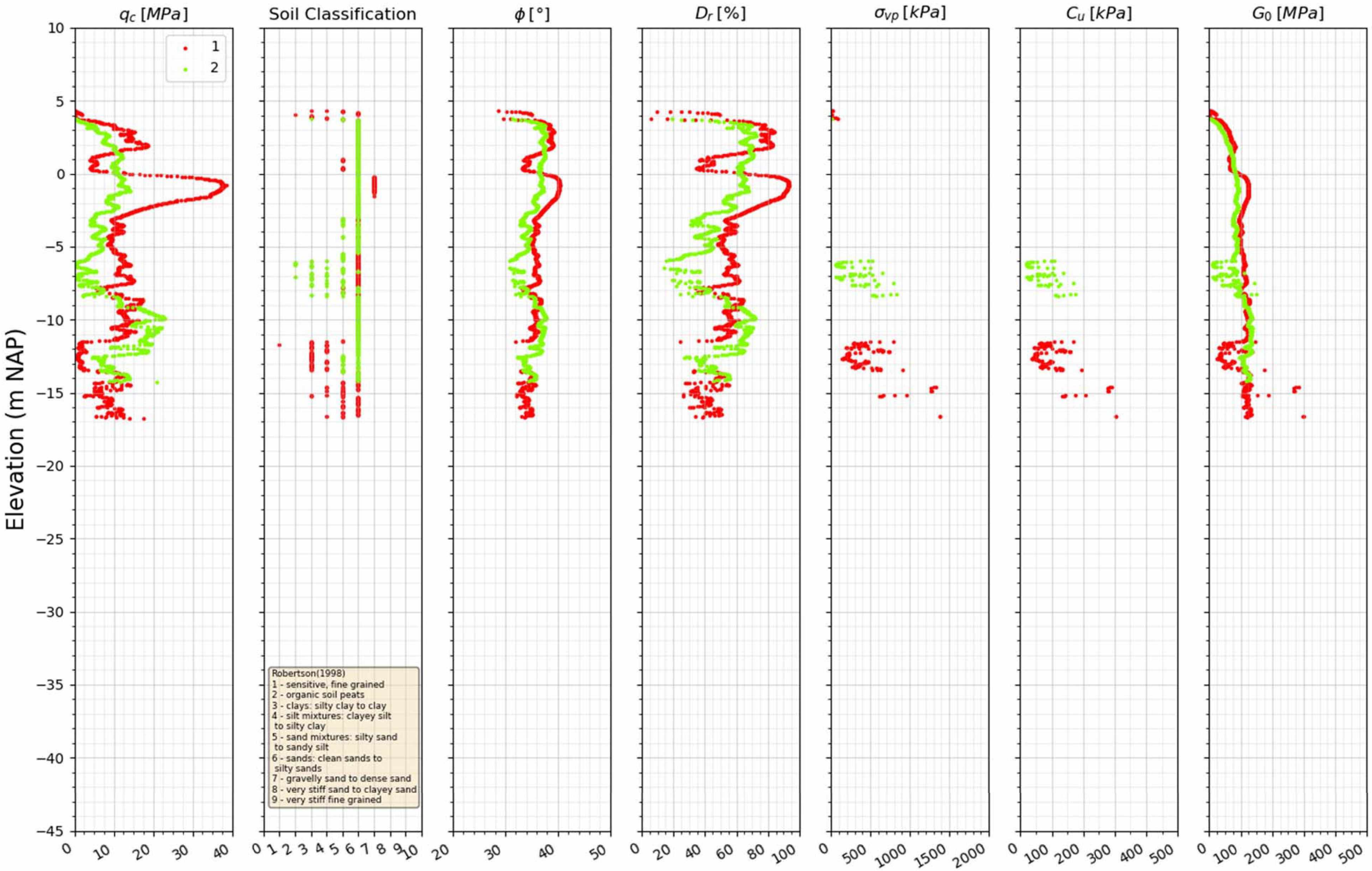


Attach.3 Figure 15- CPTs

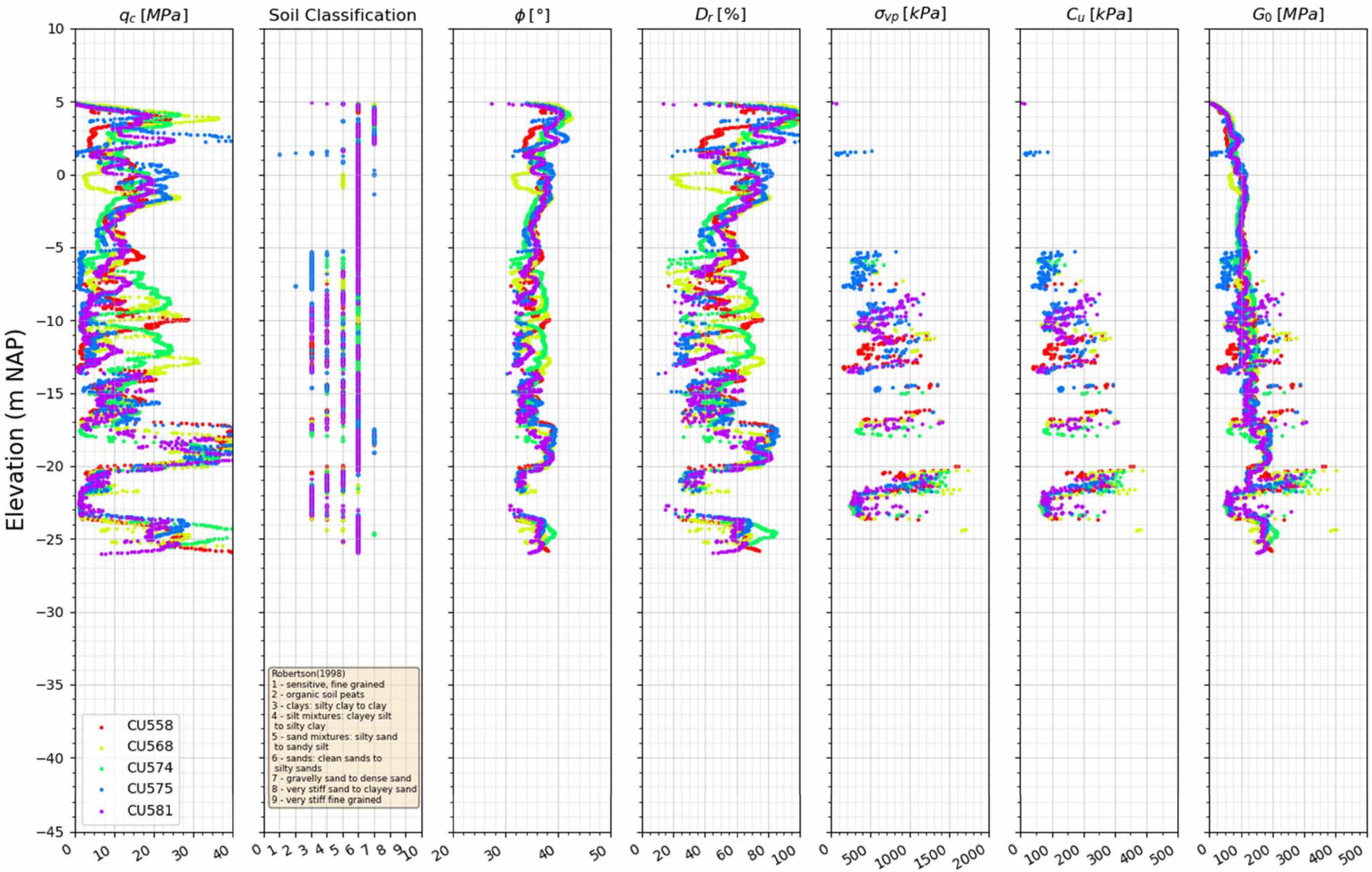


Attach.3 Figure 16- CPTs

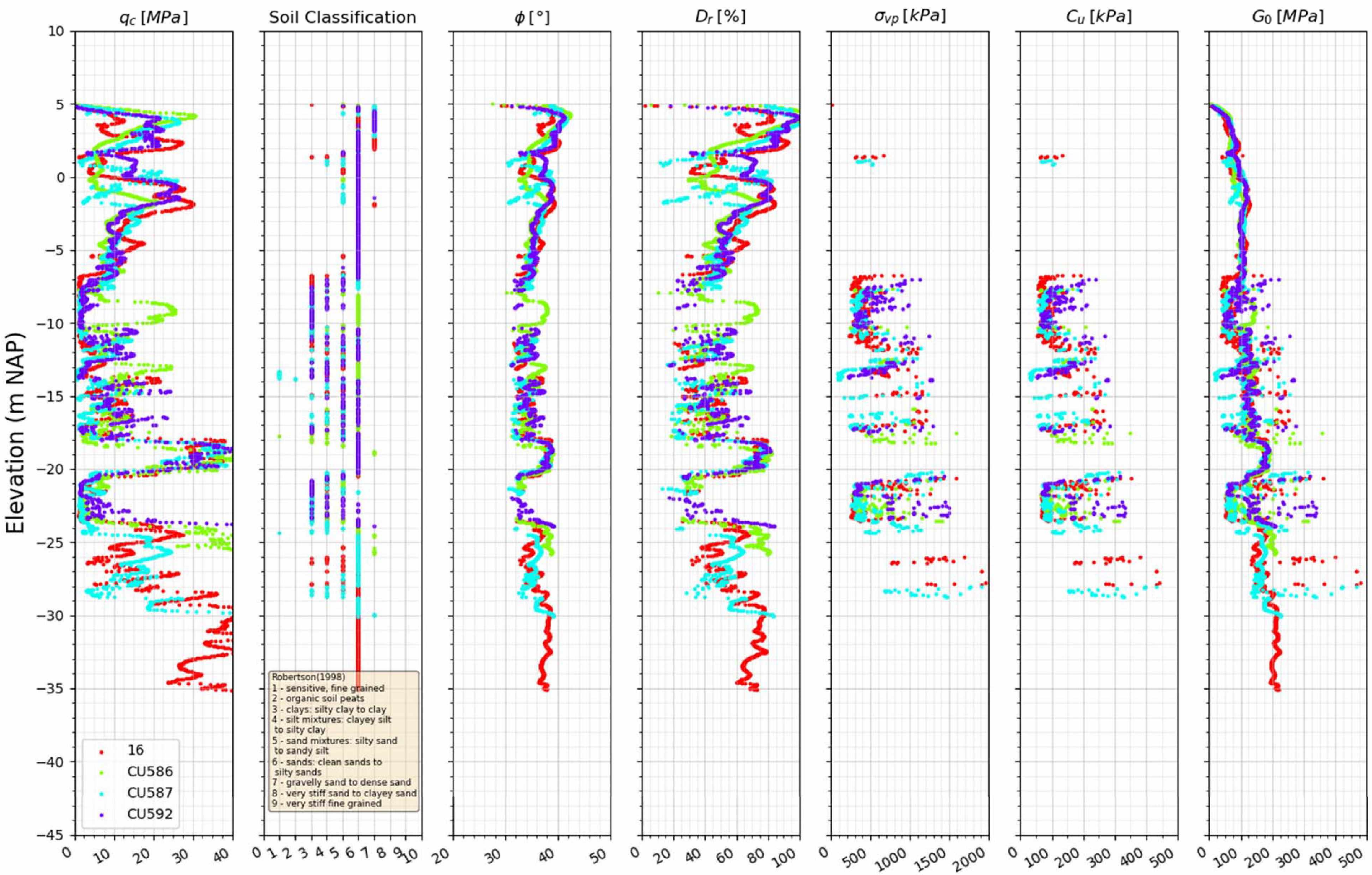
RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



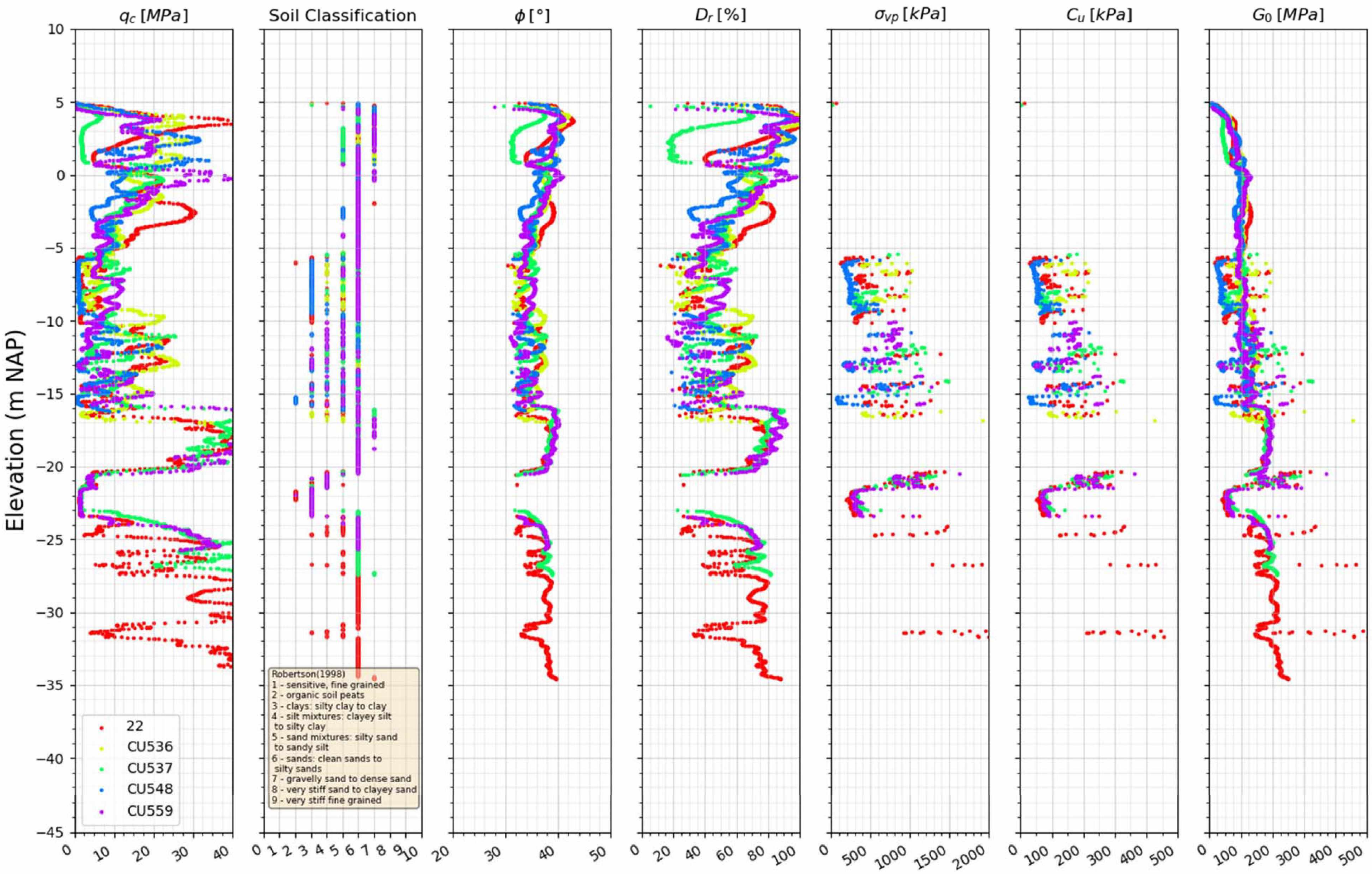
Attach.3 Figure 17- CPTs



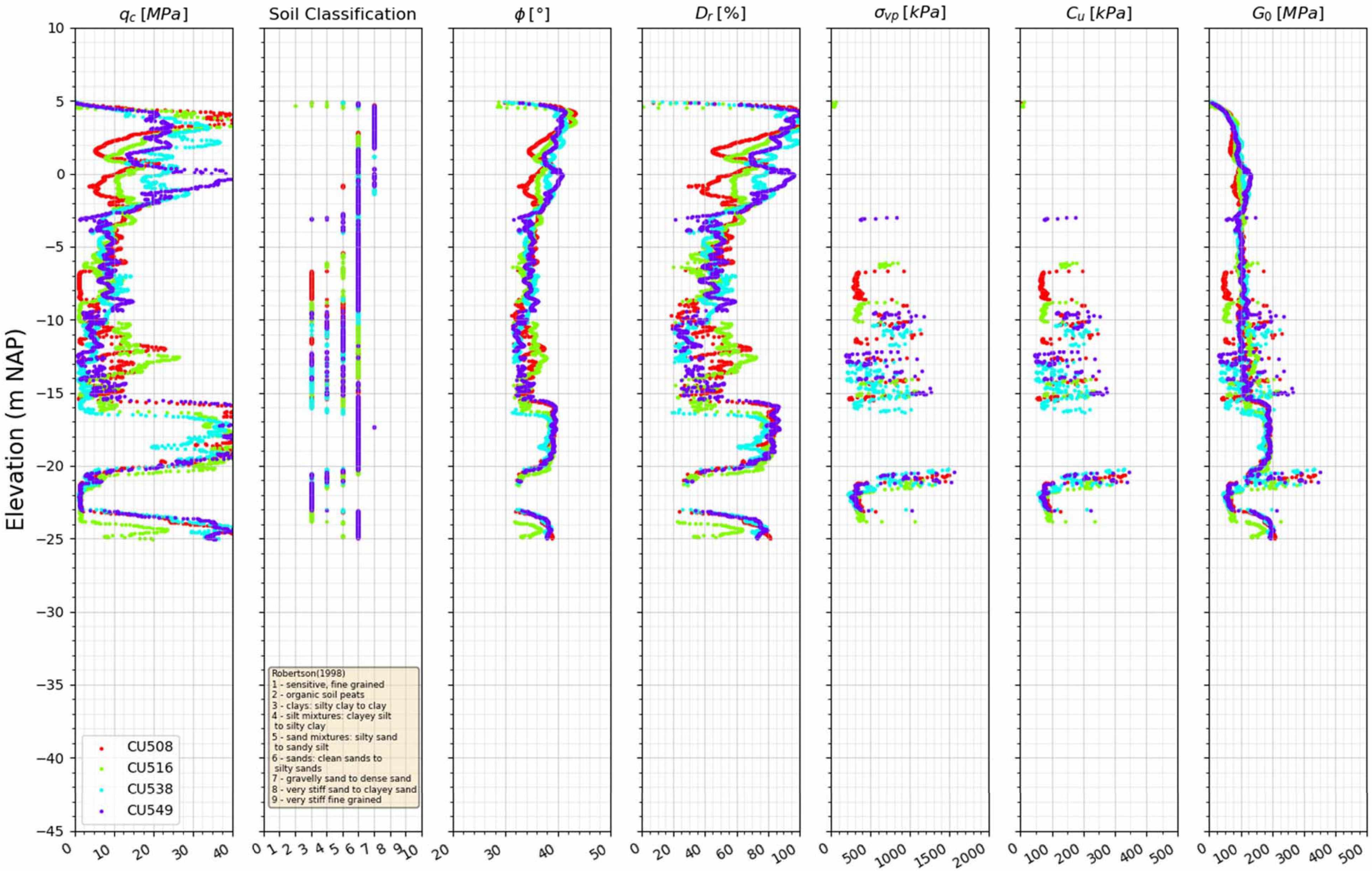
Attach.3 Figure 18- CPTs



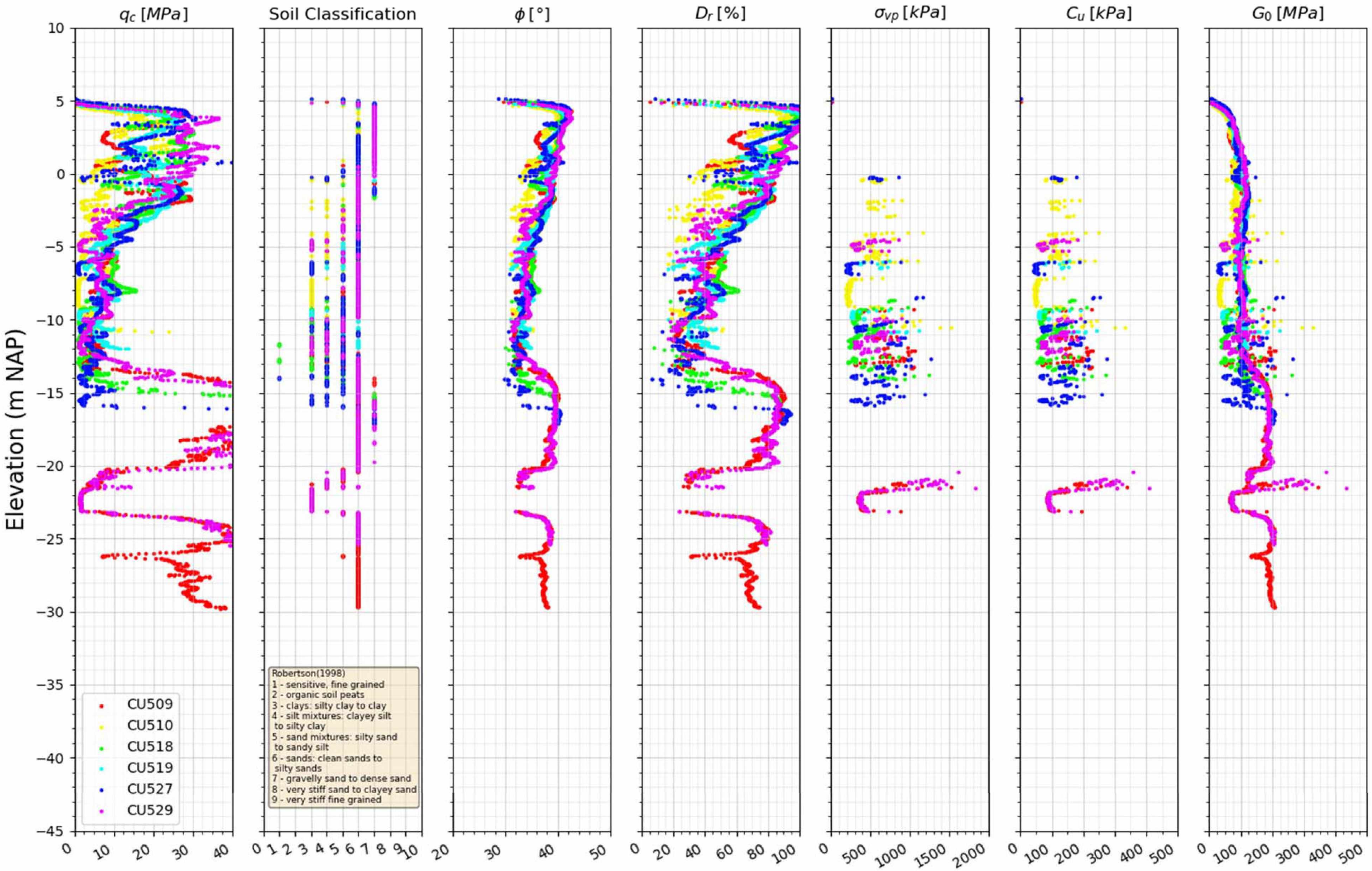
Attach.3 Figure 19- CPTs



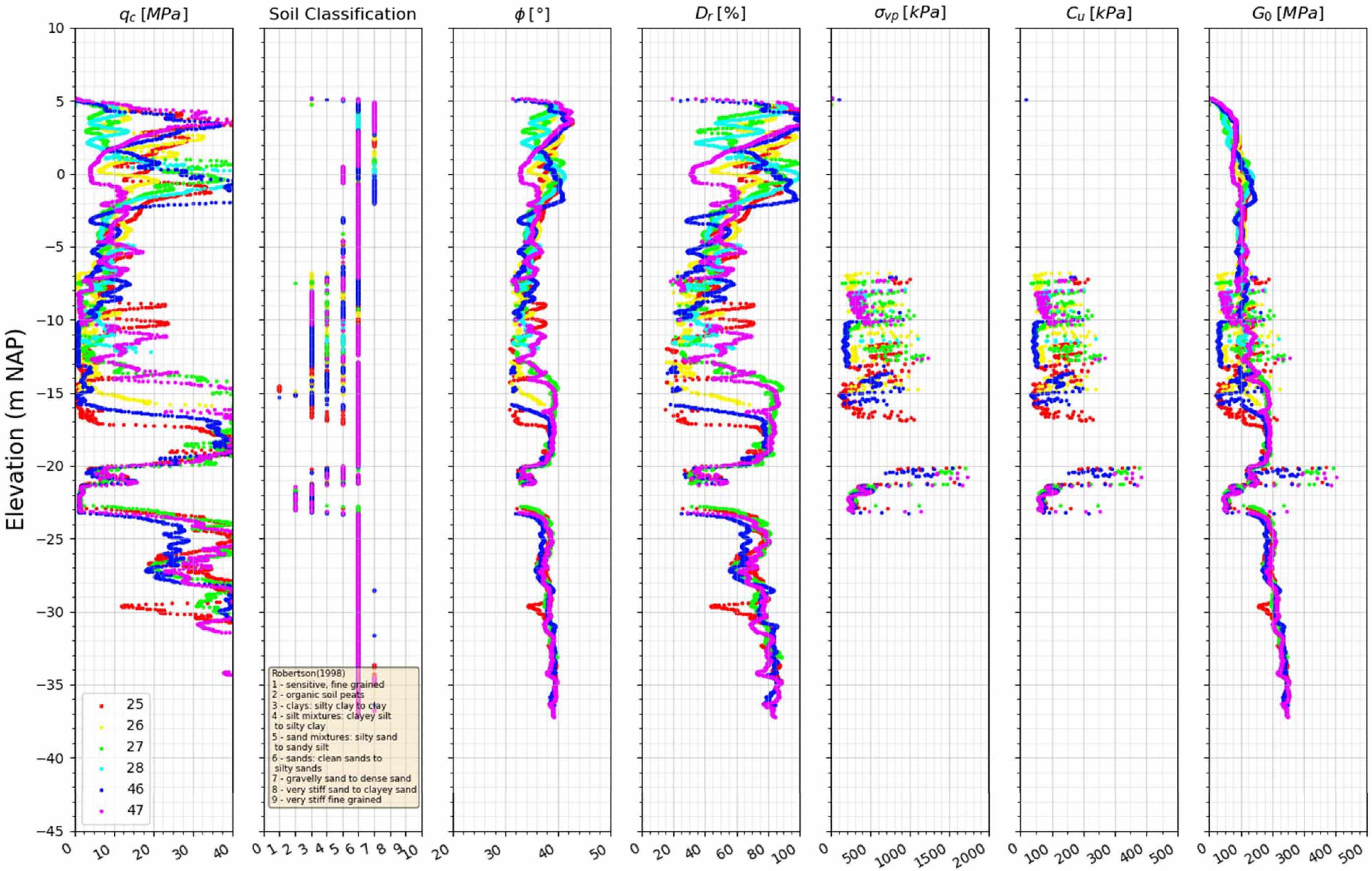
Attach.3 Figure 20- CPTs



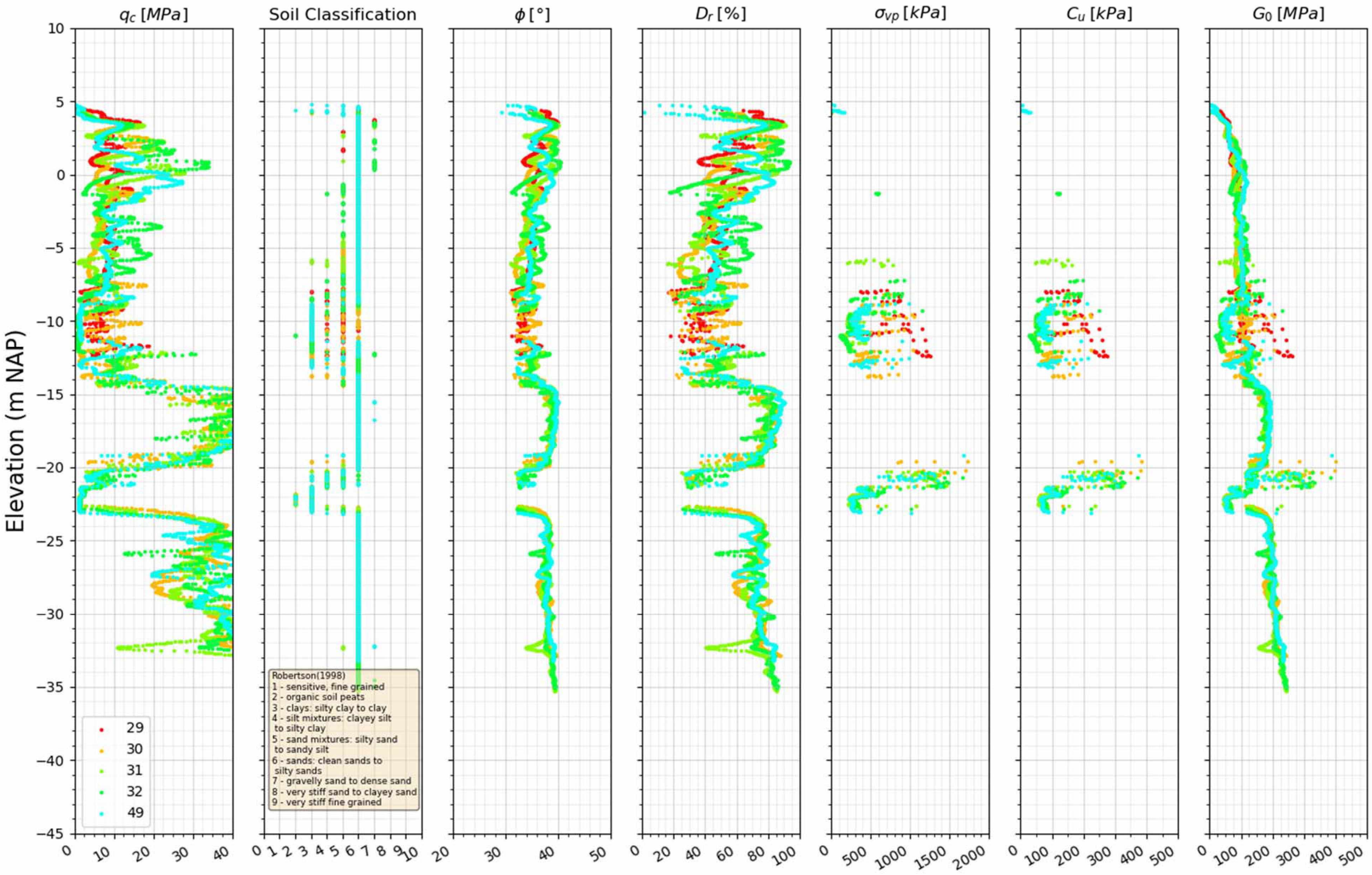
Attach.3 Figure 21- CPTs



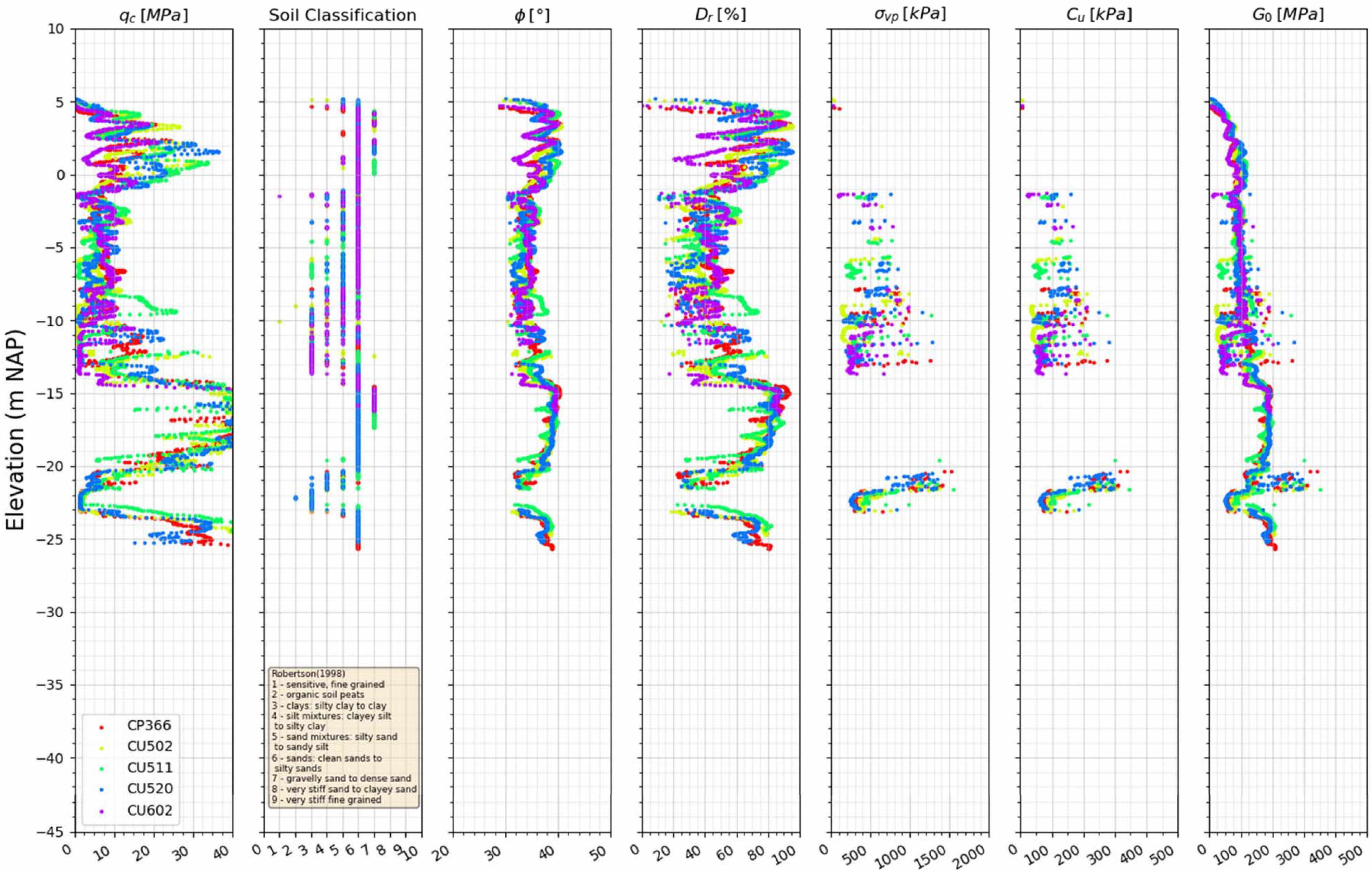
Attach.3 Figure 22- CPTs



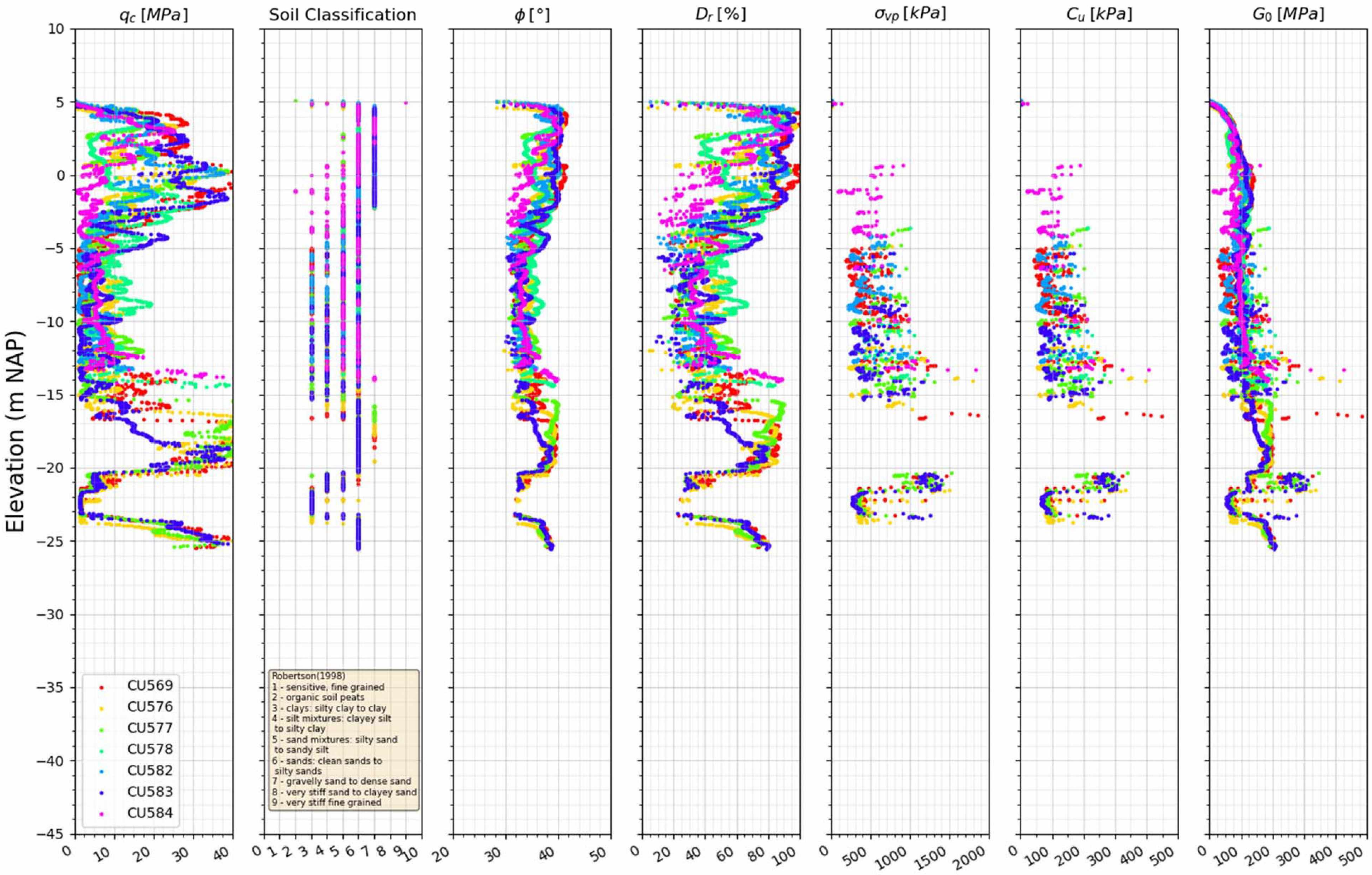
Attach.3 Figure 23- CPTs



Attach.3 Figure 24- CPTs



Attach.3 Figure 25- CPTs



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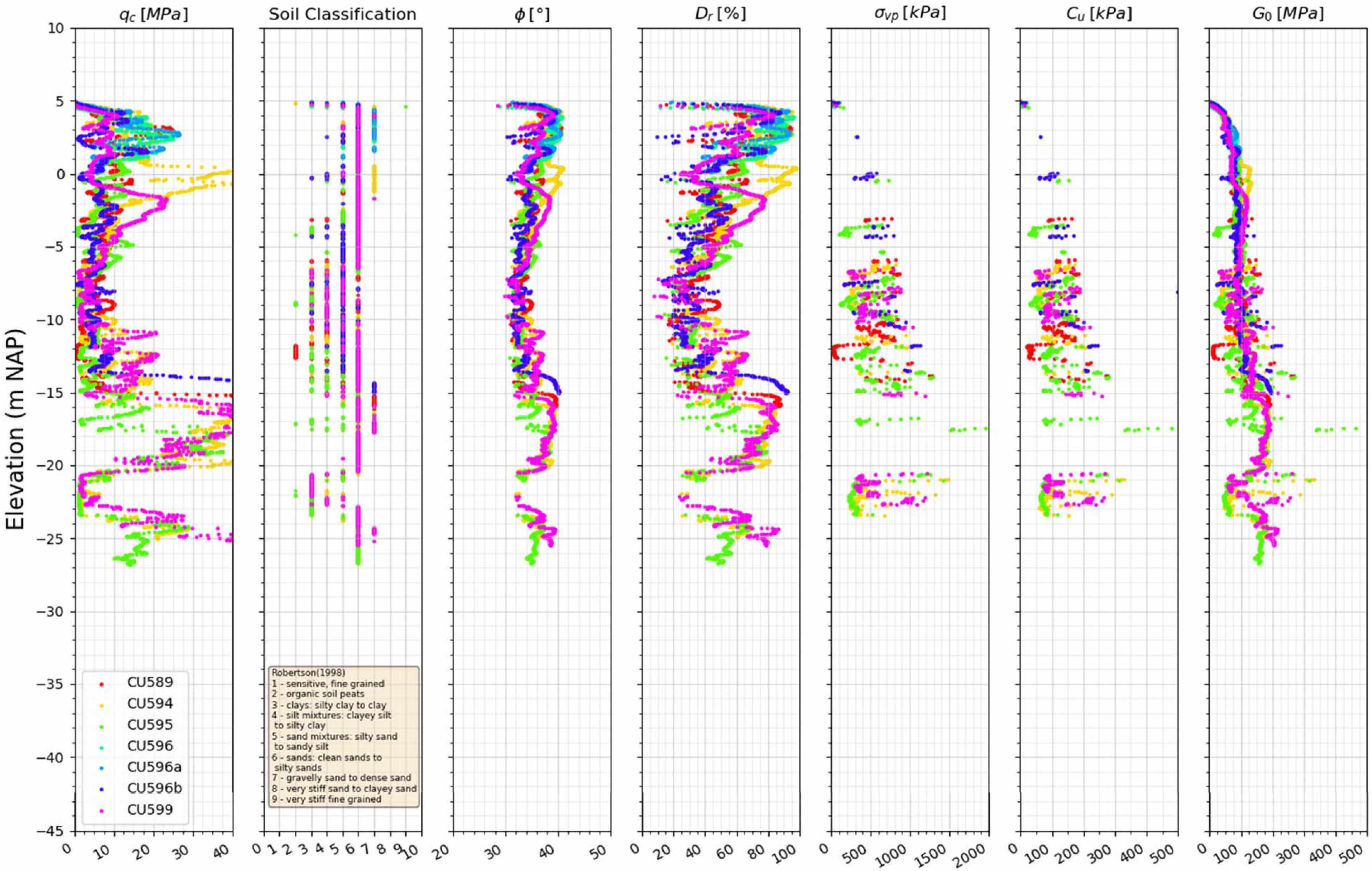

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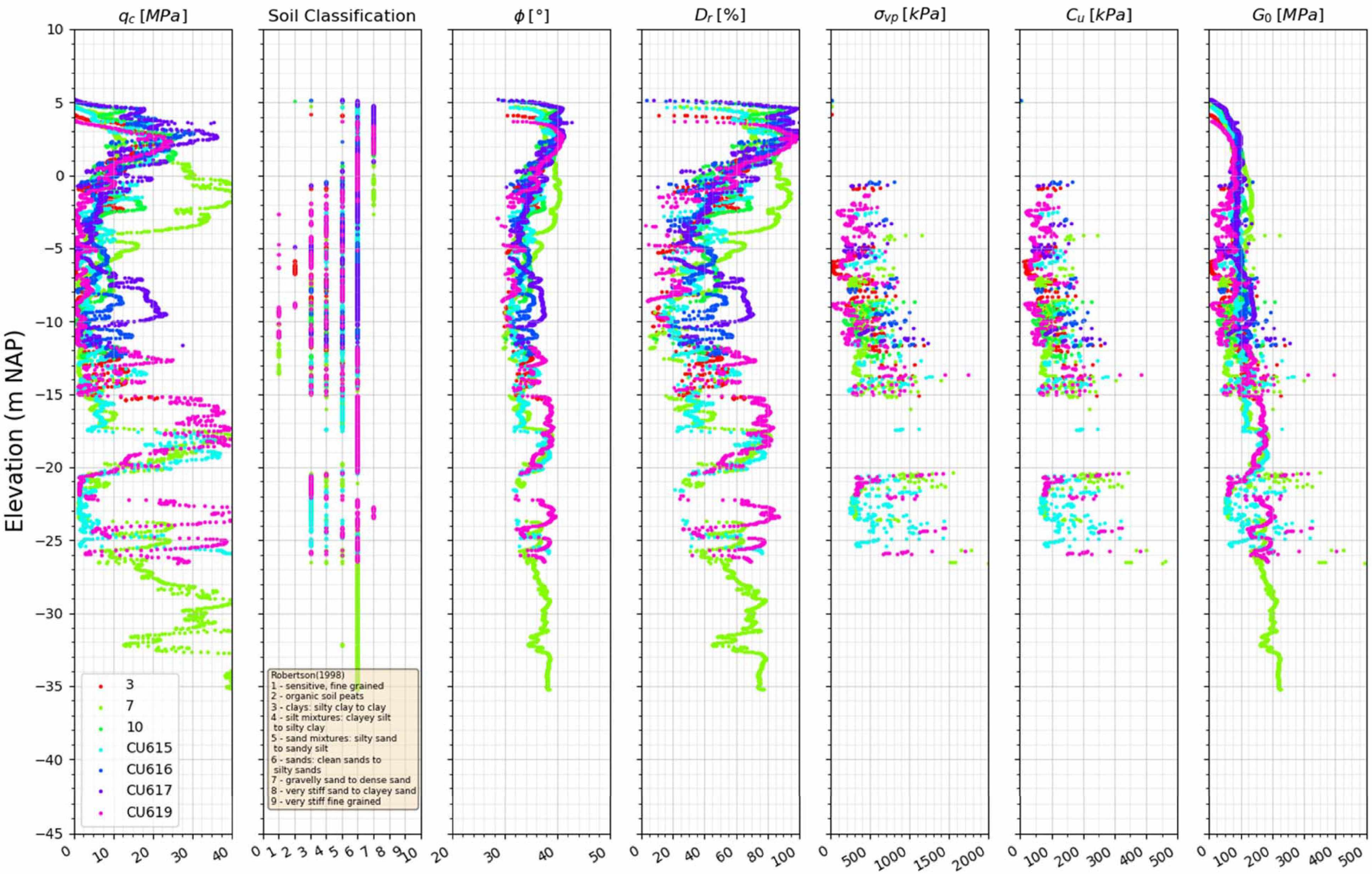

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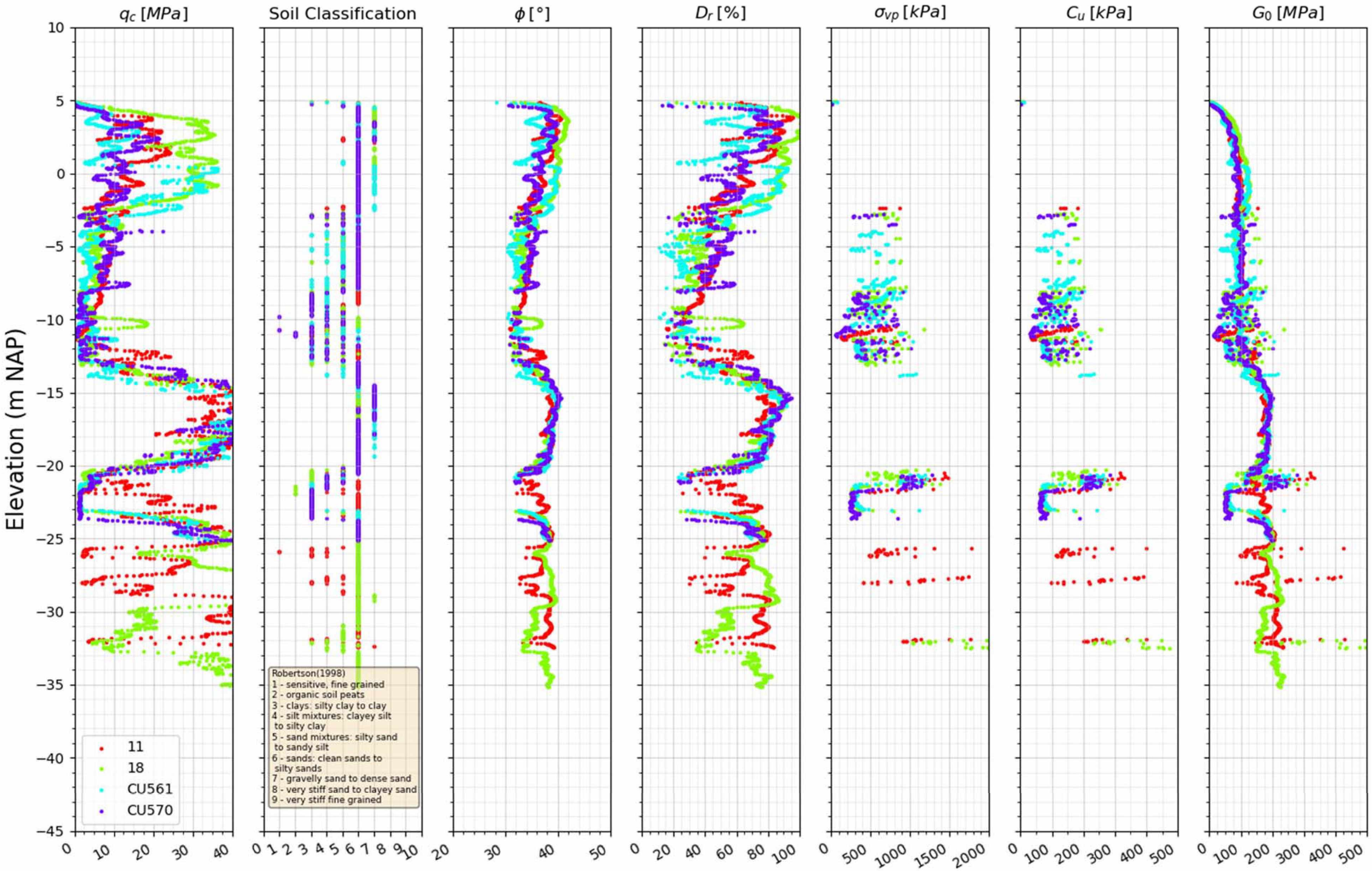

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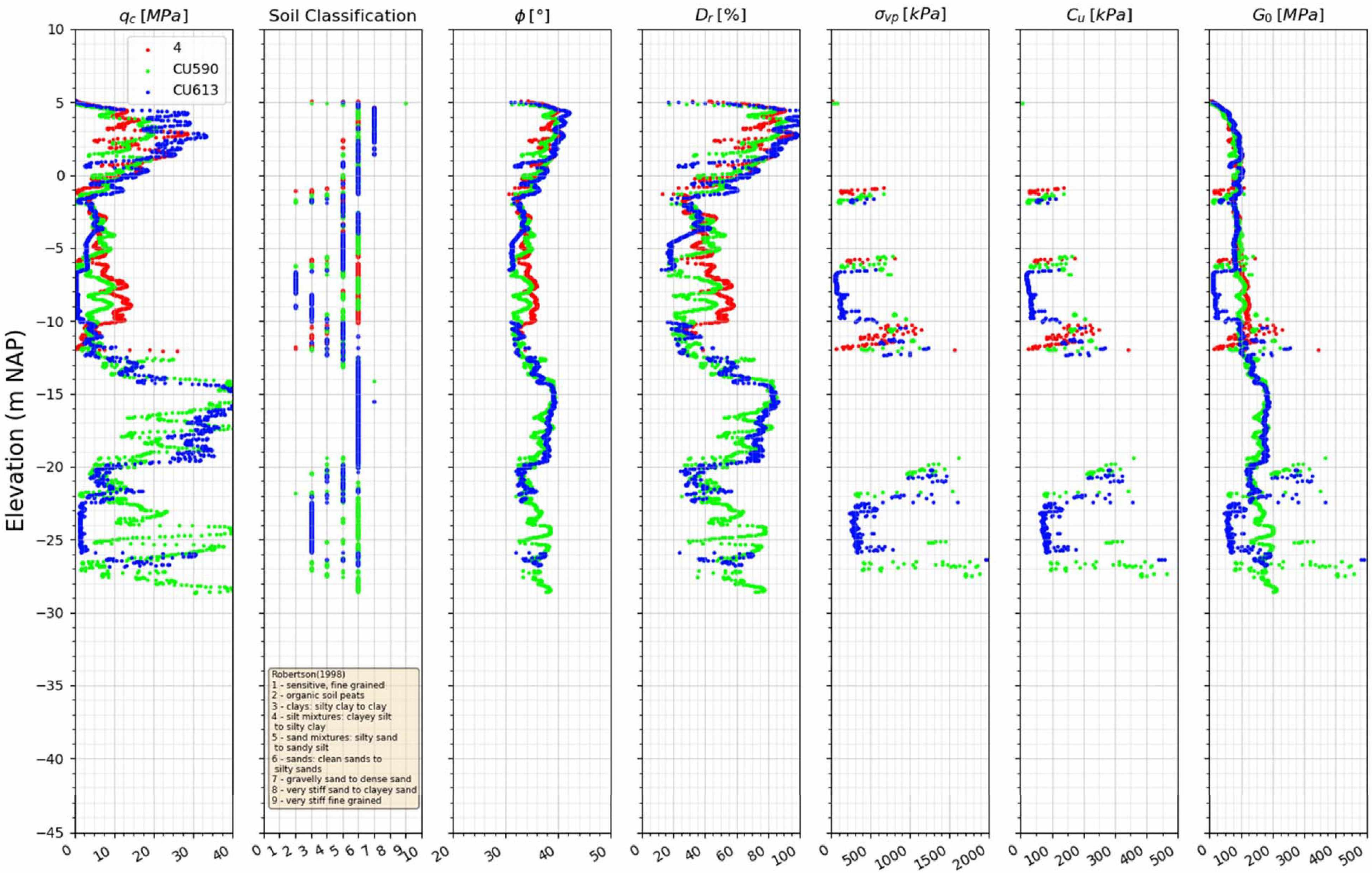

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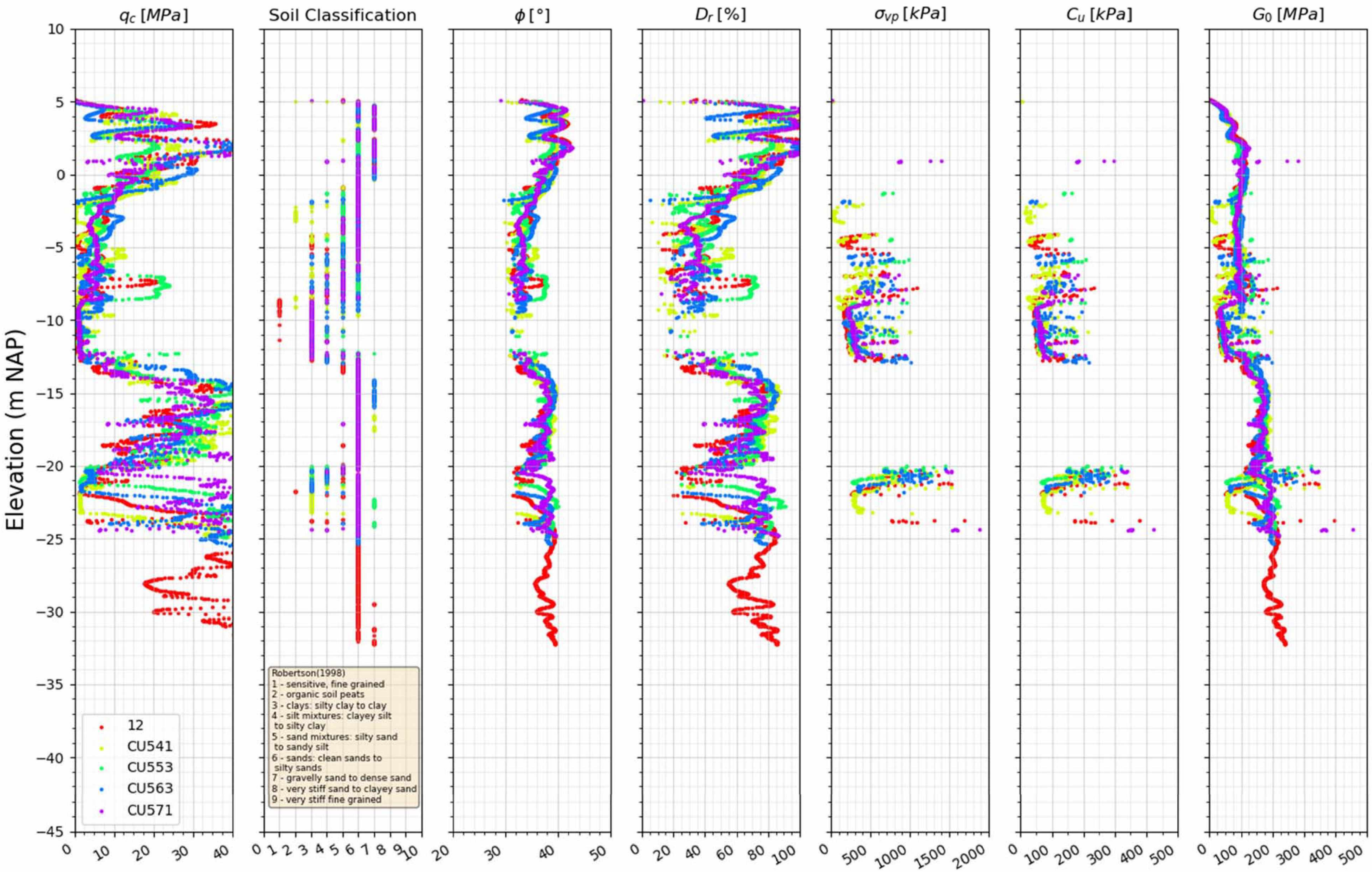

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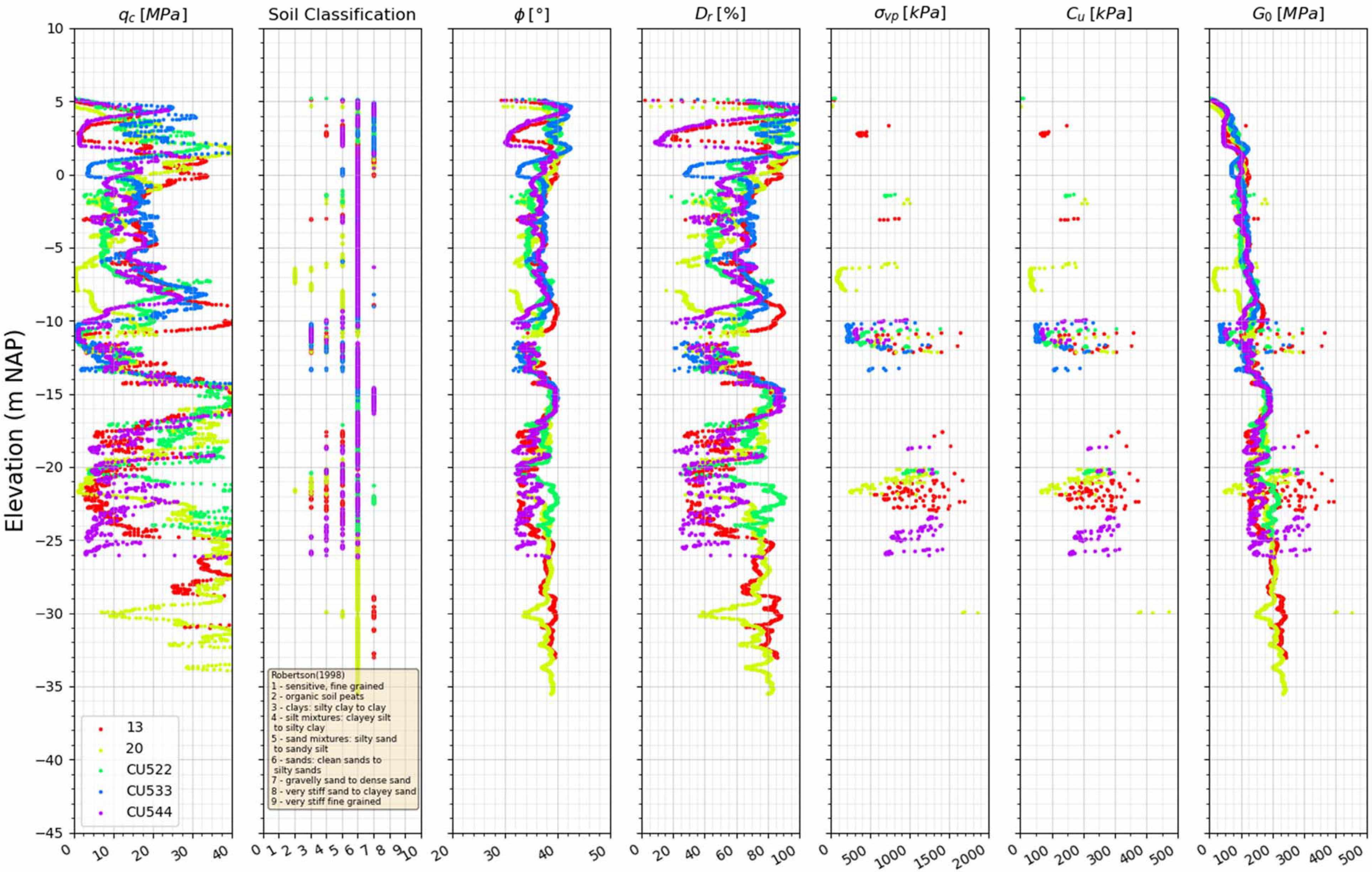

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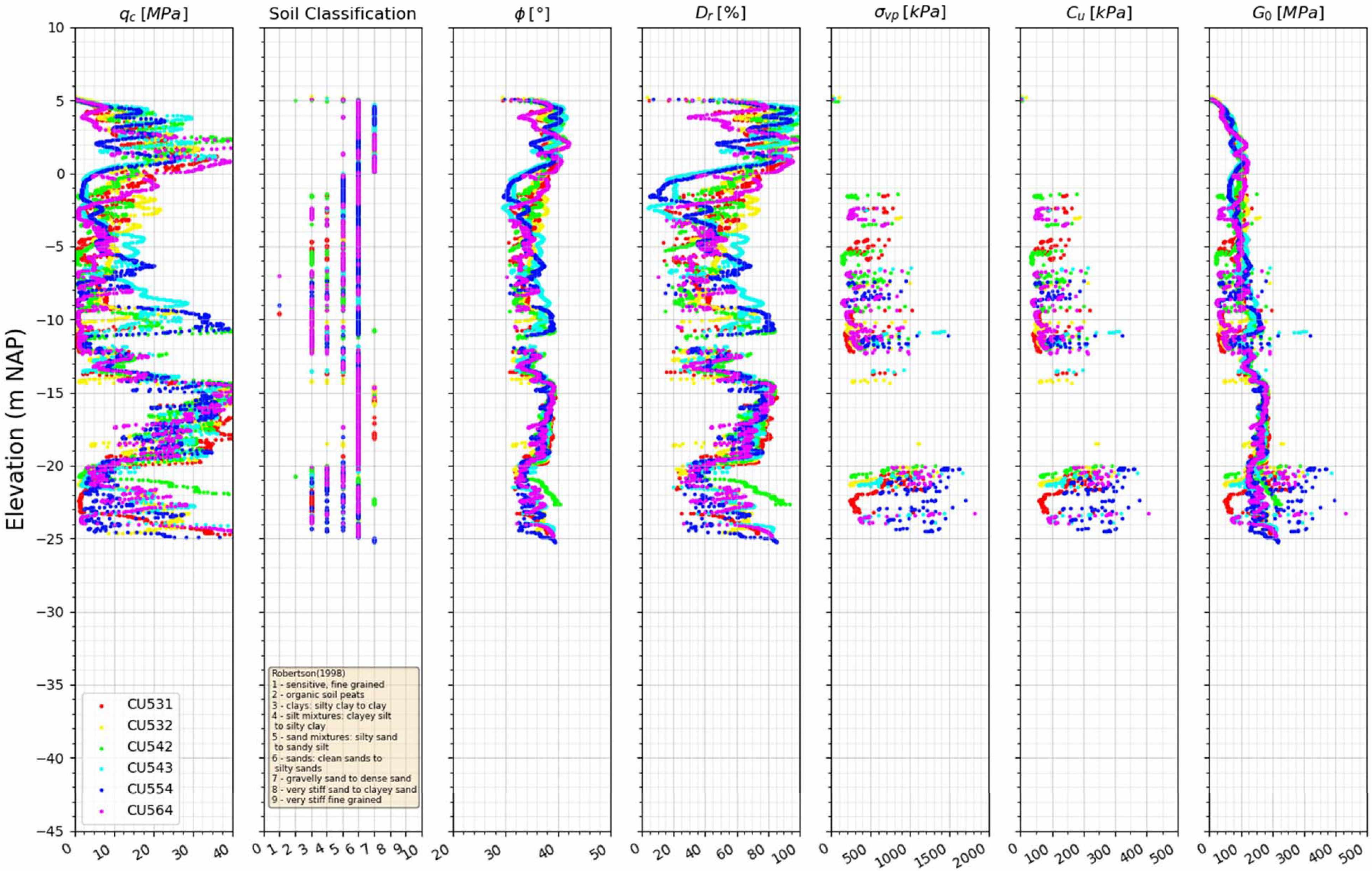

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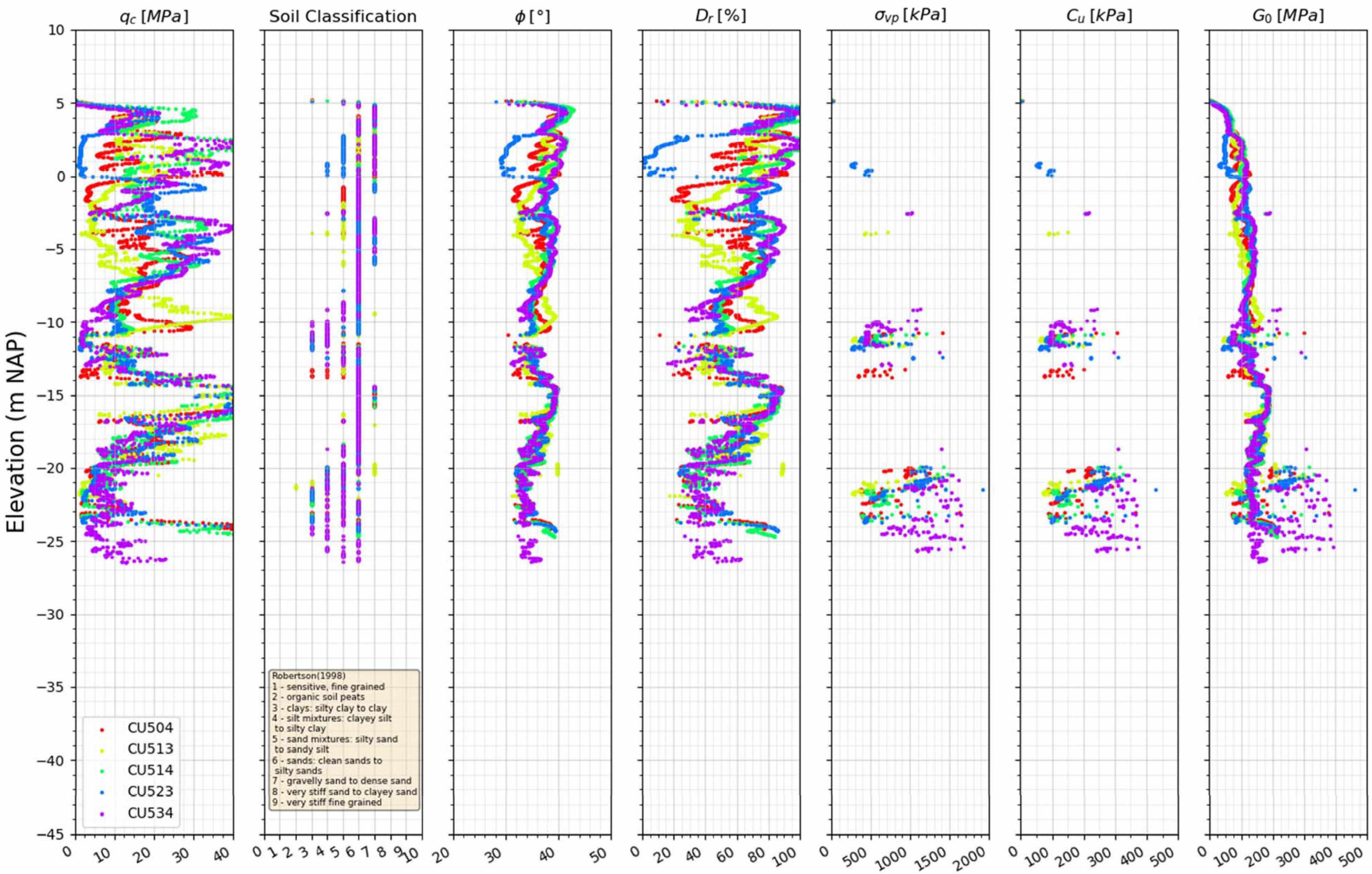

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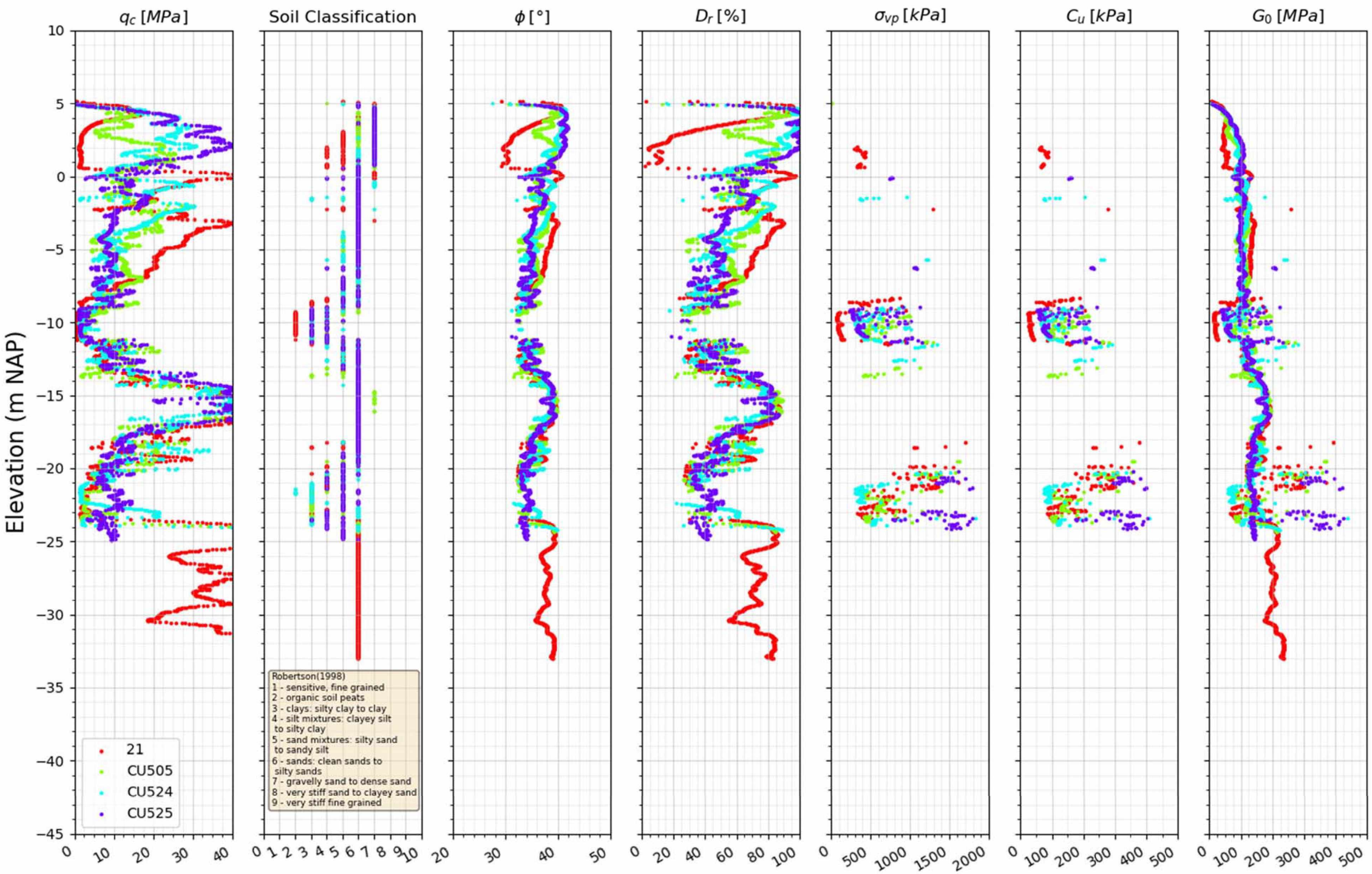

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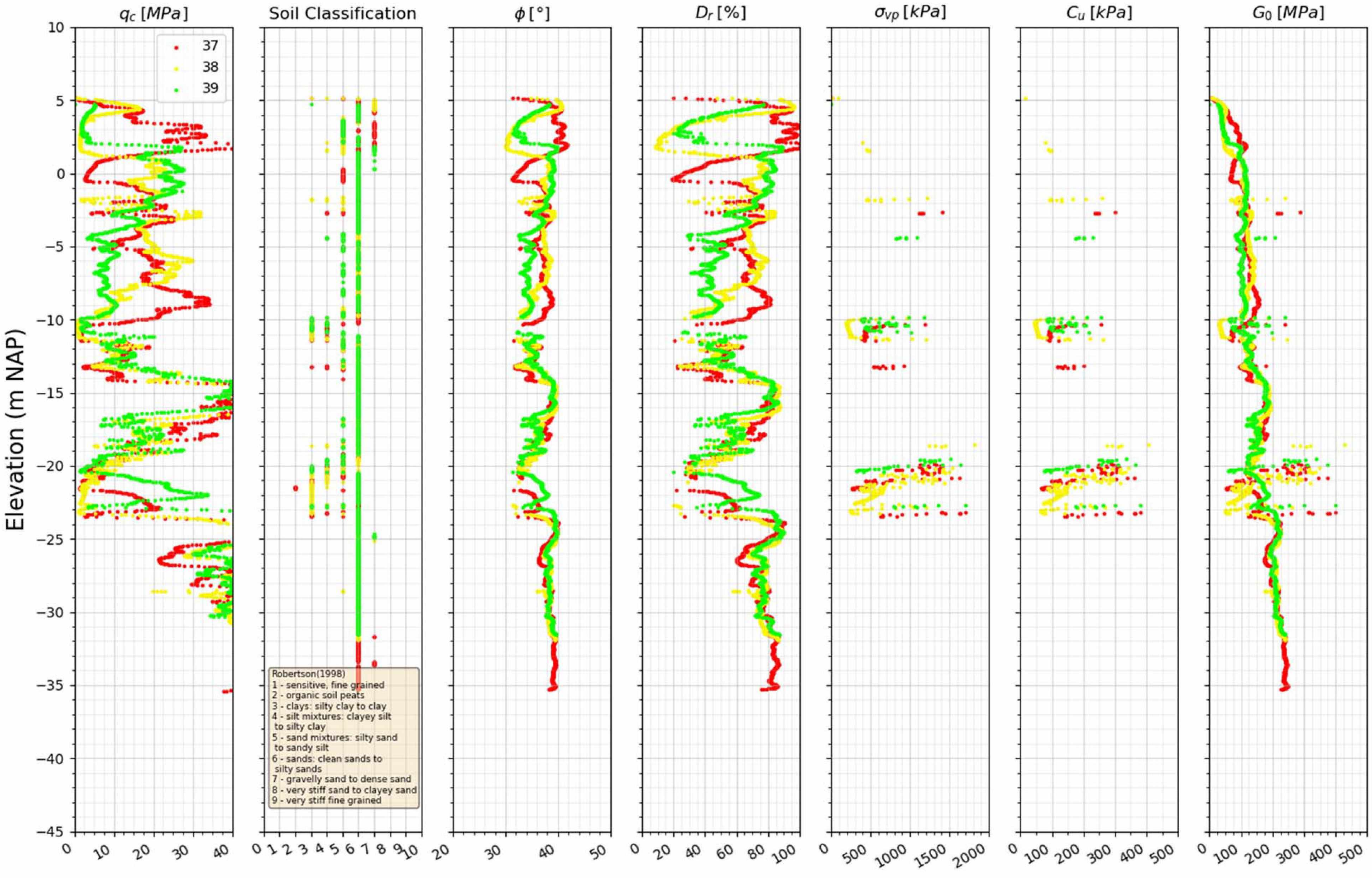

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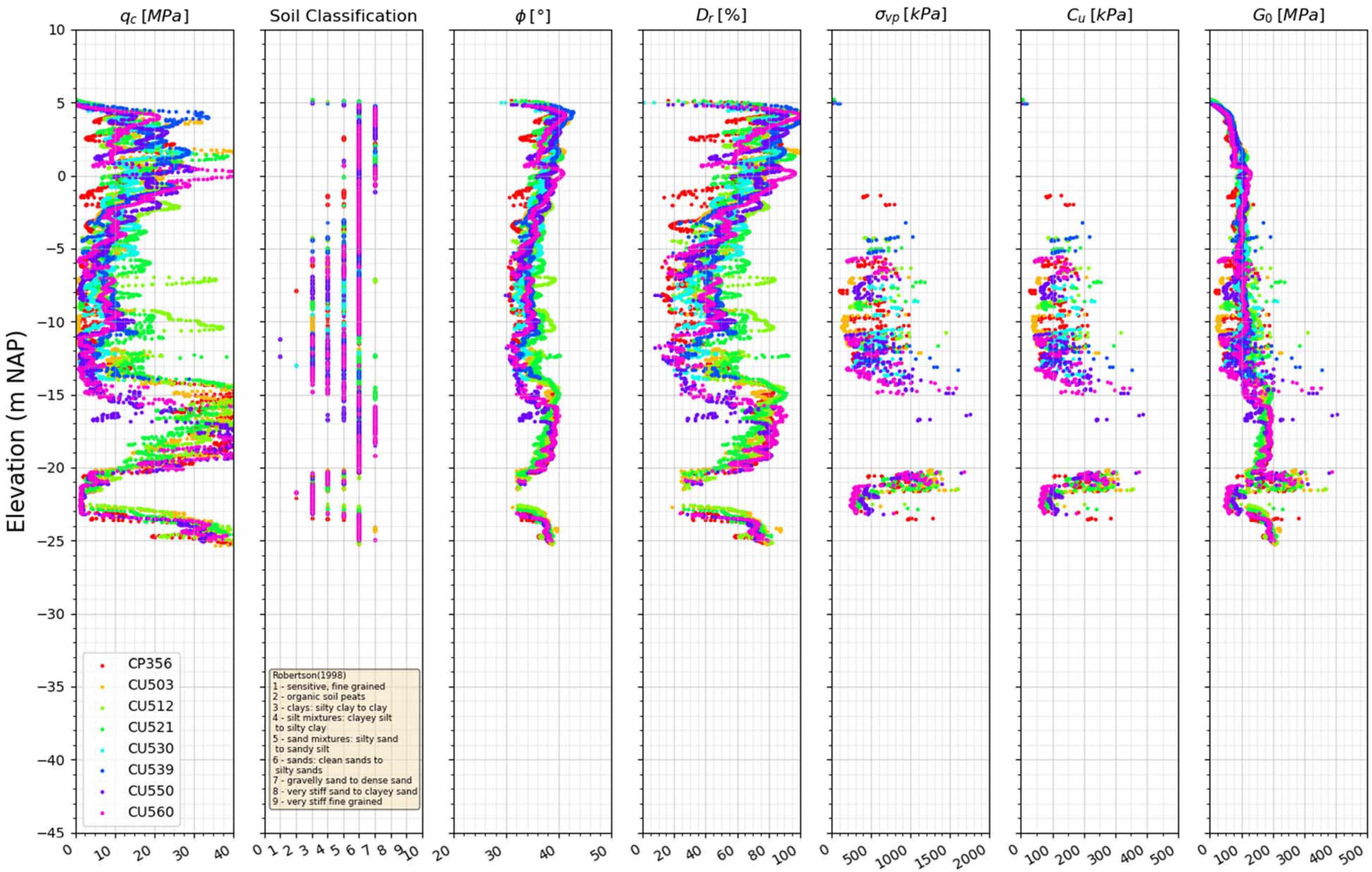

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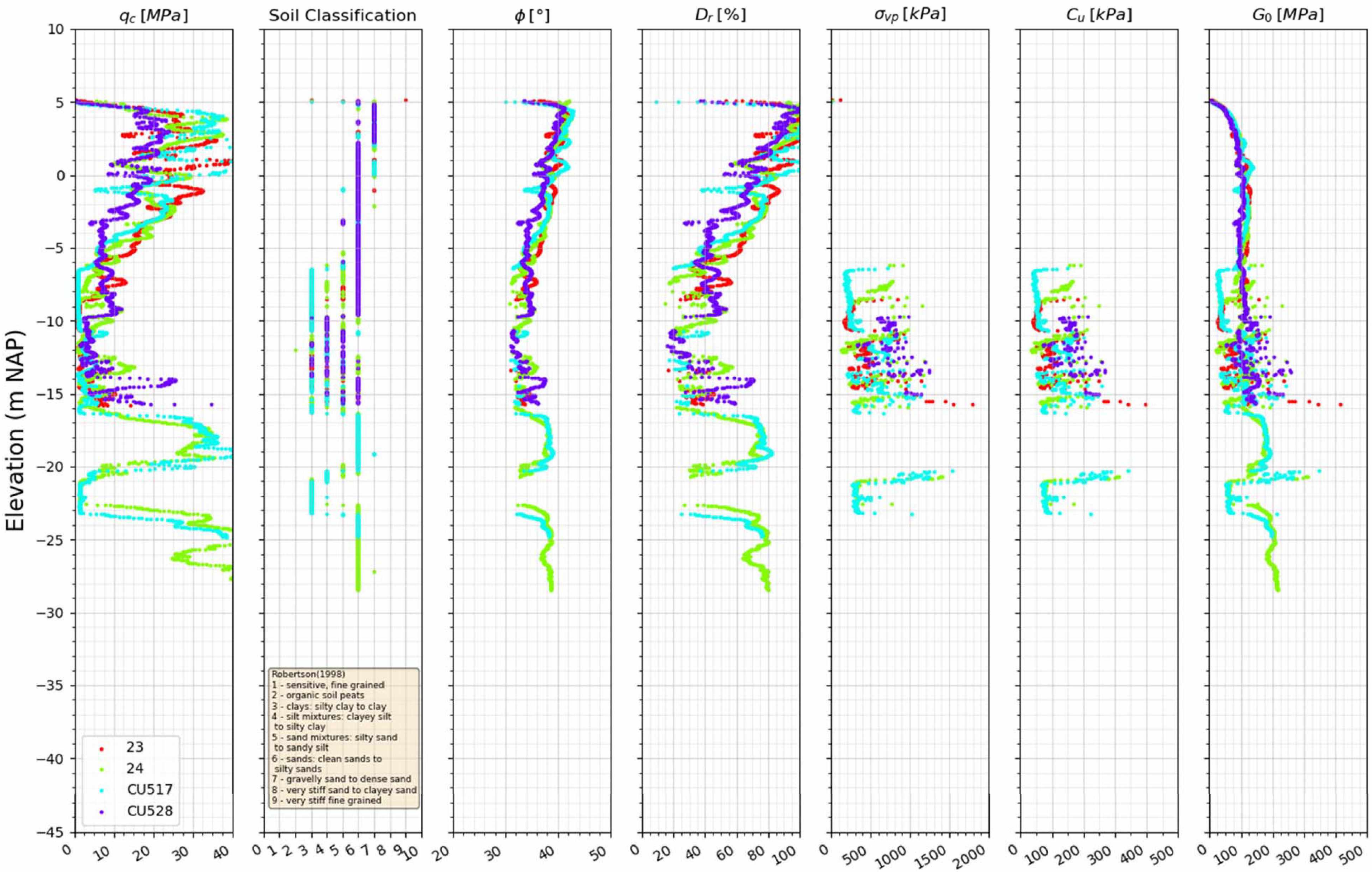

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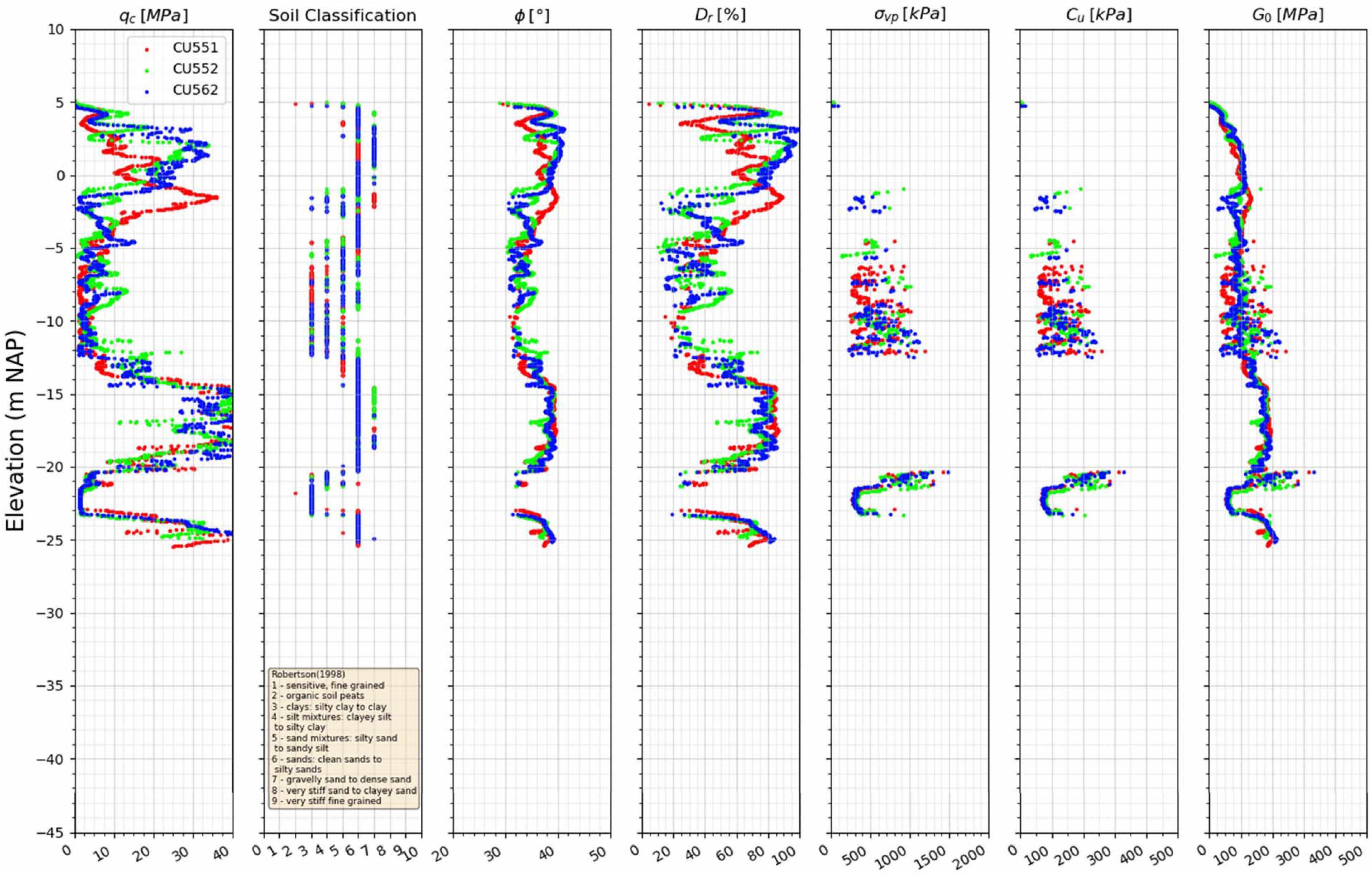

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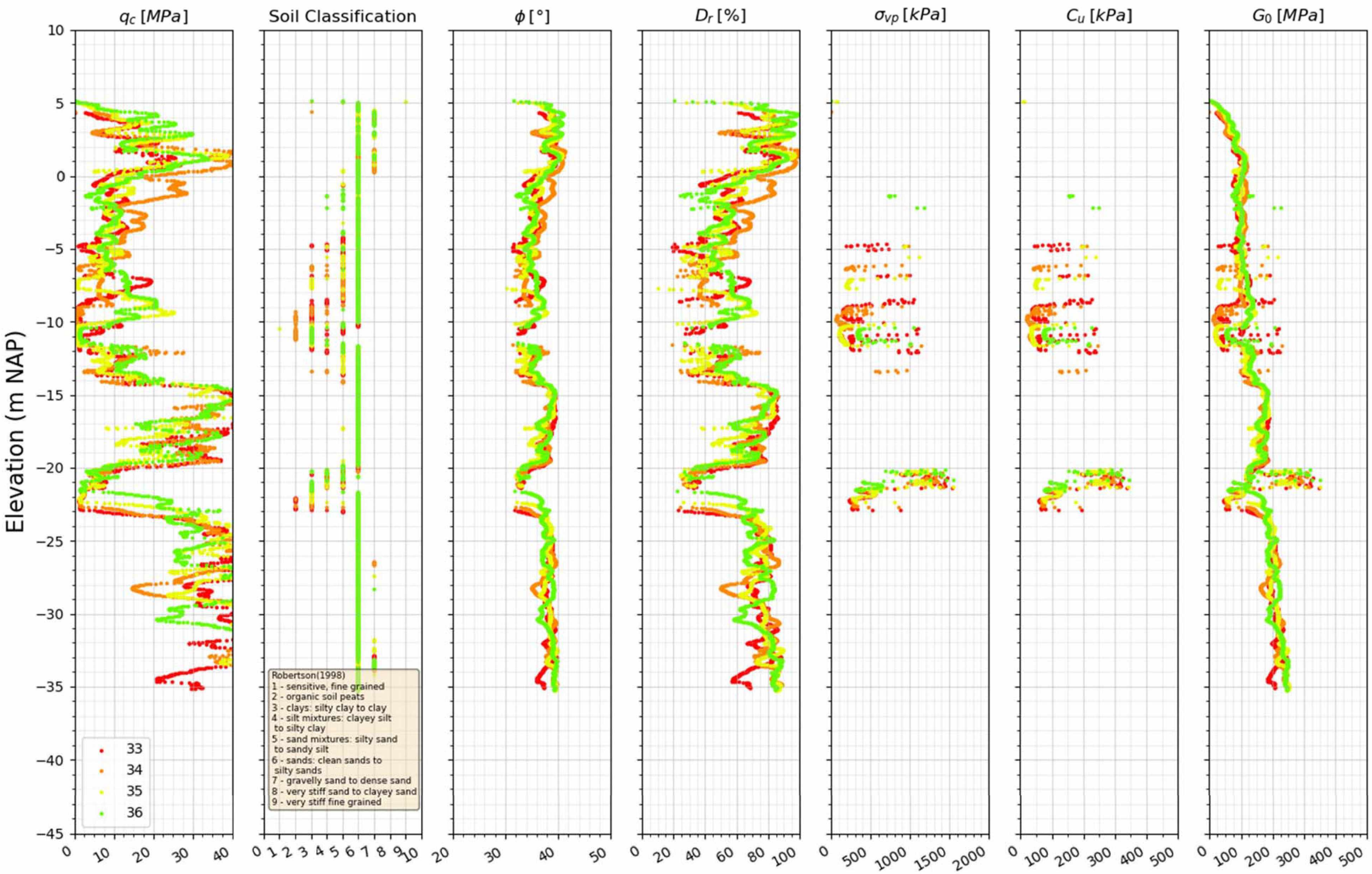

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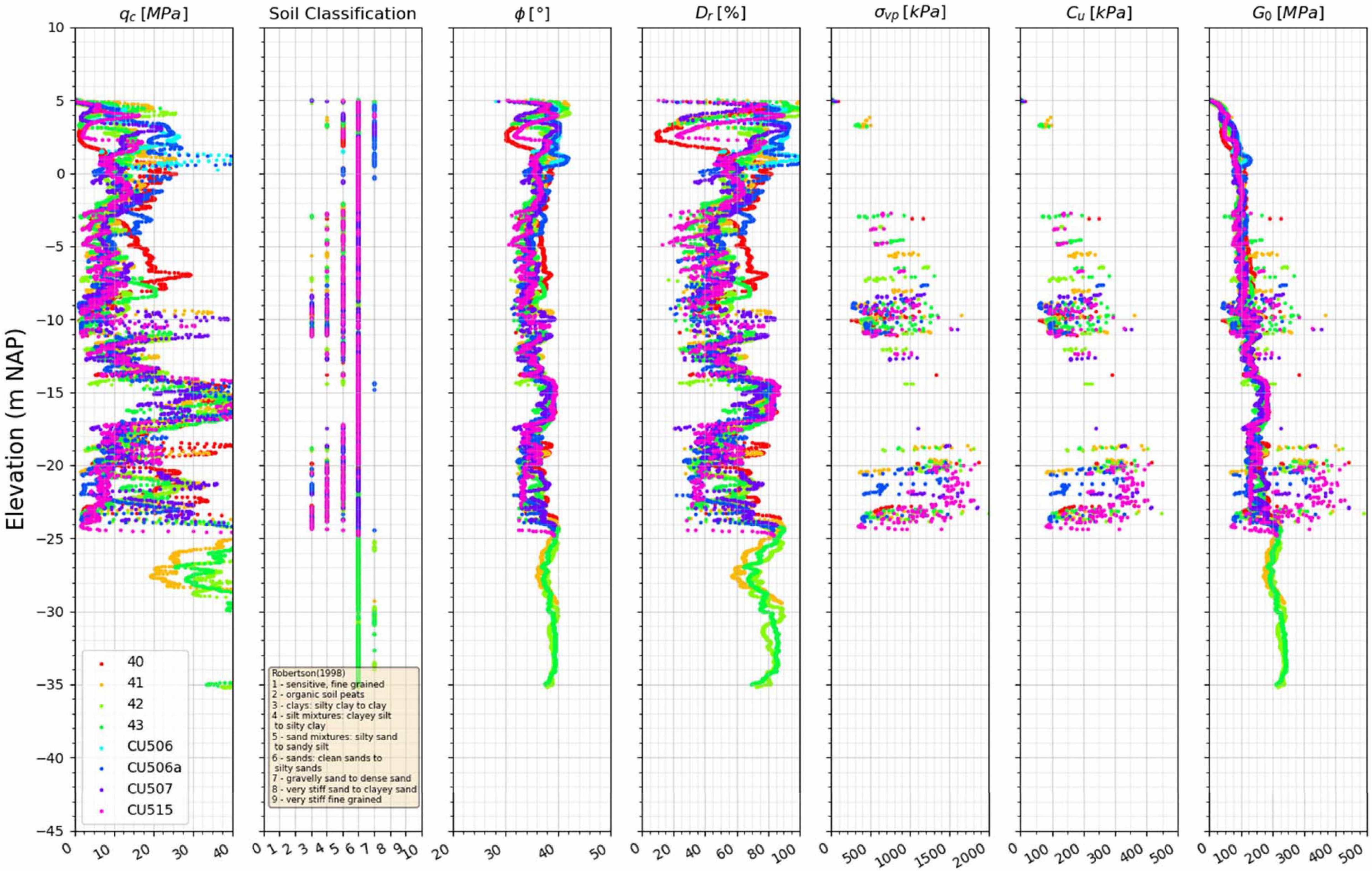

Attach.3 Figure 41- CPTs