RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE

JOB DESIGN SPECIFICATION FOR GEOTECHNICAL RECOMMENDATIONS

FEASIBILITY PHASE

RDCG REFINERY PLANT AREA

	2-9-2021	issued for local authorities		2E	
B	26/02/2021	IBD	SGI		
A	22/12/2020	ISSUED FOR REVIEW	SGI		
REV.	DATE	STATUS	WRITTEN BY (name & visa)	CHECKED BY (name & visa)	APPROV./AUTHOR. BY (name & visa)
DOCUMENT REVISIONS					

INDEX

1. FOREWORD

The present report deals with the geotechnical and environmental 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 Technip is responsible for :

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

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

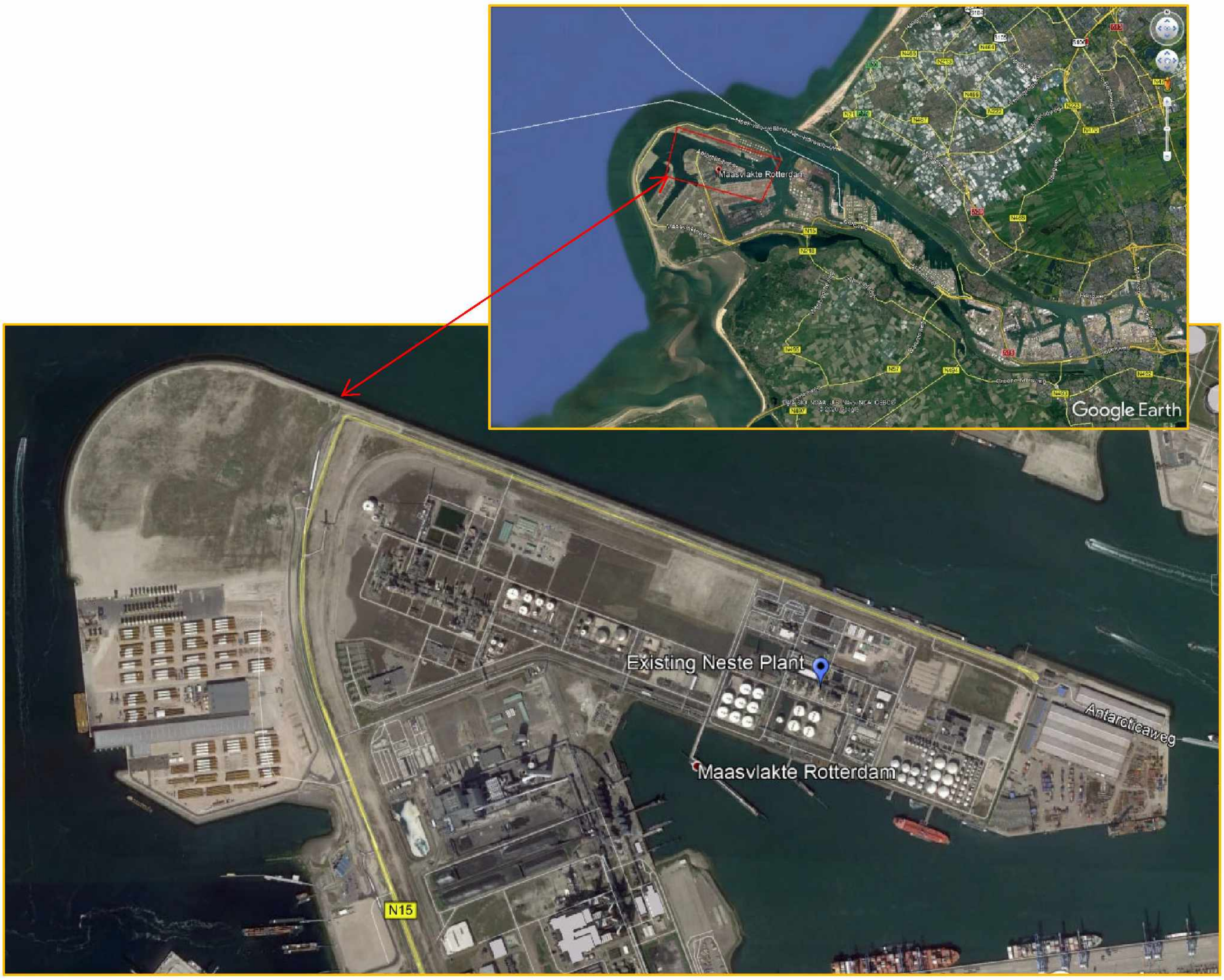


Figure 1.1: Maasvlakte industrial site (from Google earth).

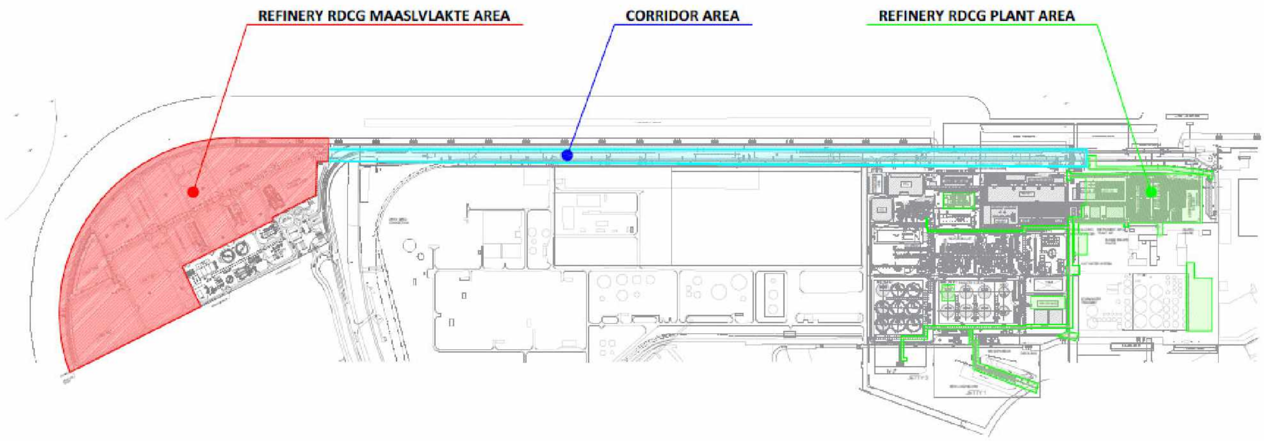


Figure 1.2: Locations of Areas scope of Technip work.

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.

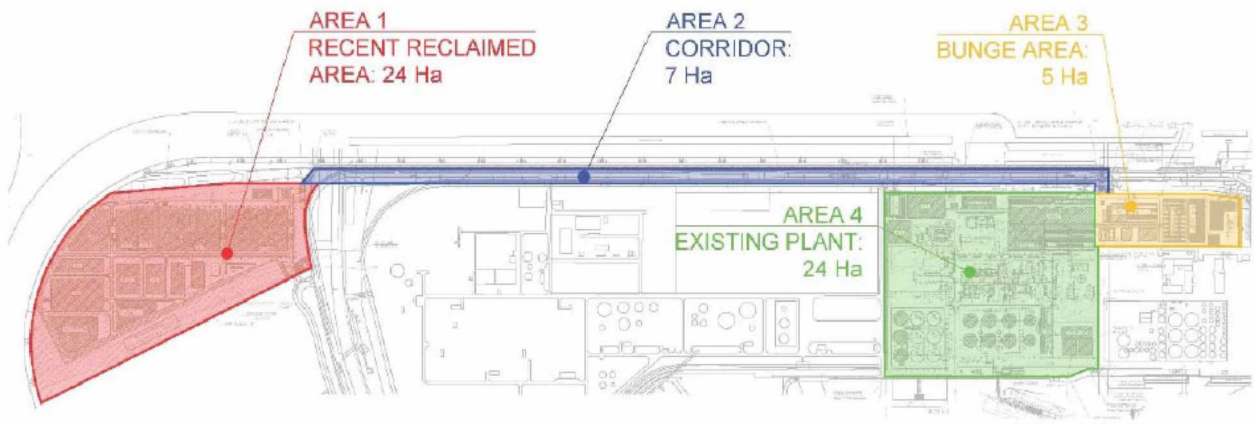


Figure 1.3: Locations of Areas scope of RSK campaign 2020

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

The present report focuses only on **Refinery RDCG Plant Area**. Separate reports have been developed for the other areas.
Currently the **Refinery RDCG Plant Area** development involves the construction of the following main units:

- Unit 040 – Existing Tank Farm (Refinary) – Naptha Tank and Propane Storage
- Unit 041 – New Tank Farm (Refinary+Blake) – RJF Storage, Diesel Storage and Feed Tanks
- Unit 052 – Utilities
- Unit 075 – Technical Building - Main Substation and Secondary Substation
- Unit 080 – Interconnecting rack
- Unit 070 – Existing Control Building → only Refurbishing
- Unit 045 – Jetty 1
- Unit 046 – Jetty 2

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;
- suitability of excavated material to be used as fill;
- description of thermal and electrical condition at the site;
- 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. REFERENCE DOCUMENTS

2.1. Reference documents

[1] 080871C 000 RT 1410 001- Soil investigation Final Factual Report - Maasvlakte Industrial Site in Rotterdam, the Netherlands - RSK Netherlands, 20 June 2020
[2] GeoDelft – “Neste Oil B2G Project, Rotterdam – Soil investigation Report” – 428840-021 ver.01 dated September 2007
[3] GeoDelft – “Neste Oil Ovj B2G Project, Rotterdam – Geotechnical Recommendations” - 428840-0013 ver. 03 dated January 2008
[4] Deltares – “Additional Soil Investigation Neste Oil / NExBTL 2E [redacted]” – 428841-0031 ver. 02 Final dated May 2008
[5] Deltares – “History and general soil aspects of the B2G Plant (Neste Oil), Rotterdam” – Co-428841-0027 ver. 02 Final dated May 2008
[6] Deltares – “Additional Soil Investigation Part 2 Neste Oil / NExBTL 2E [redacted]” – 428841-0058 Final dated July 2008
[7] Technip - “Job design specifications for geotechnical recommendations – Complementary document” - 2305-000-JSD-1422-02 revision A.
[8] Co-428841 v02 - NExBTL Plant Project Neste Oil/ Technip Italy S.p.A / Vertical seismic profiling (2008)
[9] https://www.dinoloket.nl/en - Data and Information on the Dutch subsurface

2.2. Reference standards

[10] EN 1990. Eurocode: Basis of Structural Design
[11] EN 1997–1:2013. Eurocode 7. Geotechnical design – Part I: General rules.
[12] NEN EN 1997–1+C1+A1: 2016/NB:2019. National Annex to Eurocode 7. Geotechnical design – Part I: General rules

2.3. Bibliography

[13] Ahlvin R.G. and Ulery H.H. (1962), “Tabulated values for determining the complete pattern of stresses, strains and deflections beneath a uniform load on a homogeneous half space”. Highway Research Board, Bulletin 342, pp.1-13.
[14] Atkinson J.H. and Sallfors G. (1991), “Experimental determination of soil properties. General Session 1”, Proc. X ECSMFE, Florence, Vol. 3, pp.915-956.
[15] Baldi G., Bellotti R., Ghionna V.N., 2E [redacted] M., Pasqualini E. (1986) “Interpretation of CPTs and CPTUs: 2nd part, Drained penetration of sands” 4th International Geotechnical Seminar, Singapore.
[16] Baldi G. et al. (1989), “Modulus of sands from CPT’s and DMT’s”, Proc XII ICSMFE, Rio de Janeiro.
[17] Berardi R. and Lancellotta r. (1991), “Stiffness of granular soils from field performance”, Geotechnique, Vol.41, n.1, pp.149-157
[18] Bolton (1986) “The strength and dilatancy of sands” Geotechnique 36 , n° 1.
[19] Brinch-Hansen (1970), “A revised and extended formula for bearing capacity”, Bull.n.28, Danish Geotechnical Institute, Copenaghen.
[20] Boussinesq, J. (1885), “Application des potentiels `a l’é de l’equilibre et du mouvement des solideselastiques”, Gauthiers-Villars, Paris;

[21] Chen B.S. e Mayne P.W. (1996), "Statistical relationship between piezocone measurements and stress history of clays", Canadian Geotechnical Journal, n.3, pp.488-499.

[22] Elson W.K "Design of laterally loaded piles", Report n°103 – Ciria, London;

[23] Florin V.A. (1959), "Oshovi Mekaniki Gruntov", Mosca;

[24] [redacted] R. (1991), "Some recent development on the behaviour of shallow foundations", Proc. X ECSMFE, Florence, Vol. 3, pp.1003-1030.

[25] Garizio, G.M. (1997). "Determinazione dei parametri geotecnici ed in particolare di Ko da prove penetrometriche". Tesi di Laurea, Politecnico di Torino.

[26] Grant R. et al. (1974), "Differential settlement of buildings", JSMFED, ASCE, Vol.100, pp.973-991

[27] [redacted] D.E. (1976) "A statistical study of static cone bearing capacity, vertical effective stress and relative density of dry and saturated fine sands in a large triaxial test chamber" Master's Thesis, University of Florida.

[28] Hazen A. (1892) "Some physical properties of sands and gravels with special reference to their use in filtration" Massachusetts state Board of Health, 24th Annual Report

[29] Houlsby G.T. and The, C.I. (1988) "Analysis of the Piezocone in Clay" Penetration Testing 1988, Vol. 2, [redacted] The Netherlands, pp.777-783

[30] [redacted] M., Ladd, C.C., [redacted] J.T. and [redacted] R. (1985) "New developments in Field and lab Testing of soils" Proceedings of 11th International conference on Soil Mechanics and Foundation Eengineering, Vol.1, San Francisco, Calif., pp.57-154.

[31] [redacted] M., [redacted] V.N., [redacted] R., [redacted] E. (1988) "New correlations of penetration tests for design practice" Proceedings of I International Symposium on Penetration Testing, ISOPT I, [redacted]

[32] [redacted] M., Lo [redacted] D.C.F. and [redacted] G.M. (2001). "Correlation between Relative Density and Cone Resistance for Silica Sands". Jubilee Volume 75th Anniversary of K. Terzaghi's. [redacted] [redacted]

[33] Kulhawy F.H. e [redacted] P.W. (1990), "Manual on estimating soil properties for foundation design", Cornell University, Geotechnical Engineering Group, Ithaca, New York, EL-6800, Research Project 1493-6

[34] [redacted] T., [redacted] P.K., [redacted] J.J.M. (1997). "Cone Penetration Testing in Engineering Practice" Blackie Academic and Professional, London.

[35] [redacted] P.W., [redacted] G.J. (1993) "Gmax - Qc relationship for clays" Geotechnical Testing Journal, GTJODJ, vol.16, n°1, pp.54-60

[36] Matlock and Reese (1960), "Generalised Solution for Laterally Loaded Piles", Journal of Geotechnical Engineering Division", ASCE, October 1960

[37] [redacted] G.G. (1953), "The bearing capacity of foundations under eccentric and inclined loads". Proc. III ICSMFE, Zurich, Vol.1

[38] [redacted] G.J. e [redacted] K.H. (1991), "Correlation of initial tangent modulus and cone resistance". Proc. International Symposium on Calibration Chamber Testing, Postdam, New York, 351-362, Elsevier

[39] Robertson [redacted] [redacted] R.G., [redacted] A. (1983) "SPT-CPT Correlations" Journal of the [redacted] Division, ASCE, Vol. 109.

[40] [redacted] P.K., [redacted] R.G. (1984) "Guidelines for use and interpretation of the electric cone penetration tests" Soil Mech. Series n° 69, Department of Civil Engineering, University of British Columbia.

[41] [redacted] P.K., [redacted] R.C. (1986) "Guidelines for use, interpretation and application of the CPT and CPTU" Soil Mechanics Series 105, Department of Civil Eng., Univ. Of British Columbia, Vancouver.

[42] [redacted] P.K., [redacted] R.G., [redacted] D., [redacted] J. (1986) “The interpretation of Begemann friction jacket cone results to give soil types and design parameters” Proc. VII ECSMFE, Brighton (U.K.).

[43] [redacted] P.K. (1990). "Soil Classification using the Cone Penetration Test". Canadian Geotechnical Journal, n° 1.

[44] [redacted] K. and [redacted] R.B. (1948), “Soil mechanics in engineering practice”, Wiley & Sons, New York.

[45] [redacted] N.A. (1951), “Soil mechanics”, Edit. Stroitielstvo i Archiketura, Moscow (in russian).

[46] [redacted] W.C.B., [redacted] J.K. (1981) “Cone resistance, relative density and friction angle” Specialty Session on Cone Penetration Testing and Experience, ASCE, St Louis, Norris & Holtz editors, pp.178-208.

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 started in 2008 and was completed in 2013.

Refinery RDCG Plant Area object of the present document is located on the old reclaimed area of Maasvlakte, within and adjacent (eastward) to the Existing Plant,.

Originally water depth was about 5 to 8 m below mean sea level (NAP), because of the reclamation and landfill surface was raised to about +4 ÷ +5 m NAP.

Surface level of the area is located at approximately +5.3 m to +5.6 m NAP, at the time of the ground investigation campaign. It is assumed that this area will be levelled to a Final Ground Level elevation +5.20 NAP (corresponding to HPP), as for the Existing Plant.

4. GROUND INVESTIGATION

4.1. Soil investigation RSK Netherlands 2020 for Neste Expansion Project

DEFINITION-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.[1]), for the area object of the present document the geotechnical investigation comprised:

AREA 3 - Bunge Area

- N° 27 cone penetration tests with pore pressure measure (CPTU tests) performed up to a depth of 35 meters below ground level;
- N° 3 boreholes, all drilled to a depth of about from about 30 m below ground level. Undisturbed samples have been collected during boring;
- N° 1 borehole was equipped with standpipe piezometers at different depth for the groundwater level measurement. Water samples were taken for chemical analyses;
- N° 1 in situ electrical resistivity tests;
- N° 1 in situ thermal resistivity tests;
- Geotechnical laboratory tests on selected remoulded and undisturbed

4.2. Previous available soil investigations for the original NexBTL Plant Project

4.2.1. Desk study - NexBTL Plant Project

For the original Neste Plant Project, Deltares provided a Report (Ref.Doc.[5]) with the description of the geology and geological history of the area, with particular attention to historic data on reclamation, original water depths and deposited materials.

4.2.2. First investigation - Deltares 2007

The following geotechnical ground investigation activities were carried out at the site by Deltares in September 2007, as detailed in their Final Report Ref.Doc.[2] in support of the design of the NexBTL Plant Project:

- N° 109 CPT's with measurement of cone resistance and local friction up to a depth of 30 m below surface level;
- N° 3 electrical resistivity measures with special probe.

4.2.3. Second investigation - Deltares 2008

An additional site investigation has been carried out by Deltares in April/May 2008, as detailed in their Final Reports Ref. Doc.[4] and [6], in order to fully clarify the detailed geotechnical soil profile at the site and the design parameters to be used for geotechnical calculation in support of the design and construction of the NexBTL Plant Project.

The soil investigation consisted in:

- N° 6 cone penetration tests with pore pressure measure (CPTU tests) performed up to a depth of 40 m below surface level;

- N° 15 boreholes, all drilled to a depth of about 30 m below ground level. Undisturbed samples have been collected during boring;
- N° 8 water standpipes in 4 boreholes for groundwater level measurements. From these standpipes water samples were taken for chemical analysis;
- Laboratory tests on selected remoulded samples from SPT and on undisturbed samples.
- N° 5 in situ electrical resistivity tests;
- N°2 tests with seismic cone were performed in Process area to a depth of approximately 20 m below present ground level, to evaluate the dynamic properties of foundation soil and to estimate the deformability parameters of granular material
- N°2 permeability tests;
- N°7 dissipation tests at 2 different locations.

Further CPT tests have been carried to refine the pile design calculations in order to comply with NEN 6740 requirements. Accordingly, the final soil investigation consisted in:

- N° 275 CPT with measurement of cone resistance and local friction up to a maximum depth of 35 m below surface level.

4.3. Data available from National Website

For the units not covered by data coming previous specific campaign, information extracted from the National Website https://www.dinoloket.nl/en (which collects available geotechnical information on the Netherlands territory) have been used for preliminary evaluation.

In particular the following investigations have been selected from the National database:

- N° 2 cone penetration tests (CPT tests) performed up to a depth of 25 and 37 m below surface level

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

5. GROUND AND GROUNDWATER CONDITIONS

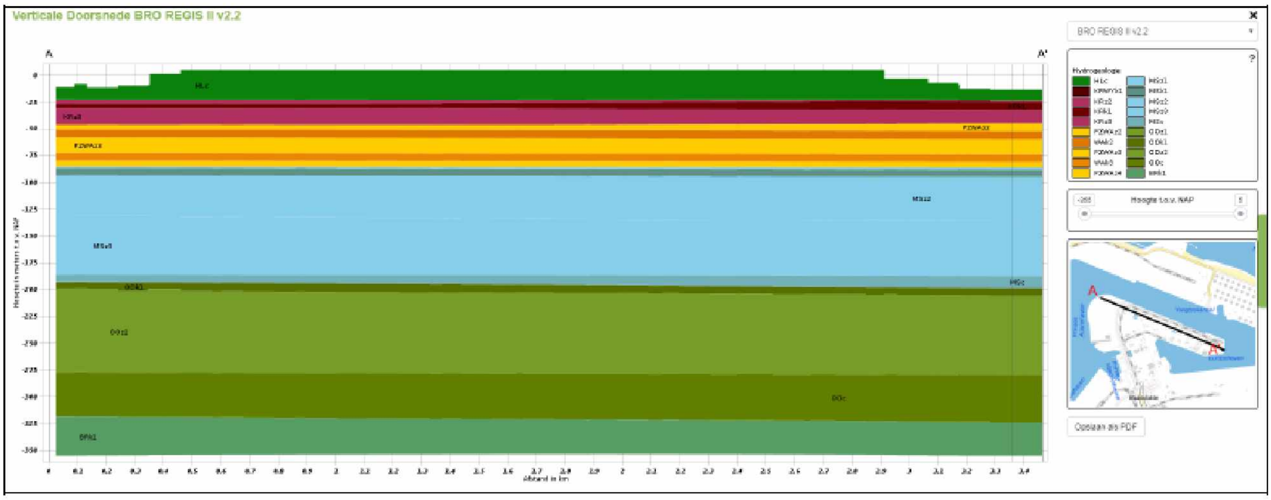
5.1. Geology

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 sands** unit consists of Holocene deposits. They are encountered from surface to up to a depth from about -20 to -21 m NAP, locally up to -18 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 $D_r \geq 60\%$. Levels/lenses of sandy silt/silty sand from loose to medium dense are recovered at different depths.
- **Clay to clayey silt** lenses of variable thickness (generally modest) within the above deposits. These lenses are found frequently at about -2 m ÷ -3 m NAP, with a maximum registered thickness of about 1.5 m, locally they are encountered at about -12 m ÷ -13 m NAP, with a maximum registered thickness of about 0.5 m. A thicker layer, of about 1.5 m to 3 m thick, is often encountered beneath -18 ÷ -20 m NAP. Possible random presence of localized minor levels of fine grained material can be found even in the first meters from the FGL. Same minor lenses of peat have been singled out during boring at various depths.
- **Pleistocene sands,** medium to coarse, densely packed, placed generally beneath -23 m NAP, but locally the top of this layer raise up to -21 ÷ -22 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 in the recently investigated Area 3. Elevations are in agreement with those revealed in the Existing Plant Area.

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

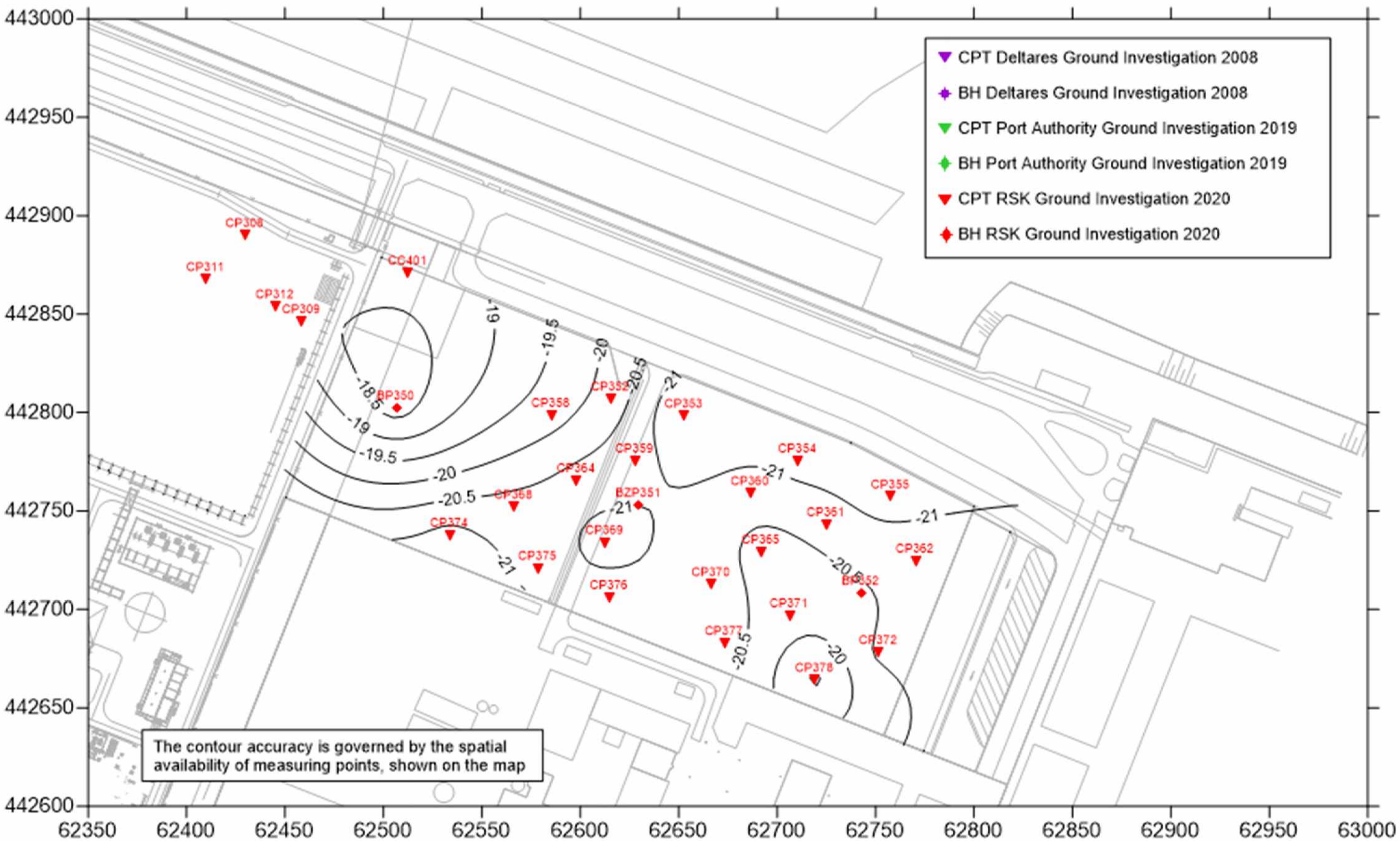


Figure 5.1: Contours of the top of the Clay Layer that overlies the Pleistocene sands in the 2020 investigated Area 3

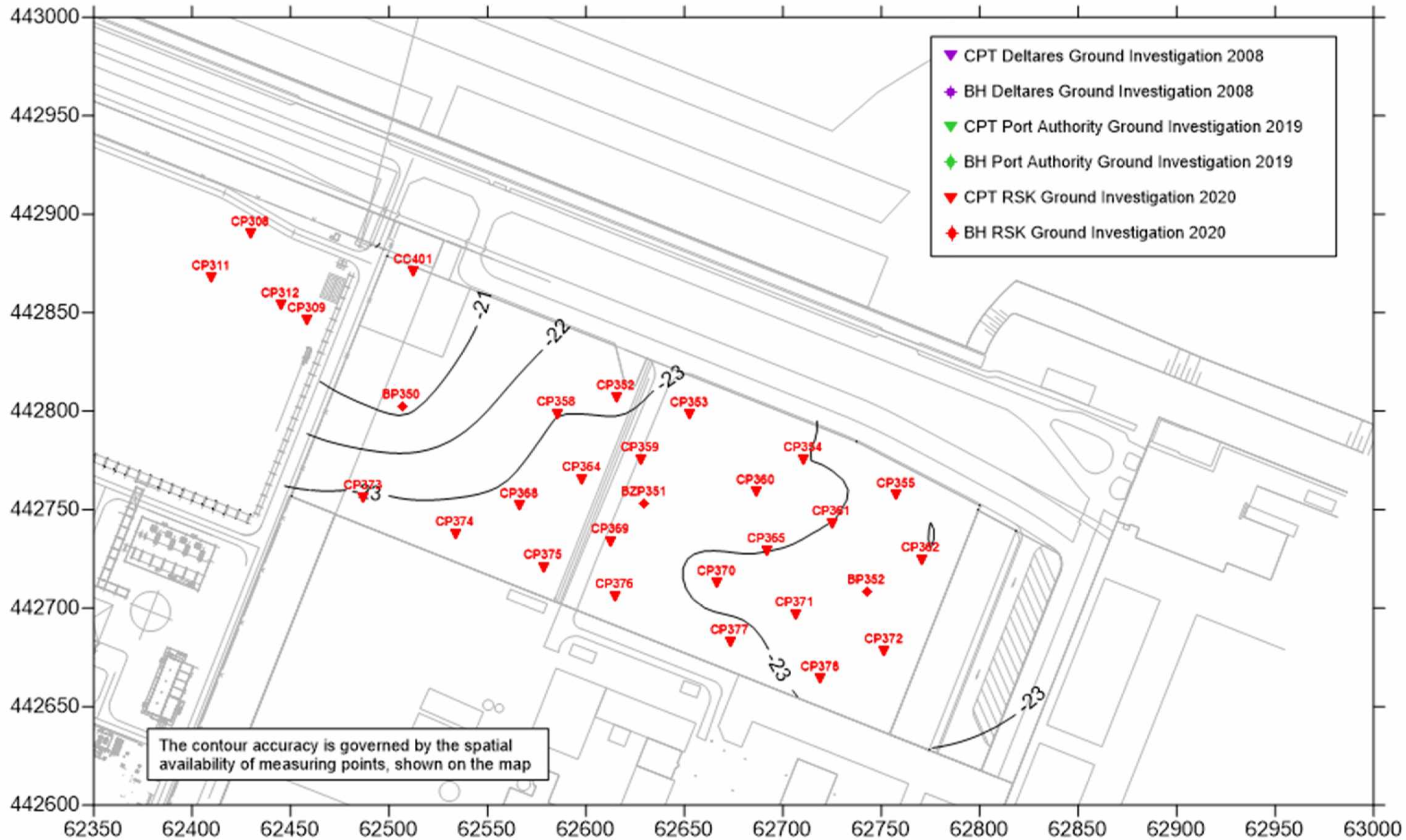


Figure 5.2: Contours of the Top of the Pleistocene sands in the 2020 investigated Area 3

5.2.1. Design Soil Profiles

In accordance with the available ground investigation data, typical soil profiles to be used for design are summarizes in Table 5.1 (see in key plan of ATTACHMENT 1).

Table 5.1: Typical soil profile

UNIT	Structure	Reference CPTs/boreholes	Clay to clayey silt levels	Top of Pleistocene sand
-	-	-	m NAP	m NAP
40	Propane Storage	M260, M271, M270, M112, M113, M114, M117 W004, W006, W008, W010 S25, S26, S27, S116 B26	+1.5 ÷ +1 +0.5 ÷ 0.0 -2 ÷ -3 -9.5÷ -10.5 -18 ÷ -20	-20 ÷ -21
	Naphtha Tank	CP322, CP323 S32	-3 ÷ -5 -19 ÷ -21.5	-21.5 ÷ -22
41	RJF Storage	CP350, CP351, CP352, CP357, CP358, CP363, CP364, CP367, CP368 BP350	-2 ÷ -3.5 -18 (-20) ÷ -21(-23)	-21 ÷ -23
	Diesel Storage	CP353, CP354, CP355, CP359, CP360, CP361, CP362, CP365, CP369, CP370, CP371, CP372, BP352, BZP351	-2.5 ÷ -3.5 -21 ÷ -23.5	-23 ÷ -23.5
	FEED Tank	CPT000000134202, CPT000000029201	-8.5 ÷ -9 -18.5 ÷ -20.5	-20.5 ÷ 23
75	Main Substation	NA	NA	NA
	Secondary Substation	CPT000000134202	-13.5 ÷ -14 -19 ÷ -23	-23
80	New racks and/or sleepers	CP375, CP376, CP377, CP378, M111, M110, M115, M116 M121	-2.5 ÷ -3 -21 ÷ -23.5	-23.5 ÷ 24
	Existing rack and/or sleepers (Revamping)	CP327, M120, M124, M130, M133, M135, M136, M141, M143, M146, M149, M150, M151, M152, S16, S17, S85, S86, S87, M097	+0.5 ÷ 0.0 -9.5÷ -10.5 -18 ÷ -21	-21 ÷ -23

It is anticipated that, at the present design stage:

- for shallow foundations two calculation soil profile has been identified for relevant ULS and SLS checks (see Section 14);
- for the tanks the specific profile at their location has been considered in calculations (see Section 15)

5.3. Groundwater conditions

The available groundwater level investigation includes standpipes piezometers, to measure the piezometric level in the lower sand and 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 also considered.

Table 5.2 summarise all the available data in terms of elevation (in m NAP) of measured ground water level. Regarding 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 latest RSK ground investigation campaign.

Table 5.3 shows the trend of the water level data from 24/02/2020 to 07/12/2020 in BZU351.

According to available information groundwater table in **Refinery RDCG Plant Area** is localized at minimum elevation of +0.4 m NAP (about 4.8 m below present g.l.) to a maximum elevation of +1.7 m NAP (about 3 m below present G.L.). A representative value of +1.5 m NAP is assumed in the design.

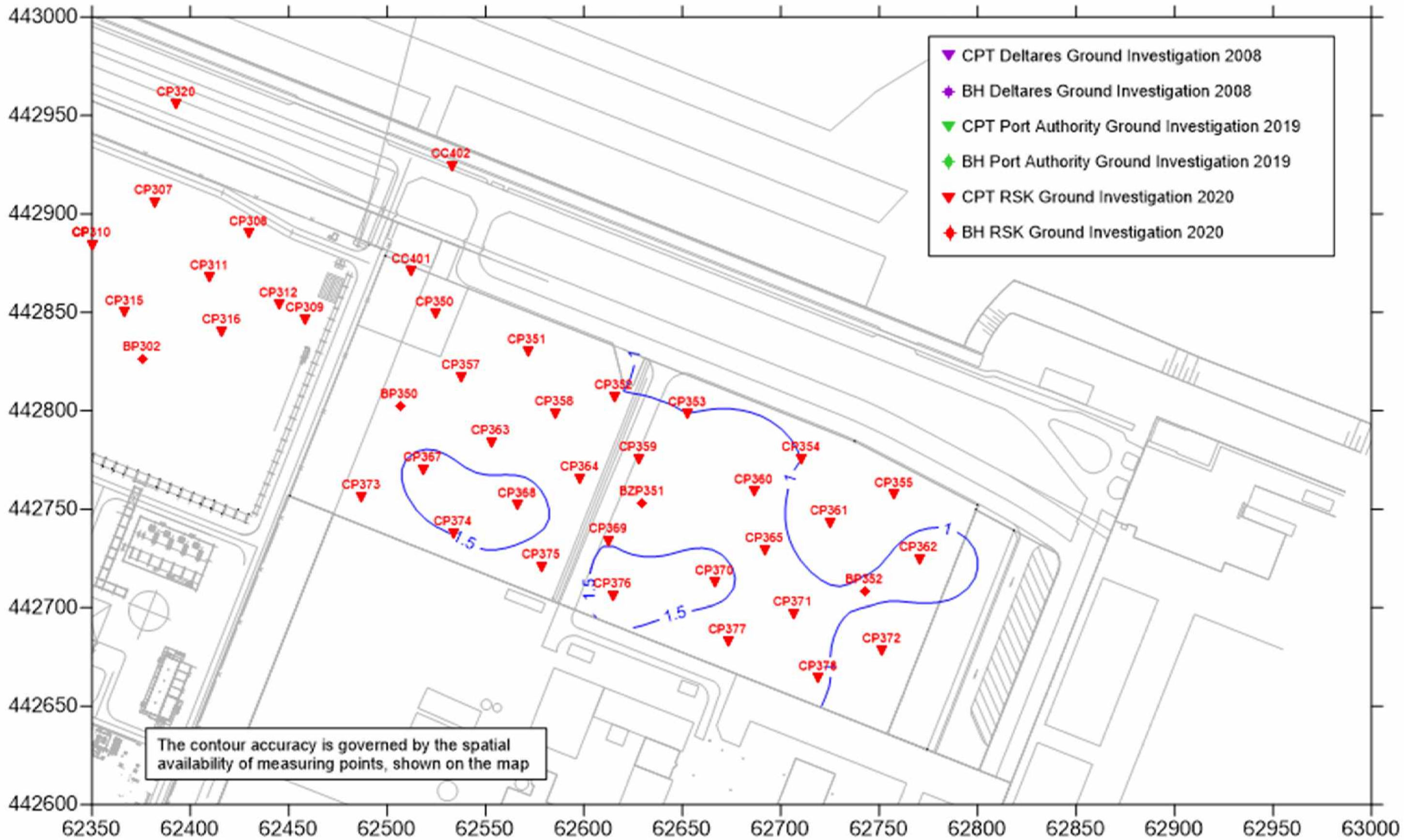


Figure 5.3: Contours of the groundwater level, based on RSK 2020 ground investigation in Area 3.

Table 5.2: Piezometric data

ID	E	N	GL Elevation (m NAP)	TYPE	BH/CPT TOE (m NAP)	GWL (m NAP)
BP350	62506.56	442802.27	5.34	BH	-24.66	1.04
BP352	62742.69	442708.09	5.55	BH	-24.45	1.05
CC401	62512.64	442870.95	5.04	CPTU	-20.96	1.40
CP322	62081.92	442723.03	5.10	CPTU	-27.04	0.70
CP323	62069.53	442689.76	5.13	CPTU	-30.13	0.70
CP350	62524.48	442849.15	5.18	CPTU	-14.36	1.40
CP351	62571.42	442830.05	5.27	CPTU	-16.07	1.40
CP352	62615.45	442806.92	5.22	CPTU	-23.88	1.00
CP353	62652.87	442798.24	5.05	CPTU	-25.13	1.00
CP354	62710.63	442775.53	5.23	CPTU	-24.77	1.00
CP355	62757.83	442757.22	5.30	CPTU	-26.74	0.80
CP357	62537.88	442817.05	5.50	CPTU	-14.24	1.00
CP358	62585.33	442798.54	5.10	CPTU	-27.38	1.30
CP359	62627.76	442775.46	5.21	CPTU	-24.39	1.20
CP360	62686.45	442758.94	5.25	CPTU	-28.05	1.20
CP361	62724.98	442743.07	5.33	CPTU	-24.95	0.60
CP362	62770.24	442724.59	5.33	CPTU	-23.85	1.15
CP363	62552.89	442783.96	5.32	CPTU	-16.86	1.20
CP364	62598.21	442765.42	5.38	CPTU	-24.62	1.20
CP365	62691.98	442729.23	5.35	CPTU	-24.09	1.20
CP367	62518.49	442770.26	5.25	CPTU	-25.21	1.75
CP368	62566.35	442751.85	5.29	CPTU	-30.11	1.75
CP369	62612.56	442734.03	4.92	CPTU	-24.64	1.50
CP370	62666.61	442712.65	5.38	CPTU	-24.62	1.70
CP371	62706.30	442696.70	5.24	CPTU	-24.16	1.15
CP372	62751.64	442678.10	5.48	CPTU	-23.94	0.75
CP373	62487.14	442756.19	5.30	CPTU	-25.54	1.10
CP374	62533.73	442737.65	5.34	CPTU	-25.16	1.50
CP375	62578.56	442720.95	5.34	CPTU	-25.26	1.30
CP376	62614.89	442706.12	5.20	CPTU	-25.40	1.60
CP377	62673.24	442682.63	5.19	CPTU	-24.03	1.10
CP378	62719.25	442664.10	5.13	CPTU	-24.25	1.05

Table 5.3 – Summary of the water level data to date

Piezometer	BZU 351 PP-01	BZU351 PP-02
Ground level (m NAP)	5.37	5.37
Height top of standpipe above ground level (m)	0.25	0.24
Level top of well screen (m bgl)	4.00	21.00
Level bottom of well screen (m bgl)	6.00	24.00
Date	Water level (m NAP)	
24/02/2020	1.15	0.16
25/02/2020	1.18	0.53
26/02/2020	1.17	0.44
27/02/2020	1.18	0.31
02/03/2020	1.23	0.65
03/03/2020	1.24	0.68
05/03/2020	1.27	0.38
09/03/2020	1.31	0.51
10/03/2020	1.33	0.35
11/03/2020	1.34	0.92
12/03/2020	1.26	0.58
13/03/2020	1.37	0.49
31/08/2020	0.70	0.43
30/09/2020	0.70	0.45
31/10/2020	1.10	0.58
30/11/2020	1.19	0.56
07/12/2020	1.18	0.53

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 **Refinery RDCG Plant Area** have been estimated mainly on the basis of the CPT tests. Available data from laboratory tests performed during 2020 campaign have been integrated with the results of the laboratory tests performed during investigation 2008 on the Existing Plant, in consideration of the comparable nature of the natural soils and reclamation history of the fill deposit.

7.2. Coarse grained soils

7.2.1. Recent sands and fill

The q_c values are generally greater than 10 MPa, locally value between 5 and 10 MPa are measured in correspondence of layers with a higher fine content. Values up to 20 ÷ 30 MPa, and more, are generally measured in the top fill layer above the +2 m NAP and between -14 m NAP and -15÷ -20 m NAP.

7.2.1.1. Grain size distribution

According to the results of grain size analyses summarized in 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 0.2 mm with occasional values up to 0.4 mm;
- D_{10} in the range 0.06 to 0.18 mm;
- uniformity coefficient generally in the range 1.4 to 2.5, with sporadic higher values;
- fines content is less than 10%, with some higher values.

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 18.0 ÷ 20 kN/m^3 with occasional lower values (see Attach.3 Figure 5).

7.2.1.3. Natural water content

The natural water content is generally in the range 10 to 25% with occasional higher values (see 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; 2E 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), [redacted] 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 22 and Attach.3 Figure 36. 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 NAP and between -14 m NAP and -18 ÷ -20 m NAP.

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 22 and Attach.3 Figure 36 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 sand layer:

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

  $G_0 = 50 + 5\,z$ (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 $\nu=0.20$:

- 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

7.2.1.6. Effective strength parameter

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

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

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

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	$\phi_{cv}' = 38°$	$\phi_{cv}' = 34°$
Rounded sands	$\phi_{cv}' = 33°$	$\phi_{cv}' = 30°$

Based on the relative density data estimated from CPT tests and assuming $\phi_{cv}' = 30°$, the values of ϕ' for the different areas of the site are reported in Attach.3 Figure 22 and Attach.3 Figure 36. The estimated values range generally between 35° and 38°. Locally, in presence of loose levels of silty sands/sandy silts values in the order of 33° are estimated.

For practical purpose, the following conservative representative values shall be adopted:

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

The value of Φ' evaluated from the CPT tests is aligned with the values obtained from the TX-CIU tests performed during the 2008 campaign.

In case of loose levels of silty sands/sandy silts $\phi' = 30°$ 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 x 10^{-5} to 4 x 10^{-4} m/s (see Attach.3 Figure 7).

Some permeability tests have been carried out during the campaign of 2008. The available tests results on the sands of the top fill layer (about 2.7 m below present g.l.) indicate a permeability coefficient of about 5 x 10^{-5} m/s, in good agreement with those obtained from empirical correlations indicated above.

7.2.2. Pleistocene sands

No Lab test are available for Pleistocene sand samples, therefore the data of the previous investigations performed on the adjacent Area4-Existing Plant have been used as reference.

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 summarized in 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;
- 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 4.6;
- fines content is less than 12% , with some higher values up to 14% and local values of about 40%.

Therefore the 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 in Attach.3 Figure 22 and Attach.3 Figure 36.

The estimated values are generally greater than 80%, frequently between -24/ -26 m NAP and -28/ -30 m NAP lower values in the range of 45 ÷ 75 % are found.

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 Area 4-Existing Plant. Consequently, the shear wave velocity profiles reported in Attach.3 Figure 37 and Attach.3 Figure 38 have been assumed as reference to evaluate small strain shear modulus G_0 of Pleistocene sands.

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 parameter

According to the available CPT results and interpretation (see Attach.3 Figure 22 and Attach.3 Figure 36), the angle of internal friction ϕ' ranges between 35° and 38°.

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 (see Attach.3 Figure 18) 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 14 kN/m^3 and 19 kN/m^3 (see Attach.3 Figure 8). Some lenses of peat are encountered with bulk unit weight of about 10.5 kN/m^3.

7.3.3. Natural water content

According to the laboratory results the natural water content is in the range of 30 ÷ 65% (see Attach.3 Figure 6), with occasional higher values on the peat lenses.

7.3.4. Plasticity characteristics

The values of Atterberg Limits are (see Attach.3 Figure 10 to Attach.3 Figure 12):

- Liquid Limit ranging from 40 % to 110%;
- Plastic limit ranging from 20 % to 40%, with a single value of 74%;
- Plasticity Index ranging from 12 % to 35% with some values up to 40 to 55% and others between 60% and 65% in the peat lenses, with an occasional value of 87%;

The soils' classification on the Casagrande Plasticity Chart indicates that soils are generally inorganic clays of high plasticity and locally inorganic silts of high compressibility and organic clays (see Attach.3 Figure 13).

7.3.5. Void ratio

The measured void index ranges between 0.9 and 1.5 (see Attach.3 Figure 14) with occasional higher values up to 2.4.

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 22 and Attach.3 Figure 36. Representative values for the most significant layers can be taken to range from 50 to 400 kPa. Moreover, 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 to 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 Go has been estimated from CPT tests according to the empirical relationship proposed by Mayne & Rix (1993). The values of Go, reported in Attach.3 Figure 22 and Attach.3 Figure 36, range from a minimum of 20 to 50 MPa, increasing with depth, and a maximum of 50 to 150 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 15 and Attach.3 Figure 16. The following can be inferred:

- measured values of compression index generally range between 0.16 and 0.22, with occasional higher values from 0.28 to 0.42 encountered especially at about -18 to -20 m NAP;
- measured values of recompression index generally range between 0.03 and 0.085;
- the RR/CR ratio ranges between 2 and 5: 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 silty clay lenses above -18 m NAP: CR = 0.18 and RR = 0.05;
- for silty clay lenses beneath -18 m NAP: CR = 0.35 and RR = 0.05.

7.3.8. Undrained shear strength

The undrained shear strength cu values estimated from CPT tests (Lunne et al.,1997) are reported in Attach.3 Figure 22 and Attach.3 Figure 36 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.

7.3.9. Effective strength parameters

According to the available laboratory results from TXCIU tests and direct shear tests (see Attach.3 Figure 19 and Attach.3 Figure 20) the following effective strength parameters can be evaluated:

- angle of internal friction ϕ' ranging from 28° to 34°, depending on the different clay and silt content;
- drained cohesion c' varying from 15 to 24 kPa.

7.3.10. 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 $6 \cdot 10^{-12}$ m/sec and $3.5 \cdot 10^{-10}$ m/sec, with occasional lower values ranging between $1.5 \cdot 10^{-12}$ m/sec and $3.5 \cdot 10^{-12}$ m/s (see Attach.3 Figure 17).

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 ÷ 2.5 m^2/year (see Attach.3 Figure 21), with sporadic higher values.

The coefficient of consolidation for horizontal flow c_h, can be estimated considering the anisotropy of permeability (k_h/k_v) which, for highly stratified deposits can ranges from 5 to 15 (see [redacted] et al. ,1985).

7.4. **Characteristic geotechnical parameters**

Waiting for the results of laboratory tests of soil investigation 2020, in the following Table 7.3 the 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

Parameters	Recent sands and fill	Clay to clayey silt levels	Pleistocene sand
γ (kN/m^3)	18 ÷ 20	16 ÷ 18	19 ÷ 20
D_r (%)	≥ 60 30 ÷ 50$^{(3)}$	-	> 70 Locally 45 ÷ 70
ϕ (°)	35 ÷ 38 33$^{(2)}$	20 ÷ 30	35 ÷ 40
c' (kPa)	-	15 ÷ 24	-
Cu (kPa)	-	20 ÷ 50 lower bound 50 ÷ 150 upper bound	-
OCR (-)	1	1.05 ÷ 1.5 Locally up to 2.4	1
CR (-)	-	0.18 above -18 m NAP 0.35 below -18 m NAP	-
RR (-)	-	0.05	-
G_0 (MPa)	$30 + 3 \cdot z^{(1)}$ lower bound $50 + 5 \cdot z^{(1)}$ upper bound 30$^{(2)}$	20 ÷ 50 lower bound 50 ÷ 100 upper bound	250 ÷ 350
E_0 (MPa)	$2.4 \cdot G_0$	$2.4 \cdot G_0$	$2.4 \cdot G_0$
$E'_{operational}$ (MPa)	$E_0/6$ $^{(3)}$ $E_0/8$ $^{(4)}$	-	$E_0/4$

(1) z = m from FGL
(2) loose levels of sandy silts/silty sands
(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 new plant area (2020 investigated Area 3) 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
EP350	Apparent Resistivity [Ωm]	North-South	692	880	758	446	68
		East-West	642	882	919	423	57
	Corrosivity Index		Sl. C	Sl. C	Sl. C	Sl. C	MC
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

Thermal resistivity data are available on the Factual Report issued by RSK (Ref.Doc.2.1 Annexure 6).

10. GROUND AND GROUNDWATER 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
- Magnesium ion content in water

The results of tests on soils and water performed during 2020 investigation in Area 3 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	$CaCo_3$	pH	SO_3	Cl^-
[n°]	[m]	[%]	[%]	[-]	[mg/l]	[mg/l]
BP350	1.50	<0.1	<0.5	9.12	11.7	<7
BP352	3.00	<0.1	2.4	9.08	<8.3	<7
BZP351	2.00	0.1	1.5	8.98	9.2	<7

Table 10.2: Results of water chemical analyses

BPZ	pH	SO_3	Cl-	Mg
[n°]	[-]	[mg/l]	[mg/l]	[μg/l]
BZP351 D	7.6	192	64	31000
BZP351 S	7.6	242	32	13000

Table 10.3: Summary of soil chemical test results in soil

Test	Maximum measured value	Minimum measured value	Average measured value
Water Soluble Sulphate as SO_3 (%)	0.00117	<0.00083	<0.00097
Water Soluble Chloride Ion Cl^- (%)	$<7x10^{-4}$	$<7x10^{-4}$	$<7x10^{-4}$
Organic Matter Content % (%)	0.10	<0.1	<0.1
pH	9.12	8.98	9.06
Calcium Carbonate $CaCo_3$ % (%)	2.40	<0.5	<1.23

Table 10.4 –Summary of soil chemical test results in water

Test	Maximum measured value	Minimum measured value	Average measured value
pH	7.60	7.60	7.60
Water Soluble Sulphate as SO_3 (g/l)	0.24	0.19	0.22
Water Soluble Chloride Ion Cl^- (%)	$6.4x10^{-3}$	$3.2x10^{-3}$	$4.8x10^{-3}$

11. GEOTECHNICAL RECOMMENDATIONS FOR FOUNDATION DESIGN

11.1. Shallow Foundations

Direct shallow foundations may be adopted for light 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 14.1).

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 13.4).

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 15.

From a practical point of view, the depth of foundations should be limited to 1.5 ÷ 2.0 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 are given in Section 13. Results of the performed evaluations are reported in Section 14.

11.2. Deep foundations

Deep foundation may be taken into consideration for heavy structures or structures which are sensitive to differential settlements and also for other items in case the expected total and differential settlements of shallow foundations (see Section 14) 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.

11.3. Shallow foundations on improved soil

Soil improvement may be evaluated as a suitable solution, to 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 and/or the total and differential settlements of large raft/slab or tanks in specific Plant areas (for example in Unit 40 below the Propane Storage raft foundation).

It is anticipated that at the present design phase <u>rigid inclusions</u> are the solution most adequate to the scope (see Section 12).

12. SOIL IMPROVEMENT

As anticipated at the present design phase rigid inclusions are the solution most adequate to the soil improvement technique to reduce within acceptable limits the expected long term settlements of shallow isolated foundations and/or the total and differential settlements of large raft/slab or tanks in specific Plant areas.

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 12.1.

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

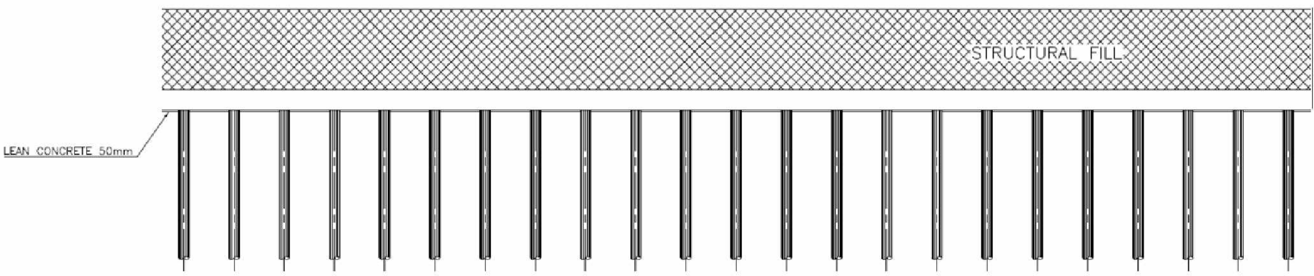


Figure 12.1: Residual total and differential settlements from mechanical completion

12.1. Intensity of treatment

Concerning the intensity and length of treatment, the following can be considered as preliminary evaluation:

- rigid inclusions diameter D = 457/510 mm or D = 356/365 mm
- spacing ~(4 ÷ 5)·D on a square grid or an equivalent triangular grid;
- assuming a pile head at ~3.5 m below F.G.L., length from L ~ 9 ÷ 10 m (pile tip elevation about -7.5 ÷ -8 m NAP) to L ~ 15.5 ÷ 16.5 m (pile tip elevation about -14 ÷ -15 m NAP)

With reference to the Propane Storage raft foundation (Unit 40) it will be realized in the ex WWT area of the Existing Plant, in which the structures/equipments were founded on rigid inclusions.
The already installed piles could be reused as foundation elements, provided that the treatment is adequately extended to cover the whole raft/slab foundation of the new structures.

Existing soil treatment in the future Unit 40 (ex WWT area) consists in driven cast in situ concrete piles:

- diameter D = 356/365 mm
- spacing = 1750 mm (~5·D) on a square grid
- L = 9.2 m assuming a pile head at ~3.5 m below F.G.L. (pile tip elevation about -7.5 m NAP)

A preliminary assessment in terms of beneficial effects of a soil treatments by means rigid inclusions has been provided at point 14.2.

13. BASIS OF GEOTECHNICAL DESIGN FOR SHALLOW FOUNDATIONS

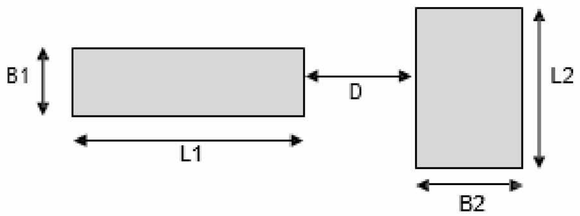
13.1. Premise

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 13.2 and 13.3. Larger foundations are herein considered as "*slab/raft foundations*". For the design of these foundations reference shall be made to paragraph 13.4.

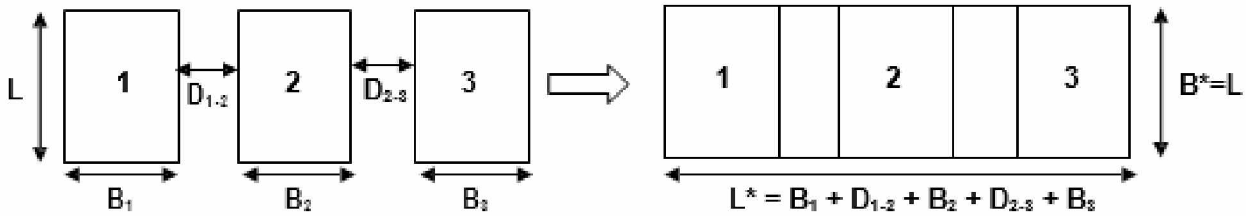
13.1.1. Definition of isolated foundation

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

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).



Three shallow foundations not isolated

Equivalent single shallow foundation for settlement evaluations

13.1.2. 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.

13.1.3. 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 13.1 and Figure 13.2).
- 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 (Figure 13.1):

$B' = B - 2 \cdot e_x$

$L' = L - 2 \cdot e_y$

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 13.2.

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.

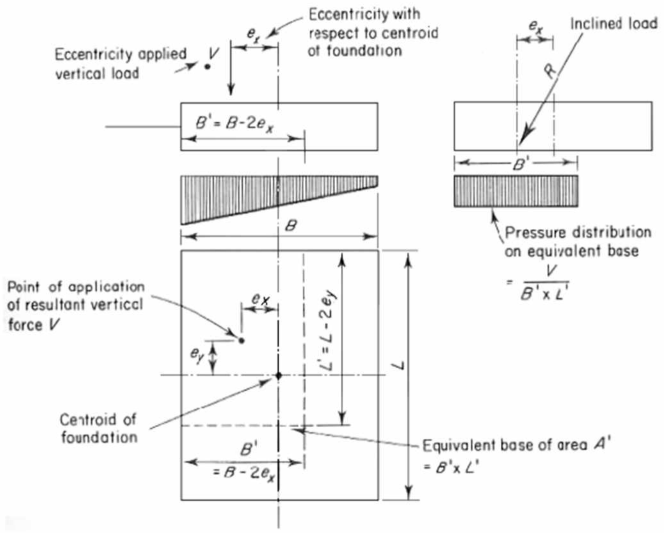


Figure 13.1: Effective footing dimensions for rectangular foundations (Meyerhof 1953).

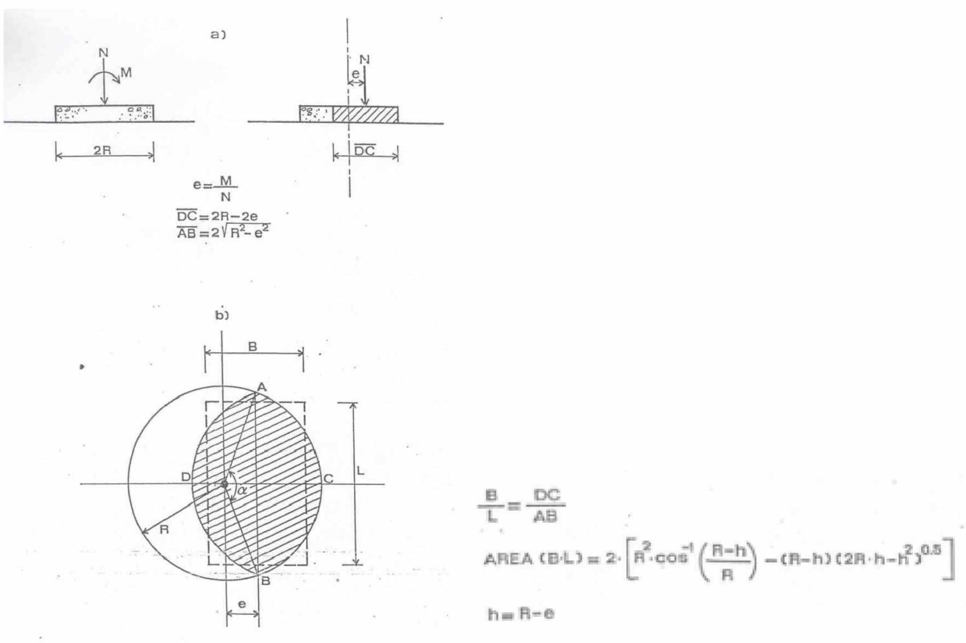


Figure 13.2: Effective footing dimensions for circular foundations (Meyerhof 1953).

13.1.4. 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 14 then the analyses need to be revised accordingly.

13.1.5. Gross and net pressures at the bottom of foundation

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

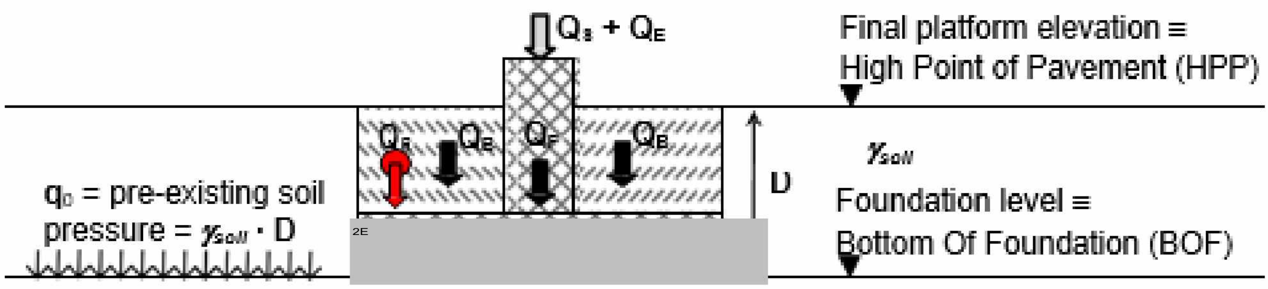
The gross pressure at the Bottom Of the Foundation (BOF) is defined as the total pressure acting at the foundation level due to loads from structures aboveground (Q_S), equipment aboveground (Q_E), foundation (Q_F), backfilling (Q_B) resting above the foundation and possible further loads (Q_R) acting 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



13.2. 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.[12], 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 13.1.

Values of sets M1 are given in Table 13.2.

Partial factors on soil resistances (R3) are given in Table 13.3.

<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 13.1: Partial factors on actions or the effects of actions (table A.3 – Ref.Doc.[12])

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 13.2: Partial factors on actions or soil parameter (table A.4a – Ref.Doc.[12])

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 13.3: Partial factors for spread foundations (EN 1997-1, table A.5 – Ref.Doc.[12])

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

13.2.1. Design bearing capacity

According to Ref.Doc. [12], 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 13.1.

R_d design resistance to be calculated as follows

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

According to Ref.Doc.[12], the q_{lim} shall be evaluated as per method described in the following, considering the soil partial factor γ_m given in Table 13.2. Values of partial factor $\gamma_{R,v}$ are provided in Table 13.3 according to the selected Design Approach and Combination (where applicable).

13.2.1.1. 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 13.2;

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

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

D = foundation embedment, Figure 13.3;

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

$B' = B - 2 \cdot e_x$	= effective foundation width, function of load eccentricity, Figure 13.1 and Figure 13.2;
$L' = L - 2 \cdot e_y$	= effective foundation length, function of load eccentricity; Figure 13.1 and Figure 13.2
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 13.3;
$s_c = \frac{s_q \cdot N_q - 1}{N_q - 1}$	= 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

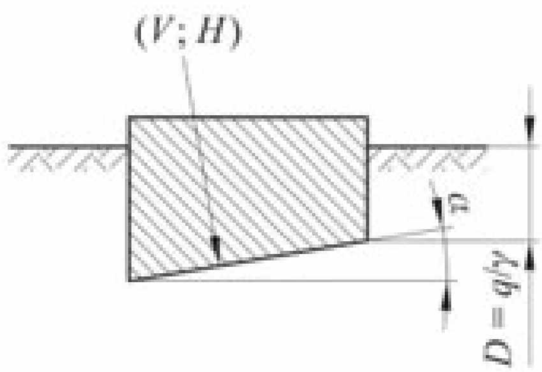


Figure 13.3: Foundation embedment level and foundation inclination.

13.2.2. Design sliding resistance

According to Ref.Doc.[12], 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 13.1.

R_d design resistance to be calculated as follows:

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

According to Ref.Doc.[12], the R_c shall be evaluated as per methods described in the following paragraphs, considering the soil partial factor γ_m given in Table 13.2. Values of partial factor $\gamma_{R,h}$ are provided in Table 13.3 according to the selected Design Approach and Combination (where applicable).

13.2.2.1. 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 13.2.

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

13.3. 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.

13.3.1. 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.

13.3.2. 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.

13.3.3. 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 (see point 14.1.3), 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}).

13.4. 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 criterion 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.

14. DESIGN OF SHALLOW FOUNDATIONS

14.1. 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 foundation of new structures/equipment to be realized in **Refinery RDCG Plant Area.**
Evaluation reported in the next points 14.1.1 and 14.1.3 can assumed applicable at each new structure and structure to be upgraded included in the scope of works of the **Refinery RDCG Plant Area** as indicated in the plan reported in ATTACHMENT 1

14.1.1. Design bearing pressure (ULS checks)

Design bearing resistance R_d for ULS checks has been evaluated according to point 13.2.1.1.

The following assumptions have been considered in calculations:

- foundation level at 1.0, 1.5 and 2 m below FGL (+5.2 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 at the site, the following calculation parameters for bearing capacity evaluations have been assumed:

- $\phi' = 35°$
- g = 19 kN/m^3

The ground water level is taken at +1.5 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 A1** and **Annex A2 respectively for** new structure within the New Expansion Area (see Figure 14.1) and the new structure and/or structure to be upgraded within the Existing Plant (see Figure 14.1).

Values have to be compared with the design gross pressure, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019 (see Table 13.1).

Based on the results reported in **Annex A1** and **Annex A2** it can be concluded that values of pressures coming from soil failure criterion show very high values and that the limit values of applied pressure will be controlled by the settlements criterion.

14.1.2. Sliding checks (ULS checks)

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

$\delta = 2/3 \cdot \phi' = 23.3°$

to be factorized with the appropriate partial factors γ_{M2} according to Table 13.2. This leads to a design friction coefficient:

$\mu = 0.375$

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

14.1.3. Settlements evaluation (SLS checks)

Settlements evaluations have been carried out in accordance with point 13.3.1

The following assumptions have been considered in calculations:

- foundation level at 1.0, 1.5 and 2 m below FGL (+5.2 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 profiles indicated in Table 14.1 and Table 14.2 have been assumed in calculations, which represent, on the safe side, the general ground conditions encountered in the New Expansion Area and the Existing Plant (see Figure 14.1). In the same Table 14.1 and Table 14.2 the geotechnical parameters used in the calculations are summarized.

The ground water level is taken at +1.5 m NAP.

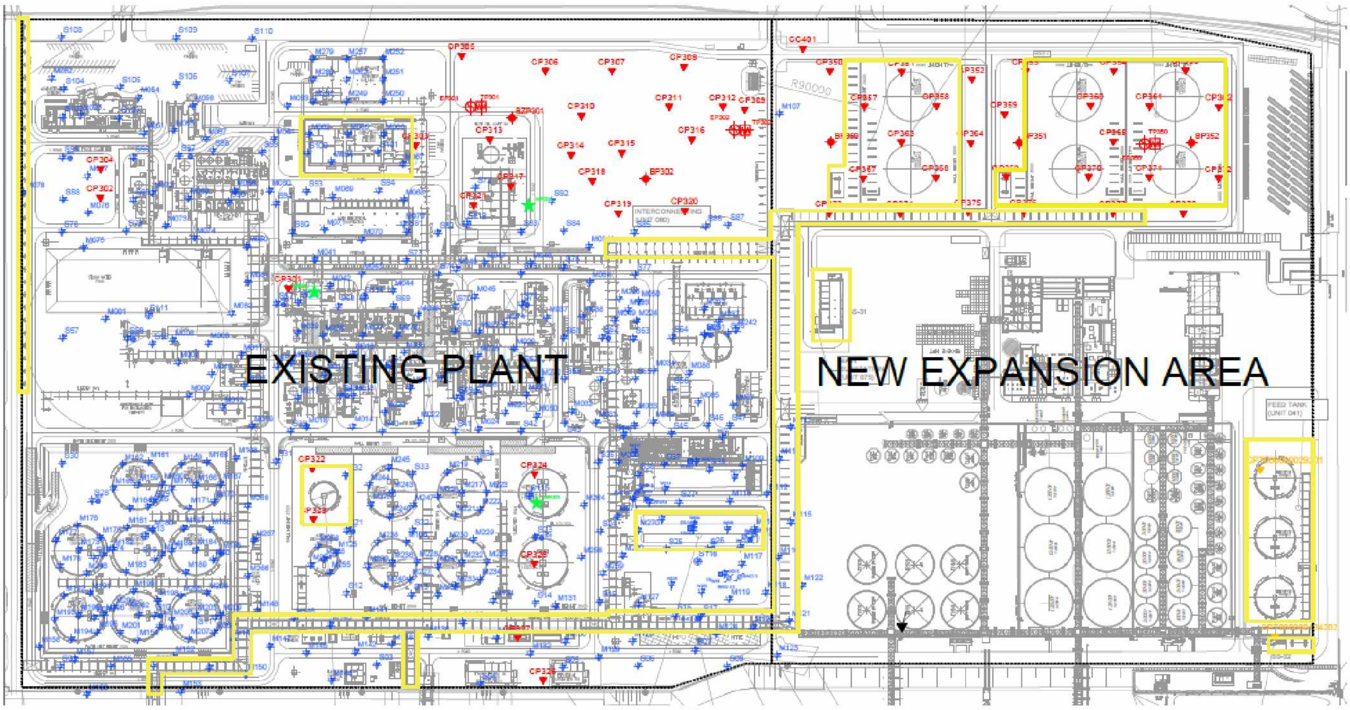


Figure 14.1: Reference areas for shallow foundation analyses

The results of calculations in terms of expected settlements for each examined applied gross load (q_{gross}) are reported in **Annex A.**

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

Table 14.1: Settlements evaluations – Reference calculation soil profile and parameters New Expansion Area

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	$E'_{operational}$ (MPa)
FGL	-2.0	Recent sand	19	-	-	-	$16+1.6\cdot z^{(1)}$
-2.0	-3.5	Clay to clayey silt	17	1.1	0.05	0.18	-
-3.5	-18.0	Recent sand	19	-	-	-	$16+1.6\cdot z^{(1)}$
-18.0	-21	Clay to clayey silt	17	1.1	0.05	0.35	-
-21	→	Pleistocene sand	19.5	-	-	-	150
$^{(1)}$ z = m from FGL							

Table 14.2: Settlements evaluations – Reference calculation soil profile and parameters Existing Plant (new structures and structures to be upgraded)

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	$E'_{operational}$ (MPa)
FGL	+1.5	Recent sand	19	-	-	-	$16+1.6\cdot z^{(1)}$
+1.5	+1	Clay to clayey silt	17	1.1	0.05	0.18	-
+1	-1	Recent sand	19	-	-	-	$16+1.6\cdot z^{(1)}$
-1	-1.5	Clay to clayey silt	17	1.1	0.05	0.18	-
-1.5	-5.5	Recent sand	19	-	-	-	$16+1.6\cdot z^{(1)}$
-5.5	-6.0	Clay to clayey silt	17	1.1	0.05	0.18	-
-6.0	-9.5	Recent sand	19	-	-	-	$16+1.6\cdot z^{(1)}$
-9.5	-10.5	Clay to clayey silt	17	1.1	0.05	0.18	-
-10.5	-18.5	Recent sand	19	-	-	-	$16+1.6\cdot z^{(1)}$
-18.5	-20	Clay to clayey silt	17	1.1	0.05	0.35	-
-20	→	Pleistocene sand	19.5	-	-	-	150
$^{(1)}$ z = m from FGL							

14.2. 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 12.1, 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
- for the above applied load expected settlements are lower than 2.5 cm.

Any assessment of greater detail will be performed in the next design phase.

15. TANK FOUNDATIONS

15.1. General

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

- N° 1 Naphta tank D = 19 m and height H ~ 14 m
- N° 3 Feed tanks D = 28 m and height H ~ 16 m
- N° 2 RJF storage tanks D= 35 m and height H ~ 16 m
- N° 4 Diesel storage tanks D= 35 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 12) 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:

- Naphta tank D = 19 m → S32, CP323
- Feed tanks D = 28 m a → CPT000000134202, CPT000000029201
- RJF storage tanks D= 35 m → CP351, CP358, CP363, CP368
- Diesel storage tanks D= 35 m → CP353, CP355, CP360, CP361, CP370, CP371

<u>Please note that currently available CPTs do not comply in terms of location (centre and perimeter) and number 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 15.1 to Table 15.8.

The groundwater has been assumed at +1.5 m NAP.

Table 15.1: Settlements evaluations – Reference calculation soil profile and parameters
Naphtha tank – S32

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-3	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-3	-5	Clay to clayey silt	16	1.1	0.05	0.18	-
-5	-19	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-19	-21.5	Clay to clayey silt	16	1.1	0.05	0.35	-
-21.5	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.2: Settlements evaluations – Reference calculation soil profile and parameters
Naphtha tank – CP323

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.9	Backfill	19	-	-	-	20
+4.9	-3.5	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-3.5	-4.5	Clay to clayey silt	16	1.1	0.05	0.18	-
-4.5	-19.5	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-19.5	-21.5	Clay to clayey silt	16	1.1	0.05	0.35	-
-21.5	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.3: Settlements evaluations – Reference calculation soil profile and parameters
Feed tanks – CPT000000134202

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.5	Backfill	19	-	-	-	20
+4.5	-13.5	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-13.5	-14	Clay to clayey silt	16	1.1	0.05	0.18	-
-14	-19	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-19	-23	Clay to clayey silt	16	1.1	0.05	0.35	-
-23	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.4: Settlements evaluations – Reference calculation soil profile and parameters
Feed tanks – CPT000000029201

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.7	Backfill	19	-	-	-	20
+4.7	-8.5	Recent sand	18.5	-	-	-	12+1.2·z(1)
-8.5	-9	Clay to clayey silt	16	1.1	0.05	0.18	-
-9	-18.5	Recent sand	18.5	-	-	-	12+1.2·z(1)
-18.5	-20.5	Clay to clayey silt	16	1.1	0.05	0.35	-
-20	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.5: Settlements evaluations – Reference calculation soil profile and parameters
RJF storage – CP351

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-2	Recent sand	18.5	-	-	-	12+1.2·z(1)
-2	-4	Clay to clayey silt	16	1.1	0.05	0.18	-
-4	-20.5	Recent sand	18.5	-	-	-	12+1.2·z(1)
-20.5	-23	Clay to clayey silt	16	1.1	0.05	0.35	-
-23	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.6: Settlements evaluations – Reference calculation soil profile and parameters
RJF storage – CP358

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.9	Backfill	19	-	-	-	20
+4.9	-2.5	Recent sand	18.5	-	-	-	12+1.2·z(1)
-2.5	-3.5	Clay to clayey silt	16	1.1	0.05	0.18	-
-3.5	-20	Recent sand	18.5	-	-	-	12+1.2·z(1)
-20	-22	Clay to clayey silt	16	1.1	0.05	0.35	-
-22	-22.5	Recent sand	18.5	-	-	-	12+1.2·z(1)
-22.5	-23	Clay to clayey silt	16	1.1	0.05	0.35	-
-23	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.7: Settlements evaluations – Reference calculation soil profile and parameters
RJF storage – CP363

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-2	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-2	-3	Clay to clayey silt	16	1.1	0.05	0.18	-
-3	-19.5	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-19.5	-22	Clay to clayey silt	16	1.1	0.05	0.35	-
-22	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.8: Settlements evaluations – Reference calculation soil profile and parameters
RJF storage – CP368

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-2	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-2	-3	Clay to clayey silt	16	1.1	0.05	0.18	-
-3	-21.5	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-21.5	-23.5	Clay to clayey silt	16	1.1	0.05	0.35	-
-23.5	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.9: Settlements evaluations – Reference calculation soil profile and parameters
Diesel storage – CP353

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+4.8	Backfill	19	-	-	-	20
+4.8	-2.5	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-2.5	-3.5	Clay to clayey silt	16	1.1	0.05	0.18	-
-3.5	-21.5	Recent sand	18.5	-	-	-	$12+1.2 \cdot z^{(1)}$
-21.5	-23.5	Clay to clayey silt	16	1.1	0.05	0.35	-
-23.5	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.10: Settlements evaluations – Reference calculation soil profile and parameters
Diesel storage – CP355

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-21.5	Recent sand	18.5	-	-	-	$12+1.2\cdot z^{(1)}$
-21.5	-23	Clay to clayey silt	16	1.1	0.05	0.35	-
-23	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.11: Settlements evaluations – Reference calculation soil profile and parameters
Diesel storage – CP360

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-2.5	Recent sand	18.5	-	-	-	$12+1.2\cdot z^{(1)}$
-2.5	-3	Clay to clayey silt	16	1.1	0.05	0.18	-
-3	-22	Recent sand	18.5	-	-	-	$12+1.2\cdot z^{(1)}$
-22	-23.5	Clay to clayey silt	16	1.1	0.05	0.35	-
-23.5	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.12: Settlements evaluations – Reference calculation soil profile and parameters
Diesel storage – CP361

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-21	Recent sand	18.5	-	-	-	$12+1.2\cdot z^{(1)}$
-21	-23.5	Clay to clayey silt	16	1.1	0.05	0.35	-
-23.5	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.13: Settlements evaluations – Reference calculation soil profile and parameters Diesel storage – CP370

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-2.5	Recent sand	18.5	-	-	-	$12+1.2\cdot z^{(1)}$
-2.5	-3	Clay to clayey silt	16	1.1	0.05	0.18	-
-3	-21	Recent sand	18.5	-	-	-	$12+1.2\cdot z^{(1)}$
-21	-23	Clay to clayey silt	16	1.1	0.05	0.35	-
-23	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

Table 15.14: Settlements evaluations – Reference calculation soil profile and parameters Diesel storage – CP371

from (m NAP)	to (m NAP)	Soil type (-)	γ (kN/m³)	OCR (-)	RR (-)	CR (-)	E' (MPa)
FGL	+5	Backfill	19	-	-	-	20
+5	-20	Recent sand	18.5	-	-	-	$12+1.2\cdot z^{(1)}$
-20	-23	Clay to clayey silt	16	1.1	0.05	0.35	-
-23	→	Pleistocene sand	19	-	-	-	150
(1) z = m from FGL							

15.2. 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 15.15.

The maximum differential settlements are also given in Table 15.15, according to the following relationship (see Figure 15.1):

- 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.

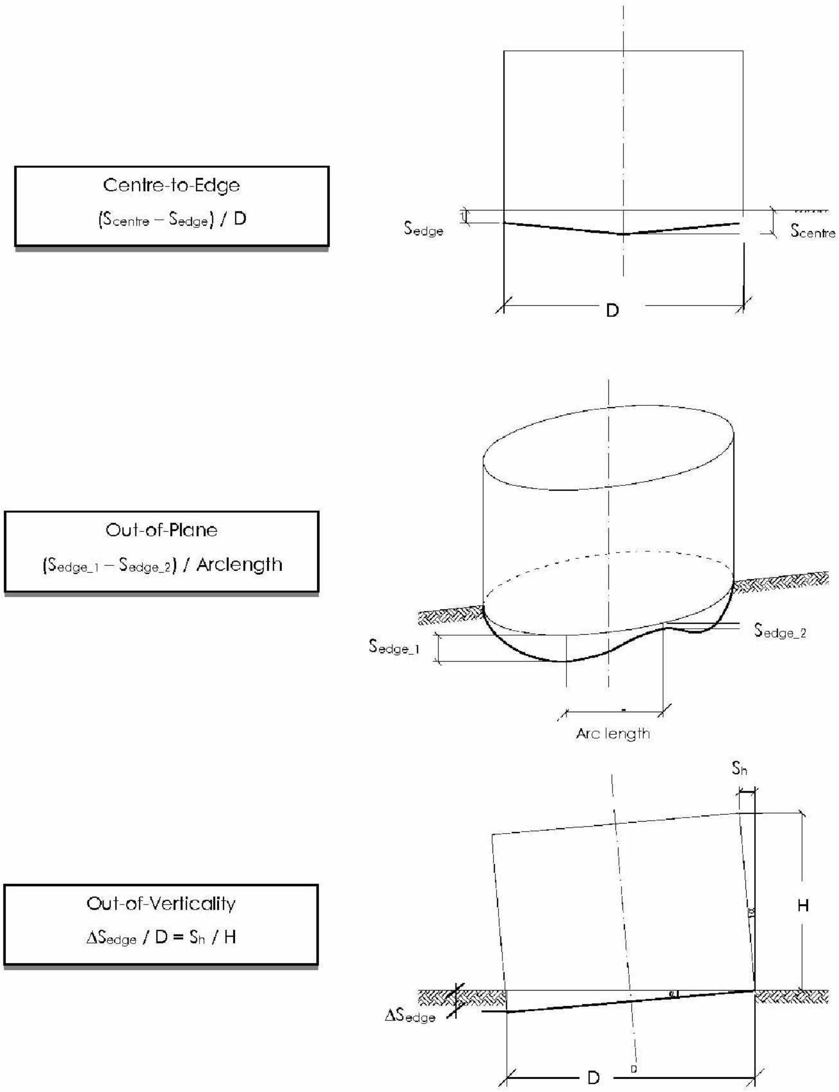


Figure 15.1

Table 15.15: Maximum estimated settlements of tanks under water test loads

Tank	CPT	D	Total settlements		Centre-to-edge	Out-of-plane (*)	Out-of-verticality (*)
		(m)	centre	edge	(%)		
Naphta	S32	19	24.7	15.8	0.47	Not valuable	Not valuable
	CP323	19	19.1	12.2	0.36		
Feed	CPT000000134202	28	20.6	12.3	0.30	Not valuable	Not valuable
	CPT000000029201	28	20.0	11.8	0.29		
RJF Storage	CP363	35	28.0	16.0	0.34	Not valuable	Not valuable
	CP368	35	26.5	15.4	0.32		
	CP351	35	34.4	19.9	0.41		
	CP358	35	28.6	16.6	0.34		
Diesel Storage 1	CP370	35	22.9	13.2	0.28	Not valuable	Not valuable
	CP360	35	21.9	12.6	0.27		
Diesel Storage 2	CP355	35	17.8	10.3	0.21	Not valuable	Not valuable
	CP361	35	18.8	10.9	0.23		
	CP371	35	20.2	11.5	0.25		

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.

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 15.16 and Table 15.17 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.

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 15.16: 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
Naphta	S32	4.83	5.92	6.30	6.62
	CP323	5.10	6.18	6.54	6.82
Feed	CPT000000134202	5.89	6.63	6.76	6.84
	CPT000000029201	6.17	7.20	7.38	7.49
RJF Storage	CP363	6.44	8.10	8.70	9.17
	CP368	6.57	8.19	8.76	9.20
	CP351	6.25	7.85	8.43	8.91
	CP358	6.71	8.56	9.13	9.58
Diesel Storage 1	CP370	6.69	8.30	8.63	8.81
	CP360	6.79	8.38	8.68	8.85
Diesel Storage 2	CP355	6.16	6.59	6.71	6.80
	CP361	6.14	6.58	6.70	6.80
	CP371	6.14	6.60	6.73	6.84

Table 15.17: Expected residual settlements at tank edge after hydraulic test

Tank	CPT	End of tank filling	Hydro test 30gg	Hydro test 45gg	Hydro test 60gg	Unload/reload settlements in operative
		cm	cm	cm	cm	cm
Naphta	S32	10.94	9.85	9.47	9.15	0.8
	CP323	7.09	6	5.64	5.37	0.85
Feed	CPT000000134202	6.42	5.68	5.55	5.46	0.98
	CPT000000029201	5.59	4.56	4.38	4.27	1.03
RJF Storage	CP363	9.53	7.87	7.28	6.81	1.07
	CP368	8.84	7.22	6.65	6.2	1.09
	CP351	13.69	12.08	11.5	11.02	1.04
	CP358	9.87	8.02	7.45	7	1.12
Diesel Storage 1	CP370	6.47	4.85	4.53	4.35	1.11
	CP360	5.86	4.27	3.96	3.8	1.13
Diesel Storage 2	CP355	4.12	3.69	3.58	3.49	1.03
	CP361	4.72	4.28	4.15	4.06	1.02
	CP371	5.37	4.91	4.77	4.66	1.02

16. SUITABILITY OF EXCAVATED MATERIAL TO BE USED AS FILL

Excavations for foundations and buried services to presumed maximum depth 2.5 m will encounter Recent Sand unit. Based on the characteristics of this Unit, excavated material may be classified generally as A3 and A2-4 (marginally), according to the AASHTO classification.

With reference to typical requirements for earthworks, it is anticipated that the materials arising from the excavations may be reused only for backfilling of foundation pits and trenches, or, more in general small confined excavated area, subject to verification of environmental compatibility.

As a results of its uniform grading, the use as ordinary fill for general site elevation 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.

16.1. Soil stabilisation for fills

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.

17. RECOMMENDATIONS FOR TEMPORARY EXCAVATIONS

According to available information, the maximum perched groundwater level is encountered at a minimum depth of about 3.5 ÷4 m below final grade (+5.2 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 the typical soil profile at the site, a maximum temporary slopes inclination of 3H/2V 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

ANNEX A1
New Expansion Area

New Expansion Area
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	560	505	490	480	465
1.5	615	575	560	550	540
2	670	640	630	625	615
2.5	725	705	700	695	
3	780	775	770	770	
4	860	875	880		
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	435	390	375	365	350
1.5	470	435	425	415	405
2	510	480	470	465	460
2.5	550	530	520	520	
3	585	575	570	570	
4	640	645	650		
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	325	290	280	270	260
1.5	355	320	310	305	295
2	380	355	345	340	335
2.5	405	385	380	375	
3	430	415	410	410	
4	465	460	460		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2\cdot e_x$ and $L'=L-2\cdot e_y$					

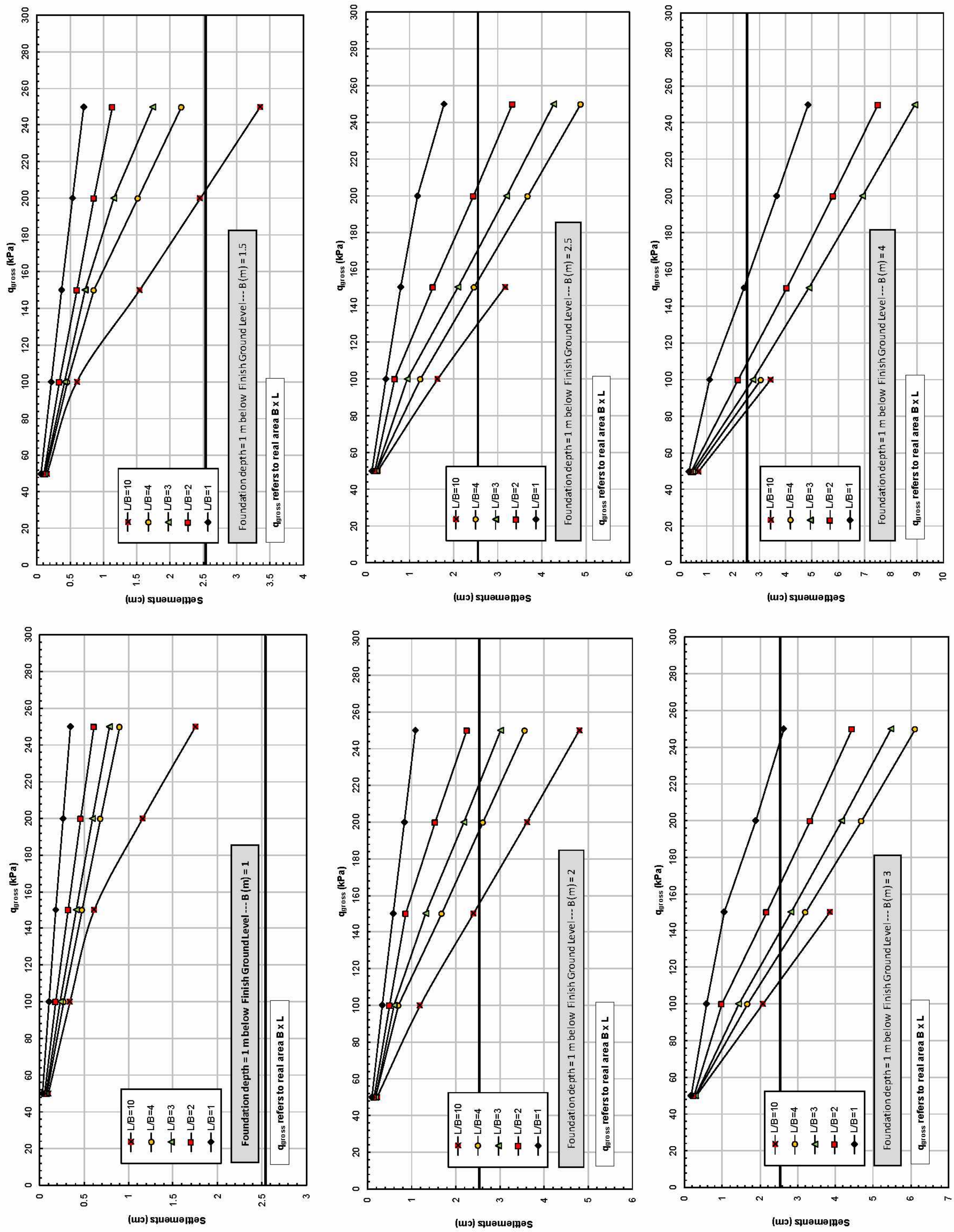
Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

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

New Expansion Area
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	0.03	0.05	0.07	0.08	0.10
100	0.10	0.18	0.23	0.27	0.34
150	0.18	0.32	0.42	0.48	0.61
200	0.26	0.46	0.59	0.69	1.16
250	0.35	0.61	0.79	0.90	1.75
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	0.07	0.10	0.12	0.13	0.15
100	0.21	0.34	0.42	0.46	0.60
150	0.37	0.60	0.72	0.85	1.54
200	0.54	0.86	1.16	1.52	2.45
250	0.71	1.13	1.74	2.18	3.35
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	0.10	0.15	0.18	0.19	0.21
100	0.33	0.50	0.60	0.70	1.19
150	0.58	0.87	1.31	1.68	2.40
200	0.84	1.53	2.19	2.62	3.61
250	1.09	2.24	3.01	3.56	4.80
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	0.14	0.20	0.23	0.25	0.26
100	0.46	0.66	0.94	1.24	1.63
150	0.79	1.52	2.09	2.46	3.16
200	1.18	2.45	3.20	3.69	
250	1.78	3.34	4.28	4.90	
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	0.18	0.25	0.29	0.30	0.31
100	0.58	0.99	1.44	1.67	2.08
150	1.06	2.19	2.82	3.21	3.86
200	1.90	3.33	4.18	4.69	
250	2.63	4.44	5.49	6.13	
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	0.31	0.42	0.45	0.53	0.69
100	1.10	2.20	2.76	3.06	3.43
150	2.41	4.04	4.90		
200	3.64	5.81	6.94		
250	4.85	7.52	8.90		
q_{gross} refers to the real area B x L					

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



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

New Expansion Area
GROSS BEARING PRESSURE FOR ULS VERIFICATIONS

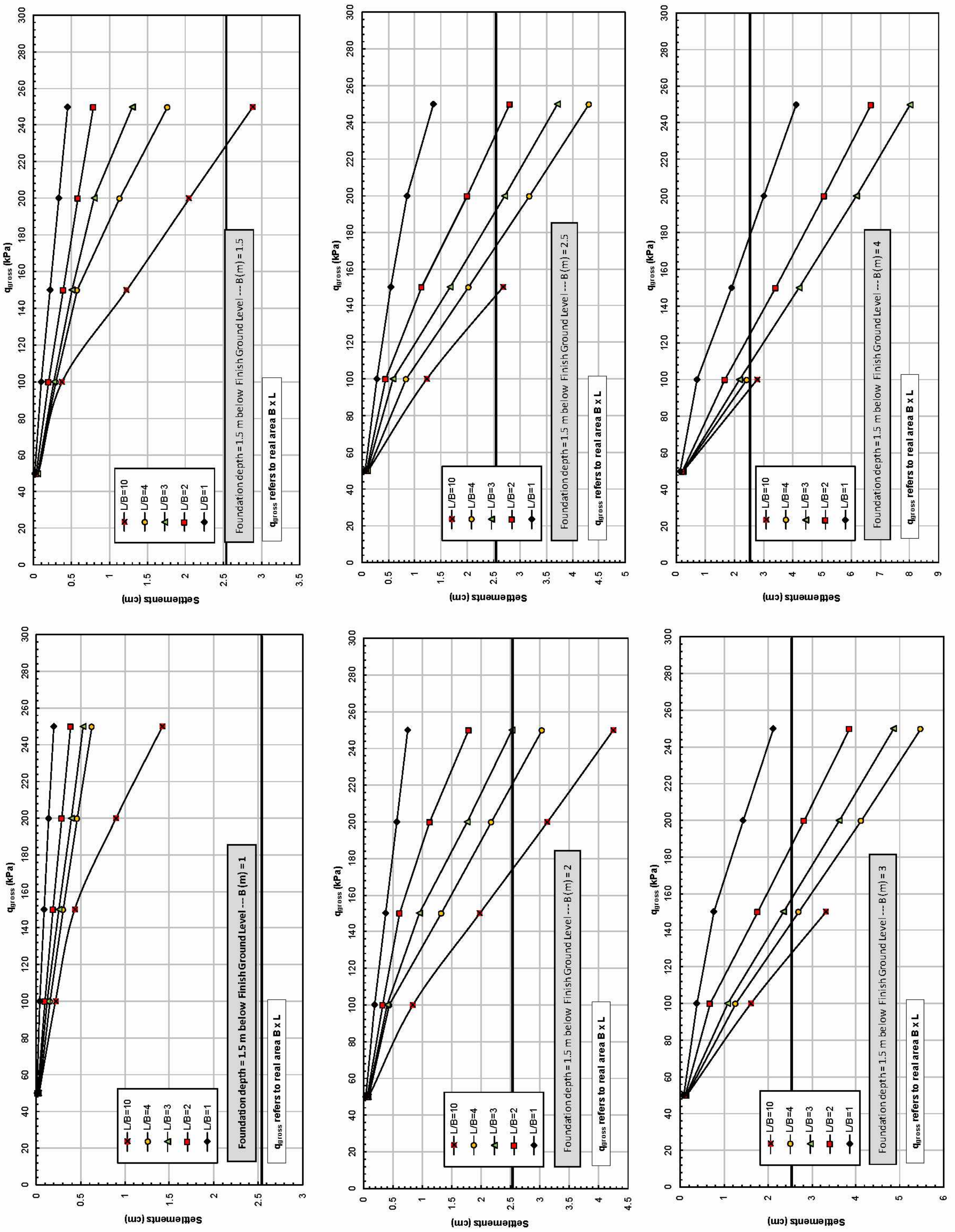
$q_{d,gross}$ (kPa)					
Foundation depth = 1.5 m below Finish Ground Level - H/N =0.1					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	785	695	665	650	620
1.5	840	760	735	720	695
2	895	825	805	795	770
2.5	950	895	875	865	
3	990	945	930	920	
4	1020	1000	995		
Foundation depth = 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	610	535	510	500	475
1.5	650	585	560	550	530
2	690	630	610	600	585
2.5	725	675	660	650	
3	750	710	695	690	
4	770	750	740		
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	465	405	385	375	355
1.5	490	435	420	410	395
2	515	470	450	445	430
2.5	545	500	485	480	
3	560	520	510	505	
4	570	545	535		
$q_{d,gross}$ 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 settlement criterion (see SLS verification in the following)

New Expansion Area
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	0.00	0.01	0.02	0.03	0.03
100	0.04	0.09	0.13	0.16	0.22
150	0.09	0.19	0.26	0.31	0.43
200	0.14	0.29	0.39	0.47	0.90
250	0.19	0.39	0.53	0.63	1.42
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	0.02	0.04	0.05	0.05	0.06
100	0.11	0.20	0.27	0.30	0.37
150	0.21	0.39	0.51	0.57	1.22
200	0.33	0.59	0.80	1.13	2.05
250	0.45	0.79	1.30	1.76	2.88
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	0.03	0.07	0.08	0.08	0.09
100	0.19	0.33	0.40	0.44	0.84
150	0.37	0.62	0.95	1.32	1.97
200	0.57	1.12	1.77	2.18	3.13
250	0.75	1.79	2.52	3.04	4.24
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	0.06	0.09	0.11	0.11	0.12
100	0.28	0.45	0.59	0.84	1.23
150	0.54	1.13	1.68	2.02	2.68
200	0.86	1.99	2.71	3.18	
250	1.35	2.81	3.71	4.31	
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	0.08	0.12	0.14	0.14	0.15
100	0.38	0.69	1.08	1.27	1.61
150	0.77	1.76	2.36	2.71	3.33
200	1.44	2.82	3.63	4.13	
250	2.12	3.85	4.87	5.48	
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	0.15	0.21	0.23	0.24	0.25
100	0.73	1.68	2.17	2.44	2.77
150	1.91	3.42	4.23		
200	3.02	5.09	6.17		
250	4.12	6.69	8.03		
q_{gross} refers to the real area B x L					



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

New Expansion Area
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	1015	880	840	815	775
1.5	1070	950	910	890	855
2	1120	1015	980	960	930
2.5	1140	1045	1015	1000	
3	1140	1060	1035	1020	
4	1130	1080	1060		
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	790	685	650	630	600
1.5	830	730	700	685	655
2	865	780	750	735	705
2.5	875	800	770	760	
3	875	805	785	770	
4	860	815	795		
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	605	520	495	480	455
1.5	630	550	525	515	490
2	655	585	560	545	525
2.5	660	595	575	560	
3	655	595	575	570	
4	640	595	580		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2\cdot e_x$ and $L'=L-2\cdot e_y$					

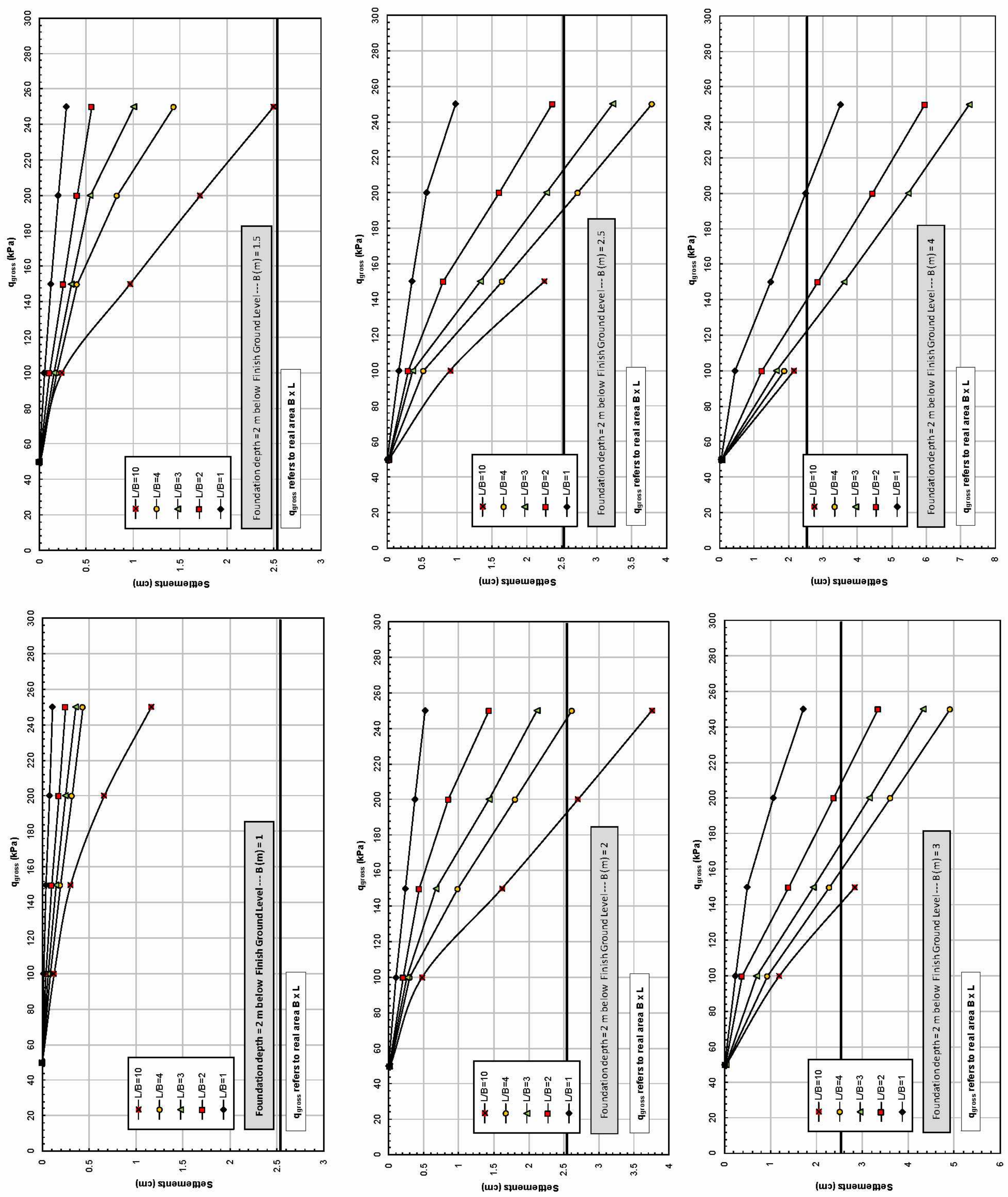
Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

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

New Expansion Area
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	0.01	0.04	0.07	0.08	0.13
150	0.04	0.10	0.15	0.19	0.30
200	0.08	0.18	0.25	0.32	0.66
250	0.11	0.25	0.36	0.44	1.17
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	0.00	0.01	0.01
100	0.05	0.11	0.15	0.19	0.24
150	0.12	0.26	0.35	0.40	0.97
200	0.20	0.40	0.54	0.83	1.72
250	0.29	0.56	1.01	1.43	2.50
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	0.01	0.01	0.02	0.02
100	0.10	0.20	0.26	0.30	0.48
150	0.24	0.43	0.67	0.99	1.62
200	0.38	0.85	1.43	1.81	2.70
250	0.52	1.43	2.11	2.62	3.76
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	0.02	0.03	0.03	0.03
100	0.17	0.30	0.37	0.52	0.90
150	0.36	0.81	1.33	1.65	2.26
200	0.57	1.61	2.29	2.74	
250	0.98	2.36	3.23	3.80	
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	0.01	0.03	0.03	0.04	0.04
100	0.24	0.39	0.71	0.93	1.19
150	0.50	1.39	1.93	2.28	2.84
200	1.07	2.38	3.15	3.62	
250	1.72	3.34	4.32	4.91	
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	0.03	0.06	0.06	0.07	0.07
100	0.45	1.23	1.65	1.88	2.16
150	1.49	2.86	3.61		
200	2.49	4.44	5.48		
250	3.51	5.95	7.25		
q_{gross} refers to the real area B x L					

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



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

ANNEX A2
Existing Plant
New structures and Structures to be upgraded

Existing Plant
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	560	505	490	480	465
1.5	615	575	560	550	540
2	670	640	630	625	615
2.5	725	705	700	695	
3	780	775	770	770	
4	860	875	880		
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	435	390	375	365	350
1.5	470	435	425	415	405
2	510	480	470	465	460
2.5	550	530	520	520	
3	585	575	570	570	
4	640	645	650		
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	325	290	280	270	260
1.5	355	320	310	305	295
2	380	355	345	340	335
2.5	405	385	380	375	
3	430	415	410	410	
4	465	460	460		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2\cdot e_x$ and $L'=L-2\cdot e_y$					

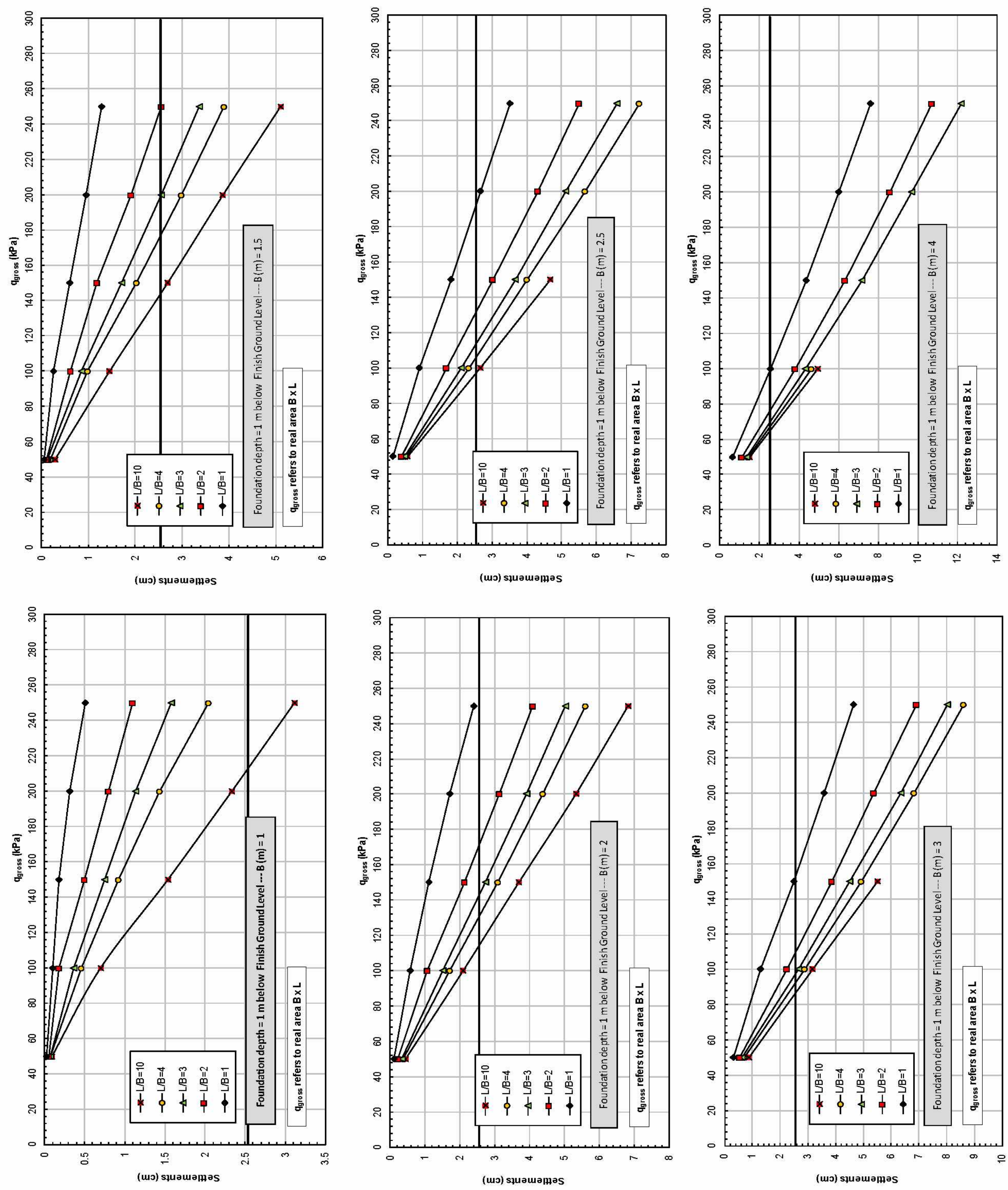
Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

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)

Existing Plant
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	0.03	0.05	0.07	0.08	0.10
100	0.10	0.18	0.36	0.46	0.70
150	0.18	0.50	0.75	0.93	1.54
200	0.32	0.80	1.14	1.43	2.34
250	0.51	1.10	1.58	2.04	3.12
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	0.07	0.10	0.12	0.19	0.28
100	0.26	0.63	0.85	0.98	1.45
150	0.61	1.18	1.72	2.03	2.69
200	0.95	1.91	2.55	2.99	3.87
250	1.29	2.57	3.38	3.90	5.10
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	0.10	0.21	0.33	0.38	0.43
100	0.58	1.06	1.50	1.71	2.09
150	1.12	2.14	2.73	3.08	3.69
200	1.70	3.13	3.91	4.37	5.32
250	2.40	4.08	5.02	5.58	6.83
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	0.14	0.39	0.49	0.53	0.58
100	0.92	1.69	2.11	2.34	2.66
150	1.82	3.04	3.67	4.02	4.68
200	2.68	4.31	5.13	5.69	
250	3.53	5.51	6.61	7.24	
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	0.31	0.53	0.63	0.72	0.87
100	1.29	2.24	2.67	2.88	3.16
150	2.49	3.87	4.51	4.94	5.50
200	3.59	5.38	6.34	6.84	
250	4.64	6.90	8.02	8.63	
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	0.67	1.12	1.35	1.43	1.50
100	2.57	3.83	4.33	4.64	4.96
150	4.35	6.31	7.17		
200	6.01	8.60	9.70		
250	7.61	10.72	12.19		
q_{gross} refers to the real area B x L					

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



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

Existing Plant
GROSS BEARING PRESSURE FOR ULS VERIFICATIONS

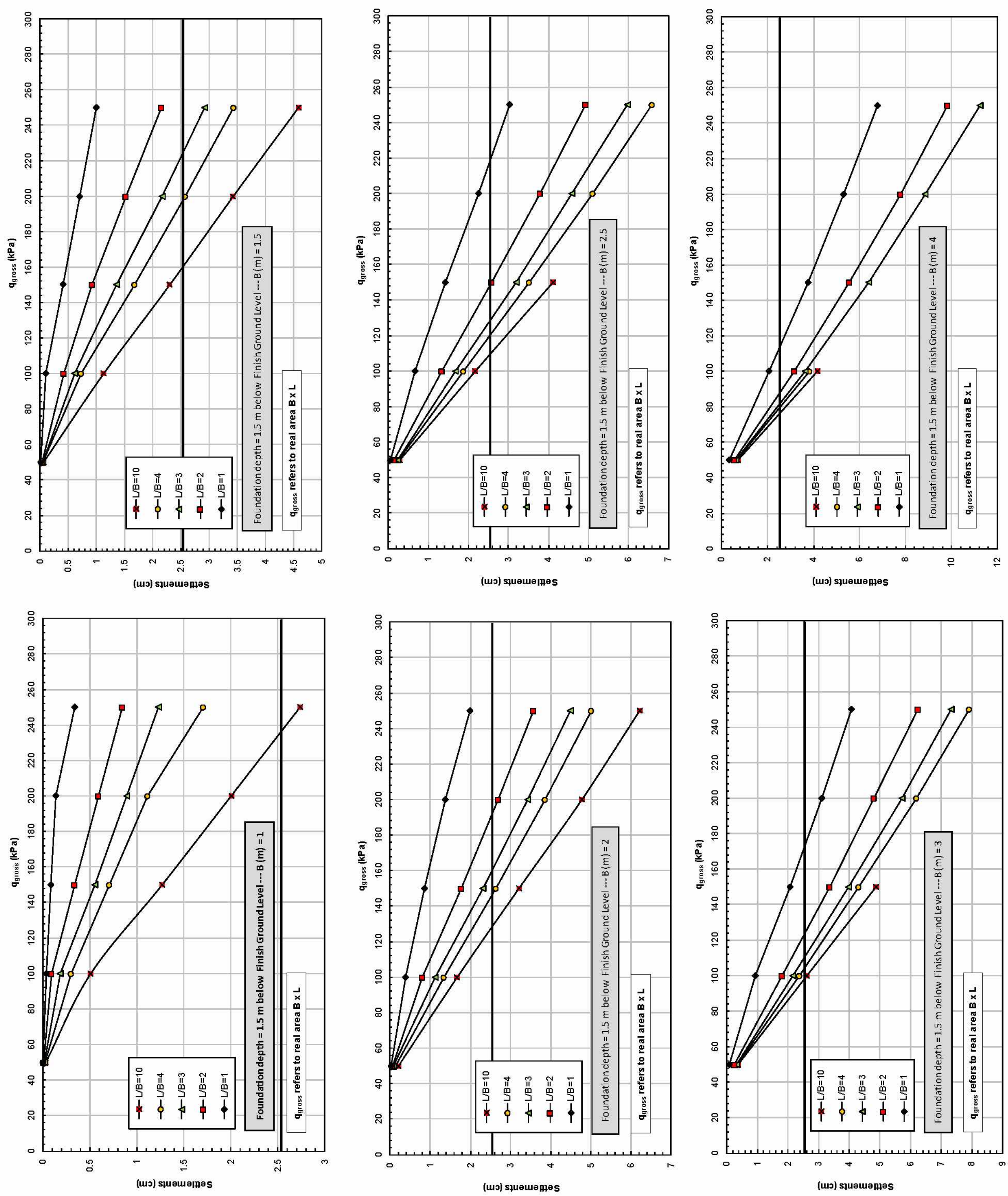
$q_{d,gross}$ (kPa)					
Foundation depth = 1.5 m below Finish Ground Level - H/N =0.1					
B' (m)	**L'/B'=1**	**L'/B'=2**	**L'/B'=3**	**L'/B'=4**	**L'/B'=10**
1	785	695	665	650	620
1.5	840	760	735	720	695
2	895	825	805	795	770
2.5	950	895	875	865	
3	990	945	930	920	
4	1020	1000	995		
Foundation depth = 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	610	535	510	500	475
1.5	650	585	560	550	530
2	690	630	610	600	585
2.5	725	675	660	650	
3	750	710	695	690	
4	770	750	740		
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	465	405	385	375	355
1.5	490	435	420	410	395
2	515	470	450	445	430
2.5	545	500	485	480	
3	560	520	510	505	
4	570	545	535		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where B'=B-2·e_x and L'=L-2·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)

Existing Plant
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	0.00	0.01	0.02	0.03	0.03
100	0.04	0.09	0.19	0.30	0.51
150	0.09	0.34	0.55	0.71	1.27
200	0.14	0.59	0.90	1.11	2.01
250	0.34	0.85	1.23	1.71	2.74
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	0.02	0.04	0.05	0.05	0.06
100	0.11	0.43	0.62	0.73	1.13
150	0.42	0.92	1.36	1.68	2.30
200	0.71	1.53	2.16	2.58	3.42
250	1.00	2.15	2.92	3.44	4.58
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	0.03	0.07	0.08	0.13	0.20
100	0.38	0.80	1.13	1.35	1.67
150	0.85	1.77	2.31	2.64	3.21
200	1.37	2.69	3.43	3.87	4.77
250	1.99	3.58	4.48	5.02	6.21
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	0.06	0.14	0.23	0.26	0.28
100	0.66	1.33	1.67	1.88	2.17
150	1.43	2.59	3.19	3.51	4.11
200	2.26	3.79	4.58	5.10	
250	3.04	4.93	5.98	6.58	
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	0.08	0.26	0.32	0.34	0.36
100	0.94	1.80	2.17	2.37	2.61
150	2.07	3.37	3.97	4.31	4.88
200	3.11	4.81	5.72	6.19	
250	4.08	6.25	7.34	7.92	
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	0.32	0.55	0.61	0.64	0.71
100	2.05	3.18	3.63	3.82	4.16
150	3.75	5.55	6.39		
200	5.32	7.79	8.85		
250	6.77	9.83	11.24		
q_{gross} refers to the real area B x L					



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

Existing Plant
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	1015	880	840	815	775
1.5	1070	950	910	890	855
2	1120	1015	980	960	930
2.5	1140	1045	1015	1000	
3	1140	1060	1035	1020	
4	1130	1080	1060		
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	790	685	650	630	600
1.5	830	730	700	685	655
2	865	780	750	735	705
2.5	875	800	770	760	
3	875	805	785	770	
4	860	815	795		
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	605	520	495	480	455
1.5	630	550	525	515	490
2	655	585	560	545	525
2.5	660	595	575	560	
3	655	595	575	570	
4	640	595	580		
$q_{d,gross}$ refers to the effective Meyerhof area B' x L', where $B'=B-2\cdot e_x$ and $L'=L-2\cdot e_y$					

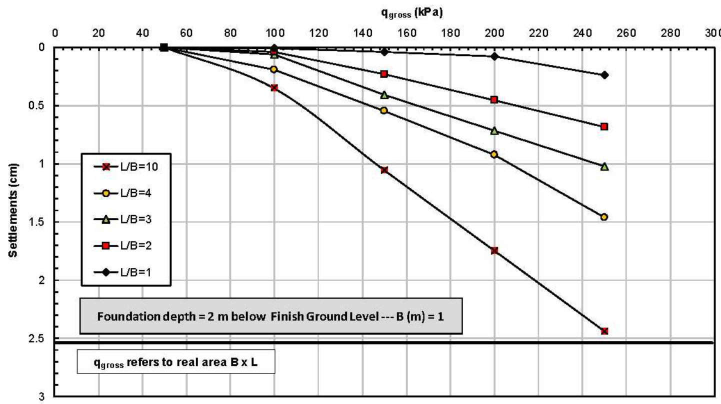
Values have to be compared with the design gross applied pressure for ULS checks, obtained applying the partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019

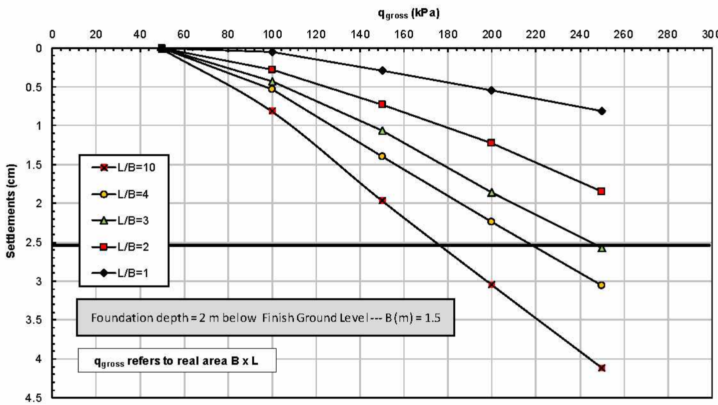
Values of pressures coming from soil failure criterion show very high values, the limit values of applied pressure will be controlled by the settlements criterion (see SLS verification in the following)

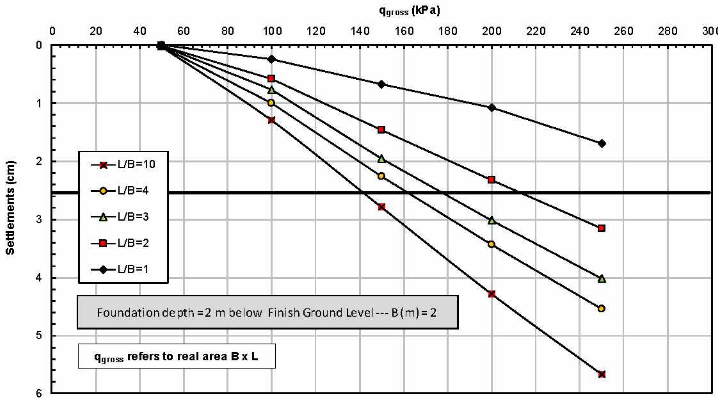
Existing Plant
SETTLEMENTS EVALUATION FOR SLS VERIFICATIONS

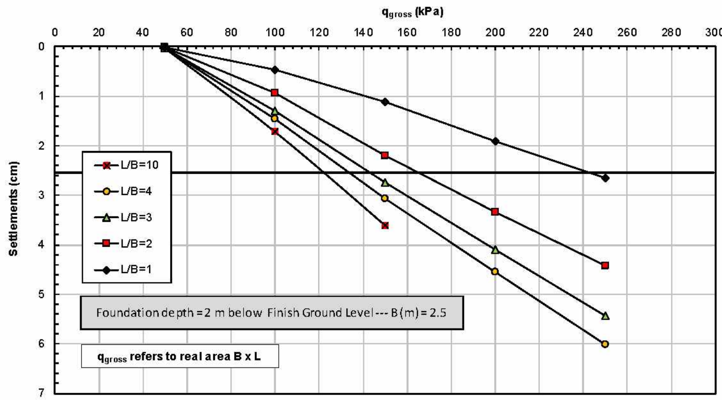
Settlements (cm)					
Foundation depth = 2 m below Finish Ground Level - B = 1 (m)					
$q_{d,gross}$ (kPa)	**L/B=1**	**L/B=2**	**L/B=3**	**L/B=4**	**L/B=10**
50	0.00	0.00	0.00	0.00	0.00
100	0.01	0.04	0.07	0.19	0.35
150	0.04	0.23	0.41	0.55	1.05
200	0.08	0.46	0.72	0.92	1.75
250	0.24	0.68	1.02	1.46	2.43
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	0.00	0.01	0.01
100	0.05	0.27	0.43	0.53	0.81
150	0.29	0.73	1.07	1.40	1.96
200	0.55	1.22	1.85	2.24	3.04
250	0.81	1.84	2.57	3.06	4.11
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	0.01	0.01	0.02	0.02
100	0.25	0.58	0.77	0.99	1.29
150	0.67	1.46	1.95	2.26	2.79
200	1.08	2.33	3.02	3.44	4.28
250	1.69	3.16	4.02	4.54	5.67
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	0.02	0.03	0.03	0.03
100	0.47	0.94	1.30	1.47	1.72
150	1.11	2.20	2.75	3.06	3.61
200	1.91	3.35	4.09	4.55	
250	2.64	4.42	5.43	6.01	
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	0.01	0.03	0.03	0.04	0.04
100	0.70	1.39	1.72	1.89	2.10
150	1.72	2.90	3.47	3.76	4.31
200	2.70	4.30	5.16	5.61	
250	3.62	5.63	6.72	7.28	
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	0.03	0.11	0.18	0.19	0.20
100	1.59	2.58	2.97	3.15	3.36
150	3.22	4.84	5.66		
200	4.71	7.05	8.07		
250	6.10	9.01	10.37		
q_{gross} refers to the real area B x L					

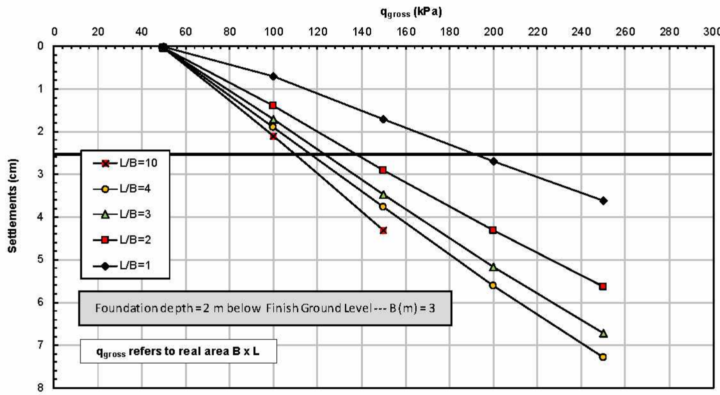
Values have to be referred to the gross pressure for SLS checks.

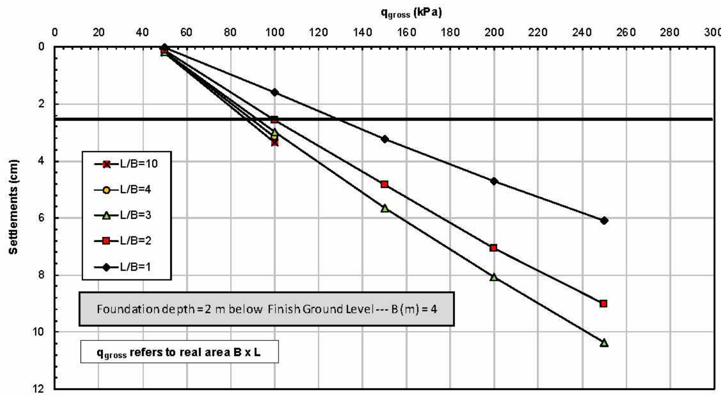






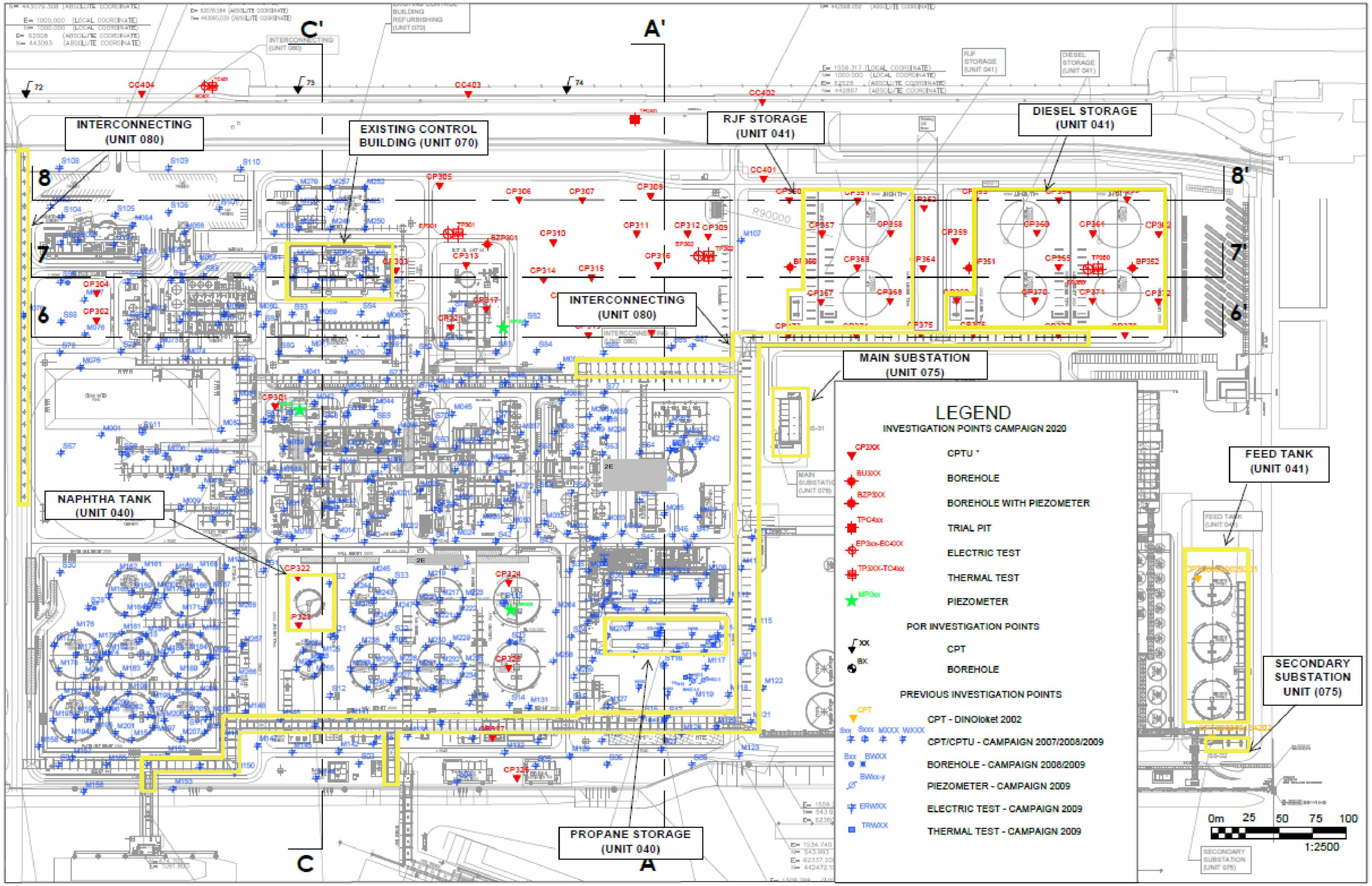




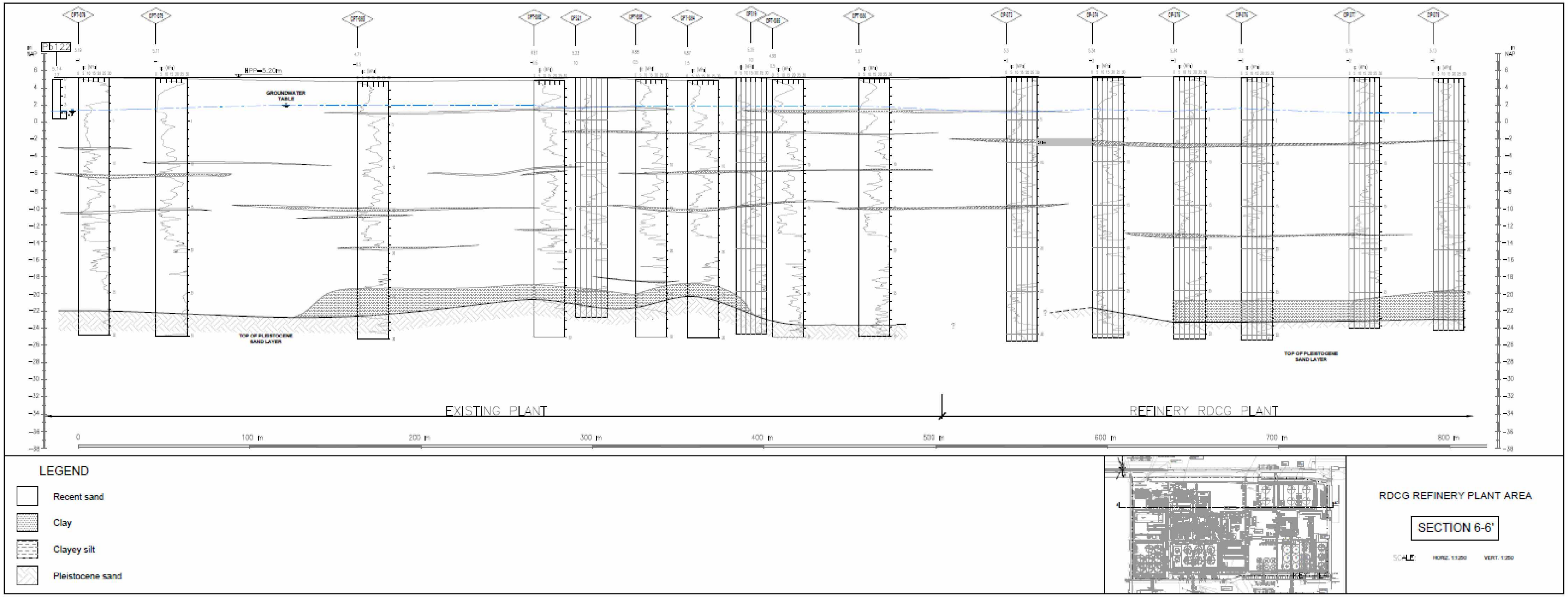


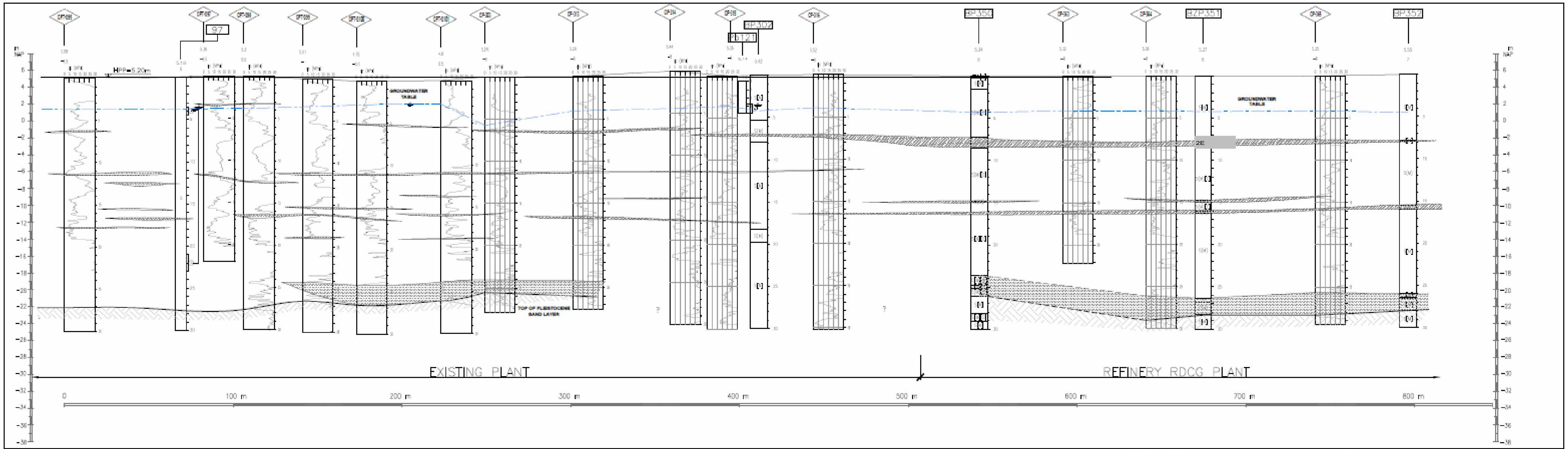
ATTACHMENT 1
Ground Investigation Location Map

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



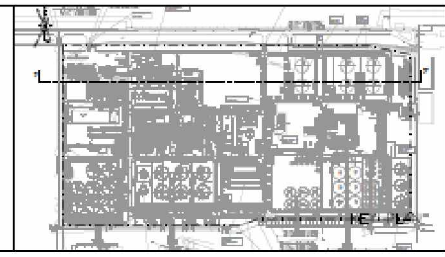
ATTACHMENT 2
Geotechnical Sections


GROUNDWATER TABLE
TOP OF PLEISTOCENE SAND LAYER
EXISTING PLANT
REFINERY RDCG PLANT
0
100 m
200 m
300 m
400 m
500 m
600 m
700 m
800 m
LEGEND
Recent sand
Clay
Clayey silt
Pleistocene sand
RDCG REFINERY PLANT AREA
SECTION 6-6'



LEGEND

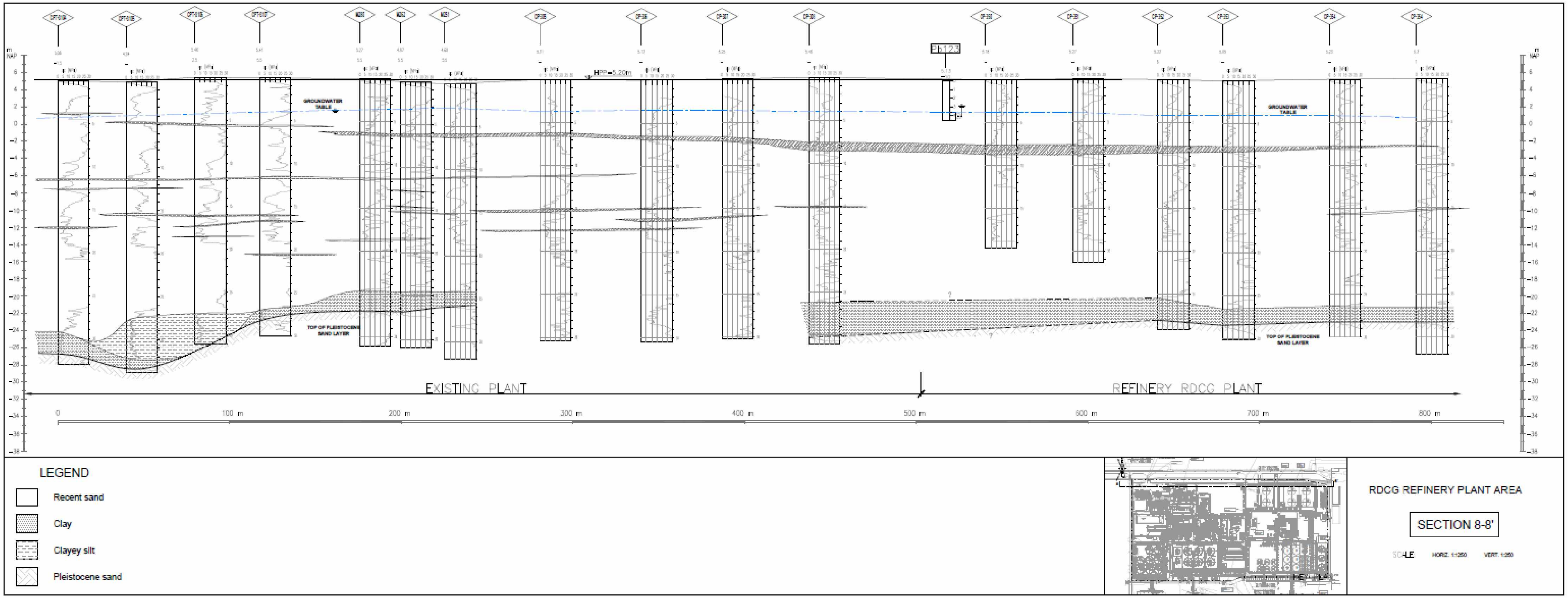
- Recent sand
- Clay
- Clayey silt
- Pleistocene sand

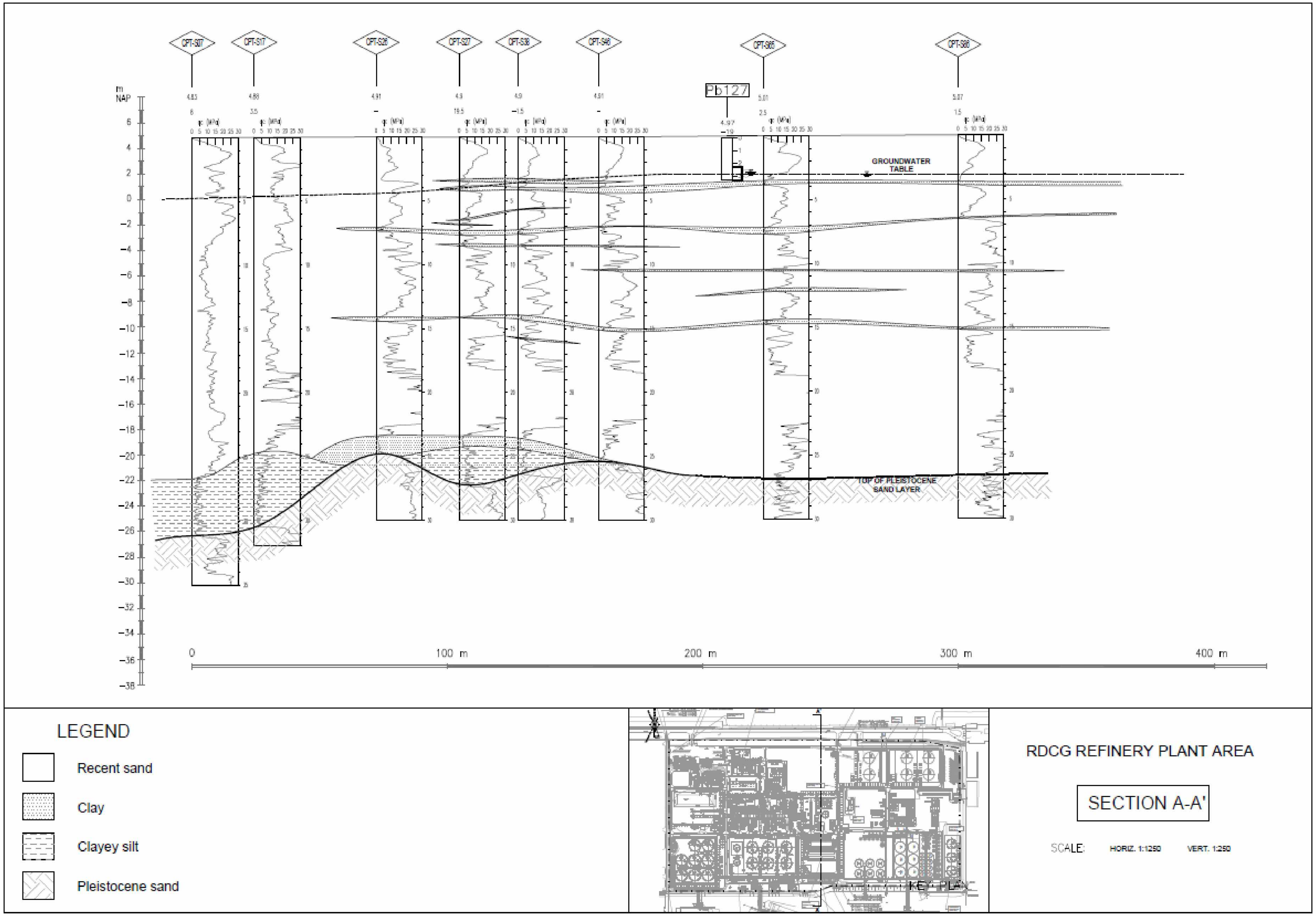


RDCG REFINERY PLANT AREA

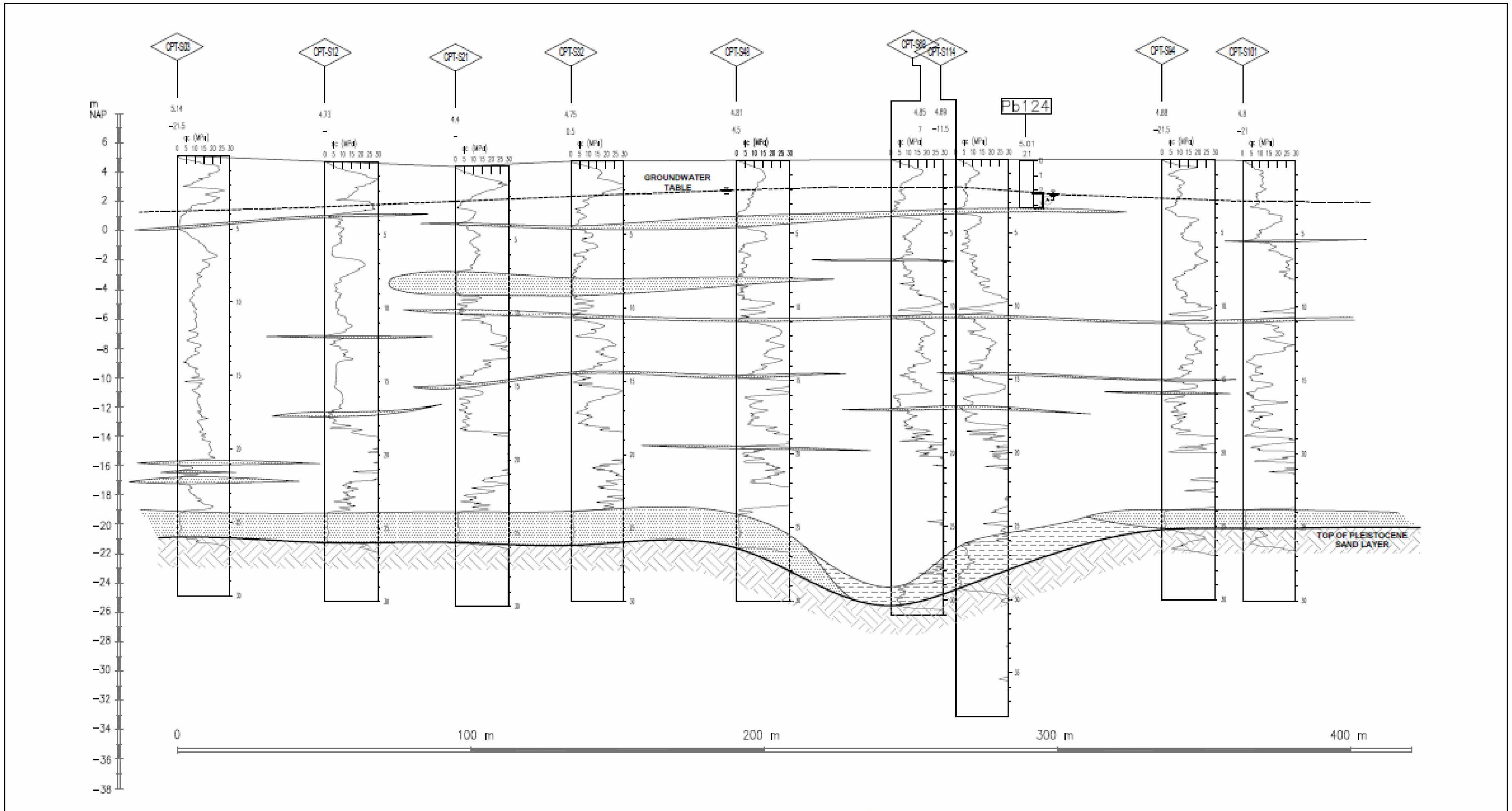
SECTION 7-7'

SCALE: HORIZ. 1:1250 VERT. 1:250


GROUNDWATER TABLE
TOP OF PLEISTOCENE SAND LAYER
EXISTING PLANT
REFINERY RDCG PLANT
0
100 m
200 m
300 m
400 m
500 m
600 m
700 m
800 m
LEGEND
Recent sand
Clay
Clayey silt
Pleistocene sand
RDCG REFINERY PLANT AREA
SECTION 8-8'

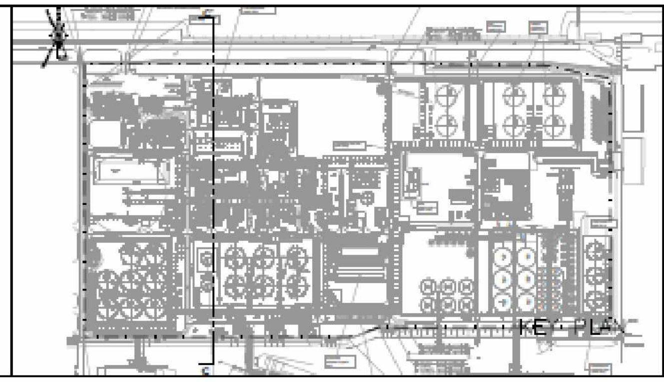

m NAP
Pb127
GROUNDWATER TABLE
TOP OF PLEISTOCENE SAND LAYER
0
100 m
200 m
300 m
400 m
LEGEND
Recent sand
Clay
Clayey silt
Pleistocene sand
KEY PLAN
RDCG REFINERY PLANT AREA
SECTION A-A'
SCALE: HORIZ. 1:1250 VERT. 1:250

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



LEGEND

- Recent sand
- Clay
- Clayey silt
- Pleistocene sand

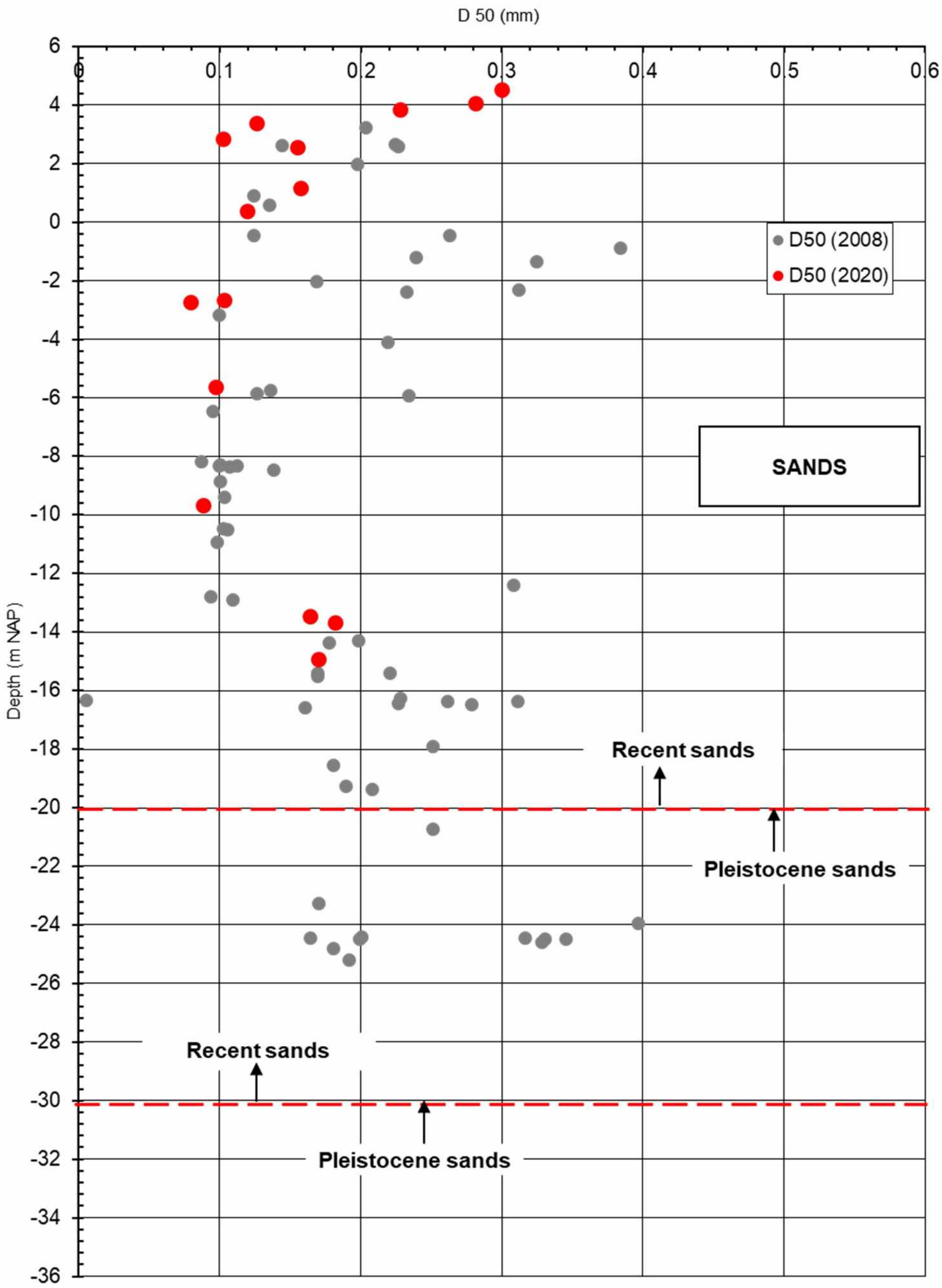


RDCG REFINERY PLANT AREA

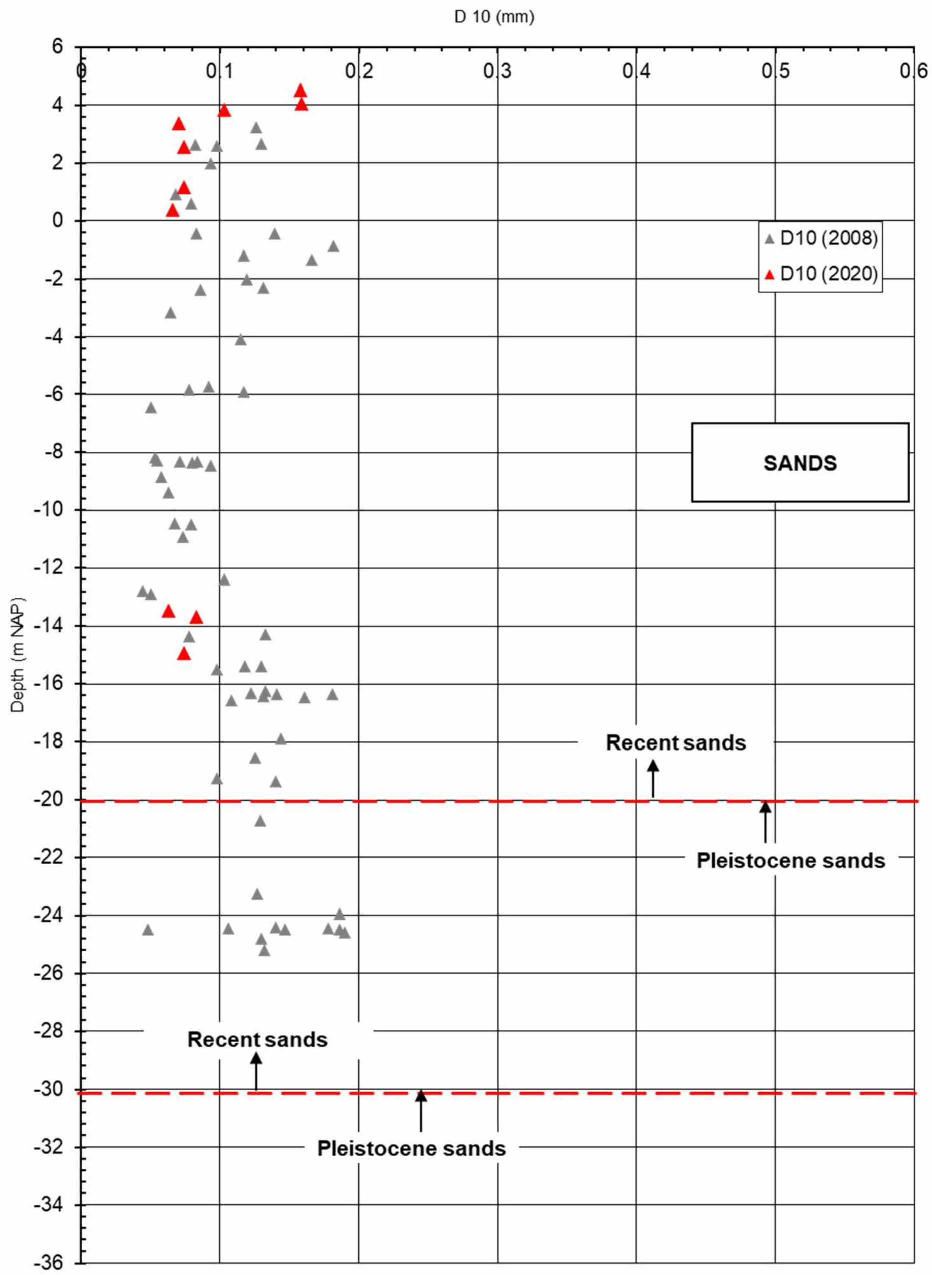
SECTION C-C'

SCALE: HORIZ. 1:1250 VERT. 1:250

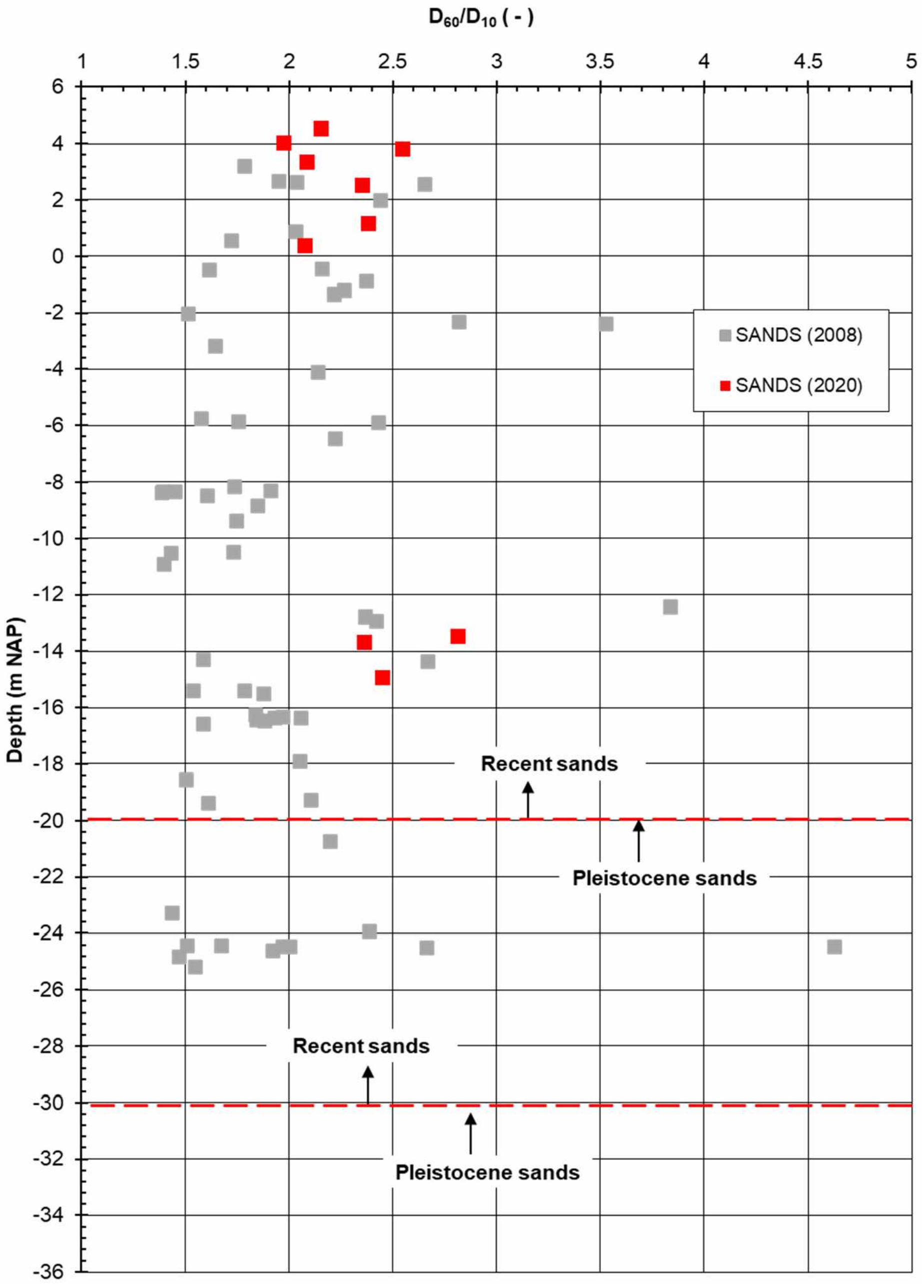
ATTACHMENT 3
Laboratory tests results
CPT results and interpretation



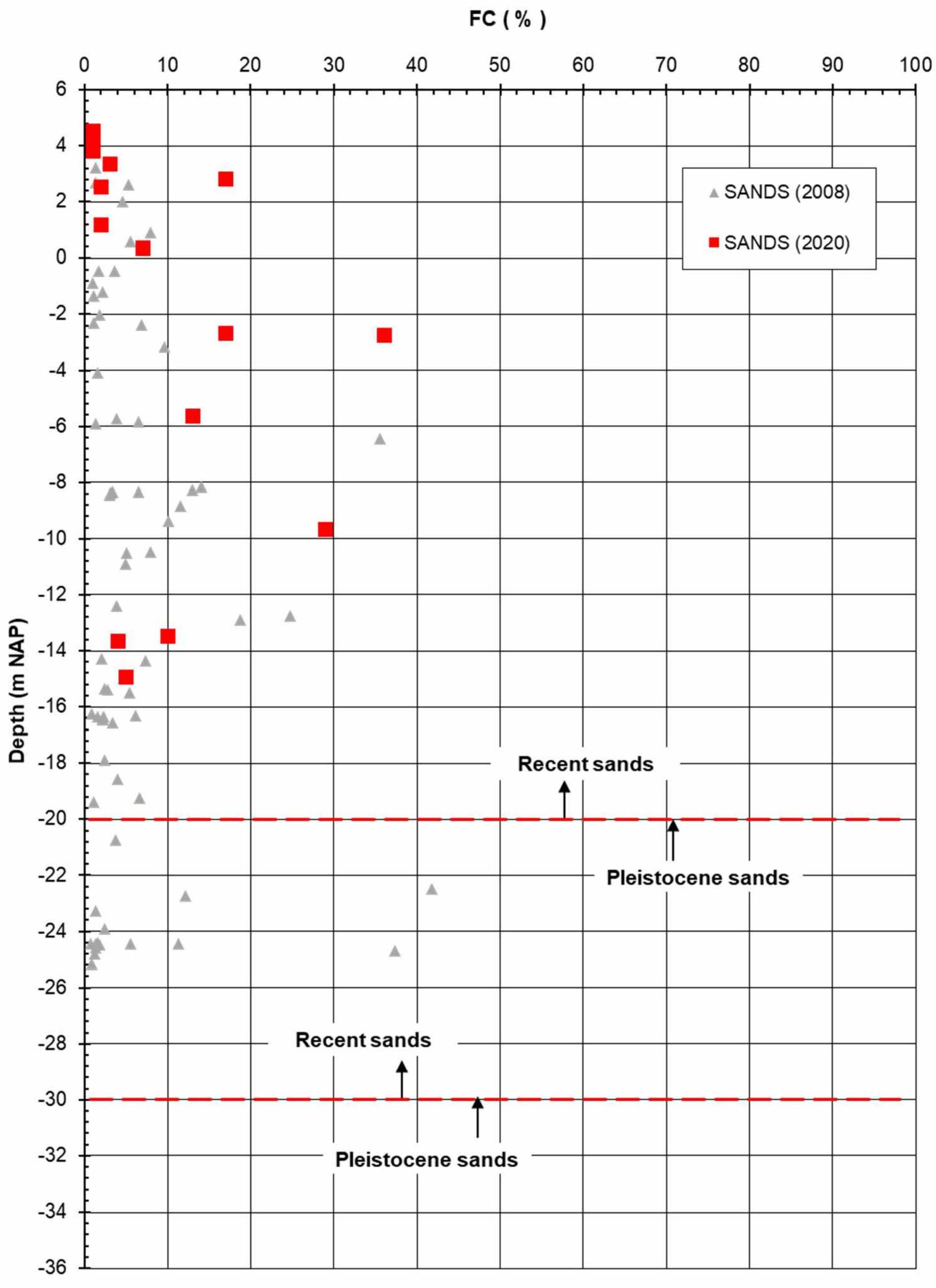
Attach.3 Figure 1- D50



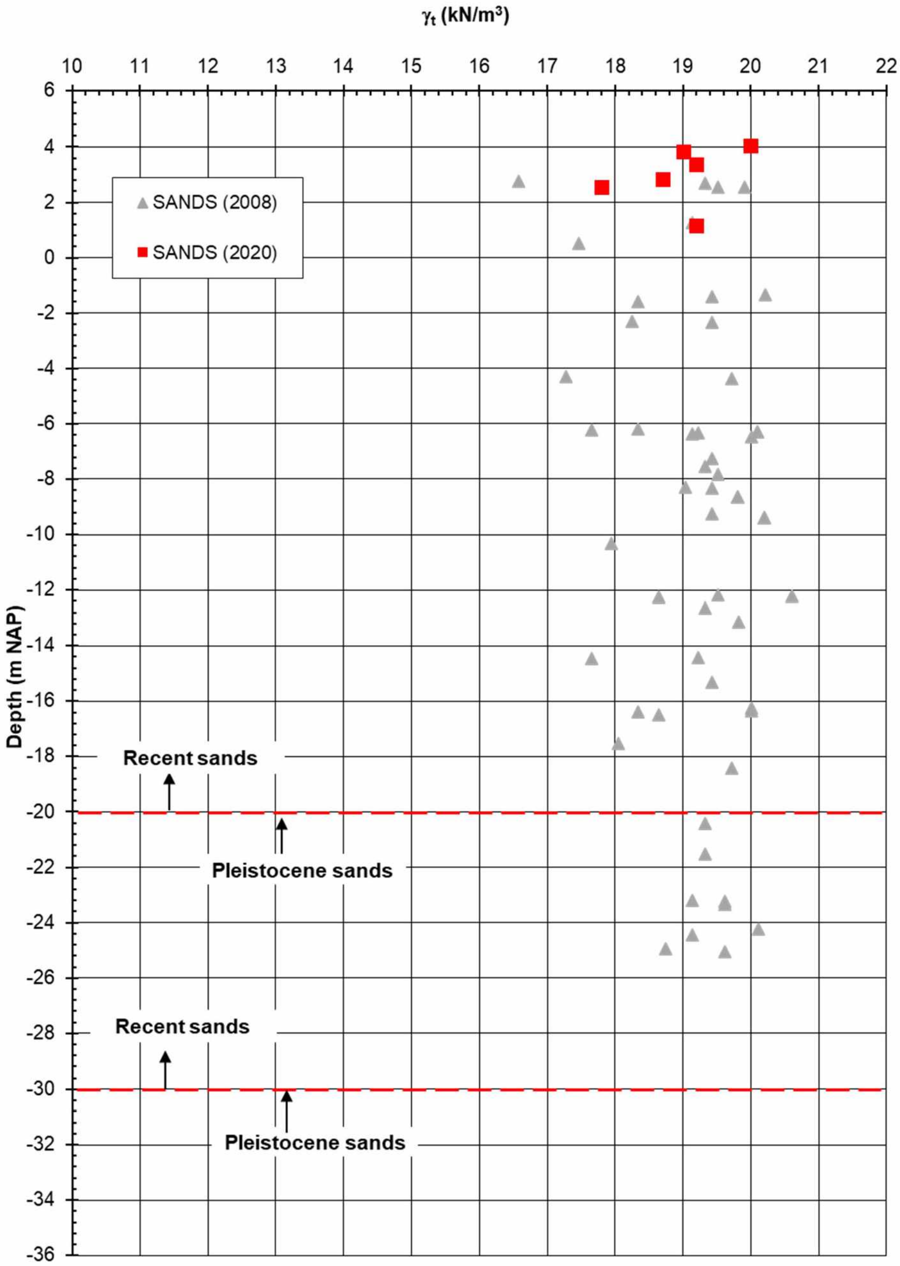
Attach.3 Figure 2- D10



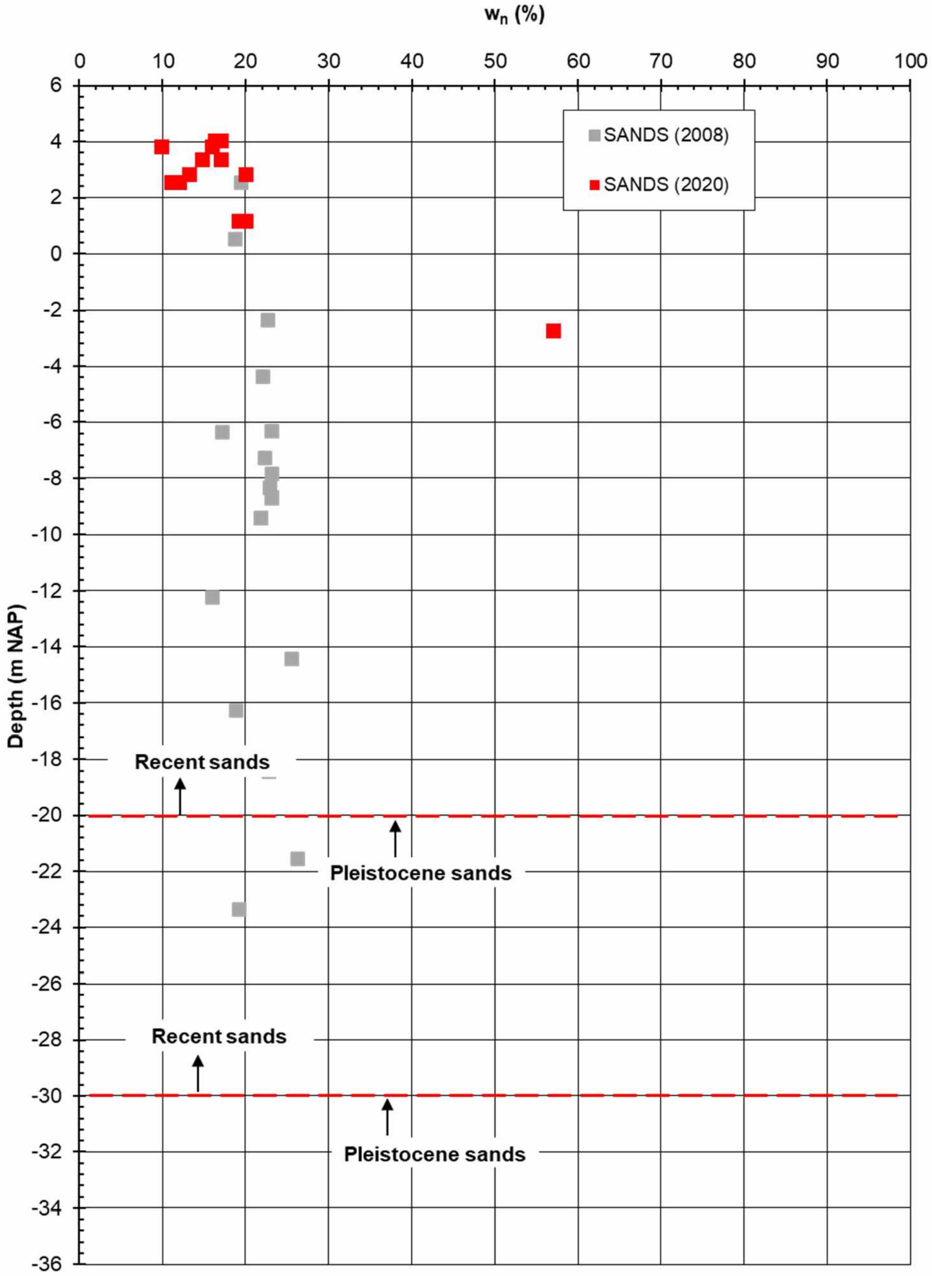
Attach.3 Figure 3- Uniformity coefficient



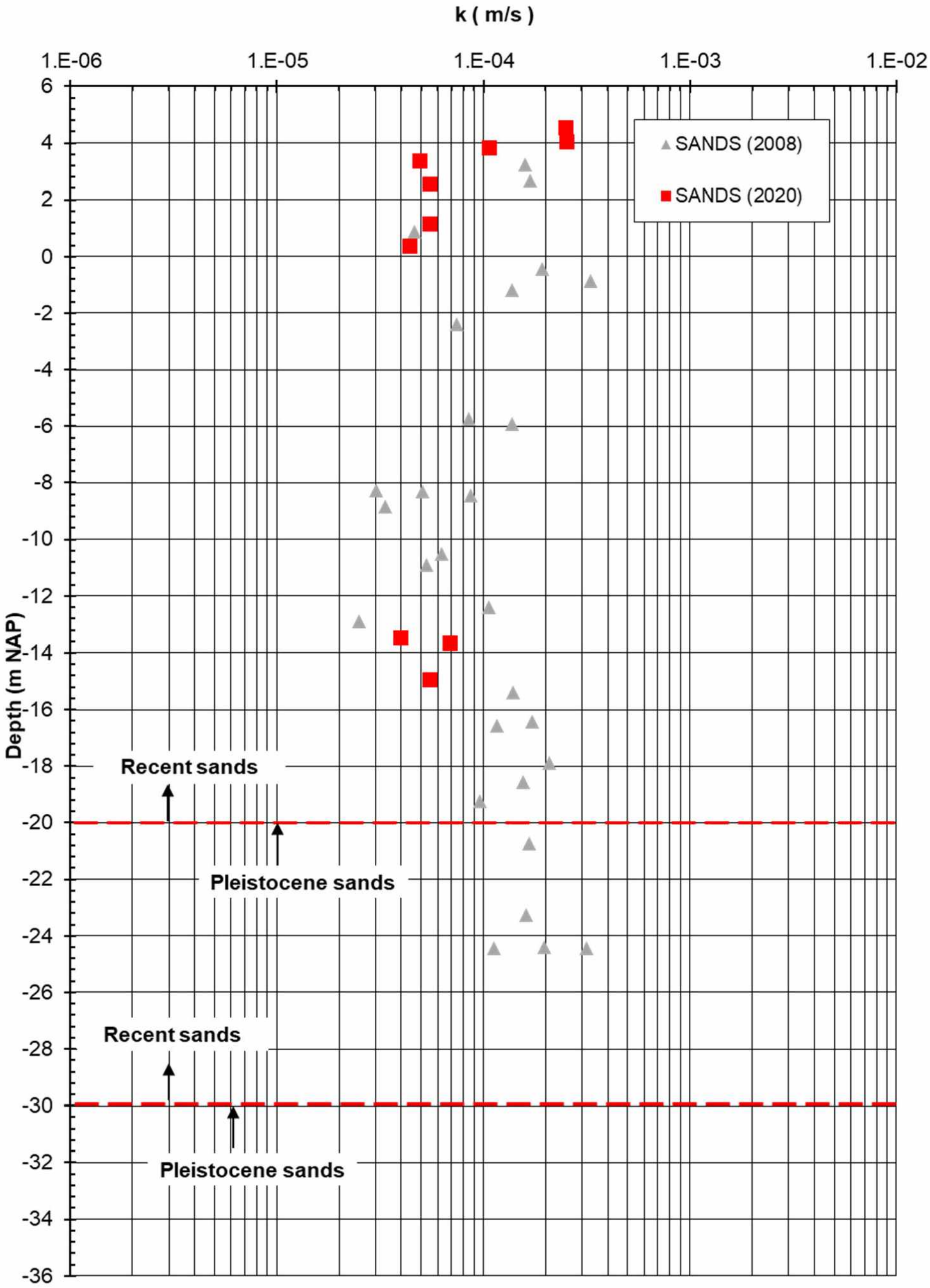
Attach.3 Figure 4- Fine content



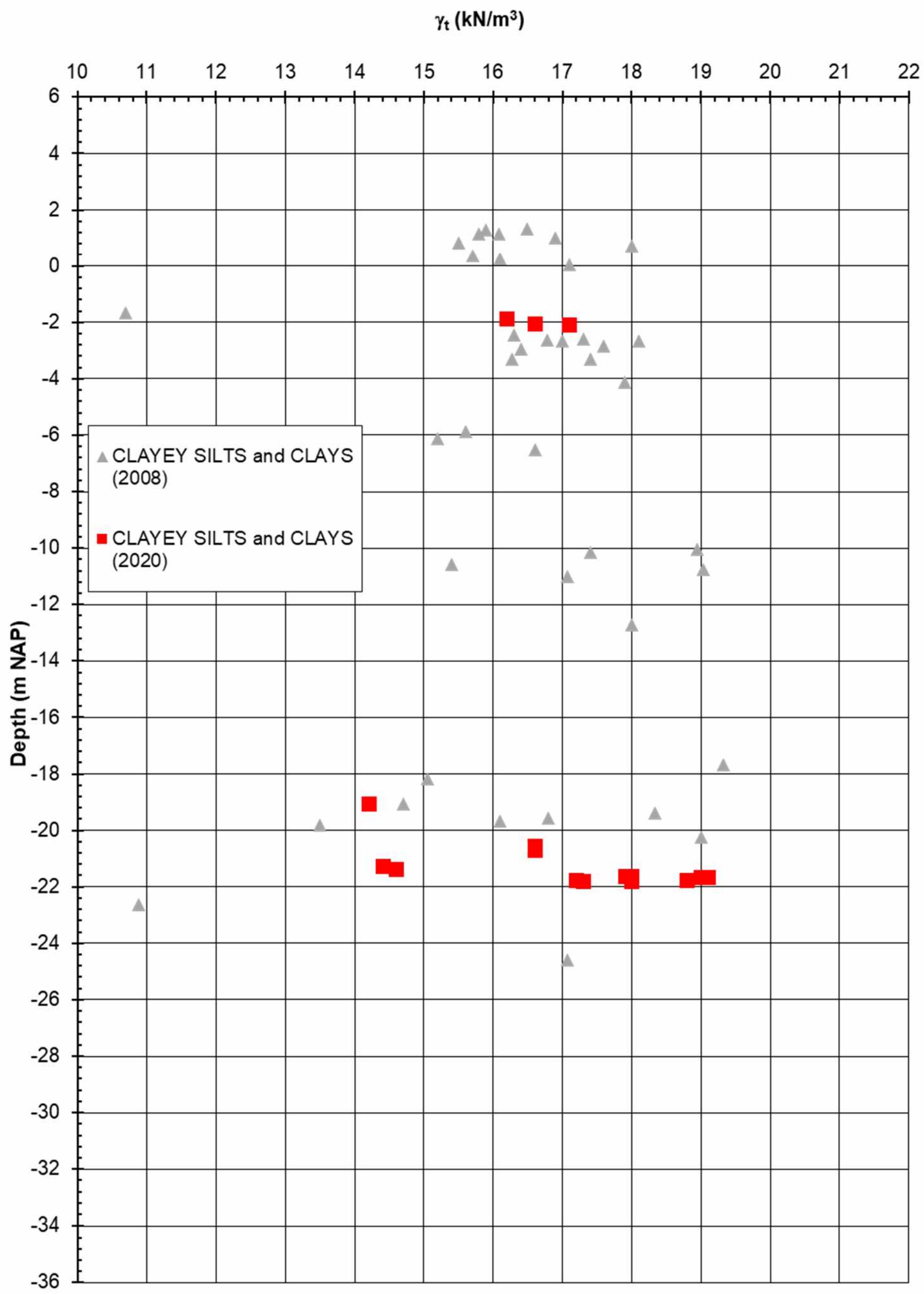
Attach.3 Figure 5- Total unit weight



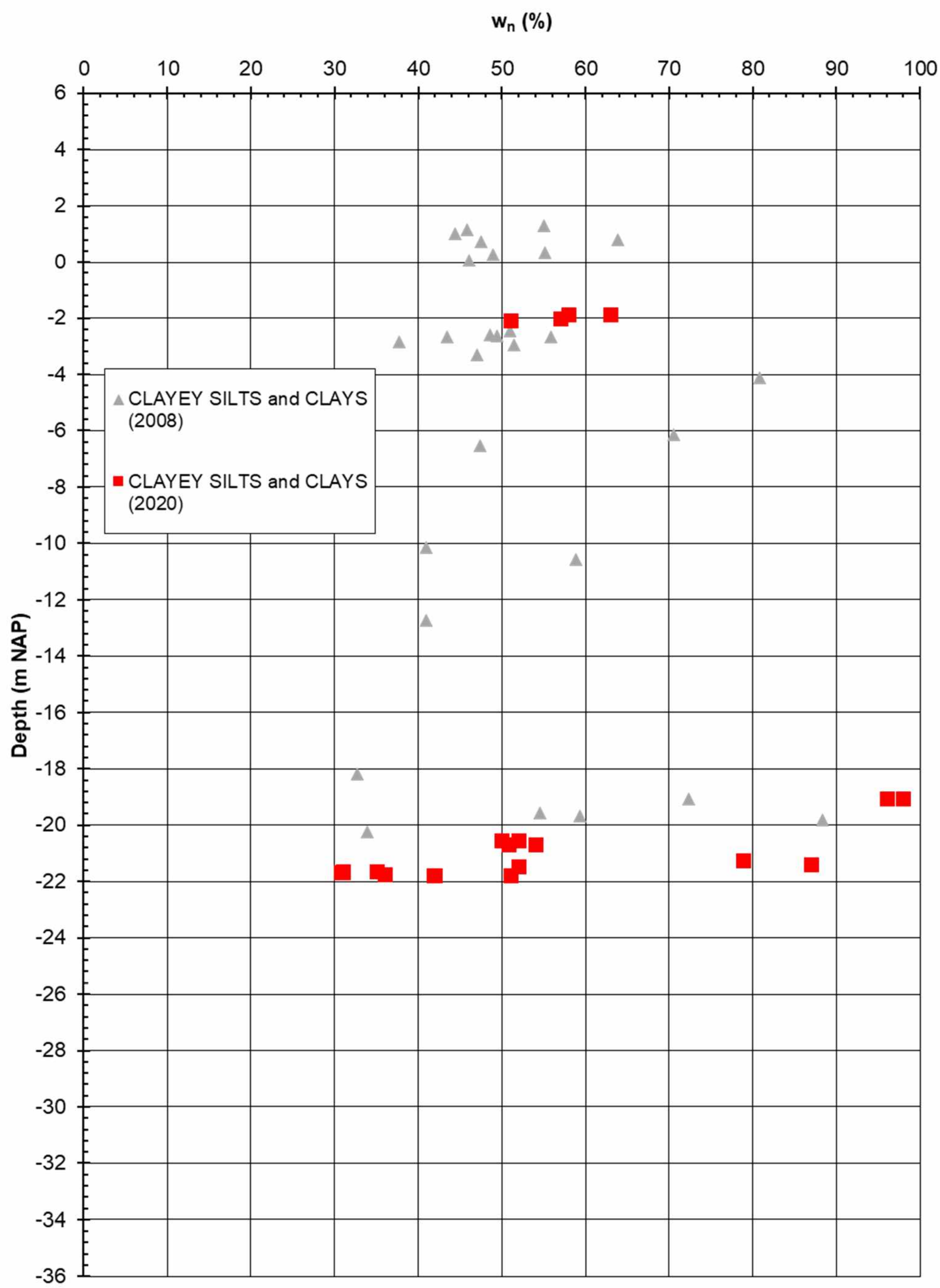
Attach.3 Figure 6- Water content



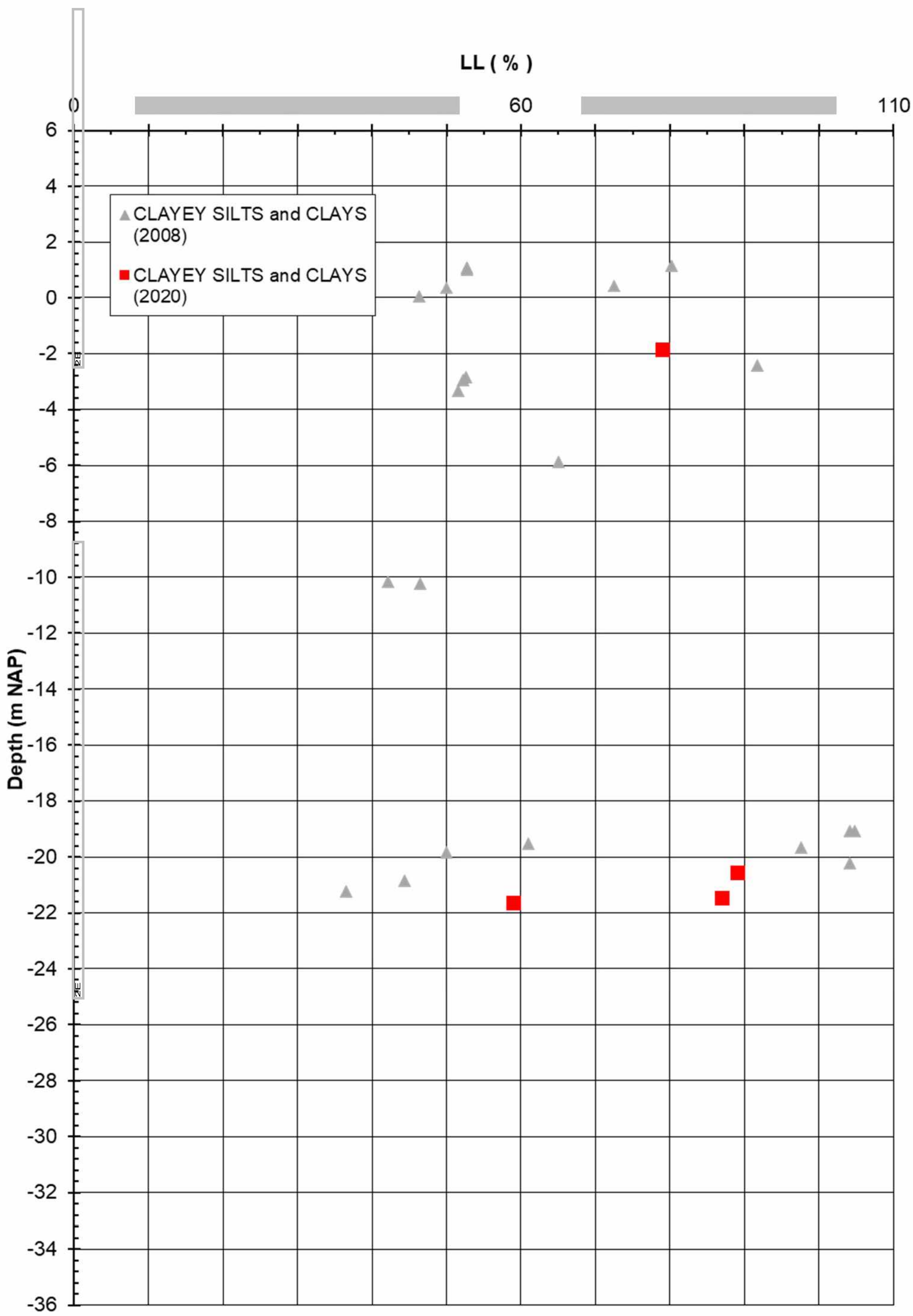
Attach.3 Figure 7- Coefficient of permeability (estimated from D10)



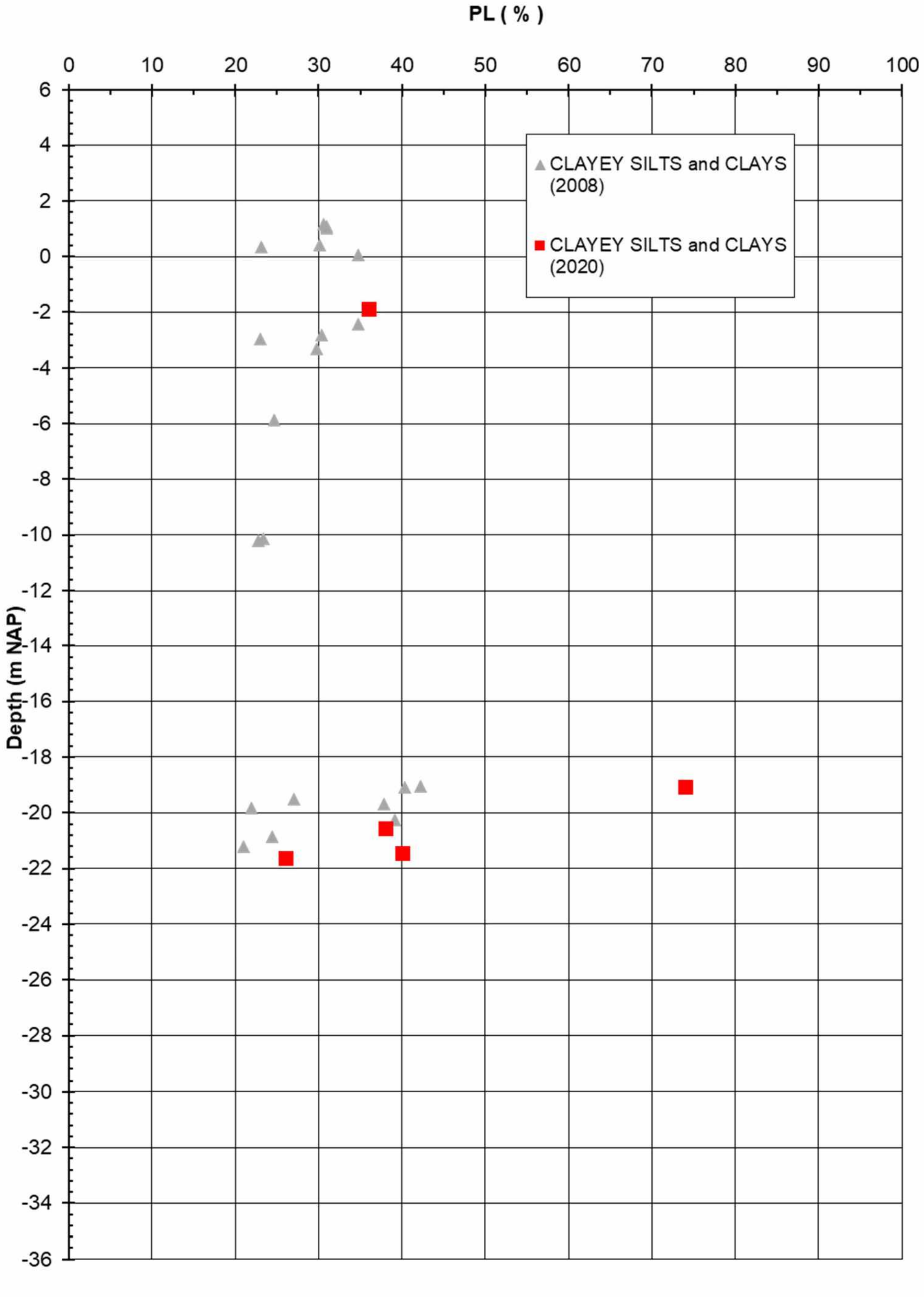
Attach.3 Figure 8- Total unit weight



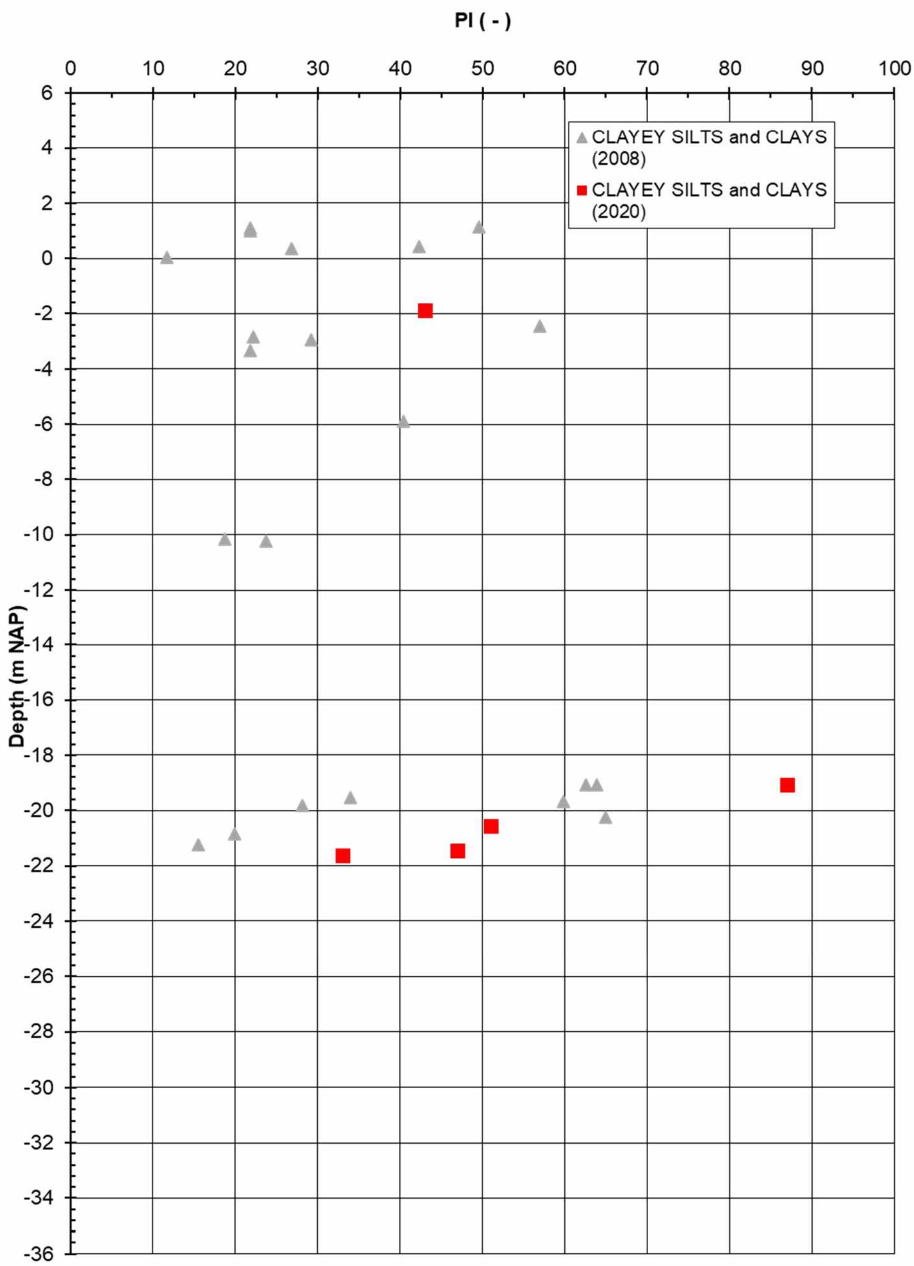
Attach.3 Figure 9- Water content



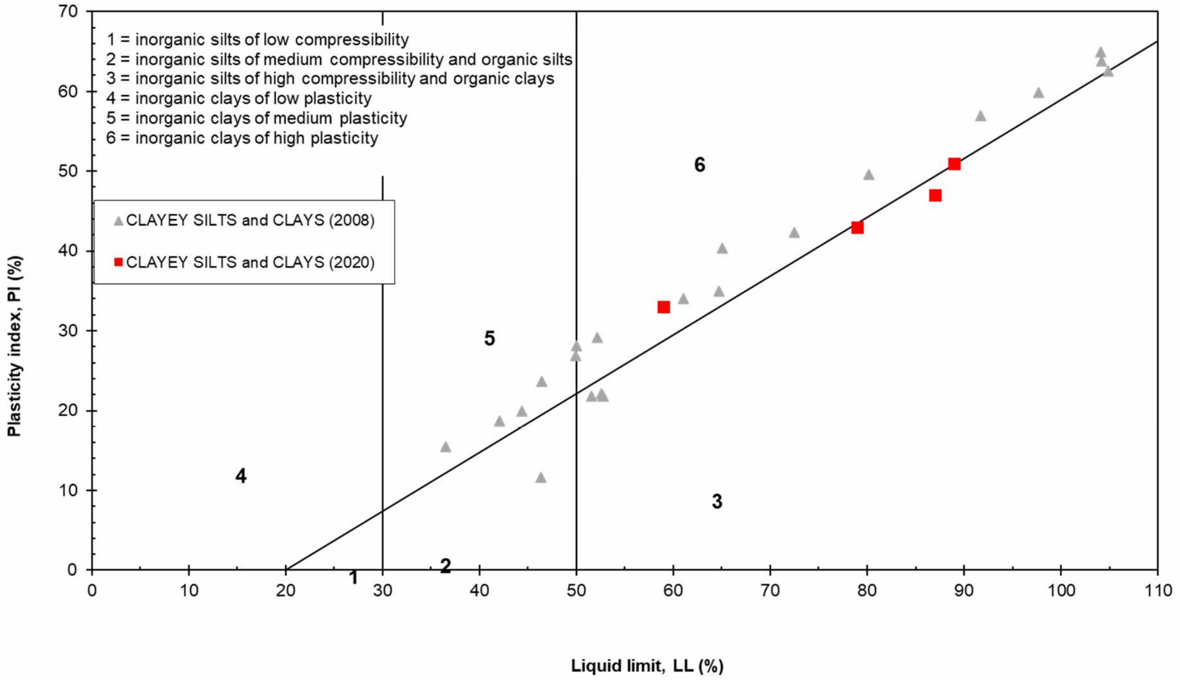
Attach.3 Figure 10- Limit liquid



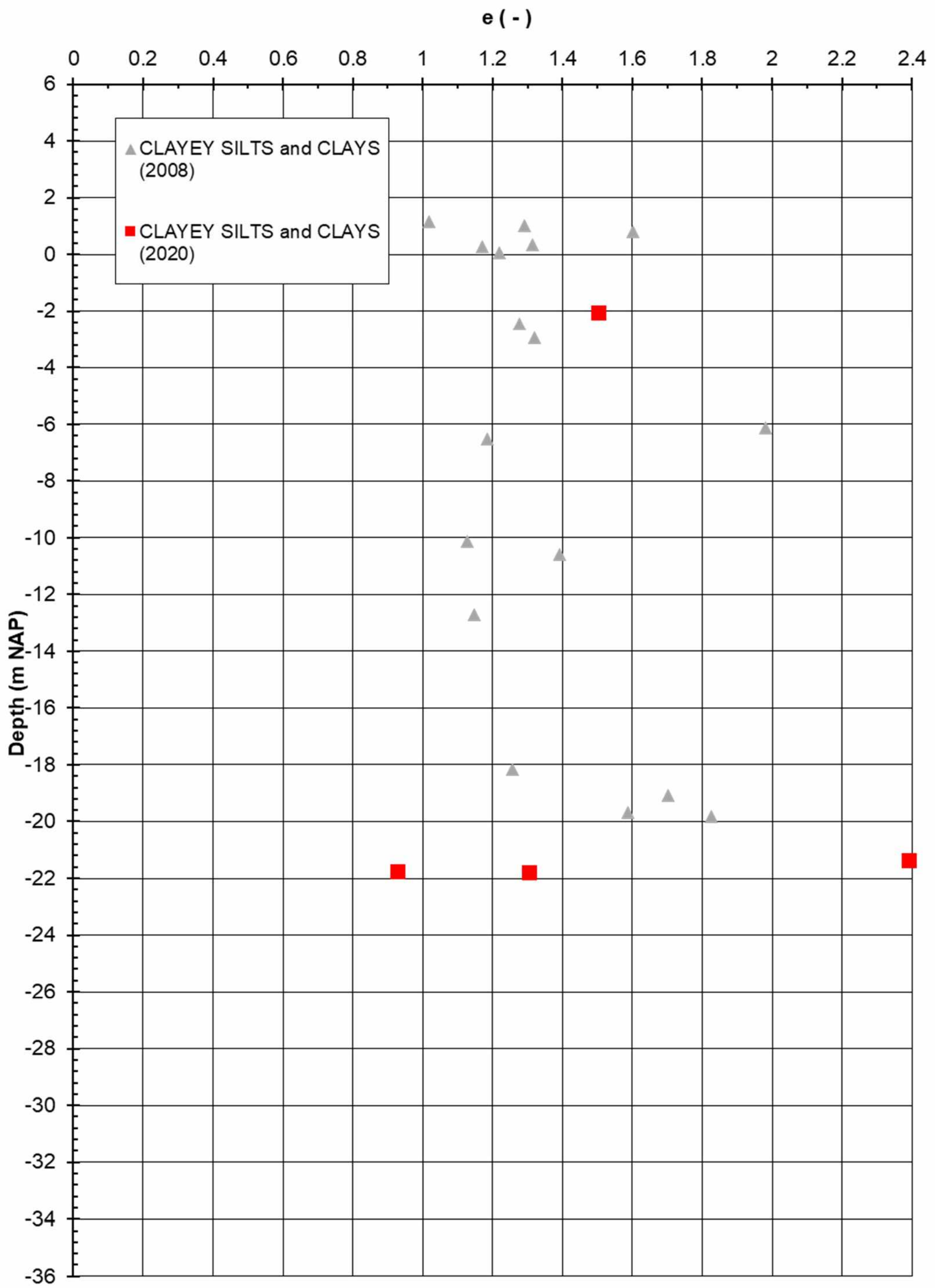
Attach.3 Figure 11- Plastic limit



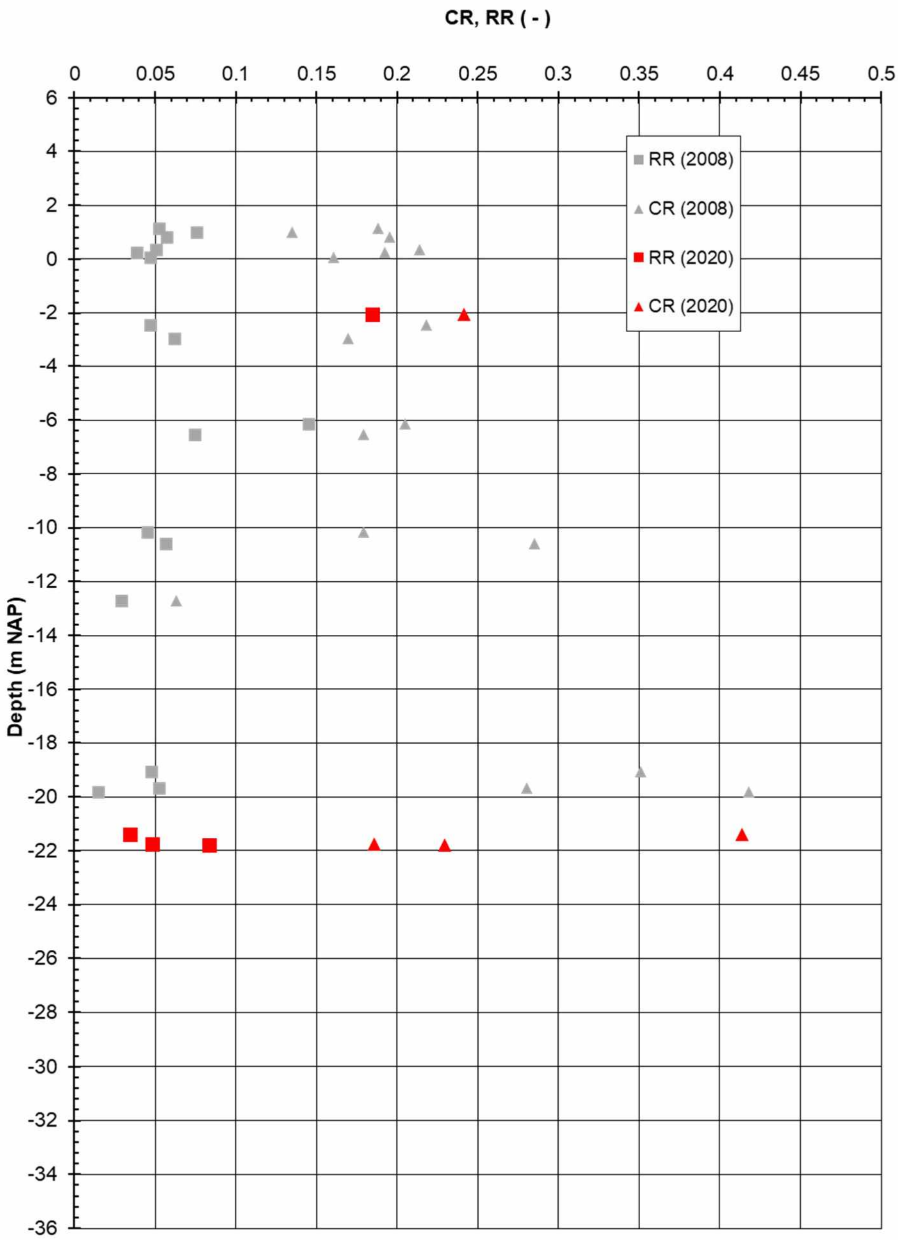
Attach.3 Figure 12- Plasticity index



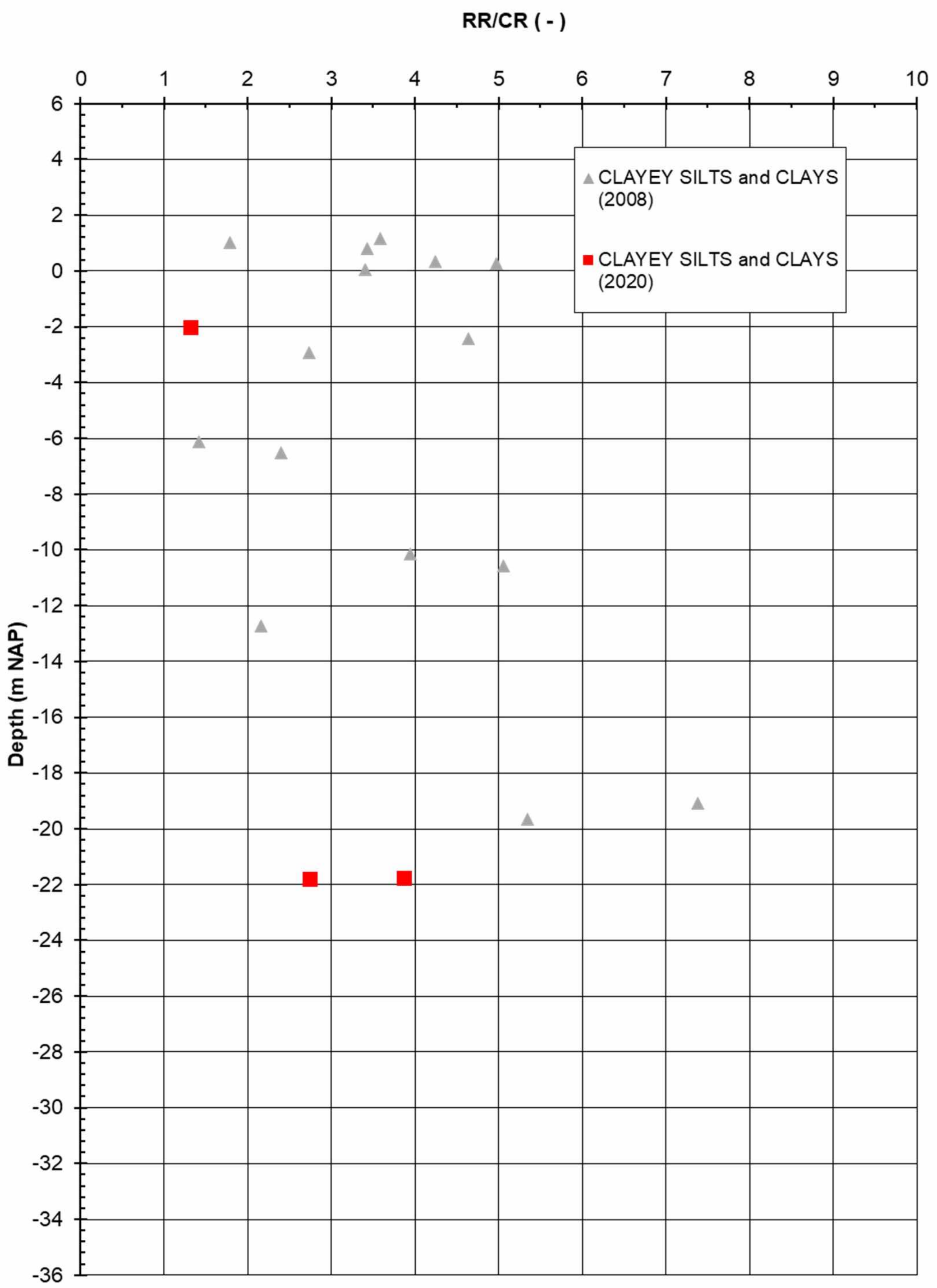
Attach.3 Figure 13- Casagrande plasticity chart



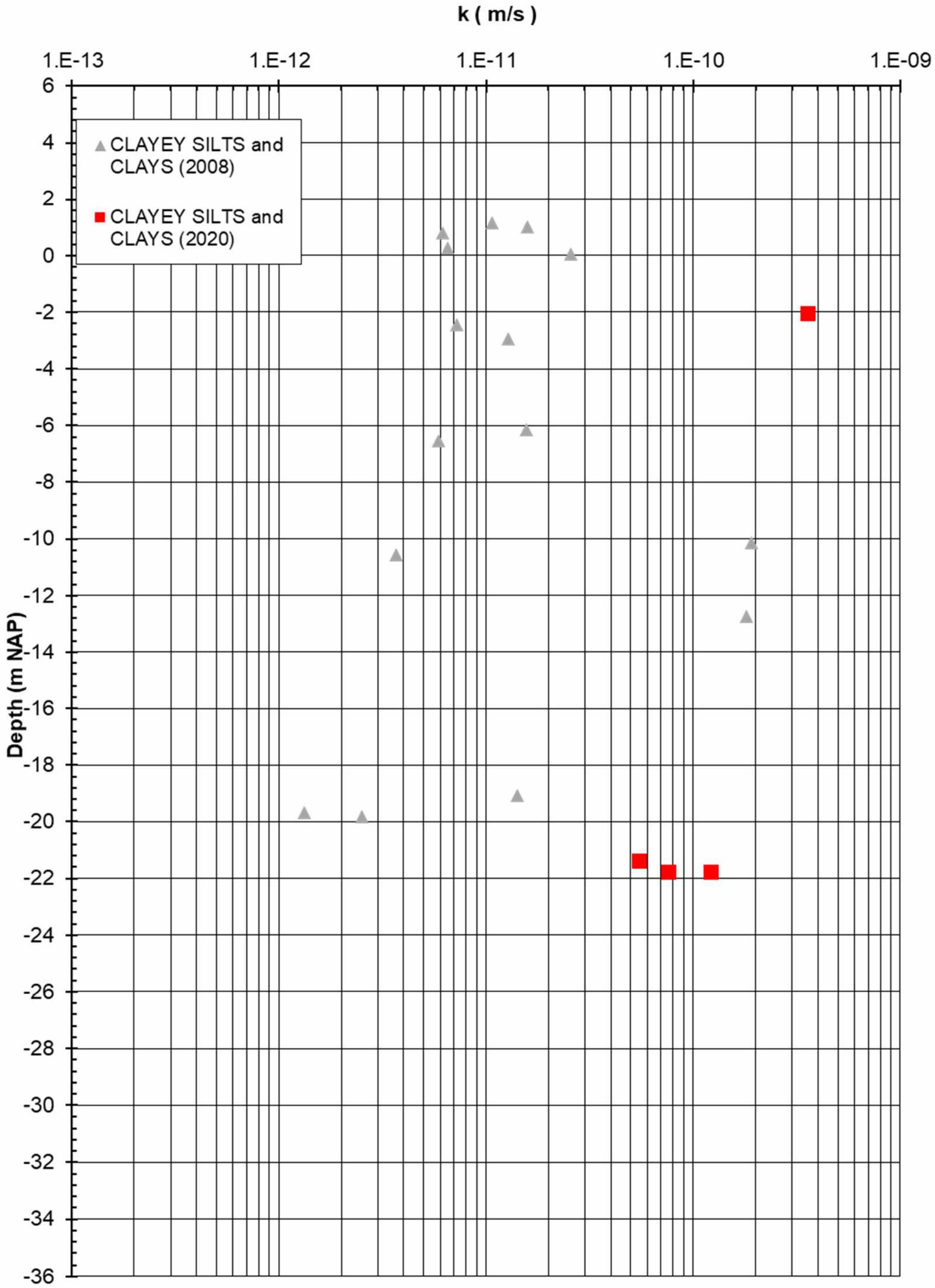
Attach.3 Figure 14- Void ratio



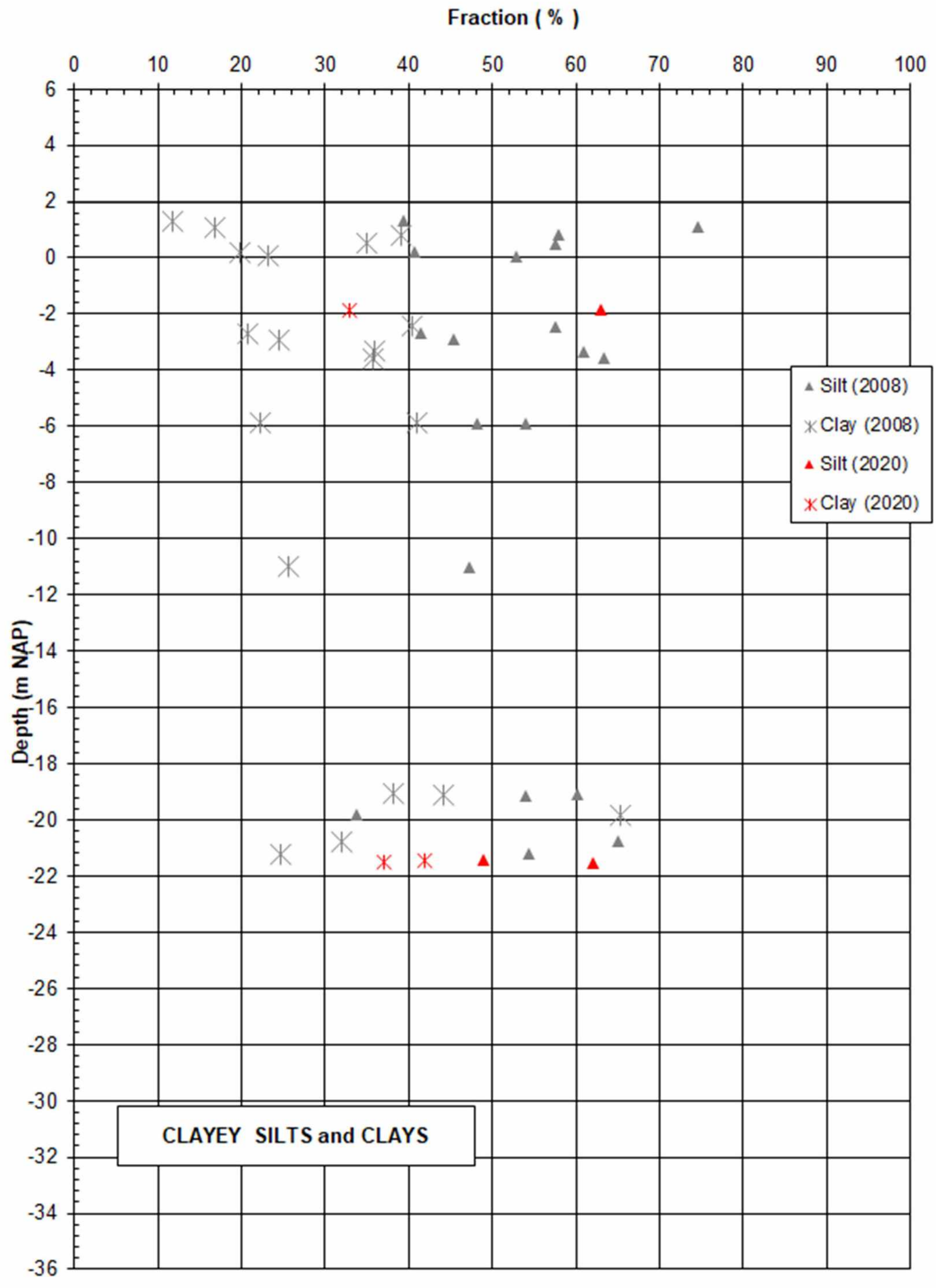
Attach.3 Figure 15- Compression and recompression indexes



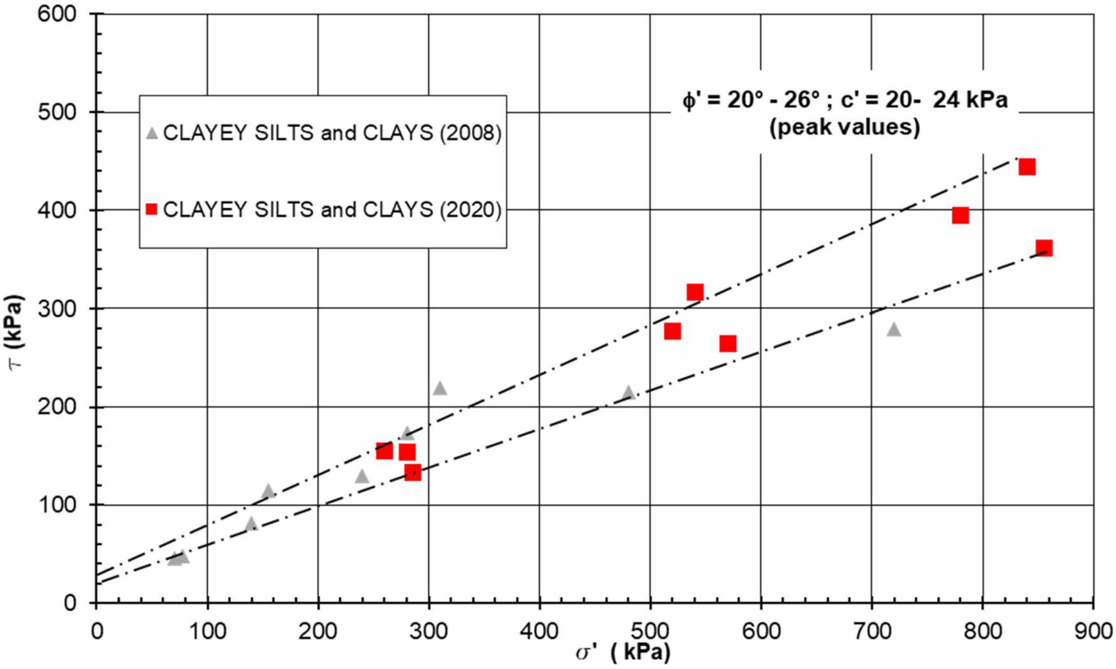
Attach.3 Figure 16- RR/CR ratio

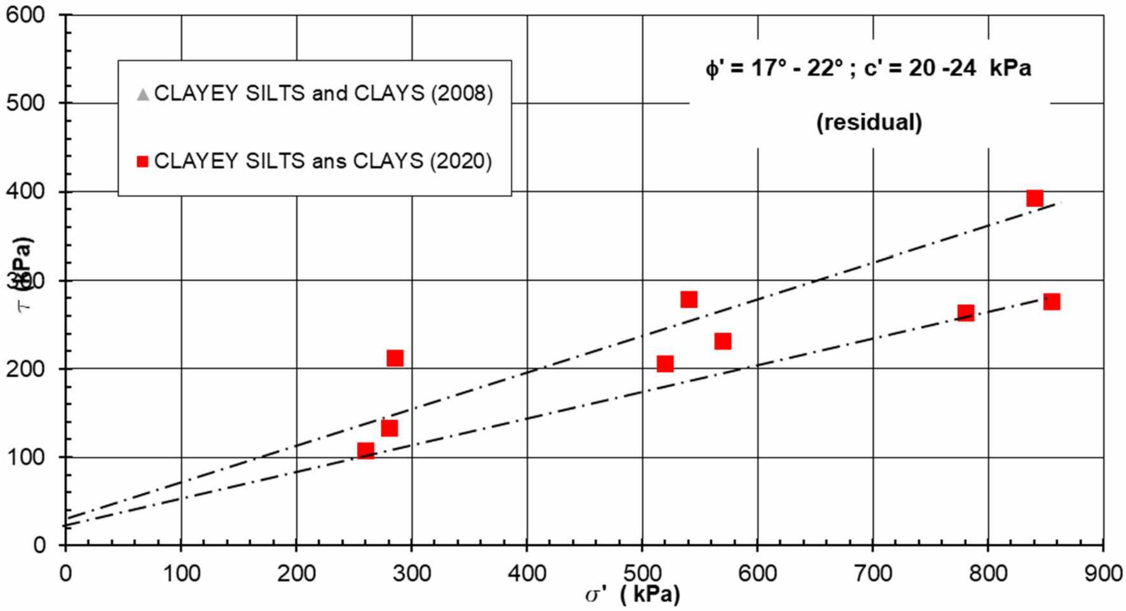


Attach.3 Figure 17- Coefficient of permeability



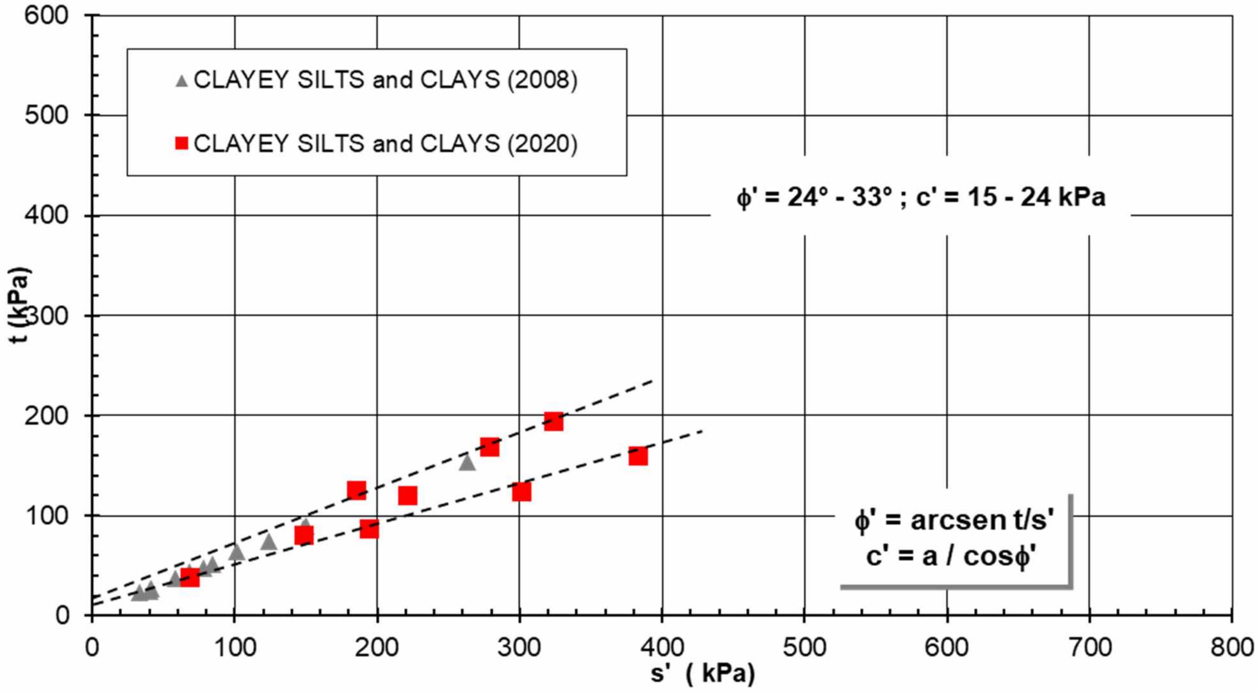
Attach.3 Figure 18- Fine grain distribution in clay samples



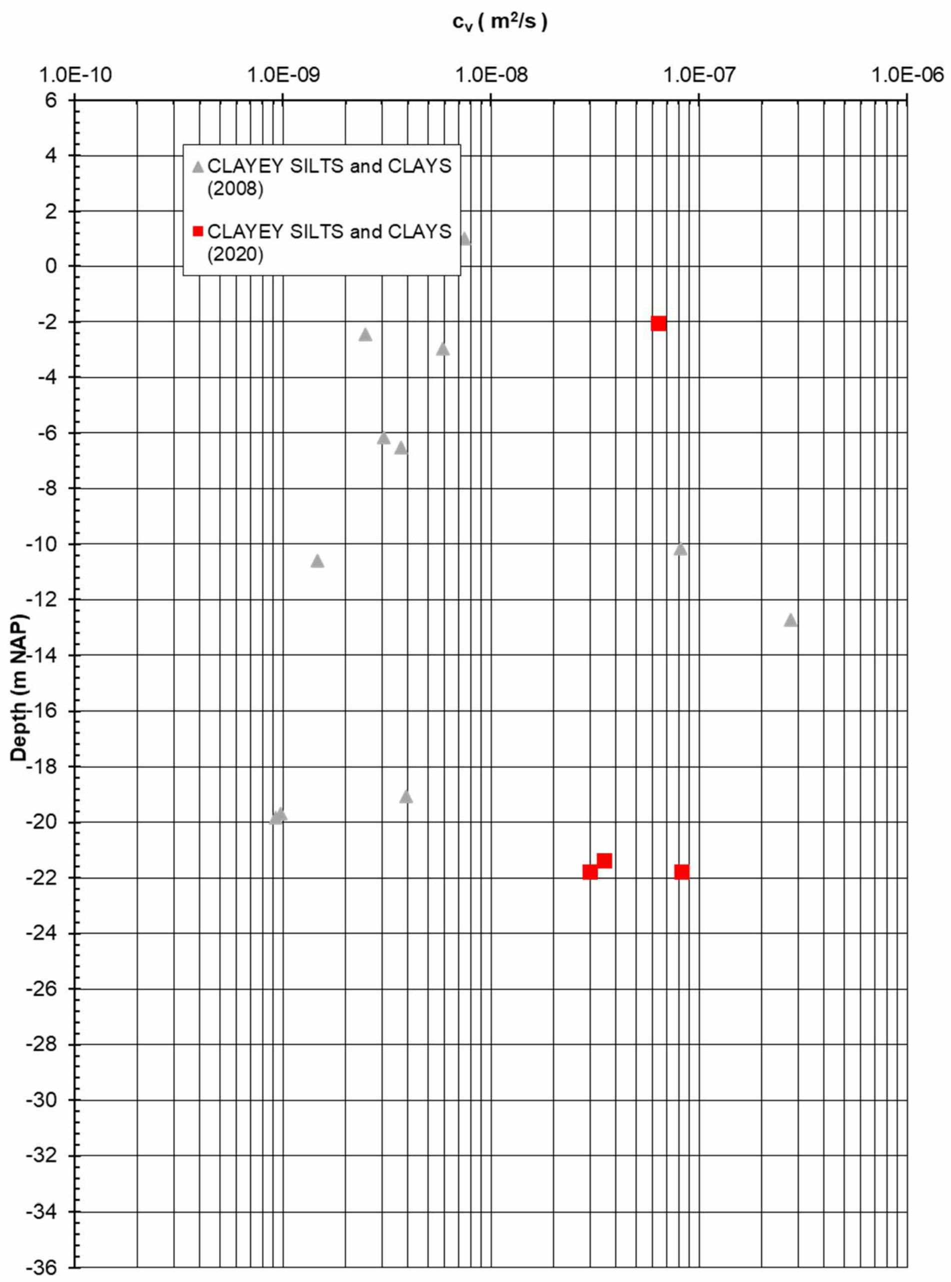


Attach.3 Figure 19- Direct shear test results on clay samples

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE

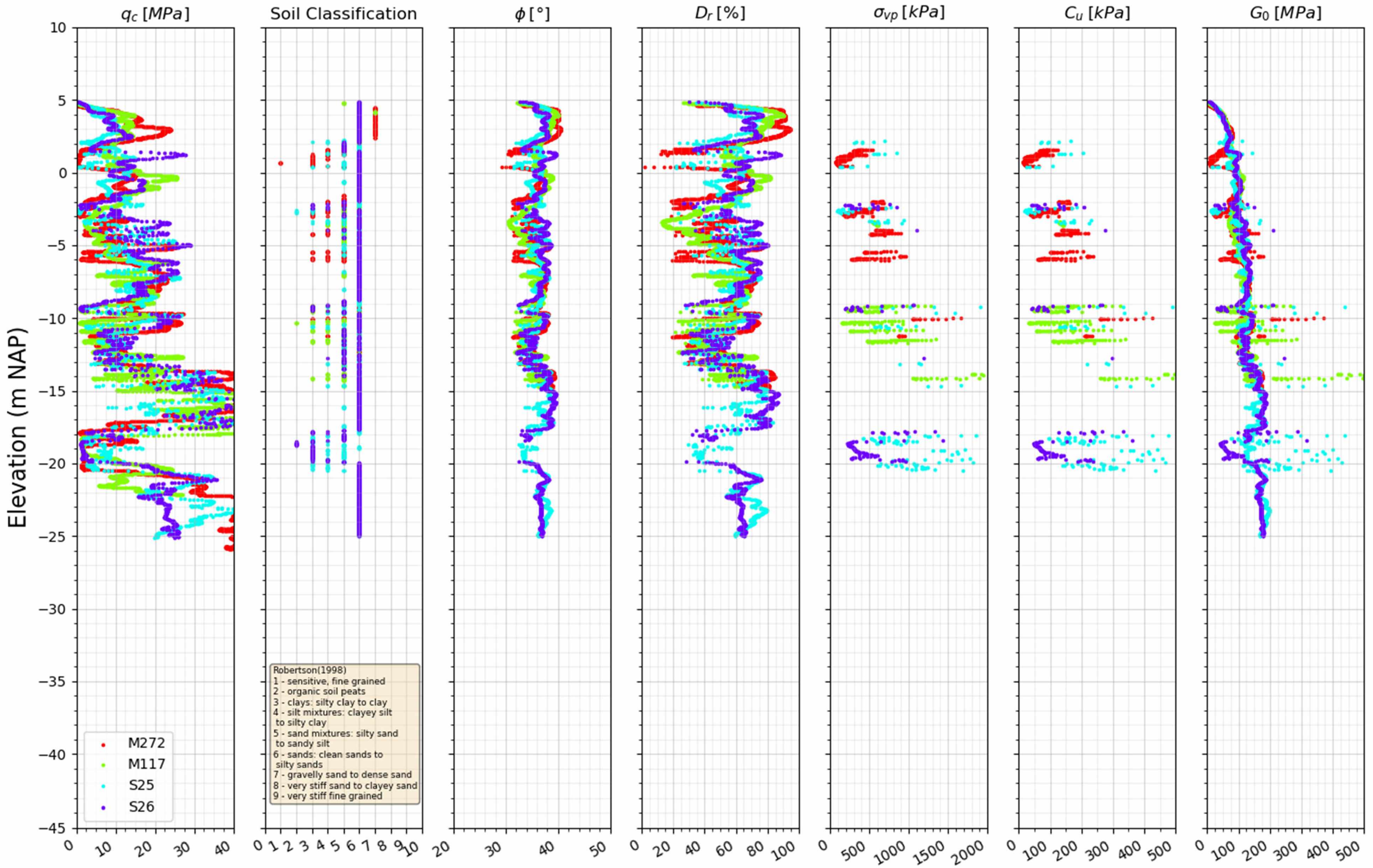


Attach.3 Figure 20- TX-CIU test results on clay samples



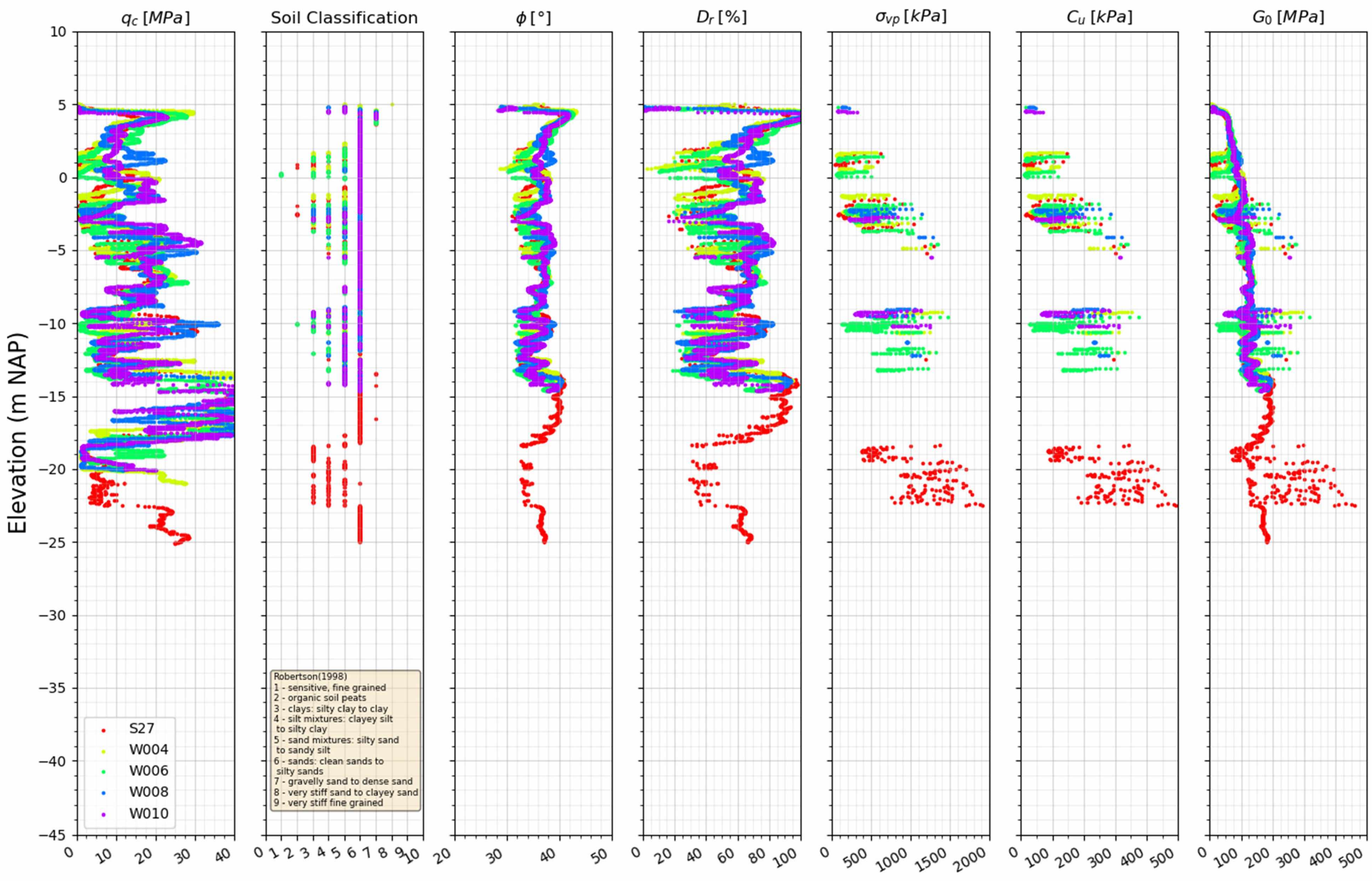
Attach.3 Figure 21- Coefficient of vertical consolidation

RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE

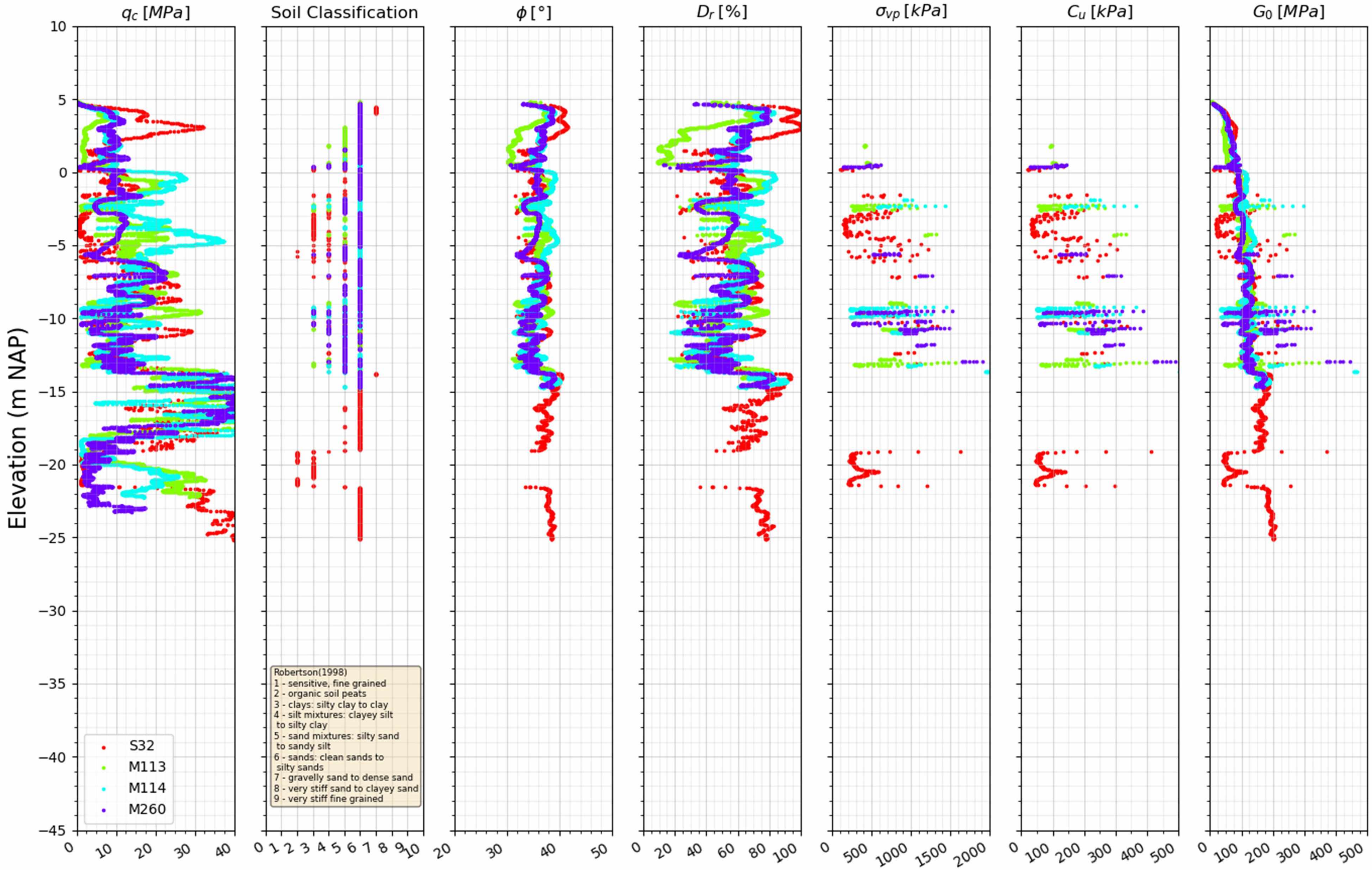


Attach.3 Figure 22- CPTs in Refinery RDCG Plant Area

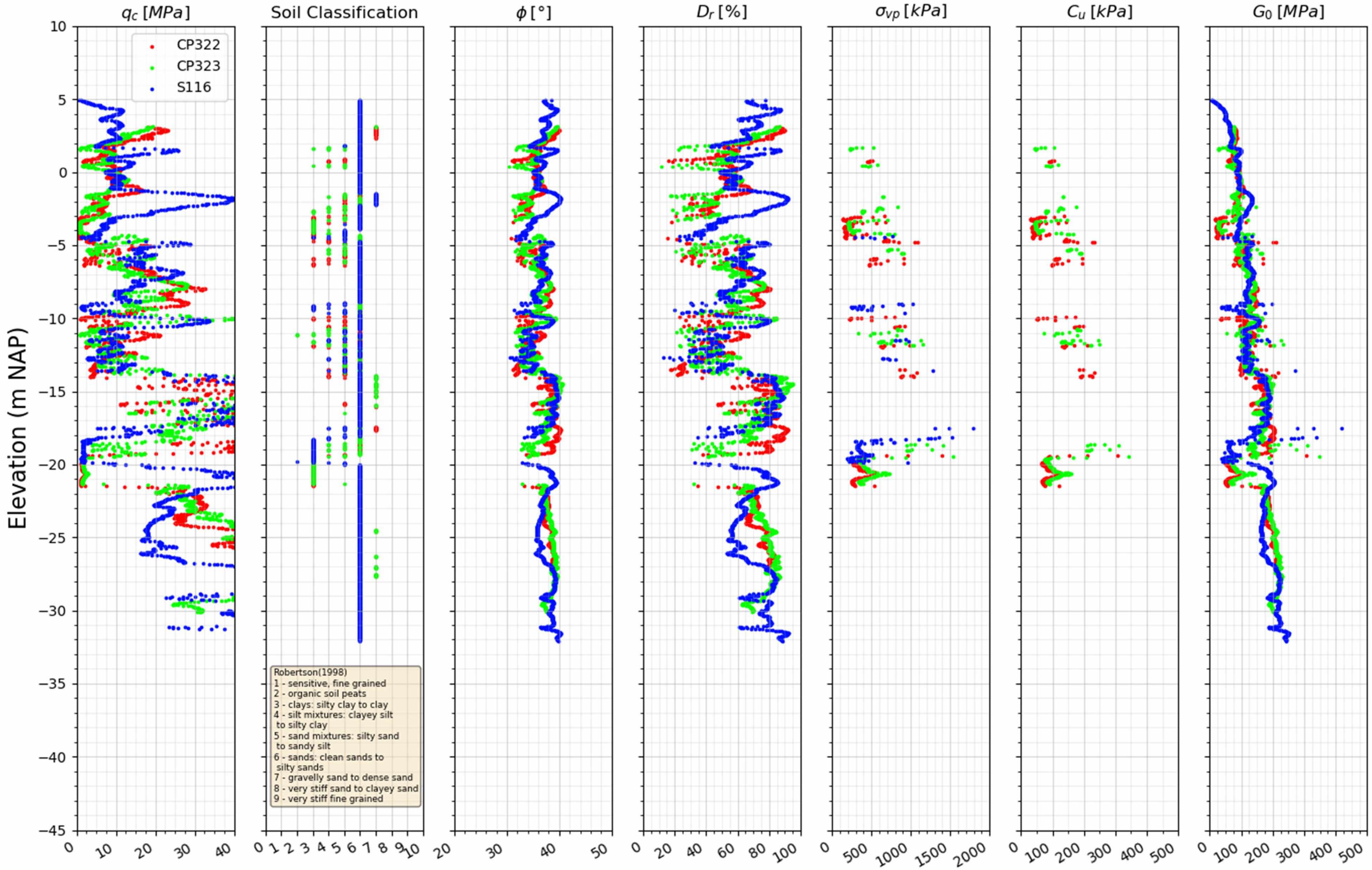
RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



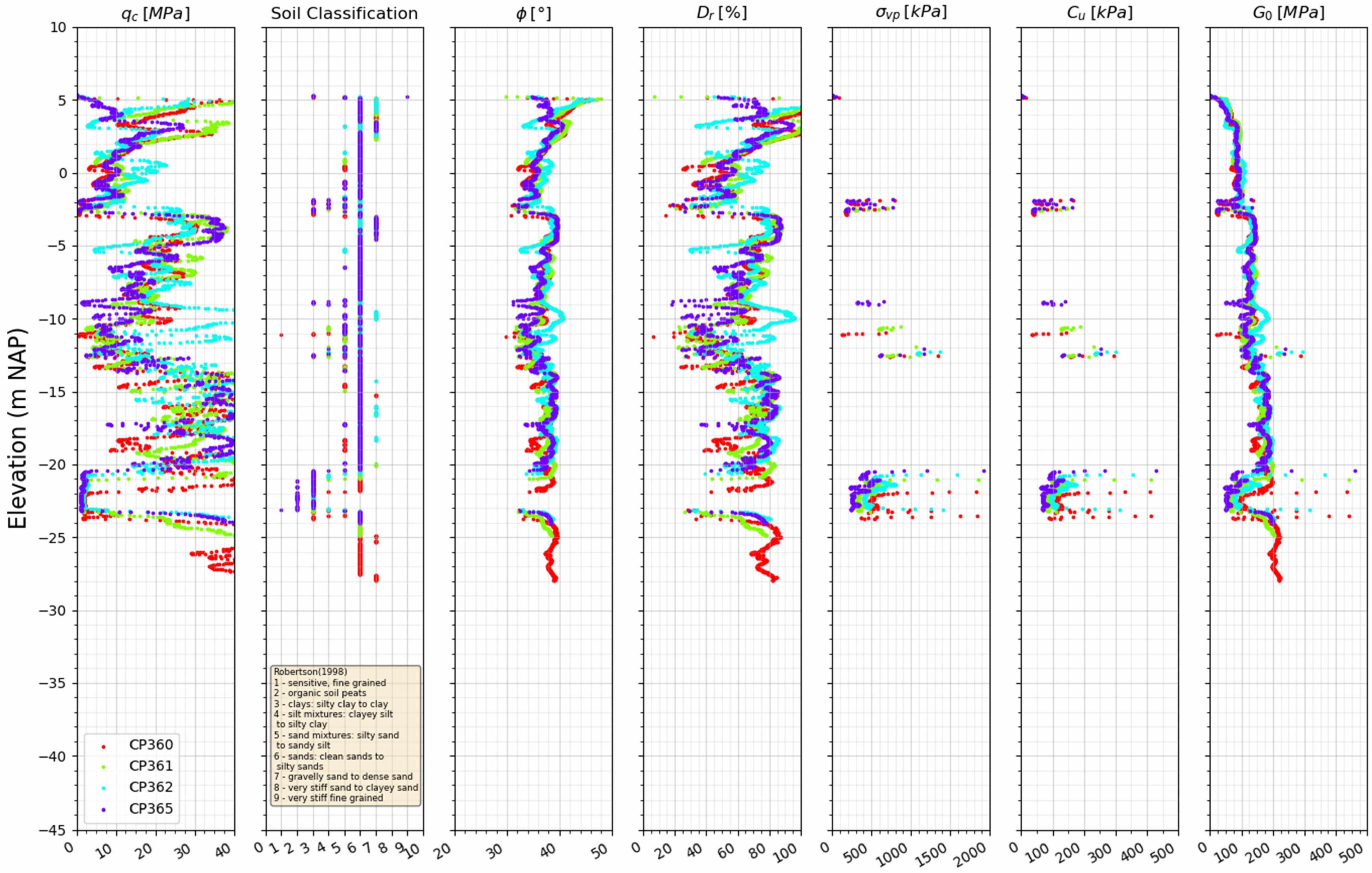
Attach.3 Figure 23- CPTs in Refinery RDCG Plant Area



Attach.3 Figure 24- CPTs in Refinery RDCG Plant Area

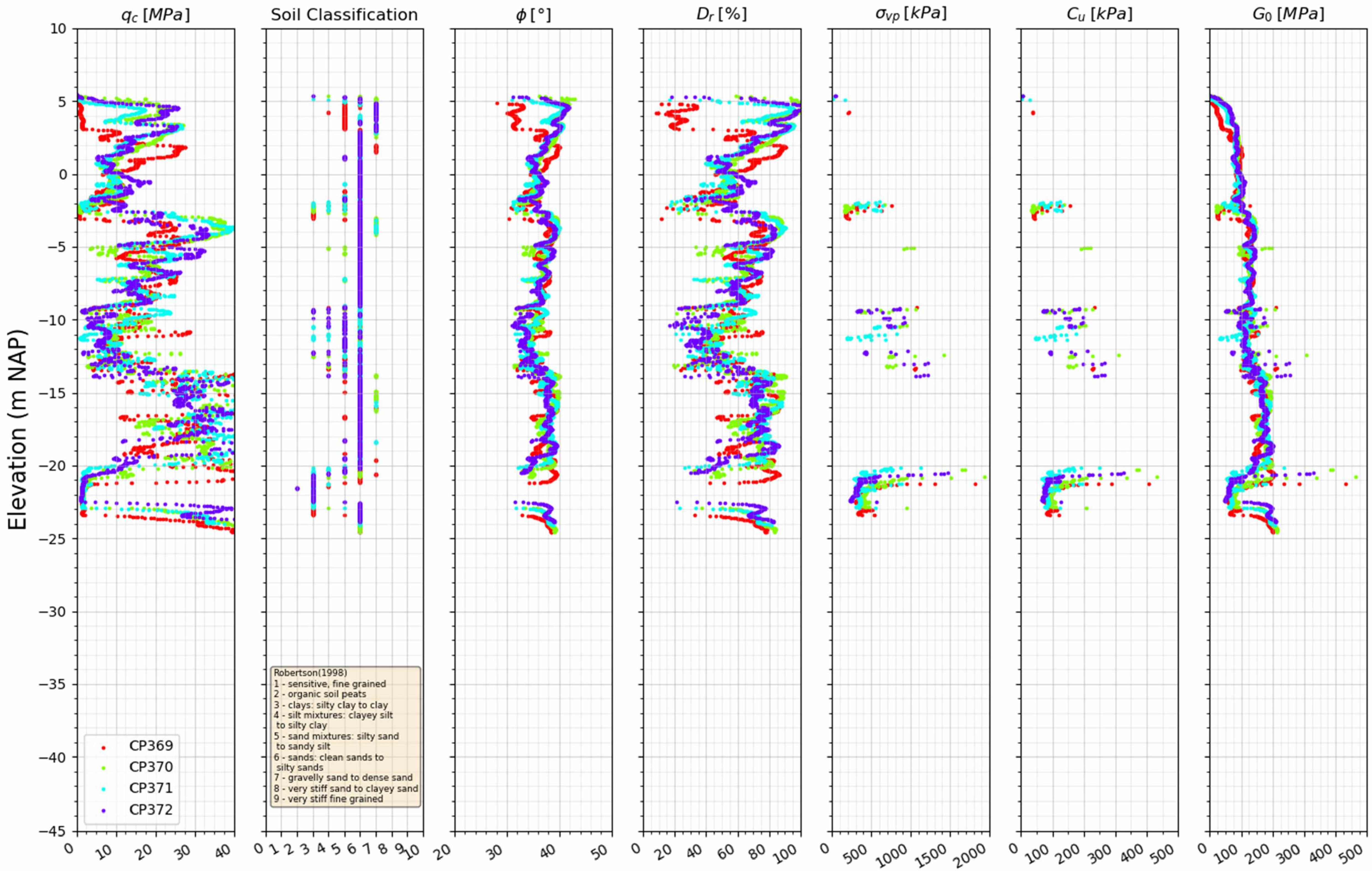


Attach.3 Figure 25- CPTs in Refinery RDCG Plant Area

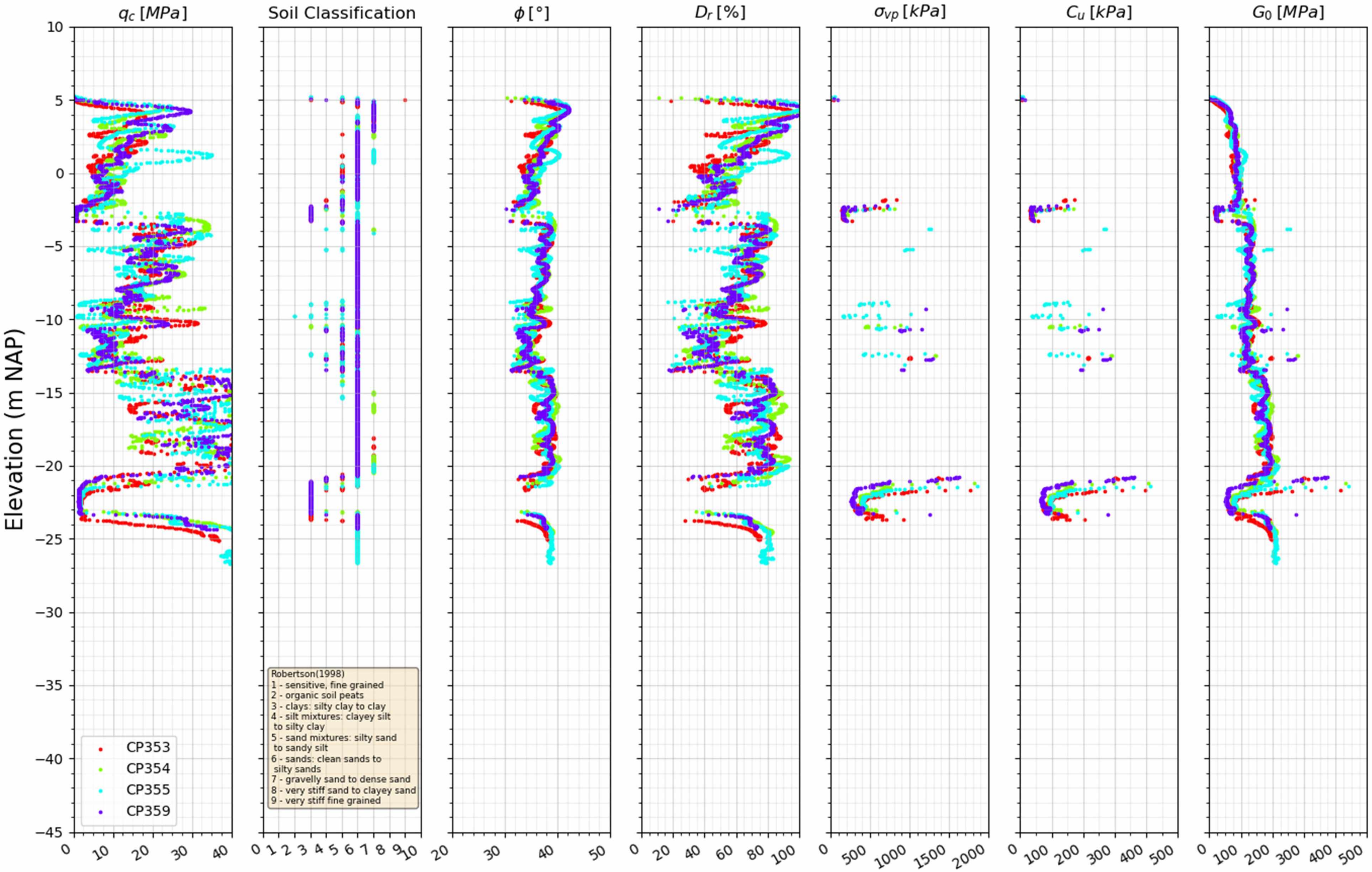


Attach.3 Figure 26- CPTs in Refinery RDCG Plant Area

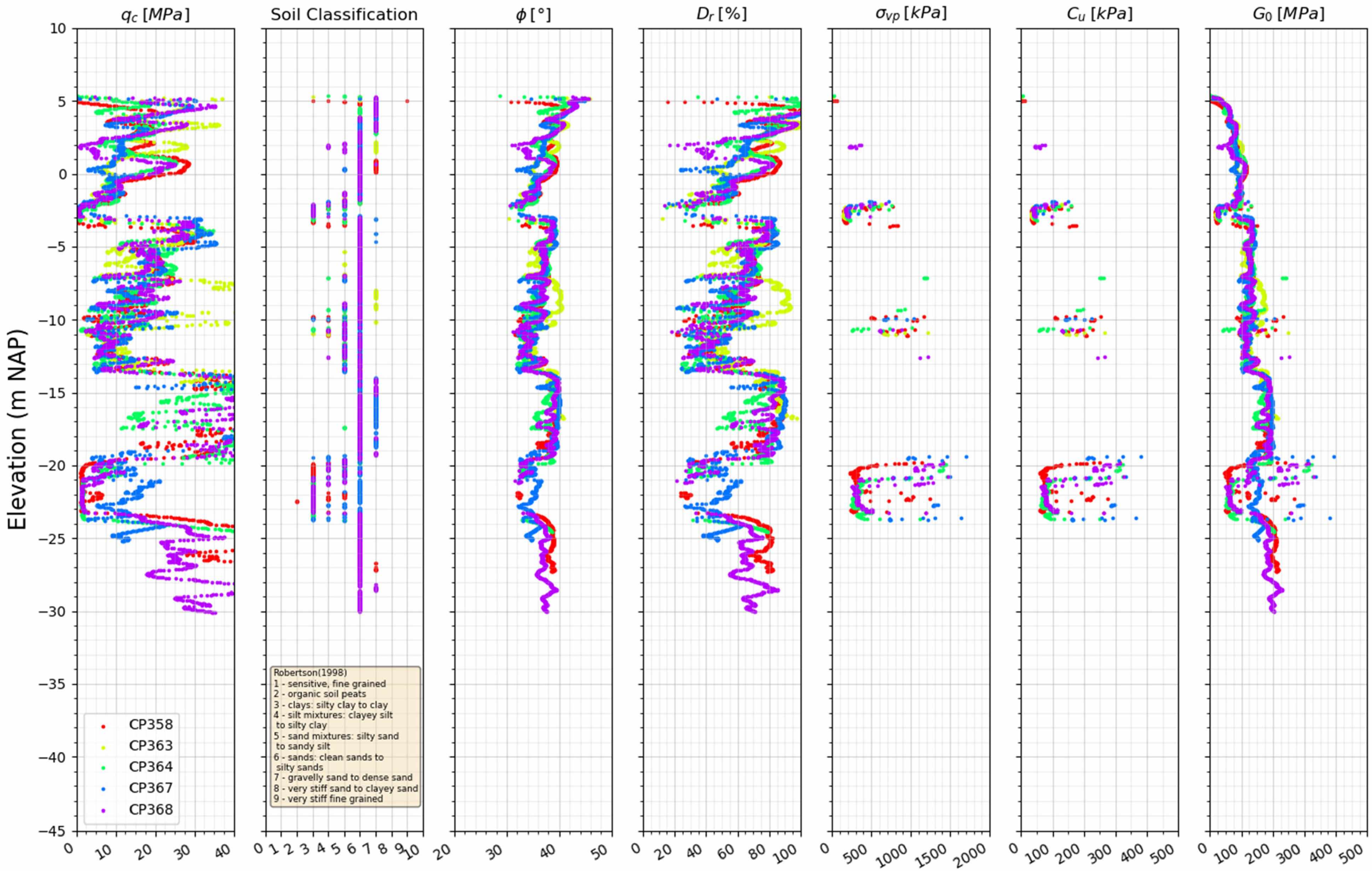
RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



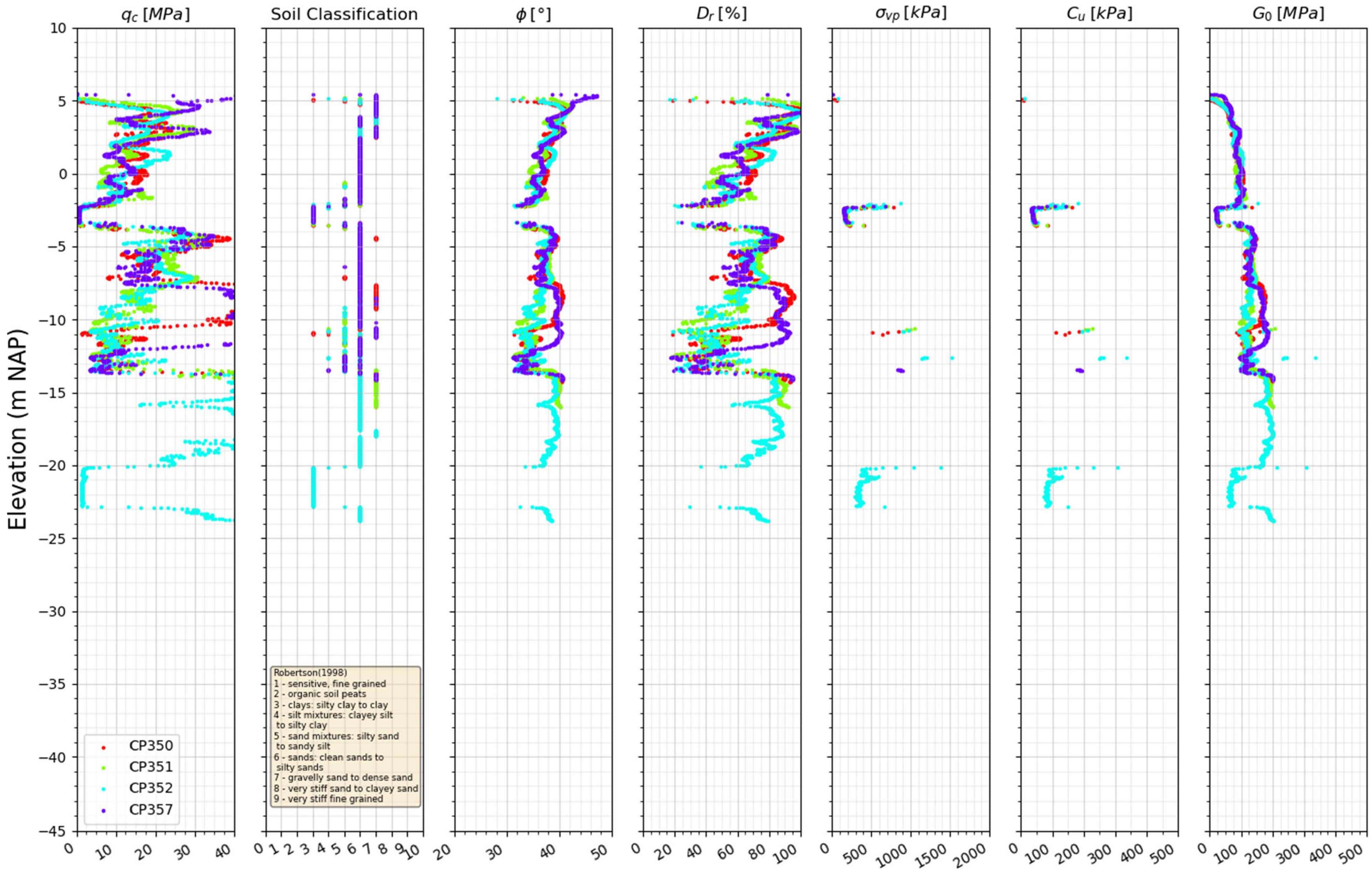
Attach.3 Figure 27- CPTs in Refinery RDCG Plant Area



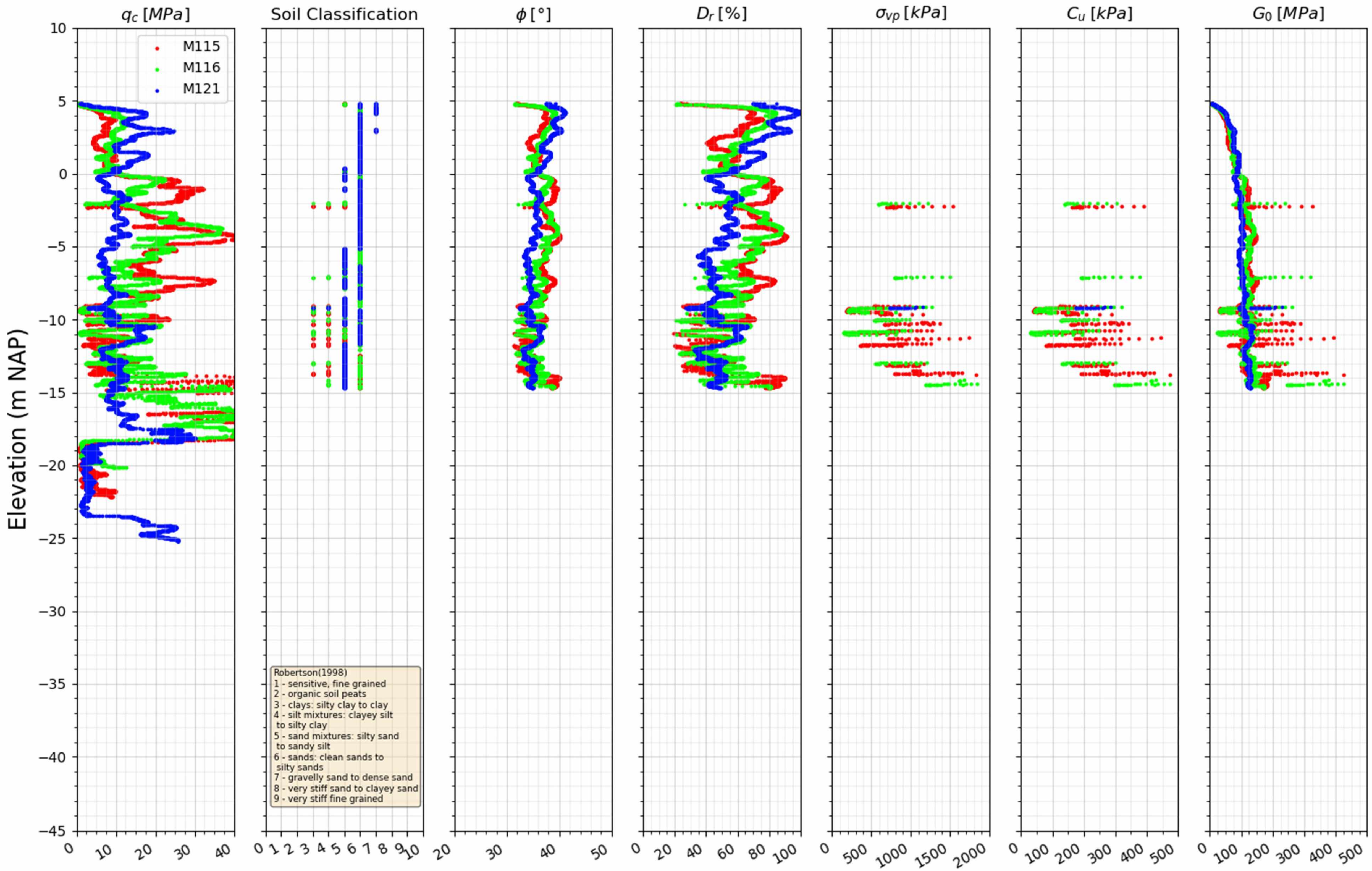
Attach.3 Figure 28- CPTs in Refinery RDCG Plant Area



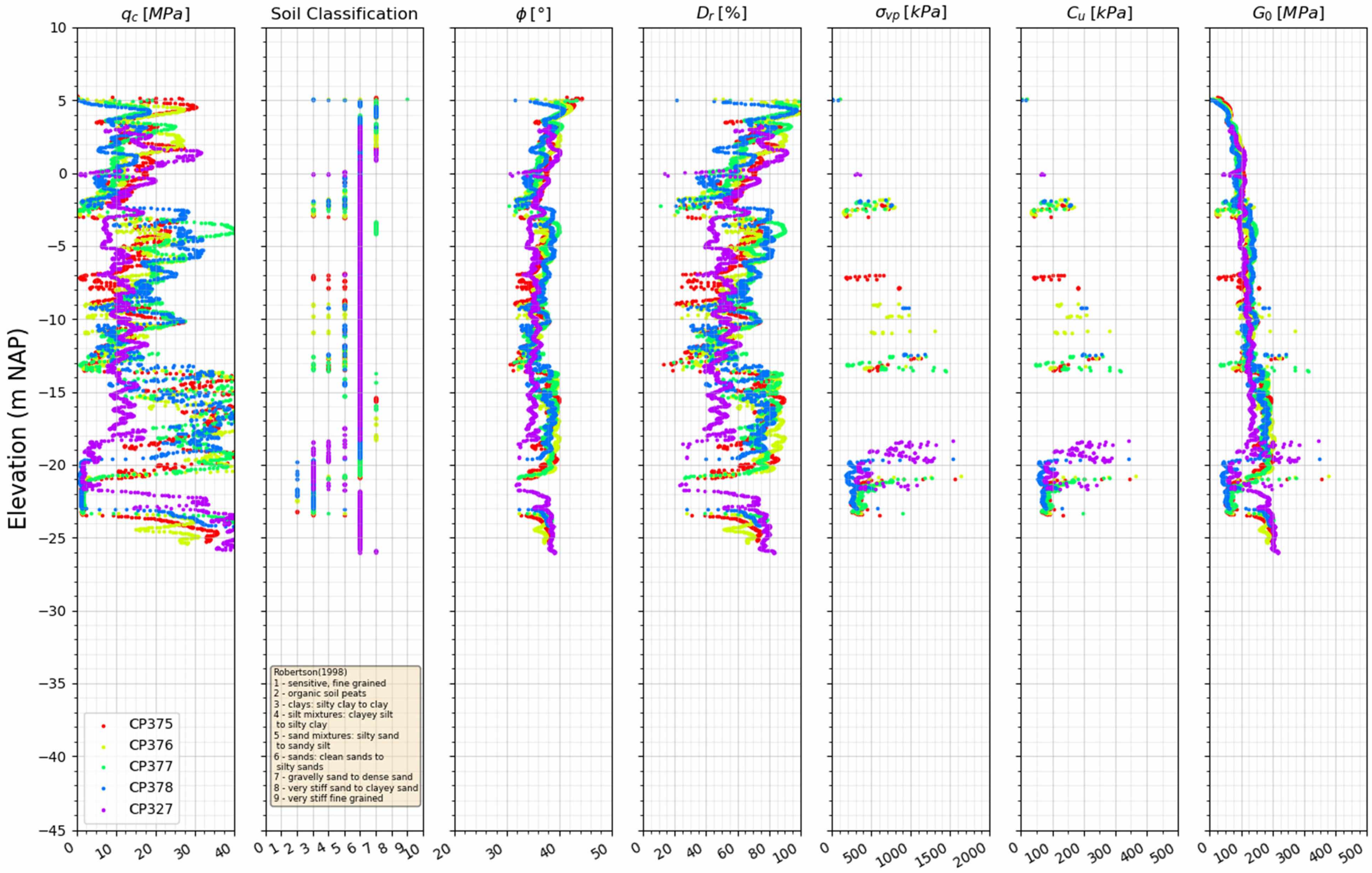
Attach.3 Figure 29- CPTs in Refinery RDCG Plant Area



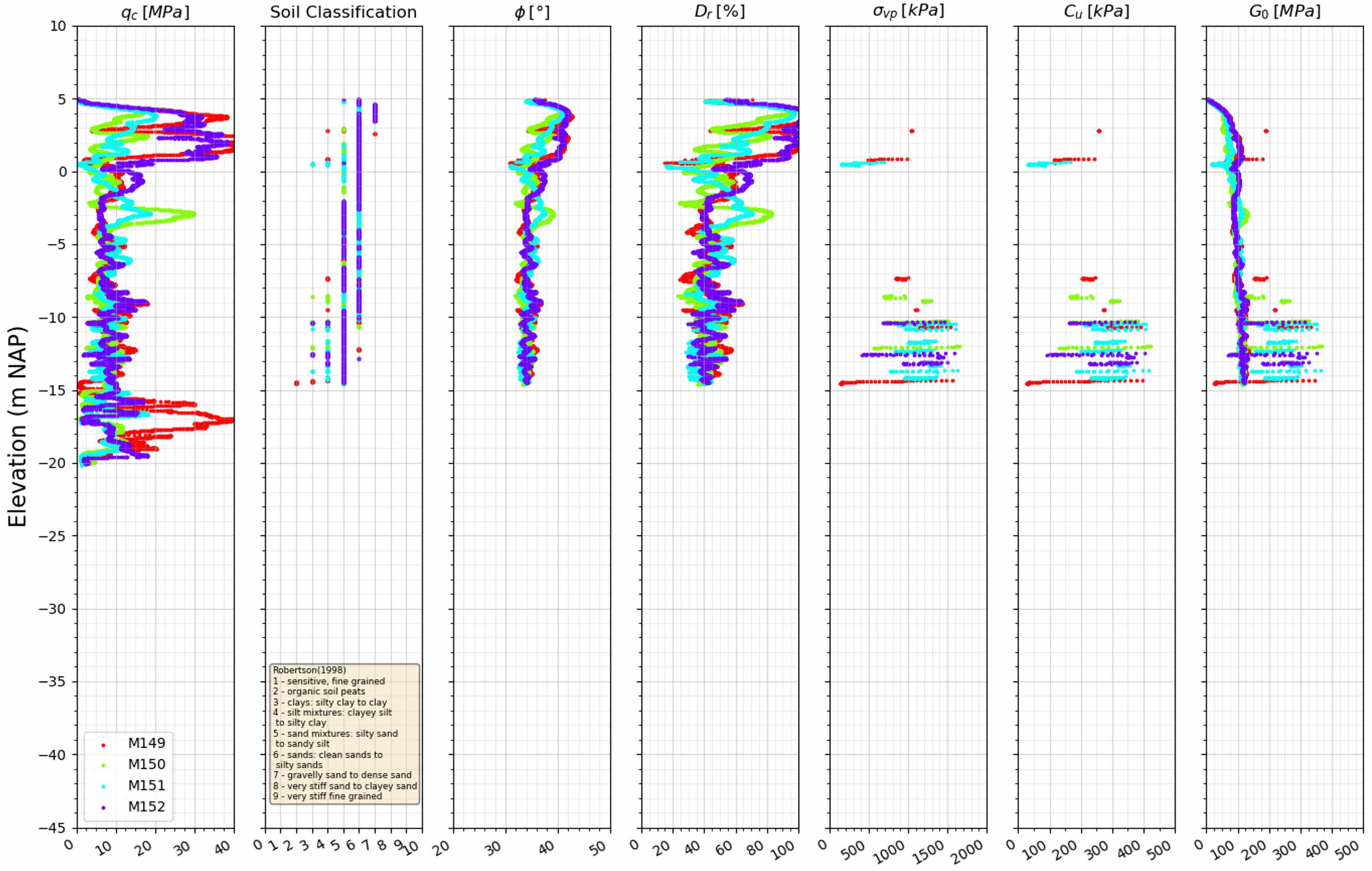
Attach.3 Figure 30- CPTs in Refinery RDCG Plant Area



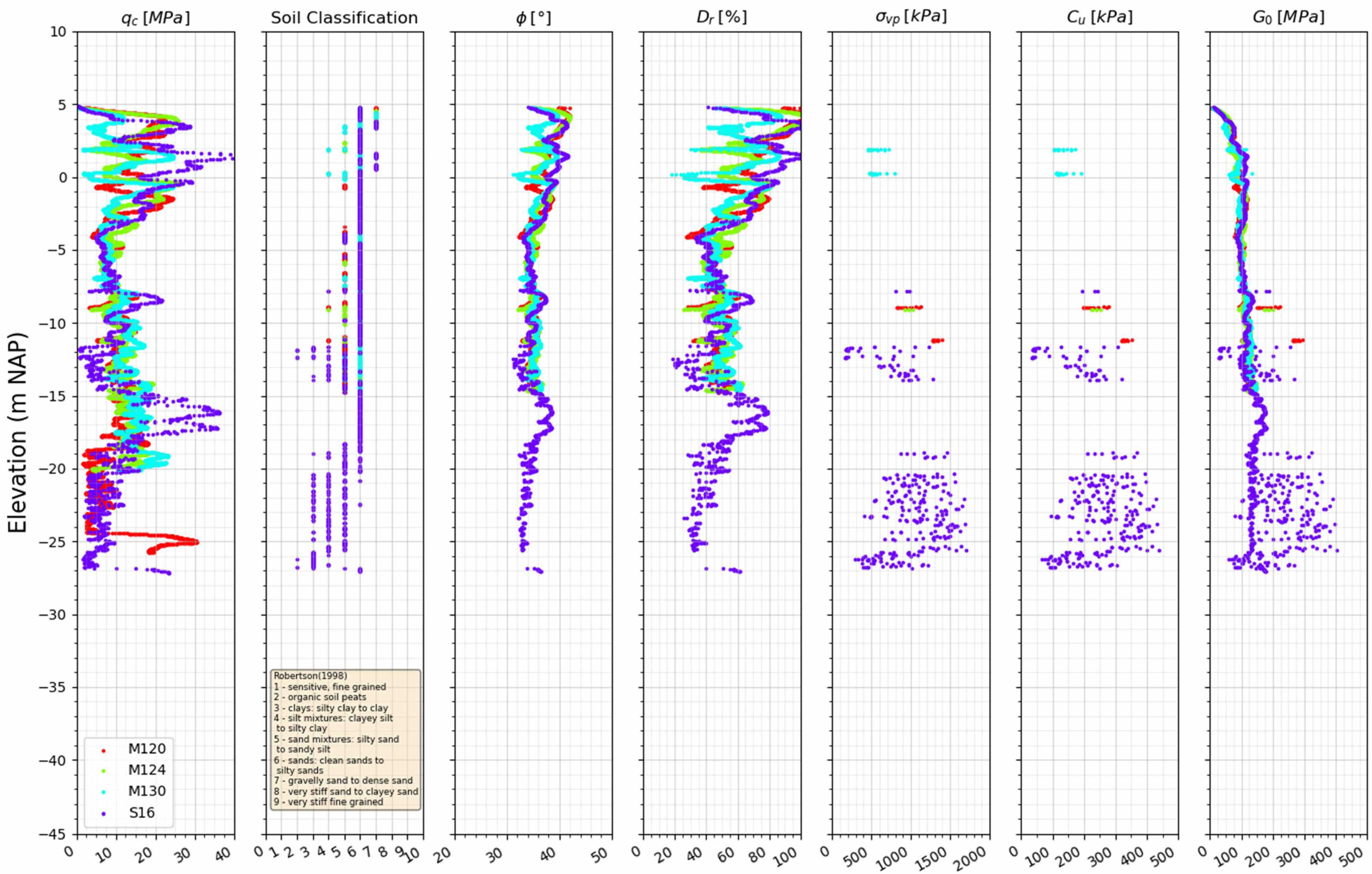
Attach.3 Figure 31- CPTs in Refinery RDCG Plant Area



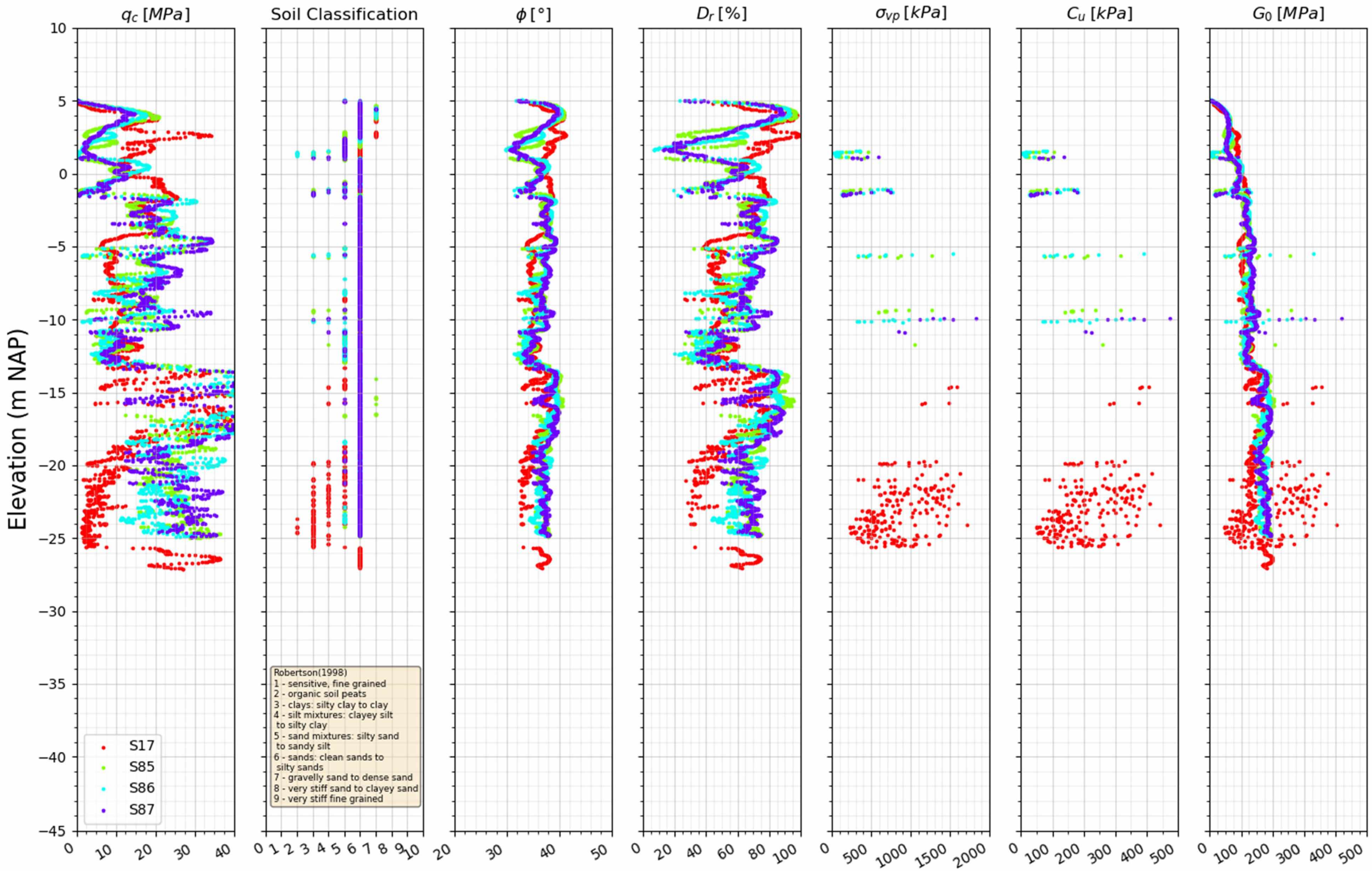
Attach.3 Figure 32- CPTs in Refinery RDCG Plant Area



Attach.3 Figure 33- CPTs in Refinery RDCG Plant Area

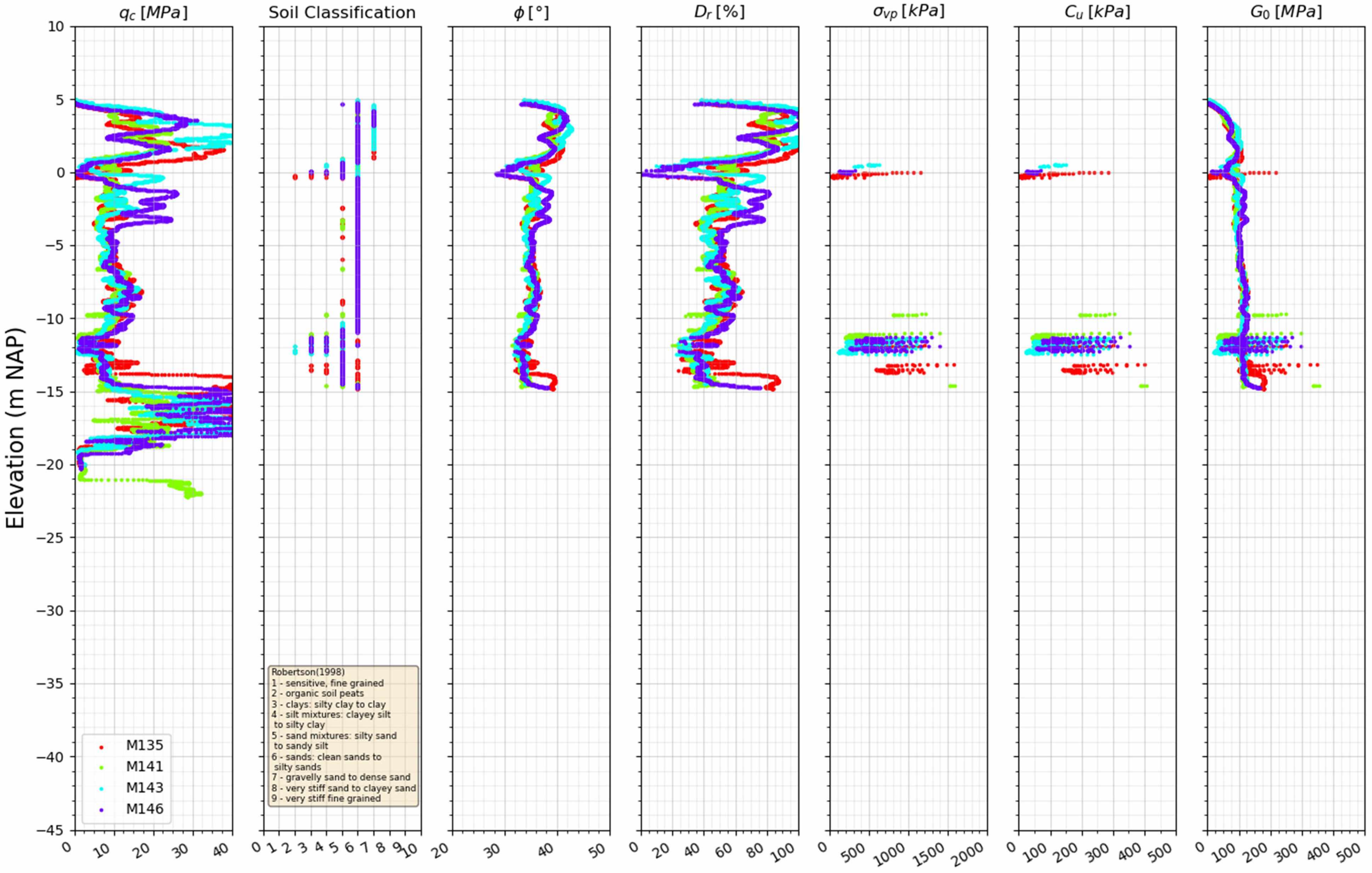


Attach.3 Figure 34- CPTs in Refinery RDCG Plant Area

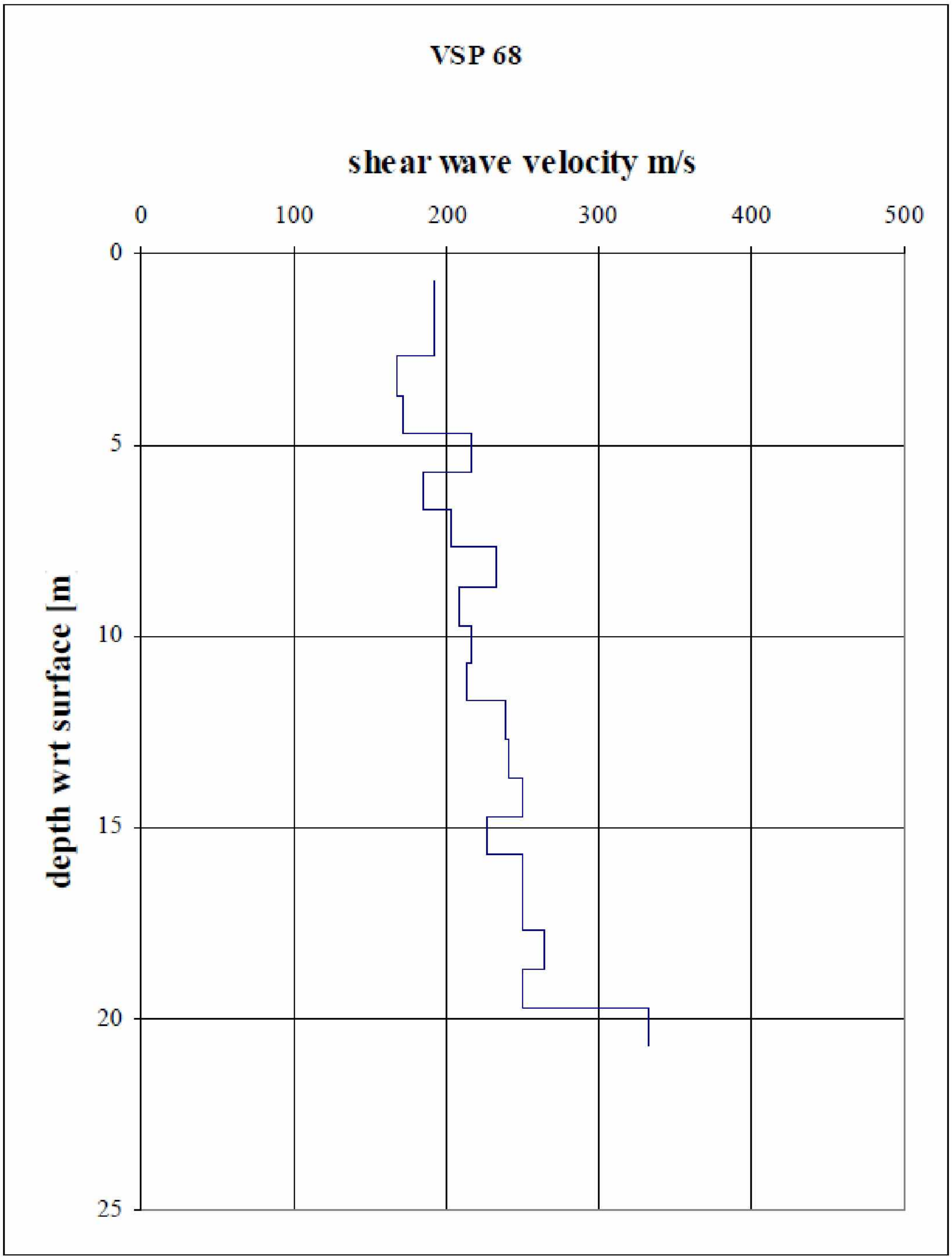


Attach.3 Figure 35- CPTs in Refinery RDCG Plant Area

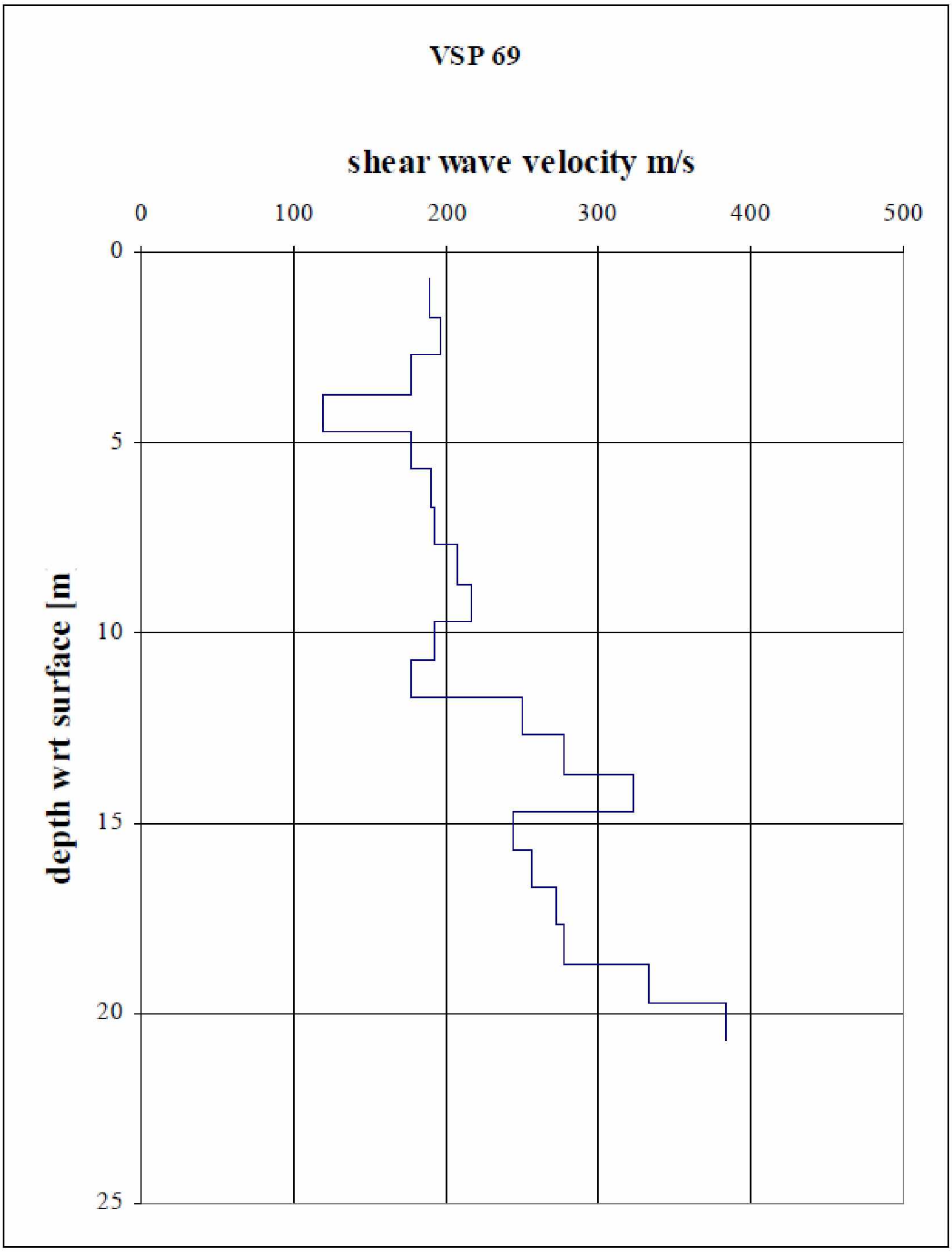
RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE
NESTE



Attach.3 Figure 36- CPTs in Refinery RDCG Plant Area



Attach.3 Figure 37- S-wave velocity profile location S68a (from Co-428841 v02, 2008)



Attach.3 Figure 38- S-wave velocity profile location S69a (from Co-428841 v02, 2008)