**RDCG (Rotterdam Capacity Growth) – FEASIBILITY PHASE**
**NESTE**

# JOB DESIGN SPECIFICATION

# FOR PILES

# RDCG REFINERY MNA AREA

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## INDEX

# 1. SCOPE

The present report deals with *Pile Works*, which might be required for heavy structures/equipments, structures/equipments subjected to uplift and/or overturning and structures/equipments sensitive to differential settlements in RDCG Area of the proposed Expansion Project of the Neste Plant located in Maasvlakte industrial site, in the harbour area of Rotterdam - Netherlands (refer to Figure *1.1*)

Here below the different area whose development TechnipFMC is responsible for:

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

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


Figure 1.1 Maasvlakte industrial site (from Google earth).

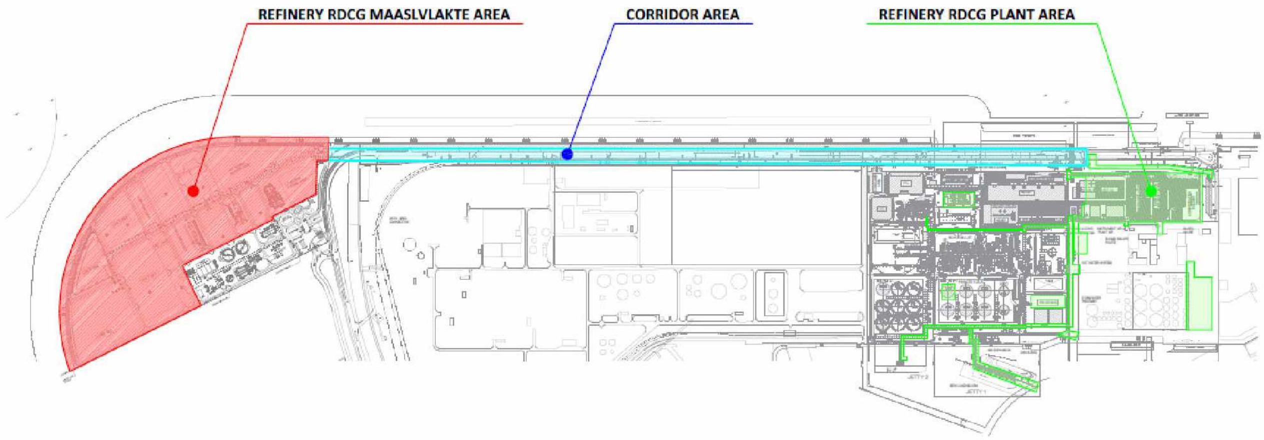


Figure 1.2: Locations of Areas scope of Technip work.

Pile design discussed in this document is related to RDCG REFINERY MNA AREA and is based on the results of the soil investigation campaign and ground characterization, presented in the following documents:

| | |
|---|---|
| 082755C-000-JSD-1410-0001 | JSD FOR GEOTECHNICAL RECOMMENDATIONS - FEASIBILITY PHASE – RDCG REFINERY MNA AREA |

# 2. REFERENCE DOCUMENTS

## 2.1. Project documents

[1] 082755C-000-JSD-1410-0001 JSD For Geotechnical Recommendations - Feasibility Phase – RDCG Refinery MNA Area
[2] 082755C-000-CN1410-01 - Calculation Report for Platform Settlements Evaluation for Recent Reclaimed Area (Maasvlakte2)

## 2.2. Reference documents

[3] 080871C 000 RT 1410 001 Soil Investigation factual report
[4] R1902305-02 Uitgevoerd grondonderzoek en laboratoriumonderzoek - I.001330 Neste Biofuels te ROTTERDAM MAASVLAKTE

## 2.3. Reference standards

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

## 2.4. Software

[8] LPILE (2015), Computer program for analysis of piles and drilled shafts under lateral loads. ENSOFT, INC.

# 3. BASIS OF GEOTECHNICAL DESIGN FOR PILED FOUNDATIONS

## 3.1. Single pile subjected to vertical loads - ULS GEO

With reference to deep foundations this document provides the geotechnical design resistances of single pile ($R_d$) defined according to the results of CPT tests in accordance to Ref.Doc.[7].

According to §7.6.2 (compressive ground resistance) and §7.6.3 (ground tensile resistance) of EN 1997-1, to demonstrate that the pile foundation will support the design load with adequate safety against compressive and tensile failures, the following inequalities shall be satisfied for all ultimate limit state load cases and load combinations.

- In compression $P_d \leq F_{r,net,d}$
- In tension $T_d \leq F_{r,max,d,ten}$

where:

$P_d$ is the design axial compression load on a pile (partial factors on actions according to Table *3.1*)

$T_d$ is the design axial tensile load on a pile (partial factors on actions according to Table *3.1*)

$F_{r,net,d}$ is the design value of the compressive resistance of a pile (considering possible occurrence of negative adhesion along pile shaft, where pertinent)

$F_{r,max,d,ten}$ is the design value of the tensile resistance of a pile

Values of sets A1 are given in Table *3.1*

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.

Table 3.1 Partial factors on actions or the effects of actions (table A.3 – Ref.Doc.[7])

| Action | Symbol | Set A1 | Set A2 |
|---|---|---|---|
| Permanent (Unfavourable) | $\gamma_G$ | 1.35(*) | 1.00 |
| Permanent (Favourable) | | 0.90 | 1.00 |
| Variable (Unfavourable) | $\gamma_Q$ | 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 | | | |

### 3.1.1. Design axial resistance ($F_d$)

The design value of the maximum bearing capacity in compression ($F_{r;max;d}$) is computed as follows:

$$F_{r;max;d} = F_{r;max;rep}/\gamma_{m;b}$$

where

$\gamma_{m;b}$ = partial material factor (Ref.Doc.[7], 7.6.3.2 (2)P, table A.8)

In this case: $\gamma_{m;b} = \gamma_{r;t} = 1.20$ calculation of piles from CPTs.

The design value of the maximum bearing capacity in tension ($F_{r,max,d,ten}$) is computed as follows (Ref.Doc. [7], 7.6.3.3 (8) a):

$$F_{r,max,d,ten} = \int_0^L O_{s;gem} \cdot q_{s;z;d} \cdot dz$$

where:

$O_{s;gem}$ = perimeter of the pile;

$q_{s;z;d}$ = design unit skin friction;

$q_{s;z;d}$ is calculated as $q_{s;z;d} = \alpha_t \cdot q_{c;z;d}$, with $q_{c;z;d}$ calculated as follows (Ref.Doc.[7], 7.6.3.3 (8) d):

$$\frac{q_{c;z;a}}{\gamma_{s;t} \cdot \gamma_{m;var;q_c} \cdot \xi}$$

where:

$\alpha_t$ = factor for influence of the execution and pile type according to Ref.Doc.[7], 7.6.2.3 (i), Tables 7c and 7d.

In this case: for sands $\alpha_t = 0.009$ for full displacement auger piles

$\alpha_t = 0.0045$ for bored piles

$\alpha_t = 0.01$ for driven cast-in situ piles;

for clays $0.02 \le \alpha_t \le 0.03$.

$q_{c;z;a}$ = measured cone resistance at depth z, determined according to Ref.Doc.[7], 7.6.2.3 (i).

$\gamma_{s;t}$ = partial material factor (Ref.Doc.[7], 7.6.3.2 (2)P, table A.8)

In this case: $\gamma_{s;t} = 1.35$ calculation of piles from CPTs.

$\gamma_{m;var;qc}$ = partial factor taken as the maximum value equal to 1.5 (Ref.Doc.[7], 7.6.3.3 (8) d).

$\xi$ = factor depending on the number of CPT's, according to Ref.Doc.[7] 7.6.2.3 (10) and 8.5.2 (3), Table 9.a. For the purposes of the preliminary calculation of pile bearing capacity carried out for this report, $\xi = 1.39$.

In the bearing capacity calculations, allowance is made for the occurrence of negative adhesion along pile shaft.

The design value of negative skin friction load ($F_{s;nsf;d}$) is calculated with the partial factor ($\gamma_{f;nk}$) as mentioned in Ref.Doc.[7], 7.3.2.2 (7):

$$F_{s;nsf;d} = F_{s;nsf;rep} \cdot \gamma_{f;nk}$$

$F_{s;nsf;rep}$ is the characteristic value of negative skin friction

$\gamma_{f;nk}$ is the partial material factor (Ref.Doc.[7], 7.3.2.2 (7)).

In this case: $\gamma_{f;nk}$ = 1.00 in accordance with point 7.3.2.2(d);

$\gamma_{f;nk}$ = 1.20 in accordance with point 7.3.2.2(e).

The design value of the net bearing capacity in compression ($F_{r;net;d}$) is computed as follows:

$$F_{r;net;d} = F_{r;max;d} - F_{s;nsf;d}$$

### 3.1.2. Characteristic pile axial resistance ($F_{rep}$)

The characteristic value of the maximum bearing capacity in compression ($F_{r,max;rep;i}$) can be determined with the following formula (Ref.Doc.[7], 7.6.2.3 (10)):

$$F_{r;max;rep;i} = \frac{F_{r;max,i}}{\xi}$$

with:

$\xi$ = factor depending on the number of CPT's, according to Ref.Doc.[7], 7.6.2.3 (10) and 8.5.2 (3), Table 9.a. For the purposes of the preliminary calculation of pile bearing capacity carried out for this report, $\xi$ = 1.39.

The characteristic value of negative skin friction load ($F_{s;nsf;rep}$) is determined according to Ref.Doc.[7], 7.3.2.2 (7d) for free standing piles:

$$F_{s;nsf;rep} = O_{s,ave} \cdot \sum_{i=1,n} d_i \cdot k_{0;i;rep} \cdot \tan\delta_{i;rep} \cdot \left(\frac{\sigma'_{v;i-1;rep} + \sigma'_{v;i;rep}}{2}\right)$$

where:

$d_i$ = thickness of the soil layer i;

$k_{o;i;rep}$ = characteristic value for the neutral earth pressure in layer i assumed equal to (1-sen $\phi'_{i;rep}$);

$\phi'_{i;rep}$ = characteristic value for the friction angle of soil in layer i, assumed equal to 35° in sand and 25° in clay;

$\delta_{i;rep}$ = characteristic value for the friction angle between the pile shaft and the soil in layer i, assumed equal to $\phi'_{i;rep}$ for concrete cast in situ piles; the value of ($k_{o;i}$ x $\tan\delta_{i;rep}$) has to be at least 0.25;

$\sigma'_{v;i;rep}$ = characteristic value for effective vertical stress at bottom of layer i;

$O_{s;ave}$ = perimeter of the pile.

When the piles are in a pile group in a foundation of a structure or part of it more or less evenly distributed along the horizontal plane, at mutual distances S ≤ 10 × D × d, the characteristic value of the total load due to negative skin friction on the shaft of a pile are determined according to 7.3.2.2 (7e):

$$\mathrm{F}_{\mathrm{s;nsf;rep;group}} = A \times \sum_{i=1}^{j=n} \left(\sigma'_{v;i;sur;rep} - \sigma'_{v;i;m;rep}\right)$$

where:

$$\sigma'_{v;i;sur;rep} = \sigma'_{v;i-1;m;rep} + d_i \gamma'_{i;rep}$$

and:

$\sigma'_{v;i-1;m;rep} = p_{sur;rep}$ for layer 1 and

$\sigma'_{v;i;m;rep} = \frac{\gamma'_{i;rep}}{m_i} \cdot \left(1 - exp(-m_i \cdot d_i)\right) + \sigma'_{v;i-1;m;rep} \cdot exp(-m_i \cdot d_i)$ for deeper layers

with:

$m_i = \frac{\mathrm{O}_{\mathrm{s;ave}}}{A} \cdot \mathrm{k}_{0;\mathrm{i;rep}} \cdot \tan \delta_{\mathrm{i;rep}}$ and $\mathrm{k}_{0;\mathrm{i;rep}} = 1 - \sin \varphi'_{i;rep}$

In which:

$A$ is the surface covered by a pile, in m$^2$;

$\sigma'_{v;i;sur;rep}$= representative value of the effective vertical stress in the bottom of layer i, as a result of the top load $p_{sur;rep}$ without the influence of other piles, in kPa;

$\sigma'_{v;i;m;rep}$= representative value of the decrease in effective vertical stress in layer *i*, as a result of the negative cohesion redistributed to the piles of a group of piles, in kPa;

$\gamma'_{i;rep}$ = representative value of the effective volume weight of the ground of layer *i:*

$d_i$ = thickness of layer *i*, in m;

$n$ = number of ground layers where negative cohesion is in effect;

$p_{sur;rep}$ = representative value of the load due to elevated ground or due to the storage of goods on the surface, in kN/m$^2$;

$\mathrm{k}_{0;\mathrm{i;rep}}$ = representative value of the neutral ground pressure factor in layer *j* with a horizontal ground surface and OCR = 1;

$\mathrm{O}_{\mathrm{s;ave}}$ = average circumference of the pile shaft, in m;

$\varphi'_{i;rep}$ = characteristic value for the friction angle of soil in layer i, assumed equal to 35° in sand and 25° in clay;

$\delta_{\mathrm{i;rep}}$ = characteristic value for the friction angle between the pile shaft and the soil in layer i, assumed equal to $\phi'_{i;rep}$ for concrete cast in situ piles; the value of ($k_{o;i}$ x tan$\delta_{i;rep}$) has to be at least 0.25;

### 3.1.3. Calculated pile axial resistance ($F_{r;max}$) from CPT tests

The maximum bearing capacity of a pile in compression ($F_{r;max;i}$) can be evaluated on the basis of the results of CPT tests according to Ref.Doc.[7], 7.6.2.3 (10):

$$F_{r;max;i} = F_{r;max,;base;i} + F_{r;max;shaft;\,i}$$

where:

$F_{r;max;base;i}$ = maximum end bearing capacity;

$F_{r;max;shaft;i}$ = maximum positive shaft resistance.

The maximum end bearing capacity of the pile tip ($F_{r;max;base}$) is determined according to Ref.Doc.[7], 7.6.2.3 (10) (see also Figure *3.1*):

$$F_{r,max,base;\,i} = A_b \cdot q_{b;max;\,i} = A_{base} \cdot \frac{1}{2} \cdot \alpha_p \cdot \beta \cdot s \cdot (\frac{q_{c;I;gem} + q_{c;II;gem}}{2} + q_{c;III;gem})$$

where:

$A_b$ = pile tip area;

$q_{b;max;i}$ = maximum point resistance;

$q_{c;I;gem}$ = average value of cone resistance obtained as the minimum value that results from a sum over depths ranging from 0.7·D to 4·D (D = pile diameter) below pile tip;

$q_{c;II;gem}$ = minimum value of cone resistance within thickness, below the pile tip, that minimizes the $q_{c;I;gem}$ value;

$q_{c;III;gem}$ = average value of cone resistance over a trajectory of 8·D above pile tip, calculated neglecting all values greater than $q_{c;II;gem}$;

$\alpha_p$ = pile class factor according to Ref.Doc.[7], 7.6.2.3 (f).

In this case: $\alpha_p = 0.63$ full displacement auger piles;

$\alpha_p = 0.35$ bored piles;

$\alpha_p = 0.7$ driven cast-in situ piles.

$\beta$ = pile form factor according to Ref.Doc.[7], 7.6.2.3 (g).

In this case: $\beta = 1.0$ for straight shaft piles (ignoring oversize tip);

s = form factor of a pile tip section according to Ref.Doc.[7], 7.6.2.3 (10).

In this case: s = 1.0 for circular cross sections.

The maximum positive shaft resistance ($F_{r;max;shaft}$) is determined with:

$$F_{r;max;shaft;\,i} = O_{s;\Delta L;gem} \cdot \int_0^{\Delta L} q_{s;max;\,z;i} \cdot dz = O_s \cdot \int_0^{\Delta L} \alpha_s \cdot q_{c;z;a} \cdot dz$$

where:

$O_{s;\Delta L;gem}$ = perimeter of the pile;

$\Delta L$ = length of pile shaft, where positive skin friction occurs;

$F_{s;max;z;i}$ = maximum positive skin friction;

$\alpha_s$ = factor for influence of the execution and pile type according to Ref.Doc.[7], 7.6.2.3 (i), Tables 7c and 7d.

In this case: for sands $\alpha_s$ = 0.009 for full displacement auger piles

$\alpha_s$ = 0.006 for bored piles

$\alpha_s$ = 0.012 for driven cast-in situ piles;

for clays $0.02 \le \alpha_s \le 0.03$;

$q_{c;z;a}$ = measured cone resistance at depth z, determined according to Ref.Doc.[7], 7.6.2.3(i).

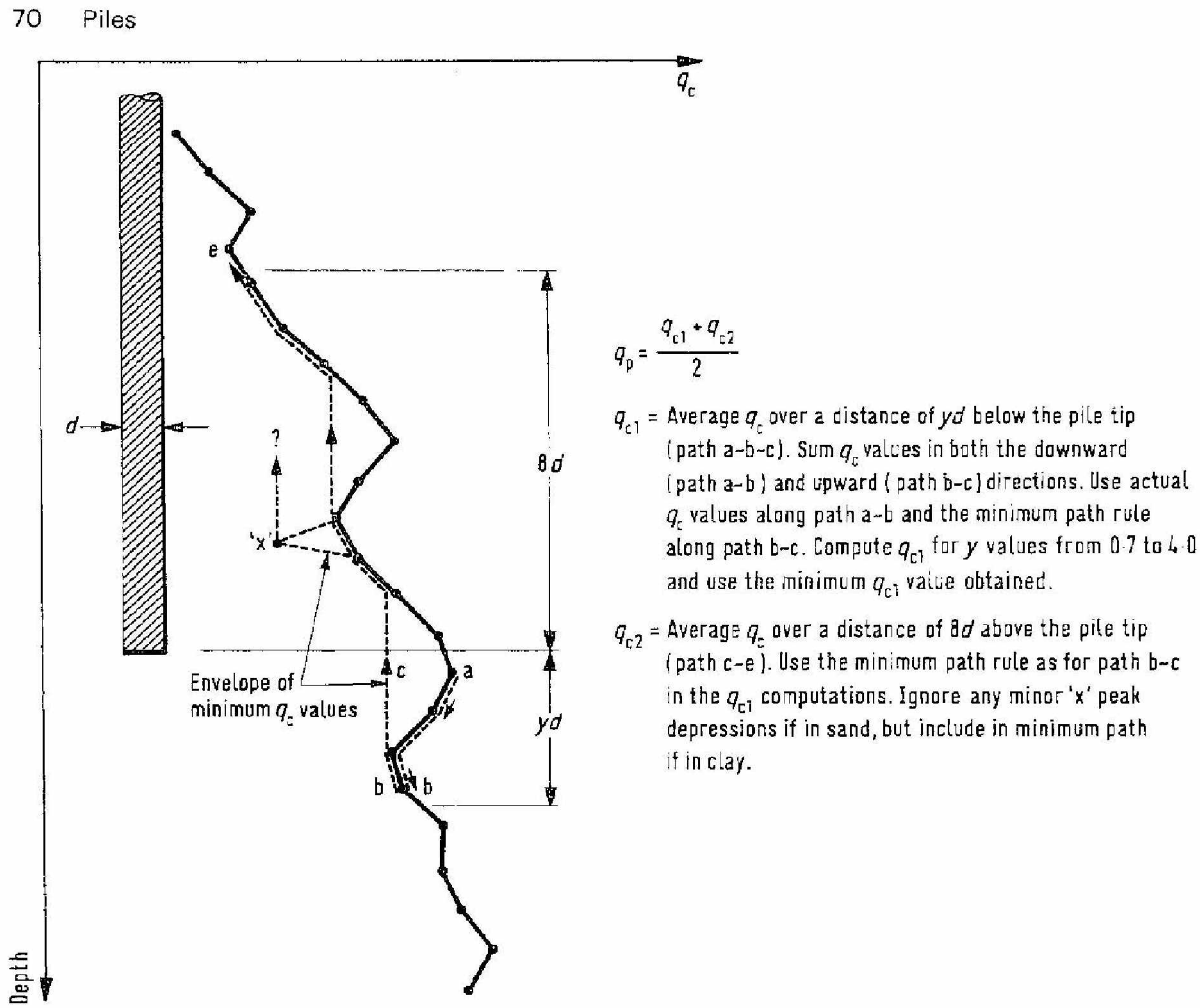


Figure 3.1: Procedure used to determine composite cone penetration test value in evaluation of pile end-bearing capacity

## 3.2. Single pile subjected to horizontal loads - SLS GEO and ULS STRU

The evaluation of the design geotechnical resistance of a single pile subject to transverse loads is omitted. The design problems of piles subjected to horizontal loads refer rather to displacements and structural resistance.
Due to the flexibility of the foundation piles under consideration, the achievement of the ultimate strength of the soil for horizontal loads corresponds to displacements of the piles that are not compatible with the serviceability of the proposed structures and the structural limits of the piles themselves.

The study of pile-soil interaction under horizontal loads is carried out in accordance with the approach proposed by Reese and Van Impe (2001) based on the well-known model of p-y curves which takes into account non-linear soil behaviour as described in detail in the technical manual of the computer code LPile (Ref.Doc.[8]).

## 3.3. Piles group foundations

It is recommended to design pile groups with a minimum centre to centre spacing between adjacent piles equal to 3 times the pile diameter, at least.

Under this condition, the distribution of vertical loads among the piles can be determined neglecting group effects.
However, in case of fixed connection between piles and pile caps, the axial loads on each pile shall be determined taking into account the additional moment transmitted by the piles to the piles cap as a result of horizontal loads. The actual load on each pile in the group determined as above must be checked according to the design resistance of the single pile.
Specific pile group analyses will need to be carried out in case smaller pile spacing is adopted.

# 4. DEEP FOUNDATIONS

## 4.1. General recommendations

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

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

Piles may be taken into consideration also for other items (buildings and utilities structures of Units 70, 76 and 53 and interconnecting rack) in case the expected total and differential settlements of shallow foundations (see Ref.Doc.[1]) 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.

Areas where piles are recommended or could be needed are shown in Figure 4.1, green zone.

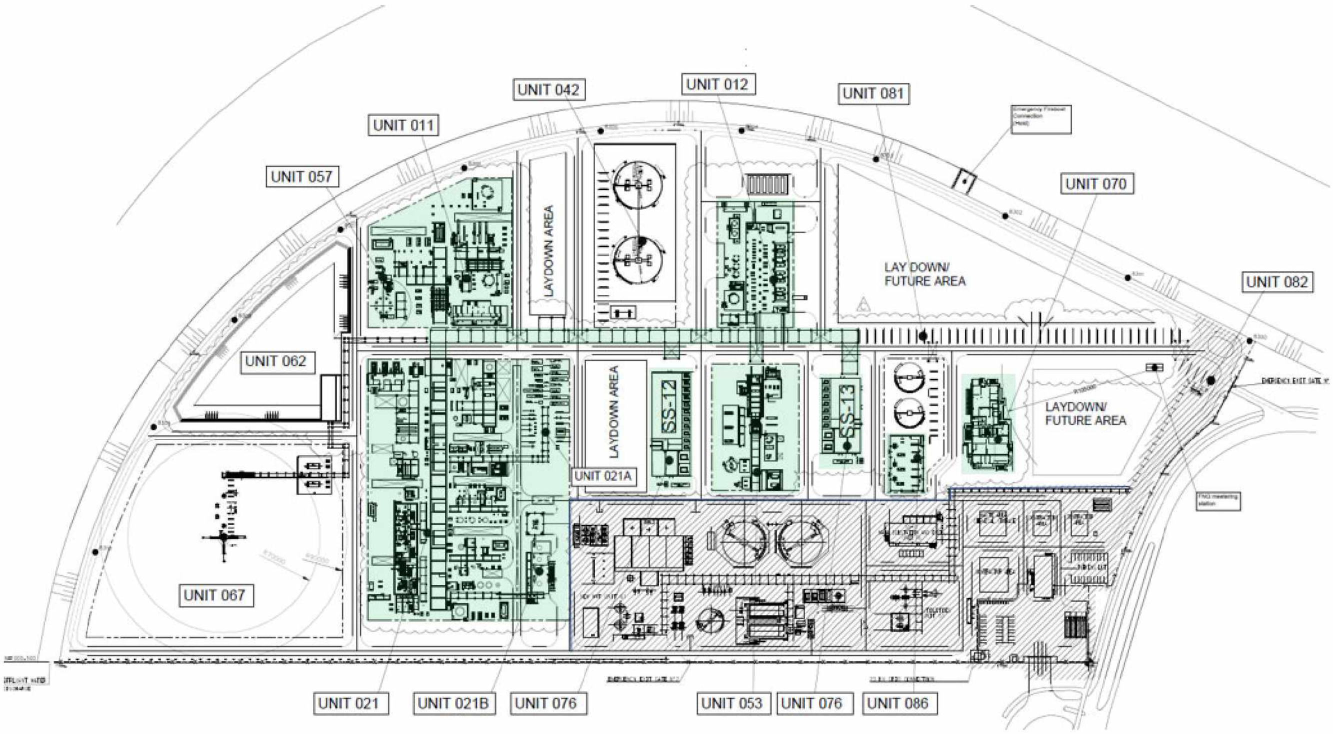


Figure 4.1

Piles shall be installed down to design penetration depth according to the required capacity.

Pile capacity under vertical loads are performed for each available CPT test within the relevant zone of each Plant Unit. Results of the performed evaluations are reported in Sections 4.3.3 and 4.3.4.

The actual load on each and every pile in a group must be checked according to the design resistances as defined in Sections 4.3.3 and 4.3.4.

Considering the soil profile type at the site and the previous experience on the existing Plant the following pile types have been selected as suitable for the new Expansion Project

- Driven cast in situ piles

Finally, it is anticipated that considering the soil profile type at the site two possible range of depths are suitable for the pile tip location:

a) Between about -12 m NAP and -18 m NAP in the dense sandy layer
b) Below about -23 m NAP in the Pleistocene sand

In the first case, the residual settlements of the fine grained layer just above the Pleistocene sands due to the reclamation and site preparation activities have to be added to the settlements of the piled foundation.
From the detailed calculation model described in Ref.Doc.[2], it appears that the residual settlements of the above mentioned clayey layer over the lifetime of the Plant due to the reclamation and site preparation activities will be less than one centimeter and rather uniform over the whole Plant area (negligible differential settlements are expected).
To them, possible additional settlements (in the order of some centimeters) can be caused by very heavy structures on very large group of piles for which high loads are transferred to depth. In this condition it is advisable to study the behaviour of the pile group to verify the compatibility of the expected settlements with those tolerable by the structure and to evaluate whether they can be discounted in advance (structures for which, for example, a hydraulic test is foreseen). Alternatively, longer piles, passing the deep clay level, may be considered.

Concerning piles with the tip in the deep Pleistocene sands, specific analyses will be performed in the next design stage for a preliminary evaluation of the driving system necessary to install piles passing through the intermediate dense sandy layer.

## 4.2. Selected pile types and diameters

The following pile type and relevant sizes have been considered:

- Driven cast in situ piles D = 356/400 mm
  D = 457/510 mm
  D = 610/660 mm

Other piles type may also be technically suitable and may be used in partial or total substitution of the piles considered in this Report, if required, subjected to revised calculations of pile bearing capacity to take into account the different coefficients applicable to different pile types.

## 4.3. Piles subjected to axial loads (ULS checks)

### 4.3.1. Calculation soil profiles and parameters

The design pile resistances in compression and in tension acting on the piles in the long term has been evaluated on the basis of CPT tests results from the ground investigation carried out in the area of interest.

### 4.3.2. Negative skin friction

A deep foundation surrounded by soil that settles is subjected to a drag load. Load transfer is by shearing stress that develops at the soil-shaft interface, negative skin friction (NSF), and increase the load applied to the pile, which may result in increase of pile settlement.

At RDCG REFINERY MNA AREA possible natural soil settlements could be due to:

A. the recent reclamation activities up to the present ground level (from 2010 to 2013), whose effects could be not completely annulled;

B. the planned filling activities necessary to raise the ground to the Final Ground Level elevation +5.80 NAP.

Concerning possible residual settlements over the lifetime of the Plant due to the recent reclamation activities and in absence of any other increase in site elevation (point A.), they are estimated to be generally between 1 cm and 1.5 cm (see detailed calculation model described in Ref.Doc.[2]). In Figure 4.2 residual settlements with depth at different representative locations within the Plant area are shown; in the same figure also the result at the most critical location, giving a maximum residual settlement of about 2.1 – 2.2 cm, is reported. <u>Above estimated ground displacements are such that the effects of negative friction can be assumed as negligible.</u>

Concerning the effects of site preparation activities (point B.) it has to be noted that:

- according to the current construction schedule for site preparation works, only areas where piles are not foreseen will be filled to the Final Ground Level elevation, whereas pits will be excavated where piles are planned (green zones in Figure 4.1), to allow piles construction at design cut-off level and the relevant foundation slabs on them;
- most of the areas where piles are planned are very congested both of foundation piles and piles as settlements reducers (soil improvement by rigid inclusions), causing the backfill and the site preparation load ending up wholly on piles, not generating significant settlements of the ground between the piles;
- residual settlements over the lifetime of the Plant (30 years) induced on the excavated areas (where piles are planned) by the fill load applied to the areas surrounding the pits are estimated to be in the order of 1.5 ÷ 2 cm (comparable to those due to reclamation only) in the most central area of the pits, away from their edges. At the edges of the excavated area residual settlements from the start of SP to 30 years are between those due to reclamation and those due to site preparation, about 4 ÷ 5 cm at ground level.

<u>Base on the above it is considered reasonable to conclude that in the areas where foundation piles are foreseen the effects in terms of settlements induced in the ground by the site preparation loads are generally limited and consequently the negative friction issue negligible.</u> Site preparation loads could eventually affect piles of structure placed very close to the excavation's edges.

At the following point 4.3.3 the design values of vertical load in compression ($P_d$) <u>in absence of</u> negative skin friction for each pile type in each piling area are summarised.

For completeness and verification of the impacts on the design pile resistances, calculations in presence of NSF have been performed for some representative CPT tests within the relevant zone of each Plant Unit (see point 4.3.4).

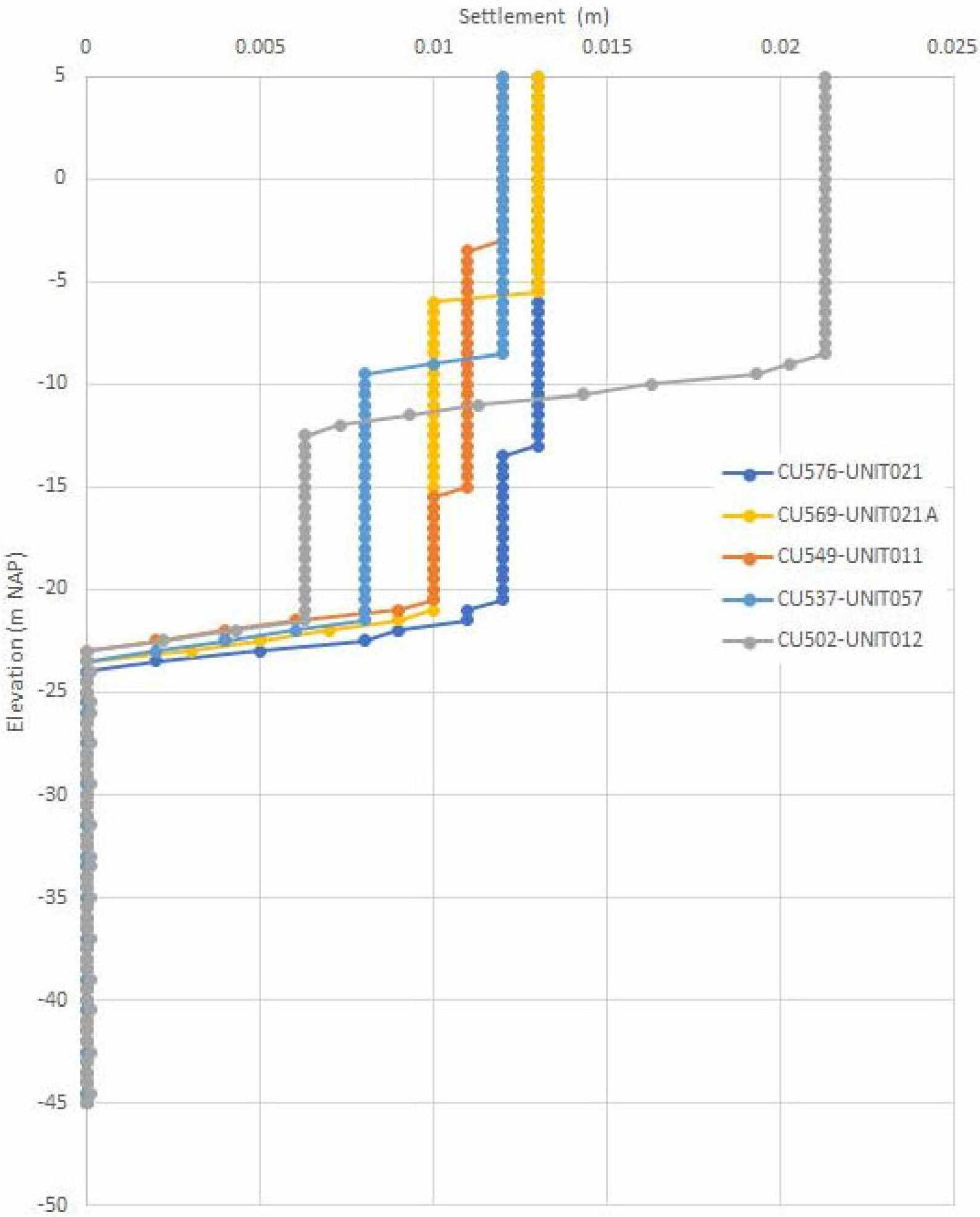


Figure 4.2 – Residual settlements due to reclamation activities at 30 years from the start of construction activities at RDCG REFINERY MNA AREA

### 4.3.3. Design value of vertical load in compression ($P_d$) and in tension ($T_d$)

The detailed calculation of the design value of net bearing resistance in compression ($F_{r;net;d}$) and in tension ($F_{r;max;d,ten}$) with depth for each pile type and each examined CPT are reported in the attached **Annex A**.

For a given design value of vertical load in compression ($P_d$) and in tension ($T_d$) for each pile type in each piling area, the piles lengths necessary to satisfy the design requirements have been evaluated. The results in terms of predicted pile tip penetration are shown in Table 4.1 to Table 4.3 for each pile type considered. I**t has to be pointed out that values in Table 4.1 to Table 4.3 are the minimum for each unit for the selected design pile length, during the following design phase more detailed zonation will be performed in order to optimized pile length and relevant design loads.**
Moreover, it has to be noted that locally the required design load could be satisfied also for a pile penetration appreciably lower than the one indicated in the mentioned figures; these local anomalous results have been ignored in order to obtain a relatively uniform distribution of pile lengths in each area.

**Design loads in compression $P_d$ and in tension $T_d$ shall be compared with the design actions at pile head, that are the characteristic actions factorized with the appropriate partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019.**

Structural analyses shall be carried out in order to evaluate the final design load of piles with regard to the foreseen reinforcement of concrete.

Table 4.1: Driven cast in situ pile D = 356/400 m (in absence of NSF)

| Unit | Pile tip elevation(1)(2) | $P_d$ | $T_d$ |
|---|---|---|---|
| - | m NAP | kN | kN |
| 21 | -16 | 1300 | 450 |
| 11 | -14.5 | 1330 | 600 |
| 57 | -14.5 | 1350 | 600 |
| 12 | -14 | 1300 | 450 |
| 81 | -14 | 1250 | 500 |
| 76 (SS-12) | -13.5 | 1200 | 450 |
| 76 (SS-13) | -12 | 1450 | 550 |
| 53 | -13.5 | 1150 | 450 |
| 70 | -12.5 | 1300 | 550 |
| 86 | -11.5 | 1250 | 600 |

(1) pile head cut off level at +3.3 m NAP
(2) see comment at the end of point 4.1 concerning pile length

Table 4.2: Driven cast in situ pile D = 457/510 m (in absence of NSF)

| Unit | Pile tip elevation(1)(2) | $P_d$ | $T_d$ |
|---|---|---|---|
| - | m NAP | kN | kN |
| 21 | -16 | 1700 | 550 |
| 11 | -14.5 | 1750 | 750 |
| 57 | -14.5 | 1800 | 750 |
| 12 | -14 | 1800 | 600 |
| 81 | -14 | 1700 | 650 |
| 76 (SS-12) | -13.5 | 1750 | 550 |
| 76 (SS-13) | -12 | 2000 | 700 |
| 53 | -13.5 | 1650 | 550 |
| 70 | -12.5 | 1750 | 700 |
| 86 | -11.5 | 1700 | 750 |
| (1) pile head cut off level at +3.3 m NAP<br>(2) see comment at the end of point 4.1 concerning pile length | | | |

Table 4.3: Driven cast in situ pile D = 610/660 mm (in absence of NSF)

| Unit | Pile tip elevation(1)(2) | $P_d$ | $T_d$ |
|---|---|---|---|
| - | m NAP | kN | kN |
| 21 | -16<br>-23 | 2400<br>3000 | 750<br>1300 |
| 11 | -14.5<br>-22 | 2400<br>3450 | 1000<br>1650 |
| 57 | -14.5<br>-22 | 2550<br>3500 | 1050<br>1650 |
| 12 | -14<br>-22.5 | 2950<br>3350 | 800<br>1550 |
| 81 | -14 | 2300 | 850 |
| 76 (SS-12) | -13.5 | 2650 | 750 |
| 76 (SS-13) | -12 | 2800 | 950 |
| 53 | -13.5 | 2500 | 750 |
| 70 | -12.5 | 2450 | 950 |
| 86 | -11.5 | 2400 | 1000 |
| (1) pile head cut off level at +3.3 m NAP<br>(2) see comment at the end of point 4.1 concerning pile length | | | |

### 4.3.4. NSF effects on the design value of vertical load in compression

A possible down-drag load has been determined considering the pile shaft resistance above the neutral plane. Based on a preliminary study of pile-soil interaction under vertical loads, carried out in accordance with the Reese and Wang (1990) approach based on the well-known model of t-z and q-s curves, and considering:

- a presumable range of design loads at pile head around 1000 -2000kN;
- ground settlements profiles taken on the very safe side equal to the residual settlements over the lifetime of the Plant induced by the site preparation works (neglecting the construction sequence and the congestion of the areas highlighted above), which are, at ground level, about 4 to 5 times greater than those of Figure 4.2;

the neutral plan position is estimated between 12 to 15 m below pile head ( -9 to -18 m NAP).

To the assessments of the design value of vertical load in compression in presence of negative skin friction, the consideration that the piles in project are within large groups (as previously mentioned, the piled areas are particularly congested) has been included. Evaluations have been done for a pile spacing equal to 3·D on a square grid.

The detailed calculation of the design value of net bearing resistance in compression in presence of negative skin friction for single ($F_{r;net;d}$) piles in group spaced 3·D ($F_{r;net;d;group}$) and in tension ($F_{r;max;d,ten}$) with depth for each pile type and the selected representative CPT are reported in the attached **Annex B**.

For a given design value of vertical load in compression in presence of negative skin friction for single piles ($P_d$) and piles in group spaced 3·D ($P_{d;group}$) and in tension ($T_d$) for each pile type in each piling area, the piles lengths necessary to satisfy the design requirements have been evaluated.
The results in terms of predicted pile tip penetration are shown in Table 4.4 and Table 4.6 for each pile type considered. **It has to be pointed out that values in Table 4.4 and Table 4.6 are the minimum for each unit for the selected design pile length, during the following design phase more detailed zonation will be performed in order to optimized pile length and relevant design loads.**
Moreover, it has to be notice that locally the required design load could be satisfied also for a pile penetration lower than the one indicated in the mentioned figures; these local anomalous results have been ignored in order to obtain a relatively uniform distribution of pile lengths in each area.

**Design loads in compression in presence of negative skin friction for single piles ($P_d$) and piles in group spaced 3·D ($P_{d;group}$) and in tension ($T_d$) shall be compared with the design actions at pile head, that are the characteristic actions factorized with the appropriate partial factors on action given in NEN EN 1997–1+C1+A1: 2016/NB:2019.**

Structural analyses shall be carried out in order to evaluate the final design load of piles with regard to the foreseen reinforcement of concrete. **It is pointed out that for ULS structural checks negative skin friction load (NSF = $F_{s;nsf;d}$ or $F_{s;nsf;d;group}$) have to be added to the external loads.**

Table 4.4: Driven cast in situ pile D = 356/400 m (in presence of NSF)

| Unit | Pile tip elevation[(1)(2)] | Single pile | | Pile in group (s = 3·D) | | |
|---|---|---|---|---|---|---|
| | | $P_d$ | $F_{s;nsf;d}$ [(3)] | $P_{d;group}$ | $F_{s;nsf;d;group}$[(3)] | $T_d$ |
| - | m NAP | kN | kN | kN | kN | kN |
| 21A – 21B | -18 | 370 | 750 | 870 | 235 | 650 |
| 12 | -16.5 | 570 | 840 | 1160 | 250 | 600 |
| 70 | -15 | 530 | 655 | 970 | 220 | 650 |
| 81 | -17 | 500 | 830 | 1080 | 250 | 650 |
| 86 | -17.5 | 280 | 810 | 760 | 245 | 850 |
| 53 | -14.5 | 260 | 700 | 700 | 225 | 500 |
| 76 (SS-12) | -16.5 | 200 | 750 | 710 | 235 | 600 |
| 76 (SS-13) | -16.5 | 310 | 665 | 730 | 220 | 800 |

(1) pile head cut off level at +3.3 m NAP
(2) see comment at the end of point 4.1 concerning pile length
(3) NSF force must be added to the external loads for ULS structural checks

Table 4.5: Driven cast in situ pile D = 457/510 m (in presence of NSF)

| Unit | Pile tip elevation[(1)(2)] | Single pile | | Pile in group (s = 3·D) | | |
|---|---|---|---|---|---|---|
| | | $P_d$ | $F_{s;nsf;d}$ [(3)] | $P_{d;group}$ | $F_{s;nsf;d;group}$[(3)] | $T_d$ |
| - | m NAP | kN | kN | kN | kN | kN |
| 21A – 21B | -17.5 | 520 | 960 | 1090 | 370 | 840 |
| 57 | -17 | 360 | 1215 | 1160 | 420 | 900 |
| 12 | -16.5 | 1120 | 1080 | 1800 | 390 | 750 |
| 70 | -15 | 900 | 840 | 1400 | 340 | 800 |
| 81 | -17 | 940 | 1065 | 1610 | 390 | 800 |
| 86 | -17 | 510 | 1035 | 1040 | 380 | 1100 |
| 53 | -14.5 | 610 | 890 | 1110 | 350 | 650 |
| 76 (SS-12) | -16.5 | 440 | 960 | 1000 | 370 | 750 |
| 76 (SS-13) | -16.5 | 520 | 850 | 1000 | 345 | 1000 |

(1) pile head cut off level at +3.3 m NAP
(2) see comment at the end of point 4.1 concerning pile length
(3) NSF force must be added to the external loads for ULS structural checks

Table 4.6: Driven cast in situ pile D = 610/660 mm (in presence of NSF)

| Unit | Pile tip elevation$^{(1)(2)}$ | Single pile | | Pile in group (s = 3D) | | |
|---|---|---|---|---|---|---|
| | | $P_d$ | $F_{s;nsf;d}$ $^{(3)}$ | $P_{d;group}$ | $F_{s;nsf;d;group}$$^{(3)}$ | $T_d$ |
| - | m NAP | kN | kN | kN | kN | kN |
| 21A – 21B | -18 | 750 | 1280 | 1500 | 630 | 1000 |
| 21 | -26 | 870 | 1525 | 1700 | 680 | 1600 |
| 11 | -16.5<br>-25 | 320<br>1310 | 1670<br>1670 | 1280<br>2190 | 710<br>680 | 1150<br>1800 |
| 57 | -16.5<br>-25 | 490<br>840 | 1620<br>1620 | 1420<br>1760 | 700<br>700 | 1150<br>1850 |
| 12 | -16.5<br>-23.5 | 840<br>2010 | 1440<br>1440 | 1530<br>2800 | 650<br>650 | 1000<br>1850 |
| 70 | -15 | 1250 | 1120 | 1820 | 560 | 1100 |
| 81 | -17 | 1280 | 1420 | 1940 | 650 | 1100 |
| 86 | -16.5 | 870 | 1380 | 1460 | 635 | 1450 |
| 53 | -15 | 990 | 1190 | 1570 | 580 | 900 |
| 76 (SS-12) | -16.5 | 650 | 1280 | 1300 | 610 | 1050 |
| 76 (SS-13) | -16.5 | 800 | 1140 | 1340 | 565 | 1350 |

(1) pile head cut off level at +3.3 m NAP
(2) see comment at the end of point 4.1 concerning pile length
(3) NSF force must be added to the external loads for ULS structural checks

## 4.4. Piles subjected to horizontal loads (SLS and ULS structural checks)

The results of pile-soil interaction analyses under horizontal load conditions are reported below for use in ULS structural checks and SLS checks.

### 4.4.1. Pile behaviour under horizontal loads

The study of pile-soil interaction under horizontal loads is carried out in accordance to point 3.2.

Calculation have been carried out assuming a representative soil profile for all the zones included in the scope of Expansion Project within RDCG REFINERY MNA AREA, considering each pile diameter and for a reference minimum pile length equal to 18 m (pile tip -15 m NAP). They can be considered still suitable in case of longer piles.

Both pinned and fixed end pile head conditions have been analysed.
Horizontal loads at pile head from 50 kPa to 250 kPa have been considered.

Results of calculations are shown in **Annex C**, in terms of pile deflection, bending moment and shear force curves vs pile length. In addition, pile deflection vs shear force for pinned condition and bending moment vs shear force curve for fixed end conditions are presented.

# 5. PRESENCE OF BOULDERS/STONES OF HISTORIC SEA BARRIER/BREAKWATER

An historic sea barrier/breakwater was identified to be present in the past within the area object of the present document. According to the competent authority (Port of Rotterdam) the barrier was disassembled and removed during the reclamation of the Maasvlakte2. However, from the CPTU results a number of early refusals were identified in the approximate location of this sea barrier. This might indicate that some residual boulders/stones were not completely removed during the disassembly. Figure 5.1 shows the location of CPTUs with early refusal. Areas with different risk of boulders/stones presence are identified:

- Zone a → high risk
- Zone b → medium risk
- Zone c → low risk

The issue of boulders/stones possible presence shall be taken into account for pile installation in case of tips pile levels deeper than -15 m NAP are selected.
It has to be noted that the new structures and equipments in the Planned Expansion area fall within the zone c with low risk for early refusal due to boulders/stones of historic sea barrier/breakwater.

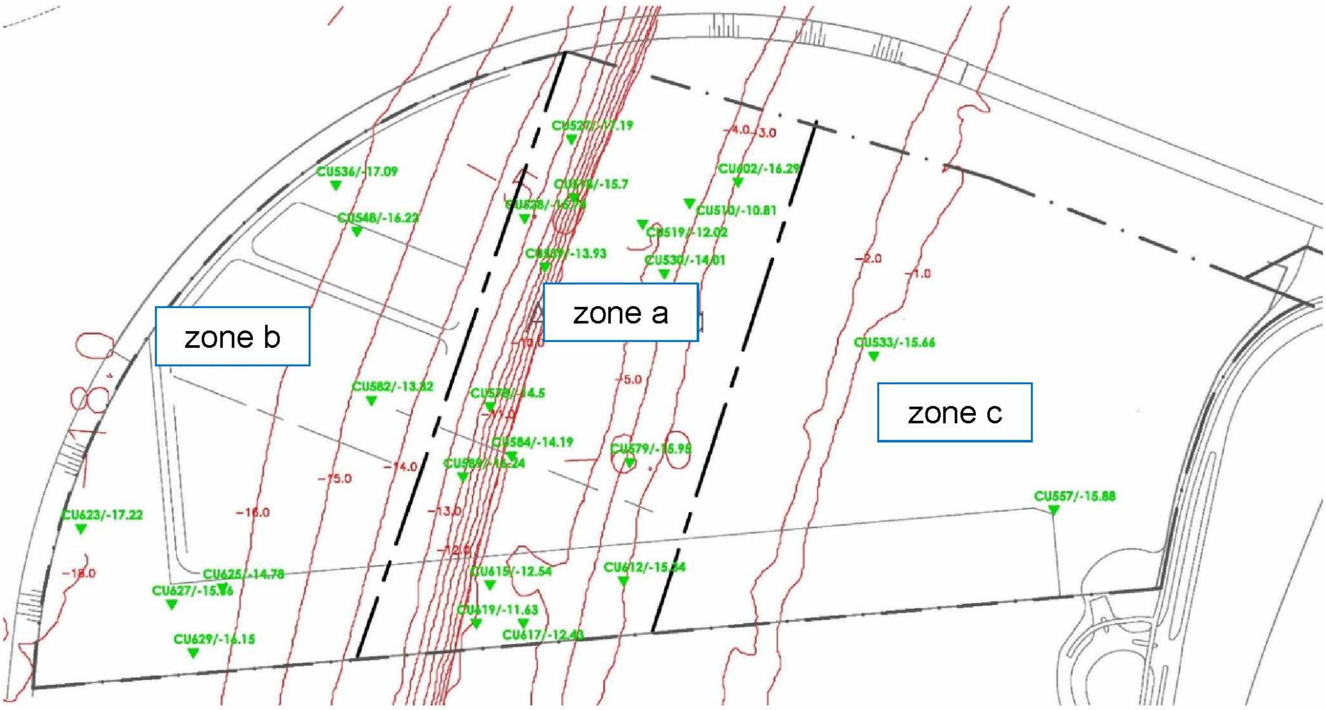


Figure 5.1: CPTU with early refusal