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Geotechnical Aspects of Underground Construction in Soft Ground, Kastner, Emeriault, Dias, Guilloux (eds)
© 2002 Spécifique, Lyon. *ISBN 2-9510416-3-2*

Prediction of long-term behaviour of a bored tunnel in soft soil

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ABSTRACT: For the realisation of RandstadRail a tunnel of 3-km length will be built in the city area of Rotterdam. 0.6-km will be realised by using cut and cover techniques and 2.4 km by shield tunnelling. Halfway the alignment station Statenweg is planned about 17 m below surface level at the transition of soft organic clay layers to the stiff Pleistocene sand. The design is based on two bored single-track concrete tunnel tubes with an outer diameter of 6.5 m each. Due to the shallow depth of the alignment near the existing metro-tunnel, built around 1960 by cut and cover techniques, and for optimising the building costs of the start and receiving shafts, the bored tunnel will be placed partly in the very soft soil layers adjacent to the shafts. In all the mentioned situations the tunnel is founded with an embedment of 180^0 in the Pleistocene sand as a minimum. During the lifetime of the tunnel construction it is expected that the soft layers will settle 1.0 m due to consolidation and creep. Because of this settlement the external forces on the tunnel lining will get larger than the vertical overburden pressure. The time dependent extra force, called the negative skin friction, has been analysed analytically as well as numerically. Physical modelling using the Delft geocentrifuge is in progress.

1 INTRODUCTION

RandstadRail is a future light-rail link between the cities of Rotterdam, The Hague and Zoetermeer in the Netherlands. It connects the local public transportation systems in these cities in such a way that travelling between the inner cities is possible without change. In Rotterdam RandstadRail will be connected to the North-South line of the local metro system. This North-South line now ends at Rotterdam Central station, which is the main train station of Rotterdam. The adjacent Rotterdam metro-line was built in the sixties by submerged tunnelling. Due to the inconveniences of submerged tunnelling and cut and cover techniques the tunnel boring techniques are fashionable nowadays. Therefore the underground parts of RandstadRail in Rotterdam will be built mainly by shield tunnelling.

1.1 Shield tunneling

Shield tunnelling is a new way of tunnel construction in the city centre of Rotterdam. The most suitable configuration of the tunnel was studied. This led to the recommendation for a two single-track tunnel of 6.5 meters radius (Fig. 1) instead of one double track tunnel of 11.5 meters radius. This is due to the soft soil conditions and the high water table in the Rotterdam area. A double track tunnel has a higher risk profile due to influencing the surroundings in a greater extend. Especially the crossing of the railway embankment near Rotterdam Central Station with a Tunnel Boring Machine (TBM) over 11.5 meters radius is considered to be risky. Also the costs of the project will be higher due to the deeper building pits for the station Statenweg and the start and receiving shafts of the TBM.

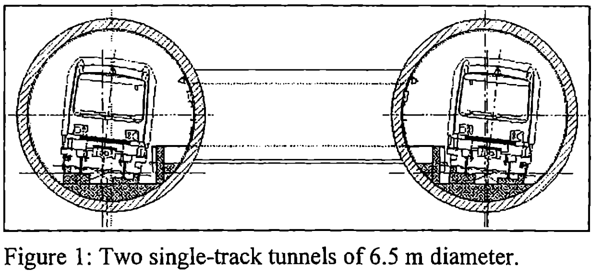

Figure 1: Two single-track tunnels of 6.5 m diameter.

1.2 Tunnel dimensions and embedment

The design is based on two single-track tunnels with an internal diameter of 5.8 meters each and a concrete lining of 0.35 meters thickness. The tunnel lining consists of 6 precast concrete segments and 1 keystone. The segments have a width of 1.5 meters.

The tunnel is mainly located in the Pleistocene sand layer because of the good properties of this stratum for boring the tunnel and supporting the lining. Due to the shallow depth of the alignment near the existing metro-tunnel, built around 1960 by cut and cover techniques, and for optimising the building costs of the start and receiving shafts, the bored tunnel will be placed partly in the very soft soil layers (Fig. 2). At the junctions to the station Statenweg the tunnel tubes are located partly in the soft clays as well, so realising the most public friendly use of this station. In all the mentioned situations the tunnel is founded over 180^0 in the Pleistocene sand as a minimum.
In this paper two cases will be considered: (Table 1).

Table 1. Depth tunnel lining (m minus reference level) and embedment (degrees) in the Pleistocene sand layer.

	Case 1	Case 2
Crown level	−11.75	-10.5
Bottom level	−18.25	−17.0
Embedment	180^0	135^0

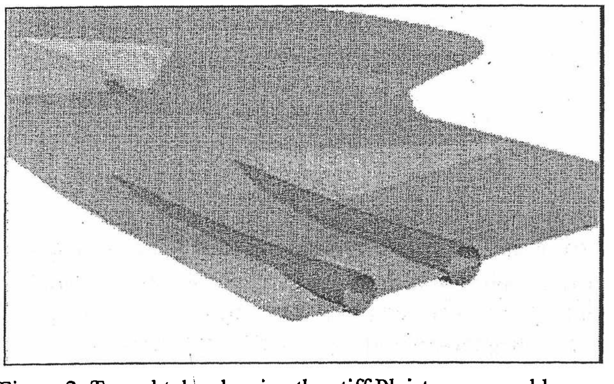

Figure 2: Tunnel tubes leaving the stiff Pleistocene sand layer

2 GEOTECHNICAL CONDITIONS

The geotechnical profile of the Rotterdam city area consists from surface level to approximately 15 meters below reference level of anthropogenic layers (0 to 5 m – ref. level), soft Holocene peat (5 to 10 m – ref. level) and organic clay layers (10 to 15 m – ref. level). Below this level Pleistocene sand layers are encountered till 35 / 40 meters below ref. level. Underneath an overconsolidated clay/sand layer 'Kedichem' is encountered that reaches until 60 meters below reference level. In Figure 3 a typical result of a Cone Penetration Test (CPT), taken at the location of station Statenweg, is shown. The measured cone resistance is presented at the right, the friction ratio at the left. On the vertical axis the depth is given in m – reference level (NAP).

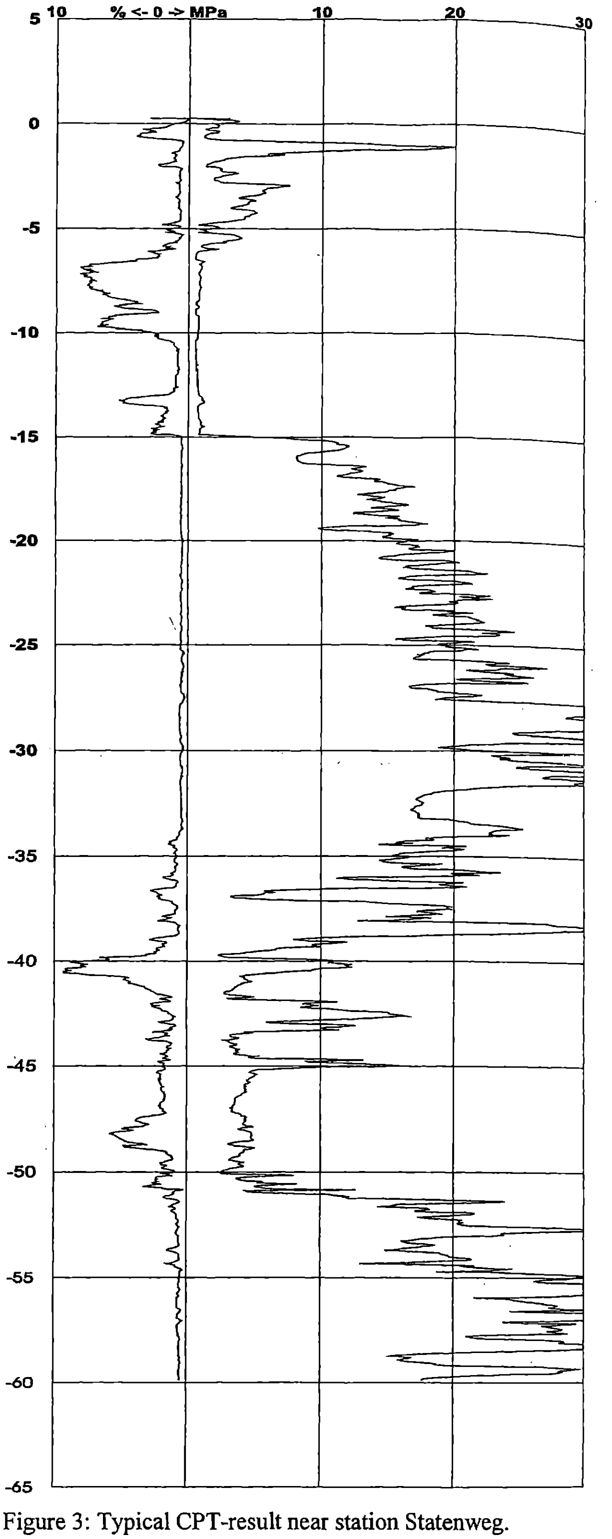

Figure 3: Typical CPT-result near station Statenweg.

The water level along the line is approximately 2 meters below reference level. Table 2 shows some soil (classification) parameters.

Table 2. Soil parameters (mean values)

	γ_{sat} kN/m^3	w %	PI -	c_u kPa	K_0 -	OCR -
Antropogene	18	-	-	-	-	-
Peat 'Holland veen'	10.5	457	-	41	0.4	1.2
Clay 'Gorkum klei a'	13.5	87	47	41	0.5	1.3
Clay 'Gorkum klei b'	16.5	56	37	30	0.5	1.3
Pleistocene sand	20	-	-	-	0.5	1.0
Kedichem clay	21	24		86	0.8	1.7
Kedichem sand	20	-	-	-	0.8	1.7

3 CONSOLIDATION AND CREEP SETTLEMENTS OF THE AREA

During the design lifetime of the tunnel the soft layers will settle some 1.0 m due to consolidation and creep. A regular sand supply on ground level is therefore necessary in order to maintain the surface level on a fixed level. To study the settlement behaviour of the organic clay and peat layers extensometer gauges were installed. Two extensometers are placed on the railway embankment near Rotterdam Central Station. Two others in the vicinity of the future station Statenweg. In Figures 4 and 5 the results of Extensometer 2 (Vroesenlaan), near the Statenweg station, are shown.

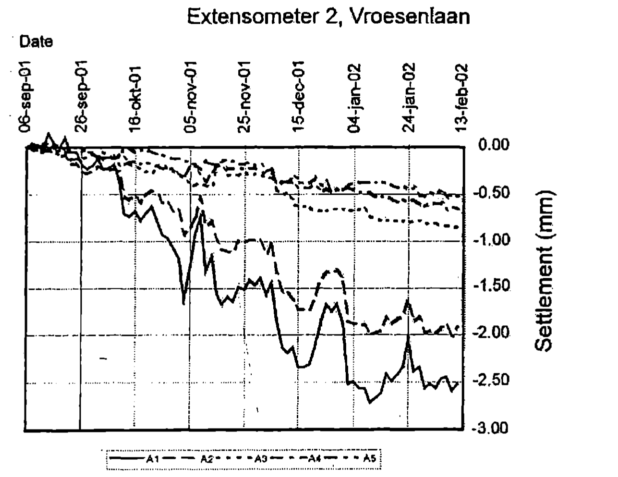

Figure 4: Extensometer measurements. Horizontal axis; time. Vertical axis; settlement in mm.

Extensometer A1: -2.5 m ref. level
Extensometer A2: -6.0 m ref. level
Extensometer A3: -10.0 m ref level
Extensometer A4: -12.0 m ref. level
Extensometer A5: -14.0 m ref. level.

From Figures 4 and 5 it can be seen that at surface level the settlement in six month reaches 2.5 mm. At the crown level of the tunnel, 12 m below reference level for case 1, the settlement is 0.7 mm over this period. Levelling results of the manhole covers of the sewerage system in this part of the city indicates however that over a period of 20 years a mean surface settlement 8.8 mm a year (standard deviation 0.2 mm) is reached.

In this stage of the project a yearly settlement of 10 mm for the surface and 2 mm over the height of the tunnel in the soft layers is adopted. Over a period of 100 years these design values result in a surface settlement of 1.0 m and a settlement of the crown level of the tunnel of approximately 0.2 m (case 1). The Pleistocene sand will not settle.

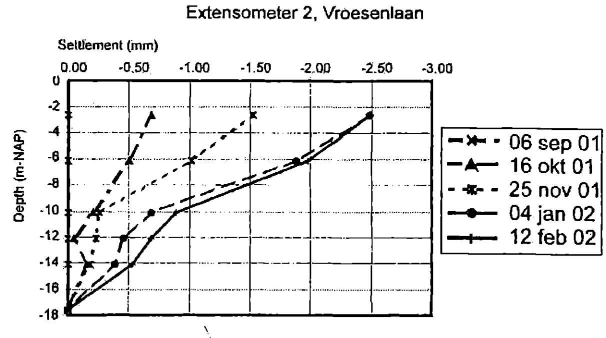

Figure 5: Layer specific settlements over a period of 6 month (same extensometer as Fig.4).

4 FORCES ON THE LINING DUE TO LONG-TERM SETTLEMENTS

Because the tunnel is a relative stiff element the bending moments and axial forces in the tunnel lining will increase with time due to the long-term settlement. The vertical and horizontal forces on the tunnel lining will get larger than the vertical overburden pressure. The time dependent extra forces, called the negative skin friction, has been analysed analytically as well as numerically.

In Figure 6 the extra loading stresses Δp_v and Δp_h due to the long term settlements are shown in addition to the normally used loading scheme. The water pressures are not indicated.

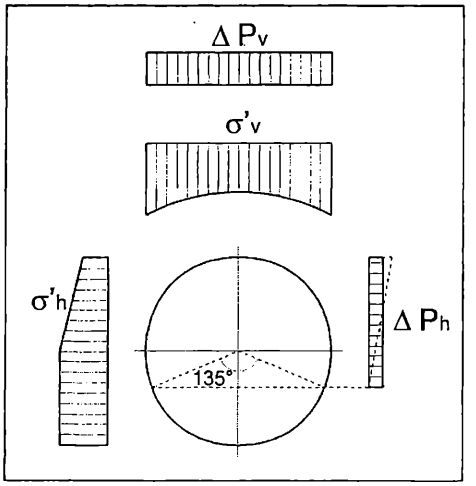

Figure 6: Loading scheme on the lining; Δp_v and Δp_h are the extra loading stresses due to long term settlements.

4.1 Analytical modeling

By using the stress distribution function of Airy the negative skin friction loading Δp_v and Δp_h are determined for a linear elastic situation. Along the lining no horizontal soil displacements are allowed.

4.1.1 Embedment of 180^0

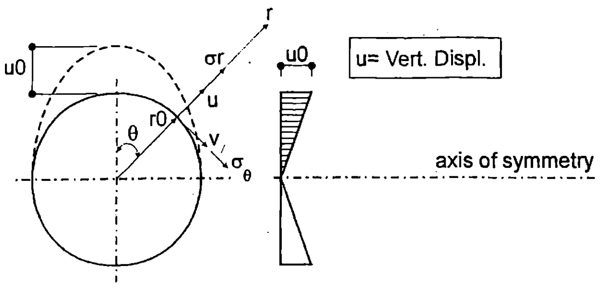

Fig 7: Embedment of 180^0; stresses on the lining and vertical ground displacements.

In Figure 7 the stresses σ_r and σ_θ on the circular boundary and the displacements u and v of the boundary are shown. On the right the vertical ground displacements, outside the sphere of influence of the tunnel, are given with a maximum of u_0 at crown level. The following formulas have been derived.

$$\Delta p_v = 0.8 \cdot \frac{u_0}{R} \cdot \frac{E}{(1+\nu)} \tag{1}$$

$$\Delta p_h = 0.2 \cdot \frac{u_0}{R} \cdot \frac{E}{(1+\nu)} \tag{2}$$

For:

$$K_0 \approx 0.5 = \frac{\nu}{1-\nu} \rightarrow \nu = \frac{1}{3}$$

And:

$$E = E_{oed} \cdot \frac{(1+\nu)\cdot(1-2\nu)}{(1-\nu)} = \frac{2}{3} \cdot E_{oed}$$

The additional loading on the tunnel lining is determined as:

$$\Delta p_v = 0.4 \cdot \frac{u_0}{R} \cdot E_{oed}$$
$$\Delta p_h = 0.1 \cdot \frac{u_0}{R} \cdot E_{oed} \tag{3}$$

4.1.2 Embedment of 135^0

In Figure 8 the stress-displacement scheme is given for an embedment of 135^0.

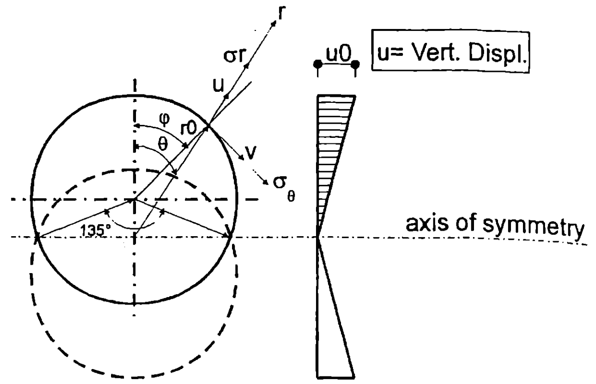

Figure 8: Embedment of 135^0; stresses on the lining and vertical ground displacements.

In the same way as described in chapter 4.1.1 it was derived that for an embedment of 135^0 the elastic solutions for a infinite half space are as follows:

$$\Delta p_v = 0.50 \cdot \frac{u_0}{R} \cdot E_{oed}$$
$$\Delta p_h = 0.06 \cdot \frac{u_0}{R} \cdot E_{oed} \tag{4}$$

4.2 Numerical modeling

A numerical analysis, based on the finite element method, was used in order to include plasticity of the soil and to derive an upper limit for the time dependent forces on the tunnel. This part of the study was performed using the finite element program PLAXIS. Two-dimensional finite element analyses with the Soft Soil Creep model were done considering the situation near the station Statenweg.

An overview of the selected model parameters is given in table 3.

No reduced interface strength around the tunnel was adopted.

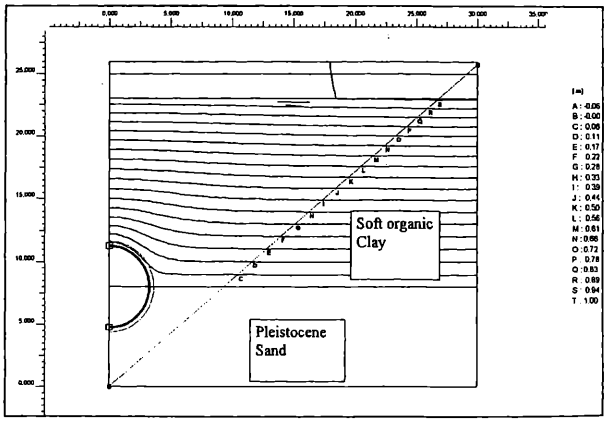

Figure 9: Contour lines vertical displacements around tunnel tube.

Table 3. Model parameters for the organic Clay (Gorkum klei b); PLAXIS (Soft Soil Creep model.

Parameter	Symbol	Value	Unit
Modified compression index	λ^*	0.083	-
Modified swelling index	κ^*	0.017	–
Modified creep index	μ^*	0.011	-
Stress ratio in 1D compression	K_0	0.5	-

To compare the numerical results with the elastic formulas given before it is necessary to derive the Young's modulus from the Soft Soil Creep parameter. The method given by Brinkgreve *et al.*(2001) was used to transform the soft soil creep parameter λ^* in a modulus of elasticity. Instead of Brinkgreve *et al.* (2001) not the parameter κ^* (swelling index) was used, but the compression index λ^*. This was done because the stress path of the area above the crown of the tunnel shows that the stresses increases during the creep phase. Therefore not the swelling index but the compression index is the ruling factor. For an isotropic mean stress of 75 kPa an E_{oed} value of 1350 kPa was obtained (organic clay; Gorkum klei b).

The bending moments show a significant increase due to creep of the soil layers. For the concrete lining with a thickness of 0.35 m the maximum bending moment increases with a factor 2. The normal forces increase with approximately 15%.

4.3 *Physical modelling*

To check the negative skin friction mechanism as described before and to derive optimum design values a test in the Delft geocentrifuge will be performed. The geocentrifuge test will evaluate the effect of the consolidation settlements. Unfortunately it is impossible to model the creep settlement in the centrifuge. Because the expected consolidation part of the total settlement for the Rotterdam area is approximately 65% it was decided that the test could give valuable information. Especially a plastic upper limit for the lining forces is an important design aspect. The test results will be available in October this year. Figure 10 gives an idea of the test set up.

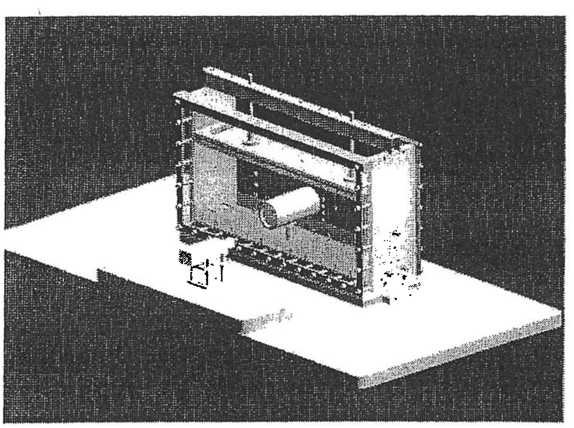

Figure 10: Geocentrifuge test set up.

5 EVALUATION

On behalf of the project RandstadRail the following general formulation is adopted to calculate the negative skin friction on the tunnel lining:

$$\Delta p_v = \frac{\alpha \cdot u_0 \cdot E_{oed}}{R} \qquad \Delta p_h = \frac{\beta \cdot u_0 \cdot E_{oed}}{R} \tag{5}$$

Using a subgrade reaction model with linear axial springs for case 2 (135^0 embedment; see Table 1) it was proved that the effect of the vertical load Δp_v is the opposite to Δp_h. Therefore in Table 4 the ($\alpha-\beta$) value is determined, in order to compare the described calculation models.

Table 4. Calculated α and β values.

Model	α	β	α–β
Analytical 135^0 *	0.50	0.06	0.44
Analytical 135^0 **	0.40	0.09	0.31
Numerical 135^0 **	0.37	0.17	0.20

* with infinite elastic half space
** finite elastic half space (location station Statenweg)

Table 4 shows that the applied models give a reasonable agreement. Obviously the plastic upper limit is not reached and therefore the negative skin friction mechanism is described reasonably well with the linear elastic model. However the numerical model gives a much higher horizontal component (β–value) of the negative friction.

6 CONCLUSIONS

Long term settlement leads to additional loads on the tunnel lining and needs to be considered in the design. To determine the additional loads, called the negative skin friction forces, the settlement of the soil layers during the adopted life-span of the construction and the soil stiffness has to be known.

The negative skin friction influences not only the vertical effective stresses on the tunnel lining, the study shows a change of the horizontal forces as well.

In this stage of the project the elastic approach, Eq. (3) and (4), is used for design purposes. A detailed evaluation of the described mechanism, using the finite element- and geocentrifuge- techniques, is in progress in order to reach optimum α– and β–values for the final design. Especially an upper limit of the negative skin friction due to plasticity in the soil and the influence of the soil strength parameters, for instance the angle of internal friction and the effective cohesion, are important issues for further investigation.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Mr. Geissler of the consulting firm IMM and Messrs. J. Gerritsen and E. Taffijn of Rotterdam Public Works who performed the analytical modeling mentioned in Chapter 4.

Mr. R. Berkelaar was helpful in preparing the extensometer measurements (Ch. 3).

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APPENDIX I. NOTATION

The following symbols and abbreviations were used:

γ_{sat}	: saturated unit weight	[kN/m^3]
w	: water content	[%]
PI	: Plasticity Index	[-]
c_u	: undrained shear strength	[kPa]
Δp_v	: vertical loading due to negative skin friction forces on tunnel lining	[kPa]
Δp_h	: horizontal loading due to negative skin friction forces on tunnel lining	[kPa]
u_0	: vertical soil displacement at crown level; determined outside the sphere of influence of the tunnel	[m]
E	: modulus of elasticity	[kPa]
E_{oed}	: constrained modulus of elasticity	
ν	: Poisson ratio	[kPa]
R	: radius of tunnel tube	[m]
K_0	: coefficient of horizontal earth pressure at rest	[-]
$K_{0;NC}$	: coefficient of horizontal earth pressure at rest for a normally consolidated situation	[-]
OCR	: overconsolidation ratio	[-]
CPT	: Cone Penetration Test	