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by

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Vertical Equilibrium of Sheet Pile Walls with Emphasis on Toe Capacity and Plugging

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Aalborg University, June 2010

Abstract

Constructions including retaining walls are normally established in areas where it is impossible to conduct an excavation with inclined sides. Due to large excavation depths and due to restrictions on the deformations of the wall, it is often necessary to anchor the wall. The limited space makes it impossible to design the anchorage with anchor plates, and the anchorage is therefore often designed as bored and injected soil anchors. Reasons for design and establishment make it is necessary to construct these anchors with an inclination. Inclining anchors imply that larger forces need to be transferred at the pile toe to fulfil vertical equilibrium. The paper describes a case study of sheet pile walls in Aalborg Clay, and the amount of loads transferred as point loads at the pile toe for free and anchored walls is estimated. A parametric study is made for the free wall with regards to the height and the roughness of the wall. Due to limitations of the calculation method, the study of the anchored wall only includes variation of the roughness. For the case study, it is found that the vertical equilibrium is fulfilled for the considered free wall. An anchored wall needs a plug forming at the pile toe to fulfil the criterion of vertical equilibrium. However, the suggested theoretical model shows that the extent of the plug is much smaller than needed.

1 Introduction

Retaining walls are typically established in areas where limited space, stability or ground water conditions make it impossible to conduct the excavation with inclined sides. This is often the case in the cities. Due to requirements on the deformations of constructions nearby, it is often necessary to anchor these walls.

The limited space condition frequently implies that it is impossible to design the anchorage with anchor plates; there is simply no available area to establish anchor plates in the required distance from sheet pile wall. It is therefore necessary to design the anchorage as an injected soil anchor.

To achieve the necessary roughness of the interface between soil and anchor, these are injected with concrete after installation. Further, they are often injected again one day after the first injection, inducing a more rough interface between the soil and the anchor.

The grouting and the design of these anchorages normally make it necessary to install them with an inclination. This induces a larger overburden pressure at the fixed length of the anchor which increases the bearing capacity. Further, the inclination makes it easier to place the anchors deep below existing constructions minimising the risk of damage of neighbouring constructions.

If the anchors were established in a hor-

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izontal plane, this would complicate the installation of the anchor, as the concrete used for injection would drift from the fixed zone of the anchor into zone of the apparent free length and even back into the excavation pit. The total stability of the wall would not be ensured if the failure mechanism 7of the anchor coincides with the likewise from the wall.

Designing sheet pile walls with inclined anchors implies that larger vertical force is introduced as a point load in the pile toe to fulfil vertical equilibrium. The load, which can be transferred through the cross section area of pile toe, is small compared to the load needed to be transferred to the soil. It is therefore interesting to investigate whether the effective area of a sheet pile toe can be assumed larger than the cross section area of the steel. This is the case if a plug forms inside the cross section of the sheet pile wall.

Nowadays, vertical equilibrium of sheet pile walls is often neglected or restricted to investigations on whether the tip resistance is negative, i.e. an upwards directed force is to be transferred at the pile toe. An upward directed force implies that a tension force between pile toe and the underlying soil occurs, which the soil does not handle well and the force cannot be transferred. However, a downward directed force implies a compression force, which the soil is able to transfer. According to Ovesen et al. (2007) and Harremoës et al. (2003) the vertical projection is of minor importance when designing sheet pile walls.

In Denmark sheet pile walls are mainly designed using Brinch Hansen's earth pressure theory (Brinch Hansen, 1953). This theory provides a formula to calculate the tangential earth pressure acting on the wall and this formula is generally applied. It is accepted that upwards directed forces cannot be transferred at the pile toe. However, there is not yet established a practice for estimation of the transferral of a downwards directed force to the soil if the steel area of the sheet pile wall cannot provide sufficient bearing capacity.

The literature provides many different suggestions on how the effective area should be calculated. According to Ovesen et al. (2007) only the steel area can be used when considering sheet pile walls. Lundgren and Brinch Hansen (1958) describes that the total area of the smallest circumscribed convex polygon can be used, cf. Fig. 1, while Harremoës et al. (2003) and Hansen (1978) uses 80% of this area.
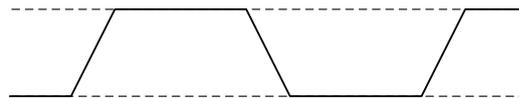

Figure 1: The solid line shows the sheet pile wall, while the dashed line shows the circumscribed polygon.

The Piling Handbook describes that 50% of the smallest circumscribed convex polygon can be applied when the sheet pile toe is placed in clay (Arcelor, 2005). Tomlinson and Woodward (2008) also describes that the base area of open-ended piles in clay should be calculated as 50% of the total circumscribed area.

In Germany, the area is calculated according to the section angle α, cf. Fig. 2. The reduction factor κ_F is calculated according to Eq. 1 (Weissenbach, 1977, 2008) where α should be entered in degrees.

$$\kappa_F = 0.015 \cdot \alpha - 0.35 \qquad (1)$$
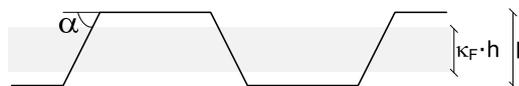
Figure 2: Calculation of the effective height for a sheet pile wall according to Weissenbach (1977, 2008).

Hoesch sheet pile walls are generally constructed with α between 50º and 81.5º (Hoesch Spundwand und Profil, 2010). This provides an effective area of 40% to 87% of the area of the circumscribed convex polygon.

The literature provides rather different effective areas. Unfortunately, most literature does not describe how the presented effective

Table 1: Plug size of sheet pile walls placed in clay.

Author/editor	Plug size of sheet pile wall	Originates from	Theory for
Ovesen et al. (2007)	0 % for steel sections	Unknown	Sheet pile walls
Lundgren and Brinch Hansen (1958)	100 %	Unknown	Sections of cast iron
Harremoës et al. (2003)	80 %	Unknown	Sheet pile walls
Hansen (1978)	80 %	Unknown	Sheet pile walls
Arcelor (2005)	50 %	Unknown	Piles
Tomlinson and Woodward (2008)	50 %	Unknown	Piles
Weissenbach (2008)	40 - 87.3 %	Empicical data	Sheet pile walls
Weissenbach (1997)	40 - 87.3 %	Empicical data	Sheet pile walls

areas are found, i.e. theoretical, numerical, or based on empirical values, cf. Tab. 1. Therefore, this paper theoretically investigates the plugging effect based on theory described for open-ended circular pipe piles.

Before the theoretical evaluation of plugging, the magnitude of the forces at the pile toe is investigated for both anchored and free sheet pile walls. According to EN1997-1, a sheet pile wall has no roughness in the soil-to-steel interface which implies that the tangential earth pressure is equal to zero in the undrained state (European Committee for Standardization, 2007). However, in the drained state the roughness is normally adjusted so the criterion of vertical equilibrium is fulfilled. Plugging is therefore investigated in the drained state.

Aalborg Clay is used as a medium for the investigations in this paper. The strength of Aalborg Clay is defined by Iversen et al. (2010), and summarised in Tab. 2.

2   Magnitude of Point Load at Pile Toe

Before investigating the plugging effect, the necessity of plugs formed in the toe of sheet pile walls is documented in the studied case in Aalborg Clay.

The excavation depth h_2 is $5m$ and $12m$ for the free and the anchored sheet pile wall, respectively, cf. Fig. 3. The water level is placed at excavation level on the front side,

Table 2: Strength of Aalborg Clay according to Iversen et al. (2010).

	Characteristic value	Design value
φ' [^{o}]	28.1	24.0
c' [kPa]	13.0	10.8
c_u [kPa]	$0.446 \cdot \sigma'_v \left(\frac{\sigma'_{pc}}{\sigma'_v} \right)^{0.72}$	NA
$c_{u,average}$ [kPa]	100.8	56

and at surface level on the back side, marked by the dashed line in the figure. The design value of the surface load is $20kN/m^2$ and it is placed on the back side. In the anchored model the anchor is placed $2m$ below excavation level and the anchor is inclined 20° to horizontal.

Point loads in the wall at the pile toe acting upwards are considered negative.
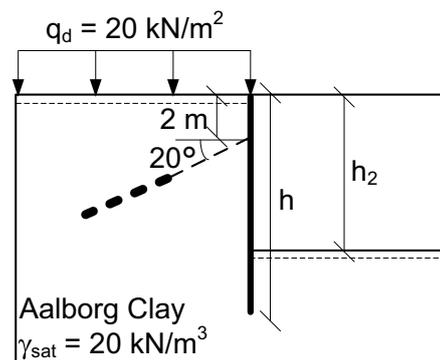
Figure 3: Sketch of the geometry used when calculating the vertical equilibrium of sheet pile walls.

The tangential earth pressure F is calculated according to Eq. 2, which theoretically connects the earth pressure to the movements

of the wall during failure.

$$F = E_\gamma \cdot \tan \delta_\gamma + (E_p + E_c) \cdot \tan \delta_p + a \cdot h$$
$$(2)$$

E is the earth pressure, δ is the mobilised soil-to-steel angle of friction, h is the total height of the wall and a is the mobilised adhesion between the soil and the wall. F is positive in the upward direction.

The values of $\tan \delta_\gamma$, $\tan \delta_p$ and a are connected to the failure mode. Brinch Hansen has developed diagrams to determine the values of the parameters depending on the internal angle of friction for the soil φ', the relative placement of the point of rotation, and the direction of rotation (Harremoës et al., 2003).

When designing a free wall according to Brinch Hansen's earth pressure theory, the relative placement of the point of rotation is zero, indicating that the wall rotates around the toe. Using this information makes it possible to calculate the vertical equilibrium for varying height and roughness, cf. Fig. 4. The roughness is chosen to be identical on the front and the back side of the wall.

The average dead-load of sheet pile walls are found for sheet pile walls from the manufacturer Hoesch Spundwand und Profil (2010) to $8.86 kN/m$. This dead-load should also be transferred to the soil via the pile toe, and is included in Q_p shown in the figure.

In the red areas of the figure, a positive load must be transferred to the soil, whereas the yellow and blue areas indicate that a negative load must be transferred. The latter cannot be accepted. The average cross sectional area of the sheet pile walls are found to $1.51 m^2/m$ (Hoesch Spundwand und Profil, 2010), which provides an end bearing capacity of $9 \cdot c_{ud,average} \cdot A_{steel} = 7.6 kN/m$. The positive load, which is to be transferred to the soil, is seldom larger than that.

For an anchored wall, the earth pressures are found by use of SPOOKS (GEO, 2010). This program calculates the total earth pressure E. It is therefore assumed that $\tan \delta_\gamma$ is
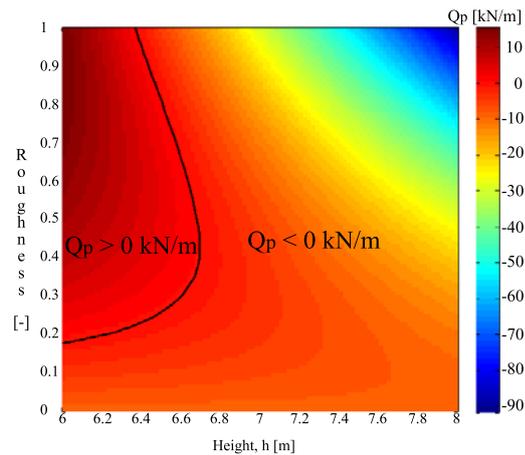

Figure 4: Parametric analyses of the point load Q_p at the pile toe as a function of the height and the roughness for a free sheet pile wall.

equal to $\tan \delta_p$, which is a fair assumption according to Fig. 5. The average between the two parameters is used in the calculations. The figure shows the values for a perfectly rough wall. For a roughness lower than this, linear interpolation is used between 0 (smooth wall) and the value for a perfectly rough wall.
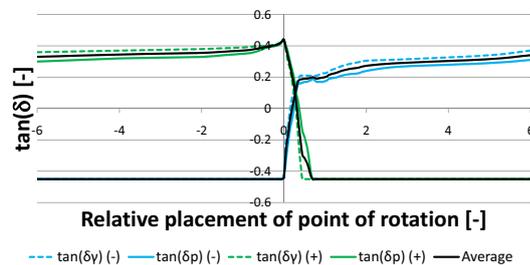
Figure 5: Difference between $\tan \delta_\gamma$ and $\tan \delta_p$ for a perfectly rough wall.

Together with the calculations of the tangential earth pressures, the maximum bending moment in the sheet pile wall is calculated. A sheet pile wall from the manufacturer Hoesch Spundwand und Profil (2010) is found fulfilling the condition $\frac{M_{max}}{f_{y,steel}} < W_y$. The design yield strength of steel is found to $390 MPa$. As the anchor is installed with a $20°$ inclination to horizontal, a vertical component of this is included when calculating vertical equilibrium.

According to Ovesen et al. (2007) an anchored sheet pile wall can fail in three modes. The maximum bending moments for the fail-

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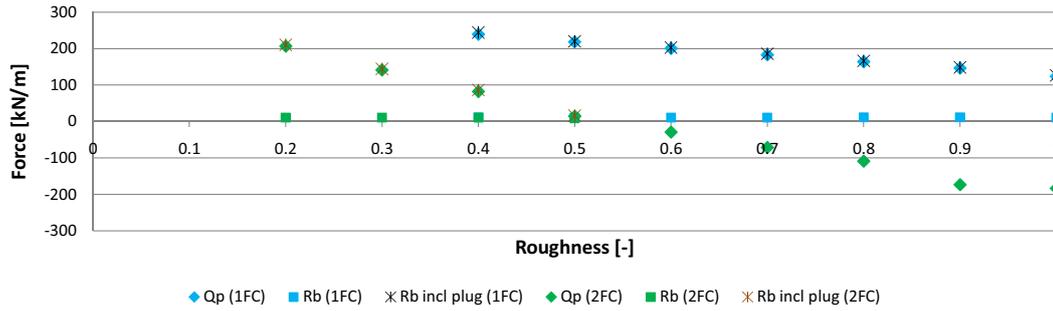
Figure 6: Parametric analysis on the roughness on a anchored sheet pile wall. Q_p is the force needed to be transferred to the soil to fulfil vertical equilibrium. R_b and R_b incl. plug is the base resistance of the steel and the plugged area, respectively.

Table 3: Necessary extent of plug to fulfil the vertical equilibrium equation for the case study in Aalborg Clay.

Roughness	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Section height, $h_w all$ (1FC) [mm]			485	485	485	485	440	440	440
Plug size (1FC) [mm]			485.0	436.5	402.6	368.6	330.0	294.8	250.8
Relative plug size (1FC)			100%	90 %	83 %	76 %	75 %	67 %	57 %
Section height, $h_w all$ (2FC) [mm]	485	485	440	350					
Plug size (2FC) [mm]	417.1	286.2	171.6	31.5					
Relative plug size (2FC)	86 %	59 %	39 %	9 %					

ure mode without any yield hinges (0FC) are very high, and it has not been possible to find sheet pile walls with sufficient moment of resistance. For the failure modes with one (1FC) and two (2FC) yield hinges the roughness of the wall must be larger than 0.4 and 0.2, respectively, to find sheet pile walls with sufficient moments of resistance. A parametric analysis of the roughness of anchored sheet pile walls can be seen in Fig. 6.

The bearing capacity calculated over the steel area is never enough to ensure vertical equilibrium for the 1FC failure mode, cf. Fig. 6. To fulfil the vertical equilibrium it is necessary to assume that a plug forms in the pile toe. The size of the plug needed to fulfil equilibrium in per cent and millimetre is seen in Tab. 3. Further, the section height of the chosen profile is noted.

For the 2FC failure mode Q_p becomes negative when the roughness is 0.6 and larger. This load cannot be transferred to the soil. To fulfil the criterion of vertical equilibrium it is necessary to assume that the roughness is below 0.5. The size of the plug needed to fulfil vertical equilibrium can be seen in Tab. 3.

In the present case, plugging is of great importance to be able to fulfil the vertical equilibrium equation. Especially when the 1FC failure mode is used, large forces need to be transferred through the sheet pile toe. In the following section, a theoretical approach to the plugging phenomenon is taken.

3 Theoretical Expression for Plugging

The plugging phenomenon is investigated by Paikowsky and Whitman (1990) for open-ended pipe piles. They have plotted the ratio of the static end bearing capacity R_p to the total bearing capacity of a closed-ended pipe pile against the penetration depth D to the diameter d ratio, cf. Fig. 7. This curve indicates that for high ratios of the penetration to diameter ratio the end bearing capacity is of minor importance of the total bearing capacity.

Further, they have plotted the ratio of the static bearing capacity of an open-ended pipe R_{open} to the bearing capacity of a closed-ended pipe R_{closed}. It is found that a full plug
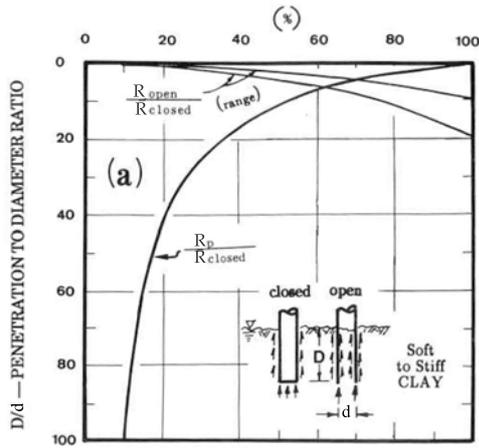
Figure 7: The effect of a plug upon the static bearing capacity. After Paikowsky and Whitman (1990).

occurs for a penetration to diameter ratio of $10 - 20$. An equivalent diameter of a sheet pile wall is found by letting the circumscribed area of a sheet pile be equal to half the area of a circle, cf. Fig. 8. This leads to an equivalent diameter of $0.62 - 0.87m$ for sheet pile walls from the manufacturer Hoesch Spundwand und Profil (2010). The penetration depth for fully plugged sheet pile walls are found to $6.2 - 17.4m$. Normal penetration depth of sheet pile walls are expected to lie within this range.
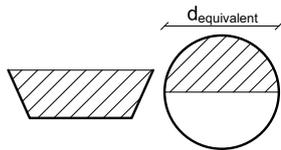

Figure 8: The equivalent diameter calculated by letting the hatched area of the sheet pile wall being equal to the hatched area of the circle.

Generally, it is expected that plugging occurs to a larger extent as the soil-to-steel angle of friction δ increases and is more likely to happen for narrow cross sections.

The geometry of a sheet pile wall resembles an H-section more, compared to a circular tube. This is due to both the H-section and the sheet pile having an open cross section, whereas the tube has a closed cross section, cf. Fig. 9. For a sheet pile wall a full plug occurs if the extent of the plug e_{plug} is larger than or equal to the section height of the wall

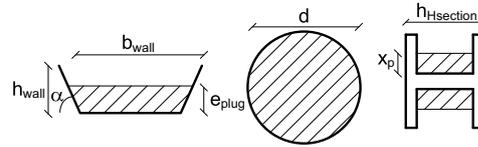h_{wall}. Further, a partial plug forms, if e_{plug} is smaller than the section height of the wall.

Figure 9: Sketch of the areas in which plugging occurs for different geometries.

Bowles (1996) presents an expression from which the size of a plug for an H-section can be calculated, cf. Eq. 3. The equation originates from a theoretical expression, where the friction along the soil-to-steel interface is set equal to the friction along the soil-to-soil interface.

$$x_p = \frac{h_{Hsection}}{2} \cdot \left(\frac{\tan \varphi}{\tan \delta} - 1 \right) \qquad (3)$$

Here $h_{Hsection}$ is the height of the H-profile and x_p is the extent of the plug, cf. Fig. 9. The extent of the plug is plotted against the soil-to-steel angle of friction, cf. Fig. 10. It is chosen to use an H-profile with a width of $300mm$ and a $h_{Hsection}$ of $440mm$. The variation of x_p with increasing δ is found to counteract what is expected. Furthermore, Eq. 3 implies that the extent of the plug x_p increases with increasing $h_{Hsection}$, which also is contrary to what is expected.
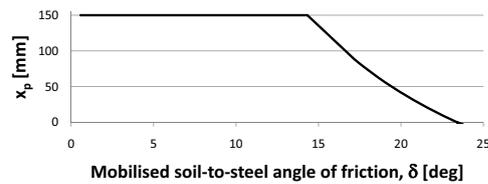
Figure 10: The plugged length x_p as a function of the mobilised soil-to-steel angle of angle of friction an H-section placed in Aalborg Clay.

The expression found by Bowles (1996) is therefore rejected, and plugging of sheet pile walls is then investigated by studying literature from open-ended pipe piles. According to Paikowsky and Whitman (1990) plugging occurs if the skin friction calculated over the plug height Q_τ reduced for soil weight G_{soil} is larger than the static end bearing capacity

Table 4: Summation of the four methods used to calculate the unit skin friction.

Expression	Variables	Reference
$\tau_{\text{Coulomb}} = K_0 \sigma_v' \cdot \tan(\delta)$	K_0, δ and σ_v'	(Ovesen et al., 2007)
$\tau_{\text{Randolph et al.}} = \beta \sigma_v'$	δ, φ' and σ_v'	(Randolph et al., 1991)
$\tau_{\text{API}} = \alpha_{API} \cdot c_u$	c_u and σ_v'	(API, 2002)
$\tau_{\text{Meyerhof}} = 1.5 \cdot K_0 \cdot \sigma_v' \cdot \tan(\varphi')$	K_0, φ and σ_v'	(Meyerhof, 1976; Randolph and Murphy, 1985)

beneath the plugged zone R_b, cf. Eq. 4.

$$Q_\tau - G_{soil} > R_b \tag{4}$$

The point bearing capacity is calculated as for piles in clay, cf. Eq. 5 (Ovesen et al., 2007; Harremoës et al., 2003; API, 2002; Arcelor, 2005; Pelletier et al., 1993).

$$R_b = 9 \cdot c_u \cdot A_b \tag{5}$$

Here, c_u is the undrained shear strength and A_b is the end bearing area including the plugged area. The end bearing capacity is calculated in the undrained state, as no expression is found for calculating R_b in the drained state in clay. According to Hansen (1978) the undrained state is critical when calculating R_b, i.e. a minimum value. However, in the investigations on the plugging effect, the critical situation occurs where R_b is at its maximum. A higher R_b counteracts plugging. As no maximum expression exists, R_b is calculated in the undrained state.

The dead load of the soil G_{soil} is calculated as the submerged unit weight of the soil in the plug and the volume vertically above, cf. Eq. 6.

$$G_{soil} = D \cdot A_b \cdot \gamma' \tag{6}$$

The skin friction is calculated as the unit skin friction τ times the area of the plug-to-wall interface. This area is calculated by means of the vertical height of the plug h_{plug}. The unit skin friction can be calculated in various ways. Four expressions for the unit skin friction is used here, cf. Eq. 7 to 10 (Ovesen et al., 2007; API, 2002; Randolph et al., 1991; Meyerhof, 1976).

$$\tau_{\text{Coulomb}} = K_0 \cdot \sigma_v' \cdot \tan(\delta) \tag{7}$$

$$\tau_{\text{API}} = \alpha_{API} \cdot c_u \tag{8}$$

$$\tau_{\text{Randolph et al.}} = \beta \cdot \sigma_v' \tag{9}$$

$$\tau_{\text{Meyerhof}} = 1.5 \cdot K_0 \cdot \sigma_v' \cdot \tan(\varphi') \tag{10}$$

Here, K_0 is the earth pressure coefficient at rest, σ_v' is the effective insitu vertical stress and δ is the soil-to-steel angle of friction. The API-method calculates the unit skin friction empirically by the factor α_{API}, cf. Eq. 11, which depends on the ratio ψ calculated as the ratio of the undrained shear strength to the effective insitu vertical stress.

$$\begin{aligned} \alpha_{API} &= 0.5\psi^{-0.5} \quad \text{if} \quad \psi \leq 1.0 \\ \alpha_{API} &= 0.5\psi^{-0.25} \quad \text{if} \quad \psi > 1.0 \end{aligned} \tag{11}$$

The method proposed by Randolph et al. (1991) uses the β factor. This factor depends on the ratio of the horizontal to the vertical effective stress inside the plug after installation. This can be hard to determine, and for design purpose a minimum value of β can be used, cf. Eq. 12.

$$\beta = \frac{\sin\varphi \cdot \sin(\Delta - \delta)}{1 + \sin\varphi \cdot \cos(\Delta - \delta)} \tag{12}$$

$$\Delta = \arcsin\left(\frac{\sin\delta}{\sin\varphi}\right)$$

The expression established by Meyerhof (1976) resembles Coulomb's failure criterion, Eq. 7. Coulomb's failure criterion is a theoretically established equation. Randolph and Murphy (1985) refers to τ_{meyerhof} in the shown form, and describes that the factor 1.5 allows for the increase in horizontal effective stress due to pile installation. It has afterwards been indicated that the internal angle of friction should be replaced by the angle of friction measured by simple shear, which tends to be somewhat lower than the deduced from triaxial testing (Randolph and Murphy, 1985).

The four proposed expressions are summarised in Tab. 4, where the incorporated variables in the expressions are noted. According to EN1997-1 the amount of shear stress which can be mobilised between a wall

and the soil should be dependent on the soil-to-steel angle of friction δ (European Committee for Standardization, 2007). According to Tab. 4 this is only the case for the first and second expression. The third and fourth expression is therefore eliminated at this stage.

Bea and Doyle (1975) investigated the soil-to-steel angle of friction for different types of soil by both a triaxial rod shear test and a direct shear test. Their general observation is that the interface strength increases as the plasticity index I_P decreases. They tested two types of clay and a dense sand, and the results can be seen in Tab. 5.

Table 5: Failure value of the soil-to-steel angle of friction found by triaxial rod shear test (TRST) and direct shear (DS). (Bea and Doyle, 1975)

	I_P [%]	δ_{TRST} [°]	δ_{DS} [°]
Gulf of Mexico Clay	$40-70$	6	14
Silty Clays	$20-30$	10	20
Dense Sand	NA	20	30

According to Iversen et al. (2010) the average plasticity index for Aalborg Clay is $I_P = 22.5\%$. Comparing this value to those found in Tab. 5 indicates that the failure value of the soil-to-steel angle of friction is between $10°$ and $20°$.

EN1997-1 prescribes that the mobilised soil-to-steel angle of friction for at sheet pile wall may not exceed $\delta = 0.67 \cdot \varphi'$ (European Committee for Standardization, 2007). Using the maximum allowed value of δ according to EN1997-1 implies that the failure value of the soil-to-steel angle of friction for Aalborg Clay should be $0.67 \cdot 24.0° = 16.0°$. This value is the interval indicated by Bea and Doyle (1975) for a plasticity index of 22.5%. In the following $\delta_{failure}$ is set to 16.0°.

4 Verification of Theoretical Expression

The previous section describes the theory behind the plugging phenomenon. The theory

is applied for a given cross section of a sheet pile placed in Aalborg Clay, and the extent of the plug is estimated based on the two suggested methods for calculating the unit skin friction. A sheet pile wall with the dimensions in Tab. 6 is used.

Table 6: Dimensions of sheet pile wall according to Fig. 9.

h_{wall}	$440mm$
b_{wall}	$675mm$
α	$56°$

The strength of Aalborg Clay is defined by Iversen et al. (2010) and summarised in Tab. 2. The coefficient of earth pressure at rest is calculated as $K_0 = 1 - \sin \varphi'_d = 0.593$. The undrained shear strength is applied as stress dependent according to the SHANSEP formula (Ladd et al., 1977). Iversen et al. (2010) defines the regression coefficients for Aalborg Clay to $A = 0.446$ and $\Lambda = 0.72$, and the applied shear strength is seen in Eq. 13.

$$c_{uk} = 0.446 \cdot \sigma'_v \cdot \left(\frac{\sigma'_{pc}}{\sigma'_v} \right)^{0.72} \qquad (13)$$

Previous triaxial tests conducted on Aalborg Clay show an *OCR* of approximately two and a unit weight of $20kN/m^3$ (Ibsen, 2007; Luke, 1994; Geoteknisk Insititut, 1985; Geoteknisk Institut, 1963). The preconsolidation stress is estimated as $\sigma'_{pc} = \sigma'_v \cdot OCR = 200kPa$.

Partial coefficients according to EN1997-1 DK NA:2008 are introduced on the strength, i.e. 1.2 on the drained strength and 1.8 on the undrained strength (European Committee for Standardization, 2008). It is assumed that the driving depth of the sheet pile wall is $D = 10m$ and the height of the soil plug is $1m$.

The soil-to-steel angle of friction is used as a measure of the plugging effect. The plugging effect is increased if the soil-to-steel angle of friction necessary to form a plug decreases and vice versa. The verification includes that Eq. 4 shall fulfil the following:

1. The plugging effect increases when the

section height h_{wall} decreases. This implies that partial plug easier forms compared to a full plug. However, if the height is zero no plug is expected.

2. The skin friction reduced for soil weight must increase with increasing soil-to-steel angle of friction (European Committee for Standardization, 2007). Further, if plugging occurs for a certain value of the soil-to-steel angle of friction the pile must remain plugged for higher value of the soil-to-steel angle of friction.

3. The plugging effect is increased when the section angle α increases.

4. The plugging effect is decreased when the section width b_{wall} of the sheet pile wall is increased.

Add 1)

The section height of the sheet pile wall h_{wall} is plotted against the mobilised soil-to-steel angle of friction of the wall to form a plug for the two methods of calculating τ, cf. Fig. 11.
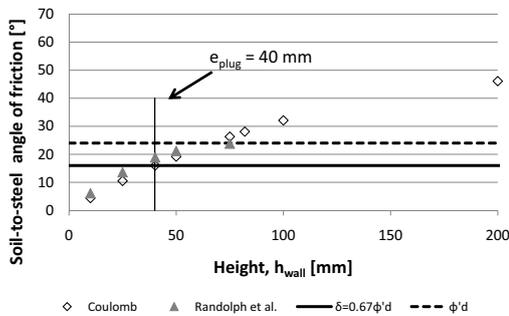
Figure 11: The height of the cross section of the sheet pile wall as a function of the soil-to-steel friction angle for the case study in Aalborg Clay.

When the height of the wall is zero, indicating a straight section, it is expected that no plug forms. However, even for a straight section a friction between the soil and the steel occurs. Thus, a plug is theoretically formed, indicating that a soil toe forms for straight sections.

However, in the normal range of h_{wall} $(200-500mm)$ the needed soil-to-steel angle

of friction increases as h_{wall} increases. This implies that a plug forms easier when h_{wall} is decreased. Generally, it is found that a plug of $e_{plug} = 40mm$ forms in the studied case, cf. the thin black line in the figure. In the following investigations, the extent of the plug is set to $40mm$ to be able to investigate the plugging effect.

Add 2)

The skin friction reduced for soil weight is plotted against the soil-to-steel angle of friction, cf. Fig. 12.
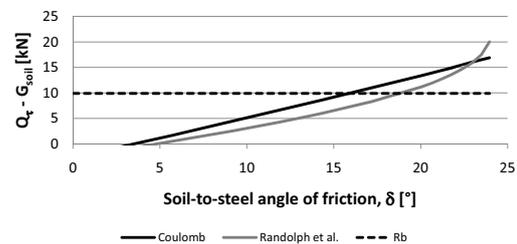

Figure 12: The skin friction reduced for soil weight and the static end bearing capacity as a function of the soil-to-steel angle of friction for the case study in Aalborg Clay.

For the considered case, with dimensions of the sheet pile wall as described in Tab. 6 and $e_{plug} = 40mm$, the static end bearing capacity is found to $R_b = 9.9kN$. Fig. 12 shows that the two methods for calculating τ nearly provides the same skin friction, and the skin friction increases with increasing δ for both expressions.

Add 3) and 4)

It is tried to vary the two parameters α and b_{wall} to see the influence on the plugging. Both parameters are plotted against the mobilised soil-to-steel angle of friction to form a plug, cf. Fig. 13 and 14.

Fig. 13 shows that the mobilised soil-to-steel angle of friction increases as the section angle α increases. At an angle of approximately $50-60°$ the graph has a peak, indicating that the plug hesitates to build up at this
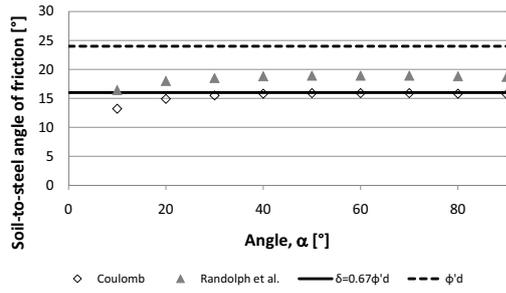
Figure 13: The angle α as a function of the mobilised soil-to-steel angle of friction for the case study in Aalborg Clay.
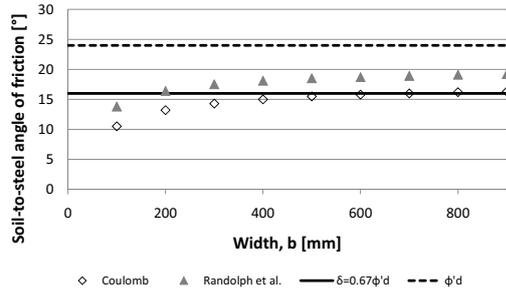
Figure 14: The width b_{wall} as a function of the mobilised soil-to-steel angle of friction for the case study in Aalborg Clay.

angle. For further increasing values of the soil-to-steel angle of friction the plug forms easier.

Normally, sheet pile walls are constructed with section angles of 50° to 81.5° (Hoesch Spundwand und Profil, 2010). In this range, the mobilised soil-to-steel angle of friction is more or less constant, implying that the section angle is of minor importance for the plugging effect. It should be noted that the extent of the plug is 40mm, which is the reason for the very small variation. However, the trend of the curve is the same for larger extents of the plug, and the angle is still of minor importance for the plugging effect.

Fig. 14 shows that the mobilised soil-to-steel angle of friction increases as the section width of the sheet pile wall increases. This implies that a higher mobilised soil-to-steel angle of friction is necessary to form a plug for increasing b_{wall}. For a certain width of the section, the mobilised soil-to-steel angle of friction increases above the failure value, implying that a plug no longer forms.

Comparison

For the considered dimensions of the sheet pile wall, cf. Tab. 6 and an extent of the plug of $e_{plug} = 40mm$, a plug will form easier according to the Coulomb failure criterion when the soil-to-steel angle of friction is below 23.0°, cf. Fig. 12. However, above 23.0° the plug forms easier if τ is calculated according to Randolph et al. (1991). The exponential increase in skin friction implies that for high soil-to-steel angle of friction a larger plug can form, cf. Fig. 15.
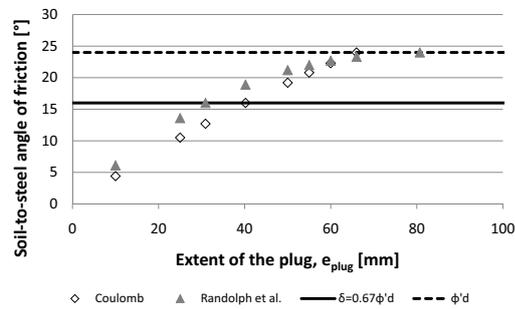
Figure 15: The extent of the plug e_{plug} as a function of the mobilised soil-to-steel angle of friction when the skin friction is calculated by Coulomb's failure criterion and Randolph et al. (1991), respectively, for the case study in Aalborg Clay.

However, using the restriction that $\delta_{max} = 0.67 \cdot \varphi'$ implies that the largest plug can be found if τ is calculated by Coulomb's failure criterion, cf. Tab. 7.

5 Shortcomings of Theoretical Model

The above analyses of the plug size show that a boundary condition is ignored because the theory shows that a plug is formed for a linear cross section.

The boundary problem concerning h_{wall} can be explained by the magnitudes of the soil and wall area, as the extent of the plug is zero. The soil area is zero for $e_{plug} = 0$ and the static end bearing capacity is found to zero. The area of the wall, however, does not

Table 7: Plug size when $\delta = 0.67 \cdot \varphi'$, $b_{wall} = 675mm$ and $\alpha = 56°$. The relative plug size is calculated based on a section height of 485mm, 440mm, and 350mm, respectively. These are the section heights used for the sheet pile walls in Tab. 3.

Method	Plug size [mm]	Relative plug size $h_{wall} = 485mm$	Relative plug size $h_{wall} = 440mm$	Relative plug size $h_{wall} = 350mm$
Coulomb's failure criterion	40.2	8.3%	9.1%	11.5%
Randolph et al. (1991)	30.9	6.4%	7.0%	8.8%

approach zero, and the value of the skin friction is larger than zero, implying that a plug occurs. However, for $e_{plug} > 0$ the plug forms easier for shorter extents indicating that partial plugs form easier than full plugs.

The dead-load of the plug is calculated as the weight of the soil in the plug and vertically above the plug. Due to the open cross section, the weight acting on the plug might be larger, as a result of a pressure distribution of the surrounding soil. Piles in sand should generally be placed with a distance to the nearest of 2.5 times the smallest length to act as a single pile (Ovesen et al., 2007). A similar standard is not found for clay, but it is assumed that the distance approximately is the same. Thus, the inclination of the pressure distribution with vertical is assumed small.

In the case study the extent of the plug is found to be very small, i.e. between 30.9 (7% of a 440mm high cross section) and 40.2mm (9.1% of a 440mm high cross section). The plugs are generally found to be much smaller compared to what is estimated from the literature, where the plugs are between 40% and 100%. However, recent literature (from after 1980) only predicts up to 90% plug.

It is tried to calculate the plug size of a circular tube to see if the open cross section of the sheet pile wall causes the small extents of the plug. For a 1m high plug and a mobilised soil-to-steel angle of friction of $0.67 \cdot \varphi'$ it is found that a plug can form in a tube with a diameter of $123 - 161mm$, depending on the calculation method for τ. The extent of the plug is again found to be very small. Thus, the open cross section of the sheet pile wall is not the reason for the small plugging formation.

According to Ovesen et al. (2007) the effective end area of sheet pile walls can only be estimated as the area of the steel, implying that no plug occurs. Compared to this, $7 - 9\%$ plug is an improvement which is predicted to occur for the used sheet pile wall.

Calculating the static end bearing capacity for the investigated sheet pile wall using the steel area gives that the sheet pile wall can resist $7.2kN/m$. If the static end bearing capacity in the studied case is predicted by both Coulomb's failure criterion (Ovesen et al., 2007) and by Randolph et al. (1991) an end bearing capacity of $15.5kN/m$ and $20.2kN/m$ is found for the two calculation methods, respectively. This implies an increase of the static bearing capacity of 115% and 181%, respectively.

The increase of the static end bearing capacity is found to be large, but it should be compared to the large point load that needs to be transferred to the soil at the toe of the wall. According to Tab. 3 it is necessary to form a plug of at least 250.8mm to fulfil vertical equilibrium of a sheet pile wall designed to fail with one yield hinge in the considered Aalborg Clay.

In the above presented results it is assumed that the plug is 1m high. However, the height of the plug has a large influence on the results. It is of first order when calculating the skin friction reduced for soil weight. However, the static end bearing capacity is not influenced by the height of the plug. A higher plug therefore leads to a larger e_{plug}.

Using Coulomb's failure criterion it is found that a plug with a extent of $e_{plug} = 40.2mm$ can form for a height of the plug of

1m. Increasing the height of the plug to 2m implies that a plug of 89.6mm can form, i.e. the extent is more than doubled. The extent of the plug is plotted as a function of the height of the plug, cf. Fig. 16. The extent of the plug increases with a polynomial for increasing heights of the plug.

It should be noted that the unit skin friction is calculated at the pile toe and this value is applied over the entire length of the plug. To evaluate the precise influence of the plug height, it would be necessary to divide the plug into a number of horizontal discs and calculate the total skin friction as the sum of the skin friction in the discs.
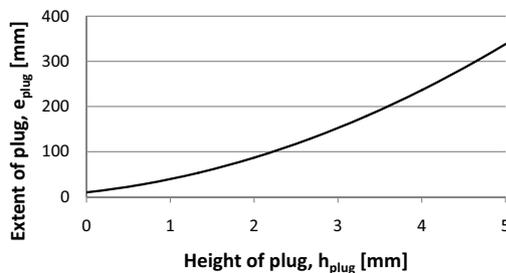
Figure 16: The section height h_{wall} as a function of the height of the soil plug h_{plug} for the studied case.

Heerema (1979) describes observed pile plugging for open-ended pipe piles driven into stiff clay. Piles were driven without an internal driving shoe, and the internal soil column were seen to be 0.9m to 3.4m below the surface outside the pipe, averaging to 2.4m. The pipe pipes were driven to a depth of $12.5 - 23.5m$.

Brucy et al. (1991) describes pile plugging in sand. Open-ended pipe piles were driven to a depth of approximately 22m. The internal soil column is measured while the pile were driven, and when the pile reaches its final penetration depth the soil column is measured to approximately 13m.

The observed and measured heights of the soil plugs described by Heerema (1979) and Brucy et al. (1991) are very large compared to the height of 1m used in these calculations. However, if the toe of the sheet pile wall is plugged, the plugged part can-

not be used when calculating the tangential earth pressure. The tangential earth pressure acts downwards for active earth pressure (on the back side of the sheet pile wall), while it acts upwards for passive earth pressure. Generally, the passive earth pressures are larger compared to the active. This implies that a larger point resistance needs to be transferred though the pile toe if the height of the plug increases.

The calculations here is based on a soil-to-steel angle of friction of $0.67 \cdot \varphi'$, which is the maximum allowable design value of δ according to EN1997-1 (European Committee for Standardization, 2007). The magnitude of the soil-to-steel angle of friction is of great importance for the size of a plug. It is therefore of great interest to perform triaxial rod shear tests or direct shear tests with shearing along a soil-to-steel interface.

Bea and Doyle (1975) indicate that the index of plasticity is important for the magnitude of the soil-to-steel angle of friction, and their general observation is that the magnitude of the soil-to-steel angle of friction decreases with increasing index of plasticity. However, they conducted tests on clay materials from the Gulf of Mexico with $I_P = 40 - 70\%$ and a silty clay (unknown location) with $I_P = 20 - 30\%$. It could be interesting to test the soil-to-steel angle of friction between normal Danish clays and sheet pile walls.

6 Conclusions

The plugging effect is investigated in this paper. For the studied case, it is found that it is necessary with plugs of up to 485mm to fulfil vertical equilibrium, i.e. a full plug is necessary. However, it is generally accepted to adjust the roughness of the sheet pile wall to achieve vertical equilibrium. For a sheet pile wall failing with one yield hinge, the roughness can be increased to 1, which lowers the needed plug size to 250.8mm.

A theoretical expression for soil plugging

is suggested, where equilibrium of the soil plug must be fulfilled. The equilibrium prescribes that a plug occurs if the skin friction mobilised between the soil and the wall in the plugged area is larger than the static end bearing capacity acting under the plugged zone. The skin friction is reduced for the weight of the soil in the plug and vertically above the plug.

Four expressions for calculating the skin friction in the soil-to-steel interface are assessed. Two expressions fail to fulfil the requirement from EN1997-1, which prescribes that the mobilised skin friction should be determined from the soil-to-steel angle of friction δ.

The remaining two expressions are compared with Aalborg Clay as the case material. The two expressions provide results close to one another. However, a larger plug forms if the skin friction is calculated by Coulomb's failure criterion. Using the limitation from EN1997-1 that δ may not exceed $0.67 \cdot \varphi'$ provides that a plug of $30.9mm$ and $40.2mm$ can form for the two calculation methods for the described dimensions of the sheet pile.

The theoretical model fails to fulfil one boundary condition; when the height of the cross section is chosen to zero, the model predicts a plug. This is not expected as a section height of zero implies a straight section. However, in the normal range of used section heights for sheet pile walls, a higher mobilised soil-to-steel angle of frictions is needed to form a plug for increasing heights. This fact corresponds to what is expected.

The theoretical model provides very short plugs compared to what is needed to fulfil vertical equilibrium. The extent of the mobilised plug is very dependent on the height of the plug, and the mobilised plug is found to increase with a polynomial for increasing height of the plug.

It has not been possible to investigate the height of plugs forming in sheet pile walls. A high plug provides a large extents of the plug but the plugged zone cannot be used to transfer tangential earth pressures. This implies that the point load at the pile toe might increase and larger plugs are needed. These effects are not investigated further.

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