Earth Pressure from the Nearby Buildings on Sheet Pile Walls

Pression des sols provenant des bâtiments voisins sur les murs de palplanches

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ABSTRACT

When a sheet pile wall is installed, a robust design is necessary to maintain the integrity of the structures at vicinity. This is accomplished by designing the wall in order to support all the pressures transmitted through the soil, including the loads on the ground surface behind the wall. The rugged design represents an inherent resistance against bending and calls for a precise quantification of the stresses on the wall. Besides soil and water pressures, the loads from the building foundations contribute with an extra pressure. For the ultimate limit state analyses, this pressure is often estimated approximately by very simple means. The paper deals with the evaluation and quantification of this pressure. The wall considered is unanchored with a strip load on the ground surface behind the wall. The problem is solved by means of Coulomb's method compared with solutions by finite element modelling applied to a number of representative examples considering different strengths for the cohesion-less soil and different load scenarios. After a discussion of the results calculation procedures are proposed.

RÉSUMÉ

Quand un mur de palplanches est installé, une conception robuste est nécessaire pour maintenir l'intégrité des structures à proximité. Cela se traduit en concevant un mur de palplanches qui inclut le soutien de toutes les pressions transmises par le sol, y compris les charges au sol derrière le mur. Une conception robuste doit définir la résistance inhérente au ploiement, et nécéssite une quantification précise des contraintes sur le mur. Outre les pressions du sol et de l'eau, la charge des fondations des bâtiments voisins contribue à une pression supplémentaire. Pour les analyses d'état limite ultime (ELU), cette pression est souvent estimée approximativement par des moyens très simples. Ce document traite de l'évaluation et la quantification de cette pression. Le mur est considéré comme sans ancrage avec une bande de charge au sol derrière le mur. Le problème est résolu par la méthode de Coulomb que l'on compare avec les solutions de calculs sur éléments finis appliquées sur un certain nombre d'exemples représentatifs, en tenant compte des forces différentes pour la cohésion du sol et des scénarios différents de charges. Des procédures de calcul sont proposées après une discussion des résultats.

Keywords: Sheet pile wall, continuous footing, earth pressure, excavation, Coulomb's method, finite element method, theory of plasticity, sand, stress distribution, principle of supperposition.

1 INTRODUCTION

Driven sheet pile walls play an important role in many ways, both to overcome topological differences and allow for excavations near existing buildings. When the wall is driven in a layer of hard soil overlain by a layer of cohesion-less soil a robust design is necessary to maintain the integrity. This means that a substantial resistance against bending is available to resist the pressure on the back side of the wall. This pressure may partly caused by a surface load from trafficking or from foundations of nearby buildings.

The influence on the wall pressure from especially shallow footings is often difficult to assess in practice and crude estimates are often used in lieu of more precise methods.

The paper describes methods to calculate the extra earth pressure on the wall from a strip loading or continuous footing behind the wall. This means that the problem is only two dimensional (2D). Different aspects in connection with loads behind walls are mentioned in this paper. However, focus is made on a free (unanchored) wall where the top of the wall moves into the excavation during rupture.

The results can be used in connection with any method to calculate the ordinary wall pressure but is presented in the paper with the Danish sheet pile wall design method as an example. The reason for this is that the method is based on actual rupture figures in the soil with respect to the predicted movements of the wall.

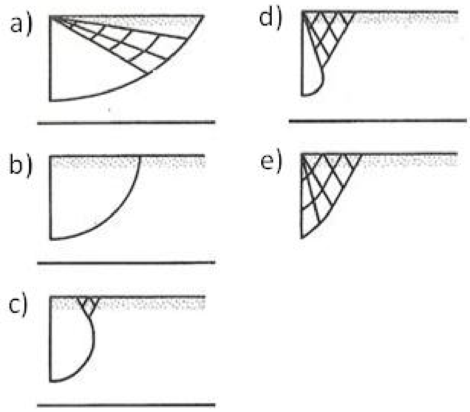

Figure 1 Rupture figures with different rotation points. Rough and rigid wall that rotates anticlockwise ($\varphi = 30°$, $c = 0$). Type (e) with the rotation point below the tip of the wall is investigated in this paper.

In this method the principle of superposition is used. As a consequence of this the additional pressure from the continuous footing load can be calculated separately and subsequently be added to the pressure from the self weight of the soil, the influence of a ground water table and cohesion of the soil if any.

An often used conventional method to calculate the additional pressure from a strip loading based on simple straight rupture lines is presented and few examples of the application are shown. The results are compared with finite element (FE) calculations carried out with the computer program Plaxis. Finally, a simple approximation to the additional pressure is proposed.

2 EARTH PRESSURE CALCULATION

The earth pressure calculation on a wall is illustrated by the Danish method denoted as EPC. This method has been proposed by J. B. Hansen [1] and used in Denmark for half a century. The pressure on the wall (e) is calculated as a sum of three terms as given in equation (1).

$$e = \gamma' d K_\gamma + c K_c + p K_p \qquad (1)$$

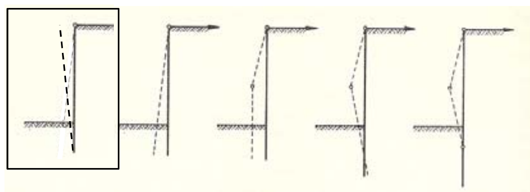

Figure 2 A wall in failure composed of one or more rigid segments connected by yield hinges in failure. This paper deals with the one to the left (the free wall).

Those terms and other parameters used in the calculation are: γ' the effective unit weight of the soil; K the earth pressure coefficient (different for the three terms); c the cohesion of the soil; p the surface load behind the wall; d the depth along the wall from the soil surface;

In the Danish method the wall is considered composed of several rigid parts interconnected by yield hinges. Each part is assumed to rotate about a point and the earth pressure coefficients are functions of the position of this point and the direction of rotation (besides the friction angle of the soil, φ). A few examples of rupture figures used for calculation of K are shown in figure 1. Examples of walls with yield hinges are shown in figure 2. The result of each calculation is the total force on the wall and the point of application. The normal component of this force (E) is distributed along the wall in a way to obtain a safe design. E.g. if the upper part of a wall (above an anchor level) moves against the soil in failure large part of E is applied near the top corresponding to a passive Prandtl rupture zone. A pressure jump near the top is often assumed to ensure that the effect of the distribution (in terms of total force and moment) is equal with the results from the calculations of the rupture figure. The method has been described by Mortensen & Steenfelt [2] and results of calculated examples are compared with FE calculations.

3 COMPUTER PROGRAM 'SPOOKS'

Although J. B. Hansen has made a complete set of diagrams to find the values of K, the earth pressure calculation for a specific design situation is rather time consuming. To this end GEO-Danish Geotechnical Institute has made a commercially available computer program named 'SPOOKS' to overcome this problem.

Here, apart from the geometry of the excavation, the soil conditions and water tables, only a selection of the total wall movements (as shown in figure 2) is necessary as input. The results are a distribution of both earth and water pressures, curves of bending moments along the wall, tip level, and anchor force (if any). All together ready for the final selection of the sheet pile profile and anchor.

3.1 *Partial surface load*

The problem when a surface load is present starting at a certain distance from the top of the wall can be calculated by 'SPOOKS'. This is if the load is active at an infinite width, which means that b in figure 3 continuous to infinity. This is incorporated by applying the full surface load at a certain depth beneath the soil surface, and a facility to do this is included in the program. This depth corresponds to the distance from top of the sheet pile wall to the edge of the applied load and is indicated with a in figure 3. If b is limited, different approximations of the additional pressure can be only manually inserted.

For an anchored wall the rotation during rupture is often about the height of the anchor. The rupture is then assumed to be a circular line rupture (figure 1b). A complete set of formulas has been presented by Steenfelt & Hansen [3] to solve this problem. In case of a low rotation point of the wall simple approximate formulas can be used as shown in this paper.

4 COULOMB'S EXTREME METHOD

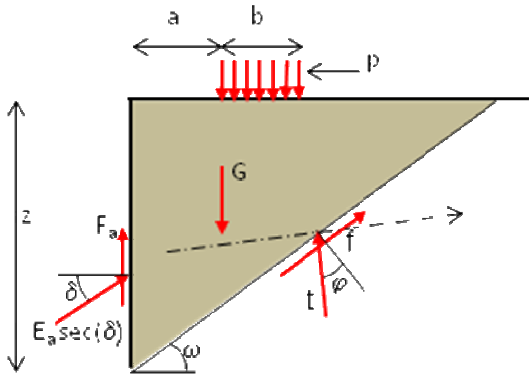

Figure 3 Coulomb's Method.

Coulomb's extreme method was presented as early as in 1776 [4]. The principle is that straight rupture lines are used to confine a rigid sliding body. This method can be used as an indication of the influence of a partial surface loading on a wall. The method will be outlined in the following as it is a serious candidate to a solution of the problem.

In figure 3 the method is outlined for a vertical wall and a partially loaded horizontal soil surface. The geometry appears from the figure and G is the weight of the shaded body, t is the total force from friction on the rupture line, f is the total shear force from cohesion of the soil. The outer support on the wall consists of a normal force E_a and a shear force F_a. The latter consists of the effect of a wall adhesion (a_h). The frictional roughness is described by a wall friction angle (δ). As the problem is 2D all forces (single arrows in the figure) have units as force/length whereas the load has the dimension force/length^2.

The principle is now that the forces and the load are projected on a line perpendicular to t (the stipulated arrow) and equilibrium is required. This means that the value of t vanishes. With a given value of ω the force E_a can be determined as E_a (ω). The value of ω is now varied and max$E_a(\omega)$ found as the necessary pressure to maintain equilibrium. The figure is made corresponding to a sliding movement to the left. This means that the results correspond to the active pressure. For the passive pressure, corresponding to sliding to the right, then $E = \min E_p(\omega)$ applies.

If this procedure is repeated for different values of z the pressure distribution can be found as $e(z) = \mathrm{d}E / \mathrm{d}z$ and only applied when e is positive.

The rupture line may not meet the soil surface in the so-called correct angle (i.e. it is not possible to construct a Mohr's circle for this point). For this reason the static conditions are not generally fulfilled for the solution. Furthermore, the straight rupture line is in most cases a crude approximation to the far more complex boundary rupture line for a more correct rupture figure (see figure 1).

It is a Danish experience that reasonable solutions are found for wall problems with active ruptures, whereas usable solutions are only found for nearly smooth walls for the passive rupture.

5 FE MODELLING AND RESULTS

In order to validate the method a number of load scenarios have been calculated by the FE program Plaxis [5].

2D mesh has been generated using triangular finite elements (15-noded). Sand is modelled in drained conditions using Mohr-Coulomb constitutive model. The hard clay below the excavation level is modelled in undrained condition using the Tresca constitutive model.

The sheet pile is modelled weightless and rigid. The model is constructed in such a way that the active pressure on the wall does not interact with the passive one. The initial geostatic conditions are calculated first. Mesh sensitivity analyses have been carried out and the optimal mesh with respect to element size and obtained accuracy has been chosen for the final analyses.

Plaxis plastic analyses (small deformation theory) and Updated Mesh (large deformation theory) have been applied. The calculations are carried out in different ways considering the impact the staged construction (excavating after, before or at the same time with the load application) has on the results.

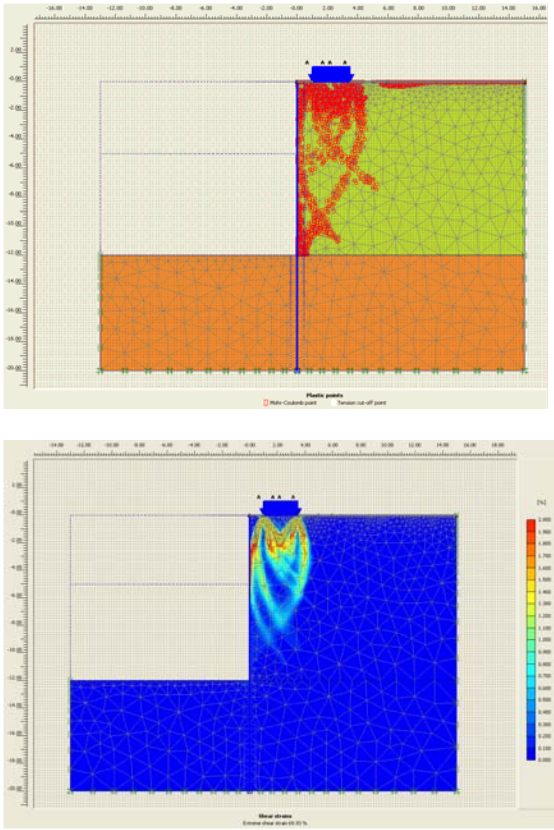

Figure 4 FE model example, (φ=30∘ a=1.0 m b=2.5m or a/b=0.4 p=125kPa). Plastic points (up) and shear strains (down).

5.1 *Load Scenarios Investigated*

Some different load scenarios are modelled and calculated to illustrate the problem. A unit weight has been applied to the soil to provide a realistic stress distribution near the top of the wall. The influence of the load on the wall has thus been derived as the difference between results of calculations of the wall with load and unit weight and with unit weight alone. As mentioned previously, the calculations have been made for the active case, which means that the wall moves away from the load about a low point of rotation. For one of the calculated load scenarios the FE model and results are shown in figure 4.

When a in figure 3 is large ($a>z$), the influence from the weight of the soil dominates and the effect from the load will vanish. An excessive height of the wall will not change the problem as long as the rotation point is low (beneath the foot of the wall). The corresponding rupture figure is the one shown in figure 1e.

Table 1. Results for φ = 30°, $N = 2p / (\gamma\, b) = 7.14$

a (m)		1.0	1.0	2.5	5.0
a / b		0.4	1.0	2.5	5.0
p (kPa)		125	50	50	50
	Cou*	0.47	0.44	0.40	0.37
$E_a/(p\ b)$	FEM	0.66	0.70	0.64	0.52
	App**	0.70	0.65	0.56	0.47
	Cou*	1.07	1.07	1.07	1.13
$d_E/(a+b)$	FEM	1.14	1.25	1.00	1.05
	App**	1.25	1.06	0.88	0.77

* Coulomb's method
** Proposed Approximation (see Section 6.2)

Table 2. Results for φ = 40°, $N = 2p / (\gamma\, b) = 40.7$

a (m)		1.0	1.0	2.5	5.0
a / b		0.4	1.0	2.5	5.0
p (kPa)		713	285	285	285
	Cou*	0.41	0.39	0.35	0.32
$E_a/(p\ b)$	FEM	0.51	0.62	0.56	0.38
	App**	0.59	0.55	0.47	0.40
	Cou*	1.51	1.48	1.39	1.37
$d_E/(a+b)$	FEM	1.00	1.50	1.10	1.00
	App**	1.25	1.06	0.88	0.77

* Coulomb's method
** Proposed Approximation (see Section 6.2)

For each set of parameters two calculations have been made: Coulomb's method and FE method. Table 1 and table 2 show results from calculations with different values of a, b, φ and p. The cohesion (and wall adhesion) is assumed to be zero ($c = a_h = 0$) and the wall completely rough ($\delta = \varphi$). The unit weight is assumed equal to γ = 14 kN/m^3 considering the presence of some water table at some depth.

A complete presentation of the different calculations carried out is not included due to lack of space. However, the FE results, consisting of additional shear force on the sheet pile wall, are summarized in figure 5 and 6.

The proposed approximations are presented in the table 1 and 2 in terms of the relative resultant normal force E_a / (p*b) and the relative point of application (d_E) measured from the top of the wall and shown as d_E / (a+b).

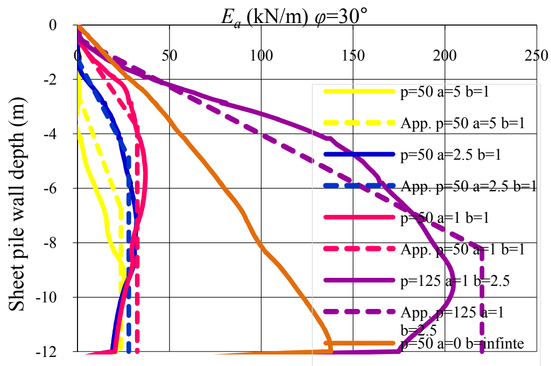

Figure 5 FE and approximated additional shear force E_a on the wall (φ=30°). See table 1 for units applied.

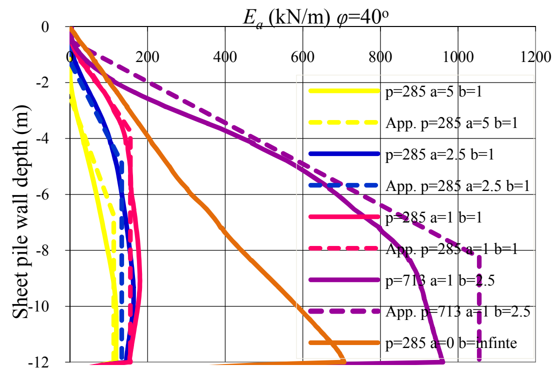

Figure 6 FE and approximated additional shear force E_a on the wall (φ=40°). See table 1 for units applied.

5.2 Proposed Approximations

A study of the results of the FE analyses reveals that the distribution of the additional stress on the wall can be approximated as a constant stress (e_a) applied at a certain depth interval:

$$e_a = F\, p\, K_p \ \text{ where } F = b \,/\, (0.25a+b)$$
$$d_1 = 0.5a$$
$$d_2 = d_1+0.25a+3b \qquad (2)$$

K_p is the earth pressure factor in equation (1). It is noticed that generally $F \leq 1$. For $a = 0$, $F = 1.0$. In the evaluation of F the value of K_p is not determined from the expression for a rough wall as K_p= (1-sin(φ)) exp((φ-0.5π)tan(φ)). Instead, by FE with a=0 and b infinite, see figure 5 and 6, in order to reduce a possible bias from the method.

6 DISCUSSION

It should be emphasized that the local bearing capacity of the soil under the load (or continuous footing) is first controlled and ensured. The wall will somehow confine the rupture figure developed under the load as shown in figure 4.

The ratio between the applied load and the unit weight has some influence on the solution. This ratio is defined as $N = 2p \,/\, (\gamma\, b)$ and indicated in table 1 and 2. With this definition N resembles N_γ from the bearing capacity formula. When choosing the different load scenarios modelled by FE, the N values were pre-calculated ensuring that the load scenarios for same φ corresponded to the same N value and the failure ratios of the all footings were equal. This was verified by the FE analyses where the loads were applied over a weightless rigid plate modelling the continuous footing.

The results are valid (in terms of the stresses p, e, etc.) for other values of a, b and γ (denoted a_1, b_1 and γ_1) if:

- The ratio $a_1 / b_1 = a / b$ and $z_1 / b_1 = z / b$
- The friction angle $\varphi_1 = \varphi$
- The forces are multiplied by a_1 / a (e.g. $E_1 = E\, a_1 / a$)
- The soil unit weight $\gamma_1 = \gamma\, a / a_1$.

However, the solution is not very sensitive to variation of the unit weight. It should also be remembered that the current analyses are based on the following assumptions: (1) cohesion-less soil, (2) active pressure and (3) rough wall.

It is observed from table 1 and 2 that the results from Coulomb's method yield less additional pressures compared to results from FE analyses. It might be possible that the current approximations apply also for a smooth wall and for the passive case. However, these cases need to be further investigated.

7 CONCLUSIONS

From the above paragraphs conclusions can be drawn:

- Coulomb's method yields less additional pressure than FE analyses.
- A simple and qualitatively correct approximation to the FE results is presented.
- This approximation should only be applied for values of parameters reasonably covered by the calculations.

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