03 Jun 1993, 10:30 am - 12:30 pm

# Behavior of a Braced Sheet Pile Wall in Soft Clay

T. Edstam

Leif Jendeby

Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 5.14

# Behavior of a Braced Sheet Pile Wall in Soft Clay

**T. Edstam**
Chalmers University of Technology, Gotherburg, Sweden

**L. Jendeby**
Chalmers University of Technology/NCC, Nordic Construction Company, Gothenburg, Sweden

SYNOPSIS

The earth pressure distribution against, and the displacement of, a braced sheet pile wall in soft clay have been examined. In connection with the construction of a fly-over, on the west cost of Sweden, two sections of a braced sheet pile wall were instrumented. The instrumentation consisted of earth pressure cells and inclinometer pipes mounted on three sheet piles. The sheet pile wall was also analyzed by means of the FLAC code in which the interaction between the sheet pile wall, the struts and the soil was studied. The movements and the calculations are compared with a conventional design method according to Peck. The results indicate that the earth pressure distribution and the deflection of the wall is strongly dependent on the construction procedure. The FLAC code was found to be a useful tool for parameter studies, but can hardly be used for design.

## INTRODUCTION

In areas with deep deposits of soft soil, deep excavations often require expensive constructions, e.g. braced sheet pile walls. The design of such a construction is often decisive for the economy of the project as a whole. Further, the design of the construction is a question of safety for the personel working down in the excavation.

Numerous computer programs are today available for the design of retaining structures. However, almost exclusively the main effort is devoted to the design of the wall itself and the struttings, while the load acting on the construction, i.e. the earth pressure, is determined in a very simplified way. This is, for example, often done according to Rankine's theory, despite that one knows that the pressure acting on the wall has a different distribution.

The knowledge within this very central geotechnical domain is, however, still limited. In 1972, Bjerrum et al stated that "The number of meetings on the subject seems at least somewhat out of proportion with the progress being made in the field", [1].

The lack of analytical models describing the problem is explained by the extreme complexity of the problem. Except that the problem is three-dimensional and requires information concerning the properties of the soil as well as the structure, it is also influenced by time and by the construction procedure at the site. The difficulty of predicting the actual stress distribution is also increasing when the construction is statically undetermined, which is the case for walls with more than two bracing levels.

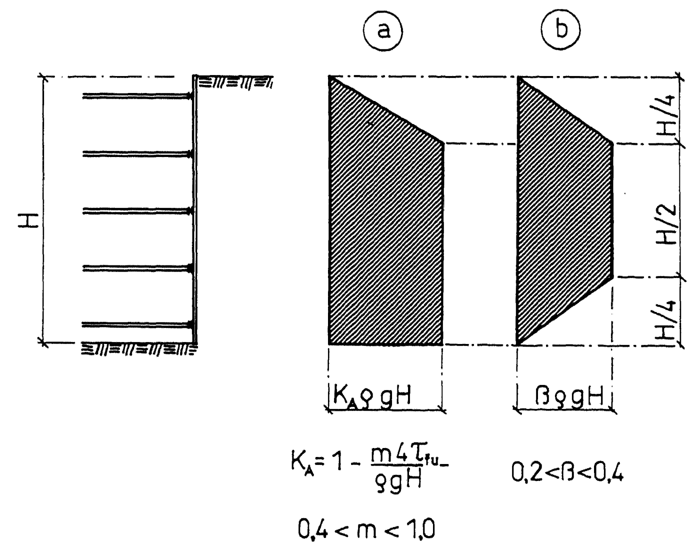
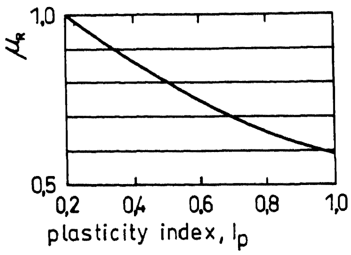



*Fig. 1 Maximum envelope for earth pressure to be used for design of supports*

*a) soft clay, $N \geq 4$ to 6*
*b) stiff clay, $N < 4$*
*$N = \rho gH/\tau_{fu}$*
*(After Peck, 1969)*

Extensive work by Peck [2] during the forties resulted in principal earth pressure distributions for strut design, see fig. 1. This is not, however, an attempt to describe the actual stress distribution, but instead an envelope of the maximum pressure that can occur at any time during the whole excavation process.

Numerous field studies in Oslo, about 30 years ago, led to further understanding of the problem. Especially the influence of the stability of the excavation as a whole was examined, and thereby the true strength properties of the soil became important parameters,[1]. A model that accounts for anisotropy as well as time effects was presented, see fig. 2.




| | $\mu_A$ active | passive |
|---|---|---|
| soft clay | | |
| $I_p$ <0,5 | 1,5 | 0,5 |
| $I_p$ < 0,5 | 1,5 | 0,8-1,0 |

*Fig. 2 Correction of undrained shear strength due to rate of loading and anisotropy*

$\tau_{fu} = \mu_A \mu_R \tau^v_{fu}$

$\tau_{fu}$ = *undrained shear strength from field vane test*

$\mu_R$ = *correction factor for rate of loading*

$\mu_A$ = *correction factor for anisotropy*

*(After Bjerrum et al, 1972)*

Except the properties of the soil, there is two factors governing the magnitude and the distribution of the earth pressure acting on the wall, namely the deflection of the wall, and the geometry of the excavation (which influences the overall stability).

A sheet pile wall driven into soft clay will rather soon after installation be exposed to earth pressures close to the at-rest conditions, which corresponds to a $K_0$-value of about 2/3. As the excavation then progresses, the wall will move towards the excavation leading to decreased earth pressure against the backside of the wall, which in turn means that the shear stresses within the soil increase. If the deflections get large enough, the shear stresses will reach the shear strength,and the earth pressure will then decrease to its minimum value, which is normally called the active earth pressure (Rankine). However, due to a limited deflection, variations in soil or construction stiffnesses, or limited stability, the earth pressure distribution will differ from the Rankine distribution. Such phenomena are shown in fig. 3.

When excavation is carried out from level I to level II, the earth pressures will be changed due to two principal effects;

i) the earth pressure on the "passive side" caused by the soil between levels I and II is removed, bringing the wall to deflect towards the excavation. The pressure on the "active side" at the same level will then decrease due to the wall deflection. B arching the stresses then will increas above level I, and below level II.

ii) since the excavation of the soil will de crease the vertical stresses below the bas of the excavation, the horizontal stresse between levels II and III will also de crease. This, in turn, leads to a movemen of the soil below level II to the left. Th earth pressure on the active side wil therefore be redistributed by means of arch ing, and in this case it will be transferre partly to the lowest strut, partly to th stiffer strata at level III.

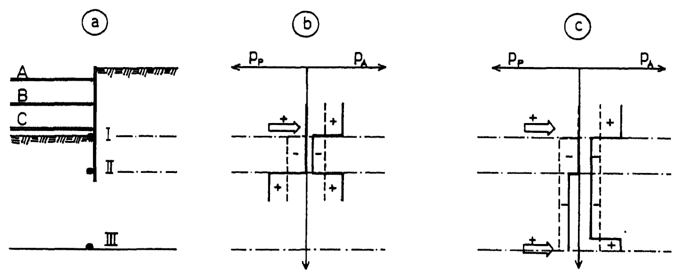



*Fig. 3 Arching effects due to excavation*

*a cross section*
*b schematic change of earth pressur due to the disapperance of eart pressure between I and II on th excavation side.*
*c schematic change of earth pressur due to decrease in vertical stresse on the excavation side.*
*(After Bjerrum et al, 1972)*

In order to study those effects, and to get a overall understanding of the load-displacemen relations for braced sheet pile walls in sof clay, three instrumented full scale sheet pile were manufactured. The instrumentation made i possible to measure earth pressure against th "active side" of the piles, and the deflectic of the piles above as well as below the dredg level.

The results of these measurements are presente in the following, and the problem is also ana lyzed by means of the FLAC code [3].

**FIELD STUDY**

The three instrumented sheet piles were drive in a soft clay deposit where a six meter deep 19 m wide, and more than 40 m long excavatic was to be carried out. A cross section of th excavation is shown in fig. 4.

Since the instrumented sheet piles were place close to the center of the wall, two-dimensiona conditions can be assumed. The subsoil consist of soft marine clay overlying a thin layer o frictional material, which in turn rests o rock. The thickness of the clay layer is ap proximately 15 m. The properties of the clay i shown in fig. 5.

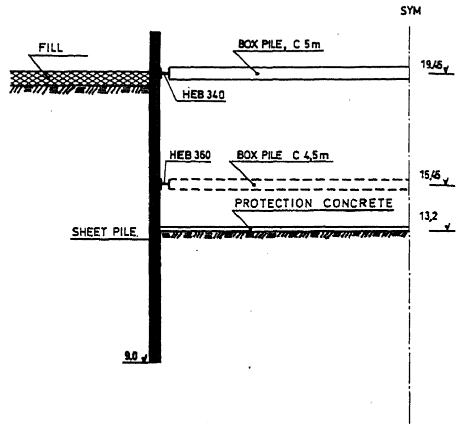



*Fig. 4 Cross section of the excavation*

The clay can be considered normally consolidated. The oedometer modulus for stresses below the preconsolidation pressure (CRS tests), varies from about 1000 kPa at ground level, to 3500 kPa in the lower part. Unloading-reloading tests indicate a modulus approximately 2-3 times as high (average from unloading-reloading).

The three instrumented sheet piles were equipped with earth pressure cells and inclinometer tubes, fig. 6. The earth pressure cells were of Glötzl type (E17KF20). At sheet pile 1 and 2 the cells measured the pressure against the "crest of wave", while the pressure against the "wave trough" was registered on pile 3.

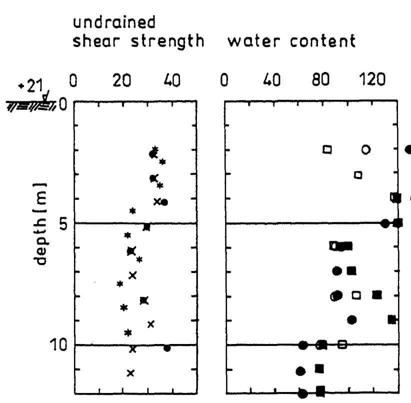



*Fig. 5 Geotechnical parameters of the subsoil at test site*
*left diagram*
- ● *fall cone test, Borehole 35*
- x *field vane test, Borehole 35*
- * *field vane test, Borehole 1*

*right diagram*
- ▽ *natural water content, Borehole 35*
- o *liquid limit, Borehole 35*
- ▼ *natural water content, Borehole 1*
- ● *liquid limit, Borehole 1*

## RESULTS OF FIELD MEASUREMENTS

Readings of the earth pressure were taken at totally 33 occasions; before, during and after the excavation work. The displacements of the sheet pile wall were measured at 15 occasions. The observations continued for a period of 107 days.

The readings indicate that the initial horizontal stresses increased linearly with depth, and corresponded to a $K_0$-value in the order of 0.7, see fig. 7. One can also observe a lower pressure against pile 3 compared to piles 1 and 2. This is due to the difference in location of the pressure cells, cf. fig. 6.

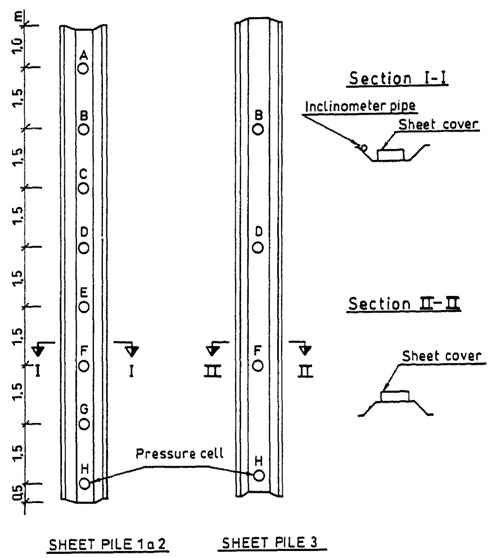



*Fig. 6 The instrumented sheet piles (PU12)*

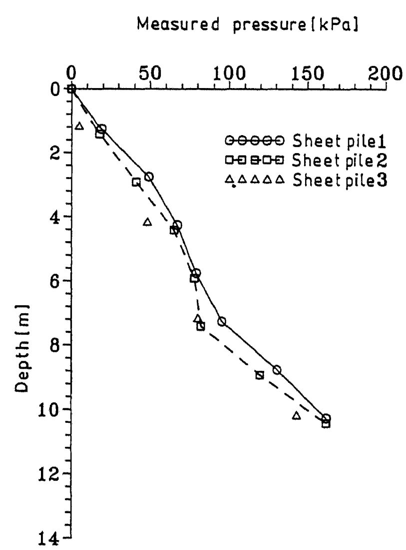



*Fig. 7 Measured initial horizontal earth pressure against the sheet piles*

When the excavation started the sheet piles moved horizontally, resulting in a decrease of the earth pressure against the sheet piles.

As long as the sheet piles were unsupported, the displacement decreased with depth, while the earth pressure increased roughly linearly with depth.

However, the insertion of the struts prevented further displacement at the strut levels. The subsequent excavation therefore resulted in a different shape of the displacement curve. At the same time the total "active force" decreased and a redistribution of the earth pressure was also observed.

At the completion of the excavation the earth pressure on the active side showed both an obvious decrease and a redistribution, fig. 8. The pressure appears to have been transferred from the parts below the dredge level towards the upper parts and to the tip level.

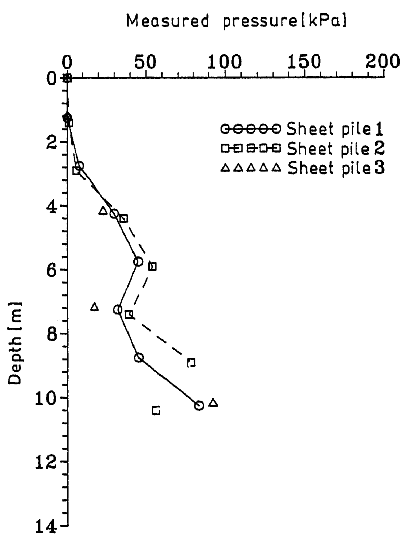



*Fig. 8 Measured horizontal earth pressure against the sheet piles at the end of excavation*

The horizontal displacements were almost zero at the pile tips and varied roughly linearly below the dredge level. Above the dredge level the pile wall showed an obvious curvature due to the struts, see fig. 9.

During the period after the completion of the excavation, only marginal displacements of the sheet piles were observed (at the lower parts). Neither did the earth pressures change much.

**NUMERICAL SIMULATIONS BY MEANS OF FLAC**

The FLAC code [3] was used in order to simulate the behaviour of the sheet pile wall during the excavation. The code can handle two dimensional problems and makes it possible to study the interaction between the sheet pile wall and the soil.

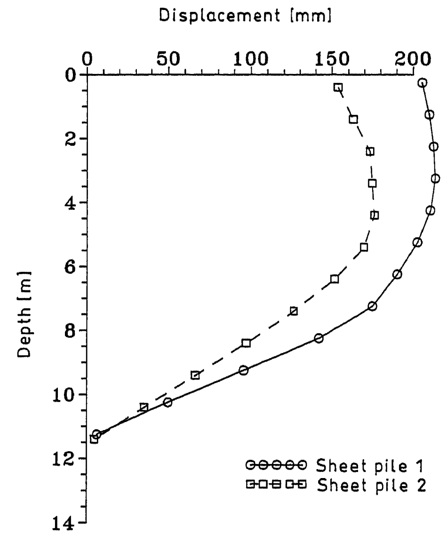



*Fig. 9 Measured horizontal displacement c the sheet piles at the end of excava tion*

In this case the soil was modeled as an isotrc pic, elasto-plastic material (no plastic harder ing; Mohr's yield criterion), while the shee pile wall and the struts were modeled as elasti beams.

Undrained conditions were assumed for the ana lysis bringing that the soil was described k means of the undrained shear strength, $\tau_{fu}$, ar Poisson's ratio, $\nu$, approaching 1/2. The shea modulus was determined according to (Larsson Mulabdic [4]).

$$G_o = 504\ \tau_{fu}/w_L \quad ; \quad w_L = \text{liquid limit}$$

However, this relation is just valid for th initial shear modulus. Hence, the modulus ok tained in this way has been reduced by a factc of 5 to 10 before it was used in the simulatior

Totally 6 simulations were performed, and th soil parameters used are given in table 1.

Table 1. Input variables in FLAC simulation

| Simulation No | G [kPa] | $\nu$ | Relative axial stiff ness for lower strut |
|---|---|---|---|
| 1 | $G_o/10$ | 0.45 | 1 |
| 2 | $G_o/10$ | 0.45 | 1 |
| 3 | $G_o/10$ | 0.33 | 1 |
| 4 | $G_o/5$ | 0.45 | 1 |
| 5 | $G_o/10$ | 0.45 | 10 |
| EL | $G_o/10$ | 0.45 | 1 |

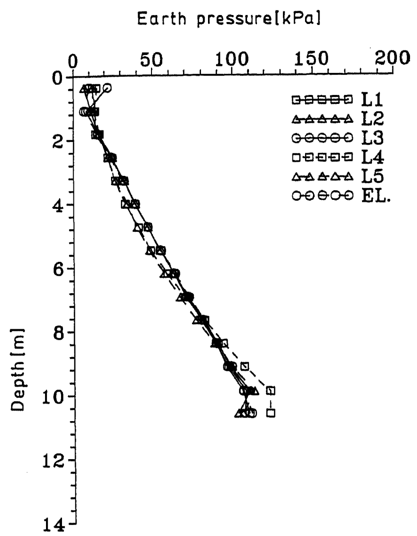



*Fig. 10 Horizontal earth pressure against the sheet pile wall at the end of excavation according to FLAC*

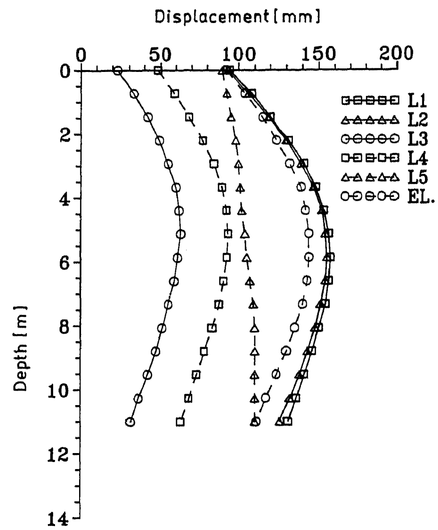



*Fig. 11 Displacement of the sheet pile wall according to FLAC at the end of excavation.*

The calculated earth pressures against the sheet pile wall were found to be rather insensitive with respect to the parameters varied, and was in all cases found to increase linearly with depth, see fig. 10. Neither did the assumtpion of an elastic soil result in any significant change of the earth pressure.

The calculated <u>displacement</u> showed strong dependence of the deformation characteristics of the soil and the struts, cf fig. 11. However, the <u>curvature</u> of the sheet pile varied only marginally with the soil properties. A decrease of Poissons ratio, or an increase of the shear modulus, resulted in a decrease of the horizontal displacement. It would also be seen that unlimited shear strength, i.e. elastic soil, only decreased the displacement slightly.

The calculations also showed that the soil had plastified in a zone near to, and below the tip of the sheet pile wall. However, the extent of the zone was limited and had little effect on the results.

**COMPARISON OF EARTH PRESSURES OBTAINED FROM FLAC ANALYSIS, FIELD TEST AND CONVENTIONAL ANALYSIS**

A comparison between the FLAC-results and the measurements clearly shows that FLAC did not simulate the actual earth pressure distribution. Neither did an earth pressure distribution obtained by Rankine theory agree with the actual distribution. It is, however, interesting to compare the resulting "active forces" calculated from the three distributions, see table 2.

From this it can be concluded that the FLAC-simulations overestimate the "active force" by approximately 80%. When calculating the Rankine pressure, a correction of the shear strength may be appropriate in order to take account of time effects and anisotropy, cf. fig. 2. The calculated Rankine force will underestimate the "active force" 10% if the shear strength is only corrected for time effects. Furthermore, if a

Table 2. "Active" force [kN/m] against the sheet piles

| $F_{measured}$ | $F_{FLAC}$ | $F_{Rankine}$ time corr. | $F_{Rankine}$ time + anisotropy corr. |
|---|---|---|---|
| -308 | -550 | -442 | -276 |

correction is done also for anisotropy the "active force" will be overestimated by 40%.

Even though the pressure on the active side is of great importance, it is the resulting net pressure which is actually needed for the design of struts and sheet piles. Since no measurements were made of the pressure on the passive side it is not possible to determine the actual net pressure. However, considering the deflection curve of the sheet piles, it may be concluded that the net pressure below the dredge is almost zero.

Thus, the "measured" net pressure can be compared with the net pressure according to FLAC and the "apparent net pressure" according to Peck (cf fig. 3), fig. 12.

From this figure it can be concluded:

* The resulting force according to Peck varies from 10% underestimation to 80% overestimation, in comparsion with the measured value. The distribution is also totally different from the measured.

* The resulting force according to FLAC overestimates the actual force in the order of 8 to 23%. However, the distribution show some resemblance with the measured distribution.

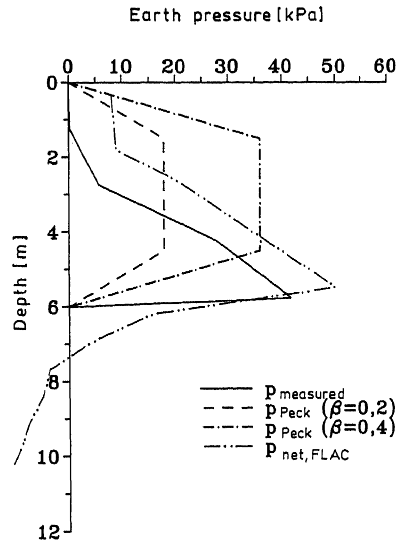



*Fig. 12 Calculated and measured values of net earth pressures. Except for the FLAC-analysis, the net pressure below dredge level is assumed to be zero.*

**COMPARSION BETWEEN MEASURED EARTH PRESSURES AND DISPLACEMENTS**

In order to preserve the stability during the operation stage, the excavation was carried out in stages. Consequently, the struts were placed successively during the excavation. The excavation was also carried out in two benches, so that the upper struts were placed during the first, while temporary struts were used at the lower level (until the protection concrete hardened).

It was found that most of the wall displacement occurred during the time lag between excavation of one stage, and placing of the corresponding strut. The displacement of the wall was therefore rather large, while the deflection (curvature) was more moderate. The displacement of the wall becomes thereby very hard to predict, since it depends on the extension of the excavation stages, as well as the time interval between excavation and bracing. An important factor is also the accuracy with which the struts are cut out.

If looking at the sheet pile wall as a loaded beam, the above described phenomena is to be compared to a displacement of the supports. It can therefore be expected that the deflection of the wall compares with the applied load, i.e. the earth pressure. Most of the actual load is known since the earth pressure on the "active side" is measured. The earth pressure on the "passive side" is, however, unknown.

Calculations have therefore been carried out with the measured pressures on the active side, and with different assumptions concerning the earth pressure on the passive side. Furthermore different assumptions concerning the bending resistance of the sheet piles were used in the analysis.

The results of these calculations are shown i fig 13, which also shows registered deflectio (the discrepancy from a straight line drawn fro top to bottom of the sheet pile).

One can see that two of the assumptions giv fairly good correspondence between measured an calculated deflection, viz;

i if the passive earth pressure is calculate based a shear strength reduced in accordanc with Bjerrum (fig. 2), and if the bendin resistance of the sheet pile wall is assume to be 0.5 times the nominal value.

ii if the passive earth pressure is calculate based an average of reduced and unreduce shear strength, and if the nominal value o the bending resistance is assumed.

The latter is however less likely, since thi solution gives tension load in the lower struts which cannot be carried by the protection concrete.

That the net load below excavation is limited i also supported by the fact that the sheet pile are almost perfectly straight below the excavation level.

Consequently, it is likely that the actual bending resistance of the sheet pile is less tha the nominal value. This is explained by the limited possibility to transfer shear forces a the interlockings of the sheet piles. A decreas of bending resistance with 50% means, however, that no shear forces at all are transferred.

**CONCLUDING REMARKS**

From the project as a whole, the following conclusions can be drawn;

- the earth pressure against "crest of wave" is about 10-15 kPa higher than those actin on "wave trough"
- the registered coefficient of earth pressur at rest, $K_o$, was 0.67-0.77
- the pile top moved 15-20 cm, while the pile tip did not move at all. The top displacement corresponds to 1.4-1.9% of the pile length
- the sum of the measured earth pressures was close to what is obtained from active Rankine earth pressure. The earth pressure above the excavation level was, however, higher than active pressure, while the pressures below this level were less than the active pressure
- the wall displacements mainly occur during the time lag between excavation and struttting. Decisive for the magnitude of the displacements is thereby what displacements are allowed before the struts are put in place. The wall displacements are therefore also very hard to predict.

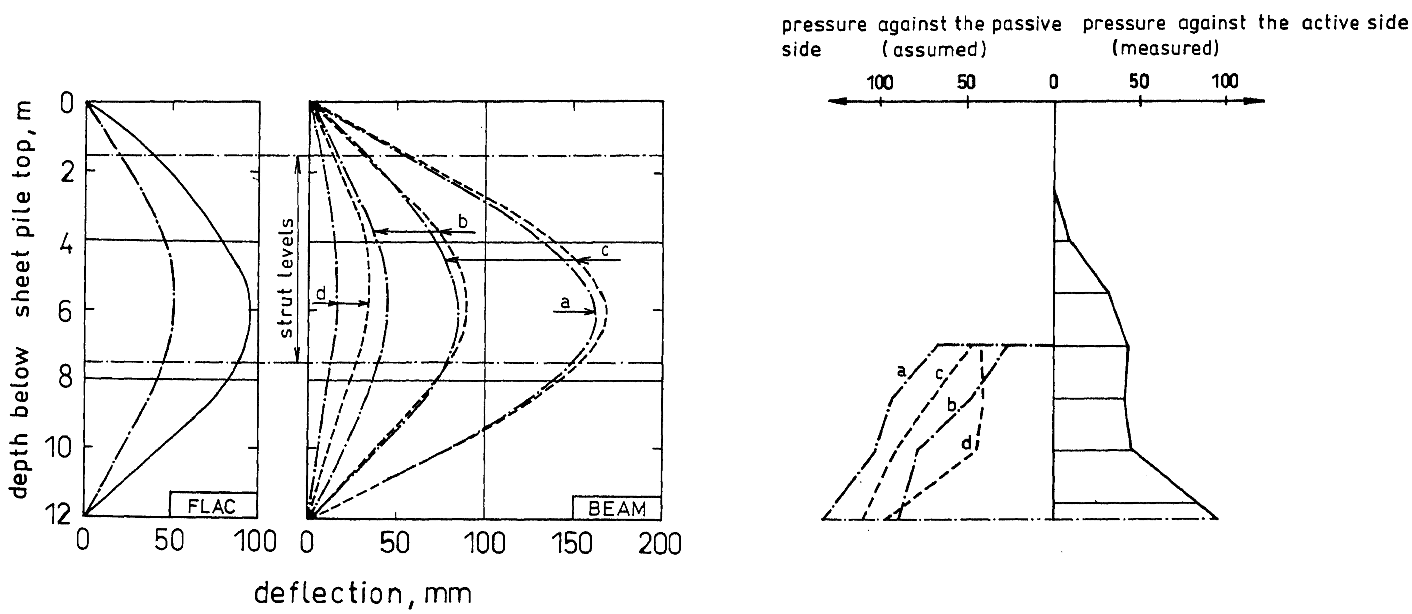



*Fig. 13 Measured and calculated deflection of sheet pile wall (excavation to full depth)*

*a. measured deflection, and deflection obtained from FLAC-analysis*

*——— measured*
*—·—— FLAC (case L2)*

*b. deflection calculated according to theory of beams, by using measured pressure on the active side, and assumed pressure on the passive side, (cf fig. c)*

*——·—— bending stiffness assumed as nominal value*
*------ bending stiffness assumed as half the nominal value*
*a passive earth pressure assumed as curve a (fig. c)*
*b passive earth pressure assumed as curve b (fig. c)*
*c passive earth pressure assumed as curve c (fig. c)*
*d passive earth pressure assumed as curve d (fig. c)*

*c. pressure distributions used in the analysis according to theory of beams*

*a passive earth pressure according to Rankine, and assuming unreduced shear strength*
*b passive earth pressure according to Bjerrum et al, cf fig. 1*
*c passive earth pressure assumed as average between a and b*
*d passive earth pressure assumed equal to measured pressure on the active side (i.e. $p_{net}$ = 0 below excavation)*

- FLAC analysis gives a good over all picture of the principal displacements pattern, but do not predict the displacement in detail. The earth pressures obtained from FLAC overestimates the actual case by more than 60%.

- the earth pressure calculated by means of FLAC is just marginally dependent on the soil modulus, Poisson's ratio, and the stiffness of the struts.

- the displacements calculated by means of FLAC are strongly dependent on soil modulus and Poisson's ratio.

## REFERENCES


[1] Bjerrum, L., Frimann Claussen, C.J., Duncan, J.M. 1972. Earth pressure on flexible structures. Norwegian Geotechnical Institute, Publication No 91. Oslo.

[2] Peck, R.P. 1969. Deep excavation and tunneling in soft ground. State-of-the-art volume. VIIth ICSMFE,pp 225-290. Mexico.

[3] ITASCA Consulting Group Inc. 1991. Users manual FLAC version 3.0. Minneapolis.

[4] Larsson, R., Mulabdic, M. 1991. Shear moduli in Scandinavian clays. Swedish Geotechnical Institute. Report No 40. Linköping.

[5] Edstam, T., Jendeby, L. 1992. Sheet pile wall at an excavation in soft clay (in Swedish). Department of Geotechnical Engineering, Chalmers University of Technology, Gothenburg, Sweden.


## ACKNOWLEDGEMENT


This project has been made possible by grants from the following;

Hercules Grundläggning
NCC, Nordic Construction Company
Development Fund of the Swedish Construction Industry
Swedish Board of Building Research