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OBSERVED AND PREDICTED RESPONSE OF A BRACED EXCAVATION IN SOFT TO MEDIUM CLAY,

Observed and predicted response of a braced excavation in soft to medium clay

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A test section was established next to a 5.5 m braced excavation in made ground consisting mainly of partially weathered clay elements. A sewer trunk line had to be installed at Pietrafitta (Central Italy), where the Italian Electricity Board (ENEL) is building a new thermo-electric plant. Surface ground movements, strut loads, sheet-pile deflections and strains were measured. The relationships between the in situ observations and the construction sequences showed the strong influence of the construction activities on the sheet-pile performance. The measured ground and bracing system responses are compared with results from standard design procedures and finite element analysis. The empirical relationships by Peck (1969) and the method by Mana and Clough (1981) were used to predict the ground response, while the free earth support method and the Peck (1969) earth pressure envelopes were used to evaluate the loads in the bracing system. A plane strain, total stress finite element analysis was carried out by modelling the excavation geometry, the construction sequences and the soil-sheet pile interface. The FEM capability of accounting for the construction procedures provides a much closer evaluation of the deflection profile, if compared with the predictions obtained by the classical design methods. The ground-surface settlements are matched better by the empirical relationships than by the finite element method, probably because of soil non-linearity or strain localisation phenomena not accounted for in the analysis. The standard design analyses appear to underestimate the loads measured in the bracing system, while a better prediction is obtained through finite elements, in spite of the simple soil model selected.

Introduction

The design of a braced excavation in soft clay requires the evaluation of loads in the bracing system and the prediction of the ground movements associated with the excavation process. The former, in order to satisfy the required safety factors against structural failure, the latter to evaluate the excavation effects on nearby structures or existing underground services.

Different methods of analysis are available, standard design procedures and finite element analyses being the most widely used. The standard procedures consider a condition of limit equilibrium for an idealised mechanism of soil–structure interaction, or make use of empirical relationships based on the back-analyses of a large number of documented case histories; in this case, soil movements or earth pressures are related to the main variables of the problem such as excavation geometry, soil characteristics, stiffness of the bracing system, etc. Apart from the classical works by Peck (1969) and D'Appollonia (1971), Mana and Clough (1981) recently proposed a simple method for predicting the ground movements associated with braced excavations in soft to medium clays, by comparing field data and FE parametric studies. They provide non-dimensional charts for estimating magnitudes and distributions of wall movements and surface settlements as a function of the factor of safety against basal heave (as defined by Terzaghi (1943)), soil and bracing system stiffness, struts preload and excavation geometry.
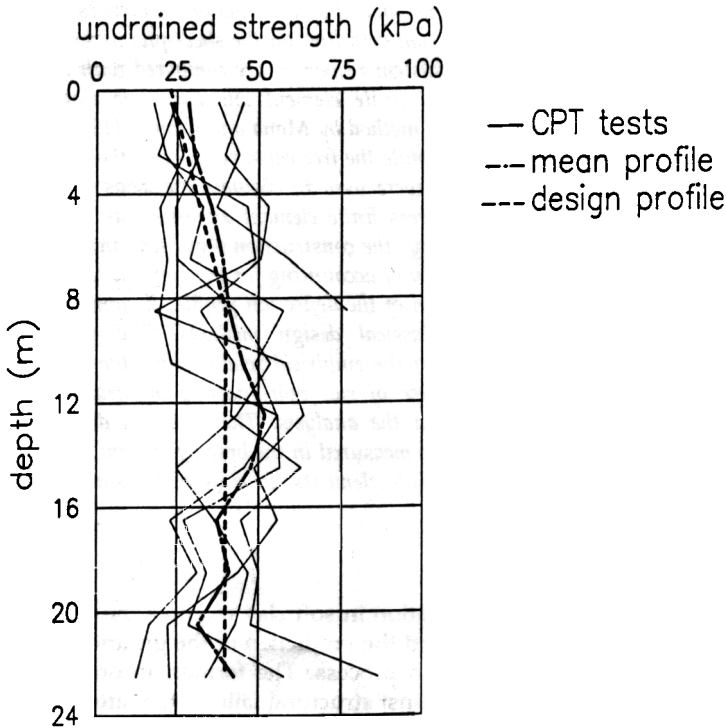

Fig. *Profile of the undrained strength from the CPT results*

The finite element method has been used extensively since the early 'seventies to study the behaviour of braced excavations. Due to its ability to model the construction procedures, the FE analysis has been successfully used to evaluate their influence on the response of the ground and the bracing system. Several comparisons of observed performances and finite element analyses have been published (e.g. Palmer and Kenney, 1971; Clough and Mana, 1976; Mana, 1978; Borja, 1990; Finno and Harahap, 1991; Finno et al., 1991). In the last decade finite element analyses have become increasingly widespread in the design practice, due to the rapid increase in computer power and the consequent diffusion of efficient and versatile FE packages on desktop computers.

The analysis of the braced excavation presented herein was performed using both standard procedures and the finite element method. Among the classical methods, the empirical earth pressure diagrams proposed by Peck (1969) and the free earth support method (FESM) have been selected to evaluate the strut loads and the sheet-pile bending moments. The ground-surface settlements and the sheet-pile deflections were estimated using the method by Mana and Clough (1981) and the normalised surface settlement profiles given by Peck (1969). The CRISP package developed at Cambridge University (Britto and Gunn, 1987) was used for the finite element analysis. The comparison of the predicted and observed response of the case-history to hand allowed for an assessment of the predictive capability, the advantages and the shortcomings of the above-mentioned methods of analysis.

Site description

The excavation was performed in made ground 20 to 25 m thick, mainly consisting of soft to medium clay. The natural clay, originally overlying a lignite layer, was excavated in the sixties for the exploitation of a coal-mine. Once the exploitation activities were concluded, about 15 m of the excavated clay were re-placed in situ in 1964–65, while the remainder was re-placed 10 years later. Although the silty–clayey fraction of the made ground is predominant, it contains inclusions of silty-sand and lignite. The soil heterogeneity is caused by the irregular mixing of the excavated soil, and, according to the activities of the mine exploitation, by the different influences of swelling and weathering processes on the excavated clayey elements which are a few decimetres of average size.

The effects of the above-mentioned mine activities are evident in the soil properties. The unit weight is 16.8 to 20.9 kN/m^3, the mean value being 18.4 kN/m^3; an average water content of 38.5% was measured, the coefficient of variation being as high as 36%. A high scattering

characterised the measurements of the undrained strength c_u, the coefficient of variation being 42 to 44% for both laboratory and in situ tests. The undrained strength measured on 54 TX-UU tests was within the range of 10 to 110 kPa, the mode being 30–35 kPa. The high scatter and the lowest c_u values may be attributed to occurred contacts, at the sample scale, between the excavated clay elements. Due to the partial swelling and weathering of the element surfaces, a sort of welding occurs between the clay elements, still undisturbed in the inner part. The made ground thus appears as a continuum deposit, in which however the contacts represent weakness lanes forming an irregular net. From the whole set of available experimental data the made ground may be considered as a heterogeneous, lightly overconsolidated, silty–clayey deposit.

The results of undrained strength measurements from six CPT tests performed close to the excavation are shown in Fig. 1. The undrained strength was evaluated through the cone resistance q_c by the relationship:

$$c_u = \frac{q_c - \sigma_v}{n}$$

with $n = 15$. The c_u profiles are shown in Fig. 1; the mean and the design profiles are also reported in the figure. The decrease in the undrained strength observed below 12 m of depth can probably be attributed to a consolidation process still in progress in the lower part of the clay layer caused by the two-staged land re-filling operations.

Instrumentation and excavation stages

A plan view and a section of the trench and the installed instrumentation is shown in Fig. 2. The ground instrumentation consisted of two inclinometer casings, a topographic benchmark net and two displacement transducers. The inclinometer TIV1, 12 m deep, was grouted into rings welded to the sheeting, while the inclinometer TIV2, 9 m deep, was installed at 3.5 m south of the southern sheet-pile. Pins A to F gave horizontal and vertical displacements of the ground surface, while pins H, K and Z provided vertical displacements only. The displacement transducers were connected to the top of the south sheet-pile.

The response of the bracing system was monitored by measuring the sheet-pile deflections by means of the inclinometer TIV1, and the strain distribution obtained from strain gauges glued to the sheet-pile, opposite to the excavation side, at regular intervals of 1 m. The strut loads were measured on two struts of the first level by means of strain gauges.

The strain gauge readings were automatically monitored every 60 min while, due to the excavation activities, only three readings were carried out at the inclinometers and the survey net.
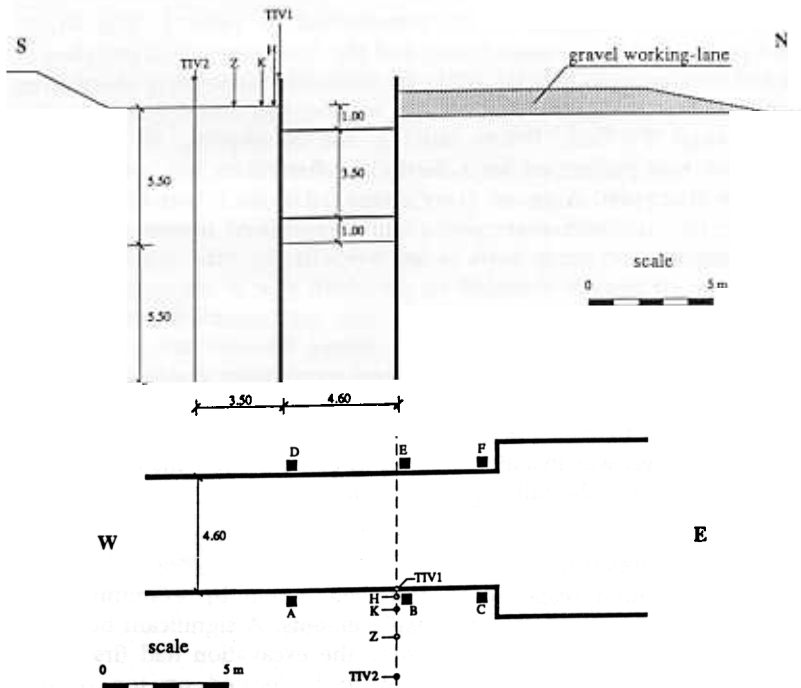
Fig. 2. Plan and section of the excavation

Table 1. Excavation sequences

Date	Stage	Event
23/1/91	A	Excavation face 15 m east of the test section. Drive sheeting in south and north sides. Install the instrumentation.
24/1/91	B	Excavation face 5 m east of the test section, readings of the installed instrumentation.
25/1/91	C	Excavate to a depth of 1 m at the test section.
28/1/91	D	Install the first strut level at a depth of 1 m. Excavate to a depth of 2 m at the test section. Perform inclinometer and survey net readings
29/1/91	E	Excavate to a depth of 5.5 m at the test section. Perform the survey net readings. Install the second strut level at a depth of 4.5 m. Perform the inclinometer readings.
30/1/91	F	Excavation face 5 m west of the test section. Pour the mud slab at the test section. Perform inclinometer and survey net readings.

The excavation sequences are summarised in Table 1. The BU-20 sheet-piles, 12 m long, were driven and the instrumentation installed at the test section on 23 January 1991, the excavation face being about 15 m far east. The test section, 4.5 m wide, was located about 5 m west of a square shaft $7 \times 7 \, m^2$. Before driving the sheet-piles, an extended excavation was performed for a depth of 1.5 m, from 5 m south of the southern sheet-pile. A gravel layer about 0.5 m thick was then placed north of the northern sheet-pile as a working-lane basement for the heavy construction equipment, a surcharge being thus applied to the surface. No equipment operated on the south side of the excavation.

Excavation began to the east of the instrumented section and proceeded west as detailed in Table 1. When the excavation reached a depth of 1 m, the first level of wales and struts were installed to brace the sheet-piles. The struts, spaced at an average of 2.7 m, consisted of 220 mm diam $\times$ 10 mm pipes, and abutted on HEB-280 wales. The second strut level was installed at a depth of 4.5 m after the completion of the excavation at the full depth of 5.5 m.

Observed behaviour

The overall ground response is described herein by examining the sheet-pile and the ground surface displacements. A significant horizontal displacement toward the area where the excavation had first been made (east) was observed at pins A to F, located on top of the
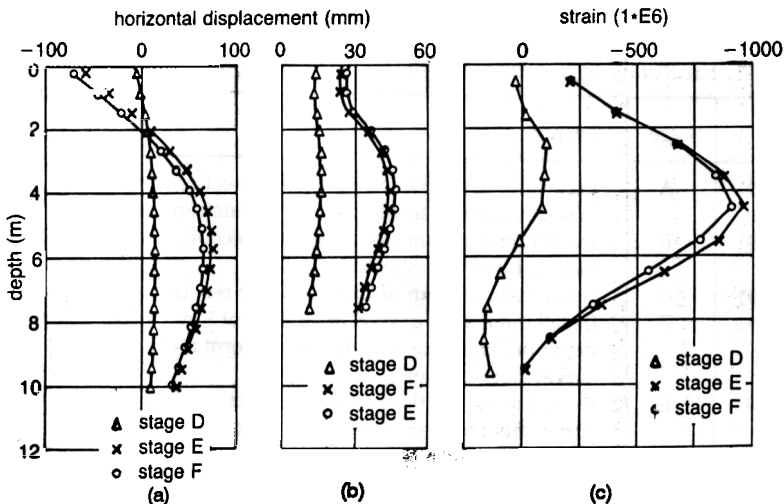

Fig. 3. Measured displacements at inclinometer TIV1 (a) and TIV2 (b) and strain distribution in the instrumented sheet-pile (c)

sheet-piles 1 m above the ground surface; it was 8–13 mm on the south side, and 10–14 mm on the north side. The displacement parallel to the excavation axis mainly occurred at stage D, at an excavation depth of 2 m; minor changes were measured thereafter. A different behaviour was observed between the north and the south sheet-piles when considering the horizontal displacement perpendicular to the excavation axis. The north sheet-pile, where the construction equipment was operating, showed maximum inward surface movements of 140 to 330 mm, while the south sheet-pile moved opposite to the excavation at 30 to 150 mm.

The above observations are consistent with the presence of the construction equipment on the north side of the excavation, and the wider shaft 5 m east of the test section. The working-lane surcharge and its higher stiffness were likely to produce the inward movement of the north sheet-pile, even after the strut installation. The shaft was probably the cause of the major displacements measured at pins C and F, and of the significant displacements parallel to the excavation axis.

Ground movements measured by the inclinometer TIV2, 3.5 m south of the southern sheet-pile, showed a maximum displacement of about 40 mm toward the excavation (Fig. 3(b)). The displacements are virtually constant with depth at stage D, while at stages E and F they gently increase from 28.6 mm at a depth of 2 m to about 40 mm at a depth of 3 m, then showing small gradients with depth.

Figure 3(a–c) shows the deflections and the strain distribution of the south sheet-pile. The point of zero displacement, 1 m deep at stage D, moved to a depth of 2 m at stage F. The movements were oriented toward the excavation below it, and opposite to the excavation in the upper few metres. About 0.5 m below the excavation base, a maximum inward movement of $\simeq$71 mm occurred at stage E, before the mud slab was poured. The strain distribution did not show any curvature change when the first strut level was installed.

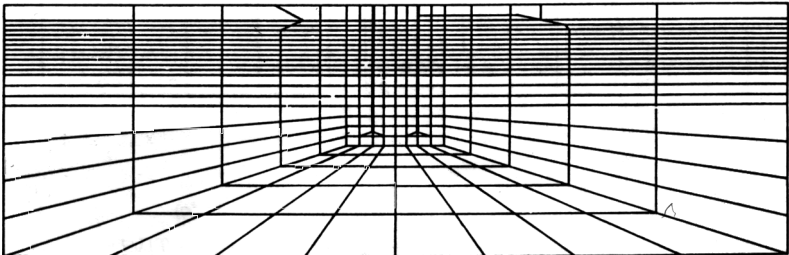The strut loads may be conveniently related to the sheet-pile deflec-

Fig. 4. Finite element mesh

tions, as they increase with them. The east strut was forced when installed at stage D due to the significant inward movements of the sheet-piles next to the wider shaft; a load of about 500 kN was then measured on it, while on the west strut a load of 70 kN was obtained. The strut loads steeply increased at stage E, when the full depth of the excavation was attained, and remained constant at 770 and 670 kN thereafter.

Finite element analysis

The FE mesh adopted in the analysis is shown in Fig. 5; it consisted of 472 8-noded Linear Strain Quadrilateral (LSQ) elements and 42 3-noded beam elements which model the sheet-piles and the struts. Due to the lack of axial symmetry, the mesh was extended to the whole domain on both the excavation sides, to a horizontal distance of ≈8.5 times the excavation width. The bottom boundary of the FE mesh was placed at the top of the lignite layer, assuming this as an infinitely rigid boundary. Both horizontal and vertical displacements were restrained at the lower boundary, while horizontal displacements alone were restrained at the sidewalls. The soil-sheet pile interface was modelled through 8-noded LSQ thin layer elements, with an aspect ratio of 10 to 30 (Pande and Sharma, 1979).
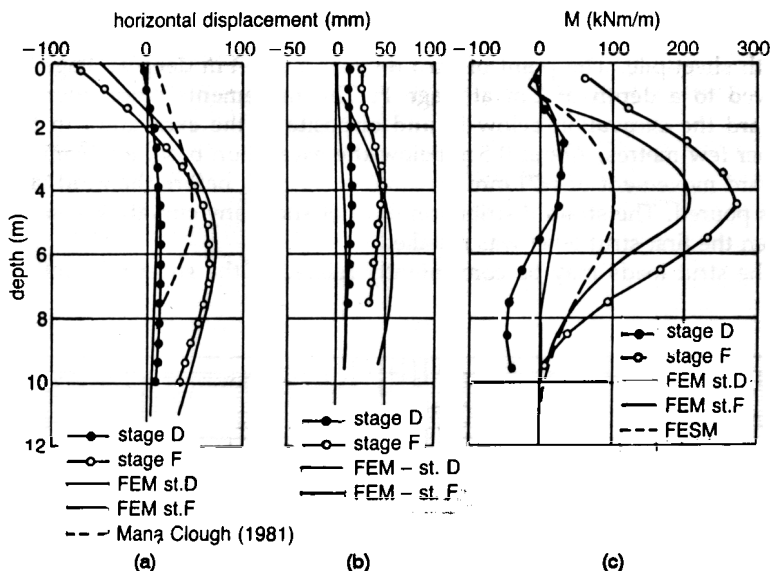

Fig. 5. Measured and predicted displacements: (a) inclinometer TIV1, (b) inclinometer TIV2; (c) measured and predicted bending moments

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Due to the low soil permeability, and to the short duration of the excavation process (7 days), undrained behaviour was assumed in the clay layer throughout the analysis, as confirmed by the work of Osaimi and Clough (1979). Among the existing techniques developed to handle problems involving media subjected to incompressibility constraints (e.g., see Borja (1992)), the simple total stress approach was used for a variety of reasons: the difficulty of obtaining a reliable evaluation of the effective stress–strain behaviour, due to the soil heterogeneity; the uncertainties associated with the pore pressure field in the made ground, due to a consolidation process still in progress; the aim of using the FE method as a design tool for a problem the provisional character of which would hardly justify a more refined numerical analysis in terms of effective stresses.

The soil and the soil-sheet pile interfaces were modelled as an elastic perfectly plastic material obeying the Von Mises yield criterion. The undrained strength profile of Fig. 1 was adopted in the analysis; a reduced c_u profile, of a factor 4, was attributed to the interface elements. A modulus multiplier M ($E_u = M \cdot c_u$) of 100 was selected for the undrained stiffness of the soil and the interfaces. Such a value might appear low if compared with those typical of natural soft to medium clay deposits (e.g. see Duncan and Buchignani (1976)). However it reflects the full-scale behaviour of the made ground, which is strongly influenced by the previously described replacement procedure. The gravel working-lane was modelled by an elastic material with a Young modulus of 1×10^5 kPa and a Poisson ratio of 0.25. The initial state of stress in the soil was evaluated by assuming a horizontal to vertical total stress ratio K = 1.

The geometrical and mechanical properties considered in the analysis for the sheet-piles and the struts are summarized in Table 2. Since each sheet-pile element could bend independently from the adjacent ones, due to the poor workmanship in the driving operations, the sheet-pile moment of inertia J was evaluated by assuming a zero efficiency of the joints.

The FE analysis was performed in two separate sections: the removal

Table 2. Properties of the structural elements

Element	A[1] (m²/m)	J[1] (m⁴/m)	E (kPa)
BU-20 sheet-pile	1.86E-2	1.001E-4	2.0E8
Pipe struts	2.19E-3	6.450E-6	2.0E8

(1) per unit length of excavation

Table 3. Maximum horizontal displacements of the south sheet-pile

	Stage D		Stage F	
	u_{max} (mm)	error (%)	u_{max} (mm)	error (%)
Measured	14.0	–	65.0	–
Mana and Clough (1981)	8.7	−37.9	49.2	−24.3
FE analysis	10.7	−23.6	74.4	14.5

of a layer 1.5 m thick and the construction of the working-lane were simulated first under undrained conditions; the trench excavation was then modelled under plane strain conditions following the construction sequence shown in Table 1. The excavation was completed in 7 stages by removing subsequent rows of elements 1 m thick, and by applying surface tractions at the excavation boundaries; the tractions are equivalent to the pre-existing state of stress of the removed elements (Mana, 1978).

Comparison of observed and predicted responses

Sheet-pile and ground displacements

Table 3 summarises the maximum horizontal displacements of the south sheet-pile. Comparison between measured and evaluated deflections are considered at stages D and F (2b and 7 in the FE analysis). In applying the method by Mana and Clough (1981) the factor of safety against basal heave was computed for an average undrained strength of 30 kPa. The percentage differences between the computed and the measured displacements are also reported in the table. A fair agreement is obtained by both the methods, the finite element analysis being slightly closer to the measurements.

For the same stages as those above, the deflection profiles as measured at inclinometers TIV1 and TIV2 are compared in Fig. 5(a–b) with the predicted displacements. The deflection profile by the method of Mana and Clough (1981) was obtained by the maximum horizontal displacement and one of the normalised profiles as suggested by Mana (1978). A close agreement is observed for the FE results, the analysis being capable of correctly locating the point of maximum inward movement and the occurrence of large displacements opposite to the excavation side, in the first 2 m of the sheet-pile. The method by Mana and Clough (1981) implies significant errors at the sheet-pile edges and predicts a quite different deflection profile. The difference can possibly

be attributed to the presence of the gravel working-lane and the construction equipment acting on the north side of the excavation; the consequent surcharge applied to the surface yields a non-symmetric displacement field oriented towards the south. The central portion of the final deformed mesh next to the excavation zone clearly supports this interpretation (Fig. 6).

The ground-surface settlements, normalised by the excavation height, are compared in Fig. 7 at stage F. The curves proposed by Peck (1969) and the surface settlement envelope suggested by Mana and Clough
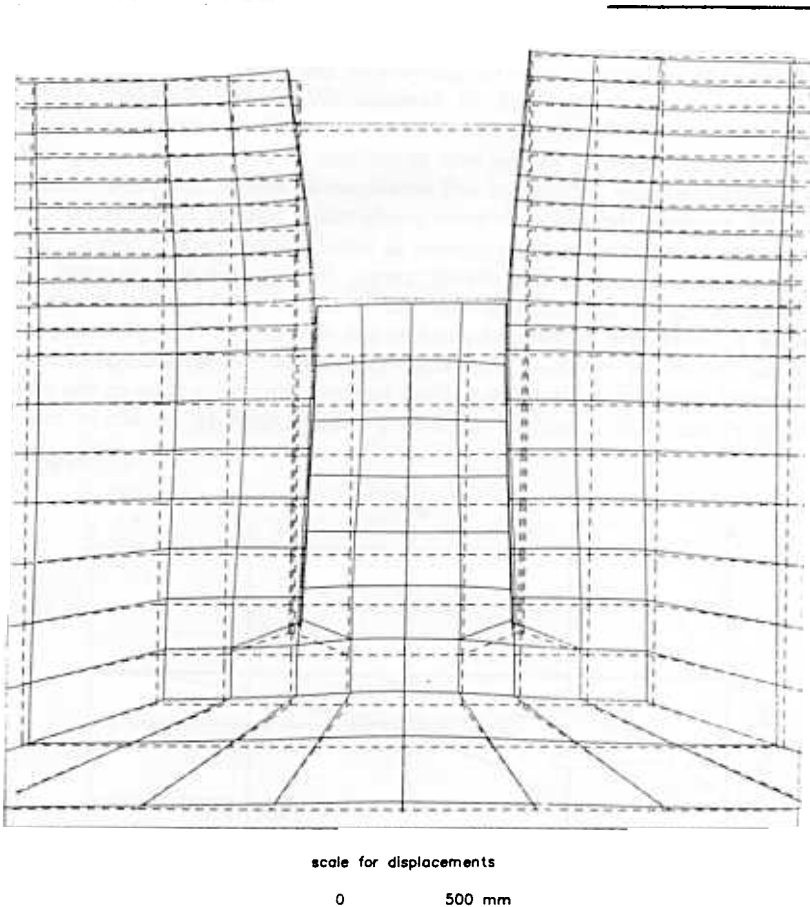

scale for displacements

0   500 mm

Fig. 6. Detail of the deformed mesh; displacements are magnified by a factor of 5

(1981), both on an empirical basis, are also shown in the figure. Except for measurements from pins B and H, next to the sheet-pile, the data points plot close to the boundary between Peck's zones I and II, consistently with the low stability number $N = \gamma H/c_u$ equal to 3.4 (by assuming a constant c_u value of 30 kPa). The normalised curve by Mana and Clough (1981) is also in fair agreement with the measurements, the maximum normalised settlement being about 0.67% against a measured one of 0.45% to 0.89%.

A worst agreement is observed with the profile of the surface settlements obtained by the finite element analysis. Next to the top of the excavation the ground-surface heaves at about 0.75% of the excavation height and the settlement profile extends to a much greater distance than that indicated by the empirical curves. Similar results were obtained by a linear elastic FE analysis of the deep excavation for the New Palace Yard Car Park in London (Ward and Burland, 1973). Simpson et al. (1979) and Jardine et al. (1986) attributed this limitation to the assumed linearity of the soil behaviour in the elastic range, and suggested that the account of soil non-linearity within the yield surface would improve the finite element predictions. Similar conclusions can probably be applied to the problem at hand, since the soil behind the excavation remains in the elastic range throughout the analysis as shown in Fig. 8; the figure shows the extension of the plastic zone at stage F, when the excavation reaches the full depth. The presence of plastic zones in between the sheet-piles may be attributed to the assumed initial stress state and stress history, the stress state in the soil being closer to a condition of passive failure than to an active one.
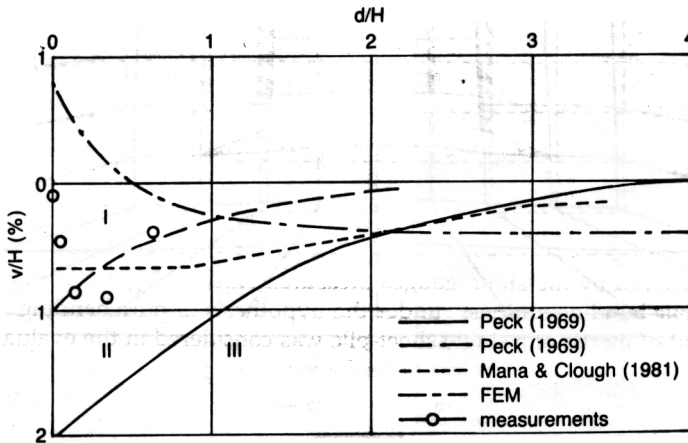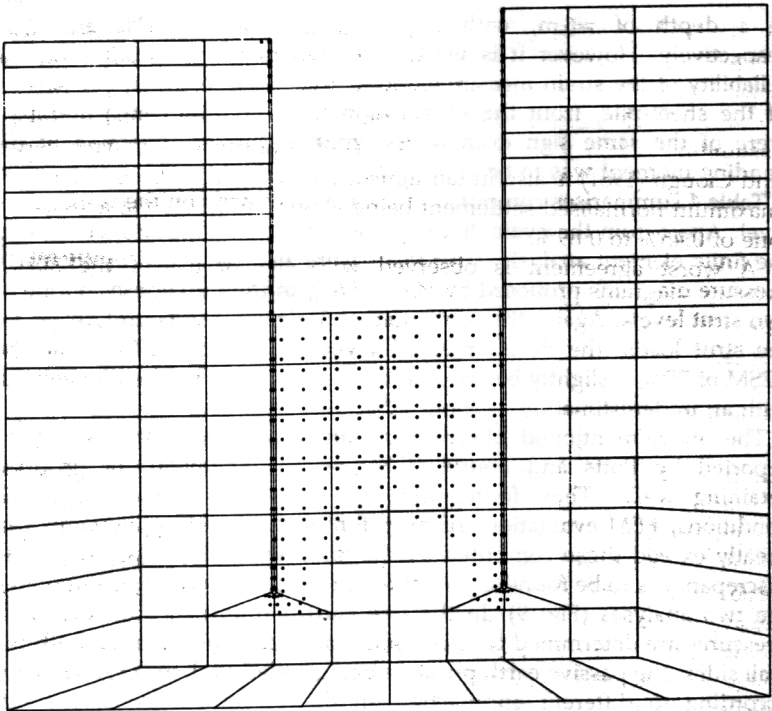

Fig. 7. Ground surface settlements

Fig. 8. Development of plastic zones at stage F

However, apart from the limits of the soil model, it is worth noting that the inability of a conventional displacement finite element procedure in modelling strain localisation phenomena could prevent a reliable evaluation of the ground movements after a certain level of strain is reached in the soil mass (Finno and Harahap, 1991).

Loads on sheet-piles and struts

The bending moment distributions computed at stage F by the finite element analysis and the free earth support method are compared with the measurements on the instrumented sheet-pile in Fig. 5(c). The latter were derived by the strain gauges measurements, taking account of the sheet-pile bending stiffness, under the hypothesis of pure bending. The moment of inertia of a single sheet-pile was considered in the evaluation of the bending stiffness as already assumed in the finite element analysis. The analyses appear to underestimate the maximum bending moment on the sheet-pile; values of 104 and 205 kN/m are obtained by the FESM and FEM computations against a measurement of 274 kN/m,

at a depth of ≈ 6 m, with a percentage error of 62% and 25% respectively. However it is worth stressing that some doubts on the reliability of the strain measurement arise, at least in the upper portion of the sheet-pile, from the observation that the strain measurements were of the same sign even in the zone where an inversion of the bending moment was to be expected.

Table 4 summarises computed and measured loads on the upper strut level. Apart from the evaluations by the free earth support method and the finite element analysis, strut loads were also calculated by the earth pressure diagrams proposed by Peck (1969), in the hypotheses of one or two strut levels. Again, the conventional design methods underestimate the strut loads, the Peck's method giving an error of 37% while the FESM of 75%; a slightly better prediction is obtained by the FE analysis, with an underestimation of about 22%.

The above-mentioned observations are consistent with the results reported by Potts and Fourie (1984) on the behaviour of propped retaining walls. They found that for excavated walls in high K_0 conditions, FEM evaluations of prop forces and bending moments can greatly exceed those computed by the limit equilibrium approach. The discrepancy is to be found in the different earth pressure distributions of the two analyses (Fig. 9). In the free earth support method the earth pressures are determined for a condition of limit equilibrium on both the wall sides, the passive earth pressure being scaled back by a safety factor according to different approaches. In the present analysis, the undrained shear strength was reduced on the passive side by the actual factor of safety. On the contrary, in the finite element analysis the earth pressures are the result of the soil-structure interaction; the progressive deflections of the sheet-pile towards the excavation promote the redistribution of earth pressures in the embedded part of the sheet-pile and the restrained zones, following the well-known mechanism of arching (Bjerrum et al., 1972). In the presence of a large degree of

Table 4. Loads on the first strut level

	strut load[1] (kN/m)	error (%)
Measured	265.1	–
Peck (1969)[2]	83.57 (167.0)	68.5 (37.0)
FESM	66.4	75.0
FEM	206.4	22.1

(1) per unit length of excavation
(2) bracketed figures refer to the case with a single strut level
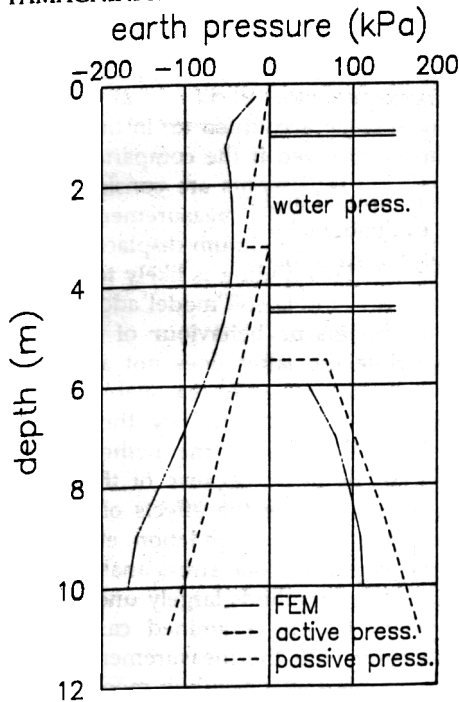
earth pressure (kPa)

Fig. 9. *Horizontal stress distribution*

horizontal restraint (as in multi-propped or strutted excavations) or in high K_0 conditions, not even large horizontal movements of the wall are sufficient to bring the earth pressures to the classic active distribution, while passive conditions may be partially or completely reached in front of the sheet-pile wall.

Conclusions

In the present work, the behaviour of a 5.5 m deep braced excavation in soft to medium clay was analysed by means of both standard design procedures and a simple elastic–perfectly plastic, total stress finite element analysis.

From the comparisons of the analysis results with the observed response of an instrumented test section the following conclusions can be drawn:

(a) The maximum lateral movement of the sheet-pile is successfully predicted by both the method by Mana and Clough (1981) and the finite element analysis, the latter matching slightly better with the

measurements; however, as far as the deflection profile is concerned, the FE analysis shows its higher predictive capability, the influence on the structure behaviour of factors such as non-symmetric loads and construction sequences being accounted for in the analysis.

(b) A different trend is observed if the comparison of predicted and measured ground-surface settlements are considered. In this case a less satisfactory agreement with the measurements is obtained by the finite element analysis for the maximum displacement and the shape of the settlement trough. This finding is likely to be the result of the initial stress state and the simple soil model adopted in the analysis; the former leads to the elastic behaviour of the soil behind the retaining structure, while the latter does not account for soil non-linearity within the elastic range. A reasonable assessment of the ground movements has been obtained by the normalised surface settlement profiles (Peck, 1969) and by the method proposed by Mana and Clough (1981). This is likely because of the empirical basis of these relationships which include the effects of factors such as soil non-linearity, strain localisation, consolidation, etc., not considered in a simple elastic–perfectly plastic, total stress analysis.

(c) The conventional design methods largely underestimate the loads on the support system for the examined case-history. Although qualitatively in agreement with the measurements, limit equilibrium analysis yields considerably lower bending moment distributions on the sheet-pile. A better prediction is given by the finite element analysis, with an error of about 25% on the maximum bending moment. Similar results are found for the strut loads, for which much closer evaluation is obtained by the finite element computations than by any of the considered standard design procedures. The latter finding may probably be attributed to the high K value selected for evaluating the initial stress state; in similar conditions, Potts and Fourie (1984) found that conventional limit equilibrium approach can severely underestimate both maximum bending moment and strut load.

(d) In agreement with the recent findings of Ho and Smith (1991), the use of the finite elements in the analysis of braced excavations, can capture most of the physical features of the soil-structure interaction process, even performing simple elastic–perfectly plastic total stress analyses by using standard computer packages on desktop computers.

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