DEEP EXCAVATIONS AND TUNNELING IN SOFT GROUND

EXCAVATIONS PROFONDES ET CONSTRUCTION DE TUNNELS EN TERRAINS DE FAIBLE RESISTANCE.

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SYNOPSIS Rational design of a project involving open cutting or tunneling requires the ability to judge whether the proposed work is feasible under various methods of construction, to estimate the settlements and other movements of the adjacent ground surface and structures, and to provide adequate strength and appropriate flexibility or rigidity in the final structure. In this report, observational data are assembled with respect to each of these requirements, and in some instances procedures are suggested for design. Particularly, in the design of tunnel lining, recommendations are made for taking advantage of the strength of the soil rather than for ignoring it in accordance with most present design practice.

FOREWORD

The state of the art of excavating and tunneling in soft ground has undergone substantial change during the last decades. New methods of construction and support have been devised for braced cuts, and new techniques have been developed for machine tunneling.

Deep substructures of buildings are usually constructed in excavations having sides supported by bracing or tiebacks. Sewers, water-supply conduits, and transportation tubes are sometimes built in tunnel and sometimes in open cut. On many such projects, the decision as to whether the work should be done by tunneling or in cut is the most significant single step in the design. The relative economy of the procedures themselves is but one of the considerations in the decision. The effects of the construction on overlying and adjacent properties must also be evaluated. In urban areas, the effects of construction on utilities and on the flow of traffic, and the necessity for underpinning neighboring structures may be the overriding factors. There is urgent need for reliable means to estimate the extent and nature of the movements and disturbances associated with each type of construction because, without such estimates, there can be no rational basis for decision.

This report attempts to summarize the present state of our knowledge of the feasibility and the consequences of tunneling and open-cutting in various types of soil, so that intelligent appraisals of the alternatives can be made. It also summarizes the basis for design of the required temporary and permanent supports. In connection with tunnels, present knowledge appears to call for a radical change from the traditional approach. Data from full-scale field observations provide the principal framework upon which the conclusions rest.

The Report is divided into two sections: Part A - Tunneling; and Part B - Open Cuts. In the preparation of Part A, the writer was assisted by Mr. Birger Schmidt, and in Part B by Mr. Harvey Parker.

1. TUNNELING

1.1 Intruduction

The number of soft-ground tunnels has increased rapidly during recent years. The most obvious change in the art appears to be the widespread adoption of excavating machines. Nevertheless, machines have not yet proven successful in all types of soft ground and more conventional methods of excavation are still widely used.

In spite of the proliferation of soft-ground tunnels and the consequent great increase in experience, the art of tunneling has failed to keep pace with modern requirements. This failure is due largely to a lack of communication and understanding between tunnel designers and tunnel constructors. It might even be said that the failure has its origins in the view, held by many engineers, that design and construction of tunnels are separable endeavors.

Probably in no area of applied soil mechanics are design and construction actually so inextricably interwoven. Yet, by long tradition the contract documents for soft-ground tunnels are usually concerned with the structural features of the completed tunnel

2. DEEP EXCAVATIONS

2.2 Introduction

This discussion is restricted to excavations with vertical sides requiring lateral support. It deals primarily with the movements of the surrounding ground and the means for reducing their magnitude, and with the forces in systems of bracing required to restrict the movements or to prevent collapse of the side walls. The phenomena of base failure and bottom heave are also considered.

The movements of the soil surrounding a deep excavation are responsible for settlement of the adjacent ground surface. To avoid damage to surface installations and to assess the need for underpinning nearby structures, the engineer needs to estimate the magnitudes of the settlements and their pattern of distribution. The settlement depends on the properties of the soil, on the dimensions of the excavation, on the general procedure of excavation and bracinc employed, and on the workmanship. As it does for tunnels, the latter factor places a severe limitation on the possibility of making valid predictions of settlements solely on the basis of theory and soil tests. Observational data are needed as a guide to judgment.

The observational data are not yet plentiful. They consist in some instances of measurements of settlement at various points on the ground surface adjacent to open cuts, and of the results of observations of settlements of buildings. In other instances measurements have been made of the lateral movement of the retaining structures as excavation proceeds. In a few instances, both types of observations have been made at the same location. The available data are assembled in Chapter 7.

It is well known that deep excavations with vertical sides cannot be made in soft cohesive soils beyond a critical depth at which base failure occurs. Heaves of the bottoms of large excavations with accompanying adjacent settlements have, however, also been noted in several instances in extremely stiff clays. These phenomena and their causes are discussed in Chapter 8.

If an estimate of settlement indicates that the movements will be excessive or if calculations indicate the likelihood of bottom heave or base failure, refinements in workmanship alone cannot appreciably reduce the undesirable consequences. Instead, radical alteration in the method of excavation and of providing support is mandatory. Several of the more successful alternatives with discussions of their advantages and limitations are considered in Chapter 9.

Finally, the loads are considered for which the temporary supports may be designed. A substantial body of literature already exists (Flaate 1966) concerning the loads to be expected in struts supporting the vertical sides of excavations in soft to medium saturated clays and in cohesionless sands. This information, summarized by Terzaghi and Peck (1967), is brought up to date and is supplemented by the results of similar observational data for cuts in other types of materials, particularly stiff clays and cohesive sands.

2.2 Lateral Movements and Settlements

2.2.1 Characteristic Movements

It has been observed that saturated plastic clays experience a consistent pattern of deformation as material is removed from the space between the walls of a temporary bracing system. Excavation reduces the load on the soil beneath the cut, whereupon the underlying soil tends to move upward. The soil alongside the sheeting or other supporting side walls tends to move inward, even at levels below that to which the cut has progressed, before cross-bracing or other types of support can be provided. On account of the rise of bottom and the inward lateral movement, the ground surface surrounding the cut subsides.

Several sets of comprehensive observations have recently been carried out in Oslo and described in a series of Technical Reports by the Norwegian Geotechnical Institute (NGI 1962-66). The results for one cut are shown in Figs. 14 and 15. Fig. 14 shows the manner in which the settlements of the ground surface and the inward movements of the sheet-pile walls developed in relation to the insertion of struts as the excavation deepened. In Fig. 15 the solid line represents the area beneath the successive settlement curves in Fig. 14, or the volume of settlement per unit length of cut, as a function of time. The depth of excavation with respect to time is shown by the dash line. The separate points in the diagram represent the areas under the various curves of lateral displacements of the sheeting. The proximity of the points to the settlement-area curve demonstrates that the volume of settlement surrounding the structure is approximately equal to the volume of lost ground associated with the inward movement of the vertical walls. The latter volume in turn is related to the volume of heave below the bottom of the excavation between the limits of the vertical walls. The observations lead to the conclusion that settlement near an open cut can be reduced only if the inward movement of the sheeting and the heave can be substantially reduced. In soils other than saturated clays the volume of settlement and the volume of lateral movement of the sheeting may not be equal. Nevertheless, even for these materials, reduction in settlement can be achieved most effectively by reducing the lateral movements of the walls.

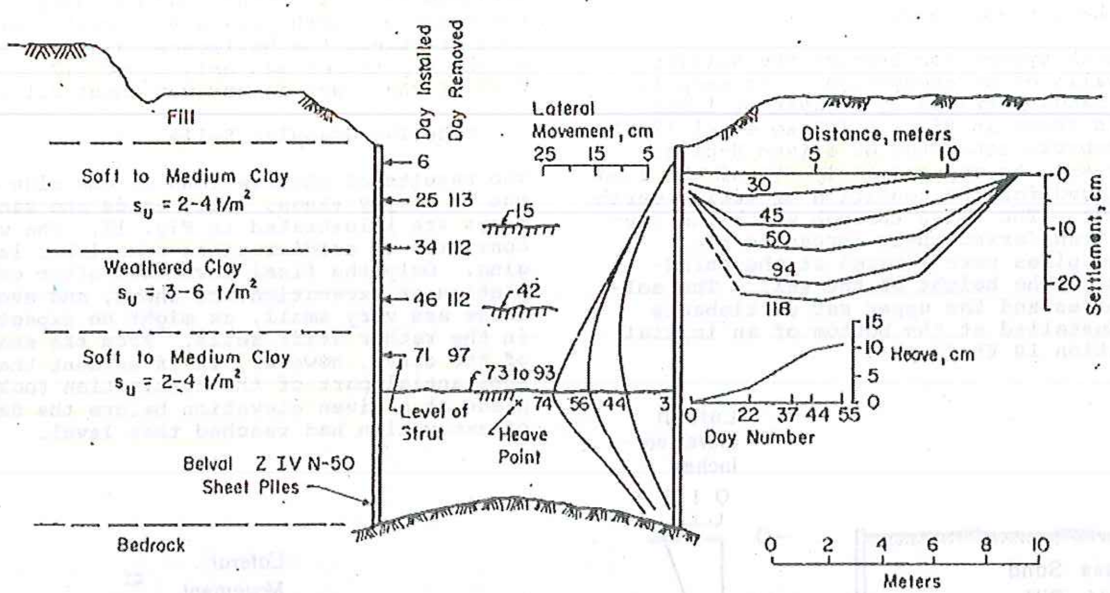


Fig. 14. Relation of Progress of Construction to Lateral Movements of Sheeting and Settlements Adjacent to Open Cut in Soft Clay in Oslo (Vaterland 1)

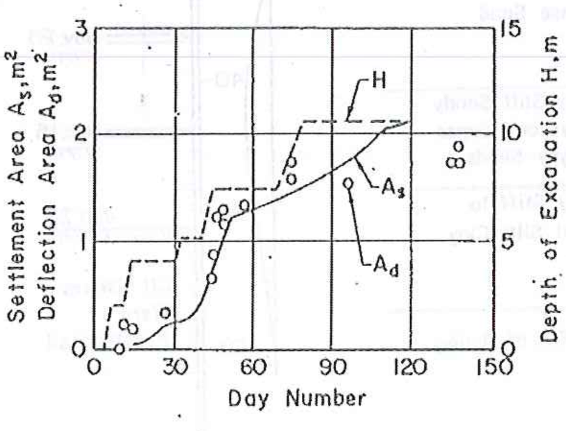


Fig. 15 Comparison of Volume of Settlement A_s and Volume A_d Represented by Lateral Displacement of Sheeting for Cut Illustrated in Fig. 14

Experience has also indicated that the stiffnesses of ordinary soldier piles and of steel sheet piling, even of heavy section, are not usually great enough to have a significant effect on the magnitude of the lateral movement of the wall. The lateral movement of walls of these types may, however, be substantially reduced by the insertion of supports such as struts at relatively close vertical spacing. In recent years several methods have been developed for constructing much stiffer boundary walls before the excavation is made. These walls also require horizontal support at vertical intervals but, under comparable conditions, the intervals need not be as small as for more flexible types of exterior walls. Nevertheless, benefits of the more rigid walls may not always be as great as anticipated by the designers. This point is discussed further in Chapter 9.

The most important variable that determines the amount of movement is not the stiffness of the exterior walls or the vertical spacing of the bracing, but the characteristics of the surrounding soils. Hence, in the following summary of lateral movements of the walls of support systems, the information is classified in accordance with the principal types of subsurface materials.

2.2.2 Summary of lateral Movements of Vertical Earth Supports

Cohesionless Sands

A tieback system for bracing the soldier-pile walls of an excavation 37 ft deep in dense sands overlain by a layer of loose sand is shown in Fig. 16 (Rizzo et al 1968). The tiebacks consisted of driven H-piles prestressed to approximately 50% of the load calculated for the condition of active earth pressure. The wales through which the tie-backs transferred their forces to the soldier piles were located at the third-points of the height of the wall. The soldier piles and the upper set of tiebacks were installed at the bottom of an initial excavation 10 ft deep.

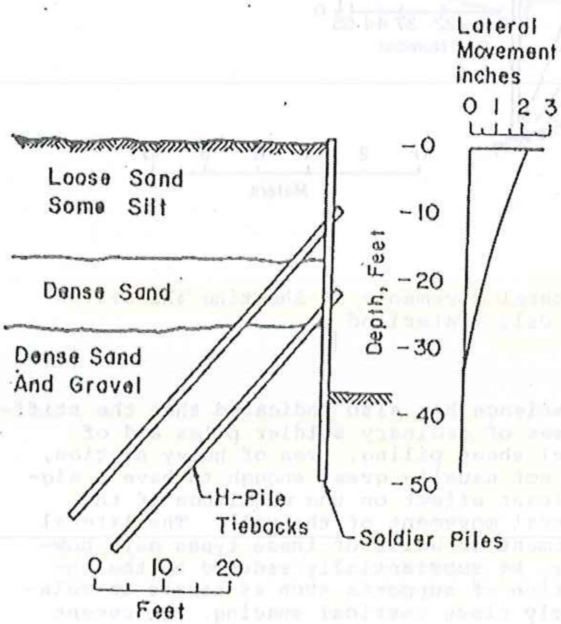


Fig. 16. Section through Cut in Sand Supported by Tieback System; Lateral Movement of Soldier-Pile Wall during Excavation

The lateral displacement of the wall at the end of the excavation period is shown at the right-hand side of the figure. Subsequent movements were negligible. The horizontal displacement of the top anchor reached about one inch. The soldier piles moved inward by amounts decreasing with depth from a little over 2 inches at the ground surface to about zero at the bottom of the cut.

For an opposite wall in the same cut, excavated to a depth of 21 ft, a similar bracing system was used. It, too, consisted of two sets of tiebacks with wales subdividing the vertical wall into equal thirds. In this instance, however, the tiebacks were prestressed to 110% of the load calculated on the basis of earth pressure at rest. With this prestress the horizontal displacement of the top anchor was only about 0.2 inch. That of the lower anchor was about 0.1 inch.

Cohesive Granular Soils

The results of observations of one side of a cut in clayey sands, silty sands and sandy clays are illustrated in Fig. 17. The walls consisted of soldier piles and timber lagging. Only the final movements after completion of excavation are shown, and even these are very small, as might be expected in the rather stiff soils. From the shape of the curve, however, it is evident that a substantial part of the deformation took place at a given elevation before the depth of excavation had reached that level.

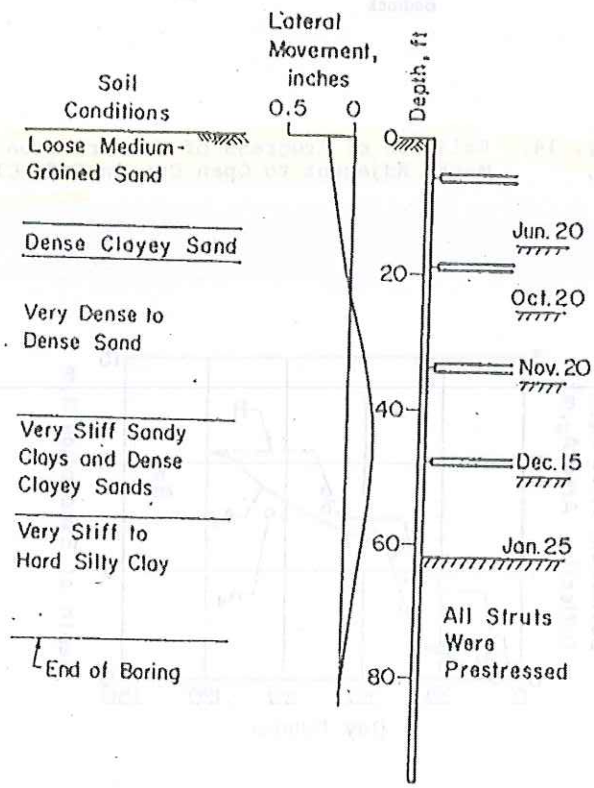


Fig. 17. Section through Cut in Dense Cohesive Sands and Stiff Silty or Sandy Clays, Supported by Cross-Lot Bracing, in Oakland, California; Lateral Movement of Soldier-Pile Wall during Excavation

An unusually deep cut beneath sloping ground, extending to 78 ft below street level on the uphill side, was made through very stiff clays, dense sands and silts, and dense sands in Seattle (Shannon and Strazer 1968). Most of the sandy materials displayed some cohesion. The bracing consisted of soldier piles and lagging with tieback anchors at 8 levels as shown in Fig. 18. Lateral movements were measured by means of horizontal extensometers and surface surveys. The movements shown in Fig. 18 are those observed during excavation between the two levels shown in the figure. The previous lateral movements were less than 0.5 inch except at the top of the soldier piles where they reached 1.5 inch. Those values followed in the figure by a "+" are smaller than the actual movements because the reference points for the extensometers did not extend beyond the zone of influence of the excavation.

Although the movements and accompanying settlements shown in Fig. 18 were small for a cut of such large dimensions, the settlements of the side streets near the uphill ends of the cut were even smaller. It is believed that construction many years ago of the railroad tunnel shown in Fig. 18 disturbed the overlying soil severely, as settlements due to tunneling were noted at that time well outside the limits of the street (see Fig. 4).

Soft and Medium Clays

A considerable amount of information has become available concerning the lateral movements of cross-braced walls in plastic clays of very soft to medium consistency. One example, representing an open cut in Oslo, has been illustrated in Fig. 14. Additional profiles of the deflection of bracing systems for several cuts in Oslo,

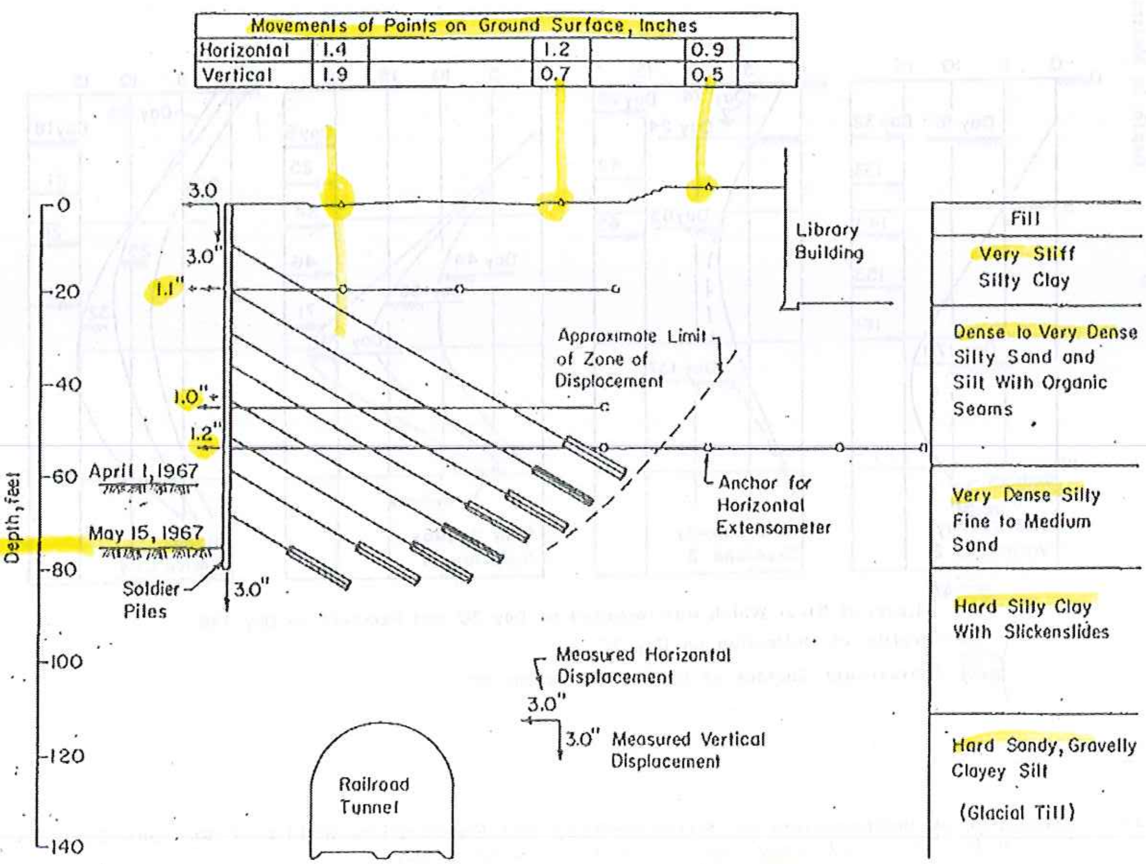


Fig. 18. Section through Cut in Very Stiff Clays and Dense Slightly Cohesive Sands, Supported by Tieback System, in Seattle; Movements of Soldier-Pile Wall during Excavation between Levels Shown

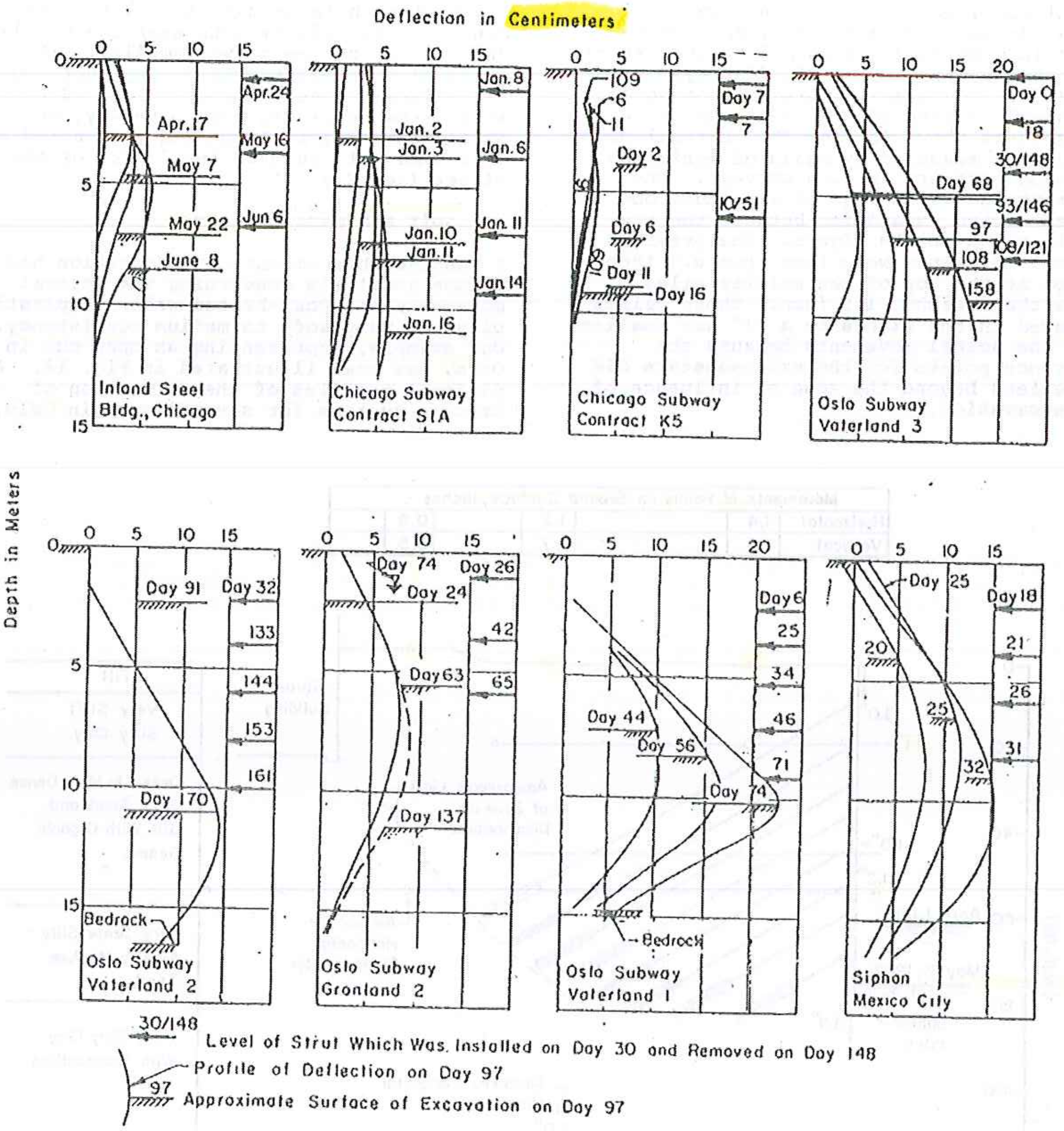


Fig. 19. Profiles of Deflections of Soldier-Pile and Sheet-Pile Walls of Various Open Cuts in Plastic Clays of Very Soft to Medium Consistency

Chicago and Mexico City (Flaate 1966, Rodríguez and Flamand 1969) are shown in Fig. 19. The significant events are related by numbers indicating the number of days since the beginning of excavation.

The diagrams show clearly that the lateral movements associated with very soft to medium plastic clays substantially exceed those in cohesive granular soils or in cohesionless soils, although admittedly there is only one example of each of the latter. They also indicate that large and often excessive movements develop if excavation proceeds too deeply before the uppermost strut is placed. In two of the cuts (Chicago Contracts S1A and K5), a substantial part of the movements could have been prevented if the top strut had been inserted while excavation was at a shallower depth. The observations appear to justify the recommendation of Peck (1943) and Ward (1955) that the top strut should be installed before the excavation exceeds a depth equal to $2s_u/\gamma$.

At those cuts in which the uppermost struts were inserted before the depth of excavation had exceeded $2s_u/\gamma$, most of the deformation occurred below the excavation level prevailing at any given time. The inward movement accumulated in this manner during the complete excavation of the cut represents inevitably lost ground and settlement associated with the construction procedure in spite of good workmanship.

In wide excavations it is customary to leave a sloping berm of earth against the wall of the bracing system while the central portion of the site is excavated to base level and the foundation slab cast. Inclined or raking struts are then installed from the edge of the foundation slab, parallel to the slope of the berm, to a wale installed near the ground surface. Excavation then proceeds in sections as the berm is cut away and additional raking struts are inserted as shown in Fig. 20. Although the stability of such a berm in a cohesive soil may be fully adequate, the movements at the top of the wall may be excessive because of deflection of the berm. The pattern of movements is exemplified by Fig. 20 (Lacroix 1966). To reduce the movements adjacent to the cut, it may be necessary to provide generous berms that should not be removed until the uppermost line of raking struts has been installed.

The lateral movements of the soil adjacent to a deep basement excavation in clay in St. Louis are shown in Fig. 21 (Lacroix and Perez 1969). The effect of prestressing the upper raker is apparent. The raker effectively prevented further inward movement at the top. Below the upper raker the inward movement of the wall followed the usual pattern for cross-lot bracing without berms.

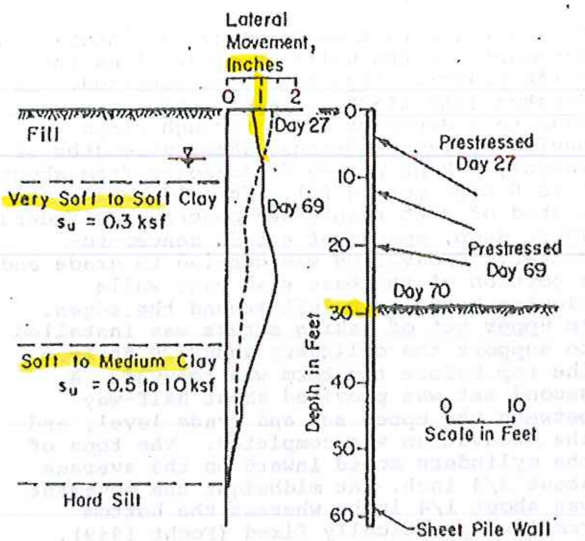
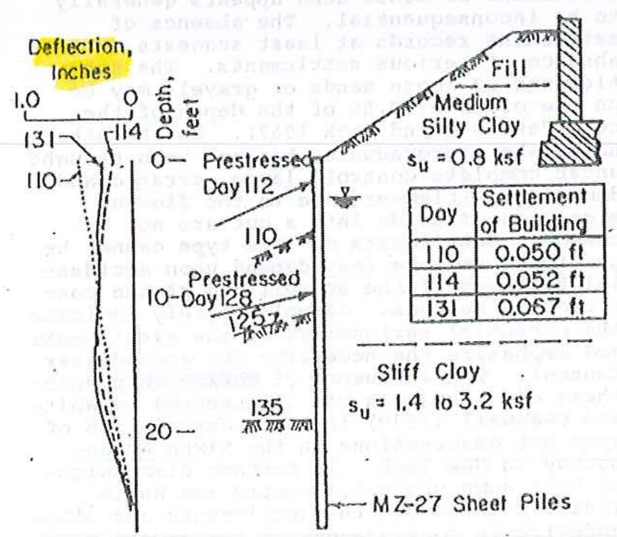


Fig. 20. Raker Support System for Cut in Very Soft to Soft Clay in Chicago, and Deflected Position of Sheeting before Prestressing Top Raker and Immediately after Prestressing the Lower Street



Day	Settlement of Building
110	0.050 ft
114	0.052 ft
131	0.067 ft

Fig. 21. Raker Support System for Cut in Medium Clay in St. Louis, and Deflected Positions of Sheet-Pile Wall before and after Prestressing Top Raker and Shortly before Completion of Excavation

Stiff Clays

Only a few direct measurements of lateral movements of the walls of excavations into stiff plastic clays have been reported. In Houston (ENR 1968), a large excavation was made to a depth of 60 ft through clays having average undrained shear strengths of roughly 3 kips per sq ft (ranging from about 1 to 6 kips per sq ft). The side walls consisted of 3-ft reinforced concrete cylinders 75 ft deep, spaced at 4.5 ft center-to-center. Excavation was carried to grade and a portion of the base slab cast while sloping berms were left around the edges. An upper set of raking struts was installed to support the cylinders about 20 ft from the top before the berm was removed. A second set was provided about half-way between the upper set and grade level, and the excavation was completed. The tops of the cylinders moved inward on the average about 3/4 inch. At midheight the movement was about 1/4 inch, whereas the bottom remained practically fixed (Focht 1969).

2.2.3 Summary of Settlements

Cohesionless Sands

Few records are available of the settlement of the ground surface adjacent to cuts in cohesionless sands. Projects of this type appear to fall into two categories. On the one hand, if the sand is above water table or if the groundwater has been lowered and brought under complete control, adjacent settlement of dense sand appears generally to be inconsequential. The absence of settlement records at least suggests the absence of serious settlements. The settlements of loose sands or gravels may be on the order of 0.5% of the depth of the cut (Terzaghi and Peck 1967). On the other hand, where groundwater has not been brought under complete control, large, erratic and damaging settlements due to the flow or migration of sands into a cut are not uncommon. Settlements of this type cannot be predicted because they depend upon accidental features of the subsoil and of the construction methods. Examples merely indicate the potential seriousness of the settlements and emphasize the necessity for groundwater control. The influence of workmanship under these circumstances was illustrated by White and Paaswell (1939) in their description of open cut observations on the Sixth Avenue Subway in New York. In further discussions of this same project, Prentis and White (1950) pointed out the settlements and other undesirable consequences of the upward seepage pressures of water flowing toward the bottom of a cut in sand from beneath a tight timber cutoff as compared to that through more permeable horizontal sheeting provided with drainage slots.

Cohesive granular Soils

Little information is available concerning the distribution of settlements of the ground surface adjacent to cuts in such materials. Again, the lack of information suggests that the movements are generally small. The settlements along the property lines, about 20 ft from the edge of the subway station cut illustrated in Fig. 17, did not exceed 0.5 inch and were usually less than 0.2 inch. The soils consist of stiff clayey sands and sandy clays. The small settlement adjacent to the cut in Seattle, in spite of the previous disturbance of the soil by tunneling, has been noted (Fig. 18).

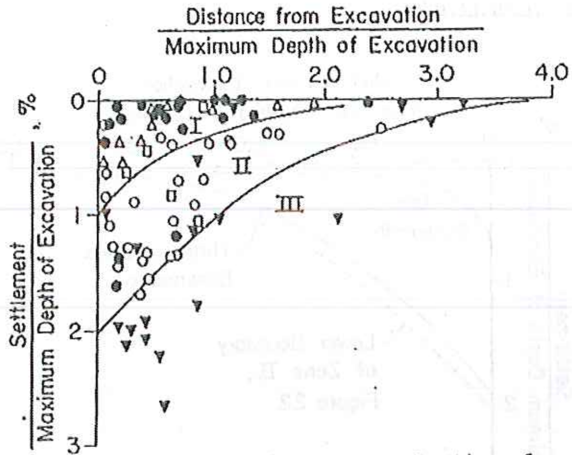
The cohesion possessed by materials in this category greatly reduces their sensitivity to seepage pressures. Since raveling is not associated with open cutting to the extent that it is in tunneling, the excavation of deep cuts in such materials is commonly a straightforward operation. On the other hand, the lateral support cannot usually be eliminated because the materials tend to spall if unsupported, and slices may descend from the sides into the cut.

Saturated Plastic Clays

Considerably more information is available concerning the immediate settlement adjacent to cuts in plastic clays than in the other materials discussed above. This fact is itself indicative that the settlements associated with plastic clays are likely to be appreciably greater than those associated with most other soils. In addition, appreciable delayed settlement may develop on account of consolidation.

Both the magnitude of the settlements and their distribution as a function of distance from the cut are of practical importance. Data from various cuts in several types of materials appear to define the zones sketched in Fig. 22, in which settlements and distances are plotted in dimensionless form as fractions of the depth of the cut. The plot permits rough estimates of the settlements that might be expected under various conditions. Settlements due to consolidation within the construction period are included. For soft clays, settlements as great as 0.2% of the depth of the cut may be encountered at distances equal to 3 or 4 times the depth.

Most of the conclusions concerning the magnitude of settlement and the distance to which it extends from the sides of the cut are necessarily somewhat indefinite because of the small number of observations available. In contrast, a comprehensive study of settlements adjacent to building sites in downtown Chicago (Ireland 1955) includes the results of several thousand observations. For many years it was the custom in Chicago to make detailed settlement observations on all structures within about one city block of a site for the construction of a new building. At all the buildings included in the study, the foundation consisted of hand-excavated, Chicago-type "caissons" to the hardpan, encountered at a



Zone I
Sand and Soft to Hard Clay
Average Workmanship

Zone II
a) Very Soft to Soft Clay
1) Limited depth of clay below bottom of excavation
2) Significant depth of clay below bottom of excavation but $N_b < N_{cb}$ (See Chapter 8)
b) Settlements affected by construction difficulties

Zone III
Very Soft to Soft Clay to a significant depth below bottom of excavation and with $N_b \geq N_{cb}$

	Depths of Excavation, ft.
Soft to Medium Clay	
● Chicago, Illinois	30 - 63
o Oslo, Norway, Excluding Vaterland 1,2,3	20 - 38
▼ Oslo, Norway, Vaterland 1,2,3	32 - 35
Δ Stiff Clay and Cohesive Sand	34 - 74
□ Cohesionless Sand	39 - 47

Note:
All data shown are for excavations using standard soldier piles or sheet piles braced with cross-bracing or tiebacks.

Fig. 22. Summary of Settlements Adjacent to Open Cuts in Various Soils, as Function of Distance from Edge of Excavation

depth of about 75 ft below the street surface, or to the underlying rock. Some of the buildings had a single basement, some possessed two basements, and a few possessed three or more. The settlements associated with approximately 20 building sites are plotted in Fig. 23 as a function of the distance from the edge of the excavation. The figure is subdivided into three parts according to the number of basements. Since the general features of the caisson foundations themselves did not differ depending upon the number of basements, the significant difference in settlement evidently depends upon whether the structures possessed one, two, or three basements.

The upper 15 ft of soil in downtown Chicago usually consist of fill and sand. Below these materials is a stiff clay crust from 2 to 4 ft thick, underlain by very soft to soft clays. The underlying clays are successively stiffer until the hardpan is reached. Thus, buildings with single basements do not involve excavation into the soft clays. Most of the settlements associated with such construction may be attributed to the excavation for the caissons. Buildings with two basements ordinarily extended to a depth of about 25 ft or roughly 10 ft into the clay deposit. Those with three basements were on the average about 10 ft deeper. The curves in Fig. 24, derived from those shown in Fig. 23 and plotted in dimensionless form, are approximately the envelopes of settlements associated with excavations to depths of approximately 25 and 35 ft respectively. It is apparent that significant settlements were observed at considerable distances from the excavations. The curve for 3 or more basements agrees substantially with that in Fig. 22 corresponding to the lower boundary for soft clays of limited depth.

The cuts for the basements were made by a variety of procedures. Under most circumstances, a simple sheet pile wall was driven to the proposed depth of the excavation, with a small allowance for embedment, and was supported by cross-bracing or, more often, by inclined rakers. Since most of the buildings were constructed within the period 1920-1940, the settlements correspond to procedures commonly used in that era. The settlement observations were carried out during and for a period of a few weeks or months after excavation for the basements. Hence, it is unlikely that the movements represent consolidation to a significant degree.

Buildings on piles are not immune to settlement due to open cutting in the vicinity, even if the piles extend to resistant materials. For example, the Rand McNally Building in Chicago was supported on timber piles driven into very stiff clays about 60 ft below ground level. When the D-8 open cut (Wu and Berman 1953) was excavated alongside, the building settled as shown in Fig 25. The tendency of the soils to settle near the cut developed negative skin-friction loads on the piles; these loads, in turn, caused penetration of the piles.

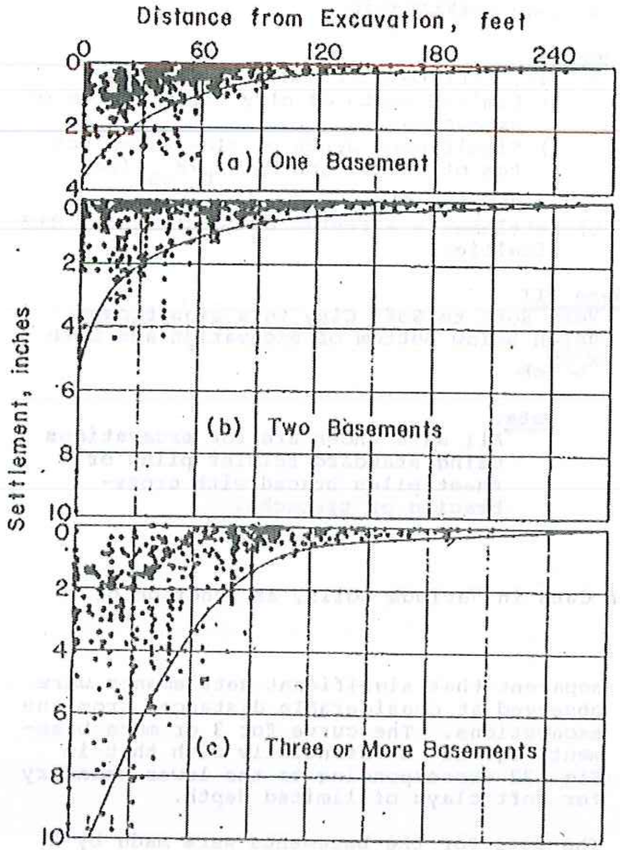


Fig. 23. Settlements Associated with Foundation Construction of Buildings in Chicago on Hardpan or Rock Caissons, and with (a) one basement; (b) two basements; (c) three or more basements

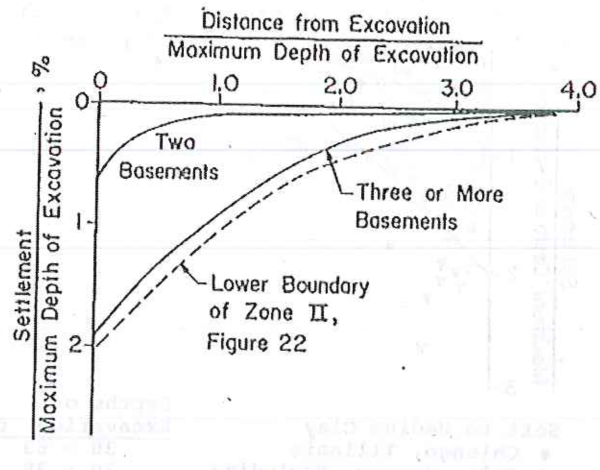


Fig. 24. Summary of Settlements due to Basement Excavations in Downtown Chicago

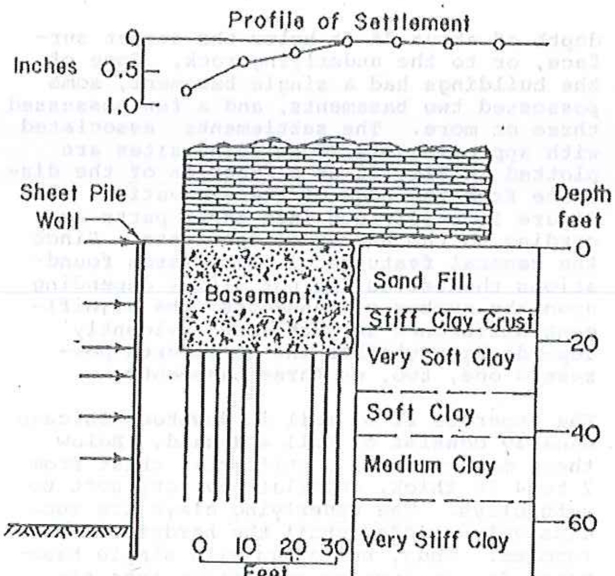


Fig. 25. Section through Pile-Supported Rand McNally Building in Chicago, Showing Settlements Caused by Adjacent Deep Open Cut

Loss of ground and settlement adjacent to cuts in soft clays may occasionally be radically altered by what may appear to be minor changes in construction details. For example, Fig. 26 indicates arrangements for supporting the lagging aginst H-section soldier piles. Method (b) has the advantage of including the soldier pile in the permanent reinforcing of the structural wall. Nevertheless, the settlements adjacent to an excavation made according to this procedure were almost three times as great as those next to a cut made by method (a) by the same contractor at a site in similar clays only a block away. In method (a) the clay outside the cut yields toward the spaces between soldier piles as the clay inside is trimmed

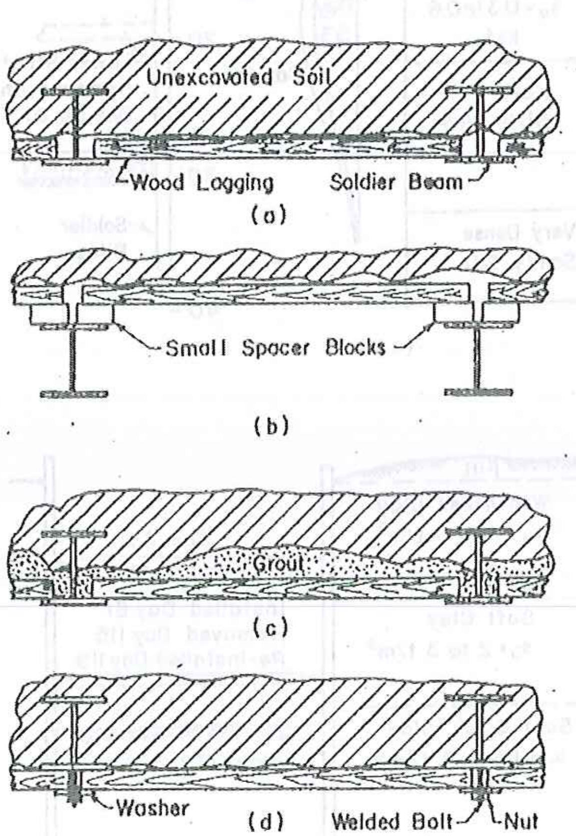


Fig. 26. Methods for Transferring Earth Pressure from Lagging to Soldier Piles; (a) Lagging Wedged Against Inside Flanges of Soldier Piles; (b) Lagging Set Behind Outside Flanges of Soldier Piles; (c) Grout or Mortar Filling Between Lagging and Soil; (d) Contact Sheeting

away to make room for the lagging. The earth pressure is transferred directly through the clay to the H-piles; very little pressure is transferred to the piles by the lagging. In method (b), pressure is similarly transferred to the H-piles as excavation is carried up to the face of the piles, but the heavily loaded soil behind the piles is then cut away to make room for the spacers and lagging. As the supporting, highly stressed clay is removed, the mass of clay behind the soldier piles moves energetically inward with corresponding loss of ground. Poor contact or space between soil and lagging may permit additional undesirable loss of ground. Where it will not interfere with drainage into the cut, grout or mortar may be used to fill the voids (Fig. 26c). Contact sheeting (Fig. 26d) may permit a tight fit because the surface of the exposed soil can be trimmed accurately.

Stiff Plastic Clay

The reduction in vertical pressure due to the excavation of a cut of given depth becomes less instrumental in causing settlements as the strength and stiffness of the clays increase. In some instances the predominant movement is a rise of the ground surface instead of a settlement. The rise is a more or less elastic response to the general reduction in load.

In Houston, several large excavations were made to depths ranging from 39 to 60 ft in the stiff clays typical of the area. Vertical movements of the nearby ground surface were generally within the tolerance of ordinary engineering surveys. Indeed, small movements of bench marks throughout the city, due to areal subsidence caused by groundwater lowering, are commonplace and are of the order of the indicated movements near the construction sites. The observations, if taken at face value, in some instances suggest a slight rise and, in others, a slight settlement. The groundwater lowering necessary during construction undoubtedly reduces the potential rise. The soldier piles themselves, however, rise appreciably. At one excavation 53 ft deep, the piles rose 1/2 inch. At the site of the 60-ft excavation previously described, the cylinder wall rose about 1 inch during excavation (Focht 1969).

2.2.4 Settlements due to Removal of Struts

Because inward movement at and below excavation level (and, consequently, settlement) can be kept to the minimum compatible with the dimensions of the cut and the soil conditions only if struts are inserted promptly after each stage of excavation and at closely spaced vertical intervals, the maximum permissible vertical spacing is often set forth in contract documents. Each strut, as soon as it is inserted, serves to restrict the movements of the sheeting or soldier piles while the next stage of excavation is

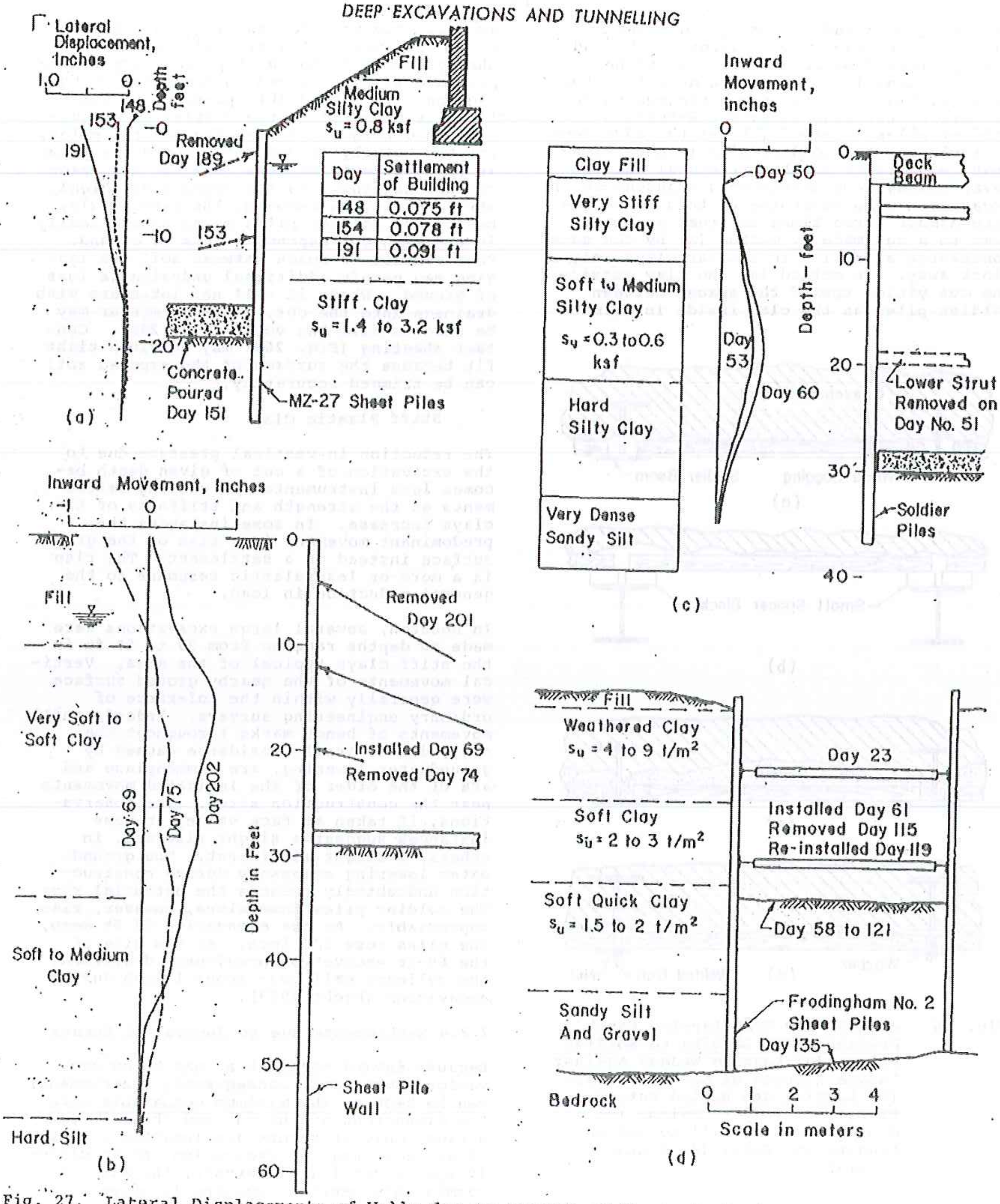


Fig. 27. Lateral Displacements of Walls due to Removal of Struts in Various Open Cuts in Clay: (a) and (b) Raker-Supported Cuts in St. Louis and Chicago respectively; (c) Chicago Contract K5; (d) Telegraph Building, Oslo. Parts (a), (b) and (c) are drawn to the same scales.

carried out. As soon as the next lower strut is placed, its predecessor no longer is so effective in reducing deep-seated movements. In many instances it could be removed without appreciably altering the pattern of deformation, provided the soldier piles, wales and remaining struts could carry the added load to which they might be subjected. Hence, if the excavation has been carried to grade and all struts have been placed, one or more of the intermediate struts can often be removed without causing large enough inward deflections of the sheeting to produce significant settlements. The removal may greatly facilitate construction that is to be carried out in the cut.

Deflections due to the removal of struts are illustrated in Fig. 27. Fig. 27a refers to the cut in St. Louis illustrated in Fig. 21. Removal of the lower raker caused a maximum deflection of less than 0.2 inch and a settlement at the building line within the accuracy of the leveling observations. Removal of the top raker was accompanied by a considerably larger movement of a pattern suggesting compression of the backfill or the closing of void spaces between the permanent basement wall and the soil.

Similar information is shown in Fig. 27b for the Chicago cut illustrated in Fig. 20. Excavation of the final portion of the berm before completion of the base slab, and subsequent removal of the lower raker, caused an inward bulge of about 0.3 inch. Removal of the top two rakers and the transferral of load from these rakers to the permanent structure again led to considerably greater movements.

When the strut at a depth of 19 ft was removed from Chicago cut K5, for which the movements during excavation were shown in Fig. 19, an inward movement of about 0.5 inch occurred (Fig. 27c). The span of the soldier piles was increased by removal of the strut from about 12 to about 22 ft.

Finally, in Fig. 27d is shown a cross-section through the excavation for the Telegraph Building in Oslo. At one level, four adjacent struts were temporarily removed. Upon removal the total inward movement of the two sides was about 2 cm; at one location it reached 3.3 cm. The lateral movement of one side of the wall at the same level during excavation was about 4 cm. It is probable that the total inward movement of the two sides during excavation was about 8 cm.

One set of observations in a cut in sand indicated an inward deflection of the soldier piles of 3 mm when the lowest strut at a depth of 8.20 m was removed. The strut above the one removed was at a depth of 4.50 m; the depth of the cut was 11.20 m (Müller-Haude and von Scheibner 1965).

2.2.5 Conclusions

The minimum settlements that can be expected, corresponding to the best open-cut construction practice, vary considerably with the type of soil. They are likely to be negligibly small adjacent to cuts in dense sands and relatively stiff cohesive granular materials. On the other hand, they are likely to be excessive adjacent to cuts in soft plastic clays. Such settlements can be reduced only by a radical change in construction procedures. Several alternatives are considered in Chapter 9.

Settlement adjacent to open cuts in sand may be caused by the erratic loss of ground associated with seepage or by runs of strictly cohesionless material. The location and magnitude of such settlements cannot be predicted. Their avoidance lies in improved control of the groundwater and in careful attention to construction details. If settlements of the foregoing type do not develop, more regular and predictable settlements adjacent to the cuts may be anticipated. The settlements are a consequence of strains in the mass of soil associated with the relief of stress caused by removal of the excavated material. To some extent, the settlement can be reduced by introducing points of support for the side walls of the bracing system as soon as practicable and at appropriately close vertical spacing. If these steps are taken and if, in addition, the workmanship is good so that unfilled voids are not left outside the supports, the settlements will be reduced to the minimum compatible with the soil conditions and the general bracing procedure.

2.3 Base Failure by Heave

2.3.1 Soft Clays

The soil surrounding an excavation tends to act as a surcharge beside that remaining below excavation level. If the surcharge is great enough, a bearing capacity failure may occur. The danger of a base failure of this type arises only when the soil beneath excavation level behaves essentially as a frictionless material under undrained conditions.

The extent to which a state of failure below the bottom of the cut is approached may be judged by values of the dimensionless number $N_b = \gamma H/s_{ub}$, where s_{ub} is the undrained shear strength of the soil below base level. If the strength of the soil constituting the surcharge is ignored, and if the cut is considered to be infinitely long, theoretical studies indicate that a plastic zone should start to form at the lower corners of the cut when N_b reaches 3.14. The zone should spread with increasing values of N_b until base failure takes place. At this stage, N_b equals the critical value $N_{cb} = 5.14$.

Accordingly, it might be anticipated that for values of N_b less than about 3.14, upward displacement of the bottom of the excavation would be largely elastic and of relatively small magnitude. For values of N_b greater than 3.14 the rise for a given increase in depth of excavation might tend to increase significantly until, at $N_b = N_{cb} = 5.14$, it would occur continuously and base failure or failure by heave would take place.

In reality, cuts are not of infinite extent and the strength of the material acting as a surcharge is not negligible. Simple procedures for estimating the factor of safety against bottom heave in excavations of various rectangular shapes and depths have been proposed (Bjerrum and Eide 1956) that take these factors approximately into account. The values of N_{cb} for cuts of ordinary shapes are usually in the range of 6.5 to 7.5 instead of 5.14. The value of N_b at which the plastic zones first begin to form would similarly be expected to be somewhat greater than 3.14.

The discussion in the preceding chapter has indicated the interdependence of settlement, lateral movement of walls, and upward movement of soil beneath excavation level. As yet, no consistent theory has been developed to describe the transition from elastic to plastic states of a homogeneous material that extends from the ground surface to depths well below the zone of influence of the cut. Furthermore, the influence of the lateral supports, and particularly of the embedded portions of the sheet piles or other retaining side walls, has not yet been taken properly into account. Progress in understanding the problem requires theoretical developments; promising starts have been made with the aid of finite element analyses. In particular, a theoretical basis is needed to permit judgment of the influence of the stiffness and the depth of embedment of sheet piles below excavation level, whether the piles do or do not reach a firm stratum. The beneficial results of such piles are often overestimated.

An interesting series of small-scale laboratory tests to investigate movements of the soil behind a sheet-pile wall was carried out by Whitney (1967). The model sheet piles were rigid. They extended various distances below the bottom of the cut, which was excavated in an extremely soft clay. The general pattern of the movements in one of the experiments is shown in Fig. 28. The pattern shown by the displacements of the entrapped air bubbles is slightly distorted near the glass side of the apparatus because of the flanges on the channel used as the model wall. The results indicated a markedly beneficial effect of embedment but, on account of the

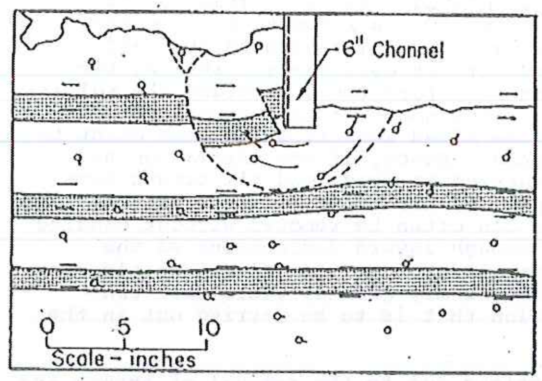


Fig. 28. Displacements of Clay Adjacent to Rigid Model Wall after Excavation on Right Side of Wall. Heavy Dashes Indicate Original Position of Shaded Layers. Symbols Indicate Magnitude and Direction of Movement of Air Bubbles in Clay Mass.

completely unyielding character of the model sheet piles, the quantitative findings are not likely to be applicable directly to the flexible walls actually used in practice.

2.3.2 Stiff Clays

Substantial upward movements of the bottoms of excavations in stiff clays have been reported in the literature. They have in some instances considerably exceeded those that could be considered elastic. The bottoms of the cofferdams for the Waterloo Bridge in London rose as much as 3 inches (Cooling 1948). More recently, a rise of the bottom of an excavation 300 by 25 m in plan and 15 m deep reached as much as 10 cm (Garde-Hansen and Thernøe 1960). In both, the value of N_b at the end of excavation was far less than N_{cb}. Hence, it was concluded that the rise must have been a consequence of artesian pressures in pervious horizontal partings beneath the excavation. Nevertheless, direct evidence of such partings was not evident.

More recent experiences (Bjerrum 1969) suggest that the rise of the bottom of cuts in stiff clays may be associated with passive failure of the soil beneath the excavated zone produced by large lateral pressures existing in the soil mass before excavation. The existence of such pressures associated with values of earth pressures at rest greater than unity has been well established in many localities. Cut slopes in strongly overconsolidated clays with large initial horizontal pressures often fail, when the excavation reaches a certain depth, by sliding along a surface that passes nearly horizontally through or slightly below the toe of the slope. This observation appears closely related to the rise at the bottom of excavations with vertical sides protected by a wall. The wall somewhat modifies the horizontal movements but does not prevent the passive displacements. If the displacements are large enough, base failure by heave may occur.

2.4 Reduction of Settlement

2.4.1 General Principles

If the details of the bracing and of the construction procedure are well designed and executed and if the workmanship is good, settlements adjacent to an open cut can be reduced only by decreasing the lateral movements of the earth supports and the rise of the bottom. The movements have their origin in the general reduction of stress in the soil surrounding the excavation; this reduction, in turn, is caused by the removal of the weight of the excavated material. In the conventional procedures considered in Chapters 7 and 8, the insertion of the struts, rakers or tiebacks that support the sheet piles or other walls is always preceded by excavation. While this excavation is going on, the walls move. Whatever portion of the movement takes place below the bottom of the excavation cannot be prevented irrespective of the capacity of the supports or of the degree to which the bracing is prestressed.

In principle, the movements could be prevented if the entire supporting structure including the sheeting, the wales, the struts or ties, and even the base slab for the completed structure could be constructed in their final positions before the removal of the enclosed soil. Subsequently, upon excavation of the enclosed earth, the settlements of the surrounding ground surface would correspond only to those associated with the deflections of the bracing system and floor slab. These movements would be extremely small compared to those that occur during excavation before the structural systems are complete.

Such an idealized procedure can exist only in imagination. It may be approached in practice, however, in two different ways. On the one hand, the amount of material excavated may be reduced to the absolute minimum required for installation of the walls and bracing. Only after the bracing system is complete is the main mass of soil excavated. This procedure is especially effective if the bottom of the proposed excavation reaches or approaches firm material which restricts the rise of the base that would otherwise take place. The second alternative is to reduce the change in stress caused by excavation by keeping the hole full of water or slurry, or even compressed air. The permanent structure, or the temporary bracing system and bottom slab, are completed before the fluid is pumped out or the air pressure removed. Combinations of the two alternatives are also possible.

The same principles and procedures for reducing settlement are also generally applicable to increasing the factor of safety of the bottom of a cut against a base failure by heave.

2.4.2 Trench Method

The principle of reducing settlement by delaying excavation until the supporting walls and bracing are complete has been appreciated and used for several generations in localities where the subsoil consists of deep deposits of soft clay. Beginning about 1900, the so-called trench method was used in Boston, Chicago, and Detroit. According to this procedure, a trench was excavated by hand around the periphery of a building at the proposed locations of the permanent basement walls. The width of the trench was as small as practicable. The sides were supported by timber planks and large numbers of horizontal trench braces or jacks extending from one side to the other. The completely sheeted trenches were carried to the full depth of the outside walls of the structure. Reinforcement was then set in the trenches and the concrete placed directly against the sheeting which served as the form.

Simultaneously with the wall construction, cross-trenches were excavated, generally along the column lines, and were similarly timbered and braced to the same depth as the external walls. Concrete struts, later forming part of the lowest basement floor, were then cast in the bottoms of the trenches. The building columns were established at the intersections of these struts, and the floor beams for the upper levels of the basement floors were erected. The floor beams, extending from side wall to side wall, served as struts. In this manner, a complete system of cross-lot bracing was established while most of the soil within the future basement area was still in place. As a final step, the soil was excavated, but the only inward movement of the permanent walls was that associated with the elastic shortening and deflection of the permanent structure. Hence, the settlements were minimized.

In some localities such as Detroit, the adjacent settlement associated with rise of the bottom led to the development of a conservative alternative to the foregoing procedure. The work was carried out as just described, up to the stage of excavating the enclosed mass of soil. At this stage, in order to reduce the change in stress due to removal of the full weight of the soil, the excavation to subgrade level was carried out in small sections at a time, and the floor slab was cast in that section and temporarily backfilled until other sections had similarly been completed. Thus, the full weight of the overlying soil was not removed until the lowermost basement floor slab was structurally capable of resisting upward loads.

Many variations of this procedure were used to suit varying conditions. They were generally successful in greatly reducing settlements in congested areas although they were costly.

In the last few years the original trench method has been modernized by the use of various techniques for constructing a stiff outer wall of cast-in-place concrete before general excavation. These techniques include drilling slurry-filled holes and constructing a stiff outer wall of cast-in-place concrete before general excavation. These techniques include drilling slurry-filled holes and constructing overlapping vertical cylinders of reinforced concrete, or of excavating slurry-filled trenches in which reinforcement is placed and concrete introduced by tremie methods. Some of the many variations are patented.

2.4.3 Cast -in-Place concrete Walls

The principal advantages of concrete walls cast in place in slurry-filled holes or trenches are the ability of the slurry to reduce loss of ground during construction of the walls, the elimination of vibration and disturbance associated with pile driving, and the strength and stiffness of the walls themselves. In some instances, the walls may serve as the exterior walls for the finished structure.

If the objective of the use of the cast-in-place concrete walls is reduction of adjacent settlement and possibly elimination of underpinning of some adjacent structures, the results are likely to be disappointing unless lateral supports such as struts or tiebacks are placed either before the general excavation is made or in stages as excavation proceeds. Even the apparently very stiff walls formed by the various cast-in-place procedures accommodate themselves to a high degree to the movements of the mass of soil in which they are enclosed. Hence, such walls are not capable of eliminating, but only to some extent of reducing, the inward lateral movements associated with general excavation.

The Soldier Pile Tremie Concrete or SPTC wall (Thon and Harlan 1968), used recently in the San Francisco area and particularly on the BART system, exemplifies the various procedures and permits a comparison of the rigidity of such construction with that consisting of standard soldier-pile or sheet-pile walls. In its completed form it consists of a concrete wall in intimate contact with the soil on each side. The wall is reinforced vertically by steel soldier piles spaced at about twice the thickness of the wall.

According to the SPTC procedure, the soldier piles are placed in predrilled slurry-filled holes. A slot is excavated, still filled with slurry, by means of a special bucket that uses the flanges of two adjacent soldier piles as guides; the slot is then filled with tremie concrete. The diameter of the holes is equal to or slightly less than the depth of the steel wide-flange sections to be used as the soldier piles (Fig. 29). Thus, when the wide-flange sections are inserted in the holes, their flanges are in intimate contact with the soil. The holes eliminate the need for pile driving and insure the verticality of the soldier piles.

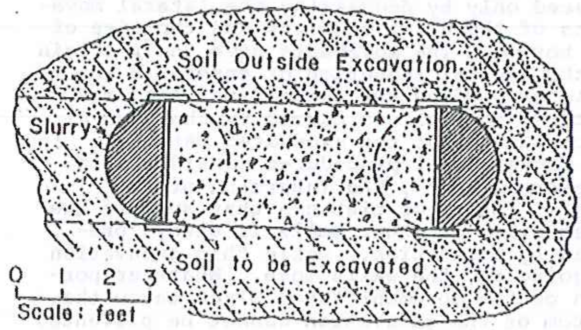


Fig. 29. Horizontal Section through Typical SPTC Wall

For fairly heavy SPTC walls, the soldier piles consist of steel beams 36 inches deep, spaced at about 6 ft. If the heaviest wide-flange section, 36WF194, is used at 6 ft centers, the stiffness of the wall, EI per ft of length, is approximately 200×10^9 lb in^2/ft. Of this stiffness, about 70% is furnished by the concrete, assumed to be uncracked, and the remainder by the steel. The corresponding stiffness of the heaviest U.S. rolled sheet-pile section, ZP38, is 8.4×10^9 lb in^2/ft. Hence, the ratio of stiffnesses of the two walls is approximately 23.8.

The deflections of the walls under a distributed loading depends, however, not only upon the inherent stiffness of the walls, but approximately on the fourth power of the distance between supports. Therefore, the relative deflections of the two walls under similar systems of loading would be approximately in the ratio of the fourth roots of the ratio of the stiffnesses, or about 2.2. Hence, it might be concluded that if a vertical spacing of struts of about 10 ft were needed in order to keep lateral movements of the sheet piles within tolerable limits as excavation proceeded downward, the substitution of the SPTC wall would not reduce these lateral movements significantly unless the spacing of the struts were less than about 22 ft.

Thus, walls of this type may alleviate the problem of adjacent settlement, but they in themselves do not permit deep excavations without the use of struts or other supports at vertical intervals less than about twice those that could be tolerated with the more flexible sheeting. Such walls should not be regarded as rigid, but might more properly be termed semi-rigid. The same reasoning leads to the conclusion that the use of semi-rigid cast-in-place concrete walls extending well below the bottom of an open cut underlain by a considerable depth of soft material will only reduce but not eliminate the loss of ground associated with inward movement of the walls as the excavation deepens. Nor will it eliminate the problem of base failure by heave. The conception that such walls are extremely strong and rigid, as might be inferred solely from the relative rigidities EI, is misleading and dangerous.

The foregoing comments apply to all the various forms of cast-in-place concrete walls. The necessary lateral support may, under favorable conditions, be provided by struts or tiebacks installed as the excavation is deepened. If the movements would be excessive, the supports must be installed before the material to be excavated is fully removed. The procedures are essentially the same as those used in the old trench method. Where cross-bracing can be utilized as part of the permanent framing for the structure, the method may prove very attractive.

Since the cast-in-place perimeter walls can be made relatively watertight, they are useful in areas where the external water level should not be lowered. Similarly, they are well adapted to the control of running sands and silts. However, if the walls are supported by tiebacks inserted through holes in the wall as excavation proceeds, significant loss of ground may occur because of caving of the anchor holes or flow of cohesionless materials into the excavation through the openings.

2.4.4 Dredging

Open dredging for excavation within cofferdams and caissons, with hydrostatic heads inside the enclosure equal to or greater than the external water level, has been practiced widely for many years. More recently, the technique has also been adapted to large-scale excavations such as those for subway systems.

The danger of settlements due to lowering the water table in loose sand containing organic matter is so great in some localities that groundwater lowering is out of the question. To avoid such difficulties in Rotterdam, for example, a trench was dredged (Plantema 1965) with floating equipment. Sections of the proposed conduits were then prefabricated, towed in, and sunk into position on piles driven after the dredging.

To investigate the feasibility of reducing settlements adjacent to open cuts in the soft Oslo clays, a test section (Di Biagio and Kjaernsli 1961; Bjerrum et al 1965) was constructed by underwater excavation. The presence of the water reduced the final value of N_b from about 5.5 to about 3.5. The movements were of about the usual magnitudes for cuts in similar Oslo soils in spite of the omission of struts in the lower half of the cut. The bottom was provided with a tremie concrete seal before dewatering.

2 4.5 Air Pressure

In the Scandinavian countries and elsewhere, use has been made of the so-called "upside-down construction". According to this construction, sheet piles are first driven along the boundaries of a proposed excavation. The soil between the sheet piles is then excavated down to the level at which the roof slab of the completed structure can be constructed. If necessary, struts are inserted between the sheet piles as excavation proceeds down to this level. If the movements associated with excavation to this level would be excessive, and if there is sufficient unoccupied ground beside the cut, the ground level on either side of the cut may be temporarily lowered until the roof slab has been placed. The roof slab is then built and supported on the sheet piles. It is then backfilled to provide the weight necessary to resist the upward pressure of the compressed air to be introduced beneath the slab. Excavation then proceeds by tunneling methods beneath the concrete roof slab with the aid of the compressed air. The air pressure reduces the inward movement of the sheet piles and also the tendency of the base to heave. After the base slab has been cast and the permanent walls completed, the air pressure is removed.

Applicability of the method is limited by the air pressure that can be supported against the roof structure without blowouts and, in the event of loss of air pressure, by the necessity for supporting the roof on the sheet piles as bearing piles.

2 4.6 Caissons

From time to time, attempts have been made to construct the substructure of buildings as caissons to be sunk to their final position by internal excavation. One such attempt over thirty years ago in Mexico City led to major disruption of the surrounding ground and to excessive settlements because of the drag-down forces exerted by the descending caisson on the surrounding soil. Difficulties were also experienced in keeping the caissons vertical.

More recently, the method has met with considerable success, particularly in Europe (CEPWR 1967) and in Japan (Takenaka 1964). The successful procedures include means for injecting bentonite as a lubricant around the periphery of the caissons, controls to detect deviations from verticality, and in some instances the use of air pressure in the working chamber at the bottom of the caisson when final cleanup is done. In Japan, use of the cast-in-place concrete wall method has largely supplanted caisson construction (Endo 1968).

2.4.7 Other Procedures

Several procedures, based on improving the controlling physical properties of the soil, have been proposed and used to a limited extent. These include freezing or grouting the soil, and electro-osmosis.

Freezing of the soil to form a retaining wall or ring of frozen ground has been accomplished successfully to permit excavation of large underground openings such as storage chambers for liquefied petroleum gas. As yet, little has been reported about the ground movements associated either with the freezing or the subsequent excavation. Although freezing has thus far been used primarily as a construction expedient, further developments may lead to a more general use (Sanger 1968).

Injection of grout into granular materials has in some instances provided successful support for adjacent structures and has permitted safe excavation by reducing the likelihood of runs. Groundwater lowering before excavation is advisable to eliminate adverse seepage pressures. Movements associated with the injections are not well documented. In many instances the ground surface rises more upon grouting than it settles subsequently because of the excavation; careful control is required.

Electro-osmosis has successfully stabilized slopes in silts and silty clays (L. Casagrande 1962), and has been used experimentally to stabilize the bottom of a cut in very soft clay aginst the possibility of a base failure (Bjerrum et al 1968). The substantial volume changes associated with the procedure often lead to settlements of the surface and the formation of fissures.

2.4.8 Summary

In this chapter it has not been possible to consider all the many procedures that have been proposed and successfully used for carrying out deep excavations with vertical sides. Rather, it has been the aim to point out the principles under which the various methods reduce loss of ground and settlement.

The most serious problems arise in very deep deposits of soft cohesive sediments. Here, the movements of the ground associated with the changes in stress caused by excavation are likely to be so extensive and so deep-seated that the use of semi-rigid walls, or the insertion of struts at closely spaced intervals as excavation proceeds, are likely to prove to be merely inadequate expedients. Significant reductions in the movements can be accomplished only by procedures that reduce the magnitude of the change in stress in the soil mass until the system of supports has been completed and is capable of withstanding the forces and deformations. These ends are accomplished either by delaying the excavation of most of the material until the supports have been completed, or by substituting a fluid pressure or air pressure for the pressures originally exerted by the removed soil.

In other types of ground where movements are likely to be smaller, use of semi-rigid walls may reduce the settlements to amounts so small that buildings need not be underpinned. In such instances, the additional cost of the semi-rigid walls may be more than offset by the reduced cost of supporting the adjacent structures.

Unfortunately, the amount of observational data concerning the magnitude and distribution of settlements is still fragmentary and does not often permit a reliable estimate of the movements to be anticipated for different types of construction in various subsurface materials.

In all methods of excavation, once the system of bracing and excavating has been decided upon, careful attention to the principles of good workmanship is necessary if the settlements are to be kept to the minimum compatible with the method chosen.

2.5 Earth Pressure

2.5.1 Introduction

The available measurements of strut loads in braced cuts were reviewed by Flaate (1966)* and condensed into semi-empirical apparent pressure envelopes by Terzaghi and Peck (1967) for estimating the maximum strut loads that might be expected in the bracing of a given cut. The envelopes, or apparent pressure diagrams (Fig. 30), were not intended to represent the real distribution of earth pressure at any vertical section in a cut, but instead constituted hypothetical pressures from which there could be calculated strut loads that might be approached but would not be exceeded in the actual cut.

The apparent pressure diagram for estimating strut loads in sand (Fig. 30a) agreed well with the data available at the time it was prepared. Recent measurements providing

*Sources of all earth-pressure data used in this study are listed in Appendix I.

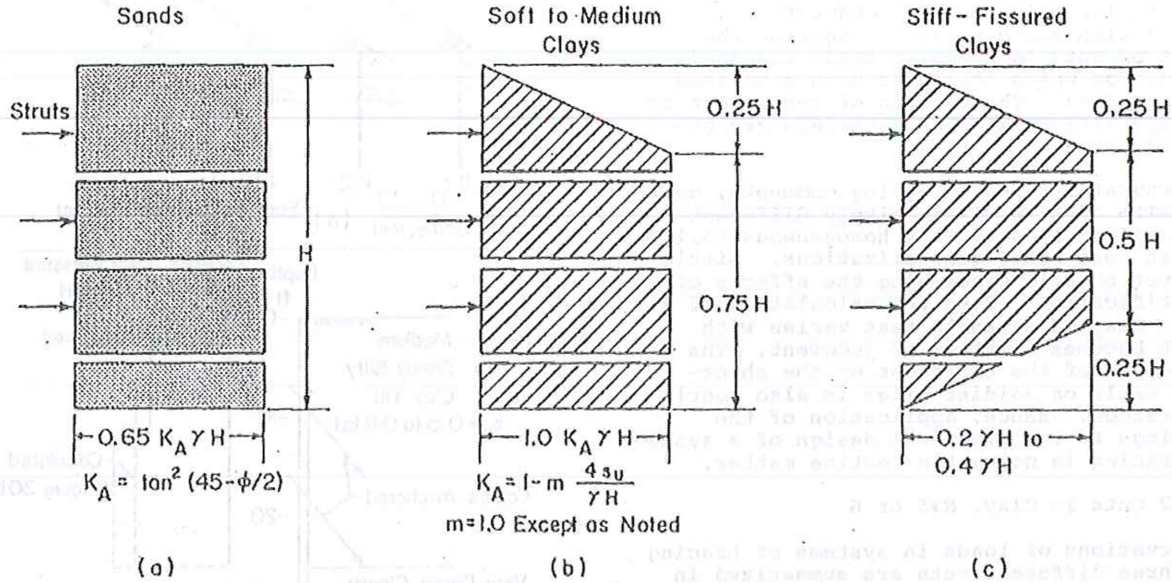


Fig. 30. Apparent Pressure Diagrams Suggested by Terzaghi and Peck (1967) for Computing Strut Loads in Braced Cuts

further data (Müller-Haude and von Scheibner 1965; Briske and Pirlet 1968; TTC 1967) are in agreement with the earlier findings and do not require further discussion.

The semi-empirical procedures for estimating strut loads in soft to medium clays were far less satisfactory. The width of the apparent pressure diagram in the procedure recommended in 1967 (Fig. 30b) is directly proportional to the coefficient of active earth pressure K_A; the same was true with respect to the original trapezoidal rule proposed by Peck (1943). Several authors (Brown 1948, Tschebotarioff 1951) noted that the original trapezoidal rule underestimated the strut loads in the Chicago cuts when the depths of the cuts were still small. The data accumulated since that time, not only in Chicago but elsewhere, support their conclusion. Moreover, the width of the apparent pressure diagram, $K_A \gamma H$, becomes negative, since $K_A = 1 - 4s_u/\gamma H$, when $\gamma H/s_u < 4$. This inequality can be satisfied in a deep cut if s_u is large enough, or in a soft clay if H is small enough. Inasmuch as experience demonstrates that the earth pressure is not zero or negative under these conditions, the approach is obviously invalid.

The information now at our disposal, including recent data to be summarized later in this chapter, appears to justify the following conclusions. The behavior of the soil and the bracing system depends on the stability number $N = \gamma H/s_u$, where s_u is the undrained shear strength representative of the clay beside and beneath the cut to the depth that would be involved if a general shear failure were to occur on account of the excavation. (The stability number N refers to all the soil involved, whereas N_b pertains strictly to the strength at levels below the bottom of the excavation at any stage.) When the depth of excavation corresponds to values of N greater than 6 or 7, extensive plastic zones have developed at least to the depth of the bottom of the cut and the assumption of a state of plastic equilibrium is valid. Semi-empirical procedures for determining strut loads, based on earth-pressure theory, then become more fitting. The movements are essentially plastic and the settlements may be large.

The distinction in behavior for cuts in clays cannot be made, therefore, solely with respect to the softness or stiffness of the clay. It must be drawn on the basis of the behavior of the cut which, for the time being, will be considered to be reflected by relative values of the stability number N. The same cut in its shallow stages may be characterized by values of N less than 3 or 4, and in its deeper stages by values greater than 5 or 6.

The final portion of this chapter is concerned with earth pressures against the sides of cuts in cohesive sands and sandy clays, for which there has been a serious lack of data. The results of one recent set of observations in such materials are presented.

Unfortunately, the foregoing comments, dealing with cuts in distinctively different types of at least fairly homogeneous soils, are at best broad generalizations. Little can yet be said concerning the effects of stratification. Even the calculation of N for a clay of strength that varies with depth becomes a matter of judgment. The influence of the embedment of the sheet-pile walls or soldier piles is also poorly understood. Hence, application of the findings to the practical design of a system of bracing is not yet a routine matter.

2 5.2 Cuts in Clay, N>5 or 6

Observations of loads in systems of bracing in three different cuts are summarized in Fig. 31. In Fig. 31a (Lacroix and Perez 1969) are shown loads in the rakers supporting the sheet-pile wall illustrated in Fig. 21. The significant dimensions and soil properties are given, but no comparison has been attempted between the measured loads and those predicted by any semi-empirical rules because of the complications introduced by the sloping ground surface and the adjacent building.

The apparent pressure diagram corresponding to the horizontal components of the forces in the tiebacks of a cut at the Pickering Generating Station in Ontario are shown in Fig. 31b (Hanna and Seeton 1967). The diagram is compared with the apparent pressure envelope, Fig. 30b, for calculating the maximum load for which the supports should be designed. The agreement is satisfactory.

The forces in several sets of cross-lot struts are shown in Fig. 31c for Chicago Contract K5 (Maynard 1969). No comparison with semi-empirical rules is given because of the uncertainty in evaluating the shear strength representing an appropriate average when a soft layer near mid-height of the cut is sandwiched between an overlying stiff clay and an underlying hard one.

In all three examples the movements of the supported walls were moderate. The loads at the Pickering Station, the only one of the cases lending itself to a simple calculation, were in good agreement with those predicted by the semi-empirical procedure. Remarkably different results were obtained from the observations made for a cut 11.5 m deep in the soft clays of Mexico City (Rodríguez and Flamand 1969). The soil profile, the general dimensions of the cut, and the diagram presented by the authors

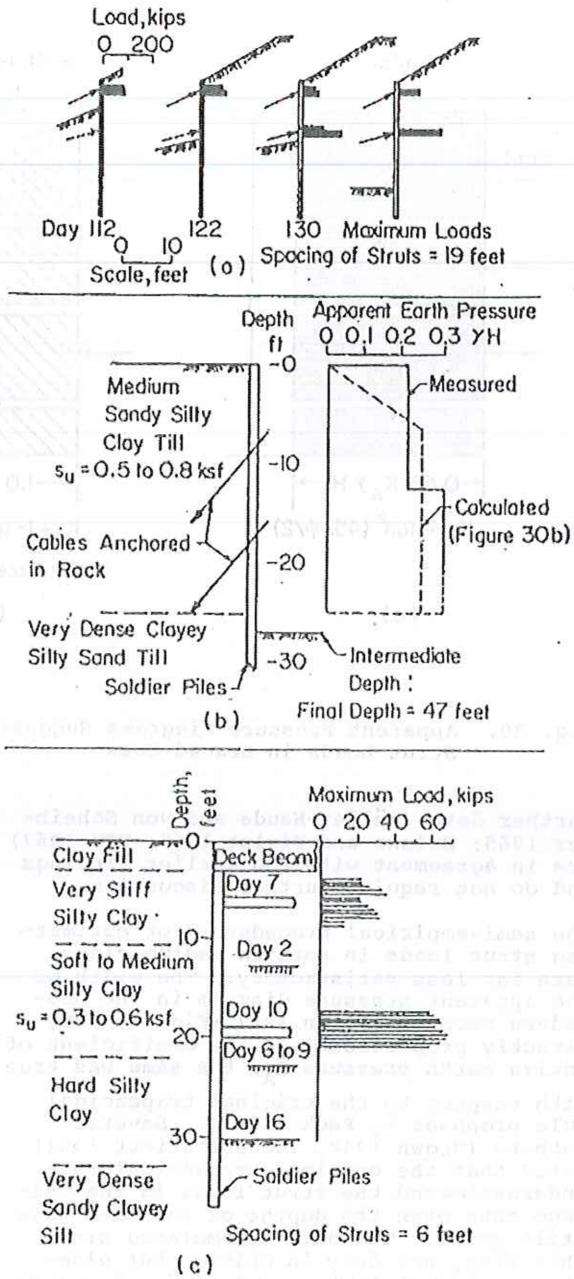


Fig. 31. Loads in Bracing Systems of Several Cuts in Clays. (a) Horizontal Components of Loads in Inclined Bracing of a Cut in St. Louis, Illustrated in Fig. 21; (b) Apparent Pressure Diagrams from Forces Measured in Tiebacks at Pickering Generating Station, Ontario; (c) Preliminary Values of Forces in Struts, Contract K5, Chicago

representing the apparent total earth pressure, as determined by the strut loads, are shown in Fig. 32. Movements of the sheeting are shown in Fig. 19. Fig. 32 also indicates the trapezoidal apparent pressure diagram computed in accordance with Fig. 30b. It is obvious that the measured apparent earth pressures greatly exceeded the calculated ones based on the value m = 1.

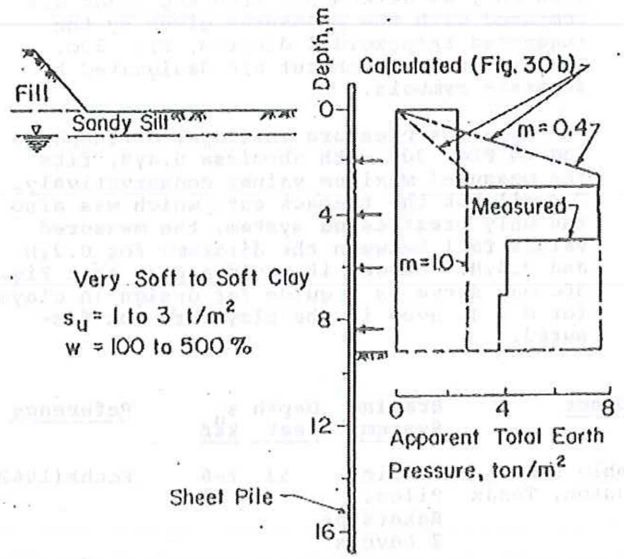


Fig. 32. Strut Loads for Cut in Soft Clays in Mexico City (See Fig. 19)

These findings are similar to those noted with respect to several of the open cuts in Oslo (Vaterland 1, 2 and 3) in which the depth of excavation corresponded to values of N equal to about 7 or 8 (Flaate 1966). In these particular Oslo cuts, the soft clays extended to a considerable depth below the bottom of the excavations. Under these circumstances, it was noticed that the settlements surrounding the cuts and the inward movements of the sheeting were extremely large and that the strut loads were very much larger than those that would be predicted by the use of Fig. 30b with the factor m taken equal to 1.0. Flaate found that the earth pressure diagram for calculation of strut loads in the Vaterland cuts would fit the observed data better if m, which constitutes a reduction factor on the shear strength, were taken equal to 0.4.

For the cut observed by Rodríguez and Flamand the calculated value of N when the cut was at its full depth was approximately 6, neglecting the effect of the surcharge remaining around the cut after some of the surface material was stripped away (Fig. 32). Since the surcharge undoubtedly had a considerable effect by the time the cut reached its full depth, the real value of N was probably closer to 7 or 8. Furthermore, the soft clay extended to great depth below the bottom of the cut and, indeed, was somewhat softer below the bottom than above. Hence, the conditions were strikingly similar to those at the Oslo cuts where large movements and loads were experienced. For Oslo, the most appropriate value of m appeared to be 0.4. If an apparent earth pressure diagram were constructed for the Mexico City cut using the value m = 0.4, as shown in Fig. 32, the maximum strut loads computed from the diagram would be in reasonably good agreement with those observed. The lateral movements of the sheeting of the cut in Mexico City, even below the bottom of the excavation, were exceptionally large, not unlike those for the Oslo cuts.

Thus the Vaterland cuts in Oslo and the cut in Mexico fall into a separate category characterized by exceptionally large pressures and movements. The values of N reached 6 to 8, but similar values have been reached in other cuts without comparable magnification of effects. In the other, more ordinary, cuts the depth of movement was, however, usually limited by a stiff layer near the bottom of the cut and the depth of the plastic zone could not greatly exceed the depth of the cut. The corresponding earth pressure appeared to be governed by the coefficient

$$K_A = 1 - \frac{4s_u}{\gamma H}$$

which is the unmodified Rankine-Résal equation for active earth pressure.

On the other hand, if at values of N = 6 to 8, there is beneath the cut a great depth of soft material, the depth of the surface of sliding is by no means limited approximately to the depth of the cut. The earth pressures may therefore be much larger (Kane 1961) and very large settlements also may be experienced. Under these conditions, any rational earth-pressure theory should take account of the greatly increased depth of the plastic zone. As an expedient pending the development of the appropriate theoretical treatment, modification of the shear strength by the reduction factor m has been suggested. For the two groups of observations at our disposal, the value m = 0.4 appears to be in agreement with the measurements.

There do not as yet appear to be any sets of observations seriously at variance with these generalizations. Unfortunately, however, the evaluation of the properly representative value of s_u for calculating either N or K_A is very uncertain and involves much judgment, especially if the clay near and below the bottom elevation of the cut consists of layers of widely different strengths.

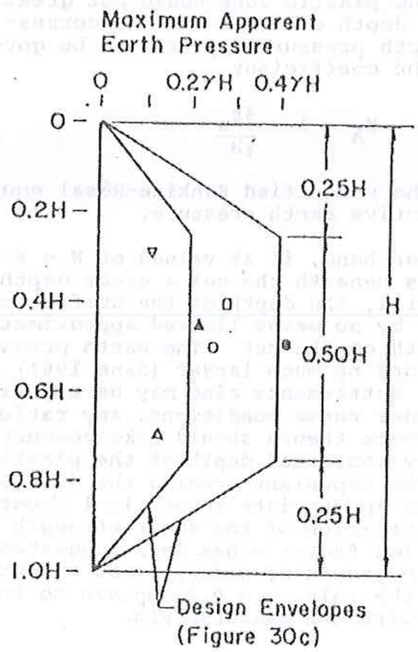
2.5.3 Cuts in Clay, N < 4

If the dimensions of the cut and the strength of the clay are such that $s_u > \gamma H/4$, classical earth-pressure theory suggests that the earth pressure against the bracing should be zero. In fact, of course, the pressure has a positive value. The discrepancy arises because of the attempt to apply a theory of plastic equilibrium to a material not in a plastic state. For values of N less than about 3 or 4, earth pressure theory should not be used, whether the cut is a shallow one in soft clay or a deeper one in stiff clay.

Measurements of strut loads in the initial stages of excavation of cuts in soft to medium clays, and the results of observations in a test trench in stiff clay in Oslo (DiBiagio and Bjerrum 1957) indicated that the shape of an apparent pressure diagram serving as the envelope of the measured strut loads at any stage could also be represented by a trapezoid. Terzaghi and Peck (1967) suggested that the width of the trapezoid would correspond to the range $0.2\gamma H$ to $0.4\gamma H$ (Fig. 30c). Few observational data were available to demonstrate the validity of the suggestion.

Measurements have now become available of loads carried by the support systems of five cuts for which N is less than 4. Two of these are in Houston, two in Toronto, and one near St. Louis. The measured maximum apparent pressures, at each strut level in each cut, as determined from the loads are compared with the pressures given by the suggested trapezoidal diagram, Fig. 30c. The values for each cut are designated by separate symbols.

The apparent pressure envelope, corresponding to Fig. 30c with abscissa $0.4\gamma H$, fits the measured maximum values conservatively. For all but the tieback cut, which was also the only prestressed system, the measured values fall between the diagrams for $0.2\gamma H$ and $0.4\gamma H$. Hence, it would appear that Fig. 30c may serve as a guide for design in clays, for N < 4, even if the clays are not fissured.



Symbol	Project	Bracing System	Depth feet	s_u ksf	Reference
o	Humble Bldg., Houston, Texas	Soldier Piles, Rakers at 2 Levels	53	2-6	Focht(1962)
□	500 Jefferson Bldg., Houston Texas	Sheet Piles, Raker at 1 Level	32.5	2-6	Focht(1962)
	Pierre Laclede Bldg., Clayton Missouri	Soldier Piles, Tieback Anchors		2.5-4	Mansur and Alizadeh (1969)
▽		a) 1 Anchor Level	29		
△		b) 2 Anchor Levels	34.5		
	Bloor-Danforth Subway, Toronto Canada	Soldier Piles, Cross Bracing at 3 levels			
●	a) B4233		39.5	1.2-5.4	T.T.C. (1967)
▲	b) B3510		39	1.5-2.5	T.T.C. (1967)

Fig. 33. Measured Maximum Apparent Pressures Compared with Prediction Based on Trapezoidal Diagram, Fig. 30c, for Five Cuts in Stiff Clays

2.5.4 Cuts in Clay, N = 4 to 6

As a cut in a homogeneous clay deepens, N is likely to increase. Between depths corresponding roughly to N = 4 and N = 6, a transition from an elastic to a plastic state should occur in the surrounding clay. Calculated values of K_A (for m = 1) may increase from 0 at N = 4 to much higher values. However, in the transition, the real earth pressure could hardly be expected to decrease to zero from values corresponding to Fig. 30c, and then to increase again. It is suggested that values taken from Fig. 30c be considered lower limits until N increases to the extent that the pressures computed from Fig. 30b become equal to those estimated from Fig. 30c.

2.5.5 Clayey Sands and Sandy Clays

The results of observations on one cut in dense clayey sands and stiff sandy clays, both with silty components, are shown in Fig. 34. The diagram indicates an arched distribution of earth pressure with center of pressure near mid-height of the cut.

Apparent pressure diagrams have been computed on the assumption that the shearing resistance of the soil consisted entirely of friction, for several different values of ϕ, by the empirical procedures recommended for cohesionless sands. The comparison is shown in Fig. 34. The drained friction angle ϕ' of the clayey sands is on the order of 35°; for this value and c' = 0 the apparent pressure diagrams agree reasonably well with the computed ones. The consolidated-undrained parameters in terms of total stresses are approximately c = 0.6 ksf and ϕ = 16°; these values give pressures much higher than the observed ones if the width of the apparent pressure diagram is taken as 0.65 $K_A \gamma H$, and K_A is computed by the expression

$$K_A = \tan^2(45^\circ - \frac{\phi}{2})\left(1 - \frac{4c}{\gamma H \tan(45^\circ - \frac{\phi}{2})}\right)$$

Predrainage was accomplished quickly at the

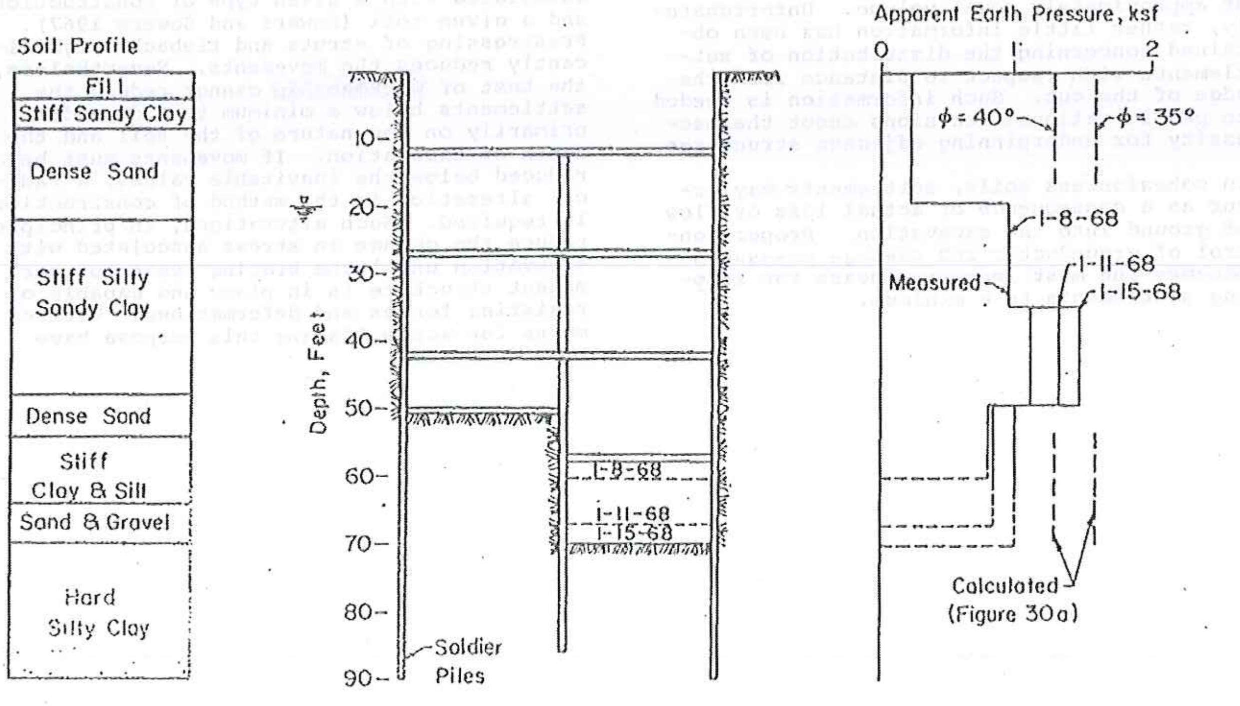


Fig. 34. Strut Loads in Cut in Dense Clayey Sands and Stiff Sandy Clays in Oakland, California

site, and it is probable that the changes in pore pressure due to the excavation also occurred quickly. Therefore, the drained parameters are probably more reasonable.

2.6 Conclusions

A substantial amount of empirical data has become available during the past decade. Consequently, our understanding of the movements and forces associated with open cutting operations has greatly improved.

Observations of the lateral movement of vertical sheet piles or soldier piles have proven particularly informative. They demonstrate clearly, at least in clays, that while excavation is actually going on, lateral movements of the piles or sheeting take place below the level of the lowest strut in place and even below the level of excavation itself. The magnitude of the movements depends strikingly on the nature of the soil and on the depth of excavation. Extraordinarily large movements appear when the depth of a cut in clay approaches that corresponding to N = 6 or 7 and when a considerable depth of similar clay also extends below the bottom of the excavation.

The lateral movements are significant because they are associated with settlements of approximately equal volume. Unfortunately, rather little information has been obtained concerning the distribution of settlements with respect to distance from the edge of the cut. Such information is needed to permit rational decisions about the necessity for underpinning adjacent structures.

In cohesionless soils, settlements may occur as a consequence of actual loss or flow of ground into the excavation. Proper control of groundwater and seepage pressures becomes the most important means for keeping settlements to a minimum.

Heave of the material beneath the bottom may also be a source of lost ground and settlement. It occurs primarily in soft clays, but may develop in stiff clays as a consequence of large lateral stresses under at-rest conditions in the ground. Heave may, of course, also take place if excess hydraulic pressures are allowed to develop below the cut.

The use of tiebacks and anchor systems instead of cross-lot bracing or rakers has become prominent. The relatively little information at our disposal suggests that the pattern of deformations differs from that associated with cross-lot bracing in that the deformations may decrease from top to bottom of the cut. It would be anticipated that under these conditions the distribution of loads among the anchors might more nearly resemble that corresponding to a triangular distribution of pressure than to an arched distribution typical of cross-lot bracing. However, the conclusion should not be drawn until adequate experience with tieback systems has been collected and discussed with a view to developing semi-empirical procedures for design.

Inferior workmanship can easily lead to larger settlements than those inevitably associated with a given type of construction and a given soil (Sowers and Sowers 1967). Prestressing of struts and tiebacks significantly reduces the movements. Nevertheless, the best of workmanship cannot reduce the settlements below a minimum that depends primarily on the nature of the soil and the depth of excavation. If movements must be reduced below the inevitable values, a radical alteration in the method of construction is required. Such alterations, in principle, reduce the change in stress associated with excavation until the bracing system or permanent structure is in place and capable of resisting forces and deformations. Various means for accomplishing this purpose have been discussed.

RESUME

Les ouvrages nécessitant la réalisation d' excavations profondes ne peuvent être conçus d'une façon rationelle que si l'igénieur est capable d'estimer les possibilités de construction par les différents moyens mis à sa disposition, les pertubations susceptibles d'être causées aux constructions voisines, les forces ou les déformations auxquelles les structures temporaires et définitives pourrent être soumises. Dans bien des cas, le choix entre la réalisation d'une fouille ou celle d'un tunnel dépend plus de la nature des pertubations prévues dans le sol que des forces auxquelles les structures seront soumises.

La construction d'un tunnel sera satisfaisante, si les immeubles, les rues ou les installations d'utilité publique avoisinates ne sont pas endommagés d'une manière excessive et si, l'ouvrage une fois terminé est capable de supporter les conditions auxquelles il sera soumis pendant sa durée d'utilisation. Parmi ces conditions, il est d'usage de considérer la poussée des terres comme la plus importante. En fait bien d'autres facteurs ont un rôle prépondérant dans le comportement des tunnels. L'ordre dans lequel les phases de la construction sont réalisées et le comportement du sol avoisinant pendant la période de construction introduisent des complications. Du fait de ces complications, le calcul du revêtement permanent ne peut être effectue en utilisant la théorie de la poussée des terres. Ce calcul devrait l'être, de préférence, en se basant sur la prévision des déformations et la connaissance des déformations tolérables.

La possibilite de construction d'un tunnel dans un sol mou a été discutée par Terzaghi en 1950. Certaines modifications ont été apportées à ses conclusions pour tenir compte de l'utilisation des machines de percement. Pour les tunnels dans des argiles non drainées, la validité des criteres de stabilité a été étudiée et les propositions de Broms et Bennermark (1967) ont été d'une maniere générale trouvées acceptables. D'autre part, pour les matériaux ayant tendance à s'éffriter ou à s'ébouler, il est difficile d'établir des critères basés sur une théorie, car l'influence des détails de construction est prépondérante.

Les résultats de mesures des tassements, associés à la construction de tunnels, dans drivers types de sols, pour des conditions différentes, sont présentés dans le rapport. Ils montrent que la distribution des tassements de la surface du sol, au dessus d'un tunnel unique, a souvent la forme d'une courbe d'erreur. L'ordre de grandeur des tassements susceptibles de se produire sous diverses conditions, peut être grossièrement estimé en se basant sur les résultats empiriques contenus dans le rapport. D'autres résultats sont condensés dans la Fig. 9 et permettent d'évaluer approximativement l'étendue latérale de la zone de tassement. Le rapport souligne les irrégularités de la distribution des tassements dues à des éboulements souvent causés par un rabattement insuffisant de la nappe dans les sols pulvérulents.

Les conditions requises pour le calcul des supports permanents et temporaires des tunnels sont étudiées en détail. L'excavation d'un tunnel est toujours accompagnée de mouvements du sol environnant vers le front d'attaque. Ces mouvements changent complétement la distribution et l'intensité des contraintes qui prévalaient dans le sol. Du fait que le revêtement temporaire ou permanent ne puisse être placé avant qu'un mouvement ne se produise, les forces auxquelles le revêtement doit résister sont sensiblement différentes de celles qui se seraient exercées si aucune déformation ne s'était produite. Si la section courante du revêtement a la forme d'un anneau à peu près circulaire, el est vraisemblable que la force de compression finale, qui s'exerce sur cet anneau ne sera pas sensiblement plus grande que celle due à l'action du poids des terres envirennantes. De plus, il est très probable que les distorsions des revêtements, même s'ils sont flexibles, seront petites pour presque tous les types de sol. Les résultats d'observation, présentés pour justifier ces conclusions, indiquent que la résistance au cisaillement, mobilisée dans le sol avoisinant, est toujours un des facteurs principaux du mécanisme de support des tunnels. La perturbation accompagnant la construction d'ouvrages adjacents, et même de tunnels voisins, ne modifie pas cette conclusion, bien qu'il faille prévoir des forces et des déformations supérieures a celles envisagées dans le cas d'un tunnel unique.

L'excavation produit, dans le sol environnant, une distribution favorable des contraintes que devrait être prise en considération lors de la conception des revêtements de tunnel. L'habitude de concevoir un revêtement primaire et un revêtement secondaire devant résister à des pressions des terres à peu près égales à celles existant dans une masse de sol au repos n'est pas justifiée et devrait être abandonnée. Il est préférable

d'utiliser une méthode de calcul qui tient compte de la résistance du sol. Une telle méthode se divise en quatre parties.

1. Evaluer la contrainte périphérique et calculer le revêtement de facon qu'il résiste à la compression. Pour toutes les argiles, en dehors des argiles gonflantes, une limite supérieure de la contraite périphérique peut être prise égale à la pression due au poids des terres au centre du tunnel, au à une valeur légerement plus grande si le coefficient latéral de poussée des terres est initialement supérieur à l'unité.

2. Evaluer la variation de diamètre qui se produirait si un revêtement fléxible était installé, et choisir unrevetement ayant une fléxibilité suffisante pour pouvoir subir la même déformation sans rupture. L'évaluation des déformations doit etre fondée sur des résultats empiriques. Malheureusement les résultats des informations disponibles ne sont pas encore applicables dans tous les cas.

3. Prévoir une rigidité et une résistance suffisante capable de supporter les efforts exercés pendant la construction tels que ceux dus à la réaction des vérins du bouclier et à la construction du revêtement. Ne pas considérer spécifiquement le cas de rupture par flabement géneralisé dans des plans perpendiculaires à l'exe du tunnel.

4. Considérer les modification à apporter dans le cas du percement consécutif d'autres tunnels ou de la construction d'autres ouvrages à proximité du tunnel. L'influence de telles activités peut aussi être évaluée, dans bien des cas, d'après les résultats empiriques présentés dans le rapport.

Le méthode de conception proposée devrait entraîner une économie considérable et, en même temps, ménager une large marge de sécurité et non sur des suppositions qui ne sont pas étayées par les observations faites.

L'excavation des fouilles blindées implique l'enlèvement de poids de terre substantiels. Les changements importants des contraintes, que y correspondent, produisent des déplacements vers le haut du fond de la fouille, des déplacements vers l'intérieur des cotés (même s'ils sont protégés par des murs de palplanches ou de pieux-soldats) et un tassement correspondant de la surface du sol avoisinant. Les relations intimes entre les différents mouvements sont exposées dans les résultats des nombreuses mesures sur des fouilles dans des argiles plastiques et dans des sables.

Le comportement des sols avoisinants les fouilles dans des argiles plastiques semble être fonction, dans une grande mesure, du nombre de stabilité $N = \gamma H/Su$, ou H représente la profondeur de la fouille, γ le poids spécifique du sol, et Su la contrainte de cisaillement dans des conditions non drainées. Pour des valeurs de N inférieures à 4 les mouvements sont assentiellement élastiques. Quand N croît de 4 à 7 où 8 des zones plastiques de plus en plus grandes se forment et les mouvements deviennent importants. Si l'argile molle s'étend sur une grande profondeur au dessous du fond de la fouille et si N est de l'ordre de 7 à 8, une zone plastique très profonde se forme et de grands mouvement, se produisant sur une grande étendue, ne semblent pas pouvoir être évités. De tels mouvements ont été observés dans plusieurs excavations dans des argiles molles à Oslo et dans une excavation profonde à Mexico. D'un autre coté, si le développement de zones plastiques est restreint par la présence de matériaux assez raides, au fond ou près du fond de la fouille, les mouvements sont considérablement réduits.

La présence de murs, soit-disant rigides, au lieu de palplanches ou de pieux-soldats habituels, réduit les mouvements, mais ne les élimine pas. Si les procédés ordinaires de construction conduisent à des mouvements excessifs, il faut apporter des modifications essentielles aux méthodes d'excavation et de blindage. Ces modifications impliquent, soit la construction complète du système de blindage avant d'excaver le sol à l'intérieur de l'enceinte, soit la diminution de la différence de contraintes qui va de pair avec l'excavation elle-même. Le premier procédé comprend la construction en tranchées, développée il y a plusieurs années et ses récentes variantes telle que la construction d'une paroie rigide moulée en tranchée de boue. Dans certains cas des étançons doivent être placés entre les parois moulées de telle manière qu'elles soient complétement étayées sur toute leur profondeur avant de retirer un important volume de terre de l'intérieur de l'enceinte. Le deuxième procédé peut être réalisé par dragage sous l'eau et par excavation a l'air comprimé après avoir construit le toit permament de l'ouvrage.

Le rapport présente des renseignements empiriques qui peuvent permettre d'évaluer l'importance de la perturbation et du tassement de la surface du sol qui accompagnent une excavation par des moyens conventionnels. Après avoir évalué ces grandeurs, l'ingénieur peut alors décider si des moyens d'excavation et de blindage plus élaborés sont nécessaires.

Les forces que le blindage d'une fouille profonde devra supporter peuvent être calculées de façon assez précise par des méthodes semi-empiriques dans le cas d'excavation dans le sable. Dans les argiles il a été prouvé que ces forces sont fonction du nombre de stabilité N Pour des valeurs de N inférieures à 4 aucune théorie de poussée des terres, faisant intervenir les contraintes de cisaillement dans l'argile, ne peut être appliquée et les forces doivent être déterminées de façon empirique. Les valeurs de N entre 4 et 6 représentent une zone de transition entre l'état élastique et l'état plastique pour lequel les théories de poussée des terres devraient pouvoir être appliquées. Pour des valeurs de N entre 6 et 8 des résultats satisfaisants peuvent être obtenus par des procédés semi-empiriques utilisant la théorie simple de Rankine-Résal et une poussée apparente ayant une distribution trapézoïdale.

Pour des valeurs de N supérieures à 8 et quand, de plus, de profondes couches d'argiles sont situées au dessous de la fouille, la zone plastique est très profonde et les forces de poussée des terres s'accroissent d'une façon impressionante. Dans ce cas la théorie classique de poussée des terres n'est plus applicable et le rapport suggére quelques autres méthodes.

L'utilisation du nombre de stabilité permet de distinguer entre les différentes types d'excavation dans les argiles et d'éliminer certaines confusions concernant les possibilités d'application de règles semi-empiriques pour des fouilles peu profondes dans des argiles molles ou pour des fouilles profondes dans des argiles raides. Dans les deux cas La condition N inférieur à 4 peut aussi bien prévaloir et une approche empirique est alors nécessaire, jusqu'à ce qu'une meilleure connaissance du problème soit acquise.

La fiche des palplanches et la profondeur à laquelle elles devraient être battues influent sur le comportement de la fouille, mais aucune conclusion définitive ne peut être apportée car la connaissance de leurs effets n'est encore que sommaire.

Le rapport présente aussi de nouveaux résultats empiriques concernant les forces qui s'exercent sur des fouilles effectuées dans plusieurs types de sols qui n'avaient pas été étudiés jusqu'a présent. Quelques renseignements sur des systèmes d'étançons inclinés et d'ancrages y sont aussi inclus.

ACKNOWLEDGEMENTS

A very large number of engineers replied to the General Reporter's circular letter requesting information and to his subsequent inquiries for additional data. Without this gratifying response many valuable items would not have come to the Reporter's attention. As the store of information grew it became necessary to limit the scope of the subject; hence, not all the contributions were finally included. For these omissions, and for the reduction of many informative projects to no more than a single sentence or an item in a table, the Reporter is indeed sorry. Several contributions arrived too late to be included.

As far as possible, the sources of information have been listed in the text and references, even though the data may have been in the form of unpublished manuscripts or personal communications. Wherever such a reference appears, it represents a contribution for which the Reporter expresses his sincere thanks.

A few individuals must be singled out for their extraordinary interest and assistance. They include Messrs. L. Bjerrum, E. D'Appolonia, M. Endo, J.A. Focht, Jr., T.H. Hanna, T. Hashiba, J.E. Jennings, B.A. Kantey, Y.H. Lacroix, B.I. Maduke, M.A.J. Matich, J.O. Osterberg, R. Parry-Davies, G. Plantema, W.L. Shannon, S. Shiraishi and W.F. Swiger.

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PECK.

APPENDIX I

Sources of all Earth Pressure Data for Braced Excavations in this Study

Soil Type	Project	Source of Information[1]
Granular Soils	Subway, Berlin	Spilker, 1937
	Subway, New York	White & Prentis, 1940
	Spree Underpass, Berlin	Klenner, 1941
	Subway, Munich	Klenner, 1941
	Subway, Berlin	*Müller-Haude & von Scheibner, 1965
	Subway, Toronto, Canada	*T.T.C., 1967
	Subway, Cologne	*Briske & Pirlet, 1968
Clays	Subway, Chicago, Contract S1A	Peck, 1943
	do. S3	do.
	do S4B	do
	do. S8A	do.
	do. S9C	do.
	do. D6E	do.
	do. D8	Wu, 1951; Wu & Berman, 1953
	do. K5	*Maynard, 1969
	Inland Steel Building	Lacroix, 1956
	Harris Trust Building	White, 1958, 1964
	Subway, Oslo, Grønland 1	NGI TR No. 1, 1962
	do. Enerhaugen	NGI TR No. 3, 1962
	do. Vaterland 1	NGI TR No. 6, 1962
	do. Vaterland 2	NGI TR No. 7, 1962
	do. Vaterland 3	NGI TR No. 8, 1962
	do. Grønland 2	NGI TR No. 5, 1966
	Oslo Technical School	NGI TR No. 2, 1962
	Oslo Telegraph Building	NGI TR No. 4, 1965
	Cofferdam, Shellhaven, England	Skempton & Ward, 1952
	Poole Power Station, England	Megaw, 1951
	T-Building, Tokyo	Kotoda, et al, 1959; Endo, 1963
	M-Building, Tokyo	Minomagari et al, 1960; Endo, 1963
	H-Building, Osaka	Endo et al, 1961; Endo, 1963
	New England Mutual Building, Boston	Terzaghi, 1941
	Laclede Building, St. Louis, Missouri	*Lacroix & Perez, 1969
	Pickering Generating Station	*Hanna & Seeton, 1967
	Siphon, Mexico City	*Rodriquez & Flamand, 1969
	Uelandsgate Test Trench	DiBiagio & Bjerrum, 1957
	Park Village East, England	Golder, 1948
	Subway, Toronto, Canada	*T.T.C., 1967
	Humble Building, Houston	*Focht, 1962
	500 Jefferson Building, Houston	*Focht, 1962
	Pierre Laclede Building, Clayton, Missouri	*Mansur & Alizedah, 1969
Miscellaneous Soils	Subway, Tokyo	Ishihara & Yuasa, 1963
	Trench, Ayer Itam Dam, Malaya	Humphreys, 1962
	Maas Tunnel, Rotterdam, Holland	van Bruggen, 1941; Tschebotarioff, 1951
	Subway, Brooklyn, New York	Miller, 1916
	Subway, Oakland, California	*Personal Files
	Subway, Toronto, Canada	*T.T.C., 1967

[1] All projects except those preceded by an asterisk were evaluated and summarized by Flaate (1966).

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