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 lining, and the manner of executing the work is either left to the contractor or is decided rather arbitrarily by the structural designers. The designers, moreover, proportion tunnel linings to withstand certain sets of loads considered appropriate for the subsurface conditions. The design loads are usually assumed to be independent or nearly independent of the type of construction and even of the properties of the soil. More often than not, except possibly for the jacking loads required to advance a shield or machine, the assumed design loads are completely unrealistic. Furthermore, the preoccupation with loads and with the design of the so-called permanent lining leads to the misconception that the design of a tunnel consists of the design of its lining. The subject is far more complex than this concept would suggest.

In the remainder of this discussion, we shall consider the requirements for a satisfactory tunnel from the point of view of applied soil mechanics. We shall examine our ability to satisfy these requirements, and shall consider the needs for improving our knowledge.

1.2 Requirements for a Satisfactory Tunnel

The first requirement for a satisfactory tunnel is that it should be able to be built. It must be possible to advance the hole safely, to maintain the integrity of the opening at least temporarily, and in some instances to preserve the integrity of the opening by such additional means as a permanent lining. There is much empirical evidence upon which to judge the feasibility of tunneling through various subsurface formations by different construction procedures. The reaction of various kinds of ground to the excavation of tunnels of various sizes by various methods has been discussed in considerable detail by several authors. Most of the published information refers to conventional methods of tunnel driving. Nevertheless, experience with tunneling machines is accumulating; it is discussed with respect to the feasibility of tunneling in Chapter 3.

The second requirement is that construction of the tunnel should not excessively damage adjacent or overlying buildings, streets or utilities. With increasing urbanization this requirement may often severely limit the types of construction suitable for a given project. It may in some instances determine whether a facility should be built by tunneling or by cut-and-cover.

The second requirement has further implications. A tunnel cannot be considered successful unless the designers have correctly decided whether it is necessary to underpin the adjacent structures. The requirements for underpinning and the cost of the work enter directly into the initial decisions with respect to location and type of tunnel. No satisfactory conclusions concerning the likelihood or extent of damage to adjacent or overlying structures can be reached unless the designer has an adequate knowledge of the magnitudes of settlements to be expected for tunnels of various sizes constructed through different subsoils by those construction methods that are considered technically feasible. Unfortunately, there is at present insufficient information to permit reliable estimates of damage or even of settlement above tunnels under all conditions. Yet the subject is of paramount importance because, if the problems of adjacent damage and disturbance are not correctly assessed, the basic concept of the design of the facility will be faulty and no amount of precision thereafter can make up for the initial errors in judgment. Because of the importance of this requirement, information concerning the magnitudes of settlements over tunnels under various conditions has been assembled and condensed in Chapter 4.

As a corollary to the preceding requirement, techniques of tunneling are to be preferred that reduce to a minimum the disturbance of the surrounding ground. Attention will be given to the prevention of lost ground and other causes of ground movement.

The third requirement for a satisfactory tunnel is that it should be capable of withstanding during its lifetime all the influences to which it may be subjected. Of these the most obvious, but possibly one of the least important, is the earth pressure between the tunnel and the surrounding soil. In reality, as we shall see, the requirements imposed by the normal earth pressures on the permanent lining of the tunnel are usually rather easily satisfied. Variations from the normal conditions introduce the most significant adverse influences on the lining: the presence, for example, of stations, ventilation shafts, cross-tunnels and other discontinuities; or distortions caused by subsequent tunneling or by carrying out large excavations alongside the completed tubes. Furthermore, the actual stresses in the permanent lining and the deformations of the lining are determined almost exclusively by the details of the method of construction, by the sequence of the construction events, and especially by the behavior of the surrounding soil during the construction period. These conditions practically invalidate the conception of designing the permanent lining for a pressure ascertained on the basis of earth-pressure theory. They indicate on the contrary that design should preferably be based on a knowledge of expected and tolerable deformations. Data will be assembled in Chapter 5 to provide the first steps in establishing such a procedure for design. It will be apparent that the state of the art involves many gaps in the information needed to permit design under all circumstances.

1 3 Feasibility of Tunneling

1.3 1 Previous Criteria

Before the present trend toward the use of tunneling machines, the feasibility of hand tunneling and the criteria for choosing between hand and shield tunneling had already received considerable attention (Terzaghi, 1950). Decisions were based on classification of the ground according to terminology used by tunnel laborers, and on the standup time associated with the type of ground and the dimensions of the drift or heading being excavated.

These criteria remain adequate for the present discussion. They need be supplemented only with respect to the feasibility of advancing a tunnel by means of an excavating machine.

Most of the excavating machines in present use consist of a rotating or oscillating cutter that chisels or shaves the soil from a circular face against which the machine is advanced. The working face may, in stable ground, be entirely unsupported. At the other extreme, in less stable materials the cutting tools may consist of knives in narrow slots in an otherwise solid circular or slightly conical cutting head. The machine is forced ahead by jacks usually reacting against lining already installed, but in some instances against reaction plates expanded against the soil at the periphery of the tunnel. Various means are provided for handling and removing the muck.

Any such device is referred to in this report as a mole. The mole may be mounted inside a cylindrical shell to provide protection against invasion of the surrounding ground. The shell serves most of the functions of a conventional shield; hence, such an assemblage is sometimes called a digger shield.

In soil designated by the tunnel man as firm ground, a heading may be advanced several feet or more, either by hand or by machine, without immediate support. Stiff clays and cemented or cohesive granular materials fall generally into this category. Tunneling in firm ground can be accomplished without danger of collapse because there is adequate time for the erection of whatever supports may subsequently be needed.

On the other hand, in raveling ground, the materials above the tunnel or in the upper part of the working face may sooner or later tend to flake off and fall into the heading. The action is progressive and may lead to open cavities above the tunnels or even to sinkholes at the surface. The raveling can be prevented if at least moderate support is provided at an early stage, before the loosening becomes extensive. Slightly cohesive sands, silts and fine sands gaining their strength from apparent cohesion, and residual soils with relict structure from the parent rock often fall into the category of raveling ground. If the necessary, although small, support is provided promptly, tunneling in raveling ground is readily accomplished. On the other hand, if raveling is permitted to start, loss of the heading may occur during construction or seriously unbalanced or non-uniform loadings may develop against the lining even after the passage of considerable time. In some instances, completed tunnel linings have failed because of the continued development of raveling initiated during mining. Seepage forces caused by the flow of water toward the working face, especially in non-uniform materials, increase the tendency to ravel. Hence, in some instances, raveling ground can be transformed to firm ground by predrainage.

Running ground consists of perfectly cohesionless materials such as dry sand or clean loose gravel. These materials run from any unsupported lateral face until a stable pile is built up at the angle of repose. However, as long as the soil above the roof is supported, as by poling boards or the hood of a shield, the running ground does not fill the tunnel but merely slopes into the heading. The development of such a sloping face may bury the lower part of a mole, stop its progress, and lead to extensive hand excavation to free the machine. Thus far, moles have not proven adaptable to running conditions.

If seepage pressures toward the working face are permitted to develop in what would otherwise be raveling or running ground, the soil may be transformed into flowing ground and advance like a thick liquid into the heading. Unlike running ground, flowing ground may fill much or even all of the tunnel. It may bury a shield or a mole. Hence, tunneling through such materials can be accomplished only with the greatest difficulty and danger unless the materials are first transformed into raveling or firm ground. The transformation may be accomplished in most instances by drainage, by use of air pressure or, occasionally, by injection of chemical grouts. Mere use of a shield, without first eliminating the flowing character of the soil, does not necessarily lead to successful tunneling because of the difficulty of preventing a flow, even if only a small portion of the face is exposed at a given time. If a run or flow begins, however, the presence of a shield greatly increases the factor of safety of the heading against complete collapse. In extremely soft running soils such as the Hudson River silts, it is sometimes practicable to advance the tunnel by shoving a shield almost blind through the materials, displacing them, and permitting the permanent lining to be constructed. Such procedures would, of course, be intolerable if the accompanying displacements of the surrounding soil would be damaging to other installations (see Chapter 4).

Squeezing soils encompass very soft to medium clays. They can be tunneled successfully by hand methods if the rate of squeeze is not excessive. If the rate would be too great to permit installation of the necessary bracing, tunneling by means of a shield may be required. The practicability of either type of construction may be greatly enhanced by the use of air pressure but, because of the impervious nature of the materials, construction is not likely to be facilitated appreciably by drainage. Even if a tunnel can be successfully constructed in squeezing ground, the effects on nighboring facilities may exceed tolerable limits, as discussed in Chapter 4.

It is well recognized that the stand-up time is a function not only of the type of soil, but of the size of the opening in which the working face is attacked. With the exception of plastic clay soils under undrained conditions, theoretical attempts to estimate the factor of safety against collapse of a heading have not yet been successful. In raveling grounds in particular, the stand-up time and the factor of safety are intimately dependent on the details of stratigraphy and on the secondary structure of the soil deposit. Since these details are unpredictable, no satisfactory correlation between factor of safety and measured soil properties can be anticipated.

On the other hand, for plastic clays at depths not less than about two diameters, the criterion for stability developed by Broms and Bennermark (1967) may be useful. According to this criterion, the ratio

$$\frac{p_z - p_a}{s_u}$$

should not exceed about 6. In this expression

p_z = total vertical pressure at depth z of center of tunnel

p_a = air pressure above atmospheric

s_u = undrained shear strength of clay.

Several examples were given by Broms and Bennermark. Further data are shown in Table I.

A study of Table I indicates that tunneling may be carried out without unusual difficulties in plastic clays if the ratio $(p_z - p_a)/s_u$ does not exceed about 5. In shield tunneling, if the ratio is much greater, the clay is likely to invade the tailpiece clearance too rapidly to permit satisfactory filling of the void. For values approaching 7 the shield may become unmanageable because of its tendency to tilt as it advances. The benefits from the use of compressed air are evident.

1.3.2 Feasibility of Use of Digging Machines

The use of a mole or digger shield does not basically alter the conclusions outlined above with respect to conventional hand and shield mining. In firm ground, such machines have often performed rapidly and economically. In slowly raveling ground, if the mole is provided with a shield, the rapid rate of advance and the support for the overlying material frequently eliminate the danger of raveling above the tunnel. Moreover, if the face of the digging machine consists of a disc or flat cone interrupted only occasionally by slots for the digging knives and for entrance of the material, considerable protection against raveling of the face is afforded. Since the face is much less susceptible to raveling than the roof, machines of this type may be highly advantageous in ground that might otherwise ravel.

On the other hand, present types of digging machines provide no better defense against flowing or running ground than do conventional procedures of hand mining or shield tunneling. Indeed, they may be more susceptible to difficulty because the erratic and unpredictable zones where migration and erosion may begin are not so readily detected in machine tunneling. A run may partly fill a digging machine before defensive measures can be taken. Hence, it is even more imperative in machine tunneling than in conventional hand or shield mining that running or flowing ground be transformed at least into slowly raveling ground by predrainage, use of compressed air, or injections. Furthermore, if predrainage is adopted to accomplish the transformation, it must be conscientiously and thoroughly carried out. Otherwise, here and there, seepage toward the face may cause, if not a full-fledged run, at least a slow and perhaps unnoticed migration of materials that may lead to caving, collapse, and even the appearance of sinkholes at the surface. Such events are not rare, but they often come as surprises to all parties.

The interrelationships among stand-up time, rate of advance, and size of opening are illustrated by the history of a tunneling machine excavating through a very stiff clay or clay-shale with well-developed secondary joint structure. The clay would, on account of the joints, be classed as raveling ground. The machine excavated a hole 23 ft in diameter at a rate up to 130 ft per day without difficulty. For one portion of the tunnel, however, it was necessary to increase the diameter. This was done by increasing the diameter of the cutting ring. Because of lower efficiency of the machine in tunneling through the section having a larger diameter, the rate of advance was decreased to 4 to 6 ft per day, but at this rate the stand-up time of the clay over the tunnel was exceeded and raveling became a problem. The raveling at one point where a thin fault zone occurred culminated in the formation of a cavity extending above the tunnel to a height of 35 ft and burial of the machine.

The feasibility of tunneling with machines is also dependent upon the uniformity or variability of the soil being excavated. Different types of excavating knives, chisels or bits are required for different materials; those that work successfully in one material may be ineffective in others. If the line of a tunnel traverses several different mater-

Table I Field Data on Stability of Tunnels in Saturated Plastic Clays

No.	Case	Reference	Soil	Depth z to Tunnel Axis, ft	Tunnel Diameter 2R, ft	Depth / diameter z/2R	Av. Undrained Shear Strength, s_u, ksf	Overburden Pressure, p_z, at Axis, ksf	Air Pressure Pa, ksf	$\frac{p_z - p_a}{s_u}$
1	London, Ashford	Tattersall et al, 1955	London Clay, fissured, plastic	90	9.3	9.7	21.	11.0	0	0.5
2	London, post office	Ward and Thomas, 1965	do.	55	7.7	7.1	7.2	7.0	0	1.0
3	London, Victoria	Ward and Thomas, 1965	do.	85	14.0	6.1	7.8	10.8	0	1.4
4	Ottawa, Sewer	Eden and Bozozuk, 1968	Leda Clay, sensitive	60	10.0	6.0	3.7	6.2	0.6	1.5
5	Antwerp, Gas Storage	deBeer and Buttiens, 1966	Boom Clay, fissured, plastic	253	17.7	14.3	7.8	31.5	0	4.1
6	Detroit, water	Housel, 1942	Plastic glacial clay	68	15.0	4.5	0.8	8.0	3.9	5.1
7	Toronto, subway	Pers. comm.	Plastic glacial clay	43	17.0	2.5	0.7	5.5	1.4	5.7
8	Chicago, subway	Terzaghi, 1943	Plastic glacial clay	36	20.0	1.8	0.44	4.3	1.7	5.9
9	Koto, Tokyo, subway	Shiraishi pers. comm.	Normally loaded sensitive clay	74	23.0	3.2	0.76	5.6	1.2	7.4
10	Osaka, Municipal Railway	Shiraishi pers. comm.	Normally loaded sensitive clay	51	23.0	2.2	0.60	5.0	1.0	6.6

Remarks:

1. Stable. Shield driven.

2,3. Face stable. Walls stood for a length of time, occasional problems with overbreak associated with fissures, stratification, etc.

4. Driven with mechanical shield; wall exposed before liner placement. No problems.

5. Hand mined. Fissured clay formed 45° talus slope at face. Wall and roof unstable except for short spans.

6. Hand mined. Concrete placed daily directly against clay. Some squeeze.

7. Stable. Only 4' clay cover, dense sand above.

8. Hand mined, horseshoe-shaped; stable with moderate squeeze at air pressure of 12 psi; excessive squeeze on drop of air pressure to 7 psi, ratio $(p_z - p_a)/s_u \doteq 7.4$

9. Shield-driven, face supported during shove. Difficult to keep shield from diving; deviations from grade as much as 14 in.

10. Shield-driven; face closed except for 2.3% opening; took in up to 80% of theoretical volume of tunnel. Inward squeeze of clay could not be controlled at air pressure of 0.8ksf. Downward deviation from grade as much as 1 ft.

Note: psi = pounds per square inch
ksf = kilopounds (kips) per sq ft

ials, it may be necessary to change the cutting elements to suit the nature of the material. Furthermore, if different soils are encountered in the same heading, as is commonly the case, it may prove impractical to find a single cutting tool capable of advancing through all the materials that must be removed.

1.3.3 Conclusion

The use of mining machines in soft ground has in some instances led to greatly increased speed of construction and decreased cost. In others, the results have been disappointing. In extreme cases, the machines have been abandoned or their use has been discontinued.

If machine excavation is contemplated, the feasibility is likely to be even more dependent upon complete control of the groundwater, if the ground is likely to ravel or flow, than would be the case in hand tunneling. The high rate of advance of mining machines, and the speed with which roof support can be provided if a mole is combined with a shield, are likely to extend the range of firm ground well into materials that would otherwise be considered of raveling character. However, if the raveling is underestimated, the ground may collapse above and in front of the machine whereupon the machine must be recovered and the heading restored under extremely difficult conditions.

1.4 Damage to Surroundings

1.4.1 Loss of Ground

The construction of every soft-ground tunnel is associated with a change in the state of stress in the ground and with corresponding strains and displacements. The displacements are observed only infrequently below ground because specialized techniques are required, but the accompanying settlements of the overlying ground surface are often noted both directly and indirectly. If they are excessive, they may cause damage to surface and subsurface installations.

Nevertheless, strain and deformation are not necessarily undesirable consequences of tunneling. They constitute part of the mechanism whereby the strength of the soil surrounding the tunnel can be mobilized to improve the stability of the tunnel faces during excavation and to reduce the structural demands on the lining, either temporary or permanent. Without the benefit of the strength of the soil itself, the hazards and costs of soft-ground tunneling would drastically increase.

Long experience has demonstrated that, except possibly in certain swelling clays, no tunneling method has yet been developed in which the strains and deformations are so small that the strength of the soil is not largely mobilized. Therefore, it is quite properly considered good practice to keep the deformations as small as possible, in order to hold the avoidable loss of ground and consequent settlement to a minimum and to prevent deterioration of the soil due to excessive local distortions or remolding. Even so, it is by no means assured that the minimum deformation compatible with mobilization of the strength of the soil will not lead to excessive loss of ground from the point of view of settlement or damage.

If the minimum deformation associated with a specific method of construction will lead to excessive loss of ground, improvement in workmanship alone cannot eliminate the undesirable consequences. The method of construction must be radically altered, possibly by such steps as the introduction of compressed air or even by replacement of tunneling by open cutting or dredging through slurry or water.

The designer is obliged to judge the extent of surface movement or lost ground that would be considered tolerable, and to specify or to grant his approval to construction procedures that will meet the requirements. He cannot fulfill these obligations unless he is able to make reliable forecasts of the inevitable settlement associated with every design or construction procedure that might reasonably be adopted for the work, with proper consideration as to type of soil, groundwater conditions, and geometry and depth of tunnel. He should also be able to estimate the additional settlement that may be associated with various deviations from the best techniques or workmanship for a given construction procedure.

The nature of the inevitable settlements and those associated with workmanship is markedly dependent on the nature of the ground. In a squeezing clay, for instance, the inevitable part of the settlement may merge imperceptibly into that due to imperfect workmanship with no change in the fundamental character of the movement. In a dense silt, on the other hand, the magnitude of the inevitable settlement may be negligible and the overlying ground surface may be nearly undisturbed, but if raveling is permitted to start and is not properly and promptly handled, a sink hole may develop with catastrophic damage within the sink hole in spite of only slight distortion outside.

Although the magnitude and even to some extent the character of the loss of ground and settlement are strongly influenced by the method and details of construction, by far the most decisive factor governing loss of ground is the nature of the surrounding soil, including the groundwater conditions. Hence, in the following sections, the presently available data regarding movements associated with tunneling will be presented for distinctive types of soil. The data are surprisingly meager, in view of the importance of the information in planning and design.

It is not yet possible, except in a few instances, to apportion the lost ground between the inevitable movements associated with a particular method of construction and the additional movements that may arise because of poor workmanship or faulty techniques. Nevertheless, even the presently limited data are useful.

Ordinarily the settlements above a tunnel, unless caused by a local disturbance such as a run into the face or stoping above the crown, are more or less symmetrical about the vertical axis of the tunnel. They form a trough-like depression with a shape roughly resembling the error function or probability curve. The maximum settlement at any cross-section perpendicular to the axis of the tunnel is denoted by δ_{max}. The value of δ_{max} is likely, however, to vary from one cross-section to another. If the soil conditions and tunneling procedures do not change significantly for a representative length of tunnel, the values of δ_{max} usually vary over a fairly well-defined range. The prevalent value will be designated as the normal settlement δ'_{max}. The consequences of tunneling can, to a considerable degree, be judged on the basis of the normal settlement. In addition, consideration must be given to the value of the greatest settlement δ''_{max} that may occur at some cross-section. Hence, in the Tables accompanying the following text, both values are given where the information is available. The tabulated values exclude settlements caused by non-routine events such as runs, blows or local collapses. The likelihood and significance of such occurrences are discussed separately.

The collected data include settlements above pairs of tunnels as well as single tunnels. Since the settlement trough ultimately formed above two tunnels is likely not to be symmetrical, the tabulated values of δ'_{max} and δ''_{max} refer to the maximum settlement at the cross-section under consideration, irrespective of the shape of the settlement curve.

Where there are sufficient data, the total volume of the settlement trough is expressed as a percentage of the theoretical volume of the tunnel excavation. The percentage of the average settlement volume with respect to the theoretical volume is a useful index of loss of ground.

Many, but by no means all soft-ground tunnels can be discussed with respect to loss of ground and settlement on the basis of four principal groupings of soils: granular soils with no cohesion other than that imparted by capillarity; cohesive granular soils; non-swelling stiff to hard clays; and stiff to soft saturated clays. In the following discussions of the effects of tunneling in these materials, only loss of ground due directly to the tunneling will be considered. Supplementary settlements such as those due to groundwater lowering, while of outstanding practical importance, are excluded.

1.4.2 Cohesionless Granular Soils

Fortunately, truly cohesionless silts, sands, or gravels are rare. Tunneling through such materials can be carried out only by complete protection of the top, sides and face of the excavation, as by full forepoling and breasting, or by rendering the materials cohesive by injection of grout. If the materials are dense and the construction procedures are expertly carried out so that no runs occur, loss of ground and settlement are usually negligibly small. On the other hand, if runs occur, particularly if the material is loose, large and erratic subsidences may develop at the surface. Runs may also be associated with the development, near or above the tunnel, of cavities that may remain open temporarily but may collapse and lead to surface subsidence at a later date. Hence, the prediction of settlement over a tunnel in such materials is extremely uncertain because the real settlements depend almost exclusively upon the smallest details of construction. The most important steps in the prediction are to judge whether the tunneling may take place through materials so dry that they will possess not even the apparent cohesion associated with soil moisture, and to assess whether the material is loose or dense.

At most sites, granular materials above the water table contain enough soil moisture to create at least small apparent cohesion. If the tunnel is below ground water level, the water table must be lowered and the soils drained to the extent that there will be no seepage gradients toward the tunnel and that some apparent cohesion will be developed at the tunnel face. According to Chapter 3, such materials after drainage permit successful tunneling by several different methods, especially if they are relatively dense. Loose materials, even though drained, may tend toward decrease in volume under the changes in stress conditions associated with tunneling, whereupon the porewater pressure may increase and counteract the beneficial influence of capillarity.

Unless the ground is adequately drained in advance, runs may occur with attendant large and irregular loss of ground. With adequate drainage, on the other hand, the use of hand mining methods with liner plates and ribs, the use of shields, or the use of moles may all lead to very moderate loss of ground. The greater the relative density of the material, the less the inevitable settlement for a given construction procedure.

If drainage is not thoroughly accomplished before tunneling begins, the consequences may be serious indeed. As we have seen in Chapter 3, a run may completely invade the heading and be associated with the formation of a sinkhole at the surface of the ground or beneath an adjacent building. Unfortunately, most deposits of granular materials consist of layers, lenses, or pockets of materials of different grain sizes, some considerably more permeable than others. Complete drainage of all the elements likely to be encountered during tunneling is difficult to accomplish. A predrainage system may succeed in draining the coarser elements of the deposit, but may leave excessive pressures in the finer lenses or layers. In any event, the time required to achieve drainage of the finer-grained portions may be considerably more than that for the coarse-grained portions. Drainage wells may be spaced at close enough intervals to provide satisfactory and quick drainage of the more permeable zones, but may not be close enough to provide drainage of the finer portions in a reasonable length of time. Hence, tunneling may proceed with little loss of ground through well-drained materials until suddenly, when a poorly drained zone is encountered, a run may develop.

In addition to the possibility of runs, there is a likelihood of slower erosion of cohesionless materials or migration of particles along lines of concentration of seepage. Such conditions are often encountered just above impervious layers or lenses of clay or silt. If the materials through which the flow takes place are in a very loose state, they may also be subject to runs.

Finally, materials possessing apparent cohesion, even if they do not invade the tunnel in a run, are likely to ravel from the roof and face. The implications of raveling are discussed in the next section in connection with tunnels through cohesive granular soils.

The use of air pressure in a perfectly cohesionless granular mass has little if any influence on the loss of ground. On the other hand, if the cohesionless materials are below normal water level, air pressure may reduce or eliminate the seepage gradients toward the tunnel and thereby reduce the tendency for erosion or flow with consequent loss of ground. Under some circumstances, however, air pressure may prove detrimental. If the permeability of the soil permits air to escape from the face or particularly from the crown of the tunnel, the escaping air may dry the soils completely by removing the soil moisture, whereupon the soil may become truly cohesionless and may run. Such runs may lead to sinkholes.

The use of grout to transform the granular material into a cohesive soil may substantially reduce the settlements. Nevertheless, if grouting is done in lieu of groundwater lowering, a danger exists that high seepage pressures may act through a small volume of ungrouted material of fine grain size and may cause a run. Inasmuch as it is rarely possible to achieve complete injection of all the materials surrounding a proposed tunnel, the likelihood of an ungrouted window is rather great.

Data are assembled in Table II on the settlements associated with tunneling in uncemented granular materials. For the most part, the settlements are rather small. It is noteworthy, however, that the data refer exclusively to fairly dense materials. The lack of similar data regarding loose sands undoubtedly reflects the difficulty of impracticality of tunneling in such materials without encountering almost continual troubles with at least local runs and losses of ground. Hence, the terms "normal settlement" and "routine" construction procedure, as used in this report, have little significance with respect to these materials.

The lateral distribution of settlements over a pair of tunnels in dense sand above groundwater level is shown in Fig. 1. The magnitude of the settlements at the measured location was unusually large for a dense sand because the material was dry and almost completely cohesionless; hence, it had a tendency to run. The spacing

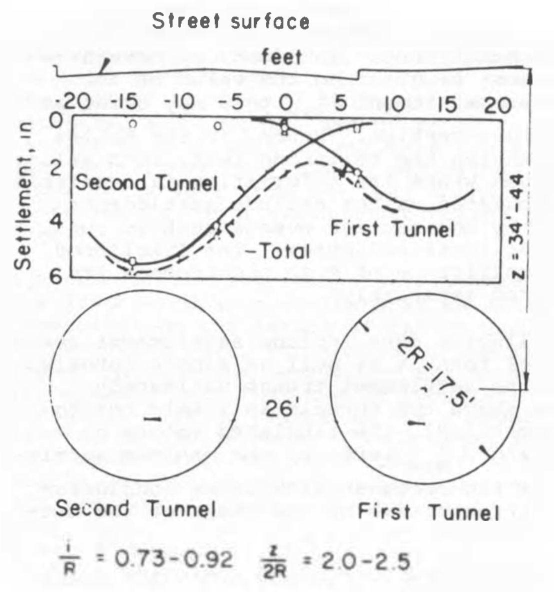


Fig. 1 Settlements over Tunnels in Dense Sand above Groundwater Level

of the tunnels is large enough that the second tunnel had little influence on the settlements above the first. For comparison, the probability or error curves best fitting the measured data are also shown in the figure. The use of these curves in estimating the distribution of settlements above other tunnels will be discussed at the end of this chapter. The settlement data for a tunnel in dense sand below groundwater level are plotted in Fig. 2a. The total settlements

Table II Settlements above Tunnels in Granular Soils (Cohesionless except for Capillarity)

No.	Case	Reference	Depth to Center, z, ft	Diameter, 2R, ft	Av. Settlement Volume, %	Largest Settlement δ"max, ft	Normal Settlement δ'max, ft	Method of Tunneling	Soil Conditions
1	San Francisco Mission Line, BART	Pers. files	36	17.5	0.5*	0.07* 0.10*[a]	0.03* 0.04*[a]	Digger shield, air 13 psi	Dense silty fine sand (N=30) with occ. thin lenses of peat. Dewatered by deep wells.
2	Toronto Subway under Parliament	Bartlett et al, 1965	49	17.0	--	**	--	Hand-mined shield air 15 psi	Crown in dense fine to medium sand, some silt. Groundwater level 15 ft above crown.
3	Toronto Subway	Pers. files	34-44	17.0	1.0	0.33	0.10	Hand-mined shield. No air.	Med. to fine uniform dense sand (N=40 to 60) above water table.
4	Toronto Subway	Matich and Carling (unpubl.)	34	17.0	1.0 2.0[b]	0.06 0.22[b]	-- --	Hand-mined shield. No air.	Dense fine to med. sand (N=36-58). Groundwater 25 ft above crown.

*Settlement due to groundwater lowering not included.

**Settlement less than about 0.01 ft; no cracks in overlying masonry building.

[a]Values for two parallel circular tunnels on 33-ft centers.

[b]Values for two parallel circular tunnels on 21-ft centers.

N = Standard Penetration Resistance (blows per ft of 140-lb hammer falling 30 inches to drive standard 2-inch O.D. sampling spoon 1 ft into undisturbed material).

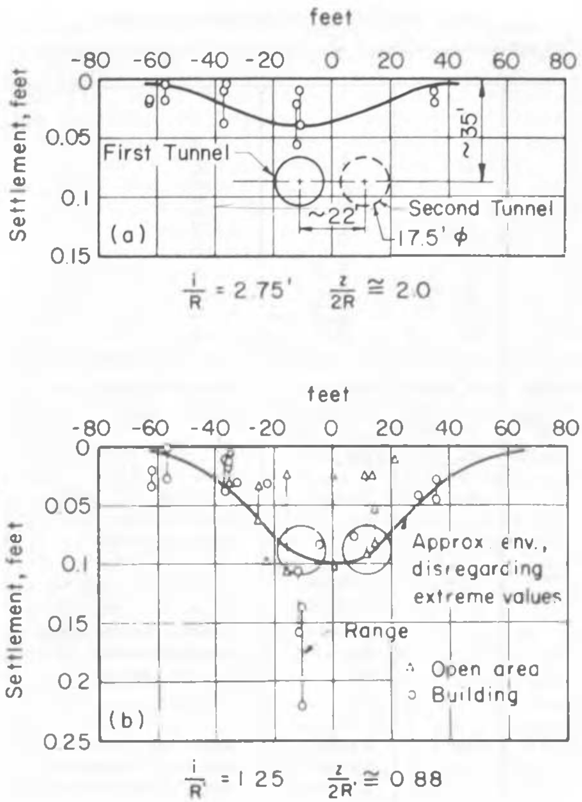


Fig. 2 Settlements Over Tunnels in Dense Sand Below Groundwater Level:
(a) After Mining of First Tunnel;
(b) Total Settlement Due to Two Tunnels

after construction of a second, adjacent tunnel are plotted in Fig. 2b.

If the second tunnel is relatively close to the first, the loss of ground associated with the second may differ appreciably in its characteristics from that associated with the first. If the first tunnel serves as a drain, the likelihood of runs in the second tunnel due to lack of groundwater control is somewhat reduced. On the other hand, construction of the first tunnel loosens the sand above the position to be occupied by the second. Hence, settlements over the second tunnel may be substantially greater than those over the first. The available data are shown in Table II.

In summary, we may conclude that the prediction of settlement associated with tunneling through cohesionless granular materials is fraught with uncertainties. If dewatering is expertly and completely done, and if there are no nearly impervious horizontal barriers above which some groundwater remains perched, the loss of ground in a dense material can be exceptionally small. On the other hand, the settlement may increase considerably as a result of erosion or migration due to seepage into the heading at localized zones, and the settlements may reach catastrophic proportions if runs develop on account of insufficient groundwater control or inadequate precautions against raveling. The likelihood of loss of ground is greatly increased if the sand is loose or contains loose zones in which positive pore pressures may be developed.

It is apparent that details of stratigraphy, details of drainage, and details or workmanship have a major bearing on the settlement. The importance of experience on the part of the workmen is discussed in the following section.

1.4.3 Tunnels in Cohesive Granular Soils

The soils in this category include a number of types ranging from clayey sands and sandy clays to **cohesive silts.** Residual soils possessing a cohesive bond, including many saprolites, often fall into this category, as do loess and certain calcareous clays with a stable cluster structure, known in some localities as marls. All these materials have several characteristics in common: they exhibit nearly linear stress-strain curves until the bond strength is approached, whereupon they fail suddenly; failure often occurs on a pre-existing surface of weakness such as an old crack or joint; if excavated without proper support during tunneling, they are likely to spall or ravel into the tunnel; the initial tangent modulus of unconfined specimens is relatively high. The binder creates a fairly **rigid** bond between the coarser particles; hence, clayey sands or sandy clays in which the clay constitutes a matrix that facilitates slip between the particles are not included in this category.

The experiences summarized in Table III, although few in number, show clearly that well-executed tunneling in such materials is accompanied by very modest or negligible loss of ground or settlement. Typical are the maximum settlements over the BART tunnels on the Mission Line, where the construction of a single tunnel caused a settlement of no more than 0.3 inch. In most instances, the stand-up time of the material permits substantial filling of the space left by the tailpiece of a shield, or permits expansion of the permanent lining against the soil. Use of the shield is generally a precautionary measure against raveling or a means to assure cutting the tunnel to proper size and regular shape. Air pressure is considered to have little direct effect on the loss of ground.

On the other hand, if raveling or piping is allowed to develop, the consequences may be catastrophic. Most materials in the categories under discussion are sensitive to adverse seepage pressures. Hence, it is axiomatic that the groundwater be kept under complete control, either by drainage or predrainage, and possibly with the aid of compressed air. Moreover, proper support must be provided to prevent the development of

Table III Settlements above Tunnels in Cohesive Granular Soils (Water Table below Tunnel Level)

No.	Case	Reference	Depth to Center, z, ft	Diameter, 2R, ft	Av. Settlement Volume, %	Largest Settlement δ''_{max}, ft	Normal Settlement δ'_{max}, ft	Method of Tunneling	Soil Conditions
1	San Francisco Mission Line, BART	Pers. files	36	17.5	0.15[a]	0.03[a]	0.01[a]	Digger shield, air 9 psi	Slightly cemented dense silty fine sand (N=40-60). Dewatered by deep wells.
2	Wilson Tunnel Hawaii	Pers. files	50	33.0	--	--	0.07	Hand, small drifts. Ribs, lagging. Horseshoe.	Residual saprolitic tropically weathered volcanics. Readily cut with air spade.
3	Wilson Tunnel Hawaii	Pers. files	100	33.0	--	--	0.20	do.	do.

[a]Values for two parallel circular tunnels on 33-ft centers. Settlements produced by first and second tunnels approximately equal.

raveling even if adverse seepage conditions are not present.

The consequences of raveling are strikingly illustrated in connection with the first Wilson Tunnel, driven by hand through residual silty clays derived from lava flows, in Hawaii. Raveling began in the roof of the tunnel at points of excessive overbreak and inadequate support. It led to the formation of a set of three sinkholes at the ground surface, each accompanied by a rush of completely broken and flowing soil into the heading. By attacking the face in drifts of small size instead of advancing the tunnel full-face for its width of 28 ft, the stand-up time was increased to the extent that the overbreak and raveling were eliminated. The settlements above the tunnel then did not exceed 1 to 2 inches. Yet, use of the same general methods to tunnel through the sinkhole area, where the cohesive bonds of the saprolite had been completely destroyed, resulted in settlements of more than a foot.

The distribution of settlements at sections along the BART Mission Line tunnels is illustrated in Fig. 3, on which the probability or error curve of best fit is superimposed. The settlements shown are those associated with the driving of only one of the tunnels. The settlements caused by either tunnel were virtually unaffected by the presence of the other, located 32 ft away, center to center.

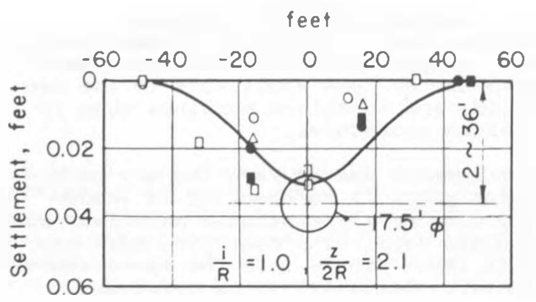


Fig. 3 Settlements over Tunnel in Drained Dense Cohesive Fine Sand on BART Mission Line, San Francisco; Air Pressure 13 psi

In connection with cohesive granular soils, we may conclude that loss of ground and settlement for single tunnels of customary sizes can usually be considered negligible, irrespective of the general construction procedure, provided the method and the workmanship are adequate to prevent raveling. If they are not, the loss of ground may be dramatic and catastrophic and is unpredictable with respect to location and magnitude. No effort should be spared to achieve groundwater control, especially in lenses of cohesionless materials that may be embedded in the more cohesive soils. The principal value of air pressure is as an aid to groundwater control, inasmuch as air pressure reduces the hydraulic gradient toward the tunnel.

In the final analysis, groundwater control and prevention of raveling are the sine qua non. If they are achieved, settlements and their influence on adjacent properties can usually be safely ignored. The prevention of raveling, moreover, is related directly to the experience of the tunneling crews. At the beginning of every job there is inevitably a period when the men, even if individually experienced, have not yet developed into a team and have not yet learned the idiosyncracies of the new job. During this period, progress is often slow and the work somewhat disorganized. Raveling, loss of ground, runs and even sinkholes may develop. Yet, the same crew, after a period of adjustment, may become highly proficient and may thereafter advance the tunnel without significant settlement. It is unrealistic to expect that a new crew in a similar tunnel in similar ground could immediately attain the degree of skill and perfection prevailing at the end of the earlier work. Allowance must be made by the engineer for a learning period. Failure to do so has resulted in serious and unanticipated movements. The best guarantee of the shortest possible and least detrimental disturbances is the availability and use of experienced foremen. Experience indicates that these comments are no less applicable to the use of shields and tunneling machines than to hand-mining procedures.

The settlements due to raveling may be delayed for years if backpacking is inadequately done or if perishable materials such as wood are used. The overlying soil may slope or ravel slowly into the space above the tunnel. Ultimately, the overlying ground may subside and cracking may develop in the permanent tunnel lining. If raveling is prevented, the strains associated with building a second tunnel alongside the first are only slightly greater than those caused by the first tunnel itself. Conversely, if the second tunnel produces stresses within a zone influenced by serious raveling in the first tunnel, large and irregular loss of ground may be expected over the first, and possibly the second tunnel.

1.4.4 Non-Swelling Stiff to Hard Clays

By and large, these materials have the desirable properties of those in the preceding category and, unless they possess a well-developed secondary structure, are rather unlikely to ravel or to be adversely influenced by seepage toward the opening. Because of their benificent character, tunnels through them in North America have from about 1940 until recently almost always been excavated by hand with nominal use of ribs and lagging or liner plates for temporary support. The loss of ground before completion of the permanent cast-in-place concrete lining was associated with general inward squeezing of the clay. The magnitude depended largely on the size of the tunnel or of the layout and sequence of small drifts with which the face was attacked, but was usually small. Before about 1940 timber sets were often used for temporary support, and loss of ground due to over-excavation and poor blocking was common.

In contrast, similar materials have been tunneled by means of shields for several generations in London. The settlements have usually been negligibly small. Data regarding one of the largest and shallowest of the London tunnels are included in Table IV, together with information from several other tunnels in stiff to hard clays. The distribution of settlement shortly after construction of the G. N. Railway tunnel in Seattle, a large hand-mined tunnel advanced in many small timbered drifts, is shown in Fig. 4.

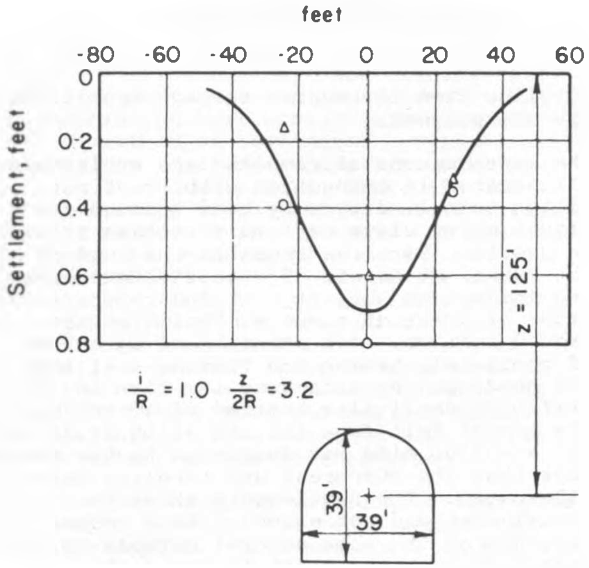


Fig. 4 Settlements over Large Railway Tunnel in Hard Clayey Glacial Till, Mined and Timbered in Small Drifts

Table IV Settlements above Tunnels in Stiff Plastic Clays (Slight to No Swelling Tendency)

No	Case	Reference	Depth to Center, z, ft	Diameter, 2R, ft	Av. Settlement Volume, %	Largest Settlement δ''_{max}, ft	Normal Settlement δ'_{max}, ft	Method of Tunneling	Soil Conditions
1	Garrison Test Tunnel	Burke 1957	121	36	--	0.14	.02 to .08	Full face, blasted. Ribs and lagging.	Hard clay (clay-shale). Unconf. compr. strength about 20 ksf
2	Heathrow Cargo Tunnel	Wood and Hill 1968	43	36	--	0.03	--	Shield, hand-mined.	Upper portion of London Clay. Min. 5-ft clay cover under wet granular material.
3	Chicago Subway Contract D3	City Chicago 1942	77	24	0.2	0.27	0.12	Hand-mined horseshoe. Face benched. Ribs and liner plates. Air 15 psi	Stiff clay for 10 ft above crown; q_u=2-4 ksf. Soft to med. clay above; q_u=0.8-1.6 ksf. Bottom half of tunnel in hard clay.
4	G.N.R.R. Seattle	Hussey et al 1915	123	39	2.6	0.8	0.6	Hand-mined; small drifts with center core. Timbered. Raveling at crown, used poling bars.	Hard clayey till

Note: ksf = kilopounds (kips) per sq ft

Materials in these categories are now commonly excavated by mole, often without a protective cylindrical tailpiece in which lining can be erected. If a cast-in-place concrete lining is to be constructed, temporary support is provided by steel ribs expanded against lagging in contact with the soil. It is becoming increasingly common, however, not to cast the lining in place. Instead, the permanent lining consists of segments of steel, cast iron, or precast concrete jacked or wedged into contact with the soil. The completion of a ring of lining effectively stops loss of ground that might otherwise take place due to slow squeezing of the clay toward the opening.

The loss of ground is undoubtedly a function not only of the strength of the clay but also of the diameter of the tunnel and of its depth. If other factors are equal, the settlement directly above the tunnel is roughly proportional to the diameter. The small settlements associated with good construction techniques in these materials can be anticipated if the ratio p_z/s_u is less than about 4. An air pressure p_a can be expected to reduce the loss of ground to an extent corresponding to a decrease of overburden pressure from p_z to $p_z - p_a$.

1.4.5 Soft to Stiff Saturated Clays

The soils in this category are characterized by values of undrained shear strength ranging from about 0.2 to 2.0 ksf (200 to 2,000 lb per sq ft) at depths of cover up to as much as about 100 ft. On the plasticity chart they are represented by points close to or above the A-line. For practical purposes they may be regarded as impervious; any visible seepage is confined to pervious inclusions. The sensitivity may range from low to very high.

Although completely undisturbed masses of such clays may possess considerable rigidity due to the presence of bonds between particles, the displacements and strains associated with tunneling seem always to be great enough to break the bonds and create a slightly disturbed zone of clay surrounding the tunnel. The soil in this zone, which may extend several diameters from the tunnel, has a modulus of rigidity considerably smaller than that of the cohesive granular soils previously discussed. Consequently, loss of ground and settlements due to tunneling are likely to be several times those of cohesive granular materials. Visible signs of distress such as spalling, raveling or piping do not ordinarily occur; rather, a barely perceptible or imperceptible squeeze takes place while excavation proceeds. Therefore, the settlements due to loss of ground often seem to be of mysterious or spontaneous origin. Nevertheless, they are by no means spontaneous and are likely to be of greater magnitude and extent than those above tunnels of similar size in cohesive granular soils, provided the settlements in the latter instance are not caused by raveling or piping.

Measurements have established within reasonable accuracy the equivalence of the volume of surface settlement and the volume of ground lost into the tunnel as a consequence of excavation. They have further demonstrated that, unless the heading is approaching a state of collapse, the movements toward the opening take place largely while the changes of stress due to excavation or jacking are occurring, and may almost cease when these operations are interrupted. Hence, unless the observations are carried out in such a manner that movements of the clay can be observed at locations not yet exposed by the excavating tools, the observer may believe that no movements are taking place.

Observations to detect the movements of the clay toward the approaching working face and toward the sidewalls of the hand-mined tunnels of the Chicago Subway were summarized by Terzaghi (1943). For example, the soft clay (unconfined compressive strength about 0.8 ksf) ahead of the working face of a 25-ft horseshoe-shaped tunnel moved inward toward the tunnel almost 2 inches before being exposed by the cutting tools, in spite of the support provided by benching the face and an internal air pressure of 12 psi. Comparable inward lateral movements of the clay alongside the tunnel probably also occurred. The corresponding loss of ground could, of course, not be prevented by any type of support installed after the clay had been exposed.

In a few instances, shields have been shoved partly blind into soft clays, with gross remolding of the soil in front of and alongside the tunnel. Heaves and settlements of the ground surface accompanied the tunneling, followed by large delayed settlements due to consolidation of the remolded clay (Terzaghi 1942). These procedures have largely been abandoned in urban areas, with a notable recent exception in Osaka, in favor of an open-faced shield equipped with breasting jacks that permit the face to be fully breasted and held by the jacks as the outer shell of the shield advances. Even with this procedure, the clay ahead of the tunnel cannot be considered unstrained. During a shove, large compressive forces are delivered to the clay in front of the shield, partly from the frictional forces developed between the soil and the outside of the advancing cylinder. After the shove and removal of at least part of the breasting, excavation of the face, even in small pockets, causes the clay to move toward the face. Hence, control is not perfect. The use of a rotary excavator that provides some support for the face while the material is cut away also involves alteration of the stresses in the clay and is accompanied by loss of ground for similar reasons.

All tunneling shields include an outer shell, usually cylindrical, that provides the protection for the men and machinery within. Usually the lining is erected within the tailpiece of the cylinder and is not exposed to the soil until the cylinder is jacked ahead. The annular space occupied by the tailpiece and necessary clearances between tailpiece and lining is generally from 2.5 to 4 inches thick. If the space is not filled, the surrounding clay moves in against the lining, whereupon the loss of ground is equal to the volume of the annular ring. As the loss of ground corresponding to the full annulus may easily be 5 per cent of the volume of the tunnel or, for example, 13 ft^3 per lineal foot of a tunnel of 18-ft diameter, efforts are always made to fill the space as much as possible by other means such as the injection of grout or pea gravel. The success of the efforts depends upon the rate at which the surrounding clay invades the space and the details of the injection operations.

The frictional forces between the skin of the shield and the surrounding soil tend to pull the clay along with the advancing shield. Consequently, a tendency toward longitudinal tensile stresses develops in the clays surrounding the rear of the shield. These stresses tend to cause failure and plastic flow of the clay into the annular space as soon as the tailpiece clears the lining. The inward flow greatly reduces the space that can be injected by grout or other materials, and places a considerable limitation on the loss of ground that can be prevented.

The movements associated with shield

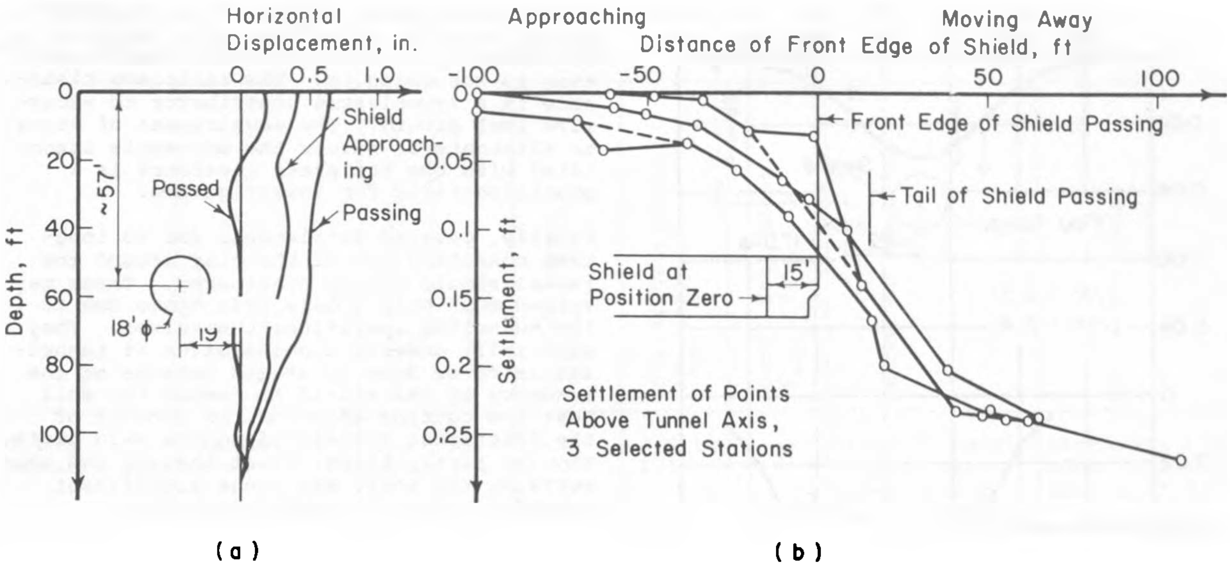


Fig. 5 Shield Tunnel in Plastic Clay: (a) Successive Positions of Originally Vertical Line in Soil Beside Tunnel as Shield Approached and Passed; (b) Settlement of Reference Points above Tunnel as Shield Approached, Passed Beneath, and Continued Beyond the Points

tunneling are well-illustrated by Fig. 5, which shows the successive positions of an originally vertical line about 8 ft from the edge of the path of a shield in plastic clays in San Francisco. On approach of the shield, the clay moved slightly away from the tunnel, but as the rear of the shield passed, the clay was drawn sharply toward the annular space. A plot of settlement of a point on the street surface as a function of the position of the cutting edge of the shield shows, Fig. 5b, that the rate of increase of settlement is greatest as the rear of the shield passes beneath the point.

Reduction of loss of ground due to invasion of clay into the tailpiece clearance requires either that the inward squeeze of the clay be prevented until the space can be filled, or else that the filling be so prompt that the invasion is insignificant. Neither requirement can be met completely at present. The amount and rate of invasion can be reduced, sometimes to insignificant values, by the use of air pressure. Attempts to grout through the lining immediately after the tailpiece clears have been hampered by the difficulty of sealing the space between the lining and the inside of the tailpiece so that the injected material is not discharged into the working chamber of the shield. Development of an adequate seal would be a substantial step in reducing loss of ground under these conditions.

The patterns of surface settlement caused by loss of ground into two different tunnels in plastic clay are shown in Figs. 6 and 7. They are roughly similar, irrespective of whether the tunnel is advanced by hand-mining, use of a shield, or use of a mole. Again, for convenience, the probability curves best fitting the data are also shown.

The influence of a second, parallel tunnel may sometimes be approximated by adding the ordinates of the two separate settlement curves. In most instances, however, the loss of ground associated with the second tube is larger than that due to the first, on account of the disturbance to the surrounding clay caused by construction of the first tube. The settlement curve for the second tube is likely to be unsymmetrical, with the greater settlement toward the first tube. However, it has been observed that, if the two tunnels have a common center wall already in place when the second tunnel is excavated, the settlement caused by the second tube is smaller than that caused by the first (Terzaghi 1943). These differences are also illustrated by the observations reported in Table V for Chicago Subway tunnels D-5 and S-6.

In short, settlements above and adjacent to tunnels in plastic clays of soft to medium consistency may be dramatically larger than those above tunnels in stiffer, more brittle cohesive granular soils, although the settlements and the movements in the tunnel are not likely to develop with such catastrophic speed that the heading might be lost and could not be entered safely for carrying out remedial work. Air pressure is an effective means of reducing loss of ground, not because it strengthens the soil but because it reduces the changes in stress

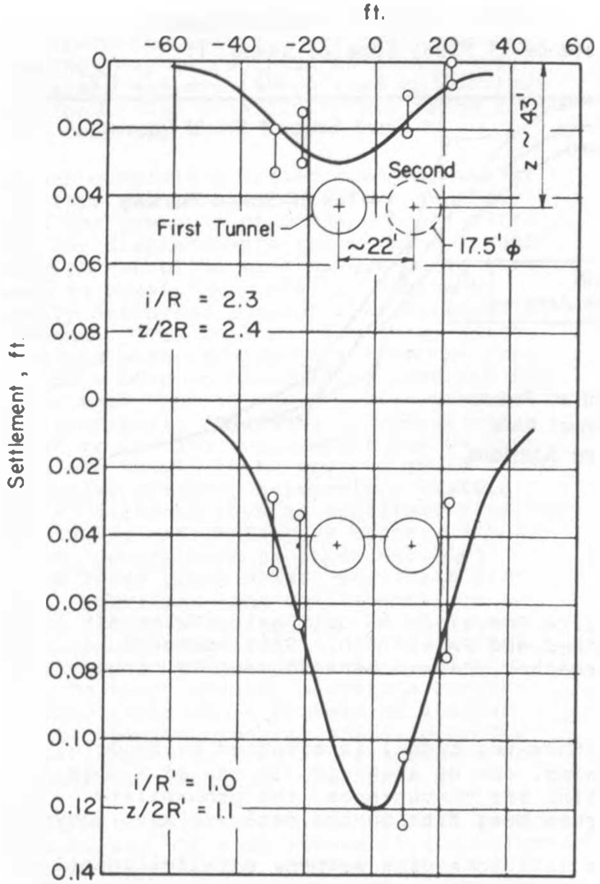


Fig. 6 Settlement above (a) Single Tunnel and (b) Pair of Adjacent Tunnels in Plastic Clay

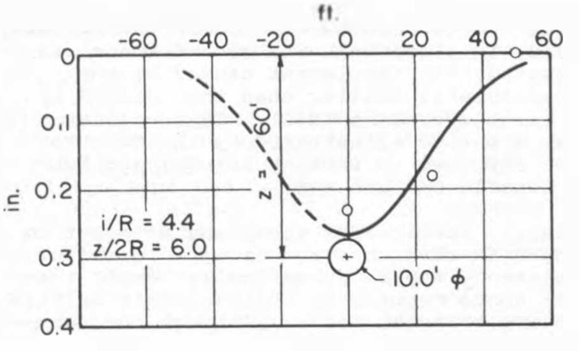


Fig. 7 Settlement above Single Sewer Tunnel in Plastic Clay in Ottawa

due to excavation. Little loss of ground occurs when the shield or mining machine is not actually advancing or when hand excavation is not going on. The tailpiece clearance is a troublesome contributor to excessive lost ground. The development of means to eliminate or reduce the movements associated with the tailpiece clearance is a promising field for investigation.

Finally, delayed settlements due to long-time consolidation of the clay around the tunnel should not be overlooked. These may spread much more widely than those due to the tunneling operations themselves. They especially deserve consideration if tunneling is to be done by shield because of the tendency of the shield to remold the soil near the cutting edge and, on account of the frictional forces, along the skin plate. Shoving partly blind, first heaving and then settling the soil, may cause significant remolding even in insensitive soils.

1.4.6 Distribution of Settlement

The available empirical data, such as the information included in Figs. 1-4, 6 and 7, demonstrate that a cross-section through the settlement trough over a single tunnel can usually be represented within reasonable limits by the error function or normal probability curve. Although the use of this curve has no theoretical justification, it provides at least a temporary expedient for estimating the settlements to be expected at varying distances laterally from the center line of a tunnel. Such an expedient is needed for judging the necessity of underpinning or shoring adjacent buildings, or of relocating vital utilities.

The pertinent properties of the error function and its relationships to the dimensions of the tunnel are shown in Fig. 8. The radius of the tunnel is represented by R, and the depth to the center of the tunnel by z. The maximum ordinate of the curve is the empirically determined maximum settlement δ_{max}. The points of inflection of the error curve are located at distances i on either side of the center line. If the value of i can be established, any table of the ordinates of the normal probability curve can be used to establish the ordinates at any other distance. The settlement ordinate at distance i is, according to the properties of the probability curve, equal to $0.61 \delta_{max}$.

Values of i have been calculated for tunnels above which reasonably reliable settlement data are available. They are assembled in Table VI and are illustrated in a dimensionless plot of i/R against z/2R, Fig. 9. The various tunnels are identified in the figure.

The plot, Fig. 9, shows reasonable trends and permits a tentative separation of the results according to the types of soil.

Table V Settlements Above Tunnels in Saturated Plastic Clays

No.	Case	Reference	Depth to Tunnel Axis, z, ft	Diameter, 2R, ft	Av. Settlement Volume, %	Largest Settlement δ''_{max}, ft	Normal Settlement δ'_{max}, ft	Method of Tunneling	Soil Conditions
1	Tyholt, Norway, R.R.	Hartmark 1964	56-72	26.0	(15)	2.0 to 2.4	--	Shield, hand-mined, air 17-23 psi	Sensitive clay with silt layers, q_u=1.2-1.5 ksf
2	Koto, Tokyo Subway	Shiraishi pers. comm.	74	23.0	(4.3)[a]	0.6[a]	0.4[a]	Shield, sectional hand-mining, breasted face, liner segments erected in shield, air 8 psi	Normally loaded sensitive clay, q_u=1.5 ksf
3	San Francisco BART	Pers. Files	59	18.0	4.3	0.2	0.15	Shield, breasted face, liner segments erected in shield. No air	Moderately sensitive clay, q_u=1.6 ksf
4	Ottawa Sewer	Eden and Bozozuk, 1968	60	10.0	1.6	0.02	--	Digger-shield, liner segments erected behind shield, air 4-5 psi	Leda clay, sensitive q_u=7.4 ksf
5	Toronto Subway	Matich & Carling (unpubl.)	43	17.5	0.6 1.3[b]	0.035	0.03 0.12[b]	Shield, hand-mined, air 10-12 psi	Crown: silty clay, q_u=1.4 ksf; Invert: till, q_u=1.6 ksf
6	Chicago D-5	City of Chicago 1942	39	20.0	0.8 1.1[c]	0.10 0.35[c]	0.06 0.20[c]	Hand-mined, ribs and liner plates, heading benched, air 14 psi	Strength of glacial clay varies from q_u=1.4 ksf at axis elevation to 0.8 ksf at 18 ft depth. Stronger material above.
7	Chicago S-6	City of Chicago 1942	36	20.0	0.6 0.4[d]	0.09 0.17[d]	0.05 0.12[d]	Hand-mined, ribs and liner plates, heading benched, air 12 psi	Strength of glacial clay varies from q_u=1.2 ksf at axis elevation to 0.7 ksf at 10 ft depth. Stronger material above.

Table V (Continued)

Notes:

Values in parenthesis estimated on basis of few observations.

[a]Values for two parallel circular tunnels on 49-ft centers.

[b]Values for two parallel circular tunnels on 21-ft centers.

[c]Values for two parallel horseshoe tunnels on 28-ft centers.

[d]Values for two horseshoe tunnels on the common center wall.

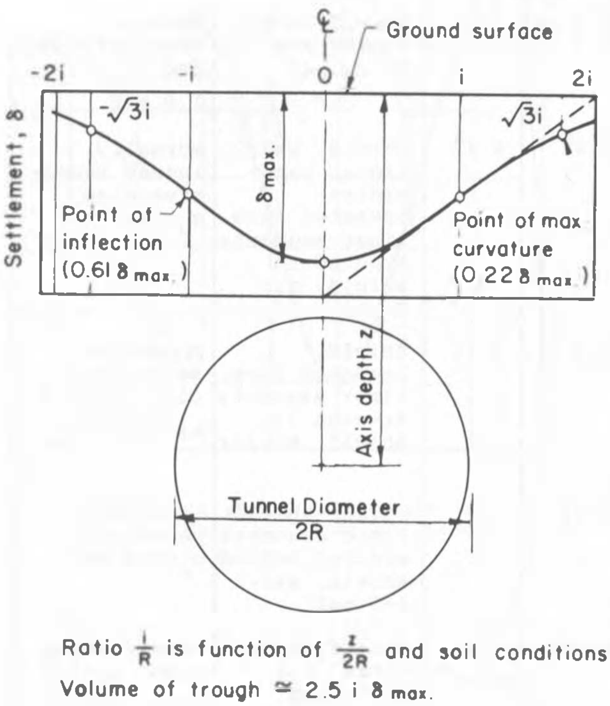


Fig. 8 Properties of Error Function or Normal Probability Curve as Used to Represent Cross-Section Through Settlement Trough Above Tunnel

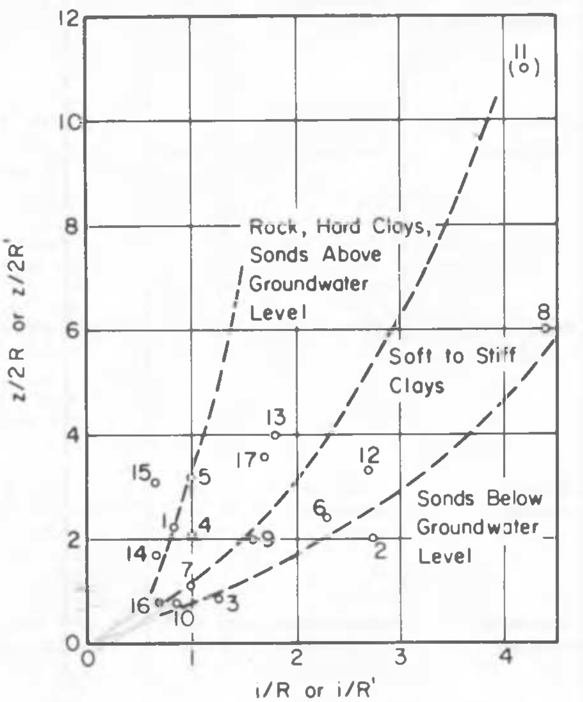


Fig. 9 Relation Between Width of Settlement Trough, as Represented by i/R, and Dimensionless Depth of Tunnel, z/2R, for Various Tunnels in Different Materials

Table VI Width of Settlement Trough Above Tunnels

No.	Case	Reference	Radius of Tunnel, R (or R'), ft	Depth of Tunnel Axis, z, ft	z/2R (or z/2R')	i, ft	i/R (or i/R')	δ_{max}, ft	Settlement Volume, ft^3/ft	Settlement Volume, %	Remark
1	Toronto Subway	Pers. files	8.75	34-44	2.0-2.5	6.4	0.73	0.28	4.5	1.9	First tunnel
			8.75	34-44	2.0-2.5	8.0	0.92	0.46	9.2	3.8	Second tunnel
	Dense sand above groundwater level										
2	do.	Matich & Carling (unpubl.)	8.75	35	2.0	24	2.75	0.04	2.4	1.0	First tunnel
3			20	35	0.88	25	1.25	0.1	6.3	1.3	Total Settlements
	Below groundwater level, crown in sand, invert in till										
4	San Francisco (BART)	Pers. files	8.75	36	2.1	18	1.0	0.03	1.35	0.56	Settlements from first and second tunnels independent and equal
	Cemented dense sand, above groundwater level										
5	G.N.R.R. Seattle	Hussey et al, 1915	19.5	125	3.2	20	1.0	0.7	35	2.6	
	Hard clayey glacial till (horseshoe)										
6	Toronto Subway	Matich & Carling (unpubl.)	8.75	43	2.4	20	2.3	0.03	1.5	0.62	First tunnel
7			20	43	1.1	20	1.0	0.12	6	1.25	Total Settlements
	Medium glacial clay										
8	Ottawa Sewer	Eden & Bozozuk, 1968	5	60	6	22	4.4	0.023	1.25	1.6	
	Medium Leda Clay										
9	Chicago Subway S-6	Terzaghi, 1943	10	39	2.0	16	1.6	0.075	3	0.75	First tunnel
	Soft glacial clay										
10	Chicago Subway S-3	Terzaghi, 1942	52	40	0.77	22	.85	--	--	--	Total settlement over two tunnels
	Soft glacial clay										
11	San Paolo	Terzaghi, 1950	4.5	100	11	19	4.2	0.67	32	50	Many construction difficulties
	Stiff clay										

Table VI (Continued)

No.	Case	Reference	Radius of Tunnel, R (or R'), ft	Depth of Tunnel Axis, z, ft	z/2R (or z/2R')	i, ft	i/R (or i/R')	δ max, ft	Settlement Volume, ft³/ft	Settlement Volume, %	Remark
12	San Francisco (BART)	Pers. files	9	59	3.3	24	2.7	0.18	11	4.3	
Medium clay											
13.	Sulphur Extraction	Deere, 1961	175	1400	4.0	320	1.8	--	--	--	
Rock											
14	Mine	Wardell, 1959	770	2620	1.7	510	0.66	--	--	--	
Rock											
15	Mine	Wardell, 1959	61	370	3.1	39	0.65	--	--	--	
Rock											
16	Mine	Pierson, 1965	620	1000	0.8	420	0.68	--	--	--	
Rock											
17	Mine	Berry & Sales, 1961	275	1970	3.6	475	1.7	--	--	--	
Rock											

Notes:

R' is one-half the width of a horseshoe tunnel or R + d/2, where d is the spacing of twin tunnels, center to center.

Volume of settlement trough is calculated by $V_s=2.5 \ _{max}i$

Appreciably greater values of i/R appear to be associated with tunnels in plastic clay than in the several varieties of granular materials. A significant exception is tunneling in sand below water table, where control of lost ground is especially difficult. As expected, the greater the depth of tunnel, the greater the spread of the settlement trough.

In several instances, the two tunnels of a pair were close enough together to produce a single settlement trough, reasonably symmetrical, that could be interpreted as the consequence of a single tunnel with depth z and radius R'=R+d/2, where d is the distance between center lines of the tunnels. Points representing the probability curves corresponding to these conditions are also plotted in Fig. 9 by replacing R by R' in the expressions i/R and z/2R. These points are also identified in the figure.

Also included in Fig. 9 are points representing the values of i/R for subsidence troughs over several mine or solution cavities in rock, where the depth of the opening is great enough to suppress the influence of the irregular cross-section of the opening. The results suggest that the subsidence troughs above granular soils (except those influenced by seepage) and rocks are roughly comparable, whereas the disturbance due to tunneling in plastic soils extends laterally significantly farther.

1.4.7 Conclusion

In this chapter, empirical information has been assembled to permit the engineer to estimate the loss of ground associated with various tunneling procedures in different types of ground. The effectiveness of various methods for reducing the loss of ground and consequently the settlement has also been discussed.

Consideration has been restricted to four principal types of subsurface conditions. Many more types are of practical interest. Furthermore, many headings involve mixed faces of cohesive and cohesionless soils.

As we have seen in Chapter 3, the feasibility of construction is much more difficult to judge in mixed faces than in faces consisting of a single material. Equipment and procedures satisfactory for one type of soil encountered in the heading may not be satisfactory for another. Once a type of construction has been tentatively selected, an estimate of the settlement is required to determine whether the procedure will be acceptable from the point of view of disturbance to surface facilities. In a mixed heading, conditions may lead to greater surface settlements than would occur if the heading were being constructed in any one of the individual materials. Little empirical information is available on which to form a basis for judgment.

One of the most urgent needs for the advancement of the art of tunneling is further detailed information about settlements associated with a variety of soil conditions and methods of tunneling. Because of the dependence of loss of ground on construction details, there seems little likelihood that theoretical investigations will prove fruitful except for some of the simplest of materials such as plastic clays. The same conclusion can be drawn concerning small-scale model tests. Full-scale field observations remain of outstanding urgency.

The information contained in this chapter is surprisingly meager. It can presently serve a useful purpose in demonstrating that lost ground and settlement occur in connection with all soft-ground tunnels, and that the magnitudes of the settlements are not always as small as designers and planners sometimes assume. In some instances the present information may permit an estimate of the settlement to be expected on a project under consideration if the work is expertly carried out according to a given procedure. Yet, the designer cannot overlook the likelihood of much greater settlements if unexpected difficulties arise. He must be aware that the unexpected is almost to be regarded as the expected in tunnel work, and must rule out designs involving construction procedures prone to difficulties under the physical conditions prevailing at the site of the project. The present margin of uncertainty in estimating the consequences of tunneling, even with the best of techniques, cannot be reduced until many more records of settlements and construction procedures become available for all types of subsurface conditions.

1.5 Design of Lining

1.5.1 Basic Concepts

As a first step toward understanding the fundamental factors governing the behavior of tunnel linings, let us consider a series of imaginary experiments. Consider first a mass of soil with a horizontal ground surface. At depth z the vertical intensity of pressure is γz. The intensity of horizontal pressure is assumed to be a constant K_0 times the vertical pressure. For the purpose of the discussion, we shall assume that K_0 is less than unity.

By some sort of technical magic, we shall now wish a circular tunnel lining into existence without disturbing the soil either outside or inside the tunnel. Since no disturbance is involved, the state of stress in the soil is no different than before the lining came into existence. Let us make the further assumptions that the circular lining is perfectly flexible but capable of supporting appreciable ring stress in compression, and that tangential or shear stresses around the lining are negligible. The initial distribution of radial or normal pressure on the lining corresponds to that in Fig. 10. We now suddenly remove the earth from inside the tunnel. Inasmuch as the circular flexible ring can be in equilibrium only if the radial pressures are everywhere equal, there must be some means whereby the intensity of horizontal pressure can increase (and that of the vertical pressures decrease) until the required equal all-around pressure is attained. The means for accomplishing the redistribution is a distortion of the lining from a circular shape to a very slightly elliptical shape. The horizontal axis becomes longer because outward deflection of the sides of the lining is required to build up the side pressure; at the same time the vertical diameter must tend to shorten and the vertical pressures correspondingly tend to decrease. The deformation takes place to whatever extent is required to bring about the virtual equality of radial stress in all directions.

If the capacity of the lining is sufficient to carry the ring stress associated with the radial pressure and if local buckling is prevented, the flexible lining is entirely satisfactory for support of the surrounding

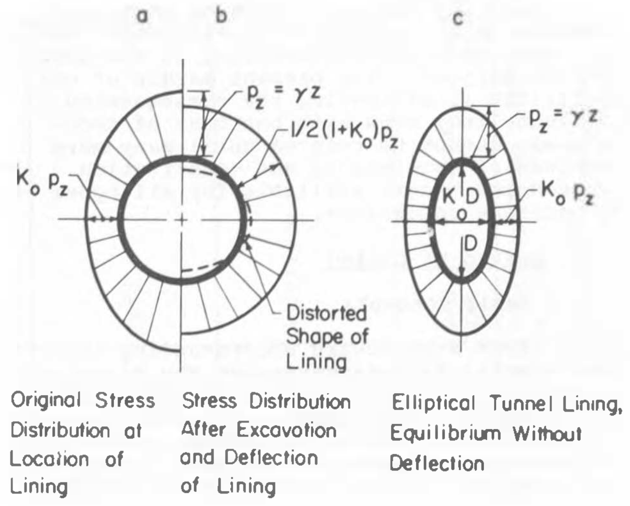


Fig. 10 Pressures Against and Deflections of Hypothetical Linings

earth. No bending moments can exist in the lining and no bending strength is required. The only practical question is whether the deformation required to establish the state of all-around pressure is within limits considered tolerable for non-structural reasons such as the preservation of clearances or the safeguarding of architectural finishes. The amount of the deformation is a function of the stress-strain characteristics of the surrounding soil, the time dependence of these characteristics, and the dimensions and depth of the tunnel.

Had the cross-section of the idealized flexible tunnel been elliptical with minor axis in the horizontal direction equal to K_0 times the vertical axis, the shape of the ellipse would have been almost perfectly matched to the equilibrium polygon for the normal forces acting on the lining. Hence, after removal of the soil from inside the tunnel, not only would no bending moments have existed in the lining, but no deformation would have been required to achieve the distribution of all-around pressure exactly compatible with the absence of bending stresses in the lining.

By similar reasoning we conclude that a flexible lining of any elliptical shape can support the surrounding soil because the soil pressures will assume the distribution theoretically necessary to produce no bending moments in the lining. The theoretically correct pressure distribution is achieved by virtue of deformation of the lining and the surrounding soil mass. The magnitude of the deformation varies somewhat depending on the shape of the cross-section of the tunnel. If the deformation is not considered excessive for some non-structural reason and if other modes of failure do not develop, the design of the lining can be considered adequate.

The favorable distribution of all-around pressure on the tunnel lining is, of course, a consequence of the development of shearing stresses in the surrounding soil mass. If the shearing stresses remain essentially constant after the necessary deformations take place, no further distortions will occur; otherwise continued movements of the lining must be expected. Experience has demonstrated that all real soils are capable of withstanding shearing stresses up to some limiting magnitudes with negligible creep or relaxation. The shearing stresses become a part of the permanent system of stress or tunnel support by which the soil surrounding the tunnel cooperates with the tunnel lining in maintaining the stability of the opening. Indeed, in most instances the soil surrounding a tunnel is not only the source of loading on the tunnel but the source of most of the ability of the tunnel to remain stable in the medium. Rational tunnel design must take into account the strength and stress-strain-time properties of the surrounding soil.

If instead of a completely flexible lining we had installed during our imaginary experiment a perfectly rigid and infinitely strong circular lining, the initial pressures acting radially against the lining would have been identical with those against the flexible lining. However, upon removal of the soil from the inside no deformation of the rigid lining would have been possible. Therefore, the distribution of external pressures would have been unchanged. The pressures would cause bending moments in the lining. Hence, in exchange for the deformations necessary to permit a flexible lining to assume the theoretically unstressed shape, the rigid lining must be designed to withstand substantial bending moments.

If the original rigid lining had been of elliptical shape with horizontal axis equal to K_0 times the vertical axis, the equilibrium polygon for the external normal forces would have almost coincided with the axis of the tunnel and the bending moments would have been practically zero. Hence, we observe that the bending moments in a rigid lining depend strongly on the shape of the tunnel.

Real tunnels cannot be wished into place without disturbance and then excavated. Furthermore, their flexural rigidity is inevitably intermediate between perfect flexibility and perfect rigidity. Hence, the idealized experiments are considerably removed from reality. Nevertheless, they will assist us greatly in visualizing the behavior and requirements of real tunnel linings. We shall consider the behavior of linings constructed in tunnels built according to two very common and rather different methods of construction. In the first of these, a light, so-called temporary lining, usually of steel, is erected to preserve the integrity of the excavation until a so-called secondary or permanent lining is installed. In the second, the temporary and permanent linings are combined so that the initial lining is the final one. Although there are many combinations of these procedures, a discussion of these two will illustrate the principles.

1.5.2 Tunnel with Primary and Secondary Lining

We shall first assume that a tunnel of circular shape is being constructed by hand methods in soft ground. We shall also assume for the moment that the method of mining and of installing initial support is suited to the ground and that appropriate means have been devised to permit advancing the tunnel and installing circular ribs and plates or lagging. All necessary measures have been carried out including possibly predrainage or the use of compressed air.

Once the circular lining is in place and blocked against the surrounding soil, or the space between soil and lining is filled by pea gravel, grout or other material, the circular rings are capable of carrying axial compression. Ordinarily they are relatively flexible with respect to bending stresses, especially because the efficiency of the joints is likely to be low. Hence, they approximate a flexible circular lining. The rings in such a lining are likely to continue to deform as the heading is advanced, but the rate of deformation usually decreases rapidly as the heading advances one or two diameters beyond the location of any particular ring. In some types of ground the rate of deformation may reduce to zero after a few days or weeks; in others equilibrium may be reached only after several months. As long as the total diameter change is not excessive, for instance, not more than 1 or 2 percent of the diameter, the behavior of the temporary lining is usually considered satisfactory. The design of the lining is properly left to the contractor because its effectiveness depends far more on the skill and care with which it is placed than on its actual dimensions.

The designer is concerned with the basis for proportioning the secondary or permanent lining to go inside the completed primary lining. Customarily the lining is designed to withstand the weight of the overburden and a side pressure equal to some fraction of the overburden pressure. Yet there is not the slightest possibility that these loads can act upon the completed permanent lining. Indeed, if the deformations of the temporary lining have ceased, it is even unlikely that significant bending stresses can ever develop in the secondary lining. Because of the deformations, shearing stresses have been mobilized in the surrounding soil to the extent that the soil and the temporary lining together have achieved equilibrium and are carrying all the loads. These conditions are not altered by the introduction of the secondary lining.

If the tunnel has been dewatered and the water level will ultimately be raised after the secondary lining has been installed, the water pressure will act upon the secondary lining. The water pressure will not be a uniform all-around pressure. However, if the drained soil has retained water by capillarity, as all soils but coarse sands and gravels do, the increase in water pressure upon restoration of the original ground water level is a uniform all-around pressure that can itself produce no bending moments in a circular lining (although there may be concurrent changes in the effective stresses leading to small moments). Similarly, if the tunnel is built under air pressure that is not removed until the secondary lining is completed, the lining will be acted on by an all-around pressure equal to the air pressure. This pressure also produces no bending moments in a circular tunnel. Hence, the only forces to which a circular secondary lining can be subjected are the nearly equal all-around pressures caused by the restoration of the water pressure or the equal all-around pressures associated with the removal of the air pressure. Even these forces must be shared by the primary and secondary lining in accordance with their rigidities. Thus, the loads actually transmitted to the secondary lining have no resemblance whatsoever to the assumed earth pressures for which such linings are usually designed. It is not surprising that defects in such linings have most often been caused by totally unanticipated influences such as cracking of the concrete due to shrinkage.

The passage of a second tunnel alongside the first significantly alters the pattern of shearing stresses in the ground and produces significant new loadings on the pre-existing tunnel. These loadings likewise have no relation to those customarily assumed for the design of rigid tunnel linings. Furthermore, if the second tunnel is excavated before the secondary lining has been placed in the first tunnel, the flexible temporary lining of the first tunnel adjusts to the new conditions by further deformation, often with complete safety. If then both tunnels are provided with their secondary linings, the secondary linings experience practically no additional forces throughout their lifetime.

1.5.3 Combined Primary and Secondary Lining

As a second example, let us consider a tunnel lined with precast segments assembled within the tailpiece of a shield. The forces acting on the shield are extremely complex. As a crude approximation it may be assumed that the radial forces acting on the cylindrical surface correspond to the at-rest conditions in the ground and it may further be assumed that the shield is a virtually rigid object. However, because of the thickness of the tailpiece and the necessary working clearance between the outside of the segmental lining and the tailpiece, an annular space several inches thick is formed around the lining as the tailpiece clears. The rate at which the surrounding soil invades the space depends upon the character of the soil, the dimensions of the shield and, undoubtedly, on numerous other factors. Nevertheless, until the soil comes into contact with the lining the radial pressure at the surface of the soil is reduced to zero or to the air pressure prevailing in the tunnel. It is customary to attempt at an early stage to fill the annular void. In soft or cohesionless soils the attempt is usually unsuccessful because the invasion occurs too rapidly. In strong cohesive material the filling may be accomplished.

The most significant aspect of these conditions is that the inward movement of the soil toward the lining, before it is prevented by the filling, is a large displacement in comparison to that likely to occur subsequently because of any radial deformations of the tunnel lining. Since the movement in soft soils may be a matter of several inches, substantial shearing stresses are set up in the surrounding soil mass and the pressure exerted against the segmental lining has no resemblance to the original state of stress in the soil. Because the inward deformations are initially only slightly restricted and because the shape of the hole is circular, the radial pressures against the segmental lining are reduced greatly with respect to the initial state of stress and are probably much more favorably distributed with respect to bending moments in the lining. The real pressures acting upon the lining depend on the stress-deformation-time characteristics of the surrounding material, the amount of deformation experienced, and the time at which the actual contact was made between the soil and the lining. In many instances the pressures are complicated by the process of filling the voids. Quite possibly the most non-uniform pressures acting on the lining are those introduced by the efforts to inject grout into the annular ring. These pressures should not be ignored in design, but they cannot logically be considered equivalent to the initial state of stress in the ground.

Since the deformation of the tunnel lining itself can be controlled by tie rods until grouting has been completed, conditions are favorable for the development of compatibility between the lining and the surrounding soil that will ultimately result in relatively small deflections of the lining or stresses to be withstood.

We are now in a position to evaluate the real factors that govern the stresses and behavior of permanent tunnel linings and to consider the presently available knowledge for solution of actual design problems. Furthermore, we see that we cannot escape the influence of details of construction procedures, even with respect to the design of secondary linings.

1.5.4 Semi-Empirical Basis for Design of Lining

The preceding discussion has demonstrated that a rational procedure for design of a tunnel lining must take into account many factors including the characteristics of the soil, the geometry of the tunnel, and the method, details, and speed of construction. The latter factors are of such importance that no purely theoretical basis for design can be envisioned. As in many other branches of applied soil mechanics, the art of tunnel design seems likely to achieve its greatest success on a semi-empirical basis in which the results of theoretical findings are suitably and consistently modified on the basis of full-scale behavior in the field.

As a first step in attempting to develop a semi-empirical basis for design, we shall consider the lining of a single circular tunnel. As we have seen, the investigation of the lining can be considered in three nearly independent steps. First of all, the lining must be designed to carry without distress whatever direct compressive forces may develop circumferentially; we shall term these forces ring stresses or ring loads. Secondly, the lining must be capable of withstanding whatever bending may occur in planes at right angles to the axis of the tunnel. We have already seen that the more flexible the lining, the less concern there may be with bending stress; on the other hand, with increasing flexibility the deformations of the lining may become significant. Finally, the lining must be able to survive local irregularities of deformation, loading or stress; that is, it should not fail by buckling or by some other detailed mode.

We shall consider these three requirements as if they were strictly independent. The approximation introduced by this assumption affords much simplification and is believed to lead to conservative design. Nevertheless, it also leads to economical design because it permits consideration of the vital factors pertaining to each aspect of the design and allows the elimination of unnecessary provisions that would be required if the behavior of the tunnel could not be so clearly defined.

The foregoing discussion has assumed that the lining of a tunnel consists of a closed ring either circular or somewhat elliptical in cross-section. The considerations that follow are based generally on the same premise. In reality, many tunnels have horseshoe-shaped sections, and the lining may not extend around the entire periphery. These conditions alter the details of behavior, but usually to a minor degree. The strains in the soil surrounding a horseshoe-shaped tunnel as excavation proceeds are as effective in mobilizing the strength of the soil as are those around circular tunnels. At the present state of knowledge, an attempt to differentiate the behavior of circular and horseshoe tunnels would be an unwarranted refinement. The data themselves suggest that no significant distinctions in behavior exist. Minor modifications in the conclusions and recommendations are, of course, required to take account of the particular geometry of the horseshoe shape.

1.5.5 Ring Stress

The basis for establishing the ring stress for which a lining should be designed may be visualized by means of Fig. 11a, in which the ring load is plotted as a function of the average radial deformation of the boundary of the tunnel toward the center. As a reasonable first approximation, the average ring load, if no radial deformation occurred, would be the radius of the lining multiplied by the mean of the vertical and

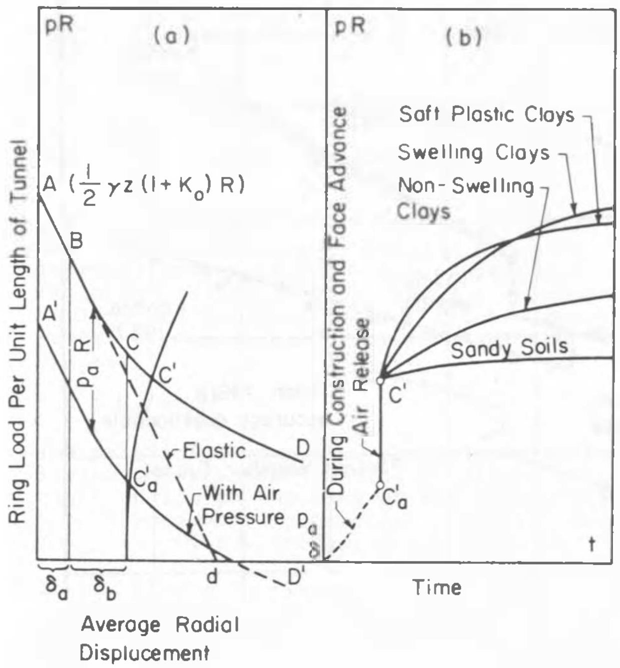


Fig. 11 Principle of Interaction Between Soil and Lining: (a) Ring Load and Average Radial Displacement During Construction; (b) Subsequent Increase of Ring Load with Time

horizontal pressures in the ground at the location of the center of the tunnel if the tunnel did not exist. The vertical pressure may be taken as γz_0 and the horizontal pressure as $K_0 \gamma z_0$. Hence, the ring load, if there were no radial deformation, would be equal to $1/2 \gamma z_0 R\ (1 + K_0) = p_0 R$, where R is the radius of the unlined tunnel. It is represented by Point A in Fig. 11a.

If the radius of the ring were decreased by a small amount δ, load would correspondingly decrease in accordance with a relation such as that indicated by the solid line in Fig. 11a. The shape and position of the line would depend on the stress-strain-time characteristics of the soil and on the time required for the construction operations. If the soil were elastic, the relationship would be linear as shown by the dash line Ad. For an inelastic material, the relationship would resemble AD. The position and shape of curve AD can be calculated only for highly idealized conditions, but they may be approximated in a few instances on the basis of field observations combined with theoretical considerations. For the moment, we shall assume that the position can be established for a particular soil with sufficient accuracy.

As the tunnel heading approaches a given point, the soil moves radially toward the tunnel and axially toward the working face. Thus, by the time the working face has reached the point, a radial displacement has already been experienced. The average value δ_a of this displacement is indicated by the thin vertical line in Fig. 11a. If the circumferential lining were placed in contact with the soil at this stage, and if it were capable of preventing any further radial displacement, the average ring load would be B, smaller than A. If, however, further radial displacement were possible before the lining could be placed, indicated by the displacement δ_b, the ring load would be C. Thus, a ring capable of preventing all further radial displacement should be designed for a ring load C. In reality, the ring will be subject to radial deflection of at least a small amount on account of rib shortening. The corresponding deflection and ring load are indicated by C'. Point C' is located on the heavy dash-dot line representing the division of load between the lining and the surrounding soil on the basis of the relative rigidities of the two media.

If the tunneling were done under an air pressure p_a, the relation between ring load and radial deflection would be displaced downward a distance $p_a R$. If the lining were placed when the radial displacement became equal to $\delta_a + \delta_b$, it would be required to support a ring load of only C_a', but this load would increase to C' upon removal of the air pressure.

The passage of time may be accompanied by a further change in ring load, depending primarily on the nature of the soil, as shown in Fig. 11b. As the distance increases from the heading to the ring under consideration, or as time passes even if there is no construction progress, the ring load is likely to increase at a decreasing rate. For many tunnels the ring load appears to increase roughly proportionally to the logarithm of time.

Empirical information regarding the ring loads in various tunnels is summarized in Figs. 12a and 12b. Because of the large quantity of data available for tunnels in the London clay, this information has been plotted separately in Fig. 12a. That for all other tunnels is shown in Fig. 12b. General data concerning the construction of the tunnels are included in Table VII. The loads are expressed in dimensionless form as p/p_z, where p is the average all-around pressure against the tunnel, as determined directly by pressure observations or indirectly by calculation from measured compressive stresses in the ring or lining, and p_z is the total vertical overburden pressure at the elevation of the center of the tunnel.

According to Fig. 12b, the ring load for a single tunnel in clay after a long period, such as 100 years, is not likely to exceed that corresponding to an all-around pressure p_z. Except for some swelling clays, and possibly for the London clays, the overburden pressure p_z would appear to be a reasonable upper limit. For many clays, however, the actual load would be considerably smaller, and for non-plastic soils the final load could be almost as small as C', Fig. 11a.

For a few tunnels, information is available from which the radial displacements δ_a and δ_b, Fig. 11a, can be estimated. On the basis of this information and on a knowledge of the ring loads, attempts have been made to construct diagrams corresponding to Fig. 11a. The results, although extremely

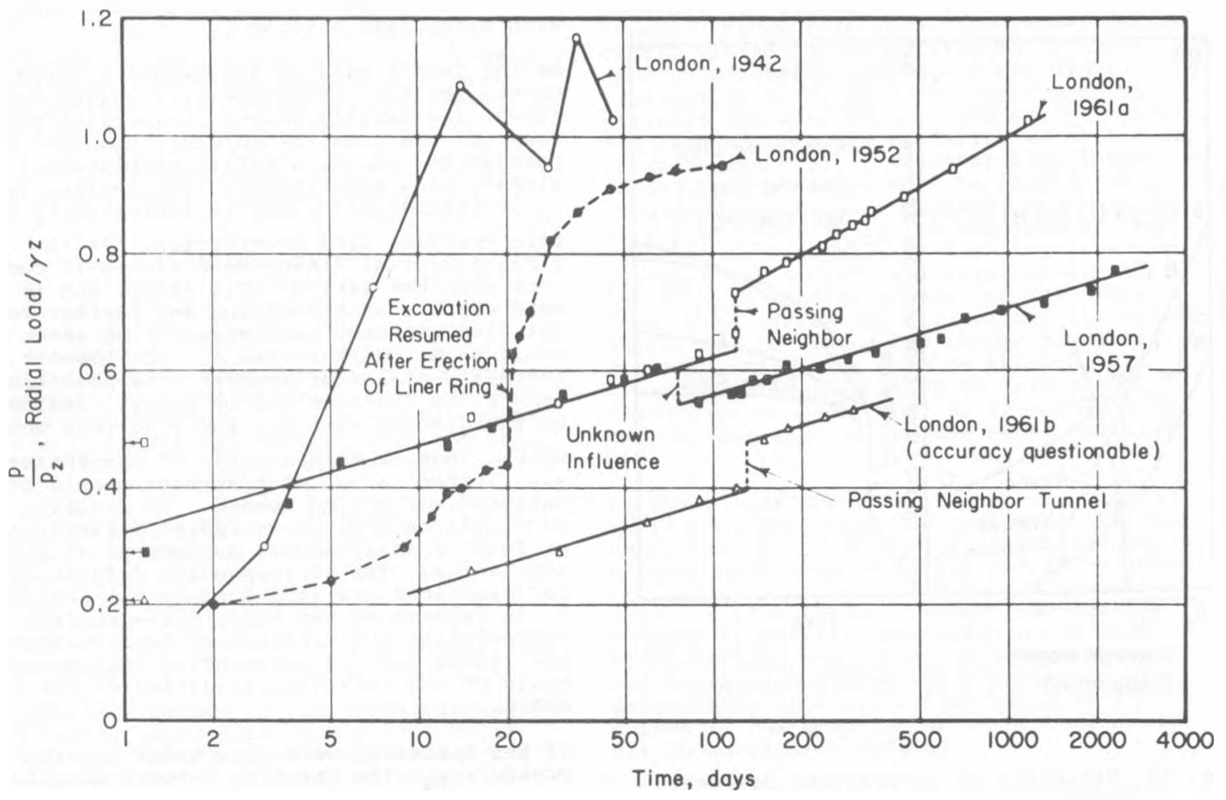


Fig. 12a. Variation of Radial Load on Liners with Time, for London Clays. Loads Have Been Determined from Direct Pressure Measurements or Calculated from Measured Ring Loads.

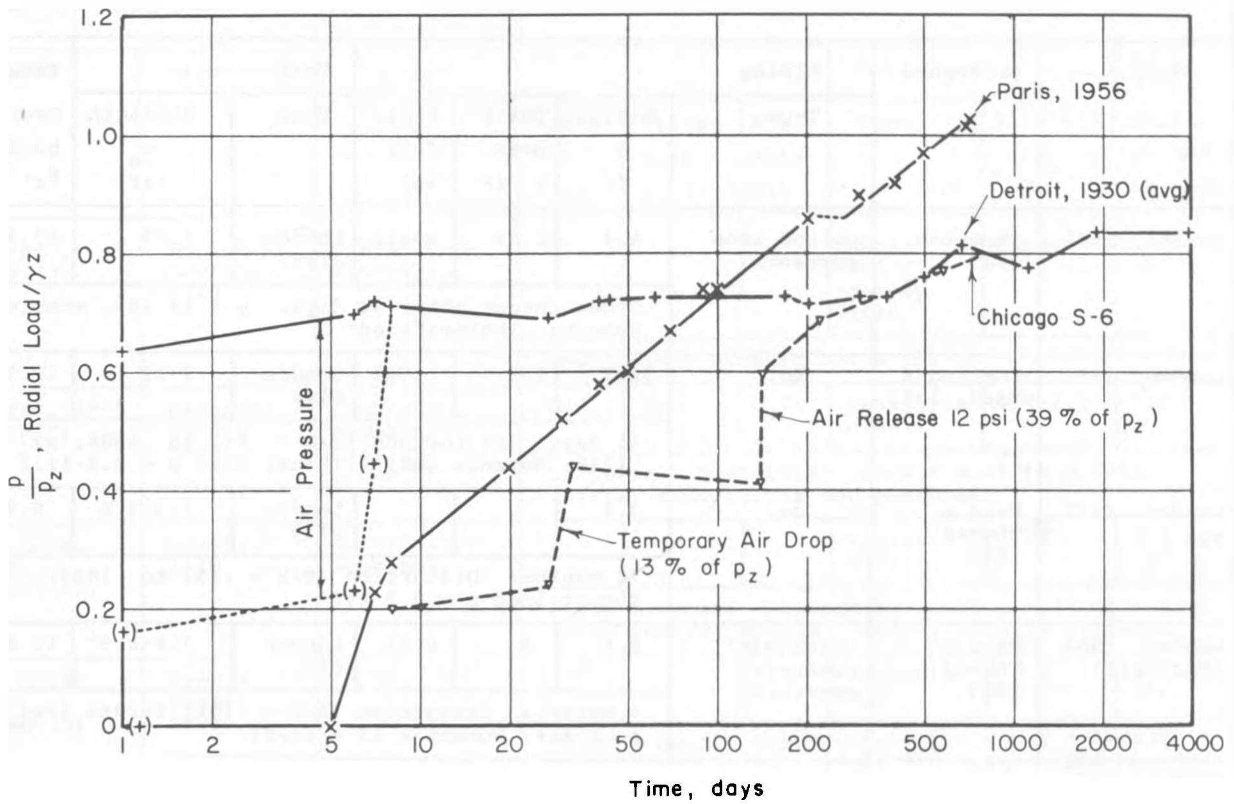


Fig. 12b Data Corresponding to That in Fig. 12a for Soils Other Than London Clay

tentative, suggest that the procedure is reasonable and could serve as a basis for improved design if supported by adequate field data. The proposed basis for present design, and implications for future improvements, are considered at the end of the chapter.

1.5.6 Bending

The principles governing the design of the lining to withstand bending in planes at right angles to the axis of the tunnel have been discussed in connection with Fig. 10. They lead to the following conclusions:

If the lining is flexible it will experience distortion that may be described with sufficient accuracy as a change in horizontal diameter equal to the change in vertical diameter but in the opposite sense. Bending stress in such a lining is irrelevant. For design it is necessary to know only whether the change in diameter will be finite and of an acceptable magnitude. The magnitude depends not only on the nature of the soil surrounding the tunnel, but on the degree to which the shape of the tunnel corresponds to the equilibrium polygon for the initial stress conditions.

Every real lining possesses some rigidity. To the extent that the rigidity reduces the diameter changes below those associated with a flexible lining of the same shape, bending moments will be induced in the lining. The moments can be reduced to a minimum by not installing the lining (or the secondary lining if there is a primary one) until most of the distortions have occurred. In any event, the lining will be adequate with respect to bending if it can be deformed, without overstress, by an amount equal to the change in diameter of a flexible lining of the same shape in the same soil. Hence, if the tunnel is circular, a conservative design would result if the tunnel could without overstress be deformed into an ellipse with horizontal axis appropriately greater than the diameter of the original circle. It is obvious that this conclusion indicates the desirability of a thin rather than a thick lining of a given material.

The basis for design, then, requires a knowledge of the diameter change of flexible tunnel linings of various shapes in a variety of soils, as a function of time. The available information is summarized in Table VII and conclusions are drawn in the following paragraphs.

Table VII Thrust and Distortion of Tunnel Linings in Soil

Case	Reference	Lining				Soil		Total
		Type	Radius R ft	Thickness t, in.	Rigidity ksi	Type	Strength s_u ksf	Overburden p_z, ksf
London, 1942	Skempton, 1943	Bolted iron segments	6.4	1.33	small	London clay	5.75	13.8
			Crown thrust after 46 days: $p \doteq 14$ ksf, stable Moment: insignificant					
London, 1952	Cooling & Ward, 1953	do.	12.7	2.3	~.003	London clay	7-22	12.5
			46 days: Distortion $\Delta R/R$ = .072 to .168%, av. .124%; Moments small; Thrust load p = 5.6-11.2 ksf					
London, 1957	Ward & Thomas, 1965	do.	3.8			London clay	7.2-9.6	6.95
			72 months: Distortion $\Delta R/R$ = .152 to .185%; Thrust load p = 5.3 ksf					
London, 1961 (Victoria) (1)	Ward & Thomas, 1965	"Flexible" concrete segments	6.6	9	0.66	London clay	7.8-8.5+	10.8
			4 months: Distortion $\Delta R/R$ = .15%; Thrust load $p < 6$ ksf; Moment $<$ 12 k ft/ft					
London, 1961 (Victoria) (2)	do.	Flexible iron segments	6.4		~0	London clay	7.8-8.5+	10.0
			4 months: Distortion $\Delta R/R$ = .16%; Thrust load p = 3.5 ksf; Moments ~ 0 at joints					
London, 1952 (Ashford)	Tattersall et al, 1955	Don-Segs	4.4	12	4.8	London clay	22	11
			14 months: Distortion $\Delta R/R$ = .024 (hor.) to .057% (ver.); Thrust load 5.2 ksf					
Charleston, 1968	Gould, 1968	Unlined, horseshoe	3.5			Silty marl	4.5+	10
			Relative horizontal diameter change $\Delta R/R$ = .25%					
Ottawa, 1961	Eden & Bozozuk, 1968	Corr. steel liner plates (primary)	5.0			Leda clay	3.7	6.1
			1 Month: Distortion $\Delta R/R$ = .08 to .17%					
Toronto, 1964 (1)	Matich & Carling, 1968	Bolted iron segments	8.5			silty clay and clayey till	≈0.7	4.4
			Distortion under air pressure (1.44 ksf) $\Delta R/R$=.06%					

Table VII (Continued)

Case	Reference	Lining Type	Lining Radius R ft	Lining Thickness t, in.	Lining Rigidity ksi	Soil Type	Soil Strength s_u ksf	Total Overburden p_z, ksf
Toronto, 1964 (2)	Matich & Carling, 1968	Bolted iron segments	8.5			sand and clayey till	dense ≈0.7	5.5
			Distortions erratic but < .1% (?)					
Norway, 1949 (Tyholt)	Hartmark, 1964 & 1968	Concrete, segments & cast in place	10.8	26	2.7	Sensitive clay	0.7-1.2	7.8
			17 years: Distortion ΔR/R = 0.24 to 0.65%; initial distortions not measured					
New York, 1906 (Penn. RR)	Jacobs, 1910	Bolted iron segments	11.5			Hudson R. silt	?	5.5 max
			First 2 weeks: Distortion ΔR/R = -.44 to -.54%; after several months: nearly back to circular					
New York, 1936 (Lincoln)	Rapp & Baker, 1936	do.	15.5			Hudson R. silt	?	6.5 max
			First 9 days: Distortion ΔR/R = -.4 to -.67% Moment 51 ft k/ft. After 175 days: ΔR/R = -.09 to -.13%; Moment 25 ft k/ft; Thrust load 5.6 ksf					
Boston, 1960 (Callahan)	C.E. Jan. 1961 Richardson	Bolted steel segments	15.4	5/8" web w. stiffeners		Boston blue clay	v. soft	
			First week: Distortion ΔR/R = -.55%					
Garrison, 1951	Burke, 1957; Lane, 1957		17.5 - 18.0			Ft. Union Shale	•	13.2 to 21.6
4A		Ribs & Lagging	18 months, distortion ΔR/R = .35%					
$4B_2$		Yielding ribs	" " ΔR/R = .42% (vert. > hor.)					
4C		Ribs & Lagging	" " ΔR/R = .43%					
4D		Slotted concrete	" " ΔR/R = .27% (3' thick)					
Other tunnel sections		Ribs	2 to 4 months, distortion ΔR/R = .07 to .19%					
			Vertical loads (single tunnels) ranged to 15% of overburden pressure.					
Chicago, S6, 1940	Terzaghi, 1943	Ribs & Liner plates, horseshoe shape	10.0			Chicago clay	0.7	4.9
			Ultimate distortion ΔR/R = .25%					
Chicago, S3, 1940	Terzaghi, 1942	Bolted steel segments, circular	12.5	3/8" web w. stiffeners	.019	Chicago clay	0.6	4.4
			First few days, distortion ΔR/R = -.33 to -.50%, then reversal of trend In second tube: ΔR/R = .05%					

Table VIII (Continued)

Notes: (1) Radius is exterior radius for steel and iron linings, average radius for concrete linings.
(2) Thickness is average or equivalent thickness.
(3) Rigidity is computed by EI/R^3, ksi.
(4) Distortion is positive when lining squats.
(5) "Thrust load" is average radial stress on lining corresponding to measured thrust, or is radial stress measured directly.

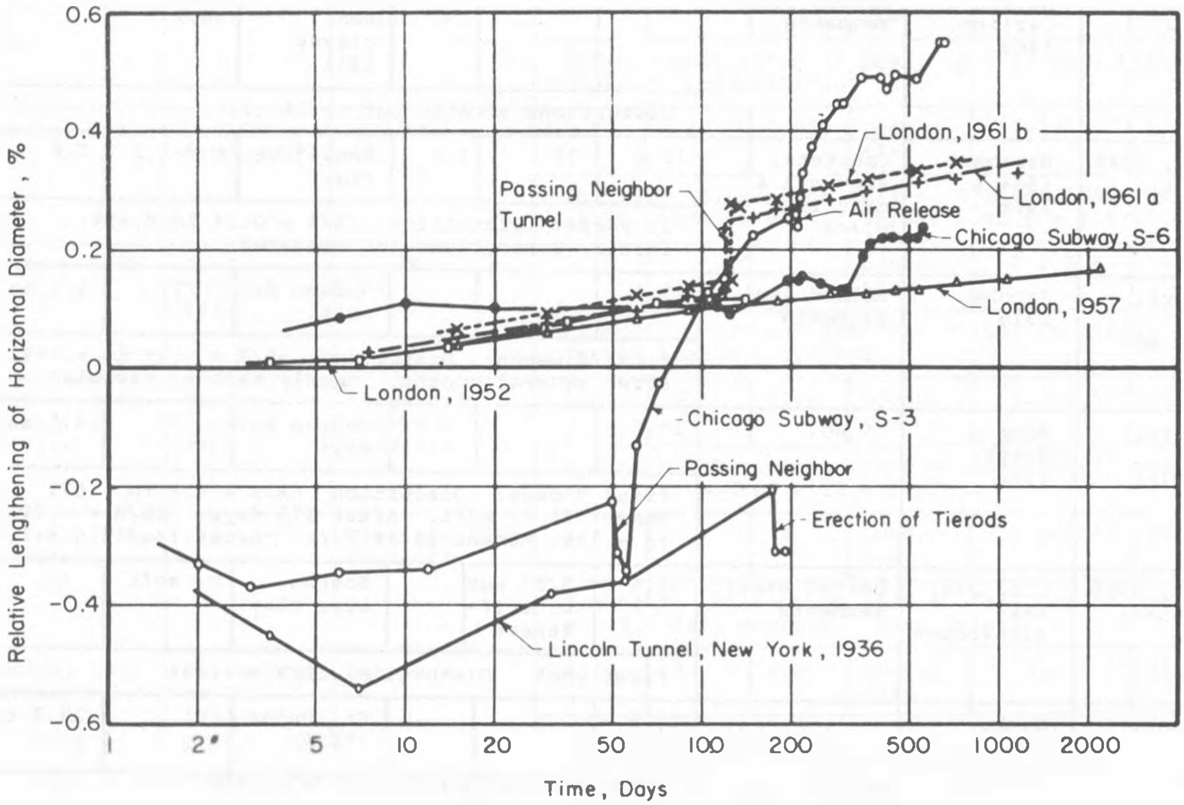


Fig. 13. Increase of Horizontal Diameter of Various Tunnels as Function of Time

Most temporary linings are relatively flexible, especially in the period when the working face is still not far away and the soil surrounding the tunnel is still deforming actively. Some linings have deliberately been made flexible, either as a whole or by the incorporation of joints. Yet, almost irrespective of the rigidity of the lining, and even in soft clays and silts, the changes in diameter of the linings have rarely exceeded 0.5%. In a few instances early distortions have been inhibited by tie rods, but after removal of the ties the distortions have similarly been small.

The findings demonstrate that, even in soft or plastic soils, distortions of more than a few tenths of a percent of the diameter of a flexible lining are effectively prevented by the strength mobilized in the surrounding ground. Moreover, in such soils the rate of distortion decreases with time, as shown in Fig. 13. Inasmuch as the distortion appears to increase roughly linearly with the logarithm of time, curves such as those in Fig. 13 can be used to judge the maximum distortion to be anticipated during the lifetime of the tunnel.

Almost all the tunnels listed in Table VII were constructed in plastic soils. Data are still scanty for tunnels in dense or slightly cohesive sands. The magnitudes of the distortions and the rate of distortion with time would, in all likelihood, be appreciably smaller for such materials.

1.5.7 Buckling

No failure has yet been reported of a tunnel lining by buckling due to earth pressures acting in planes at right angles to the axis of the tunnel, if soil or grout was everywhere in contact with the lining. Such a failure is highly improbable, except possibly in extremely soft cohesive soils providing shallow cover, because of the inability of the soil behind an individual buckling loop to continue to press against the lining without transferring much of the pressure by shear or arching to the adjacent stiffer nodal points.

On the other hand, buckling has been noted if ribs or other forms of lining are not in continuous contact with the soil but, instead, are supported irregularly or at wide spacing by blocking. It has also occurred if poorly filled large voids exist outside the lining. For instance, roof falls in raveling ground have left cavities above tunnels; the cavities have been inadequately filled with timber cribbing that may have rotted, whereupon buckling failures of the roof arch have developed.

Local buckling is also occasionally associated with twisting or ribs due to loss of ground or partial collapses near the heading, and with the forces caused by jacking forward a shield or mole.

Hence, it appears justified on the basis of experience, fortified by theory, to ignore the possibility of buckling due to forces in planes normal to the axis of the tunnel. On the other hand, the lining and its components should have a certain minimum resistance against local buckling, torsional failures, and other modes of failure that may tend to develop as a result of construction procedures and irregularities.

Buckling due to external water pressure is conceivable if the tunnel is located just below the bottom of a deep body of water, and if the soil cover of the tunnel is shallow and weak. Under these conditions the possibility of buckling should be investigated.

1.5.8 Influence of External Conditions: Multiple Tunnels

Thus far we have considered the design of a single circular tunnel unaffected by external influences other than the surrounding soil. The most significant and perhaps most frequent modification of these simple conditions is the construction of a parallel or nearby tunnel.

We have seen that the ring load in the lining of a single tunnel is likely, except for strongly swelling clays, to approach a value considerably smaller than that corresponding to the weight of the overburden. In contrast, field measurements (Table VIII and Fig. 13) in soils of widely different consistencies have shown that tunneling a second tube drastically modifies the pattern of shearing stresses around the first tube and increases the load on the lining of the first tube. In the nearly complete breakdown of arching associated with the 7 parallel hard-clay tunnels at Garrison Dam, the loads on the concreted test tunnel increased from roughly 15 percent of full overburden pressure to almost 100 percent after all 7 tunnels had been driven. If the secondary lining is in place in the first tunnel before the second passes, it is conceivable that the load on the first tunnel may reach a value somewhat in excess of overburden pressure.

The influence of a second tunnel on the bending moments in the lining of the first tunnel may also be assessed for certain conditions under which observations have been made (Table VIII). If the lining of the first tunnel was fairly flexible, as in the Chicago S6 test section and the Garrison test tunnel sections A, B, and C, notably unsymmetrical distortions occurred. The magnitudes of the distortions were well within the capabilities of the primary linings; if the secondary linings of such tunnels were not placed until after the distortions had occurred, no bending moments of consequence would have to be provided for. On the other hand, placement of a relatively rigid secondary lining before passage of the second tube, as in Garrison test tunnel section D, might lead to substantial bending moments and possible cracking.

The effects on the distortion of such a severe occurrence as the passage of a second shield tunnel close to the first (clearance of 2.5 ft between two 25-ft tunnels in soft clay) proved on the Chicago Subway to be almost negligible, and to be controllable with such temporary devices as horizontal struts in the first tunnel.

Although the multiple-tunnel problem is more complex than the single, the semi-empirical approach to design of the lining is essentially similar. Field observations provide the basis for an estimate of the distortion of the first tunnel that would occur if its lining were perfectly flexible. If the distortion is not objectionable, or can be held within reasonable limits by struts or tie rods when the second tube passes, a flexible lining would be structurally adequate for the first tube. If a semi-rigid lining is considered preferable, it is proportioned so that the stresses will not be excessive if the diameter change in any direction (not necessarily horizontal or vertical) is taken equal to that postulated on the basis of the available field evidence.

Table VIII Influence of Neighboring Tunnels, Air Pressure, and Other Factors	
London, 1961 (Victoria) (1)	Distance neighbor tunnel: 21' min. center to center Passing of neighbor caused $\Delta R/R$ = .12%; Thrust load increase 1.2 ksf After 40 months total $\Delta R/R$ = .32%; Thrust load 6.95 ksf (?) At 12 months, moments ranged -8.4 to +12.5 k ft/ft
London, 1961 (Victoria) (2)	Distance neighbor tunnel: 21' min. center to center Passing of neighbor caused $\Delta R/R$ = .11%; Thrust load increase 1.3 ksf After 40 months total $\Delta R/R$ = .35%; Thrust load 10.0 ksf Moments ~ 0 at joints
Both tunnels above	6' shaft at 3' clear distance: $\Delta R/R$ = .038%, both tunnels 6' shaft at 1 1/2' clear distance: $\Delta R/R$ = .066%, concrete tunnel $\Delta R/R$ = .087%, iron tunnel Opposite side and invert remained in position, crown descended
Toronto, 1964 (1)	Release of air pressure: $\Delta R/R$ = .12% Passing of neighbor (distance 21' center to center): $\Delta R/R$ = .09%
Toronto, 1964 (2)	Passing of neighbor (distance 21' center to center: $\Delta R/R$ = .12 to .35%
Garrison, 1951 4A 4B 4C 4D $4B_1$ Other sections 4A 4C 4D	Distance of 7 tunnels: 74' center to center Passing of neighbor tunnel caused $\Delta R/R$ = .042% Passing of neighbor tunnel caused $\Delta R/R$ = .036% Passing of neighbor tunnel caused $\Delta R/R$ = .050% Passing of neighbor tunnel caused $\Delta R/R$ = .028% (a second neighbor tunnel caused $\Delta R/R$ = .027%) Passing of neighbor tunnel caused $\Delta R/R$ = .010% (concrete lining) Passing of neighbor tunnel caused $\Delta R/R$ = .012 to .032% (concrete, 2.5 to 3' thick) Vertical load increase for multiple tunnels, from 15 to 25% of overburden Vertical load increase for multiple tunnels, from 9 to 15% of overburden Vertical load increase for multiple tunnels, from 8 to 95% of overburden. Vertical load on 4D also influenced by increase in embankment load.
Chicago, S6, 1940	Distance neighbor tunnel: 28.5' center to center Passing of neighbor caused $\Delta R/R$ = .10% (Skew distortion) Subsequent distortion increase erratic; total (stable after 400 days) $\Delta R/R$ = .25% Final pressure measured on tunnel invert 3.3 ksf
Chicago, S3, 1940	Distance neighbor tunnel: 27.8' center to center Struts placed in first tunnel during passing, subjected to very small loads Passing of neighbor tunnel caused $\Delta R/R$ = -.10% but the distortion quickly reversed at accelerated rate Release of air pressure: Distortion accelerated, may have caused increase in ultimate distortion of $\Delta R/R$ = .25% Ultimate distortion .25%

Note: For further details, see Table VII.

1.5.9 Influence of External Conditions: Other Factors

It has already been pointed out that a reduction in air pressure after a lining is in place causes a corresponding increase in ring load. If the lining is circular, it has a negligible effect on the bending moments in the lining. If the lining is not circular, the bending effects depend on the flexibility of the lining. If the lining is flexible, a slight distortion may occur, as may be judged from Fig. 13. If it is practically rigid, moments will develop corresponding to the uniformly distributed air loading. The restoration of groundwater pressures in pervious soil, after construction with the aid of drainage, has similar effects.

It is common practice to consider the effect on a tunnel lining of the weight of an adjacent or neighboring building. The lining is considered to be acted on by vertical and lateral pressures usually calculated by the theory of elasticity. That such a procedure has no basis in reality is evident against the background of the semi-empirical approach. As the tunnel heading passes the loaded area, somewhat greater radial deformations and distortions might be expected than where the load is not present. To the extent that these would take place before the primary lining is installed, before the tailpiece clearance is grouted behind a shield, or before lining is expanded against the soil behind a mole, the lining would be unaffected by the loading. Unless the movements were so much greater than usual as to lead to excessive loss of ground, their magnitude would be of no consequence. As a matter of fact, experience indicates that such loads ordinarily produce additional distortions so minor that they are not discernible in normal tunneling operations. Furthermore, if the distortions are completed before a secondary or permanent lining is placed, the presence of the loading has no influence on the secondary lining. By undergoing slightly greater distortions as the tunnel goes by, the soil develops shearing stresses capable of supporting the extra load before the tunnel is lined. The subsequently erected lining experiences these stresses only to the extent that the shearing stresses relax.

Excavation of a deep basement adjacent to an existing tunnel may lead to a radical change in the stresses or deformations in the lining. The extent of the change depends on the lateral movements permitted in the soil at the edge of the excavation. It is commonly assumed that earth pressure at rest acts on the far side of the tunnel whereas active earth pressure acts on the near side; this assumption is likely to be overly conservative, because displacement of the tunnel toward the excavation causes reduction of earth pressure on the far side as well. On the other hand, if the excavation is so poorly controlled that a slope failure occurs, the pressure on the near side may become zero and the base support of the tunnel may also be affected. Recommendations for design of a tunnel to cope with the many possibilities are discussed subsequently. Structural integrity in the longitudinal direction is at least as important under these conditions as is that in planes normal to the axis of the tunnel.

Jacking loads from advancing shields, or reactions for tunneling machines, make severe demands on linings, especially since the affected portion of the lining is near the heading where equilibrium has not yet been achieved. Nonuniform conditions introduce eccentricities tending to cause buckling, crushing and twisting. In many instances, a lining adequate to withstand these loadings is fully capable of withstanding all other conditions to which it may be subjected.

Finally, the effect of shrinkage in cast-in-place concrete linings may be significant, especially in the longitudinal direction. The effect is often aggravated by contraction caused by the temperature drop from the high temperatures that may prevail at the heading if the concrete is placed under compressed air. The effects, if inadequately guarded against, may lead to far greater cracking and deterioration of the lining then any defects due to inaccurate evaluation of the forces or deformations in planes perpendicular to the axis of the tunnel.

1.5.10 Recommended Design Procedures

Theoretical studies and the increasing storehouse of full-scale field observations lead to the conclusion that no tunnel lining is likely to be acted upon by forces corresponding to the classical states of active or passive earth pressure, or even earth pressure at rest. Design based on such calculations should be discontinued. Indeed, any design procedure based on the premise that the lining constitutes a structure acted upon by a fixed system of loads rests upon a fallacy; even procedures that allow for modifications of such loads by invoking the concept of a modulus of subgrade reaction are equally objectionable because they ignore the overriding influence of the deformations of the soil mass that occur before the lining is capable of carrying stress.

To avoid the excessive cost, and sometimes the danger, of design based on a system of fixed loads, and to gain the advantages of utilizing the not inconsiderable strength of the surrounding soils, a semi-empirical design procedure is warranted. It consists of 4 separate steps:

1. Provide adequately for the ring load to be expected.

2. Provide for the anticipated distortions due to bending.

3. Give adequate consideration to the

possibility of buckling.

4. Make allowance for any significant external conditions not included in (1) to (3) above.

For each of the steps, recommendations are given to the extent justified by the present state of the art. Lack of enough information to permit a recommendation is indicative of need for further observational data.

1. Ring Load. According to Fig. 12 the ring load in the lining of a single tunnel, except possibly in swelling clays, is likely always to be considerably smaller than that corresponding to the overburden pressure p_z. Nevertheless, it is suggested that the ring load for design be taken as that due to an all-around pressure $p_z = \gamma z$. Present knowledge is inadequate to permit a more refined estimate. Furthermore, for linings of such commonly used materials as steel, cast iron or structural concrete, design for a ring thrust to withstand an all-around pressure γz would not usually increase the minimum cross-sections that would be used for practical reasons. The design pressure γz also provides a satisfactory allowance for the influence of adjacent tunnels.

In heavily overconsolidated clays such as those in London, it might be preferable to provide for a ring load corresponding to the pressure $1/2(1 + K_0)p_z$ rather than to the overburden pressure p_z itself.

If some presently novel construction material, relatively weak in compression, should be chosen for the lining, a more detailed analysis of the ring load for a single tunnel would be justified. For such an analysis, sufficient data would be needed to allow construction of a diagram, similar to Fig. 11a, for the particular conditions of the project.

If a secondary lining is to be provided, it need be designed to support only its share of the anticipated additional ring load. This share is usually only a small fraction of the overburden load. Additional ring load due to reduction of air pressure should be apportioned to primary and secondary lining in accordance with their stiffnesses AE.

2. Bending. For a single tunnel, an estimate should be made of the magnitude of the change in diameter most likely to occur if a perfectly flexible lining of the same shape as the tunnel were installed in soil comparable to that at the site. If the change in diameter is acceptable with respect to the non-structural requirements, two courses of action are open: (1) to provide an essentially flexible lining such as one consisting of articulated blocks, or (2) to provide a continuous lining that can experience a change in shape from circular to elliptical, by an amount corresponding to the change in diameter, without overstress. The limiting stress, whether in the elastic or inelastic range, should be ascertained by the designer according to the stress-strain properties of the material. Alternative 2 is slightly conservative, because the change in diameter will be reduced by whatever stiffness the lining possesses.

If multiple tunnels are to be constructed, the same procedure should be followed except that the lining should be expected to accommodate the additional distortion associated with the subsequent tunnels. If primary and secondary linings are used, the possibility should be investigated of delaying placement of the secondary lining until all tunnels have been driven.

Studies are needed to determine the distortions that may safely be experienced by unreinforced gunite or shotcrete linings in combination with ring stresses.

3. Buckling. Provisions should be specified and enforced for uniform closely spaced blocking, uniform filling of the annular space behind shields, systematic expansion of the lining against soil, or proper treatment of similar conditions. Structural features explicitly designed to prevent buckling can safely be omitted, with the exceptions previously mentioned.

4. External Conditions. The lining should be designed with ample reserve strength for jacking loads, and for unsymmetrical or three-dimensional distortions likely at the heading itself.

Reasonable circumferential and longitudinal strength and continuity of semi-rigid linings should be provided to allow for normal adjacent operations such as pile driving or excavating on a small scale. The legal liability with respect to effects of large adjacent excavations should be investigated. If lateral movements due to excavation can reasonably be limited by application of modern techniques such as constructing stiff and strong retaining walls in slurry-filled trenches or cylinders, such measures may be preferable, as needed, to general overdesign of all tunnels to meet unknown future demands.

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.

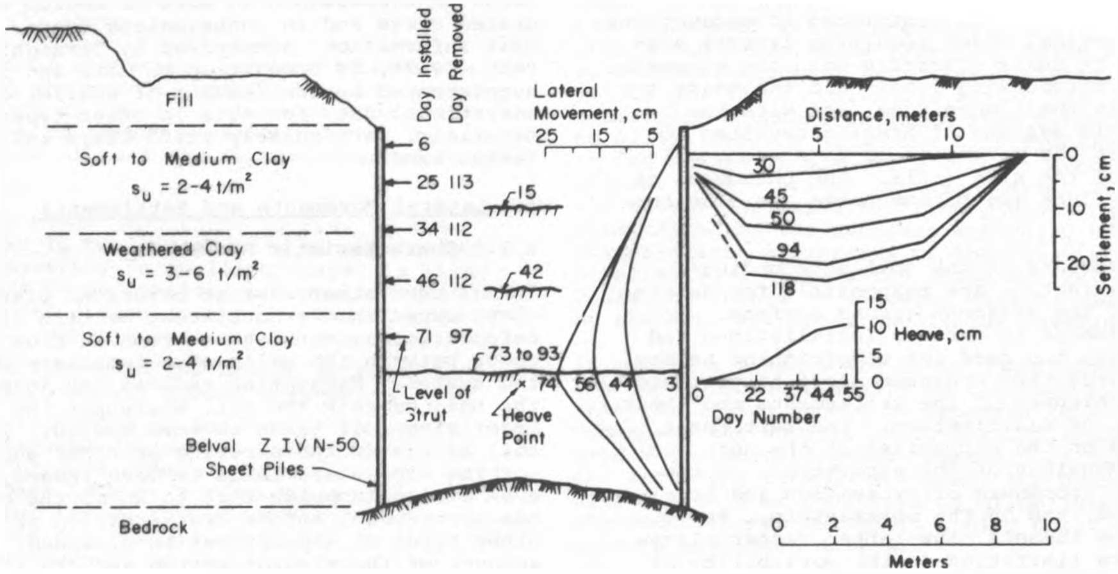


Fig. 14. Relation of Progress of Construction to Lateral Movements of Sheeting and Settlements Adjacent to Open Cut in Soft Clay in Oslo (Vaterland 1)

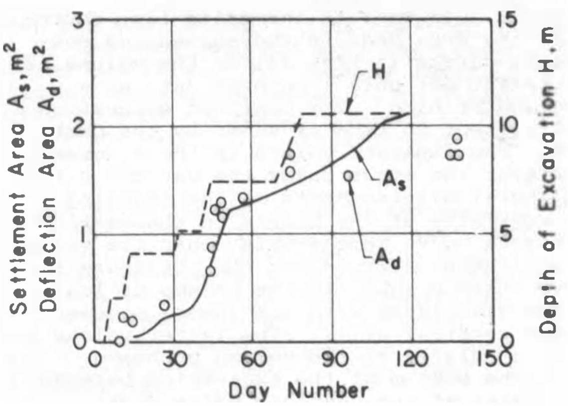


Fig. 15 Comparison of Volume of Settlement A_s and Volume A_d Represented by Lateral Displacement of Sheeting for Cut Illustrated in Fig. 14

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.

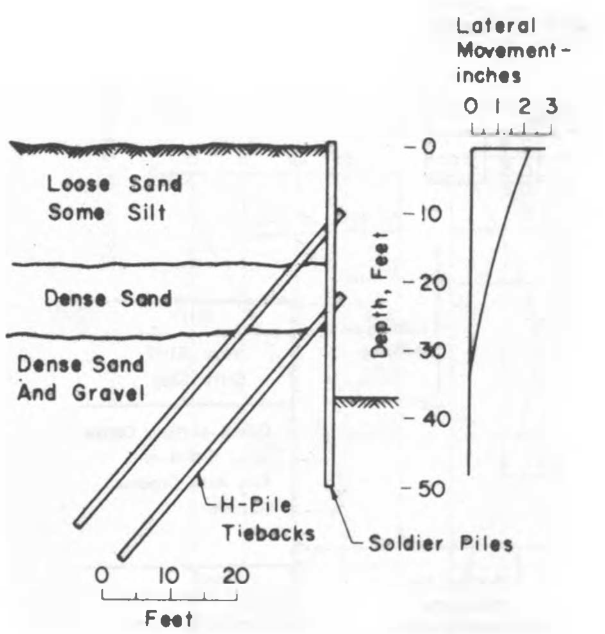


Fig. 16. Section through Cut in Sand Supported by Tieback System; Lateral Movement of Soldier-Pile Wall during Excavation

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.

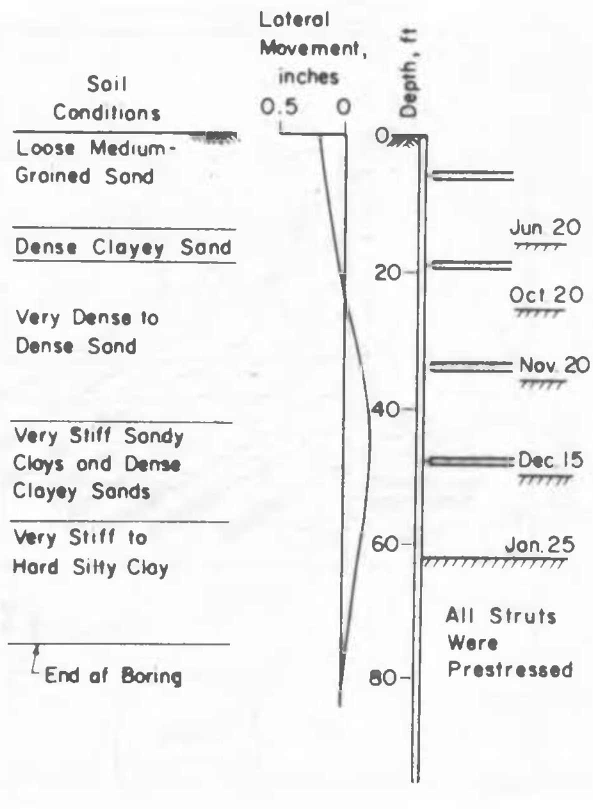


Fig. 17. Section through Cut in Dense Cohesive Sands and Stiff Silty or Sandy Clays, Supported by Cross-Lot Bracing, in Oakland, California; Lateral Movement of Soldier-Pile Wall during Excavation

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,

Movements of Points on Ground Surface, Inches							
Horizontal	1.4		1.2		0.9	
Vertical	1.9		0.7		0.5	

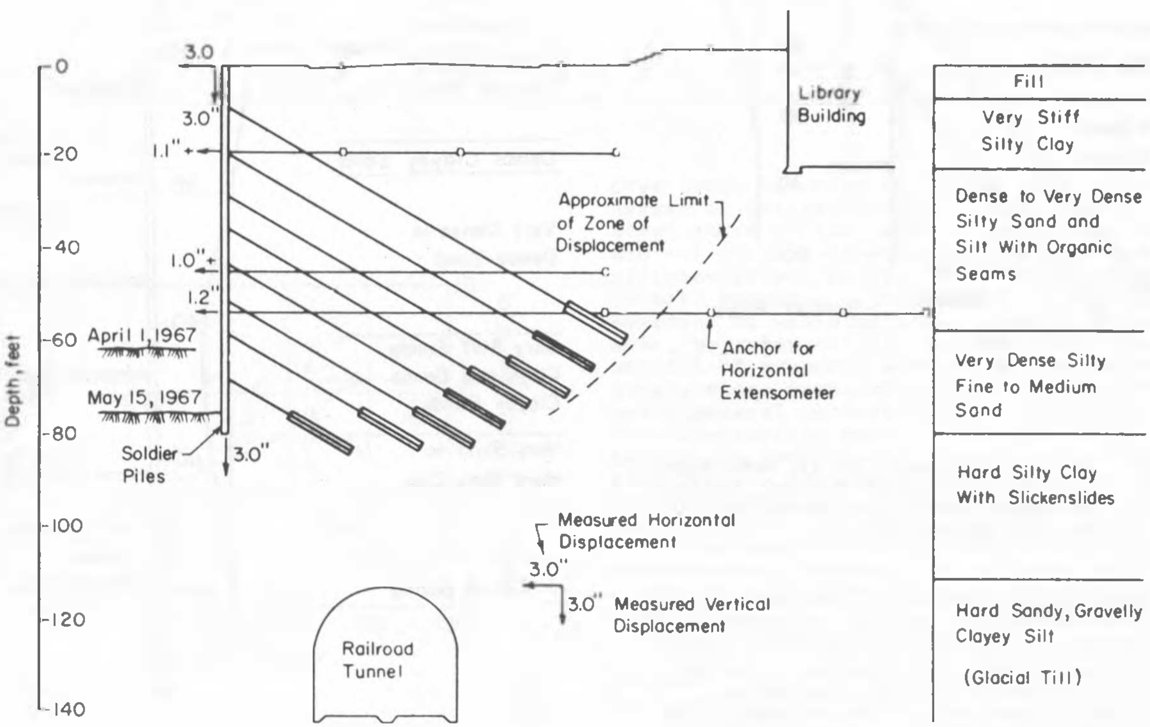


Fig. 18. Section through Cut in Very Stiff Clays and Dense Slightly Cohesive Sands, Supported by Tieback System, in Seattle; Movements of Soldier-Pile Wall during Excavation between Levels Shown

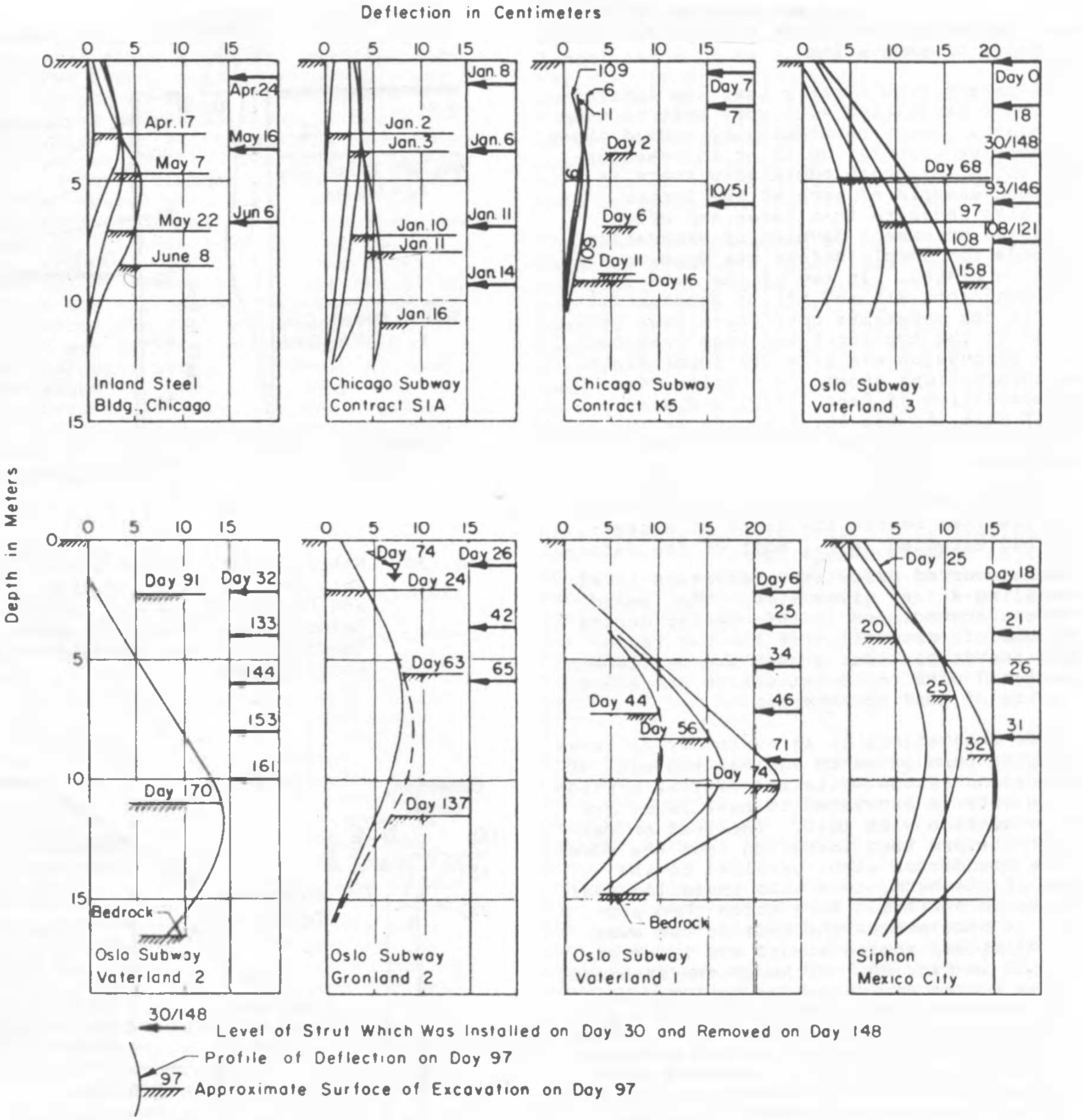


Fig. 19. Profiles of Deflections of Soldier-Pile and Sheet-Pile Walls of Various Open Cuts in Plastic Clays of Very Soft to Medium Consistency

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.

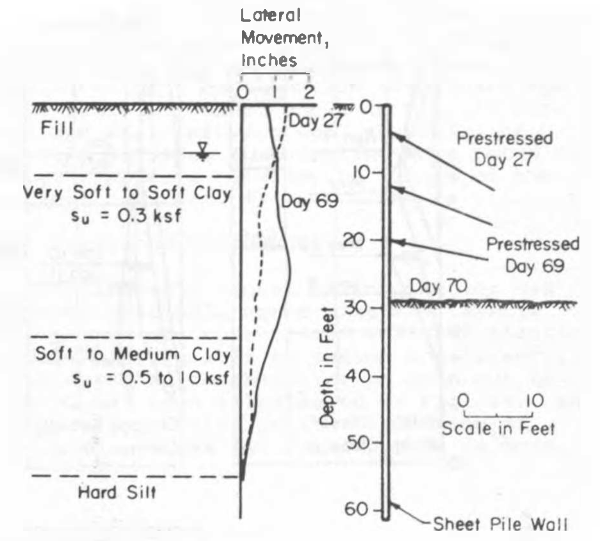
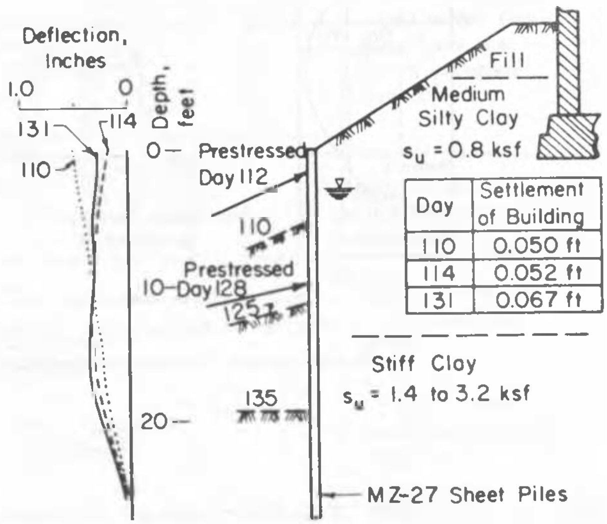


Fig. 20. Raker Support System for Cut in Very Soft to Soft Clay in Chicago, and Deflected Position of Sheeting before Prestressing Top Raker and Immediately after Prestressing the Lower Street



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).

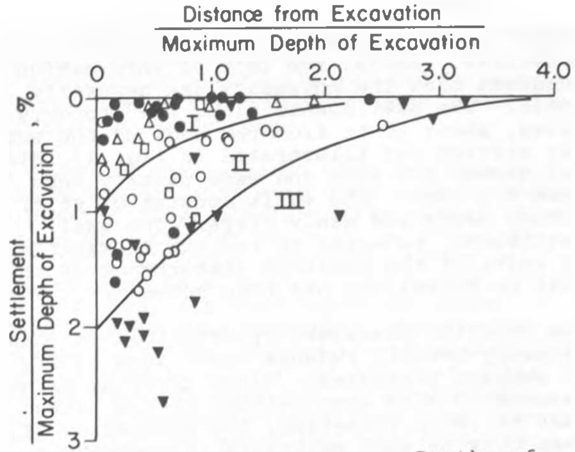
The cohesion possessed by materials in this category greatly reduces their sensitivity to seepage pressures. Since raveling is not associated with open cutting to the extent that it is in tunneling, the excavation of deep cuts in such materials is commonly a straightforward operation. On the other hand, 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



Soft to Medium Clay	Depths of Excavation, ft.
● 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

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 $N_b \geq N_{cb}$

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.

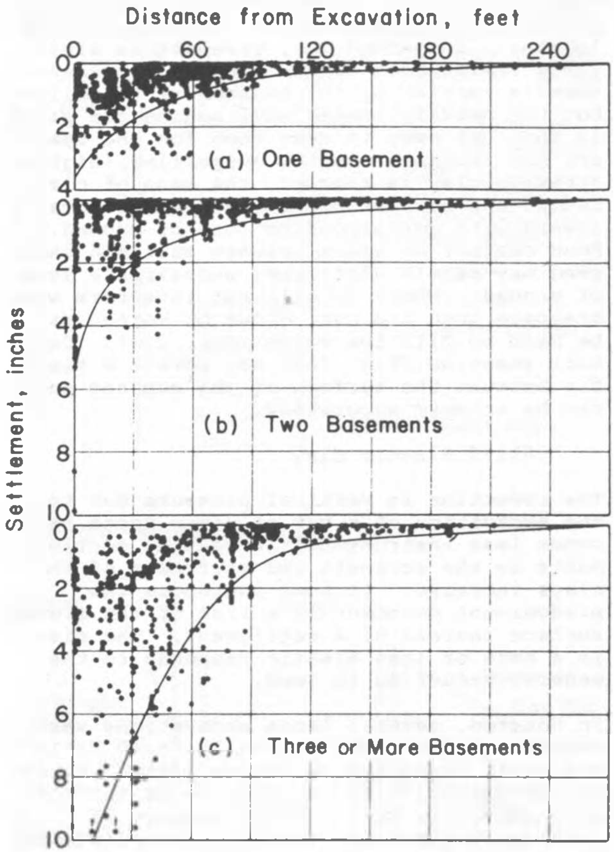


Fig. 23. Settlements Associated with Foundation Construction of Buildings in Chicago on Hardpan or Rock Caissons, and with (a) one basement; (b) two basements; (c) three or more basements

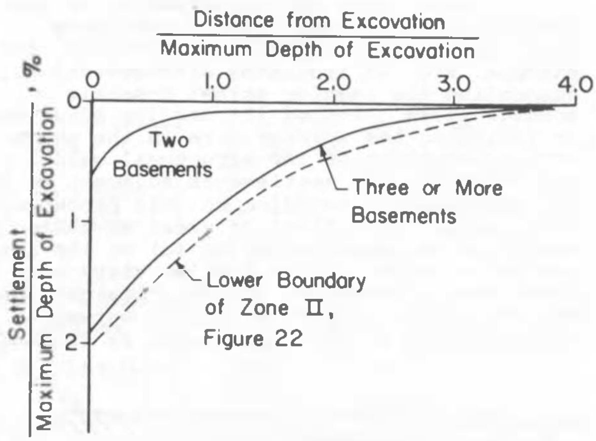


Fig. 24. Summary of Settlements due to Basement Excavations in Downtown Chicago

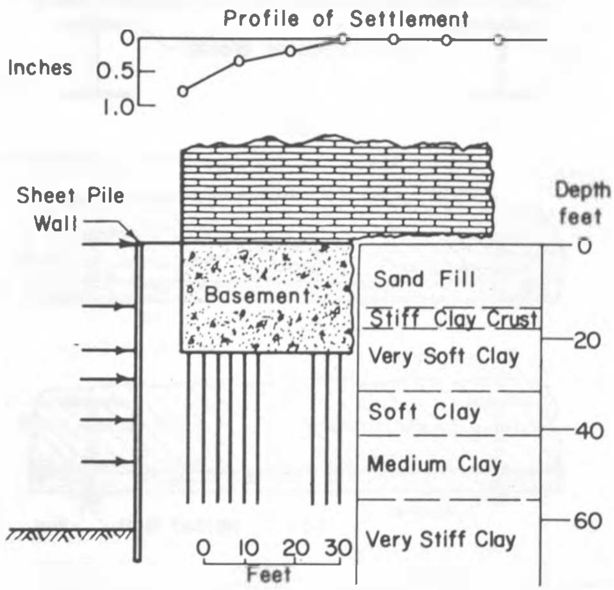


Fig. 25. Section through Pile-Supported Rand McNally Building in Chicago, Showing Settlements Caused by Adjacent Deep Open Cut

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 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.

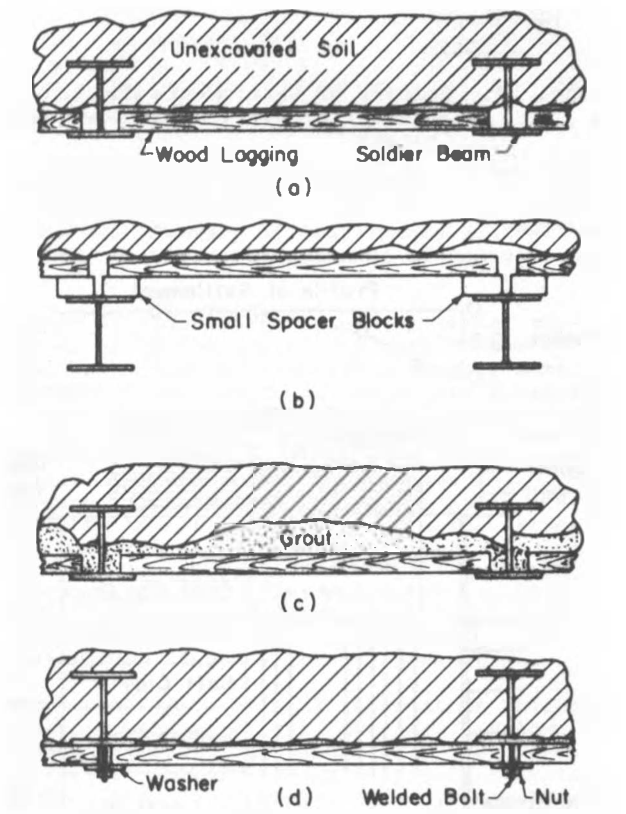


Fig. 26. Methods for Transferring Earth Pressure from Lagging to Soldier Piles; (a) Lagging Wedged Against Inside Flanges of Soldier Piles; (b) Lagging Set Behind Outside Flanges of Soldier Piles; (c) Grout or Mortar Filling Between Lagging and Soil; (d) Contact Sheeting

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

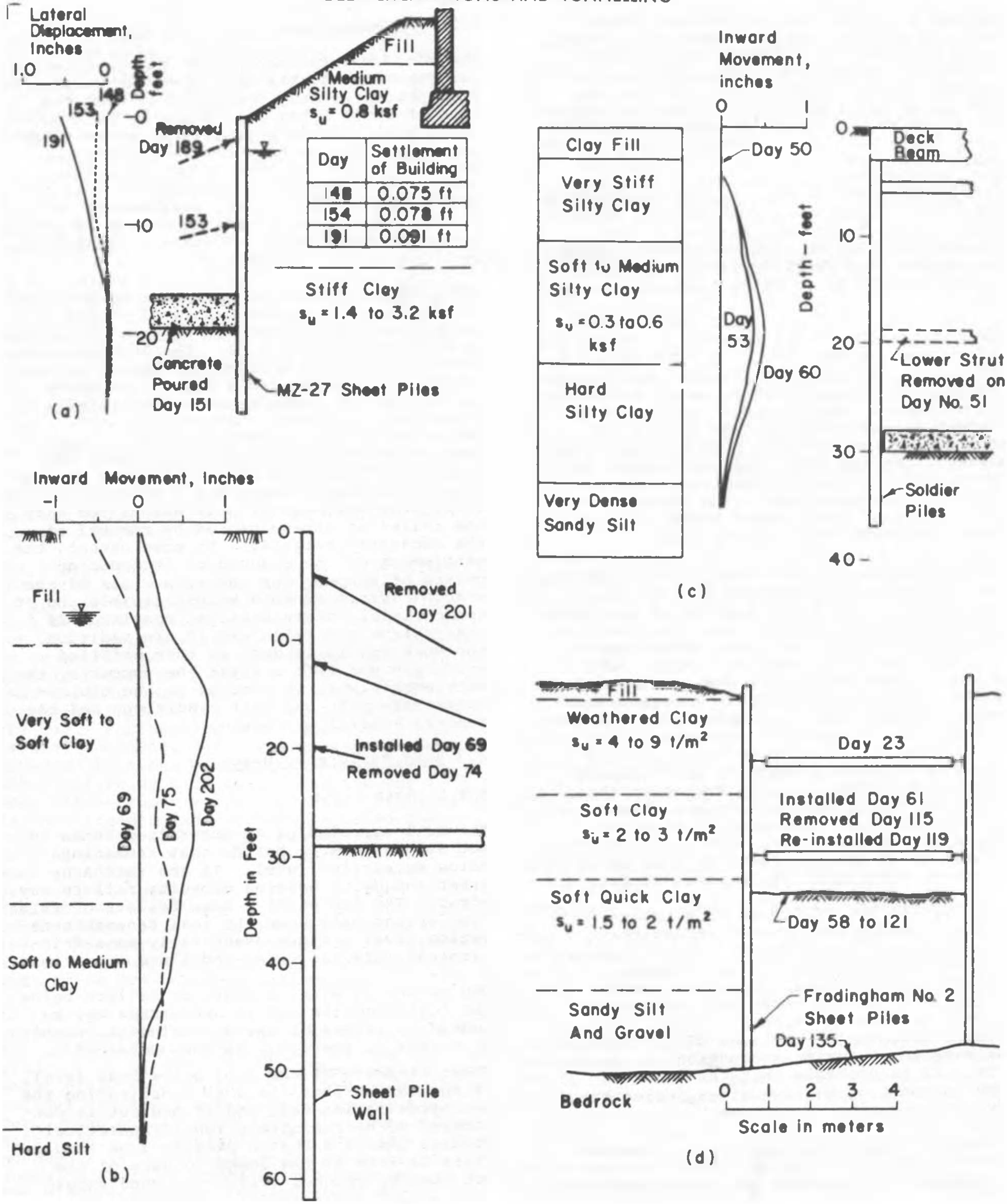


Fig. 27. Lateral Displacements of Walls due to Removal of Struts in Various Open Cuts in Clay: (a) and (b) Raker-Supported Cuts in St. Louis and Chicago respectively; (c) Chicago Contract K5; (d) Telegraph Building, Oslo. Parts (a), (b) and (c) are drawn to the same scales.

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

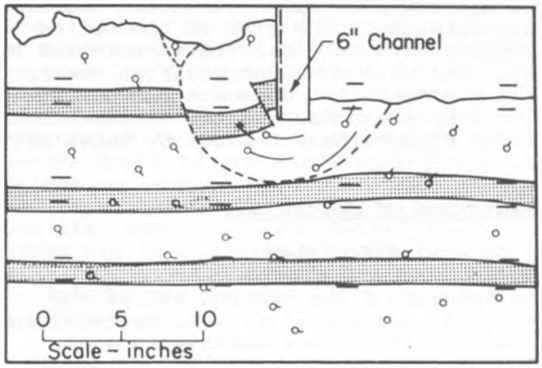


Fig. 28. Displacements of Clay Adjacent to Rigid Model Wall after Excavation on Right Side of Wall. Heavy Dashes Indicate Original Position of Shaded Layers. Symbols Indicate Magnitude and Direction of Movement of Air Bubbles in Clay Mass.

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.

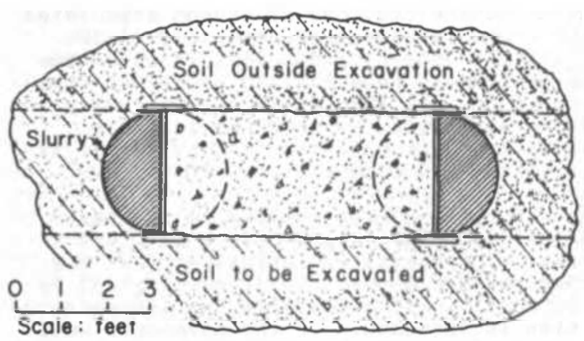


Fig. 29. Horizontal Section through Typical SPTC Wall

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 <u>Earth Pressure</u>

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.

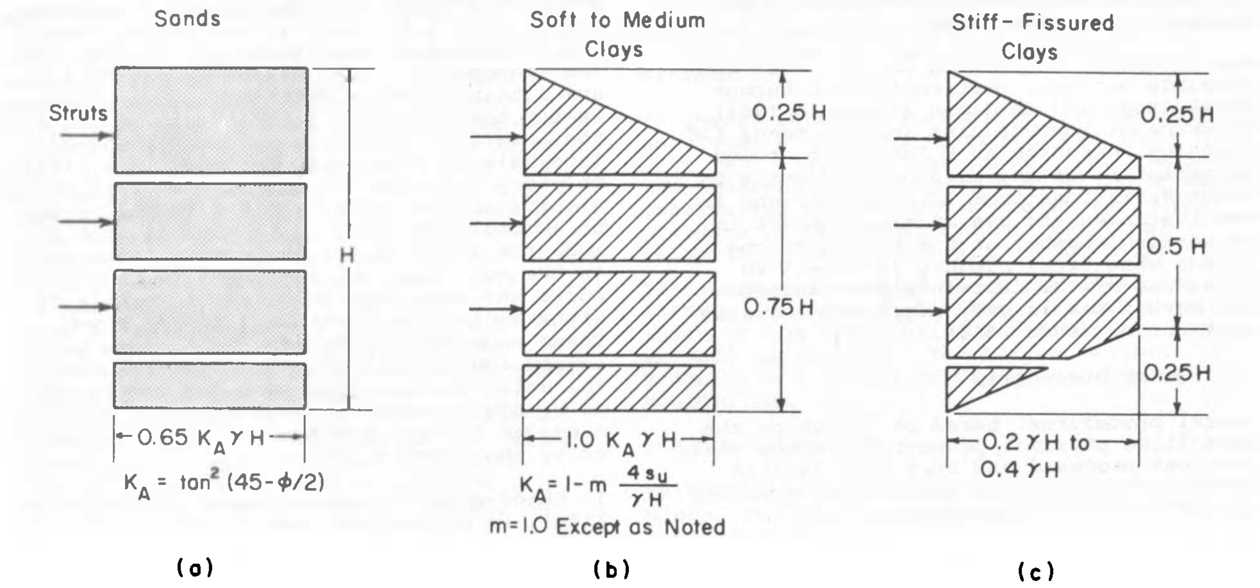


Fig. 30. Apparent Pressure Diagrams Suggested by Terzaghi and Peck (1967) for Computing Strut Loads in Braced Cuts

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

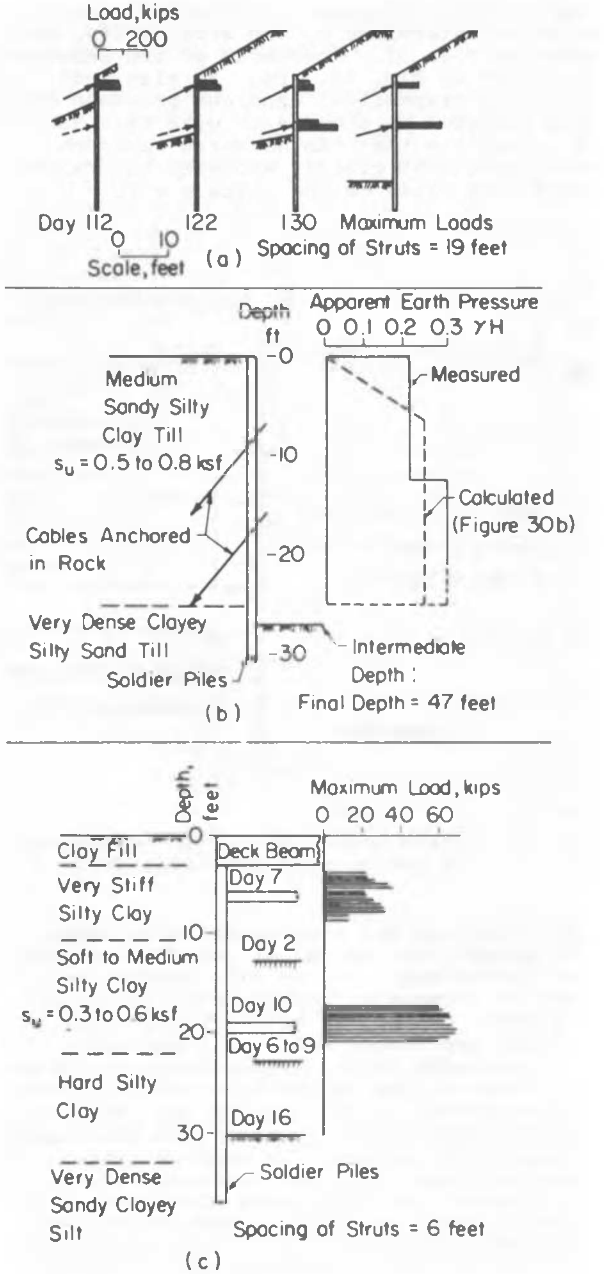


Fig. 31. Loads in Bracing Systems of Several Cuts in Clays. (a) Horizontal Components of Loads in Inclined Bracing of a Cut in St. Louis, Illustrated in Fig. 21; (b) Apparent Pressure Diagrams from Forces Measured in Tiebacks at Pickering Generating Station, Ontario; (c) Preliminary Values of Forces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

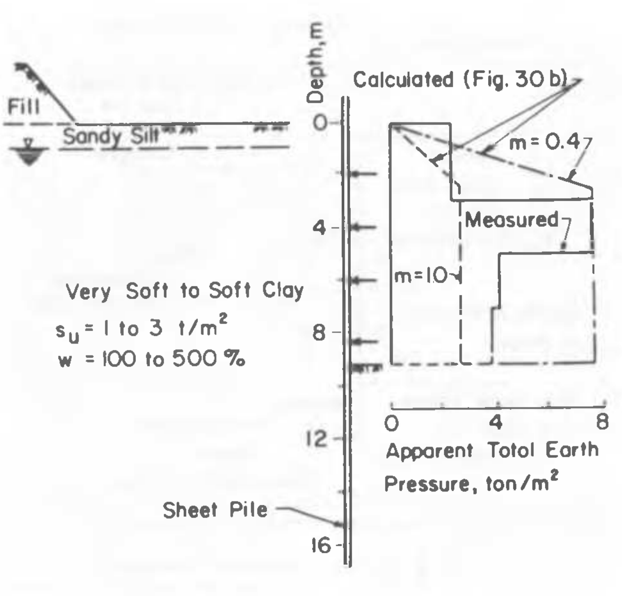


Fig. 32. Strut Loads for Cut in Soft Clays in Mexico City (See Fig. 19)

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.

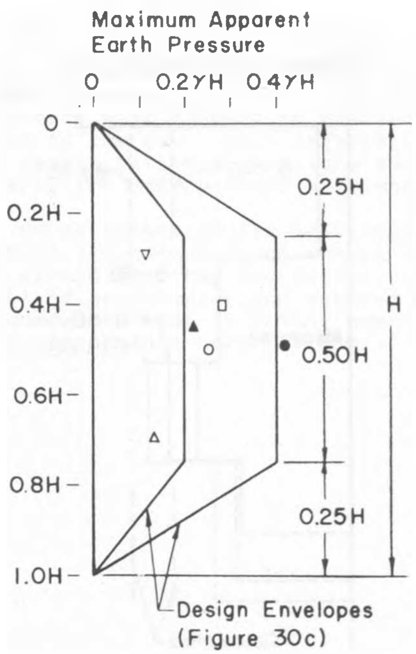
2.5.3 Cuts in Clay, $N < 4$

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
○	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-Dantorth 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

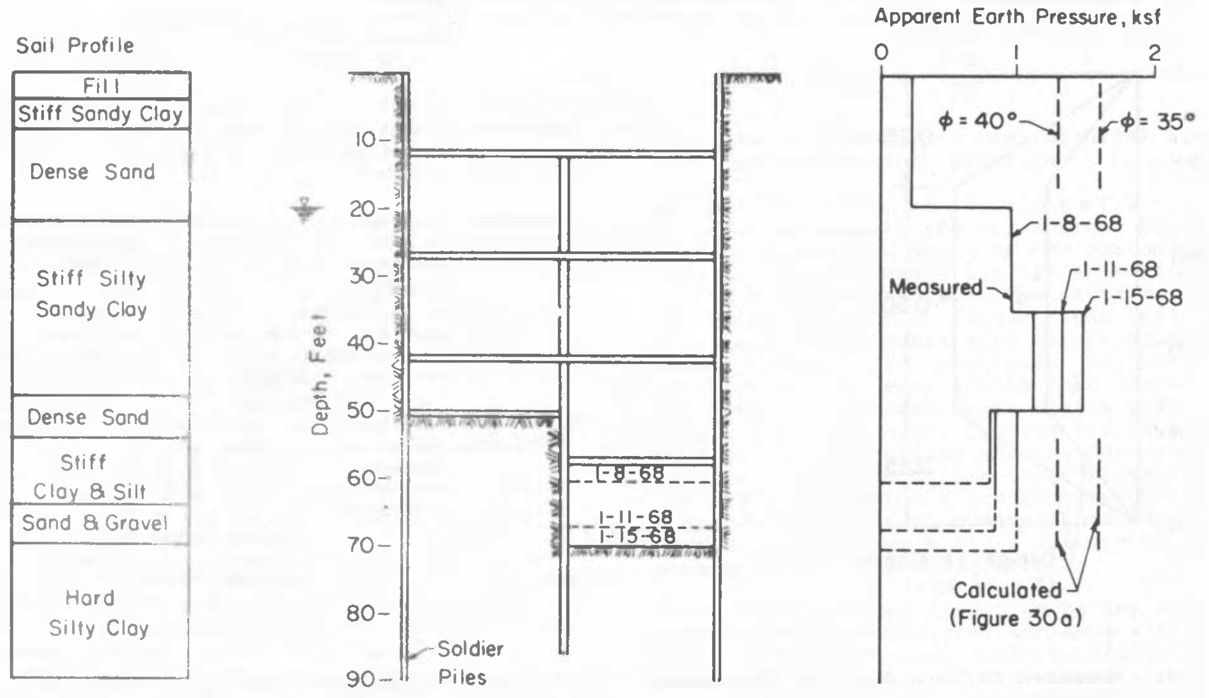


Fig. 34. Strut Loads in Cut in Dense Clayey Sands and Stiff Sandy Clays in Oakland, California

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 comprénd 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 perment 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.

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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.

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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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