

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Tunnelling in Soils

Creusement des Tunnels dans les Sols

Chairman	R.B. Peck (USA)
Co-Chairman	W. Wittke (FRG)
General Reporter	W.H. Ward (UK)
Co-Reporter	M.J. Pender (New Zealand)
Technical Secretary	H. Stille (Sweden)
Panelists	G. Aas (Norway), K. Fujita (Japan), Z. Gergowicz (Poland), B. Ladanyi (Canada), D. J. Henkel (UK)

B. Ladanyi, Panelist

Following the excellent general report on Tunnelling in Soft Ground prepared by our two General Reporters, there is not much left to add or comment. Nevertheless, among many subjects covered in the report, there is one I would like to hear more about, and that is the effect of the rate of the tunnel advance on the resulting loss of ground for tunnels in clay. This subject was discussed in some detail in several publications in the last ten years (Attewell and Boden, 1971; Attewell and Farmer, 1974; Attewell, 1978), but, apart from FEM solutions of the problem, no simple evaluation method has yet been proposed for it.

In the General Report, the subject was mentioned in connection with the excavation of tunnels in London clay, for which it was observed that "the slower the tunnel is excavated and supported, the greater is local unsupported displacement of the ground around the tunnel. But, though it seems likely, it does not appear to have been demonstrated in the field that the volume of ground lost at surface increases when the tunnel is built more slowly."

This statement, although controversial at first sight, points to the fact, in the Discussor's opinion, that when the present tunnelling machines are used in such overconsolidated clays, the majority of the loss of ground is practically instantaneous and is due to the radial movement of the clay to fill the annular void left behind the shield, the thickness of which depends on the type of the machine and the method of permanent lining installation. The response of the clay is instantaneous, whether its behaviour is elastic or elastic-plastic. On the other hand, if the tunnel face is not especially protected, either by the compressed air or otherwise, there will be a continuous time-dependent squeeze of clay towards the face. In overconsolidated clays, this is nevertheless usually only a small fraction of the total loss of ground which may be difficult to detect at the ground surface.

I would like to show in the following how such a squeeze of clay towards the tunnel face can be evaluated and related to the rate of the tunnel advance.

A simple manner to achieve that goal is to consider the face of a cylindrical tunnel, deep in the clay, to be the diametral plane of a hemispherical cavity (Fig. 1), which undergoes time-dependent closure during the tunnel advance. As we are in the short-term domain, the clay response will be practically undrained, and can be described by the equation

$$\epsilon_e = \epsilon_{e,inst.} + \epsilon_{e,creep} \quad (1)$$

saying that the total (von Mises, equivalent) strain ϵ_e change following a change in equivalent stress σ_e , is after a time t equal to the sum of an instantaneous portion, which may be elastic or elastic and plastic, and a creep portion, usually described by a creep equation of the form

$$\epsilon_{e,creep} = k f_1(\sigma_e) \cdot f_2(t) \quad (2)$$

where k is a constant and f_1 and f_2 are functions of stress and time, respectively. Since the undrained strength of clay is also time-dependent, Eqs. (1) and (2) would describe a family of isochronous stress-strain curves such as shown schematically in Fig. 2.

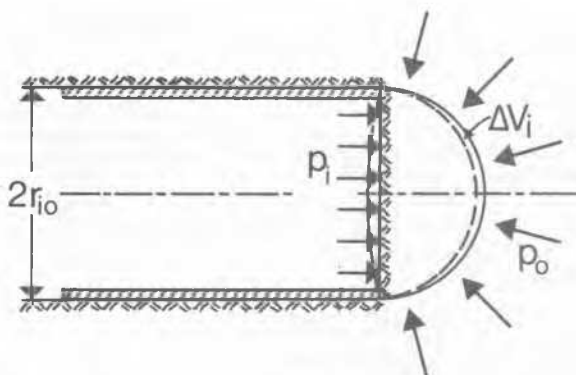


Figure 1. Scheme for estimating the creep of clay towards the tunnel face.

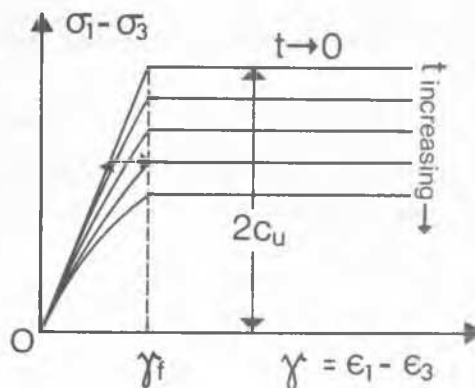


Figure 2. Isochronous stress-strain curves.

In the clay literature, the stress function in Eq. (2) has been most often expressed by an exponential law (Singh and Mitchell, 1968) but it is easy to show that a power law can be made to fit the same experimental data equally well. For the time function, either a power law or a logarithmic law can be used, but the former has been shown to represent well the undrained creep of several different clays (Singh and Mitchell, 1969). So, if power law functions are adopted, for both f_1 and f_2 Eq. (2) can be written as (Ladanyi, 1974, 1980)

$$\epsilon_{e, creep} = (\dot{\epsilon}_c/b)^b (\sigma_e/\sigma_c)^n t^b \quad (3)$$

where σ is the creep modulus corresponding to the reference strain rate $\dot{\epsilon}_c$, while b and n are the creep exponents. All the three creep parameters can be determined by plotting the creep data in log-log plots, as shown, e.g., in Chapter 5 of Andersland and Anderson (1978). Typical values for b and n for a saturated stiff clay would be, from the data shown by Singh and Mitchell, (1968; 1969): $b \approx 0.25$ and $n \approx 1.70$.

The problem of creep closure of a spherical cavity is analogous to that of a cylindrical cavity shown recently by the discussor (Ladanyi, 1980). As long as the instantaneous response remains elastic, the total radial displacement u_i of the cavity wall will be

$$(u_i/r_{io}) = (u_i/r_{io})_{el} + (u_i/r_{io})_{creep} \quad (4)$$

where r_{io} is the initial cavity radius, equal to the mined radius of the tunnel. For the first term in Eq. (4) the Lamé theory gives

$$(u_i/r_{io})_{el} = [(1+\nu)/2E] (p_o - p_i) \quad (5)$$

where p_o is the total average ground pressure at the level of the tunnel axis and p_i is the eventual compressed air pressure acting on the face. The second term may be written as

$$(u_i/r_{io})_{creep} = 1 - \exp[-C \cdot f(t)] \approx C \cdot f(t) \quad (6)$$

where

$$C = \frac{1}{2} (\dot{\epsilon}_c/b)^b (3/2n)^n [(p_o - p_i)/\sigma_c]^n \quad (7)$$

and $f(t) = t^b$

Now, since for small deformations, for a hemisphere

$$V_i/V_{io} \approx 3u_i/r_{io} \quad (9)$$

where $V_{io} = (2\pi/3)r_{io}^3$, the total "face take" ΔV_i at a given stress difference $(p_o - p_i)$ and a given time is then given by

$$\Delta V_i = 2\pi r_{io}^3 [(u_i/r_{io})_{el} + (u_i/r_{io})_{creep}] \quad (10)$$

Alternatively, since from geometrical considerations, assuming a constant volume for the clay in the hemisphere, the average displacement, s , of the tunnel face is related to ΔV_i by

$$s/r_{io} = (2/3)(\Delta V_i/V_{io}) \quad (11)$$

the differentiation of Eq. (4) with respect to time gives the average rate of squeeze of the face towards the tunnel

$$\dot{s} = ds/dt = r_{io} b (\dot{\epsilon}_c/b)^b (3/2n)^n \left(\frac{p_o - p_i}{\sigma_c} \right)^n t^{b-1} \quad (12)$$

which is seen to decrease with time for $t > 1$.

If the rate of tunnel advance is, say, 3 m/hour, the available creep time will be 20 min for each metre of advance, and ΔV_i can be calculated from Eq. (10). An increase in the tunnel advance rate will clearly decrease the available creep time and the corresponding face take due to creep, as observed.

The above analysis is obviously valid only until a creep failure occurs, after which a plastic zone starts developing. The creep failure in clay usually occurs when the accumulated shear strain attains a certain value, as shown by Singh and Mitchell (1969). After the creep failure, the problem can be treated similarly as shown by Ladanyi and Johnston (1973) in the case of deep foundations in non-linear viscoelastic-plastic ground.

REFERENCES

- Andersland, O.B. and Anderson, D.M. (1978). Geotechnical Engineering for Cold Regions. McGraw-Hill, New-York.
- Attewell, P.B. (1978). Ground movements caused by tunnelling in soil. "Large Ground Movements and Structures", Ed. J.D. Geddes, Pentech, London, 812-948.
- Attewell, P.B. and Boden, J.B. (1971). Development of stability ratios for tunnels driven in clay. Tunnels and Tunnelling, 3(3), 195-198.
- Attewell, P.B. and Farmer, I.W. (1974). Ground deformations resulting from shield tunnelling in London clay. Canad. Geotech. J., 11, 380-395.
- Ladanyi, B. (1974). Bearing capacity of strip footings in frozen soils. Canad. Geotech. J., 12, 393-407.
- Ladanyi, B. (1980). Direct determination of ground pressure on tunnel lining in a non-linear viscoelastic rock. Proc. 13th Canad. Rock Mech. Symp., Toronto, CIM Spec. Vol. 22, pp. 126-132.
- Ladanyi, B. and Johnston, G.H. (1973). Behavior of circular footings and plate anchors in permafrost. Canad. Geotech. J., 11, 531-553.
- Singh, A. and Mitchell, J.K. (1968). General stress-strain-time function for soils. Proc. ASCE, 94, SM1, 21-46.
- Singh, A. and Mitchell, J.K. (1969). Creep potential and creep rupture of soils. Proc. 7th ICSMFE, Mexico City, Vol. 1, 379-384.

K. Fujita, Panelist

ON THE SURFACE SETTLEMENTS CAUSED BY VARIOUS METHODS OF SHIELD TUNNELLING

Prof. Peck presented in his "state-of-the-art" report at the conference in 1969, an excellent article on the surface settlements caused by conventional rock tunnelling and hand-mined shield tunnelling, employing additional measures such as ground water dewatering and pressurized air when required. He showed the trough or the range of surface settlements, classifying the ground conditions into three types.

In order to reduce the surface settlement as well as to maintain the face stability, additional measures by means of pressurized air, dewatering, chemical injection, freezing, etc. are applied in the site. However, these measures may cause other problems such as air-blow accidents, oxygen-deficiency accidents to the outsiders, ground surface settlements by consolidation, ground water contaminations, movements of surrounding structures, etc.

Recently, the shield machines have been greatly improved and new types of shield machine have also been developed thereby realizing tremendous progress in tunnelling techniques.

Slurry shields and earth-pressure-balance shields have been developed aiming to maintain the stability of face and to minimize settlement even in the soft ground such as clay layer having an unconfined compressive strength of 50 kPa (0.5 kg/cm²) or less, as well as sand layer having a N-value of standard penetration test 4 or less.

In the case of tunnelling using the slurry shield, excavated soils are fed into the chamber through the slit on the face plate while rotating the cutter face. The slurry pressure resists against the ground water pressure and earth pressure acting on the face. At the same time, the bentonite and other small particles in slurry penetrate into soils around the face. In this way, a collapse of the face can be prevented.

In the case of tunnelling by the earth-pressure-balance shield, the excavated soils are taken into the chamber through slits or large openings provided on the cutter face during the rotation of cutter disk. The screw conveyor discharges

excavated materials, balancing the pressure of soils inside the chamber to both the earth pressure and water pressure acting on the face, while thrusting the shield in such a way. The pressure inside the chamber is relieved through a long passage over the screw conveyor, and soils are loaded onto the belt conveyor free from the pressure. Incidentally, it is also extensively practiced to enhance the face stability by supplying slurry or other liquid into the chamber and, at the same time, to improve the mobilization of soils inside the screw conveyor.

Regarding the surface settlements caused by shield tunnelling, about seventy examples have been presented in various published literatures for the past ten years in Japan, and among these data, the number corresponded to the examination by Peck (1969) was 43.

Fig.1 summarizes the data on the settlements caused by four types of shields; namely, manual-open, blind, slurry and earth-pressure-balance types. It shows whether any difference exists in the magnitude of the settlements depending on the type of shield machines.

Recently, the manual-open type shield is used solely in the case where the ground conditions are favorable or there are particular obstacles under the ground, therefore, the settlement of open type shield is generally large and large in its fluctuation.

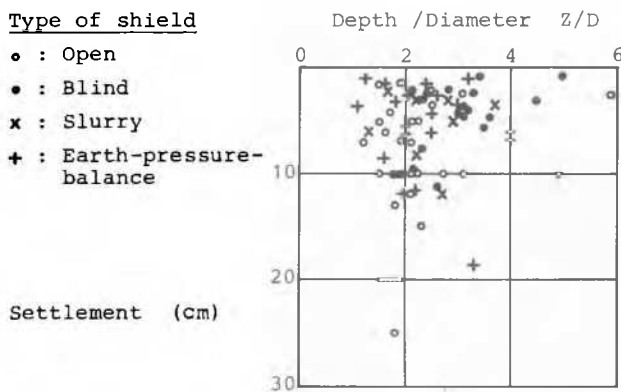


Fig.1 Max. Settlements Caused by Various Shield Tunnelling Method (FUJITA, 1981)

Type of Shield	Zones by Peck;
• :Open	A:Rock, Hard Clays, Sands above Water Level
• :Blind	B:Soft to Stiff Clays
x :Slurry	C:Sands below Water Level
+ :Earth-pressure- balance	

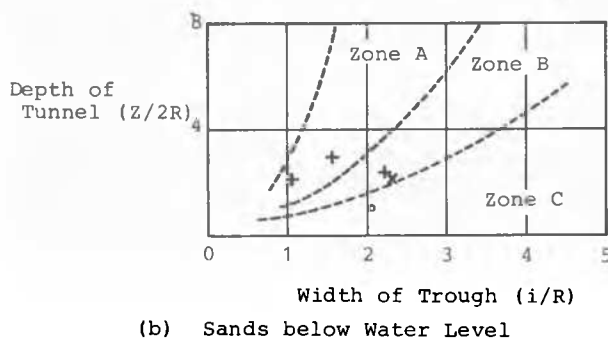
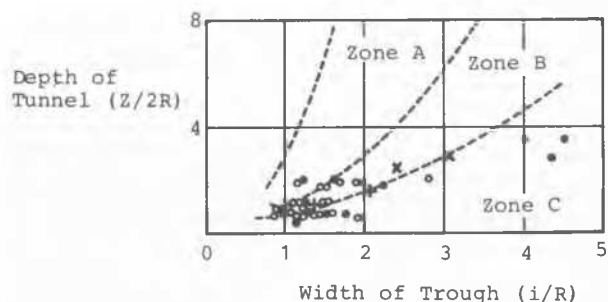


Fig.2 Width of Settlement Trough vs. Depth in Dimensionless (FUJITA, 1981)

Z.B. Gergowicz, Panelist

TUNNELS PEU PROFONDS DANS LES AGGLOMERATIONS URBAINES Shallow Urban Tunnels

Les conditions géologiques et hydrogéologiques sont des facteurs principaux qui décident aussi bien sur le projet que sur la construction des tunnels. Dans les cas des tunnels urbains, aux difficultés dues aux conditions géotechniques désavantageuses, il faut ajouter encore les problèmes des édifices hauts et situés d'une façon compact, du réseau d'installations souterraines et du système existant de communication. Pour les villes relativement jeunes certaine facilité consiste au fait que l'aménagement et toute l'infrastructure urbaine sont enregistrés et localisés dans les documents urbains:

Un autre problème se pose pour les villes anciennes, où avec les années, l'aménagement changeait et se développait; Habituellement, les installations urbaines au cours des siècles étaient construites en désordre; Les installations communales n'étant pas insérées dans les archives et cachées sous terre constituent un obstacle très difficile dans la construction des tunnels, non seulement du point de vue technique, mais parfois aussi en raison de leur grande valeur historique et culturelle.

Though the blind type is used at very soft clayey ground only, the settlement is small.

The slurry type is used where the ground conditions are not favorable. This type causes less settlement and seems to be an effective type.

As to earth-pressure-balance type, the extent of fluctuation in settlements is rather large. The reason of this seems to be that this type has less experience in the operations. Except three particular examples at the earlier application showing rather large settlements, this type assumes good performances.

Generally speaking, the width of settlement trough vs. the depth relation revealed here corresponds in great deal to that of the figure given by Peck in 1969 (Fig. 2).

In the case where the slurry shield and earth-pressure-balance shield were employed in "sands below water level", the similar tendency to Peck's case was obtained, though it was applied in better ground conditions, in spite that the tunnelling was executed in bad ground conditions without any additional measures. From this fact, though number of data is small, these types of shield machine would show excellent performances for the tunnelling in sands below water level.

The relation between width of settlement trough and dimensionless depth of tunnel proposed by Peck could be reasonable in evaluating the tunnelling methods. However, it is generally difficult to compare the advantages and disadvantages of a shield machine only from the surface settlement point of view, because the amount of surface settlement depends greatly upon the workmanship and experience of tunnelling work as well as the subsurface conditions.

Le projecteur et le constructeur ont un dilemme: est-ce qu'il faut garder l'objet historique et si oui, que faire pour que, en même temps, la fonction de l'objet construit n'en souffre pas. Très souvent, ce problème devient grave, parce qu'il apparaît pendant les travaux en cours et il oblige d'introduire des changements dans le projet et dans l'exécution des travaux.

D'après les motivations citées ci-dessus, on peut constater, qu'il serait nécessaire d'élaborer certaines méthodes de travail et de les appliquer dans les cas de ce type d'obstacles. Certainement, la meilleure méthode consisterait à la découverte complète du terrain par une large excavation; Mais il est évident que pour la plupart des cas, il faut exclure ce moyen. Il faudrait donc trouver une autre solution utilisée dans la construction souterraine, mais cette méthode devrait se caractériser par une liberté de manœuvres pendant les travaux et par possibilité d'introduire de différents changements au cours de la construction et même dans la forme définitive de l'édifice.

Comme exemple d'un tel essai, on peut citer la construction du croisement à deux niveaux de

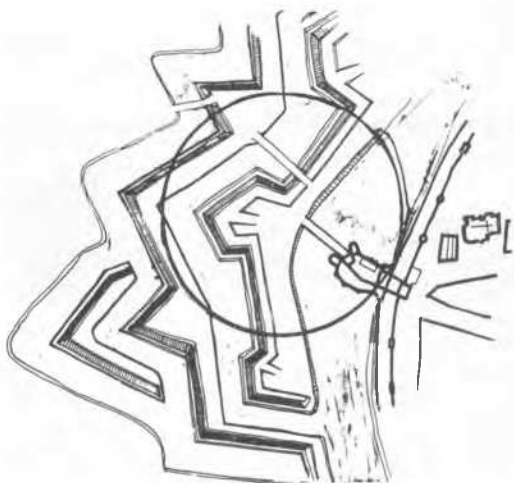


Fig. 1

l'un de principaux noœds de communication à Wrocław, capitale de la Basse Silésie (Pologne) qui compte 650 milles d'habitants.

Ce croisement est localisé sur un terrain comportant des constructions médiévales et moderne de XXème siècle et les constructions tout à fait récentes bâties sur la place des édifices détruits pendant la guerre. Ces deux zones étaient séparées par un fossé d'anciennes fortifications. Cette séparation a été rompue par l'aménagement d'un remblai servant d'infrastructure de chaussée à la place d'un vieux pont.

L'état de la région de la construction jusqu'à 1807 est présenté sur la figure 1, tandis que la figure 2 présente la situation actuelle sur laquelle on a marqué le carrefour souterrain.

Après les sondages de reconnaissance, on pouvait constater que les conditions géotechniques ne sont pas bonnes, car il y avait une grande hétérogénéité du sol qui se trouvait sur le terrain d'une future construction. La couche supérieure d'épaisseur de 3 à 6 m était formée de gravois de brique, mélangé avec du sable, des pierres, de l'argile et avec du sol organique; ensuite il y avait une couche de 3 à 5 m d'épaisseur composée de sable fin et moyen avec les inserts irréguliers de l'argile sableuse. La couche suivante d'environ 6 m d'épaisseur constituaient des graviers. Sous les graviers, il y avait une couche d'épaisseur d'environ 40 m d'argiles glaciaires. S'il s'agit de niveau d'eau souterraine, il correspondait au niveau d'eau dans le fossé et vacillait d'environ 6 m de la surface du terrain.

On savait de certaines informations historiques que sur le terrain de futures constructions doivent se trouver les restes de fortifications d'autrefois et en principe les ruines d'une porte de ville du Moyen Age. Malheureusement, on ne connaissait ni sa localisation, ni son état et ses dimensions.

Prenant en considération l'aménagement existant, la configuration du terrain (fossé), les spécifications du sol et mentionnée ci-dessus les monuments historiques du Moyen Age, on a proposé de situer le trafic routier à la surface et les passages pour piétons au sous-sol. Pour cet-

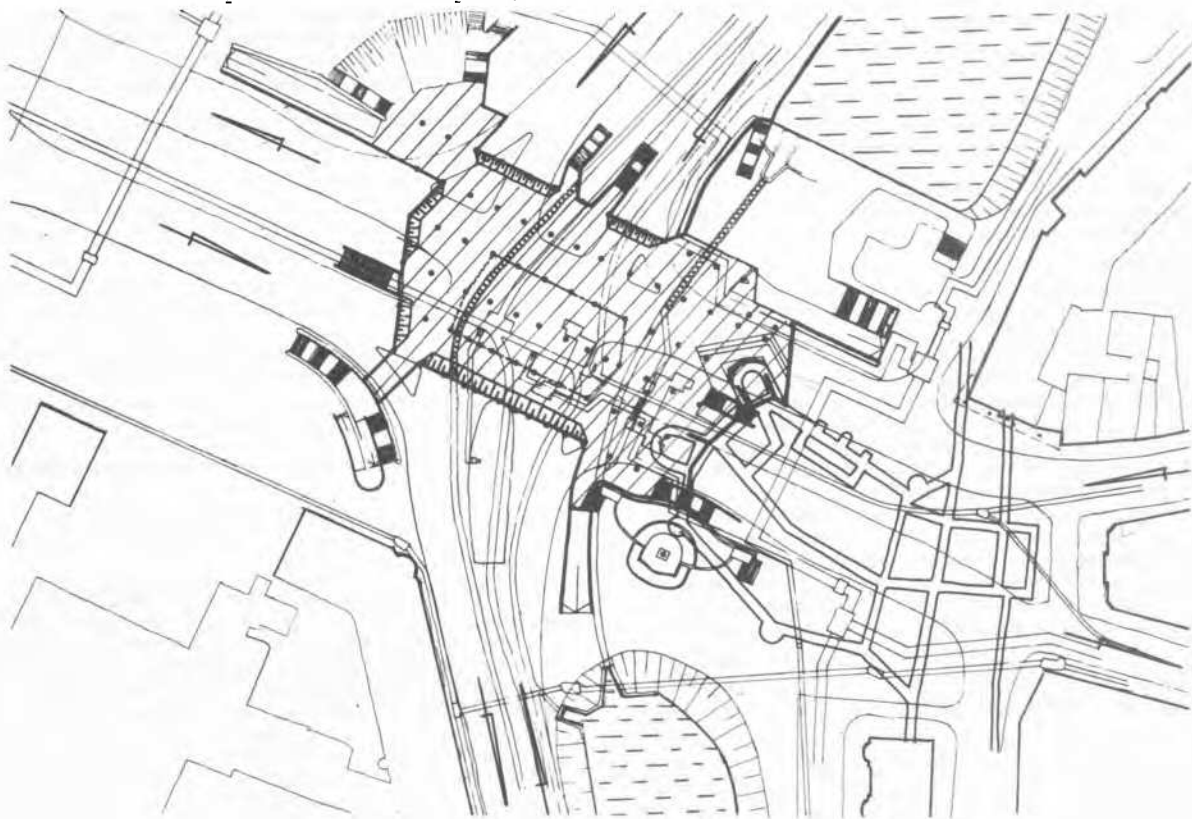


Fig. 2

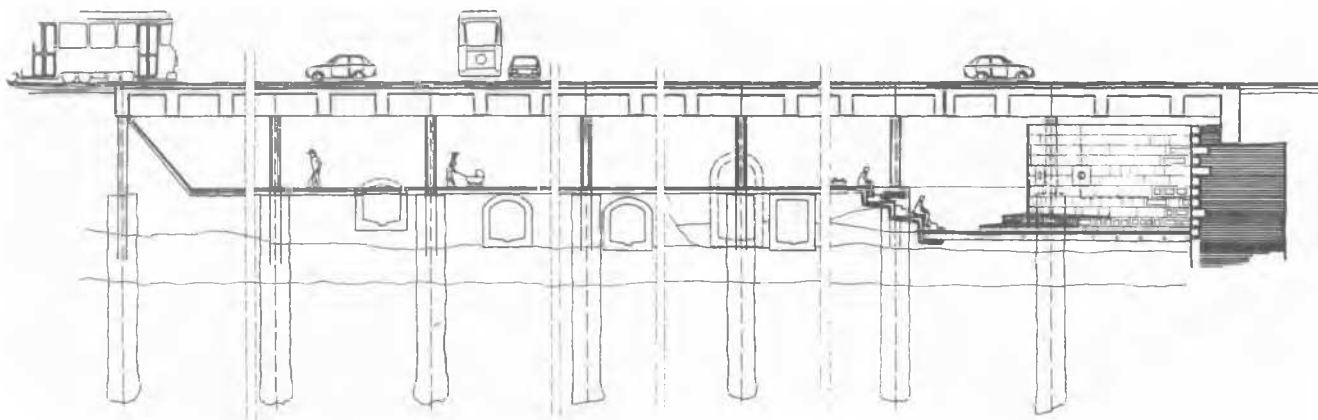


Fig. 3

te raison, on a admis la méthode suivante de construction:

- 1^{ère} étape - construction des réseaux de pieux à béton armé type pieux Franki,
- 2^{ème} étape - construction de panneau à béton armé appuyé sur les poutres qui lient les pieux particuliers,
- 3^{ème} étape - travaux souterraine sous le panneau existant,
- 4^{ème} étape - tous les travaux achevants.

La figure 3 présente la coupe longitudinale de la construction terminée.

Toutes les premisses montraient que cette méthode de construction sera la plus profitable. On y avait la possibilité d'introduire de divers changements de localisation de pieux, selon besoin et les difficultés pendant des travaux. En même temps, on croyait que l'espace relativement libre sous la chaussée permettra de mécaniser les travaux et aussi de l'utiliser plus tard selon les désirs.

La technologie de travaux prévoyait le sondage de reconnaissance sous chacun de pieux. Dans ce cas-là, il s'agissait non seulement de déterminer les spécifications du sol mais aussi de découvrir et localiser les restes historiques. Pour cette raison sur les terrains où on prévoyait de trouver les ruines, on a décidé de faire 2 ou 3 sondages sous chaque pieu. Dans le cas positif, la continuation des travaux appartenait à la géophysique qui appliquant les méthodes correspondantes devait donner les renseignements assez précis concernant la géométrie des objets trouvés. Selon les résultats obtenus, on pouvait décider de localisation de chaque pieu. Hélas, la pratique a montré que cette méthode de construction n'a pas donné tant de résultats positifs qu'on s'attendait.

D'abord, il y avaient des difficultés avec la construction de parties supérieures de pieux au cours de leur passage par la couche de gra-

vois, de pierres, restes de fortifications etc. En outre, il faut dire que cette méthode n'a pas réussi sur le terrain où il y avaient les restes de monuments historiques. On a constaté surtout que les sondages et les mesures géophysiques ne sont pas suffisants sur les terrains dont on avait un grand nombre de données (p.ex. géophysiques). Cette superfluité d'informations désorientait à un tel point, qu'on était obligé de faire une pleine excavation pour pouvoir élaborer une documentation complète et estimer la valeur de l'objet historique. Ensuite, il fallait recouvrir cette excavation et on pouvait alors, commencer à mettre les pieux. Certainement, il était possible de resoudre ce problème autrement, mais en raison du temps, on a utilisé cette forme de travail.

Un autre inconvénient consistait à ce que pendant le montage des pieux, on a détérioré tout ce qu'il se trouvait à 2 m autour. De cette façon, on a endommagé un vieux émissaire d'évacuation qui n'était pas marqué dans les documents-archives et aussi on a provoqué la panne des conduites d'eau.

Les résultats finale permettent de citer quelques avantages de cette méthode:

1. construction de panneaux (plaque) de la chaussée sans échafaudage et sans coffrage;
2. possibilité d'ouvrir assez rapidement le trafic routier sur la surface;
3. bonnes conditions pour mécaniser les travaux;
4. facilité de disposer de l'espace souterraine qui permet p.ex. de montrer les objets historiques au public.

Par contre, en basant sur les expériences obtenues, on peut conclure que la méthode présentée ci-dessus ne peut être utilisée que sur les terrains avec un vieux aménagement, mais à condition de localisation connue des objets historiques qu'on voudrait sauver.

B. Ladanyi, Panelist

When I was first asked by our Chairman, Professor Peck, about two years ago to present to this panel an interesting case history from the current Canadian soft ground tunnelling practice, it occurred to me that there are usually three kinds of such case histories:

- (1) The ones in which a conventional method was used and everything went as expected, proving the validity of the design assumptions,
- (2) The more interesting ones, in which a new and ingenious method was successfully applied for the first time, and
- (3) The most interesting ones in which something went wrong, but which cannot be discussed in public, because they are still under litigation.

The case I have finally decided to present to this panel seemed to me initially to fall clearly in the first category. However, the more I became familiar with it, mainly through some published papers and reports, the more I became convinced that the case was not quite conventional and that a lot could be learned from it.

As the case has been described and thoroughly discussed in several publications (Morton et al. 1977; Belshaw and Palmer, 1978; Palmer and Belshaw, 1979 & 1980; Rowe et al. 1980; Rowe, 1981), only a brief description will be given here.

The tunnel in question is a 3.3 km section of a 2.16 m finished diameter sanitary trunk sewer constructed during 1976 and 1977 in the city of Thunder Bay, Ontario, Canada. Although the subsoil of soft to firm clay overlain by saturated loose sand was known to be difficult for tunnelling, the contractor chose to use a technique novel to North America, i.e., a tunnel boring machine together with an unreinforced, unbolted, segmented precast tunnel lining, which was assembled in the tail piece of the tunnelling machine. The mined diameter of the tunnel was 2.47 m and the outside diameter of the completed lining 2.38 m. Clay grout was injected into the tail piece void.

The soil profile at an instrumented cross section is shown in Fig. 1. This profile is based on the data reported initially by Palmer and Belshaw (1980) and was subsequently modified by Rowe et al. (1980) in accordance with a new field investigation. Three distinct layers are apparent from the profile: Peat, which extends to a depth of 0.9 m, is underlain by 7 m of very loose silty sand and sandy silt. This is underlain by 5 m of soft to firm silty clay, resting on 11.6 m of firm to stiff varved clay. The centre-line of the tunnel is located in the soft to firm silty clay at a depth of 10.5 m. The undrained shear strength of the clay at the tunnel level varies from about 35 to 50 kPa according to Geonor vane tests and subsequent CU tests reported by Rowe et al. (1980).

Owing to the unique nature of the project, an array of instrumentation was installed close to the start of construction, and another one a little later, 1.25 km farther away along the tunnel axis. At each array observations were taken of vertical and lateral soil deformation, pore-water pressure and total pressure on the lining. Surface settlement was monitored at the first array for 22 m on either side of a 60 m length of tunnel. All other instruments were concentrated near the centre of the array. At the second array, surface settlement was monitored over the centre line of the tunnel, along a length of about 48 m and at one cross-section only. All other instrumentation was concentrated as close as possible to

SHEAR STRENGTH, KPA
PENETRATION RESISTANCE, BLOWS

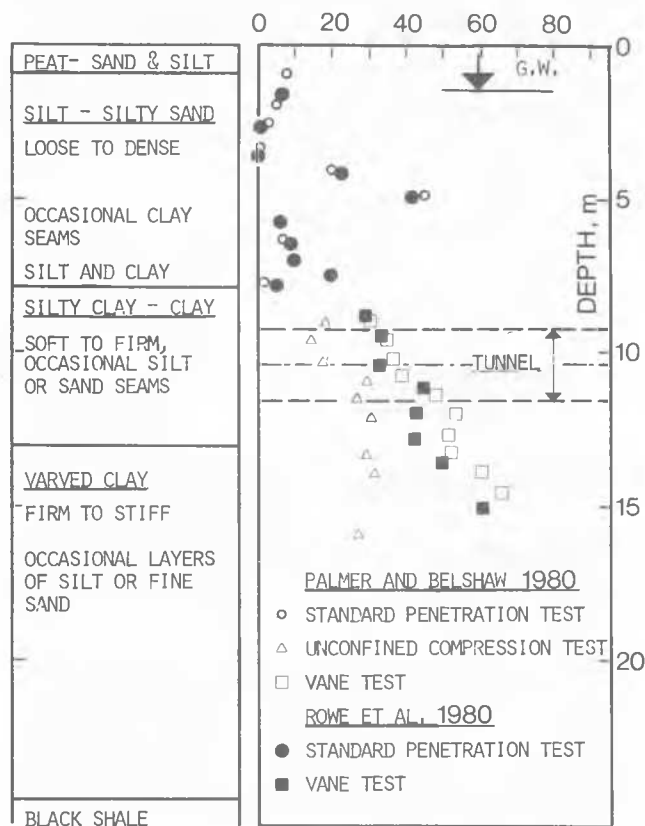


Figure 1. Thunder Bay Tunnel: Soil section and summary of field test results.

the cross-section, with emphasis on pore pressure changes and spatial soil deformations in the vicinity of the tunnel.

Results of observations

Settlement trough

The vertical soil subsidence along a section through the centre of arrays 1 and 2 is shown in Fig. 2, with the range and average centre-line surface settlement indicated. At the end of construction the shape of the trough was similar in both sections, but the average settlement at the centre of array 1 was about 34% greater than at array 2 (59 as against 44 mm). However, after about one year, while the vertical deformation at array 2 increased only less than 15%, at array 1 the settlement almost doubled in the same period (Fig. 3). There were two major differences between arrays 1 and 2. The first was the thickness of the clay cover, which was 1.2 m at array 2, and 3.5 m at array 1. The second difference was that the tunnel excavation started only 90 m from array 1. At that time the crew had very little experience with the new machine and lining technique. Alignment difficulty and grouting problems were noted by the engineer observing the tunnelling operation as the machine passed through array 1. These factors probably combined to remould the clay significantly in the vicinity of the tunnel at array 1, leading to increased long-term settlement.

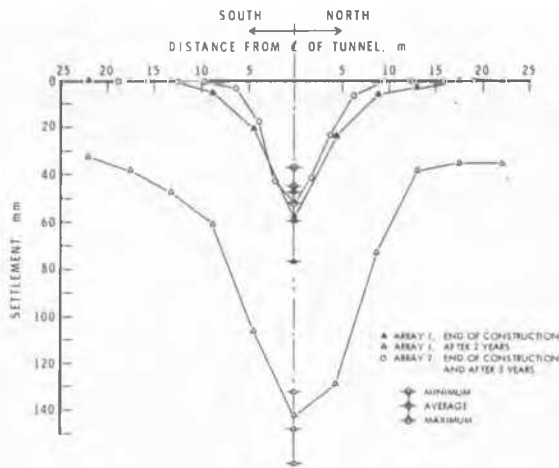


Figure 2. Composite of surface settlement (After Palmer & Belshaw, 1979).

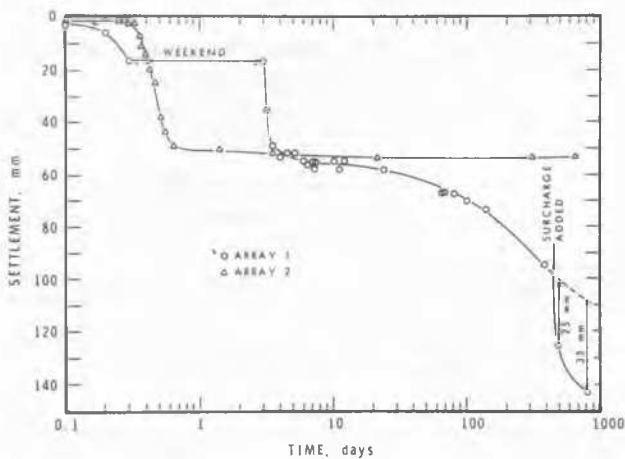


Figure 3. Increase with time of settlement over centre line of tunnel (After Palmer & Belshaw, 1979).

At array 2, the volume of settlement trough as shown in Fig. 2 is about 7.5% of the corresponding total excavated volume of the tunnel, which is approximately equal to the volume of the tailpiece void. The sequence and distribution of vertical deformation is illustrated in Fig. 4, where the observed movements are plotted relative to the position of the face of the tunnelling machine. As the tunnelling machine approached any point, a slight heave was observed, which is considered to be due to the machine thrust. At the ground surface, settlement commenced before the machine face reached the plane of the point, while at greater depths, settlement did not commence until after the face of the machine had reached the point. This indicated that a bowl of settlement precedes the tunnel face, or that there is a continuous flow of clay towards the face, in spite of the closed front doors of the machine. After the tailpiece of the machine had passed any point, settlement progressed rapidly and was practically 85% complete within 6 h (or within about 15 m of advance). Approximately 60-70% of the settlement was observed after the passage of the machine tailpiece.

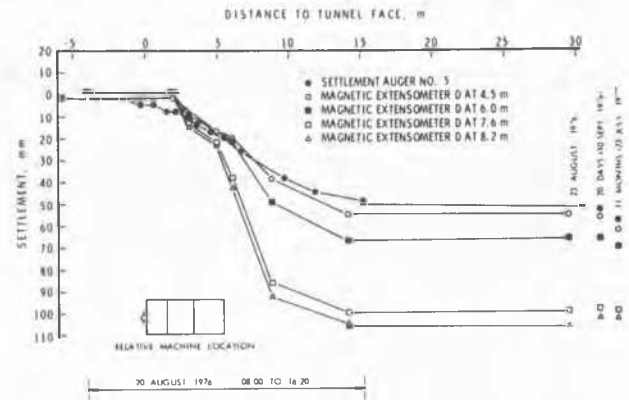


Figure 4. Vertical soil displacement over center line of tunnel (After Palmer & Belshaw, 1979).

Total pressure on the lining

The total pressure measured on a ring of lining at each array was shown in Fig. 7 of Palmer and Belshaw (1980). At each array 11 cells were still functioning after two years. The total pressure was not uniform, but did not show any preferent directions. After two years, the pressure at array 2 slightly decreased, while it increased significantly at array 1 in the same period, which was considered to be mainly due to a surcharge load of 20 kPa which was added one year after construction (Fig. 3). The maximum recorded pressure on each ring after two years was about 95% of the overburden pressure at the measuring point. The average recorded pressure was about 65% (array 1) and 50% (array 2) of the overburden pressure at the spring line. After two years, the average pressure acting on the tunnel lining was evaluated to be of the order of 45-55% of the overburden pressure, with a maximum of 90% of the overburden pressure at the spring line.

Pore-water pressure changes

The change in pore-water pressure as tunnelling progressed reflected very well the soil deformations around the tunnel shown in Fig. 4. As the machine approached the plane of the piezometer, the pore-water pressure increased by about 10%. As the machine passed the piezometer, the pressure rapidly decreased by as much as 40% and reached a minimum when the face of the machine was about 9-10 m beyond the location. Subsequently, the pore-water pressures gradually increased with distance from the tunnel face, but indicated a small net decrease after 10 months.

Discussion of the results of observation

The foregoing description represents a brief review of published data on the considered case history. The question is asked now whether any of the observed phenomena could have been predicted beforehand from the available information on the soil properties, the tunnel geometry and the tunnelling method.

If the attention is focussed only on the observations made at the instrumentation array 2, during which the tunnelling progressed normally, the most important conclusions which can be drawn from this case history are as follows:

(1) The volume of the settlement trough corresponded closely to that of the tailpiece void of 45 mm left behind the lining, although during the machine advance, a clay grout was injected into the void. In other words, the void grouting was virtually ineffective. That most of the ground loss was due to this closure of clay around the tunnel and not to a squeeze towards the face is shown also by the fact that 60-70% of the deformation occurred after the tailpiece of the machine passed the measuring point.

(2) Approximately 85% of the measured settlement occurred within 6 h after the face of the tunnelling machine passed the instrumented section. The settlement increased by less than 15% during the first year following construction. It can be concluded therefrom that the clay around the tunnel responded nearly instantaneously and in practically undrained manner. This is also supported by the pore-pressure observations, which show an initial increase due to the thrust of the tunnel face, followed by a decrease during the flow of unloaded clay towards the void. The subsequent pore-pressure increase may be due to stress redistribution, but might have also been caused by the grouting attempts.

It follows from the foregoing that an undrained analysis is quite justified for preliminary calculations in the present case.

First of all, since $c_u = 35$ to 50 kPa, $\gamma \approx 15$ kPa/m and the tunnel axis is about $z = 10.5$ m deep, the stability ratio $\gamma z/c_u = 3.15$ to 4.50 . As this is lower than 5 no particular difficulties would have been expected during the tunnel driving, but ground loss could nevertheless be high if the tunnel is left unsupported (Peck, 1969).

In fact, as pointed out by Ward (1969), small elastic movements can only be expected if the stability ratio is no more than about 1 or 2, because, without compressed air inside the tunnel, the plastic zone will form as soon as $\gamma z = c_u$.

In the present case, the plastic zone must have developed immediately, and had the tunnel been left unlined, the total radial ground loss would have been approximately equal to

$$\Delta V/V_i = 2c_u \frac{1+\nu}{E} \exp \left[\frac{p_o - p_i - c_u}{c_u} \right] \quad (1)$$

which is derived from the theory of contraction of a cylindrical cavity in an infinite elasto-plastic medium, developed by the discussor in 1961 and published in a paper by DeBeer (1964).

For $\nu = 0.40$, $E = 10,000$ kPa (Rowe, 1981), $p_o = \gamma h = 15 \times 10.5 = 157.5$ kPa, $p_i = 0$, one gets $\Delta V/V_i = 32\%$ for $c_u = 35$ kPa, and $\Delta V/V_i = 12\%$ for $c_u = 50$ kPa. As both of these values are higher than the volume of the tailpiece void, $\Delta V/V_i = 7.5\%$, this means that the void was nearly instantaneously closed after the passage of the shield, so that subsequent grouting was necessarily ineffective. This problem was clearly pointed out by Peck (1969, p. 238).

As far as the total pressure on the lining is concerned, it is interesting to make an approximate calculation by using the Caquot-Kérisel (1966) formula for the pressure on the roof of a tunnel in clay, when the plastic zone attains the clay surface. (The same theory was later shown by Atkinson and Cairncross (1973) to be a lower bound solution of tunnel collapse). According to that theory, the total vertical pressure in the roof of a cylindrical tunnel is given by

$$\sigma_r = p - 2c_u \ln(h/r) + \gamma(h-r) \quad (2)$$

where p is the overburden pressure acting on the clay surface, h is the depth of tunnel axis below the clay surface, and r is the mined tunnel radius. For the array 2, $r = 1.235$ m, $h-r = 1.2$ m, $h = 2.435$ m, $z = 10.5$ m and $p \approx 15 \times (10.5 - 2.435) = 121$ kPa, so that, for $c_u = 35$ kPa and 50 kPa, $\sigma_r/\gamma z = 66$ and 51% , respectively, which is close to the average measured values of the total pressure on the tunnel lining.

It is concluded that this kind of simple calculations may be quite useful for a preliminary assessment of the expected performance of a tunnelling method when used at a given new tunnelling site in clay.

REFERENCES

- Atkinson, J.H. and Cairncross, A.M. (1973). Collapse of a shallow tunnel in a Mohr-Coulomb material. Proc. Symp. on the Role of Plasticity in Soil Mechanics, Cambridge, 202-206.
- Belshaw, D.J. and Palmer, J.H.L. (1978). Results of a program of instrumentation involving a precast segmented concrete-lined tunnel in clay. Canad. Geotech. J., 15, 573-583.
- Caquot, A. et Kerisel, J. (1966). *Traité de mécanique des sols*. 4e édition, Gauthier-Villars, Paris.
- DeBeer, E. (1964). Spanningen en vervormingen in een kleilaag rondom een cilindrische holte met horizontale as. Proc. 4th Int. Harbour Conf., Antwerpen, 107-111.
- Morton, J.D., Dunbar, D.D. and Palmer, J.H.L. (1977). Use of precast segmented concrete lining for a tunnel in soft clay. Proc. Int. Symp. on Soft Clay, Bangkok, 587-598.
- Palmer, J.H.L. and Belshaw, D.J. (1979). Long-term performance of a machine-bored tunnel with use of an unreinforced, precast, segmented concrete lining in soft clay. Proc. Tunnelling '79, Inst. Min. Met., London, 165-170.
- Palmer, J.H.L. and Belshaw, D.J. (1980). Deformations and pore pressures in the vicinity of a precast, segmented, concrete-lined tunnel in clay. Canad. Geotech. J., 17, 174-184.
- Peck, R.B. (1969). Deep excavations and tunnelling in soft ground. Proc. 7th ICSMFE, Mexico City, State-of-the-art Vol., 225-290.
- Rowe, R.K., Lo, K.Y. and Kack, G.J. (1980). The prediction of subsidence above shallow tunnels in soft soil. Geot. Res. Rep., Univ. Western Ontario, London, Ont., 29 p.
- Rowe, R.K. (1981). Preliminary data, Array 2, Thunder Bay Sewer Tunnel. April 1981 (Unpublished).
- Ward, W.H. (1969). Discussion, Session 4, Proc. 7th ICSMFE, Mexico City, III, 320-325.

Ch. Veder (Oral discussion)

ON THE BASIC CONCEPT OF THE NEW AUSTRIAN TUNNELLING METHOD - NATM

GENERAL Even among experts it sometimes does not become quite clear what the distinguishing characteristics of NATM are and therefore I think it is indicated to present its basic concept.

First of all: "Why Austrian?" Different tunnelling methods are developed, depending upon the soil encountered, and commonly carry the name of the respective country, such as the Belgian or the English tunnelling method. Now the Austrian Alps are geotechnically very problematic as the rock there sometimes corresponds in behavior more to soil than to rock. Very complex problems are encountered here in tunnelling work, leading to the development first of the old and then the new Austrian Tunnelling Method.

THE OLD ATM The disadvantages of this process, well known to all experts, are that -

- Owing to the long duration of excavation work, the rock relaxes to a degree that large deformations occur.
- The walls of the vault must be relatively thick (about 1.20 to 1.50 m), to be able to bear the rock pressures.
- It requires a fixed number of miners and carpenters who work simultaneously (about 6 times as many as NATM).
- The death rate is much higher. 89 died during the construction of the 10.25 km railway tunnel through the Arlberg, completed in 1884, but only 15 in the new, 13.90 km parallel motorway tunnel.

THE NEW ATM The basic concept of NATM was developed in the early 60ies by L. Müller, L.v. Rabcewicz, and F. Pacher. It is not a "method" in the old sense that it strictly defines the tunnelling work in relation to the cross section of the tunnel but a certain process which still allows the use of any chosen driving-on method, as follows:

- A section of the roof, or the full tunnel profile, is excavated, preferably in one process, and immediately afterwards the rock is treated to make it load-bearing, i.e. the sides of the just completed excavation are converted into a "natural" arched vault that prevents the displacement of rock towards the cavity by:
 - o stabilization with short anchors, i.e. rock bolts;
 - o mounting of steel lattices that prevent large, loose pieces of rock from falling down;
 - o anchoring of deeper-seated rock parts with long anchors (9 m, exceptionally up to 20 m).
 - o if necessary, reinforcement of the roof with sectional-steel arches;
 - o application of shot-concrete, representing the mostly relatively thin (0.40 to 0.15 m) lining of the vault.

All these methods may be used singly or in various combinations.

- After stabilizing the roof, the benches are excavated by steps and stabilized as above.
- The floor is excavated and the rock there also stabilized; depending upon the registered rock pressure, the floor is lined with regular concrete or shot-concrete. This must be done soon, at the latest at a distance of two tunnel



Application of Shot Concrete

- diameters behind the front.
- If strong rock movements are registered, a higher roof has to be excavated ("Überfirstung") and stabilization measures are carried out only after rock movements, which serve to reduce stress peaks, have subsided. In some parts it may be indicated to carry out the "Überfirstung" by stages, in tune with the deformation velocity and the progress of excavation work.
- It may also be necessary to fix the sectional-steel arches reinforcing the roof temporarily with horizontal girders, until the sides of the benches are stabilized.

The advantages of NATM lie in the following significant factors:

- The rock has only little opportunity to expand strongly towards the cavity, thus it hardly loses strength. Strong pressures are avoided and relatively thin wall linings will suffice, as they are not exposed to a strong bending moment. A slight and controlled deformation towards the cavity is even desirable as it reduces the rock pressure upon the vault.
- Danger for the crew is reduced to a minimum.
- The completely cleared tunnel tube allows the extensive use of machinery, thus greatly reducing costly manpower.

The NATM requires a well-trained management on the site. The engineer in charge and the foreman must have a special intuitive grasp of the prevailing rock conditions and must be able to make on their own responsibility very quick and flexible decisions as to the measures required to properly stabilize the walls of the cavity.

SUMMARY Today NATM is preferably used wherever soil or loose rock, tending to strong deformations and exerting strong pressures, is encountered in tunnelling work.

REFERENCES

- Müller, L., and Flecker, E. (1978). Grundgedanken und Grundsätze der "Neuen österreichischen Tunnelbauweise". Proc. Felsmechanik Kolloquium, Karlsruhe. Trans Tech Publ. Clausthal, pp.247-62.
- Rabcewicz, L.v. (1963). Bemessung von Hohlraum-bauten. Die "Neue österreichische Bauweise und ihr Einfluß auf Gebirgsdruckwirkungen und Dimensionierung". Rock Mechanics & Engineering Geology (I), 3-4, pp. 224-245, Springer Vienna.

G. Aas, Panelist

APPLICATION OF SLURRY TRENCH WALLS FOR TUNNEL PROJECT IN OSLO CLAY

PROJECT

This paper describes the construction method for a two-storey subway and railway tunnel which was built in a deposit of soft to medium stiff Oslo clay (Fig. 1) a few years ago. The floor level of the deeper tunnel is at 15 m depth, which is about twice as deep as it is possible to carry out an open strutted excavation with sufficient safety against bottom heave failure. The maximum depth to bedrock is about 40 m.

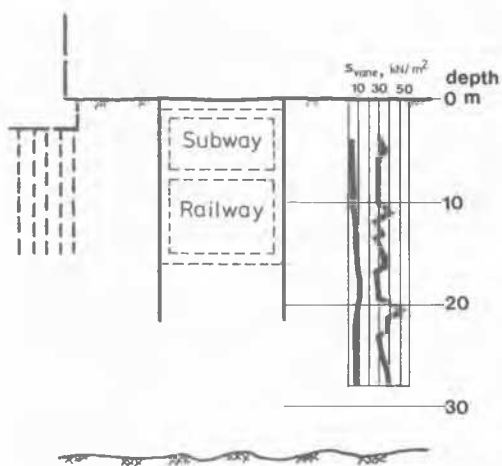


Fig. 1. Tunnel project and typical results from vane boring

The tunnel pass within a few m of the National Theatre and great care had to be taken to avoid uneven settlements on this old historical building.

A great number of construction methods were considered, including excavation under excess air pressure or in connection with freezing, and alternatives of tunnel drive. The method finally chosen was a variation of open strutted excavation. A special factor in this project was, that prior to any excavation cross-lot walls were established between the longitudinal walls below final excavation level. The purpose of these walls was both to prevent bottom heave failure and to brace the tunnel walls thus ensuring small lateral deformations when the tunnel was being excavated.

Fig. 2 shows the construction method, which was based on the use of 1 m wide slurry trench walls. The longitudinal walls were established first and extended down to a depth of 21 m. Thereafter, cross-lot slurry trenches extending to 20 m were dug at intervals of 4.5 m, one at the mid-point of each longitudinal panel. These transverse trenches were cast with concrete in the lower 5 metres below tunnel bottom. During further backfilling of the trenches, a

temporary brace was placed or cast, just below the middle deck in the tunnel.

The tunnel was supported on massive steel piles to rock installed through casings which had been mounted in the longitudinal walls prior to casting.

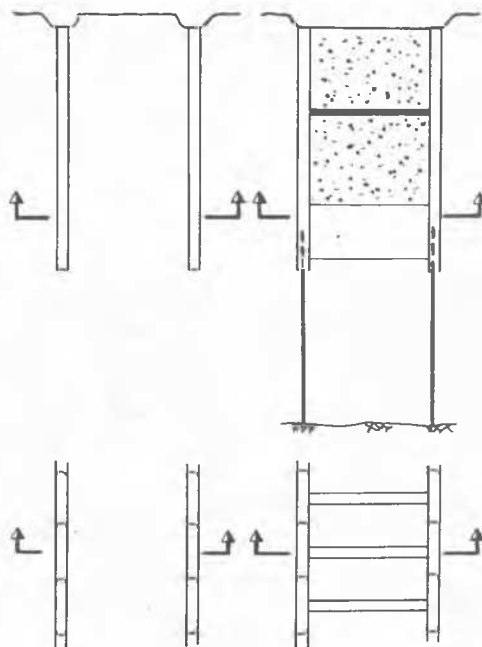


Fig. 2. Slurry trench wall constructions

The sequence of excavation operations is shown in Fig. 3. The tunnel roof was cast first and the excavation was then carried out under cover.

STABILITY AGAINST BOTTOM HEAVE FAILURE

The longitudinal and transverse walls form a stiff system of ribs below excavation level such that mobilisation of the shear strength between the clay and the concrete walls contribute to stabilize the excavation. Expressions for the factor of safety against bottom heave failure either through one single opening or for a long excavation are given in Fig. 4. In the latter case a complete failure would require local failures below each of the transverse walls. Thus, the upward pressure on the bottom of these walls represents an additional stabilizing factor.

Since the tunnel walls were founded on rock at the time when excavation took place, a shear stress between the clay and the outer wall could be mobilized as the terrain settled as a result of a tendency to bottom heave. This factor, which increases the safety factor, was not taken into consideration.

Stability calculations were based on vane strength which according to experience from former bottom heave failures (Bjerrum and Eide, 1966) gives good correlation for this type of clay.

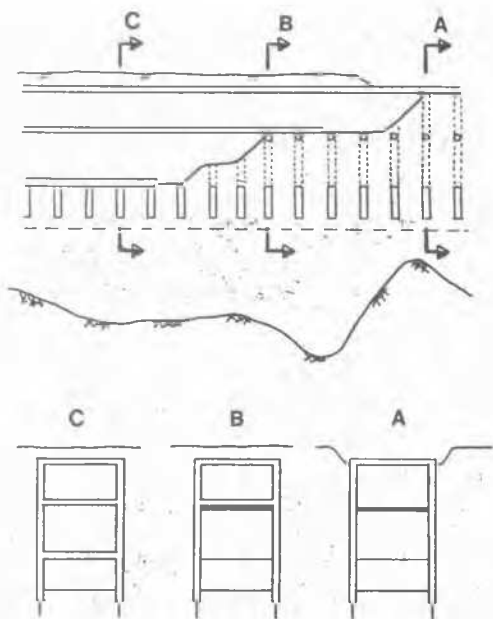


Fig. 3. Sequence of tunnel driveage

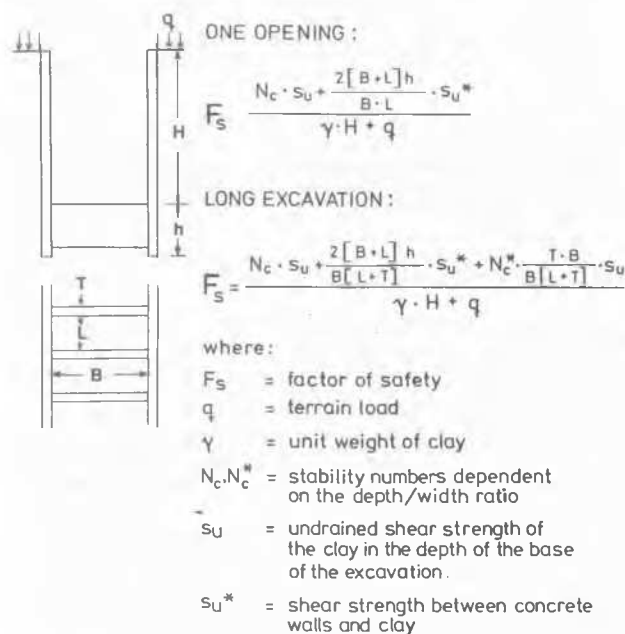


Fig. 4. Calculation of the stability against bottom heave failure

An extremely important question for this project was, however, how large a shear strength could be mobilized between the clay and the sunken concrete walls cast in one or other type of slurry. To clarify this point a comprehensive series of pull out tests on cast in-situ concrete piles had been performed. The piles of 0.25 m diameter and approximately 6 m length were cast in the depth range 16-22 m. Four different types of supporting fluid were used,

two piles being cast in each type. The four fluids were water, a slurry of local clay, a slurry of mikrosil (which is a silica dust from an industrial gas cleaning plant) and a bentonite slurry with added barytes. All slurries had a unit weight of 12.5 kN/m³. Each pile was drilled and cast within one day, and the pull tests completed in approximately 1.5 month. The pull tests were conducted by measuring the pulling force necessary to give constant rates of movements of 1.0 and 0.1 mm/min.

The results of these tests are given in Fig. 5. The conclusions to be drawn from these results were that the design value of shear strength along concrete walls cast in water, clay or mikrosil slurry should be about 90%, and in bentonite slurry about 67% of the vane boring value.

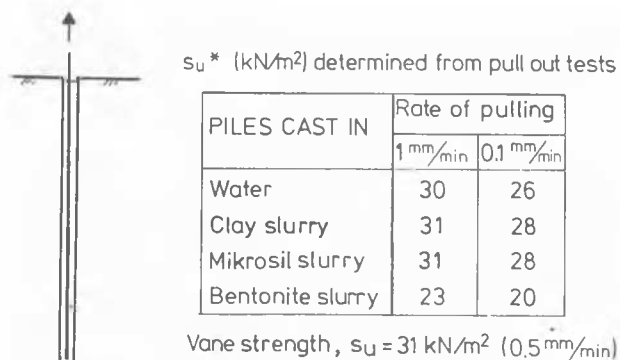


Fig. 5. Results from pulling tests on piles cast in-situ

Fig. 6 illustrates how the theoretical safety factor against bottom heave failure is affected by the presence of the cross-lot walls and the assumed value of wall adhesion. Assuming $s_u^* = 28$ kN/m² and a section length equal to about 5 m, which was actually employed, implies a safety factor of 1.33.

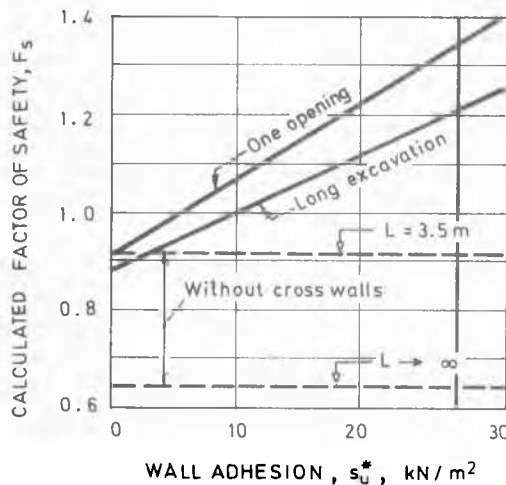


Fig. 6. Calculated factor of safety against bottom heave failure with and without system of cross-lot walls

STABILITY OF SLURRY TRENCHES

To the knowledge of the author, all slurry trench works prior to this project was carried out by means of bentonite mud. In friction soils, the primary purpose of the bentonite mud is to locally stabilize the trench walls and prevent loss of the supporting fluid in the trench. In the fairly homogeneous Oslo clay, however, it was anticipated that the main problem would be to ensure overall stability of the trenches. On the basis of stability calculations (Aas, 1976) and experience from a number of full-scale test trenches it was decided to use only water in the majority of the trenches.

The overall stability of the trenches did never cause any problems during construction of the longitudinal tunnel walls. Beforehand, however, one realized (Eide et al., 1972) that excavation of the cross-lot trenches represented a special stability problem. These problems were related to the fact that since a block of clay of only 3.5 m width was left in between neighbouring cross-lot walls (Fig. 7) and since only part of the trench had to be cast with concrete, backfilling of the remainder of a trench could adversely effect the next trench to be constructed. Originally it was therefore planned to use crushed rock as backfilling, and require that the crushed rock should be drained out completely after backfilling to ensure that its frictional resistance would be at least as high as the shear strength of the clay.

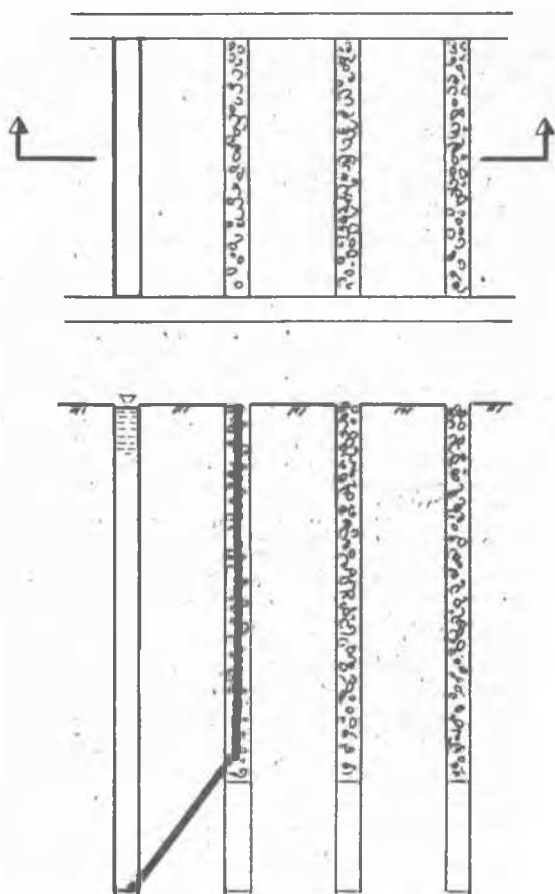


Fig. 7. Stability problems connected to the cross-lot trenches

As the work got under way, problems with fluid loss from a trench under excavation into the drained out crushed rock in the previous trench, occurred on three occasions and caused collapse of the trenches in question (Karlsrud, 1975). This method was therefore unsafe in practice. The problem was finally overcome by the use of a method where half of the trench was backfilled with low-grade concrete (Fig. 8). By first constructing alternately the southern and northern half of the trenches and backfilling with lean concrete, and then coming after with the sections to be backfilled with crushed rock, one ensured that next to a trench under excavation there would be either intact clay or a trench filled with lean concrete. It was not therefore necessary any longer to drain out the crushed rock.

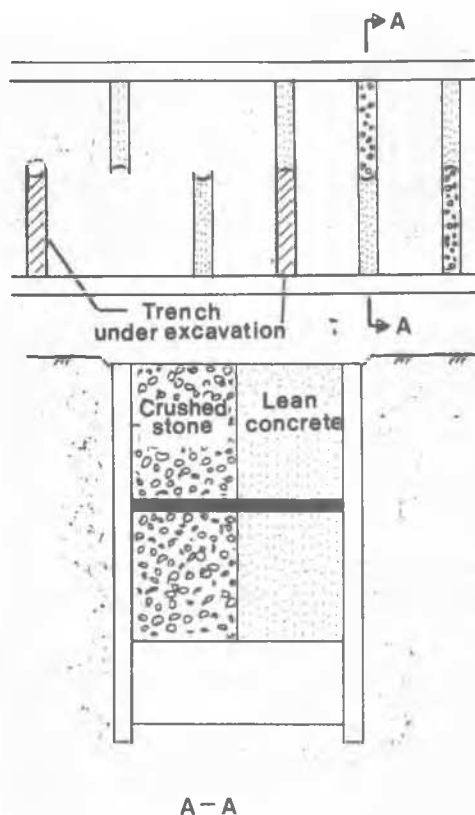


Fig. 8. Method of constructing the cross-lot slurry trench walls

PERFORMANCE OF TUNNEL STRUCTURES

Fig. 9 shows that the displacements after construction of the 21 m deep longitudinal tunnel walls were small, maximum 3-10 mm. Construction of the cross-lot walls caused surprisingly large displacement of the completed tunnel walls. The reason for these relatively large deformations was apparently that construction of the cross-lot walls caused a general reduction of lateral stress within the tunnel walls.

By the time the tunnel was completed, the surface settlement was maximum 70-80 mm and the horizontal displacement

about 40 mm. Lateral displacements of the tunnel walls at all levels indicated that one did not quite succeed in getting perfect contact between the cross-lot walls and the tunnel walls.

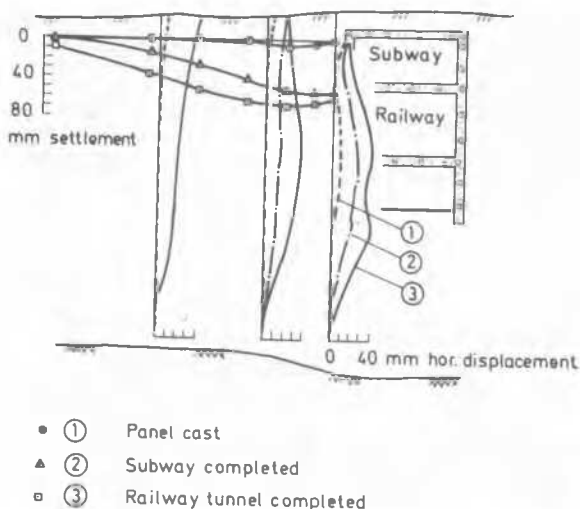


Fig. 9. Displacement patterns

However, maximum deformations amounting to less than 0.5% of the excavation depth are far below what might have been expected if supposing an alternative strutted sheet pile excavation. This is clear from Fig. 10 which gives observation data from a number of 10-12 m deep strutted excavations in Oslo (Aas, 1975). In each of these cases successive excavation and placing of a strut layer was done in limited sections, and in many of the cases the

EXPERIENCE FROM 10-12 m DEEP SHEETPILE EMBRACED EXCAVATIONS IN OSLO CLAY

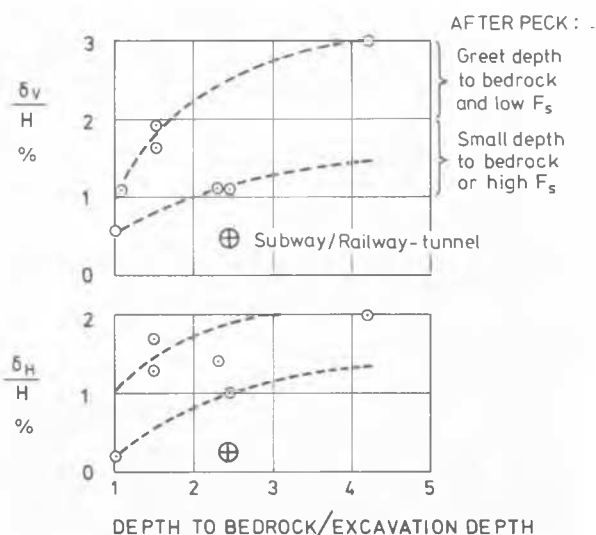


Fig. 10. Observed maximum values of settlement, δ_v , and horizontal displacement, δ_H , as a percentage of excavation depth, H

deeper part of the excavation was completed under cover in compressed air. Fig. 10 also indicates the range of maximum settlements to be expected according to world-wide experience from strutted excavations in soft clay (Peck, 1969).

CLOSING REMARKS

The special method applied on the reported tunnel project in Oslo, involving construction of cross-lot walls to prevent bottom heave and reduce lateral movements has functioned well. No stability problem arose in connection with the tunnel excavation, and ground settlements adjacent to the tunnel were quite harmless.

Contractor for the job was Kaare Backer A/S with Icos, Milan, as subcontractor for the slurry trench work. The firm Bonde & Co. A/S was the general technical consultant while NGI served as geotechnical consultants.

REFERENCES

- Aas, G. (1976) Deformasjoner i løsmasser som følge av tunnelutgravninger. (Movements in soil around tunnel excavations). Norsk Jord- og fjellteknisk forbund. Fjellsprengningsteknikk - bergmekanikk - geoteknikk. Oslo 1975. Foredrag. Trondheim, Tapir. Pp. 18.1 - 18.15.
- Aas, G. (1976) Stability of slurry trench excavations in soft clay. European Conference on Soil Mechanics and Foundation Engineering, 6. Wien 1976. Proceedings, Vol. 1.1, pp.103-110. Also publ. in: Norwegian Geotechnical Institute. Publication, 111.
- Bjerrum, L. and O.Eide (1956) Stability of strutted excavations in clay. Géotechnique, Vol. 6, No. 1, pp. 32-47. Also publ. in: Norwegian Geotechnical Institute. Publ., 19.
- Eide, O., G.Aas and T.Jøsang (1972) Special application of cast-in-place walls for tunnels in soft clay in Oslo. European Conference on Soil Mechanics and Foundation Engineering, 5. Madrid 1972. Proceedings, Vol. 1, pp. 485-498. Also publ. in: Norwegian Geotechnical Institute. Publication, 91, pp. 63-74.
- Karlsrud, K. (1975) Practical experience from the excavation of slurry trenches in Oslo clay. Norwegian Geotechnical Institute, Oslo. Publication, 110, pp. 39-47. First publ. in Norwegian in: Nordisk geotekniker-møde, Copenhagen 1975. Foredrag, Copenhagen, Polyteknisk forlag, pp. 529-542.
- Peck, R.B. (1969) Deep excavations and tunneling in soft ground; state-of-the-art report. International Conference on Soil Mechanics and Foundation Engineering, 7. Mexico 1969. State-of-the-art volume, pp. 225-290.

M.P. O'Reilly (Oral discussion):

Drs Ward and Pender are to be congratulated on their comprehensive general report on soft ground tunnelling. During the twelve years since Mexico there have been considerable advances and soft ground tunnelling is recognisably different now to that described then by Peck in his State of the Art Report.

I am very fortunate to have been intimately involved in a number of the major developments in soft ground tunnelling during this exciting period as leader of the research effort on tunnels at the Transport and Road Research Laboratory (TRRL), Department of Transport, UK. To begin, the research on tunnelling/ground interaction at the Engineering Department of the University of Cambridge by Atkinson, Mair and their colleagues under the direction of Professor Schofield, was initiated by TRRL in 1972 and has been actively sponsored there since then. That our early hopes have been fulfilled is amply demonstrated by the glowing tribute to this research work in the second paragraph of the Introduction to the General Report.

Secondly the bentonite shield which twelve years ago was just a concept is now in widespread use. In the UK the TRRL carried out the ground deformation measurements, during the experimental tunnelling using this technique, at New Cross (Boden and McCaul, 1974) and at Warrington (O'Reilly et al, 1980) on the first commercial application of the process in the UK. The bentonite shield is a significant addition to the tunneller's options in cohesionless ground or in ground with small percentages of clay.

The considerable tunnelling at Warrington, UK, much of it associated with the development of the New Town there,

provided the opportunity to compare the response of essentially the same ground to a variety of tunnelling methods - the bentonite shield, tunnelling in free and compressed air, and with chemical treatment of the ground. The results are summarised in Table I.

It must be mentioned too that despite the ground conditions at Warrington - loose sand with SPT values generally below 10 and relative compaction less than 90 per cent - and even when tunnelling below the water-table the ground losses and settlements were low by comparison with schemes elsewhere in the world. Those who suggested that the low settlements achieved on tunnel projects in the UK are confined to good tunnelling formations such as London Clay are well wide of the mark.

This brings me to my third point, the need for the promoter and designer to determine (i) the extent of the zone of ground affected by their tunnel, and (ii) the magnitude and distribution of ground movements within that zone. Their first concern will be to attempt to locate the tunnel so that its zone of influence does not impinge on buildings and services; in highly built-up areas this is often impossible and they will need to estimate the likely magnitude of settlements.

Accepting, as the General Report does, the normal distribution is a good representation of the settlement trough then the volume of soil lost at the ground surface, V , is given by the expression

$$V = \sqrt{2\pi} S_{\max} i$$

where S_{\max} is the maximum settlement on the centreline of the tunnel and i is the distance of the inflection point from the centreline (the standard deviation). From a knowledge of any two of V , S_{\max} and i the third can be determined.

TABLE I

Settlements resulting from various tunnelling methods at Warrington

Construction method	Ground conditions	Maximum surface settlement mm	Best estimate of i m	Area of trough - percentage of tunnel	Cover/diameter
Bentonite shield	Sand with some sandstone in invert	8 to 42	1.12 to 2.16	0.7 to 1.8	1.3 to 1.8
Hand excavation within shield in free air	Partially-stabilised sands and gravels	15 and 20	1.59 and 1.79	0.6 and 0.9	1.3
	Fully stabilised sands and gravels	7	2.28	0.4	1.3
	Natural sands and gravels	78	2.40	4.6	0.8
	Natural sandy clay soil	19	2.52	1.2	1.9
Hand excavation in shield in compressed air	Loose natural sand below water-table	28	3.20	7.1	3.7
Tunnelling machine	Mixed face - chemically treated sand over sandstone near invert	2 to 5	2.33 to 4.13	0.3 to 0.5	1.8 to 1.9
	Sandstone	1 to 2	3.60	0.3	2.0 to 2.1

* Excludes quite considerable settlements caused by insertion of injection tubes

An analysis currently in progress at TRRL of the data collected in the UK over the past decade at some 20 sites and covering a wide range of soil and stability conditions shows that for clay soils i is highly correlated with the depth to tunnel axis, z ; the relation is less well-defined for cohesionless soils but is nevertheless of considerable practical value.

However, determination of either V or S_{max} is much more difficult and theoretical methods are as yet not sufficiently developed to solve the problem. At the present time some form of load factor approach determining the ground loss, V , - see Fig. 8 of Mair et al and Fig. 10 of Kimura and Mair to this Session - would appear to offer the best way forward.

At this stage let me make a plea to those undertaking ground movement studies around tunnels. Much data collected in the past has often been less than comprehensive. This was particularly borne in upon me when comparing the lost ground at Warrington using the bentonite shield with similar schemes elsewhere. The minimum requirements for analysis are:

1. Complete definition of the settlement trough.
2. Details of the size of tunnel and its depth.
3. Information on ground conditions, eg undrained strength or SPT values as appropriate.
4. Details of the method of construction.

In many urban situations the alternative to tunnelling is deep trenching and here the engineer needs to know the disturbance caused by both methods of construction. The trenching side of TRRL's programme of research is again being pressed ahead on two fronts through full-scale measurement in the field by the Laboratory itself (Symons, 1980 and Symons, Chard and Carder, 1981) and with concurrent centrifuge model studies at University of Cambridge. The indications are that disturbance due to trenching is of the order of 3-4 times greater than that caused by tunnelling.

Finally the development of loads in tunnel linings is of considerable interest and I must endorse the assertion in Conclusion 6 of the General Report that they 'are not always continued long enough'. The measurements at Regents Park, begun in 1974 (Barratt and Tyler, 1976), are continuing and Fig. 1 shows the build-up of load on the lining to April this year; the load has increased to 56 per cent of overburden after seven years. Measurements over 18 years of hoop loads in the Thames-Lee Tunnel (Cooley, 1981) showed that by that time loads had built-up to 55-89 per cent of overburden; there was evidence that conditions had not altered appreciably in the final 9 years in three of the four instrumented rings.

Orr in model tests at Cambridge (Orr, 1976) showed that the load on the lining in an over-consolidated clay was 68 per cent of overburden and this was attributed to the compressibility and/or lack of fit of the lining. He suggested by reference to Bishop (1966) that creep effects might be low when the stresses in the ground were a small proportion of its strength; pore-water changes would then be the controlling factor. It will be of considerable interest to observers how the loads build-up in the tunnel lining of the recently constructed Oxford Relief Sewer as some of the measurements are being made in a length of tunnel where considerable convergence of the ground occurred on excavation.

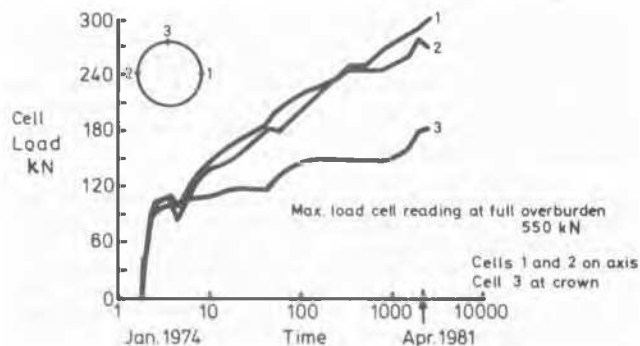


Fig. 1. Development of loads on tunnel lining at Regent Park

REFERENCES

- Barratt, D.A. and Tyler, R.G. (1976). Measurements of ground movement and lining behaviour on the London Underground at Regents Park. Department of the Environment TRRL Report LR 684. Crowthorne, (Transport and Road Research Laboratory).
- Boden, J.B. and McCaul, C. (1974). Measurements of ground movements during a bentonite tunnelling experiment. Department of the Environment TRRL Report LR 653. Crowthorne (Transport and Road Research Laboratory).
- Bishop, A.W. (1966). The strength of soils as engineering materials. 6th Rankine Lecture, Geotechnique 16, 91-128.
- Cooley, P. (1981). 86th Summer General Meeting and Conference. (Institution of Water Engineers and Scientists).
- O'Reilly, M.P., Ryley, M.D., Barratt, D.A. and Johnson, P.E. (1981). Comparison of settlements resulting from three methods of tunnelling in loose cohesionless soil. Proc 2nd Int. Conf. 'Ground Movements and Structures', Cardiff, 1980. (Pentech Press, London).
- Orr, T.L.L. (1976). The behaviour of lined and unlined tunnels in stiff clay. PhD Thesis, University of Cambridge.
- Symons, I.F. (1980). Ground movements and their influence on shallow buried pipes. The Public Health Engineer, October.
- Symons, I.F., Chard B. and Carder D.R. (1981). Ground movements caused by deep trench construction. Paper presented to Sewerage 81, London. (Institution of Civil Engineers).

Apparent stability in fine soils can create unjustified confidence and on removing temporary support after stoppages collapse due to re-established positive pore pressure may occur. The problem is acute if soil variations of a significant scale relative to the face dimensions are encountered unexpectedly.

This was envisaged when planning coal mine drifts at Huntly, N.Z. (1975) and at Selby, Gascoigne Wood, U.K. (1976). In each case the drifts were about 5.2 m diameter shield driven at 1:4 grade through water logged soils respectively 35 m and 21 m thick over rock.

At Selby the drifts ran through firm laminated silty clays saturated almost to surface, overlying Permian marls. Near rock intersection a lens of micaceous silty sand was found which could be significant for face stability.

Vacuum points were to be spaced not less than 1 m apart penetrating about 1.5 m into the soil. High vacuum for maximum effect was obtained from ejectors since volume of flow would be low (Figure 2).

In the event no serious stoppages of shield tunnelling occurred in the soils and the systems were not used.

This was a particular application of vacuum methods previously proven in tunnels through more permeable sands in glacial till at Elm Park Drift, Medomsley Colliery, U.K. (1962) and through fine hydraulic fill of sea sand at Cape Town, Table Bay Container Berth (1976). In these cases a single ejector point was used continuously to stabilise the face against adverse hydraulic gradients. Negative pressure is quickly established in the coarser material up to permeability 10^{-2} cm/sec.

The method is very simple and cheap to operate. The miners like it since they are able to adjust the apparent hardness of the soil to suit themselves by varying the position of the vacuum point marginally.

G. Pugliese (Written discussion)

UNDER-GROUND TUNNEL WITH SUPER-IMPOSED TRACKS Tunnel Metropolitain à Deux Voies Superposées

INTRODUCTION

The scope of this note is to merely illustrate how one section of the Milan under-ground second line was built, running under existing buildings, adopting an unusual tunnel section.

DESCRIPTION OF THE WORKS

The tunnel in question is of two tracks, one above the other, and passes under a 9 story, 30 m high building which has well conserved reinforced concrete foundations.

The choice of the super-imposed track section was made to minimise the area of tunnel under private properties. During the construction of the tunnel the building above was not evacuated, nor was commercial activity upset; only the basement was cleared out to make room for equipment to monitor any movement of the building that might occur.

Of alluvial origin, the sub-soil consists of sand and gravel with about 30% air spaces and a grain structure of $D_{10} = 0.7$ mm and $D_{60} = 50$ mm. The distance between the plinths of the building foundation and the excavation was about 3 metres.

The work was conducted in the following sequence of phases:

- consolidation of a first zone of soil by injection from the surface;
- excavation of a small service tunnel along the line of the upper tunnel;
- completion of soil consolidation under the building by injection from the service tunnel;
- excavation of the upper tunnel in half-metre sections, with immediate lining with steel latticed centering and shot concrete;
- final lining of the upper tunnel including an intermediate floor in reinforced concrete cast on site;
- excavation of the lower tunnel, again in half-metre sections immediately lined as above;
- final lining of the lower tunnel in 6 m stages as the work progressed.

During the building of the upper tunnel the water table level rose unexpectedly, reaching a level of two metres above the base of the lower tunnel. It was thus necessary to create an impermeable layer on the base of the tunnel.

The reinforcement of the soil was achieved injecting first a mixture of:

- cement 46% - filler 50% - bentonite 4%
- diluted in 1.5 parts of water and injecting some 80 litres per

cubic metre; there followed a mixture of:

- sodium silicate 30-40 Be 46.5 litres
 - durcisseur B, or C 7 litres - water 46.5 litres
- up to a total absorption of an average of 200 litres per cubic metre of reinforced soil.

The equipment for monitoring building movement consisted of nine long base extensimeters positioned in correspondence to the main pillars, and connected to an electrical alarm system. The instruments, with a resolution of 0.01 mm, recorded a maximum of 3 to 4 mm of movement during the works. The design and site management were undertaken by the Metropolitana Milanese S.p.A., while the work was executed by the company SOCOMET of Milan.

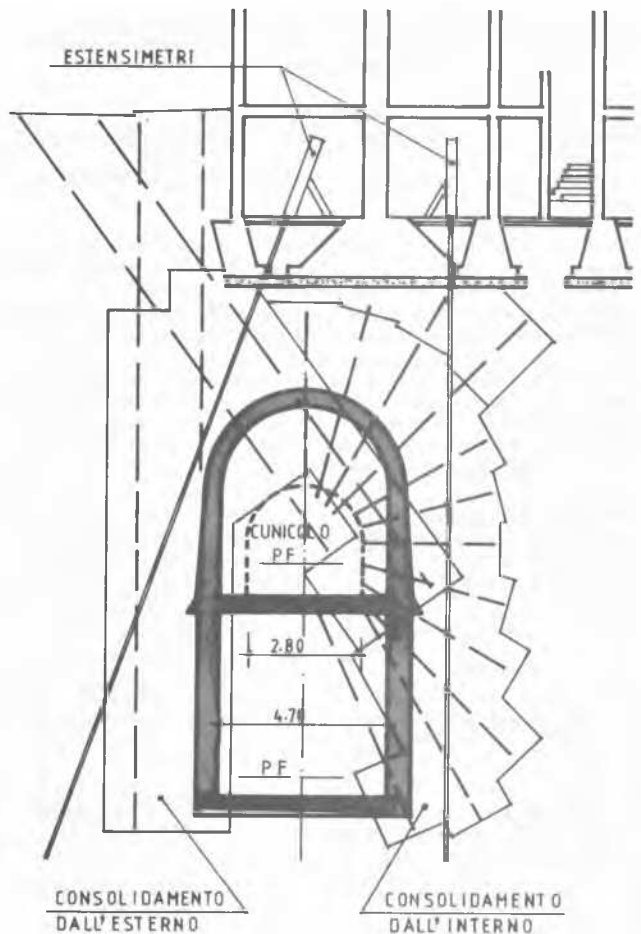


Fig. 1

J. Huder (Written discussion)

MILCHBUCK TUNNEL

A large number of contributions have been received under the theme "Saving old cities". These papers deal with a range of topics, including the well-known Tower of Pisa and other famous structures of a monumental character, like cathedrals. Work on these structures demands, amongst other things, the greatest intuition; although the materials used are to some extent

known, and possibly the cracks developed are visible, the internal stresses are largely unknown. Above all it is not known how much additional deformation such structural systems can tolerate before they become unstable. Here, however, I would like to deal with another aspect of preserving old buildings in our cities, that is, the construction of traffic systems in

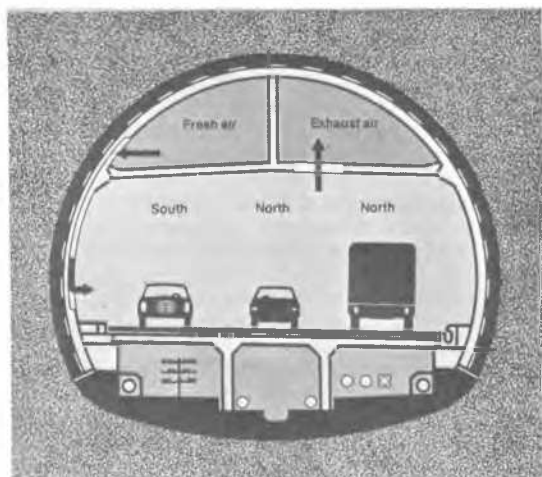


Fig. 1 Standard Cross Section

very dense urban areas near or beneath such building monuments. The engineer of today must successfully handle these problems if in the future living in the old towns and cities is to be made attractive.

In particular, a short report is given hereafter of the construction of a 3-lane highway tunnel whose roof is only 5.5 m below surface structures, roads, tramlines, sewer systems, etc. The engineers in this project were confronted with new, interesting problems. The material that had to be tunnelled through consists of moraine with zones of sand and silt with, in addition, the added complication of artesian water. After traversing a stretch of about 350 m of soil the tunnel enters rock. The project described here is the Milchbuck tunnel in the city of Zurich (see Fig. 1).

The most diverse constructional methods were investigated and their advantages and disadvantages compared. It turned out to be very difficult to predict the settlement due to excavation using the individual methods. The estimates varied from centimetres to decimetres with the same method, so that in the end it was agreed to accept a proposal of the contractor to carry out the constructional work for this tunnel with the aid of the ground freezing technique. At first, however, those concerned were not terribly happy about this proposal, as in Switzerland a tunnel of 14.5 m diameter had never been excavated in the immediate vicinity of the foundations of buildings. Thus extensive studies were undertaken, until it was finally decided to adopt this proposal. At this point in time this stretch of excavation has been completed and one can speak of a great success, in so far as no damage complaints have been made and the work progressed more or less according to plan (Fig. 2 and Fig. 3).

Various articles have been published which give information about the works carried out. [1] [2]

Today we are in a position to evaluate and

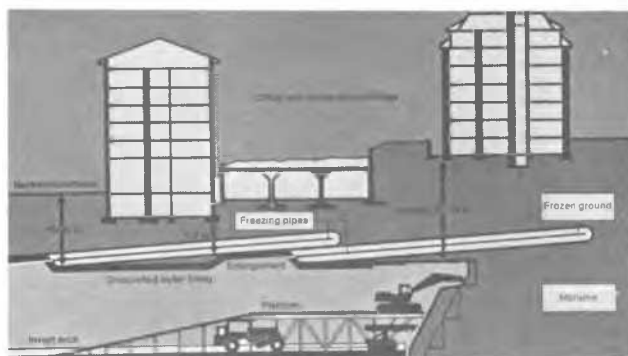


Fig. 2 Longitudinal Section for Freezing Sections 3 and 4 (see fig. 4)

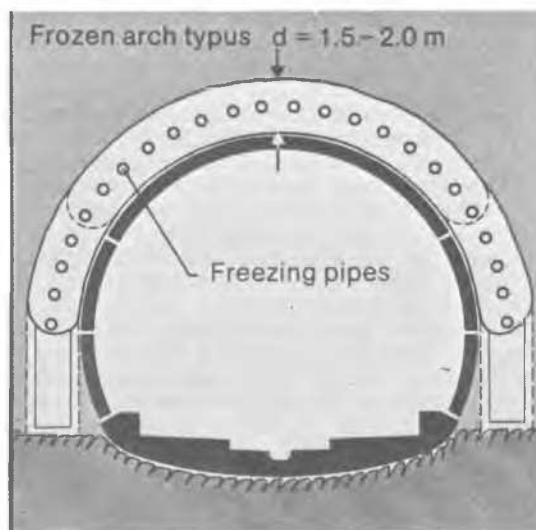


Fig. 3 Frozen Arch with Freezing Pipes

discuss the results of the measuring programme, which included surface settlement checks at about 200 points, inclinometers and extensometers in boreholes and in the tunnel and temperature longitudinally and radially. (Note, an extensive programme would have been necessary in any other method; additional is mainly temperature measurement). Here, however, many aspects can only be briefly touched upon and, above all, those questions will be considered, which arose before beginning the frozen ground section and which can be profitably discussed from the experiences gained.

At first it was doubted whether the horizontal boreholes to take the freezing tubes could be drilled with sufficient accuracy, but this was satisfactorily accomplished, albeit with the use of an exact template and a drilling carriage. Thereby, the specified tolerances of $\pm 1\%$ were actually improved on, with a maximum deviation of $\pm 0.8\%$. The control measurements were executed with a newly developed apparatus (Extensodectometer ISETH) which has an accuracy of $\pm 0.05\%$ or ± 2 cm in 40 m. [3]

The doubts about the heat of hydration of the shotcrete affecting the thickness of the frozen ring of soil to the extent that a loss of strength would result were shown on the basis of field measurements to be unfounded. This conclusion depends, to be sure, on the quality of the material in the frozen state and cannot be generalized. Likewise, the results presented here are particular to this case. To generalize much more experience is required especially in respect to different materials.

The ground water pressure under conditions of artesian head was relieved using a large number of wells drilled from the surface. The resulting loading of the subsoil caused an extended settlement depression with a maximum value of 2.8 cm and an average value of about 1.0 cm.

The deformations, in particular the heave of the building due to the freezing process could be predicted on the basis of tests and calculations. However, one unknown could not be predicted accurately, i.e. the rate of excavation which is important for the freezing period. The fact that due to the first-time nature of the work a certain time was needed before the working routine was established had the disadvantage that the frozen soil body continues to grow during the maintenance period and thus the heave keeps increasing. This may be seen from Fig. 4. Likewise, along the axis of the tunnel the "Marccianti"-like excavation of the individual freezing sections is shown, whereby the effect of small overburden here is evident. For the first section 101 days were required and in this period heaves of up to 105 mm were registered and surface settlements of 45 mm were measured after thawing. By means of suitable measures the heave could be largely reduced, whereas the settlements - without remedial measures - remained in the same order of magnitude.

The rate of expansion and thickness of the frozen soil body could be checked using the installed thermometers and they corresponded well with the predictions. In order to avoid damage, the largest heaves of the first section had to be reduced. The rate of excavation was speeded up and also ways and means of preventing the formation of ice lenses were considered. Ice lenses develop at conditions of thermal equilibrium on

the 0°C isothermal front. If this condition can be disturbed there is some hope of restricting the ice lenses forming. This was achieved in the following way: instead of using a cooling liquid of reduced temperature (-20°C) to maintain the prescribed temperature in the frozen body (after this state had been achieved using a cooling liquid at -40°C) an alternating cooling cycle has been used, in which the frozen section is subjected to a cooling period of 24 hours at -35°C with equal intermediate periods without cooling. The amount of cooling liquid remained the same, but the desired effect was clearly observed. The reduction in heave though is not only due to this intermittent cooling method and the increased rate of advance, but is also a result of the increasing overburden pressure. The reduction in the rate of advance in the last few sections was caused by the excavation of the rock at the bottom of the tunnel.

The development of deformations is illustrated for different points in a cross-section of the first working section. The location of the monitored points is given in Fig. 5a. Here the maximum values of heave and settlement are shown for this section. Noticeable is the small influence of the freezing on heave in the lateral direction. The settlements exhibit a striking depression-form.

Heaving starts with the beginning of the freezing phase (-40°) and is different from the subsequent heaving in the maintenance phase with constant temperature of -20°. (refer to Fig. 5b). As soon as the excavation has reached the desired profile, settlements start to develop (as can be seen in Fig. 5b) i.e. the frozen arch has to carry the overburden load, and a redistribution of stresses takes place. After this redistribution even a new increase in heave can be observed. This increase is in this case also due to the deeper cooling tem-

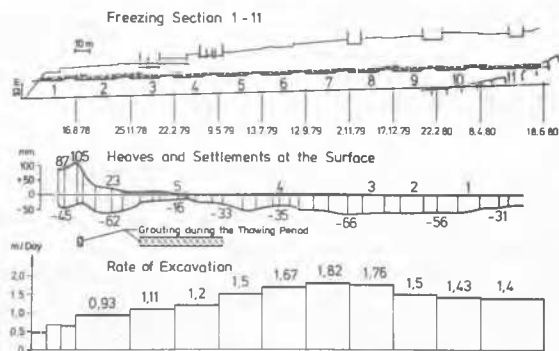


Fig. 4 Milchbuck-Tunnel, Moraine Stretch

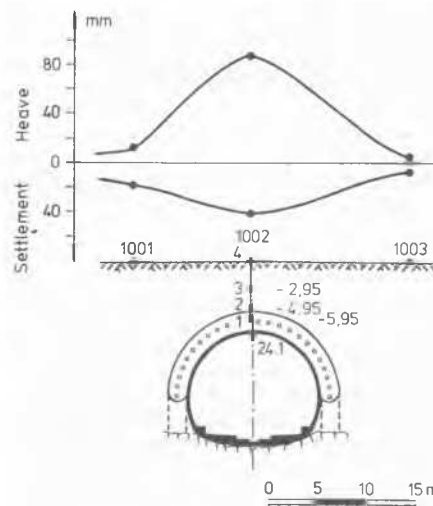


Fig. 5a Heave-Settlement Distribution Section 1

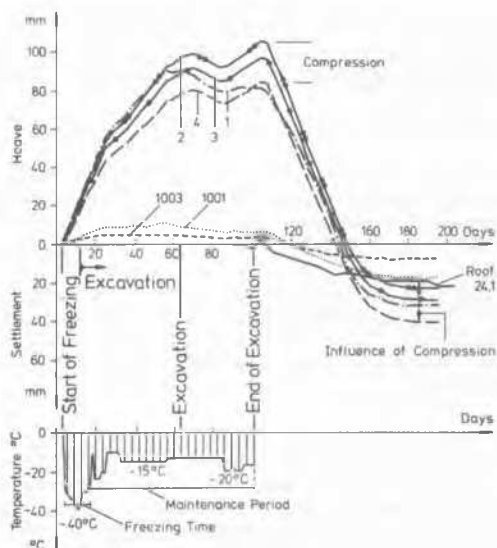


Fig. 5b Heave-Settlement Distribution Section 1

perature (see temperature curve within maintenance period). Settlements start to develop only after cooling has stopped and thawing has started.

It is noteworthy that the outer concrete lining, which has been constructed as reinforced shotcrete, begins to carry load only in this phase. The creep-deformation of the frozen soil was very small, as was to be expected from the laboratory tests. The settlements result from the thawing of the soil and from the deformation due to the stress transfer from the frozen soil arch to the concrete tunnel lining. The thawing of the soil is associated with a loosening effect, which gives a greater flexibility to the subsoil, especially in the vicinity of the arch abutments. Thereby additional settlements occur. These settlements were underestimated. In sections in which the amount of settlement appeared to be too large they were hindered by means of grouting (see Fig. 4).

In section 1 heave begins at the same time as the freezing process is started. The surface point 4 exhibits smaller heave than the points nearer to the frozen body. For these small overburden depths and a relatively loose material there is a compression during the freezing process which is also connected with the formation of ice lenses. After about 55 days the point 1 of the extensometer is fixed in the frozen body and reacts with the deformation of the same. After 65 days the excavation has reached the measuring section. The influence of the redistribution of stresses in and around the frozen body may be clearly seen. In how far these settlements were connected with creep movements was not possible to say then. By means of the low cooling temperatures, however, it was possible to stop these settlements; indeed the ground heaved once more for up to 101 days, at which time the cooling system was taken out of operation.

Immediately after introducing the shotcrete lining the measurement of the arch deformations was started. The settlement of the roof point begins first when the thawing process begins. Apart from point 2, whose measured value is affected by the freezing process, the points 3, 4 and 1002 attain the same final settlement at the roof point of the lining. The heaves and settlements at points outside the tunnel axis (points 1001 and 1003) are shown in the section (see Fig. 5a for the values of maximum heave and settlement respectively). The results for section 5 (see Fig. 6) can be seen to be quite different from those of section 1. The points are shown in the cross-section (Fig. 6a) with the max. values of heave and settlement. The time variation of heave and settlement is given in Fig. 6b. The initial settlement of about 5 mm (extensometer reading) results from the excavation for section 4. The distance from the face of section 4 to the measuring section is about 6 m.

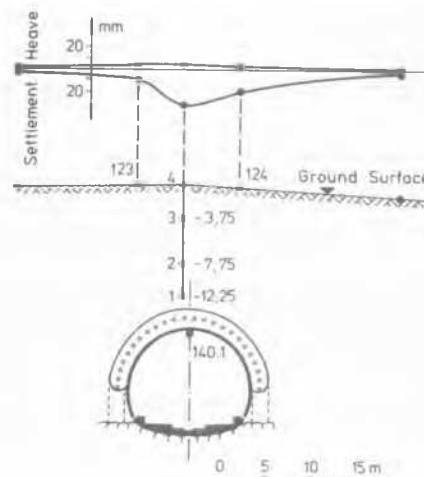


Fig. 6a Heave-Settlement Distribution Section 5 Cross-Section

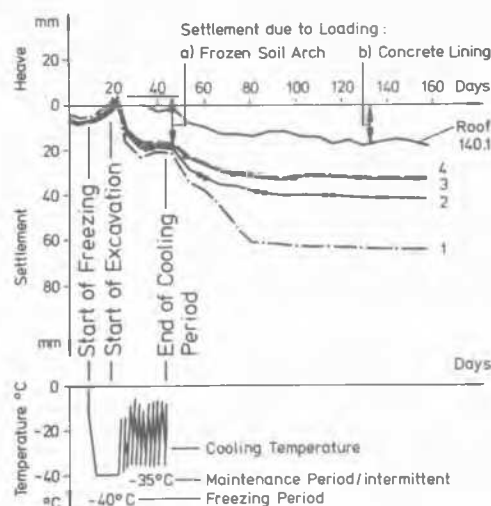


Fig. 6b Heave-Settlement Distribution Section 5

FINAL COMMENTS

The heave and settlement of buildings and the ground surface could be kept to a minimum using the ground freezing technique. By means of control measurements the deformations that were produced could be restricted with the help of remedial measures and repair work could be avoided. The experience gained in working section 1 was used to advantage in reducing the heave in subsequent sections. The increasing overburden pressure also had a beneficial effect. Besides limiting the freezing time and the specially adopted intermittent freezing technique the settlement could, when necessary, be reduced using cement-clay grouting. If the settlements of about 1.0 cm (max. 2.8 cm) due to ground water lowering carried out before construction was started are added to those for the excavation the gradient of the settlement trough in the critical sections amounts to a maximum of 1/250, but an average of 1/300. An exception is section 1.

The maximal angular distortion occurs in the cross-sections. Damage was not recorded. Only an expansion joint on a new building, which had not been properly constructed, gave rise to a damage claim.

REFERENCES

- [1] Publication No. 100, Milchbuckttunnel, 1979. Swiss Society of Soil and Rock Mechanics. Aerni, K., Mettler, K., Herzog, P., Ramholt, T., Huder, J.
- [2] Aerni, K., Mettler, K., 1980: Ground Freezing for the Construction of the Three-lane Milchbuck-Tunnel in Zurich, Switzerland. The 2nd Int. Symp. on Ground Freezing, Trondheim.
- [3] Kovari, K., Amstad, Ch., Köppl, J., 1979: Neue Entwicklungen in der Instrumentierung von Untertagebauten und anderen geotechnischen Konstruktionen. Schweiz. Ingenieur und Architekt, Heft 41, 1979.