The Umbrella Method in Tunnelling

by

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Abstract

When tunnelling through poor ground conditions or if there is shallow ground cover above the tunnel crown, the rock or soil mass needs to be reinforced ahead of the face. The umbrella method is a ground reinforcement technique in which all or part of the support of a tunnel section is placed before beginning excavation. The aim of the umbrella method is to form an arch-like shell of grouted and/or reinforced rock and soil mass around the tunnel. A number of articles and reports argue that the umbrella method has advantages in certain conditions; for example, where the tunnel has unusual dimensions, where the overburden is shallow, or where ground deformation needs to be restricted. However, there is little recorded confirmation of the method's effectiveness. There seem to be two reasons for this lack of information. One is its newness–the method has only recently been developed. The second reason is locality–the method has been applied mainly in two places, Italy and Japan.

In this thesis, the umbrella method is subdivided into three categories mainly according to the material used and/or ways of reinforcing the ground, that is, the sub-horizontal jet-grouting method, the injected steel pipe umbrella method and the pipe roof method. Then, 24 cases where the umbrella method was employed are reviewed.

The case studies lead to several conclusions: 1) the umbrella method is effective in preventing slope failures and landslides, in restricting ground surface settlement and increasing face stability; 2) ground type seems to be an important factor in the selection of the type of umbrella method used; and 3) reduction of the dimensions of tunnel supports is possible by using the umbrella method.

Thesis supervisor : Herbert H. Einstein
Title : Professor of Civil and Environmental Engineering

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Chapter 1. Introduction

1.1 Background

In urban areas, in general, poor ground prevails and because of the traffic and/or planning requirements tunnels tend to be laid at a shallow depth. On the other hand, tunnels have to be constructed in ways that cause the least amount of detrimental damage to surface structures, i.e., to minimize ground deformation as much as possible. Also in non-urban areas, tunnels are sometimes excavated in difficult ground in which it is difficult to maintain stability and limit deformation of the excavated opening. Hence, tunnel engineers have to be able to handle various problems in both urban and non-urban areas.

In the case of low ground strength and/or the need to restrict ground deformation, either shield tunnelling is used or the face is divided into small sections. However, when the latter construction procedure is implemented large construction machinery cannot be used, and as a result, the relative importance of the manual labor increases. Taking into account the present circumstances in tunnel construction, i.e., a shortage of manpower and the increase in mechanization, saving labor in tunnelling is necessary. Hence, a construction method that reinforces the surrounding ground, and then allows large-sized machinery to excavate the tunnel rapidly is particularly appealing. Such action is also necessary to ensure the safety of the construction work.

The umbrella method (see Fig. 1-1) is a ground reinforcement technique in which all or part of the support of a tunnel section is placed ahead of the cutting face before excavation is begun. So far it has been mainly employed in Italy and Japan.

The aim of this method is to preserve the face and perimeter of the excavation from disturbances as much as possible. So far three distinctively different categories of the umbrella method exist:

- sub-horizontal jet-grouting method
- injected steel pipe umbrella method
- pipe roof method

There are substantial differences among these methods in terms of the execution procedure, equipment, and reinforcing materials used. However, the basic concept and design philosophy are the same.

A number of articles and reports have been written stating that the umbrella method has advantages in certain conditions; for example, where the tunnel has unusual dimensions, where the overburden is shallow, or where ground deformation needs to be restricted. However, few of these articles confirm the effectiveness of the umbrella method by surveying cases and analyzing them systematically.

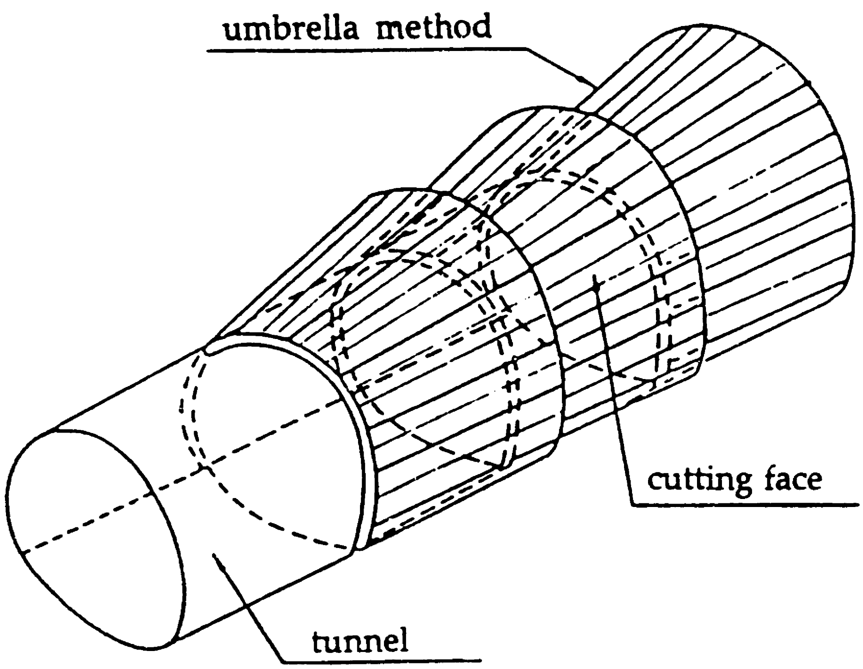


Fig. 1-1 Schematic of the umbrella method (Geo-Fronte Research Association, 1994)

1.2 Objectives of Research

The objectives in this thesis are:

- to present the principles of the umbrella method
- to introduce case studies and to create a database of the umbrella method
- to assess the effectiveness of the umbrella method

As mentioned previously, the umbrella method is a new tunnelling technique. In addition, the method is based on state of the art drilling and grouting methods. For this reason, presenting the principles of the umbrella method in connection with the drilling and grouting methods is vital to this thesis.

Because of its newness and the fact that most of the applications have been carried out either in Italy or Japan, some applications of the umbrella method are not widely known in the field of tunnel engineering. Consequently, the umbrella method is not well understood by many tunnel engineers. Therefore, case studies on the umbrella method will help in the understanding of the method and in deciding whether or not the umbrella method is feasible for particular tunnel construction. Creating a database for the umbrella method according to geological conditions, tunnel dimensions, tunnel supports, ground deformation, etc., is also important for detailed practical work. In order to understand the adaptability and effectiveness of the umbrella method it is necessary to know certain information such as the ground type in the zone where the umbrella method was employed and ground deformation induced by the tunnel excavation.

Finally, from a research point of view and on the basis of the data collected from actual construction cases, the umbrella method can be assessed in terms of the adaptability to ground types and its effectiveness in solving tunnel construction problems, especially, the limitation of ground deformation.

1.3 Approaches

Chapter 2 begins with special problems in tunnelling, before going on to deal with the umbrella method. The problems described in this chapter are problems which are usually encountered in tunnel construction where ground conditions are poor and where there is a shallow overburden. As stated previously, the umbrella method is usually employed under these circumstances. The arching theory by Terzaghi and plasticity theory are also introduced.

In Chapter 3, the umbrella method is subdivided into three categories mainly according to material used and/or ways of reinforcing the ground: the sub-horizontal jet-grouting method; the injected steel pipe umbrella method; and the pipe roof method. Then, each of these umbrella methods is described in detail. Since these three umbrella methods are closely associated with drilling and grouting methods, they are also described.

In Chapter 4, 24 case studies in which the umbrella method was employed are presented. Each case is discussed in relation to:

- environment
- geological and hydrological conditions
- problems in tunnel construction
- supplementary (in addition to the umbrella) support method of the tunnel
- structural details
- construction procedures
- field measurements

• numerical analysis (if available)

Finally, in Chapter 5, discussions and conclusions are presented. The data obtained from all cases are statistically analyzed. In particular, the reasons for adopting the umbrella method, appropriate ground types for the umbrella method and characteristics of ground deformation are discussed. Finally, information obtained in this research is summarized and future studies are suggested.

Chapter 2. Special Problems in Tunnelling

2.1 Overview

The umbrella method is generally employed under the following conditions:

- the existence of a shallow overburden above a tunnel
- the need to restrict ground surface settlement
- poor ground conditions

Of these conditions, the shallow overburden over the tunnel and poor ground conditions are discussed in this chapter.

The discussion includes brief descriptions of the arching and the plasticity theories. They are not directly related to the umbrella method, but they are helpful in understanding the conditions where the umbrella method is recommended and / or needed.

2.2 Arching Theory by Terzaghi

In general, the greater the arching effect mobilized near the perimeter of a tunnel excavation the more stable the tunnel will be. The arching effect is associated with the overburden above the tunnel. Thus understanding the relation between the arching effect and overburden is helpful in comprehending the reason why the umbrella method is usually employed under a shallow overburden.

Terzaghi (1943) defined the arching effect as follows.

If one part of the support of a mass of soil yields while the remainder stays in place the soil adjoining the yielding part moves out of its original portion between adjacent stationary masses of soil. The relative movement within the soil is opposed by a shearing resistance within the zone of contact between the yielding and the stationary masses. Since the shearing resistance tends to keep the yielding mass in its original position, it reduces the pressure on the yielding part of the support and increases the pressure on the adjoining stationary. This transfer of pressure from a yielding mass of soil onto adjoining stationary parts is commonly called the arching effect, and the soil is said to arch over the yielding part of the support.

Figure 2-1 shows arching induced by the tunnel excavation.

From his experiments using a deflecting trapdoor in the base of a soil bin, Terzaghi (1943) suggested the following expression for the earth pressure p transferred to the tunnel support, as shown in Fig. 2-2.

$$p_v = \frac{B\gamma}{2K\tan\phi}\left\{1 - e^{-K\tan\phi\frac{2H}{B}}\right\} + q\, e^{-K\tan\phi\frac{2H}{B}} \tag{2. 1}$$

$$B = 2\left|\frac{b}{2} + m\tan\left(45° - \phi / 2\right)\right| \tag{2. 2}$$

where

p_v = earth pressure
K = ratio of horizontal and vertical pressures
b = width of the tunnel
m = height of the tunnel
H = depth of overburden
γ = unit weight of the soil
q = surcharge on the surface
ϕ = internal friction angle

The results of one of Terzaghi's experiments are given in Fig. 2-3 (Bulson, 1985). The width is b, the depth of overburden is H and the vertical soil stress at a horizontal section at any depth below the surface is σ_v. The vertical stress if there were no arching, i.e., the geostatic stress, would be the σ_{vh}. It can be seen that for z/b > 2.5 there is no relief of vertical stress due to arching, but that immediately over the trapdoor, σ_v is less than 10% of σ_{vh}. If a tunnel is located at a great depth below the surface, the arching effect does not extend to the ground surface.

Next, let us consider the height of the arching zone.

Figure 2-4 shows the simplified arching zone with a height of h_o and width of B. If the unit weight of the soil is γ, pressure acting on the A-A line is $p_v = \gamma h_o$, where h_o is a height of the simplified arching zone. Assuming that this pressure is equal to the p_v expressed by the equation **(2.1)**, the following relation can be established:

$$\gamma h_o = \frac{B}{2K\tan\phi}\left\{1 - e^{-k\tan\phi\frac{2H}{B}}\right\}, \tag{2. 3}$$

where, for simplicity, surcharge, q, is assumed to be zero.

Dividing equation **(2.3)** by the depth of overburden, H, we can obtain the normalized height of the simplified arching zone as follows:

$$\frac{h_0}{H} = \frac{B}{2K\tan\phi\, H}\left\{1 - e^{-K\tan\phi \frac{2H}{B}}\right\}. \qquad \textbf{(2. 4)}$$

Let us consider a circular opening of diameter D, as shown in Fig. 2-5.

With $B = D\cot\{(\pi/4 + \phi/2)/2\}$ as shown in Fig. 2-5, equation **(2. 4)** can be rewritten as the function of the ratio of the depth of overburden H to the diameter D, H/D, and of the empirical coefficient K and internal friction angle ϕ.

$$\frac{h_0}{H} = \frac{D\cot\left\{(\pi/4 + \phi/2)/2\right\}}{2KH\tan\phi}\left\{1 - \exp\left(-K\tan\phi\frac{2H}{D\cot\left\{(\pi/4 + \phi/2)/2\right\}}\right)\right\}$$

$$= f\left(\frac{H}{D}, K, \phi\right) \qquad \textbf{(2. 5)}$$

Figure 2-6 shows the relationship between the normalized height of the simplified arching zone and the normalized depth of overburden for cohesionless soils with ϕ = 20, 30, and 40 degrees corresponding respectively to loose, medium and dense sand.

From Fig. 2-6, it is obvious that the normalized height of the simplified arching zone decreases with increasing internal friction angle ϕ or ratio H/D. Also, as the ratio H/D approaches zero, i.e., the overburden above a tunnel crown is significantly reduced, the arching zone affected by trapdoor movement or tunnel excavation extends to ground surface in all cases. Therefore, it can be imagined that a shallow tunnel has a significant influence upon existing structures on the ground surface.

As shown in Fig. 2-7, as a tunnel is placed lower and lower (the ratio H/D approaches 10), the effect of the second term on the right-hand side of the equation **(2.1)** becomes negligible. Therefore, the pressure at great depths becomes

$$p_{max} = \frac{B\gamma}{2K\tan\phi}, \qquad \textbf{(2.6)}$$

and is not affected by the depth of the overburden.

The situation in which the umbrella method is used may be explained by considering the arching effect. Figure 2-8 shows a schematic description of the arching induced by tunnel excavation in three different situations:

Case-1:

When there is a high ground resistance and a relatively deep overburden, the arching effect is mobilized and the zone affected by the trapdoor movement or tunnel excavation is small.

Case-2:

When there is a low ground resistance and a relatively deep overburden, the arching effect is mobilized. However, due to low ground resistance of the surrounding soil, the tip of the arching zone affected by the trapdoor movement or tunnel excavation reaches ground surface.

Case-3:

When there is a low ground resistance and a shallow overburden, although the arching effect might occur, it is not completely mobilized.

Figure 2-8 also contains a conceptual description of the downward movements above the tunnel crown in the three cases.

In Case-1 the tunnel crown movement does not transfer to the ground surface. However, as the depth and/or the ground resistance decrease, the tunnel crown movement transfers more and more to the surface, and finally, the surface settlement becomes almost equal to the tunnel crown settlement (Case-3). Results of experiments by Adachi (1992) who simulated tunnel excavation clearly demonstrate this behavior.

As will be mentioned later, the fact that the umbrella method is frequently employed to control the surface settlement implies that the method is usually employed in either Case-2 or Case-3.

According to the Terzaghi's definition, the arching effect should be defined as the transfer of pressure between the yielding and stationary masses of soil. Therefore, it should be emphasized that it is not always appropriate to judge the mobilization of arching only by ground deformation.

2.3 Use of Plasticity Theory

When an opening is made below the ground surface, stress redistribution occurs in the ground surrounding the opening. Thus, excavation of a tunnel is associated with significant changes in the original in-situ stress condition. If the stresses in the vicinity of the tunnel exceed the yield point of the soil, it is said that they reach a plastic state. The theory on which the computation of the stresses in a state of plastic equilibrium is based is called *the theory of plasticity*. Mohr's theory of rupture is one of the most common theories of plasticity.

For soft ground, the zone where the stresses are in a plastic state, the so called *plastic zone*, occurs as a result of stress redistribution due to relief of the initial stresses at the circumference of the excavation. It is difficult to obtain an explicit solution for the stresses and displacement under general initial stress conditions. However, assuming that the horizontal in-situ stress (p_h) is equal to the vertical in-situ stress ($p_z = \gamma H$), i.e., $K_o = 1$, an exact solution for uniform initial stresses can be developed.

For simplicity, a cohesive material ($\phi = 0$) can be considered. The geometry of the problem is defined in Fig. 2-9. Let us consider a circular opening being excavated in an infinite medium subjected to isotropic initial stresses p_o (= p_z). The excavation removes the boundary stresses around the circumference. The initial excavated radius is a. As p_i is reduced, a plastic zone develops; the radial displacement u_i occurs and the radius is now r_i. At the interface of the elastic and the plastic zone, a displacement from the elastic zone u_{ee} occurs and the radius of the plastic zone is reduced to the current value of r_e from r_{eo}. The solution of this problem can be expressed as follows (Lo et al., 1984):

(1) Radius of plastic zone: Assuming that yielding occurs when

$$\sigma_1 - \sigma_3 = 2\,c_u\,, \tag{2. 7}$$

where c_u is undrained shear strength or one half of the unconfined compression strength, q_u.

Hence, the radius r_e of the plastic zone is given by

$$\frac{r_e}{r_i} = \exp\left\{\frac{p_o - p_i - c_u}{2c_u}\right\} \tag{2. 8}$$

where

r_e = radius of the plastic zone
r_i = current radius of the opening ($r_i = a - u_i$)
p_o = isotropic initial stresses
p_i = internal (support) pressure
c_u = undrained shear strength of cohesive soil .

(2) Radial displacement u_i: The radial displacement in the elasto-plastic state is given by

$$\frac{u_i}{a} = 1 - \left\{\frac{1}{1 + \frac{2(1+\nu)c_u}{E_u}\left(\frac{r_e}{r_i}\right)^2}\right\} \tag{2. 9}$$

where

u_i = radial displacement
a = initial excavated radius
ν = Poisson's ratio of cohesive soil (ν = 0.5 for undrained case)
r_e = radius of the plastic zone
r_i = current radius of the opening ($r_i = a - u_i$)
c_u = undrained shear strength of cohesive soil .

In terms of the stability number $N = \gamma H (= p_o) / c_u$, equation **(2.8)** can be written as follows:

$$\frac{r_e}{r_i} = \exp\left\{\frac{1}{2N} - \frac{p_i}{2c_u} - \frac{1}{2}\right\}. \tag{2. 10}$$

Figure2-10 is a plot of the plastic radius, r_e, against the stability number, N, for the case of p_i = 0. This condition represents an extreme condition in which a tunnel is excavated without air pressure or support reaction pressure from a lining.

It can be seen that the plastic radius increases with an increase in the stability number. If the soil is to remain in the elastic state, N will have to be 1 or less than 1. As shown in Table 2-1, Peck (1969) suggests that tunnelling can be carried out without unusual difficulties, if N is less than 5.

Table 2-1 Tunnel stability: cohesive soils

Stability number (N)	Tunnel behavior
1	Stable
2 - 3	Small Creep
4 - 5	Creeping, usually slow enough to permit tunnelling
6	May produce general shear failure. Clay likely to invade tail space too quickly to handle

Source: Peck (1969)

Lo et al. (1984), on the other hand, suggest that N be reduced by 50 to 70%, since the undrained shear strengths used in obtaining this value correspond to those determined on specimen from conventional tube sampling, and hence, the sampling disturbance may reduce strength by a factor of 0.5 to 0.7. Thus, the actual N value lies in the range of 2.5 to 3.5.

Taking a value of N=3, Fig. 2-10 shows that the corresponding plastic radius is about 3 times the radius of the tunnel.

The plain strain radial displacement is plotted in Fig. 2-11 against the stability number, N, for three values of the ratio E_u/C_u commonly encountered in clays, where E_u and C_u are undrained modulus of elasticity and undrained shear strength, respectively. It may be seen from Fig. 2-11 that deformation increases rapidly as N exceeds a value of 3. This increase in displacement is due to an increase in the extent of the plastic zone. It should also be noted that the displacement is sensitive to E_u / C_u.

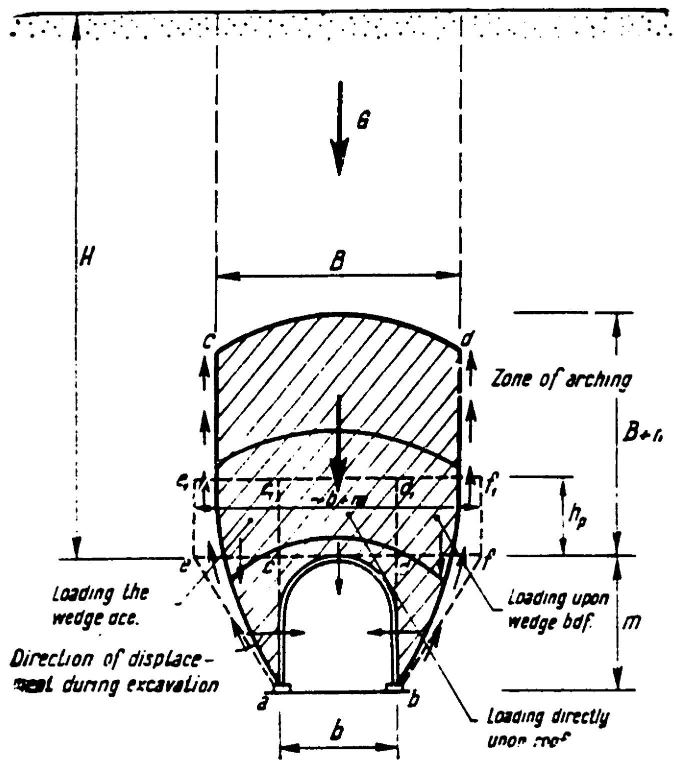


Fig. 2-1 Arching above cavity (After Terzaghi, 1943)

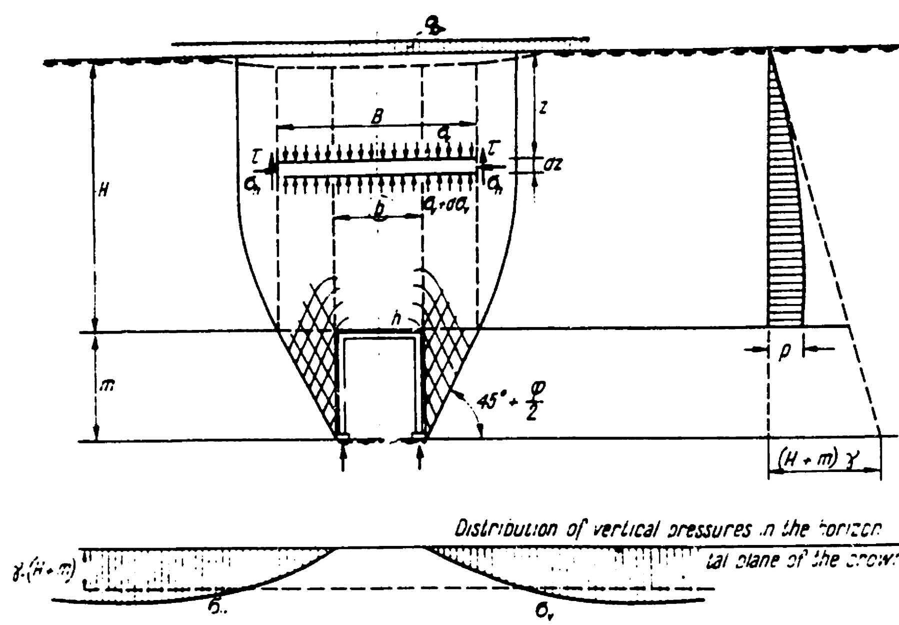


Fig. 2-2 Basic assumption of Terzaghi's earth pressure theory (Szechy, 1966)

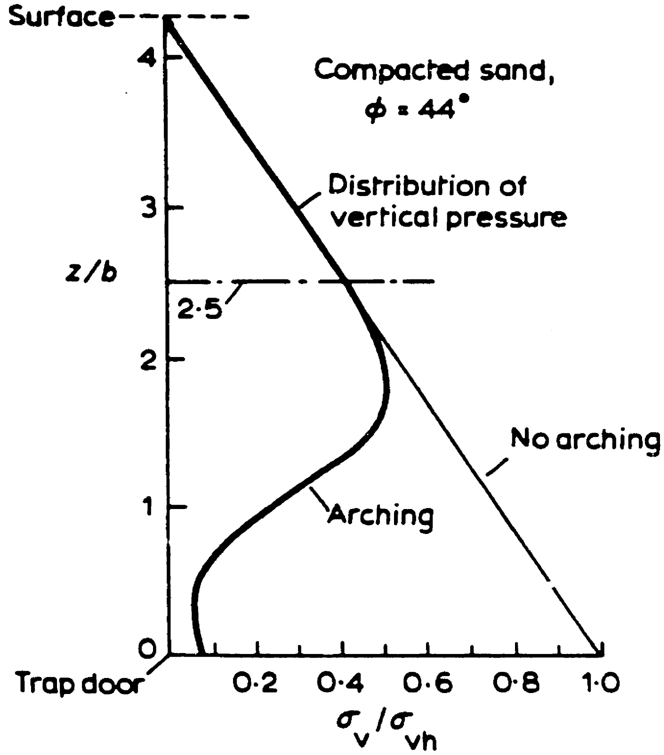


Fig. 2-3 Terzaghi's trapdoor experiments (note: z measured upwards from trapdoor) (Bulson, 1985)

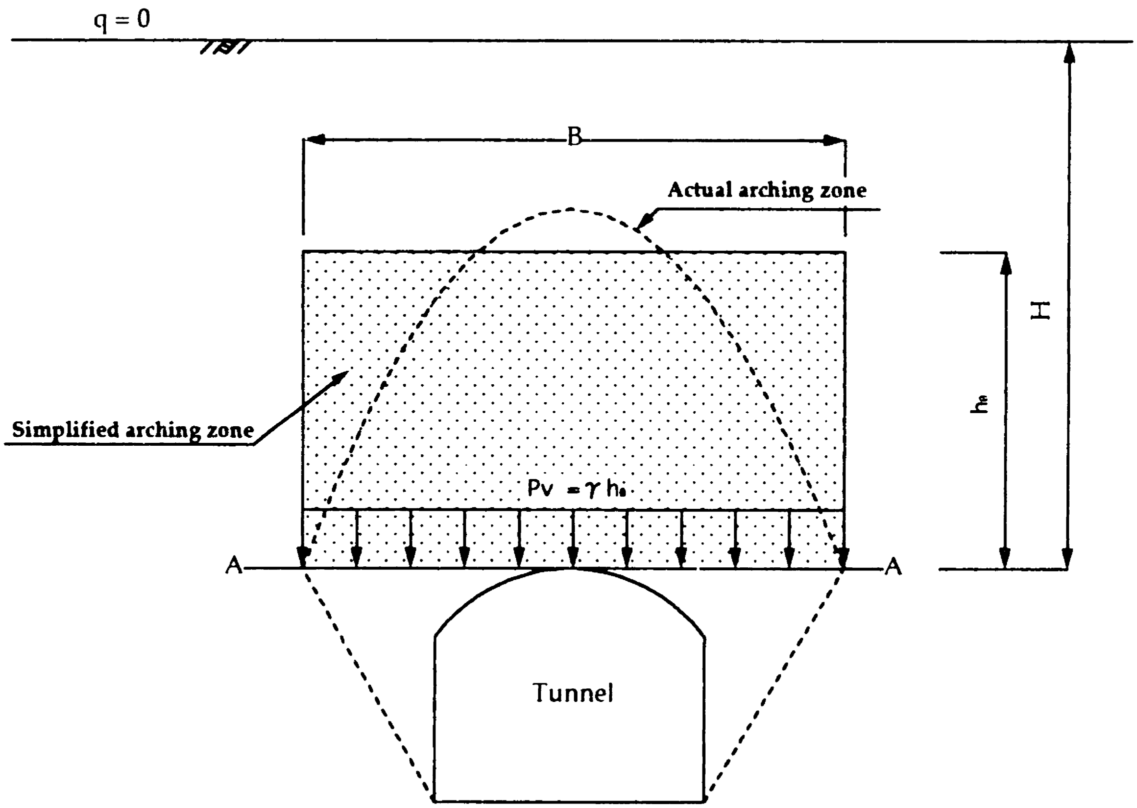
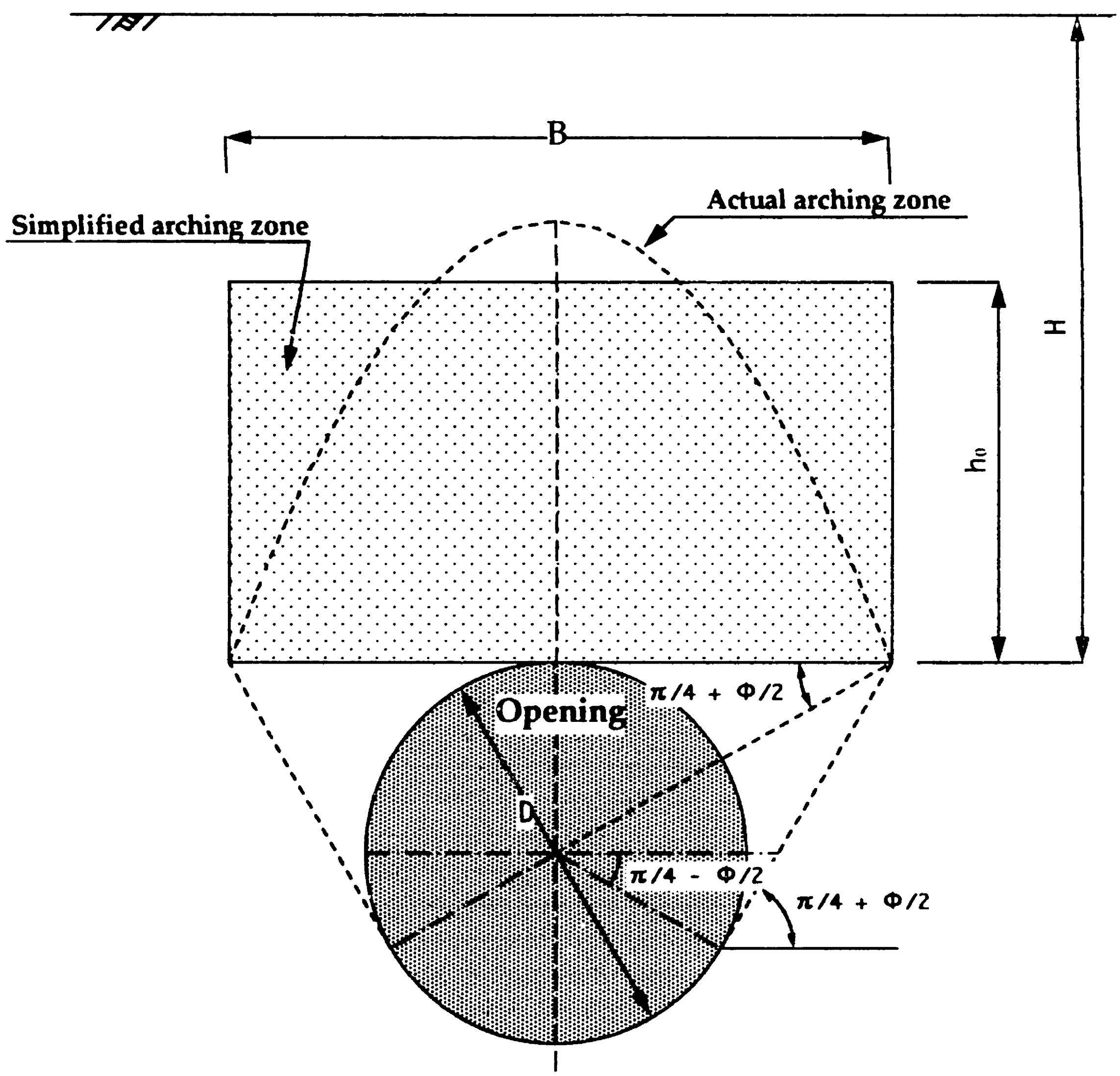


Fig. 2-4 Simplified arching zone



$B = D \cot[(\pi/4+\Phi/2)/2]$
ho: height of arching zone
H: depth of overburden
D: tunnel diamter

Fig. 2-5 Simplified arching zone around a circular opening

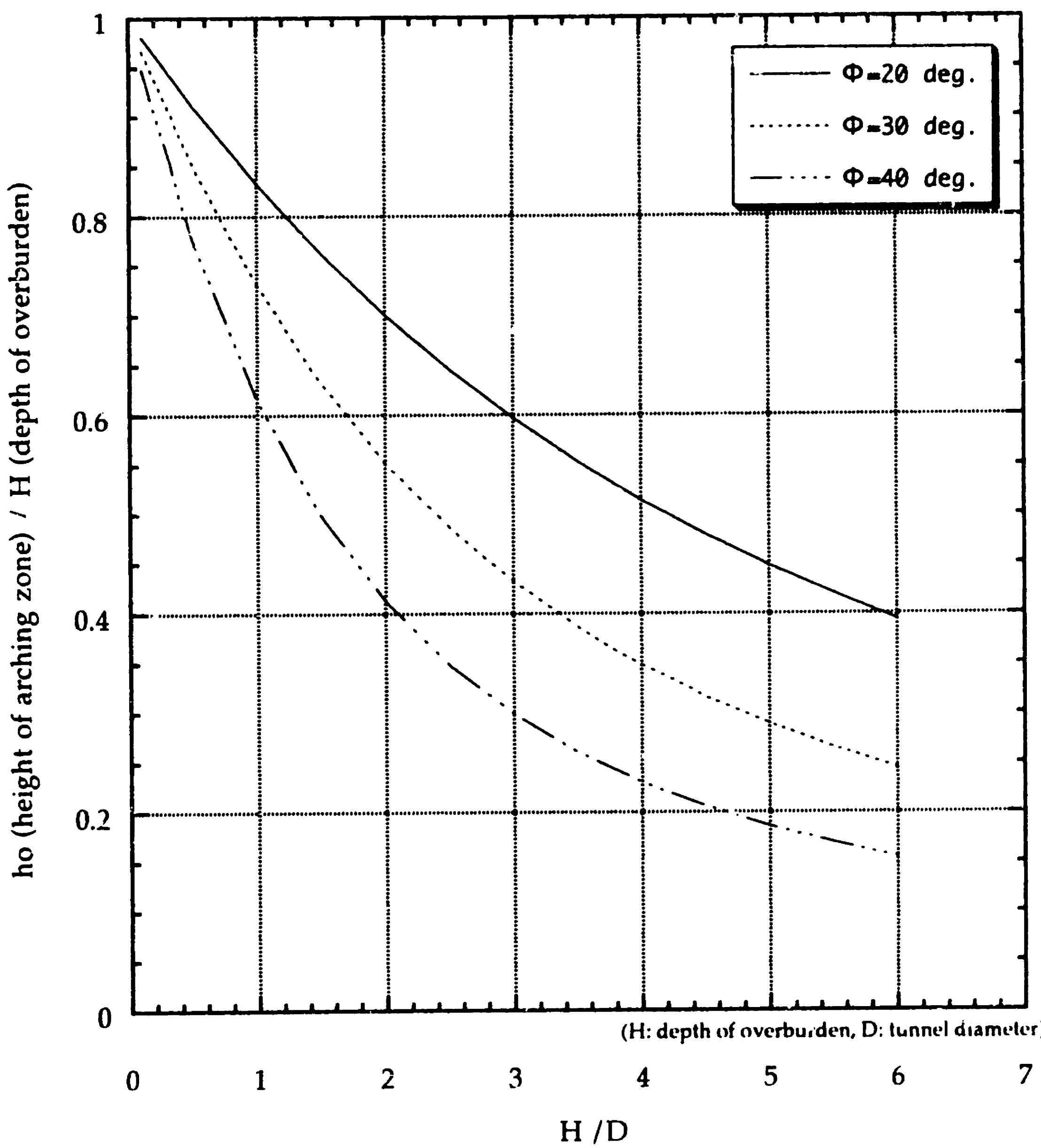


Fig. 2-6 Normalized height of arching zone versus normalized depth of overburden

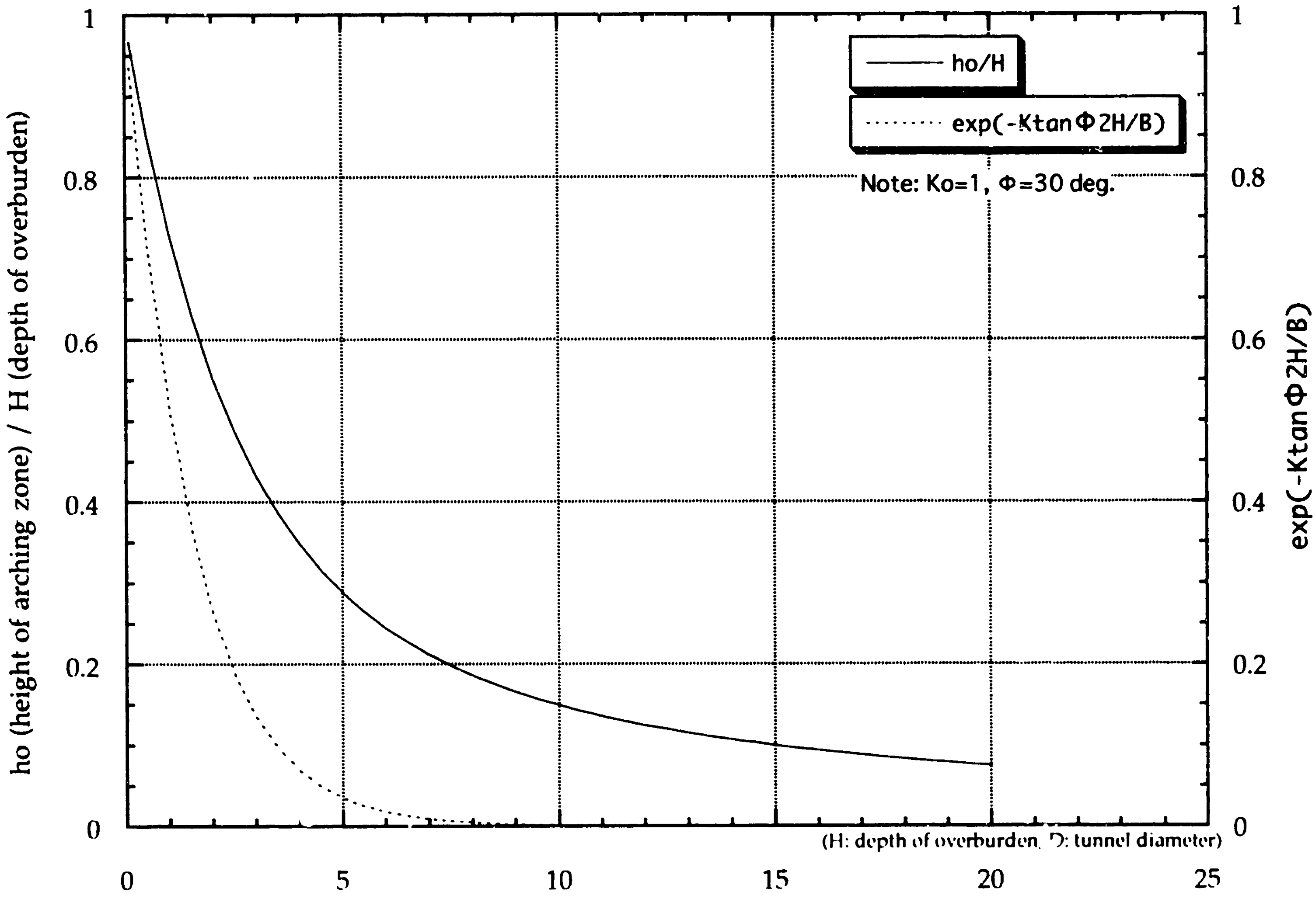


Fig. 2-7 Normalized depth of overburden versus normalized height of arching zone and value of term exp. (-K tanϕ 2H/B)

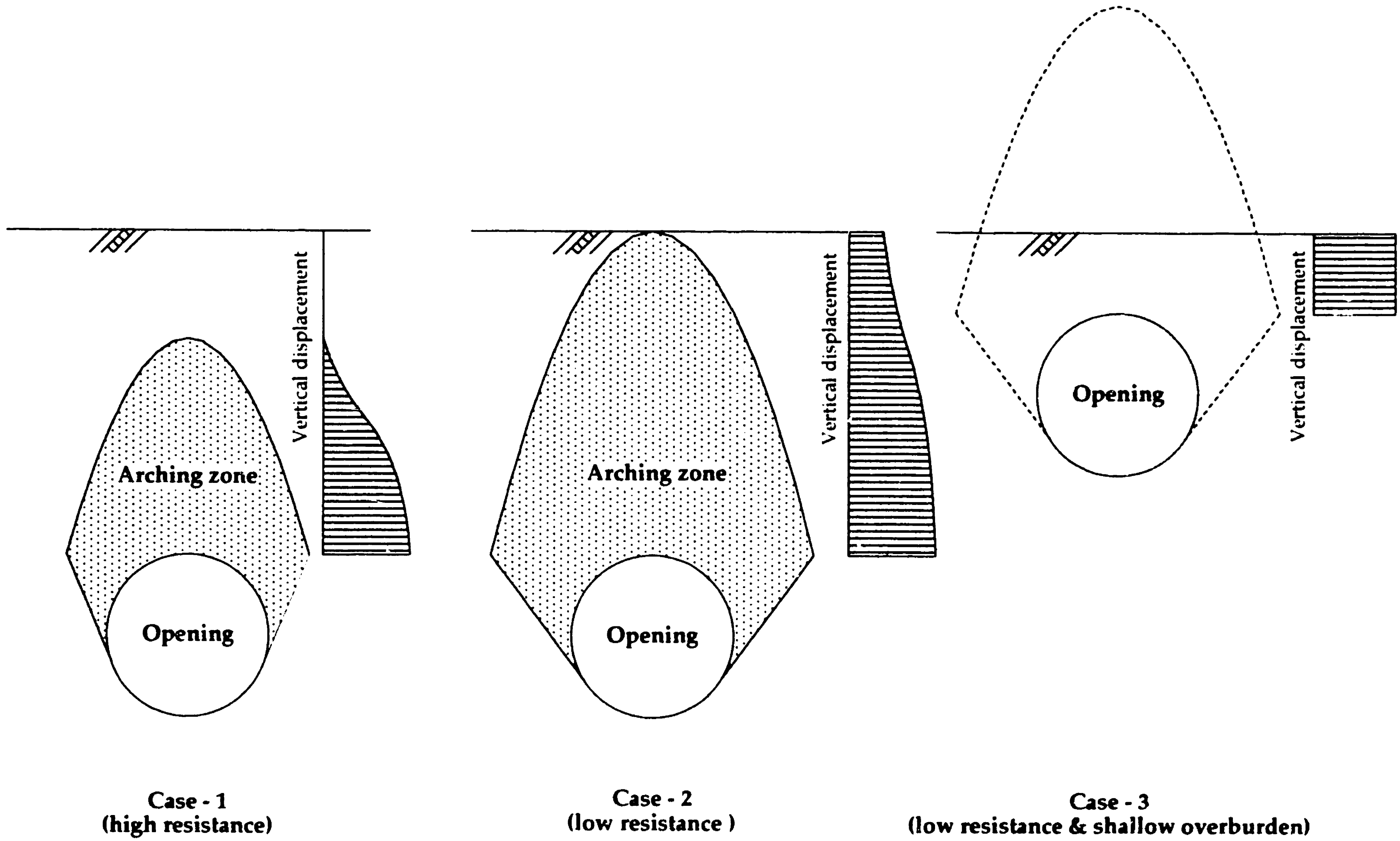


Fig. 2-8 Schematic of development of arching zone and distribution of vertical displasement above a circular opening

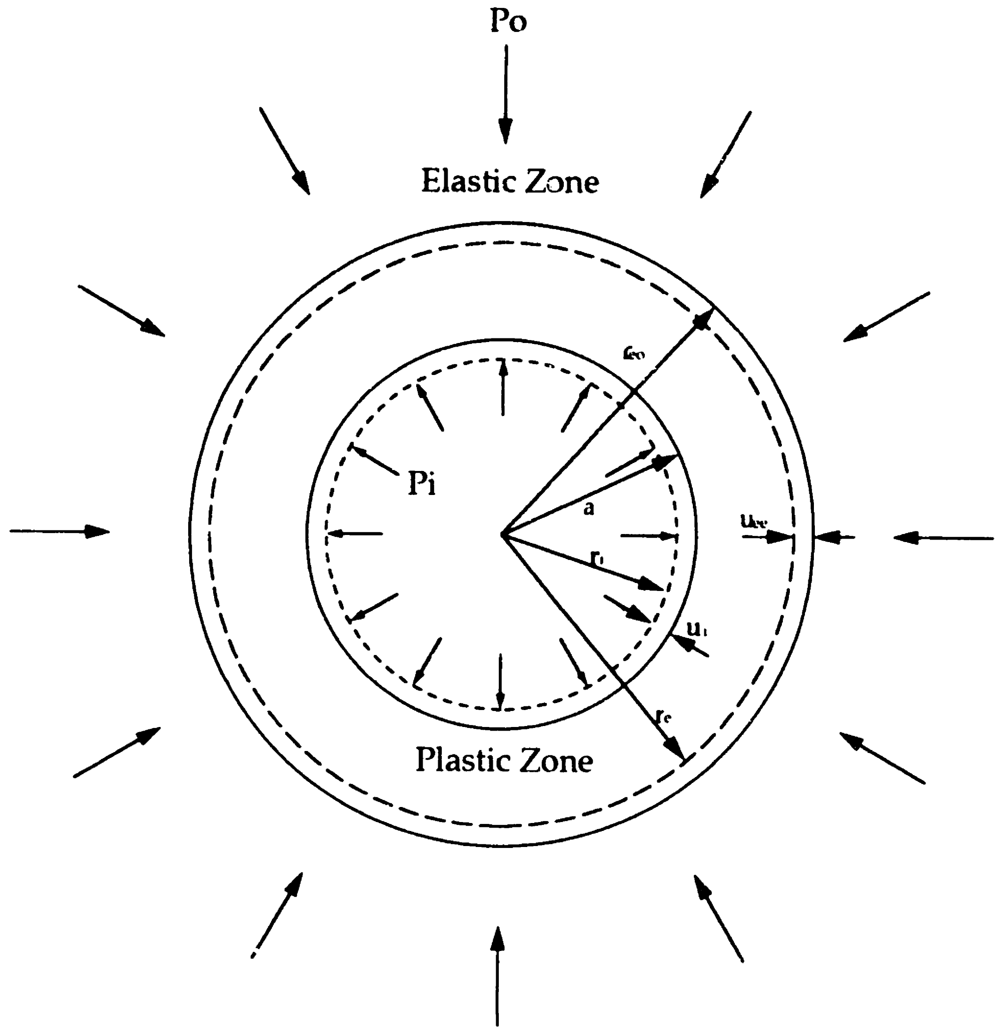


Fig. 2-9 Plastic zone around a circular opening

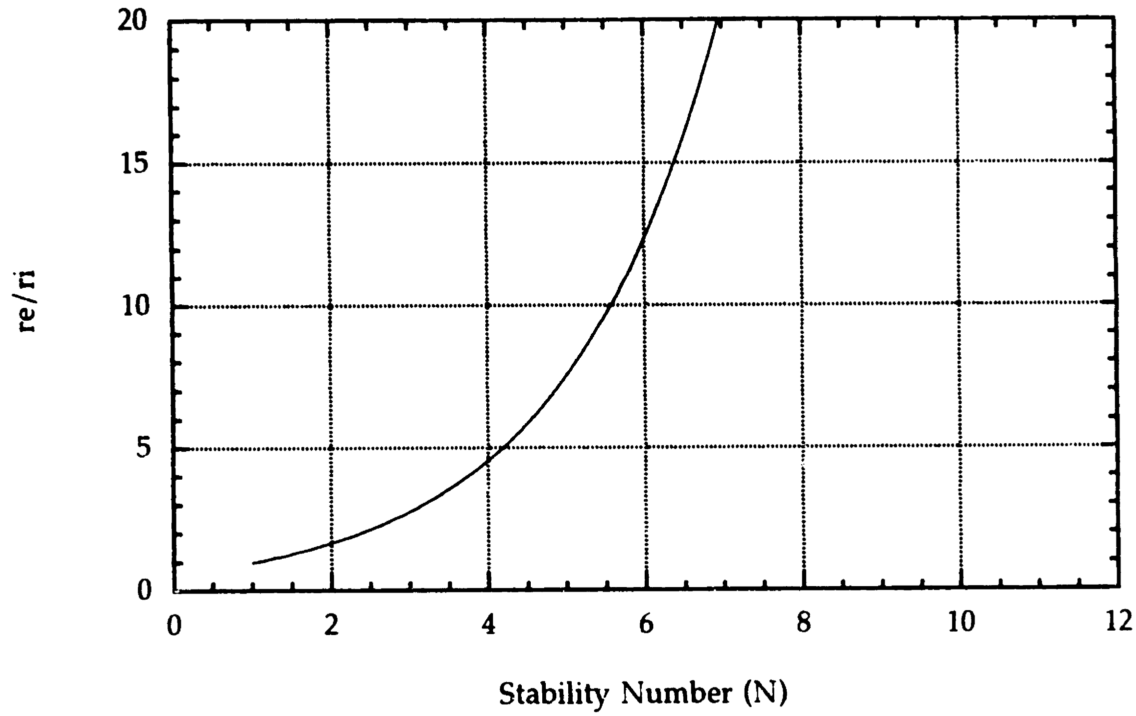


Fig. 2-10 Stability number versus normalized plastic radius (Lo et al., 1984)

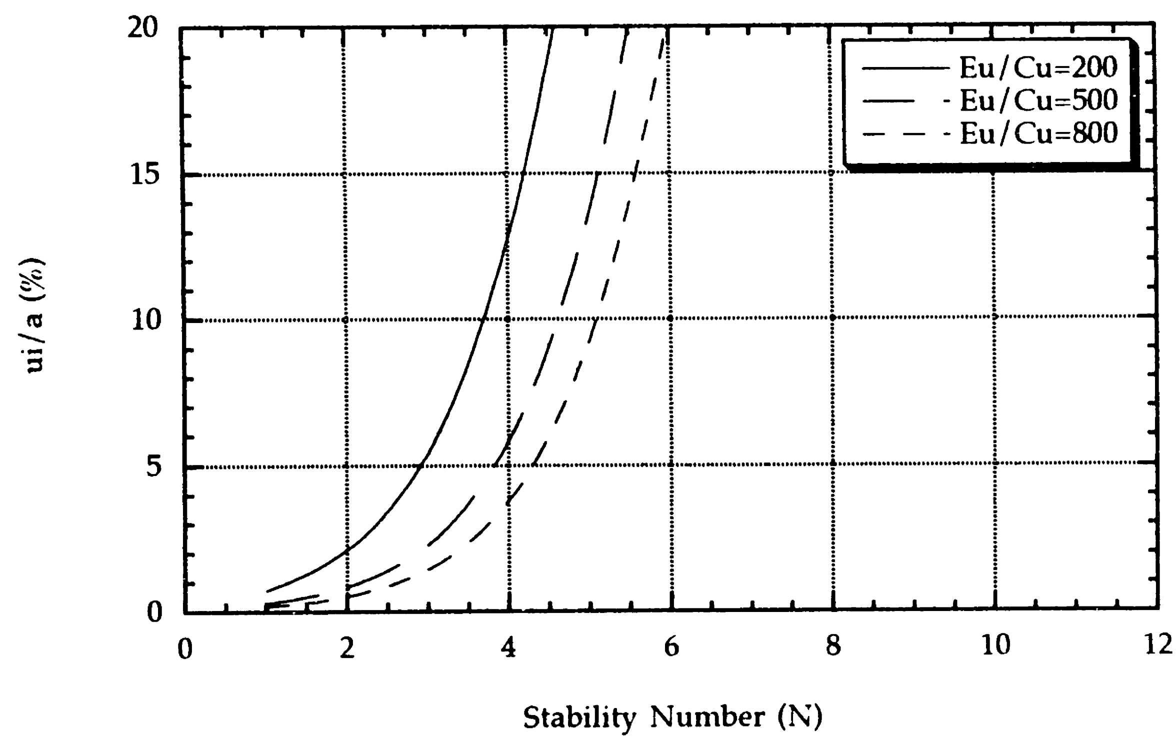


Fig. 2-11 Stability number versus normalized radial crown displacement (Lo et al., 1984)

Chapter 3. Principles of the Umbrella Method

3.1 Overview

The principles of the umbrella method are presented in this chapter.

As mentioned in Chapter 1, in the umbrella method an "arch-like" shell is created ahead of the face prior to excavation, enabling tunnel excavation to be safe and speedy under an umbrella-like structure. The outstanding characteristics of the method are the support provided by this arch-like shell structure. Thus, first of all, discussing the supporting mechanisms of the umbrella method is vital to understanding it.

Second, drilling and injecting methods are discussed, since these methods are indispensable and affect to a great extent the characteristics of each type of umbrella method.

Third, the detailed principles of the three typical umbrella methods, i.e., 1) sub-horizontal jet-grouting method, 2) injected steel pipe umbrella method and 3) pipe roof method are discussed.

3.2 Supporting Mechanisms of the Umbrella Method

Ground settlement induced by tunnel excavation gradually increases as the face approaches. When the face reaches a particular point, 30 to 40% of the total amount of the ground settlement at the particular point has occurred (see Fig. 3-1). This means that any countermeasure applied in the tunnel itself can not restrain any settlement occurring in the ground ahead of the face. Therefore, reinforcement prior to the tunnel excavation is considered to be effective in reducing the ground settlement ahead of the face.

The supporting mechanism of the umbrella method is to stabilize the excavation face in both the transverse and longitudinal directions by an arch-like reinforced zone (see Fig. 3-2).

As shown in Fig. 3-2 (a), regarding the stabilization in the transverse direction, an arch-like reinforced zone carries the ground load. Regarding the stabilization in the longitudinal direction, on the other hand as shown in Fig. 3-2 (b), the ground load of the freshly excavated portion is supported by a beam with one end supported by the tunnel support and the other end supported by the ground.

By supporting ground load in this way, the following effects on tunnel excavation occur:

- restraining ground settlement ahead of the face
- increasing face stability
- reducing dimensions of tunnel supports (shotcrete, steel arch support, etc.)

• enlarging the cutting face, which makes it possible to use large machinery, which in turn, leads to rapid excavation

Of these effects, let us first consider the increase in face stability by the umbrella method.

As shown in Fig. 3-3 (a), there is a stress concentration ahead of the face, followed by a rapid decrease of vertical stress to zero at the tunnel face (point A). The stress has to be zero along the unsupported section of the tunnel (between points A-B), assuming that the tunnel supports are installed at a certain distance behind the face. With the supports installed, there should be an increase of vertical stress again.

In contrast (see Fig. 3-3 (b)), in the case in which the umbrella method is employed, the unsupported section of the tunnel is covered with the umbrella structure and, as a result, the structure bears the overburden pressures. Similar to the relation between the lining and the ground, vertical stresses acting on the umbrella structure depend on the relative stiffness between the umbrella structure and the ground. It should be emphasized that, due to the umbrella structure, the stress concentrations both ahead of the face and behind the face are smaller.

Next, let us consider the soil element ahead of the face.

Assuming that vertical stress and horizontal stress are major and minor principal stresses, σ_1, σ_3, respectively, the initial stress state in the soil element, when the face is far from the element, is expressed by the dashed Mohr circle shown in Fig. 3-3. As the face approaches the soil element, the stress state in the soil element changes due to an increase in major principal stress, $\Delta\sigma_1$. For simplicity's sake, the minor principal stress σ_3 is assumed to be constant. Finally, as shown in Fig. 3-3 (a), when the face arrives at a certain distance from the soil element, the Mohr circle is tangent to the Mohr-Coulomb failure envelope; as a result, yielding of the soil element occurs. On the contrary, as shown in Fig. 3-3 (b), when the umbrella method is employed, the increase in major principal stress is controlled to some extent, consequently, the Mohr circle is not tangent to the failure envelope and hence failure does not occur.

Taking the above into consideration, it can be said that the effect of the umbrella method on the stress state of the soil element ahead of the face restricts the increase in the major principal stress.

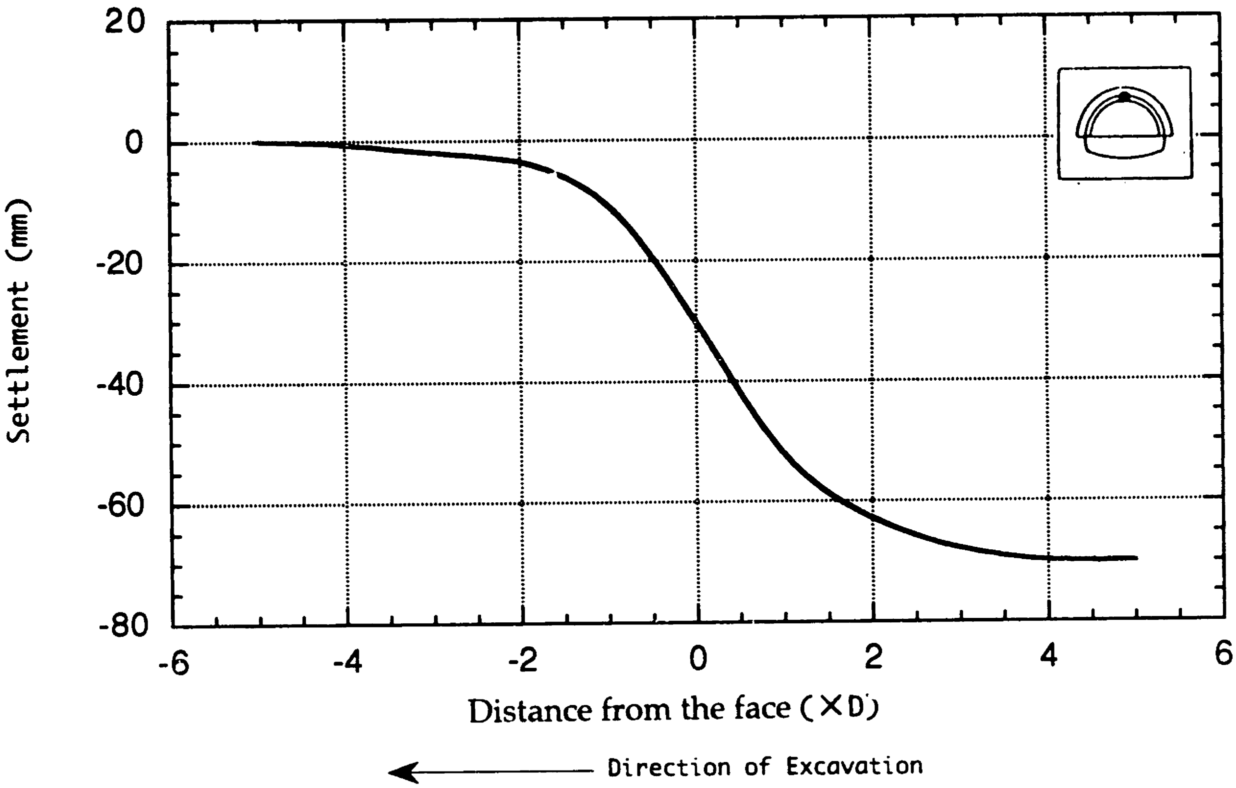
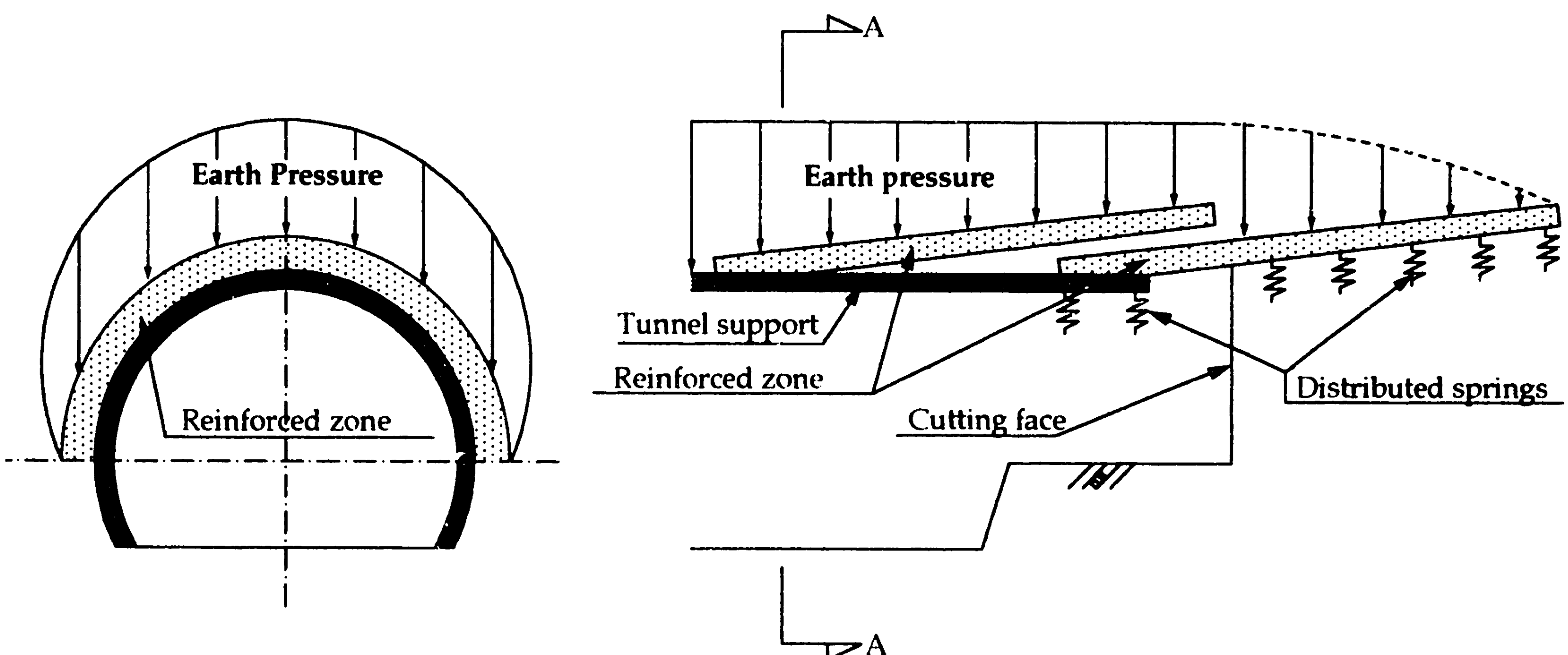


Fig. 3-1 Settlement curve



a) transverse view
(A-A section)

b) longitudinal view

Fig. 3-2 **Schematic of the supporting mechanisms of the umbrella method (Geo-Fronte Research Association, 1994)**

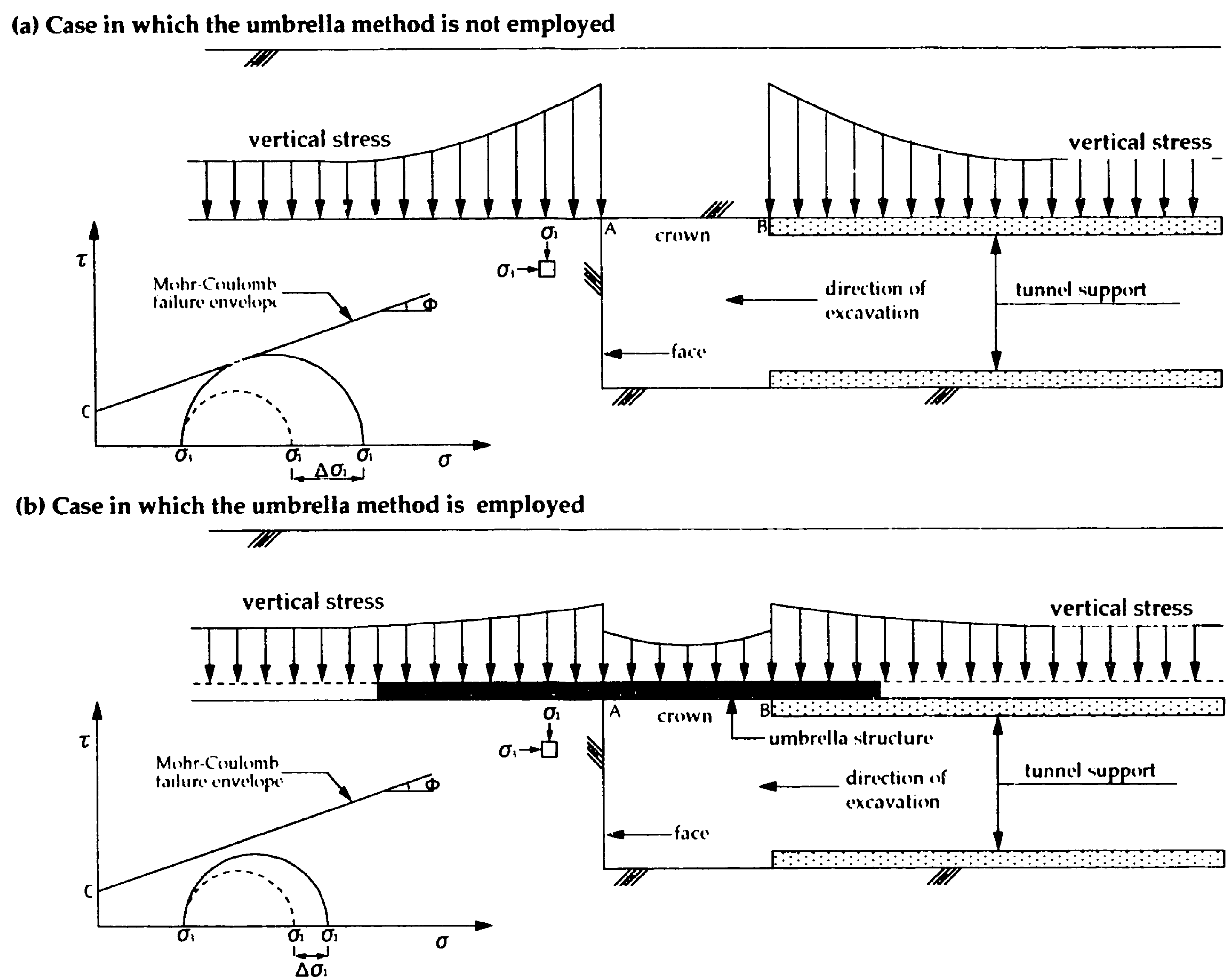


Fig. 3-3 Vertical stress distribution along a longitudinal line at the tunnel crown and stress state ahead of the face

3.3 Classification of the Umbrella Method

As mentioned in Chapter 1, the umbrella method can be subdivided into three categories according to materials used and/or ways of reinforcing ground:

- sub-horizontal jet-grouting method (Note: The method is further subdivided into two categories, i.e., the reinforced and unreinforced sub-horizontal jet-grouting methods)
- injected steel pipe umbrella method
- pipe roof method

A schematic of each of the umbrella methods is shown in Figs. 3-4 through 3-6. A description of each umbrella method will be given later.

As shown in Table 3-1, the overall stiffness of each type of umbrella method is affected by the following:

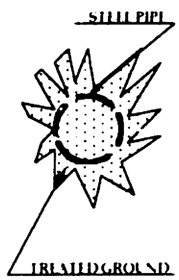
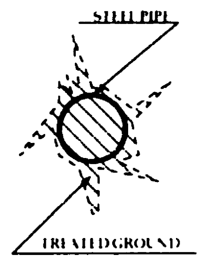
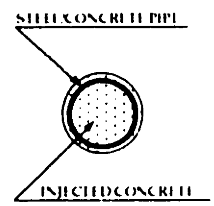
- reinforced or unreinforced sub-horizontal jet grouting method:
  [jet-grouted soil column] + [driven steel pipe] (if used) + [treated ground by permeation or fracture grouting]
- injected steel pipe umbrella method:
  [driven steel pipe] + [treated ground by permeation or fracture grouting]
- pipe roof method:
  [driven steel or concrete pipe] + [injected concrete inside a steel or concrete pipe]

It should be noted that some components such as the jet-grouted soil column or the driven steel pipe function as a beam as shown in Fig. 3-2. The treated ground, on the contrary, makes the umbrella structure function as an arch-like structure in the transverse direction.

From Table 3-1, it can be seen that each of the umbrella methods is related to a particular grouting method. In addition, although not in the table, driving steel pipes or improving the ground is directly associated with particular drilling methods. Therefore, understanding both grouting and drilling methods is crucial in discussing the principles of the umbrella method.

In the following two sections, grouting and drilling methods are briefly discussed, and then, the principles of each umbrella method are examined.

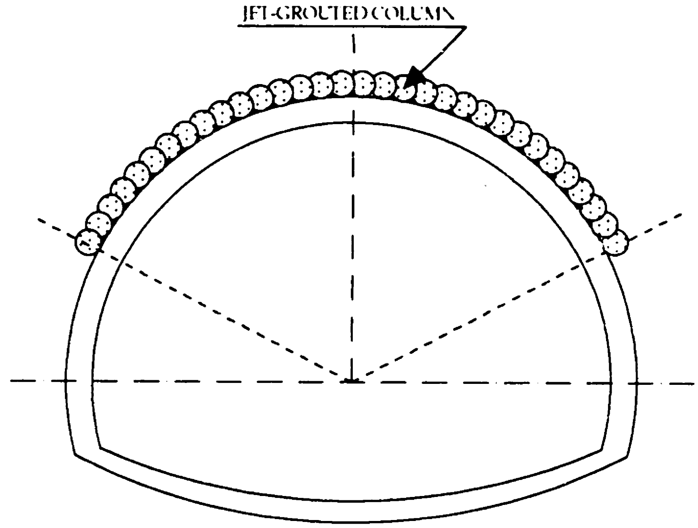
Table 3-1 Types of umbrella methods

Types of umbrella methods	Sub-horizontal jet-grouting method	Injected steel pipe umbrella method	Pipe roof method
Schematic / Stiffness			
Stiffness of the driven steel or concrete pipes	○[1)]	○	○
Stiffness of the grout filling both the inside of the driven steel or concrete pipes and the voids around the pipes		○	○
Stiffness of the treated ground around the tunnel formed by fracture or permeation grouting	○	○	
Stiffness of the treated ground around the tunnel formed by jet-grouting	○		
Names of the methods	RJFP (Italy) TREVIJET T1 (Italy) MJS (Japan)	TREVITUB (Italy) RODINTUB (Italy) AGF (Japan)	PIPE ROOF CELLULAR ARCH (Italy)

Note: 1) Reinforced sub-horizontal jet grouting method.

Source: Geo-Fronte Research Association (1995)

(a) Unreinforced



(b) Reinforced

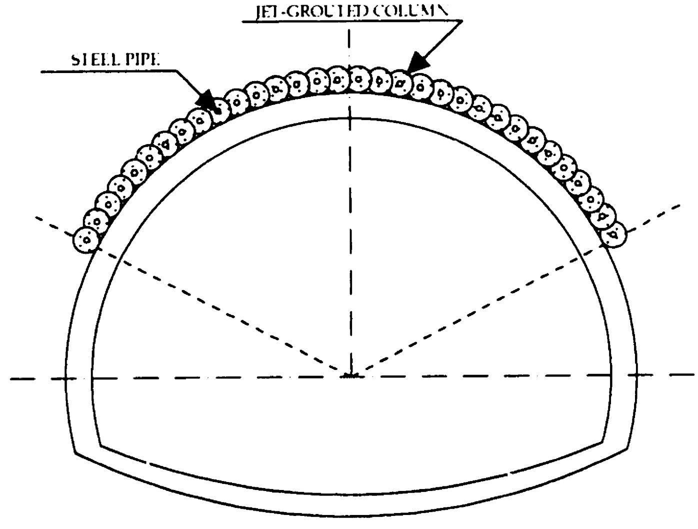


Fig. 3-4 Sub-horizontal jet-grouting method

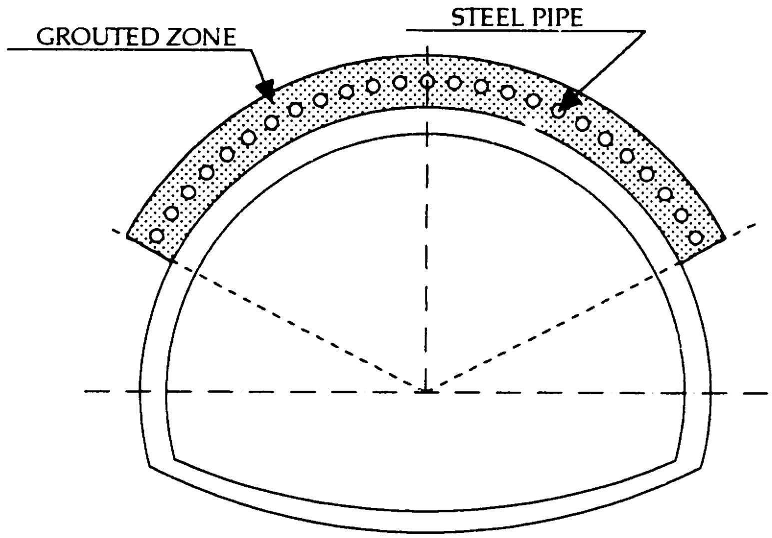


Fig. 3-5 Injected steel pipe umbrella method

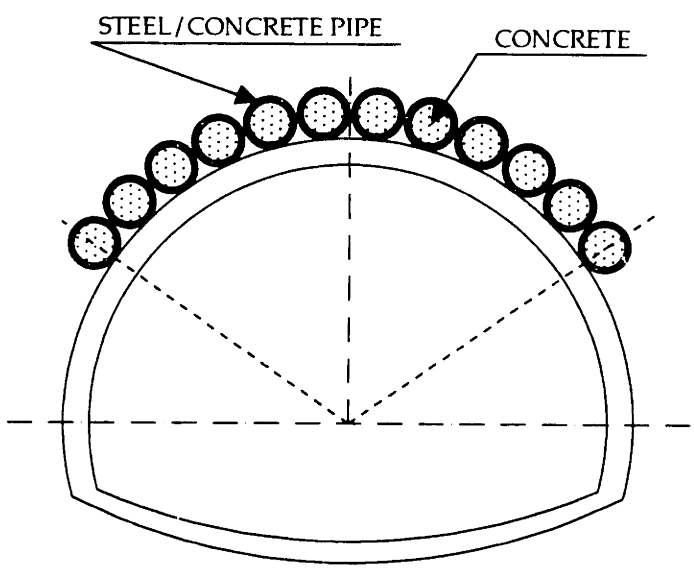


Fig. 3-6 Pipe roof method

3.4 Drilling Methods

3.4.1 Overview

A detailed discussion of all drilling methods is beyond the scope of this thesis, however, it is important that a brief summary of important aspects be given.

In general, drilling methods used in the umbrella method are grouped into three types, namely rotary drilling, rotary-percussion drilling and pipe jacking, including the so called microtunnelling. As shown in Fig. 3-7, these types can be further subdivided:

- rotary drilling
  - rotary drilling
  - double rotary drilling
- rotary-percussion drilling
  - rotary drilling with down-the-hole hammer
  - rotary-percussion drilling with top hammer
- pipe jacking (microtunnelling)
  - pilot boring method
  - jacking and boring method
  - shield tunnelling method

3.4.2 Brief Description of the Drilling Methods

(1) Rotary drilling

Rotary drilling has two distinguishing characteristics: 1) drilling fluid is forced, by means of a suitable pump, down through the inside of the drill pipe and out through the bit openings; and 2) excavation is achieved by rotating the drill stem with the bit at the bottom.

The rotary system is preferred in medium to fine grained soils, where fairly small rigs are required. In coarse grained soils or any soil including cobbles and boulders, rotary-percussion may be better suited in terms of drilling speed, since in such soils the drilling speed of the rotary drilling dramatically decreases (A down-the-hole hammer is a good complement to rotary-percussion drilling in some cases. The down-the-hole-hammer is used to avoid slowing down the operation. In most cases, the down-the-hole hammer can be fitted directly to the drill tubes with the rotary unit.).

The basic principles of the double-rotary drilling system are the same as those of the rotary drilling method, however, drilling is carried out by rotating an outer steel pipe, and rotating and hammering an inner rod. The steel pipe which is used as a casing pipe during drilling is left in the hole after the hole is drilled (see Fig. 3-8).

Compared to the conventional rotary drilling method, the length of the power unit is longer and the power system is more complicated. However, in cases in which removing spoil is difficult and/or the surrounding ground tightens around the outer pipe, so that conventional drilling methods such as rotary drilling are not applicable, double-rotary drilling is effective.

(2) Rotary-percussion drilling

Rotary drilling with a down-the-hole hammer consists of a rotary head and drill rods, with an air-operated down-the-hole hammer which is mounted at the end of the drill string (see Fig. 3-9). In this system a percussive action is applied to the bottom of the hole, whereas rotation and feed are respectively provided by a rotary head at the top of the drill string and a hoist mounted on the mast. Penetration in the ground is usually achieved through compressed air that operates the hammer and simultaneously provides flushing of the cutting out of the hole. With a down-the-hole hammer, as in rotary drilling, a steel pipe is used as a casing pipe during drilling and after the hole is drilled, it is left permanently in the hole. Removal of spoil is done with air or water.

An eccentric bit (see Fig. 3-10), the so called Odex bit, makes it possible to insert a casing tube (steel pipe) into a hole at the same time as the hole is being drilled. The Odex method is based on the principles of under-reaming which makes it possible to insert the casing tube in the hole without rotating it, while drilling is in progress.

Rotary-percussion drilling is usually employed in stiff ground such as gravel, bouldery layers, or rocky ground. In soft clay or loose sand ground, the hammer may not work properly or erosion induced by compressed air may become intense, hence another drilling system may have to be used.

In the case of rotary-percussion drilling with a top hammer, as shown in Fig. 3-11, the rotation mechanism can be incorporated in a separate rotation tool, so that the speed of the rotation can be regulated independently of the percussion mechanism. The system is applicable to ground ranging from clay to soft rock; however, since the casing tube is directly hammered into the ground, it is necessary to examine the strength of the steel pipe in stiff ground and ground with boulders to prevent failure.

A "drifter", as shown in Fig. 3-12, is, in general, classified as a "top hammer" type. As will be mentioned later, the steel pipes used in the AGF method are driven by means of a conventional "drill jumbo" with a number of drifters mounted on it (see Fig. 3-13).

(3) Pipe jacking (microtunnelling)

In pipe jacking or microtunnelling, a pipe of a relatively large diameter is jacked through the ground by means of a hydraulically operated jacking unit from a previously prepared construction pit (see Fig. 3-14). Soil is usually excavated at the face.

Characteristics of each method are shown in Fig. 3-15 and illustrations of each method are shown in Fig. 3-16 (Stein et al., 1989).

The pipe jacking method is mainly used in soils with SPT-N values of $5 < N < 50$ (Stein et al., 1989).

3.4.3 Suitable Ground for the Drilling Methods

It should be noted that the mode of drilling is selected according to soil conditions, general features of the site and design specifications in regard to length and inclination of the drill holes. See Table 3-2.

Table 3-2 Suitable ground for drilling methods

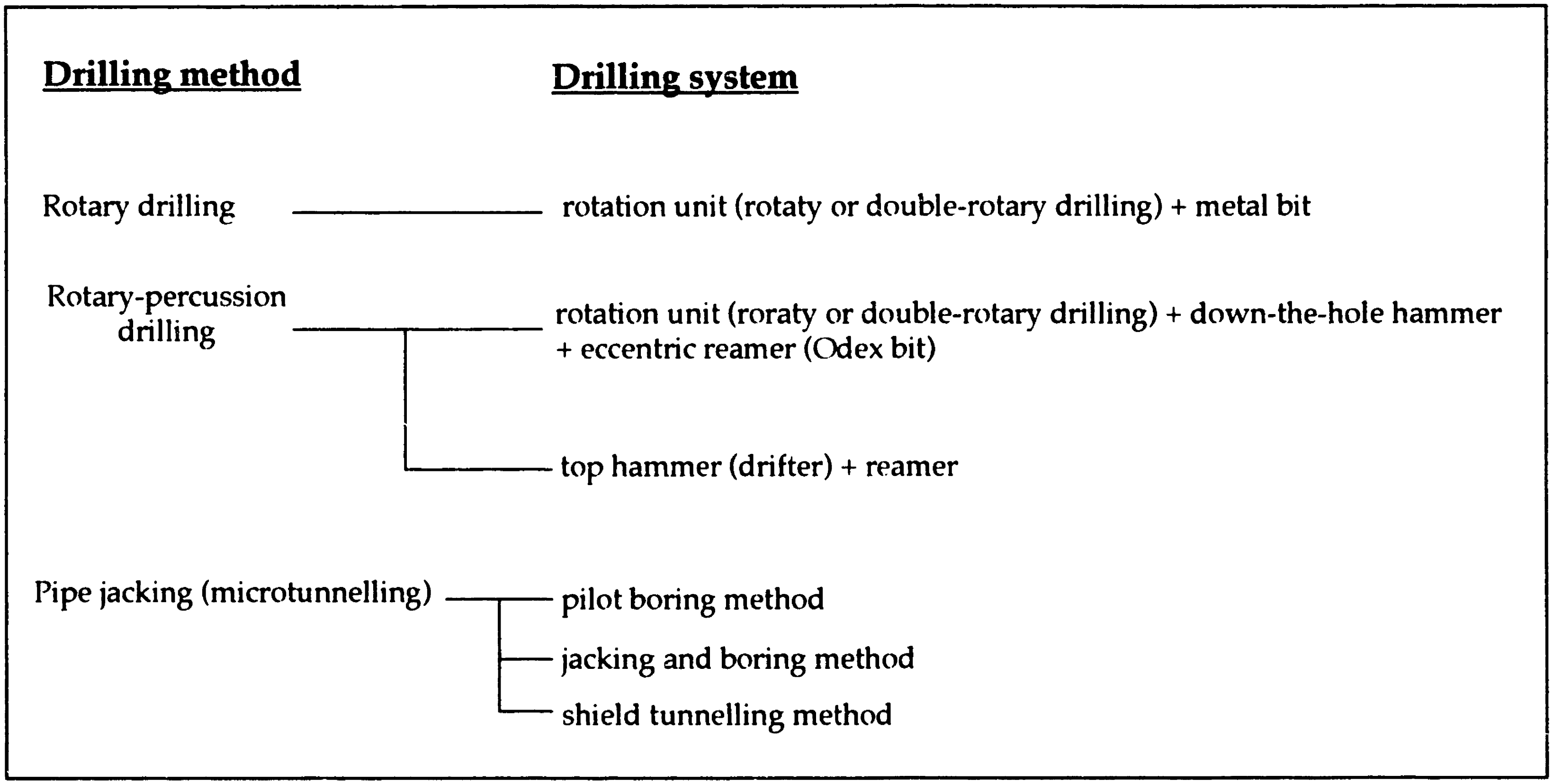
Drilling method	Soil					Talus Detritus Moraine	Weathered rock		
	Clay		Sand		Gravel	Soil with boulders	very soft	soft	medium to stiff
	N<15	N≥15	N<30	N≥30	N<50	N≥50	$\sigma_c<5$	$\sigma_c<20$	$\sigma_c<70$
Rotary	A	A	A	B	B	C	C	C	C
Double rotary	A	A	A	A	B	C	C	C	C
Down-the-hole hammer	C	B	C	B	B	A	A	A	A
Top hammer	B	A	A	A	A	B	A	B	C
Drifter	B	A	A	A	B	B	B	B	C
Pipe jacking / microtunnelling	A	A	A	A	B	C	C	C	C

Note: 1)Pipe jacking/microtunnelling was added to the table presented by the Geo-Fronte Research Association (1995).

2)N: number of blows in standard penetration test. σ_c : unconfined compressive strength (MPa).

3)Code: A = Most applicable; B = May be used; C = Not applicable

Source: Geo-Fronte Research Association (1995)



Note: Pipe jacking (microtunnelling) was added to the table presented by the Geo-Fronte Research Association (1995).

Fig. 3-7 Drilling methods (Geo-Fronte Research Association , 1995)

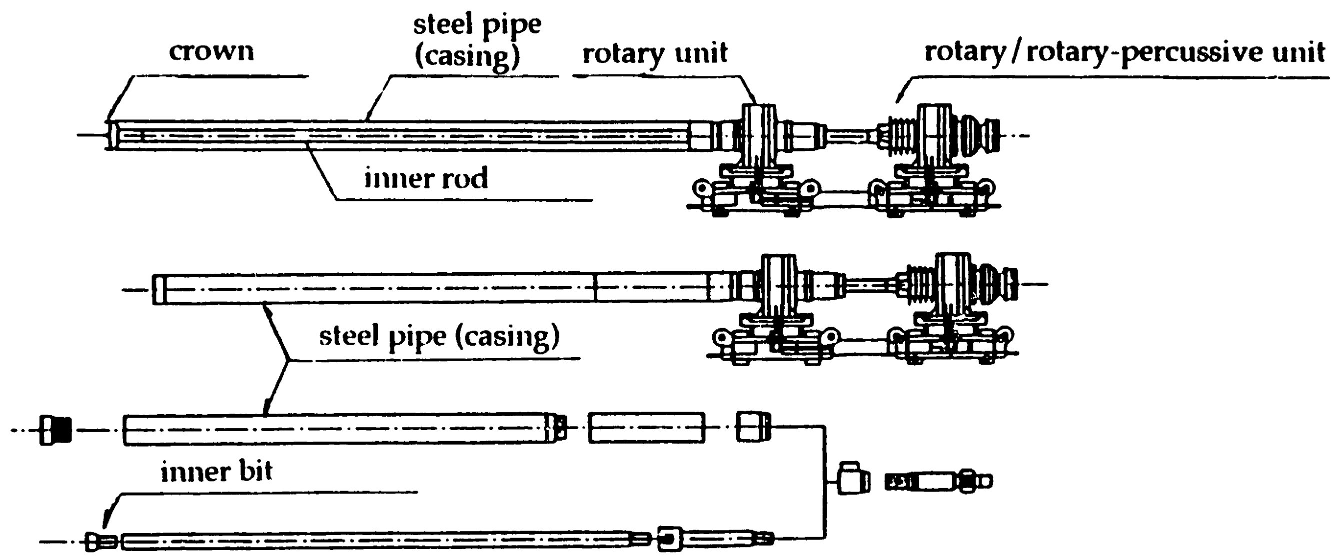


Fig. 3-8 Double rotary drilling (Geo-Fronte Research Association, 1995)

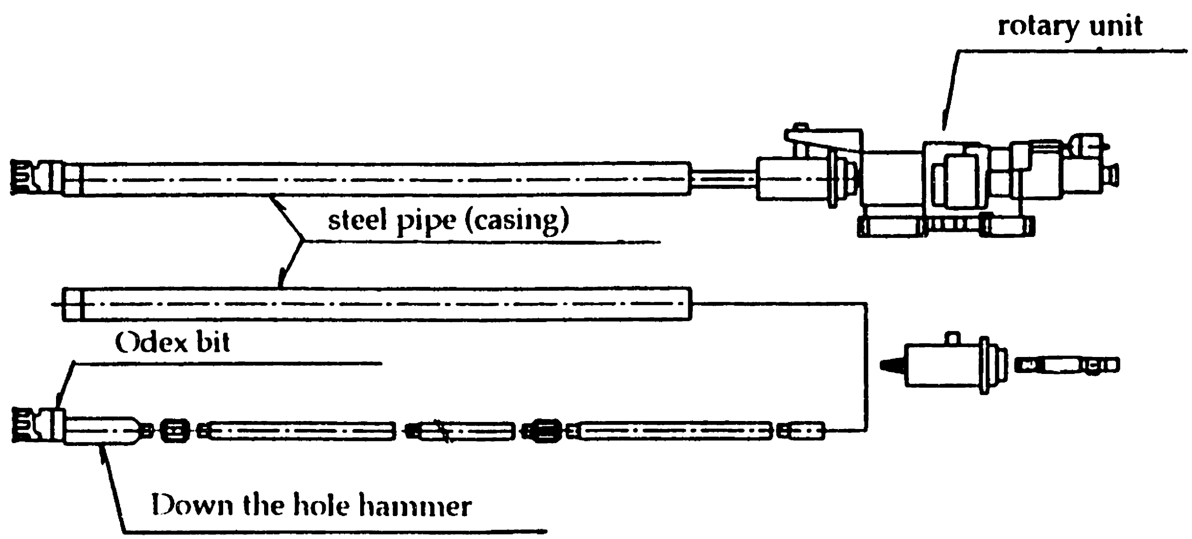
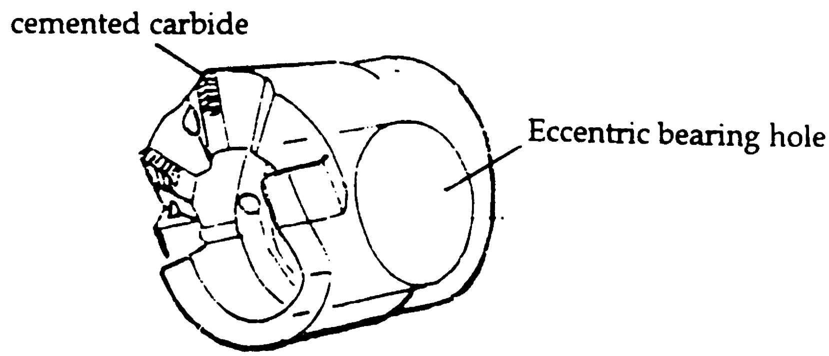
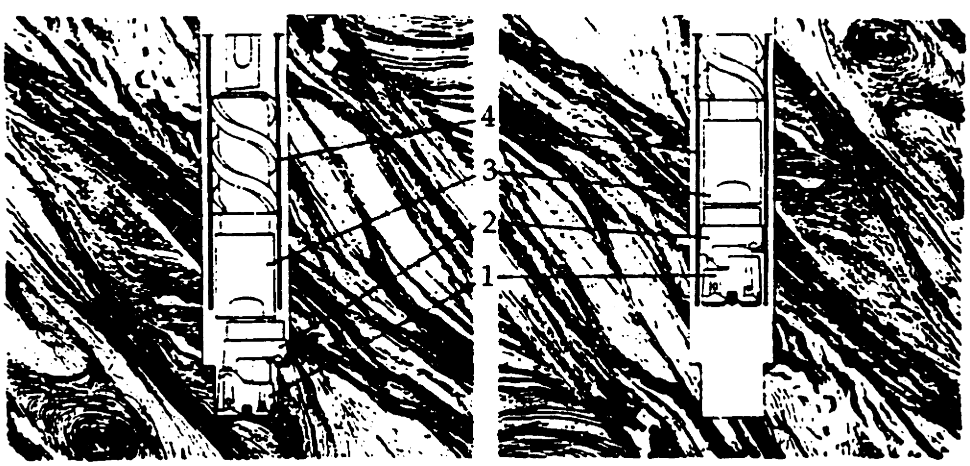


Fig. 3-9 Rotary drilling with down-the-hole hammer (Geo-Fronte Research Association, 1995)





To the left drilling: To the right removal of drilling equipment (1) Pilot bit; (2) Reamer; (3) Guide; (4) Casing tube (*Courtesy*: Atlas Copco).

Fig. 3-10 Odex method (Chugh, 1985)

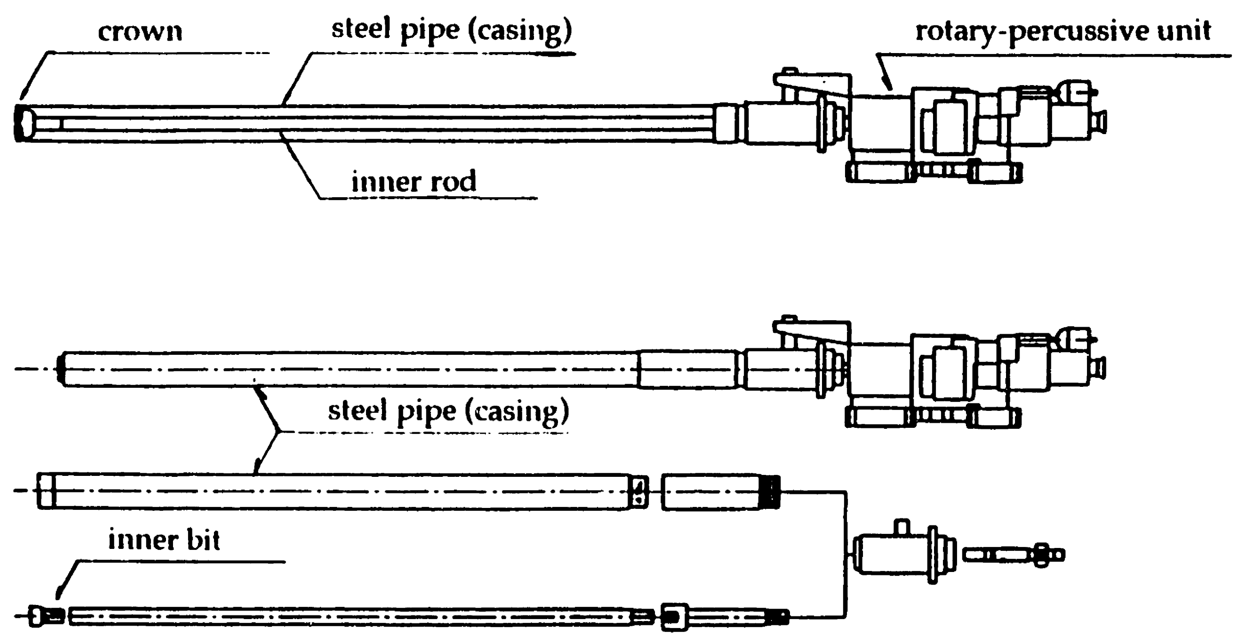


Fig. 3-11 Rotary-percussion drilling with top hammer (Geo-Fronte Research Association, 1995)

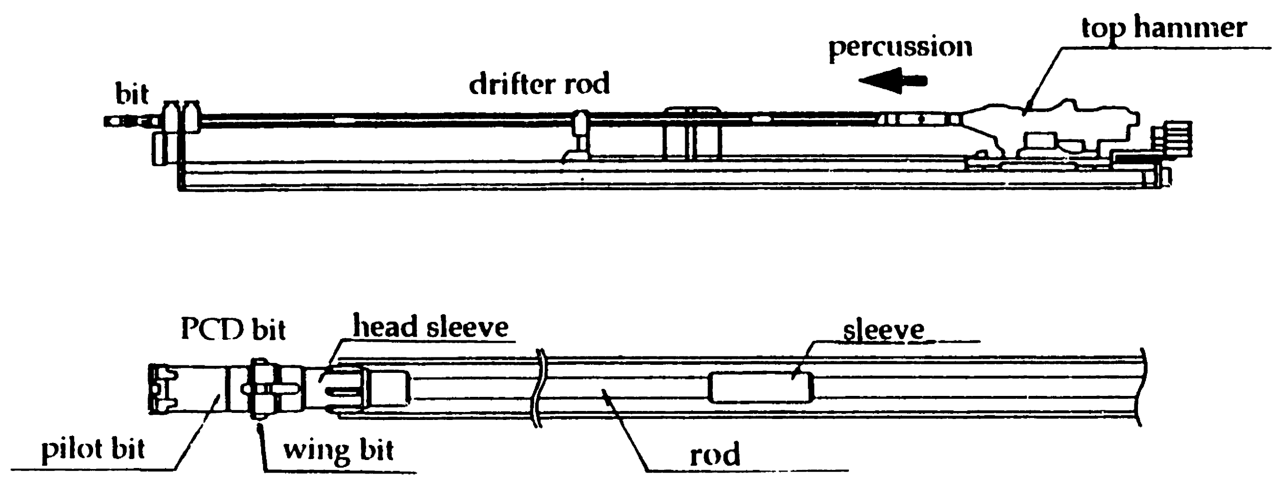


Fig. 3-12 Drifter (Geo-Fronte Research Association, 1995)

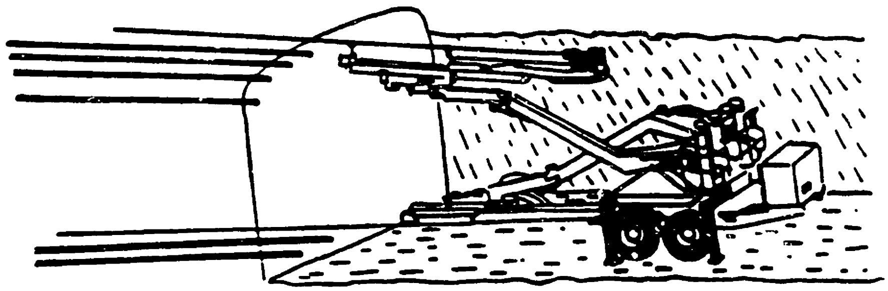


Fig. 3-13 Drill jumbo (Chugh, 1985)

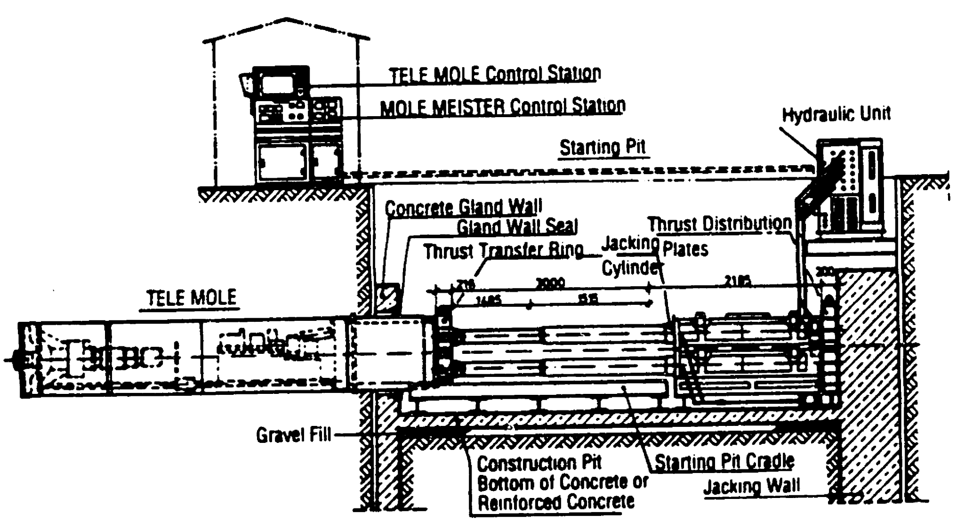
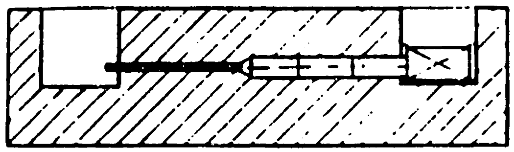


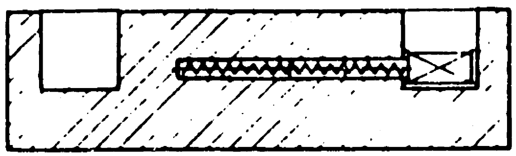
Fig. 3-14 Hydraulic pipe jacking (Stein et al., 1989)

Pilot Boring Method



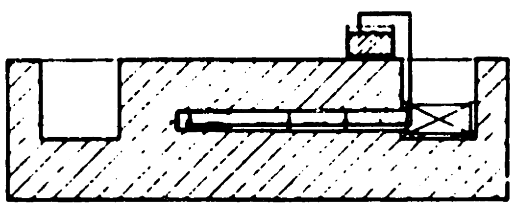
The soil displacement or soil borrow method is used for both the steered jacking / driving of a pilot pipe into the soil, and the subsequent nonsteered jacking / driving of the protective or product pipes, combined with expanding the pilot bore and pushing the pilot pipe into the target pit at the same time.

Jacking and Boring Method



Jacking of protective or product pipes, as well as simultaneous soil excavation at the face by means of a boring head, plus continuous soil removal by a conveyor screw. The boring head and conveyor screw drive unit is positioned in the starting pit.

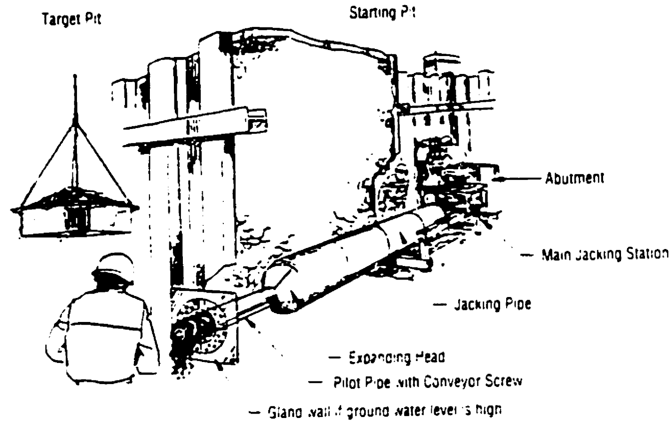
Shield Tunnelling Method



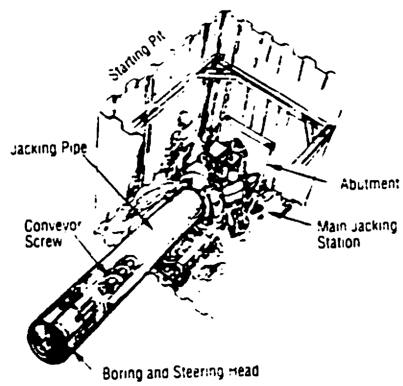
Jacking of protective or product pipes, as well as simultaneous full face soil excavation at the mechanically and hydraulically supported face by means of a boring head, and continuous hydraulic soil removal. The boring head drive unit is accommodated in the shield tunnelling machine.

Fig. 3-15 Major groups of pipe jacking methods (Stein et al., 1989)

(a) Pilot boring method



(b) Jacking and boring method



(c) Shield tunnelling method

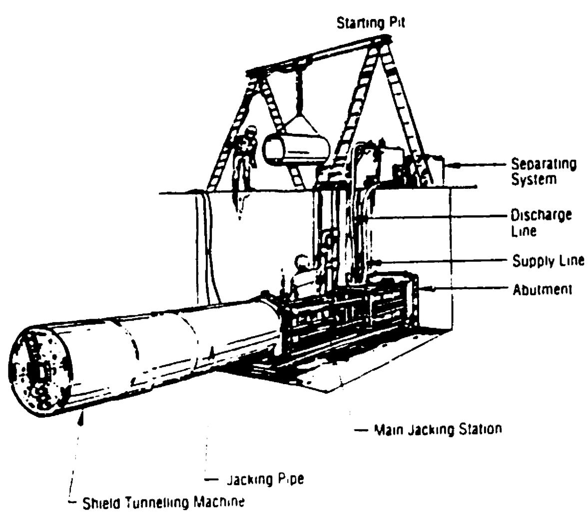


Fig. 3-16 Pipe jacking methods (Stein et al., 1989)

3.5 Grouting Methods

3.5.1 Overview

As shown in Fig. 3-17, several types of grouting are used to modify and/or stabilize in-situ soils in preparation for soft ground tunnelling. Recent improvements in grouting have enhanced its usefulness in both groundwater control and soil stabilization for tunnelling projects. Specifically it is a very effective method for tunnelling in situations such as the following (Bickel et al., 1996):

- to strengthen loose or weak soil and prevent cave-ins due to the disturbance of loose, sensitive, or weak soils by the tunnelling operation
- to decrease permeability and hence groundwater flow
- to reduce the subsidence effects of dewatering or to prevent the loss of fines from the soil
- to stabilize sandy soils that have a tendency to run in a dry state or to flow when below the watertable

In the umbrella method, grouting is mainly used for the purpose of stabilizing the ground around the excavation face.

Permeation grouting (chemical or cement grouting), soil fracture grouting and jet-grouting are methods used in the umbrella method:

Note that the following discussion on each grouting method was copied from "Verification of Geotechnical Grouting" published by the ASCE (1995).

3.5.2 Permeation Grouting

(1) Definition

Permeation grouting occurs when grout fills soil pores or rock fissures without causing significant movement or fracturing of the soil or rock formation. It refers to the replacement of water and/or air in the voids between the soil particles or rock surfaces with a grout fluid at pressures chosen so as to prevent fracturing in the grouted soil mass. Permeation grouting is used as a means for soil stabilization in many engineering projects, where cohesionless soils require that an increase in strength or reduction in permeability be utilized for the intended purpose. It is applicable in coarse to fine sands, depending on the grout mix and soil properties.

(2) Method

The method used in permeation grouting typically requires mixing of the grout components, pumping of the grout to a manifold or injection pipe and injection through an open ended pipe or sleeve port pipe.

Grout may be injected through drill rods, hollow stem augers, or a pipe driven or drilled into the ground. Where driven pipes are used, a knock-out disposable tip is provided to prevent soil from plugging the pipe.

Sleeve pipes are pipes that have ports at intervals with rubber sleeves covering the holes, e.g., Ischy's tube á manchette (see Fig. 3-18). The rubber sleeves prevent the intrusion of soil into the pipe but expand under pressure to permit grout flow. Sleeve pipes are usually placed in bored holes and are grouted into the holes with a weak grout. Inflatable packers are used to isolate an individual injection port during grouting and/or water testing. The sleeve pipes can be washed out after grouting and reused for multi-level injections or multiple injections at a single port.

The permeation grout is a fluid of sufficiently low viscosity to permeate the pores of a soil or rock matrix. The fluid grout must be injected at relatively low pressure to prevent hydraulic fracturing of the matrix and undesirable heaving of the ground surface.

(3) Design considerations

Based on the material components, permeation grouts can be divided into two categories: chemical grouts and cementitious grouts.

Chemical grouts are primarily used for stabilizing granular materials and for treating finely fissured rock. The objective may be to provide increased strength or to retard water seepage. For sandy soils, chemical grout may be used to convert a cohesionless soil into a cohesive or a nearly impervious material. Grouting with chemicals involves the filling of voids in soil or rock with a fluid chemical which sets or cures to alter the properties of the geologic mass.

Selection of the particular chemical grout is of major importance to the performance of the system. Significant items for consideration include:

- nature of the application; i.e., size of voids or fractures and types of substrate
- potential handling problems including flammability
- chemical resistance, reactivity and permanence of the grout due to environmental factors
- toxicity or other environmental issues
- viscosity and flow characteristics of the grout
- set time and curing characteristics of the grout
- cost

One of the advantages which may accrue from chemical grouts is the broad range of properties potentially available:

- strengths vary from low strength soft gels or foams to high strength
- viscosities at injection time vary from nearly water to thick oil
- stability to acids, bases or organics can vary from zero to infinite
- flexibility of the grout can vary from soft sponge-like to rock-like brittle behavior
- adhesive properties vary from very low to extremely high
- water or moisture effects on curing and other physical properties are broad

Cementitious grouts are composed of cement and water, and often contain additives such as clay, bentonite, sodium silicate, dispersants, retarders and accelerators. Most cement grouting operations are completed with ordinary Portland cement. Bleeding of cement grout can be controlled with additives such as bentonite.

The effectiveness of grouting with a particulate grout depends mainly on the following factors:

- size of the grout particle with respect to the soil pore diameter or fissure width
- grout viscosity
- grout stability
- injection pressure
- pumping rate

The main considerations of slurry grouting to seal cracks and fissures in rock, or to be injected into soils for either water control or structural improvement purposes is the grain size of the particulate grout compared to the width of the rock fissure or pore size of the soil to be grouted. Successful grouting depends on the careful selection of the grout which is most compatible with the geologic mass being treated together with injection techniques.

3.5.3 Soil Fracture Grouting

(1) Definition

Fracture grouting is the intentional fracturing of soils using grout pressures high enough to fracture the soil at the point of injection. The basic concept is to use a grout material that will not permeate the soil. Portland cement grouts are most frequently used but chemical grouts are occasionally used.

(2) Method

In fracture grouting, a stable fluid is injected under high pressure in order to control fracturing of the ground. Repeated injections after periods of curing tend to densify the adjacent ground, decrease the local permeability, stiffen and strengthen the soil due to the hardened grout lenses, and provide the capability to support footings or maintain existing elevations during an adjacent excavation.

Because the process requires that the soils be fractured and not necessarily permeated, soil fracture grouting may be used in most soil types ranging from weak rocks to clays. However, because of a rather broad range of compaction and permeation techniques for coarse grained soils, soil fracture grouting has found a particular niche in treating cohesive soils.

The injection of grout to produce lenses is performed both with open-ended pipes and with sleeve-port pipes. In the latter case, a system of injection pipes is introduced into soil in such a way that openings spaced at distances ranging from 300mm to 1000mm along the length of the pipes, are uniformly distributed in the soil layer (see Fig. 3-19). A packer capable of creating a tight seal within the grout pipe, thus isolating one zone for injection, is pushed to the selected sleeve port. To achieve uniform stabilization, injection continues in those zones in which the recorded injection pressures do not reach the required level during the first or second injection stage. In loose soils, fracturing will not occur until the initial injections densify and stiffen the zone. Prevention of soil heave has to be ensured by monitoring ground surface at regular intervals.

(3) Design considerations

Soil fracture grouting is used to increase the shear strength and resultant bearing capacity resistance of soils as well as to raise structures. To date, the most extensive applications of the process have been to remediate against settlement caused by soft ground tunnelling under structures.

Ground improvement by soil fracturing is based on three mechanisms:

- The soil unit or skeleton is reinforced by a series of hard grout lenses which extend from the injection point to form a matrix of hard grout and soil.
- The fluid grout finds and fills voids and causes some compaction in more coarse grained soils along the grout lenses created.
- The plasticity index of saturated clays decreases through the exchange of calcium ions originating from cement or other fillers.

Fracture tightening of rock is accomplished through the use of grout pressure which moves the rock within the formation by enlarging the fissures being grouted and thereby reducing the size of the fissures not being grouted. This method strengthens the formation structure and/or reduces the permeability of the formation. The technique is particularly valuable when using an unstable grout (high bleed) because, when the grout pressure is released, the rock pressure will squeeze the bleed water into the fissures too fine for the grout.

(4) Properties

Cement based grout mixes are most commonly used for fracture grouting and must be designed to minimize excess bleeding of water and hence shrinkage.

The constituent proportions of a cementitious grout mix will vary greatly depending on the application. For instance, for the compensation of settlement due to tunnelling, the grout need only be as strong as the existing soil. However, in weak soils where an improved bearing capacity is required, added grout strength is more useful.

3.5.4 Jet Grouting

(1) Definition

Jet grouting is a means of hydraulic cutting and mixing of in-situ soil materials with a fluid grout injected at high velocity to create a stabilized mixture of soil and grout.

(2) Method

Jet grouting creates a stabilized mixture of soil and grout by cutting the in-situ soil materials with a fluid mixture, which is injected at a high velocity and at pressures on the order of 35 to 40MPa.

The technique uses a drilling system to gain access to the soils requiring modification and is most often designed as a series of interconnected columns of various lengths and geometry to create a mass of mixed soil and grout. The method is applicable to a wide range of soil types as shown in Fig. 3-20.

(3) Design considerations

The strength of the soil and grout mixture is vital in underpinning and tunnelling applications. Specified compressive strength should be based on analysis, since specifying an unnecessarily high compressive strength will greatly increase costs.

Relatively small amounts of organic soils will significantly reduce strength. To achieve high strengths in organic soils, jet grouting must flush out most of the organics.

(4) Properties

The properties of jet grouting columns depend on many factors, including the system used, the jet-grouting parameters, water table location, curing time, and most importantly, soil characteristics. Depending on the application, relevant properties to specify jet grouting include dimensions, location, strength and permeability. Design compressive strengths in the range of 2 to 10MPa are easily obtainable in most inorganic soils.

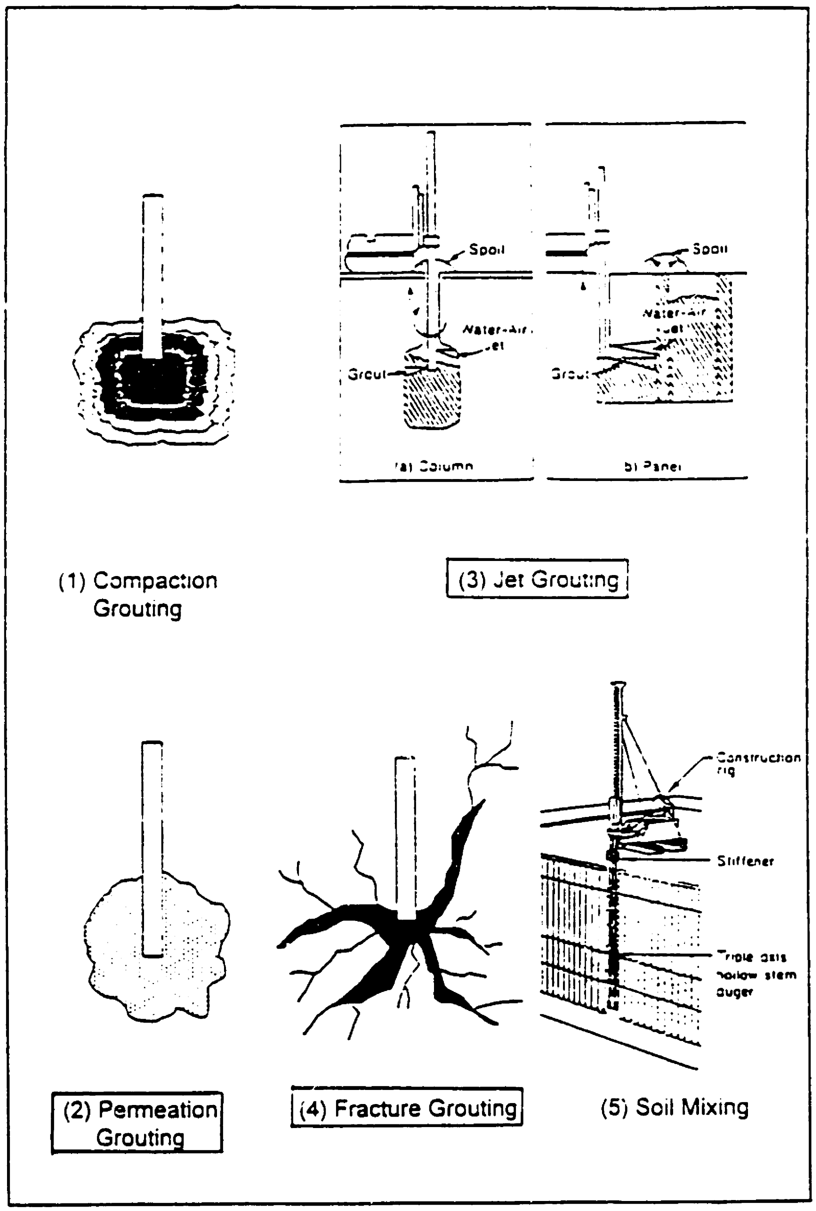


Fig. 3-17 Schematic representation of grouting methods (ASCE, 1995)

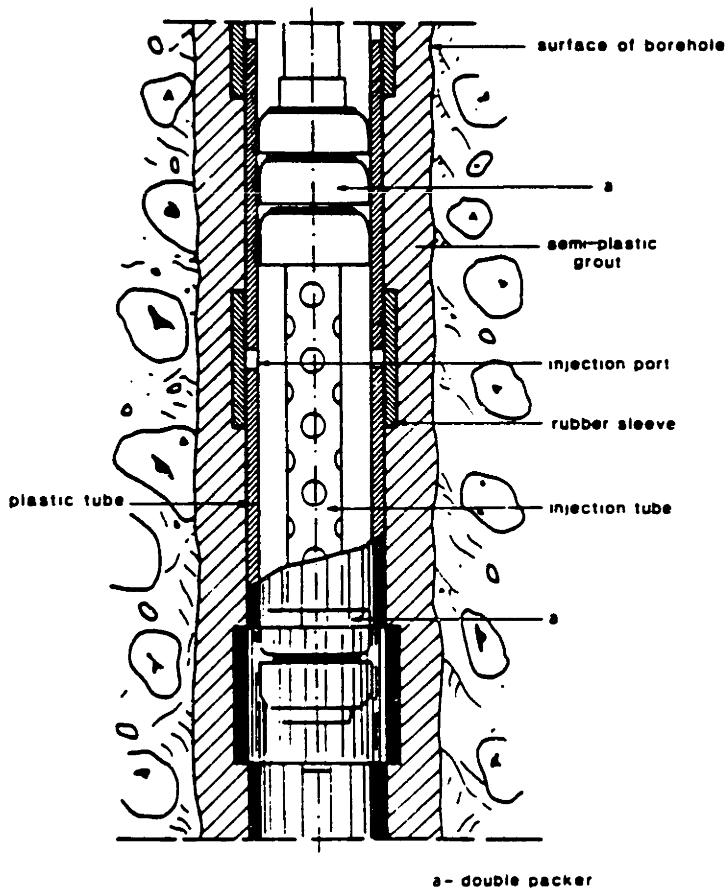


Fig. 3-18 Ischy's tube á manchette (Moseley, 1993)

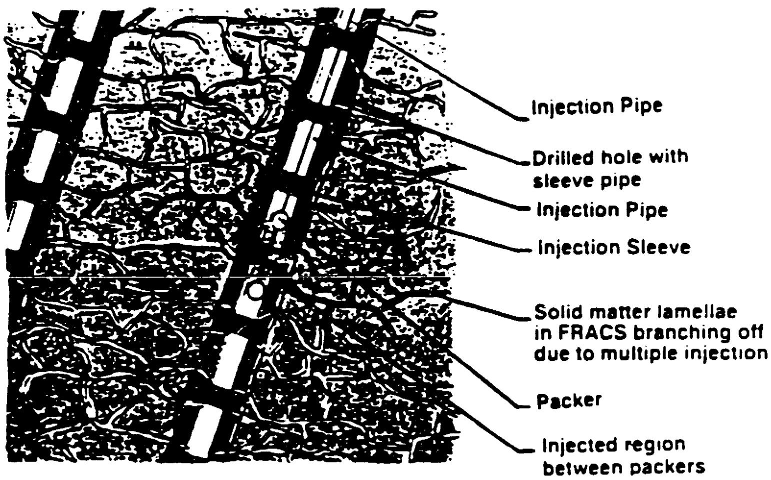


Fig. 3-19 Soil fracturing (Moseley, 1993)

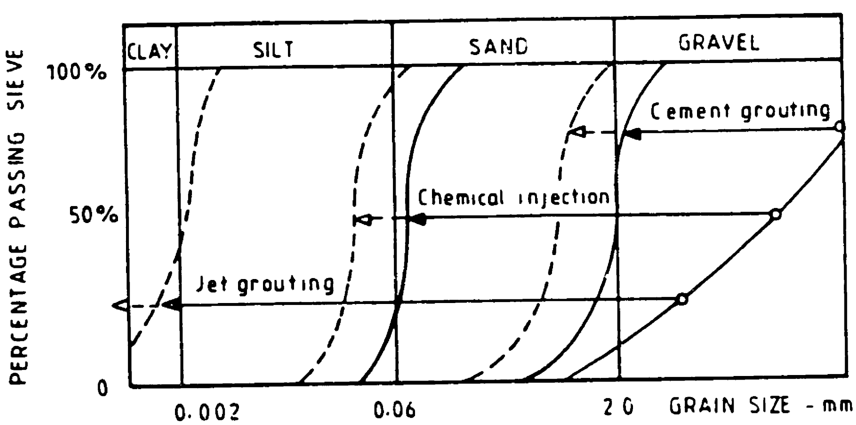


Fig. 3-20 Range of soil types applicable for jet grouting (Moseley, 1993)

3.6 Sub-horizontal Jet Grouting Method

3.6.1 Basic Principles

The sub-horizontal jet-grouting method, shown in Fig. 3-21, has been advanced in Italy in recent years and used to solve extremely difficult problems in tunnelling, e.g., the combination of both poor ground conditions and a shallow overburden above a tunnel crown.

To date, the two sub-horizontal jet-grouting methods exist, the Rodin Jet Fore-Poling method (Rodio SpA, Italy) and the TREVIJET method (Trevi SpA, Italy). Each of them uses a special patented machine for drilling and grouting; however, the basic concept and design philosophy seem to be identical. Of these methods, the Rodin Jet Fore-Poling method (RJFP) has been most widely used as a supplementary method in tunnelling.

Construction sequences of the sub-horizontal jet-grouting method are shown in Fig. 3-22. Once the arch is formed, tunnel excavation advances to a certain distance from the end of the treated length, then drilling and jetting of a new arch commences. Excavation in each section is immediately followed by the installation of tunnel supports.

The sub-horizontal jet-grouting method is carried out within a tunnel, hence, it is suitable in cases where access from the surface is difficult as a result of the presence of roads, railways, or buildings.

3.6.2 Design Considerations

(1) Applicable soil types

The main structural component of the sub-horizontal jet-grouting method is the reinforced soil column created by jet grouting. As shown in Fig. 3-20, the jet grouting method is applicable in a wide range of soils. Applications in various types of soils are discussed below.

According to Moseley (1993), sands, in general, are best suited for jet grouting, since erosion of individual particles takes little energy due to the lack of cohesion. Also, the escape of the extraneous slurry to the surface is assisted by the flowability and lack of cohesion of the ground.

Gravelly soils are also usually amenable to treatment. However, highly permeable, poorly graded gravels may lead to loss of grout and injected fluids, thus reducing the effectiveness of the intended treatment.

In cohesive soils jet-grouting is affected by the erosive efficiency which in turn is influenced by even small amounts of cohesion in soils. In relatively insensitive cohesive soils the diameter of the developed column is dependent on soil strength, specifically the higher the shear strength the smaller the volumes of affected ground. Depending on the application, soils with higher shear strength might have to be treated with abnormally high energy inputs.

The Geo-Fronte Research Association (1994) recommends the suitable grounds for the RJFP method in terms of cost and execution, as shown in Table 3-3.

Table 3-3 Suitable ground for the RJFP method

Soil type	SPT-N
Cohesive soil	N ≤ 8
Cohesionless soil (including gravel)	N ≤30

Note: 1) SPT-N: number of blows in standard penetration test (SPT)
Source: Geo-Fronte Research Association (1994)

It should be noted that Geo-Fronte Research Association concludes that the RJFP is a suitable method for relatively soft ground.

(2) Jet-grouting parameters

Diameter and properties of the jet-grouted column depend on the soil, grout mix constituents and composition, pressure and working procedure. Thus, if site conditions allow, each jet-grouting parameter should be selected by testing programs conducted at the specific construction site. The jet-grouting parameters can then be varied, if necessary to achieve the required size or shape.

On the basis of experience, the Geo-Fronte Research Association (1994) recommends the standard jet-grouting parameters shown in Table 3-4.

Table 3-4 Jet-grouting parameters

Parameter	Amount
Nozzle size (mm)	2 nozzles with 1.8/2.2 mm diameter
Grouting pressure (MPa)	40
Rotation speed of drilling rod (rpm)	18 - 22
Grout volume (liter/m)	310 - 430
Withdrawal speed (min. /m)	3 - 6

Source: Geo-Fronte Research Association (1994)

(3) Engineering properties

The strength and/or deformability of the soil and grout mixture is crucial in tunnelling applications. Engineering properties of the treated ground, such as unconfined compressive strength and modulus of elasticity, mainly depend on a number of interdependent characteristics of the soil and of the jet grouting procedure.

Table 3-5 shows the engineering properties of jet-grouted columns based on the results of a testing program carried out by the Rodio SpA.

Table 3-5 Engineering properties of jet-grouted columns

Soil type	Unconfined compressive strength: R (MPa)	Modulus of elasticity: E (MPa)	Ratio: E/R
Clayey silt	0.3 - 0.5	60 - 450	200 - 900
Sandy silt	1.5 - 5.0	500 - 2000	333 - 400
Silty sand	5.0 - 10.0	2000 - 5000	400 - 500
Gravelly sand	5.0 - 15.0	3000 - 10000	600 - 666
Sandy gravel	5.0 - 20.0	4000 - 20000	800 - 1000

Source: Mussger et al. (1986)

From the table, it can be seen that the strength of a jet-grouted column in clayey silt is much lower than that in sand or gravel. The reason for this may be that in sand or gravel a considerable amount of water is drained both from soil and grout, whereas in a cohesive soil of low permeability, poor or no drainage is likely to occur.

Fig. 3-23 shows the relationship between secant modulus and unconfined compressive strength of jet-grouted soils. From the figure, it can be seen that the modulus/strength ratio (E/R) is around 900 on average for the gravel and sand, dropping to less than 400 for medium to fine sand.

3.6.3 Practical Considerations

(1) Drilling method

As mentioned previously, jet-grouting, in general, is applicable in a wide range of soil types; however, the RJFP method may not be suitable in all ground types because of the drilling system.

For drilling, the RJFP method can use two different types of systems, the rotary drilling system or the rotary-percussion drilling system (Mongilardi et al., 1986). As shown in Table 3-2, the rotary drilling is preferred in medium to fine grained soils, where fairly small rigs are required. In coarse grained soils or any soil including cobbles and boulders, the rotary-percussion drilling may be better suited in terms of drilling speed, since in such soils the drilling speed of the rotary drilling dramatically decreases. However, the rotary-percussion drilling requires a heavier rig of a height as great as the longest rod to be used.

It should be noted that the mode of drilling is selected according to soil conditions, general features of the site and design specifications in regard to length and inclinations of holes.

(2) Grout

The grout mix constituents and composition for jet-grouted columns can be varied to meet the specific requirements for soil improvement, with quite different and far less restrictive criteria in comparison with permeation grouting (Mongilardi et al., 1986). In general, the grout consists of binary water-cement suspensions in a ratio of 1:1.1 to 1:1.25 with, if necessary, the inclusion of additives according to the Geo-Fronte Research Association (1994) which gives the standard grout composition as follows:

Composition per cubic meter of a mix with a water to cement ratio of 1:1

- water: 750liters
- cement: 760kg (Puzzolan cement)
- additive: 12kg

It should be emphasized that preliminary tests at the planned construction site might be necessary to determine the exact composition of the grout.

(3) Heaving of the ground surface

In a case of a shallow overburden above the tunnel crown, the sub-horizontal jet-grouting method may cause heaving of the ground surface due to the high injection pressures. Therefore, in cases where the overburden is not deep enough to provide confinement to the high injection pressure, the method is not recommended. The possibility of heaving is considered to be one of the disadvantages of the sub-horizontal jet-grouting method.

Figure 3-24 indicates the relationship between the amount of heaving observed during development of jet-grouted columns and the depth of overburden, where vertical and horizontal axes indicate the amount of heaving at the ground surface and the depth of overburden above the tunnel crown, respectively. For all tunnel sites shown in the figure, the adopted method was the RJFP. The data may be insufficient to draw generally valid conclusions, but one can predict from the figure that heaving of about 10mm may occur at sites with a depth of overburden smaller than 10m.

3.7 Injected Steel Pipe Umbrella Method

3.7.1 Basic Principles

Similar to the sub-horizontal jet-grouting method, the injected steel pipe umbrella method was originally developed in Italy during the 1980's, at which time it came into wide use in tunnel construction in which poor and/or settlement-prone ground conditions exist. The steel pipe umbrella method (Infilaggi method) was first utilized in tunnelling (see Fig. 3-25). Depending on the spacing between inserted steel pipes, the Infilaggi method, in general, supports the ground load only in the longitudinal direction, therefore, the arching effect does not occur in the transverse direction. Because of this, the method may be used only under relatively stable ground conditions.

In contrast, the stiffness of the injected steel pipe umbrella method, as shown in Fig. 3-26, consists of three components:

- driven steel pipe
- grout fillings inside the pipes and in the voids surrounding the pipes
- treated ground by permeation or fracture grouting

A series of upward-inclined, about 100mm diameter, 12 to 15m long holes are drilled and perforated steel pipes are inserted in the holes and then grout is injected into the pipes. In contrast to the Infilaggi method, an arch-like reinforced zone can support the ground load in the transverse direction as well, therefore the method may be used in poorer ground conditions and/or shallower overburden than those in which the Infilaggi method is used.

To date, the injected steel pipe umbrella methods are the TREVITUB (Trevi SpA, Italy), the RODINTUB (Rodio SpA, Italy) and the AGF (Japan). Differences among these methods is that the former two use a special patented machine to drill a borehole, insert a steel pipe and inject a grout mix, while the latter uses a conventional "drill jumbo" for the same purpose. Of these methods, the TREVITUB method is discussed in this thesis.

Construction sequences of the TREVITUB method are shown in Fig. 3-27.

Similar to the sub-horizontal jet-grouting method, the injected steel pipe method is applied from within the tunnel.

3.7.2 Design Considerations

(1) Applicable soil types

The ground in which the TREVITUB method can be used is strongly related to the drilling method. Basically, in the TREVITUB method, drilling can be performed with various methods such as conventional rotary drilling, rotary-percussion with top hammer and down-the-hole hammer with an Odex bit. Hence, all ground types can be handled by changing the drilling method. See Table 3-2.

(2) Applicable steel pipes

The steel pipe used in the injected steel pipe umbrella method differs in diameter, thickness and length according to the drilling method used. In addition, if the drilling method has to be changed because of the ground conditions, the thickness of steel pipes, the connection between pipes and the stiffness of the connected pipes in the longitudinal direction have to be considered in advance. Therefore, understanding the relationship between the drilling method and the applicable steel pipe is important in selecting the most suitable umbrella method under any given site conditions.

Table 3-6 shows the applicable steel pipes on the basis of the drilling method to be used (Geo-Fronte Research Association, 1995).

Table 3-6 Applicable steel pipes for drilling methods

Drilling method	OD = 101.6mm (4")	OD = 114.3 mm (4.5")			OD = 139.8 mm (5.5")		Length of a steel pipe section (m)
	t=6.0mm	t=6.0mm	t=8.6mm	t=11.1mm	t=6.0mm	t=6.6mm	
Down-the-hole hammer	C	A	B	C	A	B	6 to 13 m (if a special machine used)
Top hammer	C	A	A	A	A	A	6 to 13 m (if a special machine used)
Drill jumbo	A	A	C	C	C	C	2.5 to 3.5 m
Double-rotary	C	A	A	A	A	A	6 to 13 m (if a special machine used)

Note: 1) OD: outer diameter of steel pipe, t: wall thickness of steel pipe
2)Code: A = Applicable; B = May be applicable; C = Not applicable

Source: Geo-Fronte Research Association (1995)

3.7.3 Practical Considerations

(1) Injection (Trevi SpA, 1993)

Injection is carried out through the installed casing tube, which has perforated holes and is provided with outer valves of the "manchette" type or special "springval" type (see Fig. 3-28).

The injection of stabilizing mixes into the soil surrounding the fore-poles is an operation whose function and importance differ depending on the soils involved:

- In the case of cohesionless uniform soils the grouting is of vital importance as it has to prevent the caving of soil between adjacent fore-poles.
- In the case of morainic soils with large blocks and boulders embedded in a more or less closed matrix, the purpose of the grout is to fill possible voids in the matrix, since the soil is generally self-supporting between one fore pole and another.
- In the case of a mass with certain or apparent cohesion, soil grouting may not even be required and, if necessary, may be limited simply to filling the pipes in order to increase their shear resistance, especially around the threaded joints. This case might be considered to be almost the same as the pipe roof method (see Ch. 3.8).

When using rotary drilling with the down-the-hole hammer, the valves can be the same as those used with manchette valve tubes. A non-return valve fitted on the outer surface of the reinforced tube does not present any problems since the tube is merely pushed into the drilled hole.

On the contrary, with the Odex method (see Fig. 3-10), the valves must not protrude beyond the thickness of the tube in order to avoid being damaged during drilling. For this purpose Trevi SpA developed a special valve, called a "springval", consisting of a steel disk, seated in a gap machined in the tube and covering the openings to inject through (see Fig. 3-28).

There are basically two grouting techniques used: the single valve injection technique and the total valve injection technique.

Single valve injection:

In the single valve or one-stage injection technique each individual valve is pressured using a double packer placed astride each valve (see Fig. 3-29 (a)).

This technique is generally used for the low pressure injection of manchette pipes.

Total valve injection:

The total injection technique for each individual fore-pole uses a packer placed at the top of the pipe (see Fig. 3-29 (b)).

In order to carry out this type of injection a threaded plug screwed to the top of the pipe is used. The plug is fitted with an injection valve and with a discharge valve to which two permanent PVC pipes are attached.

(2) Grout

In general, the grout consists of binary water-cement suspensions with a ratio of 1:1.2 to 1:1.5 to which additives may be added. Trevi SpA gives the standard mix compositions with a mix of water to cement ratio of 1:1.2 (Trevi SpA, 1993).

- water: 715.00liters
- cement: 858.00kg (Puzzolan cement)
- additive: 12.00kg

Should it be necessary to achieve less bleeding, suitable additives like bentonite must be used, in this case the composition per cubic meter might be:

- water: 714.00liter
- cement: 857.00kg (Puzzolan cement)

- Bentonite: 2.00kg
- additive: 0.35kg

Depending on the specific products available, it will be necessary to conduct preliminary laboratory tests to determine the exact compositions and the physico-chemical characteristics of the mixes produced.

3.8 Pipe Roof Method

3.8.1 Basic Principles

A pipe roof is formed in the crown of a tunnel by installing a series of relatively large diameter steel or concrete pipes (40 - 80cm in diameter) in an arch or ring, as shown in Fig. 3-6. The pipes are jacked or augered into place and then filled with grout or concrete.

This method is used in Europe and especially in Japan under conditions with a shallow overburden above the tunnel crown and/or with weak soils in urban areas.

In Italy, a method similar to the pipe roof method was developed to construct the underground Venezia railroad station in Milan which is 22.8-m wide under only 4-m-loose overburden (Lunardi, 1990). In general, this method, the so called the Cellular Arch method (see Fig. 3-30), may have the same design philosophy as that of the conventional pipe roof method; however, in terms of the dimensions of the fore-poles used or regarding construction procedures, the method differs from the conventional pipe roof method. For example, the fore-poles used in the Venezia Station in Milan were reinforced concrete pipes with an outer diameter of 2.1 m; in addition they were connected to each other with reinforced concrete arches to form the vault. A detailed description of the Cellular Arch method will be given in the case study on the Venezia Station (see Ch. 4.2).

In contrast to the previously described sub-horizontal jet-grouting method (see Ch. 3.6) and the injected steel pipe umbrella method (see Ch. 3.7), the fore-poles in the pipe roof method, in general, are driven from a construction pit prepared for this purpose, because the space for driving the fore-poles is usually insufficient within a tunnel. For this reason, as shown in Fig. 3-31, the driven piles can be aligned parallel to a tunnel axis.

3.8.2 Design Considerations

(1) Applicable soil types

The ground which can be handled by the pipe roof method depends to a large extent on the drilling method. In general, drilling methods to be used with the pipe roof method are:

- horizontal boring method
- pipe jacking (microtunnelling) method

The principles of the horizontal boring method are basically the same as those in the rotary drilling method (see Ch. 3.4), that is, a pipe is put into ground by either rotating it with a cutter bit mounted in it or excavating and removing soil by augering inside a pipe. As shown in Table 3-2, medium to fine grained soils are preferred when using this drilling method. As mentioned previously, the pipe jacking method is mainly used in soils with SPT-N values of $5 < N < 50$. It can be concluded from these considerations that the pipe roof method is feasible in almost all soil types with the exception of very stiff and dense soil and rock.

(2) Analytical model for designing a pipe section

In order to be on the safe side, it is generally assumed in the pipe roof method that the pipes support the ground load only in the longitudinal direction. For practical use, the analytical model shown in Fig. 3-32 is, for example, used to determine a pile section (Saito, 1982). As we can see in the figure, one assumes that one pipe end is fixed and supported by the ground while the other hinged end is supported by a tunnel support (H or I-beam).

It is obvious that this simple analytical model cannot provide detailed information on ground deformation. In a case in which ground settlement is a primary concern, three-dimensional (3-D) finite element (FE) method has to be used. Also, in a case in which pipes are interlocked or the spacing between the pipes is so close that the earth pressure is transferred three-dimensionally to an arch-like structure, it is necessary to conduct a 3-D FE method.

3.8.3 Practical Considerations

(1) Drilling method

In general, a drilling machine for horizontal boring is employed to drive a small diameter steel pipe, say 10 to 20 cm in diameter. In contrast, in the pipe jacking method, a pipe of a relatively large diameter is jacked. For example, the shield tunnelling method was successfully employed in the aforementioned Venezia Station case to insert concrete tubes with 2.1 m in outer diameter.

Depending on the type of soil and the outer diameter of the pipe, when jacking to a maximum distance of 100 m (mean distance 70 m), tunnelling machines operated either by the soil displacement or soil excavation principle are used. In a case of jacking distances in excess of 100 m, shield tunnelling machines (micro-TBM) which may include hydraulic soil removal are used (Stein et al., 1989).

A major concern when using the pipe roof method is the directional accuracy of pipe placement. In general, when driving a relatively small diameter pipe over long distances with a conventional horizontal boring machine, deviation is likely to occur due to flexibility along the length of the pipe. Moreover, drilling is non-steerable. In contrast, in the pipe jacking method, steering is possible. For example, the steering system shown in Fig. 3-33 is employed in remotely controlled pipe jacking to change the direction of the tunnelling machine (Stein et al., 1989).

(2) Construction pit for driving a pipe

In the pipe roof method, driving a pipe is generally carried out from a construction pit prepared outside a tunnel portal. Depending on site conditions and/or the drilling method used, it is evident that larger diameter pipes require larger pits. The pit in which the jacking or boring machines are located is called the "starting pit." In pipe jacking with microtunnelling, a "target pit," in which the tunnelling machine is recovered, is usually needed.

Shapes and dimensions of a construction pit should be so designed that construction costs are reduced to a minimum. For instance, the pit length has to take into account the following (Stein et al., 1989):

- length of the installed main jacking station
- mode of operation of the jacking cylinders
- length of the starting pit cradle resulting from the length of jacking pipes and guide ring
- working space required to carry out the pipe coupling operations
- thickness of the abutment

The pit width is governed by the width of the main jacking station, as well as the requirements for clear working space on the pit bottom (Stein et al., 1989). When not enough space for the construction pit is available at the planned site, which is likely in urban areas, the pipe roof method may not be feasible.

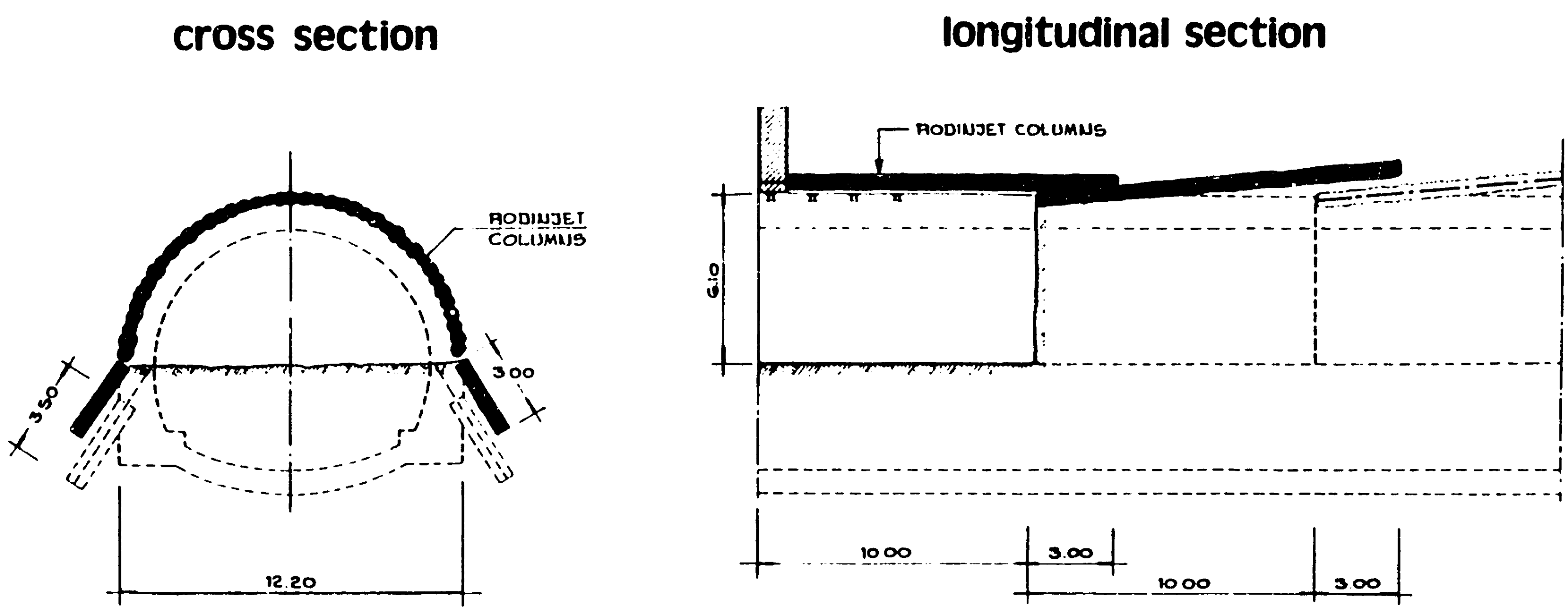
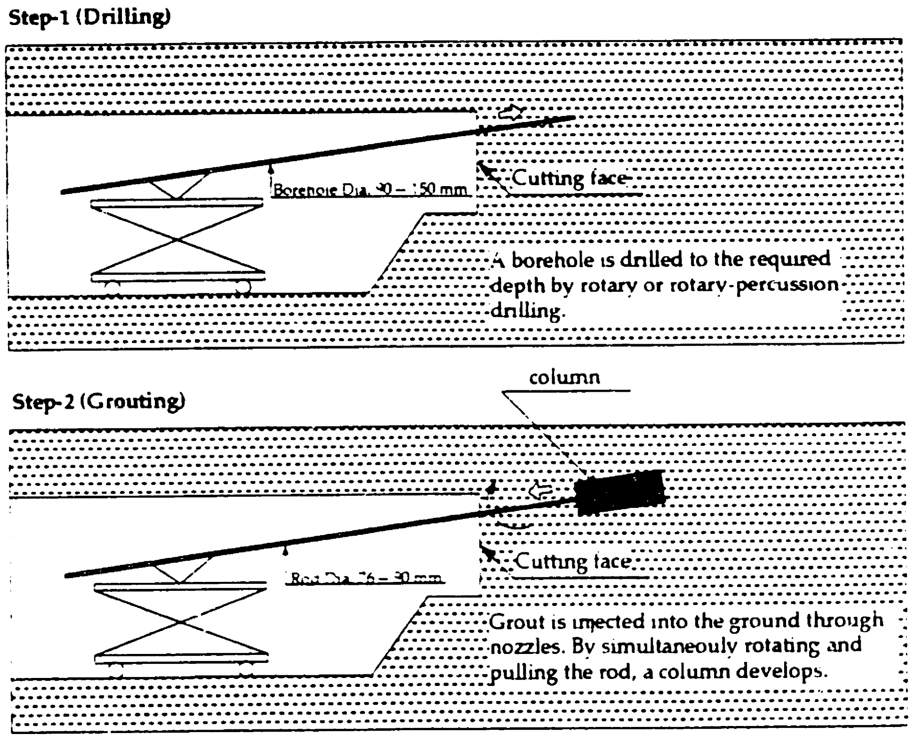


Fig. 3-21 Example of the sub-horizontal jet-grouting method (Tornaghi et al., 1985)

(a) Development of jet-grouted column (Geo-Fronte Research Association, 1994)



(b) Jet-grouting and tunnel excavation stage (Moseley, 1993)

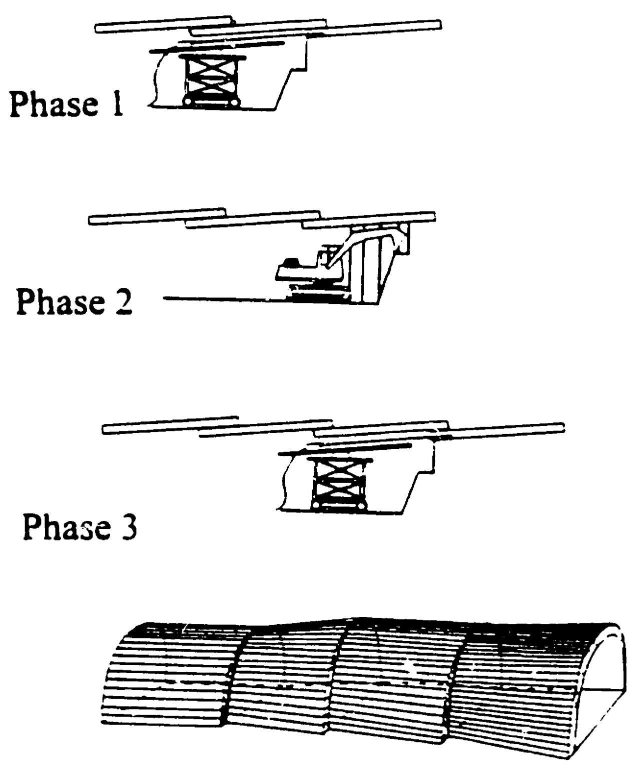


Fig. 3-22 Construction sequences of the sub-horizontal jet-grouting method

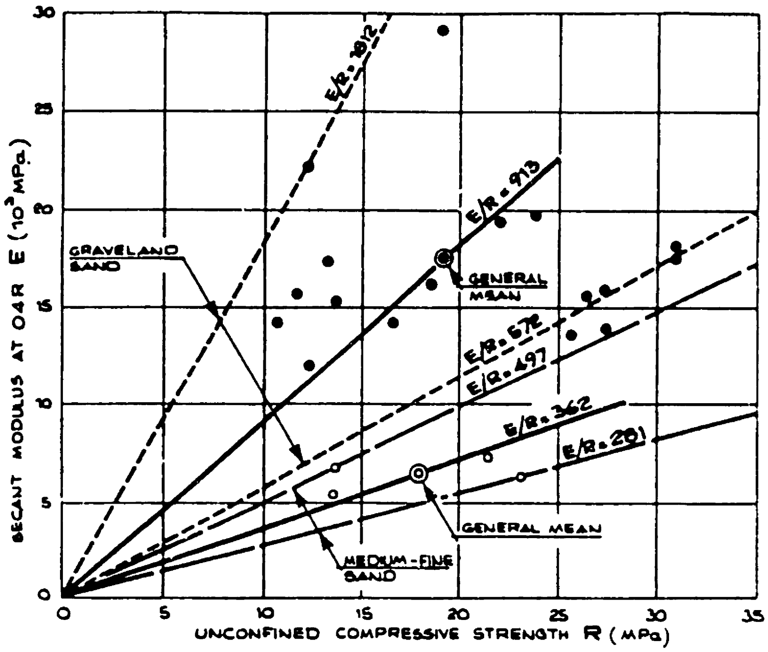


Fig. 3-23 Plots of secant modulus versus strength of jet-grouted soils (Mongilardi et al., 1986)

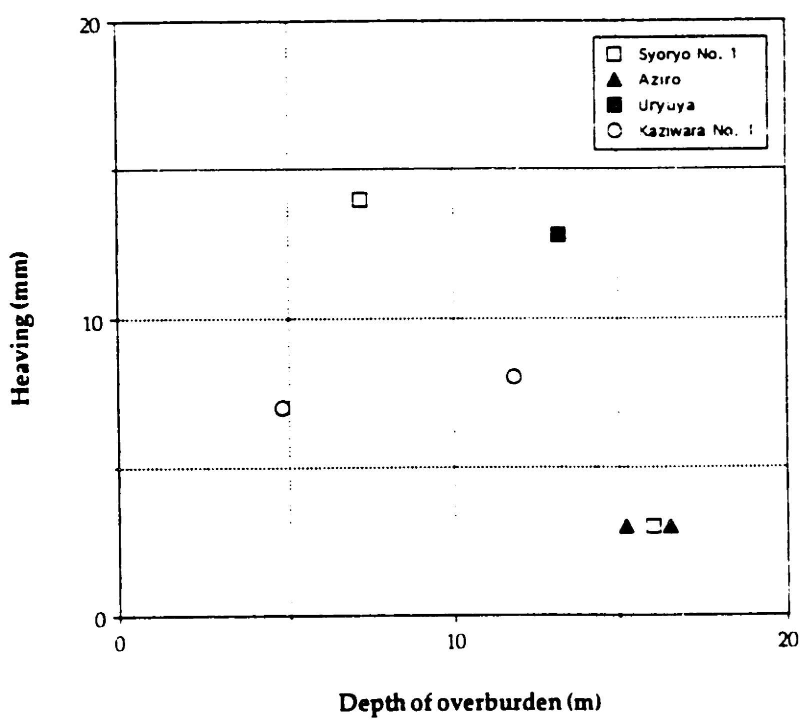


Fig. 3-24 Heaving versus depth of overburden (Geo-Fronte Research Association, 1994)

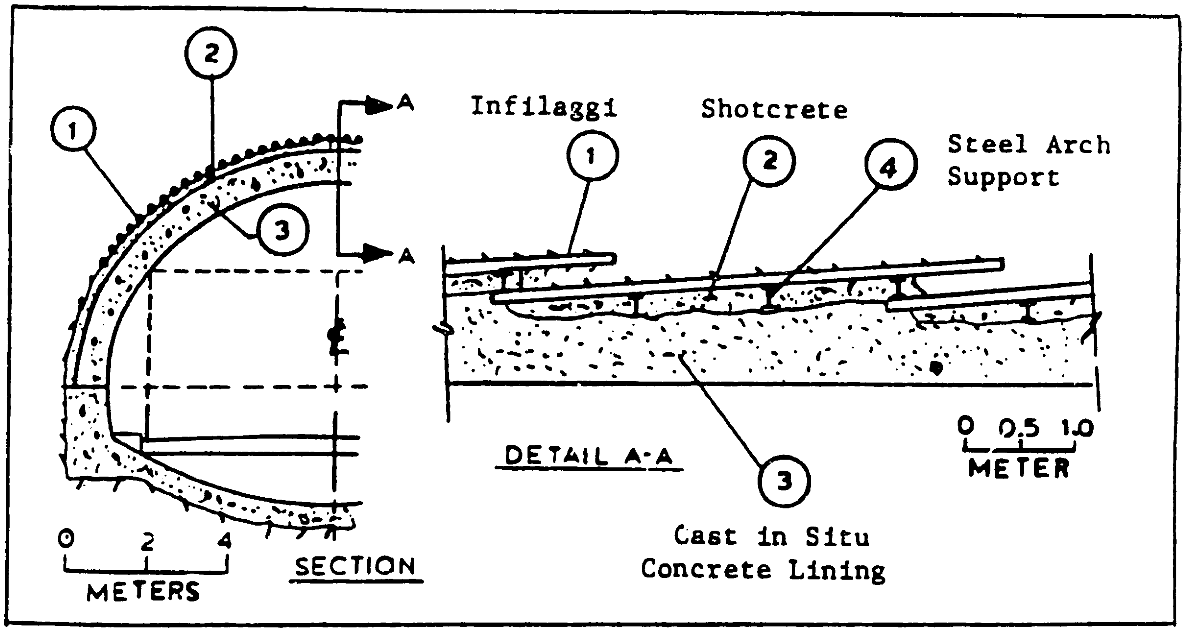


Fig. 3-25 Infilaggi method (Bruce et al., 1987)

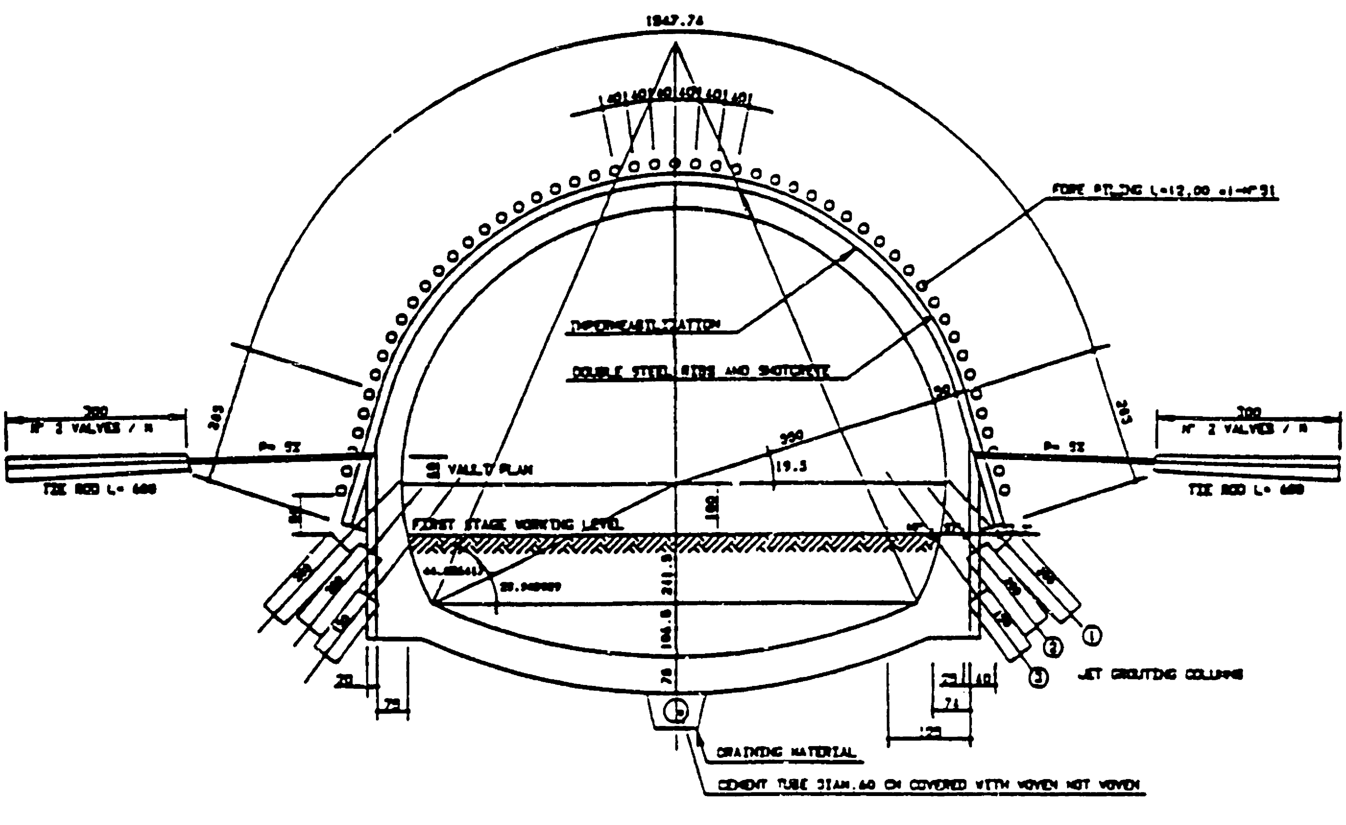
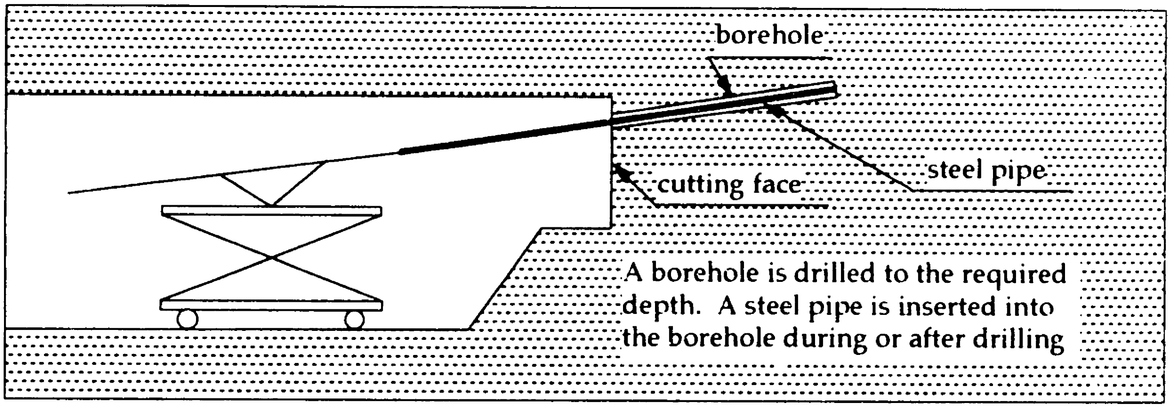


Fig. 3-26 Example of the injected steel pipe umbrella method (Trevi SpA, 1993)

Step-1 (Drilling and inserting a steel pipe)



Step-2 (Grouting)

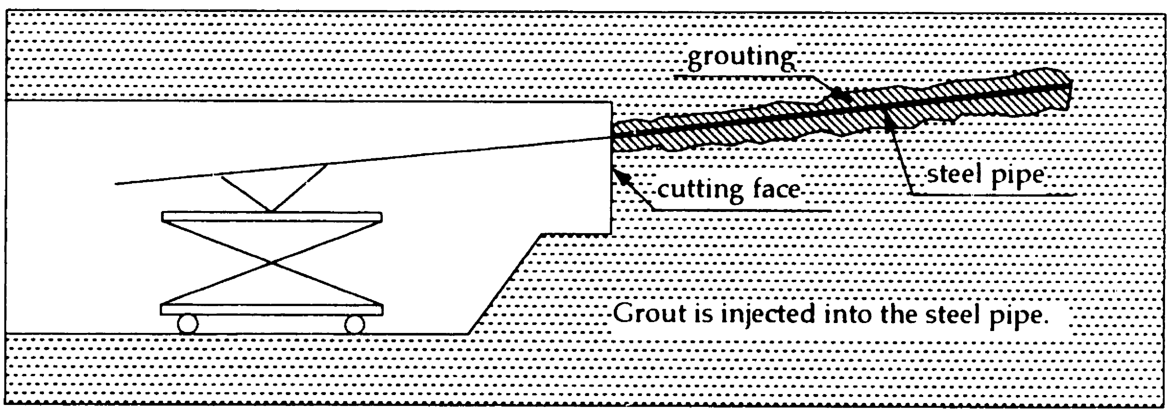
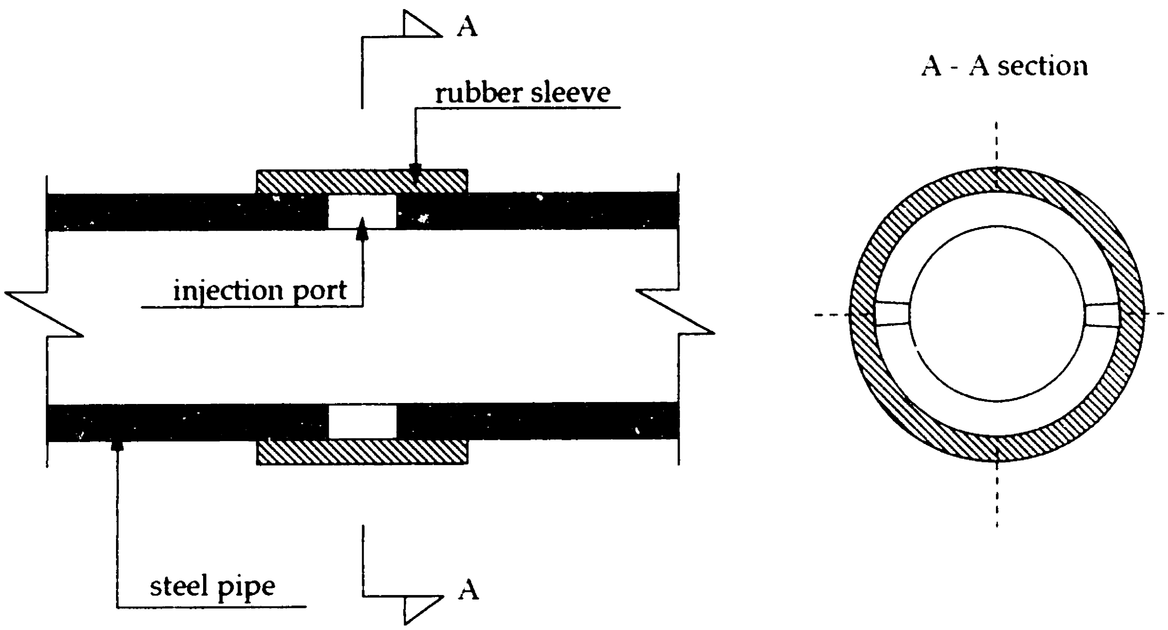


Fig. 3-27 Construction sequences of the TREVITUB method (Trevi SpA, 1993)

a) manchette type



b) springval type

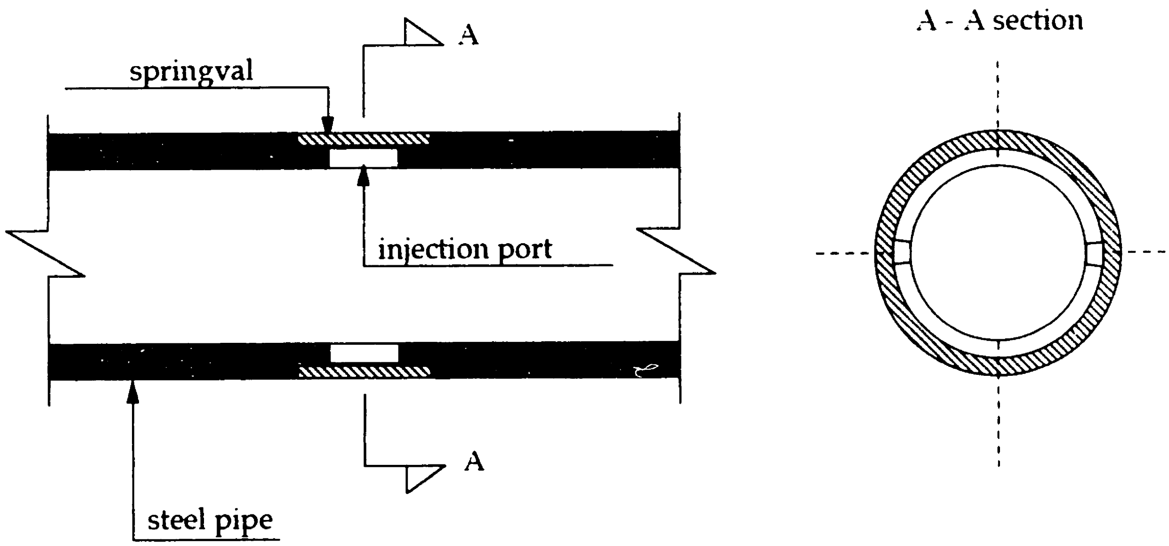
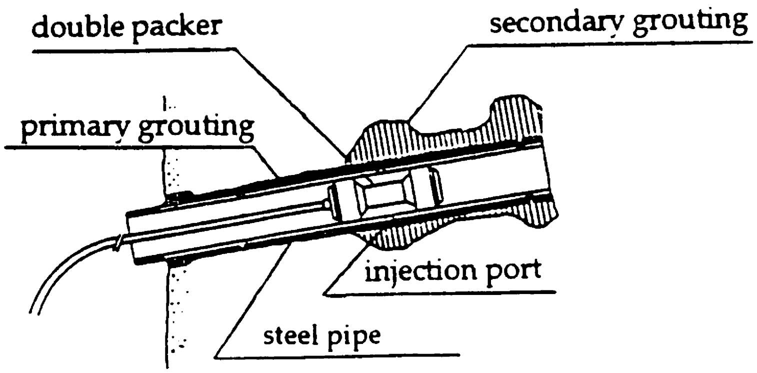


Fig. 3-28 Injection method (Trevi SpA, 1993)

(a) Single valve injection



(b) Total valve injection

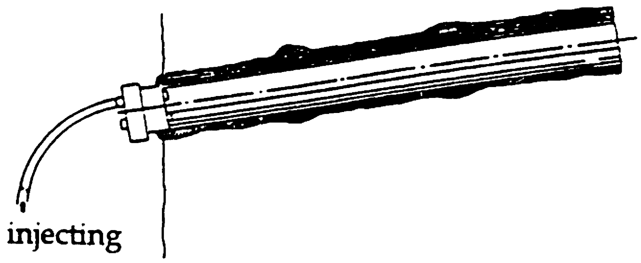


Fig. 3-29 Injection type (Geo-Fronte Research Association, 1995)

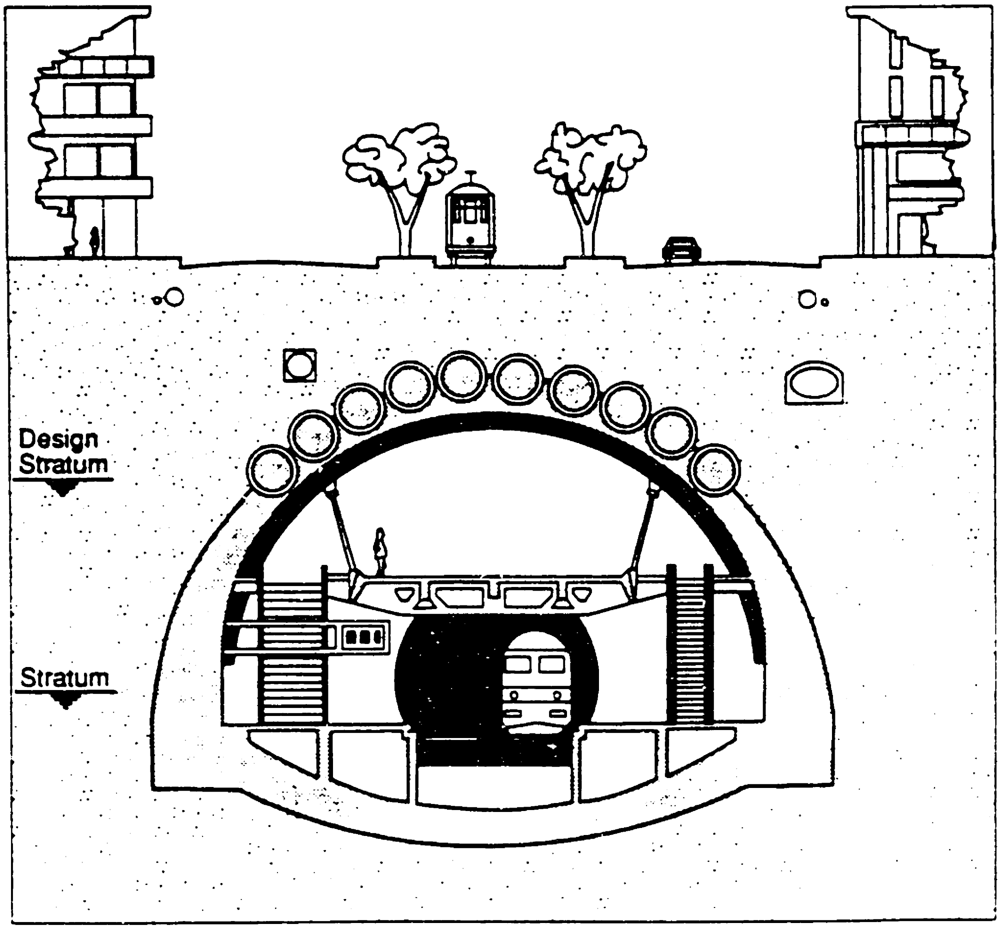


Fig. 3-30 Cellular arch method: Venezia station (Lunardi, 1990)

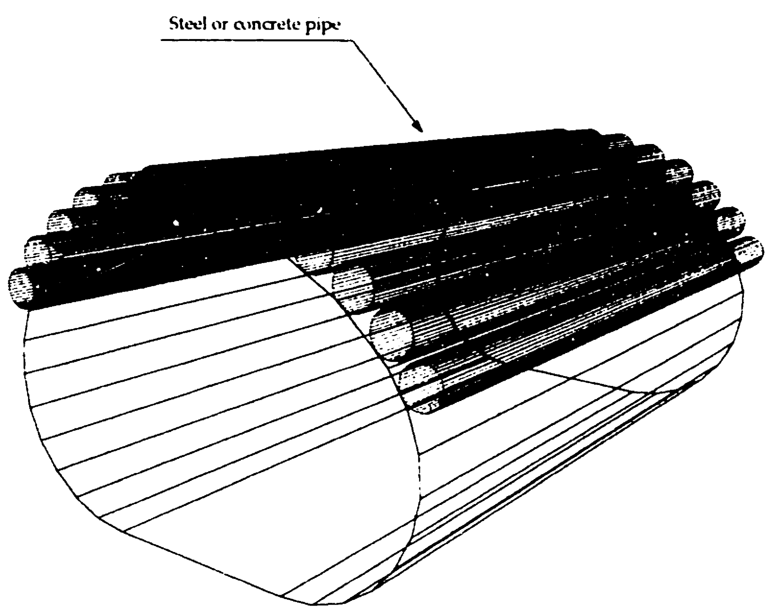
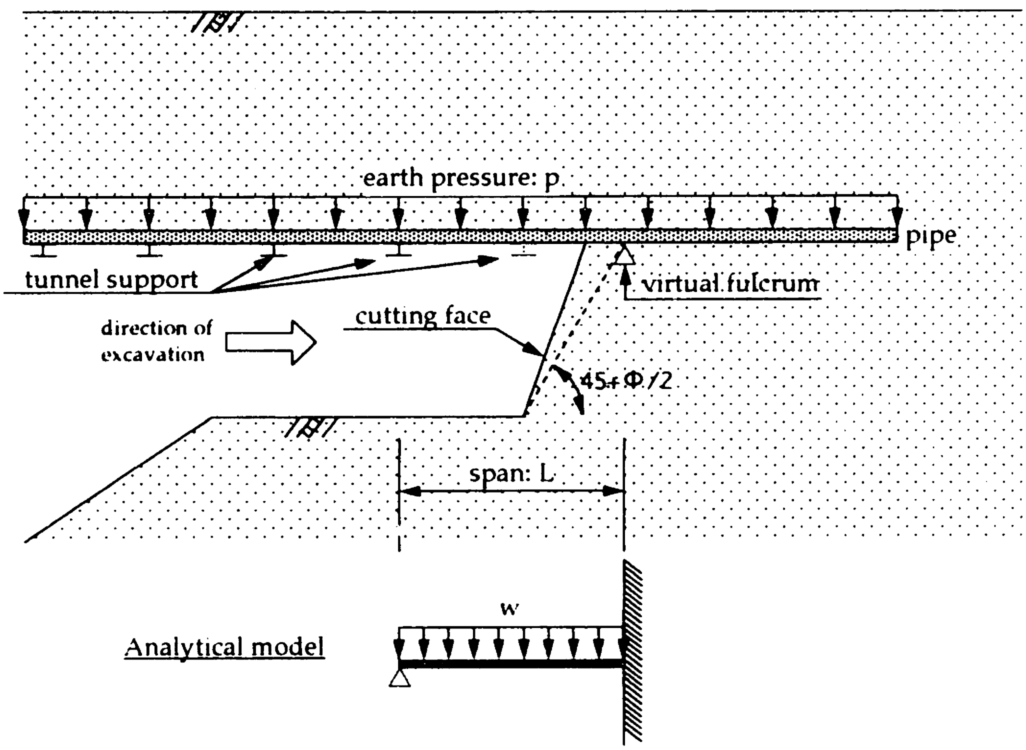


Fig. 3-31 Pipe roof method



M(max)= 1/8 × (w × L^2)
w = p × s
L: span
p: earth pressure
s: spacing between pipes

Fig. 3-32 Analytical model for designing a pipe section (Saito, 1982)

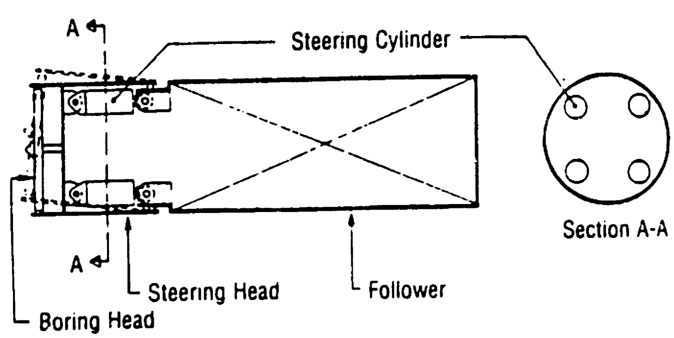


Fig. 3-33 View of steering cylinders used for moving the steering head of shield tunnelling machine (Stein et al., 1989)

Chapter 4. Case Studies on the Umbrella Method

4.1 Overview and Definition

In this chapter, 24 cases from the literature in which the umbrella method was used are studied. Each case is described with regard to the following:

- environment
- geological and hydrological conditions
- problems in tunnel construction
- supplementary support method of the tunnel
- structural details
- construction procedures
- field measurements
- numerical analysis (if available)

The tunnel or project name and type of umbrella method used in each case are summarized in Table. 4-1. Figures 4-1 through 4-3 show the location of the selected cases. The 24 cases are divided according to the type of umbrella method (see Table 4-2).

There are only three cases in which the pipe roof method was used. This is a small number compared to the number of cases with the other umbrella methods. In this thesis, new methods such as the sub-horizontal jet-grouting method and the injected steel pipe umbrella method are the focus; in addition, the case studies selected for the pipe roof method were those in which relatively large diameter steel or concrete pipes were used. Consequently, this resulted in a smaller number of cases with the pipe roof method.

The use of the umbrella method is concentrated in Japan and Italy; as shown in Fig. 4-4, about 90% of all cases are in these two countries. It is believed however that there are many other applications of the umbrella method, especially in Italy.

As mentioned previously, most of the methods discussed in this thesis were originally developed in Italy (see Table. 3-1) and then introduced to Japan as well as to other European countries around Italy. In particular, in Japan, the environmental circumstances in tunnelling such as the need to develop underground facilities (motorways, railways, etc.) in urban areas with poor ground conditions has led to many applications of the umbrella method and it is thought that such applications in Japan will increase.

Table 4-1 List of the tunnels

Case No.	Name of Tunnel / Project	Location	Type of Umbrella Method
1	Hodogaya Tunnel	Yokohama, Japan	Sub-horizontal jet-grouting method
2	Shoryou No. 1 Tunnel	Gotenba, Japan	Sub-horizontal jet-grouting method
3	Kokubugawa Tunnel	Ichikawa, Japan	Sub-horizontal jet-grouting method
4	Owani Tunnel	Owani, Japan	Sub-horizontal jet-grouting method
5	Aziro Tunnel	Itsukaichi, Japan	Sub-horizontal jet-grouting method
6	Uryuya Tunnel	Sonobe, Japan	Sub-horizontal jet-grouting method
7	Subway Vienna	Vienna, Austria	Sub-horizontal jet-grouting method
8	Rengershausen	Kassel, Germany	Sub-horizontal jet-grouting method
9	Campiolo Tunnel	Moggio Udinese, Italy	Sub-horizontal jet-grouting method
10	Lonato Tunnel	Verona, Italy	Reinforced sub-horizontal jet-grouting method
11	Kaziwara No. 1 Tunnel	Ibaragi, Japan	Reinforced sub-horizontal jet-grouting method
12	Les Cretes Tunnel	Aosta, Italy	Reinforced sub-horizontal jet-grouting / injected steel pipe umbrella methods
13	Maiko Tunnel	Kobe, Japan	Injected steel pipe umbrella method
14	Hirai Tunnel	Miki, Japan	Injected steel pipe umbrella method
15	Futatsui-Nishi Tunnel	Futatsui, Japan	Injected steel pipe umbrella method
16	Yakiyama Tunnel	Niigata, Japan	Injected steel pipe umbrella method
17	Ramat Tunnel	Piedmont, Italy	Injected steel pipe umbrella method
18	Poggio Fornello Tunnel	Tuscany, Italy	Injected steel pipe umbrella method
19	St. Ambrogio Tunnel	Sicily, Italy	Injected steel pipe umbrella method
20	Nango Tunnel	Hayama, Japan	Injected steel pipe umbrella method
21	Kubodaira Tunnel	Shioyama, Japan	Injected steel pipe umbrella method
22	Venezia Station	Milan, Italy	Pipe roof method
23	MARTA East Line Underpass Tunnel	Atlanta, US	Pipe roof method
24	Yokohama Subway No. 3 Line	Yokohama, Japan	Pipe roof method

Table 4-2 Classification of 24 cases by type of umbrella method

Type of umbrella method	Number of cases
Sub-horizontal jet-grouting method	11 cases, 2 of which are reinforced sub-horizontal jet-grouting method
Injected steel pipe umbrella method	9 cases
Pipe roof method	3 cases
Reinforced sub-horizontal jet-grouting method and injected steel pipe umbrella method (both used)	1 case

Since the effect on ground surface deformation or settlement is very important, this will be a major consideration when comparing the cases. Hence, it is necessary to first discuss how the settlement and other ground deformation will be described.

Figure 4-5 shows the idealized settlement curve normally observed during heading-and-benching excavation, where ***distance from face*** in the horizontal axis means the distance between the measuring point and the tunnel face. A negative value denotes that the tunnel face approaches the measuring point; in other words, the measuring point is ahead of the tunnel face. On the other hand, a positive value denotes that the tunnel face is beyond the measuring point, i.e., the measuring point is behind the tunnel face. Zero represents the location where the tunnel face has reached the measuring point. Note that in this thesis when the term ***face*** is used alone it always represents the face of the top heading, not the face of the bench.

As shown in the figure, before the face arrives at the measuring point, settlement gradually increases with advance of the tunnel ("A" in the figure). Then, immediately before and after passage of the face, settlement dramatically increases. In this thesis, settlement which occurs before tunnel face arrives at the measuring point is called ***pre-excavation settlement***.

As the tunnel face proceeds beyond the measuring point, settlement gradually subsides. Finally, changes in settlements become almost zero ("B" in the figure). In this thesis, settlement at this point (B), which was induced by excavation of the top heading, is called ***settlement after excavation of top heading***.

As the bench approaches in the heading-and-benching method, some additional settlement occurs and then, settlement subsides again ("D" in the figure). Settlement after excavation of the bench is called ***final settlement*** in this thesis. In general, the settlement due to excavation of the bench is very small compared to that due to excavation of the top heading.

In the following section, each case will be described in detail.

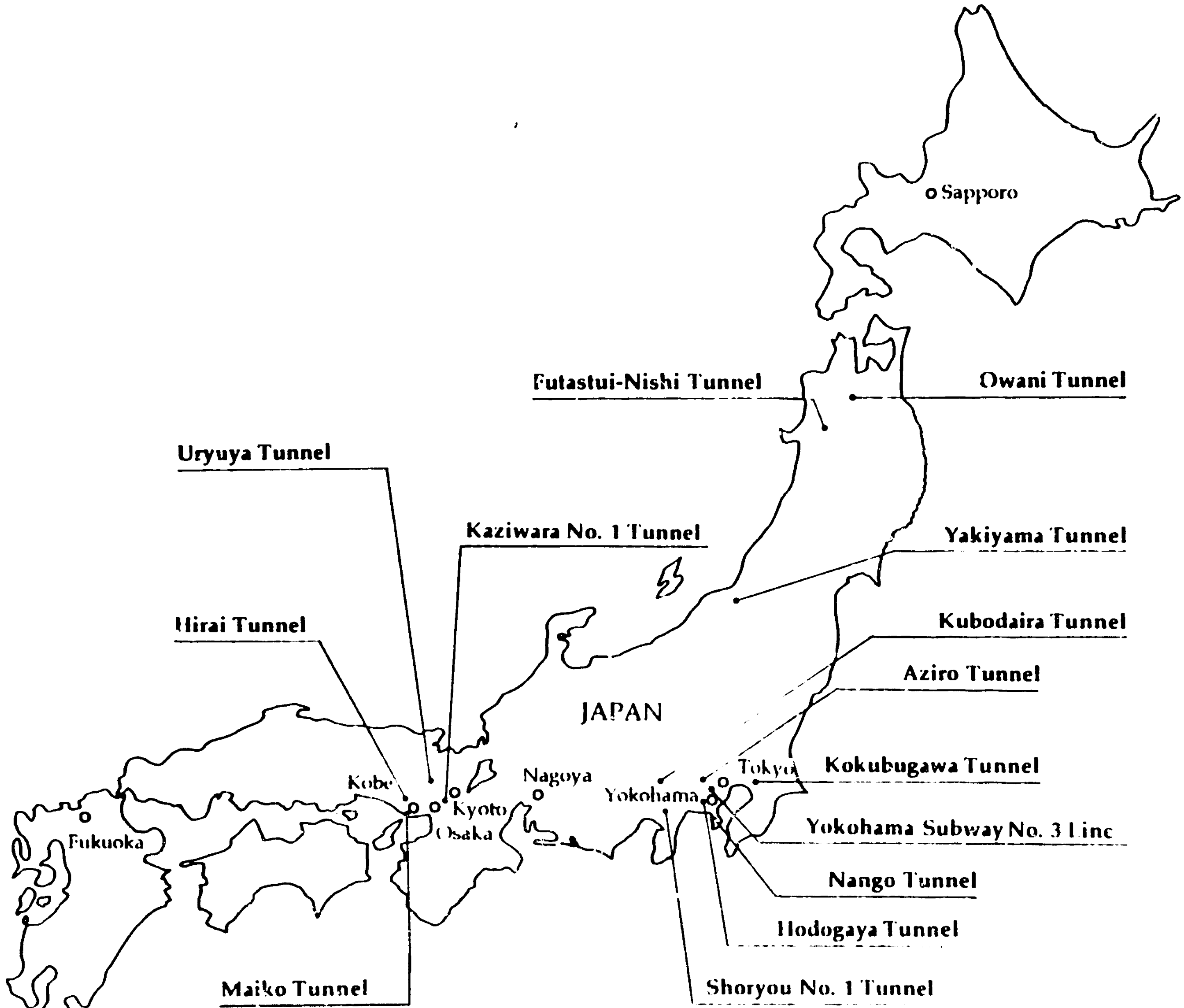


Fig. 4-1 Location of the tunnels in Japan

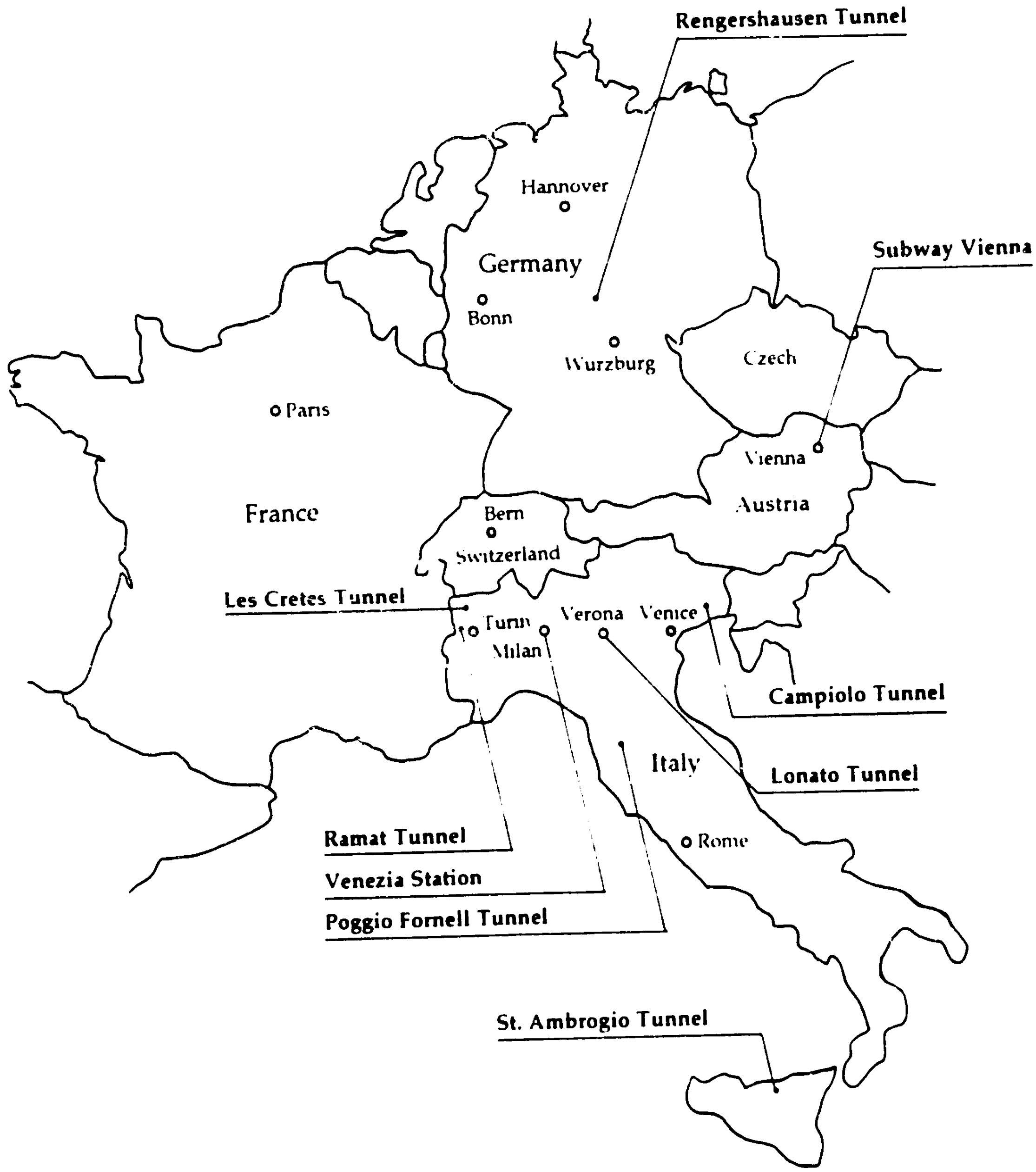


Fig. 4-2 Location of the tunnels in Europe

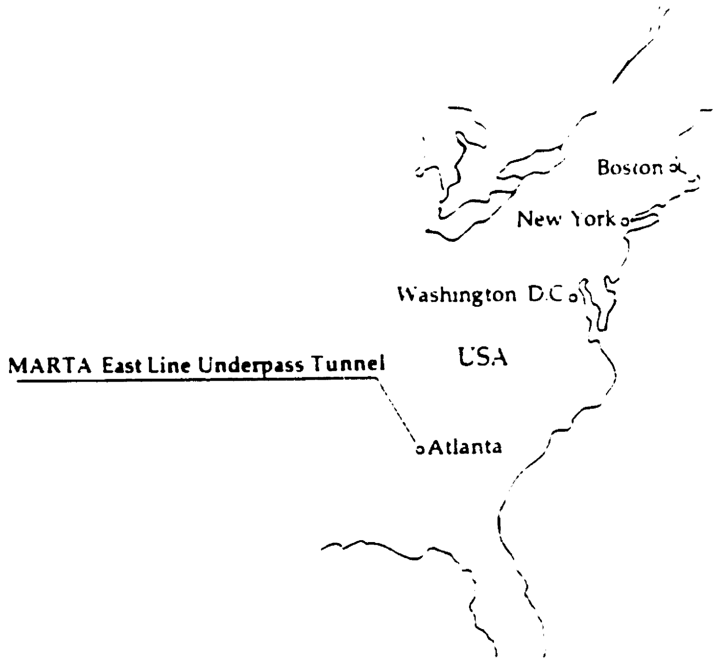


Fig. 4-3 Location of the tunnel in US

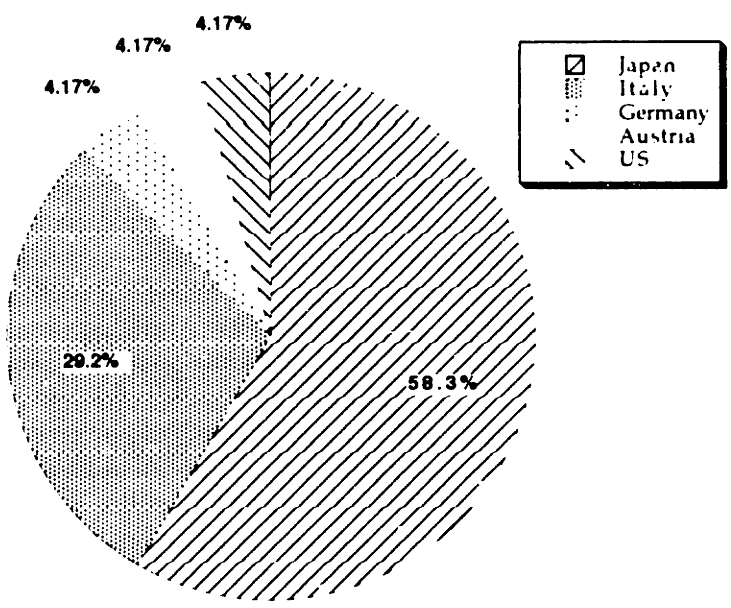


Fig. 4-4 Classification of case studies by nation

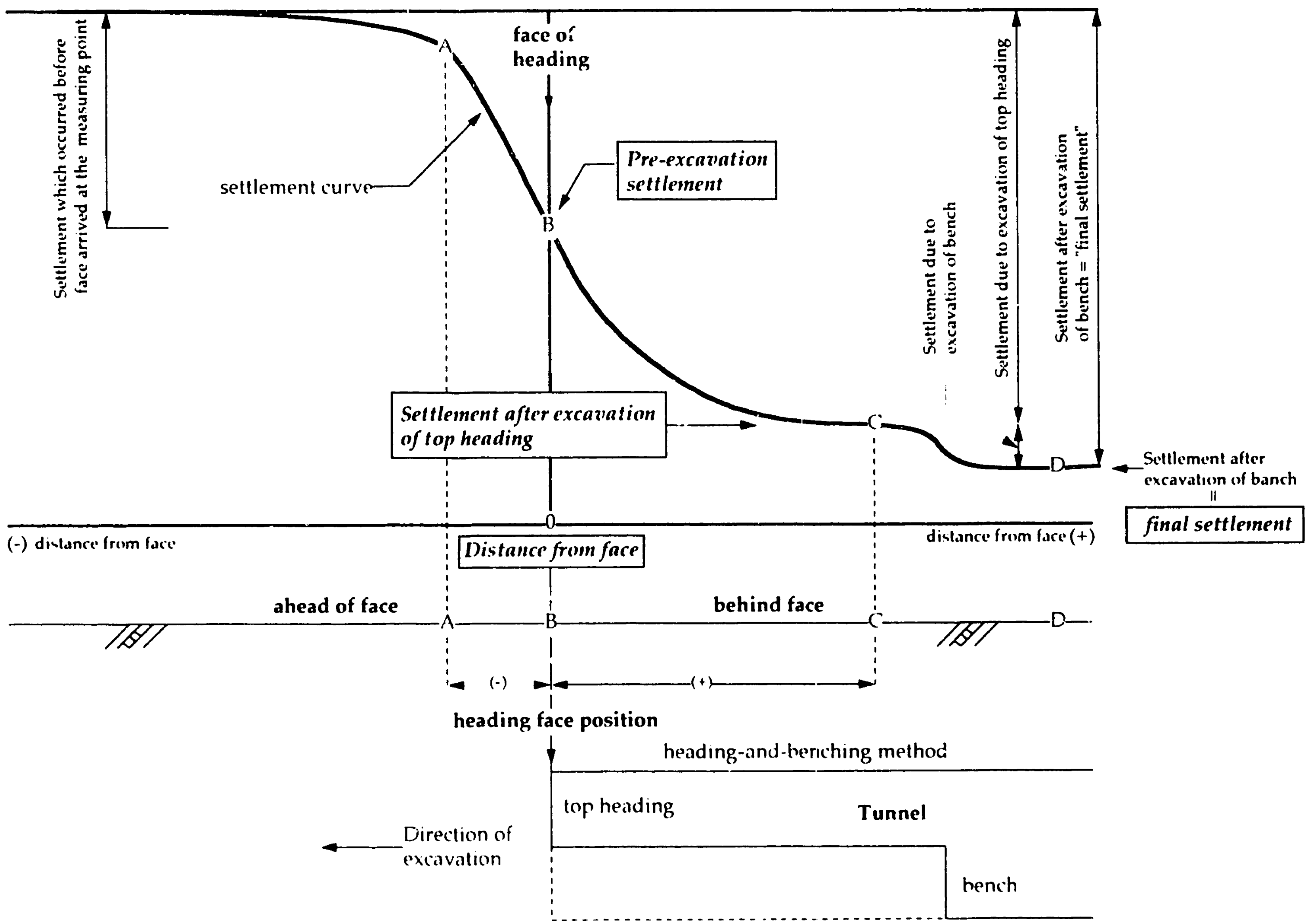


Fig. 4-5 Idealized settlement curve

4.2 Case Studies on the Umbrella Method

Case 1: Hodogaya Tunnel (Yokohama, Japan)

Environment of the Hodogaya Tunnel

The 200-m-long Hodogaya Tunnel, which is a highway tunnel with three lanes, is located in an urban area about 5km northwest of downtown Yokohama. The Hodogaya Tunnel was constructed near an existing tunnel; distances between tunnels ranged from 2.5m to 6.5m. The excavated width and height are approximately 17m and 11m respectively, and the excavated area is about 150m^2.

Geological and Hydrological Conditions

The geological conditions of the Hodogaya Tunnel are shown in Fig. 4-1-1. They consist of a bedrock of the tertiary to early diluvial Kazusa Formation, an early diluvial Sagami Formation and Kanto loam. Distributed above the tunnel section are an embankment, loam, and loamy clay.

Most of the tunnel section is the Sagami Formation. It is 10m to 18m thick and mostly consists of solid to sandy silt with an unconfined compressive strength of 0.5 - 0.8MPa and SPT-N values of 9 - 45.

The engineeringproperties of the above layers are shown in Table. 4-1-1.

Table 4-1-1 Engineering properties: Hodogaya Tunnel

Layer		Soil type	Modulus of deformation (MPa)	C (kPa)	φ (°)	SPT-N
embankment	Sf	loam, sand	15	30	10	3
Kanto loam-1	Pt	volcanic cohesive soil	15	40	5	4
Kanto loam-1	Lm	loam	20	40	5	4
Kanto loam-2	Lc	volcanic cohesive soil	25	40	5	4
Sagami Formation	By	silt, sandy silt, fine sand, gravel	210	200	12	18
Kazusa Formation	Kh	mudstone, fine sand	390	1400	15	>50

Source: Zaitsu et al. (1994)

The groundwater table was not observed in the vicinity of the tunnel. There was an outflow of some groundwater in the existing tunnel, but there was very little outflow in the new tunnel.

Problems in Tunnel Construction

The Hodogaya Tunnel was constructed under exacting conditions:

- The tunnel had a large cross section.
- The depth of overburden above the tunnel crown was small, being no greater than 17m.
- The tunnel was located under a residential area and under roads.

Supplementary Support Method of the Tunnel

The sub-horizontal jet-grouting method (the RJFP method) was employed to restrict surface settlement, improve face stability and minimize damage to the existing tunnel.

No information on whether or not alternative methods were assessed was available from the references.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-1-2.

The jet-grouted columns are 10m long and 60cm in diameter. The reinforced range of the tunnel perimeter is 120°. The number of jet-grouted columns per cross section is 32.

Tunnel support consists of a 20-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 60cm thick.

Construction Procedures

The excavation was by side drift method.

For the jet-grouted columns, a length of 13m was drilled, which was divided into 10m of improved zone and 3m of lost drilling (drill hole length which will be eventually excavated in conjunction with tunnel excavation). 9m of the tunnel section was excavated before installing the next umbrella arch to maintain a 1-m overlap for the protection of the face.

The jet-grouting parameters and grouts used are summarized in Table 4-1-2.

Table 4-1-2 Jet-grouting parameters: Hodogaya Tunnel

Grouting pressure (MPa)		50
Grout volume (liter/m)		395
Grout (kgf/m^3)	cement	700
	additive	12
	water	750

Source: Zaitsu et al. (1994)

Field Measurements

Figure 4-1-3 shows the distribution of ground surface settlements above the tunnel centerline in the longitudinal direction, where O stands for the settlement after excavation of the top heading, while □ stands for the final settlement.

A maximum final settlement of 39mm was monitored at a point 60m from the Tokyo-side portal, while under the residential areas and the road areas, final settlements were relatively small, and hence tunnel construction did not cause any serious problems to the surface structures.

Heaving occurred in a section between the Tokyo-side portal and 30m from the portal. This was due to a rise of pressure in the ground caused by insufficient removal of cutting during development of the jet-grouted columns. To prevent such heaving, holes for removal of cutting produced during development of the jet-grouted columns were drilled in the ground above the jet-grouted columns (details are unknown). Due to these countermeasures, heaving no longer occurred.

Figure 4-1-4 shows the relationship between the ground settlement above the tunnel crown and the distance from the face. From the figure, it can be seen that settlement gradually increased after the face advanced from a point about 0.5D (D: tunnel width) ahead of the measuring point. When the face reached the measuring point, large settlement took place, and settlement subsided about 1.0D - 2.0D beyond the measuring point. Taking into account the general observation in tunnel excavation that such deformation generally starts when the face is at a point about 2.0D ahead of the measuring point, and then subsides after the face is about 2.0D beyond the point, it can be seen that pre-excavation settlement in Hodogaya tunnel was reduced by the RJFP method.

Figure 4-1-5 shows the distribution of vertical displacement of the ground above the tunnel crown. From the figure, it can be seen that settlement of the ground surface was almost equivalent to that at the tunnel crown.

Numerical Analysis

(1) Analytical model

A three-dimensional (3-D) FE method was carried out in order to assess the support of the RJFP and confirm the rigidity of the jet-grouted columns (Ito et al., 1995). Figure 4-1-6 shows the analytical model, where the tunnel excavation width is 16 m and the overburden is 6.5 m.

The material constants used are summarized in Tables 4-1-3 and 4-1-4.

Table 4-1-3 Ground parameters in analytical model: Hodogaya Tunnel

Soil type	Unit weight (kN/m³)	Modulus of deformation (MPa)	Poisson's ratio	C (kPa)	φ (°)
Lm	13.7	10	0.4	40	5
By	15.2	45	0.375	180	12
Kh	18.1	500	0.3	1400	15

Source: Ito et al. (1995).

Table 4-1-4 Support parameters in analytical model: Hodogaya Tunnel

Support	Modulus of elasticity (MPa)	Sectional area (cm^2)	Polar moment of inertia (cm^4)
shotcrete (t=20cm)	4.0×10^3	2000	-
steel support (H-200)	2.1×10^5	63.53	4720
jet-grouted column	1.875×10^3	4000	-
sidewall concrete	2.35×10^4	-	-

Source: Ito et al. (1995).

The intact ground constants were set basically by reverse analysis, although the results of geological surveys and other tests were also used. In modeling the jet-grouted column, a 120° arch section was changed from intact ground rigidity to the rigidity after improvement simultaneously with the excavation.

(2) Analytical results

Figure 4-1-7 shows the comparison of the displacement curves for the analysis with measured ground surface settlement, crown settlement, and horizontal convergence, respectively.

Regarding crown settlement, the analysis results corresponded well to the measured values up to nearly 0.5D beyond the face. Regarding horizontal convergence, there was some increase and decrease in measured values. Again, there was close agreement between the analysis results and the measured data at a distance of approximately 2D beyond the face. For the settlement of the ground surface, because of insufficient field measurements, there are few measured points. However, it seems that the analyzed data points are almost equal to the measured ones.

Figure 4-1-7 also shows the comparison between the analysis results and the measured values for vertical bending strain and circumferential strain in the jet-grouted column. From the figure, bending strains in the vertical direction started at about 3m ahead of the face and the maximum tensile strain occurred near 1m ahead of the face. Then after the face passed the measuring point, tensile strains toward the outer side of the jet-grouted column occurred, and once again the direction of the strain shifted toward the inner side of the jet-grouted column.

The analysis values roughly follow this movement. For strain in the circumferential direction, as the tunnel face advanced the compression strain tended to increase. The analysis results showed the same trend.

(3) Consideration of simple analytical model

It became evident from the comparison between the analysis results and the measured values that a 3-D FE method is a useful method for ascertaining the behavior and supporting effect of the jet-grouted columns, but from a practical and economical point of view, it seems difficult to make use of the 3-D FE method as a design tool for the sub-horizontal jet-grouting method.

Considering this, Ito et al. (1995) proposed a simple design method in which a beam on an elastic foundation takes into account the subgrade reaction both ahead of the face and beyond the face (see Fig. 4-1-8). The load is simulated by assuming that the total overburden pressure is put on one excavation portion (1m in length) and the resulting strain corresponds to an incremental strain per unit excavated length (1m). Hence in order to simulate tunnel excavation the load has to be put on the newly excavated portion at every stage. Then, strains obtained in each stage are superimposed, i.e., the final strain at a particular point on the beam is the sum of the strains from each stage.

Figure 4-1-9 shows a graph of the vertical bending strain, comparing the analysis results given by the simple model with the measured values. From the figure, one can say that the bending strains determined from the simple model correspond to the measured values.

References

Zaitsu, M., Watanabe, T., et al., "Construction Report on the Hodogaya Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), May, 1994.

Ito, J., Sano, N., et al., "Approach for Design Method of the Umbrella Method," 1995.

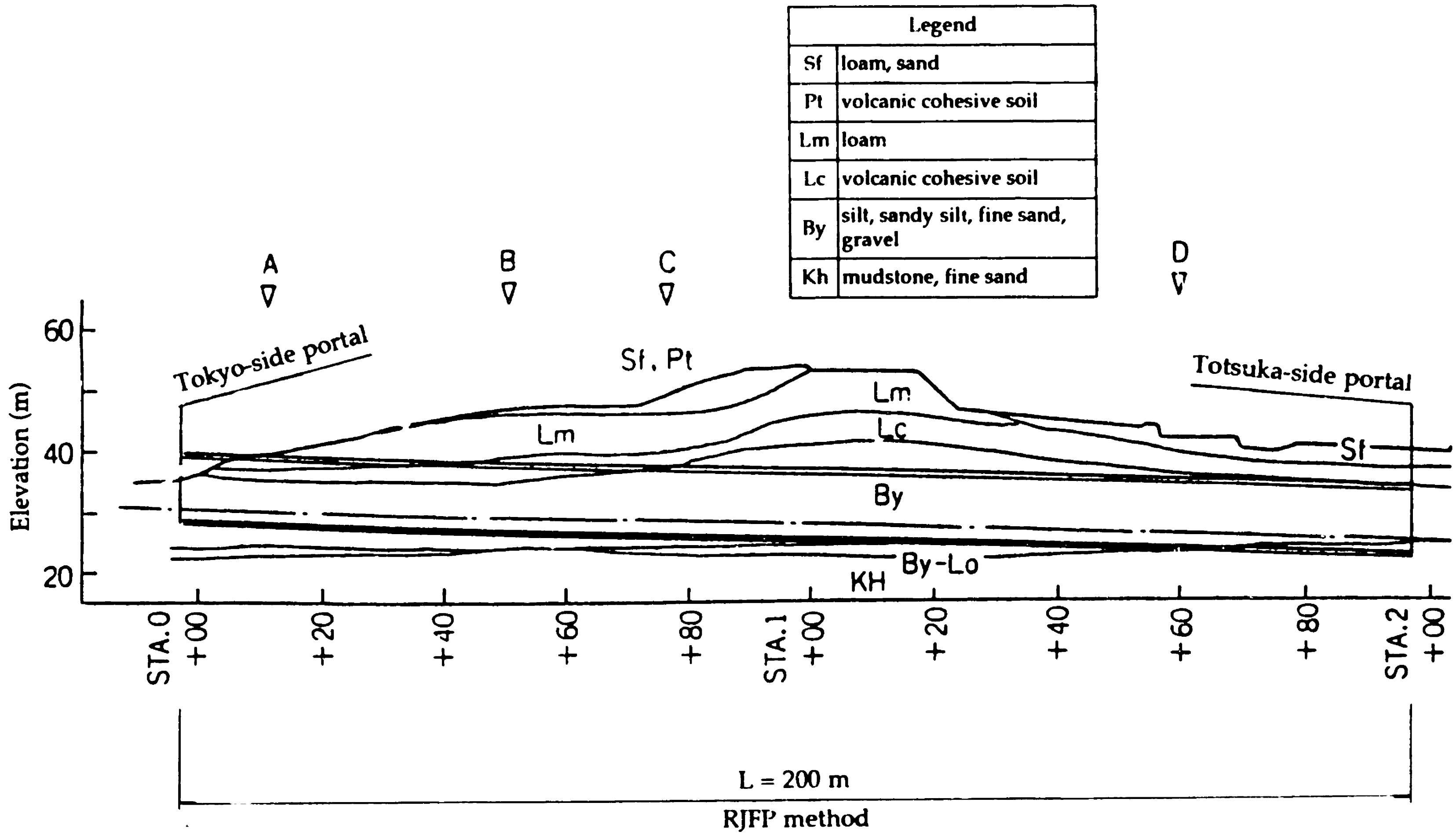
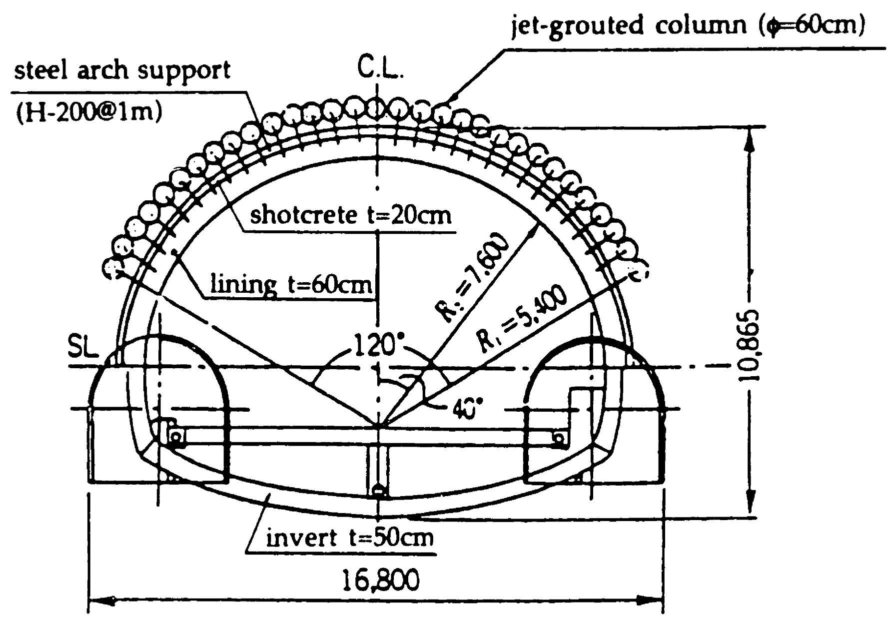


Fig. 4-1-1 Geological conditions: Hodogaya Tunnel (Zaitsu et al., 1994)

(a) Cross section



(b) Longitudinal section

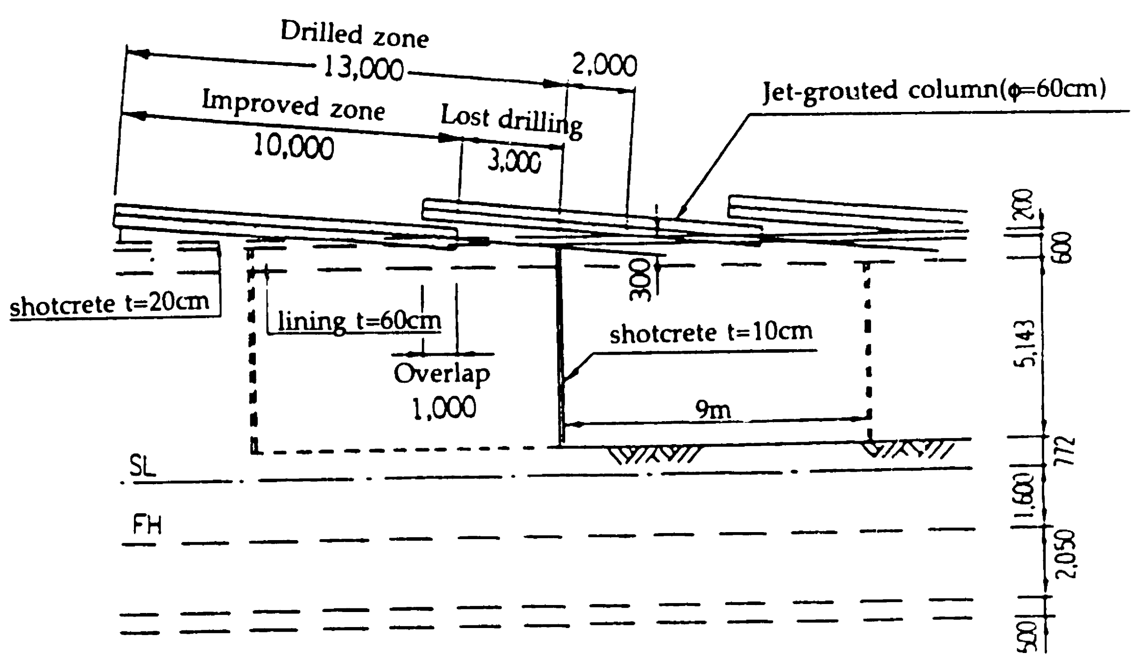


Fig. 4-1-2 Cross and longitudinal sections: Hodogaya Tunnel (Zaitsu et al., 1994)

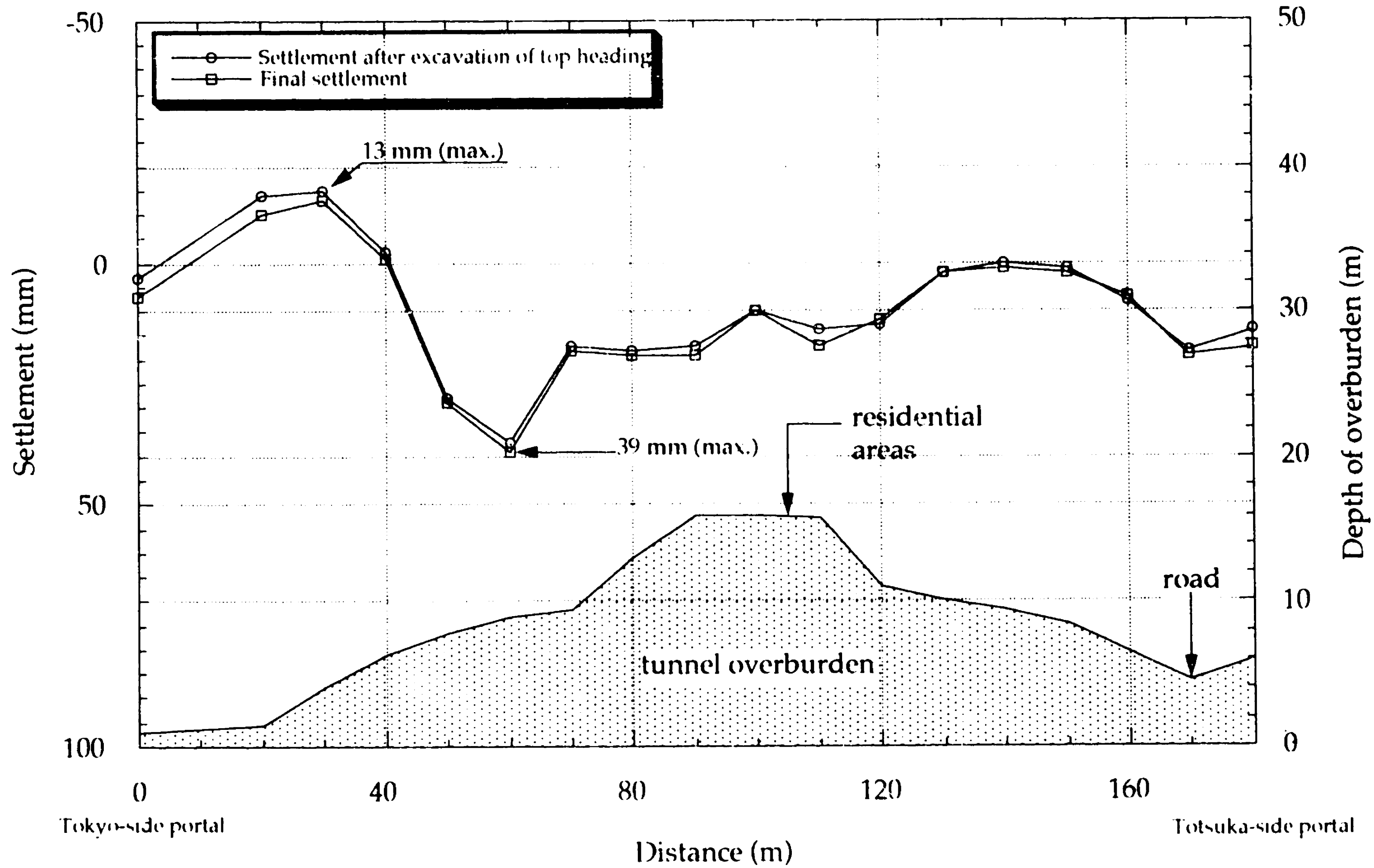


Fig. 4-1-3 Distribution of ground surface settlements: Hodogaya Tunnel (Zaitsu et al., 1994)

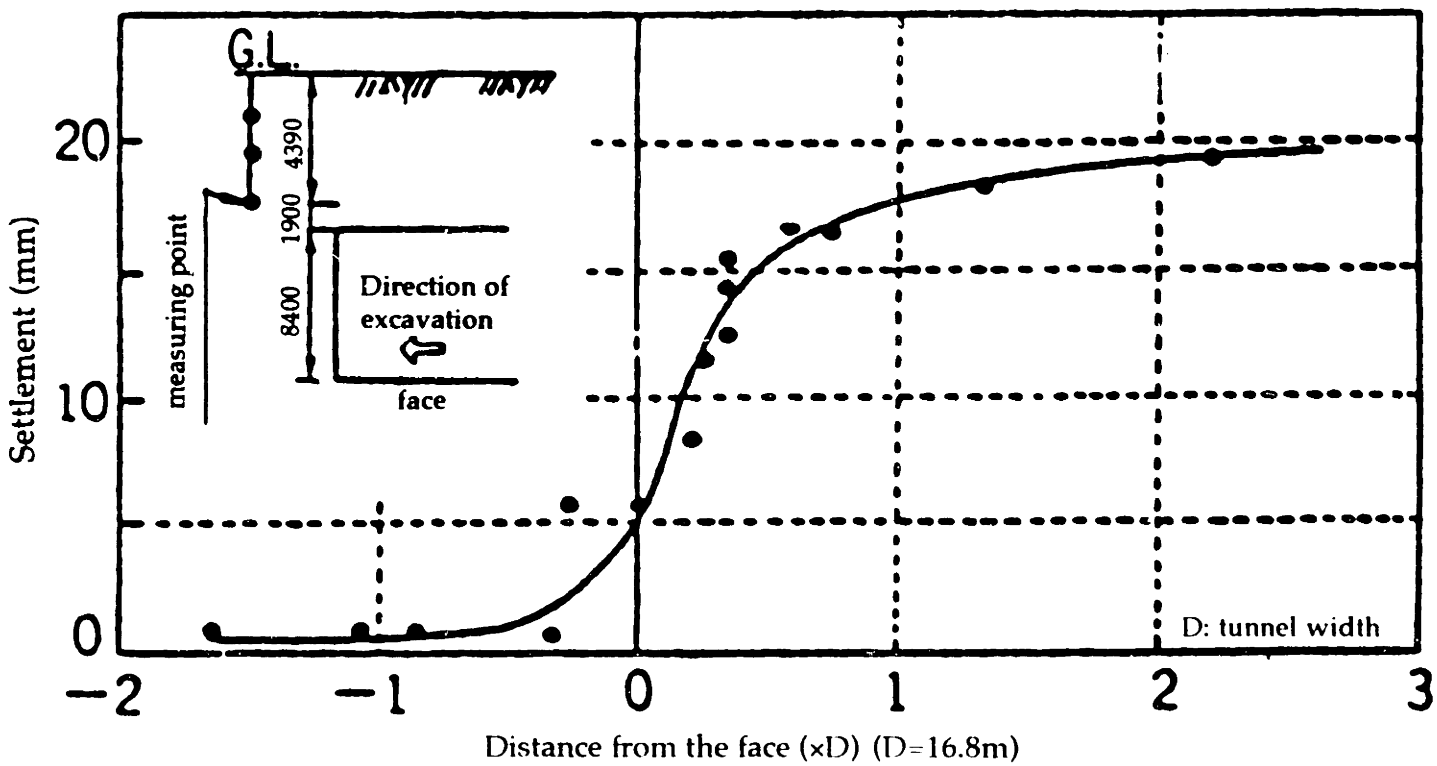


Fig. 4-1-4 Ground settlement curve: Hodogaya Tunnel (Zaitsu et al., 1994)

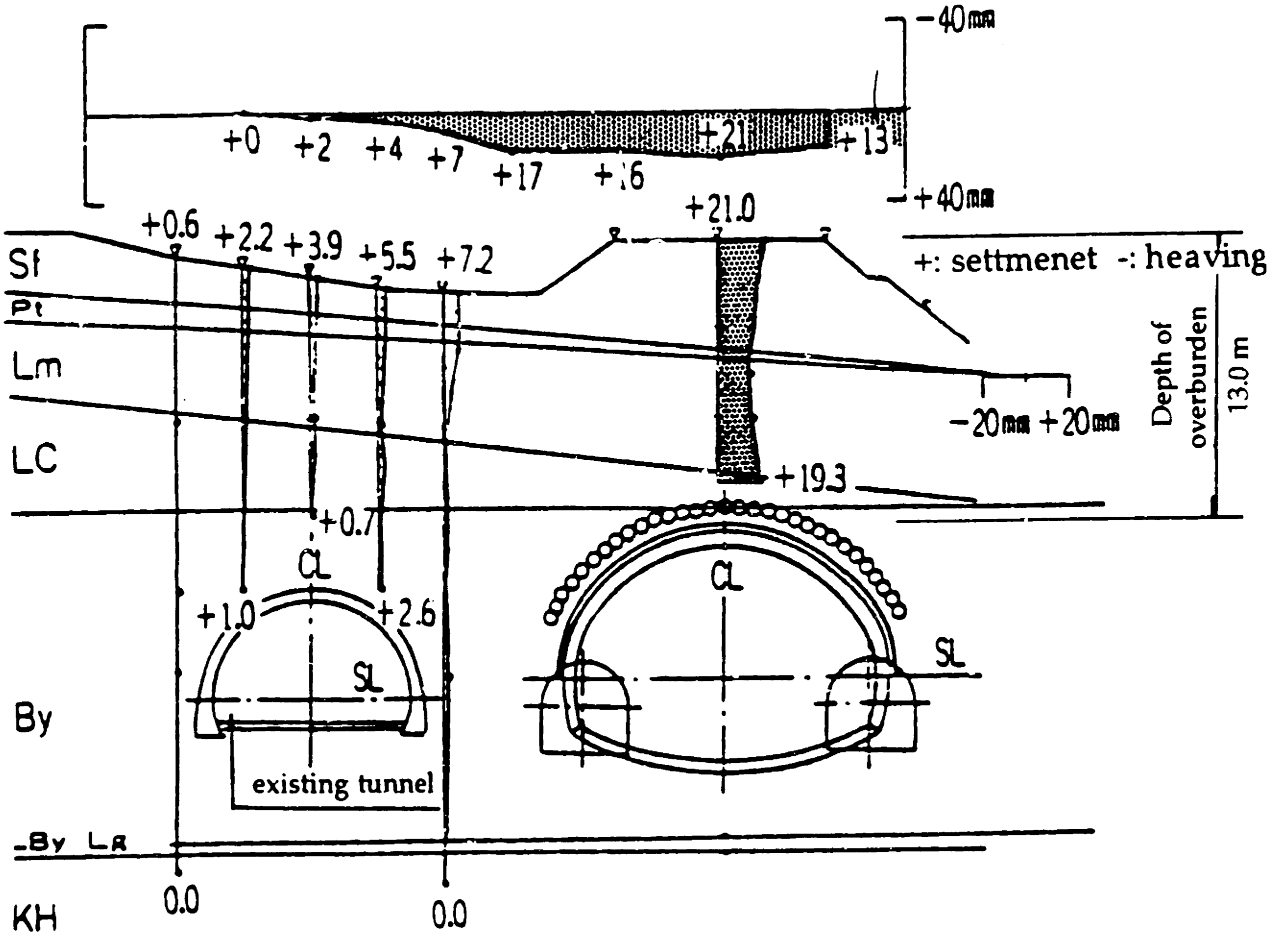


Fig. 4-1-5 Distribution of vertical displacement of the ground above the tunnel crown (Zaitsu et al., 1994)

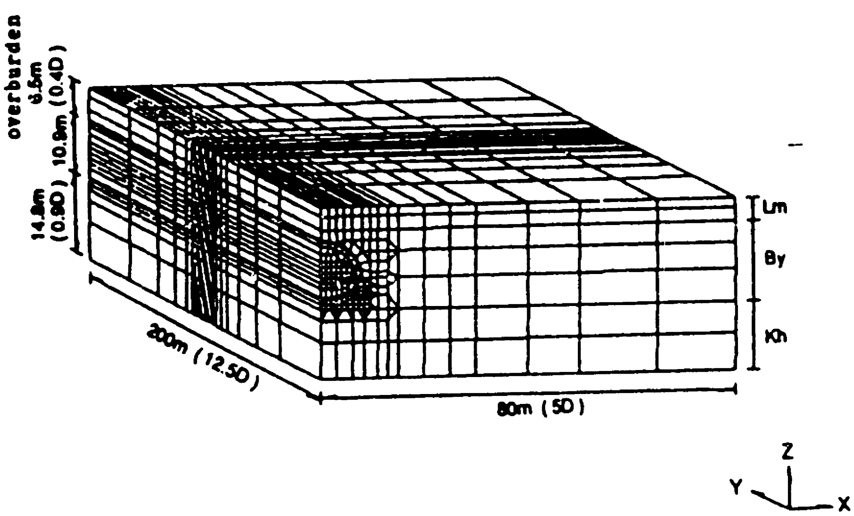


Fig. 4-1-6 Analytical model for 3-D FE method: Hodogaya Tunnel (Ito et al., 1995)

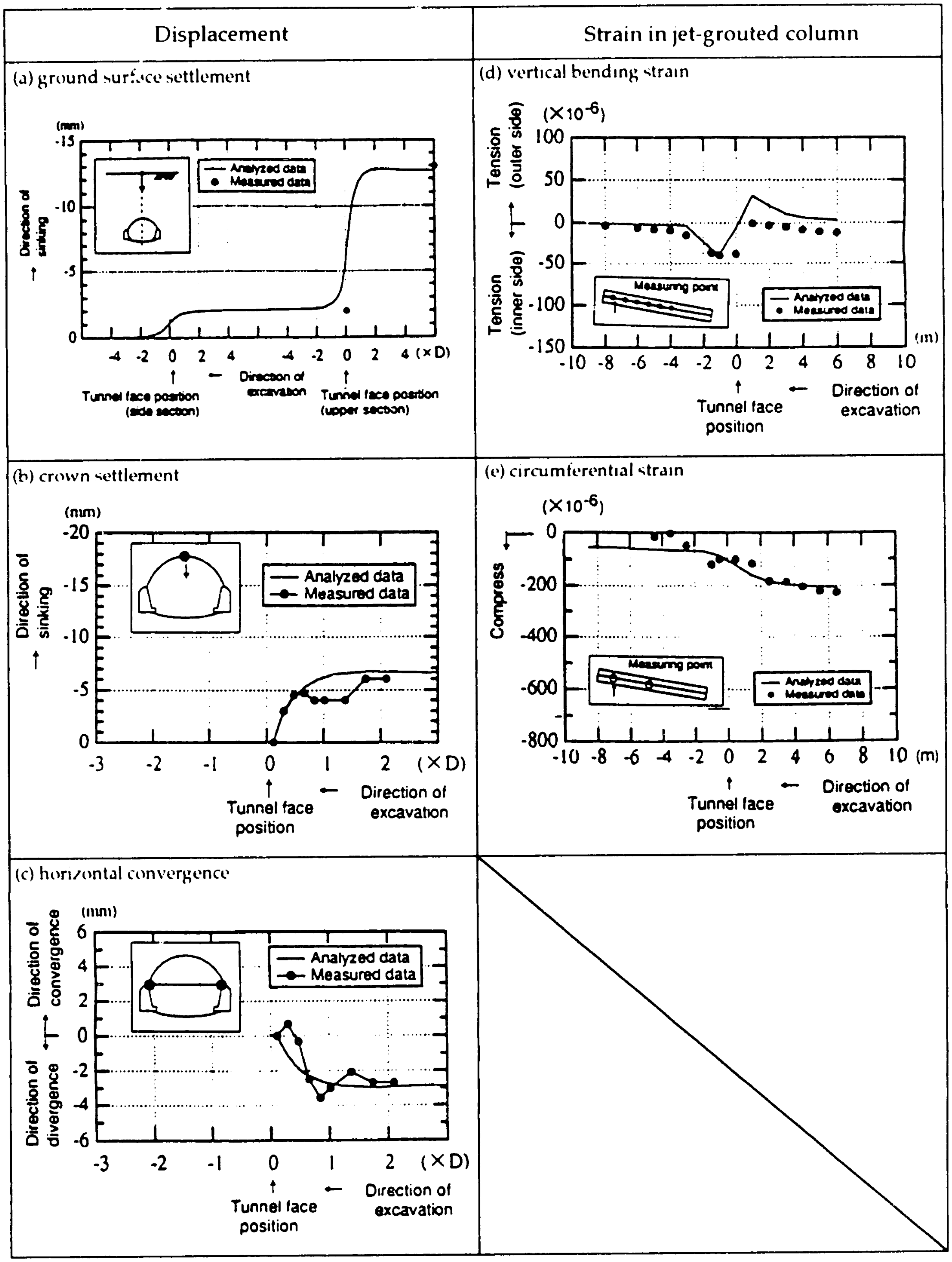


Fig. 4-1-7 Results of 3-D FE method: Hodogaya Tunnel (Ito et al., 1995)

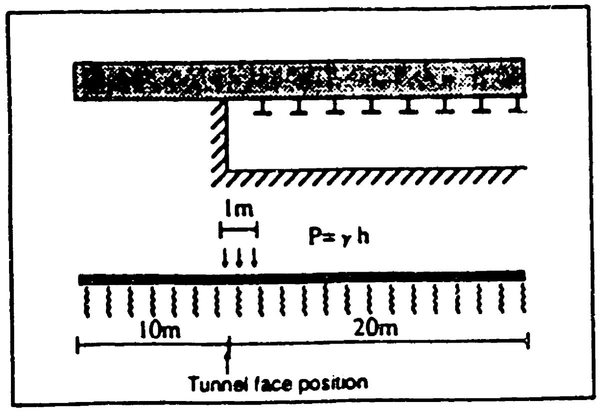


Fig. 4-1-8 Analytical model for a simple design method: Hodogaya Tunnel (Ito et al., 1995)

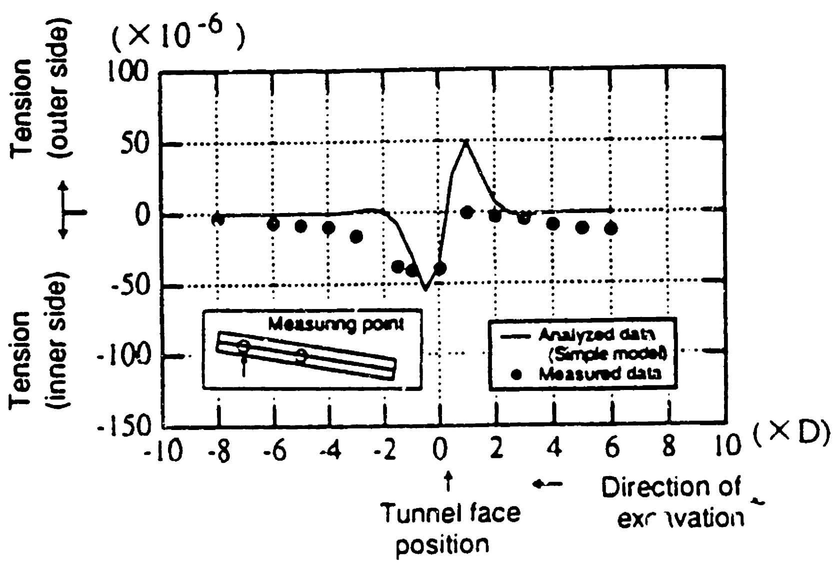


Fig. 4-1-9 Vertical bending strains in the jet-grouted column obtained from a simple design method: Hodogaya Tunnel (Ito et al., 1995)

Case 2: Shoryou No. 1 Tunnel (Gotenba, Japan)

Environment of the Shoryou No. 1 Tunnel

The 220-m-long Shoryou No. 1 Tunnel, which is located between Gotenba and Oimatsuda on the Tokyo-Nagoya Expressway, is a tunnel for a 3-lane motorway. The excavated width and height are approximately 16m and 11m, respectively, the excavated area is about 145m^2.

Geological and Hydrological Conditions

The geological conditions of the Shoryou No. 1 Tunnel are shown in Fig. 4-2-1.

The scorious loam layer (Flm2) mostly consists of scoria, loam, and volcanic sand, occasionally interposed with a thin layer of clay. The SPT-N value of this layer ranges from 3 to 20 and cohesion is about 0 to 10kPa. The volcanic sand and gravel layer (Fmf) consists of coarse sand and gravel, in which pumice with 40 to 50cm in diameter is interposed. The SPT-N value is larger than 50. The scoria layer (Hsc), which overlays the scorious loam layer (Flm2), is, in general, a loose, uncemented deposit having an SPT-N value of about 3.

Problems in Tunnel Construction

Little ground arching could be expected because of the shallow overburden of up to 18m compared to an excavation width of 16m and cross-sectional area of 145m^2. Moreover, the ground was uncemented and loose.

A supplementary support method had to be adopted to ensure face stability during tunnel excavation.

Supplementary Support Method of the Tunnel

Injected fore-poles were adopted for the Shoryou No. 2 Tunnel, which was driven with side drift method. The sub-horizontal jet-grouting method (the RJFP method) was applied at the Shoryou No. 1 Tunnel since it was considered to be a more effective way to reduce loosening of the ground and thus ensure the face stability.

Three stabilization patterns depending on stabilized zone by jet-grouted columns and rock bolts were used in the Shoryou No. 1 Tunnel (see Fig. 4-2-2 and Table 4-2-1).

Table 4-2-1 Stabilization patterns: Shoryou Tunnel

Tunnel	Pattern	Stabilized zone	Number of fore-poles per cross section	Rock bolt
No. 1 tunnel	Es-R44	180°	n=44	none
	Es-R30	120°	n=30	n=4, L=6m
	Es-R23	90°	n=23	n=16, L=6m
No. 2 tunnel	Es-2	120°	n=21	none

Structural Details

Figure4-2-3 shows the cross and longitudinal sections of the Es-R44 pattern.

The jet-grouted columns are 10m long and 60cm in diameter.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining and invert concrete are 45cm thick and 50cm thick, respectively.

Construction Procedures

The excavation method of the Shoryou No. 1 Tunnel was by heading-and-benching.

As shown in Fig. 4-2-3, a length of 13m was drilled, which was divided into 10m of improved zone and 3m of lost drilling. 9m of the tunnel was excavated before installing the next umbrella arch to maintain a 1-m overlap for the protection of the face.

The jet-grouting parameters are summarized in Table 4-2-2.

Table 4-2-2 Jet-grouting parameters: Shoryou No. 1 Tunnel

Nozzle size	2 nozzles with 18 mm in diameter
Grouting pressure (MPa)	40
Grout volume (liter/m)	365

Source: Koizumi et al. (1990)

The unconfined compressive strength of the jet-grouted columns ranged from 4 to 5MPa.

Field Measurements

Figure 4-2-4 shows the ground surface and tunnel crown settlements in relation to the progress of the face. The depth of overburden above the tunnel crown is 7.1m and the stabilization pattern is Es-R44 shown in Fig. 4-2-2. It should be noted that the ground surface settlement was monitored along with the face advance, while the crown settlement was measured after the primary supports (shotcrete and steel arch support) were installed.

Several observations can be made.

- Settlement as well as heaving occurred during development of the jet-grouted columns
- Maximum heaving of 14mm occurred during development of the jet-grouted columns, then, due to excavation of the top heading, settlement of 15mm from the peak heaving occurred, as a consequence, the final settlement was only 1mm.
- Final settlement of the tunnel crown due to excavation of the top heading was 11mm.
- Final settlements of the ground surface and the tunnel crown after excavation of the bench were 6mm and 16mm, respectively.

Let us consider the ground surface settlement in detail. Figure 4-2-5 shows the relationship between the ground surface settlement and the distance from the face. The following should be noted:

- Two curves were drawn in the figure, one of which was obtained by considering the settlement during development of the jet-grouted columns and the other of which was obtained by ignoring the settlement during development of the jet-grouted columns.
- These curves ignored heaving of 14mm, in other words, they were drawn by assuming that settlement started from the point at which the maximum heaving (14mm) was recorded.
- Hence, the settlement shown in the figure is settlement due to excavation of the top heading.

From Fig. 4-2-5, several observations can be made:

- Large settlement occurred during development of the jet-grouted columns.
- In both curves, settlement gradually occurred after the face advanced from a point about 1.0D (D: tunnel width) ahead of the measuring point and subsided about 2D beyond the measuring point.
- When the settlement during development of the jet-grouted columns was ignored, settlement after excavation of the top heading was 8mm. On the other hand, when the settlement during development of the jet-grouted columns was considered, settlement after excavation of the top heading was 15mm. Therefore, the total amount of the settlement which occurred during development of the jet-grouted columns was 7mm.
- Pre-excavation settlement of 8mm was observed in the case in which the settlement during development of the jet-grouted columns was considered. On the other hand, in the case in which the settlement during development of the jet-grouted columns was ignored pre-excavation settlement of 2mm was observed.

Figure 4-2-6 shows the ground surface and tunnel crown settlements monitored in the sections shown in Fig. 4-2-1, where geological conditions are similar (modulus of deformation ranges from 15MPa to 40MPa, and depth of overburden is between 0.3D and 1.1D). Table 4-2-3 shows the stabilization patterns at the measuring points, i.e., J-1, J-2, etc.

Table 4-2-3 Stabilization patterns at measuring points: Shoryou Tunnel

Points	J-1	J-2	J-3	J-4	J-5	I-1	I-2	I-3
Pattern	Es-R44	Es-R30	Es-R30	Es-R30	Es-R23	Es-2	Es-2	Es-2

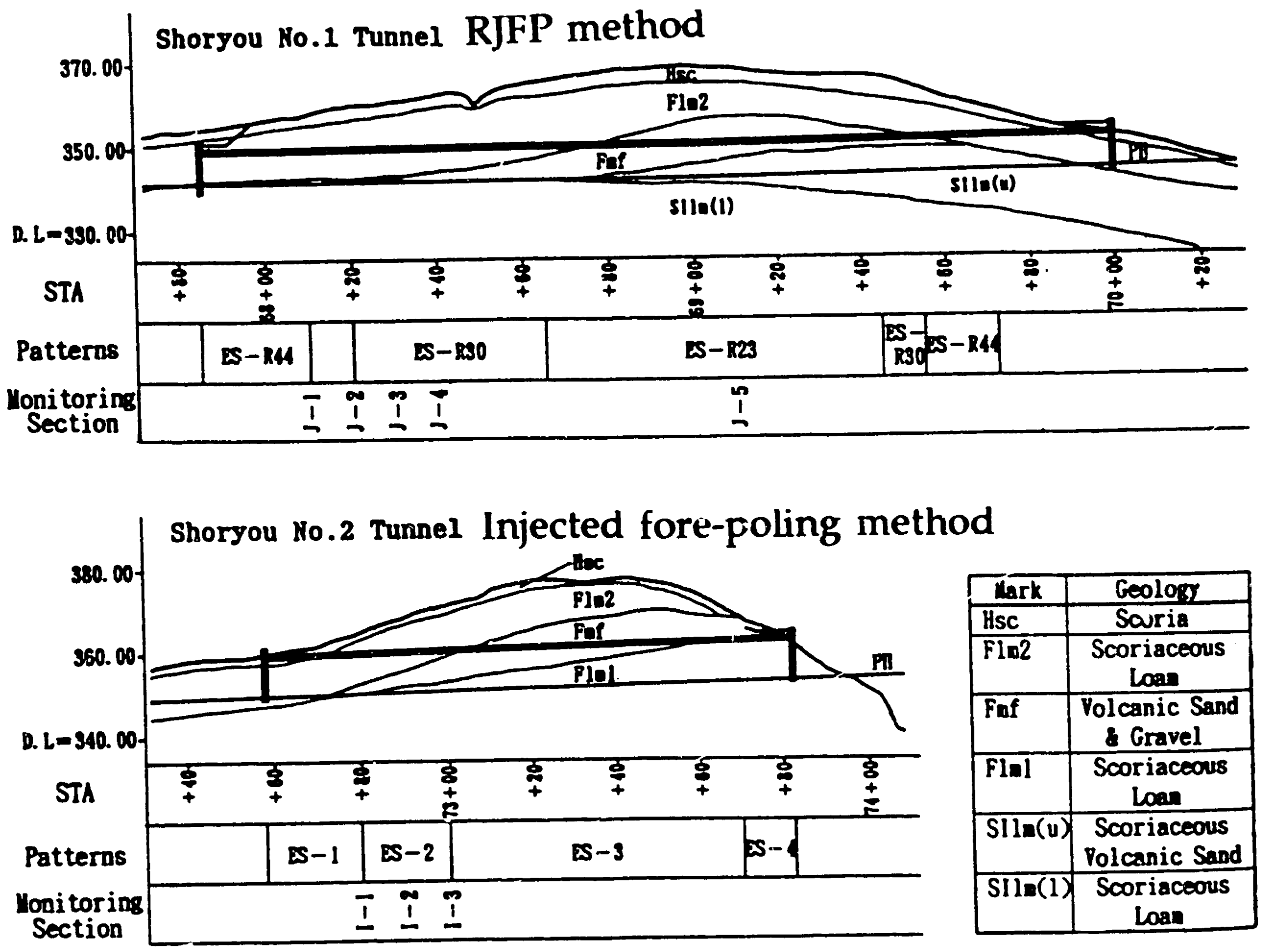
From Fig. 4-2-6, it can be concluded that:

- The pattern with jet-grouted columns ranging over 180° of the tunnel perimeter (Es-R44) showed a clear effect in the restriction of ground surface settlement.
- Between other patterns with jet-grouted columns (Es-R30 and Es-R23) and the pattern with injected fore-poling (Es-2), there was no significant difference in the restriction of ground surface settlement.
- The ground surface settlement which occurred during excavation of the bench was nearly equivalent to the tunnel crown settlement.

References

Koizumi, M., Imamura, O., "Construction of 3-lane Tunnel with Shallow Earth Covering: Shoryou No. 1 Tunnel on Tokyo-Nagoya Expressway- (Part-2)," Tunnels and Underground, Japan Tunnelling Association (JTA), June, 1990.

Kotake, N., Yamamoto, Y., Oka, K., "Design for the Umbrella Method Based on Numerical Analyses and Field Measurements," Tunnelling and Ground Conditions, Abdel Salam (ed.), Balkema, Rotterdam, 1994.



Mark	Geology
Hsc	Scoria
Flm2	Scoriaceous Loam
Fmf	Volcanic Sand & Gravel
Flml	Scoriaceous Loam
Sllm(u)	Scoriaceous Volcanic Sand
Sllm(l)	Scoriaceous Loam

Fig. 4-2-1 Geological conditions: Shoryou Tunnel (Kotake et al., 1994)

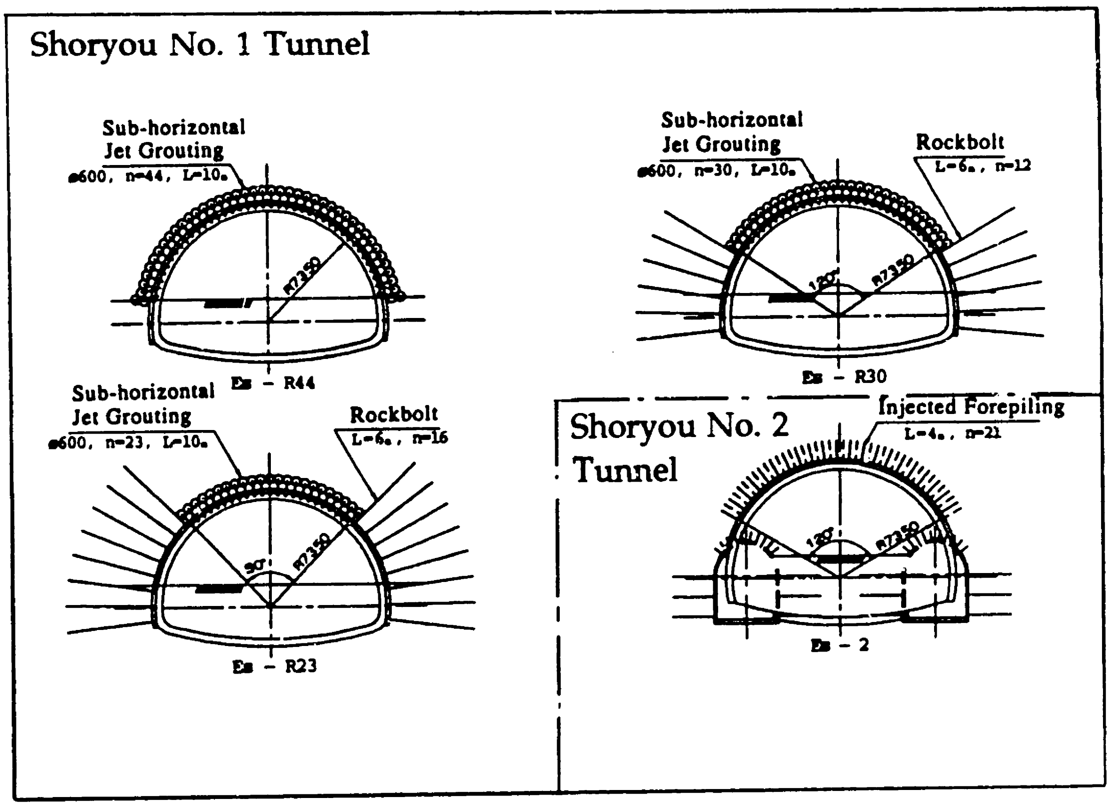


Fig. 4-2-2 Stabilization patterns: Shoryou Tunnel (Kotake et al., 1994)
(see Fig. 4-2-1 for Shoryou No. 2 Tunnel)

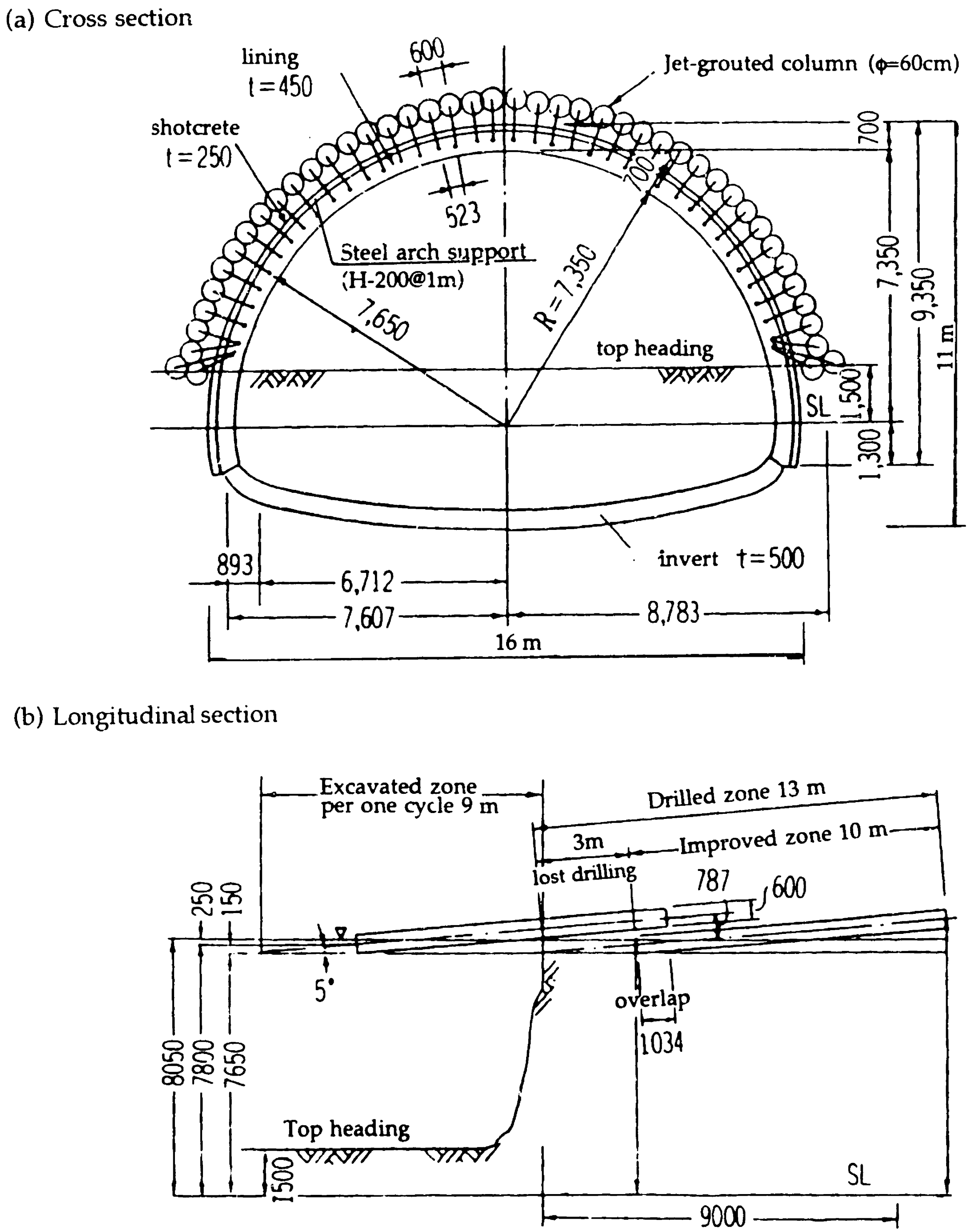


Fig. 4-2-3 Cross and longitudinal sections [Stabilization pattern: Es-R44]: Shoryou No. 1 Tunnel (Koizumi et al., 1990)

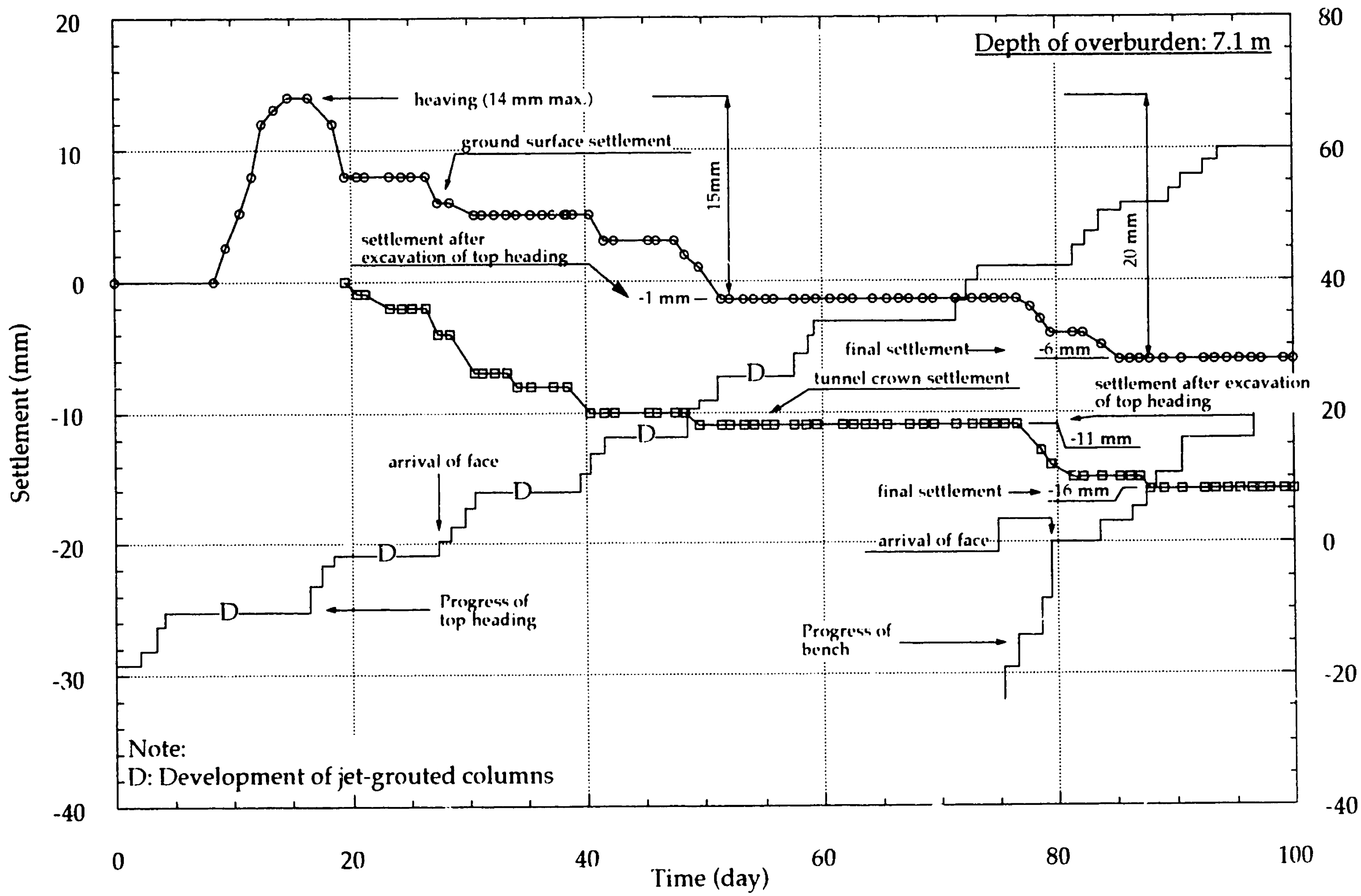


Fig. 4-2-4 Ground surface and tunnel crown settlements in relation to the progress of the face: Shoryou No. 1 Tunnel (Koizumi et al., 1990)

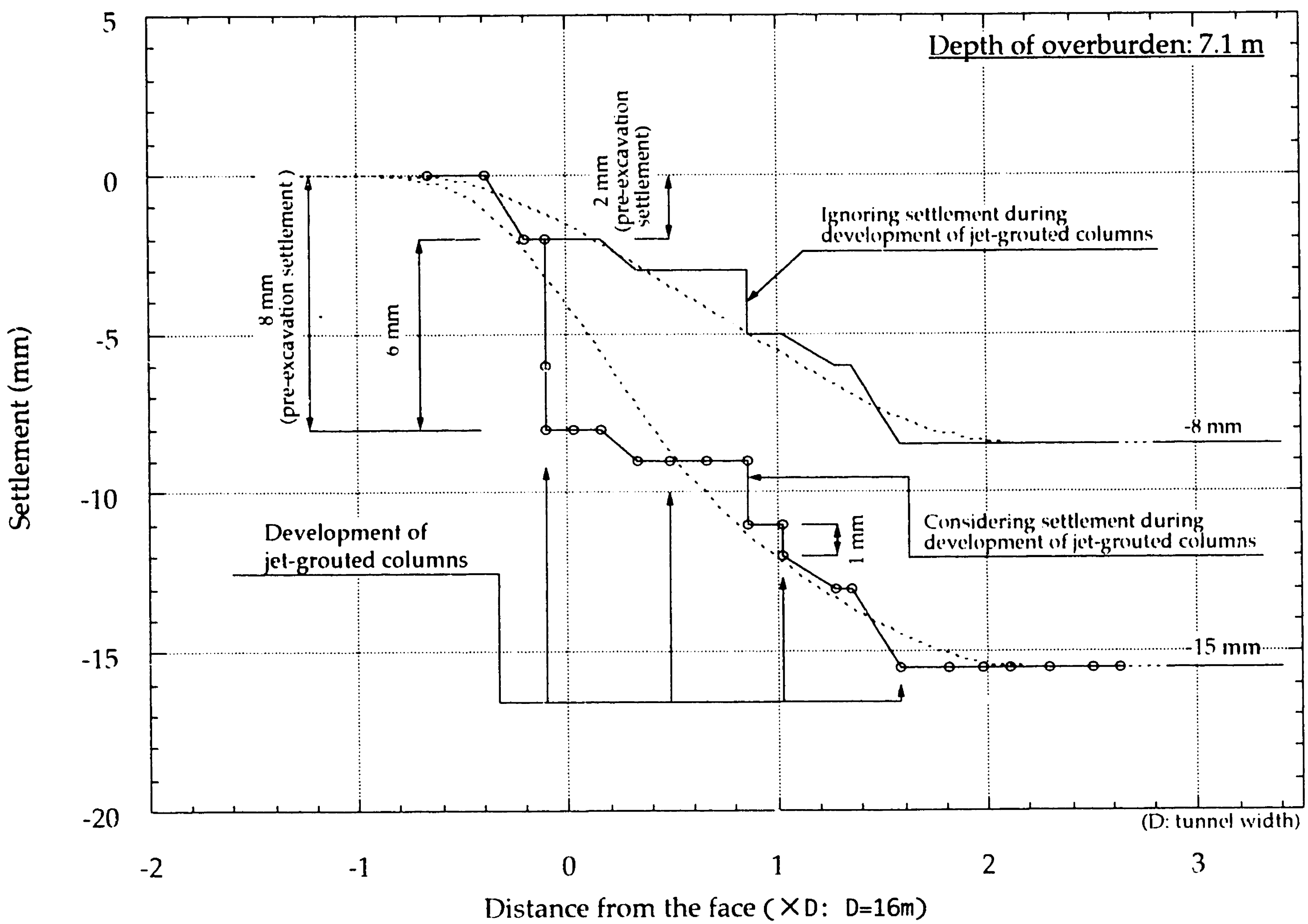
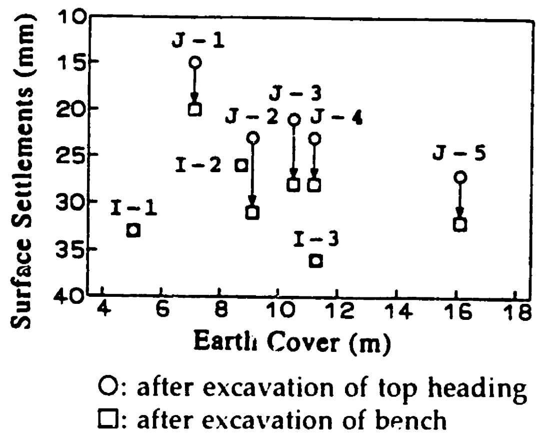


Fig. 4-2-5 Ground surface settlement versus distance from the face: Shoryou No. 1 Tunnel

(a) Ground surface settlement



(b) Tunnel crown settlement

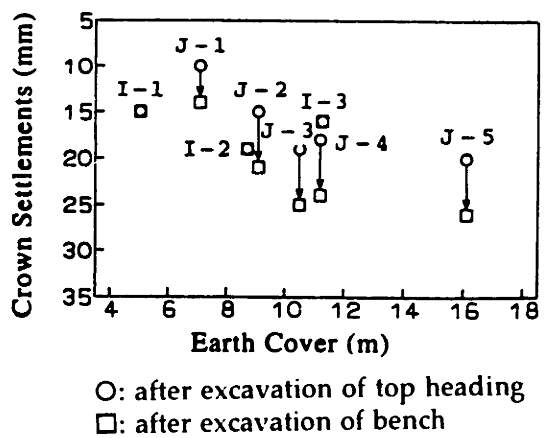


Fig. 4-2-6 Ground surface and tunnel crown settlements for each stabilization pattern: Shoryou Tunnel (Kotake et al., 1994)

Case 3: Kokubugawa Tunnel (Ichikawa, Japan)

Environment of the Kokubugawa Tunnel

The 383-m-long Kokubugawa Tunnel is a part of the underground tunnelled river which discharges into the River Edo. The excavated width and height are both approximately 9m and the excavated area is about 60m^2. The depth of overburden in the zone where the umbrella method was employed ranges from 5m to 16m.

Geological and Hydrological Conditions

The geological conditions of the Kokubugawa Tunnel are shown in Fig. 4-3-1. They consist of the diluvial Narita Formation mostly with uncemented fine, loose sand with SPT-N values of 5 - 20.

The groundwater is located near the tunnel crown.

Problems in Tunnel Construction

The tunnel had to pass under a slope, where a loose fine sand layer existed down to 30m below ground surface and the depth of overburden above the tunnel crown changed from 16m to 5m.

Instability of the tunnel face and slope failures were expected.

Supplementary Support Method of the Tunnel

The sub-horizontal jet-grouting method was employed.

Other methods such as grouting and the pipe roof method were assessed, however, the sub-horizontal jet-grouting method was selected for economical reasons.

The Kokubugawa Tunnel was the first application of the RJFP method in Japan.

Structural Details

The cross and longitudinal sections are shown in Fig. 4-3-2.

The jet-grouted columns are 7m long and 60cm in diameter. The number of the fore-poles per cross section is 34.

Tunnel support consists of a 20-cm-thick primary lining (shotcrete) and H-125 section steel arch supports installed at 1m intervals.

The thickness of secondary lining was not available from the reference.

Construction Procedures

The excavation was by heading-and-benching.

As shown in Fig. 4-3-2, a length of 9.5m was drilled, which was divided into 7m of improved zone and 2.5m of lost drilling. Rounds of 6m of the tunnel were excavated before installing the next umbrella arch to maintain a 1-m overlap for the protection of the face.

The jet-grouting parameters are summarized in Table 4-3-1.

Table 4-3-1 Jet-grouting parameters: Kokubugawa Tunnel

Nozzle size	2 nozzles with 18 mm in diameter
Grouting pressure (MPa)	40
Grout volume (liter/m)	310 - 340

Source: Kizima et al. (1989)

Field Measurements

Figure 4-3-3 shows the ground surface and tunnel crown settlements in relation to the progress of the face. The depth of overburden above the tunnel crown was 9m. It should be noted that the ground surface settlement was monitored with the face advance, while the crown settlement was measured after the primary supports (shotcrete and steel arch support) were installed.

From Fig. 4-3-3, the following can be observed:

- Settlements of the ground surface and the tunnel crown after excavation of the top heading were 47mm and 11mm, respectively.
- Final settlements of the ground surface and the tunnel crown were 52mm and 27mm, respectively.

Figure 4-3-4 shows the relationship between the ground surface settlement and the distance from the face. It should be noted that:

- Two curves were drawn in the figure, one of which was obtained by considering the settlement during development of the jet-grouted columns and the other of which was obtained by ignoring the settlement during development of the jet-grouted columns.
- The settlement shown in the figure is settlement due to excavation of the top heading.

From Fig. 4-3-4, several observations can be made:

- Large settlements occurred during development of the jet-grouted columns.
- In both curves, settlement gradually increased after the face advanced from a point about 2D (D: tunnel width) ahead of the measuring point and subsided about 2D beyond the measuring point.
- In the case in which the settlement during development of the jet-grouted columns was considered, settlement after excavation of the top heading was 47mm. On the other hand, in the case in which the settlement during development of the jet-grouted columns was ignored, settlement after excavation of the top heading was 21mm. Therefore, the total amount of the settlement which occurred during development of the jet-grouted columns was 26mm.
- Pre-excavation settlement of 27mm was observed in the case in which the settlement during development of the jet-grouted columns was considered. On the other hand, in the case in which the settlement during development of the jet-grouted columns was ignored, pre-excavation settlement of 9.2mm was observed.
- As mentioned previously, when the settlement which occurred during development of the jet-grouted columns was considered, final settlements of the ground surface and the tunnel crown were 52mm and 27mm, respectively. From the fact that the pre-excavation settlement of the ground surface was 27mm, settlement of 25mm occurred after passage of the face. From comparison of this value (25mm) with final settlement of the tunnel crown (27mm), one can conclude that the ground surface went down in accordance with the downward movement of the tunnel crown.

<u>Reference</u>

Kizima, Y., Aoki, T. et al., "Tunnel Excavation by NATM in Urban Areas: Kokubugawa Tunnel," <u>Tunnels and Underground</u>, Japan Tunnelling Association (JTA), July, 1989.

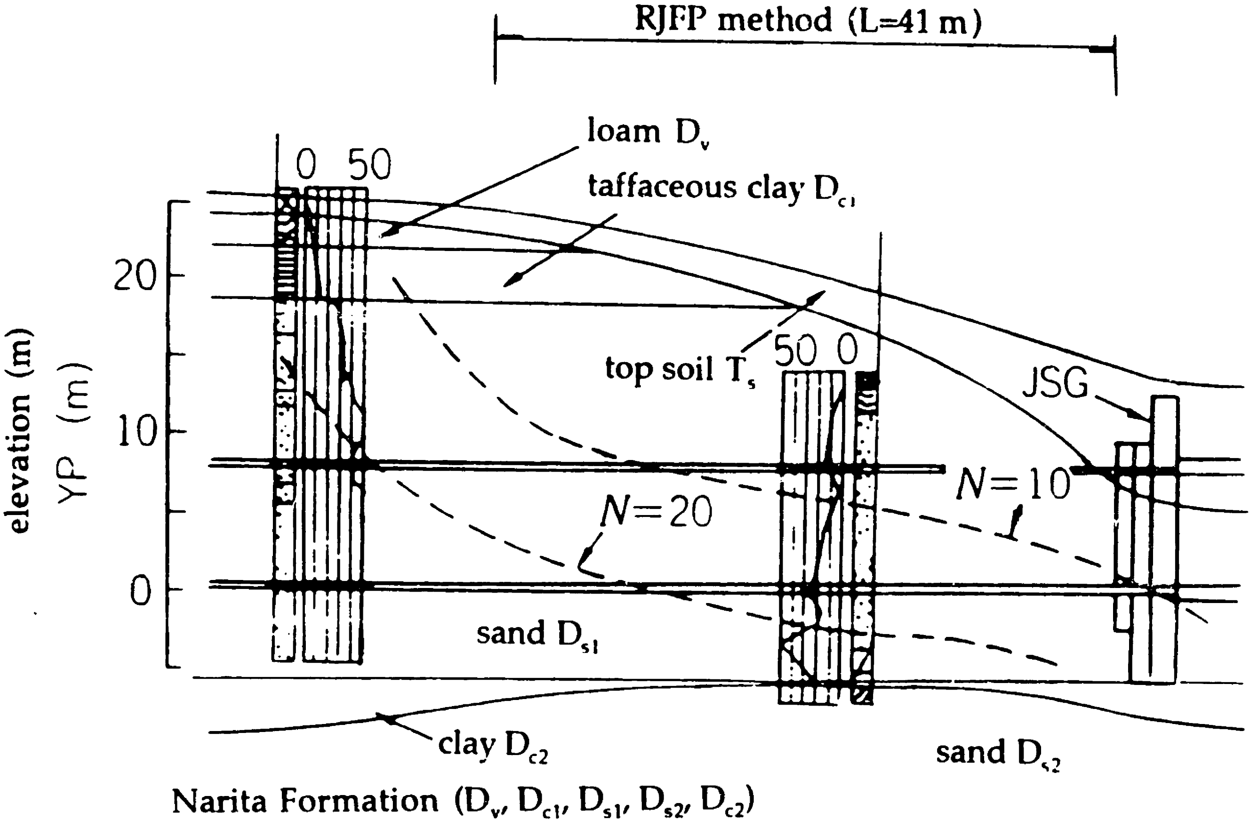
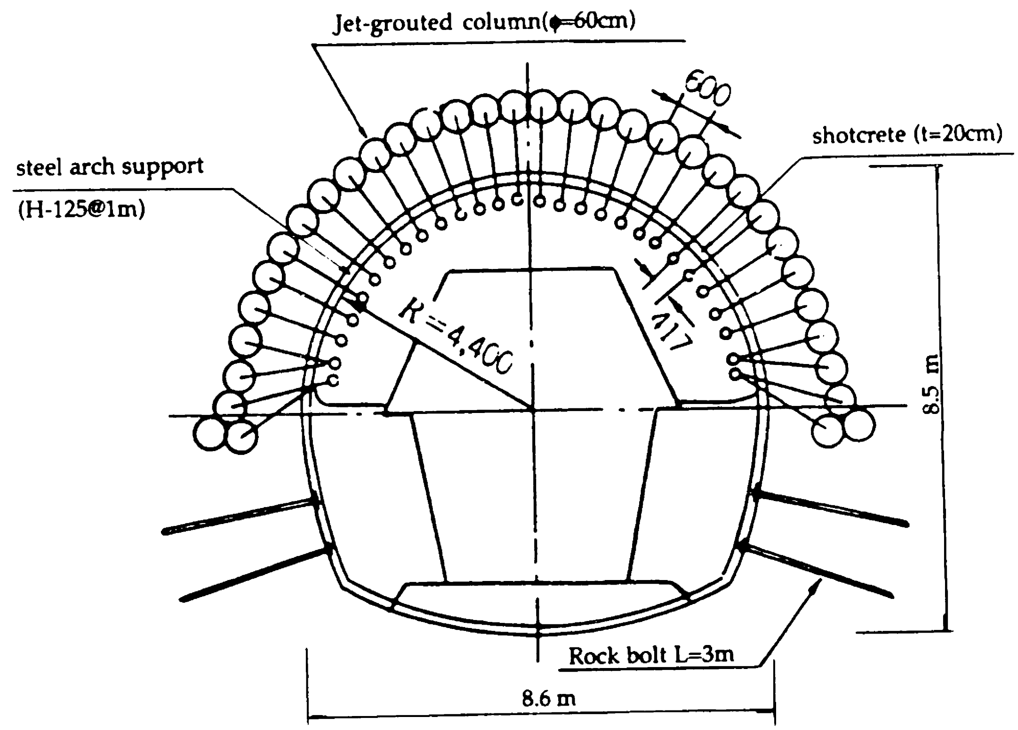


Fig. 4-3-1 Geological conditions : Kokubugawa Tunnel (Kizima et al., 1989)

(a) Cross section



(b) Longitudinal section

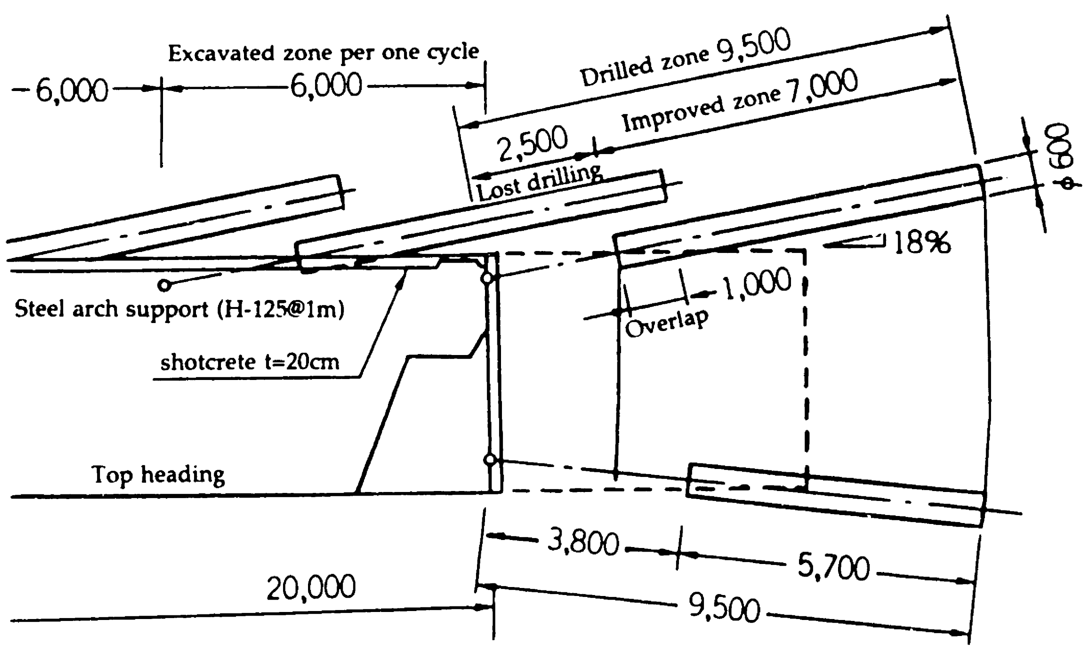


Fig. 4-3-2 Cross and longitudinal sections: Kokubugawa Tunnel (Kizima et al., 1989)

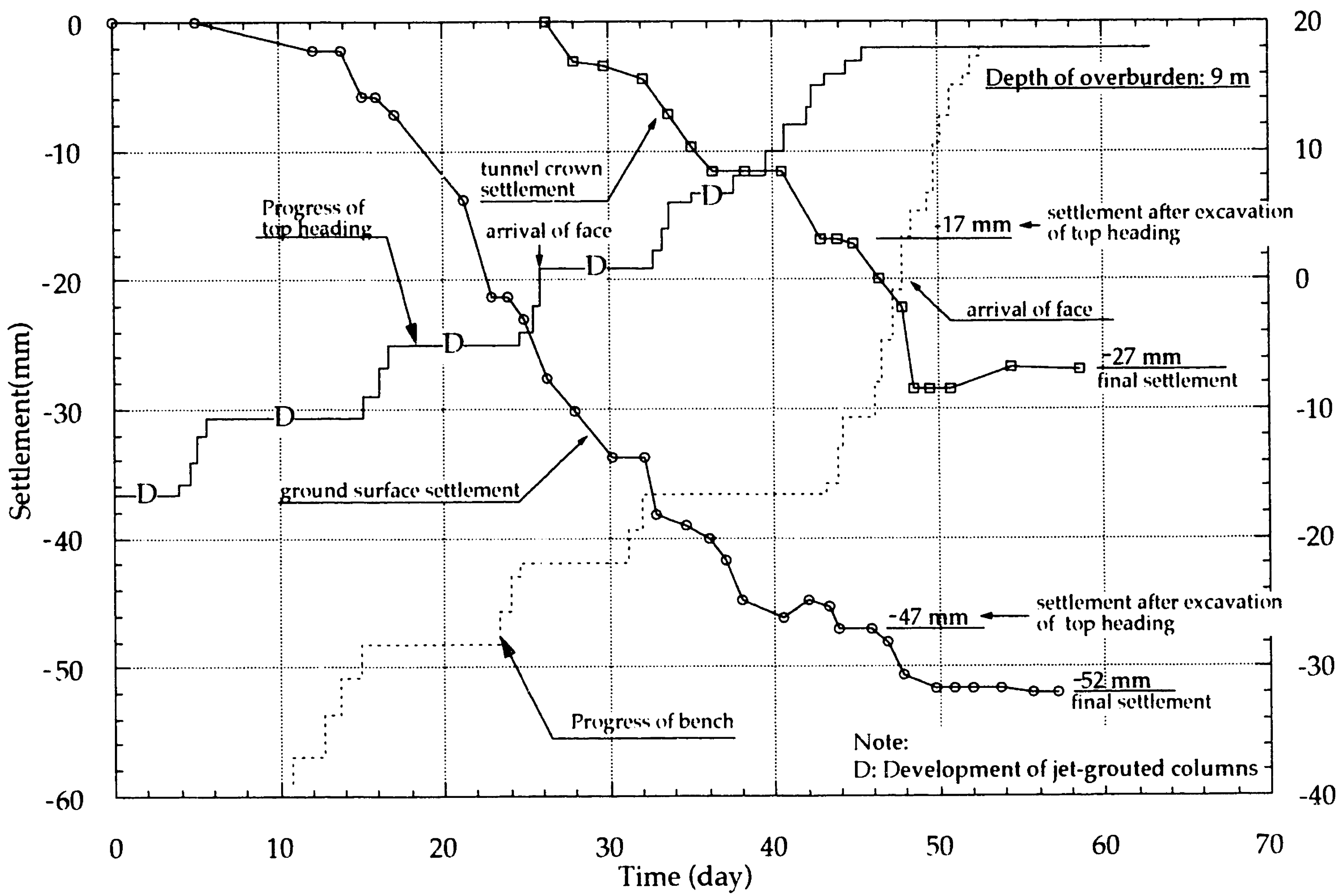


Fig. 4-3-3 **Ground surface and tunnel crown settlements in relation to the progress of the face: Kokubugawa Tunnel (Kizima et al., 1989)**

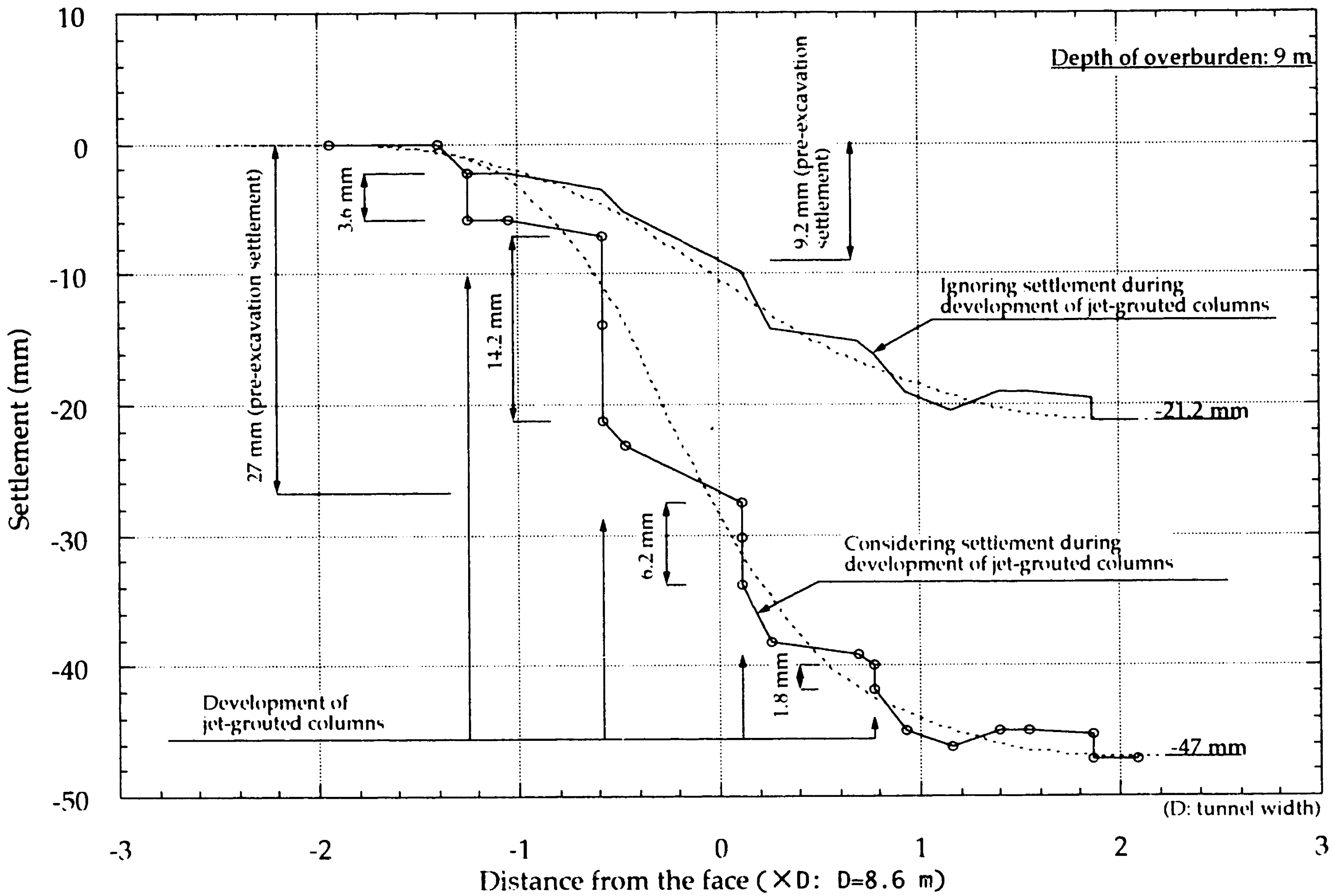


Fig. 4-3-4 Ground surface settlement versus distance from the face: Kokubugawa Tunnel

Case 4: Owani Tunnel (Owani, Japan)

Environment of the Owani Tunnel

The 145-m-long Owani Tunnel is a motorway tunnel. The excavated width and height are approximately 16m and 10m, respectively and the excavated area is 129m^2. Depth of overburden ranges from 3.5m to 15m, which is very shallow compared to the tunnel width.

Geological and Hydrological Conditions

The geological conditions of the tunnel are shown in Fig. 4-4-1.

The geology in the zone where the umbrella method was employed consists of three layers: 1) talus deposits mostly consisting of pumice and clay with gravels; 2) clay; and 3) gravel.

The average SPT-N value of the talus deposits is 18.

Problems in Tunnel Construction

Because of the shallow overburden above the tunnel crown (mean 9m) and loose ground, large ground surface settlements and instability of the face were expected.

Supplementary Support Method of the Tunnel

The sub-horizontal jet-grouting method (the RJFP method) was employed.

Other methods such as the pipe roof method, the injected steel pipe umbrella method and grouting were assessed. The sub-horizontal jet-grouting method was selected because it was considered to be most effective to both prevent slope failures and restrict ground deformation.

Structural Details

The cross and longitudinal sections are shown in Fig. 4-4-2.

The jet-grouted columns are 7m long and 60cm in diameter. The number of the fore-poles per cross section is 36.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 50cm thick.

Construction Procedures

The excavation was by side drift method.

As shown in Fig. 4-4-2, a length of 9.5m was drilled, which was divided into 7m of improved zone and 2.5m of lost drilling. Rounds of 6m of the tunnel were excavated before installing the next umbrella arch to maintain a 1-m overlap for the protection of the face.

The jet-grouting parameters are summarized in Table 4-4-1.

Table 4-4-1 Jet-grouting parameters: Owani Tunnel

Grouting pressure (MPa)	40
Grout volume (liter/m)	310 - 340

Source: Geo-Fronte Research Association (1994)

Field Measurements

Final settlements of the ground surface and the tunnel crown were 30mm and 44mm, respectively.

No other information was available from the reference.

Reference

Geo-Fronte Research Association, Technical Report on the Rodin Jet Fore-Poling Method, January, 1994.

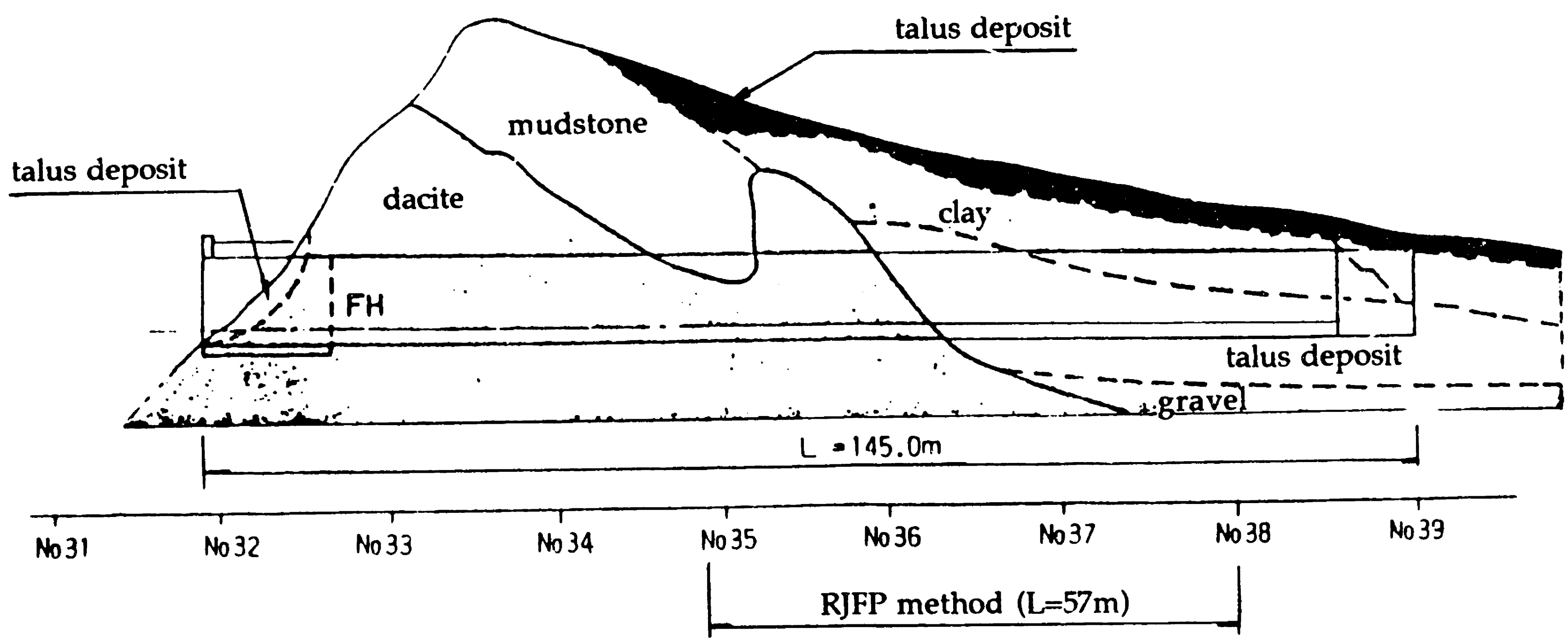
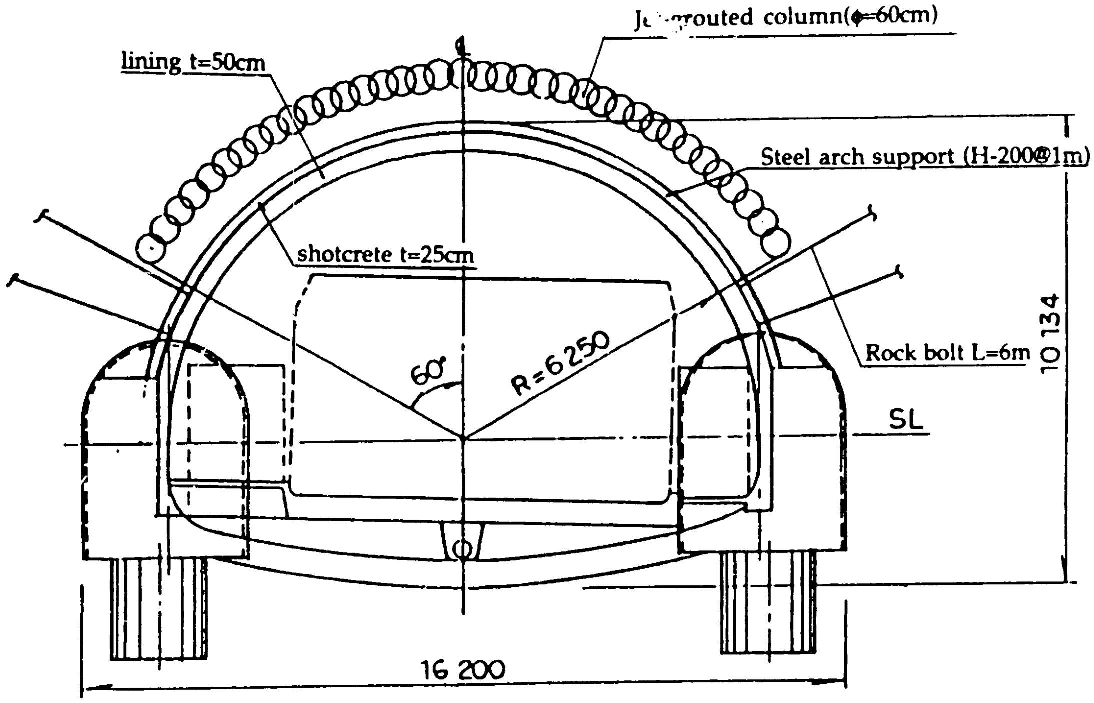


Fig. 4-4-1 Geological conditions: Owani Tunnel (Geo-Fronte Research Association, 1994)

(a) Cross section



(b) Longitudinal section

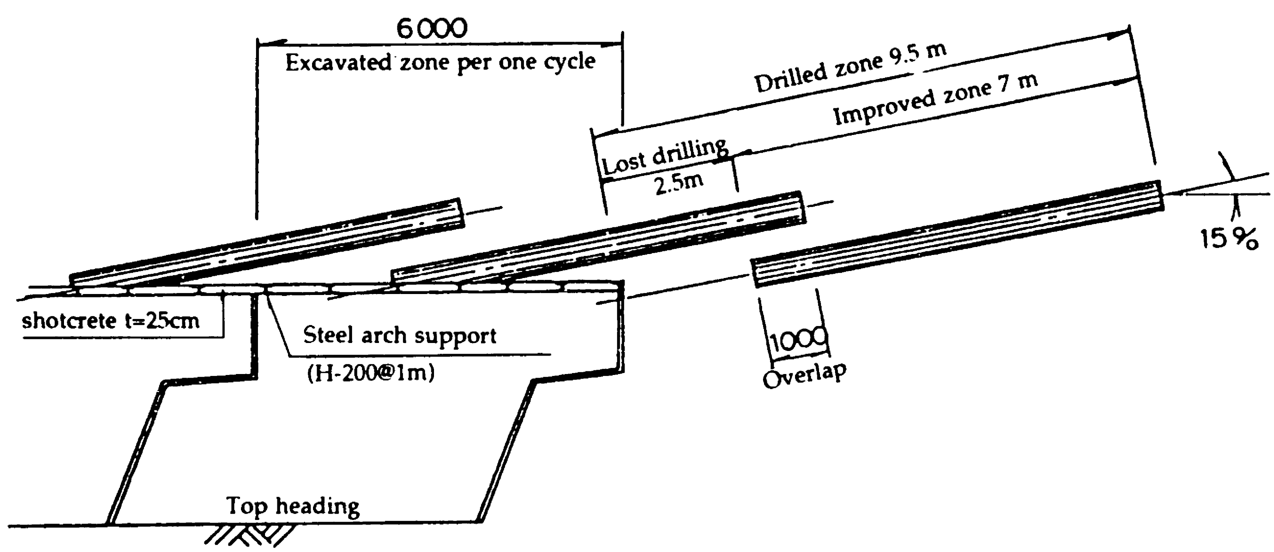


Fig. 4-4-2 Cross and longitudinal sections: Owani Tunnel
(Geo-Fronte Research Association, 1994)

Case 5: Aziro Tunnel (Itsukaichi, Japan)

Environment of the Aziro No. 1 Tunnel

The 563-m-long Aziro Tunnel is a 2-lane motorway tunnel. The excavated width and height are approximately 12m and 9m, respectively and the excavated area is about 90m^2. The depth of overburden in the zone where the umbrella method was employed is about 17m.

Geological and Hydrological Conditions

The geological conditions of the tunnel are shown in Fig. 4-5-1.

As shown in Table 4-5-1, the geology can be roughly divided into five strata, all of which except the fill stratum are the diluvial deposits.

Table 4-5-1 Soil classification: Aziro Tunnel

Soil type	SPT-N
fill (Bk)	1 - 8
loam (Lm)	2 - 5
sand with gravel, clay (Tg)	2 - 14
silt (Im)	N/A
gravel (Ig)	40 - 100

Source: Shimizu et al. (1991)

Problems in Tunnel Construction

The tunnel passed through the gravel layer (Ig) with interposed layer of silt (Im) and under the green of a golf course.

The depth of overburden was about 17m, which included the 10-m fill. A supplementary method for tunnel excavation was needed to prevent the settlement of the green.

Supplementary Support Method of the Tunnel

Four alternative supplementary methods were evaluated: the pipe roof method; grouting; injected fore-poling; and the RJFP.

The sub-horizontal jet-grouting method (the RJFP method) was selected because it was thought to be most effective in restricting ground settlement.

Structural Details

The cross and longitudinal sections are shown in Fig. 4-5-2.

The jet-grouted columns are 10m long and 60cm in diameter. The number of the fore-poles per cross section is 36.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 35cm thick.

Construction Procedures

The excavation was by heading-and-benching.

As shown in Fig. 4-5-2, a length of 13m was drilled, which was divided into 10m of improved zone and 3m of lost drilling. Rounds of 9m of the tunnel were excavated before installing the next umbrella arch to maintain a 1-m overlap for the protection of the face.

The jet-grouting parameters are summarized in Table 4-5-2.

Table 4-5-2 Jet-grouting parameters: Aziro Tunnel

Nozzle size		2 nozzles with 1.8/2.2 mm in diameter
Grouting pressure (MPa)		40
Grout volume (liter/m)		310 - 340
Rotation speed of drilling rod (rpm)		18 - 20
Withdrawal speed (min./m.)		3 - 6
Grout (kgf/m^3)	cement	760
	additive	12
	water	750

Source: Shimizu et al. (1991)

Field Measurements

Final settlements of the ground surface and tunnel crown were approximately 10mm and 8mm, respectively.

Tunnel crown settlement after excavation of the top heading was 5mm.

Reference

Shimizu, A., Takahashi, Y. et al., "Conquest of Soft Sedimentary Loam at a Tunnel Portal: Aziro Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), December, 1991.

Legend	
Bk	fill
Lm	loam
Tg	sand with gravel, clay
Im	silt
Ig	gravel

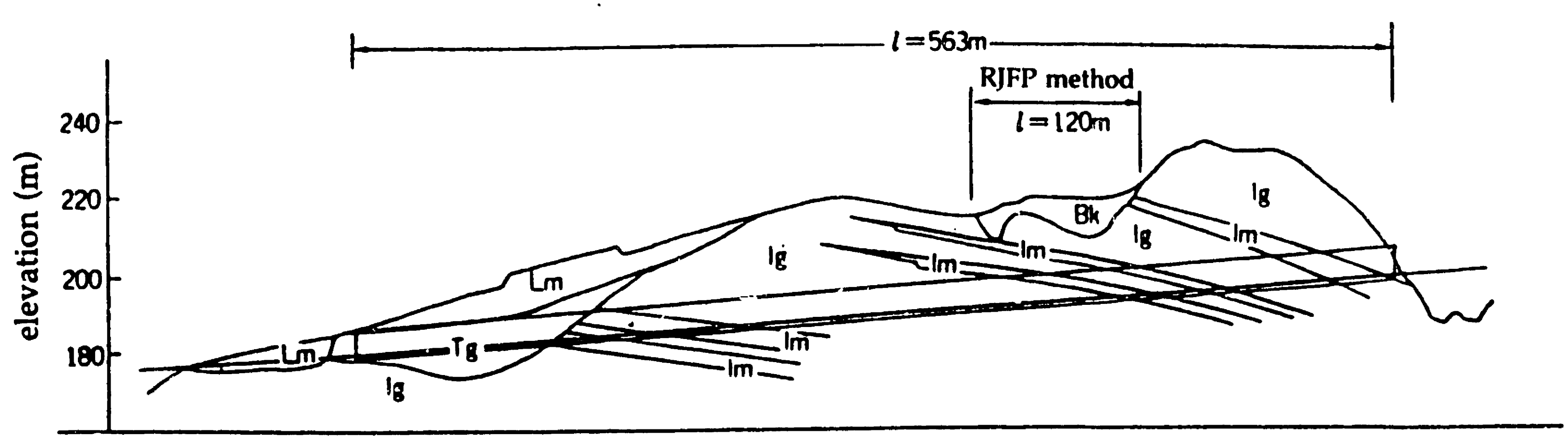
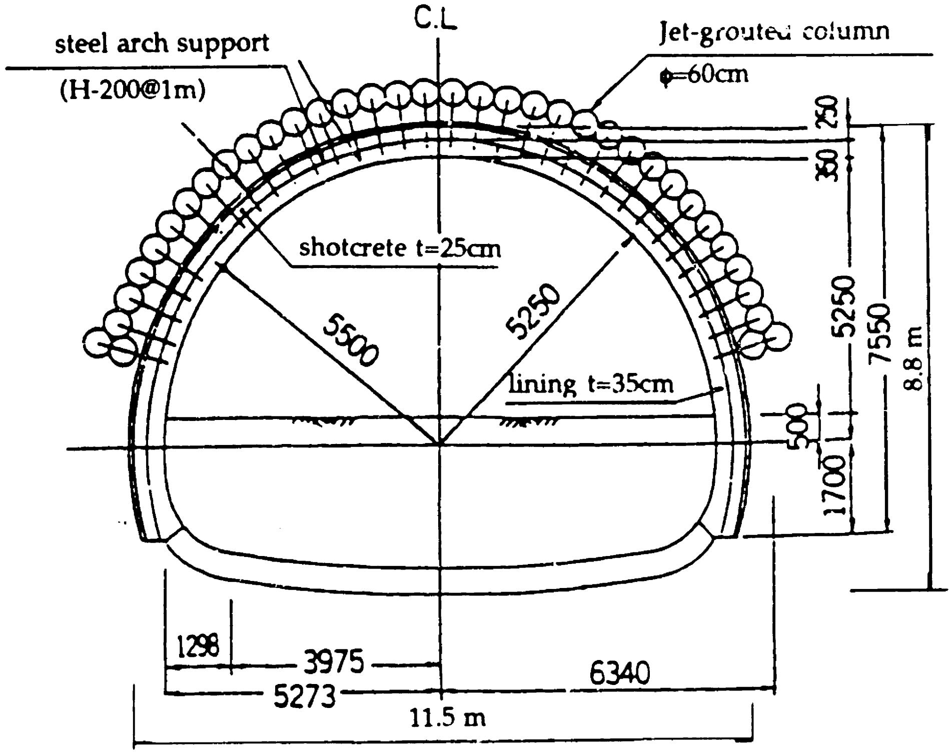


Fig. 4-5-1 Geological conditions: Aziro Tunnel (Shimizu et al., 1991)

(a) Cross section



(b) Longitudinal section

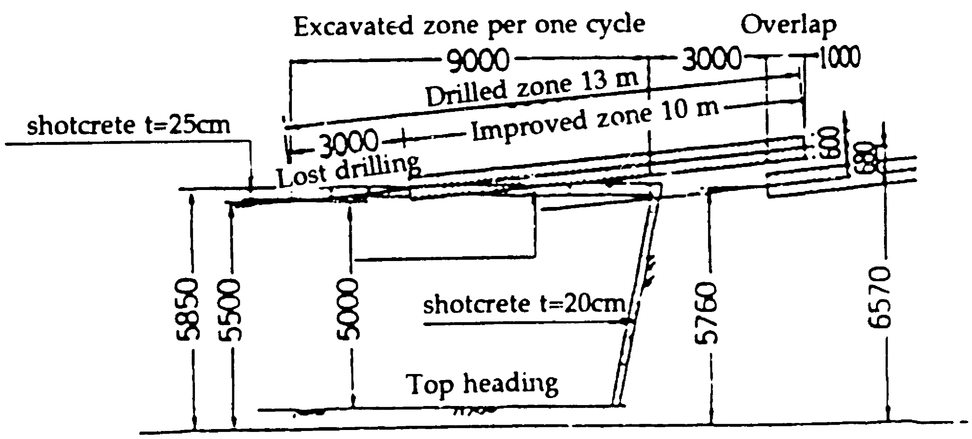


Fig. 4-5-2 Cross and longitudinal sections: Aziro Tunnel (Shimizu et al., 1994)

Case 6: Uryuya Tunnel (Sonobe, Japan)

Environment of the Uryuya Tunnel

The 185-m-long Uryuya Tunnel is a motorway tunnel with an excavated width and height of approximately 15m and 10m, respectively and an excavated area of about 120m^2. The depth of overburden in the zone where the umbrella method was employed ranges from 0m to 40m.

Geological and Hydrological Conditions

The geological conditions of the Uryuya Tunnel are shown in Fig. 4-6-1.

At the tunnel portal the top portion of the tunnel passes through the clay layer with an average SPT-N value of 10 and the middle and bottom portions of the tunnel pass through an extremely weathered shale layer with an average SPT-N value of 8.

Problems in Tunnel Construction

A slope failure occurred during cutting of a slope at the tunnel portal before tunnel excavation started (see Fig. 4-6-1). This indicated that larger failures might occur due to the tunnel excavation unless countermeasures were employed. Moreover, due to shallow overburden above the tunnel crown, instability of the face was expected.

A supplementary support method was needed to prevent such failures and to improve face stability.

Supplementary Support Method of the Tunnel

Three alternative supplementary methods were evaluated: the pipe roof method; grouting; and the sub-horizontal jet-grouting method.

The grouting method was rejected due to the very low permeability of the ground. The pipe roof method was rejected because it was estimated from the geotechnical investigation that there was a possibility that large unweathered rock blocks existed in the clay layer and hence directional accuracy for placing piles was not guaranteed. The sub-horizontal jet-grouting method (the RJFP method) was selected because of its high adaptability to the existing conditions.

Structural Details

The cross and longitudinal sections are shown in Fig. 4-6-2.

The jet-grouted columns are 10m long and 60cm in diameter. The number of the fore-poles per cross section is 29.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H section steel arch supports (dimensions are unknown) installed at 1m intervals. The secondary lining is 45cm thick.

Construction Procedures

The excavation was by side drift method.

As shown in Fig. 4-7-2, a length of 13m was drilled. This was divided into 10m of improved zone and 3m of lost drilling. Rounds of 9m of the tunnel were excavated before installing the next umbrella arch to maintain a 1-m overlap for the protection of the face.

The jet-grouting parameters are summarized in Table 4-6-1.

Table 4-6-1 Jet-grouting parameters: Uryuya Tunnel

Grouting pressure (MPa)		40 - 45
Grout volume (liter/m)		424
Rotation speed of drilling rod (rpm)		15 - 16
Withdrawal speed (min./m.)		6
Grout (kgf/m^3)	cement	760
	additive	12
	water	750

Source: Yamashita et al. (1994)

7. Field Measurements

Final settlement of the tunnel crown, where the depth of overburden was about 13m, was 7mm.

Reference

Yamashita, Y., Shiomi, K. et al., "Supplementary Methods in the Uryuya Tunnel," Technical Report on the Rodin Jet Fore-Poling Method, Geo-Fronte Research Association, January, 1994.

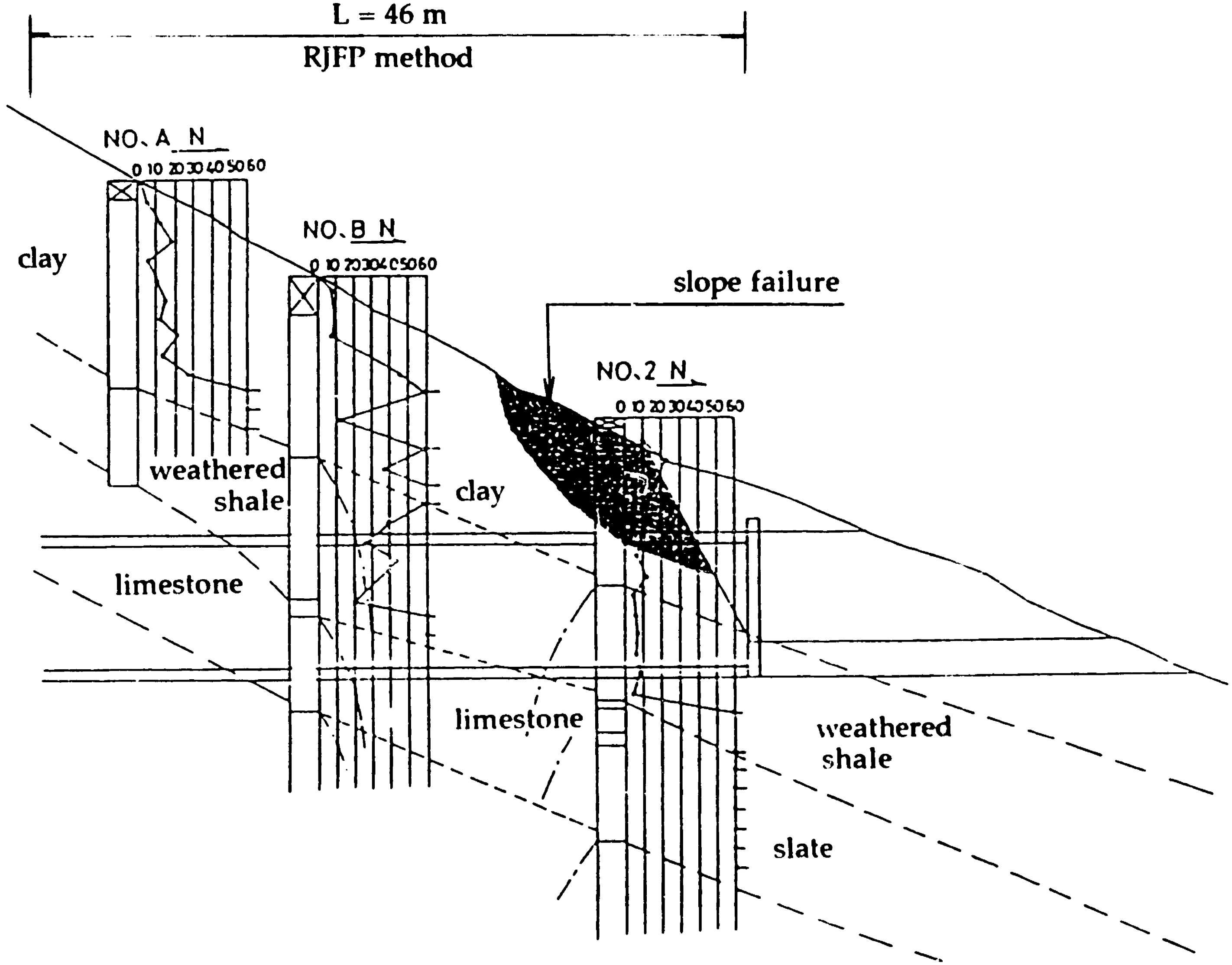
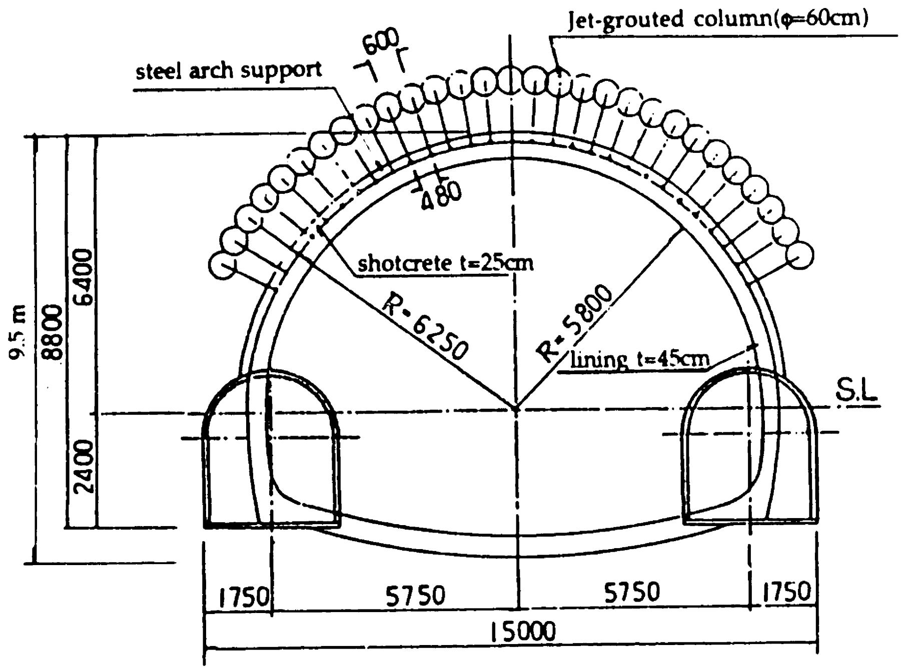


Fig. 4-6-1 Geological conditions: Uryuya Tunnel (Yamashita et al., 1994)

(a) Cross section



(b) Longitudinal section

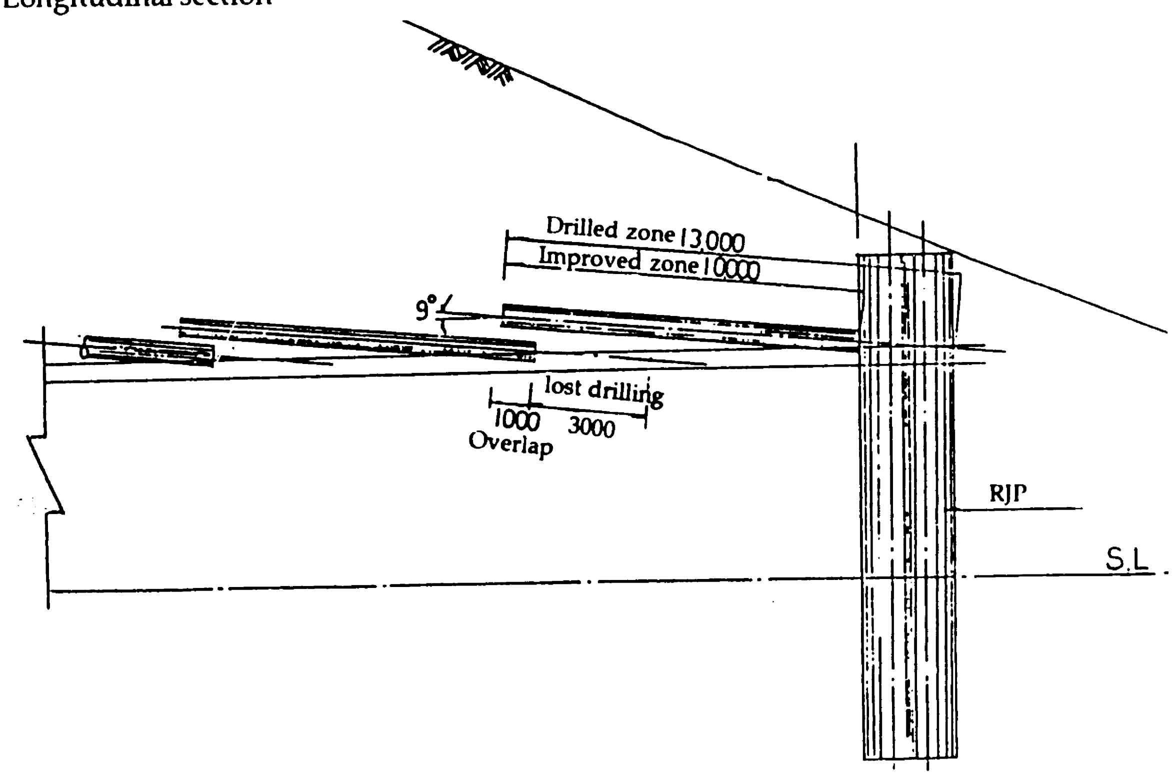


Fig. 4-6-2 Cross and longitudinal sections: Uryuya Tunnel (Yamashita et al., 1994)

Case 7: Subway Vienna (Vienna, Austria)

Environment of the Subway Vienna

As shown in Fig. 4-7-1, Rochusgasse Station in Vienna, Austria, was constructed as a mined tunnel. A portion of the station runs beneath a high-rise structure close to subsurface parking facilities. The excavated width and height of the tunnel are 11m and 9m respectively and the excavated area is about $80m^2$.

Geological and Hydrological conditions

Close to the surface are loams, overlaying layers of sandy gravels. Below these down to great depth silty sands and clayey silts alternate.

The groundwater table is approximately 12m below the ground surface. In order to reduce the tunnelling risks, dewatering was carried out.

Problems in Tunnel Construction

Because of the large cross section, the proximity to building foundations and the soil conditions, ground improvement techniques were needed to ensure safe excavation and to minimize surface settlement.

Supplementary Support Method of the Tunnel

A 2-m grouted zone around the tunnel was designed for the tender documents. In order to avoid the hazard of ground water contamination caused by chemical grouting, the sub-horizontal jet-grouting method was used as an alternative design. For design purposes a compressive strength of 5MPa and pile diameters of 0.5m to 0.8m were planned.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-7-2.

The jet-grouted columns are 10 to 15m long and 80cm in diameter. The number of the fore-poles per cross section is 26.

Tunnel support consists of a 30-cm-thick primary lining (shotcrete) and steel arch supports (dimensions and installation intervals are unknown).

Construction Procedures

There was a 3-m-long overlap between two consecutive umbrellas in the longitudinal direction.

A testing program was initiated to verify the design assumptions. When grouting pressures were 50MPa, the diameter of a jet-grouted column was 0.55m, exceeding the minimum requirement mentioned previously.

Field Measurements

The umbrella method contributed significantly to the reduction of settlement, which did not exceed 10mm.

The loading of the primary lining (shotcrete) due to earth pressures, temperature, and shrinkage was evaluated. Jet-grouted columns were found to contribute considerably to the overall support while exposing the shotcrete to relatively minor loading. Stresses in the shotcrete reached only 10% of the failure load and strains about 15% of the ultimate strain.

Reference

Mussger K., Koining J., Reischl St., "Jet Grouting in Combination with NATM," Proceedings of Rapid Excavation and Tunneling Conference (RETC), Vol. 1, 1987.

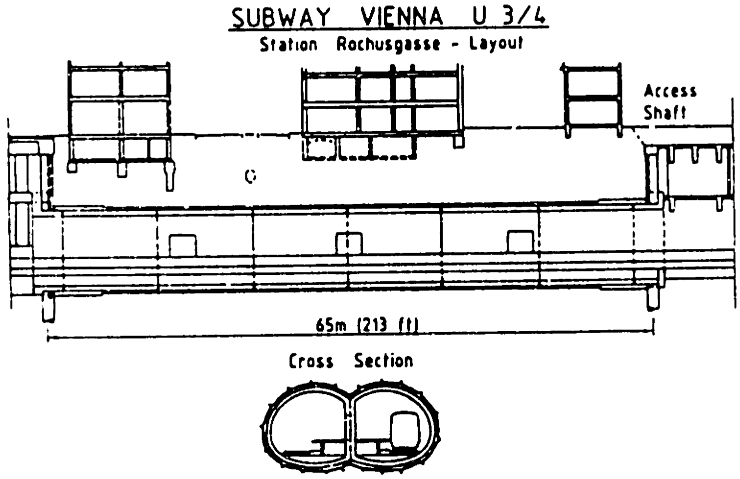


Fig. 4-7-1 Layout of the Station Rochusgasse: Subway Vienna (Mussger et al., 1987)

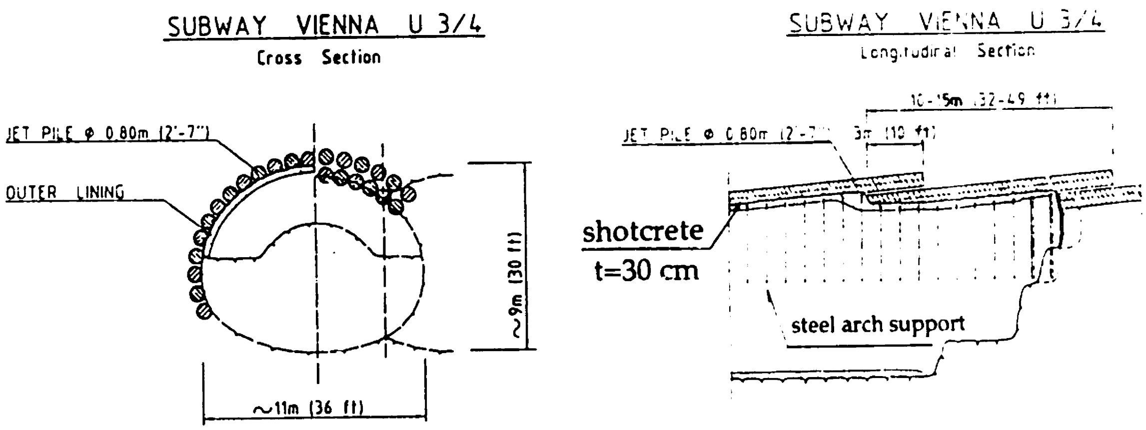


Fig. 4-7-2 Cross and longitudinal sections: Subway Vienna (Mussger et al., 1987)

Case 8: Rengershausen Tunnel (Kassel, Germany)

Environment of the Rengershausen Tunnel

The 1600-m-long Rengershausen Tunnel is part of the Germany's new railroad line between Hannover and Wurzburg. The excavated width and height are 15m and 13m, respectively and the excavated area is approximately $150m^2$.

Geological and Hydrological Conditions

The geological conditions of the Rengershausen Tunnel are shown in Fig. 4-8-1.

The tertiary formation ("T" in the figure) can be divided into several layers from bottom to top: 1) fine and medium grained sandstone strata; 2) a clay stratum with a thickness of 5 to 10m; 3) a clay stratum of 5 to 10m thickness overlaying the previous clay stratum and 4) alluvial deposits consisting of a series of sandy, silty, clayey soils and sandy gravels.

Problems in Tunnel Construction

In the northernmost section of the tertiary formation the overburden consisting of clay and sand reached the tunnel crown and was underlain by a layer of fine sand.

Excavation was in a zone with a residual friction angle of $\phi=10°$ and no cohesion. It was felt that normal excavation procedures would not be sufficient to warrant safe excavation conditions.

Supplementary Support Method of the Tunnel

To overcome the above problems, the sub-horizontal jet-grouting method was employed.

Chemical grouting was rejected due to potential groundwater contamination.

Ground freezing was expected to create problems in the invert.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-8-2.

The jet-grouted columns are 10.5m long and 60cm in diameter. The number of the fore-poles per cross section is 28.

Tunnel support consists of a 40-cm-thick primary lining (shotcrete) and steel arch supports (dimensions and installation intervals are unknown).

Construction Procedures

The excavation was by side drift method.

A length of 15m was drilled. This was divided into 10.5m of improved zone and 4.5m of lost drilling. 8.5m of the tunnel section was excavated before installing the next umbrella arch in order to maintain a 2-meter overlap for the protection of the face.

Field Measurements

No information on field measurements was available from the reference.

Reference

Mussger K., Koining J., Reischl St., "Jet Grouting in Combination with NATM," Proceedings of Rapid Excavation and Tunneling Conference (RETC), Vol. 1, 1987.

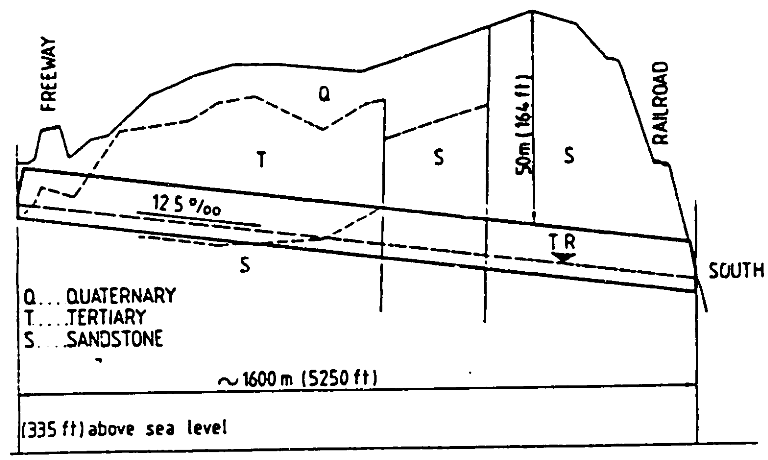


Fig. 4-8-1 Geological conditions : Rengershausen Tunnel (Mussger et al., 1987)

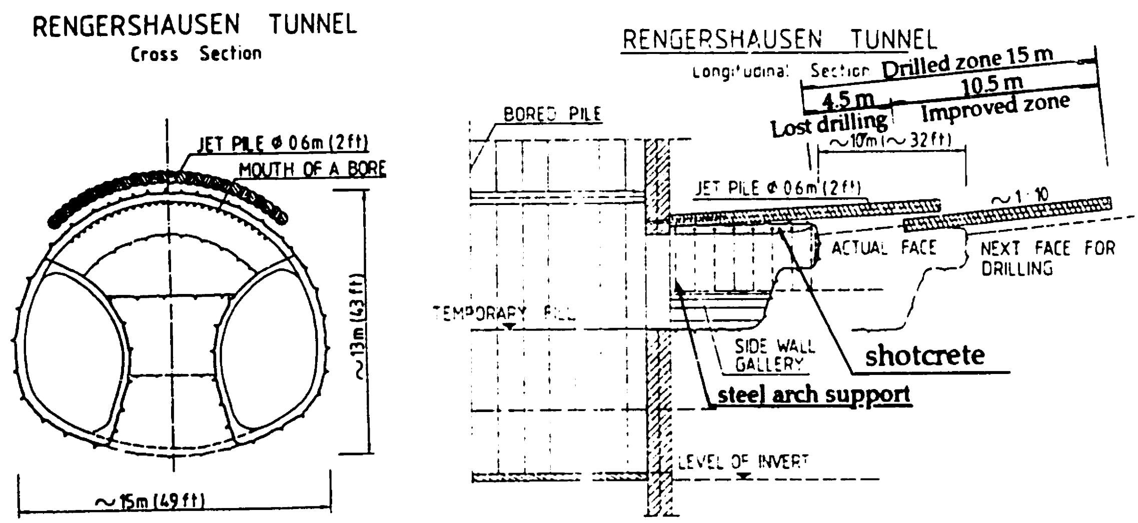


Fig. 4-8-2 Cross and longitudinal sections: Rengershausen Tunnel (Mussger et al., 1987)

Case 9: Campiolo Tunnel (Moggio Udinese, Italy)

Environment of the Campiolo Tunnel

The 170-m-long Campiolo Tunnel is a railway tunnel located in Moggio Udinese, Northeastern Italy. The excavated width and height are approximately 12m and 11m, respectively and the excavated area is approximately 100m^2. The depth of overburden above the tunnel crown ranges from 2m to 70m.

Geological and Hydrological Conditions

The ground through which the tunnel passes is detrital and mostly cohesionless soil consisting of calcareous rock fragments (up to 10 - 20cm in size) in a silty-sandy matrix.

Problems in Tunnel Construction

No information was available from the references.

Supplementary Support Method of the Tunnel

The sub-horizontal jet-grouting method (the RJFP method) was employed. At that time, it was the first major application of the RJFP.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-9-1.

The jet-grouted columns are 13m long. The number of the fore-poles per cross section is 41.

H-200 section steel arch supports were installed at 1m intervals.

Construction Procedures

10m of the tunnel section was excavated before installing the next umbrella arch to maintain a 3-m overlap for the protection of the face.

The footings of the H-section steel arch support were stabilized by means of jet-grouted columns.

Field Measurements

The continuity and mechanical properties of the jet-grouted arch were such that the steel arch supports remained virtually unloaded.

Reference

Mongilardi, E., Tornaghi, R., "Construction of Large Underground Openings and Use of Grouts," Proceedings of International Conference on Deep Foundations, Vol. 1, 1986.

Tornaghi, R., Perelli, C.A., "Soil Improvement by Jet Grouting for the Solution of Tunnelling Problems," Proceedings of the 4th International Symposium Tunnelling'85, 1985.

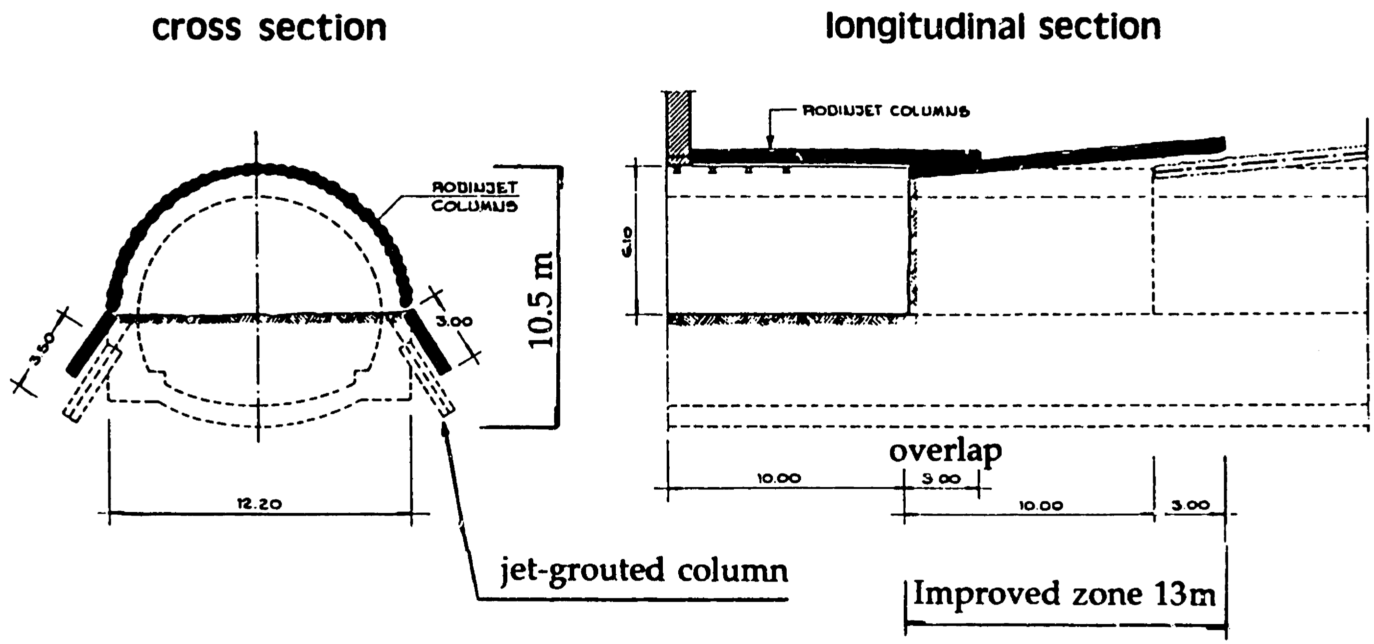


Fig. 4-9-1 Cross and longitudinal sections: Campiolo Tunnel (Mongilardi et al., 1986)

Case 10: Lonato Tunnel (Verona, Italy)

Environment of the Lonato Tunnel

The Lonato Tunnel, which is located near Verona, Italy, is a road tunnel with the excavated area ranging from $100m^2$ to $120m^2$. The excavated width and height are 12.2m - 13.6m and 9.3m - 10.0m respectively. The depth of overburden above the tunnel crown ranges from 2m to 70m.

Geological and Hydrological Conditions

The geology of the tunnel consists of chaotically mixed morainic and detrital deposits.

The morainic mixture is composed of saturated limes with sands, pebbles, and large boulders. The detrital deposits consist of large rock boulders with very high permeability.

Problem in Tunnel Excavation

Instability of both the tunnel and the face was expected.

Supplementary Support Method of the Tunnel

The sub-horizontal jet-grouting method was employed with a row of steel pipes installed beneath the jet-grouted columns. Hence, this umbrella method can be called "the reinforced sub-horizontal jet-grouting method."

No information on whether or not alternative methods were evaluated was available from the references.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-10-1.

The jet-grouted columns are 12m long and 70cm in diameter. 25 columns per cross section were placed every 60cm along the crown perimeter.

Fore-poles used are 12-m-long steel pipes 114.3mm in diameter and 7mm wall thickness. There are 40 steel pipes per cross section.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and double steel arch supports installed at 0.95m intervals (dimensions are unknown). The thickness of secondary lining ranges from 60cm to 130cm.

The footings of the double steel arch support were stabilized with micropiles (jet-grouted columns) to ensure safe bench excavation.

Construction Procedures

9m of the tunnel section was excavated before installing the next umbrella arch to maintain a 3-m overlap for protection of the face.

Construction procedures were as follows:

1) developing the jet-grouted columns and placing the steel pipes
2) reinforcing the face with jet-grouted columns and shotcrete
3) excavating the upper half section (1m steps)
4) putting in place the steel arches and shotcrete
5) supporting the footings of the steel arch support with micropiles (jet-grouted columns)
6) excavating the lower part of the tunnel
7) casting the invert arch and the final lining

Fields Measurements

No information on field measurements was available from the references.

References

Pelizza, S., Peila, D., "Soil and Rock Reinforcements in Tunnelling," Tunnelling and Underground Space Technology, International Tunnelling Association (ITA), Vol. 8, No. 3, 1993.

Mahtab, M.A., Grasso, P., Geomechanics Principles in the Design of Tunnels and Caverns in Rocks, Elsevier, Amsterdam, 1992.

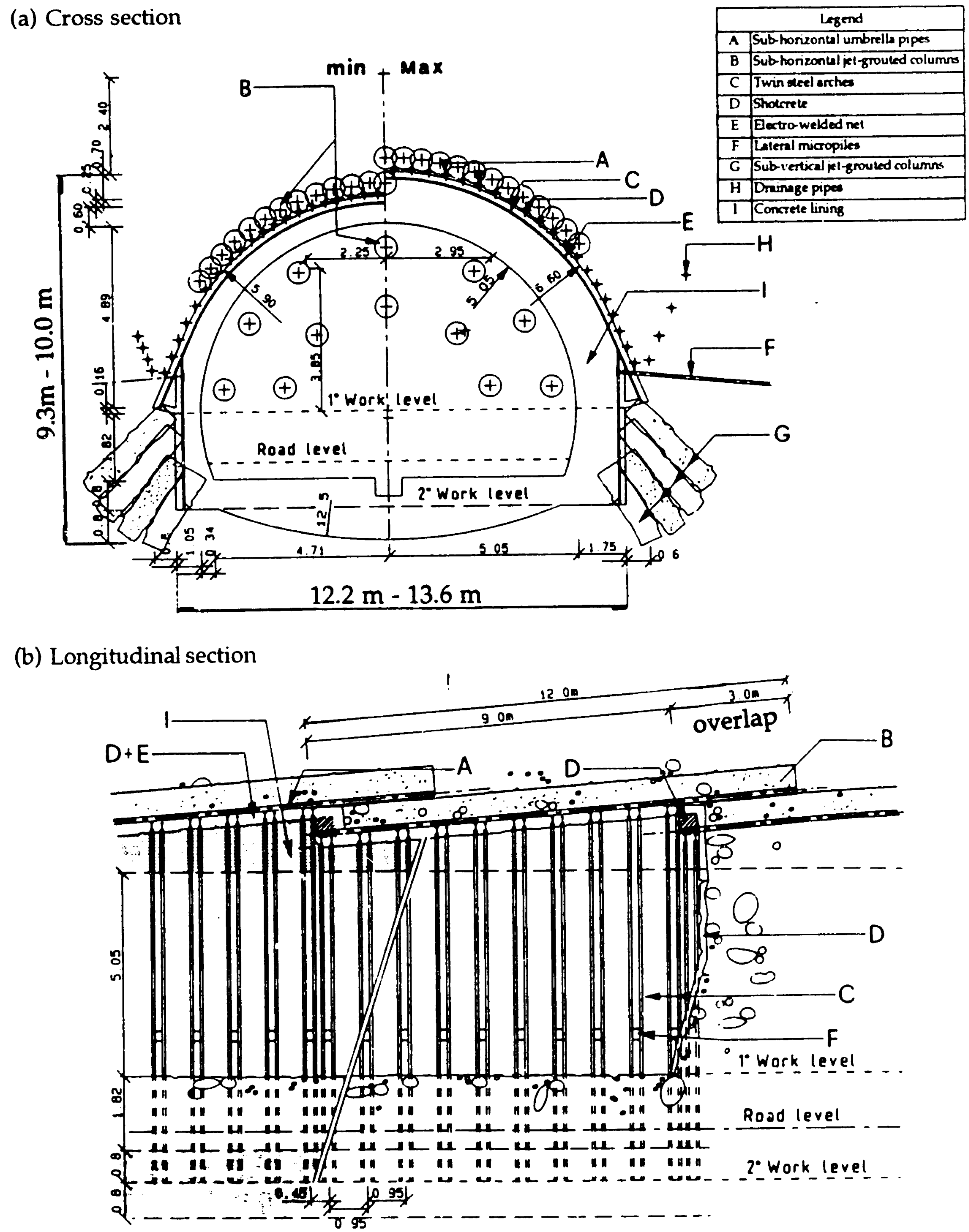


Fig. 4-10-1 Cross and longitudinal sections: Lonato Tunnel (Mahtab et al., 1992)

Case 11: Kaziwara No. 1 Tunnel (Ibaragi, Japan)

Environment of Kaziwara No. 1 Tunnel

The 703-m-long Kaziwara No. 1 Tunnel, which is located between the Ibaragi interchange and the Kyoto-South interchange on the Nagoya-Kobe Expressway, is a motorway tunnel. The tunnel was constructed near an existing tunnel. The excavated width and height are approximately 14m and 10m respectively, and the excavated area is about 110m^2. The depth of overburden in the zone where the umbrella method was employed ranges from 0m to 12m.

Geological and Hydrological Conditions

Close to the surface are talus deposits with SPT-N values of 20 - 30, and overlaying layers of sandstone and slate with an SPT-N value larger than 50.

Problems in Tunnel Construction

Little ground arching could be expected at the tunnel portal because of the shallow overburden.

Supplementary Support Method of the Tunnel

The reinforced sub-horizontal jet-grouting method (the reinforced RJFP method) was employed to prevent slope failures and to improve face stability.

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

The cross section of the tunnel is shown in Fig. 4-11-1.

The jet-grouted columns are 10m long and 60cm in diameter. The number of the fore-poles per cross section is 39.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 45cm thick.

Construction Procedures

The excavation was by heading-and-benching.

After development of a 10-m-long jet-grouted column, a jetting rod with about 90mm in diameter was re-installed in order to reinforce the columns.

The jet-grouting parameters are summarized in Table 4-11-1.

Table 4-11-1 Jet-grouting parameters: Kaziwara No. 1 Tunnel

Nozzle size		2 nozzles with 2.2 mm in diameter
Grouting pressure (MPa)		40
Grout volume (liter/m)		400
Grout (kgf/m^3)	cement	760
	additive	12
	water	750

Source: Ae et al. (1994)

Field Measurements

Final settlement of the ground surface was 40mm, which was equal to that of the tunnel crown. Pre-excavation settlement of the ground surface was 15mm.

Reference

Ae, S., Ito, K., "Construction Report on the Kaziwara No. 1 Tunnel," Technical Report on the Rodin Jet Fore-Poling Method, Geo-Fronte Research Association, January, 1994.

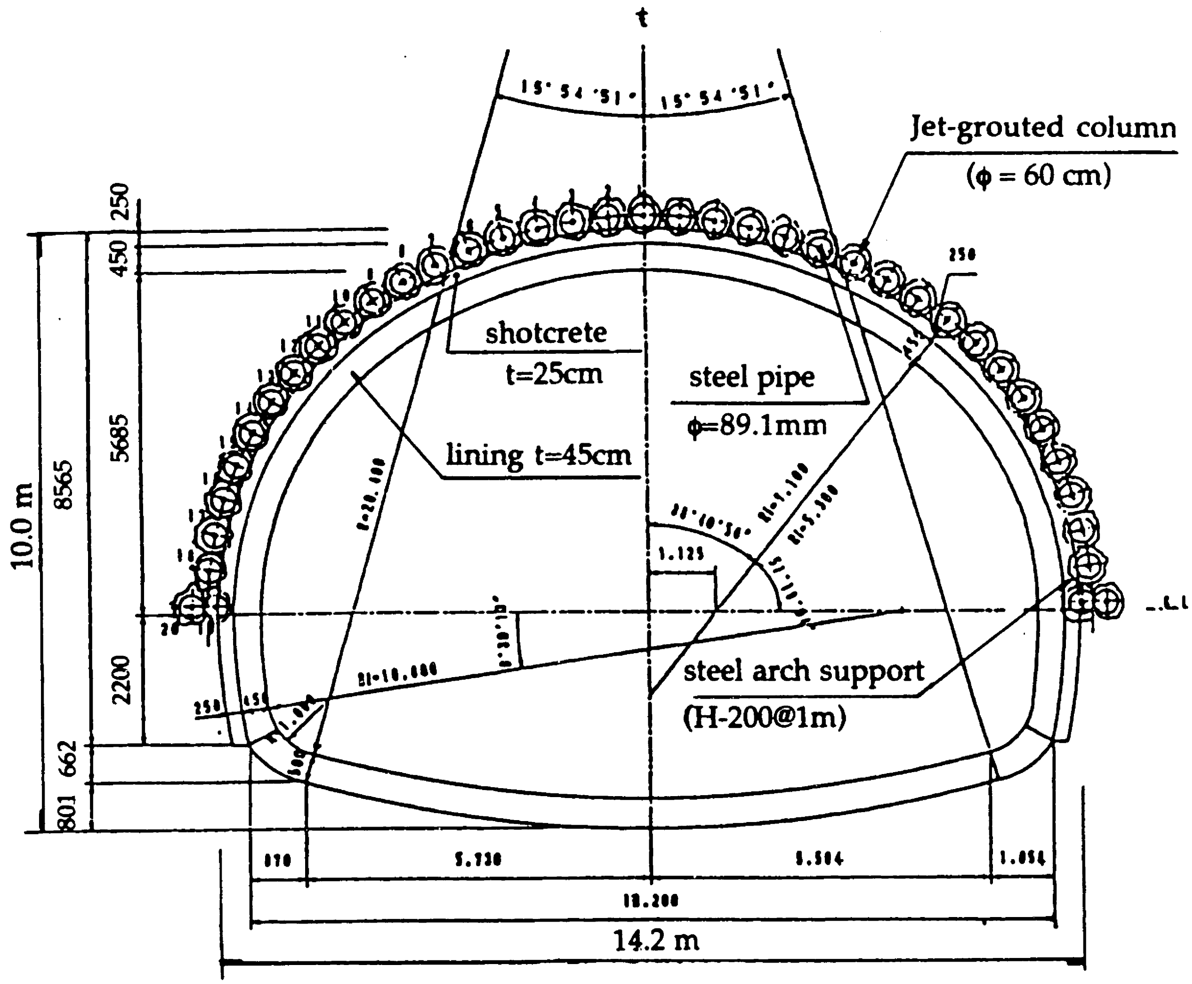


Fig. 4-11-1 Cross section: Kaziwara No. 1 Tunnel (Ae et al., 1992)

Case 12: Les Cretes Tunnel (Aosta, Italy)

Environment of Les Cretes Tunnel

The 1536-m-long Les Cretes Tunnel is a twin tunnel with two lanes in each direction. The excavated width and height of the tunnel are approximately 13m and 10m, respectively and the excavated area is about $105m^2$. The tunnel is the last of the eleven tunnels along the Mont Blanc-Aosta motorway (see Fig. 4-12-1).

Geological and Hydrological Conditions

The geological conditions of the Les Cretes Tunnel are shown in Fig. 4-12-2.

The geology of the tunnel is characterized by the presence of a thick layer of river and glacial deposits which are, in turn, overlain and interposed with present and recent alluvium.

The glacial deposits are moderately dense with medium to coarse particle sizes in a sandy-silty matrix. Inside the deposits large boulders (larger than $10m^3$) are sometimes encountered.

The present and recent alluvia are encountered in the areas facing the portals on both the Aosta and Mont Blanc tunnel sides, and it is also highly likely that they are interposed with morainic deposits even in the morphological terrace crossed by the tunnel.

Specifically, the soil can be characterized as follows: a1) gravel and pebbles in a sandy matrix; a2) silt and clayey silt, in places incorporating small boulders; m1) more or less silty sands and gravel in places mixed with boulders; m2) boulders and pebbles in a silty-clayey matrix; matrix "t") large lithoid blocks usually of limestone or schist.

The data from piezometric measurements showed the existence of a groundwater table above the level of the tunnel crown.

The engineeringproperties of these soil types are summarized in Table 4-12-1.

Table 4-12-1 Engineering properties: Les Cretes Tunnel

Soil type	ϕ (°)	C (kPa)	Modulus of deformation (MPa)	Unit weight of soil (kN/m^3)
a1	34 - 38	-	250	19
a2	24- 30	10	100	19
m1	32 - 36	-	250	21
m2	24 - 28	10	150	21
matrix "t"	30 - 36	-	350	21

Source: Pagliacci et al. (1992)

Problems in Tunnel Construction

The tunnel is on the right bank of the River Dora, therefore its entire length extends through formations consisting basically of loose alluvial or morainic deposits. Thus, problems such as instability of the excavation face and ground surface settlement were expected to occur during tunnel excavation.

Supplementary Support Method of the Tunnel

Two types of umbrella methods, i.e., the injected steel pipe umbrella method (the *TREVITUB*) and the reinforced sub-horizontal jet-grouting method (the *TREVIJET T1*) were employed depending on the ground conditions.

According to the reference, the geological conditions in which each method was employed are summarized in Table 4-12-2.

Table 4-12-2 Geological and hydrological conditions in which the umbrella method was employed: Les Cretes Tunnel

Type of umbrella method	Number of fore-poles per cross section	Geological and Hydrological conditions
Injected steel pipe umbrella method	N=35	•morainic soils with large masses •limited water infiltration
	N=51 (mean)	•slightly cohesive alluvial or morainic soils with a small particle size •limited water infiltration
Reinforced sub-horizontal jet-grouting method	N=45	•alluvial soils, mainly loose and cohesionless •water infiltration

Source: Pagliacci et al. (1992)

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

Figure4-12-3 shows the standard stabilization pattern of the injected steel pipe umbrella method.

Fore-poles used are 12-m-long steel pipes 114.3mm in diameter.

The number of the steel pipes per cross section varies from 35 to 60 placed every 40cm along the crown perimeter. It should be pointed out that a smaller number is used in morainic soils with large boulders but limited water infiltration. In slightly cohesive alluvial or morainic soils with a smaller particle size but in the presence of scant water infiltration, the number is increased on average to 51.

Tunnel support consists of a 20-cm-thick primary lining (shotcrete) and H-160 double steel arch supports (at unknown installation intervals). The secondary lining is 50cm thick.

The footings of the steel arch support were stabilized with micropiles (jet-grouted columns) to ensure that the bench excavation could be performed in safety.

Figure 4-12-4 shows the standard stabilization pattern of the sub-horizontal jet-grouting method.

The jet-grouted columns are 12m long and 60cm in diameter. 45 jet-grouted columns per cross section are placed every 45cm along the crown perimeter.

As shown in the figure, a row of steel pipes is installed beneath the jet-grouted columns. It was also reported in the reference that steel pipes were inserted into the jet-grouted columns. For this reason, this umbrella method can be called "the reinforced sub-horizontal jet-grouting method."

Tunnel support consists of a 20-cm-thick primary lining (shotcrete) and H-160 double steel arch supports (at unknown installation intervals). The secondary lining is 50cm thick. Therefore, tunnel supports used in the reinforced sub-horizontal jet-grouting method are the same as those in the injected steel pipe umbrella method.

Similarly, the footings of the steel arch support were stabilized with micropiles (jet-grouted columns) to ensure safe bench excavation.

Construction Procedures

In both the injected steel pipe umbrella and the reinforced sub-horizontal jet-grouting methods, 9m of the tunnel was excavated before installing the next umbrella arch to maintain a 3-m overlap for the protection of the face. No information on excavation method used was available from the reference.

Specifically, two consolidation techniques pertinent to each umbrella method were carried out.

One technique was only used with the injected steel pipe umbrella method.

Where large amounts of water were encountered, waterproofing treatment, as shown in Fig. 4-12-5, was carried out in the crown by inserting above the steel pipes, fiberglass manchette pipes through which the total injection (see Ch. 3.7) was performed. The pipes have an outer diameter of 60mm and an inner diameter of 40mm, and were fitted with valves every 50cm.

This treatment was not conducted with the reinforced sub-horizontal jet-grouting method, since in this method the umbrella arch, in general, acts as 'hydraulic barrier" (Trevi SpA, 1993).

Another technique was used with both the reinforced sub-horizontal jet-grouting method and the injected steel pipe umbrella method. Where the face showed signs of instability, the face itself was consolidated with manchette-type fiberglass pipes which were then injected under pressure.

Field Measurements

No information on field measurements was available from the reference.

Reference

Pagliacci, F., Yamamoto, M., "New Construction Methods for Tunnels in Difficult Soils: Les Cretes Tunnel, In-house Document of Trevi SpA, Cesena, Italy, 1993.

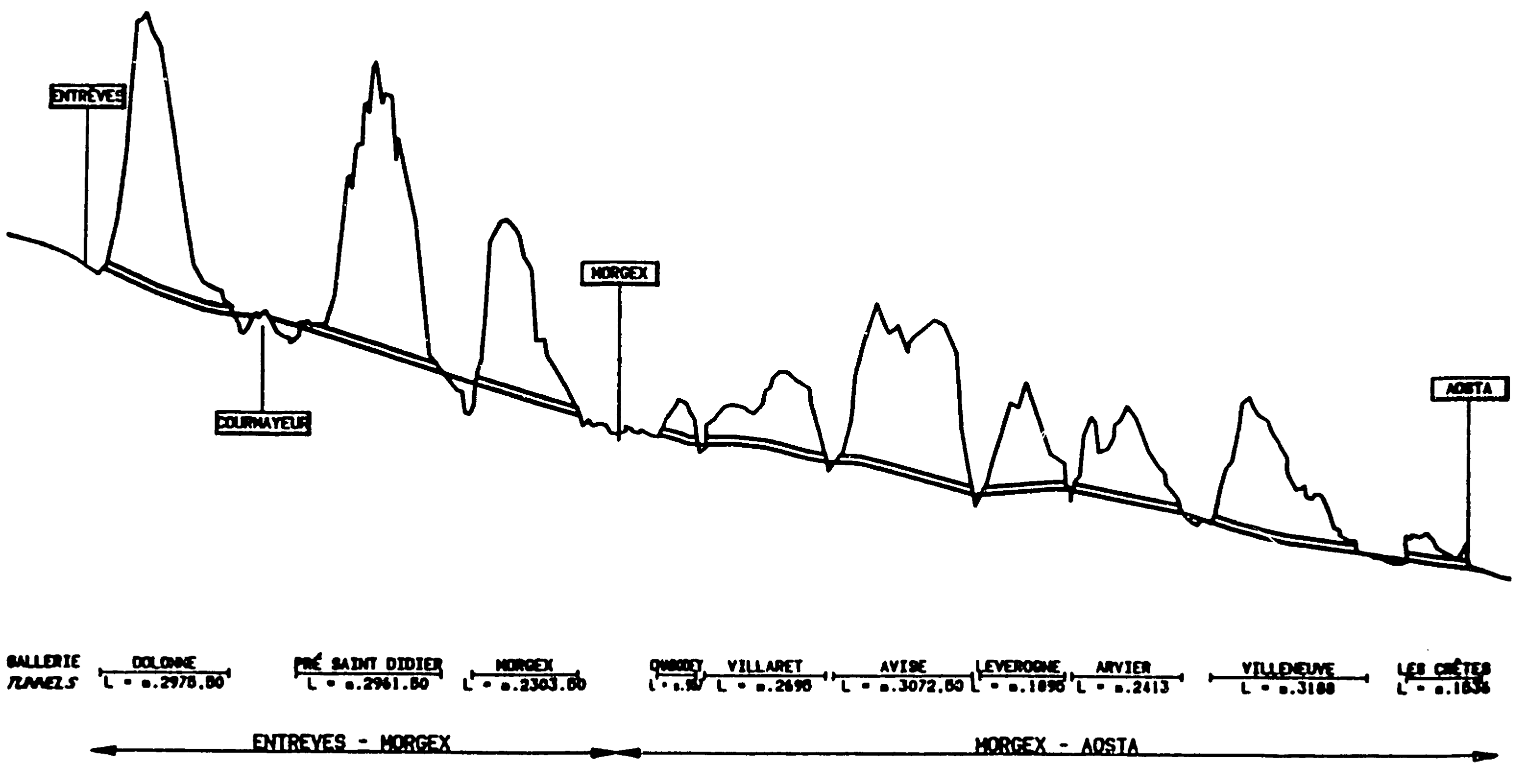


Fig. 4-12-1 Aosta - Mont Blanc tunnel motorway: Les Cretes Tunnel (Pagliacci et al., 1992)

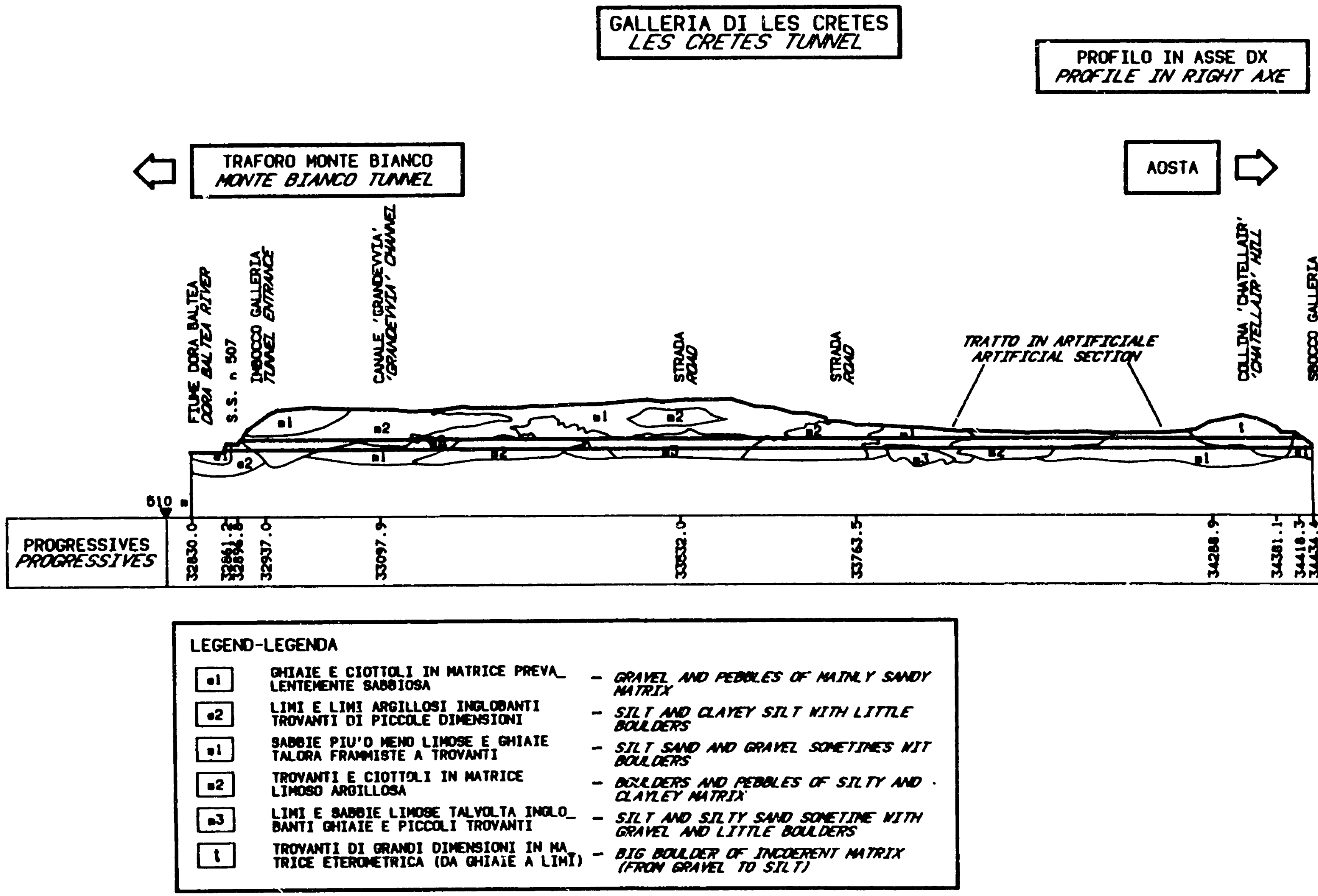


Fig. 4-12-2 Geological conditions: Les Cretes Tunnel (Pagliacci et al., 1992)

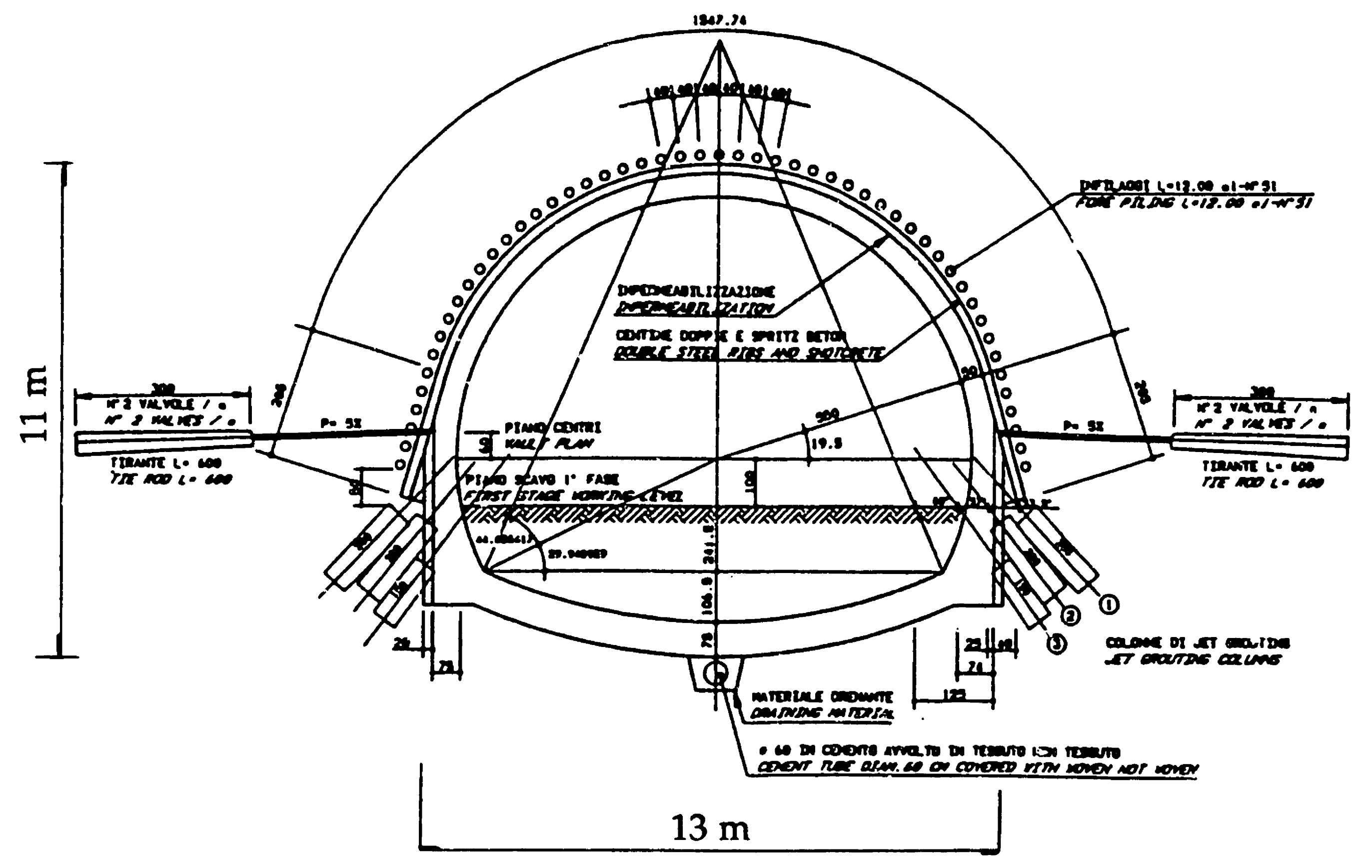


Fig. 4-12-3 Injected steel pipe umbrella method: Les Cretes Tunnel (Pagliacci et al., 1992)

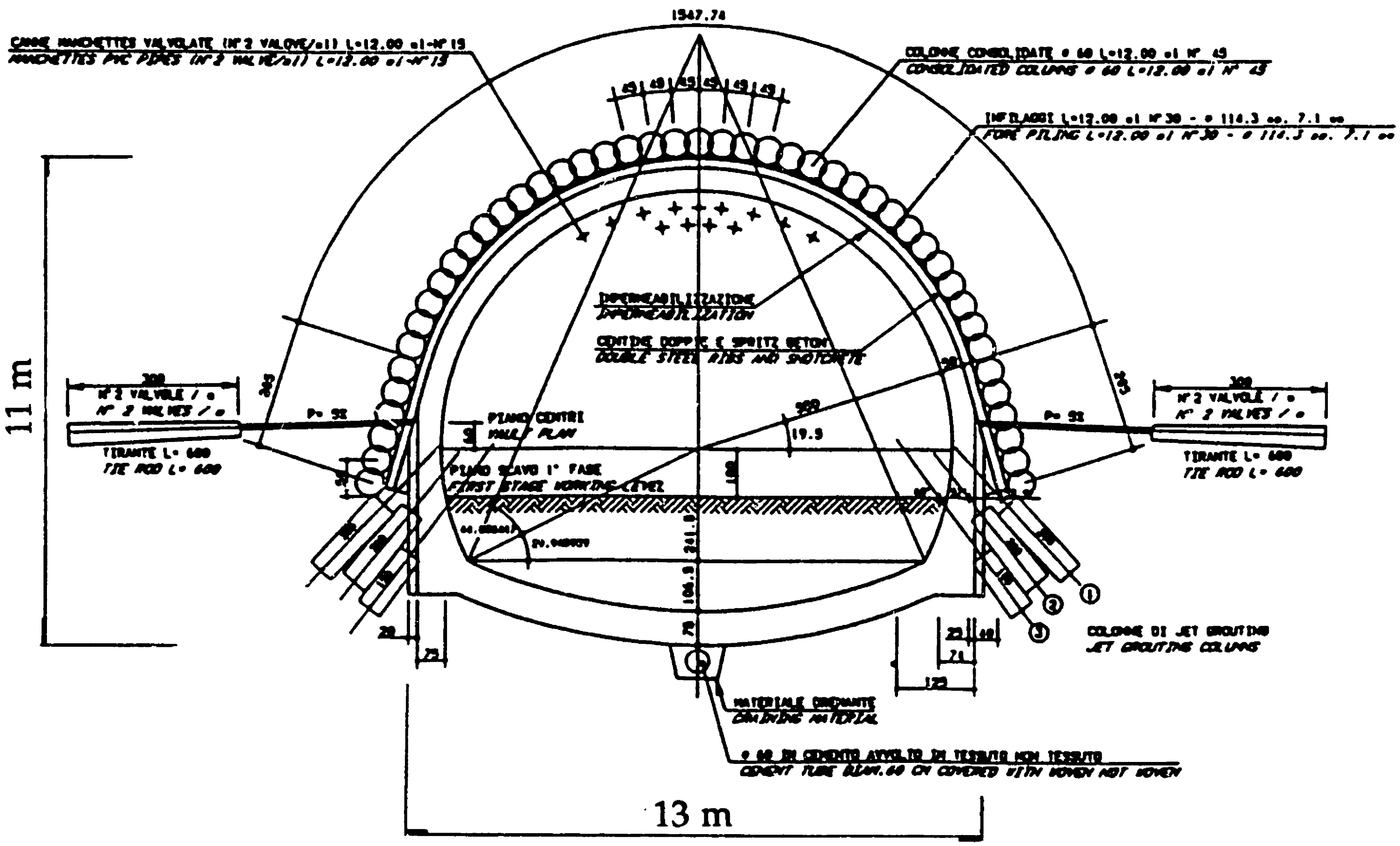


Fig. 4-12-4 Reinforced sub-horizontal jet-grouting method: Les Cretes Tunnel (Pagliacci et al., 1992)

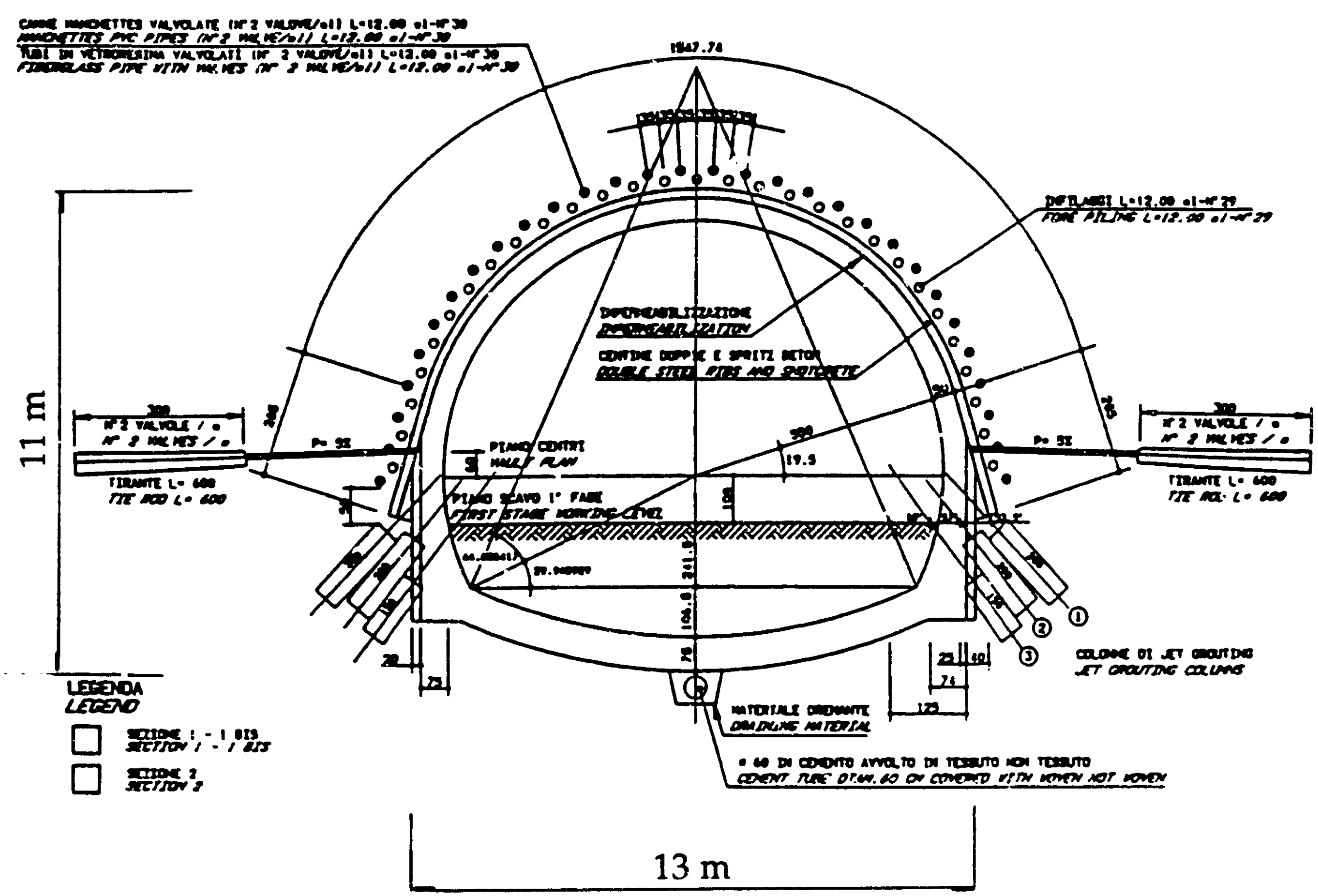


Fig. 4-12-5 Injected steel pipe umbrella method with waterproofing treatment: Les Cretes Tunnel (Pagliacci et al., 1992)

Case 13: Maiko Tunnel (Kobe, Japan)

Environment of the Maiko Tunnel

The 3.3-km-long Maiko Tunnel is a twin highway tunnel with three lanes in each direction. The excavated width and height of the tunnel are about 16m and 11m, respectively and the excavated area is about $150m^2$. The tunnel forms a part of the Kobe approach to the Akashi Bridge (see Fig. 4-13-1). Construction of that bridge, together with construction of the Maiko Tunnel, began in 1988, and is scheduled for completion in 1998.

Geological and Hydrological Conditions

The geology of the Maiko Tunnel can be roughly divided into three sections (see Fig. 4-13-2).

Section 1 :

Section 1, nearest the Akashi Straits, consists of Pleistocene sedimentary deposits that comprise the Osaka Formation. Sand and gravel predominate, and the deposits mostly consist of chert and rhyolite gravel in sizes ranging from 4 to 100mm, with interposed layers of sand and clay. SPT-N values exceed 40, and groundwater is not present. This section extends for approximately 600m.

Section 2 :

This section consists of granite of the later Cretaceous period. Although decompression and discoloration due to weathering action are to be seen at the surface, this Rokko Granite, as a whole, is relatively hard, notwithstanding the presence of jointing. This section extends for approximately 800m.

Section 3 :

This section consists of the same Osaka Formation as is found in section 1 ; however, in this formation, groundwater is present to a level 10m above the tunnel crown. Moreover, alluvial gravel and fill are encountered in some portion of the section. This section extends for approximately 1,800m.

Engineering properties of each layer in section 3 are summarized in Table 4-13-1.

Table 4-13-1 Engineering properties: Maiko Tunnel

Layer	Soil type	SPT-N	Modulus of deformation (MPa)
fill	gravel with clay	5 - 49	0.5 - 4
alluvial layer	gravel	3 - 7	0.5 - 30
terrace layer	volcanic gravel	-	-
Osaka formation	gravel	20 - 70	38 - 230
	gravel	30 - 60	20 - 50
	clay	28 - 60	10 - 60
	gravel with clay	20 - 60	30 - 50
Kobe formation	shale, sandstone	>70	410 - 640

Source: Taisei Co. (1996)

Problems in Tunnel Construction

The adverse conditions for excavation of the Maiko Tunnel were:

- a large cross-section
- uncemented ground
- excavation below groundwater level
- an urban site
- a shallow depth of overburden.

In Japan, the first application of the TREVITUB method, one of the injected steel pipe umbrella methods, was carried out for the Maiko Tunnel. After this was decided and the reduction of ground surface settlement in the Maikodai section was confirmed, the method was employed at the Fukuda junior high school section as well (see Fig. 4-13-2). The geology of each section corresponds to the aforementioned sections 1 and 3, respectively.

Detailed descriptions of each section follow.

Maikodai section:

The Maikodai district, near the south portal of the Maiko Tunnel, is a residential area. In order to preserve this residential environment, the highway was planned to be inside a tunnel to the greatest extent possible. As a result, the depth of overburden above the tunnel crown is shallow (only 55m at the maximum). In the built-up section of the city, near the Akashi Straits, the depth of the overburden often decreases to 10 to 20m. The ground surface above the tunnel is occupied by multi-unit housings, stores and other buildings, and streets with buried utility pipes for gas and water. Moreover, the twin tunnels, which are separated by 54m center-to-center in the standard section, gradually draw together in this vicinity and become binocular at the portal. The distance of the two tunnels ranges from 1.1 to 13.4m in this section. This Maikodai section extends for approximately 370m.

As shown in Fig. 4-13-2, the geology of the Maikodai section mostly consists of gravel with SPT-N values of 40 to 50.

A construction method had to be selected that would ensure face stability and prevent collapse of the tunnel crown and minimize the effect on surface structures. Consequently, the injected steel pipe umbrella method was employed.

Fukuda junior high school section:

Near the north portal of the Maiko Tunnel, the tunnel passes 4 to 7m beneath the athletic field of the Fukuda junior high school and, very close to the school buildings (10m at the minimum). This section extends for approximately 200m.

As mentioned previously, the ground in this section consists of the Osaka Formation, which is mostly gravel with silt and clay, however, alluvial gravel and fill are encountered in some portion of the section.

To minimize ground surface settlement and the effects on the nearby school buildings, the injected steel pipe umbrella method was employed here, also.

Supplementary Support Method of the Tunnel

As mentioned previously, this was the first application of the TREVITUB method in Japan as a supplementary support method for tunnel construction.

The sub-horizontal jet-grouting method was rejected because there was a risk that high pressures might have a detrimental effect upon the surface structures. The pipe roof method was rejected because there was a problem of directional accuracy in pipe placement due to long distance of jacking.

Structural Details

(1) Maikodai section

The cross and longitudinal sections are shown in Fig. 4-13-3.

Fore-poles are 12-m-long steel pipes 114mm in diameter and 6mm wall thickness. The number of the fore-poles per cross section is 37.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-250 section steel arch supports with wing ribs installed at 1m intervals. The secondary lining is 70cm thick. Side piles made of the same 114-mm-diameter steel pipe were installed.

(2) Fukuda junior high school section

The cross and longitudinal sections are shown in Fig. 4-13-4.

As in the Maikodai section, fore-poles are 12-m-long steel pipes 114mm in diameter and 6mm wall thickness. The number of the fore-poles per cross section is 42.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports with wing ribs installed at 1m intervals. The secondary lining is 45cm thick. Footing piles are jet-grouted columns; however no side piles were installed.

Construction Procedures

(1) Maikodai section

The excavation method was by heading-and-benching.

8m of the tunnel section was excavated before installing the next umbrella arch to maintain a 4-m overlap for the protection of the face.

The grout used was a binary high early strength cement-water suspension. Drilling and grouting were carried out every two pipes to prevent the collapse of drilled hole and leakage of grout.

At the beginning of the experimental construction of the Maiko Tunnel, footing piles made of the same 114-mm-diameter steel pipes were used to support the steel arch supports and to prevent the collapse of the side wall during bench excavation. However, settlement of the tunnel crown reached an excess of 30mm. It was thought that drilling for the footing piles disturbed the ground beneath the steel arch support. Thus, the method was changed and the footings of the steel arch support were stabilized by urethane injection before excavation of the top heading (see Fig. 4-13-5). Unconfined compressive strength and modulus of deformation of this reinforced ground improved up to 5 - 7MPa and 1200 - 1600MPa, respectively. As a consequence, the tunnel crown settlement decreased to less than 20mm.

Pre-loading was induced to both the ground above the tunnel crown and the footings of the steel arch support (see Fig. 4-13-6), which was done by injecting non-shrinkage mortar into sacks which were placed: 1)between the steel arch support and the umbrella arch; 2) between the umbrella arch and the ground; and 3) beneath the footings of the steel arch support. This was done to make the steel arch act as a fulcrum for the driven steel pipes as soon as possible after installation, which led to a reduction of ground surface settlement.

(2) Fukuda junior-high school section

The excavation was by center drift method (CD method).

8m of the tunnel section was excavated before installing the next umbrella arch to maintain a 4-m overlap for the protection of the face.

In the Maikodai section, the footings of the steel arch support were stabilized by urethane injection. This procedure produced a relatively small reinforced zone, so that the injection had to be implemented at each steel arch support (at 1m intervals). As a consequence, this operation affected the progress of the work. This experience led to a change in the procedure in the Fukuda junior high school section. Here, the footings of the steel arch support were stabilized by jet-grouted columns. These jet-grouted columns had a diameter of 600mm.

At the beginning of the tunnel excavation, the grout used was a binary high early strength cement–water suspension. However, when soft alluvial ground was encountered, large ground surface settlement occurred. Therefore, ultrafine cement was employed instead of the high early strength cement. This cement is, in general, used in grouting fine-grained soils (Henn, 1996). Also, horizontal jet-grouted columns were developed to reinforce the face. The athletic field of the school was also reinforced by casting a concrete slab (20cm thick, wire mesh reinforced). The above measures are shown in Fig. 4-13-7.

Field Measurements

(1) Maikodai section

Figure 4-13-8 shows the relationship between the settlement ratio defined as U / U_{max} and the distance from the face, where U_{max} is the amount of the ground surface settlement when the face passed a distance of 3D (D: tunnel width) beyond the measuring point.

As the figure shows, there is close agreement between the measured data and the results obtained from a 3-D FE method elastic analysis. Hence, one may conclude that the ground was excavated in an almost elastic state.

(2) Fukuda junior high school section

This section consists of two geologically different zones. The A-zone is formed by soft ground consisting of fill with clay and an alluvial deposit and the other (B-zone) consists of relatively stable ground with gravel and clay. In both zones the depth of overburden is almost the same ($H \approx 6m$).

The relationship between the ground surface settlement and the distance from the face is shown in Fig. 4-13-9. Note that the settlement shown in the figure is settlement due to excavation of the top heading.

From the figure, the following can be observed.

- In the A-zone, settlement after excavation of the top heading was 122mm. Pre-excavation settlement was 90mm. Thus, about 70% of the settlement after excavation of the top heading occurred before the face reached the measuring point.
- In the B-zone, settlement after excavation of the top heading was 22mm. Pre-excavation settlement was about 6mm. Thus, about 30% of the settlement after excavation of the top heading occurred before the face reached the measuring point, which was very small

compared to that of the A-zone (70%). Since it was reported that characteristics of the measured settlements in the B-zone were similar to those obtained from a 3-D FE method elastic analysis (Okazawa et al., 1995), one may conclude that the ground was excavated in an elastic state.

- In the A-zone, settlement gradually increased after the face advanced from a point about 2D (D: tunnel width) ahead of the measuring point and subsided about 1.5D beyond the measuring point. In the B-zone, on the other hand, settlement started after the face advanced from a point about 1D (D: tunnel width) ahead of the face and subsided about 1.5D beyond the measuring point. Therefore, it can be seen from these results that the settlement characteristics depend on the ground conditions.

Let us consider the settlement in the A-zone in detail.

Figure 4-13-10 shows the relationship between the ground surface settlement and the distance from the face. The following should be noted:

- Two curves were drawn in the figure, one of which was obtained by considering the settlement during drilling and installation of the fore-poles and the other of which was obtained by ignoring the settlement during drilling and installation of the fore-poles;
- The settlement shown in the figure is settlement due to excavation of the top heading.

The following observations can be made from Fig. 4-13-10:

- Large settlements occurred during drilling and installation of the fore-poles. It seems that such large settlements were induced by ground disturbance or loosening by drilling and/or injecting.
- When the settlement during drilling and installation of the fore-poles was ignored, settlement after excavation of the top heading was 70mm. On the other hand, in the case in which the settlement during drilling and installation of the fore-poles was considered, settlement after excavation of the top heading was 122mm. Therefore, the total amount of the settlement which occurred during drilling and installation of the fore-poles was 52mm.
- As stated previously, pre-excavation settlement of 90mm was observed in the case in which the settlement during drilling and installation of the fore-poles was considered. On the other hand, in the case in which the settlement during drilling and installation of the fore-poles was ignored, pre-excavation settlement was 44mm.

Figure 4-13-11 shows the relationship between the tunnel crown settlement and the tunnel base settlement (Taisei Co., 1996). Note that the settlement was measured in the B-zone during excavation of the top heading.

From the figure, it can be seen that the tunnel base settlement was approximately 80% of the magnitude of the tunnel crown settlement. Therefore, one can conclude that the whole tunnel structure sank as a rigid structure.

References

Henn, R.W., Practical Guide to Grouting of Underground Structures, American Society of Civil Engineers (ASCE), 1996.

Japan Tunnelling Association, "Challenges and Changes: Tunnelling Activities in Japan 1994," Tunnelling and Underground Space Technology, International Tunnelling Association (ITA), Vol. 10, No. 2, 1995.

Okazawa, T., Hamamura, Y., Fujii, T., "Construction of a Large Cross-section Tunnel with the Umbrella Method under Residential Areas: Maiko Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), February, 1996.

Taisei Corporation, "Development of Design Method for a Supplementary Method in a Large Cross-section Tunnel," In-house Document of Taisei Corporation, Tokyo, Japan, 1996.

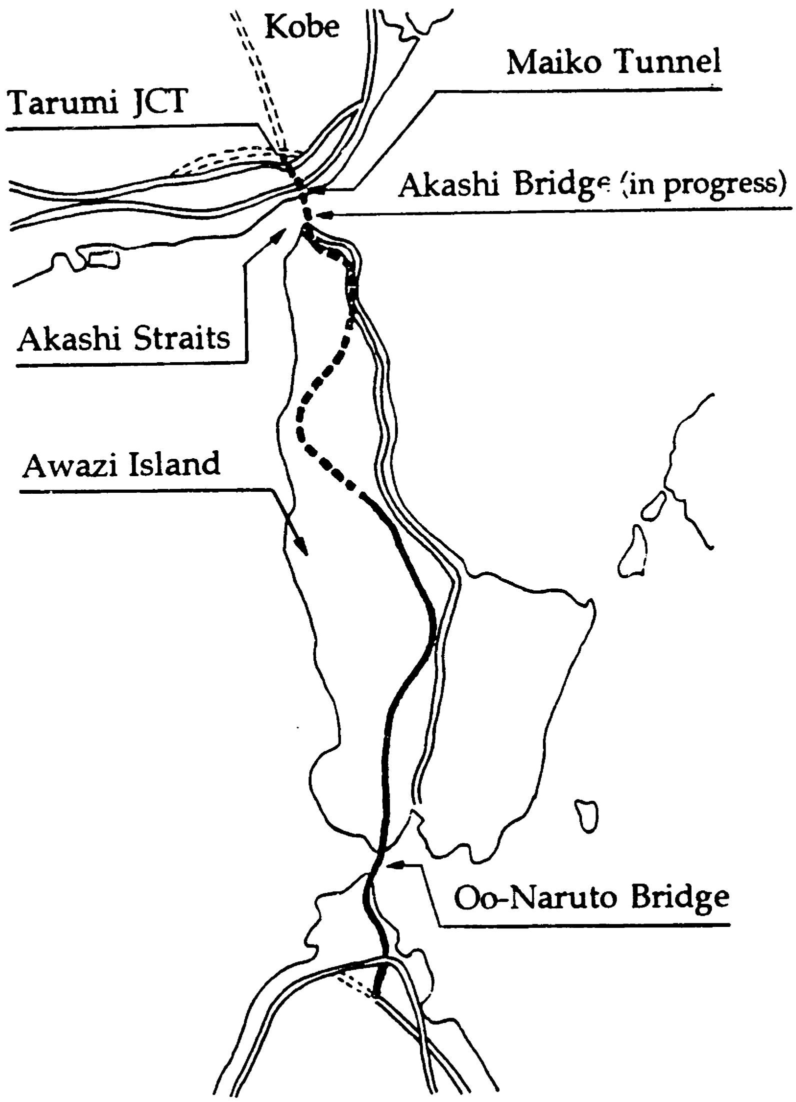


Fig. 4-13-1 Location of the Maiko Tunnel

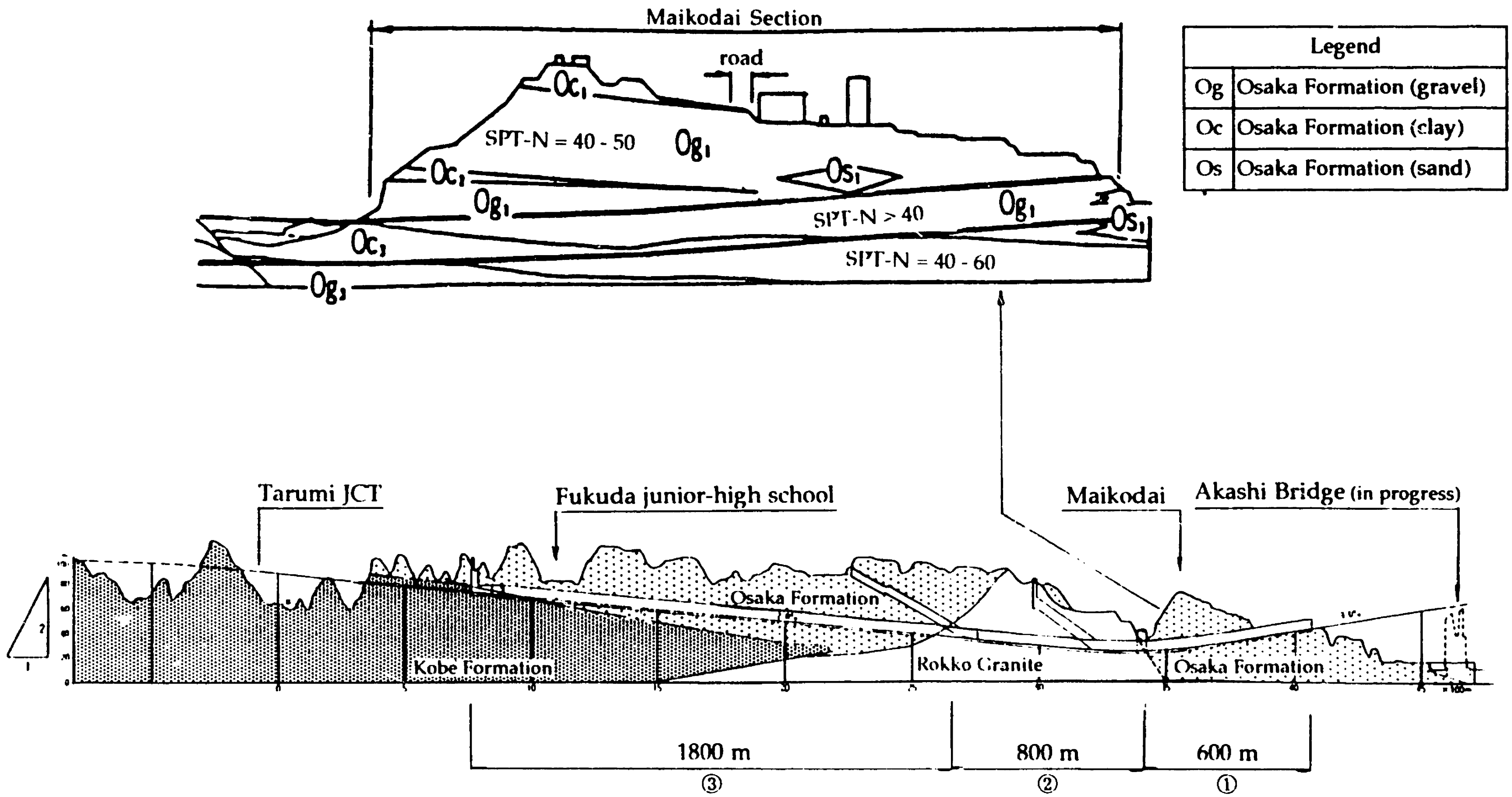
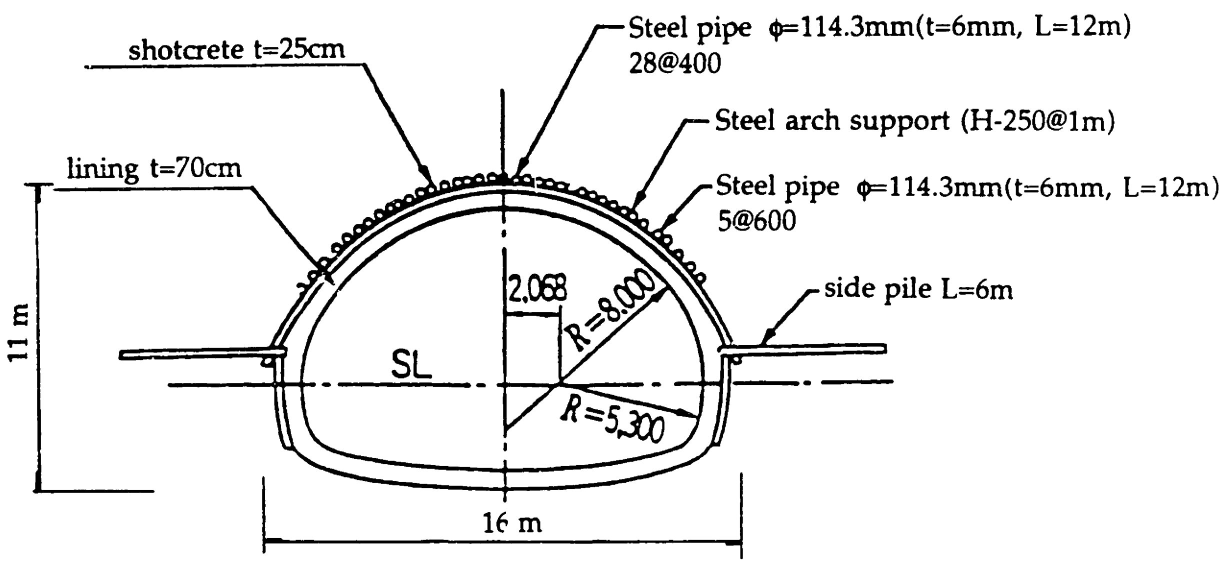


Fig. 4-13-2 Geological conditions: Maiko Tunnel (Okazawa et al., 1996)

(a) Cross section



(b) Longitudinal section

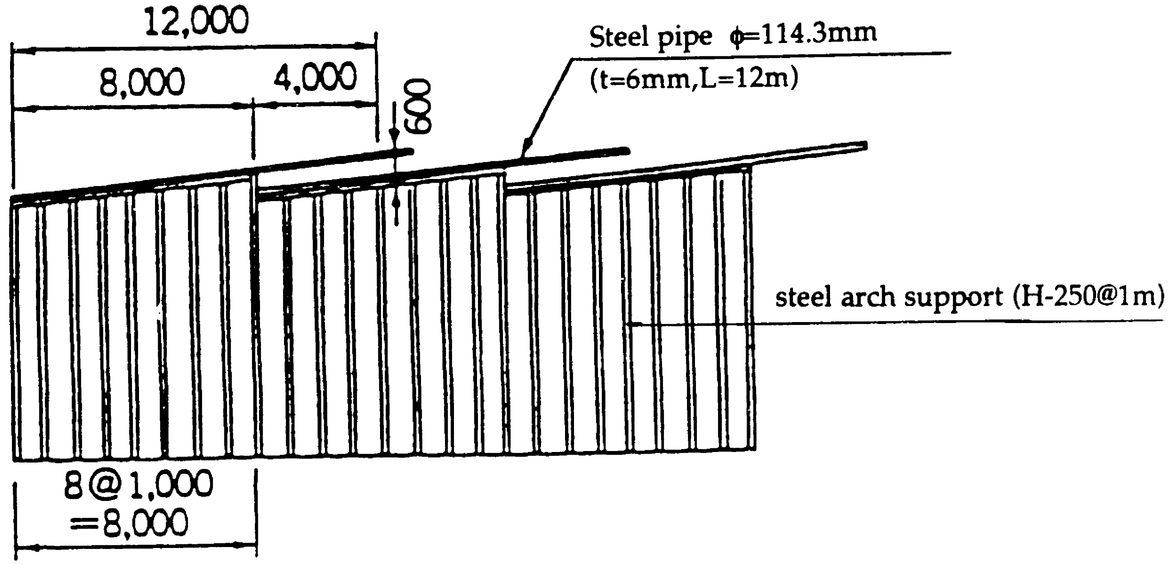
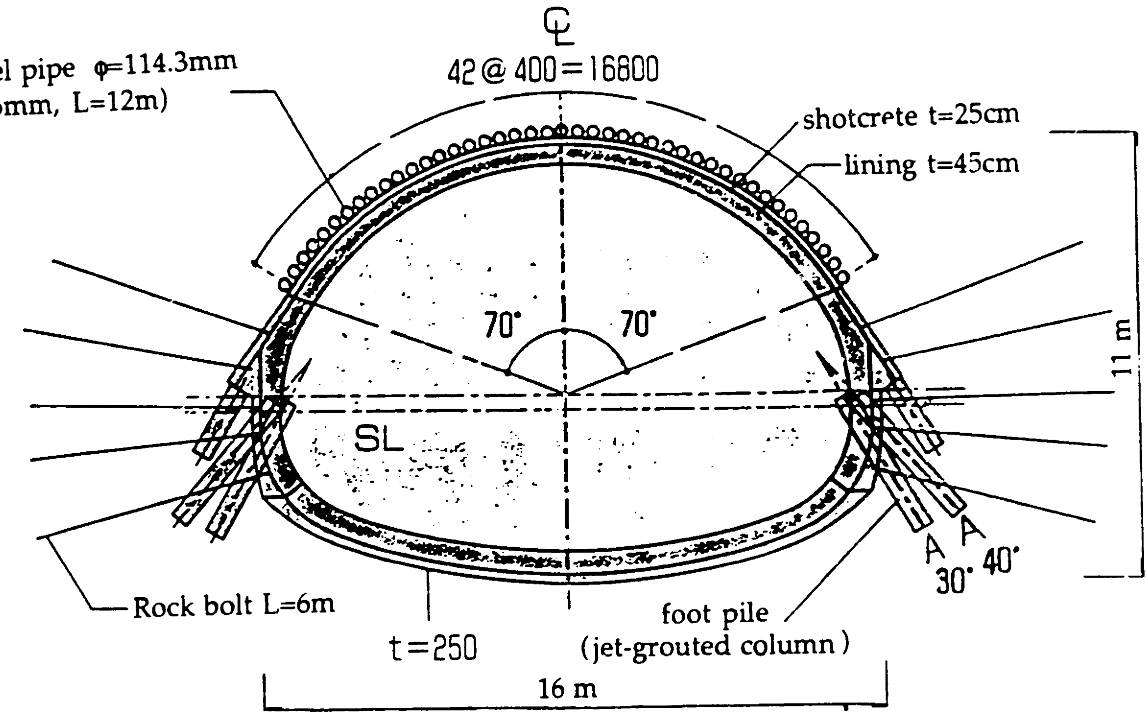


Fig. 4-13-3 Cross and longitudinal sections: Maiko Tunnel [Maikodai section] (Okazawa et al., 1996)

(a) Cross section



(b) Longitudinal section

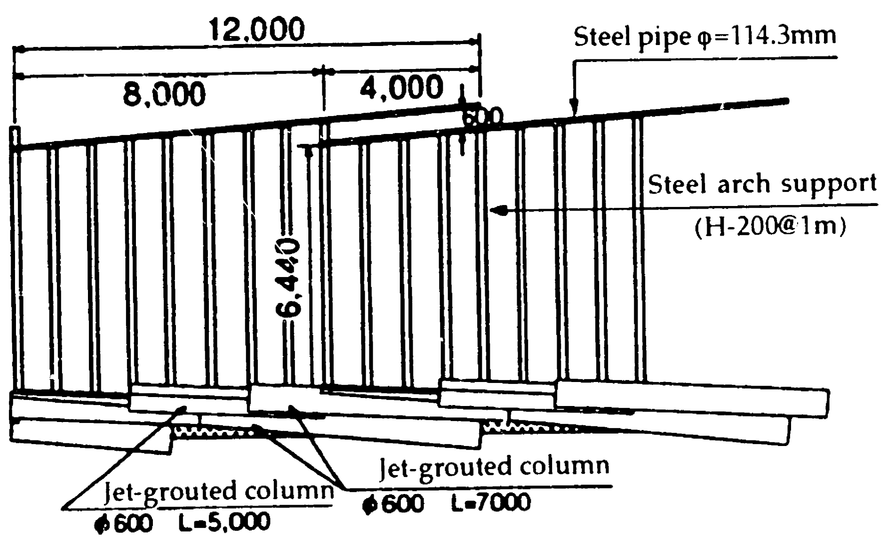


Fig. 4-13-4 Cross and longitudinal sections: Maiko Tunnel [Fukuda junior high school section] (Taisei Co., 1996)

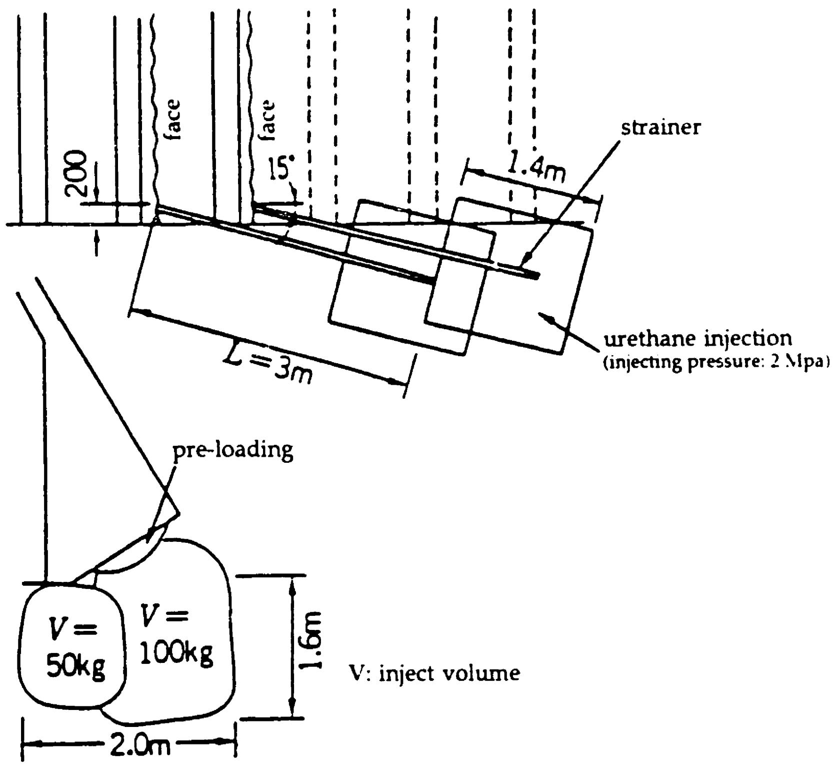


Fig. 4-13-5 Urethane injection beneath steel arch supports: Maiko Tunnel (Okazawa et al., 1996)

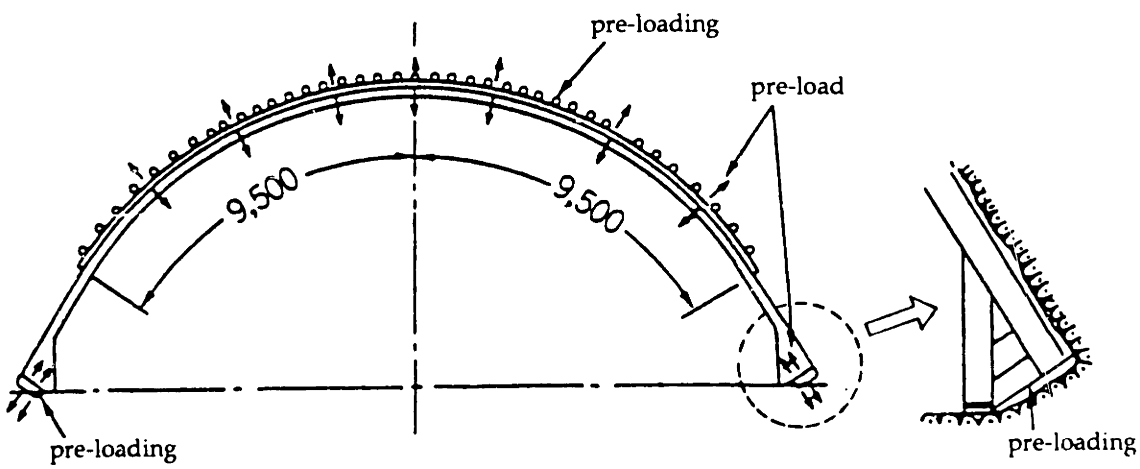


Fig. 4-13-6 Pre-loading: Maiko Tunnel (Okazawa et al., 1996)

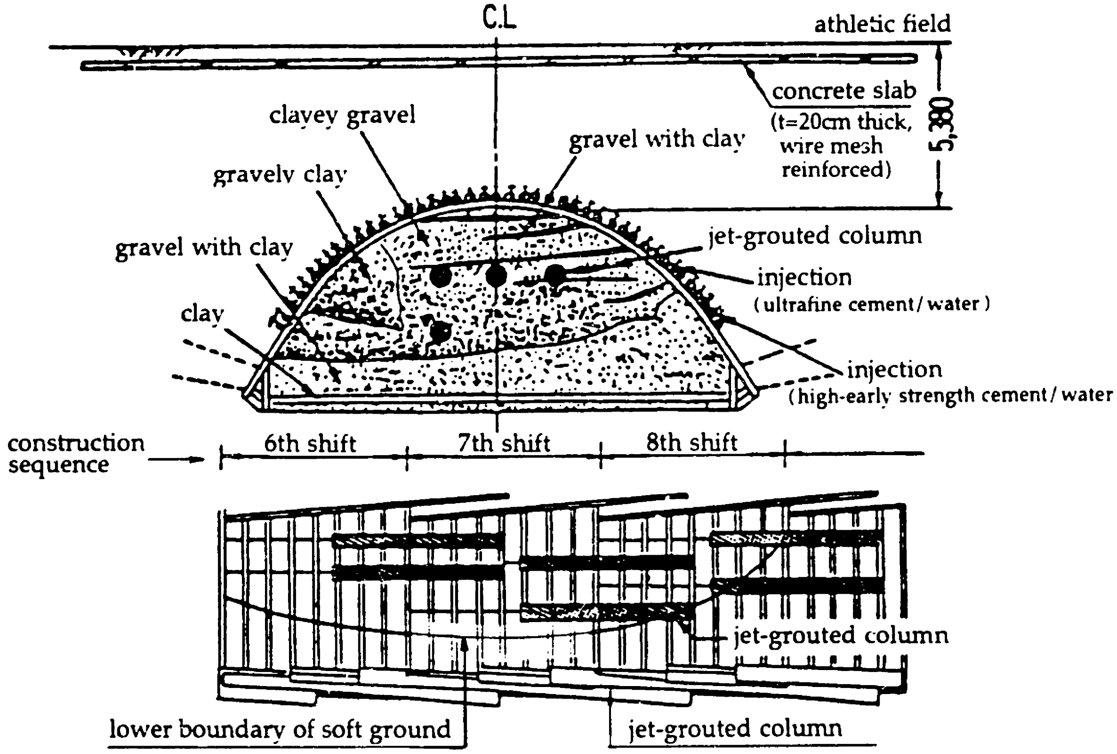


Fig. 4-13-7 Measures in Fukuda junior high school: Maiko Tunnel (Okazawa et al., 1996)

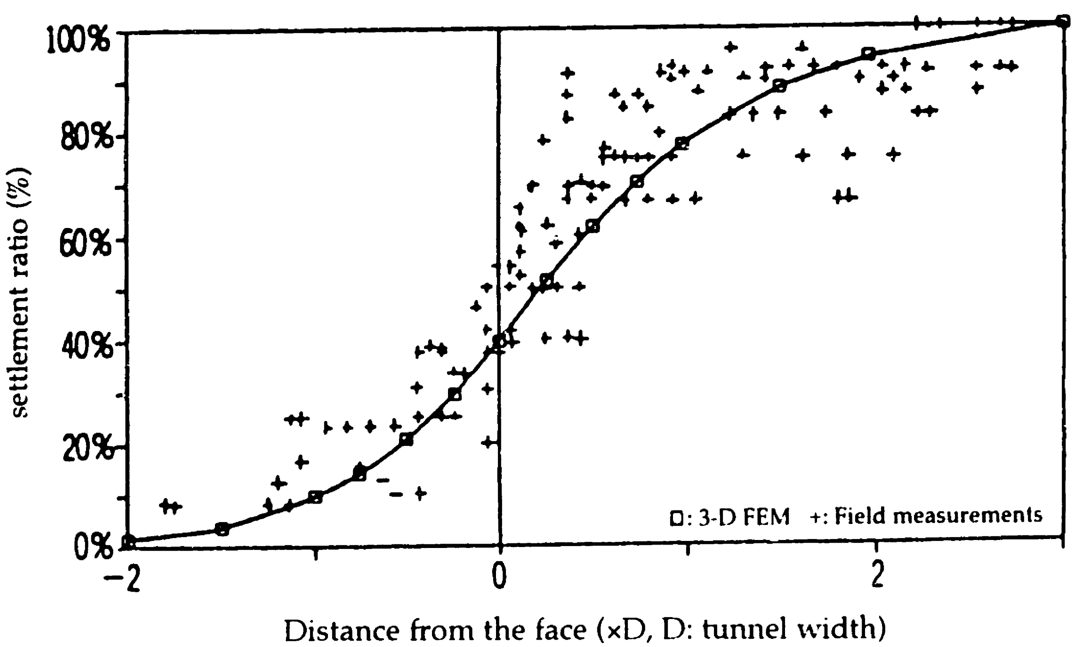


Fig. 4-13-8 Ground surface settlement curve: Maiko Tunnel [Maikodai section] (Okazawa et al., 1996)

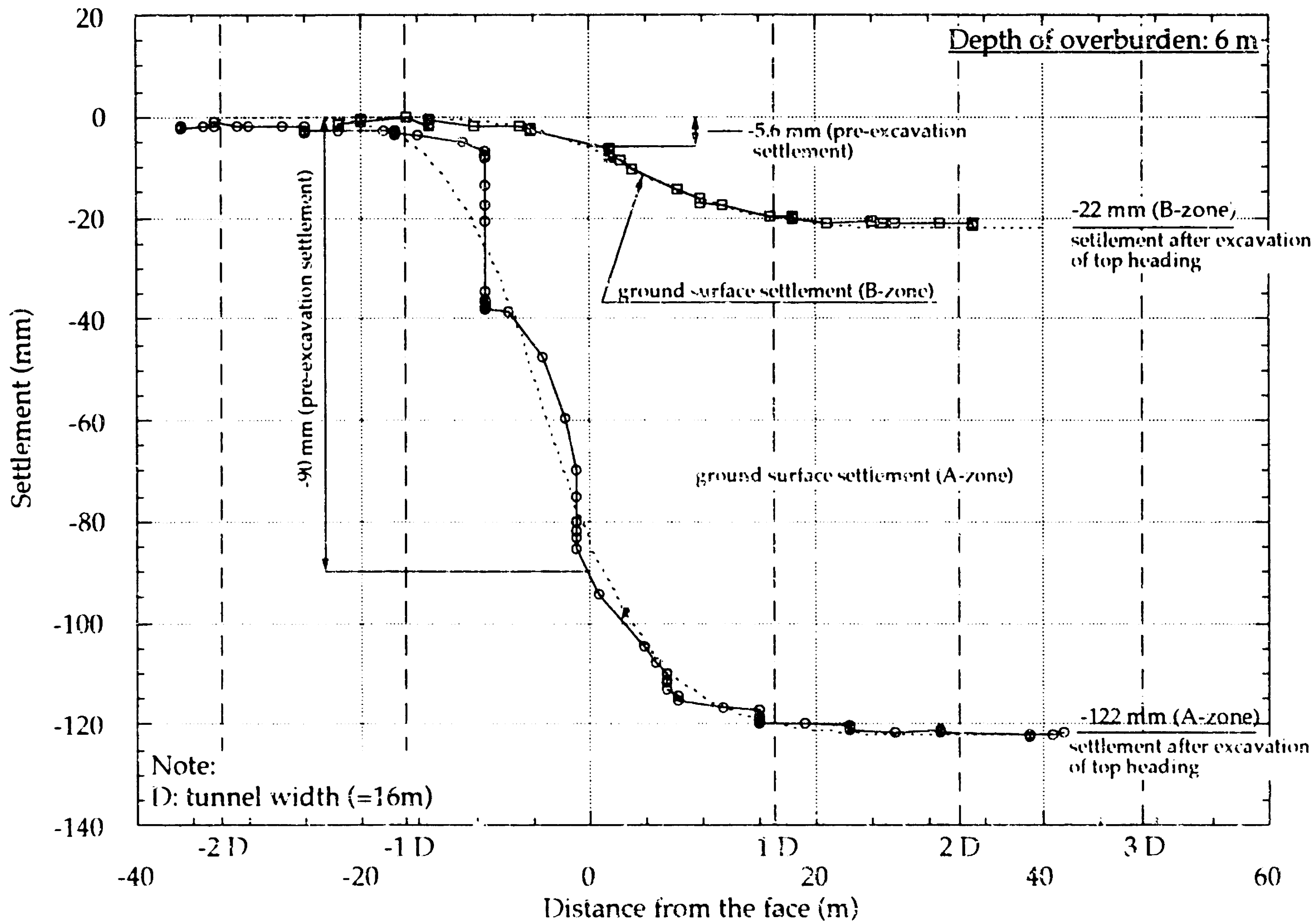


Fig. 4-13-9 Ground surface settlement versus distance from the face: Maiko Tunnel [Fukuda junior high school section] (Taisei Co., 1996)

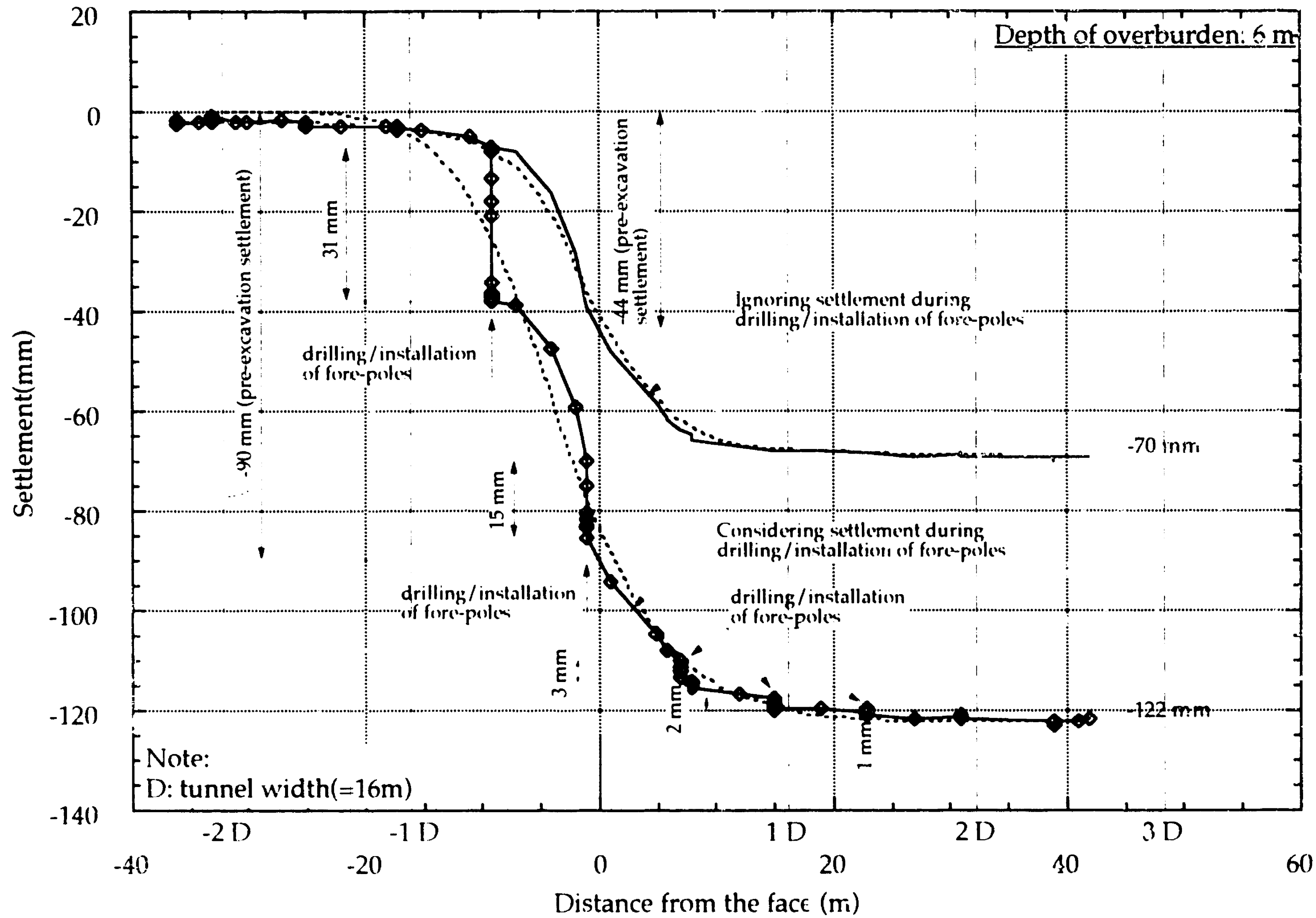


Fig. 4-13-10 Ground surface settlement versus distance from the face: Maiko Tunnel [Fukuda junior high school section, A-zone] (Taisei Co., 1996)

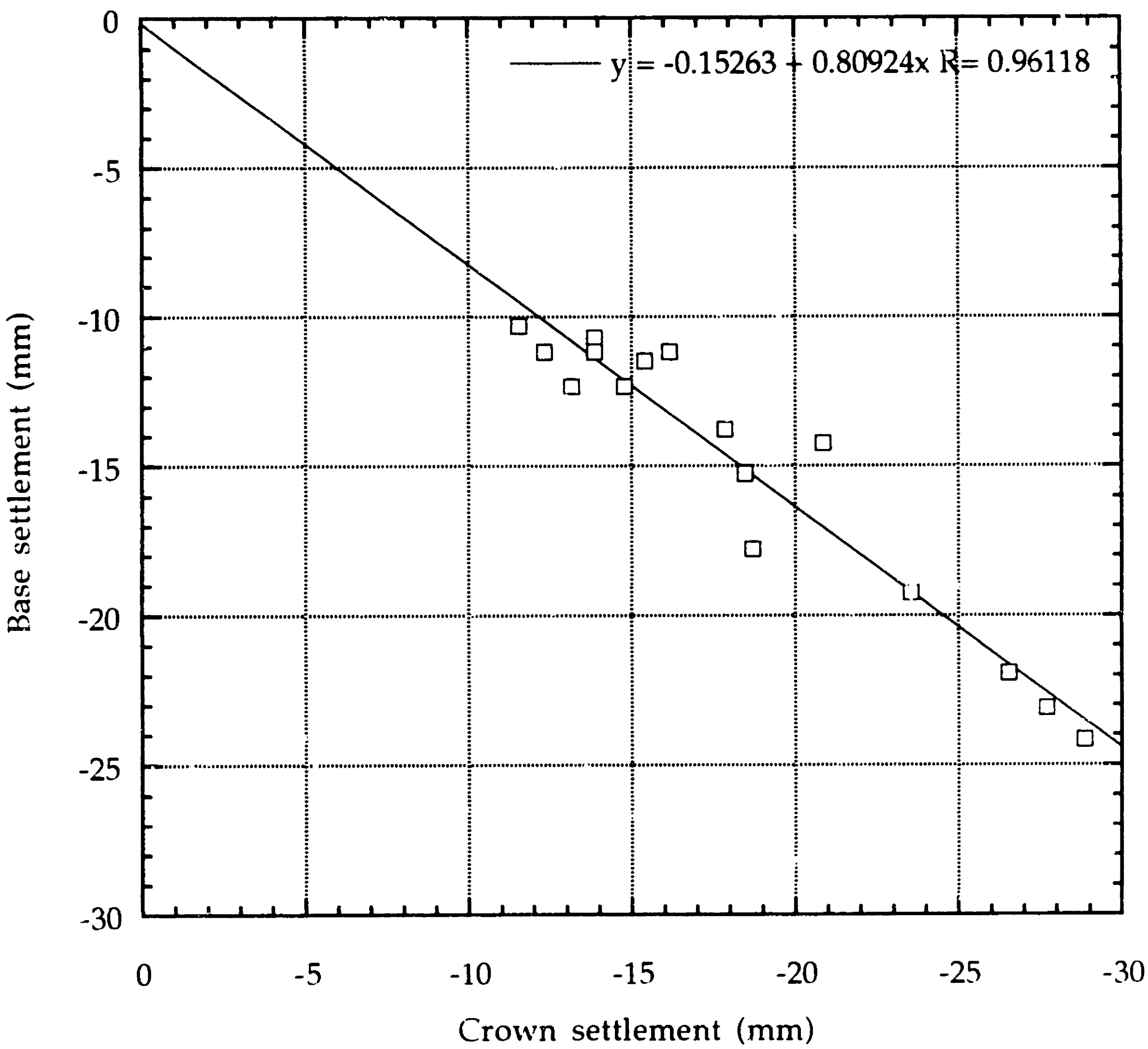


Fig. 4-13-11 Crown settlement versus base settlement: Maiko Tunnel [Fukuda junior high school section, B-zone] (Taisei Co., 1996)

Case 14: Hirai Tunnel (Miki, Japan)

Environment of the Hirai Tunnel

The 800-m-long Hirai Tunnel on the Sanyo Expressway, which starts in Suita and ends in Yamaguchi, is a twin highway tunnel with three lanes in one direction and two lanes in the other direction. The tunnel is located almost in the center of Miki, a suburb of Kobe, in which vineyards and golf courses are close to the tunnel. The excavated width and height of the flat, large cross-sectional 3-lane tunnel are about 15.5m and 10m, respectively and the excavated area is about 130m^2.

Geological and Hydrological Conditions

The geological conditions of the Hirai Tunnel are shown in Fig. 4-14-1.

The ground can be roughly divided into the following three strata:

- talus deposits (D_t) consisting of clayey gravel (not shown in the figure)
- Osaka Formation (O_g), Tertiary / Quaternary deposits, consisting of clayey gravel and gravel with clay
- Kobe Formation, Neogene deposits, consisting of tuff (K_t), sandstone (K_s), conglomerate (K_g) and mudstone (K_m)

The Hirai Tunnel mainly passes through the clayey gravel layer of the Osaka Formation (O_g), having SPT-N values of 30 - 50 and a modulus of deformation of 82 - 270MPa. Gravel mostly consists of chert smaller than 5cm in size.

The groundwater table is located about 1 - 2m above the Kobe Formation. However, no inflow of the groundwater within the tunnel was observed.

Problems in Tunnel Construction

The 3-lane Hirai Tunnel with a flat, large cross section of about 130m^2 passed through the aforementioned clayey gravel layer (O_g). The depth of overburden in the section between the west-side portal and a point of about 270m from the west-side portal ranged from 0 m to 25m, which was very shallow compared to the excavated width of 16m. Therefore, a construction method was needed which could ensure face stability.

Supplementary Support Method of the Tunnel

The All Ground Fasten (AGF) method, one of the injected steel pipe umbrella methods, was adopted to satisfy the above demands.

No information on whether or not alternative methods were considered was available from the reference.

Structural Details

The cross section of the 3-lane Hirai Tunnel is shown in Fig. 4-14-2.

The fore-poles are 3-m-long steel pipes 60.5mm in diameter. The number of the fore-poles per cross section is 53.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 75cm intervals. The secondary lining is 45cm thick.

Construction Procedures

The excavation was by heading-and-benching.

Before excavation of the tunnel, in order to maintain a depth of overburden larger than 5m above the tunnel crown, three valleys under which the tunnel passed (see Fig. 4-14-1) were filled with soil cement with SPT-N values larger than 50 and a modulus of elasticity of 90MPa.

Field Measurements

It should be noted that the following measurements are all related to the 3-lane tunnel.

Figure 4-14-3 shows the ground surface and the tunnel crown settlements in relation to the depth of overburden in the zone in which the umbrella method was adopted.

Note that the settlement in the figure is final settlement.

From the figure, the following can be seen:

- Settlement magnitude depends on the depth of overburden, that is, the shallower the overburden the larger the settlement, and vice versa.
- The magnitude of the ground surface settlement was almost equivalent to that of the tunnel crown settlement.

Figure 4-14-4 shows the tunnel crown settlement versus the time. Note that the settlement was measured after the primary supports (shotcrete and steel arch support) were installed.

Several observations can be made from the figure:

- Settlement subsided when the face proceeded almost 1D (D: tunnel width) beyond the measuring point.
- Settlement of 24mm occurred after excavation of the top heading.
- Final settlement was 35mm.

The differential settlement gauge installed about 1m above the tunnel crown gave the following measurements (not shown as a figure or table):

- Pre-excavation settlement of 18mm occurred.
- When the face proceeded 1D beyond the measuring point, settlement of 43mm was observed, then the settlement reached a final value of 53mm.
- Therefore, settlement which occurred after the face passed the measuring point was 35mm (= 53mm - 18mm). This value corresponded to the final settlement of the tunnel crown.

To assess the effect of the umbrella pipes, five strain gauges were installed at 50cm intervals in the crown-steel pipe.

Figure 4-14-5 shows the loads (axial force, bending moment and shear force) in the steel pipe, where each step involves a 1-m-long excavation and installation of supports.

The following behavior of the pipe can be observed.

- Although it is not clearly observed, according to the reference, in the first step (step-1 in the figure), the pipe deflected downward and 0.2 - 1.0ton of tensile force occurred as an axial stress. A maximum shear force occurred in the middle of the pipe.
- In the second step (step-2 in the figure), the inflection points of both axial force and bending moment moved to the deep ground. The axial force was small.
- In the third step (step-3 in the figure), since the face moved beyond the tip of the pipe, compressive force occurred along almost the entire section of the pipe. Few changes were observed in both bending moment and shear force.
- It can be seen from the figure that in the steps 1 and 2, earth pressures acted on the pipe in the direction of the arrow (→) shown in the figure. Therefore, there was a supporting effect of the umbrella pipe against the loosening of the ground. In the step-3, on the other hand, from the fact that the compressive stresses occurred in almost the entire pipe and few changes in both bending moment and shear force were monitored, some axial forces in the direction of the arrow were added to the pipe.

Reference

Sakayama, Y., Igarashi, M., "Construction Report on the Hirai Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), June, 1993.

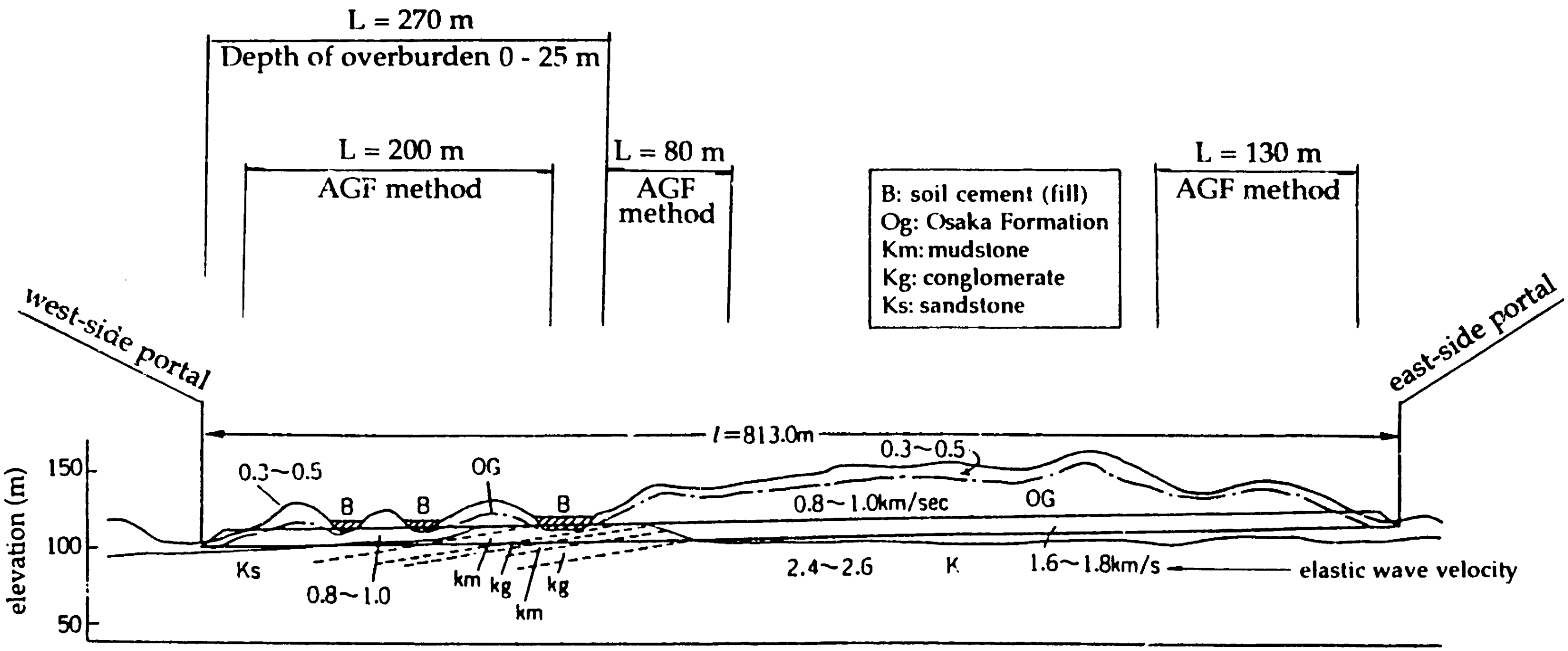


Fig. 4-14-1 Geological conditions: Hirai Tunnel (Sakayama et al., 1993)

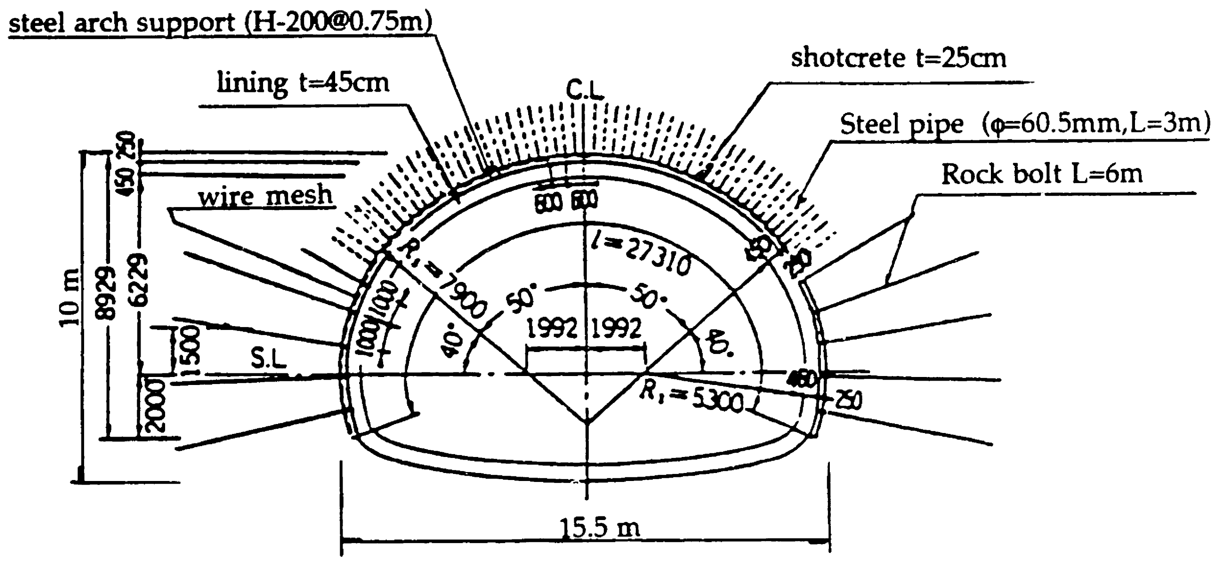


Fig. 4-14-2 Cross section: Hirai Tunnel (Sakayama et al., 1993)

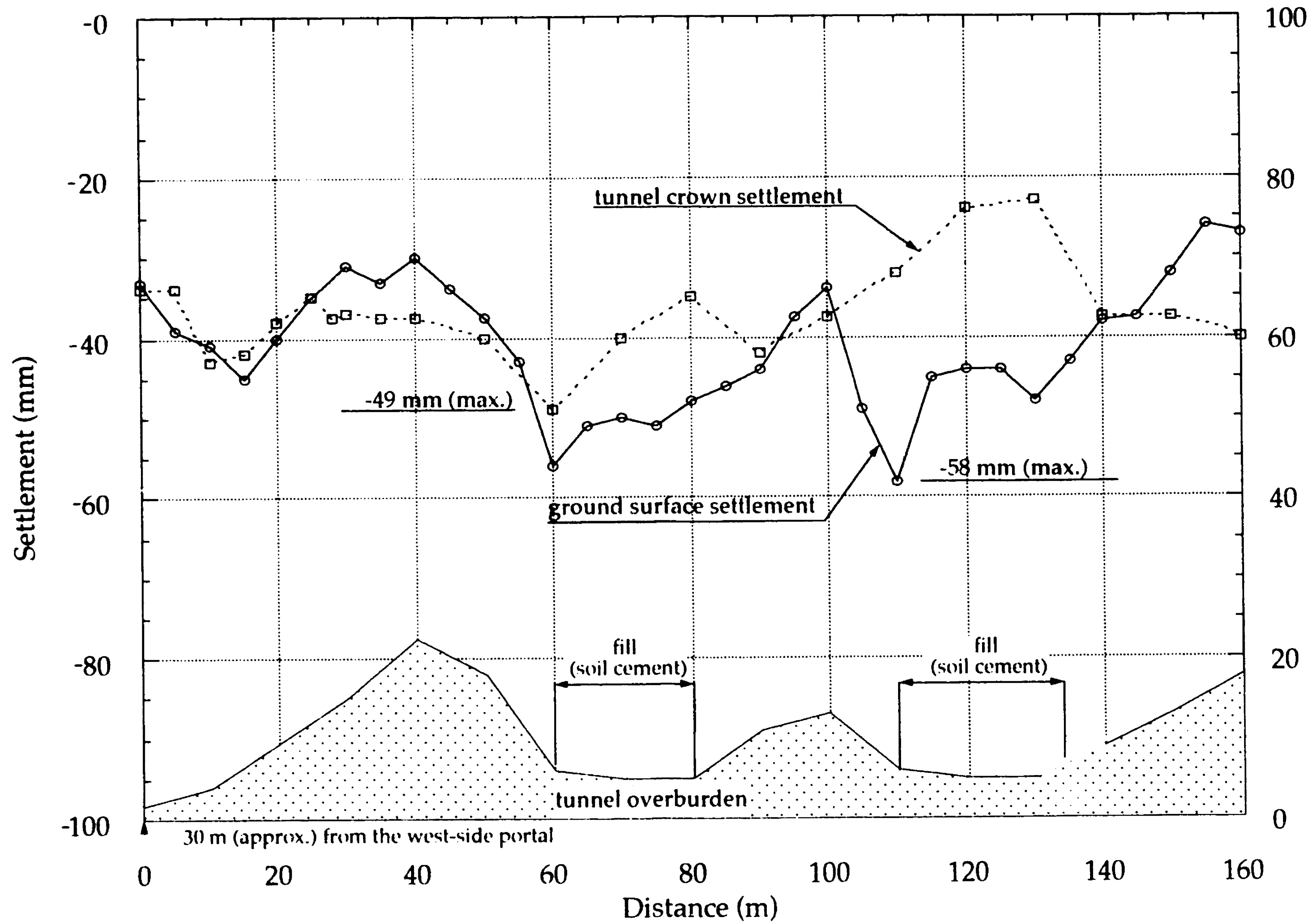


Fig. 4-14-3 **Distribution of final ground surface and tunnel crown settlements: Hirai Tunnel (Sakayama et al., 1993)**

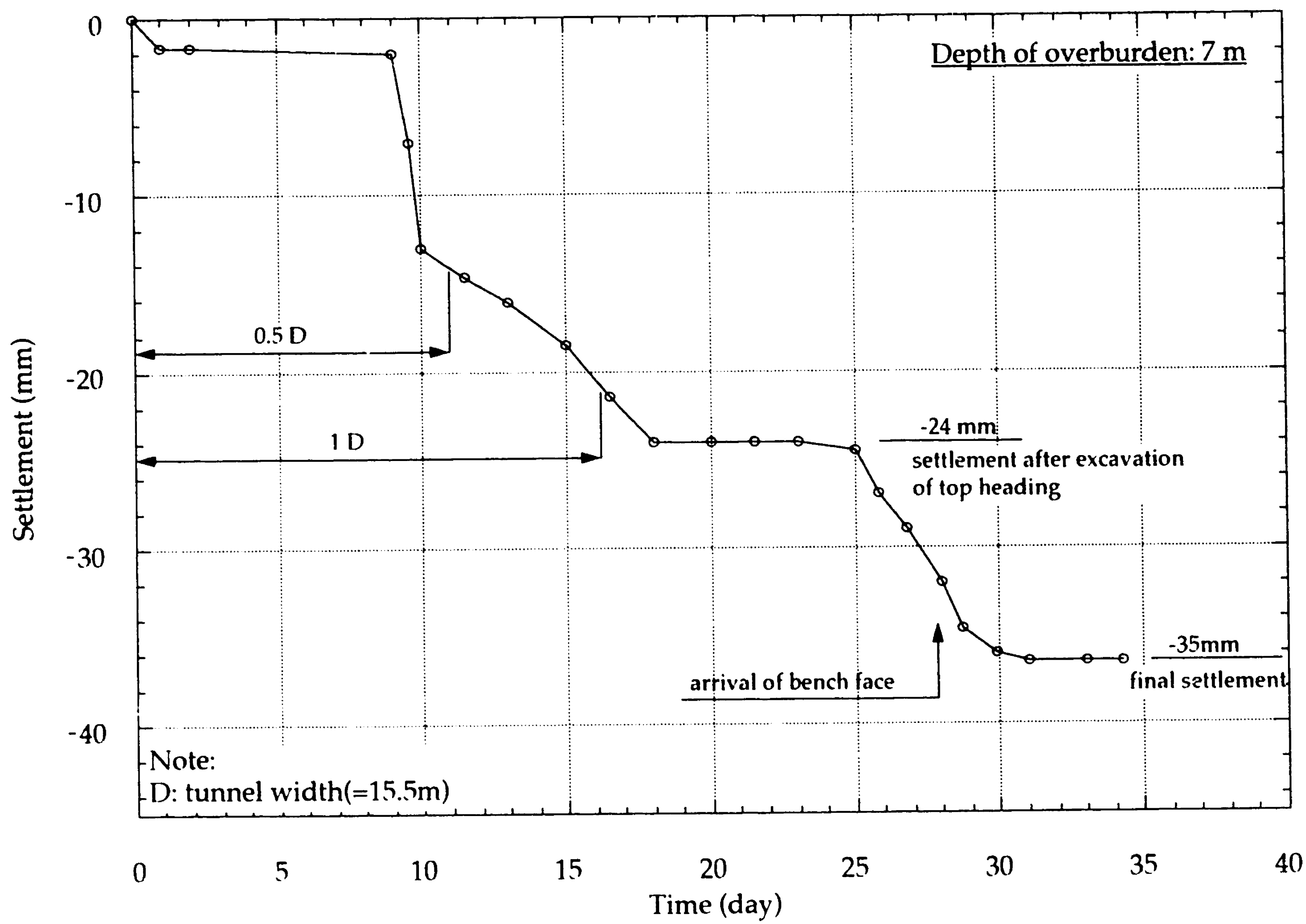
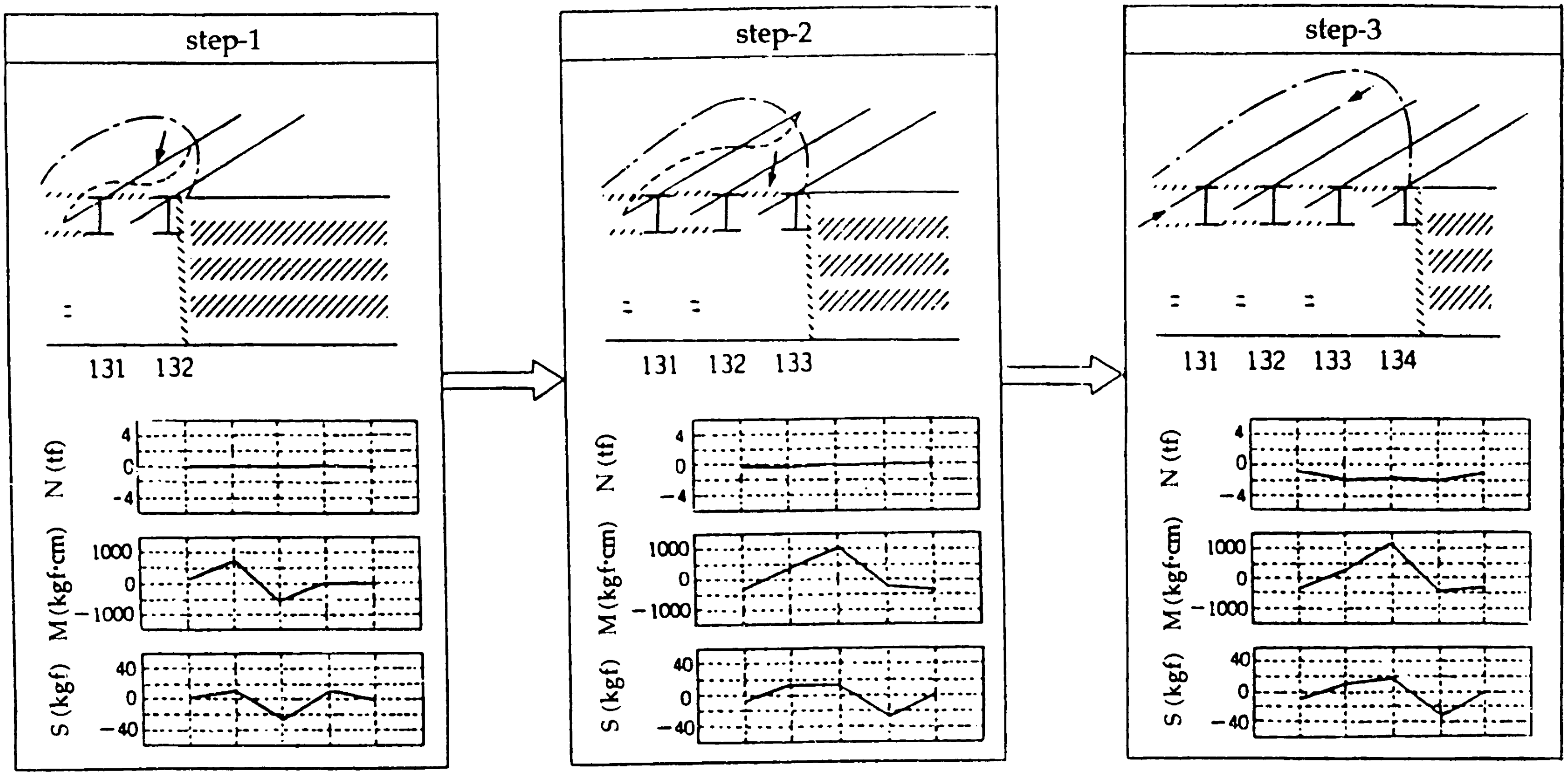


Fig. 4-14-4 **Tunnel crown settlement versus time: Hirai Tunnel (Sakayama et al., 1993)**



N: axial force M: bending moment S: shear force

Fig. 4-14-5 Loads in the crown-steel pipe: Hirai Tunnel (Sakayama et al., 1993)

Case 15: Futatsui-Nishi Tunnel (Futatsui, Japan)

Environment of the Futatsui-Nishi Tunnel

The 644-m-long Futatsui-Nishi Tunnel is a 2-lane motorway tunnel. The excavated width and height of the tunnel are about 12m and 9m, respectively and the excavated area is about $90m^2$. The depth of overburden in the zone in which the umbrella method was employed ranges from 0m to 9m.

Geological and Hydrological Conditions

The geological conditions of the tunnel are shown in Fig. 4-15-1.

As shown in the figure, a dip slope structure exists at the tunnel portal, in which tuffaceous sandstone (KS2, KS3, KS4) and weathered mudstone (Km2, Km3, Km4) alternate with one another.

Talus and highly weathered tuffaceous sandstone prevail at the tunnel portal.

Problems in Tunnel Construction

Landslides were expected to occur at the portal during tunnel excavation.

Supplementary Support Method of the Tunnel

The AGF method, one of the injected steel pipe umbrella methods, was employed at the tunnel portal to ensure safe excavation.

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

The cross section of the tunnel is shown in Fig. 4-15-2.

The fore-poles are 2.5m-and-3m-long steel pipes 101.6mm in diameter and 4.2mm wall thickness. The number of the fore-poles per cross section is 45.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 35cm thick.

Construction Procedures

The excavation was by heading-and-benching.

Steel pipes 11.5m in length were driven from an outside construction pit and steel pipes 8.5m in length were driven within the tunnel. The 11.5-m-long steel pipe consisted of three 3-m-long steel pipe sections and one 2.5-m-long steel pipe section, while the 8.5-m-long steel pipe consisted of two 3-m-long steel pipe sections and one 2.5-m-long steel pipe section.

The length of the tunnel section in which the umbrella method was employed was 18m, thus the overlap between the two consecutive umbrella arches was 2m.

The composition of the grout used is shown in Table 4-15-1.

Table 4-15-1 Injected grout: Futatsui-Nishi Tunnel

Grout (kgf/m^3)	cement	600
	bentonite	120
	water	800

Source: Wakasa et al. (1992)

Field Measurements

Figure 4-15-3 shows the tunnel crown settlement in relation to the progress of the face. Also, Fig. 4-15-4 shows the relationship between the tunnel crown settlement and the distance from the face. Note that the tunnel crown settlement was measured after the primary supports (shotcrete and steel arch support) were installed.

From the figures, the following can be seen:

- Settlement after excavation of the top heading was 5mm.
- Slight heaving of about 2mm occurred after the face proceeded about 2D beyond the measuring point.
- The settlement was characterized by the fact that settlement subsided relatively fast after excavation, that is, when the face reached a distance of about 1.5D (D: tunnel width) beyond the measuring point.

Fig. 4-15-5 shows the relationship between the maximum ground surface settlement and the depth of overburden.

From the figure, it can be seen that maximum ground surface settlement in the zone in which the umbrella method was employed ranged from 21mm to 38mm.

Earth pressures measured at the tunnel crown reached a maximum of 30kPa.

Assuming that the unit weight of soil above the tunnel crown is $17kN/m^3$ and considering that the depth of overburden at the portal is approximately 6.5m, overburden pressures acting on the tunnel crown, if no arching occurs, will be $110.5kN/m^3$. From the comparison between the measured value (30kPa) and the assumed value (110.5kPa), it can be seen that the arching effect occurred in spite of the shallow overburden above the tunnel.

Reference

Wakasa, R., Okamoto, S., Kawakami, K., "Construction Report on the Futatsui-Nishi Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), September, 1992.

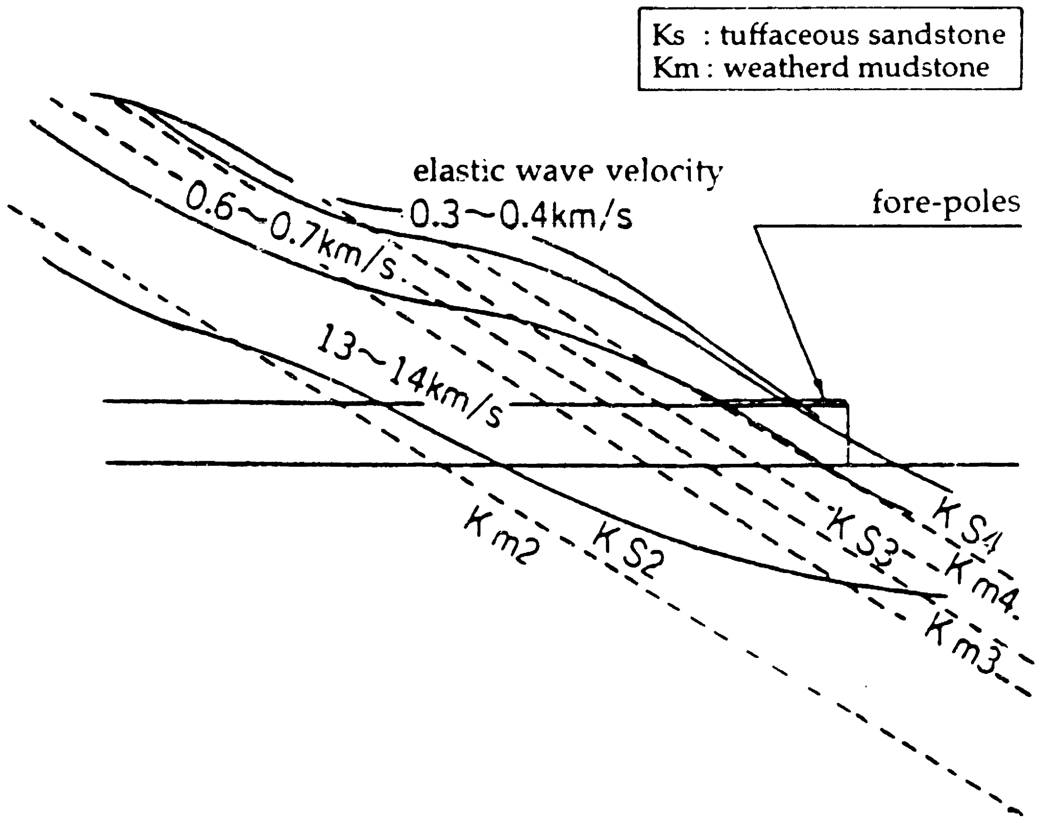


Fig. 4-15-1 Geological conditions: Futatsui-Nishi Tunnel (Wakasa et al., 1992)

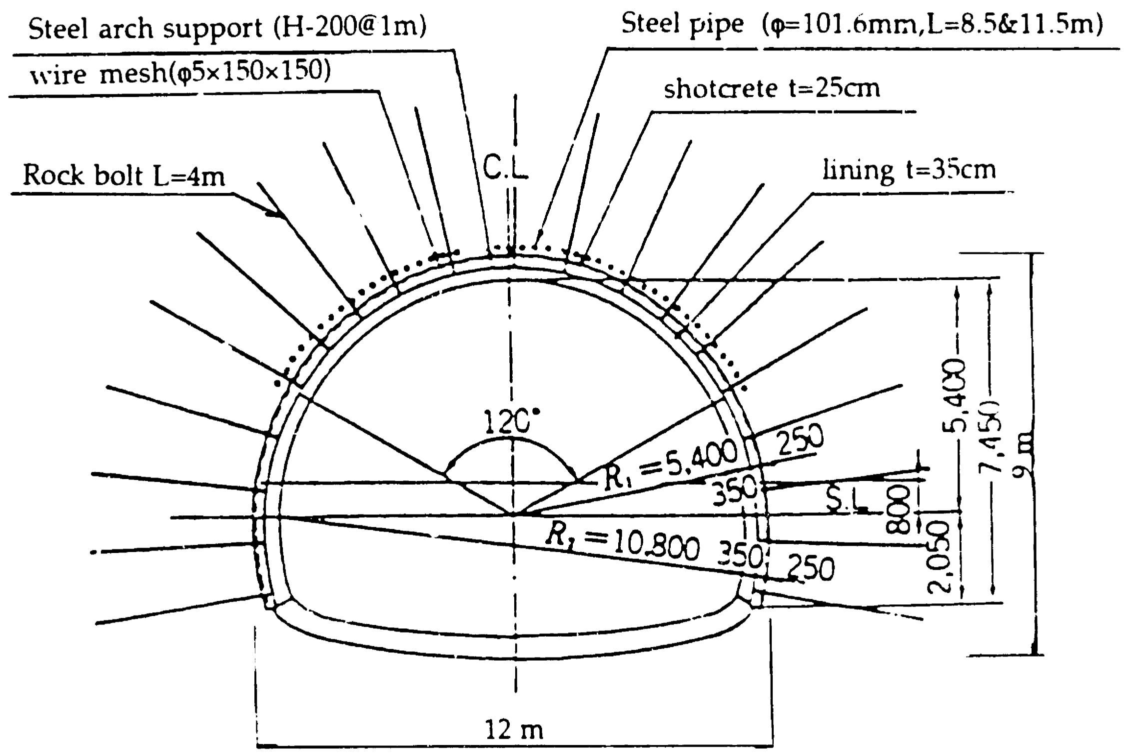


Fig. 4-15-2 Cross section: Futatusi-Nishi Tunnel (Wakasa et al., 1992)

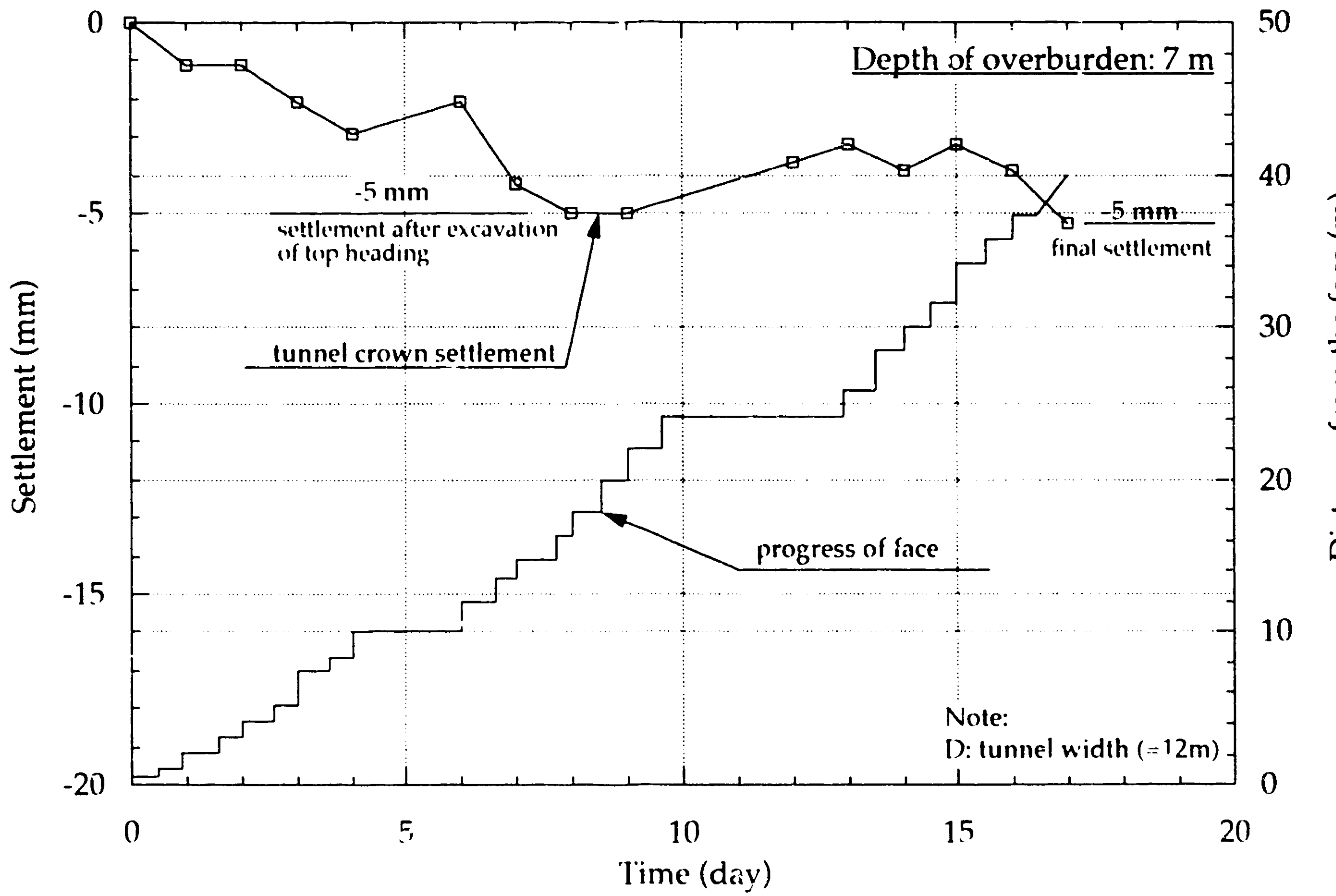


Fig. 4-15-3 Tunnel crown settlement in relation to the progress of the face: Futatsui-Nishi Tunnel (Wakasa et al., 1992)

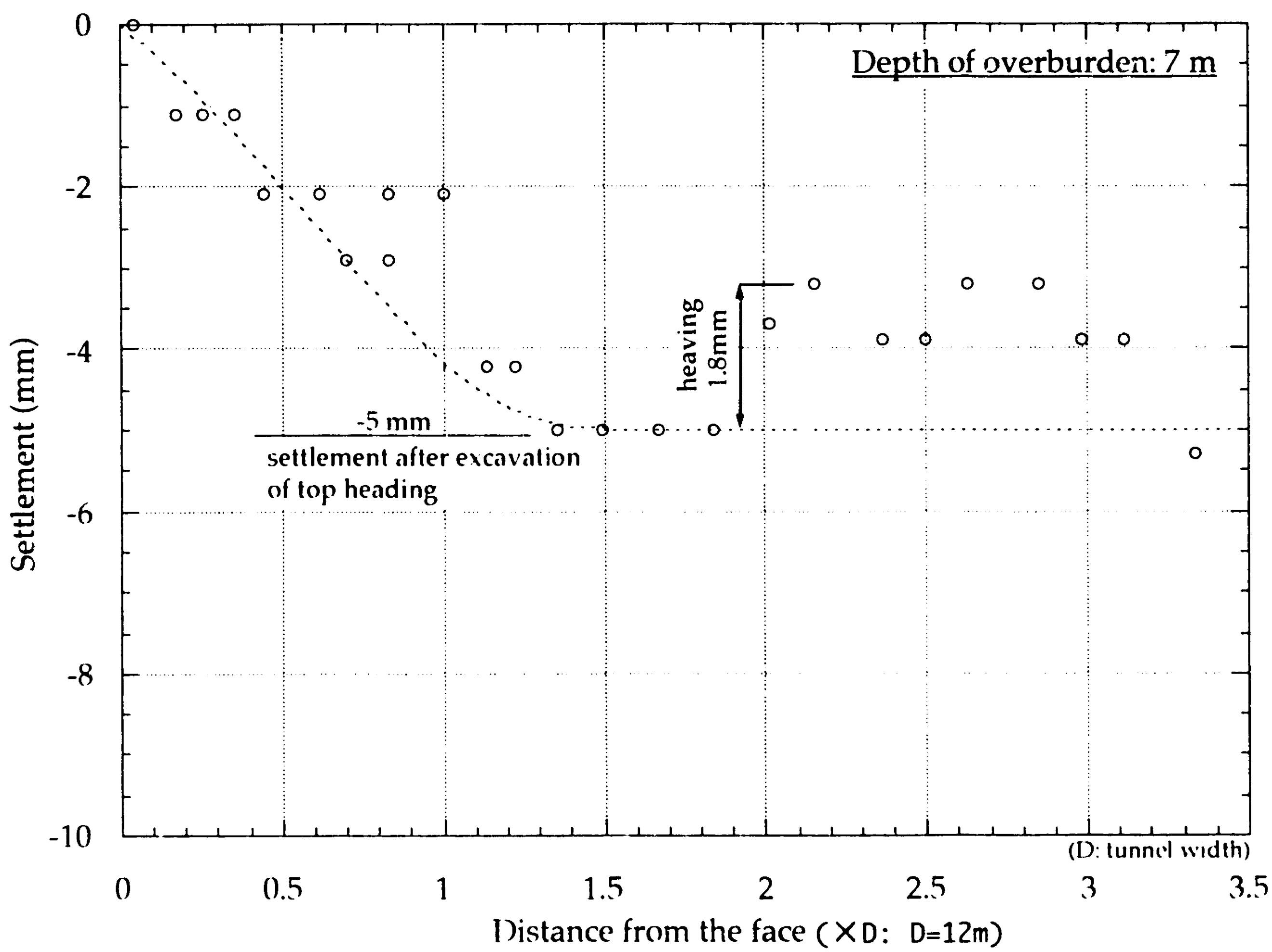


Fig. 4-15-4 Tunnel crown settlement versus distance from the face: Futatsui-Nishi Tunnel (Wakasa et al., 1992)

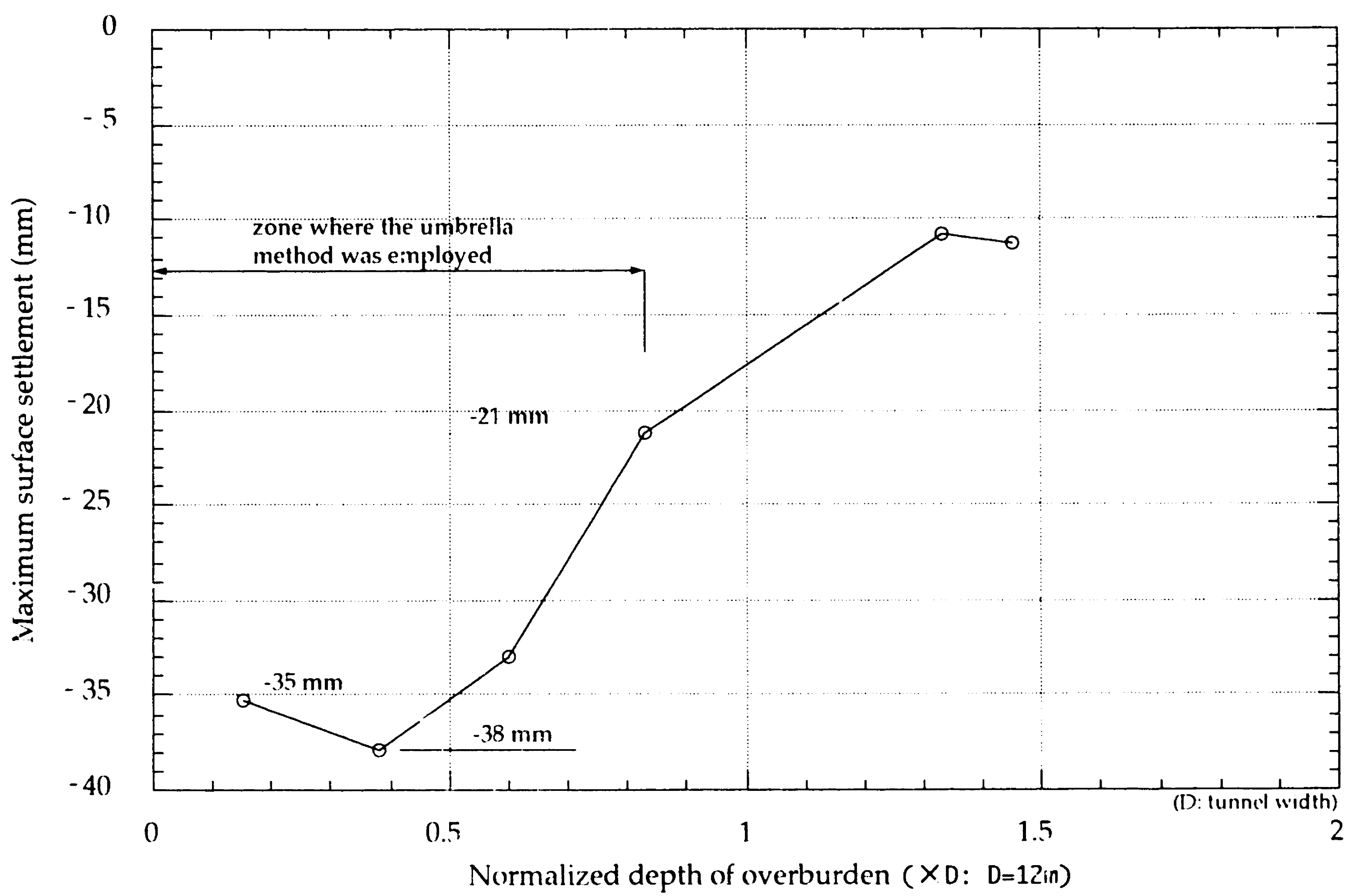


Fig. 4-15-5 Maximum surface settlement versus normalized depth of overburden: Futatsui-Nishi Tunnel (Wakasa et al., 1992)

Case 16: Yakiyama Tunnel (Niigata, Japan)

Environment of the Yakiyama Tunnel

The 2986-m-long Yakiyama Tunnel with two lanes is a motorway tunnel on the Ban-etsu Expressway. The excavated width and height of the tunnel are approximately 12m and 10m, respectively and the excavated area is approximately 100m^2. The depth of overburden in the zone where the umbrella method was employed ranges from 0m to 9m.

Geological and Hydrological Conditions

The geological conditions of the tunnel are shown in Fig. 4-16-1.

As shown in the figure, at the tunnel portal sound sandstone bedrock is covered by talus deposits and weathered sandstone. A fracture zone comprising fault clay and fault breccia with only 7-m-deep overburden was predicted to exist approximately 10 m from the entrance.

Inflow of groundwater was expected not to be much although a high water head was observed.

Problems in Tunnel Construction

During slope-cutting for the neighboring abutment construction of a bridge conducted before the tunnel construction, the weathered sandstone of the slope collapsed in a small area. There was a weathered sandstone layer at the tunnel portal as well, so that the face was expected to have poor self-supporting characteristics.

Supplementary Support Method of the Tunnel

The injected steel pipe umbrella method was applied to stabilize the face and to preserve as much as possible the natural stress state of the ground.

The initially planned 3m-long rock-bolt fore-poling, for crown-stabilization, was considered insufficient.

Structural Details

The cross section of the tunnel is shown in Fig. 4-16-2.

The fore-poles are 15-to-18-m-long steel pipes 114.3mm in diameter and 6mm wall thickness. The number of the fore-poles per cross section is 33.

The pipes were installed parallel to the tunnel axis at intervals of 40cm. Each of the 15-to-18-m-long steel pipes was composed of 3m and 1.5m long components that were connected during installation.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports (at unknown intervals). The secondary lining is 35cm thick.

Construction Procedures

The tunnel portal was excavated by full-face method with 4m round length driven with a 3-boom drill jumbo.

All of the steel pipes were driven from an outside construction pit before tunnel excavation. Pipe drills and injection procedures are shown in Fig. 4-16-3. The diameter of the "improved bulb" shown in the figure was assumed to be 40cm, which was equal to the pipe interval. The amount of cement milk to be injected was 30% of the bulb volume. After the cement milk in the clearance between the pipe casing and the surrounding ground had half-hardened, more cement milk pressurized at up to 2.0MPa was injected into the ground.

Field Measurements

Figure 4-16-4 shows the settlement of the tunnel-crown steel pipe.

From the figure, it can be seen that the maximum steel pipe settlement moved ahead with the excavation. The peak settlement of about 22 mm was reached at a distance about 10 m from the portal.

Figure 4-16-5 shows the relationship between the settlement and the distance from the face in the section where the maximum settlement appeared.

From Fig. 4-16-5, the following can be seen:

- Pre-excavation settlements of the ground surface and the crown steel pipe were 12mm and 10mm, respectively.
- Both settlements gradually increased after the face reached a point about 3m ahead of the measuring point and subsided about 9m beyond the measuring point.
- The ground surface settlement was as large as the crown steel pipe settlement.

Reference

Kotake, N., Yamamura, K., Manabe, T., 'A Case History of Portal Excavation with Injected Steel-pipe Fore-poling," Proceedings of the South East Asian Symposium on Tunnelling and Underground Space Development, Japan Tunneling Association (JTA), 1995.

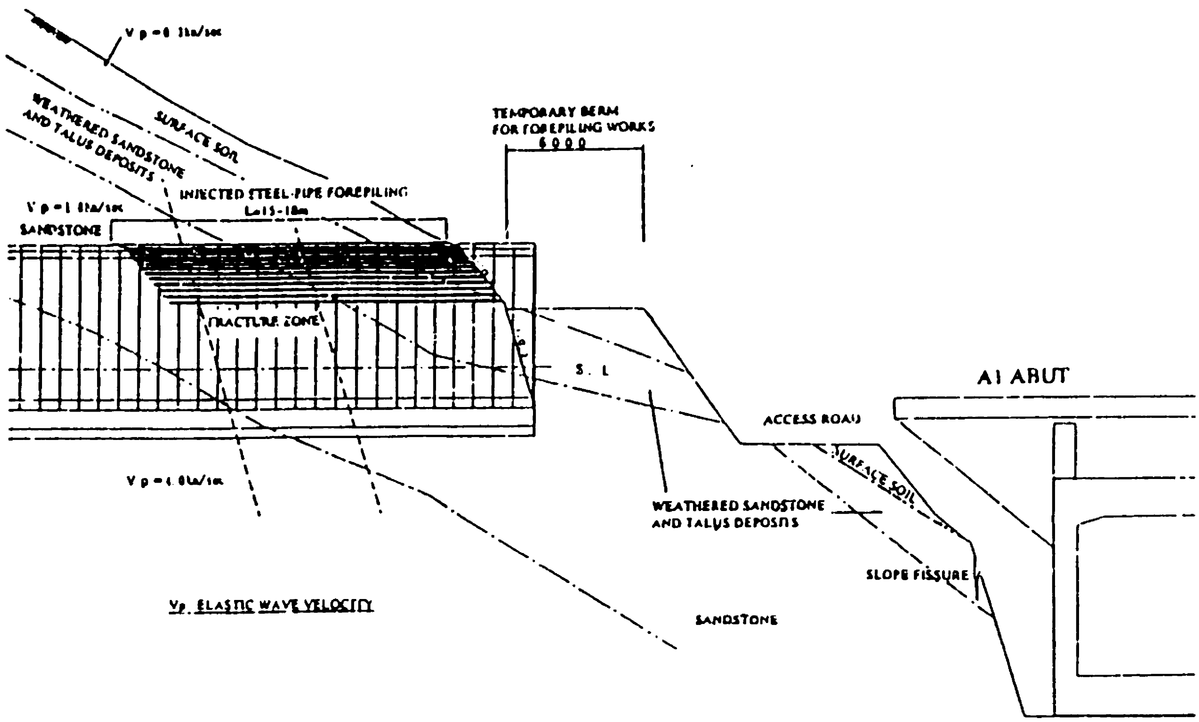


Fig. 4-16-1 Geological conditions: Yakiyama Tunnel (Kotake et al., 1995)

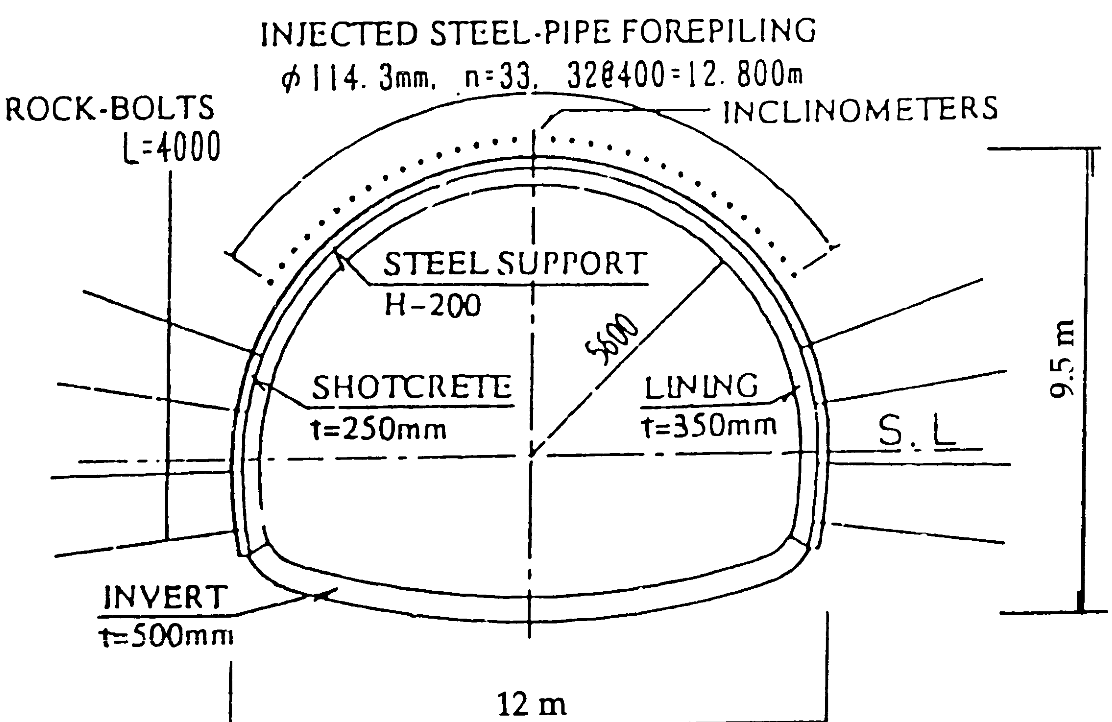


Fig. 4-16-2 Cross section: Yakiyama Tunnel (kotake et al., 1995)

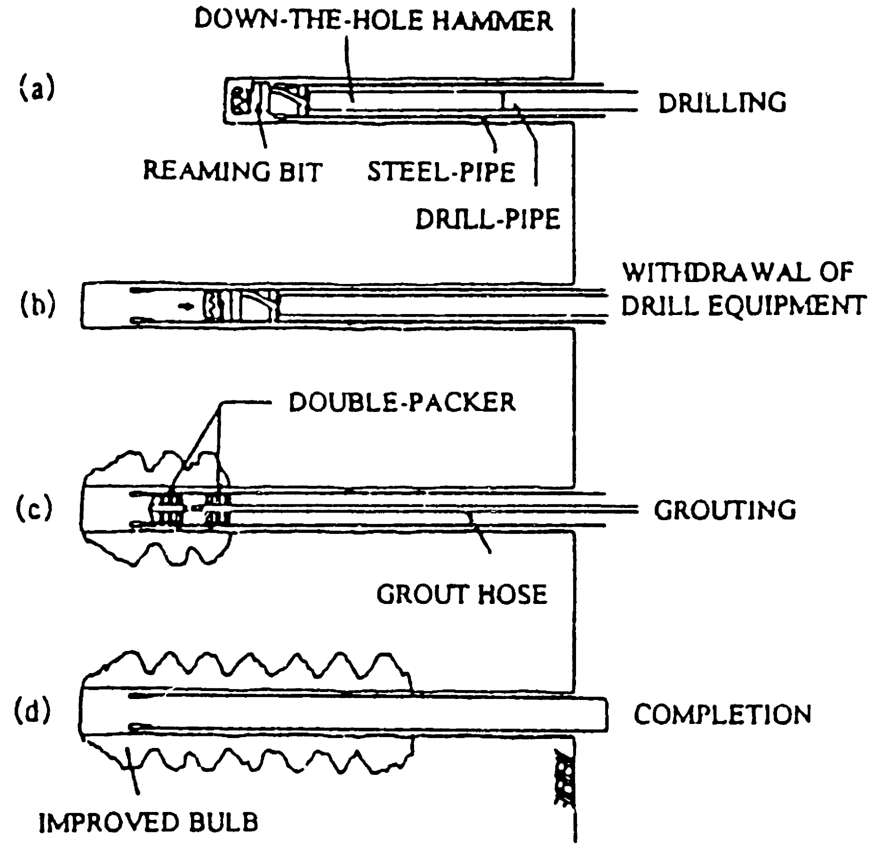


Fig. 4-16-3 Pipe drills and injection procedures: Yakiyama Tunnel (Kotake et al., 1995)

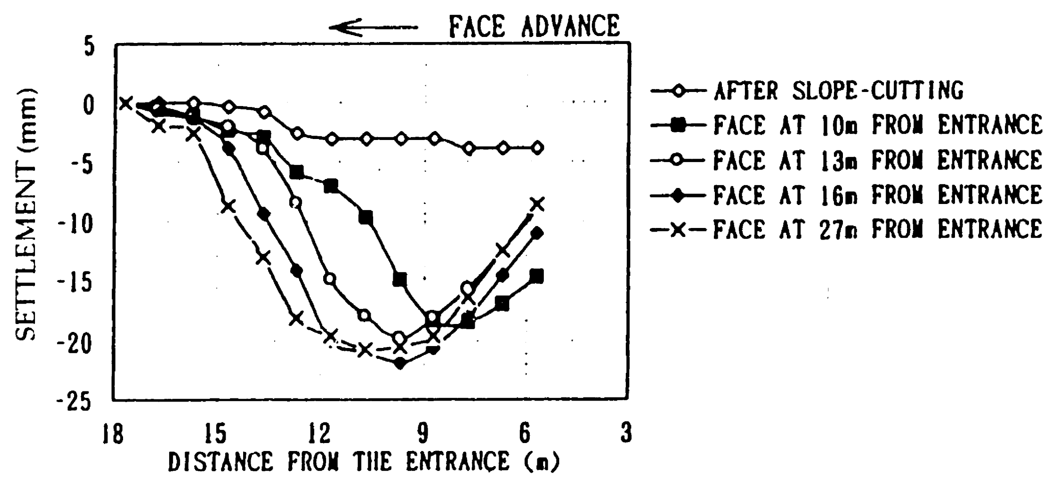


Fig. 4-16-4 Crown-steel pipe settlement: Yakiyama Tunnel (Kotake et al., 1995)

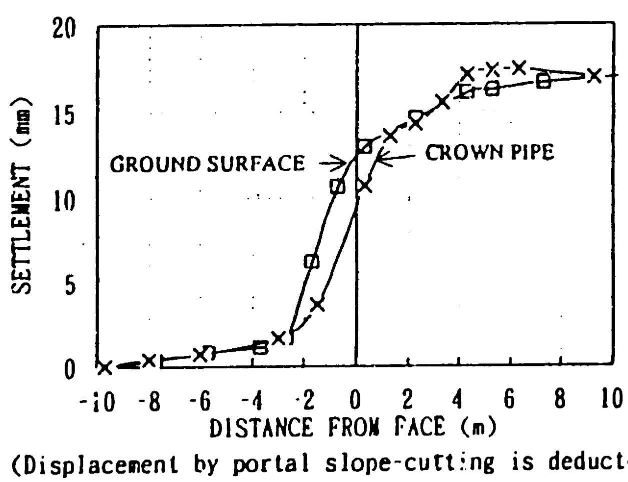


Fig. 4-16-5 Ground surface and crown-steel pipe settlements in the section of 10m from the tunnel portal: Yakiyama Tunnel (Kotake et al., 1995)

Case 17: Ramat Tunnel (Piedmont, Italy)

Environment of the Ramat Tunnel

The Turin-Bardonecchia motorway runs through the Susa valley in the east-west direction. The motorway includes five twin-bore tunnels with a total of 26km under ground. The 500-m-long Ramat Tunnel is the easternmost tunnel of the project. The excavated width and height are approximately 12m and 10m, respectively and the excavated area is approximately $90m^2$.

Geological and Hydrological Conditions

The ground through which the tunnel passes is a water-bearing deposit of glacial moraines composed of sand, gravel and large boulders.

Problems in Tunnel Construction

Tunnelling was complicated by the following factors:

- The tunnel had a shallow overburden.
- The site was of archaeological interest.
- Settlement of the ground was not permitted.
- The environmental regulations required that the large boulders above the tunnel be left in place.

Supplementary Support Method of the Tunnel

The injected steel pipe umbrella method was employed.

The sub-horizontal jet-grouting method was rejected because of the presence of large blocks of rock. Concrete and chemical grouting were rejected because a successful result was not guaranteed due to the heterogeneous morainic deposits with big voids or clay lenses.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-17-1.

The fore-poles are 12-m-long steel pipes 88.9mm in diameter. The number of the fore-poles per cross section ranges from 33 to 45.

Tunnel support consists of a 30-cm-thick primary lining (shotcrete) and H-180 section steel arch supports installed at 1m intervals.

The footings of the steel arch support were stabilized with micropiles (not shown in the figure).

Construction Procedures

9m of the tunnel section was excavated before installing the next umbrella arch to maintain a 3-m overlap for the protection of the face.

A special drilling machine was designed to drill the holes and install the pipes. This was a crawler-type drilling machine with the Odex eccentric bit system (see Ch. 3.4).

The injected pipes were equipped with an injection valve every 60cm. The valves, called "Microtrevi type", were made of thin steel sheetmetal bands which were embedded in the thickness of the pipe (see Fig. 4-17-1). The grout consisted of a binary water-cement suspension with a ratio of 1:1 to which additives were added.

As shown in Fig. 4-17-2, additional grouting was carried out from the ground surface at the low overburden section below the archaeological site.

Field Measurements

No information on field measurements was available from the reference.

Reference

Pelizza, S., Barisone, G., Campo, F., "Neolithic Site Kept Safe under Italian Umbrella," Tunnels and Tunnelling, Miller Freeman, March, 1990.

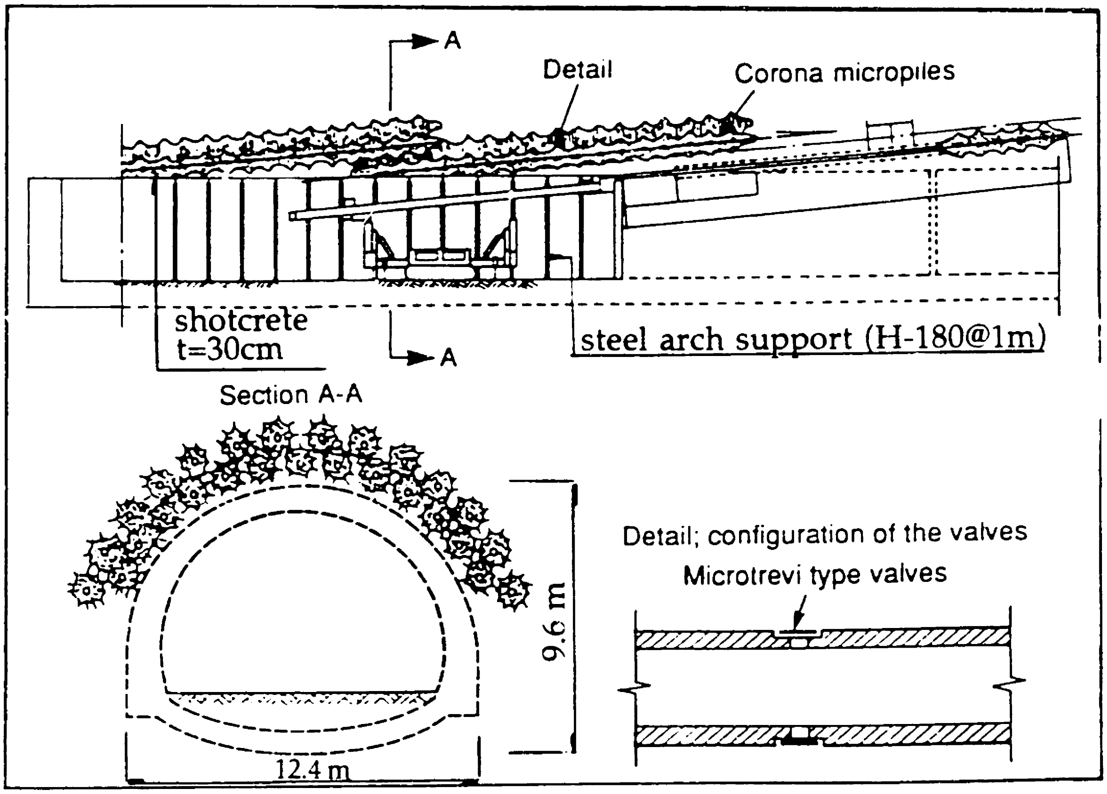


Fig. 4-17-1 Cross and longitudinal sections: Ramat Tunnel (Pelizza et al., 1990)

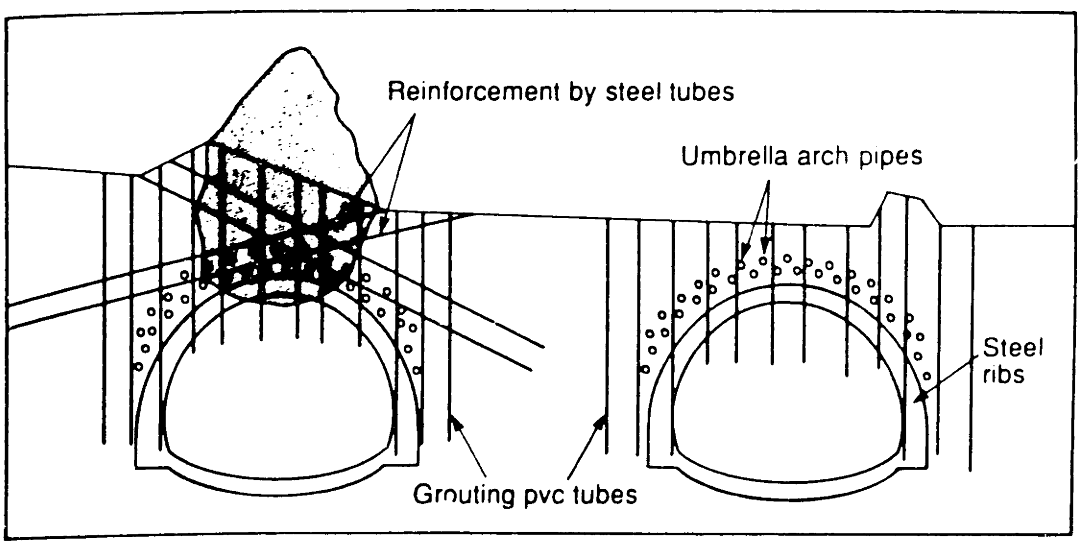


Fig. 4-17-2 Grouting from ground surface: Ramat Tunnel (Pelizza et al., 1990)

Case 18: Poggio Fornello Tunnel (Tuscany, Italy)

Environment of the Poggio Fornello Tunnel

The 550-m-long Poggio Fornello Tunnel is a tunnel on the state road "Aurelia" in Tuscany, Central Italy. The excavated width and height are approximately 11m and 9.5m, respectively and the excavated area is approximately 104m^2. The maximum overburden is 50m.

Geological and Hydrological Conditions

The tunnel passes through a rock mass formed by *Cretaceous flysch,* which belongs to a group of sedimentary rocks known as *turbiditic formations.* Turbidites were formed by the sedimentation from turbidity currents, which carried and deposited erosion materials coming from a rapidly uplifting alpine belt, at the mouth of channels and in submarine canyons within subsiding basins.

In the *Cretaceous flysch* the strata are less than 10cm thick. The predominant stratum is argillaceous slate, which alternates with calcareous and arenareous layers. Based on the Rabcewicz-Pacher classification, this rock belong[illegible] 5b class, which implies that the rock is very weak.

The engineering properties of the rock encountered in this tunnel construction are summarized in Table 4-18-1.

Table 4-18-1 Engineering properties: Poggio Fornello Tunnel

Rock type	Modulus of deformation (MPa)	C (kPa)	φ (°)
flysch	2000	50	20

Source: Pelizza et al. (1994)

Problems in Tunnel Construction

Severe problems of tunnel stability were expected during tunnel excavation due to the poor ground conditions.

Supplementary Support Method of the Tunnel

The injected steel pipe umbrella method was employed.

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-18-1.

It should be pointed out that the stabilization pattern in the upper part of the figure was adopted for heading-and-benching excavation; the stabilization pattern in the lower part of the figure, on the other hand, was for full face excavation.

In both patterns, the fore-poles are 12 and 9-m-long steel pipes. The number of the fore-poles per cross section is 32.

Tunnel support consists of a 30-cm-thick primary lining (shotcrete) and coupled steel arch supports with double T of 200 mm installed at 0.75m intervals.

Construction Procedures

For about the first 60m of the tunnel from the entrance, excavation was by heading-and-benching.

9m of the tunnel section was excavated before installing the next umbrella arch to maintain a 3-m overlap for the protection of the face. 6 to 8-m-long radial reinforcements were carried out as the face advanced for 8 m. The reinforcements were cemented and sometimes prestressed steel bars.

For the last 170m of the tunnel, the ground conditions became worse, with a maximum depth of overburden of 25m. Large deformations of the upper section of the tunnel occurred and required local propping to prevent the tunnel support from buckling. It was judged to be impossible to carry on excavation with heading-and-benching method.

In order to overcome the geotechnical difficulties, in this section the excavation method was changed from heading-and-benching method to full face excavation.

In this section of 170m, a face reinforcement was applied with fiber-glass pipes which were inserted into the ground and were placed parallel to the tunnel axis. It was thought that these pipes would stiffen the ground and increase its resistance to shearing stresses.

Field Measurements

No information on field measurements was available from the reference.

Reference

Pelizza, S., Corona, G., Graffi, G., Grasso, F., Raineri, R., "Improvement of Stability Conditions from Half to Full Face Excavation in Difficult Geotechnical Conditions," Tunnelling and Ground Conditions, Abdel Salam (ed.), Balkema, Rotterdam, 1994.

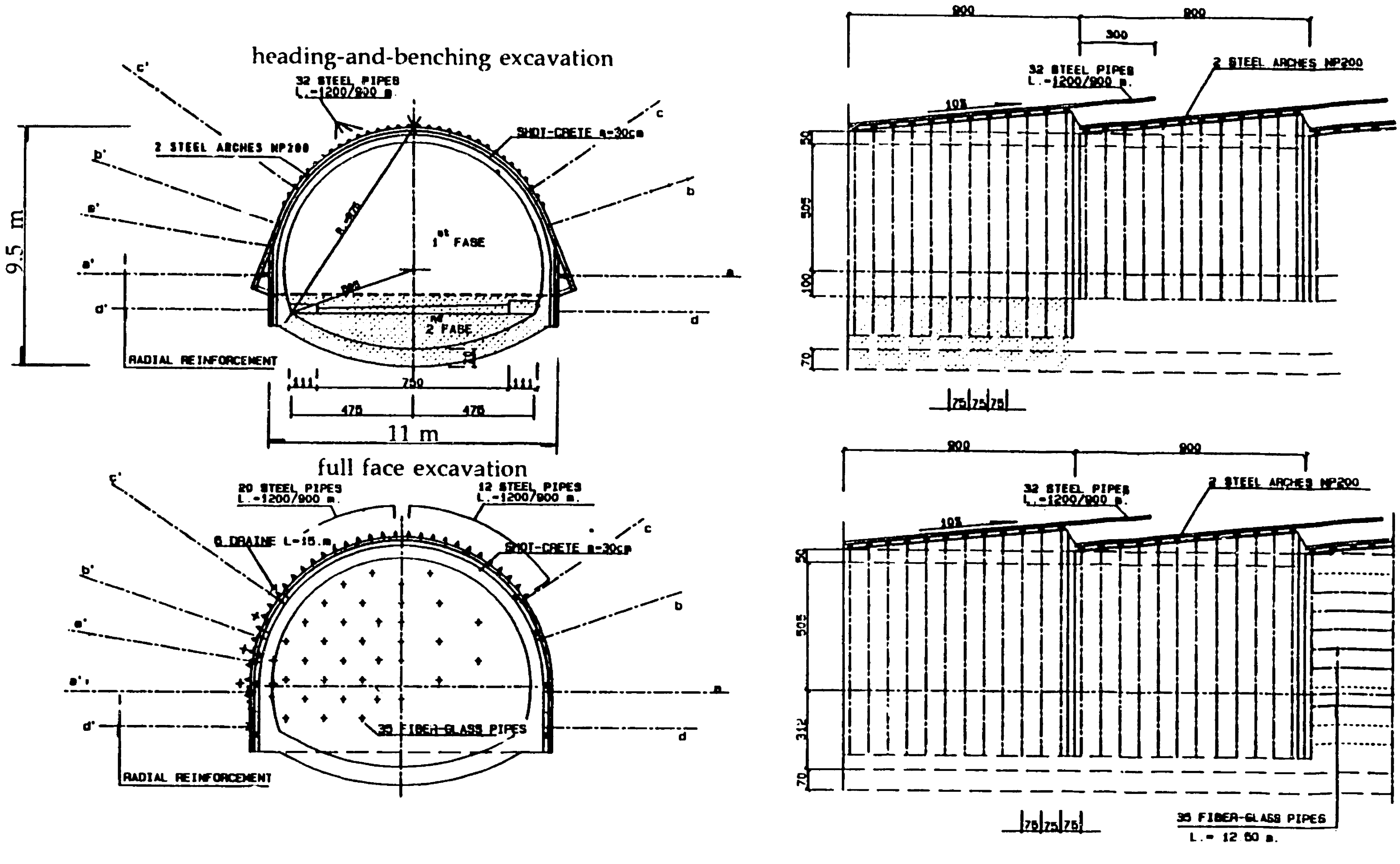


Fig. 4-18-1 Cross and longitudinal sections: Poggio Fornello Tunnel (Pelizza et al., 1994)

Case 19: St. Ambrogio Tunnel (Sicily, Italy)

Environment of the St. Ambrogio Tunnel

The Messina-Palermo motorway runs along the north coast of Sicily for a total length of 172.3km. As the area is mountainous the motorway involves the construction of many viaducts and tunnels. Of four tunnels, the St. Ambrogio Tunnel is used as a case study. The 1780-m-long St. Ambrogio Tunnel has an excavated width of approximately 14m and height of approximately 10.5m. The average excavated area is $110m^2$. The maximum overburden is approximately 200m.

Geological and Hydrological Conditions

The most common geological formation in the construction area is known as *Flysch Numidico,* which belongs to a group of sedimentary rocks known as *turbiditic formations.*

Flysch Numidico includes arenaceous-pelitic sediments of the Miocene age. The arenaceous facies consists of very hard quartzstone in thick layers, while the pelitic facies consists of a sequence of thin mudstone and claystone layers alternating with occasional siltstone levels, a few centimeters thick.

Turbiditic formations are defined as structurally complex formations.

Problems in Tunnel Construction

The flysch composed of fissured claystone and weathered sandstone makes up most of the rock mass. This is a very complex rock, which is often very poor and very sensitive to water. Therefore, many instability problems in tunnel construction were expected to occur.

Moreover, at that time the St. Ambrogio Tunnel was one of the first modern large size tunnels, dug in Italy through such geological formations, so that there was no direct experience concerning the behavior of the rock mass during excavation.

Supplementary Support Method of the Tunnel

The injected steel pipe umbrella method was employed to overcome the difficulties in tunnel construction.

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-19-1.

The fore-poles are 12 to 16-m-long steel pipes. The number of the fore-poles per cross section is 24.

Tunnel support consists of a 25 to 30-cm-thick primary lining (shotcrete) and H-180 or 200 section double steel arch supports (at unknown installation intervals).

Construction Procedures

Excavation was by the multiple-face method. The face was divided into at least three phases: heading, bench and invert arch.

9 to 12m of the tunnel section was excavated before installing the next umbrella arch to maintain a 3 to 4-m overlap for the protection of the face.

The footings of the steel arch support were anchored to the rock with horizontal anchors because of insufficient bearing capacity beneath the footings. In particularly bad conditions sub-vertical steel pipes were also installed beneath the footings.

Field Measurements

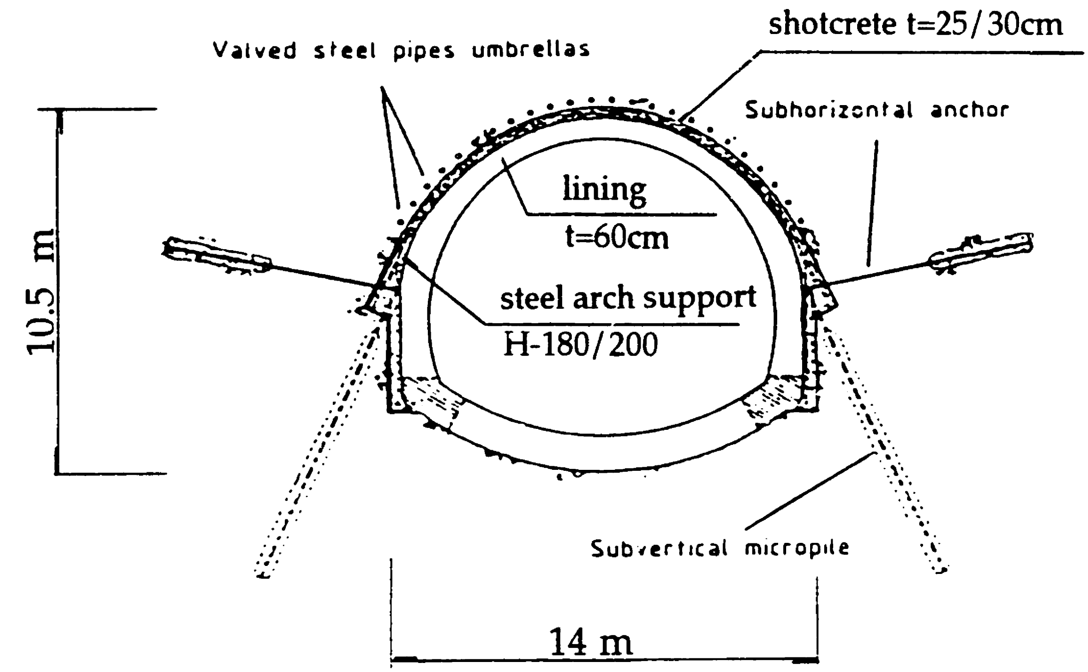
Convergence and settlement were generally less than 10cm, and only occasionally up to 20cm.

No other information was available from the reference.

Reference

Pelizza, S., Gangale, G., Corona, G., "The Messina-Palermo Motorway: Complex Rock Masses and Tunnelling Problems," Towards New Worlds in Tunnelling, Vieitez-Utesa and Montanez-Cartaxo (eds.), Balkema, Rotterdam, 1992.

(a) Cross section



(b) Longitudinal section

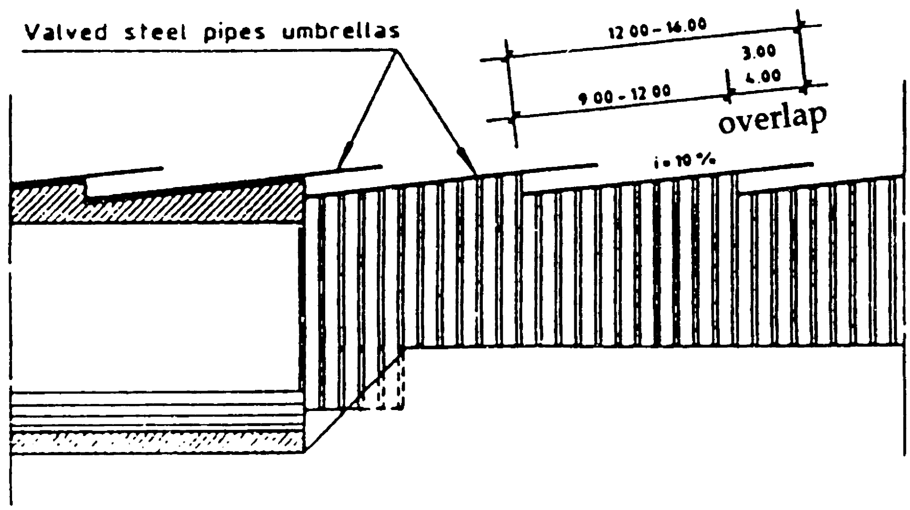


Fig. 4-19-1 Cross and longitudinal sections: St. Ambrogio Tunnel (Pelizza et al., 1992)

Case 20: Nango Tunnel (Hayama, Japan)

Environment of the Nango Tunnel

The 890-m-long Nango Tunnel is a motorway tunnel with an average excavated area of approximately $110m^2$. The excavated width and height are approximately 15m and 10m, respectively. The depth of overburden in the zone where the umbrella method was employed ranges from 3m to 21m.

Geological and Hydrological Conditions

The tunnel passes under a school. The ground material is fill, mostly consisting of mudstone blocks and clays.

Problems in Tunnel Construction

Instability of the face and ground surface settlement due to the shallow overburden above the tunnel crown and very loose ground conditions was expected.

Supplementary Support Method of the Tunnel

The injected steel pipe umbrella method (the TREVITUB method) was employed to improve face stability and restrict ground surface settlement.

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-20-1.

The fore-poles are 12-m-long steel pipes 114.3mm in diameter and 11mm wall thickness. The number of the fore-poles per cross section is 43.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 40cm thick.

The footings of the steel arch support were stabilized with footing piles.

It should be pointed out that the thickness of the steel pipes is larger than that of the pipes usually used (t = 6 - 7mm). This is so because the tunnel passes through the very loose fill layer, so that the ground arching effect is not likely to occur.

Construction Procedures

9m of the tunnel was excavated before installing the next umbrella arch to maintain a 3-m overlap for the protection of the face.

The grout consisted of a binary high early strength cement-water suspension with a ratio of 1:0.75 and the total injection technique was employed (see Ch. 3.7).

Field Measurements

The following was observed during and after tunnel excavation:

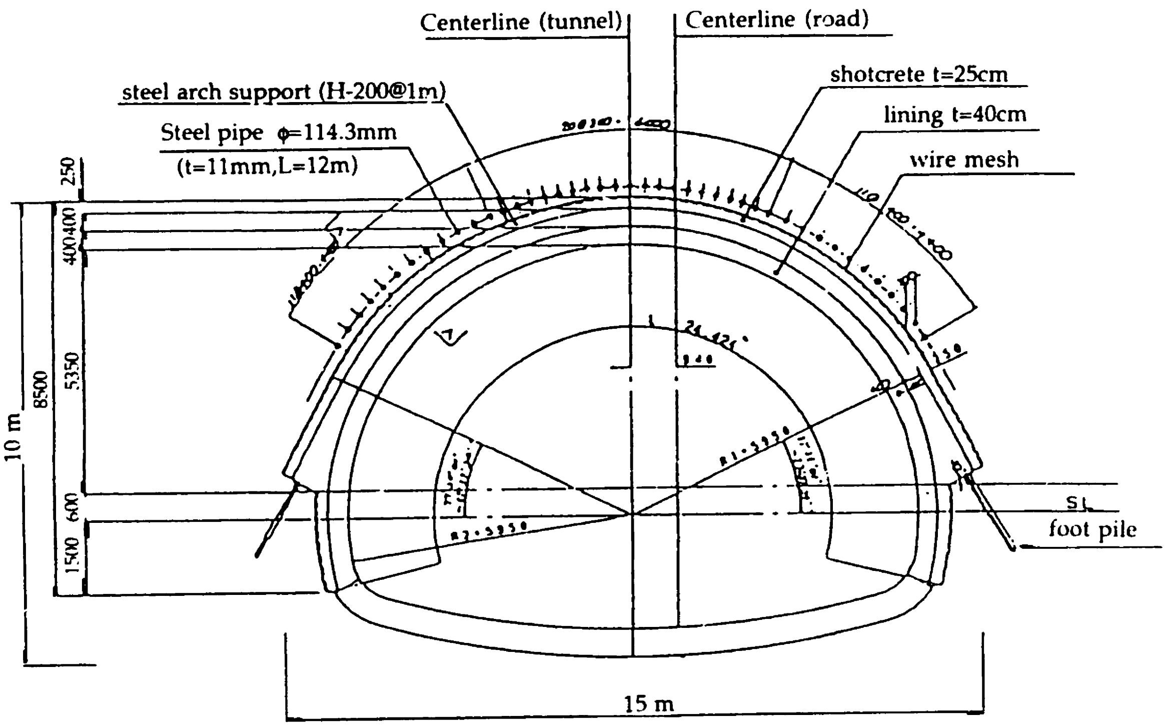
- After passage of the face, the ground surface settlement was almost equivalent to the tunnel crown settlement.
- The ground surface settlement subsided after the face proceeded 4D - 5D (D: tunnel width) beyond the measuring point.

No other information regarding the absolute settlement magnitude was available from the reference.

Reference

Geo-Fronte Research Association, Achievements of the TREVITUB Method, December, 1995.

(a) Cross section



(b) Longitudinal section

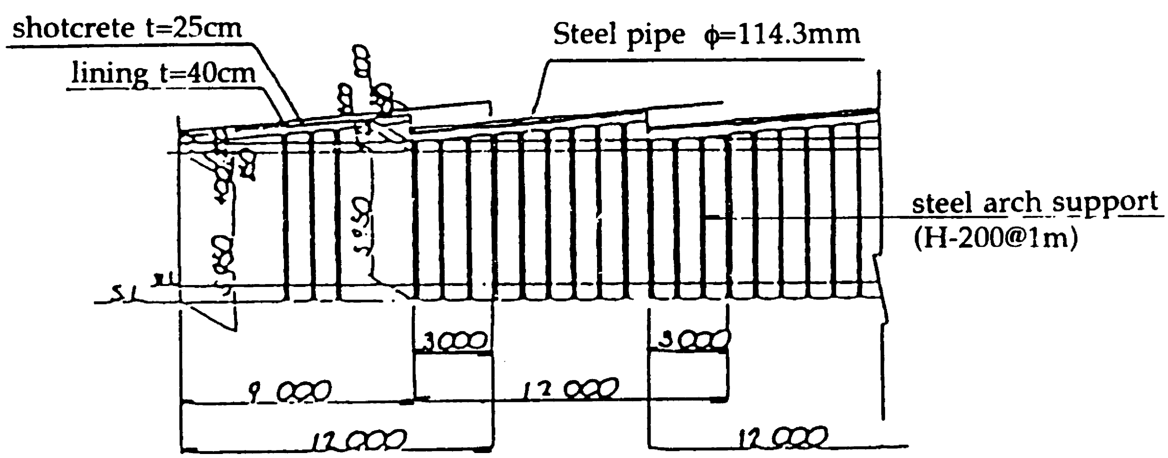


Fig. 4-20-1 Cross and longitudinal sections: Nango Tunnel

(Geo-Fronte Research Association, 1995)

Case 21: Kubodaira Tunnel (Shioyama, Japan)

Environment of the Kubodaira Tunnel

The 680-m-long Kubodaira Tunnel is a motorway tunnel with an excavated area of approximately $105m^2$. The excavated width and height are approximately 13m and 9.5m, respectively. The depth of overburden in the zone where the umbrella method was employed is about 5m.

Geological and Hydrological Conditions

The geology of the tunnel consists of river bed deposits including boulders and cobbles.

Problems in Tunnel Construction

Residential areas were adjacent to the tunnel, and a main road crossed above the tunnel. Therefore, a construction method was needed which would restrict settlement of surface structures such as the buildings and the road.

Supplementary Support Method of the Tunnel

The injected steel pipe umbrella method (the TREVITUB method) was employed to minimize ground surface settlement.

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

The cross and longitudinal sections of the tunnel are shown in Fig. 4-21-1.

The fore-poles are 12-m-long steel pipes 114.3mm in diameter and 6mm wall thickness. The number of the fore-poles per cross section is 31.

Tunnel support consists of a 25-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 45cm thick.

Construction Procedures

9m of the tunnel was excavated before installing the next umbrella arch to maintain a 3-m overlap for the protection of the face.

The grout was urethane because it was estimated that a cement suspension was not applicable in permeable ground with much groundwater flow.

Field Measurements

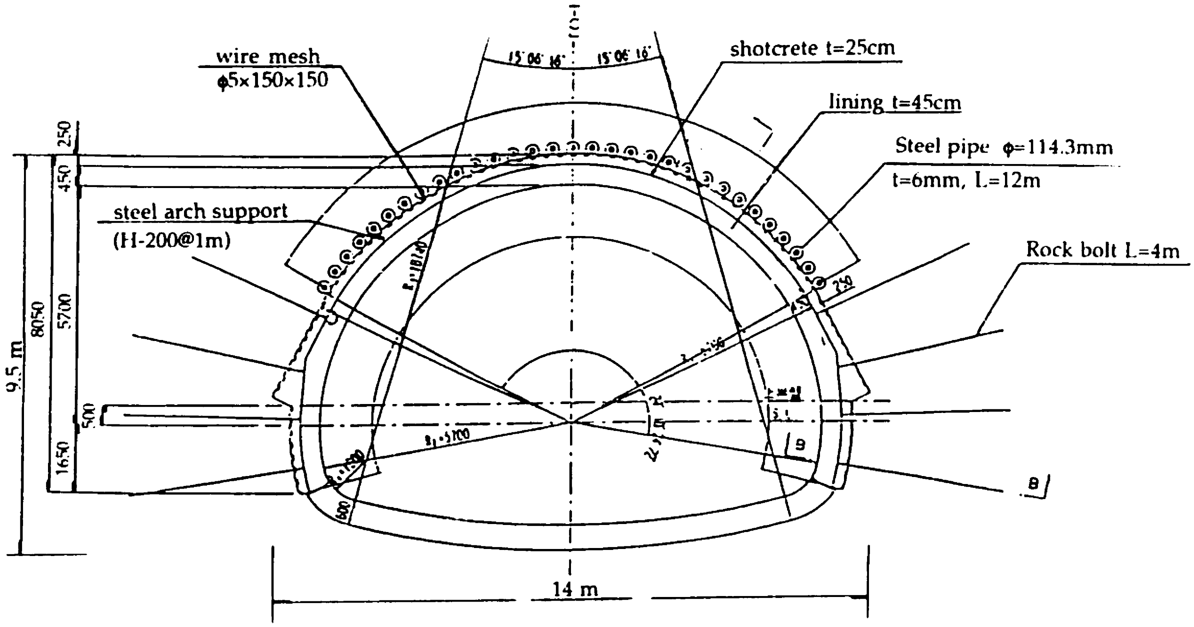
Below the zone in which the umbrella arches were overlapped (3m in length), tunnel excavation was successfully carried out. However, after the face passed the zone the face became instable. Hence, lengthening the overlap was considered.

No other information regarding the absolute settlement magnitude was available from the reference.

Reference

Geo-Fronte Research Association, Achievements of the TREVITUB Method, December, 1995.

(a) Cross section



(b) Longitudinal section

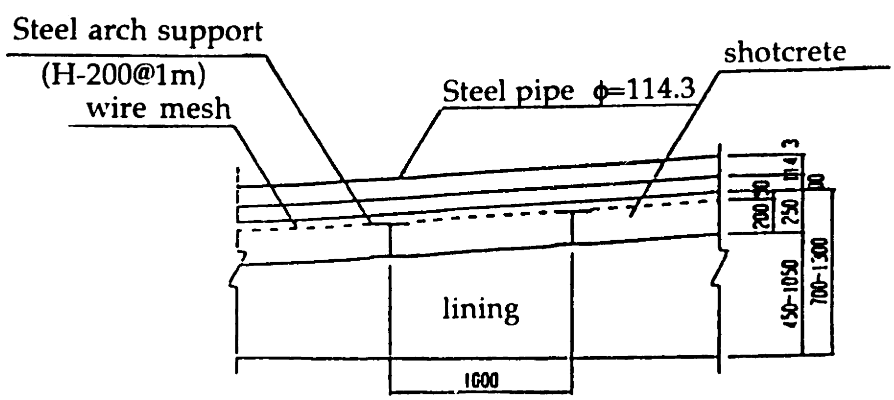


Fig. 4-21-1 Cross and longitudinal sections: Kubodaira Tunnel (Geo-Fronte Research Association, 1995)

Case 22: Venezia Station (Milan, Italy)

Environment of the Venezia Station

The Milan Urban Link Line connects the main railway stations to the underground transport system, thus creating a link between the Milan Northwest and Southeast transportation lines. Venezia Station, an underground cavern with an excavated width of 29m approximately and height of 18m, is the most important station connecting the railway line to the Line 1 of the Milan Metro (see Fig. 4-22-1). The depth of overburden above the station is about 4m.

Geological and Hydrological Conditions

The ground of Milan consists of a very thick layer of mostly quite coarse alluvial deposits (recent diluvium, Quaternary glacial epoch), with alternating levels and strata of variable thickness (from a few centimeters to a few meters). The granular composition of this soil varies from silty sand to sandy gravel. The permeability of the soil varies depending on the granular composition, with a minimum of about 10^{-4}cm/s. The density of the ground varies with the depth, from quite dense for about the first ten meters, i.e., SPT-N values are 20 to 30, to very dense at more than 30m depth.

According to these characteristics, the ground is classified as non -cohesive soil with an average internal friction angle of 35°.

Problems in Tunnel Construction

Problems in the construction of the Venezia Station were:

- unusually large dimensions of the tunnel
- a shallow overburden
- presence of foundations of old multistory buildings
- unconsolidated nature of the ground
- presence of groundwater
- need to restrict ground deformation to a few millimeters

Supplementary Support Method of the Tunnel

Initially, the construction of the Venezia Station was planned to be carried out by means of the multiple-drift method. However, an FE method simulation of the excavation showed the impossibility of obtaining, using conventional techniques, both sufficient tunnel stability (due to soil failure during excavation of the tunnel crown) and adequate limitation of the ground surface settlement (the resulting settlement was estimated to be on the order of 10^{-2} m). Two main factors contributing to this were the shallow overburden and the tunnel primary lining (shotcrete), which was extremely flexible. Thus, a supplementary method had to be considered to overcome these difficulties, that is, to achieve safe construction and minimization of ground surface settlement. The Cellular Arch Method, which was developed for the construction of the Venezia Station, was a method which satisfied the above demands.

Structural Details

The cross and longitudinal sections of the Venezia Station are shown in Fig. 4-22-2.

The dimensions of the Venezia station are: internal width: 22.8m; excavated width: 28.8m; internal height: 16.3m; and excavated height: 17.8m.

The fore-poles are 2-m-long reinforced concrete pipes 2.1m in diameter and 15cm wall thickness. The number of the fore-poles per cross section is 10.

The Cellular Arch is a composite structure similar to a semi-cylindrical section grid, in which the longitudinal elements (cells made of reinforced concrete pipes) are connected by means of large ribs (arches) placed every 6m in the longitudinal direction. Hence, the main supporting structure in the Cellular Arch system is formed by transverse arches and the pipes fixed in the ground along a semi-cylindrical profile and parallel to the longitudinal axis of the tunnel. It should be noted that in terms of design philosophy the Cellular Arch Method may differ from the other umbrella methods such as the sub-horizontal jet-grouted method, the injected steel pipe umbrella method and the pipe roof method. This is so because in the Cellular Arch method, the pipes function not only as an "umbrella" created both ahead of the tunnel face and above the tunnel crown, but also as a main supporting structure in the tunnel crown.

Construction Procedures

Construction of the arch involved the following (see Fig. 4-22-3):

<u>Steps 1 and 2</u>

- grouting, from a central service drift, of the ground around the perimeter of the side drifts.
- heading-and-benching excavation of the side drifts and completion of grouting around the perimeter of the station tunnel, followed by the pouring of concrete to form the "posts." The side drifts were 7 m high and 5 m wide and shaped to match the future tunnel cross-section. The side drifts, which were larger than many road or rail tunnels, were supported by steel ribs and shotcrete lining.
- jacking, from a thrust pit measuring 10m by 12m, of 10 microtunnels, consisting of reinforced concrete pipe segments 2.10m in diameter.

• excavation of tunnels connecting the side drifts to the 10 microtunnels. These tunnels formed the molds within which cellular arches were cast.

Step-3

• placing of steel reinforcement and pouring of concrete to form the arches and microtunnels.

• excavation of the tunnel in several stages and simultaneously finishing of the upper section.

Step-4

• mezzanine casting.

Step-5

• excavation of the bottom section.

Step-6

• excavation and casting by sections of the invert.

The execution of the 10 microtunnels was carried out by the pipe jacking method. The excavation equipment consisted of an 8-m-long steel shield machine divided into three parts. The first section was 1.30m long and movable; it was provided with a cutting edge that allowed the operator to control vertical and horizontal movements. In order to limit cavities outside the pipes, the shield machine had a computer-controlled front drilling head about 3cm smaller than the outside diameter of the shield. Grouting in the area where microtunnels would be driven was conducted to avoid decompression and instability of the ground during pipe jacking and the excavation of the connecting tunnels.

During pipe jacking, the vertical deviations did not exceed 3.0cm, the horizontal deviations did not exceed 2.5cm and no surface settlement occurred.

It should be pointed out that it is the non-cohesive nature of the soil and the lack of overburden that required the grouting and hence grouting is not always necessary for the Cellular Arch Method.

Field Measurements

Figures 4-22-4 and 4-22-5 show the locations of the measuring points and the instruments installed at these points. Figure 4-22-6 shows the settlement of the surface topographical markers located at the main measuring points.

Results of the field measurements of the ground deformation can be summarized as follows.

During grouting ("A" in Fig. 4-22-6) upward movements of 30 - 40mm were observed, then ground movements remained practically constant during cellular arch construction ("B" in

Fig. 4-22-6). The subsequent stage involving excavation of upper section and roof arch resulted in more marked settlement.

Results from the incremental extensometer located in section 3 are shown in Fig. 4-22-7. It should be noted that since the plotted data start in March 1990 the data do not include initial upward movements.

From the figure, it can be seen that large settlements occurred during construction of the upper section ("B" in Fig. 4-22-7). Also, the deeper the bases the smaller the movements.

Figure 4-22-8 shows maximum ground surface settlements in the four sections measured from the incremental extensometer. Maximum surface settlement of 14mm was observed at the right side point above the tunnel crown in section 4. Taking into account the large dimensions of the tunnel and shallow overburden above the tunnel crown, it can be said that the Cellular Arch Method restricted ground surface settlement to a great extent.

Figure 4-22-9 shows the effect of the later stages of construction (stages 4 through 6 in Fig. 4-22-3) on arch settlement. The markers were installed as shown in Fig. 4-22-4. It can be seen from the figure that maximum settlements of about 1.5mm were observed at the sides (A, C) and the crown (B) of the crown arch for the bottom section excavation. These values were practically the same for all three measuring points. Therefore, it seems that a rigid movement of the structure occurred.

References

Lunardi, P., "The Cellular Arch Method: Technical Solution for the Construction of the Milan Railway's Venezia Station," Tunnelling and Underground Space Technology, International Tunnelling Association (ITA), 1990.

Lunardi, P., "Cellular Arch Technique for Large Span Station Cavern," Tunnels and Tunnelling, Miller Freeman, November, 1991.

Lunardi, P., Colombo, A., Pizzarotti, E.M., "Performance Observations during Construction of the Large Span Milan Metro Station," International Congress "Option for Tunnelling," 1993.

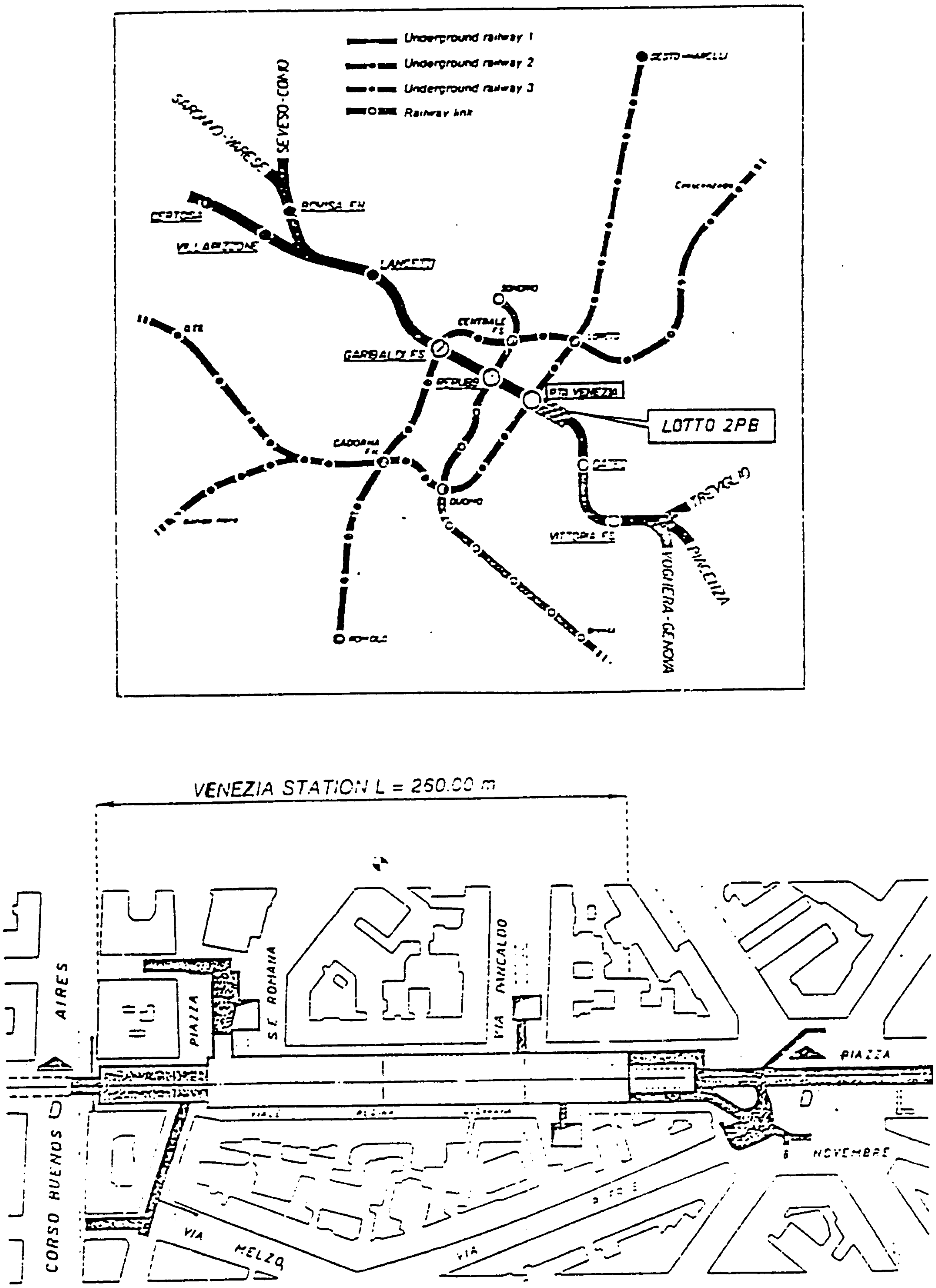
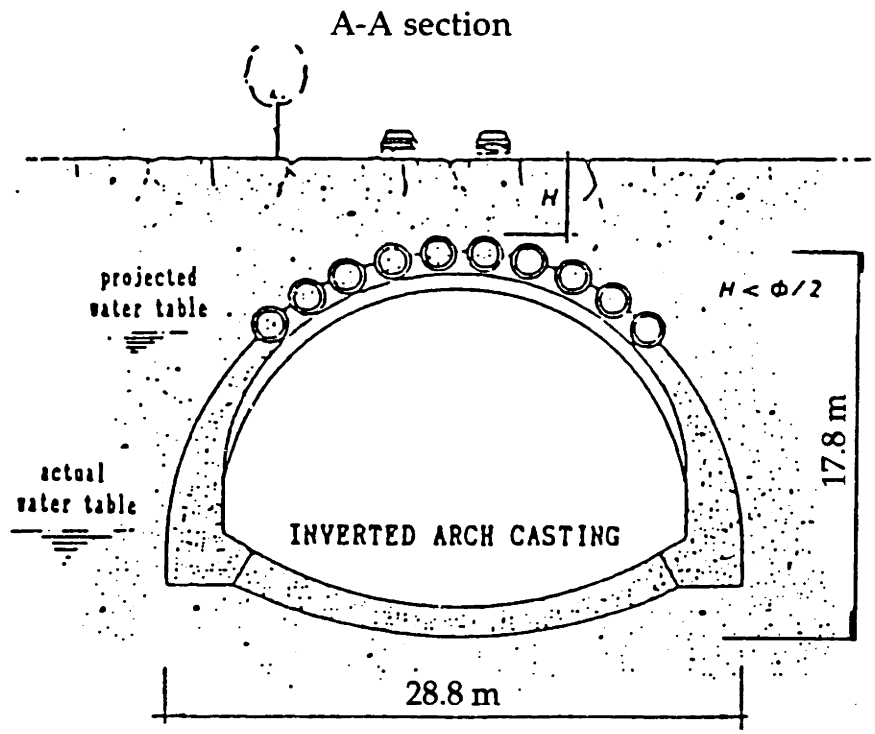


Fig. 4-22-1 Location of the Venezia Station (Lunardi, 1990)

(a) Cross section



(b) Longitudinal section

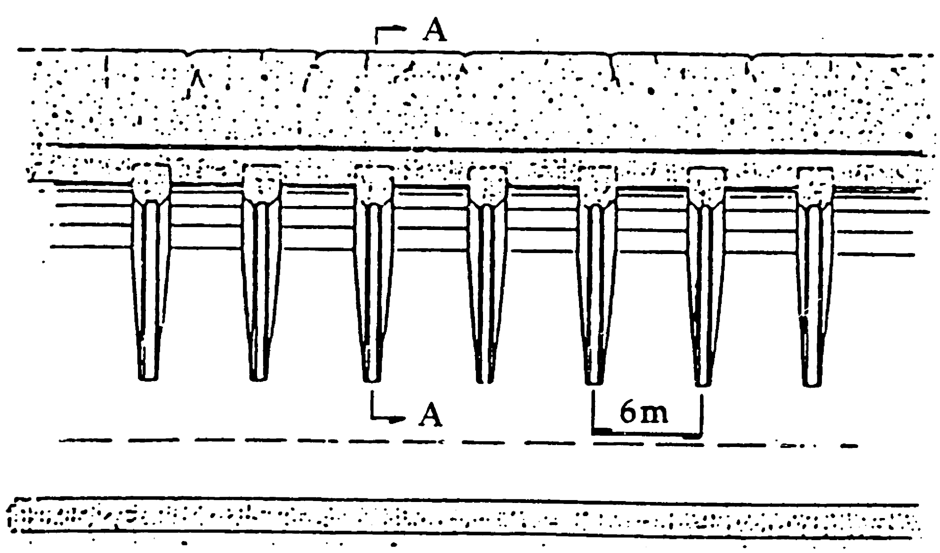


Fig. 4-22-2 Cross and longitudinal sections: Venezia Station (Lunardi, 1990)

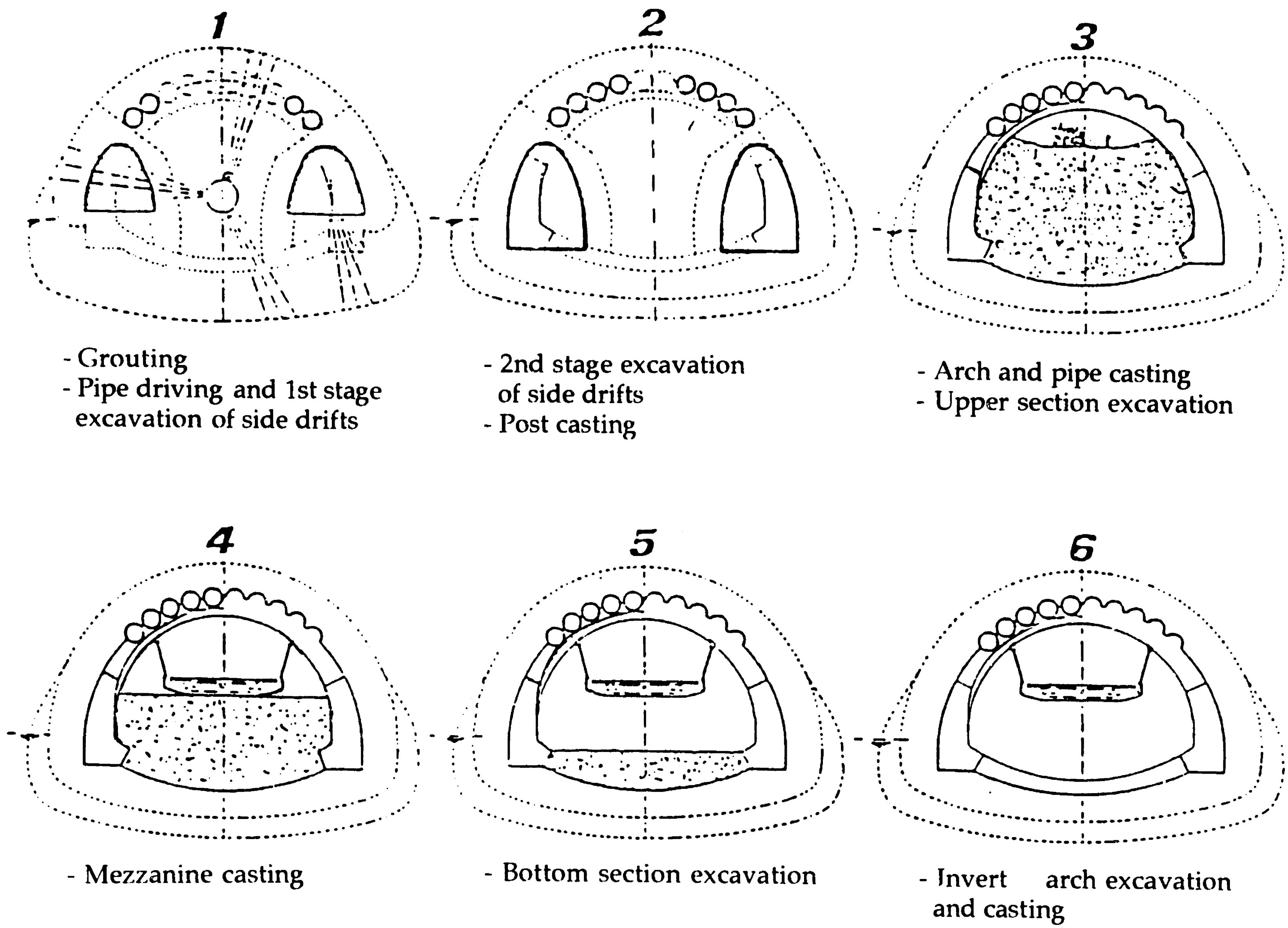


Fig. 4-22-3 Construction procedures: Venezia Station (Lunardi et al., 1992)

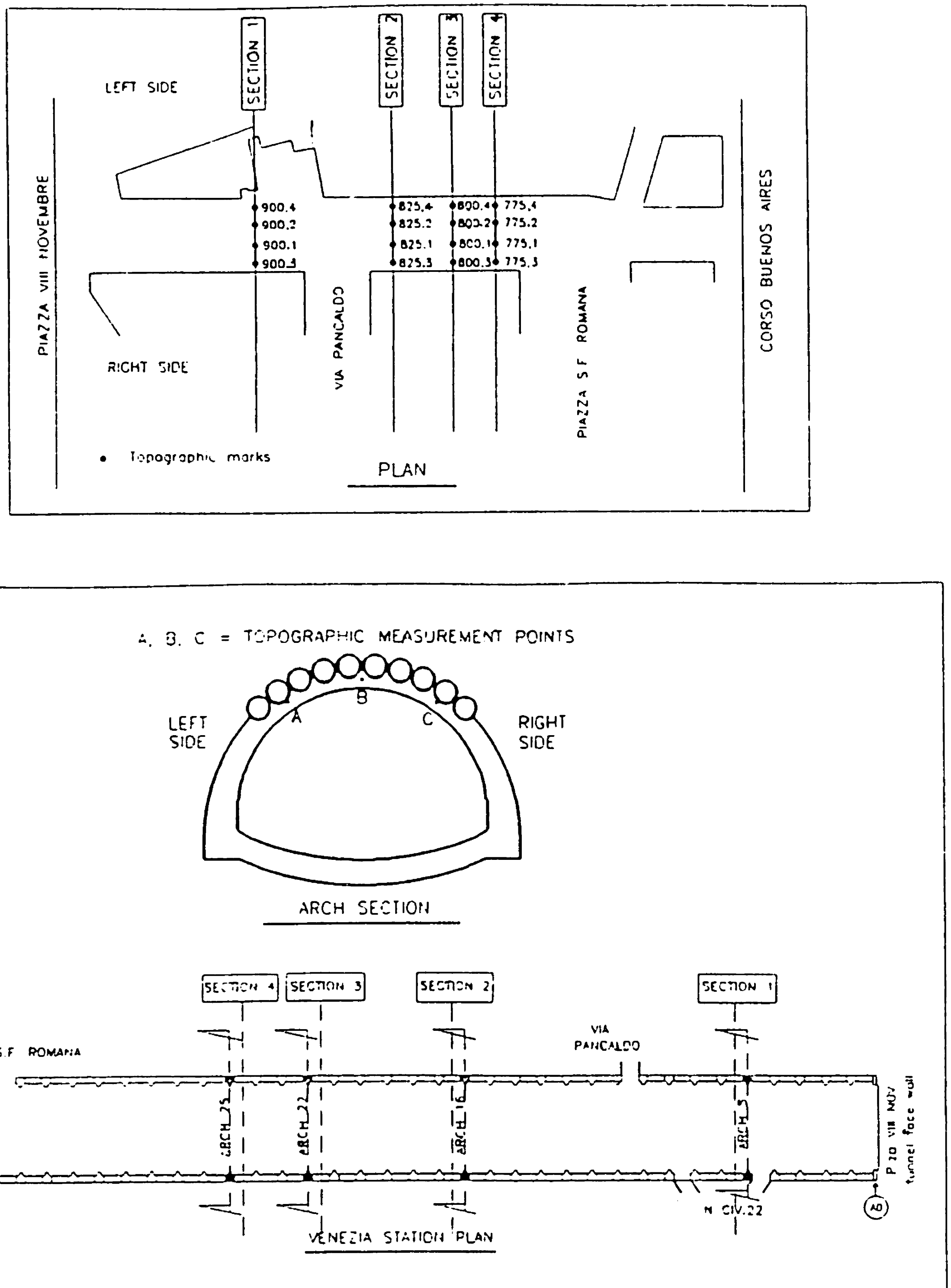
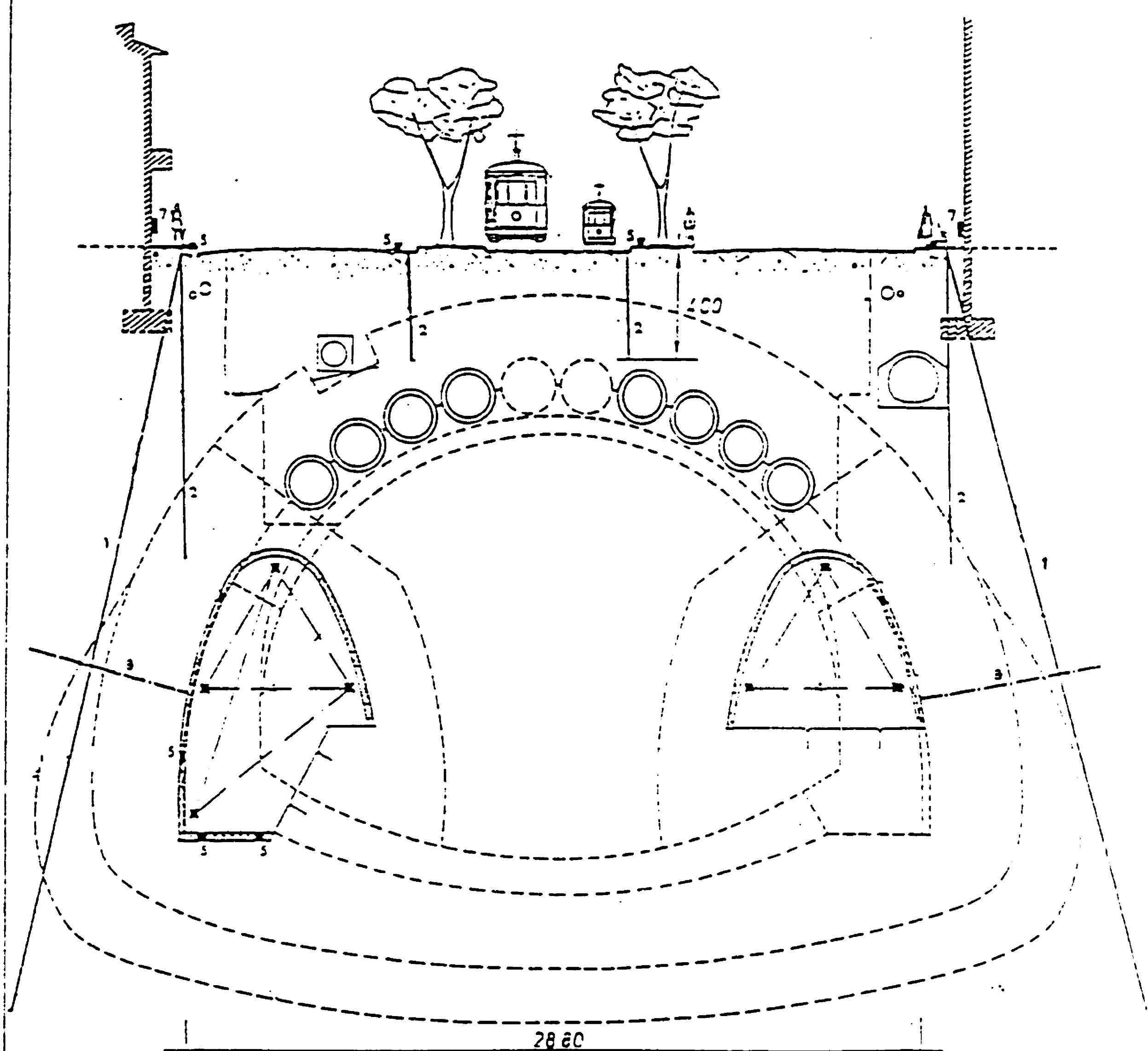


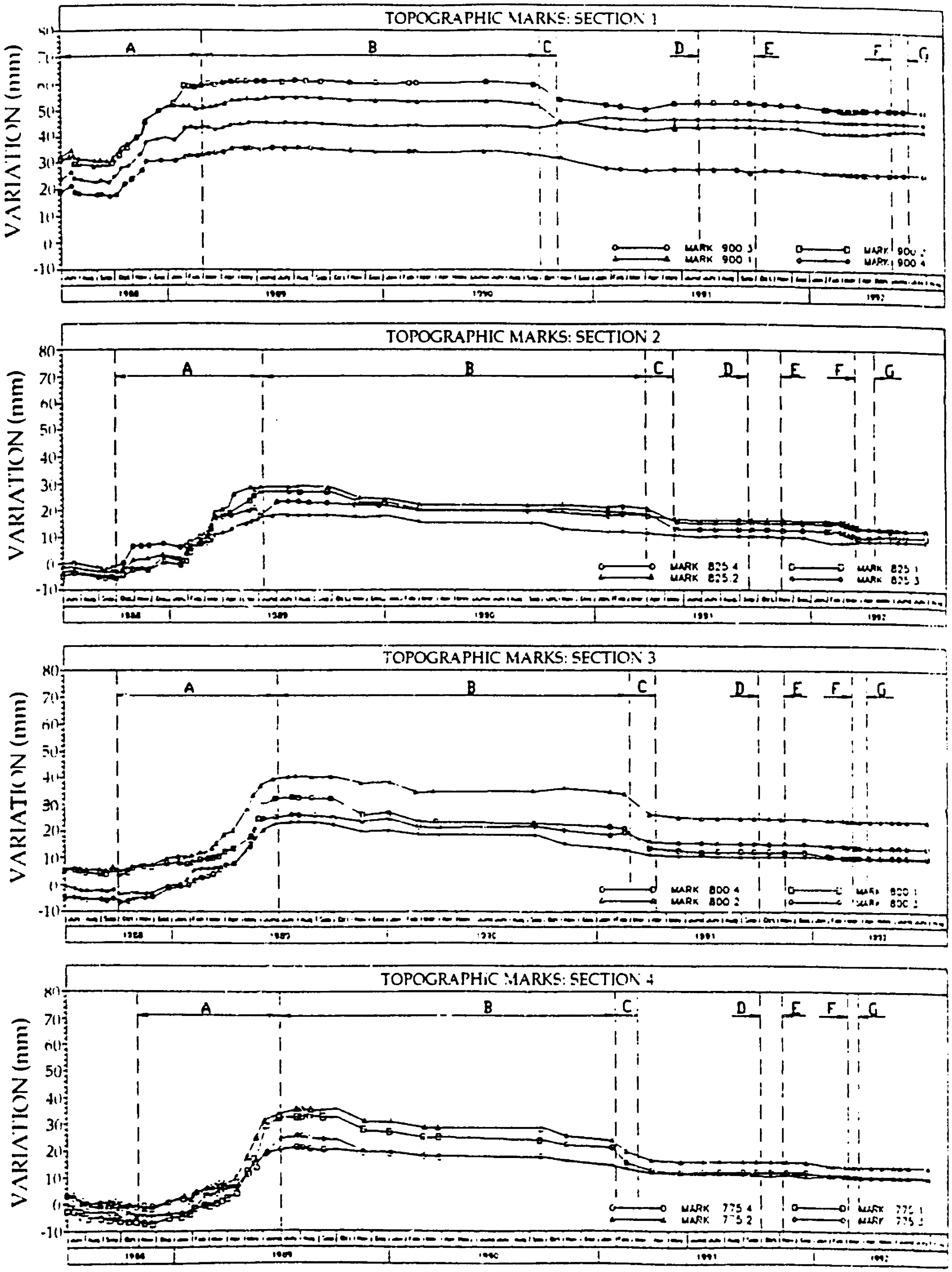
Fig. 4-22-4 Measuring points: Venezia Station (Lunardi, 1992)



PLOT KEY

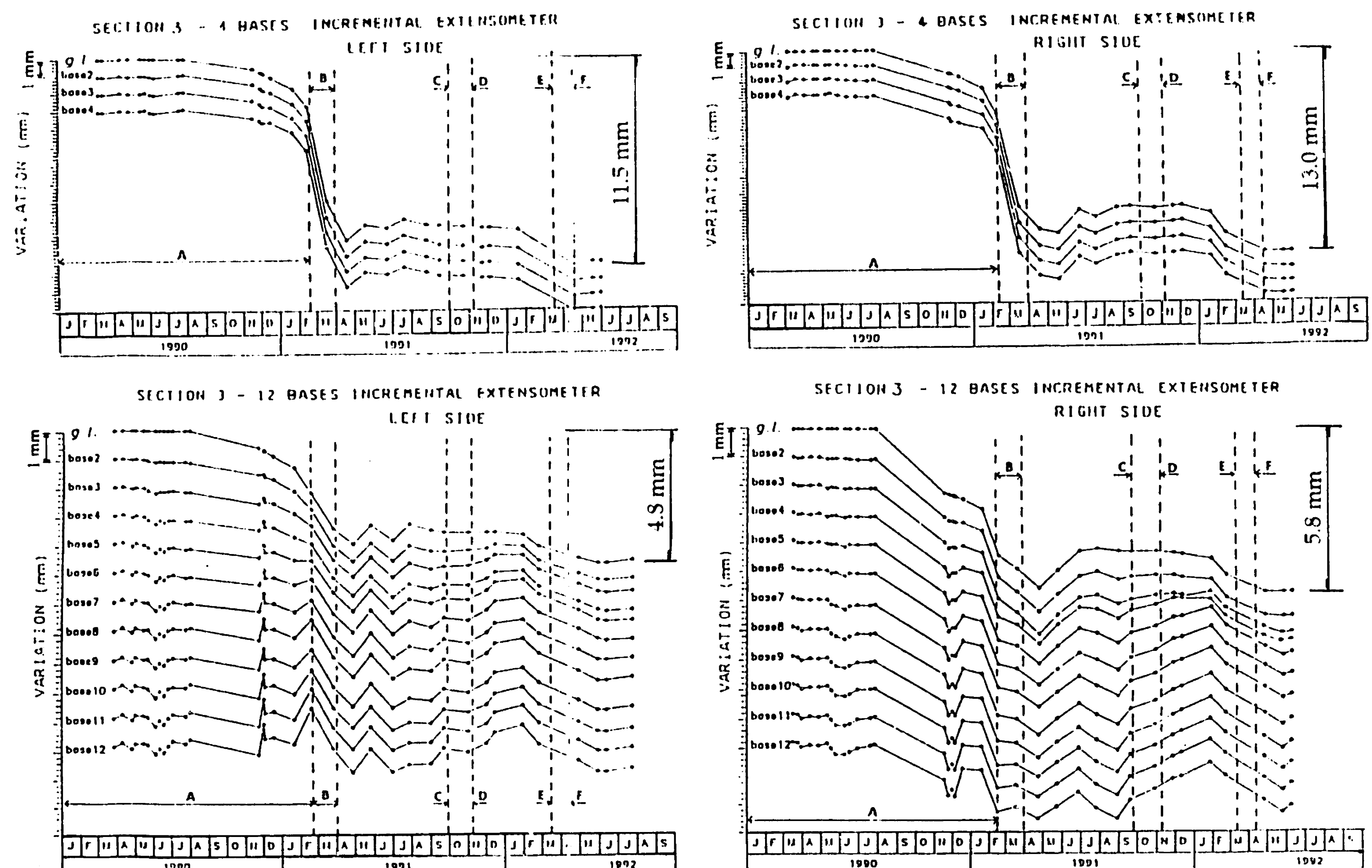
1) Inclinometer and incremental extensometer
2) Incremental extensometer
3) Multibase extensometers
4) Convergence bolts
5) Pressure cells
6) Topographic marks
7) Levellometers

Fig. 4-22-5 Field measurement program: Venezia Station (Lunardi, 1992)



A) GROUTING; B) CELLULAR ARCH CONSTRUCTION; C) UPPER SECTION EXCAVATION; D) MEZZANINE CASTING; E) BOTTOM SECTION EXCAVATION; F) INVERTED ARCH EXCAVATION; G) INVERTED ARCH CASTING

Fig. 4-22-6 Topographic marker settlement during construction: Venezia Station (Lunardi, 1992) [see Fig. 4-22-4 for location of the markers]



A) CELLULAR ARCH CONSTRUCTION; B) UPPER SECTION EXCAVATION; C) MEZZANINE CASTING; D) SECTION EXCAVATION;
E) INVERTED ARCH EXCAVATION; F) INVERTED ARCH CASTING

Fig. 4-22-7 Surface and deep settlements at section 3: Venezia Station (Lunardi et al., 1992)
[see Fig. 4-22-4 for location of section 3]

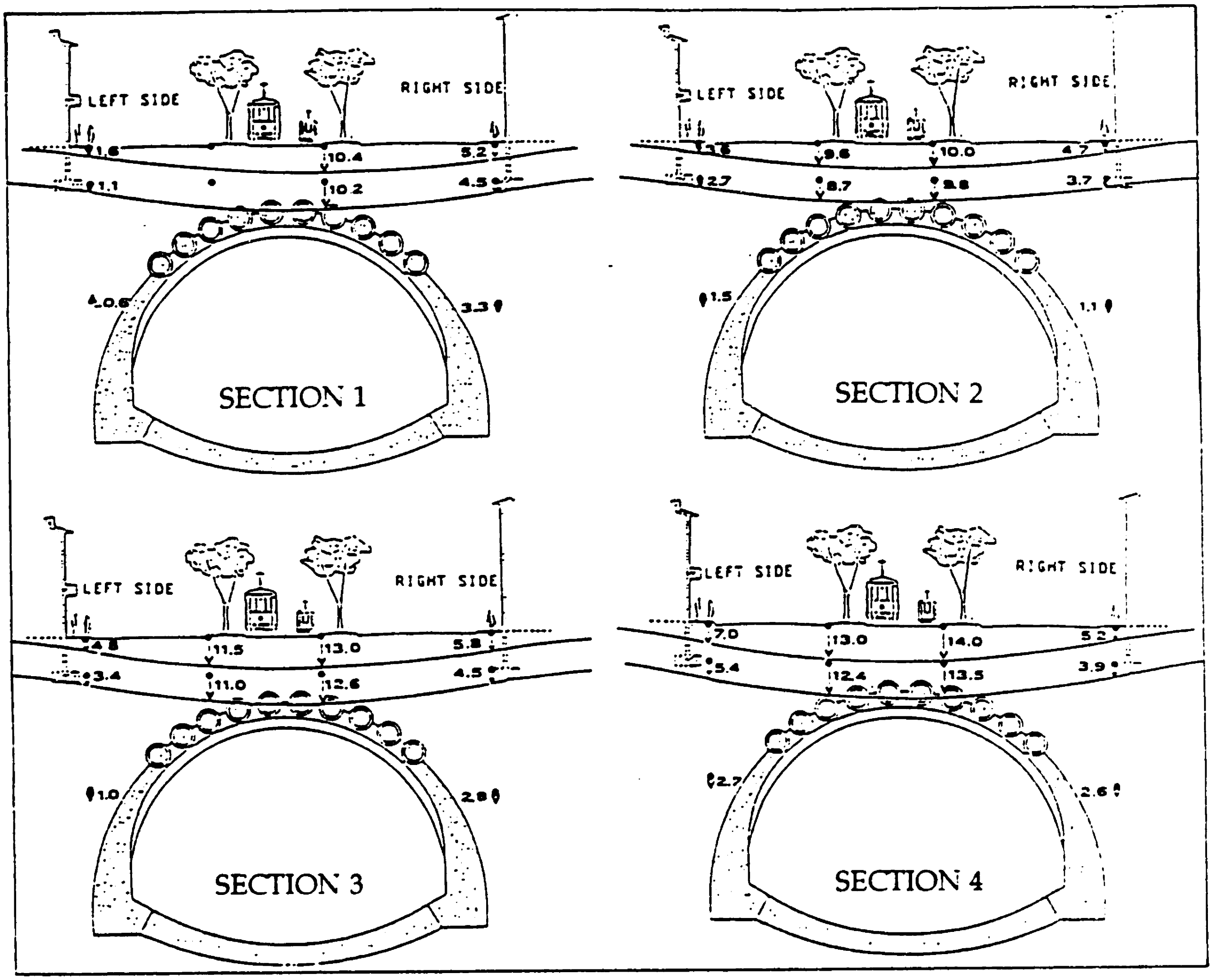


Fig. 4-22-8 Maximum settlement at the end of construction: Venezia Station (Lunardi, 1992) [see Fig. 4-22-4 for location of the sections]

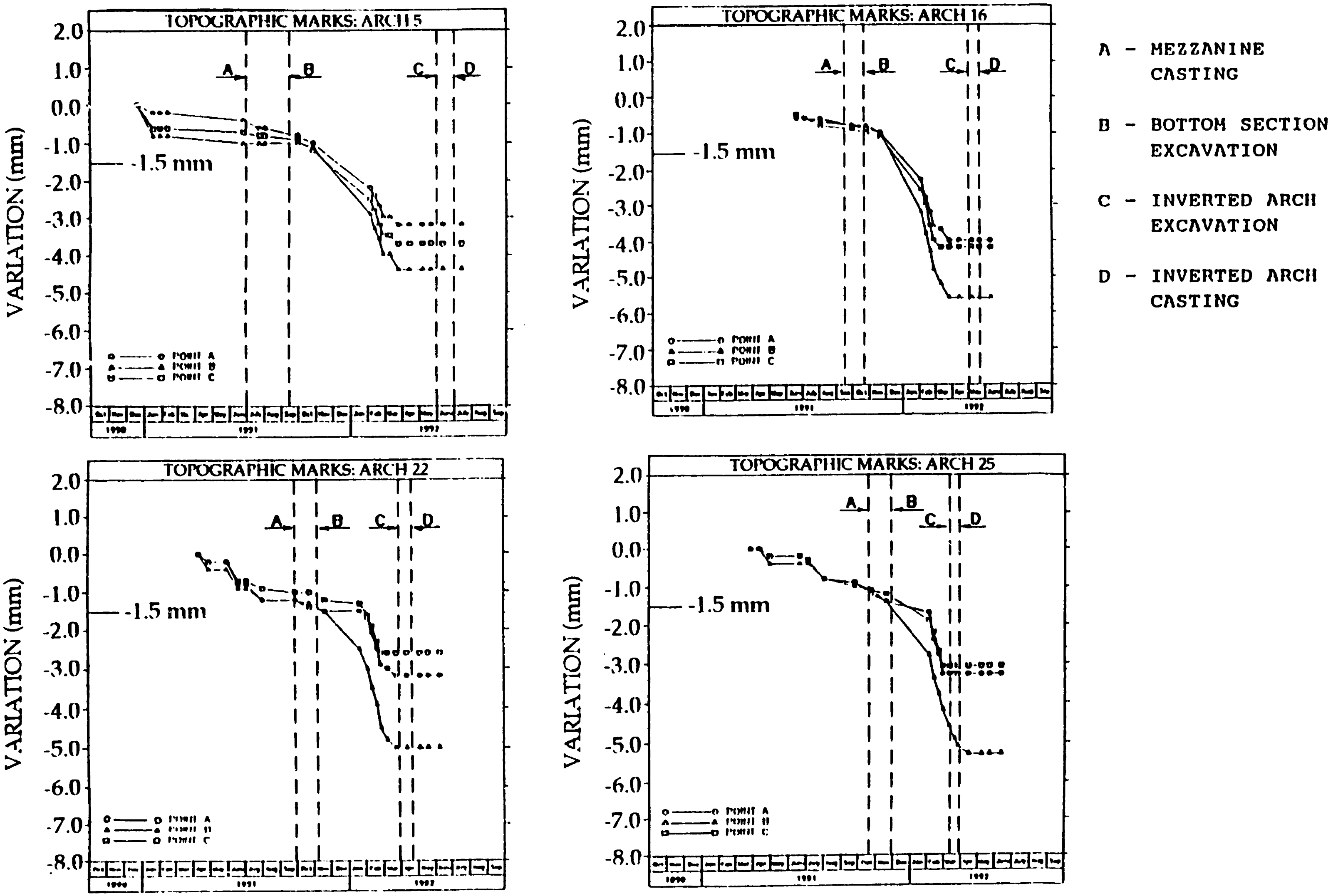


Fig. 4-22-9 Arch topographic marker settlement during construction: Venezia Station (Lunardi et al., 1992) [see Fig. 4-22-4 for location on the arch]

Case 23: MARTA East Line Underpass Tunnel (Atlanta, USA)

Environment of the MARTA East Line Underpass Tunnel

A short 55-m-long twin-tube tunnel was constructed under Interstate Highway 285 (I-285), an 8-lane highway, in Atlanta, Georgia. This underpass was required by the Atlanta Rapid Transit Authority (MARTA) to carry a 6.5-km surface extension of the existing East Line through the highway's foundation embankment (see Fig. 4-24-1).

Geological and Hydrological Conditions

The geology at the tunnel site consists of three layers from top to bottom: 1) fill mostly consisting of clayey silt with sands and gravels with random boulders ($\phi = 27°$, no cohesion); 2) residual fine sand, sandy silt ($\varphi = 29°$, no cohesion); and 3) partially weathered gneiss bedrock

The top portion of the tunnel passes through the fill layer and the middle portion passes through the residual soil. The tunnel invert passes through the partially weathered bedrock.

Much of the tunnel excavation lies above the groundwater table.

Problems in Tunnel Construction

The tunnel had to pass at a minimum distance of 1.2m under I-285. To allow excavation of the tunnel, the highway department specified that there should be no speed restrictions or obstruction of road traffic and that settlement should be minimized and be zero if at all possible.

Supplementary Support Method of the Tunnel

Five alternative tunnelling construction techniques were evaluated: 1) the pipe roof method (called the *multiple pipe arch* in the reference); 2) the sub-horizontal jet-grouting method; 3) the jacking of a precast lining; 4) ground freezing; and 5) the New Austrian Tunnelling Method (NATM).

Taking into account previous applications and availability, the pipe roof method was adopted.

Structural Details

The cross section of the tunnel is shown in Fig. 4-23-2.

21 steel pipes 54m long and 75cm in diameter were placed symmetrically about the tunnel centerline. As shown in the figure, each steel pipe has a T frame interlock. The T frame interlock prevented material infiltration from above the pipe and provided guidance for pipe installation. Steel arch supports, the principal static and dynamic load-bearing elements in

direct contact with the arch of exposed concrete-filled steel pipes, were installed at 1.2m intervals.

The lining (secondary support) is 50cm thick. Mini piles up to 21m deep and with at least a 2.5m rock socket or tie into the underlying bedrock carry the axial load imposed by steel arch supports.

Construction Procedures

The 21 steel pipes were installed prior to tunnel excavation by means of jacking (microtunnelling). For this purpose, a Herrenknecht AVN30 (600) slurry system with a rock cutting head was used. The rock cutting head was needed to bore through boulders and the hard, partially weathered rock. From the control cabin, alignment and level of the remotely controlled unit was maintained within a tolerance of 30mm and 15mm, respectively.

In addition, the excavation face was stabilized by using fifteen 60-cm-diameter horizontal jet-grouted soil cement shafts in the soil mass between the invert and the soil support pipes (not shown in Fig. 4-23-2).

Subsequent excavation of the full tunnel face was carried out by a backhoe.

Contact grouting was carried out radially through the interlocking section of the umbrella arch to fill the annulus between the soil and soil support pipes and to compensate for any ground loss or settlement.

Field Measurements

During microtunnelling, heaving of 40mm occurred because of the microtunneller's slurry pressures.

No information on other results of the field measurements such as ground surface settlement or stresses in the driven steel pipe was available from the reference.

References

Atta-alla, A.L., Iseley, D.T., "Pipe Arch Horizontal Drift Method for MARTA's Transit Extension under I-285," Proceedings of Rapid Excavation and Tunneling Conference (RETC), Vol. 1, 1991.

Wallis, S., "Micro Assistance for Macro Undertaking in Atlanta," Tunnels and Tunnelling, Miller Freeman, January, 1992.

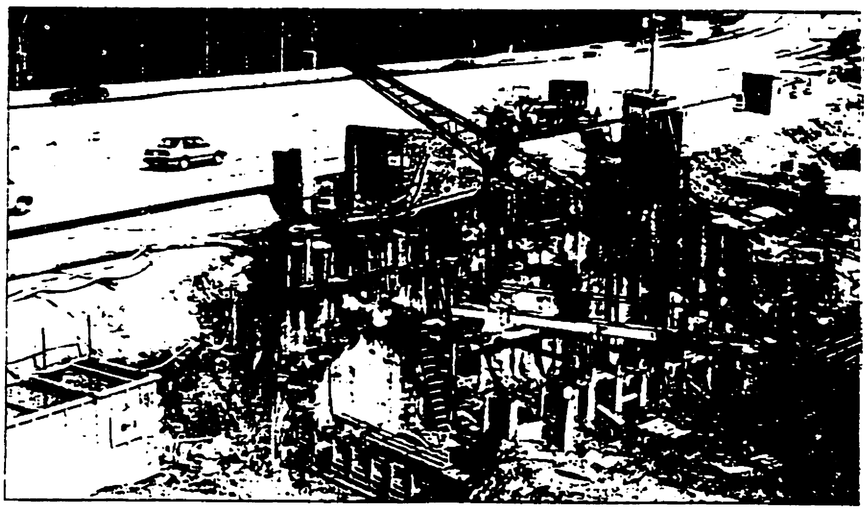


Fig. 4-23-1 Construction site of the MARTA East Line Underpass Tunnel (Wallis, 1992)

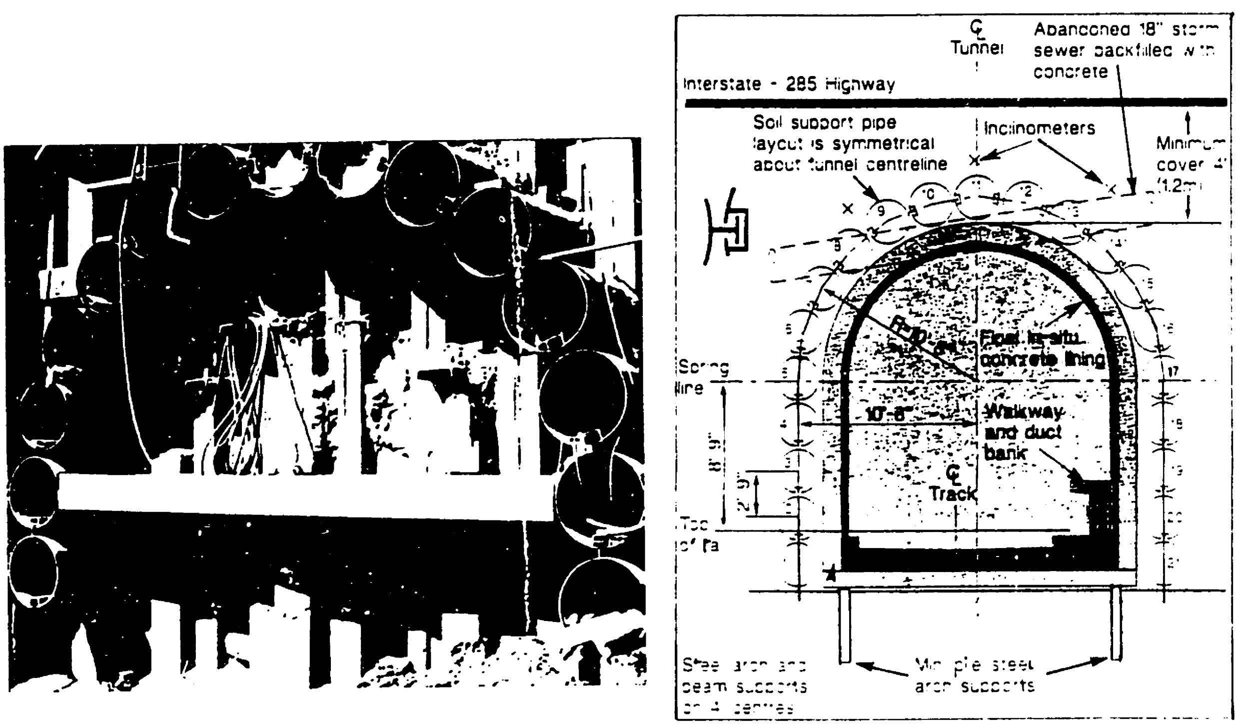


Fig. 4-23-2 Cross section: MARTA East Line Underpass Tunnel (Wallis, 1992)

Case 24: Yokohama Subway No. 3 Line (Yokohama, Japan)

Environment of the Yokohama Subway No. 3 Line

A 10.7-km extension of the Yokohama Subway No. 3 Line, connecting Shin-Yokohama Station to Azamino Station, was planned. This project included construction of an 830-m-long tunnel passing under the 6-lane No. 3 Tokyo-Yokohama Expressway with a width of 31m, which is one of the main arteries in the Greater Tokyo/Yokohama area. The excavated width and height were 10m and 8.5m, respectively and the excavated area ranged from $66m^2$ to $74m^2$ (mean $70m^2$).

Geological and Hydrological Conditions

Fig. 4-24-1 shows the geological conditions at the tunnel site.

The ground where the tunnel passes under the motorway can be roughly divided into the following strata:

- Sagami Formation, late diluvial deposits consisting of loam (L_1, L_{1v}, L_2), clay (D_{2c})
- Tsurumi Formation, middle diluvial deposits consisting of clay (D_{1c}), sand (D_{1s}) and gravel (D_{1g});
- Kazusa Formation, early diluvial deposits, alternating sand (T_s) and mudstone (T_m)

The engineeringproperties of the above strata are summarized in Table 4-24-1.

Table 4-24-1 Engineering properties: Yokohama Subway No. 3 Line

Layer	SPT-N	q_u (kPa)	ϕ (°)	C (kPa)	Modulus of deformation (MPa)
loam (L_2)	3 - 23	41 - 83	11.3 - 13.0	37 - 75	-
loam (L_1)	2 - 12	98 - 155	18.1 - 18.5	39 - 50	-
clay (D_c)	3 - 22	166 - 490	0 - 16.5	58 - 200	19.2 - 20.7
sand (D_s)	8 - 50	N/A	35.5	N/A	-
mudstone (T_m)	>50	2140 - 3400	20 - 25.3	880 - 1780	210 - 1100
sand (T_s)	>50	N/A	N/A	N/A	300 - 327

Source: Ogino et al. (1991)

Problems in Tunnel Construction

As mentioned previously, the tunnel crossed under a heavily-used motorway. Therefore, tunnel construction could not obstruct the road traffic and ground surface settlement had to be minimized.

Supplementary Support Method of the Tunnel

The pipe roof method was adopted as a supplementary support method to the NATM to minimize the effects on the motorway due to tunnel excavation.

No information on whether or not alternative methods were evaluated was available from the reference.

Structural Details

The cross section of the tunnel is shown in Fig. 4-24-2.

Fore-poles are 64.5-m-long steel pipes 67.5cm in diameter and 9mm wall thickness. The number of the fore-poles per cross section is 18. Each steel pipe consisted of six 10-m pipe sections and one 4.5-m pipe section.

Tunnel support consists of a 20-cm-thick primary lining (shotcrete) and H-200 section steel arch supports installed at 1m intervals. The secondary lining is 55cm thick.

Construction Procedures

The excavation was by heading-and-benching.

21 steel pipes were installed prior to tunnel excavation by means of the jacking and boring method (see Ch. 3.4). After jacking, the pipes were filled with concrete.

Field Measurements

Figure 4-24-3 shows the relationship between the ground surface settlement and time. The settlement was monitored at both lanes of the highway crossing above the tunnel. The depths of overburden above the tunnel crown at the two lanes were 6.7m and 7.9m, respectively.

From Fig. 4-24-3, the following conclusions can be drawn:

- Some settlement of the road surface occurred during pipe installation. After completion of jacking, the settlement gradually increased and reached up to 6 - 7mm one month after the completion. It was thought that the increment in the weight due to concrete injected into the pipes caused this settlement.
- During excavation of the top heading, additional settlement of 2 - 3 mm of the road surface occurred, while during bench excavation, little or no settlement occurred.

Reference

Ogino, Y., Watanabe, M., Hikabe, S., "Construction Report on the Yokohama Subway No. 3 Line (Kitanoyato Site)," Tunnels and Underground, Japan Tunnelling Association (JTA), December, 1991.

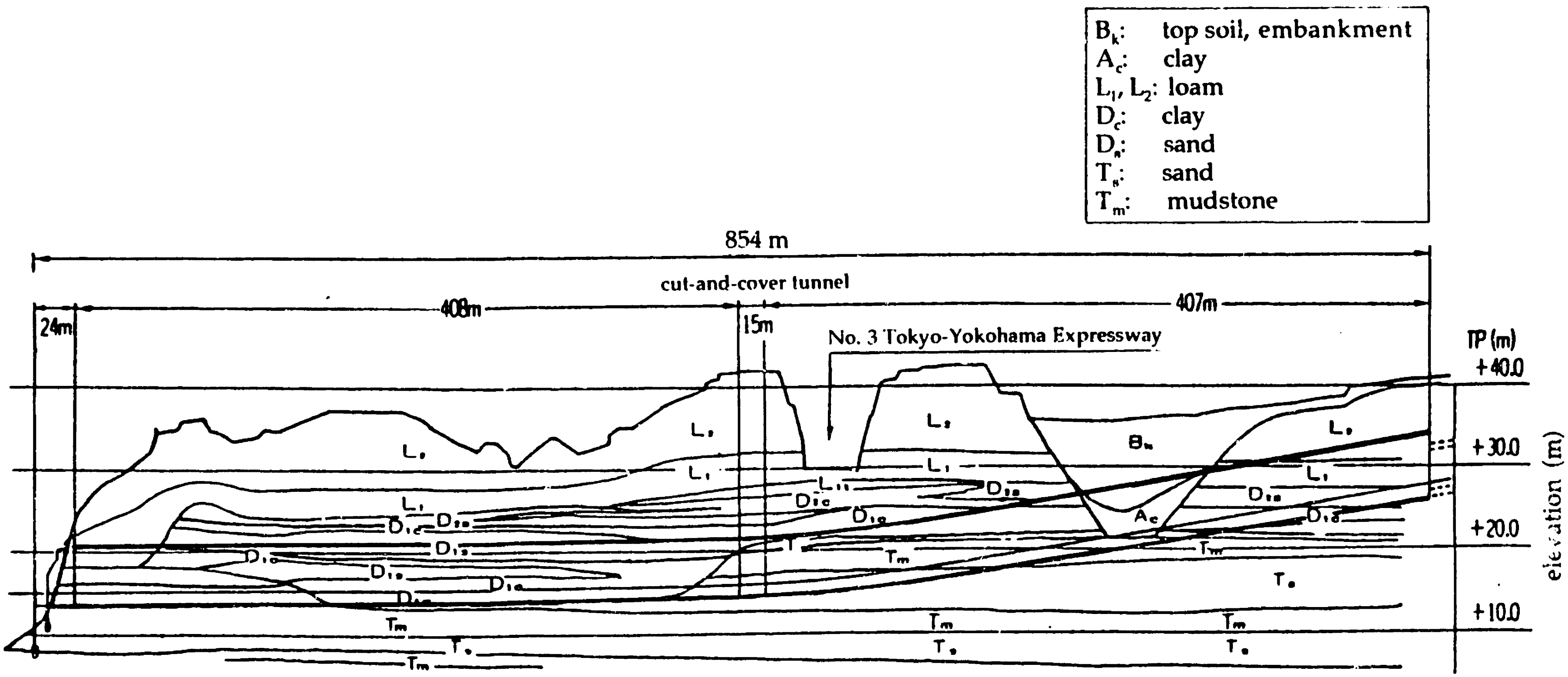


Fig. 4-24-1 Geological conditions: Yokohama Subway No. 3 Line (Ogino et al., 1991)

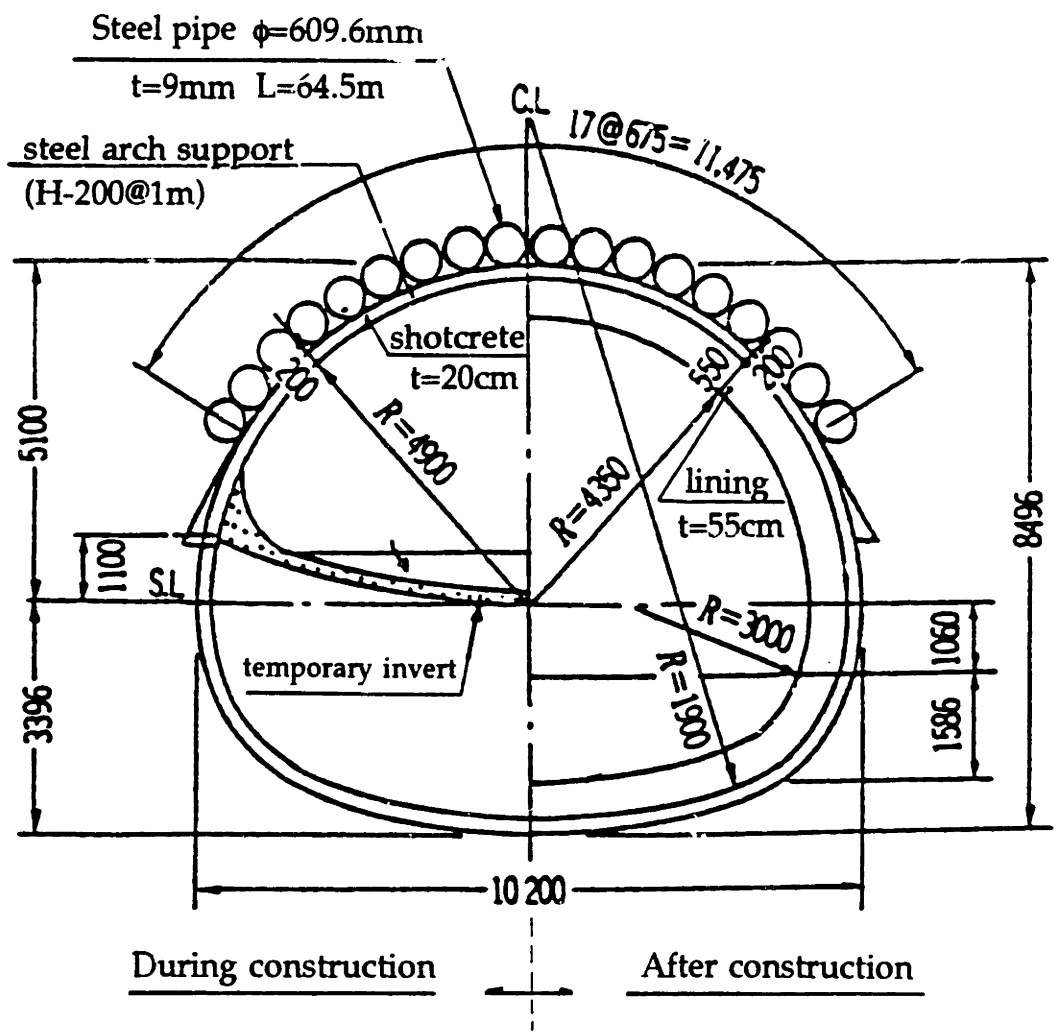


Fig. 4-24-2 Cross section: Yokohama Subway No. 3 Line (Ogino et al., 1991)

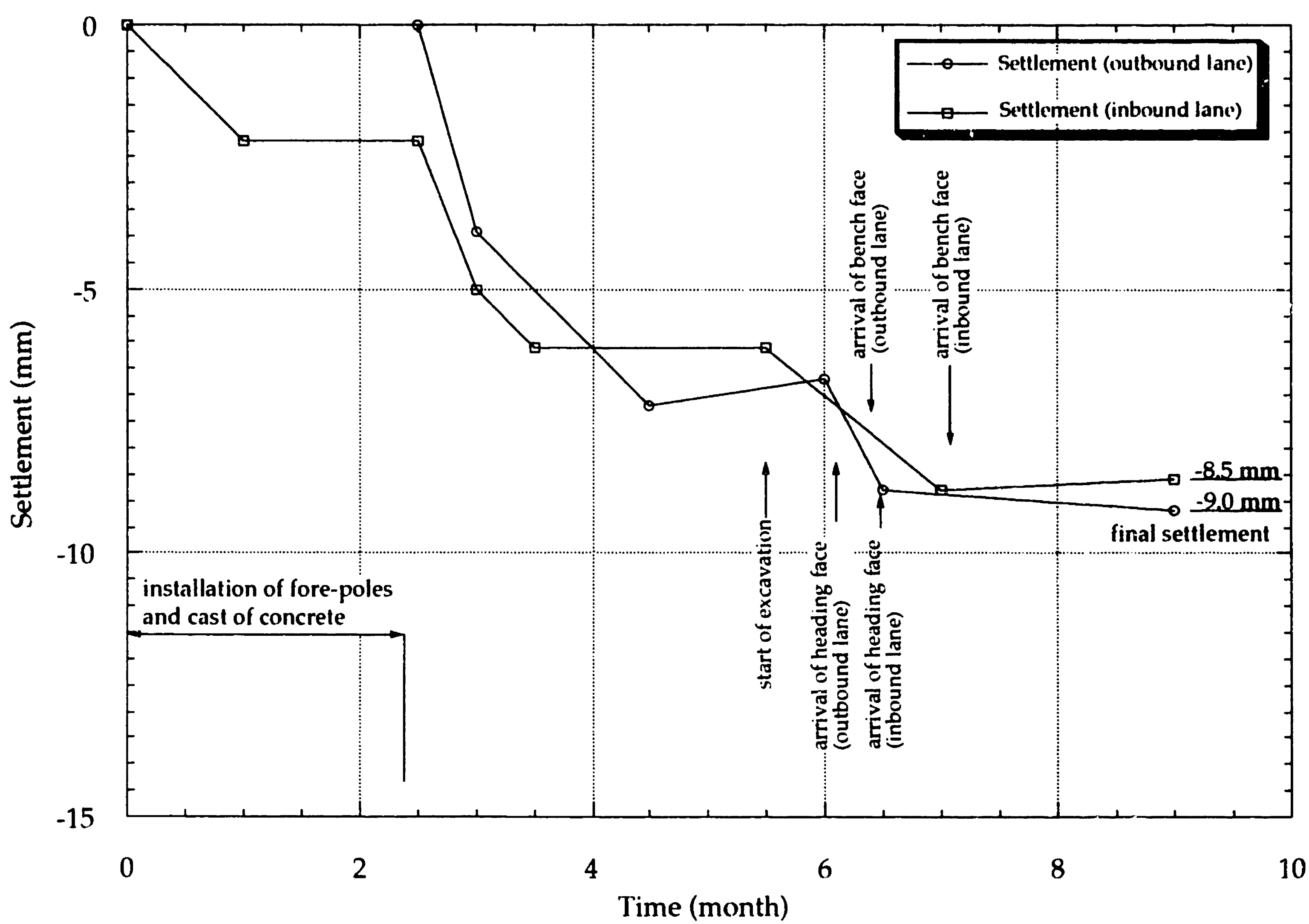


Fig. 4-24-3 Ground surface settlement versus time: Yokohama Subway No. 3 Line (Ogino et al., 1991)

Chapter 5. Discussions and Conclusions

5.1 Discussions of Results Obtained from Case Studies

5.1.1 Overview

In Chapter 4, 24 cases using the umbrella method were described.

In this chapter, the cases are further discussed with specific attention to the following aspects:

- tunnel dimensions, tunnel supports and details of the umbrella structure
- purpose of the umbrella method
- ground conditions in which the umbrella method was employed
- settlement characteristics
- tunnel behavior
- heaving and settlement during development of the umbrella arch
- analytical methods

5.1.2 Tunnel Dimensions, Tunnel Supports and Details of the Umbrella Structure

Table 5-1 summarizes tunnel dimensions, tunnel supports and details of the umbrella structure for all cases.

Figure5-1 shows the distribution of the excavated areas, which average $120m^2$. Most of the collected construction cases were for motorway tunnels with 2 or 3 lanes; as a result, dimensions of the tunnels were very similar to one another.

The following tunnel support pattern seems to be about average regardless of any differences in the tunnel dimensions, ground conditions and umbrella method used.

- primary lining: 20-to-30cm thick
- steel arch support: H-200 at 1m intervals
- secondary lining: 40-to-60cm thick

Roughly speaking, it can be seen that this is the same support pattern commonly used for tunnels in loose, uncemented ground where the umbrella method was not chosen (Sakurai et al., 1988). From this, one can conclude that when the umbrella method was employed, it was used as a supplementary method to improve face stability or to minimize ground surface settlement, not as a load-bearing structure to reduce dimensions of the tunnel supports.

Table 5-1 Summary of tunnel dimensions, tunnel supports and details of the umbrella structure

Case No.	Name of tunnel/project	Tunnel dimensions			Tunnel supports			Umbrella structure			
		Height (m)	Width (m)	Exc. Area (m²)	Thickness of primary liming (cm)	Steel arch support	Thickness of secondary lining (cm)	Type of umbrella method	Dia. of fore-pole (cm)	Length (m)	No. of fore-poles per cross section
1	Hodogaya	10.9	16.8	150	20	H-200@1m	60	Sub-horizontal	60	10	31
2	Syoryou No. 1	11	16	145	25	H-200@1m	45	Sub-horizontal	60	10	23~44
3	Kokubugawa	8.5	8.6	60	20	H-125@1m	N/A	Sub-horizontal	60	7	34
4	Owani	10.1	16.2	129	25	H-200@1m	50	Sub-horizontal	60	7	36
5	Aziro	8.8	11.5	90	25	H-200@1m	35	Sub-horizontal	60	10	36
6	Uryuya	9.5	15	120	25	N/A	45	Sub-horizontal	60	10	29
7	Subway Vienna	9	11	80	30	N/A	N/A	Sub-horizontal	80	10 ~ 15	26
8	Rengershausen	13	15	150	40	N/A	N/A	Sub-horizontal	60	10.5	28
9	Campiolo	10.5	12.2	100	N/A	H-200@1m	N/A	Sub-horizontal	N/A	13	41
10	Lonato	10	13.6	100~120	25	N/A	60~130	Reinforced sub	70	12	25
11	Kaziwara No. 1	10	14.2	110	25	H-200@1m	45	Reinforced sub	60	10	39
12-a	Les Cretes	10.4	12.5	105	20	double H-160@N/A	50	Reinforced sub	60	12	45
12-b								Injected steel	11.43	12	35~60
13-a	Maiko	11	16	150	25	H-250@1m	70	Injected steel	11.43	12	37
13-b						H-200@1m	45	Injected steel	11.43	12	42
14	Hirai	10	15.5	130	25	H-200@0.75m	45	Injected steel	6.05	3	53
15	Futatsui-Nishi	9	12	90	25	H-200@1m	35	Injected steel	10.16	8.5,11.5	45
16	Yakiyama	9.5	12	100	25	H-200@N/A	35	Injected steel	11.43	15, 18	33
17	Ramat	9.6	12.4	90	30	I-180@1m	N/A	Injected steel	8.89	12	33~45
18	Poggio Fornello	9.5	11	104	30	double T-200@0.75m	N/A	Injected steel	N/A	12	32
19	St. Ambrogio	10.5	14	110	25~30	double H-180/200@N/A	60	Injected steel	N/A	12~16	24
20	Nango	10	15	110	25	H-200@1m	40	Injected steel	11.43	12	43
21	Kubodaira	9.5	13	105	25	H-200@1m	45	Injected steel	11.43	12	31
22	Venezia Station	17.8	28.8	440	-	-	-	Pipe roof	210	214.5*	10
23	MARTA East Line	7.2	6.3	35	None	N/A@1.2m	50.8	Pipe roof	75	54*	21
24	Yokohama Subway No. 3 Line	8.5	10.2	66~71	20	H-200@1m	55	Pipe roof	60.96	64.5*	18

Notes: 1) * : total length of the umbrella arch.

2) Sub-horizontal: sub-horizontal jet-grouting method, Reinforced sub: Reinforced sub-horizontal jet-grouting method,

Injected steel: Injected steel pipe umbrella method, Pipe roof: Pipe roof method

Dimensions of the umbrella structures vary with the type of umbrella method used. In particular, the cases in which the pipe roof method was employed are significantly different from the other cases, in terms of diameter and length of the fore-poles. However, in both the sub-horizontal jet-grouting method and the injected steel pipe umbrella method, the lengths of the umbrella arches are, in general, 10 - 12m long. Also, it seems that jet-grouted columns of 60cm in diameter and steel pipes of 114.3mm in diameter are commonly used in the sub-horizontal jet-grouting method and in the injected steel pipe umbrella method, respectively.

5.1.3 Purpose of the Umbrella Method

From the results of the 24 case studies, one can conclude that the main reasons for adopting the umbrella method are:

- to prevent slope failures and/or landslides
- to restrict ground surface settlement
- to increase face stability

Table 5-2 and Fig. 5-2 summarize the reasons for adopting the umbrella method. From Table 5-2 and Fig. 5-2, it becomes evident that increasing face stability is the most common reason for using the umbrella method (50% of the time), followed by the restricting ground surface settlement (36% of the time), and preventing slope failures and/or landslides (14% of the time).

Figure 5-3 shows the reason for adopting a particular type of umbrella method. From the figure, the sub-horizontal jet-grouting method and the injected steel pipe umbrella method were both used for all of the reasons given above. Since it was reported in all cases that the desired results could be obtained with the umbrella method, it seems that there is no significant difference in the effects on the three problems between these two methods. The pipe roof method, on the other hand, was mainly used for the restriction of ground surface settlement. This implies that because of its flexural rigidity the pipe roof method has an advantage in restricting ground surface settlement compared to the other methods.

The reason depends to a large extent on the location in which the umbrella method was employed. As shown in Table 5-2, all cases can be grouped into the following categories by location, i.e., in urban areas; in non-urban areas; and near tunnel portals and/or under slopes. From Table 5-2, it can be seen that at each location the following problems in tunnel construction are likely to occur:

- in urban areas: ground surface settlement, face instability
- in non-urban areas: face instability
- near tunnel portals and/or under slopes: slope failures and/or landslides, face instability

For comparison, the purposes and effects of the umbrella method recommended by the Geo-Fronte Research Association (1995) are shown in Table 5-3. Although the table shows that each type of umbrella method is especially effective for particular purposes, the case studies considered in this thesis did not clearly confirm this.

Table 5-2 Main reasons for adopting the umbrella method and classification of cases by location

Case No.	Name of tunnel/project	Type of umbrella method	Main reasons for adopting the umbrella method: Prevention of slope failures/landslides	Restriction of ground surface settlement	Increase in face stability	Location where the tunnel was constructed: In urban areas	In non-urban areas	Near tunnel portals/under slopes
1	Hodogaya	Sub-horizontal jet-grouting		○	○	○		
2	Shoryou No. 1	Sub-horizontal jet-grouting			○		○	
3	Kokubugawa	Sub-horizontal jet-grouting	○		○	○		
4	Owani	Sub-horizontal jet-grouting		○	○		○	
5	Aziro	Sub-horizontal jet-grouting		○		○		
6	Uryuya	Sub-horizontal jet-grouting	○		○			○
7	Subway Vienna	Sub-horizontal jet-grouting		○	○	○		
8	Rengershausen	Sub-horizontal jet-grouting			○		○	
9	Campiolo	Sub-horizontal jet-grouting	unkown					
10	Lonato	Reinforced sub-horizontal jet-grouting			○		○	
11	Kaziwara No. 1	Reinforced sub-horizontal jet-grouting	○		○			○
12-a	Les Cretes	Reinforced sub-horizontal jet-grouting		○	○	○		
12-b	Les Cretes	Injected steel pipe umbrella		○	○	○		
13	Maiko	Injected steel pipe umbrella		○	○	○		
14	Hirai	Injected steel pipe umbrella			○	○		
15	Futatsui-Nishi	Injected steel pipe umbrella	○					○
16	Yakiyama	Injected steel pipe umbrella	○		○			○
17	Ramat	Injected steel pipe umbrella		○		○		
18	Poggio Fornello	Injected steel pipe umbrella			○		○	
19	St. Ambrogio	Injected steel pipe umbrella			○		○	
20	Nango	Injected steel pipe umbrella		○	○	○		
21	Kubodaira	Injected steel pipe umbrella		○		○		
22	Venezia Station	Pipe roof (Cellular Arch)		○	○	○		
23	MARTA East Line	Pipe roof		○		○		
24	Yokohama Subway No. 3 Line	Pipe roof		○		○		

Table. 5-3 Purposes and effects of the umbrella method

Type of umbrella method	Purposes and effects								
	Stabilization of face			Reinforcement of surrounding ground					
	Crown stability	Face stability	Base stability	Restriction of groundsurface settlement	Prevention of landslide	Prevention of slope failure	Stability of tunnel portal	Countermeasure against uneven pressure	Protection of adjacent structures
RJFP	◎	◎	○	◎	◎	○	◎	○	○
TREVITUB	◎	○	—	○	○	○	◎	—	○
AGF	◎	○	—	○	○	◎	◎	—	○
Pipe roof	◎	○	—	○	◎	○	◎	○	◎

Notes: 1) RJFP (Sub-horizontal jet-grouting method), TREVITUB, AGF(Injected steel pipe umbrella method)
2) ◎:Especially effective, ○:Effective

Source: Geo-Fronte Research Association (1995)

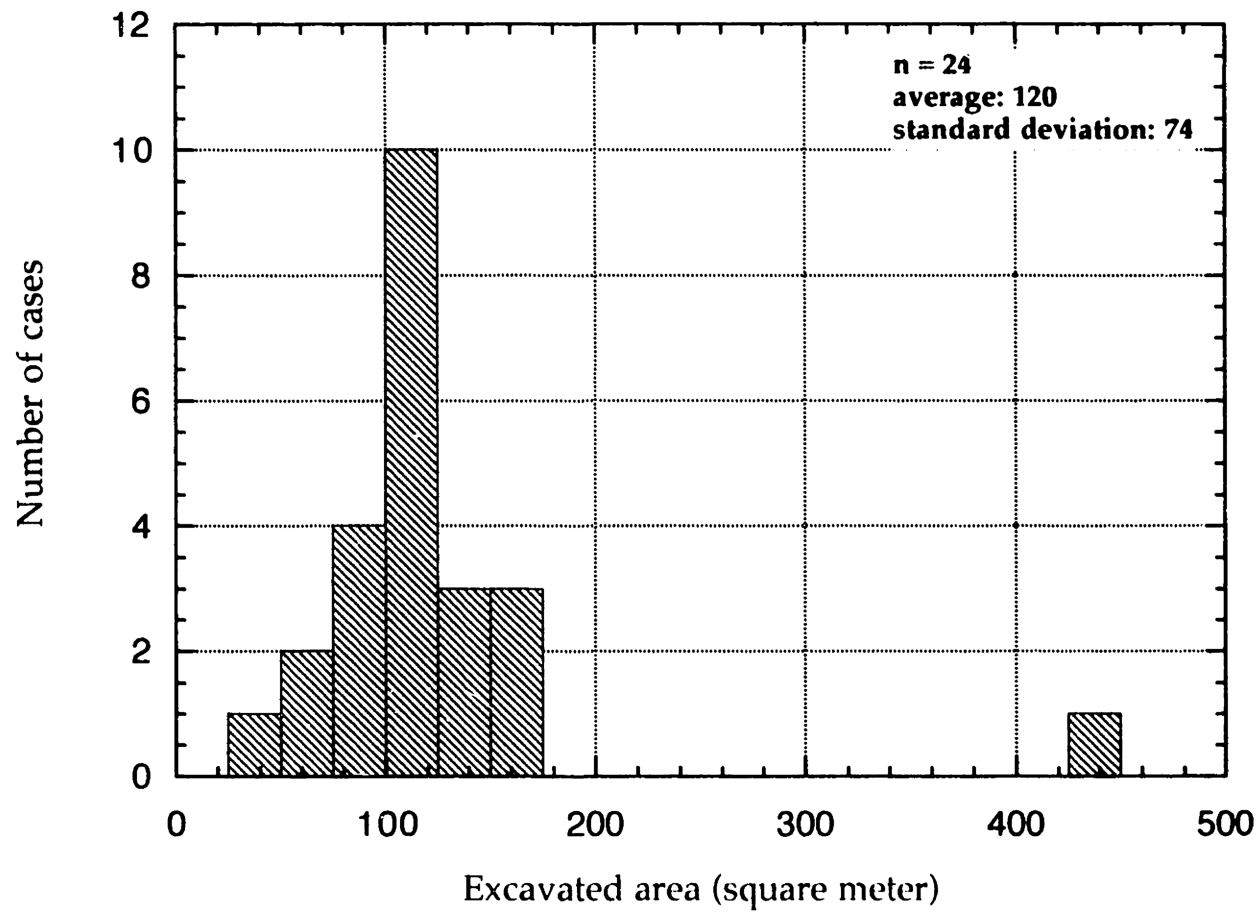
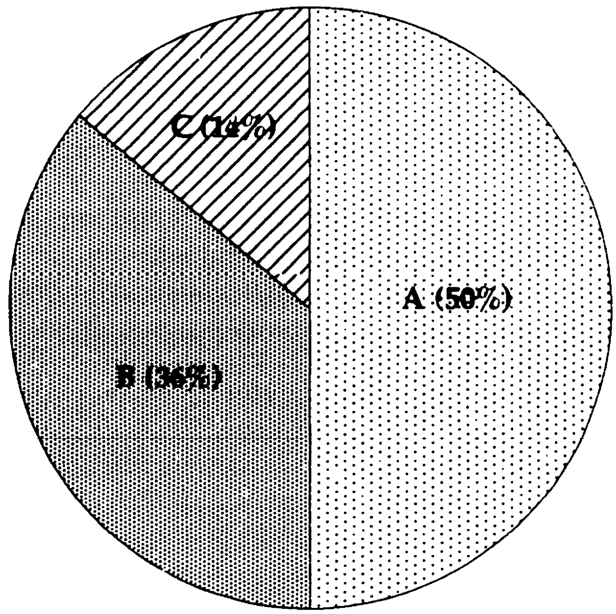


Fig. 5-1 Distribution of excavated areas



Note:
A: Increase in face stability
B: Restriction of ground surface settlement
C: Prevention of slope failures/landslides

Fig. 5-2 Reasons for adopting the umbrella method

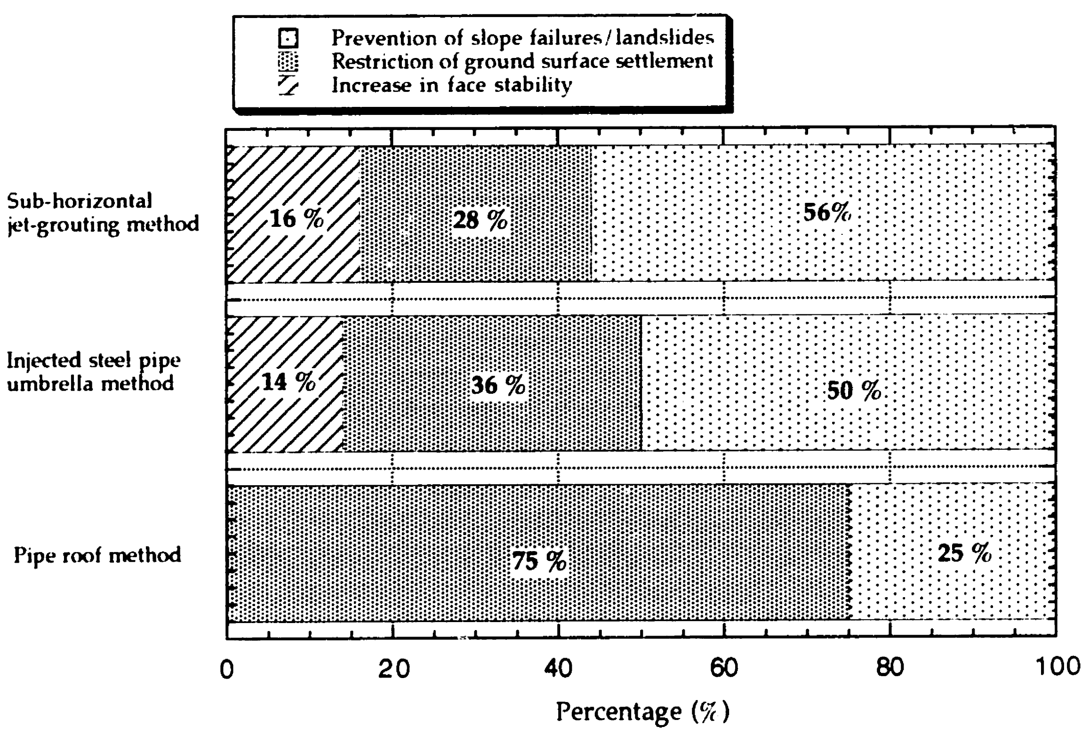


Fig. 5-3 Reasons for adopting a particular type of umbrella method

5.1.4 Ground Conditions in which the Umbrella Method was Employed

The following ground types existed in cases where the umbrella method was employed:

- sand
- clay
- gravel
- talus, moraine, detritus
- weathered rock

It should be noted that the ground mentioned here is ground in which the umbrella arch was placed, not necessarily through which the entire tunnel passes (A tunnel can pass through several different layers).

Table 5-4 summarizes the ground types for each case. Also, Fig. 5-4 shows the percentage of different ground types associated with each umbrella method. Note that in the Les Cretes Tunnel (case 12) two different types of umbrella methods were used; hence each application was counted separately. Also note that where more than one ground type for the same case existed, each ground type was separately counted as one application of the umbrella method.

Ground types in which the umbrella method was employed are discussed below according to the type of umbrella method used.

<u>Sub-horizontal jet-grouting method</u>

From the figure, it can be seen that 29% of the applications in which the sub-horizontal jet-grouting method was employed were carried out in each sand and clay, followed by gravel (25%), talus, detritus, moraine (13%) and weathered rock (4%). From the results, it can also be seen that the sub-horizontal jet-grouting method was usually employed in "soil." As mentioned previously, the Geo-Fronte Research Association (1994) suggests that this method is best suited to the relatively soft ground. The results confirm their suggestion.

It should be pointed out that two of three reinforced sub-horizontal jet-grouting cases, i.e., the Lonato Tunnel (case 10) and the Kaziwara No. 1 Tunnel (case 11), were both employed in talus, detrital or morainic deposits. The steel pipes were probably inserted into the jet-grouted columns to reinforce them because gravels and boulders included in talus, detritus, or moraine deposits made it impossible to develop a necessary diameter of jet-grouted columns.

In the Uryuya Tunnel (case 6), the sub-horizontal jet-grouting method was employed in weathered rock ground. Jet grouting is characterized by hydraulic cutting and in-situ mixing of soil with a fluid grout injected at high velocity. Because of this characteristic, weathered rock is, in general, not suitable ground for jet grouting, since it is usually too hard to cut.

Table 5-4 Ground types in which the umbrella method was employed

Case No.	Name of tunnel/ project	Type of umbrella method	Ground types				
			sand	clay	gravel	talus, dertitus, moraine	weathered rock
1	Hodogaya	Sub-horizontal jet-grouting	○	○	○		
2	Shoryou No. 1	Sub-horizontal jet-grouting	○	○	○		
3	Kokubugawa	Sub-horizontal jet-grouting	○				
4	Owani	Sub-horizontal jet-grouting		○	○	○	
5	Aziro	Sub-horizontal jet-grouting		○	○		
6	Uryuya	Sub-horizontal jet-grouting		○			○
7	Subway Vienna	Sub-horizontal jet-grouting	○	○	○		
8	Rengershausen	Sub-horizontal jet-grouting	○	○	○		
9	Campiolo	Sub-horizontal jet-grouting	○				
10	Lonato	Reinforced sub-horizontal jet-grouting				○	
11	Kaziwara No. 1	Reinforced sub-horizontal jet-grouting				○	
12-a	Les Cretes	Reinforced sub-horizontal jet-grouting	○				
12-b		Injected steel pipe umbrella		○		○	
13	Maiko	Injected steel pipe umbrella		○	○		
14	Hirai	Injected steel pipe umbrella		○	○	○	
15	Futatsui-Nishi	Injected steel pipe umbrella				○	○
16	Yakiyama	Injected steel pipe umbrella				○	○
17	Ramat	Injected steel pipe umbrella				○	
18	Poggio Fornello	Injected steel pipe umbrella					○
19	St. Ambrogio	Injected steel pipe umbrella					○
20	Nango	Injected steel pipe umbrella		○		○	
21	Kubodaira	Injected steel pipe umbrella				○	
22	Venezia Station	Pipe roof (Cellular Arch)	○				
23	MARTA East Line	Pipe roof		○			
24	Yokohama Subway No. 3 Line	Pipe roof		○			

Moreover, the rotary drilling method is usually used for this type of umbrella method. As shown in Table 3-4, this drilling method is not applicable to weathered rock. However, the ground in the Uryuya Tunnel consisted of extremely weathered rock with an SPT-N value of 8; as a result, making it possible to develop jet-grouted columns there.

Injected steel pipe umbrella method

Figure 5-4 shows that 41% of the applications in which the injected steel pipe umbrella method was employed were carried out in talus, detrital or morainic deposits, followed by clay (24%), weathered rock (24%) and gravel (11%). As mentioned previously, this umbrella method can handle all ground types by a change in the drilling method. The case studies confirm this.

None of the cases in which the injected steel pipe umbrella method was used was carried out in sand. Since sand, in general, has no cohesion, the soil can not support itself between the fore-poles if voids in the soil are not grouted and, consequently, the soil falls through the gaps between the fore-poles (The sub-horizontal jet-grouting method, in contrast, is best suited in sandy ground).

For reference, Trevi SpA (1993) recommends the following guidelines when choosing between the sub-horizontal jet-grouting and injected steel pipe umbrella methods:

- In the case of coarse sands with the presence of blocks and erratics, the injected steel pipe umbrella method is adopted due to the possibility of supporting the soil and nailing blocks and erratics.
- In the case of fine soils[1], the sub-horizontal jet-grouting method is much better suited than the injected steel pipe umbrella method because of the possibility of obtaining columns of consolidated soil which assure the support of the excavation and avoid the possibility of fine material flowing into the excavation.

Pipe roof method (only three cases were studied in this thesis, therefore it is difficult to discuss trends).

Figure 5-4 shows that two-thirds of the applications in which the pipe roof method was employed were carried out in clay, followed by sand (a third of the applications). The pipe jacking method was used for drilling in all cases. As mentioned previously (see Ch. 3.4), this

[1] It seems that term "fine" was used to contrast with term "coarse," so it does not necessarily mean clayey soil.

drilling method is, in general, used in soil. Therefore, it seems that the ground applicable for the pipe roof method basically followed the ground applicable for its drilling method.

This review of ground types allows one to make the following recommendations.

Table 5-5 Suitable ground types for a particular type of umbrella method

Type of umbrella method	Sand	Clay	Gravel	Talus Detritus Moraine	Weathered rock
Sub-horizontal jet-grouting method	A	A	B	C	C
Injected steel pipe umbrella method	B	A	A	A	A
Pipe roof method	A	A	B	C	C

Note: Code: A = Most applicable, B = May be used, C = Not applicable

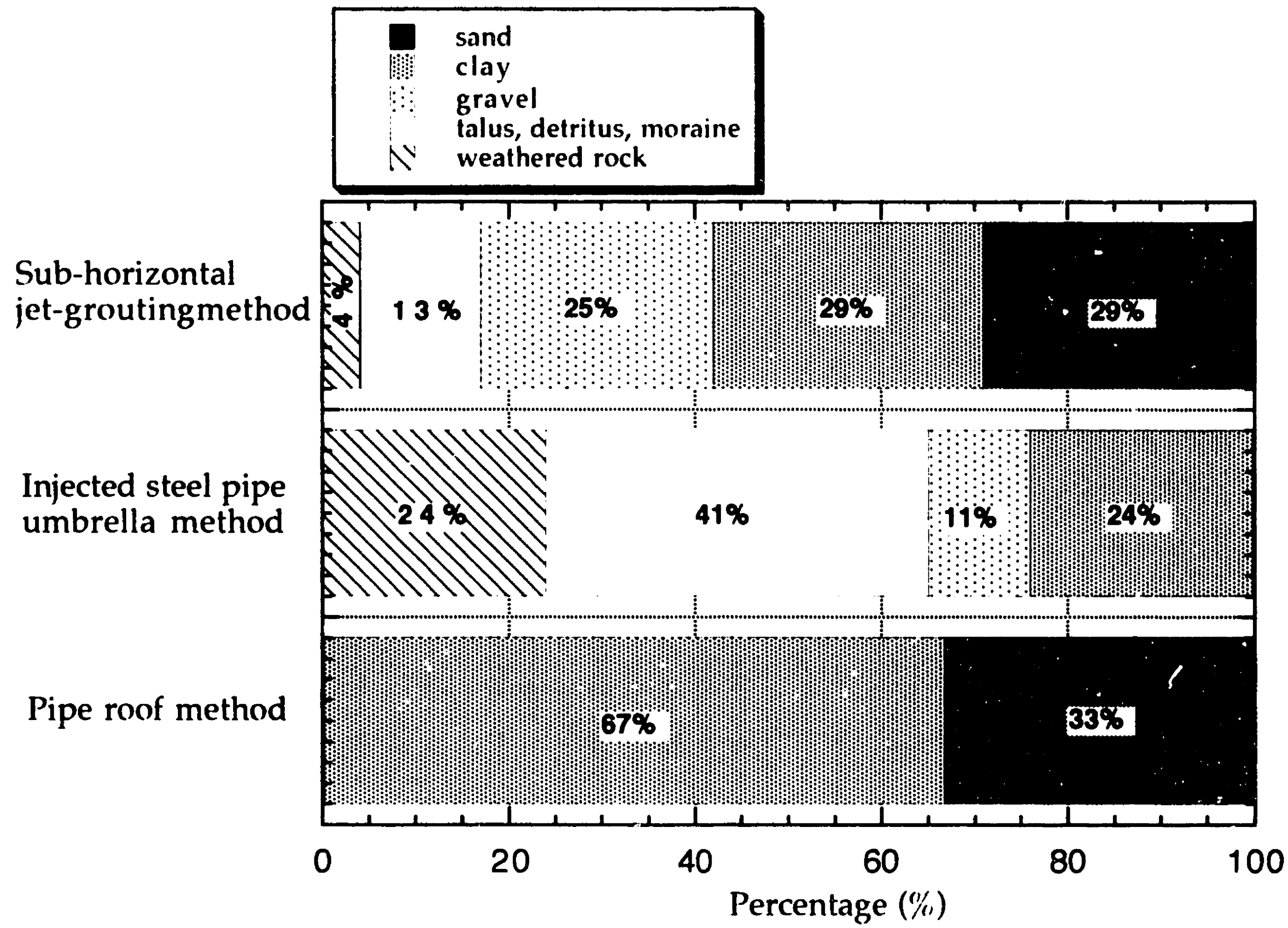


Fig. 5-4 Ground types in which each type of umbrella method was employed

5.1.5 Settlement Characteristics

(1) Ground surface settlement

As mentioned previously, in approximately a third of the cases the umbrella method was employed to restrict ground surface settlement (see Fig. 5-2). Therefore it is important to examine the effect of the umbrella method on the restriction of ground surface settlement.

Table 5-6 summarizes ground surface and tunnel crown settlements in cases where the data were available.

Figure 5-5 shows the relationship between the final ground surface settlement and the normalized depth of overburden, H/D (H: depth of overburden, D: tunnel width). In the case where the final ground surface settlement data were not available, e.g., the Maiko Tunnel (case 13) and the Yakiyama Tunnel (case 16), settlement after excavation of the top heading were plotted. In addition, although heaving occurred in the Hodogaya Tunnel (case 1), those heaving data were not plotted in the figure.

From the fact that the normalized depths of overburden (H/D) for most data were smaller than one, it can be seen that in most cases the overburden was very shallow compared to the tunnel width.

In general, the shallower the depth of overburden the larger the settlement. However, such a trend cannot be clearly observed from the figure.

Figure 5-6 shows the distribution of the final ground surface settlements.

For comparison, Fig. 5-7 shows the final ground surface settlement observed in tunnels where the umbrella method was not employed (Sakurai et al., 1988). Note that no information on ground conditions for any of the cases in Fig. 5-7 was available.

A plot of the final ground surface settlement for tunnels with and without the umbrella method is shown in Fig. 5-8. The normalized depths of overburden (H/D) for tunnels using the umbrella method are smaller than 1.5; therefore only those cases from Fig. 5-7 (without the umbrella method) with the normalized depths smaller than 1.5 are shown in Fig. 5-8.

The final ground surface settlements in tunnels with the umbrella method and those without the umbrella method are compared in Table 5-7.

Table 5-6 Summary of settlement data

Name of tunnel/project	Depth of overburden H (m)	H/D	settlement (mm)					
			ground surface			tunnel crown		
			pre-excavation settlement	settlement after excavation of top heading	final settlement	pre-excavation settlement	settlement after excavation of top heading	final settlement
Hodogaya (D=16.8m)	1.0	0.06	-	-3	-7	-	-	-
	1.5	0.09	-	14	10	-		-
	4.0	0.24	-	15	13	-	-	-
	6.4	0.38	-	2	1	-	-	-
	7.8	0.46	-	-28	-29	-	-	-
	9.0	0.54	-	-37	-39	-	-	-
	9.5	0.57	-	-17	-18	-	-	-
	13.0	0.77	-	-18	-19	-	-	-
	16.0	0.95	-	-17	-19	-	-	-
	16.0	0.95	-	-10	-10	-	-	-
	15.7	0.93	-	-14	-17	-	-	-
	11.0	0.65	-	-13	-12	-	-	-
	10.0	0.60	-	-2	-2	-	-	-
	9.5	0.57	-	0	-1	-	-	-
	8.5	0.51	-	-1	-2	-	-	-
	6.5	0.39	-	-8	-7	-	-	-
	4.6	0.27	-	-18	-19	-	-	-
	5.8	0.35	-	-14	-17	-	-	-
Shoryou No. 1 (D=16m)	7.1	0.44	-8	-15	-20	-	-10	-15
	9.1	0.57	-	-23	-31	-	-15	-21
	10.5	0.66	-	-21	-28	-	-19	-25
	11.2	0.70	-	-23	-28	-	-18	-24
	16.1	1.01	-	-27	-32	-	-20	-26
Kokubugawa (D=8.6m)	9.0	1.05	-27	-47	-52	-	-17	-27
Owani (D=16.2m)	9.0	0.56	-	-	-30	-	-	-44
Aziro (D=11.5m)	17.0	1.48	-	-	-10	-	-5	-8
Uryuya (D=15m)	13.0	0.87	-	-	-	-	-	-7
Kaziwara No. 1 (D=14.2m)	10.0	0.70	-15	-	-40	-	-	-40
Maiko (D=16m) A-zone	6.0	0.38	-90	-122	-	-	-	-
	6.0	0.38	-75	-110	-	-	-	-
	6.0	0.38	-65	-98	-	-	-	-
B-zone	6.0	0.38	-3	-17	-	-	-	-
	6.0	0.38	-4	-18	-	-	-	-
	6.0	0.38	-6	-22	-	-	-	-
Hirai (D=15.5m)	1.5	0.10	-	-	-33	-	-	-34
	4.0	0.26	-	-	-41	-	-	-43
	9.5	0.61	-	-	-40	-	-	-38
	15.0	0.97	-	-	-31	-	-	-37
	22.5	1.45	-	-	-30	-	-	-37.5
	18.0	1.16	-	-	-37.5	-	-	-40
	6.0	0.39	-	-	-56	-	-	-49
	5.0	0.32	-	-	-50	-	-	-40
	5.0	0.32	-	-	-48	-	-	-35
	11.0	0.71	-	-	-44	-	-	-42
	13.0	0.84	-	-	-34	-	-	-37.5
	6.0	0.39	-	-	-58	-	-	-32
	5.0	0.32	-	-	-44	-	-	-24
	5.0	0.32	-	-	-48	-	-	-23
	9.0	0.58	-	-	-38	-	-	-37.5
	13.0	0.84	-	-	-32	-	-	-37.5
	18.0	1.16	-	-	-27	-	-	-40
	7.0	0.45	-16	-40	-49	-	-24	-35
Futatsui-Nishi (D=12m)	2.0	0.17	-	-	-35.3	-	-	-
	4.5	0.38	-	-	-37.6	-	-	-
	7.0	0.58	-25	-30	-33	-	-5	-
	10.0	0.83	-	-	-21.2	-	-	-
Yakiyama (D=12m)	5.0	0.42	-12	-17	-	-10	-17	-
Venezia Station (D=28.8m)	4.0	0.14	-	-	-10.4	-	-	-
	4.0	0.14	-	-	-9.6	-	-	-
	4.0	0.14	-	-	-10	-	-	-
	4.0	0.14	-	-	-11.5	-	-	-
	4.0	0.14	-	-	-13	-	-	-
	4.0	0.14	-	-	-13	-	-	-
	4.0	0.14	-	-	-14	-	-	-
Yokohama Subway No. 3 Line (D=10.2m)	6.7	0.66	-7	-	-8.6	-	-	-
	7.9	0.77	-7	-	-9.2	-	-	-

Note: D: tunnel width

Table 5-7 Comparison of final ground surface settlements between tunnels with and without the umbrella method

	Tunnels with the umbrella method	Tunnels without the umbrella method
Number of data points	n = 61	n=31
Average (mm)	-29.7mm	-41.6mm
Standard deviation (mm)	23.4mm	30.0mm

Figure 5-8 shows that the ground surface settlement for tunnels with the umbrella method is, in general, smaller than that for tunnels without the umbrella method. In particular, the difference in the amount of the settlement between tunnels with and without the umbrella method becomes more distinct as the overburden becomes shallower. Table 5-7 also shows that there is an average of about 10mm difference between tunnels with and without the umbrella method. From the above comparisons, one can conclude that the umbrella method is effective in restricting ground surface settlement.

Next, the influence of ground type upon the characteristics of settlement is discussed.

Figure 5-9 shows the relationship between the final ground surface settlement for each ground type and the normalized depth of overburden. In this thesis classification of ground types is done as follows:

- "sand" includes sand, gravel
- "clay" includes clay, silt and weathered rock

From the figure, it seems that the settlement in clay is smaller than in sand, which is generally observed in tunnel excavations not using the umbrella method (Sakurai et al., 1988). Therefore, one can conclude that such a trend is true whether or not the umbrella method was employed.

Figure 5-10 shows the relationship between the final ground surface settlement and the normalized depth of overburden for each type of umbrella method. Table 5-8 summarizes the average and standard deviation of the settlement for each umbrella method .

From the comparison of the final ground surface settlements among the three types of umbrella methods, it can be seen that the pipe roof method is the most effective in minimizing settlement since the average of the final ground surface settlement is approximately 10mm (see Table 5-8). The reason for this is probably because the fore-piles used in the two cases, i.e., the Venezia Station (case 22) and the Yokohama Subway No. 3 Line (case 24), had relatively large diameters. As a result, the flexural rigidity of the umbrella arches formed by these fore-poles was much higher than that of the others. In fact, ground surface settlement had to be minimized in both tunnel constructions.

Table 5-8 Comparison of final ground surface settlements among three types of umbrella methods

	Sub-horizontal jet-grouting method	Injected steel pipe umbrella method	Pipe roof method
Number of data points	n=24	n=29	n=9
Average (mm)	-20.4mm	-43.9mm	-11.0mm
Standard deviation (mm)	13.2mm	25.6mm	1.9mm

On the basis of average ground surface settlement, one can conclude that the sub-horizontal jet-grouting method is more effective in restricting ground surface settlement than the injected steel pipe umbrella method. The relatively high average settlement for the injected steel pipe umbrella method is possibly related to the following specific cases:

- In the Maiko Tunnel (case 13), in which the ground consisted of a soft alluvial deposit, i.e., the A-zone in the Fukuda junior high school section, the settlements were extremely large, approximately 100mm. In contrast, the settlements in a relatively stable diluvial deposit (B-zone) were about 20mm.
- The settlements observed in the Hirai Tunnel (case 14) were relatively large compared to the others. In this tunnel, the length of each umbrella arch ahead of the face is short, only 3 m, so that the effect on preventing ground movement ahead of the face was low.

If the data for the Maiko Tunnel (A-zone) and the Hirai Tunnel are excluded, the average and standard deviation of the final ground surface settlement for the injected steel pipe umbrella method are -25.1mm and 8.7mm, respectively. Hence, there is no significant difference in the restriction of ground surface settlements between the sub-horizontal jet-grouting method and the injected steel pipe umbrella method.

(2) Settlement curves

The settlement curves observed during excavation of the top heading are shown in Fig. 5-11. The settlement ratio is defined as follows:

$$\text{Settlement ratio (\%)} = \frac{\text{Settlement at a particular distance from the face}}{\text{Settlement after excavation of the top heading}}$$

In all cases except for the Hodogaya Tunnel case ("A" in the figure), the measuring points were on ground surface above the tunnel centerline. In the Hodogaya Tunnel, the measuring point was in ground approximately 2m above the tunnel crown.

For the cases of the Shoryou No. 1 ("B" in the figure), Kokubugawa ("C" in the figure) and Maiko A-zone ("D" in the figure) tunnels, the settlement curves were obtained by ignoring the settlement which occurred during development of the umbrella arches, i.e., during drilling/installation of fore-poles. With respect to other cases such as the Hodogaya ("A" in the figure) and the Yakiyama ("F" in the figure) tunnels, the settlement which occurred during development of the umbrella arches was very small.

As a whole, it can be seen that in all cases the settlement started between 1D and 2D ahead of the face and subsided between 1D and 2D beyond the face. Comparing the distance from the settlement starting point to the face with the distance from the face to final settlement point, the latter seems to be a little longer than the former. As mentioned previously, the general observation in tunnel excavations without the umbrella method shows that such deformation generally starts 2D ahead of the face and subsides 2D beyond the face. Therefore, it seems that tunnel excavation using the umbrella method follows this only to a limited extent.

Next, the pre-excavation settlement ratio, which is defined as settlement ratio when the face arrived at the measuring point, is discussed.

It can be seen from Fig. 5-11 that the settlement curves can be subdivided into two groups regarding the amount of the pre-excavation settlement ratio:

- Group A: curves with small pre-excavation settlement ratio
  Hodogaya (A), Shoryou No. 1 (B), Maiko (B-zone) (E)
- Group B: curves with large pre-excavation settlement ratio
  Kokubugawa (C), Maiko (A-zone) (D) , Yakiyama (F)

Table 5-9 summarizes the characteristics of the settlement curves.

From the table, it can be seen that the average pre-excavation settlement ratios for groups A and B are about 25% and 60%, respectively. It can also be seen that for Group A the range over which settlement occurred is, in general, narrower than that for Group B.

Next, characteristics of the settlement curves are discussed. For simplicity, the following discussion does not include the influence of excavation method on ground deformation.

Table 5-9 Summary of characteristics of settlement curves

	Group A			Group B		
	Hodogaya	Shoryou No. 1	Maiko (B-zone)	Kokubugawa	Maiko (A-zone)	Yakiyama
Total range over which settlement occurred	2.8D (-1.2D to +1.6D)	3.0D (-1D to +2D)	2.8D (-1.2D to +1.6D)	4.0D (-2D to +2D)	3.3D (-1.7D to +1.6D)	1.6D (-1D to +0.6D)
Pre-excavation settlement ratio (%)	27	19	27	48	56	72
H/D	0.37	0.44	0.38	1.05	0.38	0.42
Ground type	Clay	Clay	Gravel, Clay	Sand	Clay	Weathered rock
Settlement after excavation of top heading (mm)	-20	-15 (-8)	-22	-47 (-21)	-122 (-70)	-17

Note: 1) For settlement after excavation of top heading, values within parenthesis were obtained by ignoring the settlement which occurred during development of the umbrella arches.
2) Pre-excavation settlement ratio is settlement ratio when the face arrived at the measuring point.
3) D: tunnel width, H: depth of overburden

Effect of ground conditions on the characteristics of the settlement curves

This effect can be seen in the Maiko Tunnel case in which the injected steel pipe umbrella method was implemented under different ground conditions. The depth of overburden and the excavation method were the same in both A and B zones. The ground type of the A-zone was a soft alluvial deposit consisting of gravel and clay. The ground type of the B-zone was, on the other hand, a relatively stable diluvial deposit consisting of gravel and clay.

Both curves, as a whole, subside approximately 1.6D beyond the face; however settlement in the A-zone seems to start sooner than in the B-zone. This behavior may also have led to the large difference in the settlement after excavation of the top heading between two zones and hence indicates that ground surface settlement is very sensitive to ground conditions below and/or through which a tunnel passes.

Effect of type of umbrella method on the characteristics of the settlement curves

A conclusion regarding this effect can be drawn by a comparison between the Hodogaya Tunnel case and the Maiko Tunnel case (B-zone).

Since the ground conditions and the depth of overburden were almost the same, the difference between these cases was due to the type of umbrella method, that is, the sub-horizontal jet-grouting method and the injected steel pipe umbrella method which were employed in the Hodogaya Tunnel and the Maiko Tunnel, respectively.

As seen in Fig. 5-11, the settlement curves for these cases are very similar to one another. Therefore, it seems that the effect of the type of umbrella method on the characteristics of settlement curve is small.

5.1.6 Tunnel Behavior

Figure 5-12 shows the relationship between the ratio of the ground surface settlement to the tunnel crown settlement and the normalized depth of overburden.

From this figure, it can be seen that the ratio increases when the normalized depth of overburden decreases. This behavior is, in general, observed in tunnel excavation with a shallow overburden and/or poor ground conditions (Sakurai et al., 1988).

As shown in Fig. 5-13 and greatly simplified, the following two types of behavior can occur in tunnel construction:

- mode-A: the tunnel crown deflects inwards and the wall, on the other hand, deflects outwards.
- mode-B: the whole tunnel structure sinks as a rigid structure.

It should be emphasized that in reality both modes can take place simultaneously.

The main factor that influences the above tunnel behavior is believed to be soil-structure interaction and particularly the difference between the stiffness of the tunnel and of the surrounding ground. If the tunnel structure is much stiffer than the surrounding ground, mode-B prevails. On the other hand, if the surrounding ground is much stiffer than the tunnel structure, mode-A prevails.

As stated previously, the Maiko Tunnel case (case 13) showed mode-B behavior (see Fig. 4-13-11). Regarding the other cases, the data were not available to decide which behavior took place.

Taking into account that a tunnel with an umbrella arch is much stiffer than a tunnel without an umbrella arch and, moreover, the ground condition where the umbrella method is employed is usually poor, mode-B behavior may prevail. Therefore, to prevent the tunnel from sinking, it is important to stabilize the footings of the steel arch support by means of soil improvement such as jet grouting and/or by enlarging the area of the footings to reduce the stresses acting on the ground beneath the footings. The Maiko Tunnel is a good example of these measures.

Figure 5-14 is a conceptual description of the transmission of earth pressures. As shown in the figure, earth pressures acting on the umbrella arch are first transmitted to the steel arch support and then are transmitted to its footings. It seems that in a tunnel with the umbrella method, the steel arch support carries more of the load than in a tunnel without the umbrella method where the steel arch and shotcrete supports share the load more evenly. The reason for this seems to be that the umbrella arch acts as a tunnel support, and the stiffer the umbrella arch the more load the steel arch support initially carries; as a result, the shotcrete carries less. In fact, in the Subway Vienna case (case 7), stresses in shotcrete reached only 10 % of the failure load due to the umbrella method's contribution to overall tunnel support. Also, it was reported that in the Maiko Tunnel (case 13) the ratio of the load carried by the steel arch support to the load carried by the shotcrete was larger than that in the zone where the umbrella method was not employed.

In the Campiolo Tunnel (case 9), on the other hand, the steel arch supports were virtually unloaded. In this case, the footings of the umbrella arch were stabilized by the jet-grouted columns (no information on the stabilization zone and on stresses in shotcrete was available from the reference). This implies that the earth pressures acting on the tunnel were supported only by the arch itself.

From these considerations, one can conclude that the umbrella contributes to a great extent to overall support during tunnel excavation. Moreover, it seems that the transmission of earth pressures acting on the umbrella arch depends on the stabilization beneath the footings of the umbrella arch.

As mentioned previously, the primary tunnel supports used with the umbrella method were almost the same as those without the umbrella method and hence the umbrella structure was not considered to be one of the load-bearing structures. However, there is no doubt that in reality the umbrella structure carries earth pressures acting on the tunnel. The need to reduce dimensions of the tunnel supports such as primary lining and steel arch support, and hence to minimize construction costs, will lead to a design which actively uses the umbrella.

5.1.7 Heaving and Settlement during Development of the Umbrella Arch

As mentioned in the Shoryou No. 1 (case 2), the Kokubugawa Tunnel (case 3) and the Maiko Tunnel (case 13) cases, settlement probably induced by overdrilling and/or ground disturbance due to drilling was observed for both the sub-horizontal jet-grouting method and the injected steel pipe umbrella method. For instance, it was reported that in the Maiko Tunnel drilling fluid reversed due to clogging inside the pipes and then invaded the area around the casing and, as a result, overdrilling occurred.

In the Hodogaya Tunnel (case 1) and the Shoryou No. 1 Tunnel (case 2) cases, heaving occurred during development of the jet-grouted columns. In the Hodogaya Tunnel case heaving occurred due to increased pressures in the ground induced by the insufficient removal of cutting of soil.

The umbrella method is effective in minimizing ground deformation. However, for further use of the umbrella method, quality control during development of the umbrella arch seems to require more effort.

5.1.8 Analytical Methods

In the Hodogaya Tunnel case (case 1), results of three-dimensional (3-D) FE method and measured values were compared. It seems that behavior of the fore-poles and ground deformation can be assessed by using a 3-D FE method.

In addition, a simple analytical model, that is, a beam on an elastic foundation, was described in the Hodogaya Tunnel case. Regarding the bending strains in the fore-poles, the analysis results obtained by the simple model correspond well to the measured values.

Considering that the umbrella method is frequently used to restrict ground surface settlement, an analytical method which can estimate ground deformation is needed. A 3-D FE method, which can satisfy the above demand, is, in general, an expensive design tool; for this reason, a simple analytical method needs to be implemented.

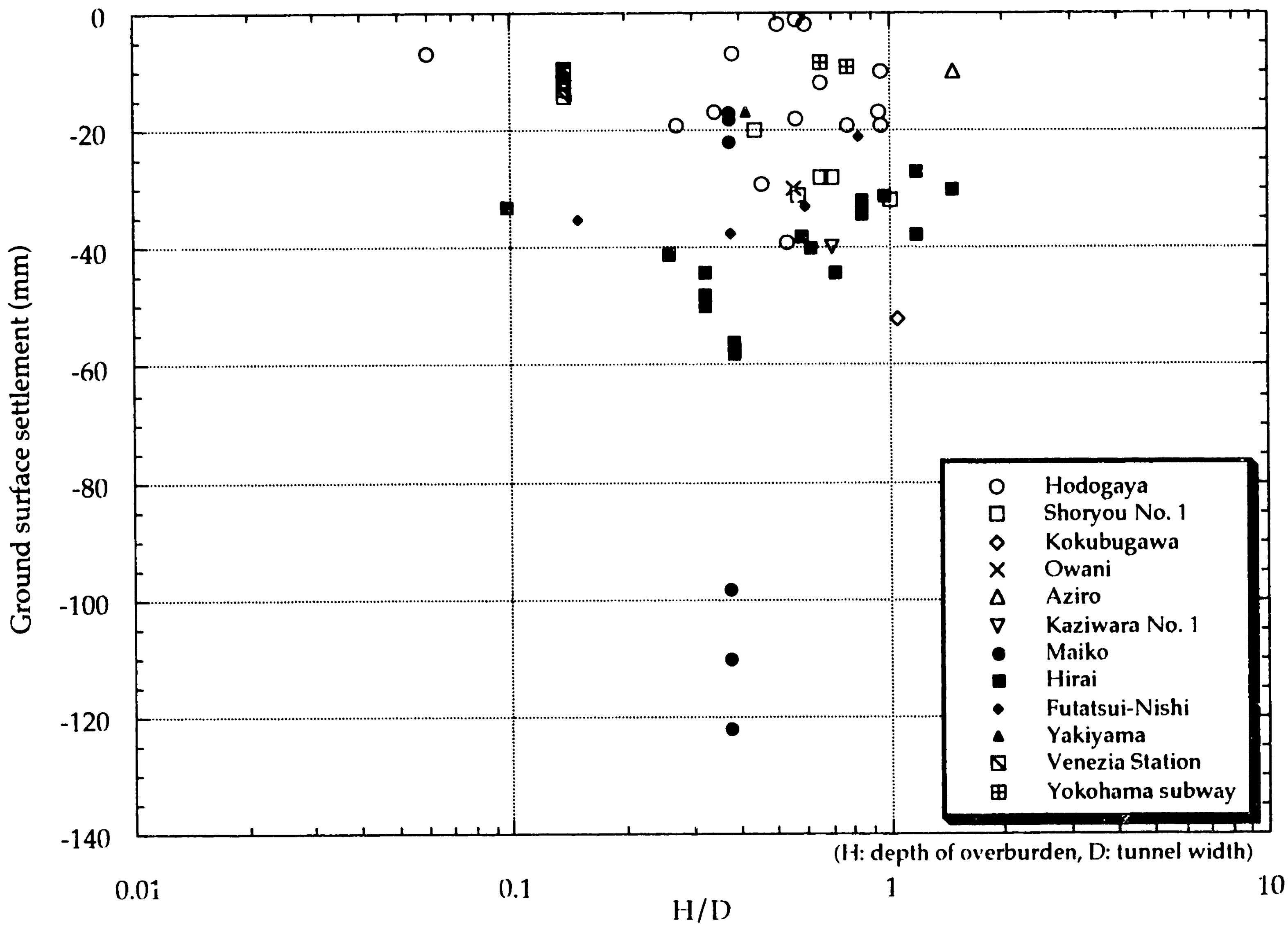


Fig. 5-5 Ground surface settlement versus normalized depth of overburden

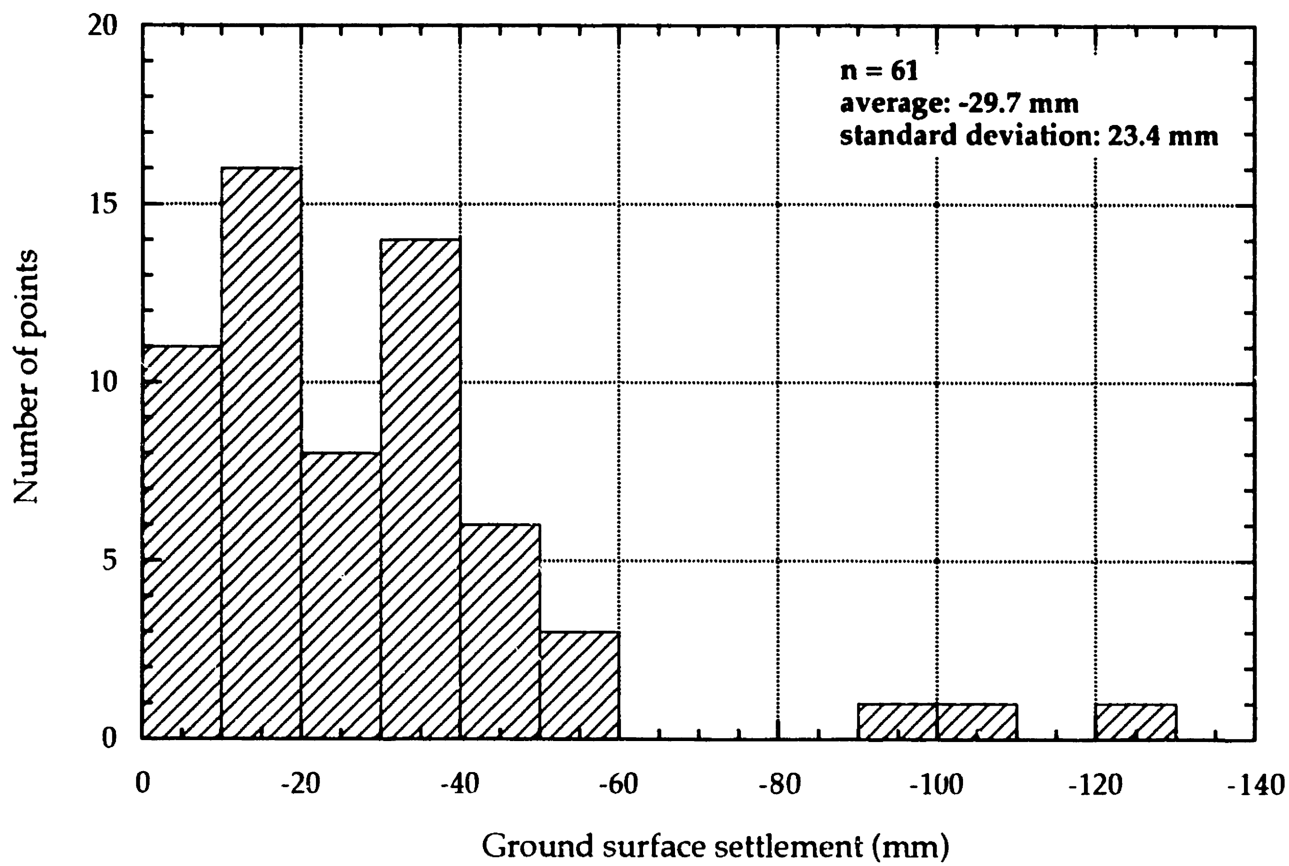


Fig. 5-6 Distribution of ground surface settlements

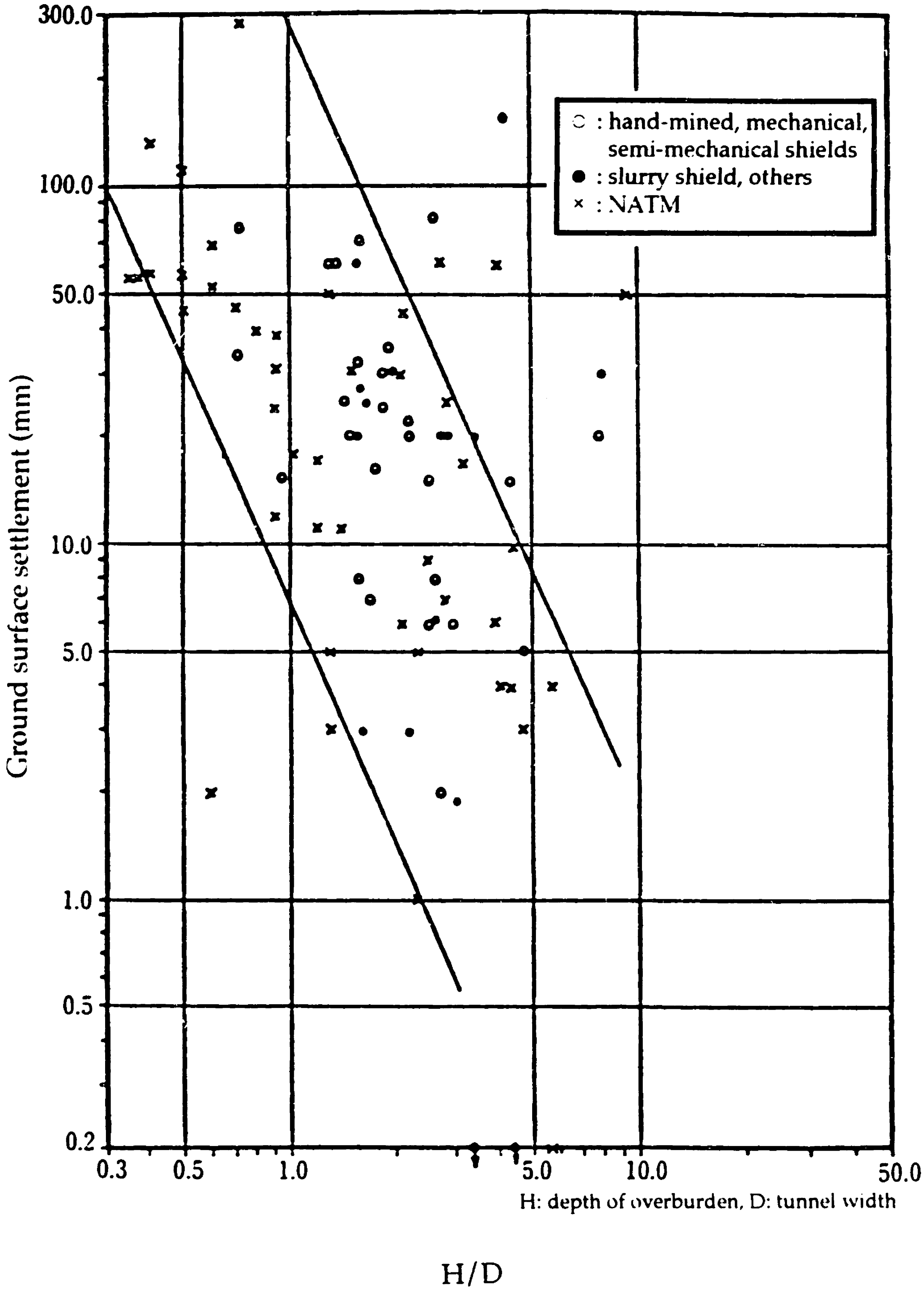


Fig. 5-7 Ground surface settlement versus normalized depth of overburden [tunnel without the umbrella method] (Sakurai et al., 1988)

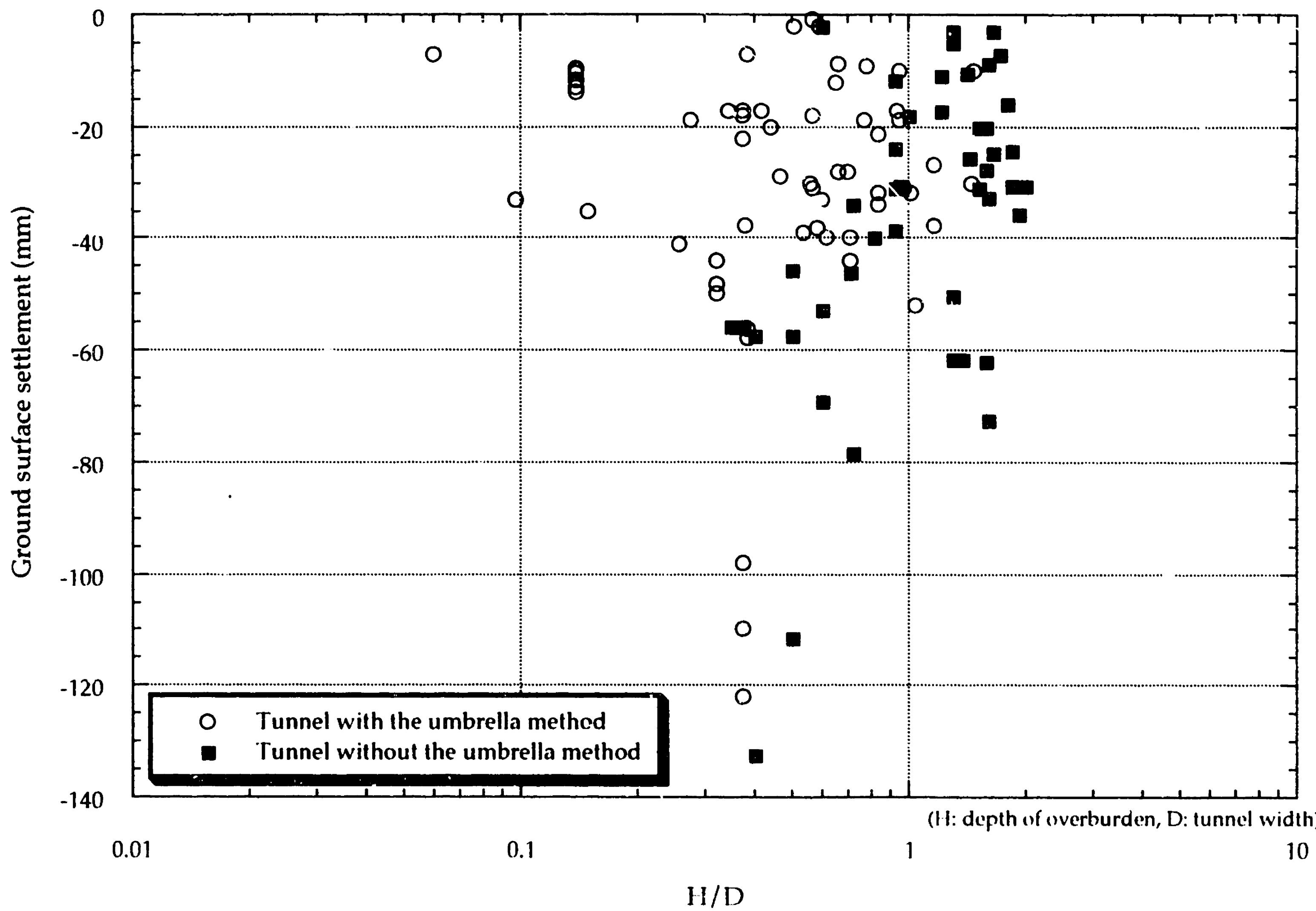


Fig. 5-8 Comparison of ground surface settlements between tunnels with and without the umbrella method

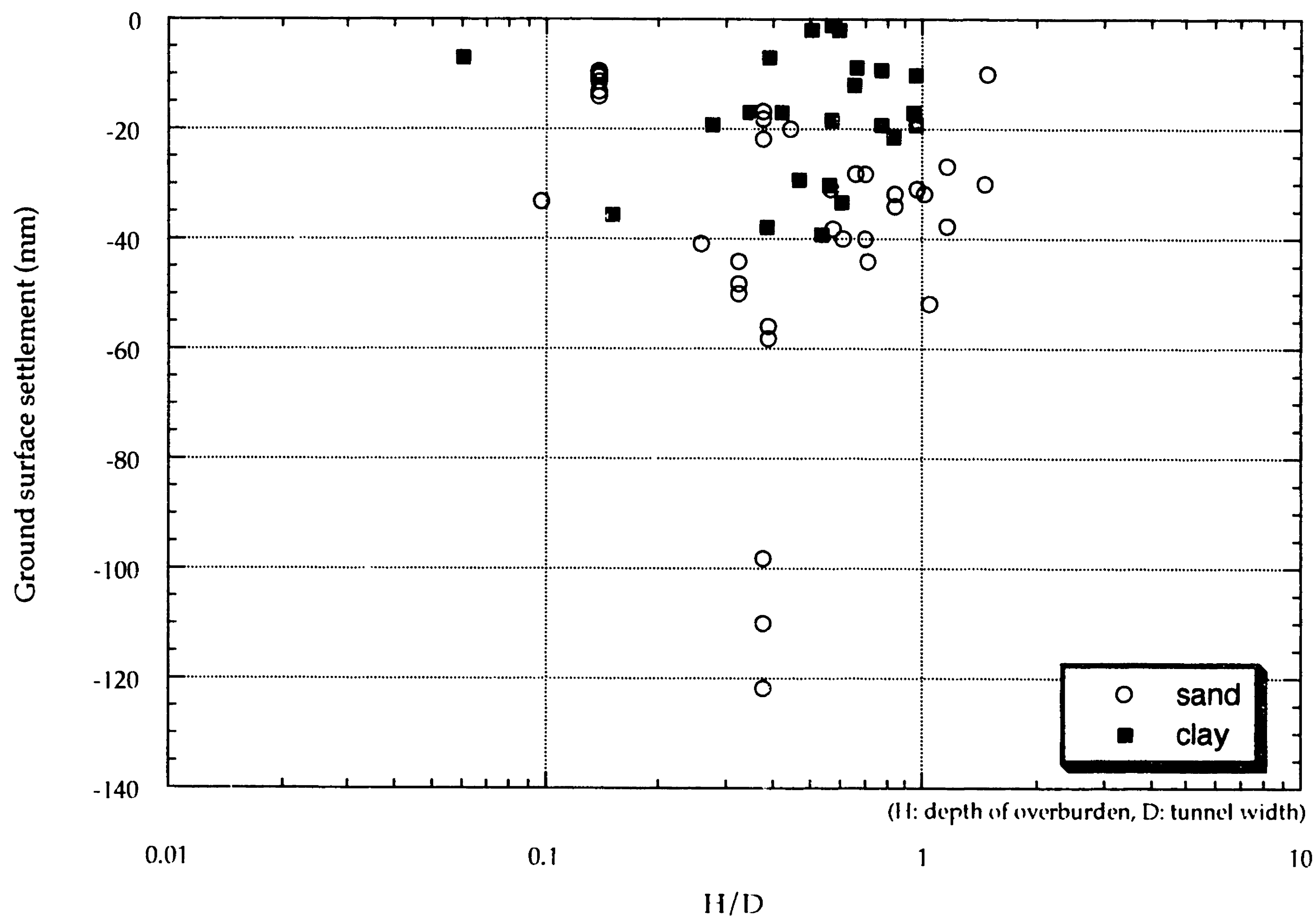


Fig. 5-9 Comparison of ground surface settlements between in sand and in clay

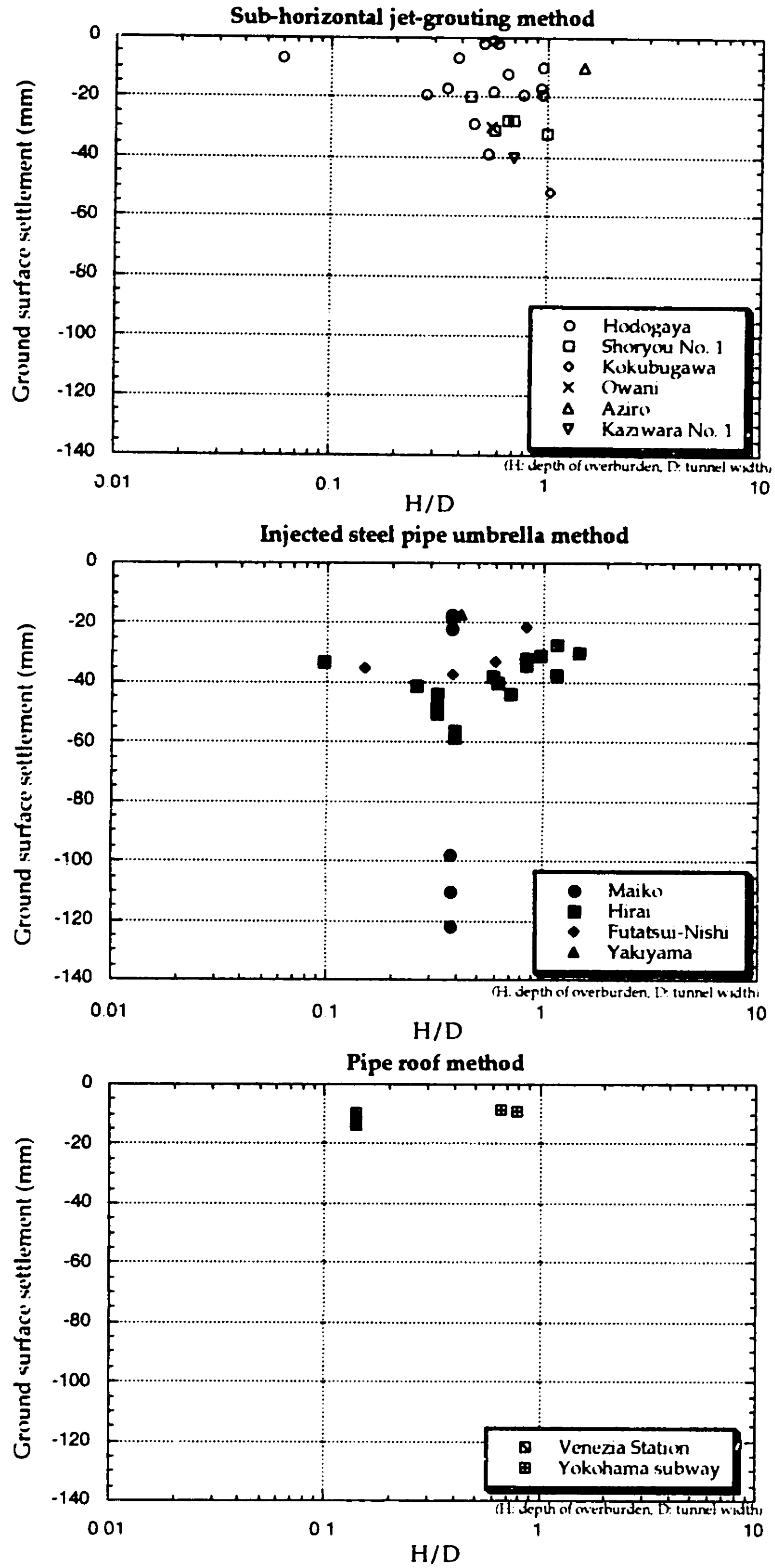


Fig. 5-10 Ground surface settlement versus normalized depth of overburden for a particular type of umbrella method

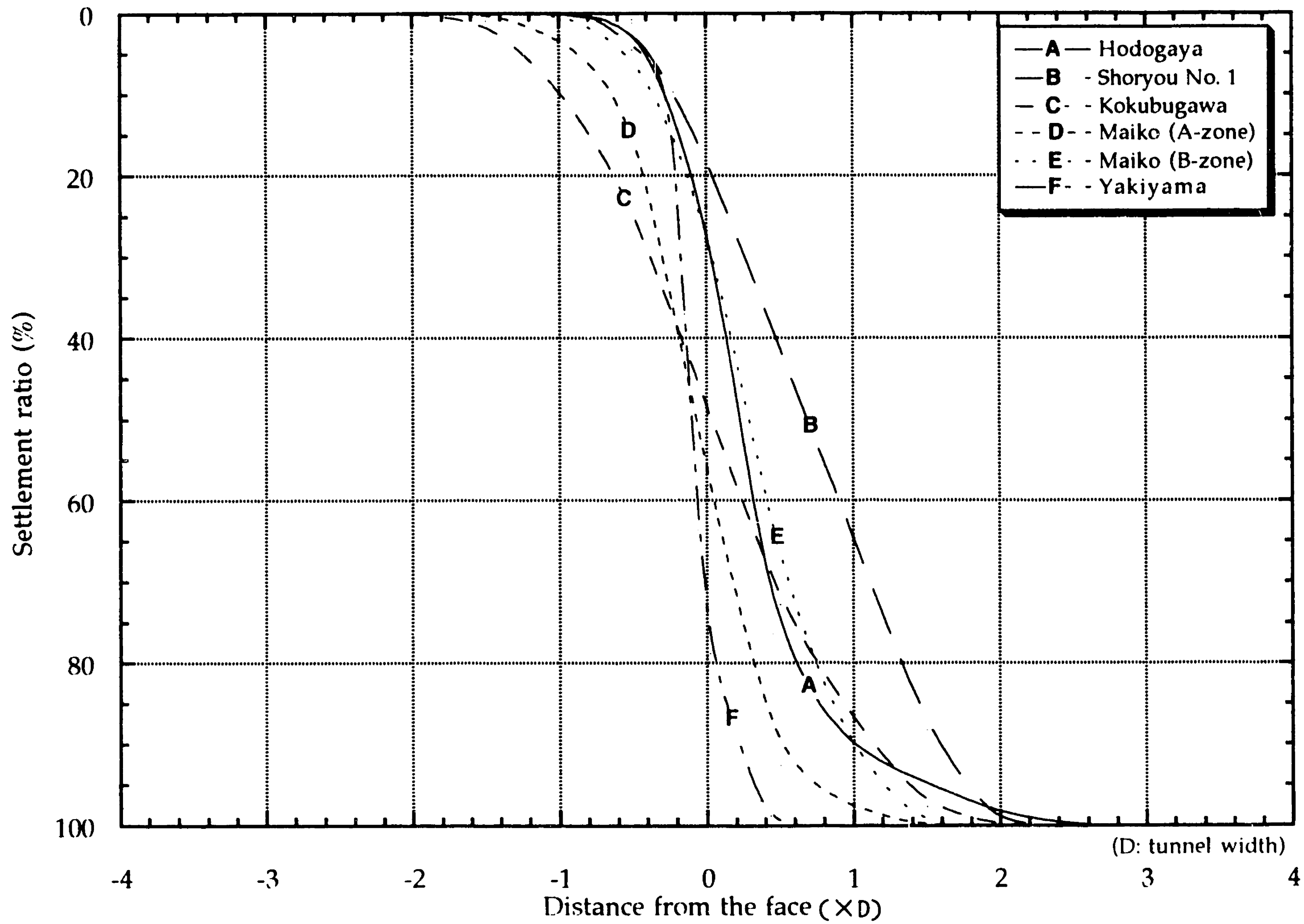


Fig. 5-11 Settlement curves

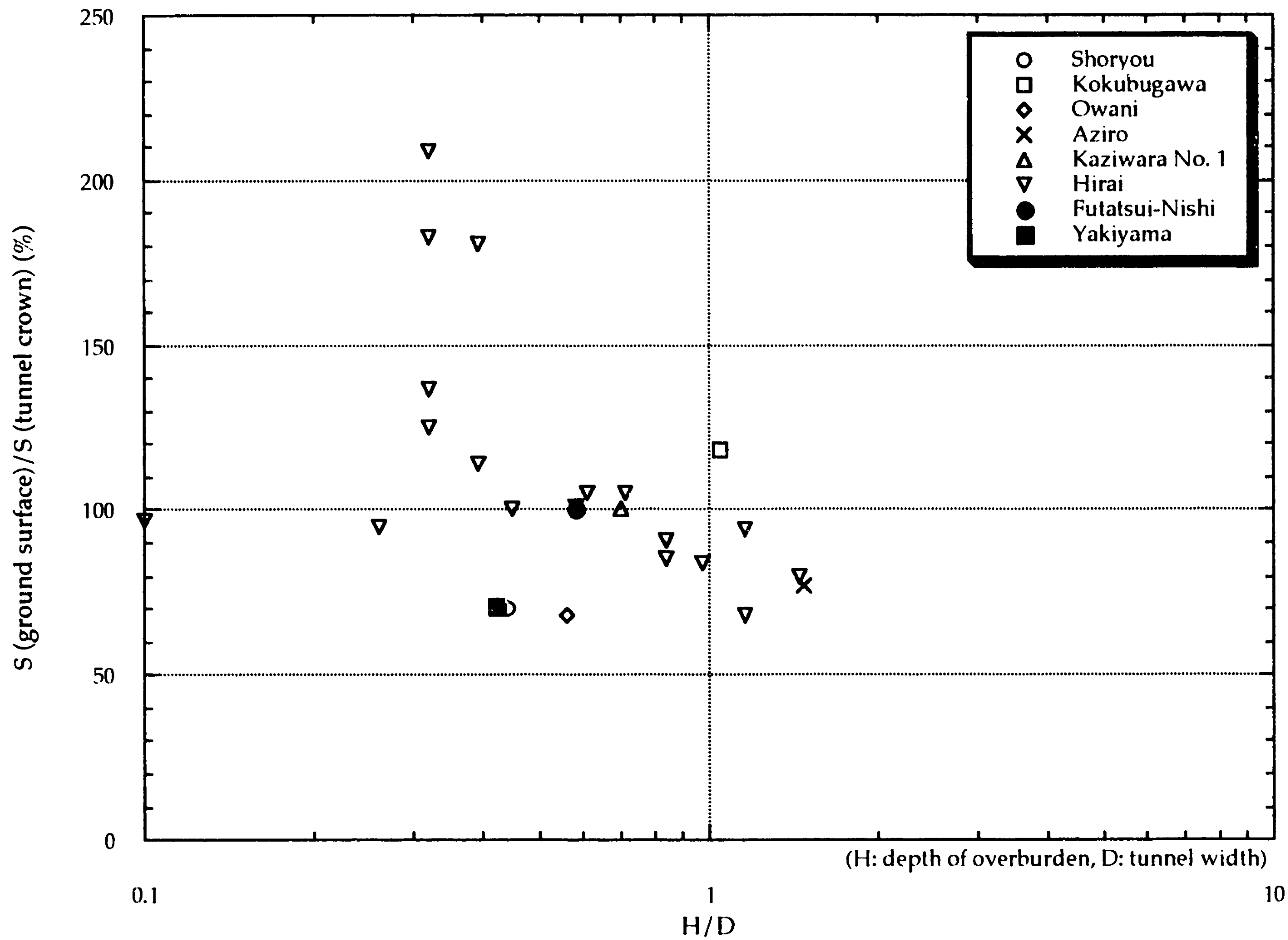


Fig. 5-12 Ratio of ground surface settlement to tunnel crown settlement versus normalized depth of overburden

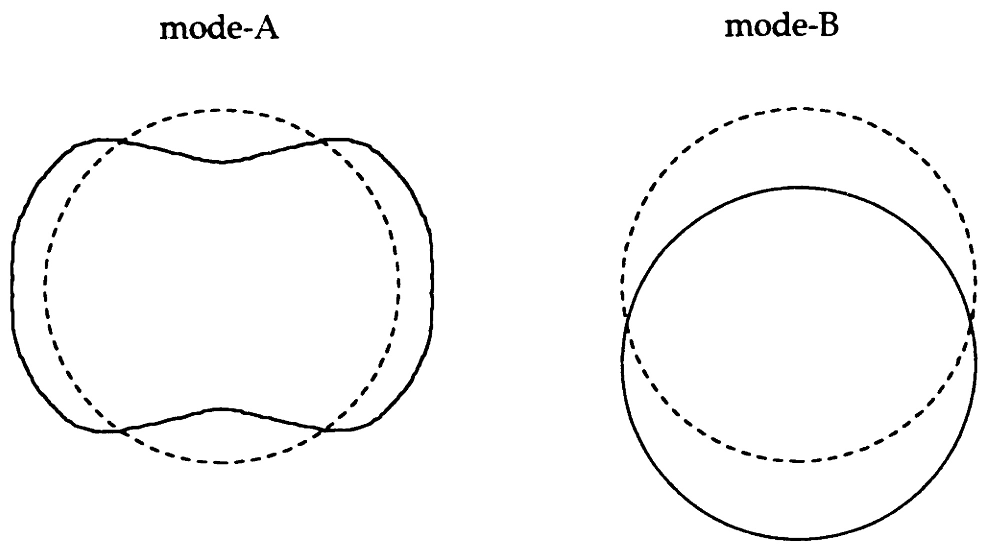


Fig. 5-13 Schematic description of behavior of tunnel structure

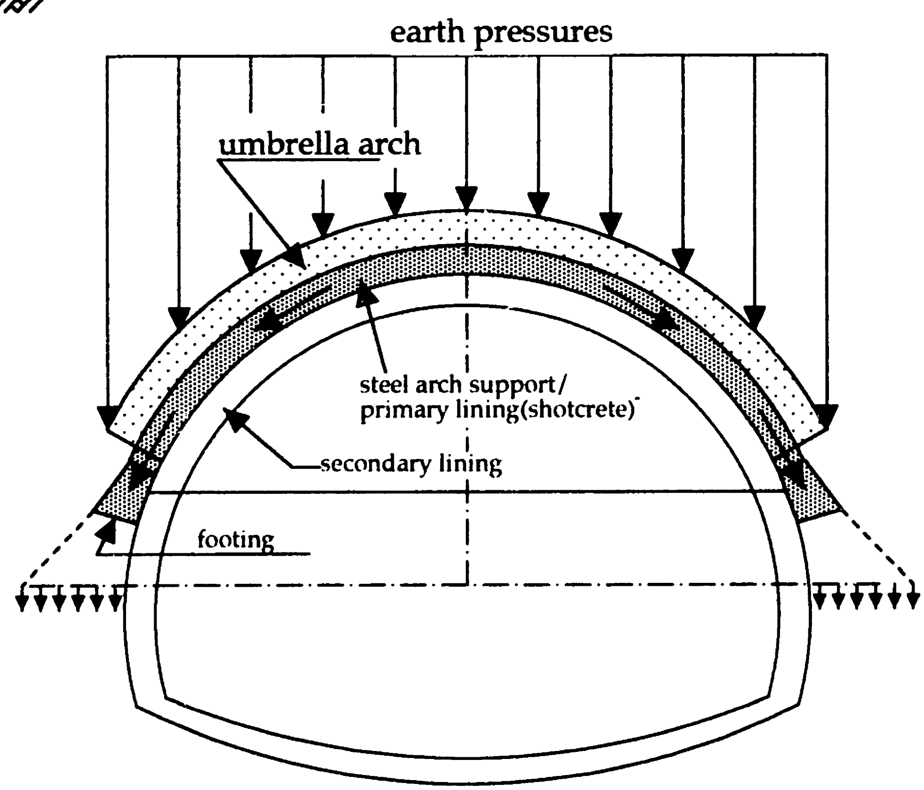


Fig. 5-14 Transmission of earth pressures

5.2 Conclusions and Future Studies

5.2.1 Conclusions

In this research, the principles of the umbrella method were first discussed and then extensive case studies on the umbrella method were carried out, which mainly considered the following aspects:

- ground conditions in which the umbrella method was employed
- reasons for using the umbrella method
- structural details of tunnel
- field measurements such as ground deformation

From the results of the 24 case studies, the following conclusions can be drawn:

Adaptability on ground type

- Ground type seems to be a distinctive factor in the selection of the umbrella method. This is mainly due to the drilling method which is used for each type of umbrella method. Roughly speaking, in soil (sand, clay, gravel), all of the umbrella methods are feasible, while in weathered rock, only the injected steel pipe umbrella method is applicable.

Effectiveness in limiting problems in tunnel construction and in contributing to overall tunnel support

- Both the sub-horizontal jet-grouting method and the injected steel pipe umbrella method are effective in: 1)preventing slope failures and/or landslides, 2) restricting ground surface settlement; and 3) increasing face stability. The pipe roof method, on the other hand, is especially effective in restricting ground surface settlements because of its high flexural rigidity.
- When the umbrella method was used, ground surface settlement was smaller compared to cases where the umbrella method was not employed. Therefore, the umbrella method is effective in restricting ground surface settlement. Of the three types of umbrella methods, the pipe roof method is most effective in restricting ground surface settlements. Between the sub-horizontal jet-grouting method and the injected steel pipe umbrella method, there was no significant difference in the restriction of ground surface settlements.
- It seems that using the umbrella method makes it possible to reduce the dimensions of the tunnel supports.

Settlement characteristics

- In tunnel excavations using the umbrella method, settlement caused by the top heading excavation started between 1D and 2D ahead of the face and subsided between 1D and 2D beyond the face. This is slightly different from general observations in tunnel excavations in which the umbrella method was not used, where such a settlement generally starts 2D ahead of the face and then subsides 2D beyond the face.
- The general observation for tunnel excavations that ground surface settlement in sand is larger than in clay is true whether or not the umbrella method was employed.
- When the depth of overburden and the type of umbrella method were the same (the injected steel pipe umbrella method), the characteristics of the settlement curves varied with ground conditions.
- Comparing the sub-horizontal jet-grouting method with the injected steel pipe umbrella method, when the ground conditions and the depth of overburden were similar, showed that the characteristics of the settlement curves were very similar to one another.
- In tunnels using the umbrella method, it is important to stabilize the footings of the steel arch support to restrict ground surface settlement.
- It is possible that heaving and/or settlement occur during development of the umbrella arches.

Numerical analysis

- A three-dimensional (3-D) FE method can assess the behavior of the umbrella structure and ground deformation. It is possible to use a simple model with a beam on an elastic foundation to estimate strain in the fore-poles.

5.2.2 Future Studies

Although this research provides valuable information to understand the umbrella method, further studies are needed, in particular, for selecting an appropriate umbrella method for tunnel construction and understanding the behavior of the umbrella structure:

For selection of an appropriate umbrella method

- Collection of information on construction costs:
  The construction cost of each umbrella method should be considered in the selection of an appropriate method.
- Introduction of new type of umbrella method:
  In this thesis, the umbrella method was subdivided into three categories: the sub-horizontal jet-grouting method; the injected steel pipe umbrella method; and the pipe

roof method. However, from a practical point of view, other methods can be defined as umbrella methods, for example, the PASS method, in which a thin mortar lining is created ahead of the face (Tsuchiya et al., 1995), and horizontal chemical grouting, which was employed in Washington D.C. to construct a subway in which extremely long (approximately 200m) chemical-grouted columns were developed (Blakita et al., 1995). Comparing such methods with the three categories of umbrella methods discussed in this thesis is desirable.

- Collection of more cases:

  For completing the database on the umbrella method, further collection of cases in which the umbrella method was used or is being used is important. There exist many more construction cases, especially in Italy.

For understanding the behavior of the umbrella structure

- Establishment of a numerical analysis method:

  As mentioned previously, a 3-D FE method is often used for assessing the behavior of the umbrella structure and estimating ground deformation. The 3-D FE method is particularly appropriate since the umbrella structure is constructed near the face, where ground behavior is always complex. Also, since the umbrella structure extends in both longitudinal and transverse directions, an analysis in only one direction is insufficient to fully understand the behavior of the umbrella structure. Although the 3-D FE method satisfies these requirements, it is often too expensive in practical design. Hence, it may be desirable to develop simpler approaches.

- Assessment of the behavior of the umbrella structure.

  The main focus was on ground deformation in this thesis. However, it seems desirable to better understand how the umbrella structure interacts with the ground and the tunnel.

Appendix - Summary Tables of Case Studies

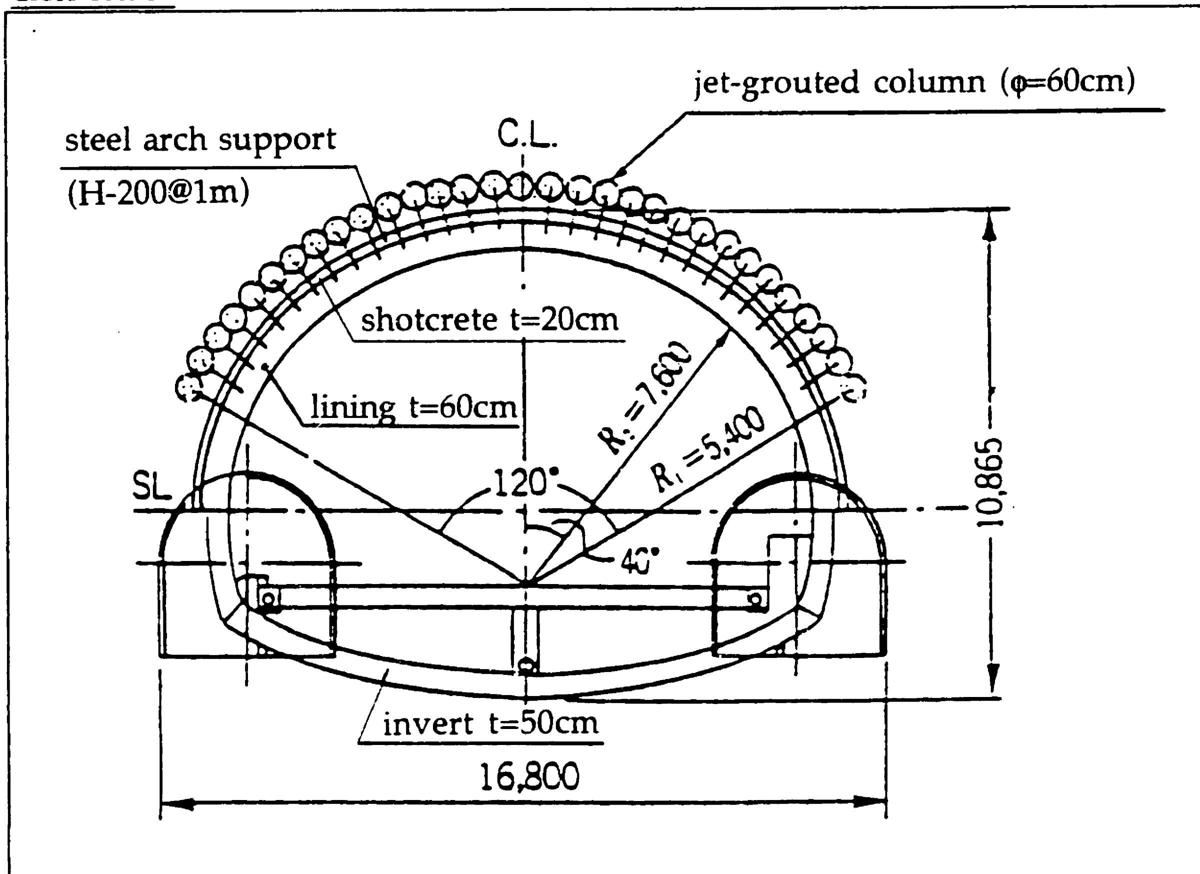
Name of project	Hodogaya Tunnel
Time frame	From 1992/3 To 1993/12
Location	Yokohama, Kanagawa Pref., Japan
Purpose of tunnel	3-lane motorway tunnel
Main contractor	Hazama Corporation
Clients	Japan Highway Public Corporation
Tunnel dimensions	Width(m): 16.8 Height(m): 10.9 Length(m): 200 Exc. area(m^2): 150
Umbrella method	sub-horizontal jet-grouting Fore-pole Dia. (cm): 60 Length (m): 10 @ N/A (cm)
	Number of fore-poles per cross section: 31
Drilling method	machine: SR-510 (Rodio SpA)
Grouting	cement/water grout (w/c=1.1)
Tunnel support	shotcrete (cm): t=20 steel arch: H-200 @ 100 (cm) secondary lining (cm): t=60
Geology/Hydrogeology	Depth of overburden(m): 5 - 17 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	loam, sand	3	15	10	0.03
layer-2	volcanic cohesive soil	4	15	5	0.04
layer-3	loam	4	20	5	0.04
layer-4	volcanic cohesive soil	4	25	5	0.04
layer-5	silt, sandy silt, fine sand, gravel	18	210	12	0.2
layer-6	mudstone	>50	390	15	1.4

Settlement (mm) Depth of overburden at the measuring point (m): 6.5

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	8	7
tunnel crown:	5	20	N/A

Cross section



Name of project Shoryou No. 1 tunnel **Time frame** From 1989/7 To 1990/4

Location Gotenba, Shizuoka Pref., Japan **Purpose of tunnel** 3-lane motorway tunnel

Main contractor Toyo Construction Co. **Clients** Japan Highway Public Corporation

Tunnel dimensions Width(m): 16 Height(m): 11 Length(m): 220 Exc. area(m²): 145

Umbrella method sub-horizontal jet-grouting Fore-pole Dia. (cm) 60 Length (m) 10 @ 60 (cm)

Number of fore-poles per cross section: 23 - 44

Drilling method machine: SR-510 (Rodio SpA) **Grouting** water/cement grout

Tunnel support shotcrete (cm): t= 25 steel arch: H-200 @ 100 (cm) secondary lining (cm): t= 45

Geology/Hydrogeology Depth of overburden(m): 7 - 17 Groundwater level (m): N/A

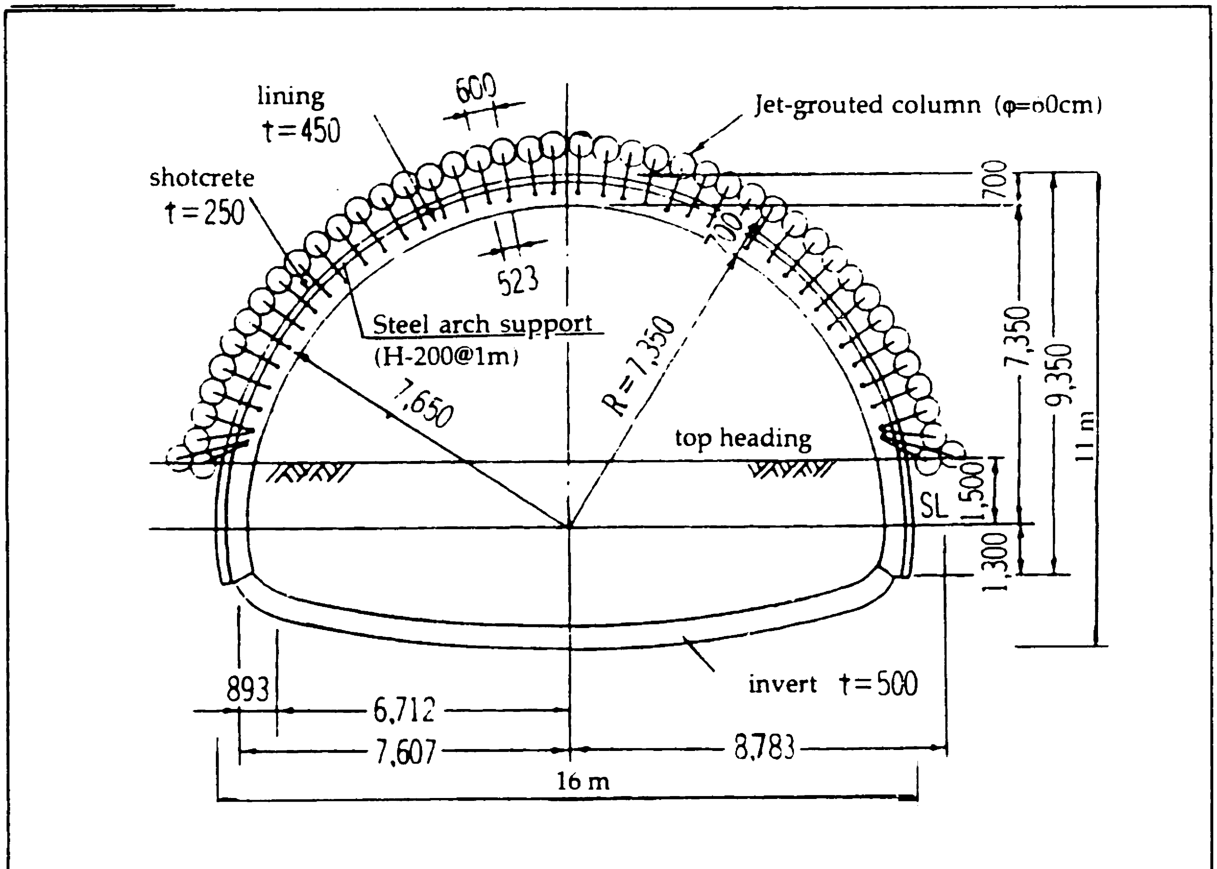
	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	Scoria	3	N/A	N/A	N/A
layer-2	Scoriaceous loam	3 - 20	N/A	N/A	0 - 10
layer-3	Volcanic sand & gravel	>50	N/A	N/A	N/A
layer-4	Scoriaceous volcanic sand	N/A	N/A	N/A	N/A
layer-5	Scoriaceous loam	N/A	N/A	N/A	N/A
layer-6		N/A	N/A	N/A	N/A

Settlement (mm) Depth of overburden at the measuring point (m): 7.1

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	8	15	20
tunnel crown:	-	9	14

Tunnel crown settlements were measured after primary supports were installed.

Cross section



Name of project	Kokubugawa Tunnel
Time frame	From 1989/1 To 1989/3
Location	Ichikawa, Chiba Pref., Japan
Purpose of tunnel	underground tunnelled river
Main contractor	Shimizu Corporation
Clients	Chiba Prefecture
Tunnel dimensions	Width(m): 8.6 Height(m): 8.5 Length(m): 383 Exc. area(m²): 60
Umbrella method	sub-horizontal jet-grouting; Fore-pole Dia. (cm) 60; Length (m) 7 @ 60 (cm); Number of fore-poles per cross section: 34
Drilling method	machine: SR-11 (Rodio SpA)
Grouting	water/cement grout
Tunnel support	shotcrete (cm): t=20; steel arch: H-125 @ 100 (cm); secondary lining (cm): t= N/A
Geology/Hydrogeology	Depth of overburden(m): 5 -20; Groundwater level (m): N/A

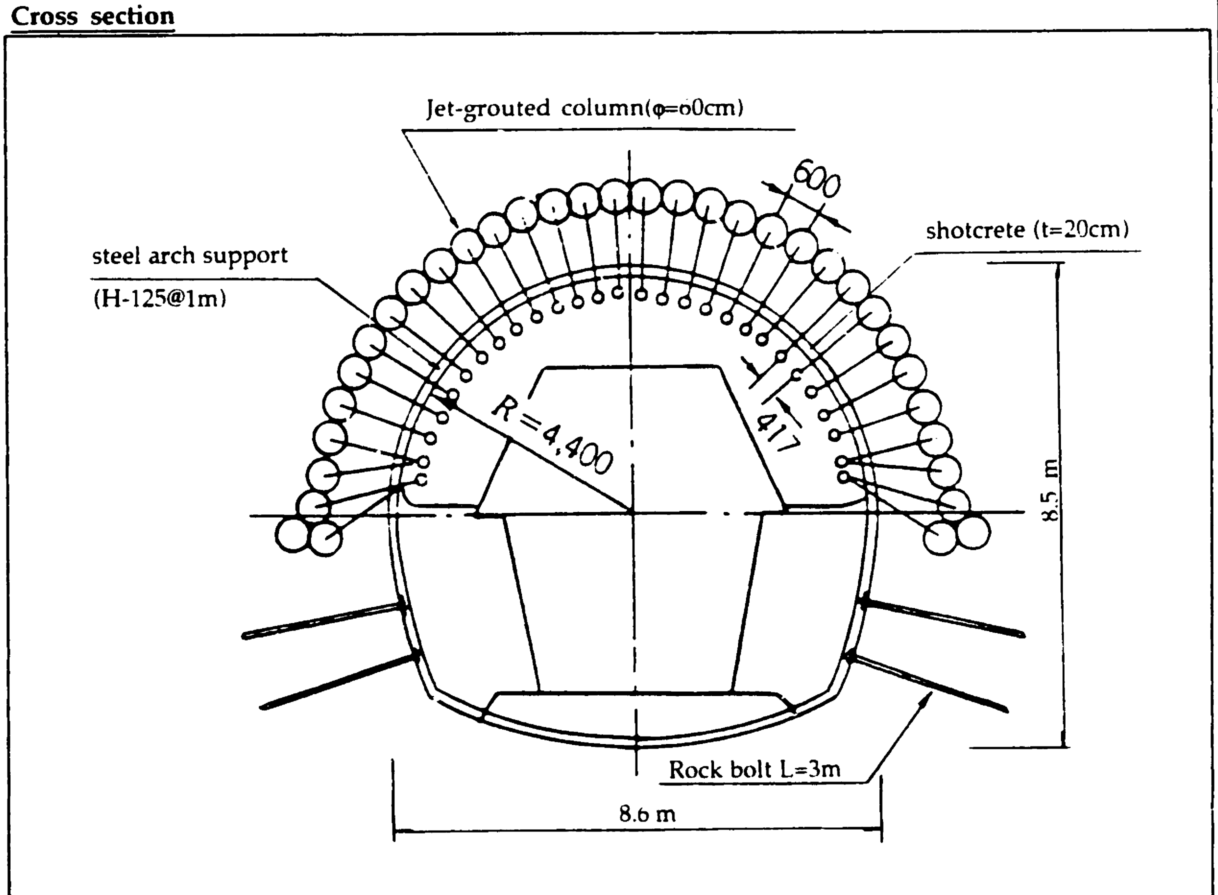
	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	top soil	< 5	N/A	N/A	N/A
layer-2	tuffaceous clay	< 10	N/A	N/A	N/A
layer-3	fine sand	<20	N/A	N/A	N/A
layer-4	sand	N/A	N/A	N/A	N/A
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 9

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	27	47	52
tunnel crown:	-	17	27

Tunnel crown settlement was measured after primary supports were installed.

Cross section



Name of project	Owani Tunnel	**Time frame**	From 1990/4 To 1990/9
Location	Owani, Aomori Pref., Japan	**Purpose of tunnel**	Motorway Tunnel
Main contractor	Daiho Corporation.	**Clients**	Ministry of Construction

Tunnel dimensions Width(m): 16.2 Height(m): 10.1 Length(m): 145 Exc. area(m²): 129

Umbrella method sub-horizontal jet-grouting Fore-pole Dia. (cm) 60 Length (m) 7 @ N/A (cm)
Number of fore-poles per cross section: 36

Drilling method machine: SR-11 (Rodio SpA) Grouting: N/A

Tunnel support shotcrete (cm): t= 25 steel arch: H-200 @ 100 (cm) secondary lining (cm): t= 50

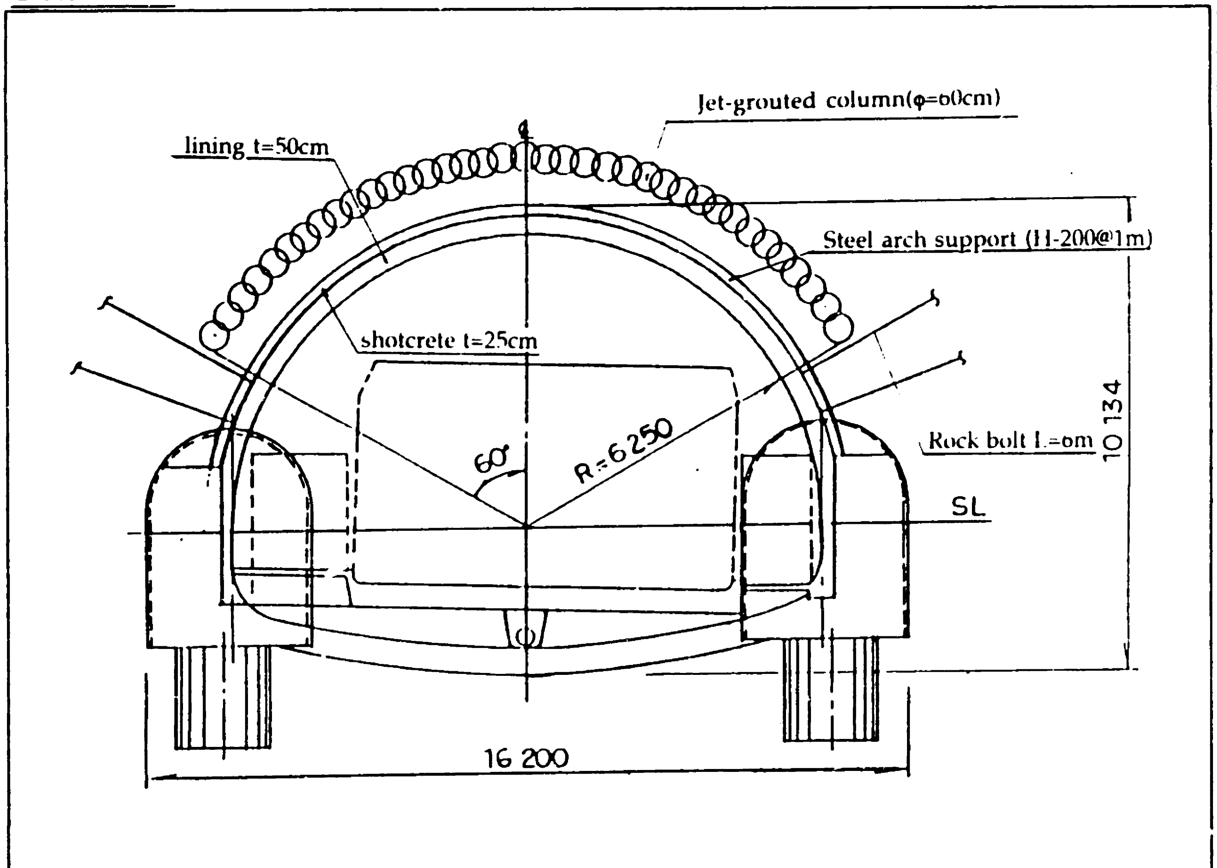
Geology/Hydrogeology Depth of overburden(m): 3.5 - 15 (mean 9 m) Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	talus deposits (clay, gravel, pumice)	18	N/A	N/A	N/A
layer-2	dacite	N/A	N/A	N/A	N/A
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 9

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	30
tunnel crown:	N/A	N/A	44

Cross section



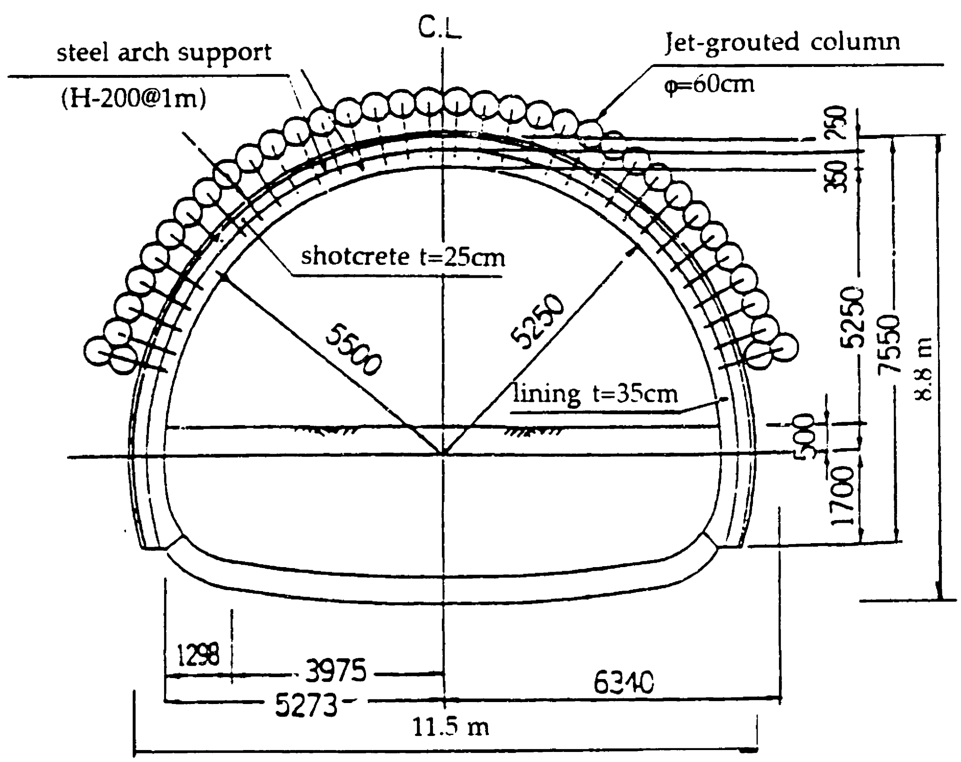
Name of project	Aziro Tunnel
Time frame	From 1990/12 To 1990/4
Location	Itukaichi, Tokyo, Japan
Purpose of tunnel	2-lane motorway tunnel
Main contractor	Tobishima Corporation
Clients	Tokyo Metropolitan Goverment.
Tunnel dimensions	Width(m): 11.5 Height(m): 8.8 Length(m): 563 Exc. area(m²): 90
Umbrella method	sub-horizontal jet-grouting Fore-pole Dia. (cm) 60 Length (m) 10 @ N/A (cm)
	Number of fore-poles per cross section: 36
Drilling method	machine: SR-510 (Rodio Spa) Grouting: cement/water grout (w/c=1)
Tunnel support	shotcrete (cm): t= 25 steel arch: H-200 @ 100 (cm) secondary lining (cm): t= 35
Geology/Hydrogeology	Depth of overburden(m): 0 - 20 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	fill	1 - 8	N/A	N/A	N/A
layer-2	loam	2 - 5	N/A	N/A	N/A
layer-3	sand with gravel, caly	2 - 14	N/A	N/A	N/A
layer-4	silt	N/A	N/A	N/A	N/A
layer-5	gravel	40 - 100	N/A	N/A	N/A
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 17

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	10
tunnel crown:	N/A	5	8

Cross section



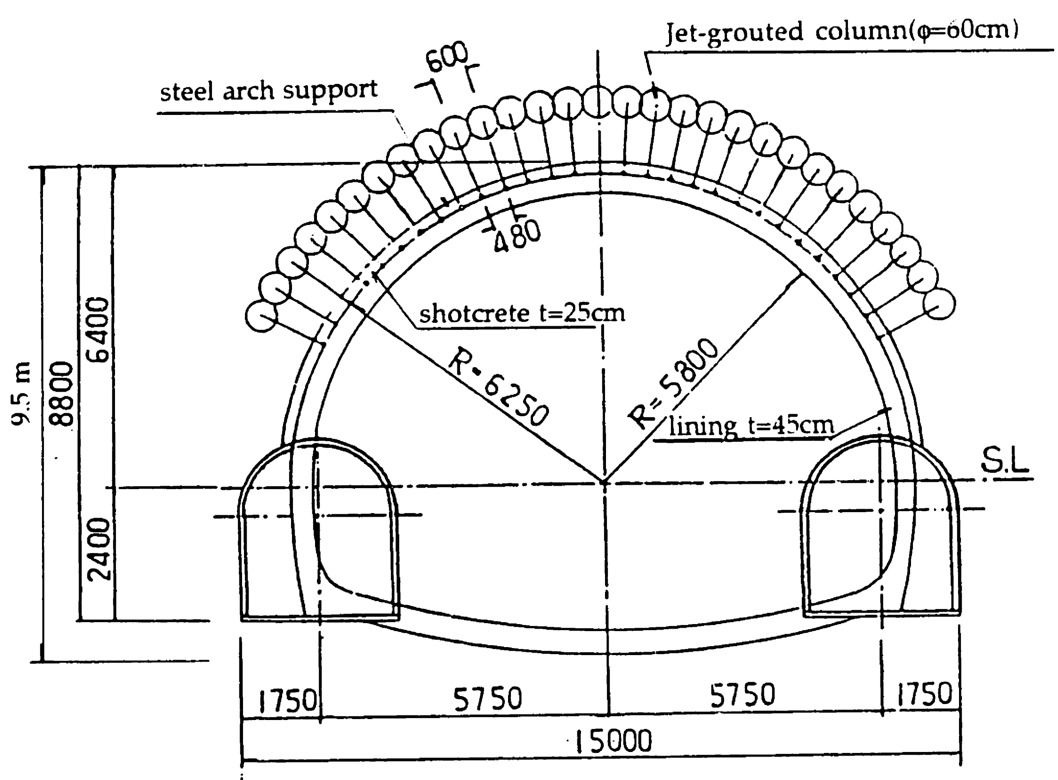
Name of project	Uryuya Tunnel
Time frame	From: 1991/12 To: 1992/2
Location	Sonobe, Kyoto Pref., Japan
Purpose of tunnel	motorway tunnel
Main contractor	Tekken Corporation
Clients	Ministry of Construction
Tunnel dimensions	Width(m): 15 Height(m): 9.5 Length(m): 185 Exc. area(m²): 120
Umbrella method	sub-horizontal jet-grouting; Fore-pole Dia. (cm): 60; Length (m): 10 @ 60 (cm); Number of fore-poles per cross section: 29
Drilling method	machine: SR-510 (Rodio Spa)
Grouting	cement/water grout (w/c=1)
Tunnel support	shotcrete (cm): t=25; steel arch: N/A @ N/A (cm); secondary lining (cm): t=45
Geology/Hydrogeology	Depth of overburden (m): 0 - 40; Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	clay	10	N/A	N/A	N/A
layer-2	weatherd shale	8	N/A	N/A	N/A
layer-3	limestone, slate	>50	N/A	N/A	N/A
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 13

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	7

Cross section



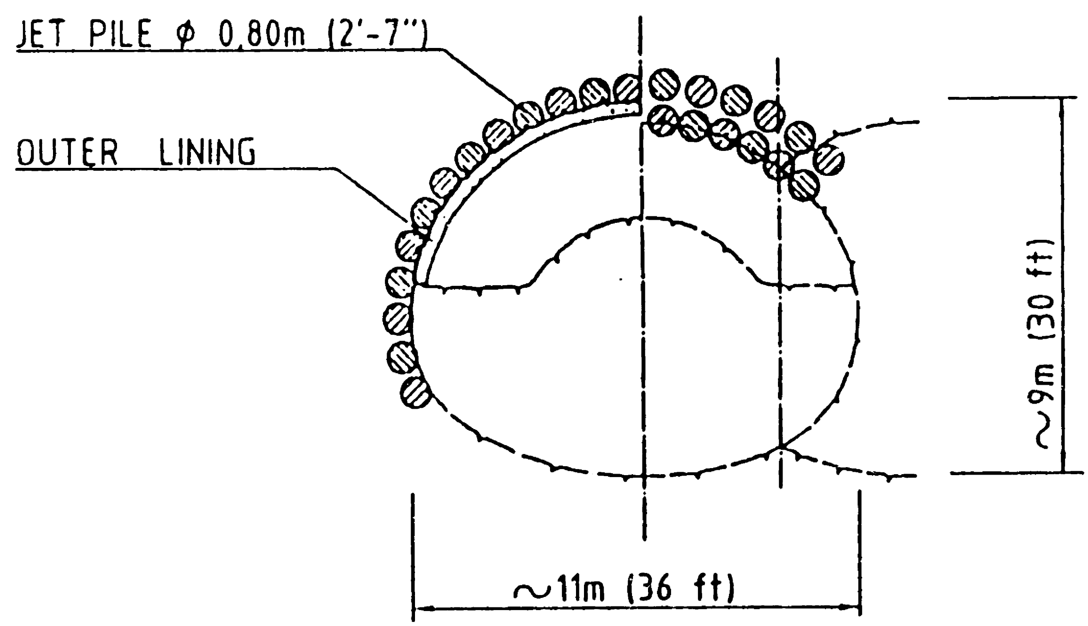
Name of project	Subway Vienna — Time frame From N/A To N/A
Location	Vienna, Austria — Purpose of tunnel: subway station (Rochusgasse Station)
Main contractor	N/A — Clients: N/A
Tunnel dimensions	Width(m): 11 Height(m): 9 Length(m): 65 Exc. area(m^2): 80
Umbrella method	sub-horizontal jet-grouting — Fore-pole Dia. (cm) 80 Length (m) 10-15 @ N/A (cm) Number of fore-poles per cross section: 26
Drilling method	N/A — Grouting: water/cement grout
Tunnel support	shotcrete (cm): t= 30 steel arch: N/A @ N/A (cm) secondary lining (cm): t= N/A
Geology/Hydrogeology	Depth of overburden(m): N/A Groundwater level (m): GL -12 m

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	loam	N/A	N/A	N/A	N/A
layer-2	sandy gravel	N/A	N/A	N/A	N/A
layer-3	silty sand, clayey silt	N/A	N/A	N/A	N/A
layer-4					
layer-5					
layer-6					

Settlement (mm) — Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	<10
tunnel crown:	N/A	N/A	N/A

Cross section



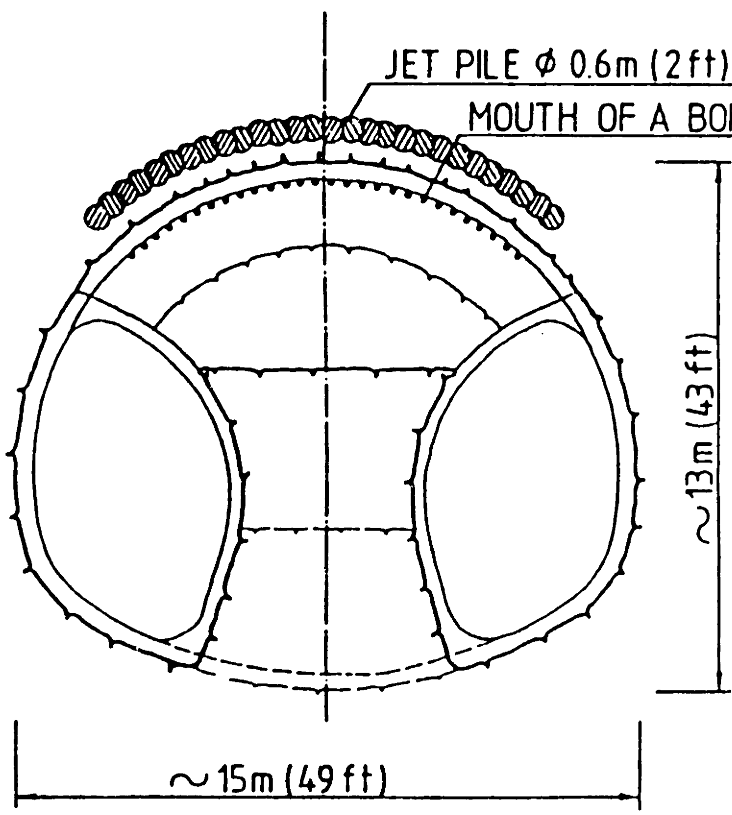
Name of project	Rengershausen tunnel — Time frame From 1985/10/01 To N/A
Location	Kassel, Germany — Purpose of tunnel: railroad tunnel
Main contractor	N/A — Clients: N/A
Tunnel dimensions	Width(m): 15 Height(m): 13 Length(m): 1600 Exc. area(m²): 150
Umbrella method	sub-horizontal jet-grouting Fore-pole Dia. (cm) 60 Length (m) 10.5 @ N/A (cm)
	Number of fore-poles per cross section: 28
Drilling method	N/A — Grouting: N/A
Tunnel support	shotcrete (cm): t= 40 steel arch: N/A @ N/A (cm) secondary lining (cm): t= N/A
Geology/Hydrogeology	Depth of overburden(m): N/A Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	clay & sand	N/A	N/A	10	0
layer-2	fine sand	N/A	N/A	N/A	N/A
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



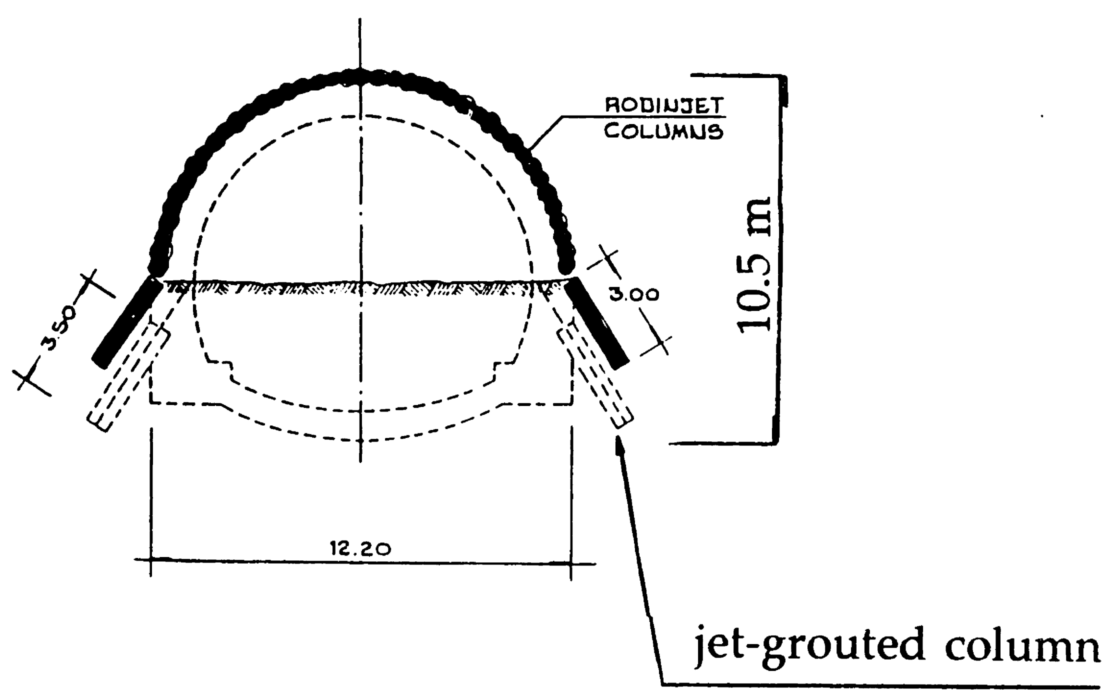
Name of project	Campiolo Tunnel	Time frame From 1983	To 1984		
Location	Moggio Udinese, Italy	Purpose of tunnel	railroad tunnel		
Main contractor	N/A	Clients	N/A		
Tunnel dimensions	Width(m): 12.2	Height(m): 10.5	Length(m): 170	Exc. area(m²): 100	
Umbrella method	sub-horizontal jet-grouting	Fore-pole Dia. (cm) N/A	Length (m) 13	@ 45 (cm)	
	Number of fore-poles per cross section: 41				
Drilling method	Rotary percussion (SR-500, Rodio SpA)	Grouting: N/A			
Tunnel support	shotcrete (cm): t= N/A	steel arch: H-200 @ 100 (cm)	secondary lining (cm): t= N/A		
Geology/Hydrogeology	Depth of overburden(m): 2 - 70	Groundwater level (m): N/A			

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	detrital and cohesionless soil	N/A	N/A	N/A	N/A
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



Name of project Lonato tunnel **Time frame** From N/A To N/A

Location Verona, Italy **Purpose of tunnel** motorway tunnel

Main contractor N/A **Clients** N/A

Tunnel dimensions Width(m): 13.6 Height(m): 10 Length(m): N/A Exc. area(m²): 100 - 120

Umbrella method Reinforced jet-grouting Fore-pole Dia. (cm) 70 Length (m) 12 @ 60 (cm)
Number of fore-poles per cross section: 25 Inserted steel pipes: Φ 114.3mm, t=7 mm

Drilling method N/A **Grouting** N/A

Tunnel support shotcrete (cm): t= 25 steel arch: N/A @ 55 (cm) secondary lining (cm): t= 60 - 130

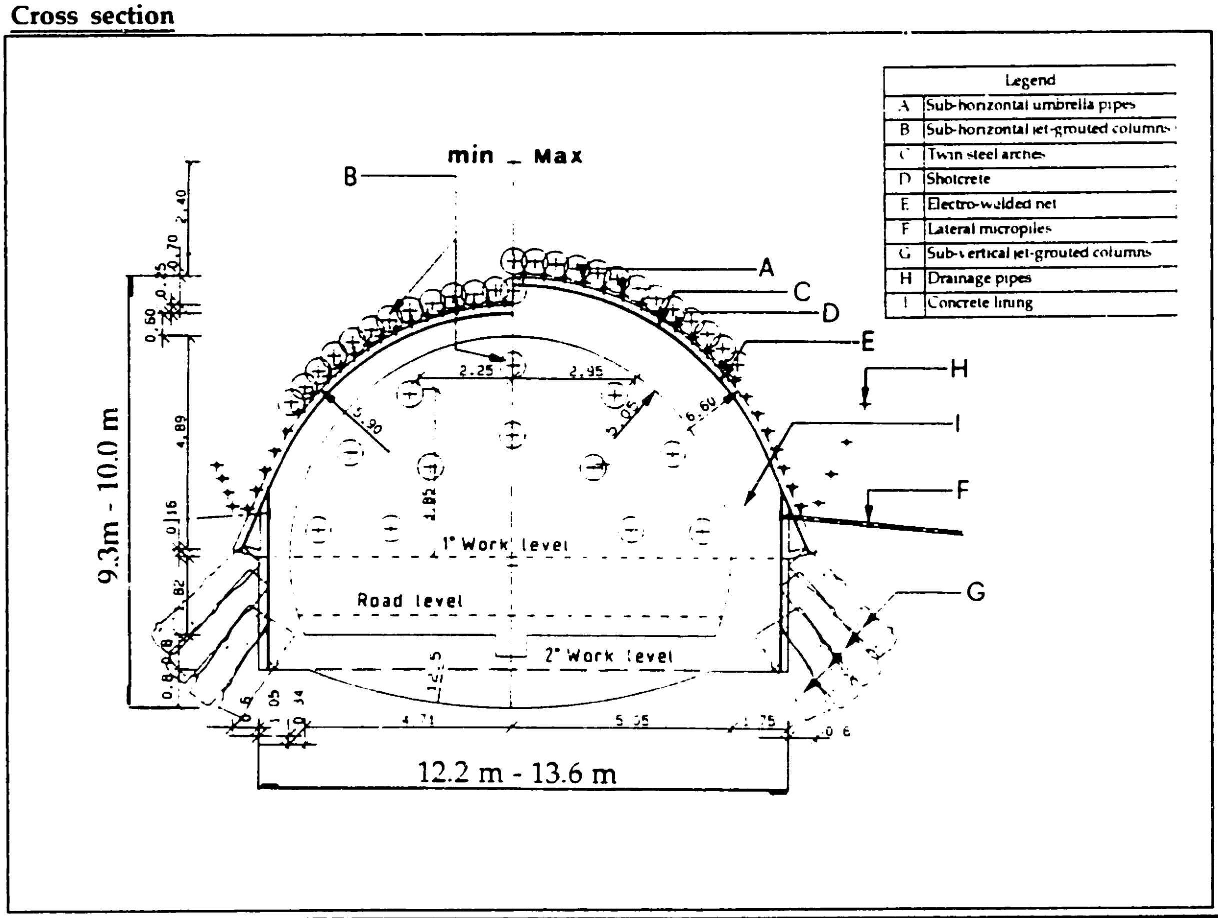
Geology/Hydrogeology Depth of overburden (m): 2 - 70 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	morainic and detrial deposits	N/A	N/A	N/A	N/A
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



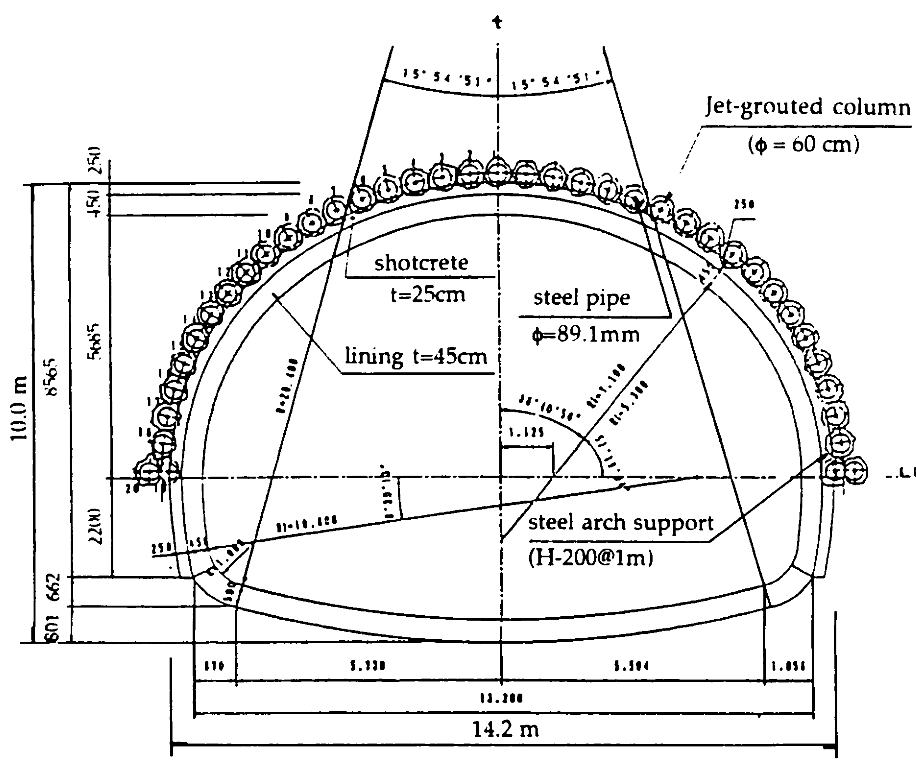
Name of project	Kaziwara No. 1 Tunnel
Time frame	From 1991/8 To 1991/10
Location	Ibaragi, Osaka Pref., Japan
Purpose of tunnel	motorway tunnel
Main contractor	Obayashi Corporation
Clients	Japan Highway Public Corporation
Tunnel dimensions	Width(m): 14.2 Height(m): 10 Length(m): 703 Exc. area(m²): 110
Umbrella method	Reinforced jet-grouting; Fore-pole Dia. (cm) 60; Length (m) 10 @ 60 (cm); Number of fore-poles per cross section: 39; Inserted steel pipes: Φ89.1mm
Drilling method	machine: SR-510 (Rodio SpA)
Grouting	cement/water grout (w/c=1)
Tunnel support	shotcrete (cm): t=25 steel arch: H-200 @ 100 (cm) secondary lining (cm): t=45
Geology/Hydrogeology	Depth of overburden(m): 0 - 12 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	Top soil	N/A	N/A	N/A	N/A
layer-2	talus	20 - 30	N/A	N/A	N/A
layer-3	Sandstone/slate	>50	N/A	N/A	N/A
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 10

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	15	N/A	40
tunnel crown:	N/A	N/A	40

Cross section



Name of project Les Cretes Tunnel **Time frame** From N/A To N/A

Location Aosta, Italy **Purpose of tunnel** two-lane motorway tunnel

Main contractor Follioley & Trevi (JV) **Clients** N/A

Tunnel dimensions Width(m): 12.5 Height(m): 10.4 Length(m): 1536 Exc. area(m²): 105

Umbrella method Reinforced jet-grouting Fore-pole Dia. (cm) 60 Length (m) 12 @ 45 (cm)

Number of fore-poles per cross section: 45 inserted steel pipes: Φ114.3 mm, t=7

Drilling method N/A **Grouting** plugging injection (30 bars)

Tunnel support shotcrete (cm): t= 20 steel arch: 2H-160 @ N/A (cm) secondary lining (cm): t= 50

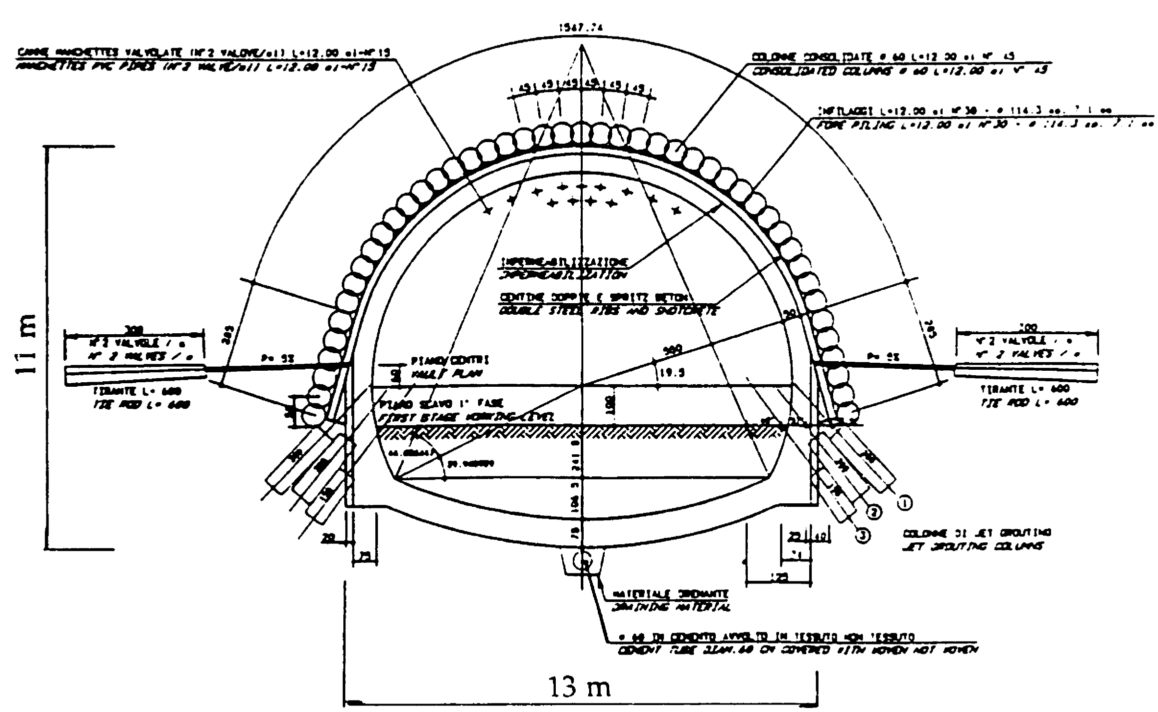
Geology/Hydrogeology Depth of overburden(m): N/A Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	gravel, pebble	N/A	250	34 - 38	N/A
layer-2	silt, clayey silt	N/A	100	24 - 30	10
layer-3	silty sand, gravel	N/A	250	32 - 36	N/A
layer-4	boulder, pebble	N/A	150	24 - 28	10
layer-5	boulder	N/A	350	30 - 36	N/A
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



Name of project Les Cretes Tunnel **Time frame** From N/A To N/A

Location Aosta, Italy **Purpose of tunnel** two-lane motorway tunnel

Main contractor Follioley & Trevi (JV) **Clients** N/A

Tunnel dimensions Width(m): 12.5 Height(m): 10.4 Length(m): 1536 Exc. area(m^2): 105

Umbrella method Injected steel pipe umbrella Fore-pole Dia. (cm) 11.43 Length (m) 12 @ 40 (cm)
Number of fore-poles per cross section: 35 - 60

Drilling method N/A **Grouting:** plugging injection (30 bars)

Tunnel support shotcrete (cm): t= 20 steel arch: 2H-160 @ N/A (cm) secondary lining (cm): t= 50

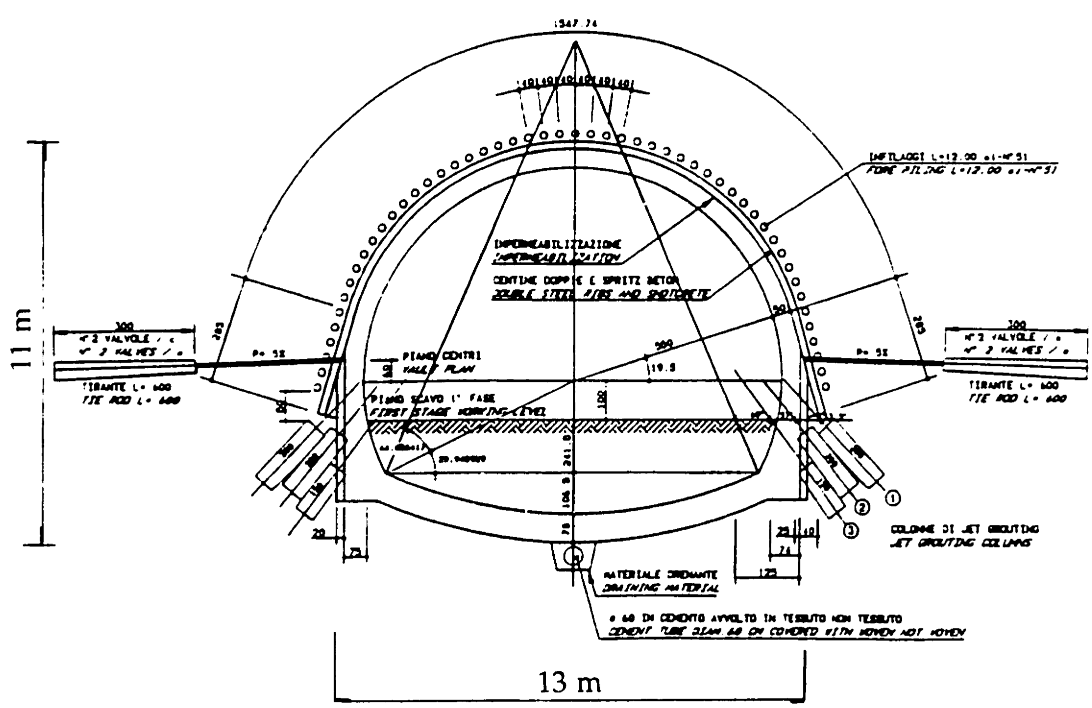
Geology/Hydrogeology Depth of overburden(m): N/A Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	gravel and pebble	N/A	250	34 - 38	N/A
layer-2	silt, clayey silt	N/A	100	24 - 30	10
layer-3	silty sand, gravel	N/A	250	32 - 36	N/A
layer-4	boulder, pebble	N/A	150	24 - 28	10
layer-5	boulder	N/A	350	30 - 36	N/A
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



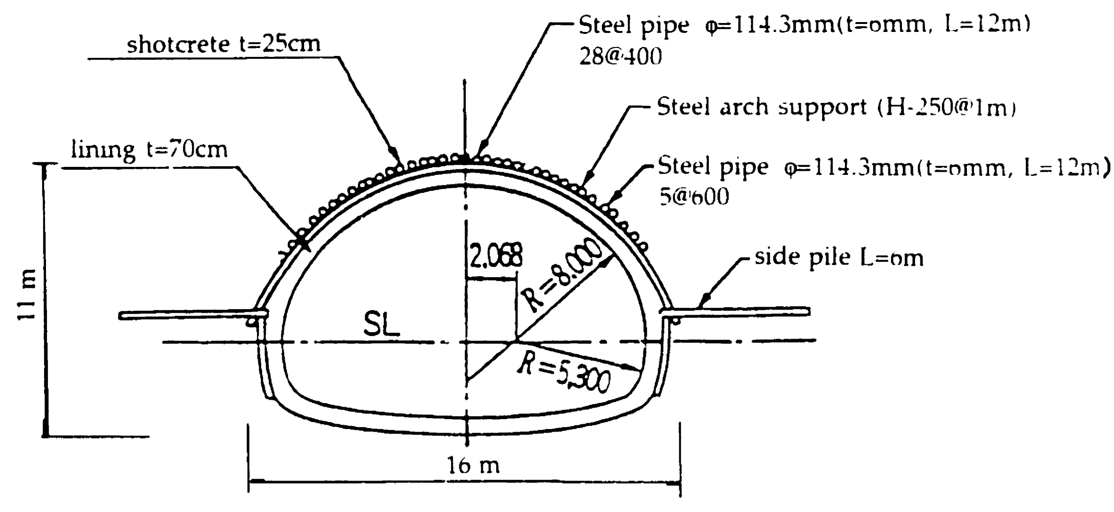
Name of project	Maiko Tunnel (Maikodai section) — Time frame From 1992/12 To 1995/5
Location	Kobe, Hyogo Pref., Japan — Purpose of tunnel: twin 3-lane motorway tunnel
Main contractor	Obayashi Co., Kazima Co. (JV) — Clients: Honshu-Shikoku Bridge Authority
Tunnel dimensions	Width(m): 16 Height(m): 11 Length(m): 3300 Exc. area(m²): 150
Umbrella method	Injected steel pipe umbrella Fore-pole Dia. (cm) 11.43 Length (m) 12 @ 40 (cm)
	Number of fore-poles per cross section: 37 — steel pipes: wall thickness = 6mm
Drilling method	machine: SM505DT (Trevi SpA) Grouting: cement/water
Tunnel support	shotcrete (cm): t= 25 steel arch: H-250 @ 100 (cm) secondary lining (cm): t= 70
Geology/Hydrogeology	Depth of overburden(m): 10 - 20 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	clay	N/A	N/A	N/A	N/A
layer-2	gravel	40 - 50	N/A	N/A	N/A
layer-3	clay	N/A	N/A	N/A	N/A
layer-4	gravel	>40	N/A	N/A	N/A
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



Name of project	Maiko Tunnel (Fukuda j-h school section)	Time frame From 1995/6	To in progress
Location	Kobe, Hyogo Pref., Japan	Purpose of tunnel	twin 3-lane motorway tunnel
Main contractor	Taisei & Nishimatsu (JV)	Clients	Honsyu-Shikoku Bridge Authority

Tunnel dimensions Width(m): 16 Height(m): 11 Length(m): 3300 Exc. area(m²): 150

Umbrella method Injected steel pipe umbrella Fore-pole Dia. (cm) 11.43 Length (m) 12 @ 40 (cm)
Number of fore-poles per cross section: 42 steel pipes: wall thickness=6mm

Drilling method machine: SM605-DT(Trevi SpA) Grouting: ultra fine cement/water grout

Tunnel support shotcrete (cm): t=25 steel arch: H-200 @ 100 (cm) secondary lining (cm): t=45

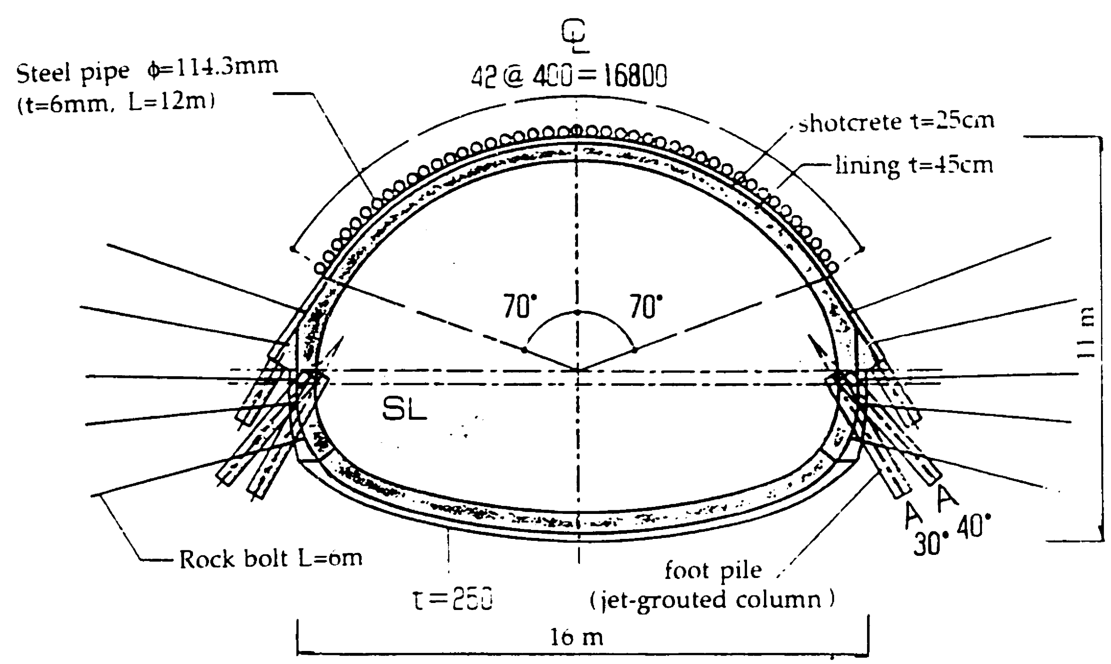
Geology/Hydrogeology Depth of overburden(m): 4 - 7 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	fill (gravel with clay)	5 - 40	0.5 - 4	N/A	N/A
layer-2	alluvial gravel	3 - 7	0.5 - 30	N/A	N/A
layer-3	volcanic gravel	N/A	N/A	N/A	N/A
layer-4	gravel	20 - 70	38 - 230	N/A	N/A
layer-5	gravel	30 - 60	20 - 50	N/A	N/A
layer-6	clay	28 - 60	10 - 60	N/A	N/A

Settlement (mm) Depth of overburden at the measuring point (m): 6

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	90	122	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



Name of project Hirai Tunnel **Time frame** From 1992/4 To 1993/5

Location Miki, Japan **Purpose of tunnel** 3-lane motorway tunnel

Main contractor Maeda Co. & Sato Co. (JV) **Clients** Japan Highway Public Corporation

Tunnel dimensions Width(m): 15.5 Height(m): 10 Length(m): 800 Exc. area(m²): 130

Umbrella method Injected steel pipe umbrella Fore-pole Dia. (cm): 6.05 Length (m): 3 @ N/A (cm)
Number of fore-poles per cross section: 53

Drilling method Drill jumbo (THMJ2400) **Grouting:** N/A

Tunnel support shotcrete (cm): t= 25 steel arch: H-200 @ 75 (cm) secondary lining (cm): t= 45

Geology/Hydrogeology Depth of overburden(m): 0 - 25 Groundwater level (m): N/A

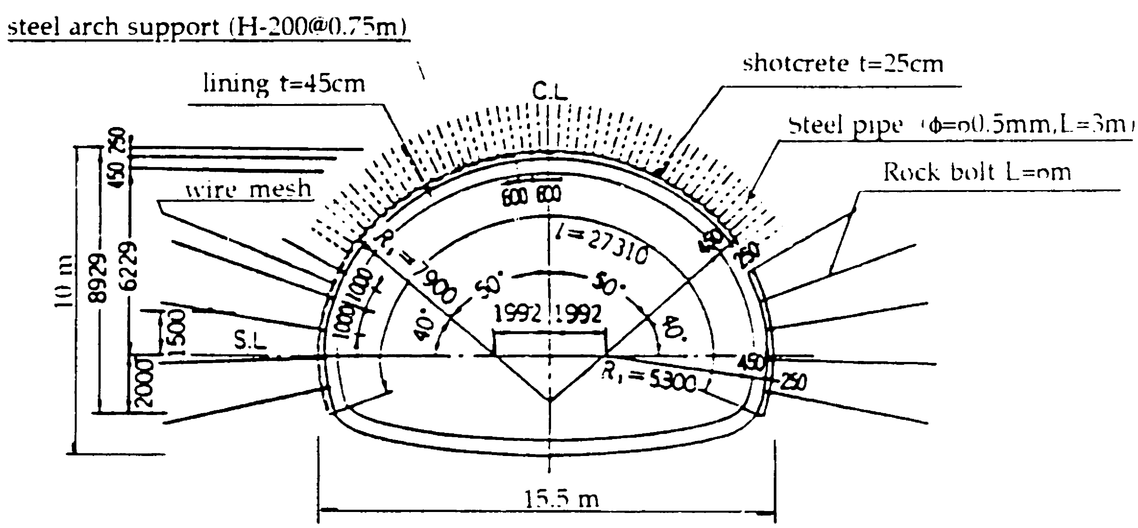
	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	fill (soil cement)	>50	94.1	N/A	N/A
layer-2	clayey gravel - 1	10 - 30	19.7	N/A	N/A
layer-3	clayey gravel - 2	30 - 50	82.4 - 122	N/A	N/A
layer-4	clayey gravel - 3	> 50	N/A	N/A	N/A
layer-5	sandstone, tuff	>70	N/A	N/A	N/A
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 7

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	16	40	49
tunnel crown:	-	24	35

Tunnel crown settlement was measured after primary supports were installed.

Cross section



Name of project	Futatsui-Nishi tunnel	Time frame From	N/A To N/A
Location	Futatsui, Japan	Purpose of tunnel	2-lane motorway tunnel
Main contractor	Goyo Corporation	Clients	Ministry of Construction

Tunnel dimensions Width(m): 12 Height(m): 9 Length(m): 644 Exc. area(m²): 90

Umbrella method Injected steel pipe umbrella Fore-pole Dia. (cm) 10.16 Length (m) 8.5,11.5 @ 30 (cm)
Number of fore-poles per cross section: 45 steel pipes: wall thickness = 4.2 mm

Drilling method Drill junbo (THCJ- 2400) Grouting: water/cement/bentonite grout

Tunnel support shotcrete (cm): t= 25 steel arch: H-200 @ 100 (cm) secondary lining (cm): t= 35

Geology/Hydrogeology Depth of overburden(m): 0 - 9 Groundwater level (m): N/A

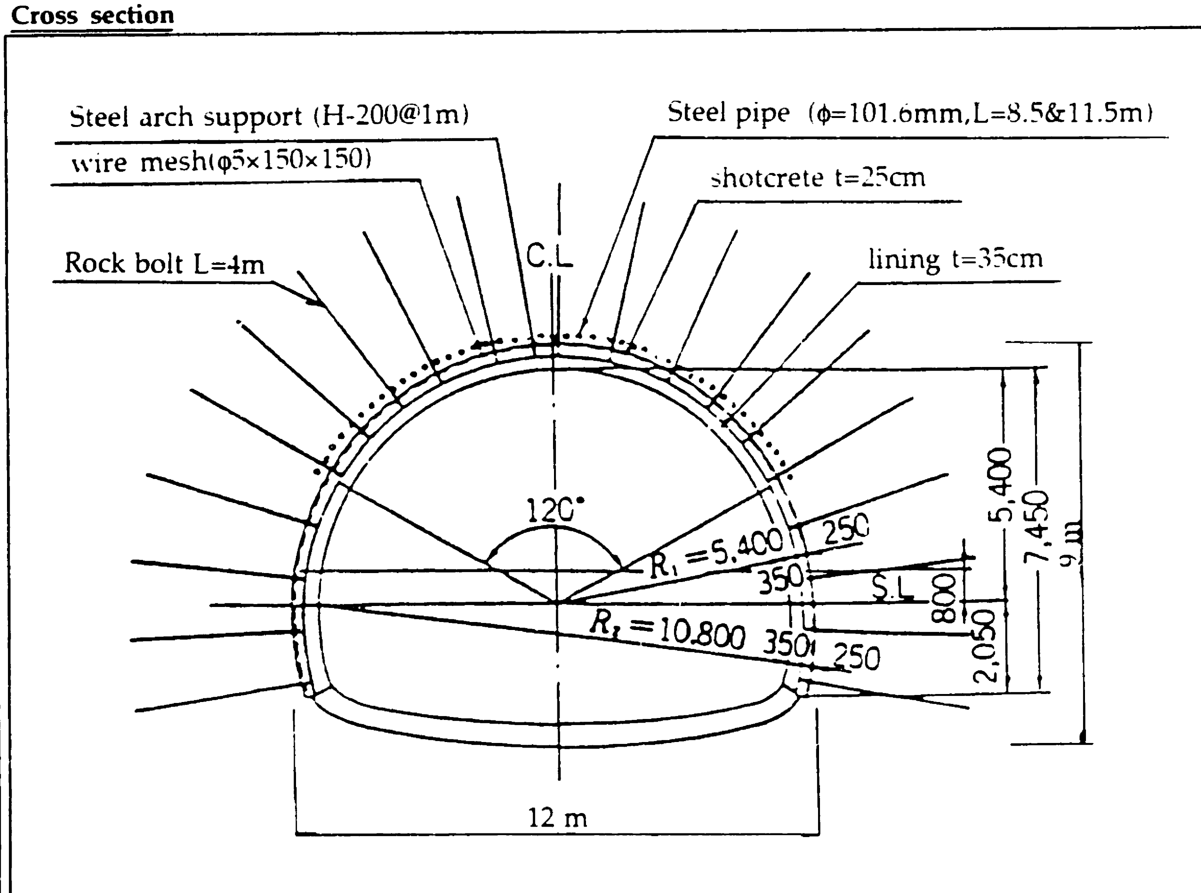
	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	talus	N/A	N/A	N/A	N/A
layer-2	tuffaceous sandstone	N/A	N/A	N/A	N/A
layer-3	mudstone	N/A	N/A	N/A	N/A
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 7

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	25	30	33
tunnel crown:	-	5	N/A

Tunnel crown settlement was measured after primary supports were installed.

Cross section



Name of project	Yakiyama tunnel
Time frame	From N/A To N/A
Location	Niigata Pref., Japan
Purpose of tunnel	2-lane motorway tunnel
Main contractor	Toyo Construction Company
Clients	Japan Highway Public Corporation
Tunnel dimensions	Width(m): 12 Height(m): 9.5 Length(m): 2986 Exc. area(m²): 100
Umbrella method	Injected steel pipe umbrella; Fore-pole Dia. (cm) 11.43; Length (m) 15, 18 @ 40 (cm); Number of fore-poles per cross section: 33; steel pipes: wall thickness = 6mm
Drilling method	down-the-hole hammer with TUBEX
Grouting	cement/water grout
Tunnel support	shotcrete (cm): t=25 steel arch: H-200 @ N/A (cm) secondary lining (cm): t=35
Geology/Hydrogeology	Depth of overburden(m): 0 - 9 Groundwater level (m): N/A

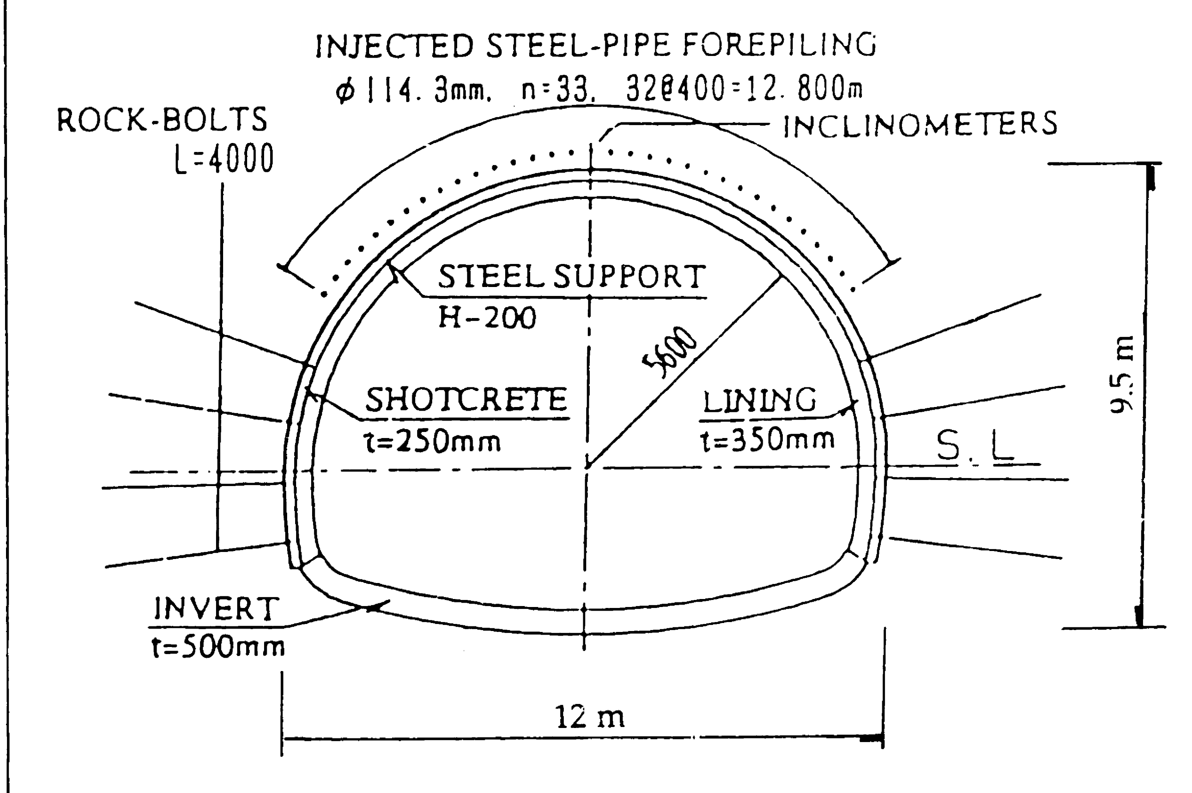
	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	surface soil	N/A	N/A	N/A	N/A
layer-2	talus deposits	N/A	N/A	N/A	N/A
layer-3	weathered sandstone	N/A	N/A	N/A	N/A
layer-4	sandstone	N/A	N/A	N/A	N/A
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 5

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	12	17	N/A
tunnel crown:	10	17	N/A

Settlements of crown-steel pipe was substituted for tunnel crown settlements.

Cross section



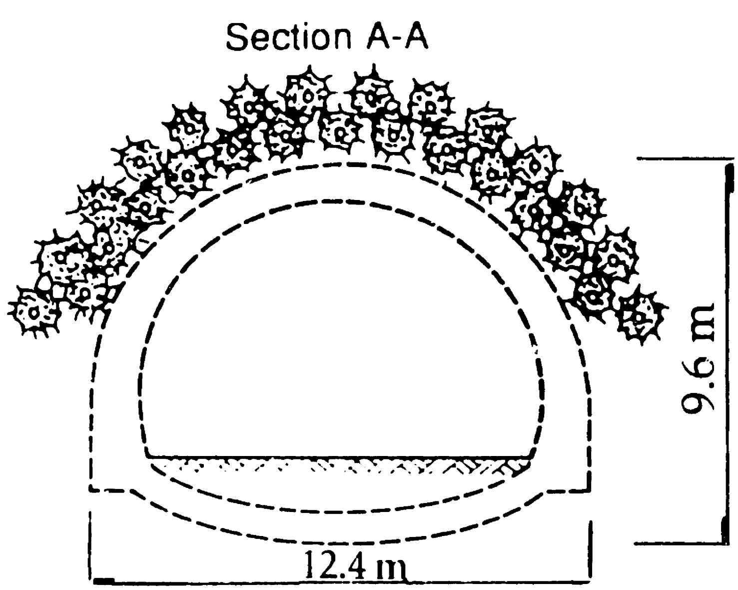
Name of project	Ramat tunnel
Time frame	From N/A To N/A
Location	Piedmont, Italy
Purpose of tunnel	2-lane motorway tunnel
Main contractor	N/A
Clients	N/A
Tunnel dimensions	Width(m): 12.4 Height(m): 9.6 Length(m): 500 Exc. area(m^2): 90
Umbrella method	Injected steel pipe umbrella Fore-pole Dia. (cm) 8.89 Length (m) 12 @ N/. (cm)
	Number of fore-poles per cross section: 33 - 45
Drilling method	Odex bit + down-the- hole hammer
Grouting	water/cement (w/c=1.0)
Tunnel support	shotcrete (cm): t= 30 steel arch: I-180 @ 100 (cm) secondary lining (cm): t= N/A
Geology/Hydrogeology	Depth of overburden(m): 2 - 4 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	moraine (sand, gravel, boulder)	N/A	N/A	N/A	N/A
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface	N/A	N/A	N/A
tunnel crown	N/A	N/A	N/A

Cross section



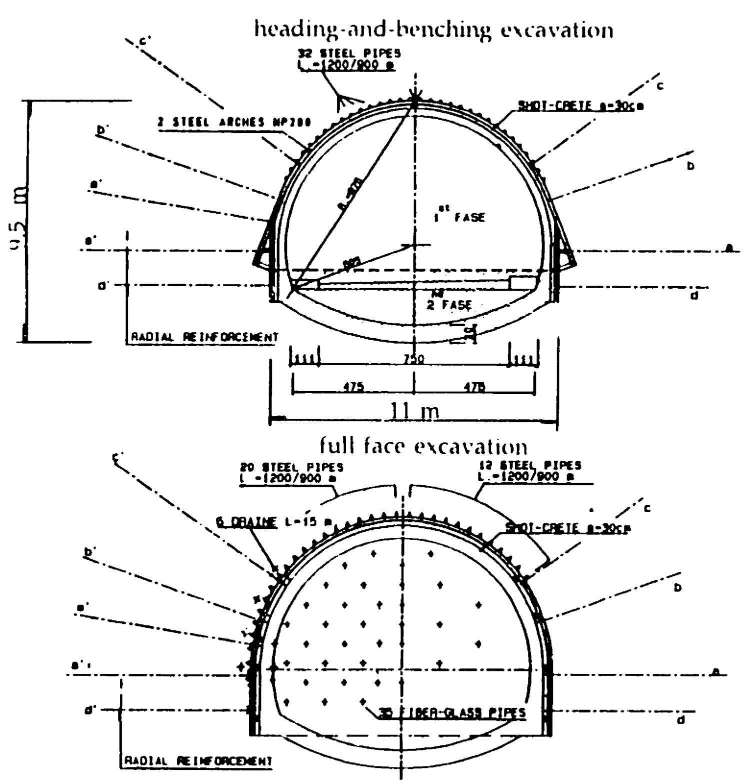
Name of project	Poggio Fornello Tunnel
Time frame	From N/A To N/A
Location	Tuscany, Italy
Purpose of tunnel	motorway tunnel
Main contractor	N/A
Clients	N/A
Tunnel dimensions	Width(m): 11 Height(m): 9.5 Length(m): 550 Exc. area(m²): 104
Umbrella method	Injected steel pipe umbrella Fore-pole Dia. (cm) N/A Length (m) 12 @ N/A (cm)
	Number of fore-poles per cross section: 20, 32
Drilling method	N/A
Grouting	water/cement grout
Tunnel support	shotcrete (cm): t= 30 steel arch: 2T-200 @ 75 (cm) secondary lining (cm): t= N/A
Geology/Hydrogeology	Depth of overburden(m): 50 (max.) Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	flysch	N/A	2000	20	50
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



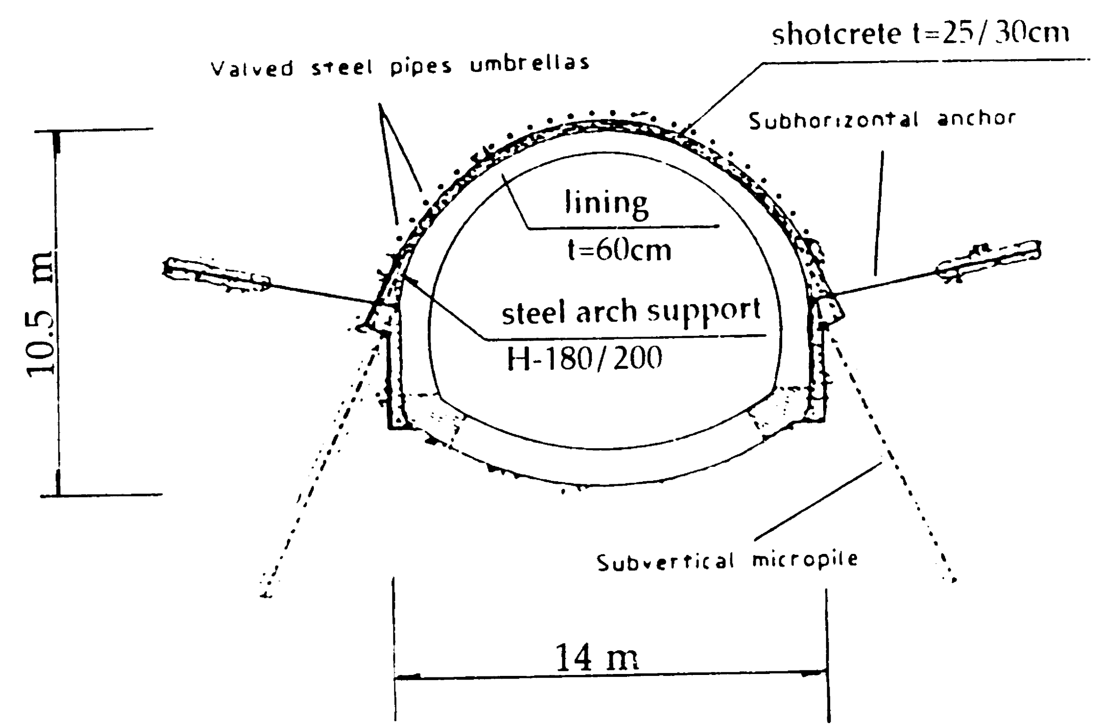
Name of project	St. Ambrogio Tunnel
Time frame	From N/A To N/A
Location	Sicily, Italy
Purpose of tunnel	motorway tunnel
Main contractor	N/A
Clients	N/A
Tunnel dimensions	Width(m): 14 Height(m): 10.5 Length(m): 1780 Exc. area(m²): 110
Umbrella method	Injected steel pipe umbrella; Fore-pole Dia. (cm): N/A; Length (m): 12 - 16 @ N/A (cm); Number of fore-poles per cross section: 24
Drilling method	N/A
Grouting	water/cement grout
Tunnel support	shotcrete (cm): t= 25 - 30 steel arch: 2H-180 @ N/A (cm) secondary lining (cm): t= 60
Geology/Hydrogeology	Depth of overburden(m): 200 (max.) Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	flysch	N/A	N/A	N/A	N/A
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	<100
tunnel crown:	N/A	N/A	N/A

Cross section



Name of project: Nango Tunnel **Time frame** From 1995/2 To 1995/9

Location: Hayama, Kanagawa Pref., Japan **Purpose of tunnel:** motorway tunnel

Main contractor: Shimizu Corporation **Clients:** Kanagawa Prefecture

Tunnel dimensions: Width(m): 15 Height(m): 10 Length(m): 890 Exc. area(m²): 110

Umbrella method: Injected steel pipe umbrella Fore-pole Dia. (cm): 11.43 Length (m): 12 @ 30, 40 (cm)
Number of fore-poles per cross section: 43 steel pipes: wall thickness =11 mm

Drilling method: Double rotary (SM505DT, Trevi SpA) **Grouting:** high early cement/water grout (w/c=1.33)

Tunnel support: shotcrete (cm): t=25 steel arch: H-200 @ 100 (cm) secondary lining (cm): t=40

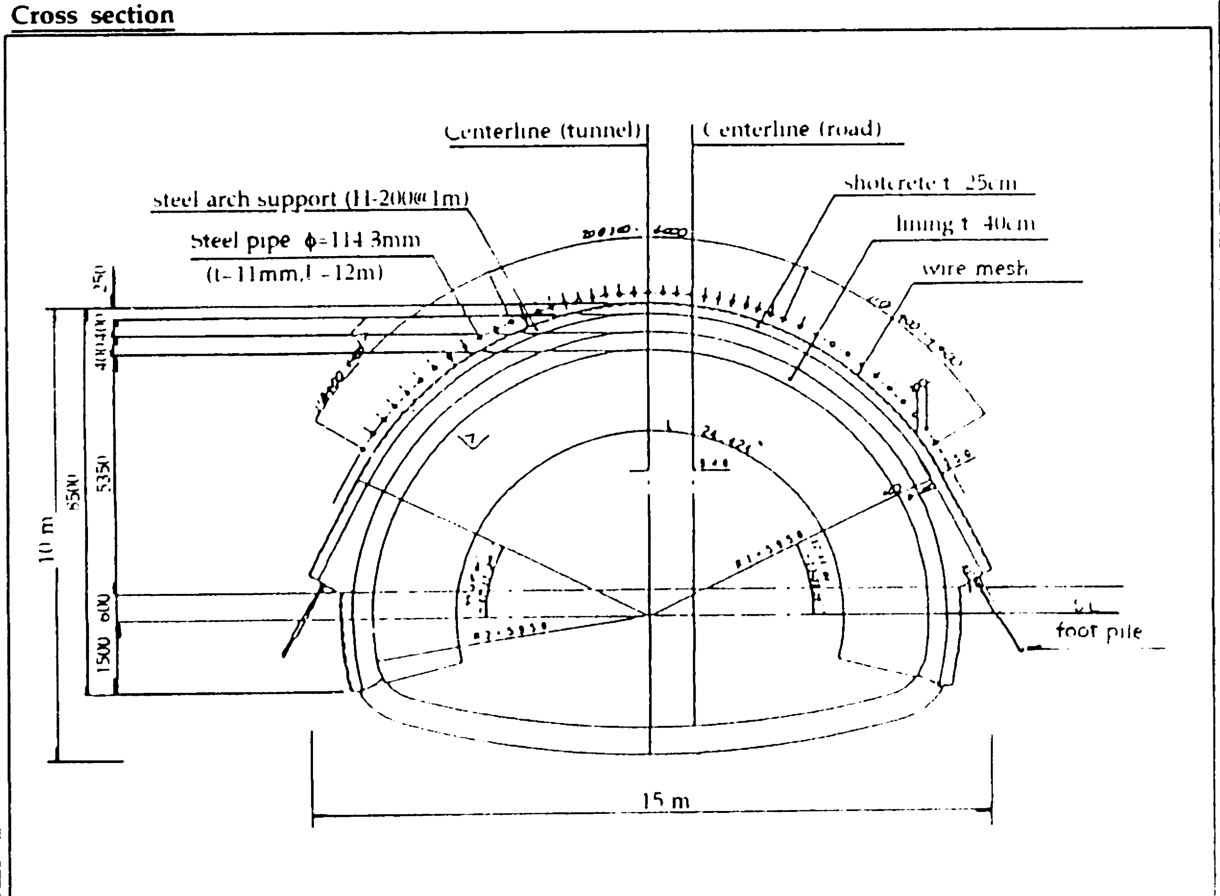
Geology/Hydrogeology: Depth of overburden(m): 3 - 21 Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	fill (clay, mudstone block)	N/A	N/A	N/A	N/A
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



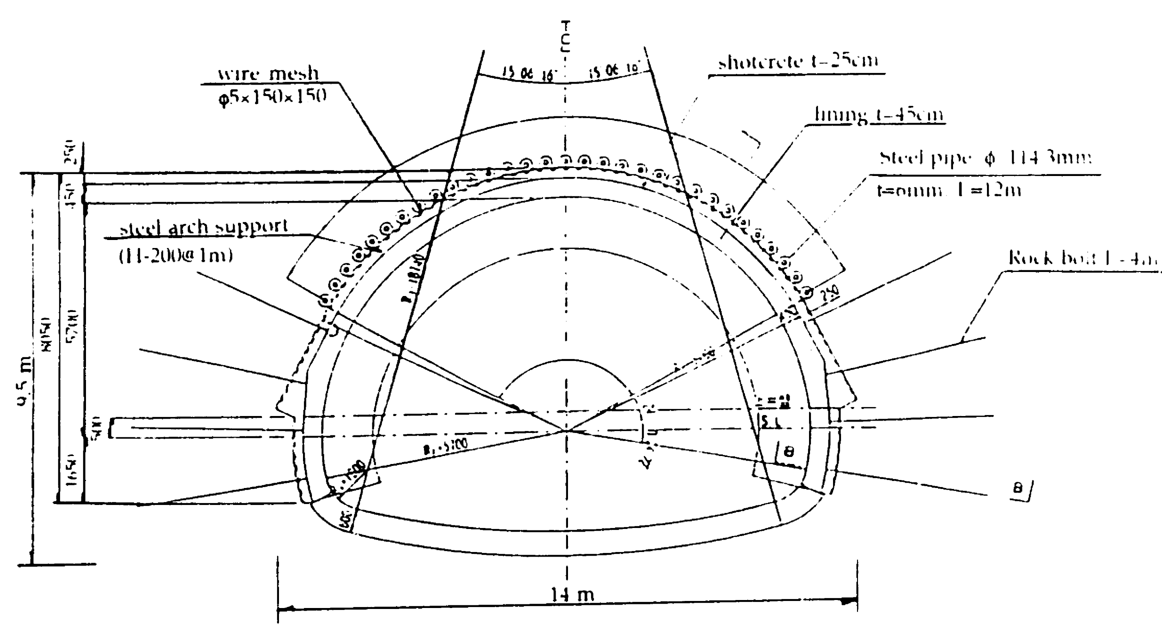
Name of project	Kubodaira Tunnel
Time frame	From 1995/2 To 1995/9
Location	Shioyama, Yamanashi Pref., Japan
Purpose of tunnel	motorway tunnel
Main contractor	Aoki Corporation
Clients	Yamanashi Prefecture
Tunnel dimensions	Width(m): 13 Height(m): 9.5 Length(m): 680 Exc. area(m²): 105
Umbrella method	Injected steel pipe umbrella; Fore-pole Dia. (cm) 11.43; Length (m) 12 @ 45 (cm); Number of fore-poles per cross section: 31; steel pipes: wall thickness=6 mm
Drilling method	down the hole hammer
Grouting	urethane grout
Tunnel support	shotcrete (cm): t= 25; steel arch: H-200 @ 100 (cm); secondary lining (cm): t= 45
Geology/Hydrogeology	Depth of overburden(m): mean 5 m; Groundwater level (m): N/A

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	river bed deposits (boulder, cobble)	N/A	N/A	N/A	N/A
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



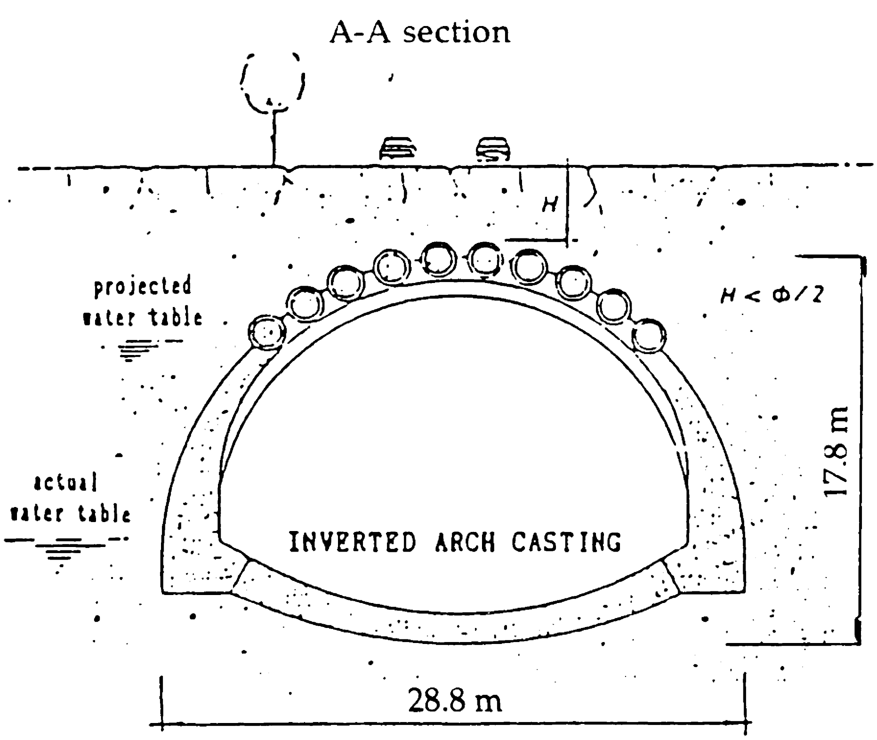
Name of project	Venezia Station
Time frame	From 1987 To 1992
Location	Milan, Italy
Purpose of tunnel	subway station
Main contractor	Consorzio GIEMME SpA
Clients	Region of Lombardy
Tunnel dimensions	Width(m): 28.8 Height(m): 17.8 Length(m): 214.5 Exc. area(m²): 440
Umbrella method	pipe roof method; Fore-pole Dia. (cm) 210; Length (m) 2 @ N/A (cm); Number of fore-poles per cross section: 10; RC pipe: wall thickness = 15 cm
Drilling method	microtunnelling (shield machine)
Grouting	concrete
Tunnel support	shotcrete (cm): t= --- steel arch: --- @ --- (cm) secondary lining (cm): t= ---
Geology/Hydrogeology	Depth of overburden(m): 4 - 5 Groundwater level (m): N/A

	ground type	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	alluvial cohesionless deposits	N/A	N/A	35	N/A
layer-2					
layer-3					
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 4

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	---	---	14
tunnel crown:	---	---	N/A

Cross section



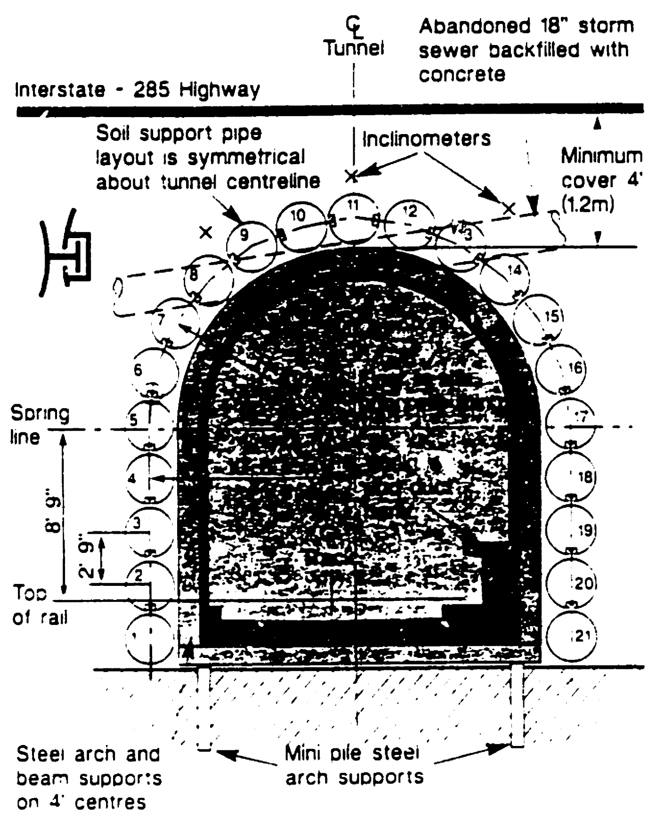
Name of project	MARTA East Line Underpass Tunnel	Time frame	From 91/1/1	To 91/9/5	
Location	Atlanta, US	Purpose of tunnel	Subway tunnel unedr I-285 highway		
Main contractor	Archer Western Co., SEC	Clients	Atlanta Rapid Transit Authority		
Tunnel dimensions	Width(m): 6.3	Height(m): 7.2	Length(m): 54	Exc. area(m²): 35	
Umbrella method	pipe roof method	Fore-pole Dia. (cm): 75	Length (m): 6	@ N/A (cm)	
	Number of fore-poles per cross section: 21				
Drilling method	microtunnellling (shield machine)	Grouting: cement grout			
Tunnel support	shotcrete (cm): t= None	steel arch: N/A	@ 120 (cm)	secondary lining (cm): t= 30.8	
Geology/Hydrogeology	Depth of overburden (m): 1.5 - 3.25	Groundwater level (m): N/A			

	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	fill	N/A	N/A	27	0
layer-2	silty sand sandy silt	N/A	N/A	29	0
layer-3	weatherd rock	N/A	N/A	N/A	N/A
layer-4					
layer-5					
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): N/A

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	N/A	N/A	N/A
tunnel crown:	N/A	N/A	N/A

Cross section



Name of project	Yokohama Subway No. 3 Line
Time frame	From N/A To N/A
Location	Yokohama, Kanagawa Pref., Japan
Purpose of tunnel	subway tunnel
Main contractor	N/A
Clients	Yokohama city
Tunnel dimensions	Width(m): 10.2 Height(m): 8.5 Length(m): 830 Exc. area(m²): 66 - 74 (mean 70)
Umbrella method	pipe roof method; Fore-pole Dia. (cm): 60.96; Length (m): 4.5, 10 @ 67.5 (cm); Number of fore-poles per cross section: 18; steel pipes: wall thickness = 9 mm
Drilling method	Jacking and boring method
Grouting	concrete
Tunnel support	shotcrete (cm): t=20 steel arch: H-200 @ 100 (cm) secondary lining (cm): t=55
Geology/Hydrogeology	Depth of overburden(m): 6 - 20 Groundwater level (m): N/A

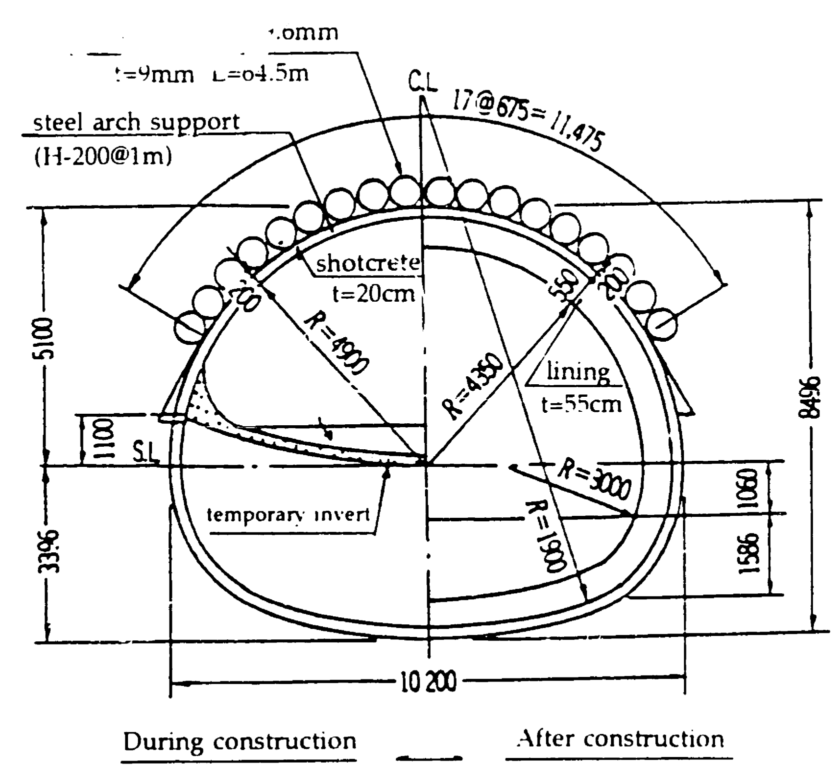
	groundtype	SPT-N	E(MPa)	Φ(deg)	C(kPa)
layer-1	loam	2 - 12	N/A	18.1 - 18.5	39 - 50
layer-2	clay	5 - 22	19.2 - 20.7	0 - 16.5	58 - 200
layer-3	sand	8 - 50	N/A	35.5	N/A
layer-4	mudstone	>50	210 - 1100	20 - 25.3	880 - 1780
layer-5	sand	>50	300 - 327	N/A	N/A
layer-6					

Settlement (mm) Depth of overburden at the measuring point (m): 7.9

location	pre-excavation	after excavation of top heading	final settlement
ground surface:	7	N/A	9.2
tunnel crown:	N/A	N/A	N/A

Settlement of 6 - 7 mm occurred before excavation of top heading.

Cross section



List of References

Adachi, T., "Deformation and Earth Pressure in Deep Underground Space," Proceedings of Forum de Geotechnical Problems on Urban Underground Space, Japanese Society of Soil Mechanics and Foundation Engineering(JSSMFE), 1992.

Ae, S., Ito, K., "Supplementary Methods in the Kaziwara No. 1 Tunnel," Technical Report on the RJFP Method, Geo-Fronte Research Association, January, 1994.

American Society of Civil Engineers (ASCE), Verification of Geotechnical Grouting, Geotechnical Special Publication No. 57, Byle, M.J., Borden, R.H. (eds.), New York, 1995.

Atta-alla, A.L., Iseley, D.T., "Pipe Arch Horizontal Drift Method for MARTA's Transit Extension under I-285," Proceedings of Rapid Excavation and Tunneling Conference (RETC), Vol. 1, 1991.

Bickel, J.O., Kuesel, T.R., King, E.H., Tunnel Engineering Handbook (second edition), Chapman and Hall, New York, 1996.

Blakita, P.M., Cavey, J.K., "Rest in Peace," Civil Engineering, December, 1995.

Bruce, D.A., Boley, D.L., Gallavresi, F., "New Developments in Ground Reinforcement and Treatment for Tunnelling," Proceedings of Rapid Excavation and Tunnelling Conference (RETC), Vol. 2, 1987.

Bulson, P.S., Buried Structures: Static and Dynamic Strength, Chapman and Hall, New York, 1985.

Chugh, C.P., Manual of Drilling Technology, Balkema, Rotterdam, 1985.

Geo-Fronte Research Association, Technical Report on the Rodin Jet Fore-Poling (RJFP) Method, January, 1994.

Geo-Fronte Research Association, Achievements of the TREVITUB Method, December, 1995.

Geo-Fronte Research Association, Recommendations for Choice of the Umbrella Method, December, 1995.

Henn, R.W., Practical Guide to Grouting of Underground Structures, American Society of Civil Engineers (ASCE), 1996

Ito, J., Sano, N., et al, "Approach for Design Method of the Umbrella Method," 1995.

Japan Tunnelling Association (JTA), "Challenges and Changes: Tunnelling Activities in Japan 1994," Tunnelling and Underground Space Technology, International Tunnelling Association (ITA), Vol. 10, No. 2, 1995.

Kizima, Y., Aoki, T. et al, "Tunnel Excavation by NATM in Urban Areas: Kokubugawa Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), July, 1989.

Koizumi, M., Imamura, O., "Construction of 3-lane Tunnel with Shallow Earth Covering: Shoryou No. 1 Tunnel on Tokyo-Nagoya Expressway- (Part-2)," Tunnels and Underground, Japan Tunnelling Association (JTA), June, 1990.

Kotake, N., Yamamoto, Y., Oka, K., "Design for Umbrella Method Based on Numerical Analyses and Field Measurements," Tunnelling and Ground Conditions, Abdel Salam (ed.), Balkema, Rotterdam, 1994.

Kotake, N., Yamamura, K., Manabe, T., "A Case History of Portal Excavation with Injected steel-pipe Fore-poling," Proceedings of the South East Asian Symposium on Tunnelling and Underground Space Development, Japan Tunneling Association (JTA), 1995.

Lo, K.Y., Ng, M.C., Rowe, R.K., "Predicting Settlement due to Tunnelling in Clays," GEOTECH 84, Lo, K.Y. (ed.), American Society of Civil Engineers (ASCE), 1984.

Lunardi, P., "The Cellular Arch Method: Technical Solution for the Construction of the Milan Railway's Venezia Station," Tunnelling and Underground Space Technology, International Tunnelling Association (ITA), 1990.

Lunardi, P., "Cellular Arch Technique for Large Span Station Cavern," Tunnels and Tunnelling, Miller Freeman, November, 1991.

Lunardi, P., Colombo, A., Pizzarotti, E.M., "Performance Observations during Construction of the Large Span Milan Metro Station," International Congress "Option for Tunnelling," 1993.

Mahtab, M.A., Grasso, P., Geomechanics Principles in the Design of Tunnels and Caverns in Rocks, Elsevier, Amsterdam, 1992.

Mongilardi, E., Tornaghi, R., "Construction of Large Underground Openings and Use of Grouts," Proceedings of International Conference on Deep Foundations, Vol. 1, 1986.

Moseley, M.P., Ground Improvement, Chapman and Hall, New York, 1993.

Mussger K., Koining J., Reischl St., "Jet Grouting in Combination with NATM," Proceedings of Rapid Excavation and Tunneling Conference (RETC), Vol. 1, 1987.

Ogino, Y., Watanabe, M., Hikabe, S., "Construction Report on the Yokohama Subway No. 3 Line (Kitanoyato Site)," Tunnels and Underground, Japan Tunnelling Association (JTA), December, 1991.

Okazawa, T., Hamamura, Y., Fujii, T., "Construction of a Large Cross-section Tunnel with the Umbrella Method under Residential Areas: Maiko Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), February, 1996.

Pagliacci, F., Yamamoto, M., "New Construction Methods for Tunnels in Difficult Soils: Les Cretes Tunnel," In-house Document of Trevi SpA, Cesena, Italy, 1993.

Peck, R.B., "Deep Excavation and Tunnelling in Soft Ground," Proceedings of 7th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1969.

Pelizza, S., Barisone, G., Campo, F., "Neolithic Site Kept Safe under Italian Umbrella," Tunnels and Tunnelling , Miller Freeman, March, 1990.

Pelizza, S., Gangale, G., Corona, G., "The Messina-Palermo Motorway: Complex Rock Masses and Tunnelling Problems", Towards New Worlds in Tunnelling, Vieitez-Utesa and Montanez-Cartaxo (eds.), Balkema, Rotterdam, 1992.

Pelizza, S., Peila, D., "Soil and Rock Reinforcements in Tunnelling," Tunnelling and Underground Space Technology, International Tunnelling Association (ITA), Vol. 8, No. 3, 1993.

Pelizza, S., Corona, G., Graffi, G., Grasso, F., Raineri, R., "Improvement of Stability Conditions from Half to Full Face Excavation in Difficult Geotechnical Conditions," Tunnelling and Ground Conditions, Abdel Salam (ed.), Balkema, Rotterdam, 1994.

Saito, T., Pipe Roof Method in Tunnelling, Riko-Tosho, Tokyo, 1982.

Sakayama, Y., Igarashi, M., "Construction Report on the Hirai Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), June, 1993.

Sakurai, S., Adachi, T., NATM in Urban Tunnelling, Kashima-Shuppankai, Tokyo, 1988.

Shimizu, A., Takahashi, Y. et al, "Conquest of Soft Sedimentary Loam at a Tunnel Portal: Aziro Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), December, 1991.

Stein, D., Mollers, K., Bielecki, R., Microtunnelling: Installation and Renewal of Nonman-Size Supply and Sewage Lines by the Trenchless Construction Method, Ernst and Sohn, Berlin, 1989.

Szechy, K., The Art of Tunnelling, Akademiai Kaido, Budapest, 1966.

Taisei Corporation, " Development of Design Method for a Supplementary Method in a Large Cross-section Tunnel," In-house Document of Taisei Corporation, Tokyo, Japan, 1996.

Terzaghi, K., Theoretical Soil Mechanics, John Wiley and Sons, Inc., New York, 1943.

Tornaghi, R., Perelli Cippo, A., "Soil Improvement by Jet Grouting for the Solution of Tunnelling Problems," Proceedings of the 4th International Symposium Tunnelling '85, 1985.

Trevi SpA, "Tunnel in Italy: Traditional and New Technology," In-house Document of Trevi SpA, Cesena, Italy, 1992.

Trevi SpA, "Reinforced Protective Umbrella Method for Tunnel Excavation: Illustrative Report," In-house Document of Trevi SpA, Cesena, Italy, 1993.

Tsuchiya, Y., Kenmochi, S., Hara, T., Nakayama, K., "New Tunneling Technique: PASS Method," Tunnelling and Ground Conditions, Abdel Salam (ed.), Balkema, Rotterdam, 1994.

Wakasa, R., Okamoto, S., Kawakami, K., "Construction Report on the Futatsui-Nishi Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), September, 1992.

Wallis, S., "Micro Assistance for Macro Undertaking in Atlanta," Tunnels and Tunnelling, Miller Freeman, January, 1992.

Yamashita, Y., Shiomi, K. et al, "Supplementary Methods in the Uryuya Tunnel," Technical Report on the RJFP Method, Geo-Fronte Research Association, January, 1994.

Zaitsu, M., Watanabe, T., et al, "Construction Report on the Hodogaya Tunnel," Tunnels and Underground, Japan Tunnelling Association (JTA), May, 1994.