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SESSION 2A

Structural design – Soil and foundation

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Computational Model and Charts for Cut-and-Cover Tunnels

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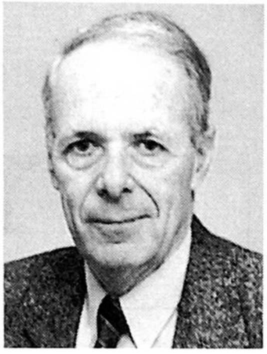


Summary

Arch-shaped concrete linings show even in cut-and-cover situations economical advantages when compared with rectangular profiles. The backfilled soil along the sides of the arch counteracts lateral movements caused by the dead weight of the soil cover above it. Thus, the computational model has to take into account the interaction between the arch and the soil. A comprehensive computational model was developed and validated by field measurements on several types of tunnel profile and at different stages of backfilling. The numerical procedure enabled charts to be developed for the design parameters for different geometrical configurations. Apart from the selection of the cross-section the following factors can be influenced by the engineer: the thickness of the concrete arch, its steel reinforcement and the stiffness of the backfill. The presented curves clearly exhibit the interaction between these factors.

1. Introduction

For environmental reasons roads and railways are being routed more and more through tunnels constructed by the cut-and-cover method. For this purpose a cut is made, the tunnel arch is constructed and then the structure is backfilled in accordance with the landscaping requirements. In Fig. 1 this method is illustrated for the most frequently encountered case of cut. Fig. 2 shows the procedure for an excavation supported by tied-back pile walls and Fig. 3 that of a cut in sloping ground. In this contribution we deal with reinforced concrete arched tunnels of large cross-section that are often met with in traffic tunnels. Rectangular frame structures or thick-walled sewage pipes are not considered. In the case of arched tunnels the deformations of the usually slender concrete construction are sufficient to activate a deformation-dependent interaction between it and the lateral backfill. This does not apply for thick-walled pipes, especially if they are of rectangular shape, since they tend to behave as embedded rigid structures.

The aim of the present work is twofold. Firstly, a computational method is described, whose validity has been confirmed by extensive field measurements in numerous projects under widely

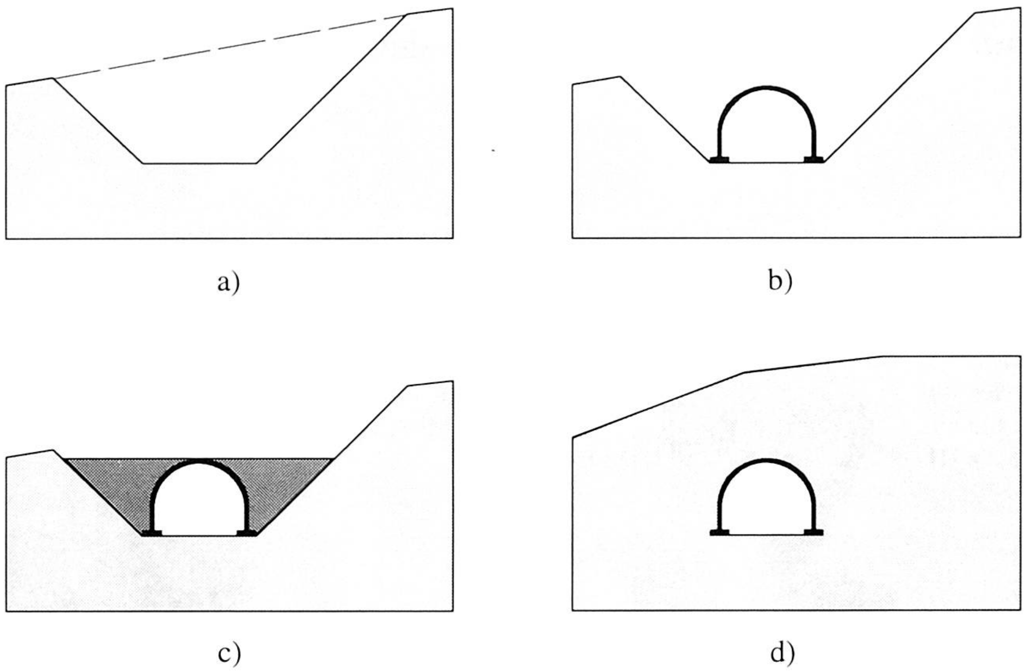


Fig. 1 The different construction stages of a typical cut-and-cover tunnel
a) Excavation *b) Construction of concrete arch*
c) Lateral backfilling *d) Backfilling over tunnel*

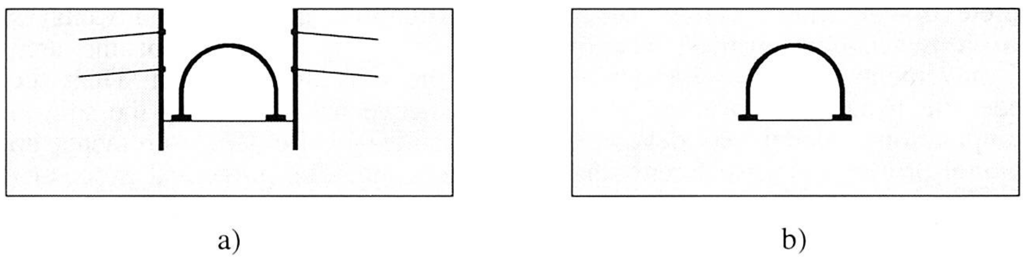


Fig. 2 Cut-and-cover tunnel in an excavation supported by pile walls

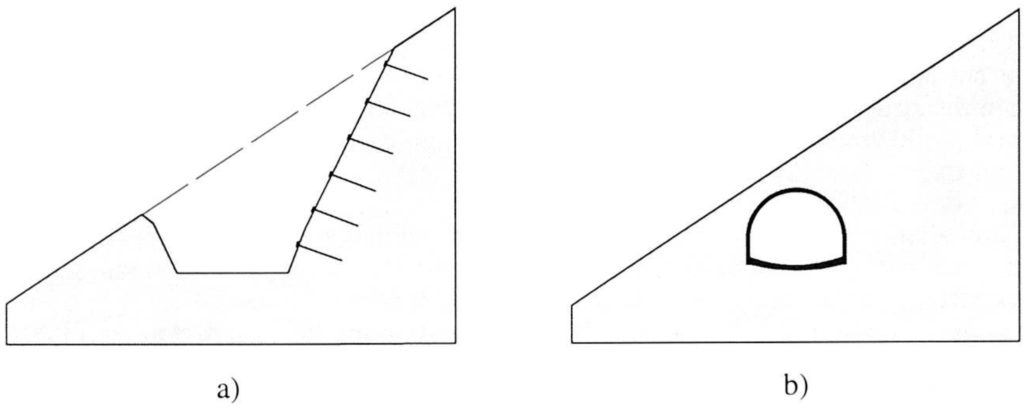


Fig. 3 Cut-and-cover tunnel in sloping ground
a) Excavation and support of slope
b) Construction of tunnel arch and backfilling

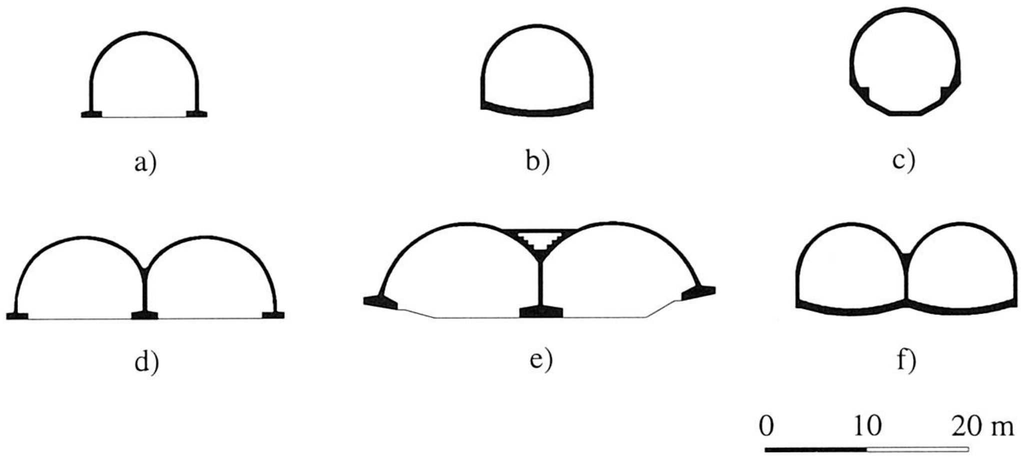


Fig. 4 Typical profiles in traffic tunnels
a) - d) Single and two lane tunnels with open and closed tunnel base
e) - f) Twin tube for two and three lane tunnels

differing conditions. Secondly, the statical behaviour of cut-and-cover tunnels is illustrated with the aid of charts. Ultimately, the economic design of such tunnels depends on a basic understanding of the way the forces act and on a suitable computational method. Several examples of typical tunnel profiles common in traffic structures are shown in Fig. 4.

In the design process for cut-and-cover tunnelling a number of decisions have to be taken. They are concerned with the tunnel shape, the thickness of the concrete lining, the amount of reinforcement and the stiffness to be achieved in the lateral backfill. These factors have a big influence on the safety of the structure, its serviceability and costs.

2. The Computational Model

The structure of a cut-and-cover tunnel consists of four elements (Fig. 5):

- the original ground
- the tunnel arch
- the lateral backfill
- the backfill above the tunnel roof.

These structural elements are deformable and interact with one another. The fill over the tunnel roof, however, only acts as a load for small heights and exhibits no real load carrying capacity. It is clear that regarding this fill simply as a vertical load acting on the system backfilled to the crown of the tunnel arch lies on the safe side. But how does the arch interact with the stepwise-backfilled material on the sides of the tunnel? It is clear that as soon as a compacted layer of backfilled earth extends over the whole of the width it becomes a component of the structural system. Thus with progressive backfilling the tunnel arch experiences an increasing embedment implying a step by step change of the structure as a whole.

Assumptions in the computational model (Fig. 5): The original ground ① and the compacted layers of backfill ③ are assumed to be elements acting in plane strain since there is no displacement normal to their plane, while the concrete arch ②, that is structurally connected to them, is assumed to consist of curved beam (i.e. flexurally stiff bar) elements. The vertical and horizontal extents of the ground considered in the analysis depend on the span of the tunnel arch and the depth of backfill. The external boundaries are allowed to have displacements, each in one direction. Fig. 5 shows the final state of the completed structure. The engineer, however, is interested above all in the intermediate stages of placement of the lateral backfill, since the internal forces and moments (M, N) and the deflections due to

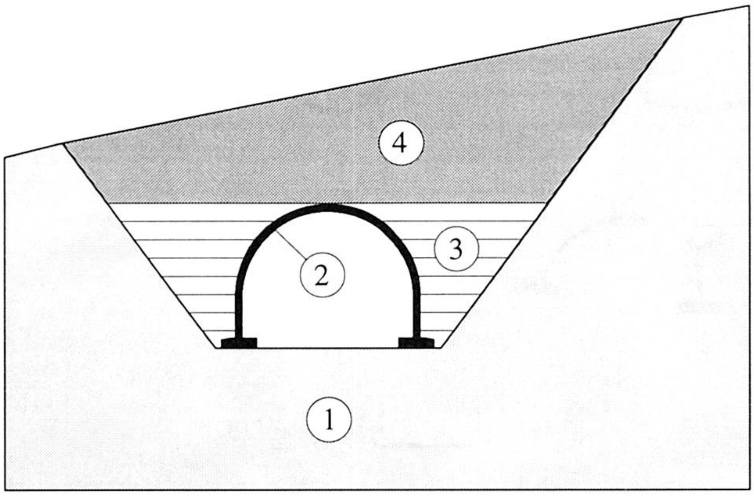


Fig. 5 The structural system for a cut-and-cover tunnel
① the original ground ② the tunnel arch
③ the lateral backfill ④ the backfill above the tunnel roof

bending action change with each new stage of backfilling. The aim is to determine for each point on the tunnel arch the critical values of M and N. Thus only a computational method which is able to take into account the changes in the structure during the stagewise construction process is of interest. The first question to be answered is what is actually the incremental load applied to the structural system. This is given by the unit weight of the layer of backfill in the process of compaction, which has a width of **a**, and the depth of **Δh** of backfill. Since a layer of lateral backfill is side-by-side with the tunnel arch, the earth pressure has to be considered there. Its value lies between the active earth pressure (λ_a) and the earth pressure at rest (λ_0) depending on the lateral movement of the arch. The earth pressure coefficients are a function of the angle of internal friction φ , i.e.

$\lambda_a = \tan^2(45°-\varphi/2)$ and $\lambda_0 = 1-\sin\varphi$.

The formula for λ_0 was proposed originally by Jáky (Jáky, 1944) and then only for cohesive soils. Fig. 6 shows that the load increment components (p_h, p_v) influence the concrete arch ②, the already compacted backfill ③ and the original ground. It is recommended to carry out separate calculations with λ_a and λ_0 and to adopt for design purposes the most unfavourable result for the pair of values (M and N). One can show that with decreasing thickness Δh of a backfill layer in a stage the influence of the difference between the active pressure and the earth pressure at rest decreases. If for any reason the backfilling is carried out asymmetrically, then due to the different heights of the lateral backfill an asymmetrical structure results. In the computational model this is taken into account automatically. Even if in subsequent backfilling the symmetry of the structure is restored in terms of geometry, the distribution of the final deformations and the sectional forces remain asymmetrical. The computational program presented here has a memory capability. An extreme case of asymmetry of final backfilling is obtained for a tunnel in sloping ground (Fig. 3).

After specifying the statical system and the loading quantities the idealisation of the deformational behaviour for the tunnel arch ① and the plane strain regions ② and ③ (Figs. 5 and 6) is discussed. As is usual in the statical analysis of reinforced concrete structures, a linear elastic material behaviour is assumed. Without limiting the validity of the computational model, in the following we exclude the development of plastic hinges. We will not consider this point further, as the structural safety of a backfilled tunnel arch is hardly affected by the presence of a few plastic hinges.

If we consider the plane strain region ① of the original ground (Figs. 5 and 6), then it is reasonable here too to assume linear elastic material properties. Its stiffness is characterised by

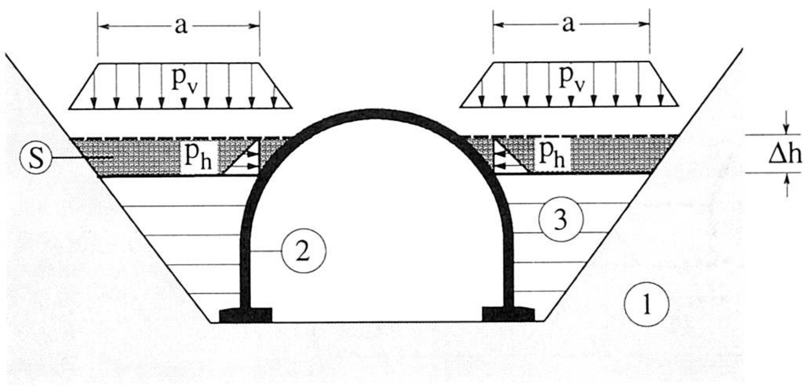


Fig. 6 The layer S during compaction
Consideration as surcharge p_v and earth pressure p_h

the value of the Young's modulus E_0, which is either estimated or determined by field tests. In the case of tunnels comprising twin tubes or a base slab the value of E_0 is of particular importance. If the ground has a low stiffness there is in the case of twin tube profiles a big differential settlement between the middle foundation and those at the sides, which leads to higher bending moments in the concrete arch. There is usually no information available concerning the Poisson's ratio ν_0 of the original ground. It also influences the settlements, however, since a higher value of ν_0 corresponds to stiffer soil. In the following computations an estimated value of $\nu_0 = 0.25$ is employed.

As the already compacted backfill forms an essential part of the structure its deformational properties are of particular interest. From the computational viewpoint there would be no difficulty in introducing a non-linear material behaviour by the artifice of assigning higher Young's moduli to the individual layers of backfill with increasing height of the backfill. Such a procedure is not adopted for two reasons. Firstly, one does not generally have detailed information about the stress-strain relationship for the backfill. Secondly, if this information were available, the advantage gained would be outweighed by various uncertainties. Thus a constant, pressure-independent Young's modulus E is used in all calculations. A question of some practical consequence is the choice of the Poisson's ratio ν for the plane strain elements ③ of the already compacted backfill. Here purely theoretical considerations are of little help. To obtain realistic values we resort to back-calculations of the measured deformation of the tunnel arch. Extensive investigations for different profiles (Figs. 4a-d) with different loading stages indicate that the best agreement is obtained with a value of $\nu = 0.5$, i.e. for an incompressible material. If an element of such material is fully restrained laterally then for a vertically applied stress σ_y a horizontal stress σ_x of the same value is induced. For $\nu \leq 0.5$ one obtains the relation:

$$\sigma_x = [\nu/(1-\nu)]\sigma_y$$

The empirical result that the already compacted backfill behaves as an incompressible elastic material whose Poisson's ratio $\nu = 0.5$ is the most important finding of our extensive field measurements.

2.1 The general case of the computational model

The computational model presented here takes into account the geometry of the structure in its various stage of backfilling (Fig. 7), the differing material properties of the original ground and the compacted backfill, as well as the normal and bending stiffnesses of the concrete arch. The shape of the latter- with or without base slab - may be chosen freely. Each stage of lateral backfilling represents a load increment. Fig. 7 shows the factors influencing the results of the computation. They are:
- the geometry (D, d, a_1, a_2, H_1, H_2, α_1, α_2, Δh) and
- the material properties (E_0, ν_0, E, ν, φ, γ).

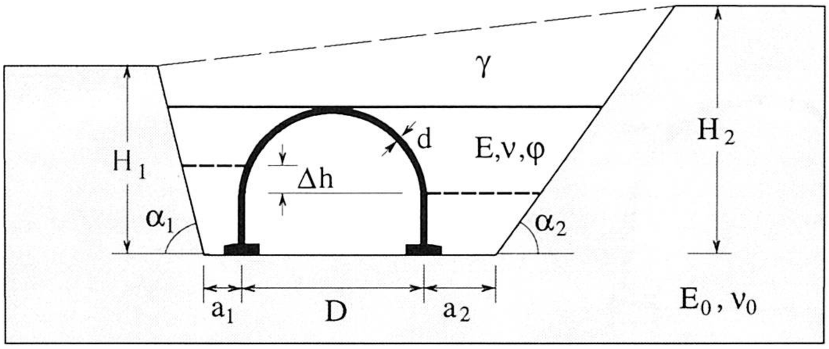


Fig. 7 The general computational model with the computational parameters

3. The Computational Procedure

The computational model consists of plane strain elements and embedded or partially embedded beam elements of the tunnel arch (depending on the stage of construction under consideration) the analysis being carried out by the finite element method. The computer program must be able to store and sum up the results of the individual computations for each stage of backfilling. Between the tunnel arch and the already compacted lateral backfill either a non-slip shear-resistant interface condition or one allowing slip (simulated by special slip elements) can be specified. It should be noted that only normal stresses can be transmitted between the beam elements and the plane strain elements.

The calculation procedure begins with the loading due to the first layer of the lateral backfill. The load quantities are different if there are different thicknesses of backfill right and left of the tunnel arch. After carrying out the calculations the internal forces and moments in the concrete arch together with the displacement components are stored. Fig. 8 shows the results for the stage i of backfilling. If the backfilling up to this point was asymmetric, this is shown by a non-symmetry in the distribution of the calculated quantities (bending moment M_i, normal force N_i, and the displacement components u_i and v_i in the tunnel arch). Here we should not forget that for small layer thicknesses the difference due to differing earth pressure coefficients is also small. This is generally the case for a thickness $\Delta h = 1.00$ m.

After lateral backfilling and over the arch the following internal forces and moments and displacements are obtained on summation:

$$M_n = \sum_1^n M_i, \qquad N_n = \sum_1^n N_i, \qquad u_n = \sum_1^n u_i, \qquad v_n = \sum_1^n v_i.$$

In this way the most unfavourable values of M and N can be determined for each section of the tunnel arch and for a chosen reinforcement content the safety against failure of the concrete arch at each section can be obtained from a corresponding interaction diagram.

It is recommended to represent the internal forces and moments and the displacements graphically as a function of the arch length. Since a translation of the arch as a whole does not produce any internal stressing or deformation it is of little interest. Thus it is advantageous to assume one of the abutments to be fixed and the other to displace only in a horizontal direction. In this way the deformation mechanism, especially in the case of twin tunnel cross-sections, is easily seen and a comparison with field measurements can be easily made.

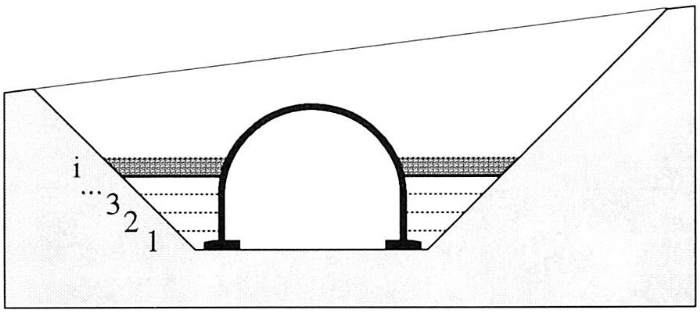


Fig. 8 The simulation of the backfilling stage i

4. Deformation Measurements and Back-calculation

Within the framework of a long-term research project extensive deformation measurements were carried out on 5 different structures with differing profile geometries and sizes of the span (Figs. 4a-d). The results were interpreted with the help of the computational model described above applying the same basic assumptions (Tisa and Kovári, 1993). The agreement between the measured and calculated deflection curves is better for more flexible concrete arches than for stiffer ones. The first would be the case for large spans and relatively small concrete thicknesses. For a particularly instructive example we consider the 550 m long Brünnen Tunnel on the Swiss National Highway N1, which with its twin four lane tunnel tubes (Fig. 9) forms a part of the by-pass around Berne. The spans are each 15.75 m, while the thickness of the concrete arch is only 0.35 m. The middle wall is 0.45 m thick. In three measuring cross-sections roughly 150 m and 250 m apart the deflection curve for the arch was measured for four stages of lateral backfilling. We pick out on the construction stage with the backfilling just to the crown of the arch and consider the displacement vectors in the three measuring cross-sections. From Fig. 9 it is clear that both side foundations experience settlements of about 4-8 mm relative to the middle wall. The three displacement vectors at a point indicate - despite a certain scatter both in their directions and magnitude - a uniform behaviour of the structure. Some deviations - especially in the area of the crown - may be traced back to differences in the procedure for lateral backfilling and compaction. A comparison between the measured and the calculated values can be best made in terms of the distribution of the displacement components u and v as a function of the arc length (i.e. the circumferential length of the arch) S.

In Fig.10 the points represent the measured values in the three measuring cross-sections. The lines represent the back-calculation, which was always carried out assuming the active earth pressure coefficient (λ_a) and the earth pressure coefficient at rest (λ_0). From the good agreement between observation and theory one may conclude that the calculated internal moments and forces, which for technical reasons cannot be measured directly, form a good basis for the design and the determination respectively of the structural safety. In the computational model the following parameters were adopted: $E_0 = 80$ MPa, $\nu_0 = 0.25$, $E = 50$ MPa, $\nu = 0.495$, $\varphi = 30°$, $\gamma = 22$ kN/m^3 and a Young's modulus of 30'000 MPa for the concrete. To experimentally determine the deflection curve the distometer instrument (Kovári and Amstad, 1979) was used, which by means of an invar steel wire is capable of measuring the change in distance between two points of the tunnel arch with an accuracy of around ± 0.05 mm.

It should be mentioned that with increasing stiffness of the arch the scatter of the measured deflections also increases leading to greater differences between theory and observation.

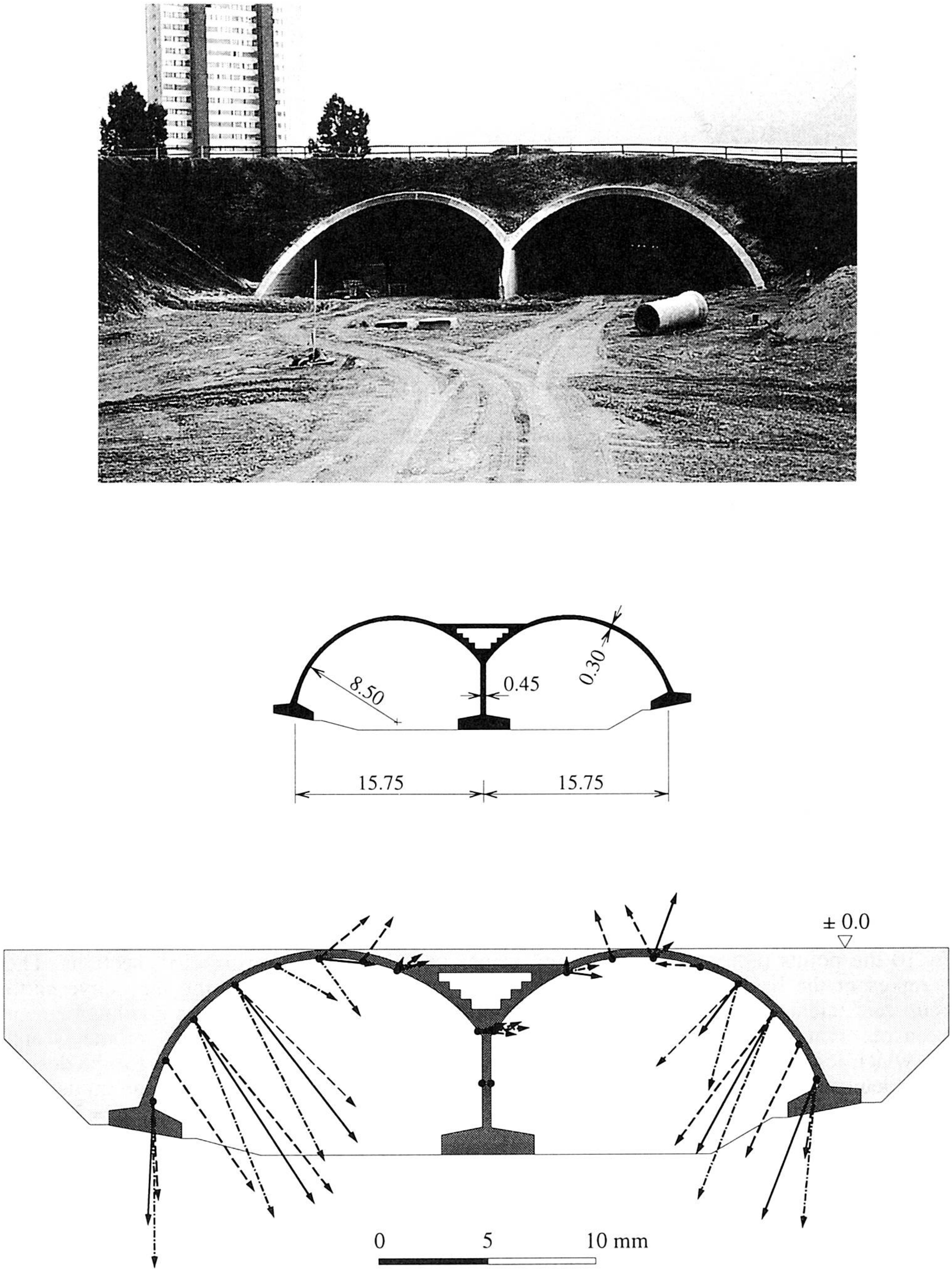


Fig. 9 Brünnen Tunnel on the by-pass in Berne (N1)
a) Tunnel entrance (by courtesy of Emch+Berger Bern AG)
b) Geometry
c) Observed displacement vectors in 3 measuring cross-sections

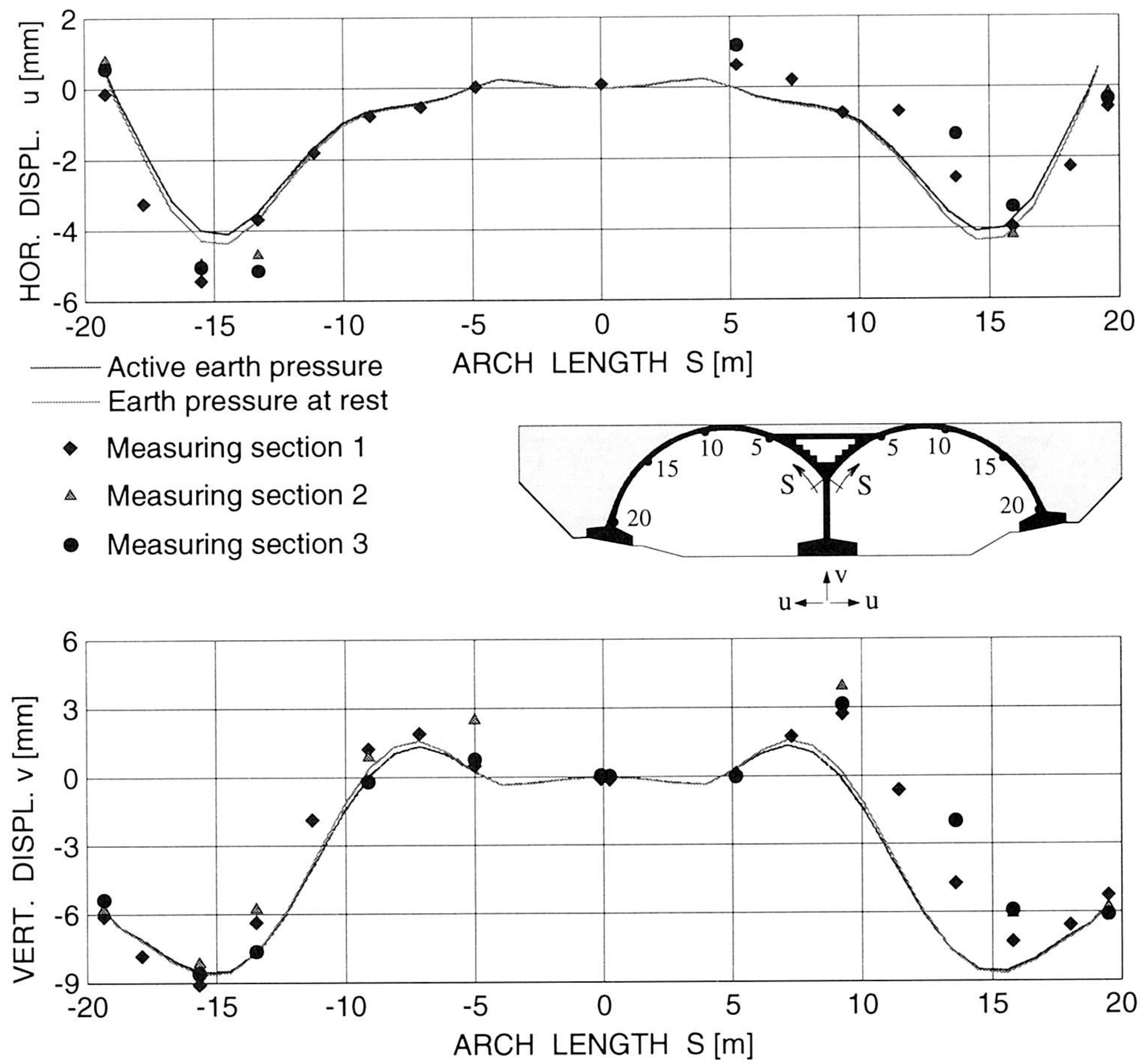


Fig. 10 Brünnen Tunnel: Comparison of measured displacement components (u,v) in discrete points with the calculated bending deflection curve

5. Design Charts

The computational model presented above permits the preparation of design charts for frequently encountered tunnel sections and deepens our understanding of the internal forces acting in cut-and-cover tunnels. In the foreground is the influence of the following factors that the engineer has to determine affecting the forces and moments at the cross-section:

- shape of tunnel section (with or without the invert arch base, etc.)
- the thickness of the concrete arch (d) and the amount of reinforcement (Fe)
- the stiffness of the lateral backfill (E) and
- position and profile of the final backfill above the roof of the tunnel.

In the following design charts for a two track tunnel (Fig. 11a) and a twin tube tunnel with four traffic lanes are presented and described (Fig.11b). For a chosen cross-section in the concrete arch (point V) the bending moment M and the normal force N are determined for different values of the above-mentioned factors and these two values are represented as a point in the interaction diagram (Figs. 13 and 14). The most important construction stage (A) corresponds to backfill to the height of the crown of the arch. The subsequent backfill was assumed to be horizontal with

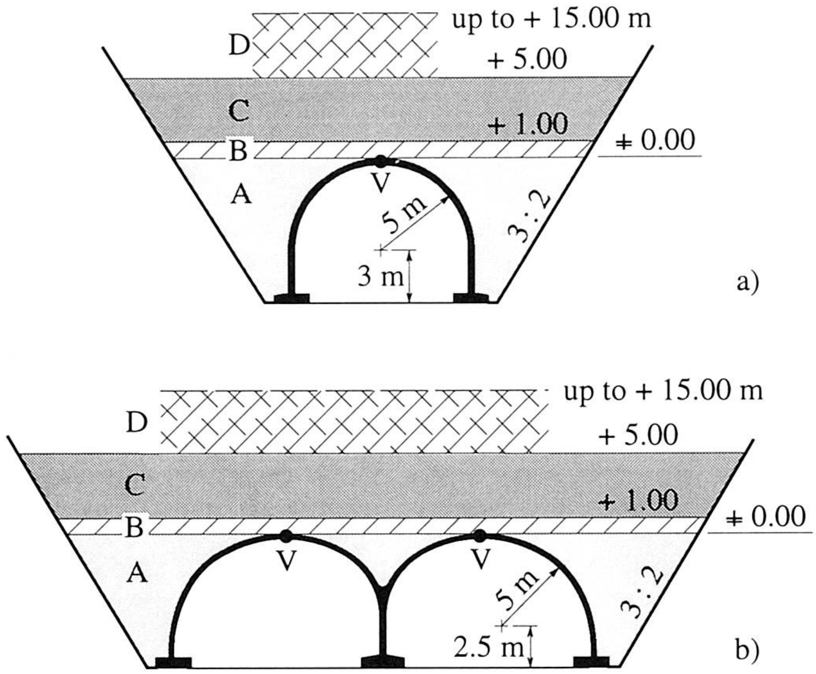


Fig. 11 Geometry of the profile and the computed backfilling stages A to D. The design charts refer to the cross-section at the crown of the arch (point V). a) Two track tunnel b) Twin tube tunnel

steps of 1, 5, 10 and 15 m. The corresponding construction stages are designated by B, C and D in Fig. 11. Each pair of values M/N of the construction stage A is the result of a series of individual computations corresponding to backfill increment of 1.0 m. The earth pressure coefficient at rest (λ_0) was always adopted. The backfilling to B, C and D was each considered as a single loading corresponding to the differential heights.

In view of the large number of parameters that appear in the model it is necessary to restrict the computations to a selected number of thicknesses of concrete arch (0.3, 0.4 and 0.5 m) and values of Young's modulus of the lateral backfill of 10, 20, 40, 60 and 100 MPa. Between the tunnel arch and the lateral backfill non-slip shear resistant interface condition (full friction) was considered. Details of the tunnel profile geometries for the profile types treated here are shown in Fig. 11.

5.1 Interaction diagrams for a two track tunnel

From previous investigations (Kovári and Tisa, 1982) it is known that in this type of profile the crown of the arch (point V) corresponds to the most unfavourable condition of internal stressing (M/N). The interaction diagrams (Figs. 12,13 and 14) relate to this cross-section of the tunnel arch. Each diagram is valid for a given thickness of concrete arch. The approximately horizontal varying series of points correspond to a particular height of backfill over the crown of the tunnel. It may be readily seen that these series of points move in the direction of higher normal force with increasing depth of backfill, i.e. the bending moments remain approximately constant despite the increased loading. Within such a nearly horizontal series of points the influence of the Young's modulus E of the lateral backfill is noticeable. With decreasing values of E a point moves to the right in the direction of greater moments. The permissible region of a pair of values M/N in the interaction diagram is marked for a given amount of reinforcement (0, 0.15, ... 0.60 %). These limit lines apply for a resistance coefficient reduced by the amount γ = 1.2 (Swiss Code SIA 162).

A look at the charts clearly shows that the construction stage with backfill up to the crown of the arch is the least favourable. Each additional backfill step results in an increase of normal force at an almost constant bending moment. Thus the points on the interaction diagram move away from the corresponding limit lines for varying amounts of reinforcement.

Considering the influence of the thickness of the arch **d** on the bending moment and on the required reinforcement content Figures 12, 13 and 14 clearly show that with increasing value of **d** the bending moment increases and the reinforcement content decreases. These relations are however only pronounced for low values of the Young's modulus E of the backfill (10 and 20 MPa).

Each point for the row 0 m (backfill to crown of the arch) required 8 separate calculations corresponding to the 8 stages of the lateral backfill. For the rows 1m, 5 m, etc. each point required an additional calculation.

In order to show the forces for the critical construction stage A (lateral backfill to crown of arch) in a more explicit way we consider Fig.15. The diagram shows the bending moment M at the crown of the arch (V) and the required amount of reinforcement Fe in % according to SIA 162 for different values of thickness of concrete arch d and different values of Young's modulus in the lateral backfill E. In this figure it may also be clearly seen that with increasing stiffness of the backfill the bending moment and the required amount of reinforcement decrease. Concrete thicknesses induce greater bending moments but require, however, smaller specific reinforcement. The influence of the reinforcement at large values of E is always smaller.

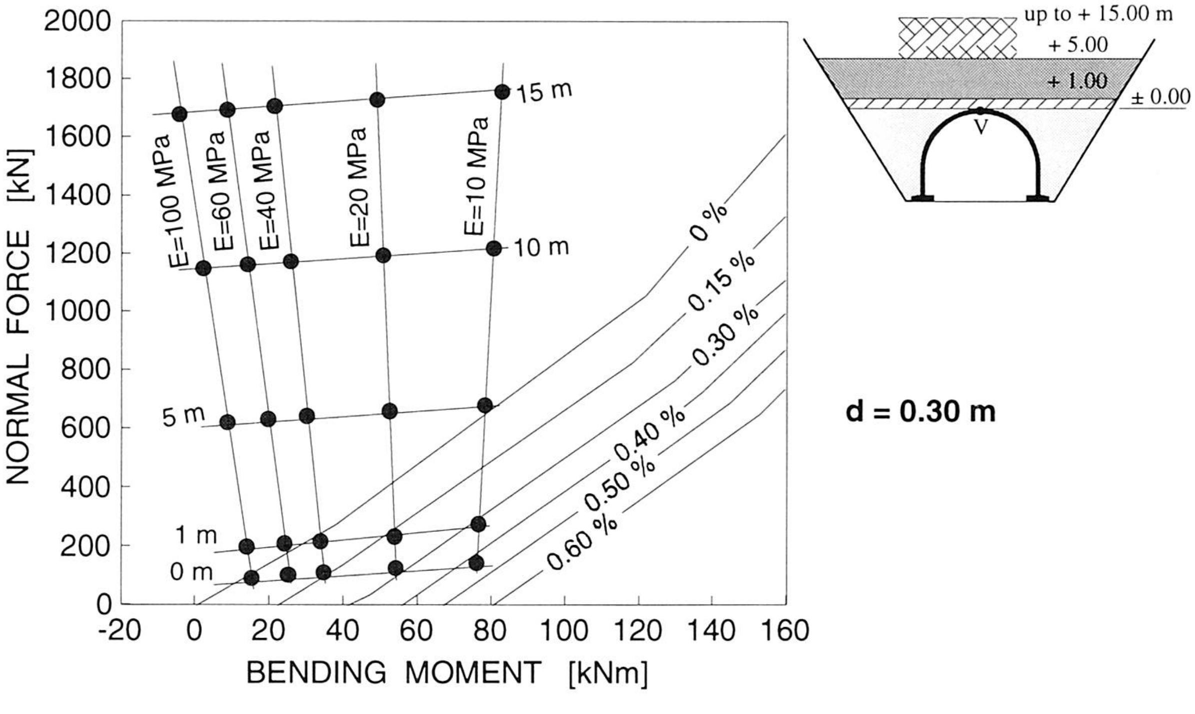


Fig. 12 Interaction diagram M/N for the crown V for the backfilling stage to the crown with backfilling increments of 5, 10 and 15 m Thickness of arch 0.30 m (reinforcement content in %)

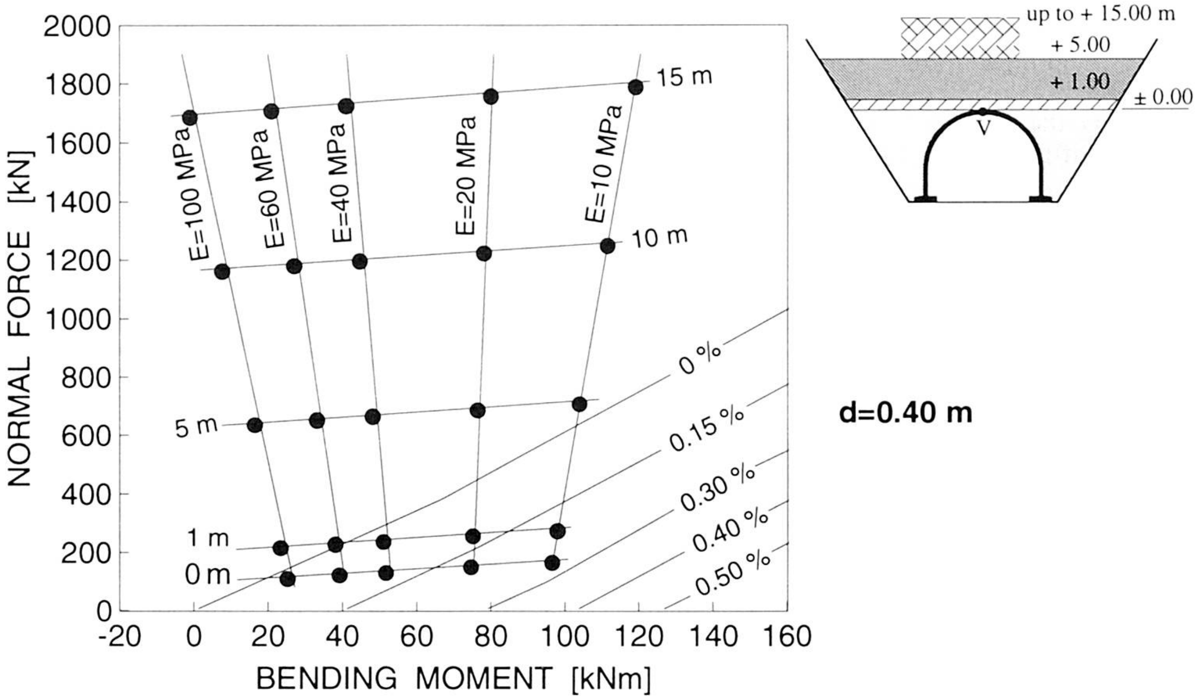


Fig. 13 *Interaction diagram M/N for the crown V for the backfilling stage to the crown with backfilling increments of 5, 10 and 15 m Thickness of arch 0.40 m (reinforcement content in %)*

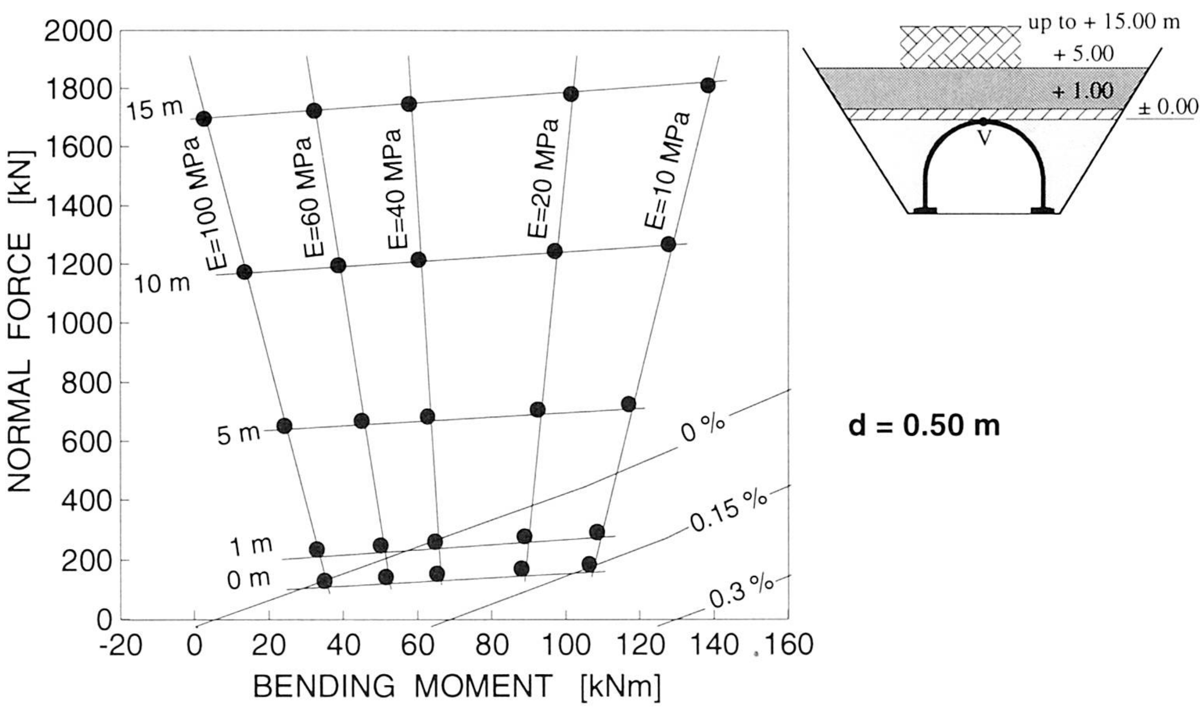


Fig. 14 *Interaction diagram M/N for crown V at the backfilling stage to the crown and backfilling increments of 5, 10 and 15 m. Thickness of arch 0.50 m (reinforcement content in %)*

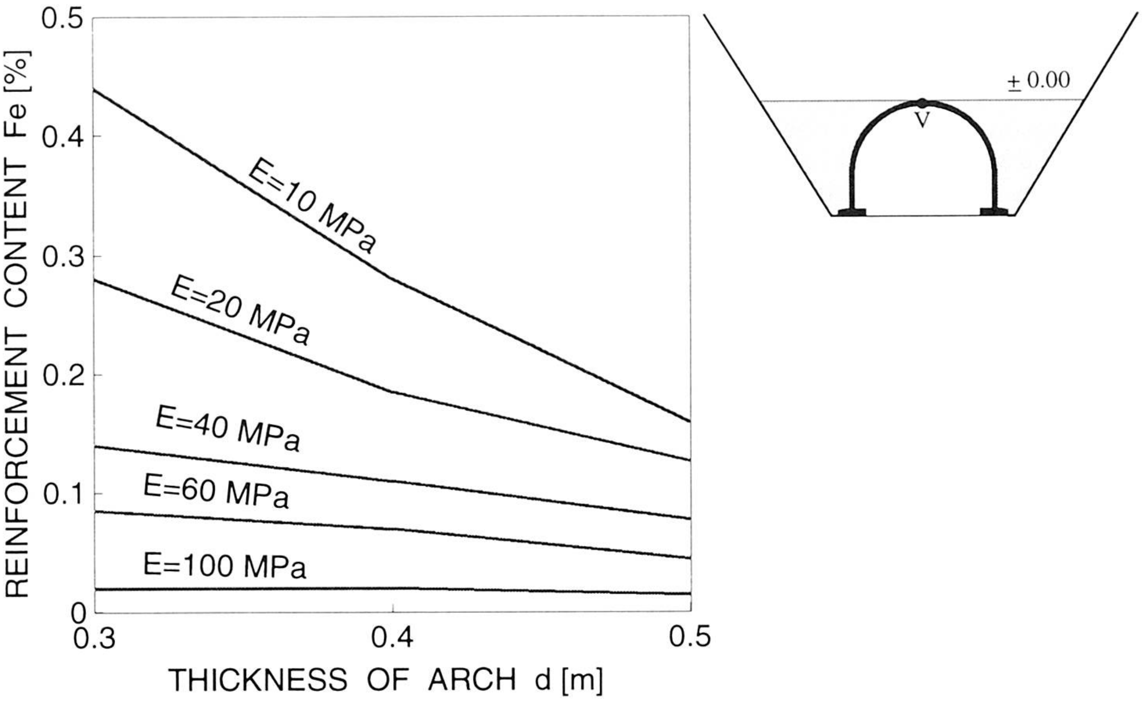


Fig. 15 Lateral backfilling up to the crown. Required reinforcement content (Fe) as a function of chosen values of E and d.

To further elucidate the way the forces act we refer to the diagrams in Fig. 16. Here too the values of normal force and bending moment are those at the crown of the tunnel V with the lateral backfill level with the crown. One clearly recognises here as well that the stiffness (Young's modulus E) of the backfill only affects the normal force slightly, but the bending moment greatly.

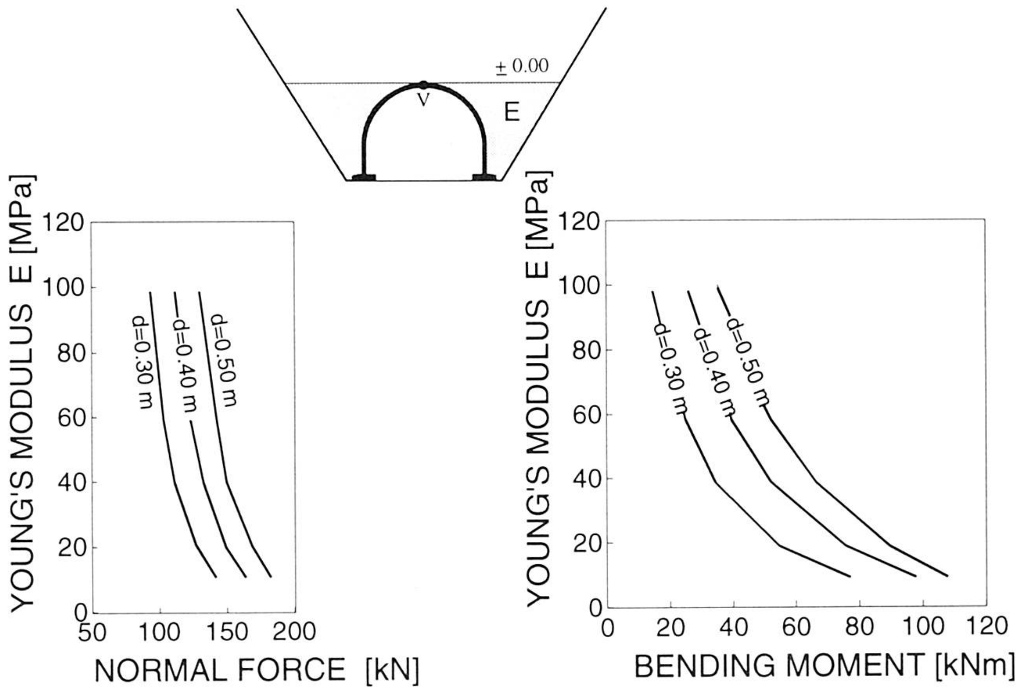


Fig. 16 The internal moments and forces M and N as a function of the Young's modulus E of the backfilling and the thickness of the arch d

5.2 Interaction Diagrams for Twin Tube Tunnels

The exact geometrical details for the tunnel profile may be taken from Fig. 11b. The diagrams in Figs. 17 and 18 apply once again for concrete arch thicknesses of 0.30, 0.40 and 0.50 m. The most striking result in Fig. 17 is the converging of the lines for a constant value of E in a point at d = 0.30 m and a height of backfill of 10 m. This means that at this stage of construction the Young's modulus E of the backfill has neither an influence on the bending moment nor on the normal force at point V. For the thickness d=0.40 m the point of convergence lies at a backfill height well over 15 m (Fig. 18). For d = 0.50 m (Fig. 18) the influence of the Young's modulus E of the backfill becomes for all heights under consideration very pronounced. Fig. 19 shows the same tendency between the crucial factors as Fig. 15.

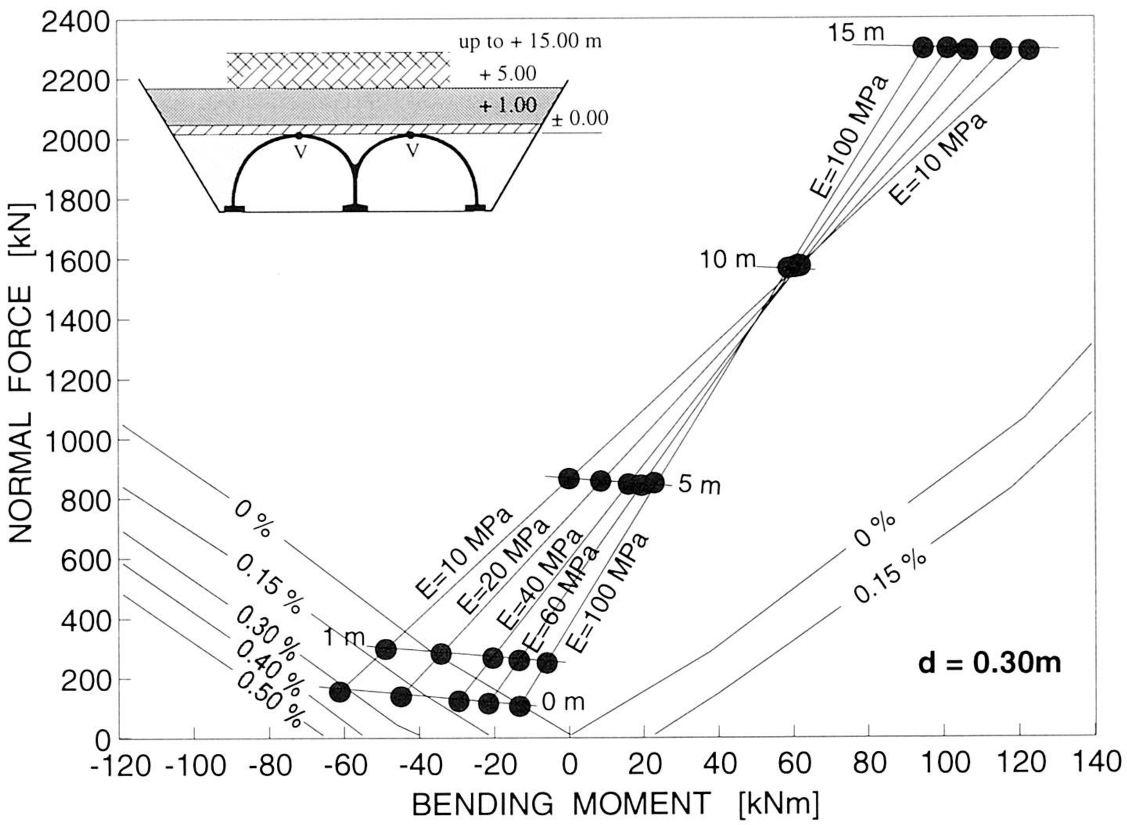
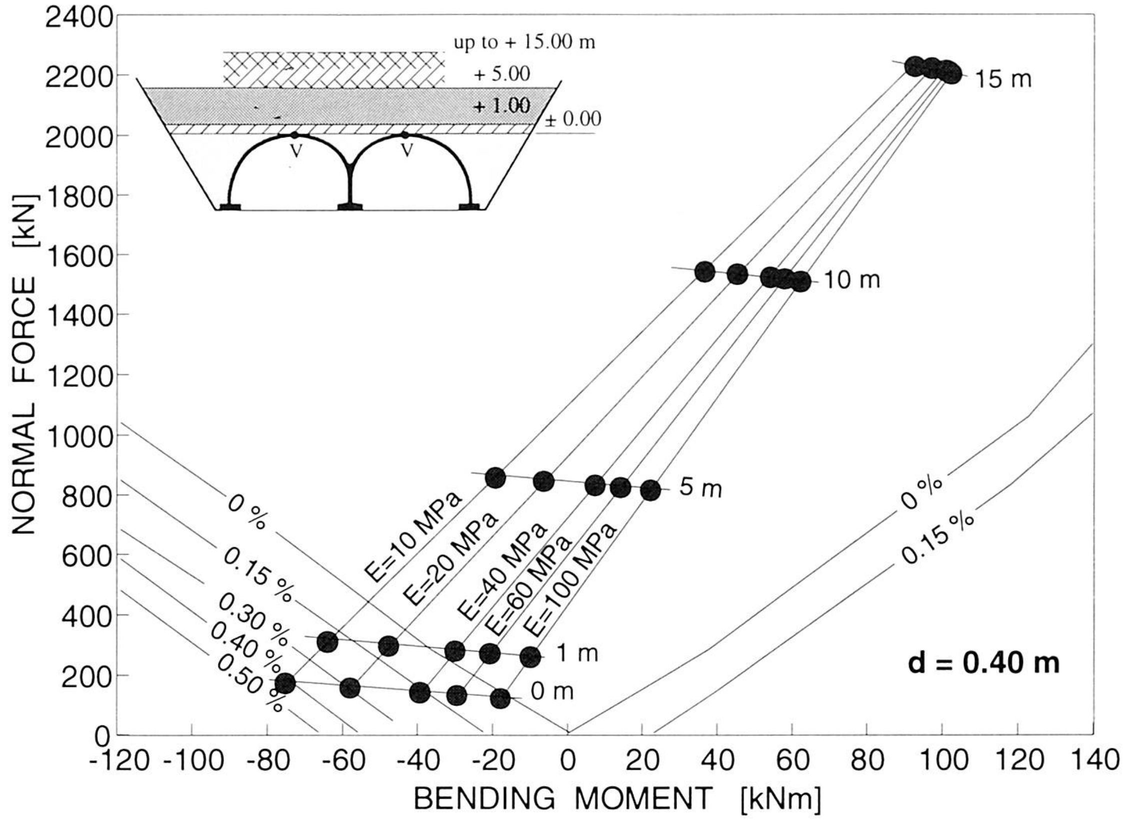


Fig. 17 Interaction diagram M/N for the crown V for the backfilling stage to the crown with backfilling increments of 5, 10 and 15 m Thickness of arch 0.30 m (reinforcement content in %)



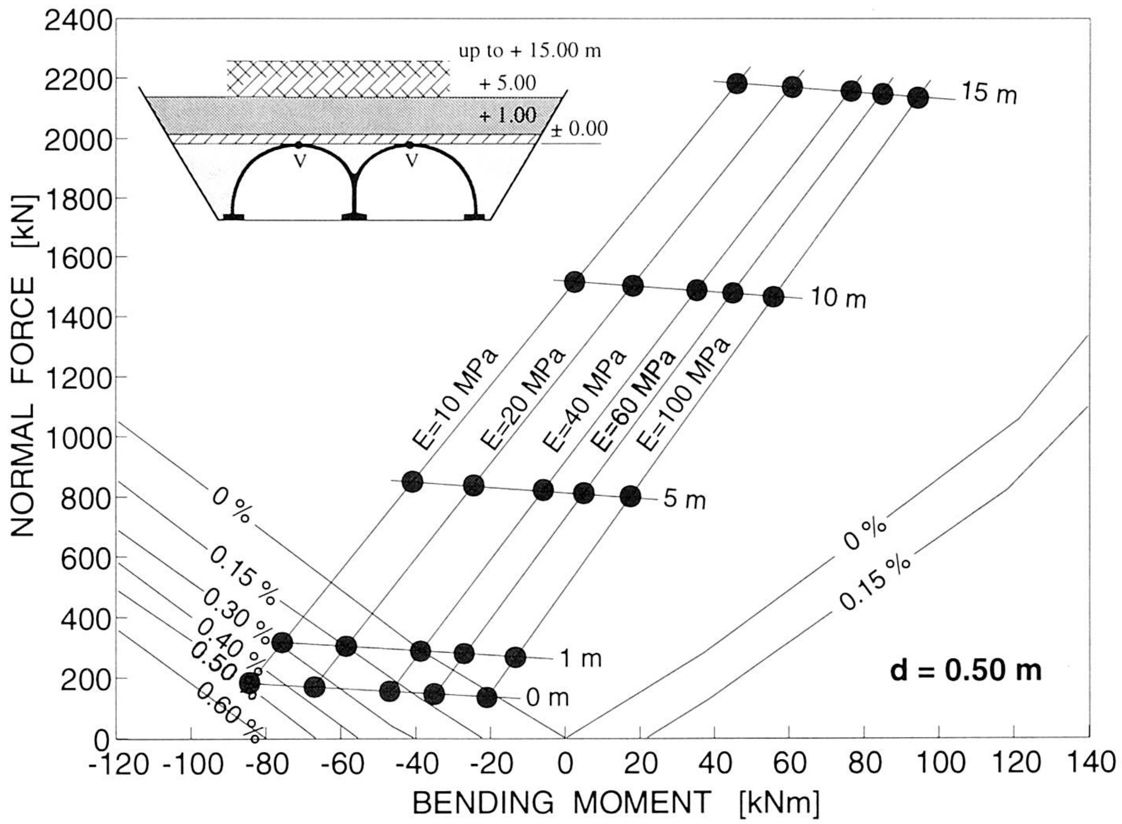


Fig. 18 Interaction diagram M/N for the crown V for the backfilling stage to the crown with backfilling increments of 5, 10 and 15 m Thickness of arch 0.40 m and 0.50 m (reinforcement content in %)

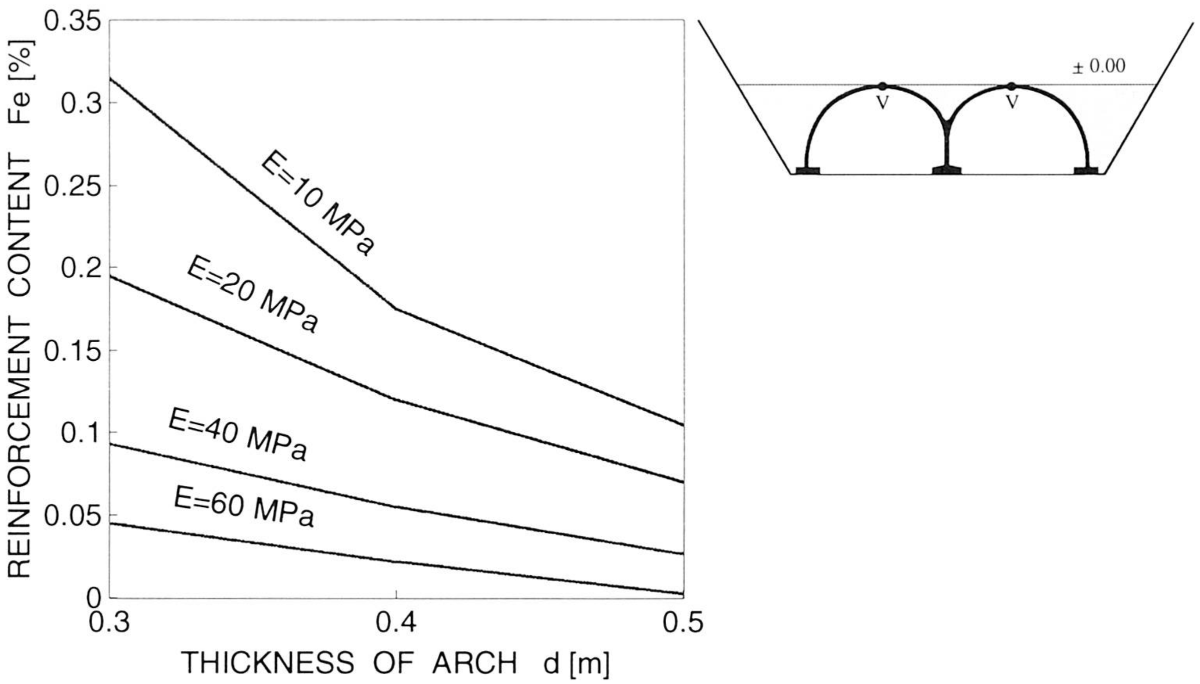


Fig. 19 Lateral backfilling up to the crown. Required reinforcement content (Fe) as a function of chosen values of E and d.

6. Final Remarks

The computational model presented in this paper was validated by measured deflections of various cut-and-cover structures in several stages of backfill. The model gives the possibility to establish a relation between the factors influencing the interaction between the tunnel arch, the backfill and the original ground. The presented charts may serve as a guideline for preliminary designs of tunnels with similar cross-sections as well as a means to understand the structural behaviour of arch-shaped concrete linings in cut-and-cover situations. In case of the final design of a given project such charts may lead to the most economical design. It is recommended to carry out computations for both non-slip resistant interface condition and for one allowing slip between tunnel arch and lateral backfill.

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Over-Crossing Concrete Tunnels in Arlandabanan

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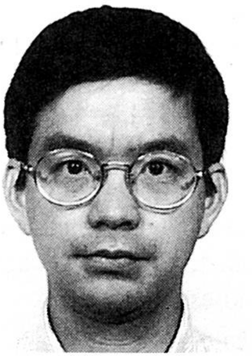

Lingfu Zeng, born 1962, received his B.Sc 1983, M.Sc 1986 and Ph.D 1991, specialized in Solid & Structural Mechanics and Finite Element Methods.

Summary

The structural analysis of over-crossing concrete tunnels in Arlandabannan, which is a new high-speed-train connection between the center of Stockholm and the Arlanda airport, is overviewed. The tunnels are of cut-and-cover type and consist of a single-track tunnel crossing obliquely over a double-track tunnel at about 20 - 30 m under the ground surface. The present paper describes a shell structure adopted for the tunnels and summarizes finite element analyses that have been carried through for the tunnels under various loading conditions. Numerical results are presented and suggestions to the structural analysis and design of similar concrete tunnels are given.

1. Introduction

Arlandabanan is a new high-speed-train railway connecting the center of Stockholm and the Arlanda airport. Near the Arlanda airport a railway of about 5,100 m long is constructed under the ground through rock and concrete tunnels. The concrete tunnels are of cut-and-cover type and consist of several different sections, see Mörnstad [1]. In a section near Märstaån a single-track tunnel (Shuttle Tunnel) crosses obliquely over a double-track tunnel (Intercity Tunnel) at about 20 - 30 m under the ground surface, see Fig. 1. The over-crossing part of the concrete tunnels, which is heavily grayed in the figure, is designed as an independent monolith connecting through dilation joints with neighboring concrete tunnels. In the following presentation this monolith will be referred to as *an over-crossing tunnel monolith*. The present paper describes a shell structure adopted for the over-crossing tunnel monolith. The concrete tunnels are subjected to various design loads which are mainly according to the Swedish design codes for building structures [2], also partly according to those for bridge structures [3], railway-bridge structures [4] and tunnel structures [5]. (See also Mörnstad [1] and references therein for the more general design specifications and requirements.) The main part of the paper is to report finite element analyses that have been carried through for this over-crossing tunnel monolith under various design loads, e.g.

underground water, overburden pressure, earth pressure, temperature changes, traffic loads and so on.

2. Thick-Shell Structure

In this over-crossing tunnel monolith the Intercity Tunnel is 97.5 m long and the Shuttle Tunnel 83.2 m. The crossing angle is about 16°. The internal dimension of the Intercity Tunnel is 11.5×7.5 m^2 and that of the Shuttle Tunnel 7.0×7.3 m^2. The thickness of different plates and other geometric parameters are given in Fig. 1. We note that the covering soils over this tunnel monolith differs about 10 m in depth from the southern end (towards Stockholm) to the northern end, see also Fig. 1. Such a 3-dimensional structure with non-uniform loading should be adequate to be modeled as a shell. In the modeling, considering the concrete plates are relatively thick, a thick-shell theory is applied so as to account for the shear deformation through the shell-thickness dimension.

3. Geomechanical Conditions

The bottom plates of both Intercity and Shuttle Tunnels in this tunnel monolith lie on compacted rockfills which are produced through in situ rock-blasting and are considered as an elastic foundation. The thickness of the rockfills is about 0.3 m and its elasticity modulus is 50.0 MPa. The rockfills are supported by hard rocks. The averaged underground water level is +20.5 m with the highest high underground water level at +21.5 m and the lowest low underground water level +19.5 m.

4. Design Loads

The design loads are divided into two groups: permanent and variable loads. The permanent loads include the self-weight of structure, ballast, (vertical and horizontal) earth pressure, water pressure. Notice that the water pressure is associated with three underground water levels. This in turn leads to six different cases in the earth pressure loads. In the group of variable loads, the following loads are considered: train loads, overburden pressure, earth pressure due to the overburden pressure (single and/or double sided), temperature changes (see below) and underground water level variations. Notice that the train loads are dynamic and moving.

The over-crossing tunnel monolith is a thick-walled structure with a closed section. In addition, it is relatively long. The effects of temperature changes have been found to be pronounced. The following cases of temperature changes are considered: (*i*) Temperature difference of ±14° between the outer and inner shell-surfaces (4 cases); (*ii*) Temperature increase or decrease of 10° in the plate shared by the Intercity and Shuttle tunnels.

The permanent and variable loads are combined with different coefficients principally according to the design code *BKR 94* [2]. The following combination categories have been considered: ordinary use, long term use, fatigue, damage. For a more detail specification of design loads and load combination we refer to Jovall [6].

4. Finite Element Analysis

A general-purpose finite element program ABAQUS [7] has been used for the structural analysis of this over-crossing tunnel monolith. In the analysis 8 node thick-shell elements are used. A finite element mesh with 1504 elements, 4607 nodes and 27000 unknown variables are used, see Fig. 2 (left). Linear elastostatics is assumed and dynamic amplification factors are employed to account for the dynamic effects of train loads.

The individual load cases are first applied and it has been found that the dominant design loads are the earth pressure and the temperature changes. This has been used as a guide when performing various load combinations. In Fig. 2 (right), the deformed structure is shown for a combined load case where a load combination for damage is considered.

The temperature load is one of the particularly concerned load cases in the analysis. In Fig. 3 the deformed structure under a temperature load and a section force contour map are plotted.

Notice that there exist seven components of section forces, i.e. two moments, a twisting moment, two normal membrane forces, an inplane shear force and two transverse (thickness direction) shear forces. Hence, huge amount numerical results/data must be processed as max-min section force envelopes must be established in order to carry through the design calculations, e.g. reinforcement calculations, see Jovial [6] and Zeng [8] for the detailed treatment.

5. Closure

Structural analysis of an over-crossing concrete tunnel monolith has been overview in the paper. A shell structure adopted and the finite element analyses carried through for the tunnel monolith have been briefly described. The temperature loads are found to be pronounced. This should be emphasized when similar concrete tunnels are studied. The importance of other practical issues, e.g. processing huge numerical results in an effective manner when structural analysis of a complicated structure is performed, should also be realized.

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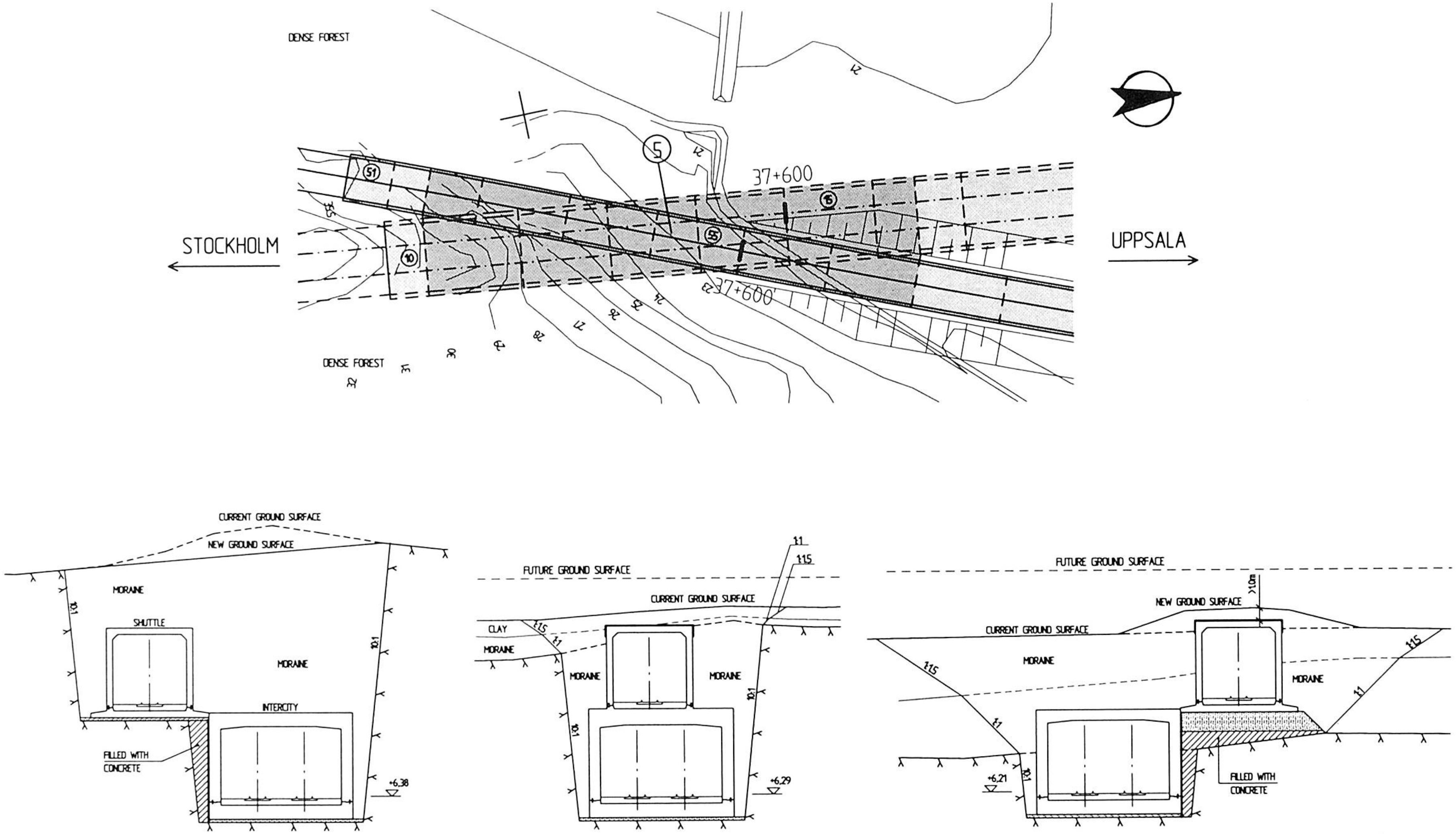

Fig. 1. Over-crossing concrete tunnels at Märstaån in Arlandabanan
- A plan view (upper) and three typical sections (lower).

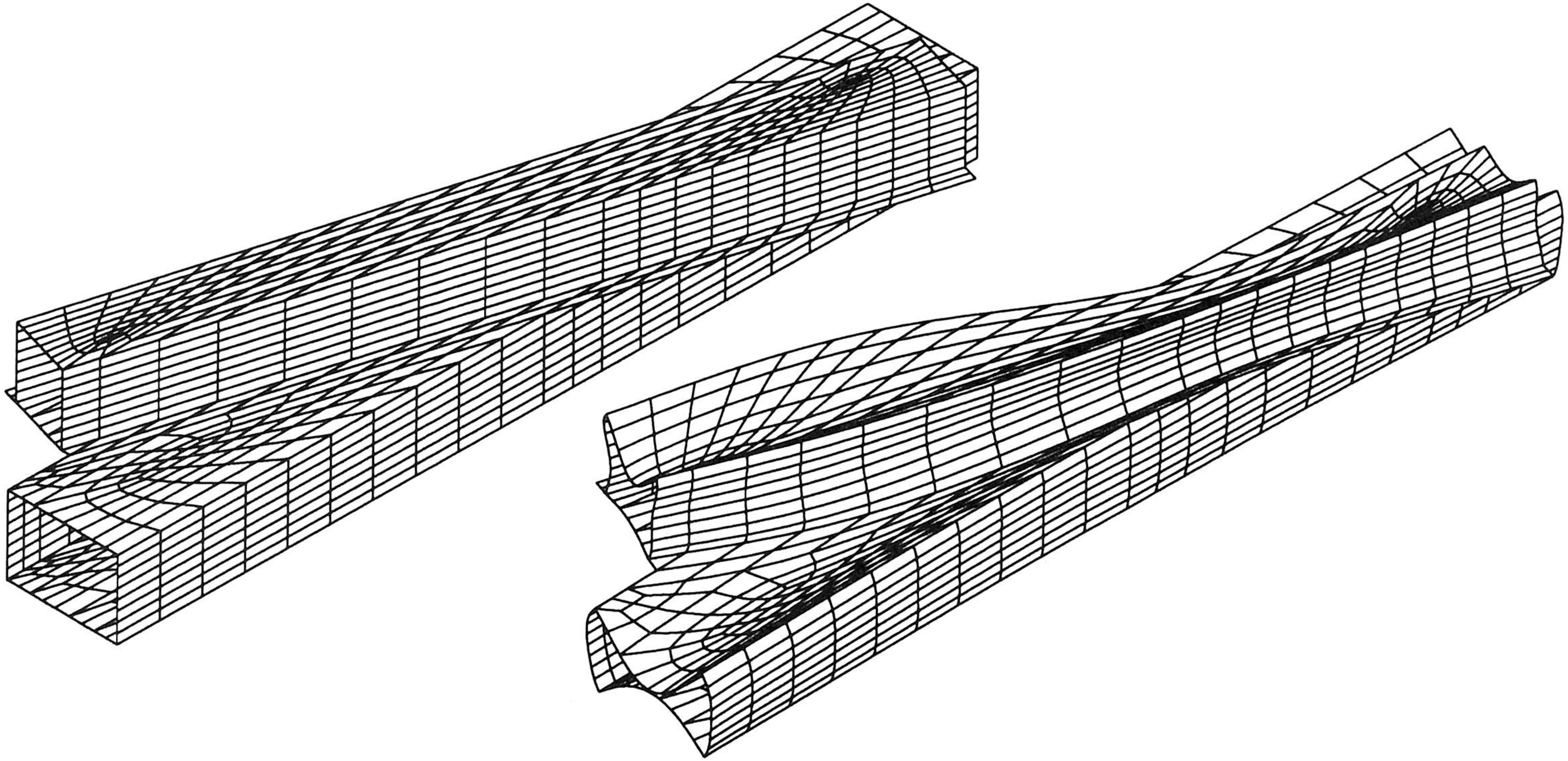

Fig. 2. Finite element mesh with 1504 thick-shell elements (left) and the deformed structure when a combined load case for damage is considered (right)

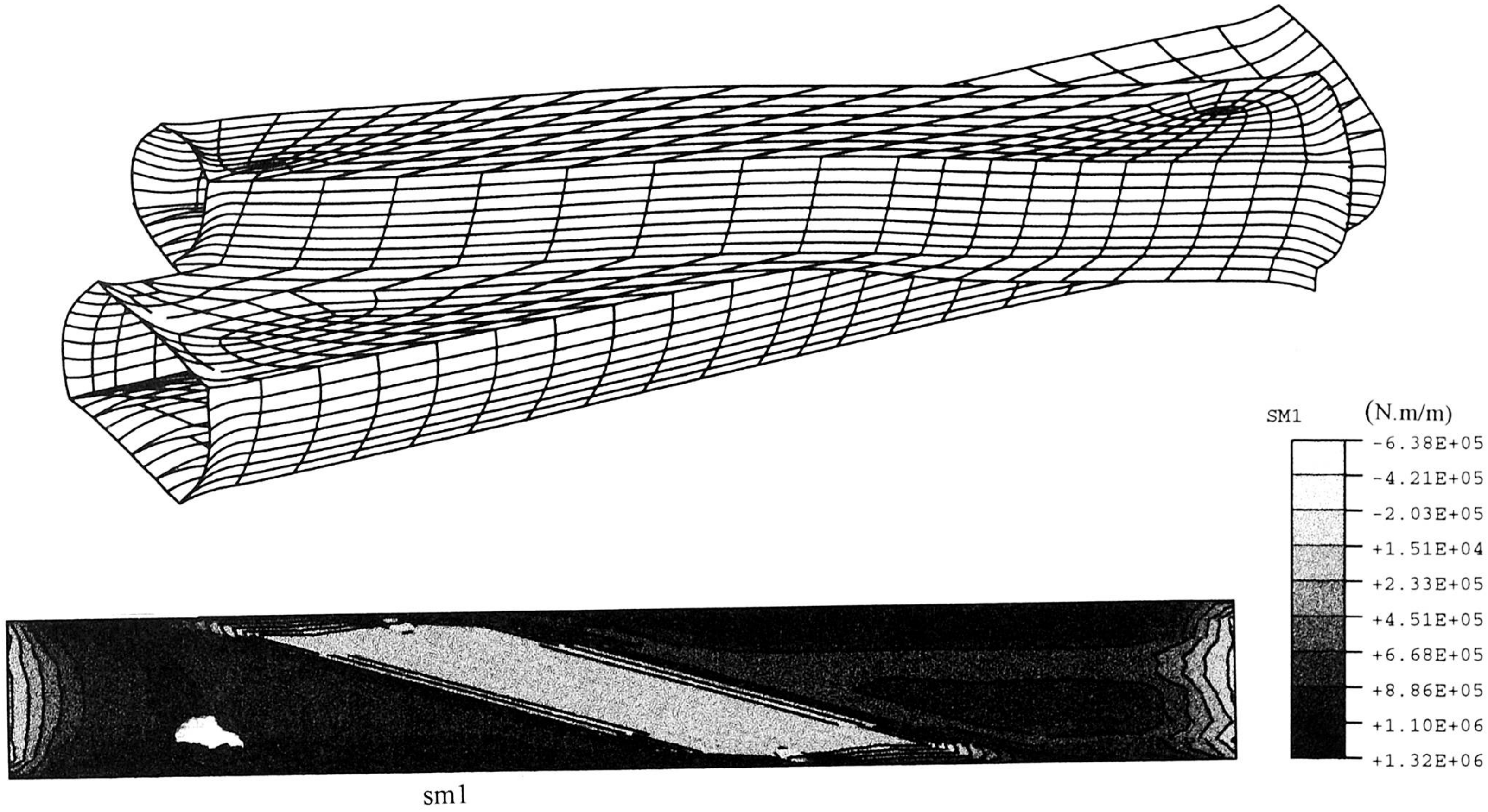

Fig. 3. Deformed structure (upper) and a moment contor map of the Intercity Tunnel roof (lower) when a load of temperature difference of 14° between the inner and outer shell-surface is applied.

Underpinning Of Red Line South Station

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Summary

The paper discusses the design and construction details for the underpinning system used for the underground Red Line Transitway station at South Station.

1.0 Introduction

The Massachusetts Highway Department's (MHD) Central Artery / Tunnel (CA/T) project in Boston is one of the largest highway projects ever undertaken to solve the traffic congestion problems of a large metropolis. This $10.5 billion project replaces the aging elevated Interstate Highway I-93 through downtown Boston and extend the Massachusetts Turnpike I-90 to Boston's Logan International Airport through a new tunnel under Boston Harbor. The project will reconstruct approximately 12 kilometer of Interstate highway-- half of which will be in tunnels. The project will keep the existing elevated roadway in service until the underground replacement is available for traffic. Of the many technical features of the CA/T project is the underpinning of the Red Line Transitway South Station which happens to be one of the more challenging and interesting construction operations.

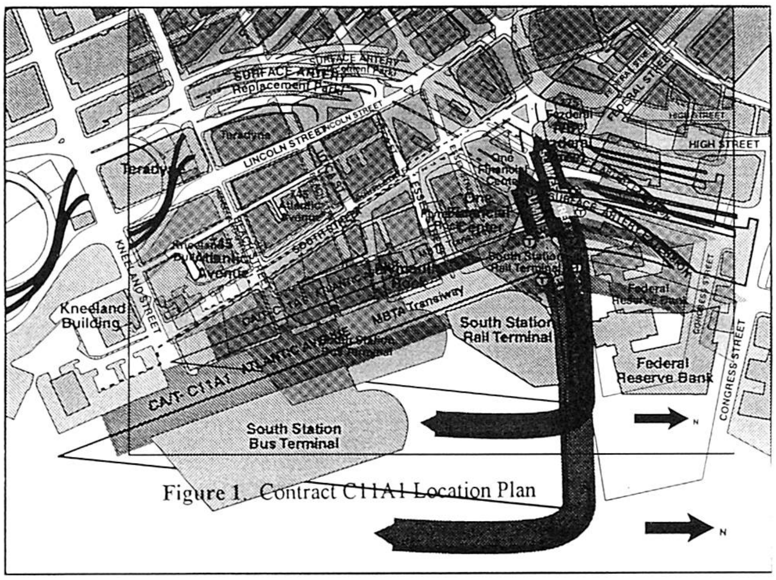

Figure 1. Contract C11A1 Location Plan

The underpinning of Red Line South Station is one of the components of C11A1 construction contract. Perini, Kiewit and Cashman, a Joint Venture (PKC) was the low bidder for a $378 million contract. The contract includes approximately 600 meter long of cut-and-cover mainline tunnel from Kneeland Street to Congress Street below Atlantic Avenue. Approximately 360 meters of the CA/T tunnel will support the new Massachusetts Bay Transit Authority's (MBTA) Transitway tunnel which will be used for electrically operated busses. Figure 1 shows the Contract C11A1 location. On the east side there are three major buildings - the recently completed two- story South Station Bus Terminal supported on pile foundation; the five story South Station Rail Terminal, approximately 80-years old and supported on timber piles; and the 32-story Federal Reserve Bank (FRB) building with a two level underground garage extending to the curb line. On the west side there are many old multistory brick historic buildings including the 10-story framed building at the intersection of Kneeland Street and Atlantic Avenue; the 46-story One Financial Center building and Dewey Square Tunnel. The Red Line underground Transit Station is located at the intersection of CA/T tunnel with Summer Street. Figure 2 shows an artist's rendering along the centerline of CA/T tunnel. At this location there will be three tier tunnels under the underground Red Line/ Transitway Lobby. The new Transitway tunnel along Atlantic Avenue crosses the existing Red Line Transitway at the mezzanine level of the existing station. Most of the existing Red line Station above the Red Line tunnel will be rebuilt.

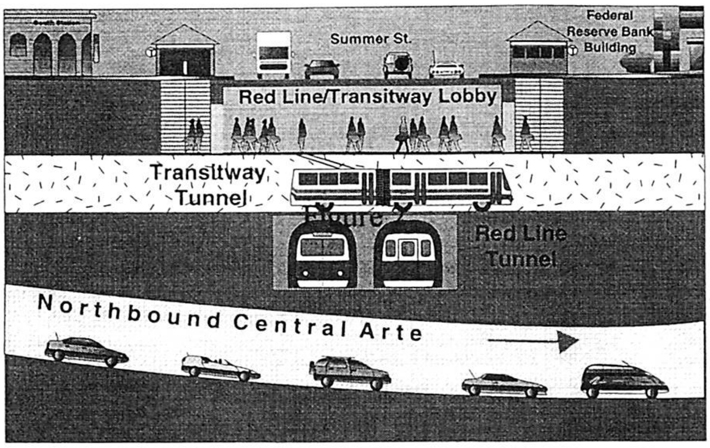

Figure 2. Artist rendition of CA/T Tunnel Crossing under MBTA Red Line Tunnel and Transitway Tunnel.

The northbound CA/T tunnel passes directly under the MBTA Red Line South Station, located below the intersection of Atlantic Avenue and Summer Street. The existing station is soil supported within the glacial till stratum. Underpinning of the existing station is required to allow excavation for construction of the tunnel structure directly below. The original underpinning system consists of post-tensioned concrete box girders jacked in place and supported on drift walls.

2.0 Site Geology

The subsurface explorations performed by GEI Consultants, Inc. from April, 1991 to March, 1992 indicated that the South Station is founded in glacial till above bedrock. The glacial till varies in particle size and density. Till in the area is very stiff and has a blow count of more than 50 (using a standard hammer), except in some pockets. Some boulders and cobbles are present in the Till. Due to a relatively high fine content, the till has low permeability.

The bed rock is classified in three categories, i.e. B1, B2 and B3.

B1	Completely weathered Argillite
B2	Severely or severely to moderately weathered Argillite
B3	Moderately or slightly weathered Argillite

Groundwater levels in the fill and organic soils are high and within 3-5 meters below surface. Piezometric water levels in the deeper observation wells are approximately 5-7 meters below grade. The Piezometric levels in the till and bedrock are lower than in the fill.

3.0 Design Criteria

The design of the underpinning structure was governed by the structural characteristics of the existing Red Line South Station foundation consisting of slab on grade, isolated column footings and the clearance between the bottom of the keel and the access tunnel. The settlement or camber of the keel and the slab supporting the tracks (both sides of keel) were important operational considerations for Red Line Transitway. Cracking of the slab would permit groundwater penetration thus flooding the station.

The protection of the station and other adjacent structures (5-story South Station Rail Terminal, Federal Reserve Bank, One Financial Center building and Dewey Square Tunnel) were of paramount importance. In addition, the MBTA station must remain operational throughout the duration of construction. Therefore, the following criteria was prepared in consultation with MBTA, the operating agency for the Red Line transit station:

Construction Conditions

	Threshold Value	Limiting Value	Operating Limits
Deflection	6-mm	10-mm	0
Camber	6-mm	10-mm	0

Maximum deflection was expected when the underpinning post-tension girders are fully tensioned, excavation for the CA/T tunnel has been completed, all post-tensioning losses have occurred and full dead/live loads are acting. Maximum camber was expected when the underpinning post-tension girders are fully tensioned, all post-tensioning losses have occurred and portions of Red Line South Station has been demolished for rehabilitation.

4.0 Existing Station Conditions

A review of the original 1914 design drawings indicate the following conditions.

1. The bottom slab reinforcing steel was placed in two layers; however, the reinforcing steel was not developed beyond the haunches. Therefore, the section cannot be considered as a reinforced concrete slab.

2. The main reinforcing steel parallel to the tracks, under the center wall were 3.81 meters long and centered under each column. Since the column spacing is 4 meters and the reinforcing is not continuous, the section cannot be considered as reinforced.

3. The column footings were individually placed on the till

4. The bending stiffness of the grade beam located in the center wall between the tracks is considerably reduced due to niches in the wall.

The most critical location or the area of possible cracking and potential water infiltration is the slab between columns.

The as-built drawings indicate that the strength of concrete is 20 Newton per square millimeter (n/sq.mm) but recent core tests from concrete from other structures of MBTA tunnel shows concrete strength in the existing structure of 40 n/sq.mm. Thus, the project utilized 30n/sq.mm as the average concrete strength to analyze the existing tunnel structure.

The analysis indicated that for a 10-mm or less settlement, no cracks are expected in the Red Line South Station bottom slab. This analysis is very conservative because it assumes that the station is simply supported with no soil springs. No analysis was performed for the structure above the base slab because the geometry of the structure is too complicated to obtain a reasonable solution and most of the structure above the roof of the Transitway will be replaced by a new facility.

5.0 Final Design

The detailed design and the preparation of the contract documents for C11A1 was carried out by Seelye Stevenson / DeLeuw Cather, a joint venture (SS/DC) under the direction of the Massachusetts Highway Department (MHD) and the Management Consultant, Bechtel / Parsons Brinckerhoff (B/PB). The final design consisted of two grouting/ access tunnels (east access tunnel, 4.88-meter wide X 4.12-meter high for jacking and west access tunnel 3-meter wide X 4.12-meter high for receiving) approximately 30 meter long directly under the Red Line Station and three drift walls on each side of the CA/T tunnel as shown in Figure 3. The drift walls were provided to support thirteen post-tensioned underpinning girders, 2.44 X 2.44 meter in cross-section with a clear span of 21.85 meters. The seven odd numbered girders jacked in place and the space between the jacked girders hand mined for the six even number girders. It was anticipated that the grouting galleries will be enlarged to form east and west access tunnels. The Contractor proposed to combine the excavation of grouting galleries and the access tunnels into one operation using 5.9-meter wide X 4.27-meter horse shoe tunnels; and substitute thirteen post-tension segmental girders with eleven cast-in-place post-tensioned horse-shoe shaped girders. The odd numbered girders have the same size as the original segmental girders, but the excavation in between is enlarged to 3-meters. The steel ribs in the crown extends beyond the effective girder depth. The girders were redesigned by SS/DC. Figure 4 shows conceptually the layout of access tunnels, multiple drifts and underpinning roof girders. The structural design of the post-tensioned girders included spare ducts for providing additional post-tensioning. It was recommended that the post-tensioning be applied in 20-percent increments symmetrical about the centerline of each girder and the tendon ducts grouted.

To monitor the movement of the existing structures and buildings around the South Station area, an extensive instrumentation program was developed. It included installation of Single Position Bore Hole Extensometers (SPBX), Displacement Monitoring Points (DMP), Vibrating Wire Strain Gauges, Tilt Meters and Piezometers. The SPBX's are also used as a bench mark for monitoring vertical movement. These instruments are read on a regular basis to determine the ground movement and groundwater levels.

6.0 Construction

The west and east access shafts 33.5 meter deep were constructed using slurry wall panels. The support of excavation include steel ribs at 900 mm on center and steel liners for the access tunnels, drifts 1 & 2 and the post-tension roof girders. The space between the steel liners and excavated ground was filled with pea gravel and cement grout soon after excavation.

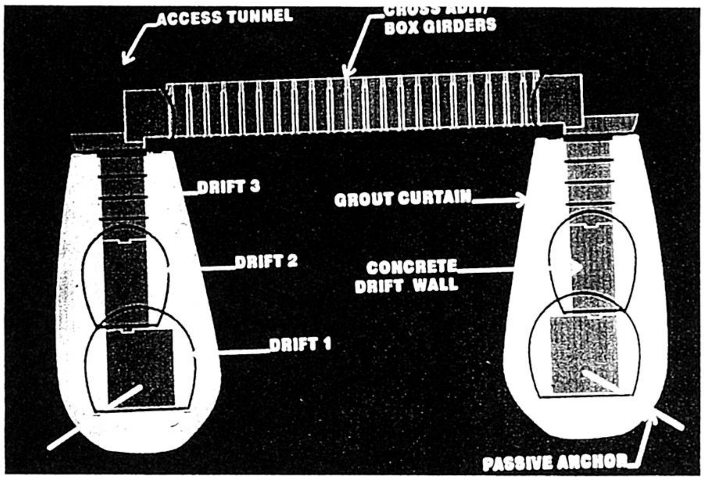

Figure 3. Cross-Section of Underpinning Systems with Drift Walls, Grout Curtain and Post-tension Box Girder Roof.

The access tunnels were excavated using multiple bench mining technique. The length of tunnel excavation in each bench was approximately 3 meters. The front face was breasted whenever the excavation was stopped for an extended period of more than 24-hours.

The water table was lowered 600 mm below the invert of the access tunnel excavation. At an average 6.3 liters per second, water was pumped from four dewatering wells. The dewatering provides good dry conditions for excavation and minimizes the chances of ground movement due to inflow of water. The access tunnel is used to grout its periphery and create an envelope around drifts 1, 2 and 3. Once grouting has been completed, the dewatering will be stopped to bring the water table to its natural position. Figure 3 shows the grout curtain.

Drifts 1 and 2 will be excavated from the access shafts but drift 3 will be excavated from the access tunnel. The supports for drift 3 consist of horizontal ribs and lagging. Then the odd numbered post-tension girders will be excavated from the access tunnels. The even number girder excavation will be supported from the framing of odd number girders.

7.0 Contingency Plan

The Contractor developed a contingency plan for the face stability during excavation. The plan includes the following steps. If the first step is not successful, the second and subsequent steps will be implemented.

I. Breasting the face
II. Shotcreting and/or additional pre-stabilizing grouting of the face
III. Pressure relief system
IV. Backfill excavation

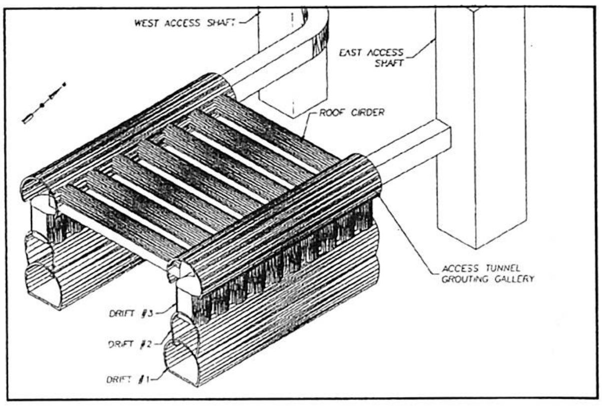

Figure 4. Conceptual Layout of Underpinning of South Station

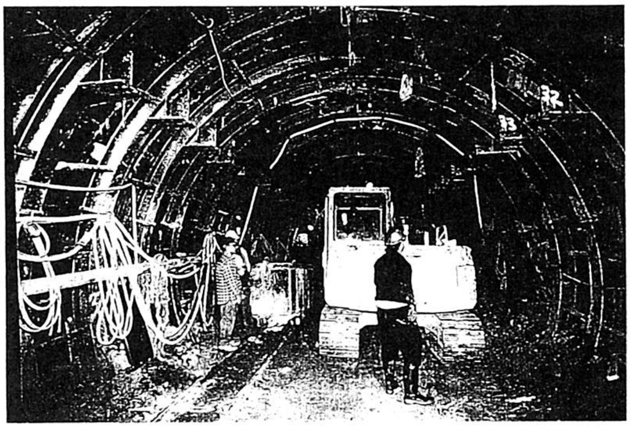

Figure 5. Access Tunnel Excavation

8.0 Conclusion

The contractor has completed construction of both access tunnels as of July 1997 and sodium silicate grout is being placed for the construction of drifts 1, 2, 3 and the post-tension girders. Figure 5 shows east access tunnel during construction with steel rib and steel liner support of excavation. In addition to sodium silicate grouting, the contractor has made multiple passes to grout the annular space around the steel liner of the access tunnel with cement grout.

Acknowledgments

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Study on Practical Application of the MMST Method

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Abstract

The MMST (Multi-Micro Shield Tunneling) method is a large-section tunneling method. Tunnel outer skin portions are excavated in advance by means of multiple small-section shields, followed by connection of excavated portions to make up external structures. Soil from inside the skin structure is then removed. Conventionally, large-section rectangular tunnels have been constructed according to the cut-and-cover method. The MMST method, on the other hand, enables construction of similar large-section rectangular tunnels without cut-and-cover, a singular advantage in projects requiring urban tunneling. This paper introduces the MMST method and reports on a construction test intended for future practical applications.

1. Introduction

Not a few expressway routes currently in service in city areas employ mainly tunnel structures. They are generally constructed according to a cut-and-cover method, but this method is not applicable to certain locations due to difficulties such as site, construction, and environmental assessment conditions. Moreover, restrictions including neighboring buildings and road alignment may make ordinary methods without cut-and-cover, such as shield method and NATM , inapplicable. In this background, a new method, not cut-and-cover, compatible with these severe conditions has come to be required.
A promising new method which can manage to achieve the target under the above restrictive conditions is the MMST method. However, no rational design and construction approaches have yet been developed and established at present, and this method is not yet been applied practically to tunneling up to now. Metropolitan Expressway Public Corporation in Tokyo has established a Research and Study Group Concerning Practical Application of the MMST Method to the Trans-Kawasaki Route (headed by Professor Konda of the Tokyo Metropolitan University). This group has conducted research concerning practical applications of the MMST method. Construction of the ventilation tunnel in the Daishi junction (temporary name) connecting the Trans-Kawasaki Route with the Yokohane Route has also been started as a construction test employing the MMST method(Fig.1) .

2. Outline of the MMST Method

2-1 Procedure and Features

The MMST method is implemented as described below (Fig. 2):

(1) Construction of individual small-section tunnel portions (shield excavation and arrangement of MMST steel shells)

(2) Connection of individual tunnel portions (excavation, reinforcing bar arrangement, and concrete placement between tunnel portions)
(3) Construction of the external structure (concrete placement in steel shells)
(4) Removal of soil from the inside of skin structure (mechanical removal with a back hoe)
(5) Internal construction work (intermediate slab, roadway surfaces)

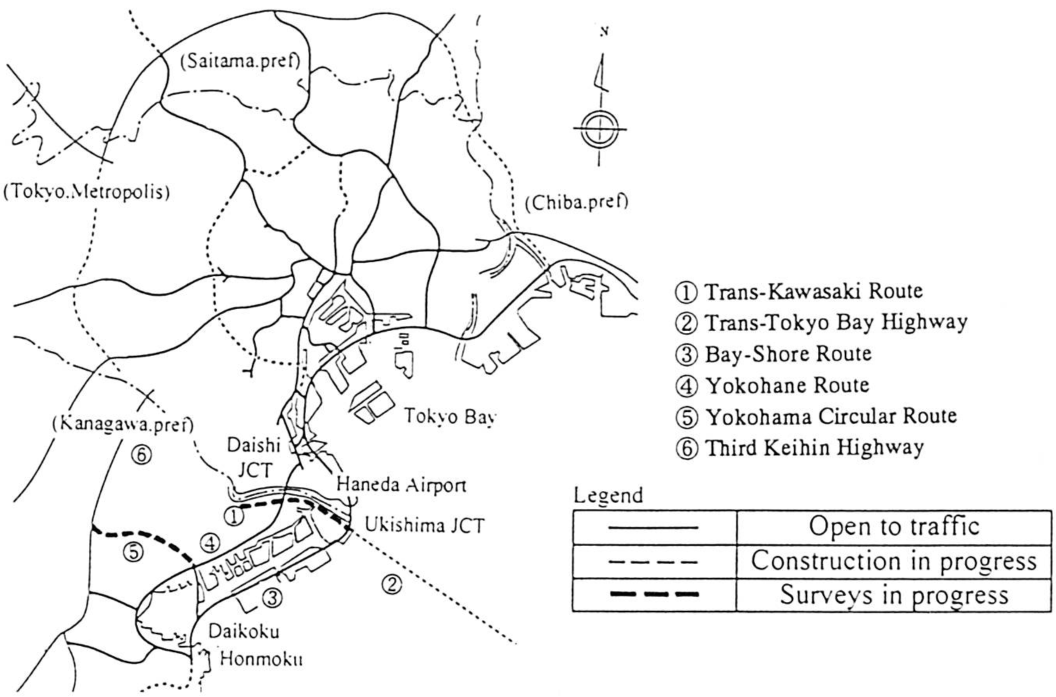


Fig. 1 Location map of Trans-Kawasaki Route

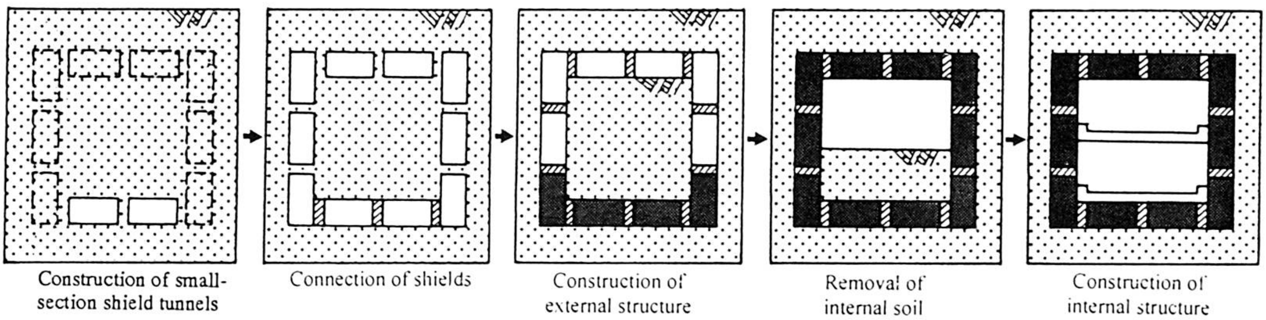


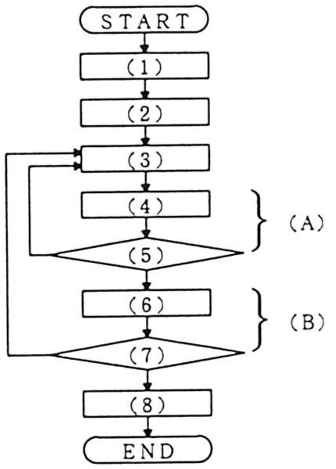
Fig. 2 Construction procedure according to the MMST method

Features and subjects of this MMST method are listed below:

- The MMST method can cope with changes in the tunnel section to a certain extent, which is favorable for matching to the road alignment.
- The use of small-section shields, which enables reduction of the size of shaft facilities while proving highly compatible with changes produced in the surrounding ground.
- Soil can be removed from the inside using an ordinary excavation machine, resulting in a decrease in industrial wastes from shield excavation.
- The shield is of a rectangular shape, long in a longitudinal or transverse direction, working in the extreme proximity of a neighboring shield.
- The external structure is of a construction in which SC and RC are combined. Besides, the force acting on members differs from one construction phase to another.
- The connection is made after removal of a part of steel shells from the tunnel inside, presenting considerable problems in terms of safety and workability. It is also necessary to take appropriate measures to adjust for errors during shield excavation.

2-2 Designing the MMST section

Fig. 3 shows the basic design flow of an MMST section.



(1) Individual tunnel portion arrangement plan
(2) Setup of design conditions
(3) Assumptions regarding the section
(4) Plane framed analysis of individual tunnel portions
(5) Verification of the stress
(6) Plane framed analysis of the MMST as a whole
(7) Verification of the stress
(8) Determination of the section
(A) The design of individual tunnel portions
(B) The design of the MMST as a whole

Fig. 3 MMST design flow

A structural system can vary greatly for each construction step. The design based on MMST is roughly classified into a design of individual tunnel portions during construction of an external structure and design of box culverts after its completion. The design of individual tunnel portions consists of designing MMST steel shells as individual shield linings on the basis of a plane framed model. In the design, four load cases shown in Fig. 4 are used. These are based on consideration of the effects of the neighboring shield according to the shield construction sequence. The design of the MMST as a whole includes the structural design of ordinary structures, which are steel-concrete composite members with concrete poured in steel shells, and connecting structures or RC members, according to the ordinary cut-and-cover tunnel design technique. Analysis is made using an overall frame model that takes the ground spring into account, as shown in Fig. 5, except that MMST steel shells are converted to reinforcing bars and designed as RC sections. The difference from the cut-and-cover tunnel is that the outer skin portions are completed first in the ground, followed by removal of internal soil. Therefore, the effect of unloading by removal of the soil, in other words, rebound expected during temporary construction of an ordinary cut-and-cover tunnel, is introduced as a load active on the tunnel invert. This effect is considered separately as a stress released effect during removal of internal soil.

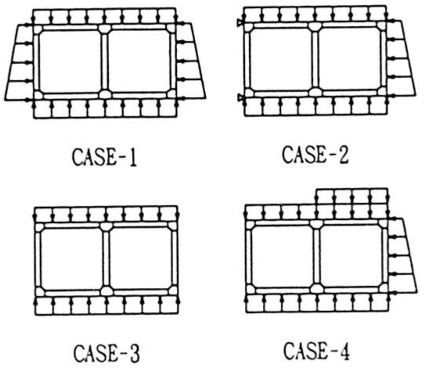


Fig. 4: MMST steel shell design model

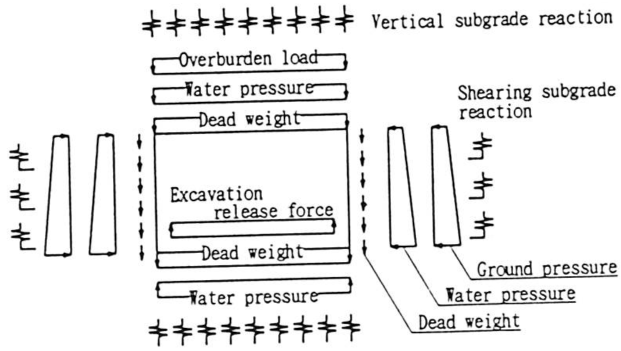


Fig. 5: MMST external structure design model

Figs. 6 and 7 show the construction of MMST steel shells and connections. MMST steel shells are made from main beams (angle steels) and skin plates (shell plates). To connect individual tunnel portions, part of steel shells is removed, and the RC structural member is constructed through excavation, a reinforcing bar arrangement, and placement of concrete between individual tunnel portions. Changing the length of connecting members can change the tunnel section.

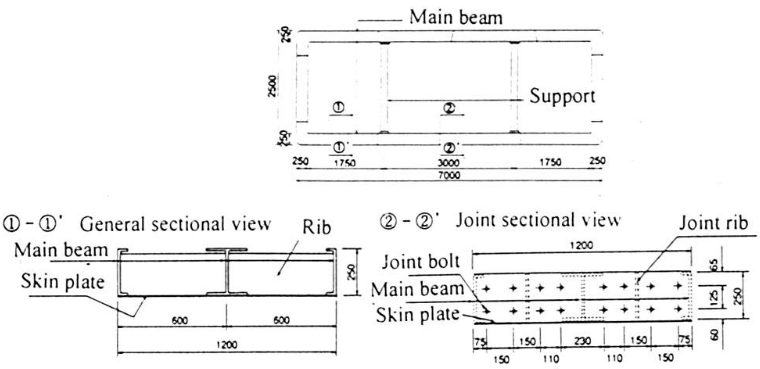


Fig. 6: Basic construction drawing of MMST steel shell

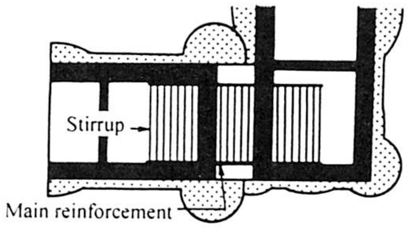


Fig. 7: Basic construction of the connection

3. Construction Test with the MMST Method

3-1 Objectives of the construction test

There are many problems to be solved before practical application of the MMST method is possible. These can be classified as follows : (1) Problems related to individual tunnel portions, (2) Problems related to connections between individual tunnel portions, (3) Problems related to the MMST structure as a whole, and (4) Other problems.

To solve these problems, the construction test was planned in parallel with numerical analysis and various experiments. Approaches including implementation of the construction test to address the above problems were classified into four kinds as shown in Table 1. Each approach plan was drafted: (1) Study through numerical analysis, (2) Study through elemental experiment, (3) Study through monitoring of construction test, and (4) Study based on the construction test state.

3-2 Outline of the construction test

The construction test using the MMST method was planned for a part of a ventilation tunnel connecting a ventilation station planned within a junction of the Trans-Kawasaki Route and the expressway tunnel. (Fig. 8)Three routes were decided upon as shown in Table 2. The following points were considered ;

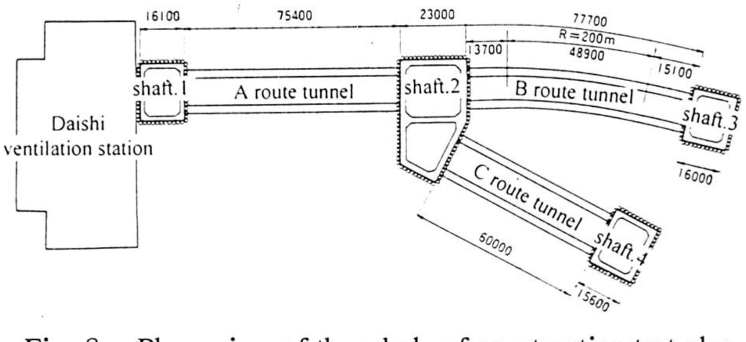


Fig. 8: Plane view of the whole of construction test plan

(1) Connection pattern of individual tunnel portions (a total of four types including longitudinal and transverse connections, and two types of corner connections),
(2) Change in the length of connection (1.6 – 0.5 m),
(3) Connector construction types (RC and PC constructions), and
(4) Linear and 200R curved sections of the plane alignment.

3-3 Specifications of MMST shield machine

The sectional shape of the tunnel being a super flat rectangle(aspect ratio approx. 1/3), the shield machines to be used for excavation of individual tunnel portions according to the MMST method had to ensure highly accurate position control, more than in the case of ordinary circular tunnel construction. This may be due to the extremely narrow distance between individual shields, and the fact that excavation errors may grow too much to offset errors in the joint and other structures.

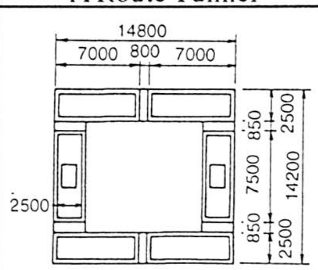
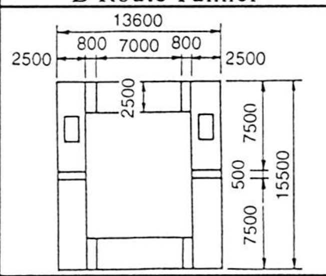
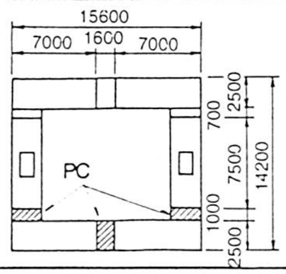
Shield machines prepared for the construction test included longitudinal and transverse shield machines for each tunnel. A drum cutter type slurry shield machine was prepared for the tunnel route A, a slurry earth-pressure type shield machine for the route B, and a slurry shield machine

Table 1: MMST Method Problem Factors and Countermeasures

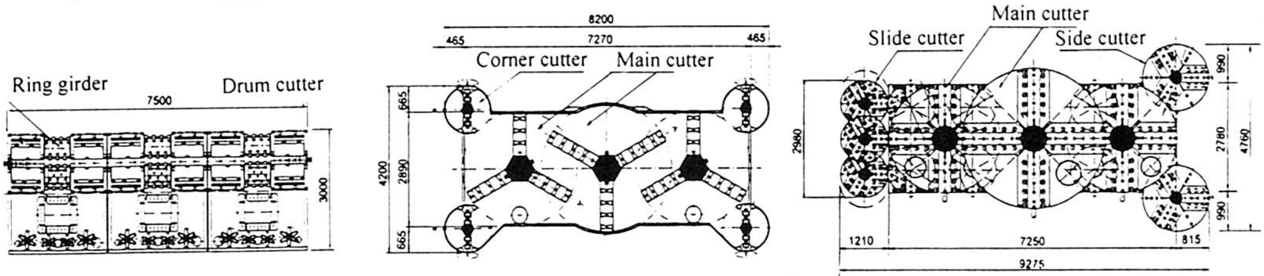
Problem factors			Countermeasures: Analysis	Countermeasures: Elementary experiment	Construction test: Monitoring test	Construction test: Construction state
Individual tunnel porting	D	Confirmation of MMST steel shell joint performance		O	O	
		Confirmation of active load during proximity construction of individual tunnel portions	O		O	
	C	Understanding of behavior of individual tunnel portions	O		O	
		Confirmation of position control of shield machine				O
Connection between individual tunnel portion	D	Confirmation of joint construction design method for linear connection	O	O	O	
		Confirmation of joint construction design method for corner connection	O	O	O	
		Confirmation of effects of individual tunnel portion construction error on proof stress	O	O		
	C	Confirmation of workability of connection		O		O
MMST tunnel	D	Confirmation of design method as a steel-concrete composite construction		O	O	
		Confirmation of effect of removal of internal soil	O		O	
		Confirmation of the stress of general members in each construction stage	O		O	
		Confirmation of opening construction design method in each construction stage	O	O	O	
		Confirmation of the effect of thermal stress generated after placement of structural concrete	O		O	
		Confirmation of the effect on overall system of advance stress occurring in steel shell during construction of individual tunnel portions	O	O	O	
		Confirmation of anti-seismic safety of overall system	O			
	C	Confirmation of workability as a whole				O
Others		Understanding of how ground deformation occurs in each construction stage	O		O	
		Confirmation of workability of ground and tunnel internal facilities				O
		Construction cost and process				O

* D: Design C: Construction

Table 2: Outline of the plan for each route

	A Route Tunnel	B Route Tunnel	C Route Tunnel
Sectional view / Outline of the plan			
Tunnel length	L=75.4m	L=77.7 m	L=60.0m
Tunnel outside dimensions	Height : 14.2m Width : 14.8m	Height : 15.5m Width : 13.6m	Height : 14.2m Width : 15.6m
Internal excavation sectional area	90m2	90m2	98m2
Excavation sectional area	210m2	211m2	222m2
Connection type	RC	RC	RCandPC
Overburden	4.7~6.9m	7.3~7.4m	5.1~6.0m
Horizontal alignment	Linear R=∞	Curved R=200m	Linear R=∞
Vertical alignment	3.0% down-grade	1.0% upgrade	3.0% upgrade

for route C (Fig. 9). Important investigation items for this construction test were the workability confirmation in terms of position control because this shield has to manage the super-flat rectangular section as well as confirmation of construction accuracy. Accordingly, each shield machine has various position control mechanisms for pitching, yawing, and rolling, which include a copy cutter, over cutter, slanting jack, articulation system, and movable flap.



(1) A route tunnel (2) B route tunnel (3) C route tunnel

Fig. 9: Horizontal shield machine

3-4 Monitoring plan

During the construction test, problems classified below were monitored:
(1) Understanding the load
(2) Understanding the behavior and structural characteristics
(3) Understanding of effects on the surrounding ground

For these problems, two monitoring sections were set for each tunnel. Measuring equipment shown in Fig. 10 were arranged for measurement for analysis and review as the work progressed.

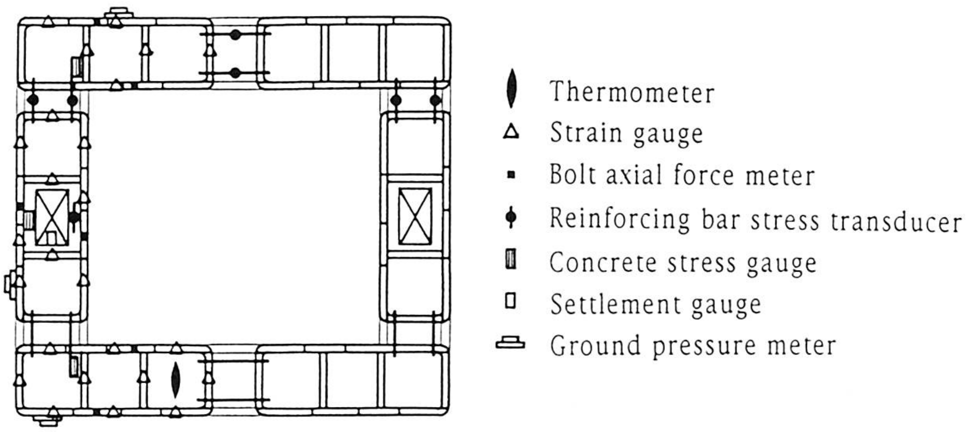


Fig. 10: Instrument arrangement plan

4. Summary

The MMST method is a new large-section tunneling method with superior features for urban areas with severe restrictions. Because of lack of a practical application record, and due to the complicated procedure, this method faces problems in terms of design and construction. This report briefly described various reviews and tests used to clarify these elements. As regards the construction test progress, the shaft was nearly completed in July 1997, and the shield machine, whose launch is scheduled for the earliest time, will be launched in September. The connections between small-section tunnel portions are planned for February 1998. In line with the construction test, various elemental experiments and FEM analysis are in progress. We plan to summarize these results to establish an economical MMST design and a construction method and practical application for the construction of Trans-Kawasaki Route.

Basel's New Motorway Link to France - Tunnel Construction Under Inner-City Conditions

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Franz A. Zahn, born 1954, received his civil engineering degree at Technische Hochschule Darmstadt (Germany) and his PhD at University of Canterbury (New Zealand). He is professor of civil engineering at Fachhochschule Konstanz (Germany) and works part-time for his former employer, Jauslin + Stebler.

Summary

The Kanton of Basel Stadt is presently building a new expressway linking the Swiss motorway A2 with the French A35, closing the 3.2 km gap in the motorway system which has created heavy traffic on inner city roads for long time. Approximately 2.5 km of the total length of the link comprise tunnels under the city. Owing to the varying conditions along its length, the structural design principles and construction methods of the tunnel vary considerably from one section to the next. The paper discusses some of the most interesting aspects of the design and construction of these tunnels, concentrating on three different sections, each of which features different problems requiring specific solutions.

1. Introduction

The city of Basel, situated at the juncture of Switzerland, France and Germany has to this day been plagued by extremely dense and heavy traffic on its inner city roads. This is due to the lack of an expressway linking the Swiss and German highway systems to the French motorway A35. Both traffic to and from France and locally generated traffic at present pass through the city, adding to by traffic from the increasingly important regional airport Basel / Mulhouse / Freiburg. To improve this situation, and to honour a bilateral agreement between Switzerland and France dating back to 1963, the Kanton of Basel Stadt is now building the missing link between the Swiss A2 and the French A35.

The 3.2 km long four lane dual carriage way will branch off the A2 via a set of curved viaducts which lead into a 1090m long tunnel under the part of the city located on the right bank of the Rhine. The road will surface to cross the river on a new 260m long bridge and disappear again on the other side in a second stretch of tunnel, 1430m long, which will end only 240m before the Swiss-French border control post. A total of five connections to the city's road system will also make the new expressway attractive to local traffic. The new link represents a total investment of about 1.1 billion Swiss francs, spent over a period of 10 years and is due for completion in 2005.

Due to its complexity, and in order to spread the work involved with the planning, design and construction of the expressway to as many local companies as reasonably possible , the owner has decided to divide the project into four sub-projects, each of which is again divided into a number of contracts . While the typical problems arising from inner city construction are common to all sections each one is faced with different local conditions, requiring specific structural solutions and construction methods. Three different types of tunnel construction are presented below. Within the scope of this paper, only the more interesting aspects of each method will be discussed

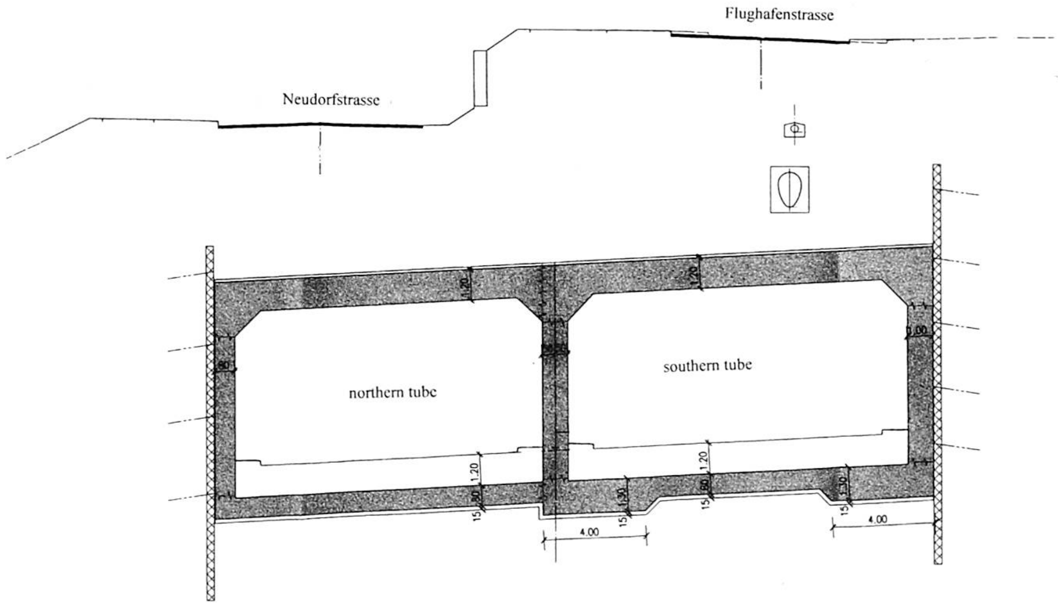
2.Tunnel 'Flughafenstrasse'

The author's direct involvement is limited to this 240 m long tunnel section under the two almost parallel roads leading to the airport and to the Swiss-French border control post. Since one of the two roads must be in operation at all times during tunnel construction, one tunnel tube has to be built ahead of the other. The finished tube is to be covered in order to rebuild the road above before excavation for the second tube can commence. Also, the overall construction program requires that the southern tube (inbound traffic) be in operation by the time work on the second tube of this section begins.

2.1 Structural Implications

Fig. 1a shows the typical cross section of the tunnel, Fig. 1b shows the cross section during the construction of the southern tube and Fig. 1c during the construction of the northern tube. During construction of the 10.5m wide second tube, the 14.5m wide inbound tube has to stand alone with an overburden of up to 9m, giving rise to very large bending moments in the frame corners and the middle wall. Also, shear forces are significant requiring shear reinforcement in the top slab.

To achieve full frame action during this governing construction load case, cut-and cover insitu concrete construction was chosen rather than top-down construction. The moment connections between the bored piles and the top slab would otherwise have been rather complex. Due to the proximity of the reconstructed road above the completed southern tube to the excavation for the second tube, the soil cover is secured using the principle of reinforced earth, allowing an almost vertical face (refer to Fig 1c). This causes the soil cover to exert a horizontal force on the tunnel roof slab, increasing the sectional moments and shear forces in the slab and middle wall. The ground anchors of the southern retaining wall are left in place fully stressed. This avoids significant unsymmetric lateral earthpressure which would further increase the frame moments and shears. The finally chosen cross section dimensions are indicated in Fig. 1a.



2.2 Partially Prestressed Roof Slab and Middle Wall

To limit the density of main reinforcement it was decided to partially prestress the roof slab and the middle wall of the tunnel using bonded prestressing tendons. Looped tendons are used for the wall, while the stressing anchorages of the slab tendons are placed in stressing pockets left in the top surface of the slab, alternate tendons being stressed from opposite sides. Fig. 2 shows the typical tendon layout. The reason for this layout is that the tunnel walls are directly cast against the anchored retaining walls of the excavation. The width of the excavation is limited by the proximity of the road and by property boundaries. It was not possible to leave a wide enough strip along the tunnel walls to allow the use of double breasted formwork.

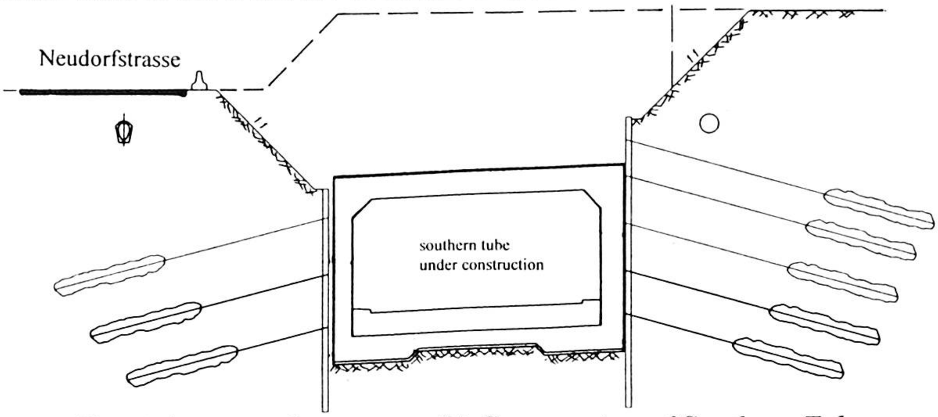


Fig. 1 (continued) *(b) Construction of Southern Tube*

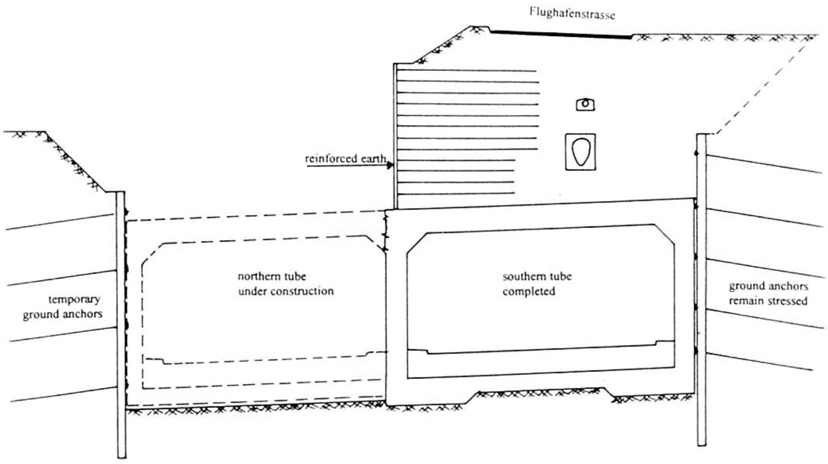


Fig. 1 (continued) *(c) Construction of Northern Tube*

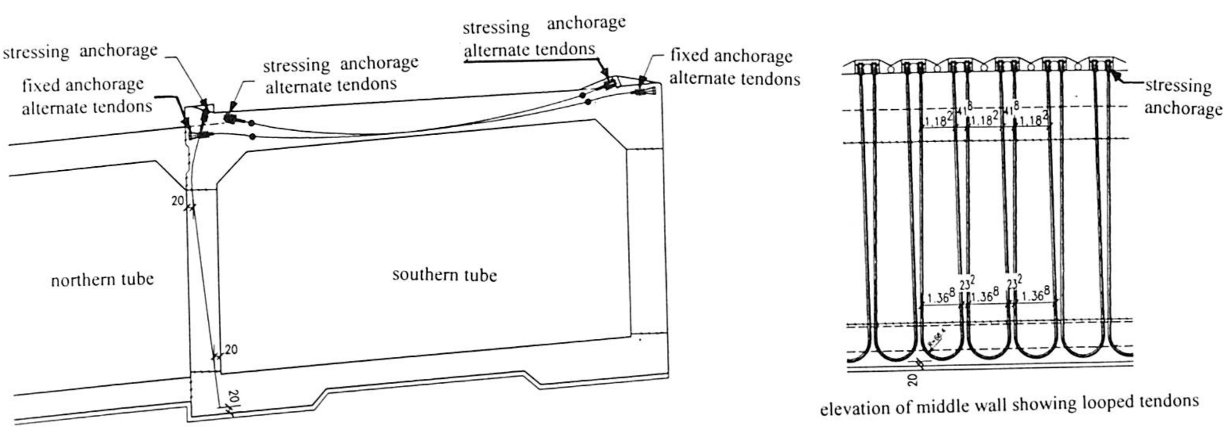


Fig. 2 Tunnel 'Flughafenstrasse': Layout of Prestressing Tendons in Typical Cross Section

3. Other Tunnel Sections

While the author is not directly involved, it is of interest to briefly present two more tunnel sections, where conditions completely different from those at the tunnel 'Flughafenstrasse' lead to quite different solutions. Both these tunnel sections are located on the right-hand side of the river and pass under densely built-up city quarters. The first, tunnel 'Horburg West' was designed by Aegerter & Bossardt, and the second, tunnel 'Horburg Mitte' by Walther Mory Maier. I would like to thank both these companies for making available the respective material.

3.1 Tunnel 'Horburg West'

This 363m long tunnel section under Horburgstrasse is characterised by the specific problems arising from the multitude of different underground services below street level, and from the rows of apartment blocks along either side of the street. Fig. 3 shows the typical cross section of the tunnel. The proximity of the foundations of the buildings to the outside walls of the projected tunnel requires underpinning measures before commencing any excavation work below foundation level. In some parts, the tunnel cuts up to 3.5m under the edge of the buildings. Two different methods of underpinning are employed: conventional section-by-section insitu concrete underpinning walls with temporary ground anchors, requiring relatively little disruption to the usage of the basements, and 15 to 25 cm ∅ micro piles installed from within the basements.

Major difficulties are caused by the various under ground services, which all have to remain fully operational during construction. These need to be temporarily diverted or fixed in place along the edges of the excavation. After completion they are rebuilt, and a services tunnel is constructed on top of the tunnel roof slab.

To minimise the duration of noise emission and disruption to traffic on this busy inner city street, the top-down construction method was chosen for this tunnel section. Three rows of ∅1.00m bored piles are installed, on top of which the top slab is constructed. This is carried out in two phases, as indicated in Fig. 4, to allow one half of the road to be operational at all times. After completion of the 1.20m thick top slab, the underside of which is at -5.0m below the road surface, the excavation is backfilled, replacing the underground services and building the new services tunnel. The tunnel is completed by excavating below the top slab to construct the insitu concrete base slab and the 30cm walls in front of the sheet pile walls. Since the ground water level is at approximately -8m, the ground water table only needs to be lowered by pumping during the construction phase below the tunnel top slab. After completion, a considerable part of the tunnel will be submerged in the ground water. A water-tight membrane is therefore placed around the entire tunnel cross section.

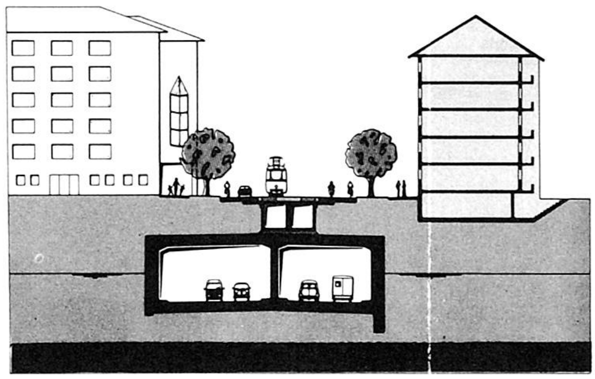


Fig. 3 Tunnel 'Horburg West': Typical Cross Section

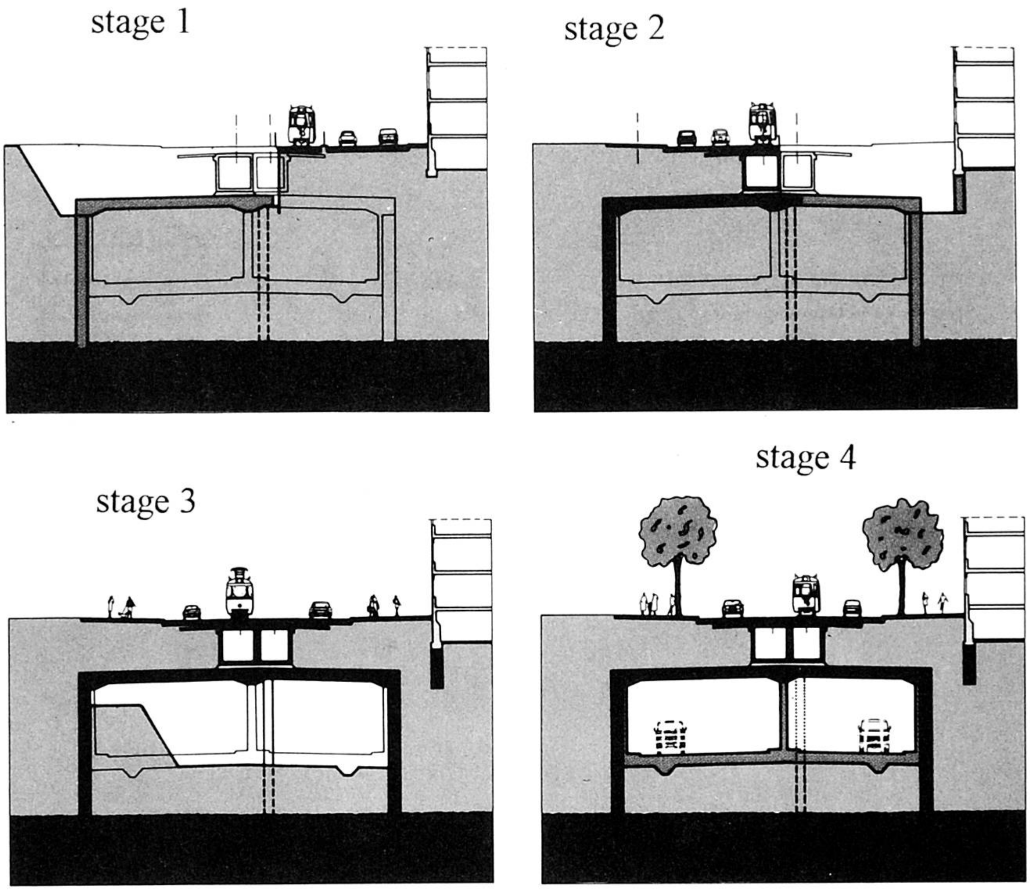
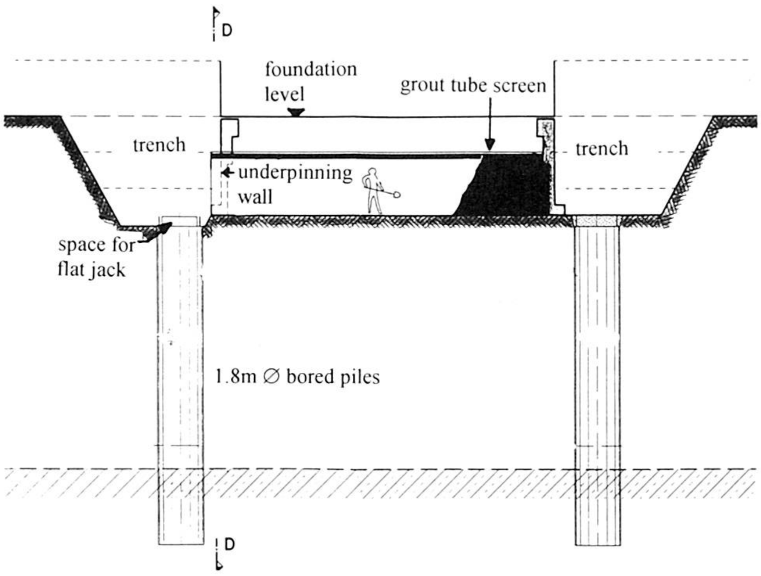


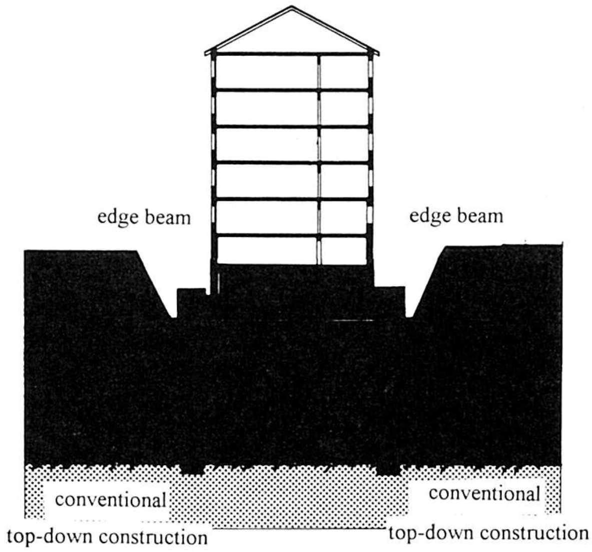
Fig.4 Construction Stages of Tunnel 'Horburg West'

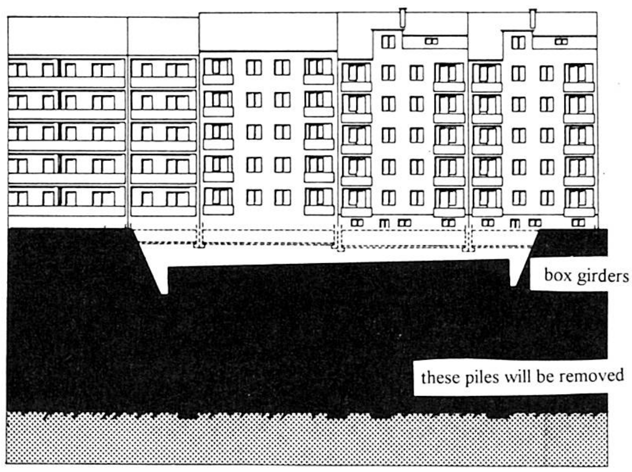
3.2 Tunnel 'Horburg Mitte'

This 247m long tunnel section is the continuation of the section 'Horburg West' and is of particular interest because it passes under a number of apartment blocks. Conventional tunnelling methods were not appropriate because the space between the top slab of the tunnel and the basement of the buildings is very small. The section 'Horburg Mitte' is also built using the top-down method of construction, hence the cross section is very similar to that of 'Horburg West'. And therefore only the specific problems of undermining the buildings shall be discussed.

Central to the selected solution is the construction of a transfer plate under the foundations. The principle is illustrated in the sequence of construction stages, Fig. 5. A trench is firstly excavated parallel to either edge of the basement (Fig. 5a). The foundations are underpinned in the process. These trenches serve two purposes: From one, injection tubes are driven horizontally under the basement to create a screen in the form of multiple cement grout arches. In addition, the soil between this screen and the basement is stabilised by cement grout injection. In the cover of the grout screen, 3.6m wide by 2.2m high tunnels are dug horizontally to build eight parallel partially prestressed concrete box girders which together form the transfer plate and the tunnel top slab (Fig. 5b). Shear keys and transverse prestressing serve to provide some load distribution between the individual box girders. After completion of the transfer plate, two partially prestressed edge beams, 2.8m high by 2.5m wide are built in the excavated trenches. These beams will later carry the load from the transfer plate to 1.8m ∅ bored reinforced concrete piles located on either side of the basement. Flat jacks are placed between the piles and the beams to transfer the load without undue settlements.







4. Conclusions

Three different sections of Basel's tunnelling project were presented, each characterised by very specific conditions and problems, leading to quite different structural solutions. At the time of writing this paper, the sections 'Horburg West' and 'Horburg Mitte' are well underway, while work on Tunnel Flughafenstrasse has commenced. So far, no major unforeseen problems have been encountered during construction.

The Design of the Øresund Tunnel Anchored Ramps

John Tomkins
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Symonds Travers Morgan
East Grinstead, UK

John Tomkins was awarded his Bachelor's degree in Engineering Science by the University of Warwick. He subsequently obtained Chartered Engineer status and a Masters Degree in Business Administration. He is an Associate Director, responsible for the bridge department. He headed the team designing all the approach works to the Øresund Tunnel.

Carl Valentine
Senior Engineer
Symonds Travers Morgan
East Grinstead, UK

Carl Valentine received his Bachelor's degree in Civil Engineering from the University of Loughborough and subsequently attained Chartered Engineer status. He is a Senior Engineer in the Civil Engineering Division. He was responsible for the design of all the Øresund Tunnel anchored ramp structures.

Gøran Nilsson
Block Engineering Manager
Øresund Tunnel Contractors
Copenhagen, Denmark

Gøran Nilsson received his Bachelor's degree in Engineering Science from Chalmers' University of Gothenburg. Prior to working on the Øresund Tunnel he was engineering manager for NCC Technology in Malmø. He was responsible for the design of all temporary works associated with the approach works to the Øresund Tunnel.

Summary

The anchored reinforced concrete approach ramps to the Øresund Tunnel provide the transitions from the earthworks cuttings to the tunnel portal. They carry both motorway and railway traffic.

The ramps are required to withstand the uplift forces from the hydrostatic pressures associated with the surrounding water table. The ramps, in conjunction with a watertight geomembrane, are also required to exclude water from the approaches.

The anchored ramp design was proposed at any early stage of the design and build tender preparation as a substitute for the gravity structures originally envisaged. This paper describes the main features of the design and how they evolved during detailed design.

1. Introduction

The 16km long Øresund Link between Denmark and Sweden comprises an immersed tube tunnel, a man made island and a series of bridges. The link carries a dual lane motorway and a twin track railway. The ramps described in this paper are part of the immersed tube tunnel contract which runs between a Peninsula of reclaimed land near Copenhagen airport and the man made island, known as Peberholm. The ramp structures extend for about 400 metres at each end of the tunnel to support the motorway and railway approaches.

The design and build tunnel contract was won by Øresund Tunnel Contractors (a joint venture of 5 major European contractors comprising NCC AB of Sweden (leader), John Laing Construction Ltd of England, Dumez-GTM SA of France, Boskalis Westminster Dredging bv of The Netherlands and E Pihl & Son A/S of Denmark). Symonds Travers Morgan were the designers during both the tender and detailed design stage. Site work started in September 1996 and is due for completion in 1999, prior to the installation of finishing works which will allow the link to open in 2000.

Early in the tender period it was found that the gravity ramp structures shown in the Owners' illustrative design would require significant excavation and subsequent replacement with concrete and ballast material. An anchored ramp solution offered savings in both material and time. The major design task was to find suitable details to provide a cost effective stable watertight approach structure. At the time of writing this paper the design is almost complete with the Owners' review still on-going.

2. General Description

2.1 Geology

The ground conditions are generally favourable for construction. From sea bed level downwards the sequence of the existing geology is marine deposits, glacial deposits and finally limestone to significant depth. The marine deposits occur in thin layers and are removed where they might affect any structures. Glacial deposits contain mainly clay till with some gravel, stone and sand. The depth of material is generally within the 2 to 5 metre range. The underlying limestone is characterised by silicification and by subvertical jointing. Near its top, there are glacially disturbed layers to a depth of 2 to 4 metres.

The founding levels of the Peninsula segments vary from about -3m from mean sea level (MSL) at the top of the ramps to -9m at the Portal entrance. The foundations are either within glacially disturbed limestone, glacial deposits, or limestone fill. On the Island the levels vary from -8m MSL at the Portal entrance to -3m at the top of the ramps. The foundations are within disturbed or undisturbed limestone, the glacial deposits or limestone fill placed as part of the Island reclamation works.

2.2 Structural Layout

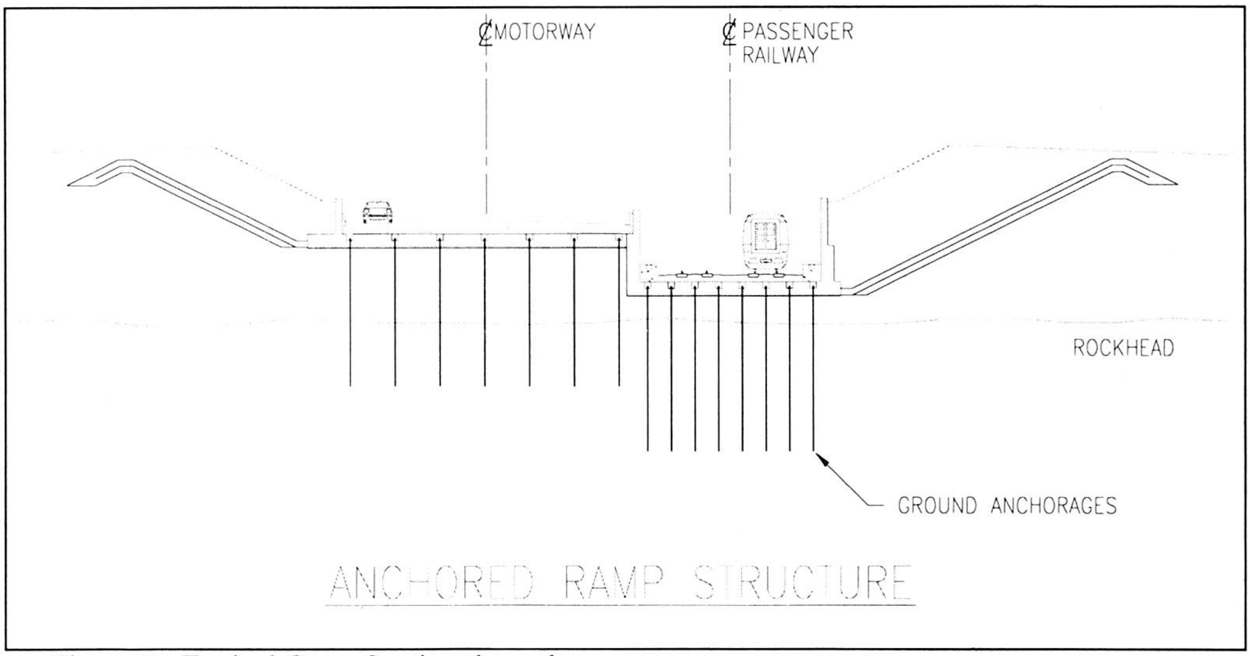

Figure 1: Typical Cross Section through ramps

The ramp base slabs and geomembrane are used to exclude groundwater from the approaches. They are subject to buoyancy forces which vary due to changes in the groundwater level predominantly caused by tidal movements, especially on the Island. Buoyancy effects are resisted by the weight of fill above the geomembrane and by the use of ground anchorages for the slabs. A total of approximately two thousand three hundred 36mm diameter grade 1080/1230 bar anchorages with lengths between ten and fifteen metres were specified for the ramps.

The vertical alignment of the segments follows the railway and motorway alignments. In cross-section the top of the motorway slabs is set horizontally below the motorway alignment by at least the full carriageway construction thickness. For the railway track the slab is set at a minimum of 800mm below the lowest rail level with a nominal crossfall to the south to provide drainage of the track ballast. The motorway is able to climb away from the tunnel at a steeper gradient than the railway. As a consequence the motorway ramps are always at a higher level than the railway ones and can be stopped sooner. A typical cross section is shown in Figure 1.

The ramps are divided into segments to mitigate the effects of shrinkage, thermal and ground movements and differential settlement with the adjoining segments. Various bay lengths were considered during the initial design for both the bases and walls. A typical length of 20 metres was eventually selected. Expansion joints in the base slabs subject to groundwater pressure are sealed with waterstops between segments. Waterstops in the transverse direction will be connected to the waterstops in the longitudinal direction via a crucifix section, to maintain a watertight seal. Shear keys are provided between adjacent sections to limit differential vertical movement between adjacent slabs and eliminate any possible harmful effects on the carriageway and railway construction.

3. Design

3.1 Loadings

A number of load cases and combinations based on the Eurocode were considered during the design of the segments. The individual loads are shown in Table 1 below.

Permanent Loads:	Variable Loads:	Accidental Loads:
Self weight Road pavement Railway ballast Hydrostatic pressure Ground anchorage loads Earth pressures Settlement Differential settlement Construction sequence	Nominal surcharge Road traffic loads Train loads Water level variations Temperature loads Snow loads	Accidental traffic collision Accidental train collision Earthquake

Table 1: Loads used in the base slab and ground anchorage design

For base and anchorage vertical displacements, the following ground conditions were considered:

- Settlement due to de-watering.
- Swell due to unloading during excavation.
- Settlement under permanent dead load including anchorage loads.
- Upward movement due to turning off the de-watering system.
- Creep settlement under permanent dead load including anchorage loads.
- Movement due to variation in the buoyancy forces.
- Self settlement of fill areas

3.2 Base Design

Typical base slabs, representative of groups of slabs, were analysed as 2-D plane finite element models. The slabs were supported on springs to model the ground conditions anticipated from the available data and from the additional ground investigation undertaken during the contract. The founding material comprises one or more of disturbed and undisturbed limestone, glacial till and limestone fill. For each material, a range of spring stiffnesses was considered. Additionally, the effects of possible variations in ground stiffness under a particular slab were considered by assigning different support stiffnesses to adjacent supports or groups of supports. All loads were applied to the model including the ground anchorage effects and the variation in groundwater pressures. The section design was completed in accordance with Eurocode 2.

3.3 Ground Anchorage Design

3.3.1 Alternative Types of Ground Anchorage Considered

During the tender stage a variety of solutions for resisting uplift pressures were considered and it was found that ground anchorages would be the most economical solution. At detailed design stage the option of using passive or prestressed anchorages was investigated. The passive anchorages have significant advantages over the prestressed anchorages from constructional aspects, but have a penalty in their generally livelier load-deflection characteristics. The inherent advantages of the passive anchorages over prestressed ones include:

- anchorages will not require stressing
- prestressed anchorage head pockets eliminated,
- steel fixing, shuttering, finishing work simplified
- a potential source of leakage removed.

For passive anchorages, the slab was found to be more susceptible to lift off from the adjoining ground with subsequent loads carried only on the anchorages. The resulting behaviour under live load was therefore almost independent of the soil and depended mainly on the stiffness of the passive anchorages. In the general case, the size of the imposed downward load needs to be assessed against the buoyancy forces to assess how the soil-anchorage system will behave.

The behaviour of the anchorages under train loads was a major consideration during detailed design, especially fatigue aspects. Based on the number of anchorages required to provide vertical equilibrium, movements were found to be typically in the order 10mm downwards under train loads. This magnitude of movement generates large stress changes within the anchorages and results in a severely limited number of fatigue loading cycles which they can withstand. In order to produce a satisfactory design, the number of passive anchorages would have needed increasing to approximately twice the number of prestressed anchorages required.

For prestressed anchorages, provided the precompression is greater than the subsequently applied upward loads, the soil stiffness affects the slab's behaviour. In weak ground, the spring stiffness of the anchorage plays a more important role in the behaviour and applied loads can cause a large change in stress in the ground anchorages. If the ground is good, as it generally is here, or even moderate the ground stiffness is dominant.

The final design adopted prestressed anchorages predominantly because they provided a more efficient solution against the train loads and the resultant risk of fatigue. The use of bar or strand tendons was also considered. For this scheme, where a large number of relatively lightly loaded and short ground anchorages were required, bars were considered the most practical and economical solution.

The design life of the anchorages, which will be either partially or totally within the groundwater range, is required by the Contract to be 100 years. The corrosion resistance of the tendon or the ability of the tendon system to exclude water and any harmful chemicals is a fundamental requirement in achieving the required design life. The selection of the tendon and corrosion protection system is therefore a major factor. The contract requirement adopts the approach of assuming that if the grout body of the anchorage is fully under compression then water will be kept away from the metal tendon within. A concern about this approach was that cracking can occur behind the end nut within the fixed anchorage with subsequent corrosion of the anchorage. The alternative principle of double corrosion protection was therefore adopted.

3.3.2 Detailed Analysis

A simple elastic model was established to investigate how the soil-structure system behaved under the design loading. This was used to assess the anchorage stressing loads such that the anchorages would not be overstressed due to subsequent upward loads and that lift-off of the slabs would not occur.

The soil and anchorages were modelled as elastic springs connected at the top by the slab, which was assumed to be infinitely stiff. Loads applied to the system cause deformations and result in changes in stress within the soil mass and the anchorages. The slab acts over a very large area compared to the depth of the anchorages, so one dimensional compression of the soil was assumed. The small deformations occurring in soils below the anchored zone were considered separately from the above using a simplified analysis. The effect of the increase in pore pressure resulting from the dewatering being switched off was modelled by applying an equivalent mechanical force to the system. The applied force equals the change in the 'effective stress' in the soil, which will change in volume accordingly (i.e. heave as pore pressures build up).

Allowance was made for the different behaviour of the system depending on whether the material behaves in a drained or undrained manner during anchorage stressing. If the soil behaves as 'undrained' during stressing, excess pore pressures will be set up which will slowly dissipate after lock-off leading to loss of anchorage force. This loss will not occur if the soil is 'drained' during stressing. The consequence is a difference in the final anchorage force with a possible overstress in the drained case but potential understress (i.e. possible lift-off) in the undrained case. In practice, the real behaviour will lie somewhere between the two extremes. The model takes this into account by allowing for different degrees of consolidation to apply during stressing, with further consolidation allowed to occur after lock-off if appropriate.

3.3.3 Calculation of Required Ground Anchorage Length

A ground anchorage consists of three main elements: a fixed length, a free length and the head. The free length was determined from the maximum of the following criteria:-

- A minimum length of 5 metres;
- Extending the distal end of the free length into sound limestone;
- Providing a total free and fixed length to mobilise sufficient mass of rock and soil to resist uplift.

For a 150mm diameter borehole and a conservative ultimate skin friction of 0.5 N/mm^2, the required fixed length was calculated to be 8 metres. However, it was considered that there was potential for reducing this length and so full scale pre-tests were carried out at locations near to the proposed alignment.

Six pull-out tests were carried out with fixed lengths ranging from 3 to 6 metres. To demonstrate that the fixed length was adequate, a pull-out load of three times the working load was used. The bar in the proposed works anchorage cannot accommodate this load, therefore nine 15.7mm diameter strands were used. In order to obtain comparable results with the proposed bar anchorage system, the same diameter corrugated sheath was used for the fixed length enclosure.

The ground anchorage installation and testing were carried out in accordance with DIN 4125. The tests demonstrated that a fixed length between 4 and 5 metres was capable of withstanding the ultimate pull-out force of 1876kN. It was decided to specify a minimum fixed length of 5 metres for the works ground anchorages to allow for variations in the limestone. To verify the pull-out tests and confirm the adequacy of the proposed ground anchorages for the main works, suitability tests were carried out in accordance with DIN 4125. These tests concluded that the proposed ground anchorages with a 5 metre free length and a 5 metre fixed length complied with the requirements of DIN 4125.

4. Conclusions

Anchored ramps can provide an extremely cost-effective solution to the problem of buoyancy effects where suitable ground conditions exist. The problem of heavy cyclical loading associated with railway loading requires special attention in developing an appropriate design. Each site with its unique ground and loading conditions requires careful consideration in developing a solution.

Design and build has benefits in stimulating considerations beyond the usual boundaries imposed on designers and contractors and permits investigation of real cost-effective design.

Challenging Use of Immersed Tubes in the Fort Point Channel, Boston, MA

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Summary

This paper is focused on the design challenges and solutions related to the Fort Point Channel immersed tubes, the first large concrete tubes in the U.S. Six tubes will be fabricated in three stages in the adjacent casting basin. Later the casting basin will be used as an excavation support for the cut-and-cover tunnel.

The tubes will be placed in two parallel rows at a skewed angle across the Channel. Dredging to a depth of 4.8m (16 ft.) will precede the placing. The constructed tubes will be placed over an operating subway tunnels just with a 1.5m. (5 ft.) vertical gap between the existing and the new tunnels. Construction of the large immersed tubes in restricted conditions created a several challenging technical problems which have been solved during the design and are discussed below.

1.0 Introduction

The Fort Point Channel (FPC) tunnels are one of the most technically challenging part of the multi-billion Massachusetts Highway Department's (MHD) Central Artery (I-93)/Tunnel (I-90) Project in Boston. The FPC crossing is located in close proximity to the viaduct of I-90/I-93 Interchange and just about 91.4m (300 ft.) from railroad tracks of the South Station. The FPC is also an important historical site in downtown, placing serious restrictions to the roadway alignment. This required that the roadways in this area had to be designed as a tunnel.

Tunnel design was complicated by many factors. One factor is the presence of two existing MBTA Red Line subway tunnels in the FPC. These tunnels were built in 1915 by the shield method under compressed air, and are 7.3m (24 ft) in diameter. Each tube has a primary wooden liner and the 0.3m (2ft)- thick secondary liner from unreinforced concrete.

2.0 ITT Alignment

Several tunnel alternatives for crossing the Red Line tunnel were considered. Cut-and-cover tunnel construction in a dewatered cofferdam was unacceptable because removal of surcharge and dewatering would have caused excessive soil displacements which the existing tunnels could not withstand. Therefore, the immersed tube method of construction (ITT) has been chosen over the Red Line Tunnel. The ITT alternative also helped to mitigate the problem of global stability of the soft Boston Blue Clays as will be discussed below. The proposed roadway alignment crosses the 1133m (400ft)-wide Channel with a skew of 45 degree angle and includes five roadway lanes for the West bound and six lanes for the East bound.

Two parallel rows of concrete rectangular immersed tubes with a width up to 48.5m (160 ft) each, accommodate this alignment. The rectangular shape of the tunnels also satisfied limited clearance between the navigational channel at El.87 and the top of the Red Line tunnel at El.56. The vertical size of the tubes was limited to 7.9m (26 ft) just to accommodate a minimal vertical roadway clearance ,4.3m (14 ft), and to provide minimum soil cover over the existing Subway tunnels, 1.5m (5 ft).

The length of the alignment for the immersed tubes (ITT) was reviewed several times during the design. At the beginning the intention was to use the immersed tubes method of construction only to cross over the Red Line Tunnels without subjecting the tunnel to the negative impact of the dewatering of the underline soils. Two parallel tubes, 121.2m (400 ft) long, would satisfy this purpose. However, later, it was recognized that the excavation support system for the westerly adjacent cut-and-cover tunnel would be excessively expensive due to instability of the deep Boston Blue Clay in this area. To avoid dewatering it was decided to use dredging and extend the immersed tubes an additional 91m (300 ft) further west. During the dredging and placement of the ITT the hydrostatic pressure is permanently applied against the unbalanced soil pressure, to mitigate instability.

The eastern closure also caused a problem. Because of the skewed ITT alignment, the eastern seawall, which will be constructed on the ITT roof, is also skewed to the ITTs. A significant unbalanced horizontal hydrostatic force would be applied transversely to the eastern ends of the ITTs. To minimize this force it was decided to extend the ITTs further into the casting basin. This solution also significantly improved the construction schedule, allowing the early construction of the cut -and-cover tunnel in the dewatered eastern portion of the casting basin. The final length of the ITT alignment was increased to 333.3m (1100 ft) , and six tubes instead of two tubes have been proposed to cross the Channel. (See Figure 1.)

3.0 Ventilation Building

Extension of the ITT's also helped to successfully solve a problem requiring deep foundation under the Ventilation Building #1, which has to be located on the west shore of the FPC. The challenge was to build the underwater portion of the building in the dry, means as a snorkel on the top of the ITT constructed in the casting basin. This allows for the elimination of the very costly and elaborate cofferdam. The foundation for the ventilation building was designed as two independent rectangular snorkels which are constructed on two parallel immersed tubes (See Figure 1). The snorkels are basically extensions of the tunnel external walls, the appropriate transverse walls, creating a large open well. The tops of the snorkel walls project above the high tide water elevation, allowing subsequent construction of the building in the dry. The snorkel walls carry all loads from the Ventilation Building, transferring them into the immersed tube tunnel foundation system (See Figure 2).

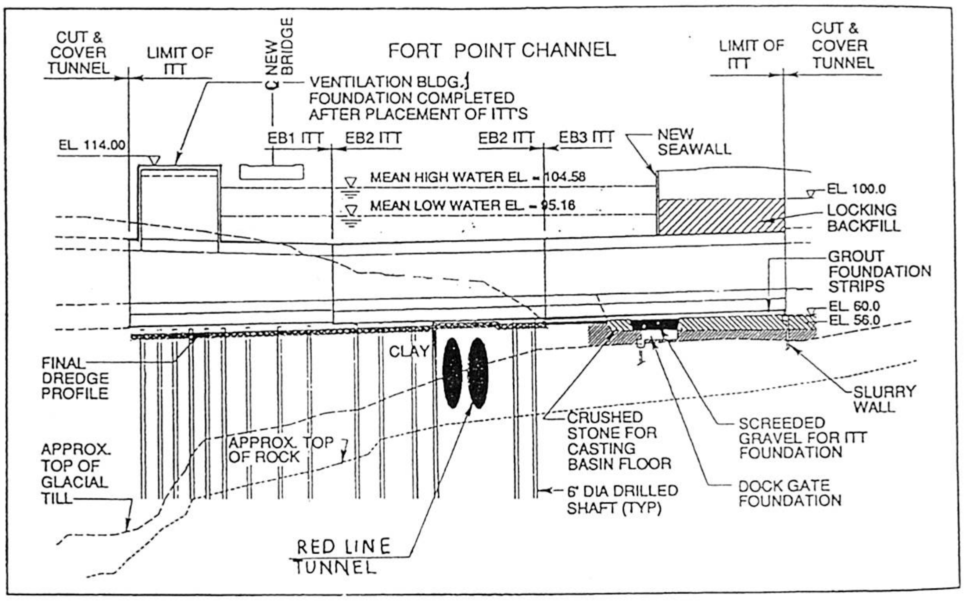

Figure 2. Longitudinal section I-90 EB in the FPC

4.0 Cofferdam

Normally immersed tubes are fabricated in offsite shipyards. In this case it was unacceptable because three upstream existing bridges lack sufficient navigational clearances for such large tubes. Therefore, it was decided to utilize the adjacent excavation of the future cut-and-cover tunnel. The available sizes for a long term excavation were not enough for fabrication of all six tubes. So, the tubes will be fabricated in three stages assuming subsequent dewatering and flooding of the basin after each stage. The casting basin, 19.7m (65 ft)-deep, was formed by reinforced concrete slurry walls with the tie-backs and the 19.7m (65ft)-diameter cellular cofferdams. The cofferdams separate the casting basin from the FPC and will be partially removed and restored after each float out. The removal of several cells provide the exit for the floating tubes from the casting basin in the Channel.

5.0 Foundations

Two types of foundations have been utilized for the immersed tubes. Two western tubes and two middle tubes are supported by drilled shafts embedded into the bedrock. Two eastern tubes have a base foundation (See Figure 2).

For the first 2 tubes the drilled shafts were required to avoid the differential settlements induced by the presence of the Ventilation Building atop the tubes. For the second 2 tubes the drilled shafts were required to provide protection for the Red Line tunnels from overloading due to accidental flooding of the ITT. Flooding of the ITT's would cause the subgrade load to increase by up to ten times and, as the analysis indicates, the subway concrete liner would be significantly overstressed. Hence, the tunnel was designed to span the Red Line Tunnels in this case and be founded on the drilled shafts.

For all four tubes, the drilled shafts were positioned under the ITT longitudinal walls. This allowed to minimize the reinforcement in the tube base slab, and also considerably simplify the performance of the connections between the ITT and the drilled shafts (up to 40 shafts for each

ITT). The 70mm (4in)-diameter plastic sleeves, will be positioned in the walls during ITT fabrication. These sleeves allow for grout to be placed in the gap between the ITT and the shafts directly from the ITT roof (See Figure 3).

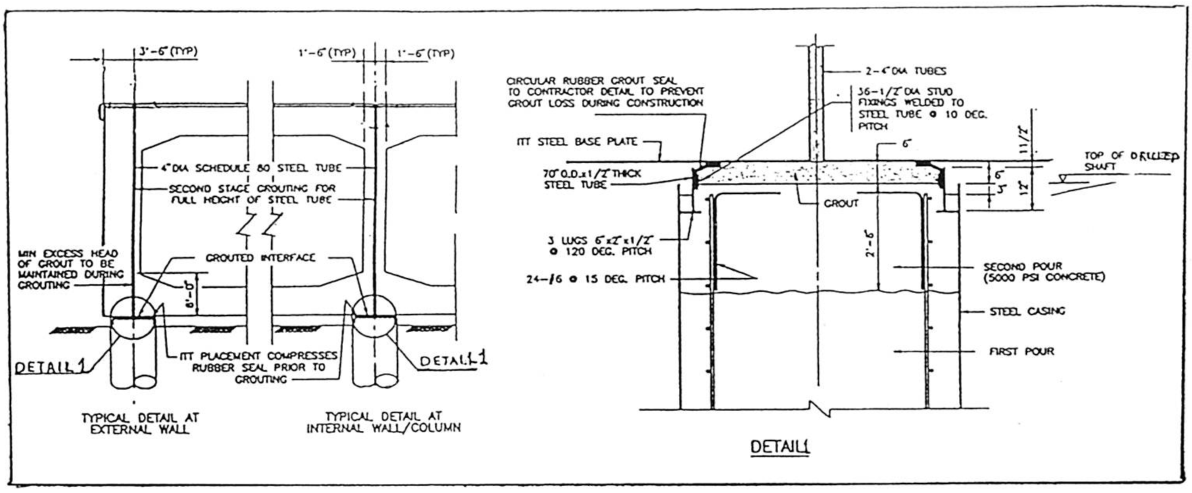

Figure 3. ITT - Drilled shaft Connection

To bridge the Red line tubes, the 36.4m (120ft.) span between the shafts must be provided. This span has different locations under each wall since the Red Line tubes are at an acute angle to the ITT alignment. This required that the tube be designed for torsion. Three dimensional finite element analysis was used to locate the drilled shafts at positions where the loads would be as equal as possible. Several runs were required to reach this objective. The final layout of drilled shafts is shown on Figure 4.

As noted previously two eastern ITT's placed in the vicinity of casting basin have a base foundation. The properties of underline soils in this area are much better than on the eastern side of the FPC. The glacial marine (till) and bedrock stratums here approach the tunnels invert elevation (See Figure 2). Several base support systems have been considered during the design, including sand jetting, flowable sand, gravel or pea-stone bed and the grouting. It was recognized that the grouting alternative is more acceptable for this particular case. The sand alternative was rejected because of potential liquefaction during earthquake. The successful placement of screeded gravel base under such wide parallel tubes was also a concern. Also access for screeding equipment into the casting basin is problematic due to the proximity of the previously placed tubes.

The grout foundation was designed as several longitudinal strips, which are located under the external and the internal walls. The required width of the strips under the external and the internal walls are 3.3m (11 ft) and 4.5m (15 ft.) respectively. A special inflatable grout bags with the sizes 3m x 4.5m (11ft x 15ft) and 4.5m x 6m (15ft x 20ft) will be used to form these strips. The bags will be attached to the tube bottom (steel plate) and properly secured before the concreting of the tubes has begun. Similar to the other tubes, the plastic vertical sleeves will be embedded in the walls to supply each bag. Using a specific sequence, the bags will be filled with grout while the ITT is temporally supported on jacks. After the grout is cured the temporary jack-supports will be released and the load from the ITT will be transferred to the foundation.

6.0 Floating and Placing

The combination of such factors as the relatively short distance of the ITT alignment 333.3m(1100 ft), large width of the ITT's up to 48.5m (160 ft), narrow channel 121m (400 ft), and small gap between the ITT rows 1.2m (4 ft) cause a several construction challenges. Six large tubes have to be placed in this very restricted area. Some of these challenging problems are discussed below.

- The traditional methods of towing and placing immersed tubes are not applicable in this design. A special system of cables and winches, which propels the floating of tubes from the casting basin to their project position, has been developed. This system assumes numerous locations of the anchor points all around the FPC and also the installation of the anchor tower in the middle of the Channel.

- Because the last two tubes have to be placed partially in the Channel and partially in the casting basin (after final removal of the cellular cofferdam), the WB2 tube has to be temporary stored in the Channel to give the other one (EB2 tube) the priority for maneuvering and placement (see Figure 1). The storage area has to be predreged to allow the moored tubes to float during low tides. This area accommodates two tubes to give the flexibility for construction schedule in case if the drilled shaft foundation should not be completed in time.

- Considering the narrowness of the channels flat pontoons or vertical cylinders should be used for the tube immersion. The pontoons or the cylinders will be installed on the top of the ITTs in the casting basin. During immersion of the two western tubes, the snorkels (underwater part of the Ventilation Building) will act as a large vertical cylinder (See Figure 5).

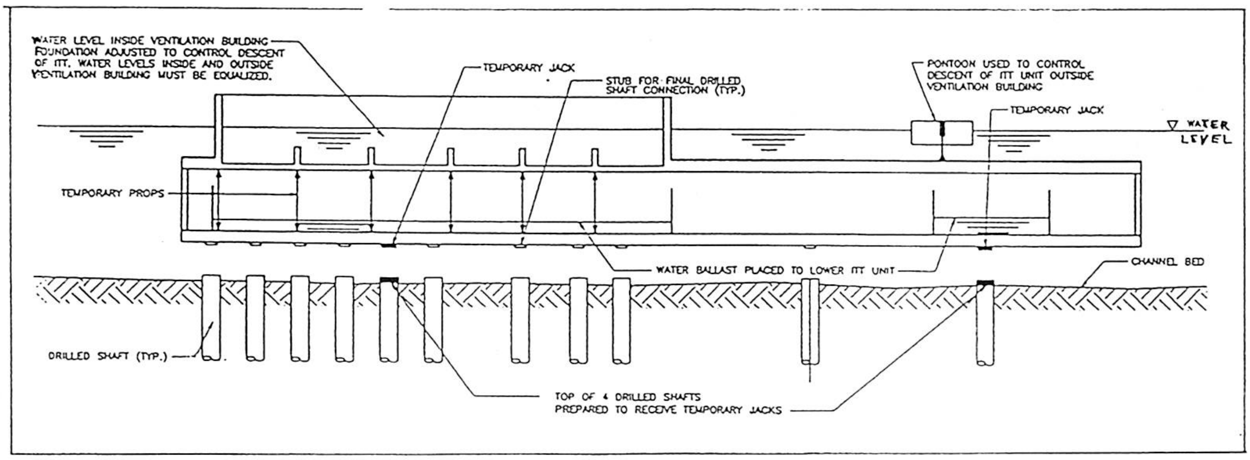

Figure 5. Immersion of the ITT with snorkel

- A challenging placement procedure has been developed for the west bound (WB) tubes, because east side of the WB2 tube has to be connected to both, the previously built cut-and-cover tunnel and to the WB1 immersed tube (See Figure 1). At first, the WB1 tube will be floated from the casting basin and placed forward of it's final position, ballasted with water and not connected to the drilled shafts. Then, the WB2 tube will be floated from the casting basin, placed into position, connected to the receiving cut-and-cover tunnel through the Gina seal, and then connected to the drilled shafts as shown on Figure 3. After that, the WB1 unit will be deballasted to be neutrally buoyant and will be connected to the WB2 unit through the Gina seal. In this operation the WB2 tube serves as a receiving unit. Even though the units are connected unsymmetrically, theoretically they are perfectly balanced. However, for safety reasons, the WB2 tube will be preballasted to increase it's stability prior to receiving the WB1 unit.

- The final stage of the ITT construction, the east and west closures, also presented a challenge. The necessity to maintain minimum vertical navigational clearance preclude placement of the backfill on the top of immersed tubes. The friction forces underneath the tubes, which are necessary for the ITT horizontal stability, is minimal. On all stages of placement the tubes are not subjected to any unbalanced horizontal forces. However, when all units will be in place and the cut-off walls will be erected on the western and eastern tubes to provide the dewatering of the east and the west closures, the situation is changed. A significant horizontal longitudinal unbalanced force directed to the west is developed. This occurs because the east and west tubes have different areas exposed to the hydrostatic pressure. Analysis indicates, that the friction forces underneath preballasted tubes, and the friction forces between the walls and the locking fill, do not safely counteract the unbalanced force. To gain a sufficient factor of safety against tube sliding the following measures have been provided.

1.The dewatering of the east and the west closures will be simultaneous with the strict control to keep the water table in the closures equal. The allowed diversity in the levels is 1.5m (5ft).

2. Temporary steel straps will be installed across all joints between the tubes, to protect the Gina seal from decompression.

3. All tubes will be sufficiently preballasted by the water tanks during dewatering of the closures.

4. To increase the safety factor against sliding, the shear keys were introduced in the connections between the western tubes and the drilled shafts.

7. Conclusion

At the time this paper was written, the Final Design of the Fort Point Channel Crossing has been completed. The Bechtel/ Parsons Brinckerhoff (B/PB) Joint Venture performed the preliminary design as well as 25% of the final design for the Fort Point Channel Crossing. ACER Engineers and Consultants Co. provided the final design for the immersed tubes and B/PB has coordinated and reviewed the final design. Recently the Massachusetts Highway Department awarded the Construction Contract to the "Modern Continental Construction Co. Inc. ". At the present time, the contractor is in the process of reviewing the proposed construction methods with the intent to reduce the construction cost and schedule. Presently, the casting basin is nearing completion and construction of the immersed tubes will start in the September 1997.

Acknowledgments

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List of Figures (5) for the paper "Challenged Immersed Tubes in Fort Point Channel, in Boston

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Geotechnical Instrumentation Supporting Central Artery Construction

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Summary

This paper focuses on the extensive geotechnical instrumentation program developed to monitor and control the effects of construction of MHD's Central Artery (I-93)/Tunnel (I-90) Project in Boston.

1.0 Introduction And Background

Boston's massive and complex CA/T Project must be designed and built with minimum disruption to existing infrastructure including buildings, subways, railroads, utilities, bridges and viaducts. From project inception, integration of design and construction were emphasized, and a proactive philosophy of designing to prevent soil deformations was pursued. To ensure that strict control and monitoring criteria were achieved, an extensive instrumentation system was designed and is being implemented.

This paper presents an overview of the project and defines the technical instrumentation needs generated by the adverse geological settings, old infrastructure, and abutting structures. Key monitoring criteria are stated, along with current experience of designing and implementing the monitoring system. Since portions of the system have been in place for some time, several valuable "Lessons Learned" are discussed, and several planned future developments presented.

2.0 Project Description And Instrumentation Requirements

The CA/T Project represents a challenging endeavor from a geotechnical engineering standpoint. Few, if any, single projects in the United States encompass both the variety and magnitude of underground construction as the CA/T. Significant features include: approximately 4 miles of cut-and-cover tunnel construction within a heavily congested urban environment where dozens of buildings lie within the zone of influence of construction. There are six locations where tunnels will be constructed either above, below, or adjacent to active underground rail transit facilities, an immersed tube tunnel, and viaduct piers with vertical and horizontal loads in the thousands of kips.

The instrumentation program is the primary means of monitoring construction activities which can result in detrimental impacts to surrounding facilities. Accurate and timely reporting of data is essential to meeting the objectives of the instrumentation efforts.

The generally adverse ground conditions found in the Boston area make the geotechnical efforts even more challenging. The high groundwater table, deep deposits of soft soils, and the presence of permeable strata combine to create an environment with the potential for large deformations resulting from excavation activities. In addition, virtually the entire project alignment falls within reclaimed land, filled by numerous processes and materials, many of which are not well documented.

The generalized profile consists of fill, soft organic and silty clay deposits, dense glacial deposits, and bedrock. In some areas, the soft silty clay extends to depths in excess of 150 ft. The bedrock properties are extremely variable, ranging from totally decomposed to very hard and massive.

Control of deformations and maintenance of prevailing groundwater levels represent the primary geotechnical objectives in CA/T tunnel construction and thus define the role of the geotechnical instrumentation program. Soft marine clays and organic deposits encountered throughout the alignment are particularly sensitive to changes in groundwater conditions. Structural foundations founded above, on, or within these strata become subject to potential deformation. Additionally, existing timber pile foundations exposed to air for extended durations as a result of lowering of the groundwater table will likely deteriorate.

Many buildings are designated as historic, and must be protected from even architectural damage, century-old transit tunnels must have structural and waterproofing integrity maintained, and multistory office buildings must not experience deformations that could over-stress structural members. In all cases, tolerances for allowable deformations are very small and thus deformations must be measured to a high degree of precision.

Construction vibrations represent another source of potential distress to adjacent structures and business activities. Buildings in poor structural condition, with attached facades, or housing sensitive electronic equipment must be protected by near real time monitoring and reporting of vibrations. The geotechnical instrumentation program is the key element of the mitigation effort protecting adjacent facilities.

3.0 The Monitoring System: Design And Implementation

Initial geotechnical site investigations were performed by Area Geotechnical Consultants (AGCs). The AGCs produced investigative data and engineering reports identifying and evaluating possible impacts of construction and subsurface conditions on adjacent facilities.

Section Design Consultants (SDCs) used these reports to create final designs for particular sections of roadway and also to design the geotechnical instrumentation program. The geotechnical instrumentation program was incorporated into the construction documents and consists of specifications and drawings. The drawings show instrument locations and installation details, and specifications include selection, installation and maintenance criteria, monitoring frequency, response values, and data reporting requirements. The SDCs based their designs on recommendations of the AGCs, as well as guide specifications and directive drawings prepared by the Management Consultant, Bechtel/Parsons Brinckerhoff (B/PB).

The geotechnical instrumentation program is designed to achieve the following objectives:

- Monitoring ground and facility deformations resulting from construction,
- Monitoring of changes in groundwater levels,
- Providing preconstruction baseline data for comparison with construction and post construction data.

The construction contractors are responsible for purchasing, installing, and maintaining the majority of the instruments. Before the construction contractors can proceed with instrument installations, they submit plans for review by B/PB to insure conformance with contract requirements. B/PB performs quality assurance of the contractors' work, reads the instruments (using a subcontractor), analyzes and reports the data.

The instrumentation program includes a Geographic Information System (GIS) consisting of Oracle and GDS. It was developed by the Project staff to improve the conventional methods of data processing and retrieval, which are too slow for the large volume of data generated. The system helps provide an early warning, so that mitigating actions can be implemented rapidly. This partially automated system provides readily accessible data to the contractors, construction managers, and other interested parties.

The GIS application consists of textural and graphic components. The textural component is an Oracle relational database. The Oracle database allows for quick and accurate entry, storage, processing, and retrieval of data and ensures that duplication of data does not occur. As soon as the data are entered and checked, they are available to many users linked by a computer network. The database has controlled access limiting users to verified/released information only, thus eliminating false alarms from data that has not been verified, while authorized users can insert, update, and delete data. The controlled access allows users unfamiliar with the system to view data, without inadvertently altering records. With the exception of a few instruments (seismographs and inclinometers), which currently have their own database programs, all the geotechnical and structural instrumentation observations are contained within the Oracle database. Oracle accepts data entered by hand, remotely monitored data and electronic data from a data logger in ASCII format transferred using bulk loading procedures.

The instrumentation monitoring program currently includes 10,000 instruments of 22 different types. The primary purpose is to provide data for construction control. The system creates a database that serves the needs of the Management Consultant (MC), the construction contractors, and the owners of adjacent properties (i.e., the abutters).

The instrumentation data would be of little value in construction control unless reduced and evaluated accurately and quickly. At the inception of the program, CA/T instruments were both installed and monitored by the construction contractors. Monitoring has since been assumed by the MC's subcontractor to collect the data and report them on a daily basis. Reduced and checked data is ready for analyses by the MC at 8:00 a.m. the day after they are collected. The MC then rechecks, analyzes, and provides copies to the Resident Engineers for forwarding to the construction contractors and selected abutters by 4:00 p.m. the same day. These hard copies provide the contractors with the information needed to monitor construction operations. When readings indicate the development of apparent problems (exceeding a predetermined Response Value), additional data are collected on an accelerated schedule and the data turnaround time is shortened to a few hours. In special cases the construction contractors can tap directly into the MC's Oracle database to facilitate rapid responses to changing conditions.

Data use requires a knowledge of when to take mitigative action. Every construction contract specification contains predetermined Response Values for each instrument. Each Response Value consists of a lower Threshold Value and a higher Limiting Value. Threshold Values provide a warning that ground or structure responses to construction operations are reaching levels of concern, and that action must be taken to prevent additional deformation. If, despite these efforts, deformation continues and the critical Limiting Value is reached, more stringent measures may be taken to bring the situation under control, including stopping construction and partially backfilling the excavation in the most serious cases.

The accuracy of the instrumentation readings is checked through a several-tiered QA/QC system, including: checking the current reading against the previous readings in the field, redundant readings by the field superintendent and others, and checking the monitoring reports for errors and consistency of data with similar type instruments in the area.

4.0 Lessons Learned

The "Lessons Learned" during implementation of the instrumentation program on the Central Artery/Tunnel Project range from broad based, such as, how to measure the potential damage to an abutting structure during construction, to very specific, such as combining two different instruments in one borehole.

Lessons specific to instrument types, locations, and installations:

- Inclinometer/Probe Extensometer - This instrument combined an Inclinometer and Probe Extensometer in the same borehole. The Inclinometer measures lateral movement of the soil mass. The Probe Extensometer measures consolidation of a soil mass. Combining the two instruments in one borehole saved installation time and cost. But, the type and shear strength of the grout required in the borehole after installation for each of these instruments differs. The Inclinometer requires a stiffer grout than the Probe Extensometer. A compromise grout did not serve the purpose, the precision of measurement for both instrument types was compromised, and the combining of these two instruments in a single borehole was discontinued.

- 'Dry' Observation Wells and Vibrating Wire Piezometers with collection zone or piezometer tip installed above the level of groundwater - Either the designed elevation for groundwater collection or measurement is above the level of groundwater after dewatering started or the interpolated geology was incorrect. The designer may not account for the amount (depth) of dewatering the contractor performed, resulting in a 'dry well' during construction. Alternatively, the geology may be different from interpolated by the designer in areas with glacial deposits or areas with human generated fill. In differing geology cases, during installation a technical person can observe the strata in the proposed collection zone, knowing the designer's intent, and make a 'field decision' to insure installation of functioning observation wells.

- Instrument base fixity established in a zone of construction's influence: Inclinometers, Probe Extensometers, and Multi Point Heave Gages - The base fixity zone, which serves as datum for each of these types of instruments, must be placed in a stratum that will not be affected by the construction activity. In cases where this is not done, there is lateral movement or settlement in the zone of anticipated fixity making it difficult to measure true movement when the data are tabulated and plotted. The base fixity zone needs to be placed approximately 10 feet below the bottom of the excavation support wall into a stratum suitable to provide base fixity, such as a till or bedrock. Further, if in fractured bedrock, the base fixity zone needs to be pregrouted.

- Deep Benchmarks (DBM) - are another example of instruments affected by construction activities. DBMs are critical for achieving accuracy for optical surveying methods. The specification for the installation of Deep Benchmarks must insure that the deep benchmark will not be subject to movement even during deep dewatering. This may require installing the tip of the DBM even deeper than a few feet into competent bedrock. During CA/T deep dewatering a few DBMs apparently moved laterally or settled, resulting in optical monitoring points referenced to these DBMs showing no settlement.

Lessons related to the effects of weather on instruments:

- It is well known that strain gages should not be read in direct sunlight. Other effects of weather that need to be considered include: freezing of water inside inclinometer casing, deformation monitoring points installed into asphalt being ripped out during snowplowing, and spurious data from tiltmeters. In some cases, the temperature can be measured and a correction factor applied to correct the instrument reading for the change. In other cases, installation of a different instrument type is necessary. During one prolonged cold spell and groundwater infiltration inside an inclinometer casing, ice built up prevented inclinometer readings from being taken. The specifications need to be written to hold the contractor responsible for maintaining the inclinometer.

Lessons related to setting realistic Response Values for instruments on abutting structures:

- Depending on instrument type, RVS are based on parameters relating to a structure, groundwater regime, loads, or allowable deformations. When a RV is exceeded, the Contractor is required to perform an action defined in the specifications or in an action plan. In some cases the designers specified the expected amount of deformation resulting from construction rather than amounts actually permitted for an abutting structure. Generally, this leads to unduly restrictive response values and evaluations. The designer's analyses must determine the permitted deformations for each abutting structure.

Lessons related to collection of data:

- Preconstruction Monitoring Data: some structures abutting construction undergo changes due to non-construction activities, e.g., seasonal weather changes, tidal changes, age of the structure, condition of the foundation. These changes require documentation by installing instruments to provide Preconstruction Monitoring Data. There are numerous late 18th century structures abutting the Project that have shown settlements prior to construction. Also, there are daily tidal fluctuations of up to 10ft between high and low tide. These data were collected, analyzed, and then used in setting the allowable deformation limits for abutting structures and water monitoring instruments

- Monitoring of instrumentation during construction must be carried out by representatives of the owner rather than the construction contractors. Under the previous monitoring arrangement for data collection and reporting two general problems occurred: 1) reduced and plotted monitoring data were not received by the MC in near real time, and 2) obtaining additional readings on instruments usually required days to implement. Therefore, rapid evaluation by the MC of changing conditions that arose from construction activities was difficult. The responsibility for collection and reporting of instrumentation data has been transferred to the MC and its subcontractor, resulting in development of a rapid and responsive monitoring system (Oracle/GDS) and collection, reduction, analyses and distribution of instrumentation data happen in near real time.

As the Project continues the lessons learned are being implemented into newly awarded contracts.

5.0 Future Developments

The project has another seven years of construction before completion. The monitoring system must continue to effectively function to 2004 and beyond. Some issues, e.g., temporary lowering of groundwater levels, and time-related deformations associated with clay subsoils, demand that monitoring continue beyond the project completion date. The system is designed to be both responsive and flexible. As we evaluate actual construction experience with the monitoring system, we seek means to modify and improve its overall performance. The Oracle/GAS combination has proven effective at providing quick and varied ways to display and plot data; relate local project geometry and geology, assess facility performance, and generate groundwater and settlement contours.

Our experience to date has also permitted us to evaluate numerical performance criteria, and instrument response values. In some cases, these were proven too conservative, and have been cost-effectively relaxed. We will continue to make these evaluations.

Another area that has advanced is data entry, manipulation and distribution. We have utilized more electronic components in accomplishing these tasks. Improved speed in these portions of the work has permitted more staff time to be available for analyses, correlations, and evaluations of interface problems. We are expanding the capability for more people, (both on project, and approved third parties) to access and read data at various stages.

Other future developments include: generating digital images of instruments and buildings they are installed in, and live camera views of the ongoing construction coupled with simultaneous viewing of instrumentation data. We consider the system a "living" entity that can be continuously modified and improved to meet the evolving monitoring needs of the largest infrastructure project in the U.S.

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Reliability-Based Design Method for Cut-and-Cover Tunnels

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Summary

The statistical results of geometrical and physical parameters for both backfill soil and structural material of cut-and-cover tunnels are given in this paper, while centrifugal model test is employed to figure out uncertainties in calculation formulae of backfill soil pressure. Then, uncertainties in load effect of structures under backfill soil pressure are determined with finite element-multiple response surface method. On the basis of above research, reliability indices for the 16 cut-and-cover tunnel structures in standard design drawing album are calculated and a target reliability index as well as partial coefficients in design equation are selected. Finally, a reliability-based design equation for cut-and-cover tunnel structures is developed.

1 Introduction

Cut-and-cover tunnel is an important type of tunnel constructions. Compared with other tunnels, cut-and-cover tunnel has more similarity with ground structures, so it is relatively more easier to make a reliability-based design for this type of tunnel than that of other tunnels.

In this paper, uncertainties in backfill soil and structural material, uncertainties in calculation formula of backfill soil pressure and probability characteristics of load effect of structures are discussed, following with reliability calculation of cut-and-cover tunnel structures as well as selection of target reliability index and partial coefficients in design equation.

The parameters affecting backfill soil pressure include thickness, slope angle, bulk density and internal friction angle, of the backfill soil. Centrifugal model test is used to help find uncertainties in calculation formulae of backfill soil pressure, by comparing experimental results with theoretical results of the earth pressure. 11 models, backfilled in both symmetric and asymmetric shapes of soil, are tested to achieve the uncertainty analysis. When small sample size occurs in statistics, statistical uncertainty is taken into consideration[1]. Accordance with so-called "structure-load" model, the uncertainties in load effect of cut-and-cover tunnel structures under backfill soil pressure are worked out with finite element-multiple response surface method[2], in which 16 structures from standard design drawing album are involved in calculation, and 4 random variables, modulus of elasticity of structural material, coefficient of rock mass reaction, backfill soil pressure on arch ring and backfill soil pressure on side wall, are taken into account in response surface. Based on the aforementioned work and the calibration method, we compute reliability indexes for the 16 structures with fractile method[3] and determine a target reliability index and partial coefficients in design equation, all being weighted with coefficients selected according to the rate of utilization of cut-and-cover tunnels. At the end of this paper, reliability-based design equation for cut-and-cover tunnel structures is given for application in practice.

2 Statistical analysis of basic random variables

The random variables influencing structural reliability of cut-and-cover tunnels could be divided into two groups. One group is on load, such as geometric shape of backfill soil (thickness and slope angle), material properties of backfill soil (bulk density and internal friction angle) and coefficient of rock mass reaction etc. Another group falls on structural capacity, for instance the bulk density, compressive strength, tensile strength, modulus of elasticity and Poisson's ratio, of structural material, etc. The statistical results of those basic random variables are shown in Table 1. Statistical data for variables on load are obtained by ourselves through in-situ investigation. To the load-relative variables, statistical uncertainties are thought over when their sample sizes are less than 50. Statistical data for capacity-related variables are mainly taken from ground structures. The capacity-related variables, having no bearing on backfill soil pressure, will play a role in load effect calculation and reliability computation of the structures.

Table 1 Statistical results of basic random variables

random variables	design value	sample size	mean	var. coeff.	distr. type
thickness of backfill soil (m)	1.5	31	1. 545	0.0906	normal
	2.0	40	1. 927	0.0610	
	2.5	10	2.516	0.0528	
	3.0	20	2.980	0.0601	
slope angle of backfill soil (°)	11.31(1:5)	40	11.07	0.0783	normal
	5.71(1:10)	12	5.79	0.0618	
	18.43(1:3)	15	19.17	0.1263	
bulk density of backfill soil (KN/m^3)	19.00	99	18.94	0.0551	normal
internal friction angle of backfill soil (°)	35.00	50	34.17	0.1216	normal
coeff. of rock mass reaction (MN/m^3)	rock mass of type II*		133	0.177	lognormal
	rock mass of type III		300	0.236	
	rock mass of type IV		733	0.225	
	rock mass of type V		1400	0.101	
bulk density of concrete (KN/m^3)			23	0.02	normal
compressive strength of concrete (MPa)	C20		20.67	0.308	lognormal
tensile strength of concrete (MPa)	C20		3.878	0.336	lognormal
modulus of elasticity of concrete (MPa)	C20		27000	0.0853	normal
Poisson's ratio of concrete	C20		0.2	0.05	normal

* The type of rock mass is from within the rock mass classification adopted by railway in China, the same blow.

3 Uncertainties in calculation formulas of backfill soil pressure

Backfill soil pressure can be verified by in-situ test, ordinary simulation model test and centrifugal model test. The in-situ test is quite expansive with results of high dispersity. In ordinary simulation model test, the model, required satisfying geometric and physical similarity rules, will be quite small in size and the backfill soil in the model will be unduly thin in thickness at the same time, which makes the measurement of backfill soil pressure uneasy. In comparison with the former two methods, centrifugal model test is relatively cheap in cost, easy in making, convenient in measuring, with a good representing of the prototype. Centrifugal model test has been widely used in geotechnical engineering and stability analysis of high slopes and hydraulic structures etc. It is the centrifugal model test that is appropriate to be employed in evaluation for uncertainties in calculation formulas of backfill soil pressure.

In this study, 11 model tests are completed. The model scale is 1:50 and structural material is C20 concrete in all tests. Other test parameters are listed in Table 2.

Table 2 Principal parameters in centrifugal model tests

type of cut-and-cover tunnel	slope of cutting	slope of backfill soil	thickness of backfill soil at crown (m)	type of backfill soil	number of survey points
type I in shape II*	1:0.75	1:5(symmetric)	1.5	clay	8
type I in shape II	1:0.75	1:5(symmetric)	2.0	clay	8
type I in shape II	1:0.75	1:5(symmetric)	2.5	clay	8
type I in shape IV	1:0.75	1:10(symmetric)	1.5	sand	8
type I in shape IV	1:0.75	1:10(symmetric)	2.0	sand	8
type I in shape IV	1:0.75	1:10(symmetric)	2.5	sand	8
type I in shape IV	no cutting	1:10(symmetric)	2.0	sand	8
type I in shape IV	1:0.75	1:10(symmetric)	2.0	clay	8
type II in shape IV	1:0.75	1:5(asymmetric)	1.5	sand	11
type II in shape IV	1:0.75	1:3(asymmetric)	1.5	sand	11
type II in shape IV	1:0.75	1:2(asymmetric)	1.5	sand	11

* Here "shape" denotes the shape of cross section of a lining, with I corresponding to symmetric cross section, II asymmetric cross section with vertical side wall, III asymmetric cross section with one inclined side wall and IV asymmetric cross section with one inclined side wall on which there is no earth pressure. "Type" means the type of rock masses in which a tunnel are constructed. The rock mass are classified from VI to I according to railway standard in China, in which the smaller the type, the worse the quality of the rock mass. For every shape, we have 4 types of structures from V to II.

The uncertainties in calculation formulas might be expressed with an uncertainty random variable of calculation formulas, which is defined as a ratio of experimental value to theoretical value of backfill soil pressure. Here, the theoretical value is calculated according to the formulas in appendix III of Design Code for Railway Tunnel in China.

Taking a look at the experimental results, we know that the backfill soil pressures at different location on arch ring have a smaller difference, between their experimental value and theoretical value, in contrast to side wall. The reason for this phenomena is suggested to be due to the lack of consideration of resistance of cutting slope in the formulas of the code. Therefore, we use two uncertainty random variables to express uncertainties in calculation formulae on arch ring and side wall separately. Statistical results of the uncertainty variable ξ on arch ring and η on side wall are given in Table 3. Here the statistical uncertainties of ξ and η are also considered.

Table 3 Uncertainties in calculation formulas on backfill soil pressure

random variables	sample size	mean	stand. dev.	var. coeff.	distr. type
ξ	42	1.0327	0.2190	0.2120	normal
η	37	0.8063	0.3123	0.3873	normal

4 Uncertainties in load effect of cut-and-cover tunnel structures

Section 2 gives probability characteristics of basic random variables, while section 3 proposes

the uncertainties in calculation formulae of backfill soil pressure. Following we come to uncertainties in load effect of cut-and-cover tunnel structures under backfill soil pressure.
We take in the uncertainty analysis of load effect 16 cut-and-cover tunnel structures, which cover all the structures from type V to type II in I-IV shapes in standard design drawing album. In load effect calculation below, thickness of backfill soil at crown is chosen as 2m and slope of cutting is 1:0.75 for all 16 cur-and-cover tunnels.
We treat the structures as a plane strain problem, being discreted into beam elements at backfill soil and beams element on elastic foundation at rock mass, respectively. 4 random variables with strong influence on internal forces of structures, modulus of elasticity of structural material, coefficient of rock mass reaction, backfill soil pressure on arch ring and backfill soil pressure on side wall, are taken into account in calculation. Other variables, for example structural gravity and geometric size etc., are regarded as constants due to their small variability.
The probability characteristics of modulus of elasticity of structural material and coefficient of rock mass reaction have been given in Table 1. The probability characteristics of backfill soil pressure on arch ring and backfill soil pressure on side wall could be brought out with Monte-Carlo method from random variables of thickness of backfill soil, slope angle, bulk density, internal friction angle and uncertainty variables of ξ and η. The probability characteristics for the 4 selected variables are obtained as in Table 4.

Table 4 Probability characteristics of basic variables in load effect computing

random variables		mean	var. coeff.	distr. type
modulus of elasticity of lining (GPa)		27	0.0853	normal
elastic coeff. of rock mass (MN/m^3)	rock mass of type V	1400	0.1770	log normal
	rock mass of type IV	733	0.236	
	rock mass of type III	300	0.225	
	rock mass of type II	133	0.101	
backfill soil pressure on arch ring (KPa)		39.347*	0.2258	normal
backfill soil pressure on side wall (KPa)		27.777#	0.4255	normal

* The value represents a mean of earth pressure at crown. The means of earth pressure at other positions on arch are different from the one at crown, while variation coefficients for all locations on arch are the same.
\# The value represents a mean of earth pressure at spring. The means of earth pressure at other positions on wall are different from the one at spring, while variation coefficients for all locations on wall are the same.

In interval $[-3\sigma,+3\sigma]$ of backfill soil pressures on arch ring and side wall, we divide whole the sampling area into 9 sub-areas to achieve a multiple response surface, which is used to calculate probability characteristics of load effect, axial force N and moment M. Here σ denotes the standard deviation of variables of interest. Calculation results at the most dangerous section of tunnel structures are shown in Table 5.

5 Target reliability index and partial coefficients

On the calculation results of load effect in section 4, it is known that the 16 cut-and-cover tunnel structures are all controlled for failure by tensile limit state, so we could, based on the tensile strength design equation of concrete in deterministic design, establish a limit state function for reliability-based design as follows

$$g=1.75\sigma_t d^2+Nd-6M \quad (1)$$

where N--axial force, KN; M--moment, KN-m; σ_t--tensile strength of concrete, KPa; d--section thickness, m.
In Eq.(1), only the variability of σ_t, M and N are considered, while d is looked upon as a constant.

According to Eq.(1), the design equation for cut-and-cover tunnel structures can be written as

$$\gamma_t^{-1} \cdot 1.75\sigma_t d^2 + \gamma_n^{-1} \cdot Nd \geq \gamma_m \cdot 6M \qquad (2)$$

in which γ_t--partial coefficient on capacity; γ_n--partial coefficient on load effect(axial force); γ_m--partial coefficient on load effect(moment); other symbols in Eq.(2) are the same as in Eq.(1).

Table 5 Probability characteristics of load effect for 16 cut-and-cover tunnels

shape	type	location	load effect	mean	var. coeff.	distr. type
I	V	arch ring	axial force N (KN)	118.64	0.3366	lognormal
			moment M(KN-m)	14.03	0.3410	
	IV	arch ring	axial force N (KN)	127.86	0.3375	
			moment M(KN-m)	20.02	0.3414	
	III	arch ring	axial force N (KN)	130.45	0.3374	
			moment M(KN-m)	24.32	0.3424	
	II	side wall	axial force N (KN)	284.71	0.3327	
			moment M(KN-m)	105.85	0.4438	
II	V	arch ring	axial force N (KN)	106.71	0.3448	lognormal
			moment M(KN-m)	22.86	0.3497	
	IV	arch ring	axial force N (KN)	102.87	0.3468	
			moment M(KN-m)	24.64	0.3503	
	III	arch ring	axial force N (KN)	111.24	0.3473	
			moment M(KN-m)	27.74	0.3540	
	II	side wall	axial force N (KN)	332.48	0.3420	
			moment M(KN-m)	243.24	0.4463	
III	V	arch ring	axial force N (KN)	118.20	0.4015	lognormal
			moment M(KN-m)	24.82	0.4110	
	IV	arch ring	axial force N (KN)	111.84	0.4037	
			moment M(KN-m)	28.38	0.4164	
	III	side wall	axial force N (KN)	278.67	0.4002	
			moment M(KN-m)	50.87	0.4120	
	II	arch ring	axial force N (KN)	220.36	0.4348	
			moment M(KN-m)	79.09	0.4472	
IV	V	arch ring	axial force N (KN)	104.82	0.3523	lognormal
			moment M(KN-m)	26.76	0.3545	
	IV	arch ring	axial force N (KN)	116.08	0.3553	
			moment M(KN-m)	31.46	0.3589	
	III	arch ring	axial force N (KN)	118.36	0.3603	
			moment M(KN-m)	33.67	0.3611	
	II	arch ring	axial force N (KN)	223.82	0.4314	
			moment M(KN-m)	103.82	0.3459	

Barring limit state function and design equation, we also need weight coefficients in calculation of target reliability index and partial coefficients. Based on investigation data of 278 cut-and-cover tunnels in use, we figure out the weight coefficients as in Table 6.

Table 6 Weight coefficients relevant to the rate of utilization of cut-and-cover tunnels

shape \ type	V	IV	III	II
I	0.000	0.002	0.073	0.107
II	0.005	0.036	0.232	0.172
III	0.000	0.014	0.038	0.038
IV	0.009	0.047	0.141	0.086

Making use of the probability characteristics of properties of structural material and load effect, Probabilities of structural failure of 16 cut-and-cover tunnel structures are computed on the limit state function with fractile method. Then, weighting the probabilities of structural failure with the coefficients from Table 6 yields an average failure probability $P_f=8.916\times10^{-5}$, Which results in a reliability index $\beta=3.748$. Therefore the target reliability index for cut-and-cover tunnel structures can be chosen as

$$\beta=3.7 \tag{3}$$

Partial coefficients for the 16 structures can be brought about from the target reliability index. Again weighting the partial coefficients with the coefficients from Table 6 finally produces the partial coefficients for cut-and-cover tunnel structures as below

$$\gamma_t=1.35, \quad \gamma_n=1.12, \quad \gamma_m=2.74 \tag{4}$$

In partial coefficient computing, the nominal value of property parameter σ_t of concrete is taken as fractile of 5% probability, and the nominal values of load effect N and M as fractiles of 95% probability.

At the end, we come to the design equation for cut-and-cover tunnel structures relevant to tensile limit state function as

$$\frac{1.75\sigma_t d^2}{1.35}+\frac{Nd}{1.12}\geq 2.74\times 6M \tag{5}$$

6 Conclusions

This paper shows the statistical results of parameters of backfill soil and properties of structural material in cut-and-cover tunnels in China. Based on centrifugal model test, uncertainties in calculation formulas of backfill soil pressure are brought out. After figuring out probability characteristics of load effect of 16 cut-and-cover tunnel structures under backfill soil pressure with finite element-multiple response surface method, the author calculates the reliabilities of the 16 structures, and according to the calibration method, selects a target reliability index and partial coefficients in design equation.

Owing to the lack of statistical data, we just considered the load produced by backfill soil, neglecting other loads, e.g. load due to collapse of side hill etc. Otherwise, because the target reliability index and partial coefficients are all computed at the most dangerous sections, and all the most dangerous sections are controlled by tensile limit state, we can only propose a design equation relevant to tensile limit state. Generally speaking, the design equation relevant to compressive limit state could be written as

$$\gamma_c^{-1}\cdot\alpha\sigma_c d\geq\gamma_n\cdot N \tag{6}$$

where N--axial force, KN; σ_c--compressive strength of concrete, KPa; α--eccentric influence factor for axial force N ($\alpha=1+0.648(e/d)-12.569(e/d)^2+15.444(e/d)^3$); e--eccentricity (=M/N), m; d--section thickness, m; γ_c--partial coefficient on capacity; γ_n--partial coefficient on load effect.

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