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Tunneling

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Transportation Research Record 1150

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Contents

Foreword

The seven papers in this Record are concerned with tunnel design, construction, and operation.

The stacked-drift liner technique and other design and construction features used to build the 1,500-ft-long Baker Ridge Tunnel in Seattle, Washington, are described by Holloway and Kjerbol. Twenty-four small (about 9 ft) concrete-filled tunnels around a 63.5-ft-diameter circle provided soil support during excavation of the stiff marine clay with sand lenses in the center and minimized disturbance of the environment. Resulting deformation of the liner during final excavation was only about 0.10 in.

Jenny, Donde, and Wagner's paper is about one of the first applications of the New Austrian Tunneling Method (NATM) in the United States. The method balances deformations during excavation to control movement yet permit the creation of a new state of equilibrium in the ground formation to minimize costs of excavation and support lining. A thorough knowledge of soil stand-up times is basic to using the NATM.

Domestic and foreign standards and their design concepts for plain concrete tunnel lining are reviewed by Gnilsen, who proposes a new concept of lining design based on soil-structure interaction of thin concrete layers.

Rehabilitation of the Holland Tunnel provided an opportunity to study the effect of temporary changes in the ventilation system. Lesser, Horowitz, and King report on a study to determine the effect of four different operational modes on visibility through smoke during a fire.

Permits and five design-construction packages for islands, tunnel sections, tub joints, ventilation, buildings and approaches, and electrical and mechanical features of the Interstate 664 submerged tunnel crossing of Hampton Roads, Virginia, are discussed by Gaddis.

Difficulties and resulting modifications in boring 14- to 21-ft-diameter tunnels for drainage of Interstate 10 in Phoenix, Arizona, are described by Smirnoff. Surface settlements in alluvial soils were minimized by the addition of poling plates, breasting boards, and a breasting table to the tunneling machine and by a compaction-consolidation soil-cement grouting scheme. Before the modifications, large ground losses (1,000 yd^3 or more) occurred frequently. After the modifications, surface settlements were limited to from $^1/_4$ to $^1/_2$ in.

Sandegren reports on a new material (cellular plastic foam) used since 1979 for insulating railroad tunnels in Sweden against the development of ice that is formed when groundwater seepage freezes in tunnels. Mats of foam 50 mm thick are bolted onto the rock face. The cost of the mats is reported to be repaid in 4 to 5 years by reduced expenses for cutting and removing ice.

Completion of World's Largest Soft-Ground Tunnel Bore

LEE J HOLLOWAY AND GEORG KJERBOL

In this paper are described the design and construction of a 1,500-ft-long tunnel through Mt. Baker Ridge in Seattle, Washington. The tunnel is part of a major improvement of Interstate 90. The net interior diameter of the bored portion, which is sized for vehicular traffic on two levels, is 63.5 ft. The soil is predominantly stiff clay. The project can be rated as the world's largest diameter tunnel in soft ground. Construction by a stacked-drift method has resulted in minimal distortion of the liner and insignificant disturbance at the ground surface above. Work was started in January 1983 and completed in May 1986. Completion of the interior roadway system is scheduled for 1988.

With a clear inside diameter of 63.5 ft and an effective outside diameter of more than 80 ft, the 1,500-ft-long Mt. Baker Ridge Tunnel has the distinction of being the largest diameter soft-ground tunnel in the world (Figure 1). It is interesting to note that this is not the first time Seattle has claimed a world record in tunneling. When the Great Northern Railroad tunnel under Seattle was constructed in the early years of this century, it was—with its 30-ft-wide by 26-ft-high horseshoe-shaped configuration—rated by its promoters as the largest soft-ground excavation opening ever attempted.

The novel technique developed for construction of the Mt. Baker Ridge Tunnel has proven successful beyond expectations. The stacked-drift system, in which a flexible liner consisting of many small concrete-filled tunnels contiguously arranged to form a compression ring is emplaced before excavation of the tunnel core, has provided a structurally safe support system and minimized the disturbance of the environment.

The diametric deformation of the liner, which was measured periodically during and after removal of the interior core of soil, came to about 1/10 in. Including an estimated initial movement of about 1 in. during construction of the liner, the total diametric deformation has been only about 0.13 percent. This small deformation confirms the effectiveness of the system.

PROJECT HISTORY

The new Mt. Baker Ridge Tunnel is an important link in a 7-mi-long freeway improvement program planned by the Washington State Department of Transportation (WSDOT) for

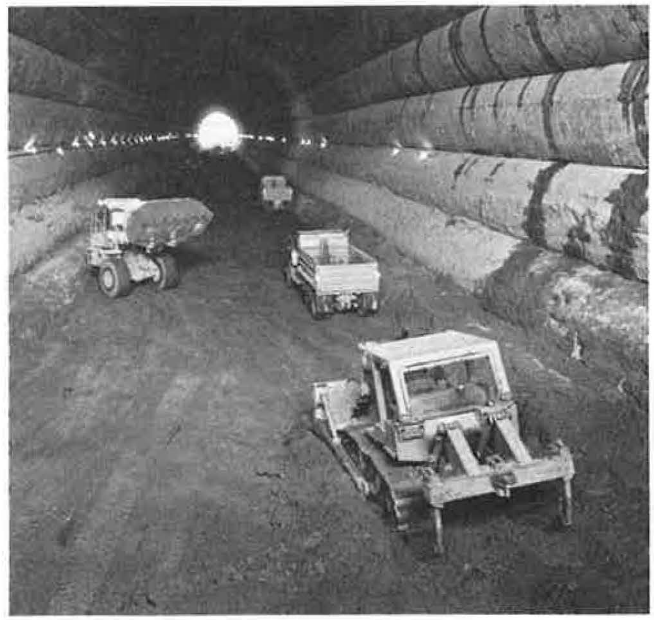

FIGURE 1 Tunnel mass excavation near completion (Richard S. Heyza/*Seattle Times*).

Interstate 90 (Figure 2). At an estimated total cost of more than $1 billion, the overall project calls for widening—generally from four to eight lanes—of the roadway system from Seattle to Bellevue.

When the tunnel interior structures are completed, they will carry traffic on three levels (Figure 3). On the lower level, two lanes will be reserved for car pools and public transit; the middle level will accommodate three lanes of mixed west-bound traffic; and the top level will provide passage for pedestrians and bicyclists. Currently, the I-90 crossing of Mt. Baker Ridge is handled through two double-lane tunnels completed in 1940; these tunnels will be converted to accommodate three eastbound lanes. The new bored tunnel and the old tunnels are of the same 1,500-ft length.

The new tunnel is a bored tunnel with cut-and-cover sections at the ends (Figure 4). The alignment is straight with a grade of 1.12 percent down toward Lake Washington. The exact total length of underground construction is 1,476 ft. The 63.5-ft inside diameter bored tunnel is 1,332 ft and two cut-and-cover sections at the ends are each 72 ft.

The I-90 expansion program concluded a 20-year history of struggles to overcome community opposition and funding difficulties. The single large tunnel concept was accepted by WSDOT in response to neighborhood concerns about the

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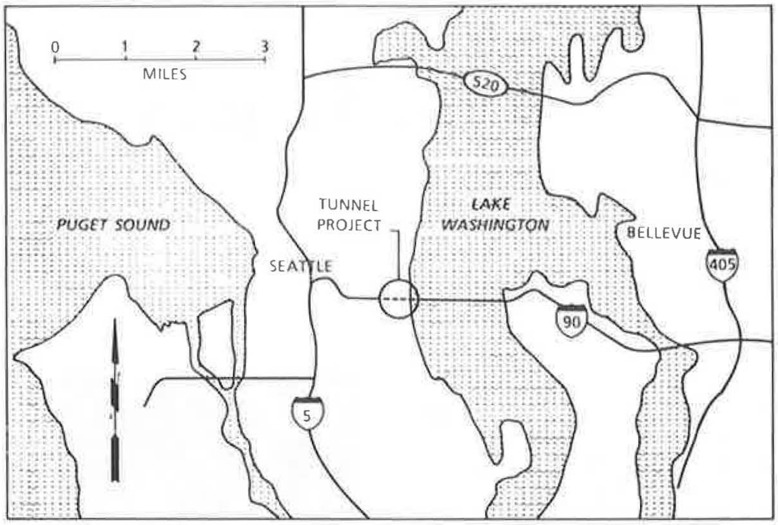

FIGURE 2 Location map.

potentially disruptive impact that would have been caused by several small tunnels, a cut-and-cover tunnel, or a large open trench.

The final compromise that led to the start of construction in 1983 included a lid, 1,900 ft long, to completely cover the freeway as it emerges from the west side of the tunnel. This resulted in a total project length of 3,400 ft and the requirement for an active ventilation system for both the new and old tunnels.

The entire Mt. Baker Ridge Tunnel complex will require 10 years to construct, and its cost is estimated at more than $200 million. Several phases of construction will be necessary to accomplish the work without significant impact on the environment and traffic. Design and construction of each phase have, therefore, been divided into separate contracts, of which the Mt. Baker Ridge Tunnel bore was the first awarded. The bored tunnel was completed in May 1986. The contract for construction of the interior roadway system was awarded in September 1986, and completion is scheduled for 1988.

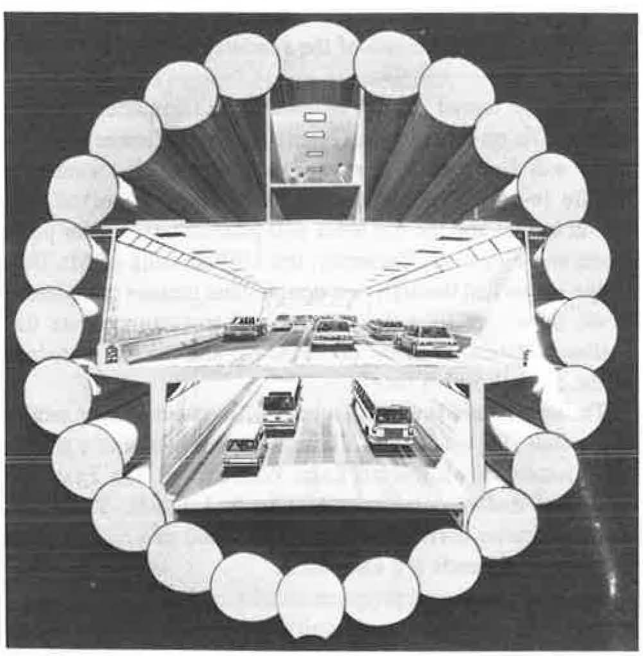

FIGURE 3 Traffic on three levels.

The tunnel was designed by Howard Needles Tammen & Bergendoff (HNTB) acting as prime consultants with geotechnical assistance from Shannon & Wilson, Inc. A. J. Hendron, Jr., acted as special advisor, and the firm of Jacobs Associates was engaged as cost and scheduling consultants. A tunnel design review board consisting of R. B. Peck, A. A. Mathews, and C. M. Metcalf was established by WSDOT to act as expert advisors throughout the period of tunnel bore design, development, and construction.

SITE CONDITIONS

Mt. Baker Ridge is a long hill trending north-south along the west shore of Lake Washington. The site is a residential area, generally with one- and two-story frame and brick buildings (Figure 5). The project alignment traverses one of the narrowest portions of the ridge.

The centerline of the new tunnel is parallel to and 129 ft north of the centerline of the nearest of the two existing highway tunnels (Figure 6). The maximum depth from ground surface to tunnel spring line is about 150 ft, which results in a maximum soil cover of about 110 ft. The bored portion of the tunnel is as long as possible. The depth of cover is about 10 ft at the ends.

A stratum of surficial sand containing perched water mantles the top and slopes of Mt. Baker Ridge. At the invert of the tunnel, a stratum of very dense sand is encountered. Otherwise, the local soils consist predominantly of overconsolidated, silty clay. The clay varies from very stiff to hard and is fissured. Typically, the clay is fractured and slickensided and is susceptible to rapid softening in the presence of water. The overconsolidated condition is an aftereffect of glacial loads of up to 4,000 ft of cover during the ice ages that have produced "locked-in" horizontal stresses within the stiff clay and silt masses. Evaluation of laboratory samples from the site and examples from construction of deep cuts in the Seattle clays indicate that the horizontal earth pressure at rest generally is

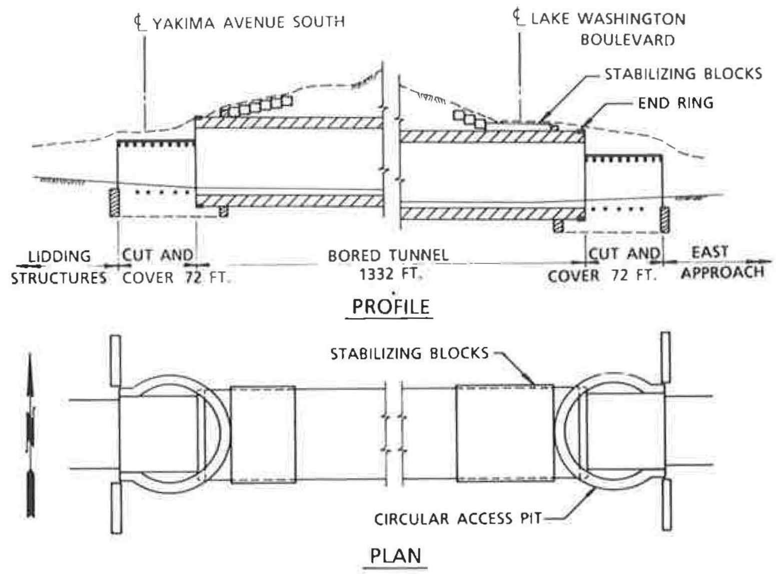

FIGURE 4 Structural arrangement.

FIGURE 5 West end of new tunnel north of existing freeway (Ralph Radford/Valley Newspapers).

higher than the overburden pressure, possibly by a factor of two to three.

Design was preceded by an extensive program of subsurface explorations, including numerous conventional test borings and test pits at both ends of the tunnel. In addition, photographs and records from construction of the two old tunnels confirmed that occasional boulders and sand lenses could be expected and that marginal slope stability would be a concern (landslides occurred at both portals during construction).

After completion of design, additional information of value in assessing ground behavior during construction was obtained as a result of three test tunnels, each 30 ft long, constructed in the vicinity of the new tunnel. These test tunnels, excavated by hand in a 12-ft-diameter horseshoe-shaped configuration, were branched off parallel to the new tunnel from a 160-ft-deep shaft at elevations roughly corresponding to its invert, spring line, and crown.

THE STACKED-DRIFT CONCEPT

When the single-tunnel concept became the only acceptable alternative, the designers soon realized that full-face tunneling for an excavation opening large enough to accommodate the 63.5-ft inside clearance requirement, if feasible at all, would incur serious risks of excessive disturbance in the form of landslides and ground surface settlement. A multiple-drift construction method was selected for the project.

A variant of the multiple-drift method was used for constructing the two old I-90 tunnels (Figure 7). The following excerpts, taken from the construction progress reports, are indicative of the difficulties encountered:

> In the start it was expected that 10″ x 14″ lining timber sets spaced on 3′-0″ centers would be ample to carry the weight of the overburden until the permanent concrete lining could be placed in front of and between timber sets. At points about 200 feet in from the portals the overburden became so heavy that heavier timbering was demanded. Placing 10″ x 14″ timbers closer together was not adequate, beside reducing the concrete lining too much. Accordingly, 12″ x 16″ timbers were used under greater depth of overburden. These were spaced generally at 3′-0″ centers although at places it was necessary to place them closer.
>
> At many places the wall plates were crushed to only a fraction of their thickness when placed. To obviate this trouble, the contractor procured hard wood corbel blocks and placed them above and below the wall-plate timbers.
>
> On account of the numerous timber joints, settlement under heavy load was experienced. Temporary lining was therefore set outside the theoretic position from 2″ to 8″ after allowing for settlement of mud sills and compression of wallplates.

The excavation opening of the new tunnel is about six times larger in area than that of each of the old tunnels. Indisputably, the construction difficulties that could be caused by using a conventional multiple-drift method would be many times more serious—possibly to the extent that constructibility could be questioned.

Clearly, a construction method that would permit minimum excavation face exposure at any one time was necessary. This

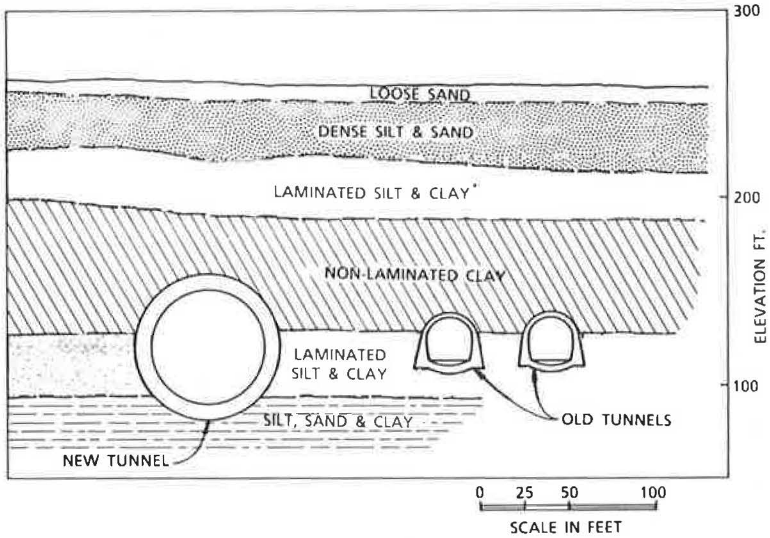

FIGURE 6 Typical subsurface section (looking east).

was accomplished by leaving the 63.5-ft-diameter interior core of soil in place until the liner had been completed. The resulting stacked-drift liner was constructed by filling individual drifts with concrete before proceeding with adjacent excavation. Each concrete-filled drift would then become a component in the final liner.

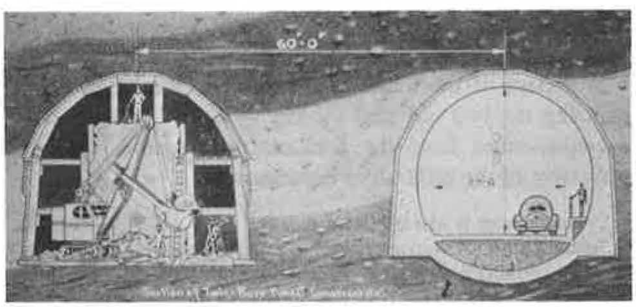

FIGURE 7 Construction of old tunnels.

The final design called for a series of contiguous drifts constructed sequentially from invert to crown of the completed tunnel liner. With this sequence, the risk of joints opening due to settlement during construction of individual drifts could be minimized, and the critical zones of concrete quality at the drift interfaces could be inspected and repaired where necessary before placement of concrete in the adjacent drift above.

There is no reinforcement across the joints between the drifts. These are held together by the earth pressure. Structural integrity is achieved by interaction with the surrounding soil.

Although no details could be found on previous applications of the particular construction technique developed for the Mt. Baker Ridge Tunnel, the overall concept of removing the soil core after completion of the liner was used as early as 1803 when the French constructed the 3,600-ft-long Tronquay Tunnel through soft sandy ground as part of the 60-mi St. Quentin Canal. More recently, the two Eisenhower Memorial Tunnels in Colorado were constructed by a similar method; rectangular drifts were used for the most difficult stretches of tunneling through zones of decomposing rock.

DESIGN

As is the case for most other bored tunnels, constructibility became the governing factor in determining the dimensions of the various components of the stacked-drift system. Also, gross construction engineering aspects with respect to type of structural elements and sequence of construction had to take priority over refined structural considerations. As a result, design generally became an investigative process with detailing limited to the establishment of a range of permissible features based on analyses of critical areas of interface conditions and evaluation of soil-structure behavior during and after construction.

In addition to the stacked-drift liner and excavation of its core of soil, the design involved construction method studies and engineering analyses of tunneling access pits, including detailing of bracing members for eventual conversion of the access pits to cut-and-cover structures.

Tunnel Liner

Because deformation of the tunnel liner would be controlled by the behavior of the surrounding ground, it was realized that axial stiffness would be the most important factor and that flexural strength would have little or no effect on the performance of the liner. Thus, the liner essentially would act as a membrane; intimate contact with the surrounding ground would be the principal requirement for achieving the desired soil-structure interaction.

The stacked-drift system, consisting of numerous interconnected small tunnels, provides a support system of flexibility sufficient to be comparable to a membrane. Adjusting to the nonuniform external loading by deforming until equilibrium is reached, it acts as a ring in compression with its individual elements held together by the thrust in the ring. The presence of a number of discrete joints at approximately equally spaced locations around the ring makes the liner's adjustment to nonuniform soil loads easier and produces a more favorable stress distribution throughout the liner.

In most tunnels, the critical condition is caused by the gravity-induced overburden loads. As mentioned earlier, the soil conditions of Mt. Baker Ridge made it necessary to consider the possibility of lateral soil pressures in excess of vertical pressures. This condition could cause a lengthening of the vertical axis, which could create a serious stability problem near the ends where the soil cover is low.

The possibility of large horizontal loads on the tunnel liner was taken into account by considering a relatively wide range of values of K_0, the ratio between the horizontal and the vertical soil pressures. The stacked-drift liner would not be structurally effective until after excavation of the core. Finite element analyses of an idealized elastically continuous tunnel structure were performed to help assess the performance of the soil and liner. The initial analyses considered K_0-values ranging from 0.5 to 2.0. The most unfavorable condition was found to be 115 ft of soil cover and a K_0 of 2.0.

Later stages of the analyses took into consideration the effects of ground disturbance during construction. Because the soils would relax with a resulting reduction of the horizontal pressure, a more narrow range of K_0, from 0.5 to 1.5, was used for the final analyses.

The eventual magnitude of ring thrust varies with depth of cover over the tunnel and with the value of K_0. A possible high of about 1,000 kips per linear foot of liner was estimated for a K_0 of 1.5 at the deepest section of the tunnel. A possible low of about 100 kips per linear foot of liner would occur at the ends of the tunnel for a K_0 of 0.5.

Liner distortion during construction would be a function of many parameters. These, in addition to the large diameter of the tunnel and the stacked-drift method of presupport, made liner distortion difficult to predict on the basis of case histories of conventionally excavated and supported tunnels. The initial finite element analyses of the liner using an estimated soil modulus of 20,000 lb per square inch indicated that an assumed K_0 of 2.0 produced a maximum distortion, measured in terms of diametric deformation, of 0.46 percent for a cover depth of 115 ft. For lesser values of K_0 and shallower depths, smaller distortions resulted. On the basis of these analyses and data on other tunnels, it was concluded that diametric deformation could reach a maximum of 0.5 percent with a more likely maximum occurrence of 0.25 percent.

The tunnel liner distortions were predicted to occur rapidly; only a small percentage of the total distortion would develop as long-term movement. This was confirmed in laboratory and field by creep and consolidation tests that indicated a fairly rapid creep rate during the first few hours of testing and a much slower rate thereafter.

The shape of the stacked-drift liner, which in the basic concept consists of a number of circular elements arranged like pearls on a string, was not desirable from a structural point of view. Initial calculations showed that overstress could occur at the narrow interface between the drifts. The problem of stress concentrations could be avoided if a constant thickness of the stacked-drift liner were used, but such a requirement would have resulted in an impractical construction procedure.

Structural investigations based on strain analyses with an allowance for higher strains caused by the effects of long-term loading on the liner resulted in specifications for a 5-ft minimum width of joints between abutting drifts with cushion strips, 6 to 9 in. wide, along the edges of the joints. For curved joints, the width would be measured along the chord between intersecting points on the circumference of abutting drifts. The purpose of the cushion strips, which were specified to be wood, was to create a confined stress transfer area sufficiently remote from the surface of the drifts to reduce the risk of concrete cracking and spalling along the joints.

The adequacy of the joint details was evaluated by quarter-scale model tests. The model tests indicated that straight or curved joint surfaces would be equally acceptable. The tests assessed performance under what were considered worst-case conditions, which included equivalent liner thrusts of up to 1,600 kips per linear foot and angular rotations across the joints ranging from 0 to 0.5 degrees. For a diametric deformation equal to 0.5 percent for a system with 30 drifts, a factor of safety of 1.1 for plain joints and 1.2 for joints with wood cushions was observed with regard to the initiation of through-going cracks. The model continued to carry design thrusts at liner distortions approaching 1.0 percent before there was a significant reduction in thrust-carrying capability. This indicated that the factor of safety against complete failure was considerably greater than the factor of safety against initiation of cracking. The joints with wood strips were shown to be more flexible, allowing about double the joint rotation for a given load eccentricity and, consequently, increasing the thrust-carrying capacity at the joints by nearly 10 percent.

The final specifications resulting from the design investigations called for a series of concrete units of approximately equal size and shape. The specified compressive strength of concrete was 4,000 lb per square inch and the minimum allowable number of drifts was set at 24. The concrete units could be constructed as intersecting cylinders with curved or straight joints or as trapezoidal blocks. A hybrid of circular and trapezoidal units, with blockouts on the inside of the completed tunnel liner, would also be allowed. For the purpose of minimizing soil disturbance, the maximum allowable size of the drifts was established at 12 ft wide by 12 ft high. For all possible shapes, the joints between the units—or chords of curved joints—were required to be oriented radially toward the axis of the completed tunnel.

Provisions at Tunnel Ends

Concrete stabilizing blocks 90 ft in length were constructed across the top of the tunnel in areas of shallow cover. The blocks were designed to provide added resistance to uplift caused by potentially high soil pressures on the liner. A large horizontal force would cause the crown to rise. The blocks, acting in cantilever over the sides of the tunnel, would mobilize sufficient weight to counterbalance the horizontal pressures.

At the extreme ends of the bored tunnel, where the liner extends into the access pit, a reinforced concrete ring was keyed into the drifts to reduce the risk of instability. The reinforcement was designed to yield should the ring reach a diametric deformation of 0.3 percent.

The risk of downslope movements of the marginally stable slopes near the ends of the tunnel, compounded with the possibility of longitudinal bending of drifts in the areas of low soil cover, resulted in a need for longitudinal steel reinforcement in the drifts near the ends of the tunnel. Such movements occurred during construction of the old tunnels and resulted in a west-ward displacement of the west tunnel portal of as much as 13 in.

Cut-and-Cover Structures

The permanent support structures at the ends of the bored tunnel consist of cut-and-cover sections with flanking wing walls at their downslope ends. Excavation to a depth of about 90 ft was required to provide launching and terminal access areas for construction of the tunnel liner. Full-face access—about 90 ft wide—for launching of the drift mining operations was selected rather than small shafts for individual drifts.

The cut-and-cover walls extend through the upper water-bearing sands and major silt and clay units and bear in or close to hard silts or very dense sands. Construction involved a large excavation volume and had to be carefully planned to keep soil movements in the marginally stable slopes along the access areas at a minimum. Prevention of downslope soil movements and consequent risk of loss of support of the hillside was a principal design consideration.

Each access pit was supported by a secant-pile wall that consisted of a series of 8-ft-diameter concrete cylinders with concrete filled intervals. The piles were aligned with their centers on a 100-ft-diameter circle to act as a compression ring during excavation of the access pit.

The cylinder piles on the north and south sides of the access pits were reinforced with steel beams in preparation for their eventual function as permanent walls between the top and bottom bracing systems. The cylinder piles on the east and west sides of the pits did not need to be reinforced.

The design involved investigation of significant changes in the characteristics of the structural system during construction. First, the access pits would be closed on all sides. Next, the wall on the upslope side toward the bored tunnel would be locally penetrated during construction of each drift. Finally, the upslope and downslope sides would be completely removed during mass excavation of the tunnel interior. Thus the structure would initially act as a ring in compression, and later the north and south walls would be converted to vertical beams between the top and the bottom bracing systems.

SPECIAL CONTRACT PROVISIONS

As a large and unique construction project, the Mt. Baker Ridge Tunnel required a special contract format to suit the complexity, risks, and duration of the work. Recognizing this, WSDOT concurred that the standard firm bid price type of contract should be modified to allow the contractor considerable discretion in determining his construction methods and to provide a basis for equitable sharing of the risks inherent in underground construction.

Performance-type plans and specifications were developed that permitted the contractor, within certain restraints and requirements, to develop details of design relating to his selected construction procedures with respect to size, shape, and number of drifts and methods of excavation and support. The contractor was also allowed considerable latitude in his selection of construction methods for the cut-and-cover approach structures.

An important piece of background information, a "Design Summary Report," was made part of the contract documents. This report contained a detailed description of the geotechnical conditions and a discussion of all pertinent design and construction criteria. In addition, all previous reports on soil conditions, design, and construction considerations were made available at a project reference library. Included was a report, "Construction Method Considerations," prepared by the designers at the beginning of the bidding period.

Construction of the test tunnels was completed during the 4-month bidding period. Prospective bidders were invited to visit the test site and make their own observations of actual tunneling conditions. A report entitled "Test Tunnel Construction Observations and Instrumentation Results" was issued in time for review by all interested parties.

In consideration of the long duration of the contract, an escalation clause was included. This provision covered 100 percent of variations in the cost of fuel and electricity and 80 percent of variations in the cost of labor and principal material items. WSDOT also elected to assume responsibility for damage caused to utilities, streets, and property by ground settlements.

The advantages of establishing an outside nonbiased authority to facilitate resolution of project disputes that might arise during performance of the work and to minimize the need for settlement of claims by litigation were recognized by the designers at an early stage. For this purpose, a three-member Disputes Review Board was established and general guidelines for its function were included in the contract documents. One member was appointed by the state and one by the contractor, and the third was selected by the other two. It was the responsibility of the board to mediate disputes or controversies between the contractor and the state. During regular visits to the project site, the board would encourage the settlement of differences at job level. If a dispute could not be resolved by informal mediation or by decision of the engineer, the board would consider written appeals and conduct hearings. After conclusion of the hearings and consideration of all evidence, the board would submit a written report to both parties with findings, reasons, and recommendations. The board's recommendations were not binding and could be appealed to the board for reconsideration. In the unlikely event that the board's recommendations did not resolve a conflict, all records and recommendations would be admissible as evidence in any subsequent litigation.

Bidders were required to provide complete background information on the basis for their proposals. Costs for each bid item (except an item identified as "fixed fee," which represented the bidder's profit) were to be clearly explained and separated. This information was confidential and the documents of the successful bidder were held in escrow until the completion of the contract. The documents could be used in case of disputes to determine the contractor's bid concept and aid in the resolution of conflicts.

CONSTRUCTION

After a 4-month bidding period, the construction contract was awarded to the Guy F. Atkinson Construction Company in October 1982 for $38.3 million. Eighteen bids were submitted. The other bids ranged from $38.5 million to $61 million. A total construction cost of between $50 million and $80 million was originally expected by WSDOT on the basis of the engineer's estimate. The fair and equitable contract conditions, an overwhelming interest in the project, and the first-hand information from the test tunnels undoubtedly all contributed to the favorable bid prices.

Monitoring Soil Movements

Concurrent with the start of construction, a surveying and geotechnical instrumentation system was installed. The system was used for monitoring access pit construction, drift excavation, and tunnel liner deformations during core removal. The data were used to verify design assumptions about soil behavior and to observe the effects of excavation on existing surface structures, subsurface utilities, and the adjacent existing tunnels. Another important purpose was to evaluate the acceptability of the contractor's construction methods and workmanship. Both raw and reduced data were furnished to the contractor for assessment of his methods and verification of project performance.

The design concept for the tunnel bore required minimal disturbance of the surrounding soil. Restrictions were placed on the allowable ground movement caused by construction. The limit on vertical movement was 1 in. measured 3 ft above each drift. For lateral movement, it was 0.5 in. measured 3 ft from the side of each drift.

The need for monitoring of the soil movements resulted in the Mt. Baker Ridge Tunnel being furnished with one of the most extensive instrumentation systems ever installed for an underground project. The instrumentation included combined inclinometer-settlement casings, concrete stress meters, strain gauges, tape extensometer reference points, and borehole extensometers. Joint meters were installed between drifts above the spring line to measure the effects of possible rotation caused by stoping of soil inside the ring of drifts.

Access Pits

The access pits were the first major items of construction activity. The contractor elected to construct these in close conformance with the details given in the plans and specifications for a circular secant-pile wall (Figure 8). The access pit walls were constructed in several steps starting with the uphill piles that would serve as retaining structures while subsequent downslope work was completed.

First, a series of 8-ft-diameter holes was drilled through the upper weak materials into the massive clay units at the location of the intervals between the 8-ft-diameter main cylinder piles. These holes were backfilled with a lean concrete to act as shoring for later operations. The main cylinder piles were then drilled to full depth, by cutting into the shoring concrete, and backfilled with structural concrete. Plastic sheeting inserted around the perimeter of the holes made the structural concrete surface relatively easy to expose and clean for subsequent interlocking at the intervals.

Last, 5-ft-diameter pilot holes were drilled through the lean concrete and continued to the full depth. The remaining lean concrete and soil were cleaned out with a special tool that exposed the sides of the main cylinder piles for a minimum width of 4 ft. The holes were backfilled with structural concrete to complete the ring of continuously interlocked piles that formed the access pit walls.

Following casting of a reinforced concrete cap beam on top of the access pit wall, the contractor excavated the soil inside the access pit in preparation for tunneling operations.

Drift Excavation

Working within the parameters set by the plans and specifications, the contractor elected to construct the liner as a series of 24 drifts (Figure 9). Machine excavation in a basic 9.5-ft-diameter horseshoe shape was used for 23 of these. The final closure drift was of an irregular shape and had to be hand excavated.

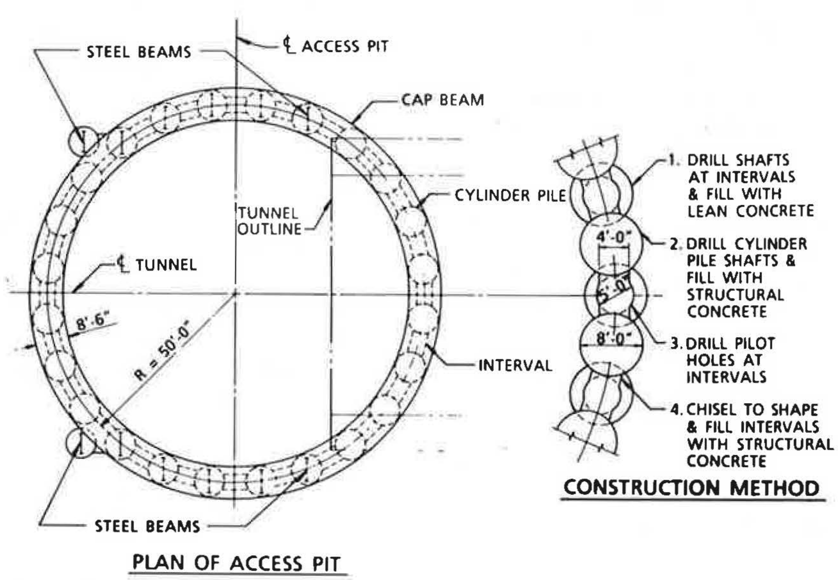

FIGURE 8 Access pit arrangement.

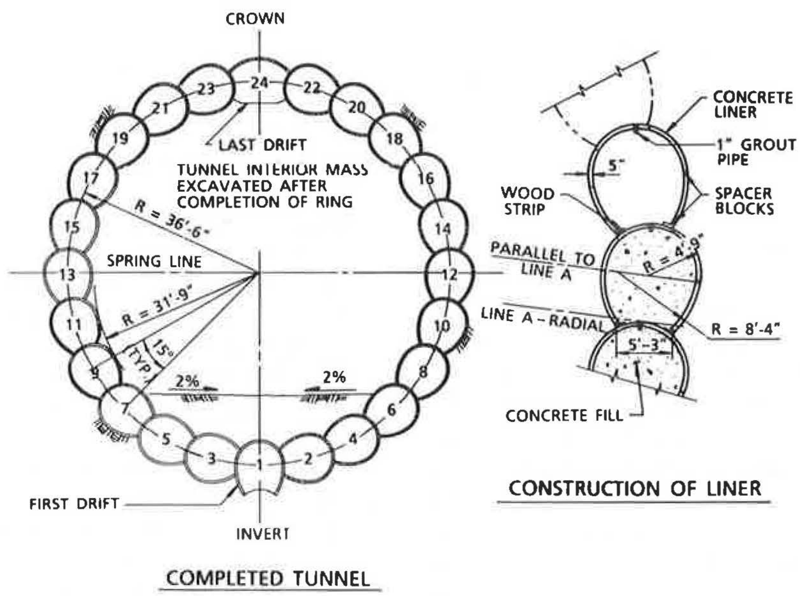

FIGURE 9 Stacked-drift tunnel liner.

The contractor acquired a pair of custom-designed tunneling machines. The shield of these machines consisted of a horseshoe-shaped outer hull provided with a dozen 100-ton-capacity hydraulic shove jacks and an inner circular hull containing an operator station, a swinghoe excavator, and the forward mounting of a conveyor belt for removal of the excavated material. The inner hull was shorter and could rotate relative to the outer hull so that the operator and equipment could always work from a level position regardless of the changing orientation of the drifts around the perimeter of the tunnel. A ½-in.-thick tail skin attached to the rear of the outer hull provided protection for erection of the ground support system.

A five-piece precast concrete liner, 4 ft long by 5 in. thick, was chosen as the support for the individual drifts. Below the spring line of the final tunnel, these segments were reinforced with polypropylene fibers. Above the spring line, No. 4 reinforcing steel in a 6-in. grid pattern was used.

As a first step in excavation of each of the drifts, a steel push ring was mounted behind the shield and anchored to the wall of the access pit to serve as a backstop for launching of the machine. The drifts were excavated in 4-ft increments, as determined by the length of the drift liner segments and the reach of the shove jacks.

A laser beam provided a reference point for steering the shield. The laser was mounted on the rear wall of the access pit and aimed into the tunnel to strike an alignment grid on the shield itself.

The drift liner segments were moved into the tunnel by train and off-loaded for positioning by means of a hydraulically operated erector arm at the rear of the shield. A total of about 37,000 segments had to be set.

The liner segments were provided with jacking pockets for expansion of the support ring after the tail skin of the shield had passed. A portable 20-ton-capacity hydraulic jack was used for this. Softwood wedges were driven into open joints to hold the expanded ring in place. The cushion strips required at the edges of the interface with the preceding drift served well both as adjustable spacers and as supports for the arch formed by the drift liner segments.

Each 4-ft advance of the shield involved about 11 yd^3 of excavation. Occasional large-sized boulders had to be broken up before removal. Muck was removed by a conveyor belt and loaded onto a train of four cars. The train, which also had one dolly for liner segment transportation, was moved on rails by an 8-ton diesel locomotive. At the west access pit, the cars were lifted out and their contents were dumped onto a stockpile for later loading into trucks. A car-passer system with two alternating lines of rails allowed maximum speed in exchanging cars and thus cut the time of each cycle. About 3,700 yd^3 of muck were removed for each drift; the total was about 90,000 yd^3 for all of the drifts.

After each drift had been holed through, the 50-ton shield was picked up at the east access pit and trucked back to the west end for the next drift. Two tunneling machines were employed, one on each side of the tunnel. When excavation of one drift had been completed, the tunnel crew shifted to the second machine. Other crews then filled the excavated drift with concrete and set up the tunneling machine in preparation for digging the adjacent drift.

Excavation of Drift 1 commenced on September 27, 1983. Progress on the first drift was intentionally slow to allow the crew to become acquainted with the particular features of this type of tunneling and was further delayed by unexpected difficulties as described later. When the initial problems with this and the two following drifts had been solved, the tunneling progress improved considerably; the highest production rate reached was 110 ft per three-shift day. Excavation of a typical drift took from 14 to 20 working days to complete. Work proceeded 24 hr a day on a five-day week schedule. Specifications allowed simultaneous excavation of two drifts after the invert drift had been completed. Because of early difficulties, simultaneous excavation did not begin until Drifts 4 and 5. Then the excavation of the drifts was staggered so as not to begin or complete two drifts at the same time.

Because of its odd configuration, the last drift had to be dug by hand. Conventional overhead steel ribs and timber lagging were used to support the soil. The tedious process of hand mining reduced the daily average advance to 37 ft. Finally, on July 9, 1985, drift excavation was completed when break-through was made to the east access pit for the 24th time.

Concrete Placement

The contractor decided to divide the excavation of the drifts and their subsequent filling with concrete into two separate operations. All drift excavation work was started from the west end. Except for a plug of concrete bulkheaded at about 150 ft from the west end, the concreting operations were from the east access pit. The west-end plug was constructed as soon as possible after holing through of a drift in order to avoid any delay in setting up the equipment for tunneling of the next drift.

The concreting equipment was a rail-mounted pump located at drift level. Concrete was fed via an inclined drop chute from the edge of the pit to a surge hopper, from where it was transferred to the pump on a swivel-mounted traveling conveyor belt and then pumped through a rail-mounted 6-in.-diameter slickline to the riser car located at the foot of the concreting face. The terminal portion of the slickline was supported on hangers, and its end was kept buried 3 to 10 ft into the concrete.

It took from five to six 8-hr working days to place the 2,800 yd^3 of concrete required to fill a drift. Typically, before this process was complete, excavation had already begun on the adjacent drift from the west access pit.

Grouting

When a drift had been concreted, it was grouted to ensure complete filling of its interior and of any external voids caused by overexcavation or cave-ins. Each drift was divided into eight sections of equal length. A 1-in.-diameter steel pipe was installed in the high point of the arch from east or west in each section to be grouted. Within the section to be grouted by that particular line, tees and nipples were installed between each standard 21-ft length of pipe. The nipples projected halfway through holes drilled through the precast liner and were provided with a plastic cap to prevent plugging of the line during the concrete filling operations.

A grout mix of 1 ft^3 of water to 1 sack of cement was pumped through the line at pressures of up to 100 psi. This pressure was higher than normally used for backfill grouting but was dictated by the concern for complete filling of all voids. There was evidence that the grout did penetrate the surrounding soils along bedding planes and fractures. Once, while grouting in Drift 13, grout began leaking into Drift 14 on the other side, some 65 ft away. As the drifts approached the portal regions of low cover, the pressure was reduced to minimize any undesirable effects of high-pressure grouting.

Completion of Tunneling Work

Mass excavation of the core, dubbed the 25th tunnel, was started on August 5, 1985. The excavation was done in lifts, or benches, up to 12 ft in depth. Bench 1 at the top was excavated using a front-end loader to haul the muck to the ends of the tunnel for dumping in stock piles, from where it was trucked to final disposal sites. On subsequent lifts, highway trucks were driven in a continuous working cycle from east to west and were loaded in the tunnel by a front-end loader.

Mass excavation took about 8 months including interruptions to install the top bracing structures that converted the access pits to cut-and-cover structures (Figure 10). A total of about 160,000 yd^3 of material were removed in a series of five lifts.

Field Experience Record

Construction was completed on schedule and within budget, although several difficult problems arose. Their solution posed some real challenges and even caused the project to be periodically shut down when ground movements exceeded the maximum allowable limits.

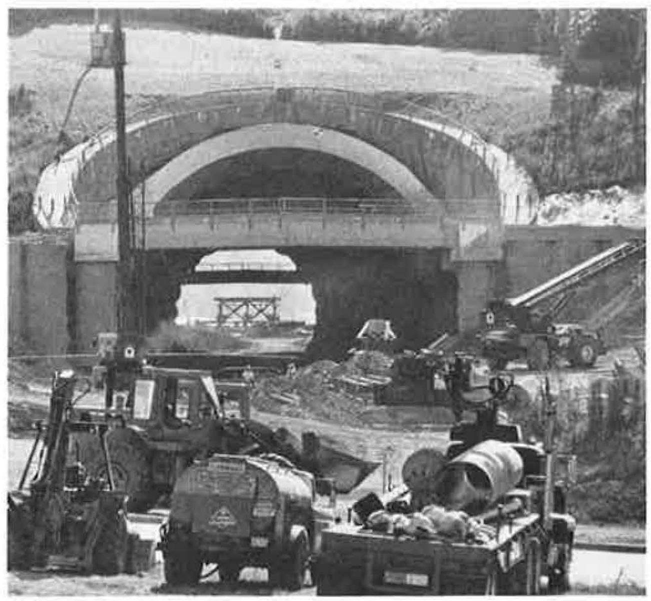

FIGURE 10 View from west through the completed tunnel (Ralph Radford/Valley Newspapers).

In driving the first drift, the shield operators, who were inexperienced with this particular type of excavator, ran into alignment control difficulties, particularly while dealing with mixed face conditions. About two-thirds of the way through excavating the first drift, dry cohesionless sand was encountered. It proved impossible to control excavation after full-face sand had been reached. Above and ahead of the shield, the dry running sand caved in up to the interface with the overlying clays. The resulting void was estimated to have a volume of about 130 yd^3 and required backfilling before drift excavation could be continued. Pneumatically placed pea gravel and grout were injected through the drift liner; later grout to completely fill the cavity was pumped through a hole drilled from above.

As a result of the encounter with running sands in Drift 1, the shield being prepared to drive Drift 2 was modified by adding a 30-in. hood to the front. This improvement enabled excavation with little difficulty through occasional lenses of running sands. The hood extension was used on both shields for all remaining drifts.

A further modification as a result of the excavation of Drift 1 was the addition of ripper teeth to the swinghoe of the excavator. The clays were considerably stiffer and harder to excavate than had been anticipated. The addition of ripper teeth to dig the face increased performance significantly.

Difficulties with steering the shield caused the operators to overexcavate at the heading of the drifts. Although they did not result in excessive soil movements around the relatively small opening for the first drift excavated, vertical movements of up to three times the allowable 1 in. occurred in Drift 2.

The specifications called for all supporting elements of the drift liner to be installed tightly against the surrounding ground and for all voids to be promptly filled with grout or pea gravel. Filling of voids during drift excavation did not prove practical because the drift liner was perforated for grip holes and had too many other escape openings at jacking pockets and at the joints in the segmented liner. Instead, it was decided to do the grouting after concrete filling, as described earlier. This improved the overall performance of the system by limiting any cumulative effects from previously constructed drifts. Of the some 60 instrumentation stations passed subsequent to Drift 2, at only one, in Drift 8, did the settlement exceed the specified 1 in.

One potential problem that arose was the "growth" of the drifts as drift excavation proceeded around and up the perimeter of the main tunnel. Control of the shield proved difficult when it was moving tightly along the adjacent drift. Also, deviations in the alignment and grade of the preceding drift had to be corrected. This meant that the shields rode slightly away from the preceding drift most of the time, which resulted in a larger arc distance excavated with each drift than had been originally planned. To partly rectify this condition, 1 in. of the 5-in. bottom plate on the shield was removed. However, a progressively increasing distance of the drifts from invert toward crown of the completed tunnel liner could not be entirely avoided. As a result, the available horizontal clearance for construction of Drift 24 decreased from the originally intended 9.25 ft to 5.5 ft at the narrowest section. This last drift was hand mined, so no particular problem arose from lack of space.

EVALUATION OF STRUCTURAL PERFORMANCE

Final validation of the stacked-drift concept used in construction of the liner for the Mt. Baker Ridge Tunnel came with the removal of its interior core of soil. Measured periodically during bench excavation of the 63.5-ft-diameter opening, diametric deformation generally did not exceed 0.1 in., which when added to an initial deformation during drift construction (estimated by inclinometer and joint meter observations to be of the order of 1.0 in.) resulted in a total diametric deformation of only 0.13 percent.

The 0.13 percent diametric deformation is about half of the 0.25 percent assumed in the design analyses. Apparently the relaxation of the lateral earth pressure that resulted from reasonably controlled disturbance during construction caused the ground to stabilize with the coefficient of earth pressure at rest near unity in value. Another observation is that comparison with the almost traditional rule of thumb—based on experience with other soft-ground tunnels—that the diametric deformation may be as high as 0.5 percent does not appear to be valid for a tunnel with the liner installed before excavation of its interior.

Ground surface settlement of up to 12 in. was predicted during the original design investigations. An actual maximum settlement of 2.5 in. recorded along the eastern two-thirds of the bored tunnel alignment and a localized settlement of about 9 in. near the west end of the tunnel were well within this tolerance. Most of the settlement occurred during construction of the drifts; there was a continuing movement of only about 0.5 in. during excavation of the interior core of soil.

Concrete stress meters installed at invert, spring line, and crown at joints between drifts in the liner showed stresses of up to 1,300 lb per square inch. The total measured ring thrusts approximately equalled the thrusts that could be calculated from full overburden weight with a coefficient of earth pressure at rest equal to unity. Maximum stresses in the liner occurred along the outside edge of the invert drifts and along the inside edge of the spring line drifts, which suggests minor ovaling of the bore with the long axis in the horizontal plane. This minor ovaling was confirmed by the tape extensometer measurements across the tunnel diameter.

CONCLUSION

The Mt. Baker Ridge Tunnel provides several valuable insights into large-diameter tunnel construction in soft ground. The design and construction experience will provide useful information for the advancement of tunnel projects of similar size and type. Adapting conventional small-sized tunneling methods for a project of record size has proved successful in minimizing the risk of ground disturbance. Performance-type contract provisions, including thorough background information on project considerations, risk sharing, and dispute settlement, plus the test tunneling program, have clearly demonstrated their value in easing this highly innovative project through its many challenges.

Through a special agreement with WSDOT about shop drawing reviews and other advisory services, HNTB was able to closely observe the construction progress. It was evident that the excellent management attitudes and understanding relations between the WSDOT project engineering staff and the contractor contributed significantly to the successful completion of the project. There were no major cost overruns and, with a low rate of inflation during later years, the total cost was actually less than the original budget. This, in itself, is an unusual occurrence for underground construction.

New Austrian Tunneling Method Used for Design of Soft-Ground Tunnels for Washington Metro

ROBERT J. JENNY, PRAKASH M. DONDE, AND HARALD WAGNER

The design of Section E-5 tunnels of the Washington Metro using the New Austrian Tunneling Method (NATM) is described. The twin-bore tunnel section, approximately 1,000 ft long, runs directly beneath Fort Totten Park in Washington, D.C., west of the District's border with Prince George's County, Maryland. In addition to a conventional tunnel design, Metro engineers directed an alternate design using NATM. Although this type of design is used extensively in Europe, this will be one of the first NATM applications to soft-ground tunnels in the United States. The NATM is a method whereby the rock or soil formations surrounding a tunnel are integrated into an overall ringlike support structure; thus the formations become part of the support system. The two main support elements are a reinforced shotcrete initial lining and an unreinforced final concrete lining. The tunnels will pass entirely through a cretaceous formation of stiff, silty clays and clayey sands. NATM appears to be a suitable method of constructing these tunnels because the soils apparently possess good stand-up time, the groundwater table is low, and the tunnels are relatively short. Even though these tunnels have not yet been constructed, it is believed that the design concepts presented will be useful to tunnel designers.

Application of the New Austrian Tunneling Method (NATM) to the Section E-5 tunnels of the Washington, D.C., Metro is described.

Washington Metro Section E-5 tunnels and Fort Totten Station are on the Greenbelt Route beneath Fort Totten Park in the vicinity of Fort Totten Drive, at the northeast border between Maryland and the District of Columbia (Figure 1). The twin-bore tunnel section is approximately 1,000 ft long with a ventilation shaft at the western end and a fan shaft at the eastern end (Figure 2). Approximately 300 ft of the station are to be constructed underground and the remaining 300 ft will be in open air. The tunnels will pass entirely through cretaceous formations of stiff, silty clays and clayey sands. Groundwater is at about the invert of the tunnels. Ground cover will range from 100 ft at the western end to 65 ft at the eastern end. The underground portion of the station is also being designed using NATM techniques. However, this paper deals only with the bored tunnels. This design will be one of the first NATM applications to soft-ground tunnels in the United States. Alternative NATM contract documents will be prepared for competitive bidding against conventionally driven tunnel-boring machine (TBM) or shield tunnels. The NATM design will incorporate techniques currently being used on the Vienna subway and those that have been used on the Munich subway. Even though the tunnels have not yet been constructed, it is believed that the design concepts presented here will be of interest to tunnel designers.

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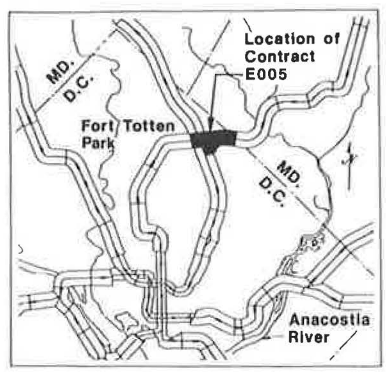

FIGURE 1 Location of tunnels.

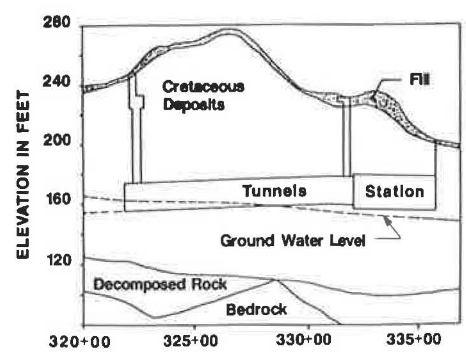

FIGURE 2 Generalized geologic profile.

NATM Design Principles

Excavation of tunnels causes the in situ balance of stresses in the rock or soil mass to be changed by deformations related to the tendency of the mass to relax and converge until a new, stable state of equilibrium is reached. This equilibrium is achieved through a succession of stress redistribution processes.

NATM design involves control of transitional stress redistribution by an economical and safe excavation and lining process. On one hand, deformation should be kept to a minimum so that the primary state of stability and compressive strength of the ground are not unduly weakened. On the other hand, geomechanically controlled deformations are necessary to the extent that the ground formation itself will act as an overall ringlike support structure. This creates a new state of equilibrium at an early excavation stage and thus minimizes costs of excavation and support.

TUNNEL CROSS-SECTIONAL GEOMETRY

The cross-sectional geometry of the E-5 tunnels is shown in Figure 3. This cross section consists of six circular arch segments and was developed using the following criteria:

- Keep excavation to a minimum while providing the required train clearances;
- Allow no tensile stresses in the final tunnel lining at any location, thus permitting the use of an unreinforced concrete final lining;
- Maintain flexibility in construction procedures and excavation sequence for various soil conditions; and
- Allow for proper tunnel drainage during and after construction.

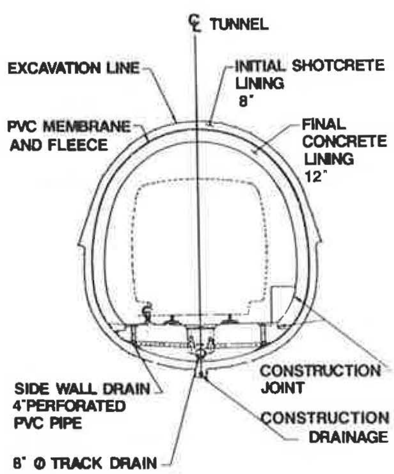

FIGURE 3 Tunnel section.

The initial support system consists of 6 to 8 in. of wire mesh–reinforced shotcrete and 6-in.-deep lattice girders, typically spaced 3 ft on center. In unstable grounds, that is, soils with inadequate stand-up time and swelling or raveling soils, forepoling with steel rods or plates may also be required. The final lining consists of 12 in. of unreinforced cast-in-place concrete. The safety walk and invert concrete shown in Figure 3 are standard Metro requirements and are not part of the structural support system. A plasticized polyvinyl chloride waterproofing membrane is installed between the initial and the final lining to prevent infiltration of water through the final lining.

FINITE-ELEMENT ANALYSIS

Determination of stresses and deformations of the support elements and surrounding soil was based on the finite-element method (FEM) developed at Innsbruck University in Austria. Design parameters for soil and stiffness reduction factors were chosen to represent the soil-structure interaction based on NATM principles taking into account the redistribution of the load to the soil surrounding the cavity.

The following nine loading conditions, one for each stage of construction, were analyzed by the FEM:

- Loading 1 (Figure 4): This condition represents in situ soil stresses before excavation.

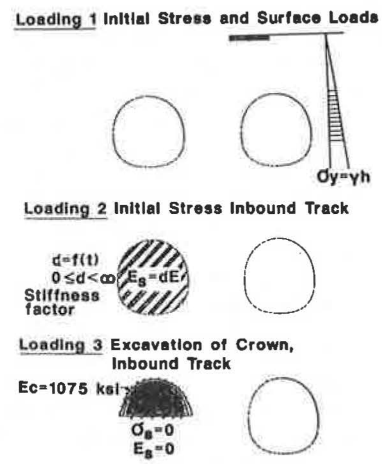

FIGURE 4 Loadings 1–3.

- Loading 2 (Figure 4): This condition represents stresses and displacements resulting from stress relief in the excavation area after removal of face support and just before the start of excavation of the inbound tunnel.
- Loading 3 (Figure 4): This loading condition represents excavation and initial support of the top heading of the inbound tunnel.
- Loading 4 (Figure 5): This loading condition represents excavation and initial support of the bench and invert of the inbound tunnel.
- Loading 5 (Figure 5): This loading condition represents the stresses and displacements resulting from stress relief in the excavation area just before the start of excavation of the outbound tunnel.
- Loading 6 (Figure 5): This loading condition represents the excavation and initial support of the top heading of the outbound tunnel.
- Loading 7 (Figure 6): This loading condition represents the excavation and initial support of the bench and invert of the outbound tunnel.
- Loading 8 (Figure 6): This loading condition represents stresses and displacements of the final lining and the soil. That the shotcrete provides partial support in combination with the concrete final support is taken into consideration. Loads for this case differ from all previous ones in that the loading takes into account the effect of water pressure as well as the dead load of the concrete lining.

• Loading 9 (Figure 6): This loading condition represents stresses and displacements of the final lining and the soil, assuming that the shotcrete initial support deteriorates with time and all loads are carried by the final lining.

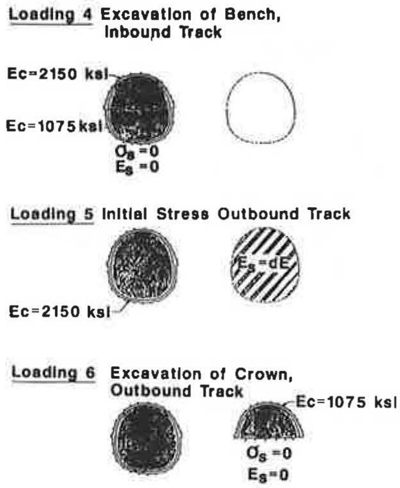

FIGURE 5 Loadings 4–6.

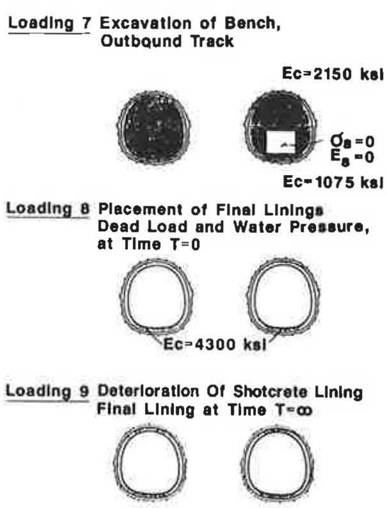

FIGURE 6 Loadings 7–9.

Results of these loading cases preformed for Section E-5 tunnels indicate that stresses in linings are within allowable limits of the applicable codes.

CONSTRUCTION SEQUENCE

The sequence of excavation and support of the tunnel is shown in Figure 7. The construction method should complete and close the ring of initial support for each tunnel as rapidly as possible, while maintaining heading stability. Closure is accomplished by excavation and support of the top heading, usually in two excavation cycles ahead of the bench. That of the bench is done one cycle ahead of the invert (Figure 8). To optimize the use of material and equipment and crew productivity, both E-5 tunnels will probably be constructed simultaneously, one lagging a few feet behind the other.

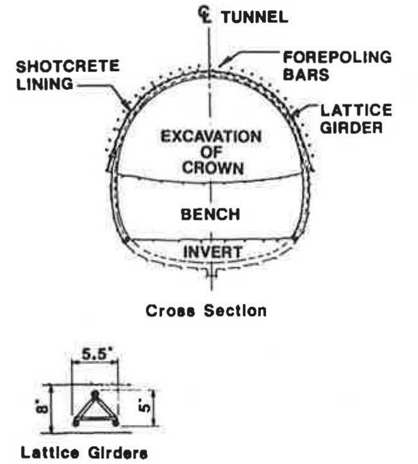

FIGURE 7 Sequence of excavation and support.

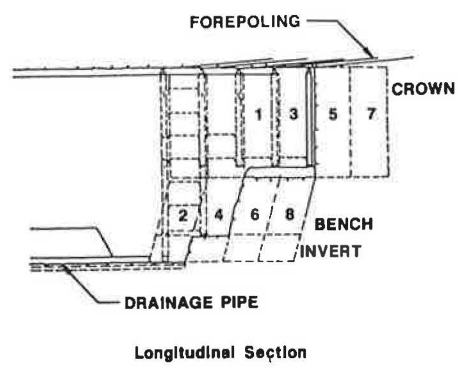

FIGURE 8 Closure.

Evaluation of available geotechnical data has indicated that, from a construction standpoint, four ground types may be encountered. The various types of ground will be identified during construction by visual observation as well as on the basis of the results of ground monitoring and instrumentation.

Ground Type 1 covers the most favorable ground condition. Stand-up time is estimated to be in excess of 3 hr. After excavation of the top heading, a 1-in.-thick layer of shotcrete is applied, followed by installation of lattice girders (only at the top heading), application of another shotcrete layer to embed the lattice girders, placement of wire mesh on the shotcrete, and application of the last shotcrete layer to a total thickness of 6 in. When the bench is excavated it is lined with shotcrete only and no lattice girders are required.

Ground Type 2 covers soils the general ground behavior of which is stable, but stand-up time is estimated to be as long as 3 hr. This ground type requires extension of the lattice girders to the bottom of the sidewalls and use of forepoling bars and spiling for additional stabilization of the ground. In addition, shotcrete thickness is increased to 8 in. and the length of tunnel advance is generally limited to 3 ft.

Ground Type 3 covers generally unstable ground with stand-up time limited to less than 2 hr. Discontinuous forepoling sheets, closure of the face with shotcrete immediately after excavation, and a complete ring of lattice girders are required.

Ground Type 4 covers cohesionless gravels, loose layered sands or plastic clays, and the presence of groundwater. This condition is similar to Ground Type 3 except that stand-up time is estimated to be about 1 hr, and interlocked forepoling sheets for every tunneling cycle are required.

SUMMARY

Section E-5 tunnels are well suited for NATM construction. The general criteria that favor application of the NATM construction technique over the conventional one are

1. Short tunnels for which TBM usually becomes expensive,
2. Soils with good stand-up time,
3. Low groundwater table,
4. Availability of construction shafts to perform construction from multiple headings, and
5. Difficult ground conditions such as presence of boulders that usually pose a problem for TBMs.

In general, tunneling by NATM requires readily available "off-the-shelf" equipment such as a backhoe; therefore mobilization of special equipment such as large and expensive TBMs is not required.

A clear, objective assessment of conditions; a well-defined unit cost payment schedule; and a cooperative effort among owner, engineer, and contractor are essential for the successful completion of an NATM project.

Plain Concrete Tunnel Lining—Design Concepts

REINHARD GNILSEN

Discussed in this paper are domestic and foreign concepts for the design of plain concrete tunnel lining. Comparisons are made in terms of the respective design capacity assumed by the concepts. A new concept, tailored to U.S. tunnel design practice, which builds on an advanced approach to defining the capacity of plain concrete tunnel lining, is developed. The first concept is the standard approach incorporated in the Deutsches Institute für Normung (DIN) building code (DIN 1045). This concept is built on strength design criteria similar to those considered in American Concrete Institute (ACI) Code 318, Strength Design. The applicability of this design concept to unreinforced concrete tunnel lining has been proven extensively during the last decade. Partial cracking of the concrete, a characteristic of strength design, is restricted by the code to the extent that stability and safety requirements are observed. The second concept discussed is incorporated in the ACI Code 318.1, Building Code Requirements for Structural Plain Concrete. In contrast to DIN 1045, the ACI code is based on working stress provisions, which thus precludes cracking of the concrete. A comparison of the permissible lining forces of the two concepts is presented. A new Combined Design Concept, based on discussions of ACI 318.1 and DIN 1045, is developed. This concept is specifically tailored to the design of underground lining and incorporates U.S. design practices. ACI Building Codes 318 and 318.1 are used for reference.

The concept used in the design of a tunnel lining should reflect boundary conditions associated with the shape of the tunnel and the construction concept used. Traditionally, tunnel shapes and load assumptions, linked to construction methods, often required a lining that could withstand bending moments. In these cases, consideration of axial forces in the lining is most often of minor design importance. However, research shows that axial thrust, rather than bending, should be the primary factor when competitive lining is required.

The promotion of axial liner thrust has proven effective. This concept has been verified through model studies performed by various authors and confirmed through theoretical analysis. An arched tunnel geometry and a highly flexible lining are shown to be the crucial factors in the design of economic structures. If these criteria are satisfied, axial thrust in the lining, rather than bending, will govern design requirements, provided that adequate construction concepts are implemented. Reinforcement or use of a thicker liner, otherwise required to accommodate bending moment, can frequently be omitted.

Consequently, a design concept is required that is tailored to thin, unreinforced, arched tunnel lining. To account for the specific boundary conditions of tunnel linings, it is necessary that this design concept differ significantly from the existing American Concrete Institute (ACI) Building Code for Plain Concrete. Like many existing building codes, the ACI code is primarily tailored for surface structures.

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The Deutsches Institut für Normung (DIN) and the ACI codes for unreinforced concrete design are discussed in this paper, and a new design concept is developed and proposed for use in U.S. tunnel design. Nevertheless, critical evaluation by potential users will be crucial to further development of the concept to enhance meeting practical design requirements.

Basically, the proposed concept is applicable to concrete linings regardless of whether the concrete is cast in place, precast, or pneumatically applied. However, some characteristics of precast or pneumatically applied concrete (shotcrete) are not specifically dealt with here. These are primarily the handling and installation requirements of precast elements and the plastic behavior of green shotcrete when subjected to early loading from the ground. The corresponding relief of stress concentrations represents safety reserves in addition to those discussed in this paper.

Also, it should be noted that this discussion is of the sizing of the lining only. Other important factors of the design process—load assumptions, selection of a calculation model, and the like—are not dealt with.

Unreinforced linings of underground structures have been constructed throughout history. Examples are numerous, ranging from ancient tunnels to today's linings of underground structures for water, conveyance, and sewage. Also, conditions similar to those of unreinforced linings prevail in segmental (hinged) precast linings. However, it was not until the introduction of the New Austrian Tunneling Method (NATM) that the use of unreinforced cast-in-place concrete liners became standard practice for highway and transit structures.

First applications of NATM for highway tunnels, including the use of unreinforced concrete lining, date back to the early 1960s. Lined unreinforced tunnels in Austria, Switzerland, and Japan were the basis for research on and experience with this innovative lining concept.

Economic considerations and experience gained abroad prompted other countries to review their traditional design standards. In the Federal Republic of Germany, for instance, extensive use of NATM for subway systems called for the development of an analytical approach to unreinforced tunnel lining. This resulted in the definition of a new design standard, incorporated in the German building code (DIN 1045) (*1*).

Effective January 1984, the DIN code for unreinforced concrete was incorporated in the general provisions for the design and construction of tunnels within the German railway network (e.g., high-speed rail line, Hannover to Würzburg). This step is significant given the exceptionally large dimensions of the tunnels involved.

The need for a U.S. standard approach was encountered during preparation of the NATM designs for the Mount Lebanon Tunnel, Stage 1, Light Rail Transit System, Pittsburgh, Pennsylvania, and the Wheaton Station and tunnels of the Glenmont Route of the Washington, D.C., subway system. The design concept developed here was first used for the design of these sections. It offers cost-effectiveness and meets the conventional safety requirements of the U.S. standards. The author is confident that the concept represents an important step forward in the evolution of American tunnel design.

DESIGN ACCORDING TO THE GERMAN STANDARD (DIN 1045)

For the design of both reinforced and unreinforced structural members, only the strength concept is specified in DIN 1045 (*1*). The working stress model is used only to check deformation behavior under service load. Thrust capacity of the tunnel lining is defined using simplified analytical tools. The equations used are contained in Manual 220 (*2*) that accompanies the DIN Code.

Ultimate Thrust in the Lining

The ultimate thrust capacity of unreinforced concrete is expressed (*2, 3*) as

$$P_e = (A_g)\,(\beta_r)\,(1 - 2\,e/h) \qquad (1a)$$

$$P_0 = (A_g)\,(\beta_r) \qquad (1b)$$

where

- P_e = ultimate thrust at given (computed) eccentricity,
- P_0 = ultimate thrust at zero eccentricity,
- e = M/P eccentricity of thrust resultant in the lining,
- A_g = $(b)\,(h)$ area of the overall liner cross section,
- b = unit width of the lining,
- h = overall thickness of the lining, and
- β_r = design strength for concrete specified in DIN 1045, Table 12.

β_{wn} is minimum strength value of any test series performed on cubes (DIN 1045, Table 1).

$$\beta_r = (0.85)\,(0.80)\,(\beta_{wn}) = 0.7\,\beta_{wn} \qquad (2)$$

β_r values account for the reductions of prism strength from cube strength and fatigue effects (*4*). Equation 2 is valid for β_r up to 17.5 MN/m^2 (2,536 psi). For higher-strength concrete, an additional reduction of β_{wn} is required.

Thrust capacity (P_e in Equation 1) is analyzed in Figure 1 according to the effect of different design assumptions. P_e-values are indicated as percentages of P_0 (e/h = 0).

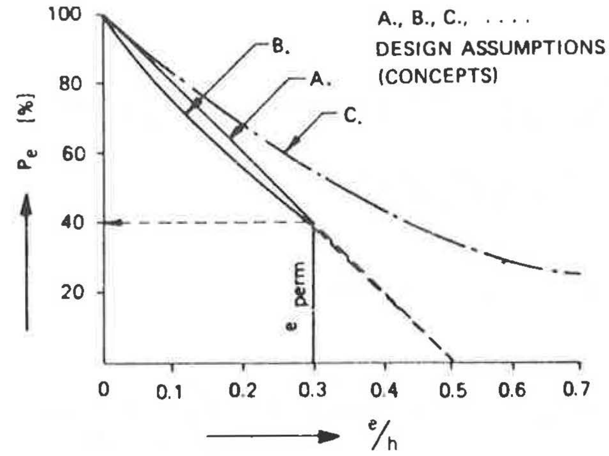

FIGURE 1 Thrust capacity of unreinforced lining according to different design assumptions.

Concept A is simplified design stress distribution according to Figure 2, bottom, and considered for Equation 1.

Concept B is parabolic stress distribution according to Figure 2, bottom, used in the strength design for reinforced concrete components (DIN 1045).

In Concepts A and B, the eccentricity input parameter (e) is conservatively based on discounting the effect of concrete tension cracking:

$e = e_l = e_{n\,.\,l}$ (see Figure 2, bottom)

where e_l is the eccentricity of liner thrust resulting from linear elastic calculations and $e_{n\,.\,l}$ is the eccentricity of liner thrust after partial cracking (nonlinear behavior) of the lining.

Concept C is stress distribution according to Concept B. In addition, a decrease of thrust eccentricity due to nonlinear behavior (cracking) is considered (*5*): $e_l > e_{n\,.\,l}$ (see Figure 2).

Figure 1 shows that the conservative design assumptions (A and B) lead to similar results and thus justify the simplified assumption (A) for determining lining capacity.

Restriction on Thrust Eccentricity and Crack Depth

According to DIN 1045 cracking from service loads must not exceed the centroid of the concrete section. This is to be achieved by limiting the thrust resultant's eccentricity (e) to 0.3 h. In comparison, the parabolic stress distribution in Figure 2, bottom, corresponds to an eccentricity of 0.294 h (*1, 4*). The difference between the two eccentricity values is considered negligible.

This relationship between the eccentricity of the thrust resultant and crack depth applies to ultimate compression only (parabolic stress distribution, Figure 2, bottom). Working stress considerations (linear triangular stress distribution, Figure 2, top) lead to a smaller crack depth (x). The eccentricity value (e) used in Equation 1 usually relates to working stress characteristics. Therefore the design approach of DIN 1045, Equation 1 linked to ultimate stress assumptions, is conservative and eliminates the requirement to check the crack depth at working stress level.

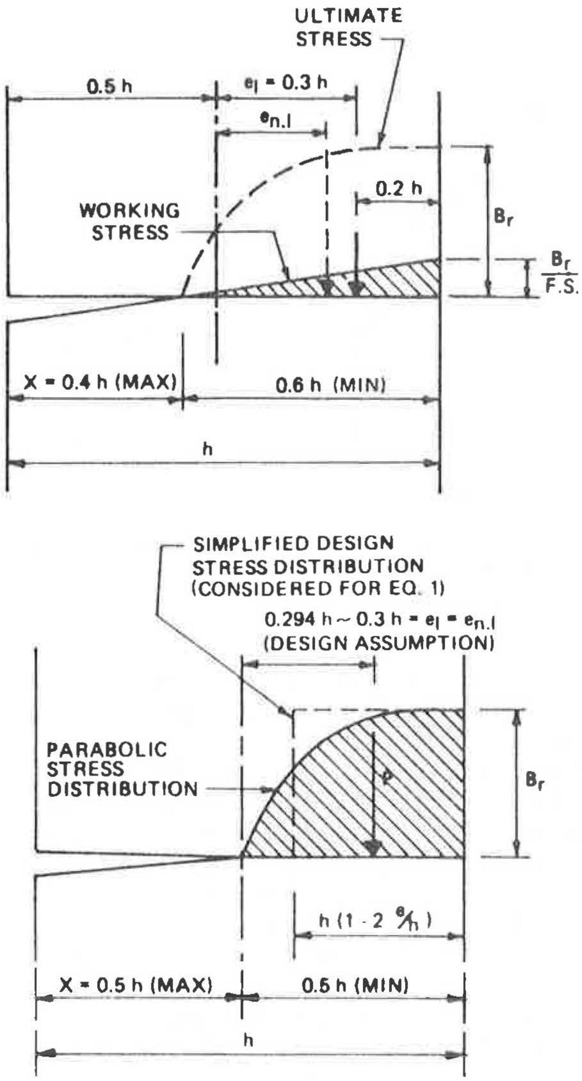

FIGURE 2 Different design assumptions at permissible eccentricity $e = 0.3\ h$: working stress design assumption and transition to ultimate stress level (*top*) and ultimate stress design assumption of DIN 1045 (*bottom*).

Safety Considerations

To account for irregularities in the structure, load uncertainties, and computational inaccuracies, a factor of safety (*FS*) is considered in arriving at a permissible thrust value:

$$P_{perm} = P_e/FS \tag{3a}$$

$$P_{perm} = P_0/FS \tag{3b}$$

where *FS* is 2.5, as specified for unreinforced concrete components in DIN 1045.

DESIGN ACCORDING TO ACI 318.1

Unreinforced concrete members are covered by the Building Code Requirements for Structural Plain Concrete (ACI 318.1) (*6*). Like most local codes, the ACI code has been developed primarily to address requirements of aboveground structures not underground structures. Consequently, working stress criteria only are provided for the design of unreinforced concrete members.

Working Stress Criteria

The principles of the working stress design concept are observed in ACI 318.1. Thus a linear elastic stress distribution in the lining is assumed. Both compression and tensile stresses must not exceed permissible values. Corresponding thrust limitations can be derived from stress restrictions in Section 6.3.6 of ACI 318.1 and Section 7.8 of ACI 322-72 (*7*):

$$(f_{au}/f_{ciu}) + [f_{bu(c)}/f_{cu}] \le 1 \tag{4}$$

with respect to limitations on compression and

$$f_{bu(t)} - f_{au} \le f_{tu} \tag{5}$$

with regard to the permissible tensile stress of the concrete where

f_{au} = compression due to axial thrust,
$f_{bu(t)}$ = tension due to bending, and
$f_{bu(c)}$ = compression due to bending.

Safety Considerations

The index (u) indicates that thrust and bending, factored by U, are considered for Equations 4 and 5. U is load factor according to ACI 318. All permissible stress values are governed by Section 6.2 of the code:

f_{cu} = permissible bending compression,
f_{ciu} = permissible axial compression, and
f_{tu} = permissible bending tension.

The permissible stress (f_{perm}), induced by service load (P_n) equals (f_n) (ϕ)/U. Therefore the factored permissible stresses as used in Equations 4 and 5 [i.e., induced by the factored load (P_u)] equal (f_n) (ϕ). f_n is nominal strength and ϕ is 0.65, strength reduction factor, according to ACI 318.1.

Permissible Thrust in the Lining

Permissible thrust values (P_{perm}), considering compression limits, can then be derived from Equation 4:

$$[(P_u/A_g)/f_{ciu}] + [P_u\,(e/S)/f_{cu}] \le 1 \tag{6a}$$

where S is $bh^2/6$, section modulus. By transformation,

$$P_{perm} = (f_{ciu})\,(f_{cu})/[(f_{cu}/A_g) + e\,(f_{ciu}/S)]\;U \tag{6b}$$

Permissible thrust values with respect to tensile capacity can be derived from Equation 5:

$$(P_u\, e/S) - (P_u/A_g) \leq f_{tu} \tag{7a}$$

and by transformation,

$$P_{\mathrm{perm}} = f_{tu}/[(e/S) - (1/A_g)]\ U \tag{7b}$$

Equations 6b and 7b require the factored permissible stress values $[(f_n)\ \phi]$ contained in ACI 318.1, Section 6.2, and ACI 322-72, Table 7.1. The nominal strength values included in these parameters are broken out. Nominal strength (f_n) according to ACI 318.1 for flexure is

$$f_{cn} = f_c' \quad \text{(compression)} \tag{8}$$

$$f_{tn} = 5(f_c')^{1/2} \quad \text{(tension)} \tag{9}$$

For axial compression it is

$$f_{cin} = 0.6\, f_c' \tag{10}$$

Instead of using Equations 6b and 7b, P_{perm} can be calculated using these nominal strength values:

$$P_{\mathrm{perm}} = (0.6\, f_c')\ (f_c')\ (0.65/[(f_c'/A_g) + e(0.6\, f_c'/S)]\ U \tag{11}$$

substituting Equation 6b and

$$P_{\mathrm{perm}} = [5(f_c')^{1/2}]\ 0.65/[(e/S) - (1/A_g)]\ U \tag{12}$$

substituting Equation 7b.

COMPARISON OF DIN 1045 AND ACI 318.1

Comparison of the two codes evaluates the impact of different design assumptions. Both concepts are expected to lead to structurally sound underground linings. However, the degree of conservatism implicit in the design assumptions varies, and this potentially affects the economy of the underground structure. The same factor of safety, $FS = 2.50$, is used for either concept to eliminate the effect of explicitly imposed safety margins. According to ACI 318.1,

$$f_{\mathrm{perm}} = f_n\ (\phi/U) \tag{13a}$$

where ϕ is 0.65 (*6*) and U is 1.63, leading to $FS = 2.5$.

The value of 1.63 represents average load conditions; compared with the full range of U from 1.4 to 1.7,

$$f_{\mathrm{perm}} = (0.65/1.63)\, f_n = 0.40\, f_n \tag{13b}$$

According to DIN 1045,

$$f_{\mathrm{perm}} = \beta r/2.5$$

$$\beta r = 0.56\ \beta_{ws}\ (8)$$

$$f_c' = 0.85\ \beta_{ws} \tag{14a}$$

$\beta_{ws} = f_w$ = cube strength, averaged from a test series according to DIN 1045.

By substitution,

$$f_{\mathrm{perm}} = 0.66\, f_c'/2.5 = 0.26\, f_c' \tag{14b}$$

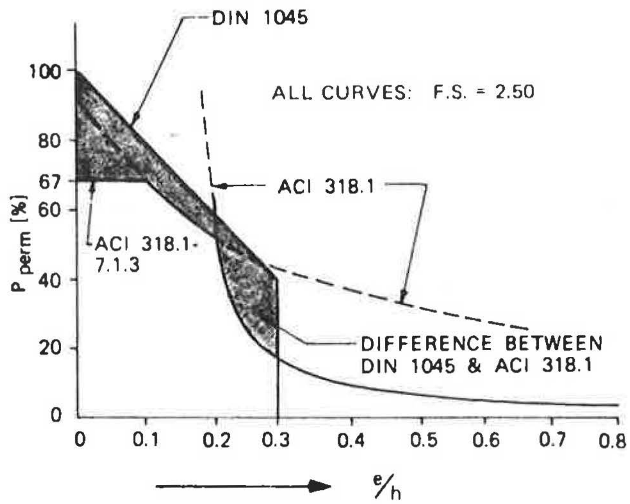

FIGURE 3 Comparison of permissible thrust according to ACI 318.1 and DIN 1045.

Permissible thrust values given in Table 1 and shown in Figure 3 are determined from Equations 1 and 3, and 11 and 12, respectively. Values are indicated as percentages of P_{perm} at zero thrust eccentricity determined according to DIN 1045.

Plain concrete provisions according to ACI 318.1 lead to a considerably smaller design capacity than do those calculated according to DIN 1045. This reflects the tensile stress limitations and the observation of a minimum eccentricity value specified in the ACI code. However, it must be noted that many local codes developed from DIN 1045 call for consideration of a minimum "unavoidable" eccentricity as well.

DEVELOPMENT OF THE COMBINED DESIGN CONCEPT

General

According to the Guidelines for Tunnel Lining Design of the Underground Technology Research Council (*9*), "structural codes should be used cautiously" because "most codes have been written for above-ground structures on the basis of assumptions that do not consider ground lining interaction." The guidelines further state: "Blind application of structural design codes is likely to produce limits on the capacity of linings that are not warranted in light of the substantial contributions from the ground and the important influence of construction methods on both the capacity and cost of linings." Similar statements have been made by other authors and organizations (*5, 10, 11*).

Careful evaluation of the applicability of existing structural codes to the design of tunnel liners was necessary. Comparison of the German and the U.S. standards for unreinforced concrete design indicates that the strength model used in DIN 1045 allows greater use of the lining capacity than do the ACI 318.1 working stress criteria. It is widely acknowledged that the strength criteria can represent a sound basis for the design of unreinforced concrete tunnel lining. Nevertheless, it is understandable that use of foreign codes (i.e., the German DIN code) is severely limited in the United States. Therefore a new concept needed to be developed to better reflect typical U.S. situations.

TABLE 1 PERMISSIBLE THRUST VALUES

ECCENTRICITY	ACI 318.1			DIN 1045
e/h	EQ. 11	EQ. 12	EQ. 11, 12	EQ. 1, 3
0	(92) %	– %	(92) %	100 %
0.1 *	67	–	67	80
0.2	53	64	53	60
0.3	43	16	16	40
0.4	37	9	9	(20)
0.5	32	6.5	6.5	(0)
0.6	28	4.9	4.9	–
0.7	25	4.0	4.0	–
0.8	23	3.3	3.3	–

* MINIMUM ECCENTRICITY ACCORDING TO ACI 318.1/SECTION 7.1.3

This design concept uses design parameters and safety definitions incorporated in ACI 318 (*12*). In addition, elements of the German standard are used, thus making use of a proven tool for applying strength criteria to unreinforced concrete design.

Figure 4 shows the sources used in developing the design approach. Elements of two codes are combined to arrive at the Combined Design Concept. Their applicability is subject to the extent of concrete cracking that corresponds to the eccentricity of the thrust resultant. For eccentricities up to a limit defined later, a strength design concept, referred to as Modified Strength Design, is used. For eccentricities that exceed the defined limit, a working stress concept is applied to the uncracked lining portion, thus averting further cracking of the concrete lining.

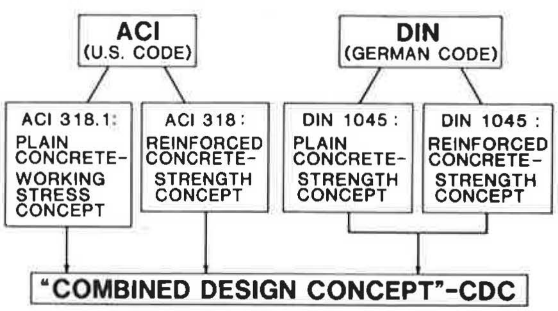

FIGURE 4 Derivation of the Combined Design Concept.

Modified Strength Design Based on ACI 318 and the German Standard

A liner's capability to sustain nonlinear behavior induced by an eccentric thrust is best modeled by the strength design approach. This is also indicated by the Alternate Design provisions in ACI 318 (*6, 12*) in which it is stressed that "the straight line theory [as it is used in the working stress method of ACI 318.1] applies only to design members in flexure without axial load." Such load conditions do not typically occur in arched structures.

The Modified Strength Design concept developed here follows the strength design concept (ACI 318) in the design assumptions made for stress and strain in the concrete. Deviating from ACI 318, however, the developed design concept considers the potential of tunnel liners to render reinforcement (*5, 10, 11*) unnecessary.

According to DIN 1045 and ACI 318, a simplified rectangular compression stress figure is recommended for design purposes unless test results call for other distributions. Simplified rectangular stress figures are derived differently in the design concepts shown in Figure 5. DIN 1045 and the Modified Strength Design define smaller ultimate compression stress blocks than are granted by ACI 318 (Figure 5, right). Corresponding ultimate strain assumptions are shown in Figure 5, left. The correlation between these ultimate stress-strain assumptions and working stress conditions has been shown in Figure 2. The effect on the conservatism implicit in the design assumptions is analyzed as follows.

According to ACI 318,

$$c = 3\,[(h/2) - e] \tag{15a}$$

$$a = 0.85c = 2.55\,[(h/2) - e] \tag{15b}$$

According to DIN 1045 and the Modified Strength Design,

$$\bar{a} = 2.0\,[(h/2) - e] \tag{15c}$$

where $\bar{a}$ is the depth of the rectangular design stress block according to DIN 1045 and the Modified Strength Design.

The conservatism in the Modified Strength Design compared with ACI 318 regarding the width of the design stress block is expressed by

$$a/\bar{a} = 2.55/2.0 = 1.28$$

In the Modified Strength Design, ultimate thrust capacity is written as

$$P_e = (b)\,(h)\,(0.85\,f_c')\,[1 - (2\,e/h)] \tag{16a}$$

for $e > 0.1\,h$ and as

$$P_0 = 0.8\,(h)\,(b)\,(0.85\,f_c') \tag{16b}$$

for $e \leq 0.1\,h$

Because these equations are based on tension cracking, their validity is limited to reaching a permissible crack width as specified later.

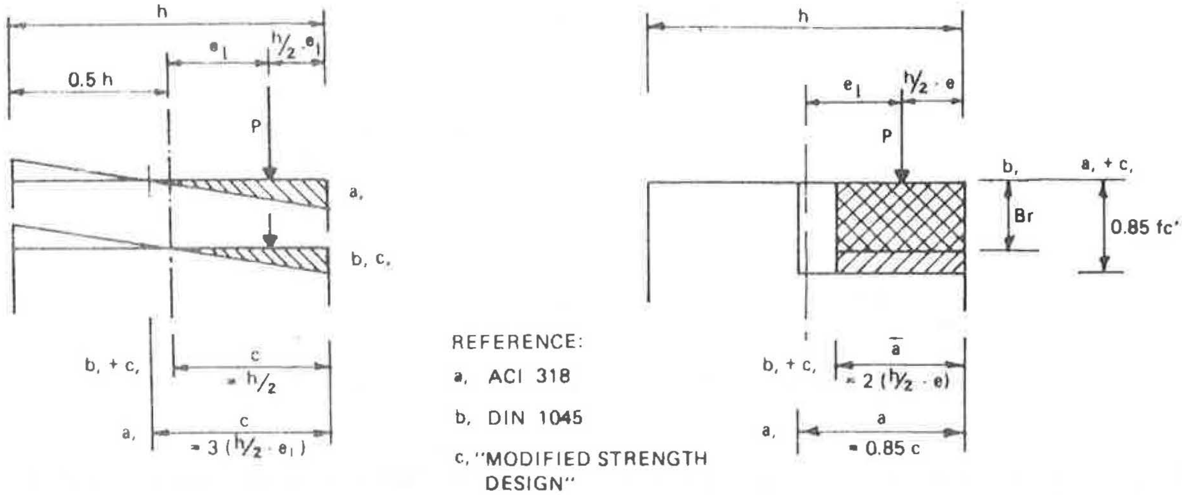

FIGURE 5 Design parameters for strength design concepts: strain design assumption (*left*) and ultimate stress design block (*right*).

Crack Restrictions

Distortion is controlled by limiting thrust eccentricity (e), to $e =$ 0.30 of the liner thickness (h). In the ultimate stress design, such eccentricity corresponds to cracking to the centroid of the cross section. In comparison, stresses from service loads lead to smaller crack depths (Figure 2, top).

Design Based on ACI 318.1 Beyond Maximum Permissible Cracking

Crack restrictions limit the use of the strength design method to a maximum eccentricity of 0.3 h. However, greater eccentricities will not cause further cracking as long as tension stresses in the uncracked liner portion remain within the elastic range. A pertinent design approach is provided by the working stress method of ACI 318.1. Observation of the permissible concrete tension (Equations 9 and 13) yields the permissible values of liner thrust at given eccentricities.

Figure 6 shows a linear stress distribution as induced by service load (P_n) at the critical eccentricity (e = 0.3 h). A comparison with ultimate stress conditions is shown in Figure 2. The depth of the compressed liner portion ($\bar{h}_{compr}$) is

$$\bar{h}_{compr} = 3 \times (0.2\ h) = 0.6\ h \tag{17}$$

When the concrete's tensile strength is considered, an even larger uncracked liner depth ($\bar{h}$) is actually provided. It is governed by the provision of ACI 318.1 that the thrust (P_n) must not cause tensile stresses that exceed permissible values (*6*). From Equation 7b it generally follows that

$$P_{perm} = f_{t\ perm}/[(e/S) - (1/A_g)] \tag{18}$$

and

$$P_{perm} = f_{t\ perm}/[\bar{e}/\bar{S}) - (1/\ \bar{A}_g)] \tag{19}$$

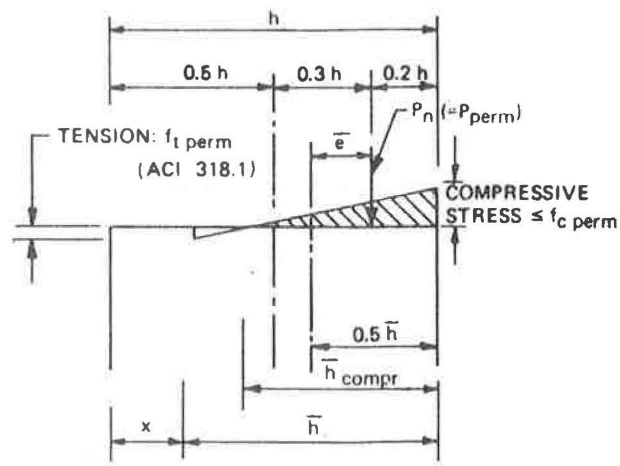

FIGURE 6 Cracking of the lining at the strength design limit, e = 0.3 h.

where

$\bar{A}_g$ = $b\bar{h}$, uncracked portion of the lining pertinent to thrust eccentricity e = 0.3 h;
$\bar{h}$ = uncracked depth of the lining pertinent to e = 0.3 h;
$\bar{S}$ = $b\bar{h}^2/6$, section modulus of the uncracked lining portion; and
$f_{t\ perm}$ = $5(f_c')^{1/2}\ (\phi/U)$, according to Equations 9 and 13.

Uncracked lining depth ($\bar{h}$), is determined by iteration from Equation 20 developed from Equation 19 after substitution and transformation. It was demonstrated earlier that the uncracked depth of the cross section is greater than 0.6 h. For most cases, a value of approximately 0.65 h will result from the iteration. The value of P_{perm} considered in Equation 20 pertains to e = 0.3 h and is calculated according to Equation 16a with the additional consideration of a safety factor (FS).

$$[2 - 1.2\ (h/\bar{h})]/b\bar{h} = f_{t\ perm}/P_{perm} \tag{20}$$

The thrust eccentricity ($\bar{e}$) can then be expressed by $\bar{h}$:

$$\bar{e} = [0.5\,\bar{h} - (0.5\,h - e)] \tag{21}$$

P_{perm} for $e > 0.3\,h$ is calculated using Equations 19–21:

$$P_{perm} = [5(f_c')^{1/2}]\,0.65/[(\bar{e}/\bar{S}) - (1/b\bar{h})]\,U \tag{22}$$

Equation 22 is reflected in Figure 7. The graph shows how P_{perm} increases with a decreasing thrust eccentricity ($\bar{e}$).

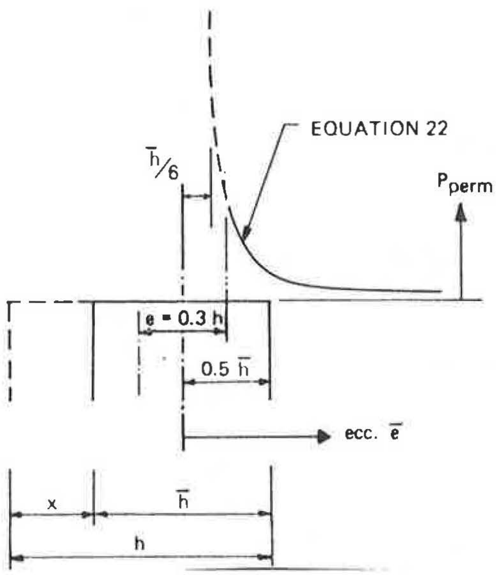

FIGURE 7 Permissible thrust versus eccentricity, $e > 0.3\,h$.

Safety Considerations

In accordance with ACI building code requirements for both reinforced and unreinforced concrete, a permissible thrust value must include an explicit safety factor that is related to the ultimate design capacity of the liner. Load uncertainities are hereby accounted for by a factor (U). Reduction of the ultimate design strength by ϕ accounts for imperfections of the structure and computational inaccuracies. A compound safety factor can thus be written:

$$FS = U/\phi \tag{23}$$

Explicit and implicit safety components are analyzed elsewhere (*8*). They are considered in determining the factors U and ϕ to be used in the proposed liner design concept. The evaluation suggests that a conservative approach will be taken by meeting the safety factor provisions of ACI 318.1 ($\phi = 0.65$ and $U = 1.4$ to 1.7). These parameters provide a compound safety factor of from 2.15 to 2.62 according to Equation 23.

Comparison of Design Concepts

The graph shown in Figure 8 reflects the computation of permissible thrust values given in Table 2. Although Table 2 covers the full range of load factors (1.4 through 1.7), $U = 1.63$ is based on the graphic display.

For the Combined Design Concept ($\phi = 0.65$) the factor $U = 1.63$ corresponds to $FS = 1.63/0.65 = 2.5$. Values in Table 2 are indicated as percentages of P_{perm} at zero eccentricity determined according to DIN 1045. Values in parentheses relate to

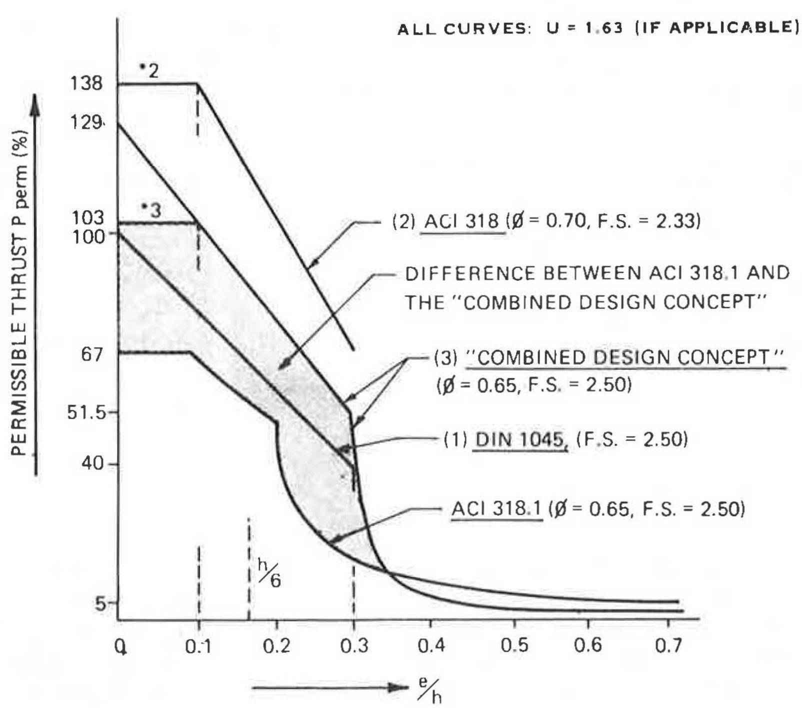

FIGURE 8 Comparison of permissible thrust according to ACI 318 ("Strength Design"), DIN 1045, ACI 318.1, and the Combined Design Concept.

TABLE 2 DESIGN FACTORS

Reference to Figure 8		Thrust Eccentricity	Design Criteria					Permissible Thrust
			Design Safety			Ultimate Design Strength	Width of Stress Figure	
Design Concept	Notes	e/h	Ø	U	F S	% of B_r	[(h/2) - e]	%
(1) DIN 1045		0.0	—	—	2.50	100	2.00	100
		0.3			2.50	100	2.00	40
(2) ACI 318	*2	0.0	0.70	1.4 (1.7)	2.00 (2.43)	129	2.00	161 (132)
		0.3	0.70			129	2.55	82 (68)
(3) "Modified Strength Design"	*3	0.0	0.65	1.4 (1.7)	2.15 (2.62)	129	1.60	120 (99)
	*1	0.3	0.65			129	2.00	60 (49)

Note: *1, equivalent to modified strength design for eccentricity $e \geq 0.3\ h$. *2, $e_{min} = 0.11\ h$, according to the strength design parameter used [see Figure 5, and refer to equation 15b in which $a = 2.55\ (0.5\ h - e)$]; $a_{max} = 2.55\ (0.5\ h - 0.11\ h) = h$. *3, $e_{min} = 0.10\ h$; according at ACI 318.1, Section 7.1.3.

the maximum load factor ($U = 1.7$), and other values relate to minimum safety factors permitted by the codes. For a comparison with P_{perm} according to ACI 318.1 refer to Figure 3.

Figure 8 indicates permissible thrust values subjected to the design parameters (Figure 5) and safety factors used. The Combined Design Concept (i.e., the Modified Strength Design for eccentricities smaller than 0.3 h) yields similar results for concentric thrust but allows higher permissible values for eccentric thrust in the lining than does DIN 1045.

A prime purpose of combining two distinctly different concepts in the Combined Design Concept was to extend the applicability of unreinforced concrete design. In particular, the combined concept allows for eccentricities greater than the strength design limit (cracking to one-half of the lining depth; $e_{max} = 0.3\ h$), if the liner thrust is small enough to satisfy working stress criteria imposed on the uncracked portion of the lining depth. This provision can be crucial to permitting use of unreinforced concrete where load cases generate low thrust values that increase the eccentricity of the thrust resultant (e.g., dead weight).

Figure 8 also makes it possible to evaluate the impact of using the working stress design according to ACI 318.1 over different eccentricity ranges. P_{perm} is evaluated for eccentricities greater than 0.3 h. It shows that ACI 318.1 yields slightly more conservative results if used only for $e > 0.3\ h$ (Figure 7 and Equation 22). In contrast, the curve describing Equation 12 is based on the unlimited use of ACI 318.1 (i.e., for all eccentricities). This results in a higher P_{perm} for $e > 0.33\ h$.

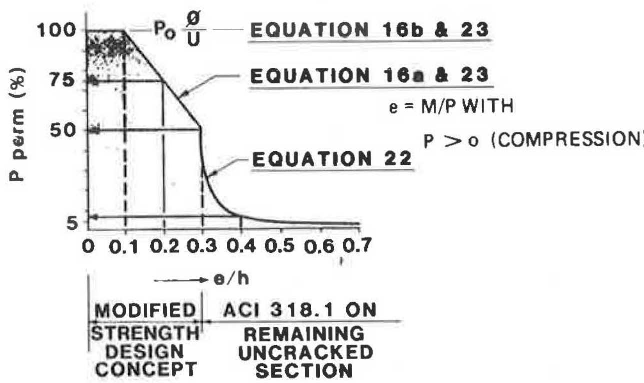

FIGURE 9 Design chart for permissible liner thrust.

compression stress constitute the criterion for P_{perm}. This is the case for eccentricities of up to 0.3 h. For eccentricities greater than 0.3 h, permissible tension stresses govern P_{perm}. According to ACI 318 and DIN 1045, permissible tension stress induced by service loads is related to compression strength characteristics by a nonlinear correlation law. As a result, limits on compression and tension, respectively, diverge with increasing concrete strength. The graph in Figure 9 is based on a concrete strength frequently used for tunnel lining design ($f_c' = 3{,}500$ psi). Therefore, for most cases, Figure 9 can be used as a design tool, which makes it possible to avoid analytical calculations. Also, deviations from other concrete strengths are negligible for most practical purposes.

Summary of the Combined Design Concept

The permissible liner thrust according to the design concept developed in this paper is shown in Figure 9. The eccentricity (e) of the compression resultant corresponds to a linear elastic stress distribution and is indicated as a variable input parameter. Permissible thrust values are given as percentages of the permissible thrust at zero eccentricity.

Figure 9 is designed from Equations 16a, 16b, 22, and 23 and is valid regardless of specific design data if restrictions on

NOTATION

a = depth of rectangular design stress block according to ACI 318;

$\bar{a}$ = depth of rectangular design stress block according to DIN 1045 and the Modified Strength Design method;

A_g = gross area of overall section;

$\bar{A}_g$ = gross area of uncracked section portion after partial cracking;

b = width of compression face of lining (unit width);
c = distance from extreme compression fiber to neutral axis, design assumption;
e = eccentricity of thrust in the lining, general;
e_l = eccentricity of liner thrust resulting from linear elastic calculations (service load condition);
$e_{n \cdot l}$ = eccentricity of liner thrust after partial cracking (nonlinear lining behavior);
$\bar{e}$ = eccentricity of liner thrust measured from the centroid of the uncracked liner portion after partial cracking of the unreinforced structure;
f_c' = compressive strength of concrete specified in ACI 318;
f_w = average cube strength of a series according to ACI 318;
f_u = compressive stress in concrete induced by P_u, general;
f_{au} = compression stress induced by P_u at zero eccentricity;
$f_{bu(t)}$ = concrete tension stress induced by M_u;
$f_{bu(c)}$ = compression stress induced by M_u;
$f_{c\ \mathrm{perm}}$ = permissible compressive stress in concrete, general;
f_{cu} = $f_{cn}\ \phi$, permissible compression stress in concrete subjected to M_u;
f_{ciu} = $f_{cin}\ \phi$, permissible compression stress in concrete subjected to P_u at $e = 0$;
f_{tu} = $f_{tn}\ \phi$, permissible tension stress in concrete subjected to M_u;
f_n = nominal concrete strength, general;
f_{cin} = nominal concrete compression strength subject to P at $e = 0$ according to ACI 318.1;
f_{tn} = nominal concrete tension strength subject to M according to ACI 318.1;
f_{cn} = nominal concrete compression strength subject to M according to ACI 318.1;
$f_{t\ \mathrm{perm}}$ = permissible concrete tension stress induced by P_n;
FS = factor of safety;
h = overall thickness of lining;
$\bar{h}$ = minimum thickness of an uncracked lining portion;
M = bending moment, general = P (e);
M_u = factured moment = P_u (e);
P = axial load, general;
P_n = nominal axial load (= service load);
P_u = factured axial load;
P_c = concrete compressive force induced by P_u;
P_0 = ultimate thrust at zero eccentricity;
P_e = ultimate thrust at given eccentricity;
P_{perm} = permissible P_n;
S = section modulus of overall section;
$\bar{S}$ = section modulus of uncracked section portion;
U = load factor;
x = depth of cracked section portion;
β_{ws} = f_w, average cube strength of a series according to DIN 1045;
β_{wn} = Minimum value of any cube strength test series according to DIN 1045;
β_r = Design strength for concrete according to DIN 1045; and
ϕ = strength reduction factor.

ACKNOWLEDGMENT

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Transverse Ventilation System of the Holland Tunnel Evaluated and Operated in Semitransverse Mode

N. LESSER, F. HOROWITZ, AND K. KING

The Holland Tunnel, which has a fully transverse ventilation system, had to have its ceiling removed because of concrete deterioration. For more than a year before the installation of a new ceiling, vehicular traffic would continue, and the ventilation system would operate in a semitransverse mode. Evaluations were conducted to ascertain the capability of the ventilation system to maintain an acceptable environment during normal vehicular usage and in the event of fire. These evaluations included computations for varying traffic patterns and actual fire testing.

Once described as the eighth wonder of the world, the Holland Tunnel opened for use in 1927 and was designated a National Historic Civil and Mechanical Engineering Landmark on May 2, 1984. One of the reasons for conferring landmark status on the tunnel was an unprecedented ventilation system that made possible the longest underwater vehicular tunnel of the time. The tunnel consists of two independent tubes, each with two traffic lanes, approximately 8,500 ft long.

Because of deterioration after nearly 60 years of operation, the ceiling, which forms a basic element of the ventilation system, had to be replaced. However, the tunnel tubes, which are a vital link in commuting and trucking routes in and out of New York City, could not be shut down for an extended period. All work—removals, measurements, and new construction—would have to be done at night and the tunnel tube restored for traffic use each morning. The simplest way to implement such a staging requirement is to completely remove the existing ceiling, take measurements, and then prefabricate and install sections. This procedure would result in the tunnel tubes being without a ceiling for more than a year.

Evaluations and field tests were conducted to determine whether, without a ceiling, the ventilation system could

- Maintain environmentally acceptable conditions under normal traffic patterns and
- Control smoke during fire emergency conditions.

The evaluations included computer simulations of contaminant levels that would result from varying traffic patterns, critical velocity computations for smoke control, and in situ fire tests.

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EXISTING SYSTEM

The existing ventilation system for the Holland Tunnel was designed as a fully transverse system. The ventilation system is divided into sections along the length of the tunnel; each supply and exhaust section is served by multiple fans; each fan is capable of operation at three or four speeds.

Outside air is supplied from the chamber under the roadway through openings spaced approximately 15 ft apart along both curbs. Vitiated air is exhausted through ports spaced 15 ft apart in the original ceiling. The ports are centered above each lane of traffic. The exhaust ports are sized to balance air quantities, and, as a result, rather small ports are located near ventilation building shafts. Figure 1 shows a longitudinal cross section of the tunnel tubes and the various ventilation sections and ventilation buildings. Figure 2 shows a transverse cross section. The vitiated air is monitored for contaminant levels in each ventilation section, and the appropriate number of fans is manually set at the speed required to maintain safe contaminant levels.

The system has proven to be fully capable of controlling contaminant levels under normal operating conditions including peak traffic flows and traffic stoppages.

During fire emergencies, when the smoke and heat generated are within the moderate range—this includes a burning automobile or even a truck that is not carrying a highly flammable or "smokey" load—the ventilation system is reconfigured to control the smoke and heat and provide life-supporting uncontaminated air to vehicle occupants and fire fighters within the tunnel. Maximum exhaust capacity is used in the section that contains the fire, and supply is provided in all sections. (Highly flammable cargo is not allowed in the tunnel.)

The basic traffic pattern is eastbound in the south two-lane tube and westbound in the north two-lane tube. However, at times, to facilitate maintenance, two-way traffic is implemented in a tube. When a two-way traffic pattern is in effect, the ventilation system serves in the normal manner.

INTERIM SYSTEM

With the ceiling removed, the ventilation system is operated in a semitransverse pattern. Computations, using the FHWA TUNVEN computer program, were performed to evaluate the resulting contaminant levels. The TUNVEN program permits the solution of coupled, one-dimensional, steady-state tunnel aerodynamics and advection equations to obtain longitudinal

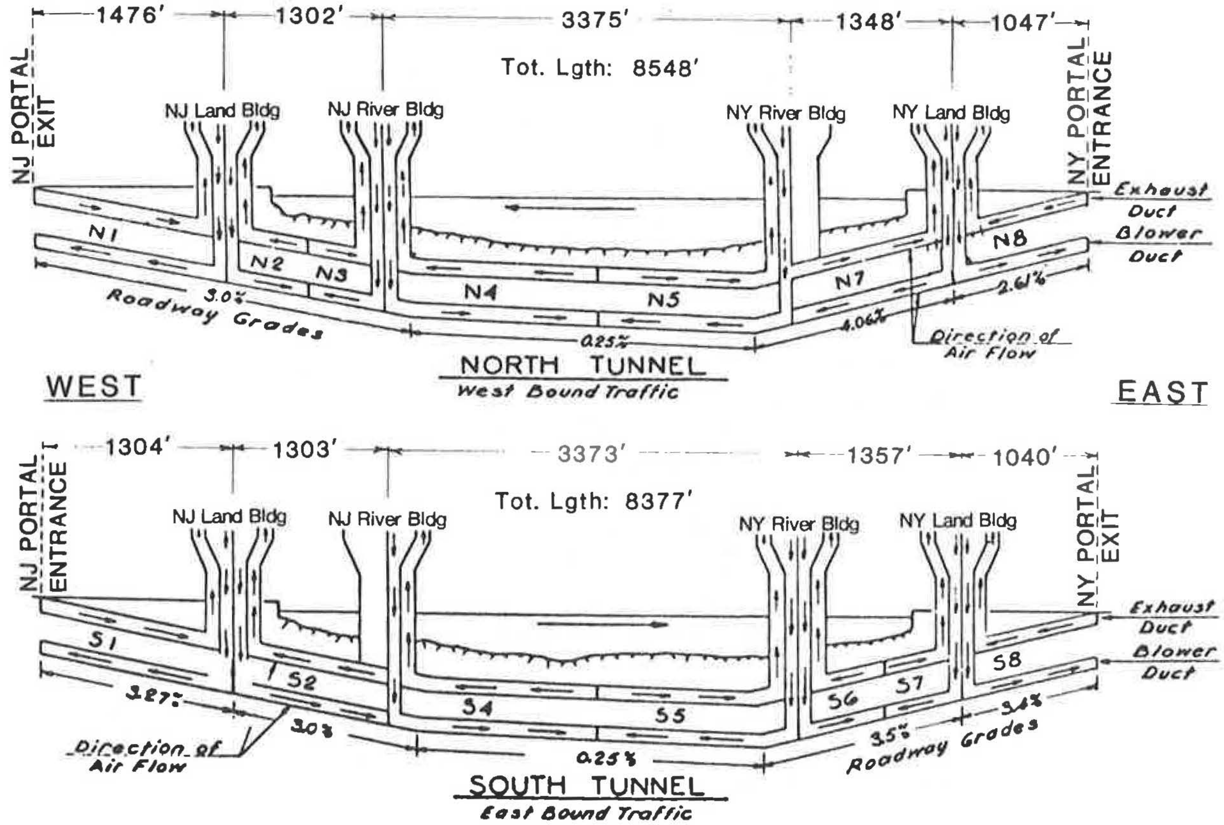

FIGURE 1 Longitudinal cross section of Holland Tunnel tubes.

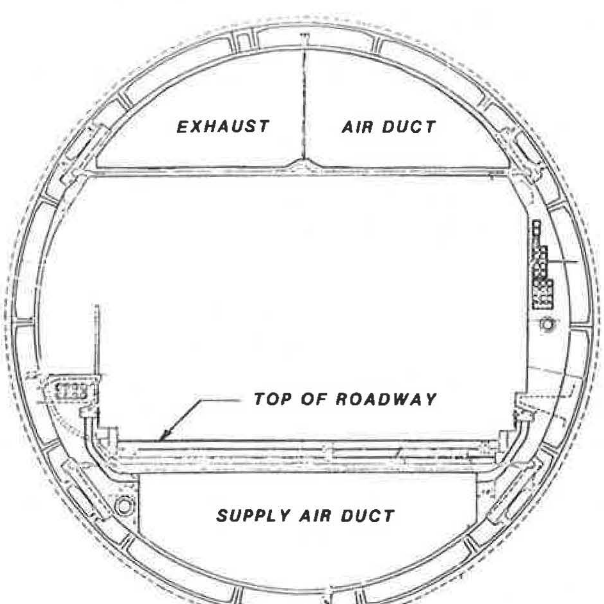

FIGURE 2 Transverse cross section of Holland Tunnel tubes.

air velocities and pollutant concentrations for a given tunnel design, traffic load, and ventilation rate. The program can be used to determine ventilation requirements for natural, semitransverse, and fully transverse ventilation systems. In this instance the ventilation rates were fixed, and the adequacy of these rates to control contaminant levels was sought. On the basis of the results, which predicted satisfactory contaminant levels, it was decided to remove the existing ceiling in its entirety. Segment-by-segment removal and replacement would not be pursued.

Figure 3 shows measured values for the fully transverse and the semitransverse configurations under heavy traffic conditions.

During a fire emergency, the ability of the ventilation system to mitigate the risks will depend, as it does with the present ceiling, on the severity of the fire. The ventilation system would be reconfigured to control smoke movement in a specific direction. This presumes that traffic is one way and that all vehicles downstream of the incident have left the tunnel. A specific reconfiguration was developed for each tunnel tube section. The underlying principle was to exhaust downstream (traffic flow) of the ventilation section containing the fire, supply air upstream of that section, and provide minimal air supply in the fire section and all downstream sections.

Longitudinal velocities of the air should be sufficient to prevent back-laying or bidirectional air flow (smoke-laden air moving above uncontaminated air moving in the opposite direction) and to be low enough in velocity to minimize turbulence. This critical longitudinal velocity was initially determined through calculations using the UMTA Subway Environment Simulation (SES) computer program. The intent is to keep, for as long as possible, the smoke-laden air, which is moving to the exhaust shaft, in the upper cross section of the tunnel. The air supplied to the fire section and all sections downstream is intended to support the desired stratification and to provide life-sustaining air at lower levels for fire-fighting personnel. Fire department vehicles would proceed to the fire

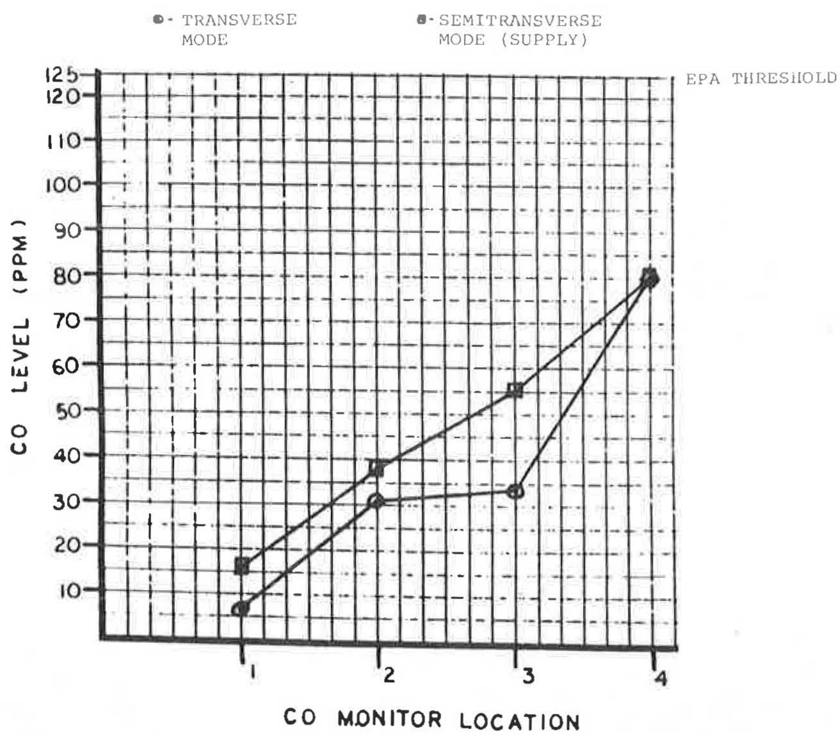

FIGURE 3 Comparison of CO levels of transverse configuration with those of semitransverse configuration under heavy traffic.

from the downstream direction, moving counter to the smoke exhaust flow. Maintaining the stratification of smoke-laden air for as long as possible is essential to providing acceptable visibility for fire department access.

There would be obvious advantages to permitting two-way traffic in one two-lane tunnel tube while the other tube is out of service for construction activity. However, the emergency ventilation approach just described for fire conditions is considered inadequate for two-way traffic. Should a major fire develop with two-way traffic, vehicles could have great difficulty exiting the tunnel in both directions. Reconfiguring the ventilation system in an attempt to control movement of smoke and heat in one direction or another would create the risk that trapped vehicles and their occupants would be enveloped in smoke. This factor, coupled with the possibility of having a more serious accident with two-way traffic, increases the life safety risk significantly. The risk could be mitigated somewhat by limiting traffic speed, stationing additional traffic control personnel within the tunnel, and substantially increasing cargo inspections or precluding truck traffic. However, short of precluding truck traffic, these measures would not modify the level of risk appreciably, and such a preclusion is impractical. Therefore the decision was made to not permit two-way traffic within a single tube. Diversion to the Lincoln Tunnel would be used instead.

FIRE TESTS

Because it was decided on rather short notice to field test the emergency reconfiguration of the ventilation systems, there was not time to instrument the tunnel.

Four configurations were tested in an attempt to evaluate the following parameters:

- Maximum supply upstream and maximum exhaust downstream,
- Minimum supply and exhaust,
- Maximum supply upstream and downstream and maximum exhaust downstream, and
- Moderate supply upstream and moderate exhaust and supply downstream.

The tests were conducted in ventilation Section S5 in the south tube. The distance from the test fire to the nearest downstream ventilation building was approximately 1,000 ft. The fire and smoke were created by the combustion of diesel fuel in three open half-sections of 55-gallon drums. The fire was estimated at 2 million Btu/hr per drum or approximately 6 million Btu/hr total. The critical velocity for a 2 million Btu/hr fire was calculated to be 250 ft/min; for 8 million Btu/hr, 400 ft/min; and for 30 million Btu/hr, 600 ft/min.

The proposed fan configuration for Test Run 1 is given in Table 1 (refer also to Figure 1).

In Test 1, the maximum air quantity that the installation was capable of supplying was provided in Ventilation Sections S1 and S2, and almost the maximum was provided in Section S4 with no exhaust. Minimum outside air was supplied in Sections S5, S6, S7, and S8. Maximum available exhaust was used in Section S5 and near maximum was used in the other downstream sections (S6, S7, and S8). The intent was to create high air velocities at the fire location and, as previously noted, prevent back-layering. The resulting effect was an entirely smoke-free tunnel upstream with smoke moving downstream

TABLE 1 EMERGENCY FAN CONFIGURATION OF SOUTH TUBE SECTION S5 (Test 1)

	Exhaust		Supply	
Ventilation Section	Fan/ Speed	ft^3/ min	Fan/ Speed	ft^3/ min
S1	None		3/4	260,000
S2	None		3/4	180,000
S4	None		3/3	325,000
S5[a]	3/4	500,000	1/1	100,000
S6	3/2	245,000	1/1	50,000
S7	3/3	180,000	1/1	50,000
S8	3/2	190,000	1/1	75,000

[a]Location of fire.

within the upper one-third of the tunnel tube cross section for approximately 400 to 500 ft and then dropping to the roadway. After the smoke dropped, visibility within the tube was poor. As a result, there was inadequate visibility for fire department vehicles to proceed safely to the fire; therefore this reconfiguration was considered unacceptable.

Test 2 (Table 2), the minimum air scheme, was an attempt to minimize turbulence to the fullest extent possible. Table 2 gives the fan configuration for this reconfiguration. Smoke moved downstream in the upper part of the tube cross section at approximately 250 ft/min; however, smoke also moved upstream and dropped to the roadway. The resulting back-layering and lack of control eliminated this approach.

TABLE 2 EMERGENCY FAN CONFIGURATION OF SOUTH TUBE SECTION S5 (Test 2)

	Exhaust		Supply	
Ventilation Section	Fan/ Speed	ft^3/ min	Fan/ Speed	ft^3/ min
S1	None		1/1	50,000
S2	None		1/1	50,000
S4	None		1/1	90,000
S5[a]	1/1	100,000	1/3	180,000
S6	1/1	60,000	1/1	50,000
S7	1/1	50,000	1/1	50,000
S8	1/1	75,000	1/1	75,000

[a]Location of fire.

Test 3, as indicated in Table 3, represented a configuration in which a high air capacity was supplied downstream of the fire as well as upstream. Also, high exhaust capacity was maintained downstream of the fire. The idea was to dilute and purge the smoke-laden air so as to allow at least 100 ft of visibility downstream. This approach was not successful because the quantity of air supplied was insufficient to dilute the smoke-laden air, which quickly cooled and dropped to the roadway. Visibility was not adequate.

In Test 4 (Table 4) a moderate quantity of outside air was supplied upstream of the fire in Sections S1, S2, and S4. In Section S5, the section with the fire, a moderate supply was maintained with the exhaust twice the amount of supply. In the more remote downstream sections minimum supply and exhaust were maintained. The velocity created within the fire section (S5) was approximately 500 ft/min.

TABLE 3 EMERGENCY FAN CONFIGURATION OF SOUTH TUBE SECTION S5 (Test 3)

	Exhaust		Supply	
Ventilation Section	Fan/ Speed	ft^3/ min	Fan/ Speed	ft^3/ min
S1	None		3/4	24,000
S2	None		3/4	180,000
S4	None		3/4	400,000
S5[a]	2/4	400,000	3/4	450,000
S6	3/2	250,000	3/4	200,000
S7	3/4	250,000	3/4	200,000
S8	1/1	75,000	1/1	75,000

[a]Location of fire.

TABLE 4 EMERGENCY FAN CONFIGURATION OF SOUTH TUBE SECTION S5 (Test 4)

	Exhaust		Supply	
Ventilation Section	Fan/ Speed	ft^3/ min	Fan/ Speed	ft^3/ min
S1	None		3/3	240,000
S2	None		3/3	180,000
S4	None		3/3	450,000
S5[a]	2/4	400,000	3/2	200,000
S6	1/1	60,000	1/1	50,000
S7	1/1	50,000	1/1	50,000
S8	1/1	75,000	1/1	75,000

[a]Location of fire.

Upstream was maintained smoke free. Smoke moved downstream within the upper cross section of the tube for approximately 450 ft, then a portion started to drop to the roadway. However, the air supply was sufficient to dilute the smoke, and adequate visibility was maintained for 250 ft. Further downstream, in Sections S6, S7, and S8, there was minimal smoke. Fire department vehicles drove to the fire location through the downstream sections without difficulty. (The fire department participated in the final testing.)

On the basis of the ventilation pattern created in Test 4, fan configurations for fires in all possible tunnel locations were developed, and operating personnel received detailed instructions for implementing such configurations in the event of an emergency.

The following points summarize the results of the fire tests.

- It was not possible to establish stratified flow of smoke in the upper tube cross section for an extended distance and also have a smoke-free condition upstream of the fire.
- Using the maximum ventilation capacity to dilute the smoke to maintain adequate visibility was not successful.
- Optimum conditions resulted from a ventilation reconfiguration that provided sufficient air velocity to prevent back-layering and, at the same time, create stratified smoke flow for a limited downstream distance. A certain portion of the smoke-laden air then dropped as a result of cooling and turbulence. However, the moderate quantity of air supplied diluted the smoke-laden air enough to provide 250 ft of adequate visibility.

NEW CEILING EXHAUST PORTS

A new ceiling will be placed in the tunnel tubes. This ceiling will, in effect, return the ventilation system to fully transverse. To improve the ability of the system to mitigate adverse conditions during a fire emergency, the exhaust ports will be as large (6 ft × 1 ft) as they can be and still maintain the structural adequacy of the precast concrete panels that form the new ceiling.

To permit balancing exhaust quantities, fusible damper plates, which would melt away under fire conditions, are being used. It is anticipated that greater smoke removal capacity in the vicinity of a fire will result.

Interstate 664 Submerged Tunnel Crossing of Hampton Roads–Newport News, Virginia

WILLIAM J. GADDIS

The Virginia Department of Highways and Transportation determined that by 1995 traffic crossing the Hampton Roads harbor area would be too great to be handled effectively by the James River Bridge and the Hampton Roads Bridge-Tunnel. A third crossing, Interstate 664, was planned to assume a major portion of the projected traffic load. The two portal islands, each reached by an approach bridge, provide tunnel access north and south of the Newport News navigation channel. Construction of the islands involved removal of the top layer of muck soil from the bay floor, backfilling with sand, and placing hydraulic sand fill within a stone dike system. Four-lane I-664 crosses the islands first at grade and then within an open approach structure until it reaches the multistory ventilation building. It then passes through a lower level of the building and enters a sunken tube tunnel that takes it under the navigation channel. The circular-steel-shell, concrete-fill construction concept was used for the double-bore tunnel 4,454 ft long face-to-face of the ventilation buildings. The structure is the fourth sunken tube highway tunnel in the United States with twin bores. Each bore contains a 26-ft-wide roadway for two lanes of traffic. The tunnel is placed in an excavated trench and then backfilled. The tunnel-island project is to be completed in 1990.

The Virginia Department of Highways and Transportation (VDH&T), on the basis of several studies and traffic projections, determined that by 1995 the volume of traffic crossing the Hampton Roads harbor in the Tidewater area of Virginia would be too great to be effectively handled by the current roadways. After a corridor location study (*1*) and an environmental impact statement (2) had been completed the location of Interstate 664 (I-664) was established in 1975. The total 20-mi route connects I-64 in the city of Hampton with I-64 in Suffolk. Included in the length is the Hampton Roads crossing that connects Newport News with Suffolk (Figure 1).

Because of both commercial and naval considerations, a tunnel was mandated to carry the highway under the Newport News navigation channel in Hampton Roads (Figure 1). The corridor study developed the general highway alignment and the approach requirements to reach the tunnel. These included a viaduct structure from the north (within Newport News) and a trestle for the remaining crossing of Hampton Roads. The study also documented the need for man-made portal islands at each of the tunnels. More important, it confirmed the need to use a submerged tube tunnel because of soil conditions at the site.

A Stage 1 (preliminary) study for the tunnel-island part of the project refined the project geometrics; established the type of sunken tube channel that would be developed for the construction contract documents; and set the basic design criteria for tunnel ventilation, electrical features, island configuration, and other principal design items. The tunnel layout permits a 55- by 1,000-ft channel to be dredged in lieu of the existing 45- by 800-ft channel. The layout and channel configurations are shown in Figure 2.

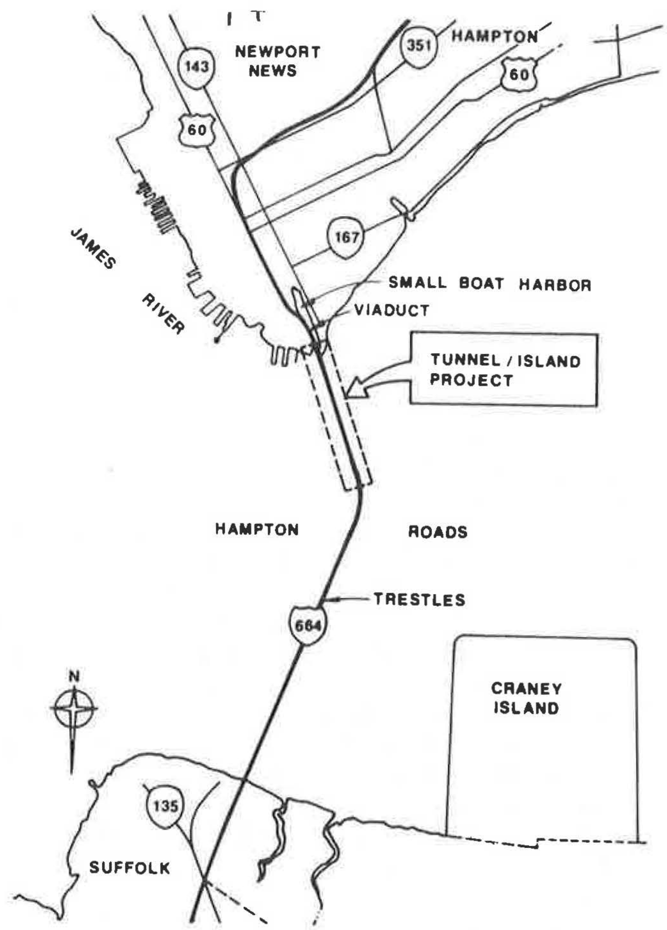

FIGURE 1 Project location plan.

Stage 2 (final) design of the tunnel-islands began in June 1979. Five construction packages were developed:

1. Islands;
2. Ventilation fans;
3. Tunnel (tubes, including finish items);
4. Buildings (and island surface completion); and
5. Electrical systems.

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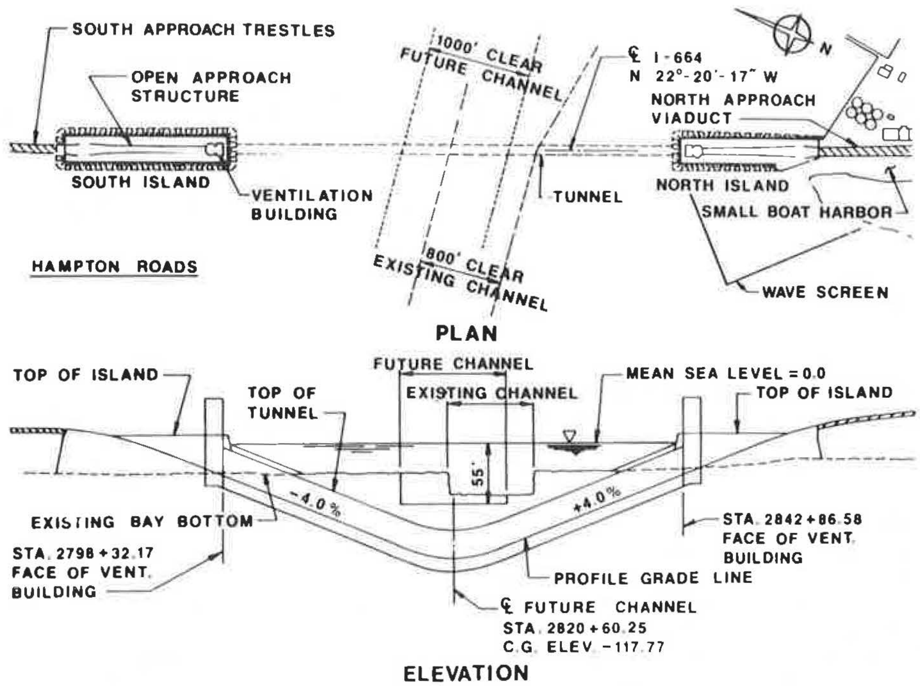


FIGURE 2 Plan and elevation.

The packaging was based on several factors, including previous experience of VDH&T and the designers with contractor interface problems, contract dollar amounts and contractor bonding requirements, and the uniqueness or specialty features of the various design and construction items.

As of this writing, all designs are complete except for the electrical systems package. For several reasons, the design project, which was essentially complete in 1982, was put on hold at that time. The electronic industry's state of the art has been experiencing considerable change. Thus it was decided that the completion of the electrical systems should be accomplished at a time closer to construction to allow inclusion of the most up-to-date systems.

In July 1985 the island construction contract was advertised. In September 1985 Tyger/Pensacola, a joint venture, started work on the island construction that was to have been complete by July 1987. The tunnel contract was awarded to Morrison-Knudsen/Interbeton in September 1986. The ventilation fan contract was to have been advertised by February 1987 and the buildings contract in January 1988.

PERMITS

As described later, the island and tunnel contracts involve a considerable amount of dredging and the placement of large quantities of hydraulic fill. The disposal of unsuitable (for hydraulic fill purposes) material was taken care of by gaining Corps of Engineers approval to use the Craney Island disposal site (Figure 1).

Borrow for hydraulic fill has come from two sources, "upland" sites and dredging the Thimble Shoal Channel just east of the Chesapeake Bay Bridge-Tunnel (about 20 mi east of the I-664 site).

LEVELS

An extensive geotechnical study was performed during the various project stages. Subsurface investigation, used for both the island and the tunnel designs, revealed that settlement and slope stability were the major design concerns for the man-made islands. Basically, the islands are built of hydraulic granular fill confined within and protected by stone dikes. The stability of these structures is essential because they support not only the island-based roadway and building facilities but also the end tubes of the tunnel.

The subsurface investigation included soil borings, a geophysical survey that used seismic reflection and seismic refraction measurements, and laboratory testing of Shelby tube and Osterberg piston samples. The results indicated that there is a layer of muck soil of variable depth on the bay bottom underlain by various layers of sand and silty clay, silty clay and clayey silt, and sand and silty sand. Laboratory consolidation tests, Atterberg limits, void ratios, and natural moisture contents were used to evaluate the compressibility of the substrata.

As a result of the conclusions of the study, complete removal of the muck soils by dredging was specified. This was necessary to reduce settlement, enhance stability, and reduce construction time. The sand and silty sand substrata are not cause for concern. However, the layers containing clay are more compressible and will require time for the settlements to stabilize. Settlements of as much as 3 ft are anticipated. To speed settlement, a 12-ft island surcharge was specified. This reduced the time between completion of the islands and placement of the first tube by about 10 months.

The island geometrics, armor protection, and a seawall around the island perimeter are based on design criteria that used the hindcasting method to predict wave heights. Thus the islands and other facilities are protected from wave forces similar to those of past hurricanes and other storms that have struck the site. About 1.6 million cubic yards of fill and 840,000 tons of armor stone are required for the construction of the two islands (Figure 3).

TUNNEL

As stated earlier, the subsurface conditions at the site required the use of a sunken tube tunnel. There are several different names for this type of tunnel construction: immersed tube, submerged tube, sunken tube, and trench. Regardless of the name, they all are based on the same concept. A trench is dredged to a specific geometric shape, a foundation course or system is placed in the trench, and prefabricated tube segments are floated to the trench and lowered onto the foundation. The tube sections have bulkheads at each end. As the segments are mated, the bulkheads are removed to allow access between the in-place segments. Finishing and other operations proceed in these units while the remaining segments are being placed.

Sunken tube tunnels have been used in the United States for 90 years. A good general description of the various alternates and methods of constructing them can be found in *Civil Engineering Practice* (3).

The I-664 tunnel is only the fourth sunken tube highway tunnel in the United States with twin bores. The last such tunnel constructed was the recently completed Ft. McHenry Tunnel in Baltimore. That tunnel included a pair of twin-bore tunnel sections. The design of that tunnel and of the I-664 tunnel proceeded almost simultaneously.

The width of the double-bore tunnels makes the proper mating of the adjoining segments difficult. Things learned during construction on the Ft. McHenry Tunnel have been incorporated in the I-664 contract documents.

VDH&T uses the high sidewalk system in their tunnels. This concept was also used in the Chesapeake Bay tunnels and elsewhere. The premise behind the high sidewalk is that it allows patrolmen to be more visible to motorists and to see vehicles more easily. The high sidewalk does, however, require that tube geometrics for circular-bore tunnels be somewhat larger. This results in a slightly higher cost because buoyancy is a principal design feature for this type of construction.

In Stage 1 various types of tunnel cross sections were studied and compared. Four groups were developed, each representing a particular method of constructing prefabricated tunnel tubes. Group I included the single-steel-shell type of tube structure, (Figure 4). A steel shell, supported by transverse and longitudinal stiffeners, forms the outer surface of a concrete structure. The shell waterproofs the concrete and, in addition, provides exterior concrete reinforcement. A coal-tar epoxy coating protects the outer steel surface. Steel trusses are placed within the interior wall sections to help resist forces from launching and concreting. Normally, the steel shell is built and launched from shipbuilding ways. After it is launched, the segment is towed to an outfitting site where concrete is placed in stages through hatches in the top of the steel plate. As the concrete is placed, the tube sinks until it has only a small freeboard when the interior concreting is completed. The segment is then floated to the site and sunk by placing ballast in bins attached to the top of the tube.

The Group II concepts included the circular-steel-shell type of construction, (Figure 4). This is the most common type of design in the United States. Tunnels in this group consist of two steel tubes supported by longitudinal stiffeners and connected at intervals by steel diaphragms. Concrete placed inside the steel shell forms the duck and roadway cavities and also provides structural strength in the in-service, backfilled position. Concrete is placed outside the circular shell plate to ballast the structure. The circular tube shape is inherently suited to supporting forces from launching and concreting. Except for the method of placing ballast, this group of tunnels is essentially the same as Group I: both are built on ways,

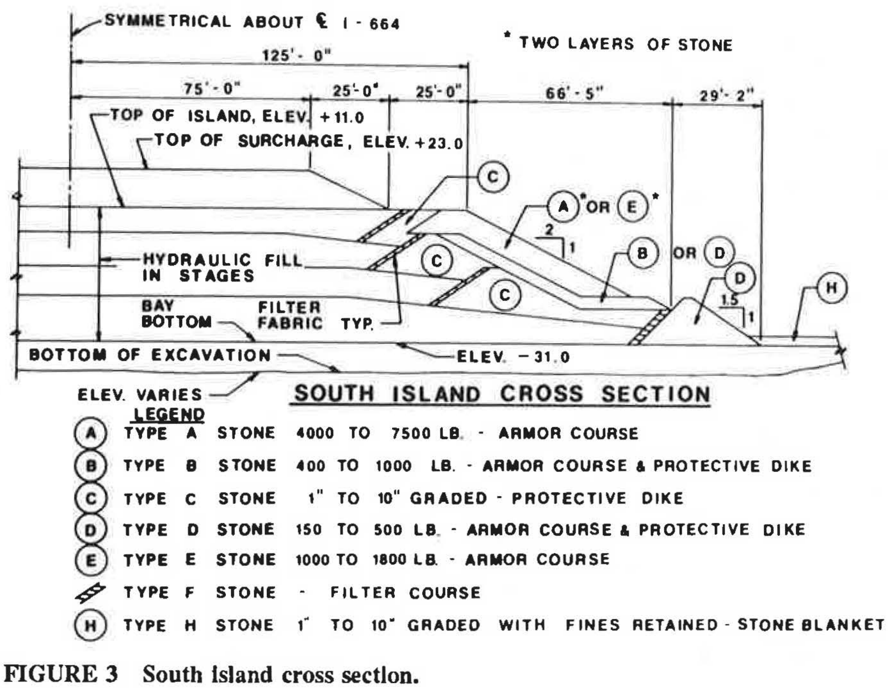
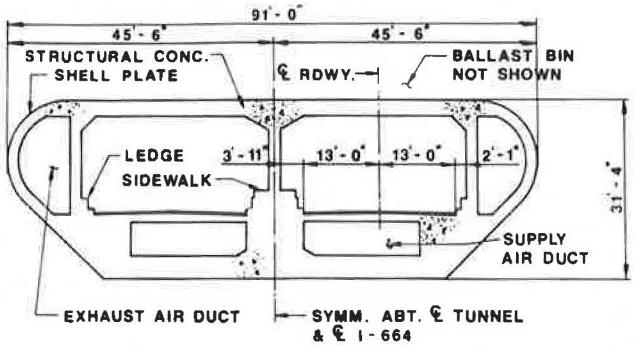
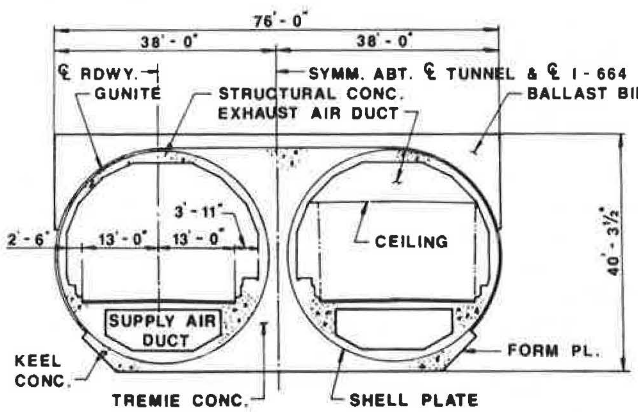


FIGURE 3 South island cross section.

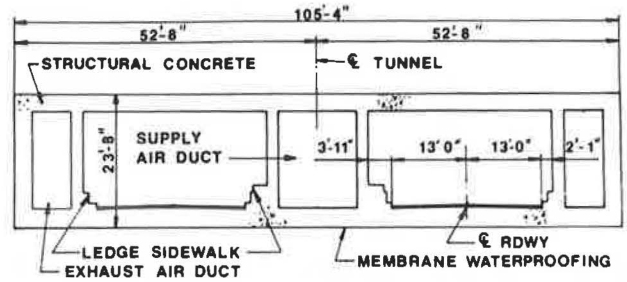


GROUP I, CONCEPT 1

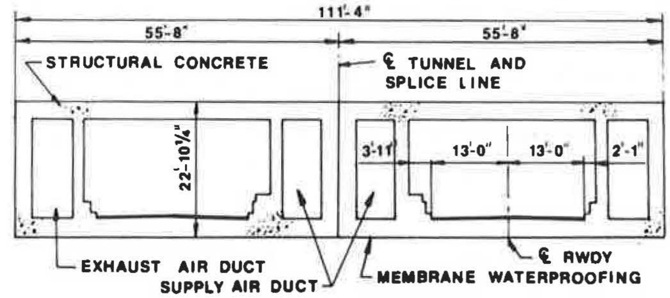


GROUP II, CONCEPT 5

FIGURE 4 Groups I and II cross sections.



GROUP III, CONCEPT 7



GROUP IV, CONCEPT 9

FIGURE 5 Groups III and IV cross sections.

launched, and towed to the site. Because many tunnels have been built using this system, contractors are familiar with general fabrication and placement methods.

Group III comprised concepts that use a casting basin for tunnel segment fabrication. An area large enough to accommodate at least one-half of the required tunnel segments is selected along a shoreline to serve as the casting basin. The size requirement is an economic consideration. The area also has to be deep enough to float the completed segments. The area is encompassed by a cofferdam and dewatered. Cut-off walls and a well-point system are installed. Then the reinforced concrete tunnel segments are fabricated in the basin (Figure 5). Posttensioning is often used to improve watertightness of the segments. A waterproofing membrane is placed on the exterior concrete faces. When the segments are completed, the casting basin is flooded and the segments are floated to the site. The segments are lowered into the trench by placing gravel in ballast pockets attached to the top of the segments.

There are several advantages to this type of construction, but a major disadvantage is the casting basin. A large area (15 to 30 acres was considered for I-664) is required. It was anticipated that the most convenient sites (for I-664) would have been in wetlands. The activities at the basin were thought likely to cause environmental changes. Because other construction methods were available, the use of a Group III proposal would have been considered only if economics indicated that environmental mitigation efforts should be pursued.

The Group IV concepts were similar to those of Group III. However, this group would have used one of the islands as the casting basin to eliminate the environmental problem just discussed. As it turned out, construction on the island itself became quite complex. Furthermore, because of the size of the islands and ventilation buildings, the width of the tunnel segment was limited. Thus, for some concepts, the section would have had to have been built in two parts (Figure 5) and then joined after floating. Also, only one segment at a time could be constructed. Last, the completion of the ventilation building would have had to wait until the last segment was completed. This would have extended the overall construction schedule.

Consideration was given to the many advantages and disadvantages of each group of concepts. The determining factor in selecting a Group II concept, a circular-steel-shell section, was cost. This concept was almost 10 percent less expensive than those in any of the other groups.

This concept was then developed into final design and the contract documents were completed. Some of the tunnel design criteria follow.

1. Cover: Ten-foot minimum within the shipping channel (protection against anchor drag). Beyond the channel, the minimum cover is 5 ft.
2. Overdredging: There is a 3-ft allowance.
3. Tube placement tolerances are as follows:

	Inboard End	*Outboard End*
Horizontal alignment	±½ in.	±1 in.
Vertical alignment	±½ in.	±1½ in.
Maximum differential of vertical measurement tilt	±½ in.	±½ in.

4. Water elevation: Mean sea level equals Elevation 0.0. Maximum storm conditions, extreme high at Elevation +8, extreme low at Elevation –4.

5. Buoyancy: Sinking ballast factor of safety to be at least 1.10; after joint dewatering and bulkhead removal, factor of safety to be at least 1.02; completed tube (in-service) factor of safety to be at least 1.25.

6. Roadway vertical clearance: 16 ft 6 in. minimum.

7. Launching and outfitting stresses shall not exceed 133 percent of the normal unit stresses in the AASHTO code. Tubes shall be investigated for side- and end-launching.

There are several features of the design that bear review.

Tunnel Configuration and Alignment and Project Details

The alignment and cross section as presented in the contract plans are shown in Figures 2 and 6. The cross section includes twin bores, each with a 26-ft-wide roadway, a high sidewalk, a ledge, an exhaust air duct above the roadway, and a supply air duct below. Exhaust air ports are located in the ceiling; supply air flues leave the supply duct and open into the roadway cavity in the sidewalk and ledge faces.

The walls of the roadway cavity will be covered with special ceramic tile. Originally, and as presented for bids, the ceiling was a series of suspended porcelain-enamel, concrete-filled panels. The roadway lighting was to have been fluorescent fixtures hung on the tunnel walls as shown in Figure 6. However, as discussed elsewhere in this paper, recent decisions about the construction and operational cost of fluorescent lights versus other lighting methods have caused a restudy of lighting types and their location. If a different lighting system is used, the lights will be ceiling mounted. Thus the new study will also include a reevaluation of the ceiling system.

There are 15 tube segments, each almost 298 ft long. The horizontal length of the tunnel is 4,454 ft measured face-to-face of the ventilation buildings. The overall length of the tunnel, portal-to-portal, is 4,782 ft. The tunnel segments will include more than 18,000 tons of structural steel.

Support and Placement of Tunnel Tubes

The geotechnical investigation indicated that no special precautionary measures were needed to support the tunnel segments. Thus two common methods of support were developed and were shown on the contract documents as contractor bid options. One was a screeded-gravel foundation. A 2-ft layer of gravel is placed in the bottom of the trench by clam shell or tremie tube to prevent segregation of the material. The gravel is then screeded to the proper elevation by a heavy beam, template, or drag. The second method presented was a pumped-sand foundation. In this method, the tubes are placed in the trench on a temporary support at each corner of the segment. For this project, a prefabricated concrete pad was used. Jacks were incorporated into the system to make alignment adjustments. A sand slurry would then be pumped under the tubes, and the tube load would be transferred to the sand foundation by gradually releasing the jacks. Different methods of placing the sand slurry have been used on other projects. No particular method was specified for the I-664 project. (Morrison-Knudsen/Interbeton chose the screeded-gravel option.)

Typical Tube Joints

The contract documents call for a rubber gasket sealing system at typical tube joints. This system requires the gaskets to be compressed to properly seal the joint. This is achieved by first having divers align the just-lowered segment with the adjoining elements using a series of jacks. Final compression is then

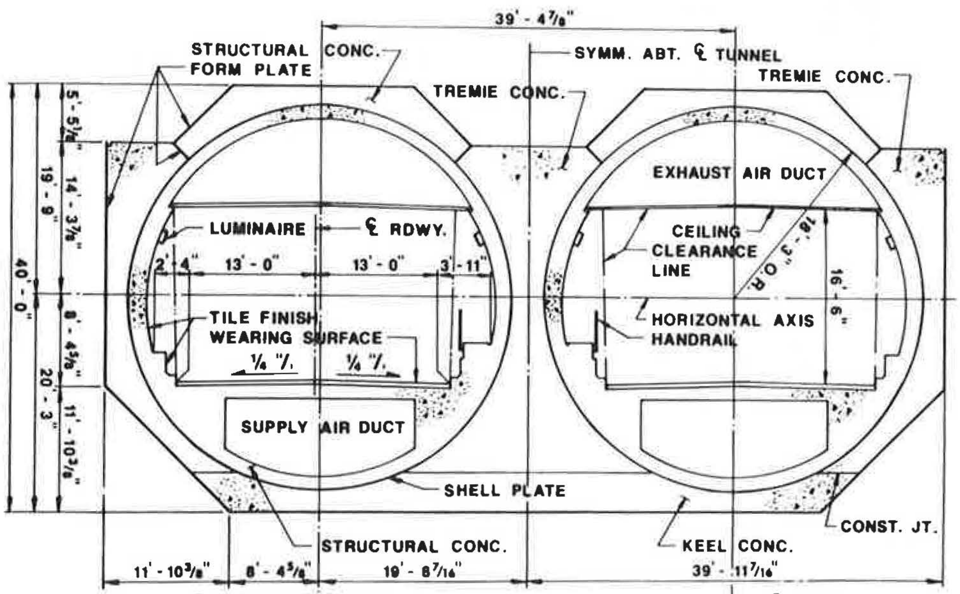


FIGURE 6 Typical cross section.

achieved by dewatering the joint area. This activates a hydrostatic force on the free tube bulkhead. The resulting pressure moves the free tube and compresses the gasket. The jacks help hold the alignment during the process.

Tube Closure and Tube-to-Ventilation Building Joint

The first three tubes adjoining each island will be placed before the remaining segments. This will allow work to proceed simultaneously on both the tunnel and the building contracts. The ventilation building foundation will be constructed within a cofferdam. The end tube details at the ventilation building joint were developed to allow the building contractor to "tie in" to the tube. This permits him to complete his cofferdam, dewater, and construct the building-tube joint in the dry. The joint details allow differential settlement between the massive building structure and the tube. To prevent movement of the first three tube segments placed, they will be backfilled before the cofferdam is dewatered.

Placing tubes from both ends results in the last tube being placed between two earlier tubes. Obviously, the compressed gasket joint system used at the typical joints can be achieved at only one end in this situation. The tremie-concrete collar joint method will be used on the last or closure joint. This joint system was used on many older tunnels that were built before the rubber gasket system was developed.

TUNNEL VENTILATION

The tunnel ventilation system will be fully transverse (i.e., it will include both supply and exhaust air fans). A ventilation building housing fans, fan drives, louvers, exhaust vents, and other equipment is located at each end of the tunnel. Each tunnel bore will have two ventilation systems, each served from the ventilation building nearest the section (Figure 7). Each ventilation system will have a supply and an exhaust duct system. The facility will have a total of 24 fans (12 exhaust and 12 supply).

The fans will be centrifugal, double inlet, and double width with bearings mounted on separate outside pedestals. The bearings are specified to be of the spherical roller type. The fans will be driven by a three-speed drive made up of two motors and two HTD belt drives. Each fan will be connected to a single-speed motor through the first drive. This single-speed motor will have shaft extensions on both ends. The shaft connection not connected to the first drive will be connected through the second drive to a smaller two-speed motor.

The "ducted fan" concept (i.e., all three fans for a ventilation system will be located in one fan room) will be used. For proper operation, metal duct work will connect the fan inlets with one of the building's concrete exhaust shafts that, in turn, connects to the tube exhaust duct. The metal duct becomes a partition that isolates the fan from the rest of the fan room. Vitiated air is exhausted by the fan through a metal vent connected to the building ceiling. A turret or nozzle on the roof dissipates the air well above the operations part of the building and avoids contaminating the fresh air intakes.

The three supply fans per section are located in a chamber that has louvers in the exterior building walls. Outside air will be pulled into the room. The louvers will not be weatherproof. Rainwater will enter during high-speed fan operations and extreme weather conditions. All fan room chambers, exhaust and supply, have floor drains to clear rainwater from the chamber.

The fan motors will be equipped with antifriction bearings. The synchronous speed ratings are 1,200 rpm maximum for the large motor and 1,800 rpm maximum for the small motor. The fans can be started from rest on any of the three speeds, and each motor is designed to be started a maximum of four times per hour. By varying the number of fans operating at a time (one, two, or three) and the motor speed (low, medium, or

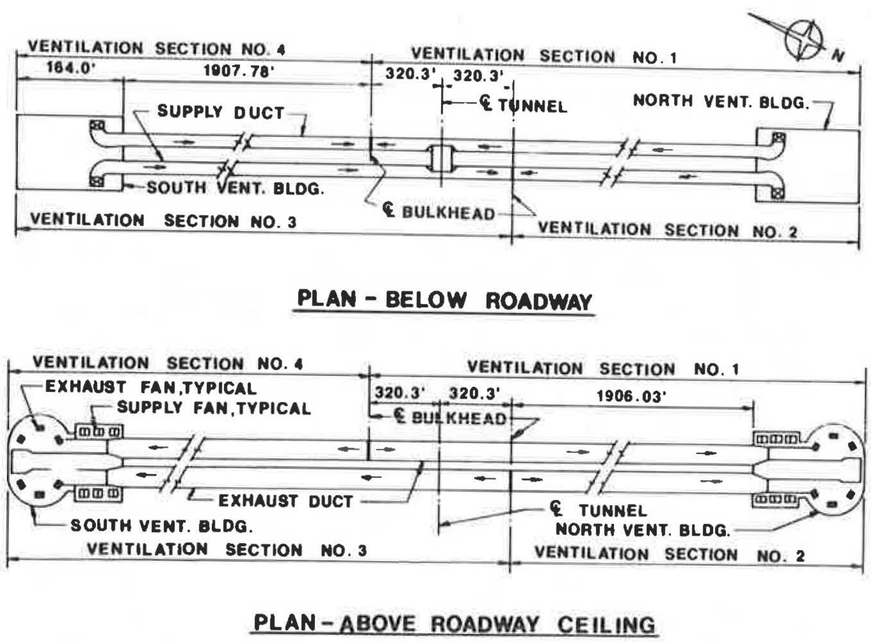


FIGURE 7 Tunnel ventilation systems.

high), a wide range of air flow can be provided. This allows efficient and functional operations. Each fan has a damper. The purpose of the damper is to isolate a nonoperating fan from the remainder of the system, thereby eliminating a backdraft through the nonoperating fan and permitting the operating fans to function properly.

Other details that complete the tunnel ventilation system are items such as stainless steel turning vanes, shotcrete-formed curved shafts, and blank-off panels that permit the supply and exhaust air flow to be adjusted.

Under normal operations, the east tunnel bore carries northbound traffic and the west bore carries southbound traffic. The ventilation system has been designed for this mode of operation. It also has been designed for two-way traffic in either bore. The total facility has been designed to accommodate a closure of either bore as a result of extreme maintenance requirements or a catastrophic situation such as a trestle closure. The parameters of the ventilation system follow.

1. Ventilation rates: under design conditions, carbon monoxide (CO) concentration rates are not to exceed 125 parts per million.
2. Design conditions:
   a. The worst possible condition of 10-mph traffic with a traffic flow of 1,500 vehicles per hour per lane.
   b. A traffic mix of 91 percent passenger cars and 9 percent trucks.
   c. Anticipated CO output from vehicles as measured by the Environmental Protection Agency's 1976 tests.
   d. Tunnel roadway grades (4.0 percent maximum).
   e. The higher ventilation amount of the two operating conditions:
      (1) Two lanes of one-way traffic or
      (2) Two lanes of two-way traffic.
   f. Design minimum ventilation rate of 100 ft^3/min per lane foot.
   g. All three fans within a ventilation system shall operate for 100 percent ventilation requirements.

BUILDINGS AND OPEN APPROACHES

The ventilation buildings were primarily developed to be functional. They contain the tunnel ventilation fans and the primary facility mechanical and electrical equipment. In addition, the north ventilation building houses the facility control center, communication room, and main facility offices.

The design of the building also sought to keep the building compact. Although this is probably the basic consideration in the design of most conventional buildings, it is even more important in this case. First, the building area required can affect the size of the island because there are specific requirements for features such as truck turning movements at the tunnel end of the building, parking areas, and sight visibility and traffic movements between the end of the open approach and the end of the island. Second, buoyancy is a design consideration. The more compact the building area is below sea level, the less is the effect of buoyancy on design and construction. Last, the below-grade building size determines the magnitude of island excavation and the cofferdam and dewatering system that is required to build the below-grade part of the structure.

In the I-664 building design the functional requirements were achieved by using procedures that maintained compactness and, in addition, presented a structure with distinctive architectural treatments. One unique detail that was used to minimize the below-grade foundation area was the cantilevering of the second and higher floors. This permitted automobile parking under the cantilevered floors. This, in turn, helped keep the size of the island down. The use of a brown-tone, rough-textured surface on all exposed concrete allowed the I-664 project to avoid the stark appearance of some other facilities' ventilation buildings. The same concrete treatment was used on the open approach structures. The premium for this aesthetic treatment is minuscule in comparison with the overall cost of the project.

The lowest level of the building contains the portal pump rooms. These rooms house the mechanical equipment and ancillary systems that collect drainage from the open approach roadways and from the tunnel low-point pump room. The portal pump system then sends the runoff overboard into the bay via an underground drainage system that terminates in an outfall headwall located within the island armor layer.

The second lowest level in the ventilation building is the roadway level that serves as part of the tunnel and connects the tubes with the open approach structure.

The open approach structure has walls 38 ft high and footings as much as 11 ft thick. This thickness is required to resist buoyancy because the roadway elevation at the building interface is almost 23 ft below sea level. A safety barrier separates the two-way traffic within the approach structure.

The problem of the "black hole" effect of tunnel portals (on a bright day) was particularly important on this project. The north portal will appear as a black hole to southbound drivers because the midday sun will, during all seasons, be to the south and in their eyes. Sun screens have proven to be of minimal benefit in this situation. A flaring transition, in the form of a sweeping curve, will be used from the building wall to the tunnel ceiling to diffuse the bright natural exterior light into the less bright artificial light of the tunnel (Figure 8). The colored, textured concrete finish will also lessen the black hole effect. Tunnel lighting will follow transition zone procedures normally used to reduce the impact of changes in light. Also, throughout the lighting transition zone, the roadway slab will be portland cement concrete instead of the normal asphaltic concrete used elsewhere in the tunnel and on the open approaches. The lighter concrete color will help reduce the outside-inside lighting contrast—at least for a while.

FIGURE 8 Portal treatment.

ELECTRICAL SYSTEM

Two 13,800-volt feeders, each capable of handling the entire bridge-tunnel facility, will be brought from switching stations in Newport News. The feeders will be connected to two independent power sources supplied by the local power company. The feeders will lead to switchgear units in the north ventilation building's switchgear room. From there power will be distributed to the various electrical and mechanical systems of the facility. The reason for having two independent power sources is obvious. Power outages for a facility such as I-664 can cause real problems. Although emergency generators can be installed (as they will be at I-664) to provide certain tunnel-oriented essential services, it is not feasible to provide units sized to drive all facility systems necessary for normal operations, such as the computer-controlled traffic control system, all tunnel lighting (about one of three lights on the sidewalk side of each bore is powered by the generator), television surveillance system, and other facility power and lighting systems. Lengthy power outages have caused other facilities to develop unique operating procedures in an attempt to maintain user safety.

As mentioned earlier, the original contract documents specified the use of what has been a conventional tunnel lighting system (i.e., beyond the transition zone, two continuous rows of fluorescent luminaires, one located on each side wall). The economic benefits, both for construction and operations, presented by sodium lighting units have led various agencies, including VDH&T, to consider the use of these systems in tunnels. Recently, the Washington State Department of Transportation, in cooperation with the FHWA, let a contract that uses high-pressure sodium luminaires installed in the tunnel ceiling. This was done after the system was tested in an existing tunnel. On the basis of economic projections for the Washington project, and estimates for I-664, VDH&T has authorized a study that will compare fluorescent and high- and low-pressure sodium lighting systems.

MECHANICAL SYSTEM

Most of the mechanical design elements for the I-664 project are routine for tunnel facilities. Special consideration was given to the open approach and tunnel drainage. There have been some problems in other facilities in keeping the tunnel drainage system clean. The I-664 tunnel design tried to alleviate the problems by providing drains that were easier to clean. Time will tell if these efforts are successful.

Water supply for the facility will be achieved by bringing a 10-in. line from a meter vault in Newport News to the north island. It will be underwater within the small-boat harbor. This avoids the freeze problem that would be presented if the line were attached to the superstructure of the approach viaduct.

The water supply line will provide domestic water to the complete facility. It will also provide fire protection water to hydrants on each island and to hydrants within each tunnel bore. The tunnel hydrants will be located in niches in the sidewalk-side tunnel wall and will be spaced about 300 ft apart. A recirculation line will be provided at each hydrant to prevent freezing.

Fire protection is divided into two basic areas, ventilation buildings and tunnel. The fuel loading in the buildings is minimal. Therefore the buildings will have only fire extinguishers as required by code. A halon sprinkler system will be provided in the central control center to protect the computer and other electronic systems.

Several fire-fighting procedures will be used in the tunnel. These include use of the exhaust fans to remove smoke and heat, stopping supply air fans when appropriate, use of portable fire extinguishers stored in niches in the tunnel, island-stationed emergency vehicles that contain appropriate fire-fighting equipment, and response of the Newport News Fire Department to all vehicle and structural fires. A standpipe, which will allow the fire department to pump sea water from the harbor if the domestic water supply is insufficient, is located on each island.

ACKNOWLEDGMENTS

The tunnel and island portion of the I-664 crossing of Hampton Roads has been discussed. Obviously, the complete facility will extend well beyond these limits. The final design of the associated construction packages that interfaced with the tunnel was performed by VDH&T or other consultants. Their efforts and cooperation are acknowledged. The project was and will continue to be a challenge. When completed, nearly 20 years after the initial VDH&T efforts, it should serve well the citizens of and the visitors to the Tidewater area.

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Design and Construction of Interstate 10 Drainage Tunnels in Phoenix

TIMOTHY P. SMIRNOFF

Drainage of the depressed Interstate 10 inner-loop extension in Phoenix, Arizona, required the construction of a conveyance system composed of pressure tunnels, 14 to 21 ft in diameter, beneath the city. The geologic and engineering parameters specific to this project and the method of construction used in stream-transported alluvial soils are described. Minimization of surface settlement and concern for lowering of the groundwater table, which is almost completely controlled by the nearby Salt River, influenced construction methods and contractual provisions.

Interstate 10, a major link between East and West, runs from Florida to California, passing through the southern tier of states. I-10 is complete except for a small portion in the Phoenix area. This last link of I-10 will pass through the city to form an inner loop through the downtown business district. Conceived as a depressed roadway, the sunken section dividing the watershed into two regions would intercept the surface drainage of the Phoenix basin. Several times a year the Phoenix area is prone to flash flooding. The runoff from these storms moves from the mountains in the North to the Salt River. To prevent inundation of the depressed roadway and flooding upstream, runoff storm water collected from the northern half of the drainage area will be conveyed in a series of pressure tunnels (siphons) to the Salt River. The tunnel system is shown in Figure 1, and the prominent features of each of the tunnels are given in Table 1.

The geologic features unique to the Phoenix area and the design and construction of the tunnels are described. Details of construction methods, equipment, and surface effects of tunneling are also included.

GEOLOGY

The project area, in the desert region of the Basin and Range Physiographic Province, is characterized by fault block mountains with intervening basins filled with deep deposits of unconsolidated sediments shed from the mountains. The topography is nearly flat and slopes at less than 1 percent to the southwest toward the Salt River. Most flows in the Salt River are controlled by a series of upstream dams operated by the Salt River Project. Release from these dams, combined with the natural drainage from below the dams, has reached peak flows of as much as 170,000 ft^3/sec in past years. However, the riverbeds are normally dry.

Tunnel and Underground Engineering, Howard Needles Tammen & Bergendoff, 9200 Ward Parkway, Kansas City, Mo. 64114.

The typical soil profile in the Phoenix Basin consists of stream-transported, alluvial, coarse-grained, poorly graded, "river run" sediments overlain by fine-grained sediments deposited in areas isolated from the main stream channels. In the Phoenix project area, the fine-grained sediments are mostly sandy clays of depths ranging from 10 to 30 ft. These sediments are underlain by sand, gravel, and cobble deposits of considerable depth. Groundwater readings obtained from borings in the project area indicate that the water table is at depths ranging from 60 to 95 ft below the ground surface.

The overlying sandy clay material can be described as light brown to brown in color and of medium plasticity (classified as CL material according to the Unified Soil Classification System). The deposit has light to moderate calcite cementation, which generally increases with depth. Dry density (γ_d) of the deposit ranges from 90 to 115 lb/ft^3, with an average value of 105 lb/ft^3. Moisture contents of the soil are generally below the plastic limit and range from 6 to 20 percent, with an average value of 12 percent. In general the sandy clay has an internal angle of friction (ϕ), ranging from 22 to 48 degrees, with an average value of 30 degrees. Similarly, the shear strength of the material is good; the cohesion intercept ranges from 0.25 to 0.9 ton/ft^2, with an average value of 0.4 ton/ft^2. It should be noted that the shear strength of the material was tested in direct shear after samples had been saturated, consolidated, and then sheared under undrained conditions.

Located below the fine-grained sandy clays is a thick deposit of sand, gravel, and cobbles (SGC). This SGC deposit is grayish brown in color and roughly stratified. Grain sizes of the deposit range from trace fines to boulders as large as 18 in. in diameter. Most of the materials that make up the SGC deposit are subrounded sands and gravels, classified as GP material according to the Unified Soil Classification System. The SGC deposit has a high relative density with a dry density (γ_d) of approximately 140 lb/ft^3. The internal angle of friction (ϕ), of the uncemented material is 40 degrees and perhaps as high as 50 degrees for the cemented material. Predominant rock types in the SGC deposit include sandstone, quartzite, and granite. A generalized soil profile with soil description and representative soil parameters is shown in Figure 2.

GEOTECHNICAL INVESTIGATIONS

A limited conventional subsurface exploration and testing program was undertaken to provide information on subsoil and groundwater along the project corridors and to verify

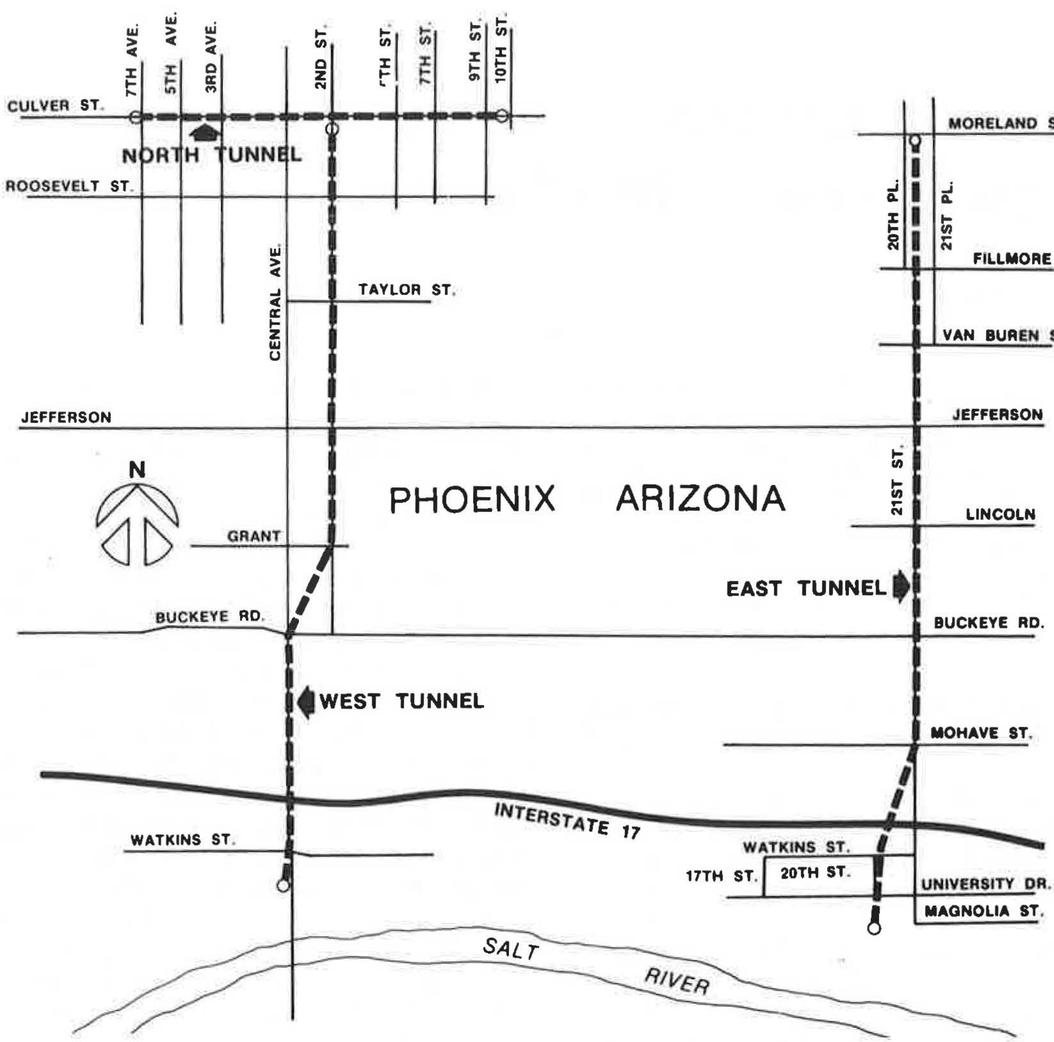

FIGURE 1 Site location.

TABLE 1 FEATURES OF I-10, PAPAGO FREEWAY, DRAIN TUNNEL SYSTEM

Tunnel and Identification	Finished Diameter (ft)	Total Length (ft)	Excavated Diameter	Total Excavated Material (yd^3)	Total No. of Segments	Finish Concrete (yd^3)	Design Flow (ft^3/sec)
North tunnel I-ID-10-3(187)	14	6,554	17 ft	56,000	6,655	11,000	2,200
East tunnel I-ID-10-3(188)	21	13,542	24 ft 10 in.	243,000	13,542	31,000	5,000
West tunnel I-ID-10-3(189)	21	13,680	24 ft 10 in.	245,000	13,680	32,000	5,000

observations and data uncovered by a literature search. Small-diameter borings were drilled to depths of 100± ft below existing grade; borings were drilled with a Becker hammer drill and advanced with the ODEX (overburden drilling with the eccentric method system). Field permeability tests were performed in selected borings at three locations.

During the prebid phase of this project, prospective bidders had the opportunity to inspect the ground within the project area from four large-diameter observation holes. These holes were drilled using a 36-in.-diameter helical auger to a depth of approximately 60 ft. A 36-in.-diameter steel casing with observation ports placed approximately every 5 ft below a depth of 20 ft was installed to provide safe entry. More than 40 prospective bidders and consultants were lowered into each of these holes (Figure 3).

EXPECTED GROUND BEHAVIOR

The behavior of the ground at tunnel level was expected to vary primarily as a function of the amount and extent of cementation encountered within the SGC deposits in which all of the tunneling was to occur. It was anticipated that firm, raveling to running ground conditions would occur at the tunneled face. These conditions might occur singly or in combination within the length of a single advance because of the erratic and variable nature of the river-deposited SGC materials.

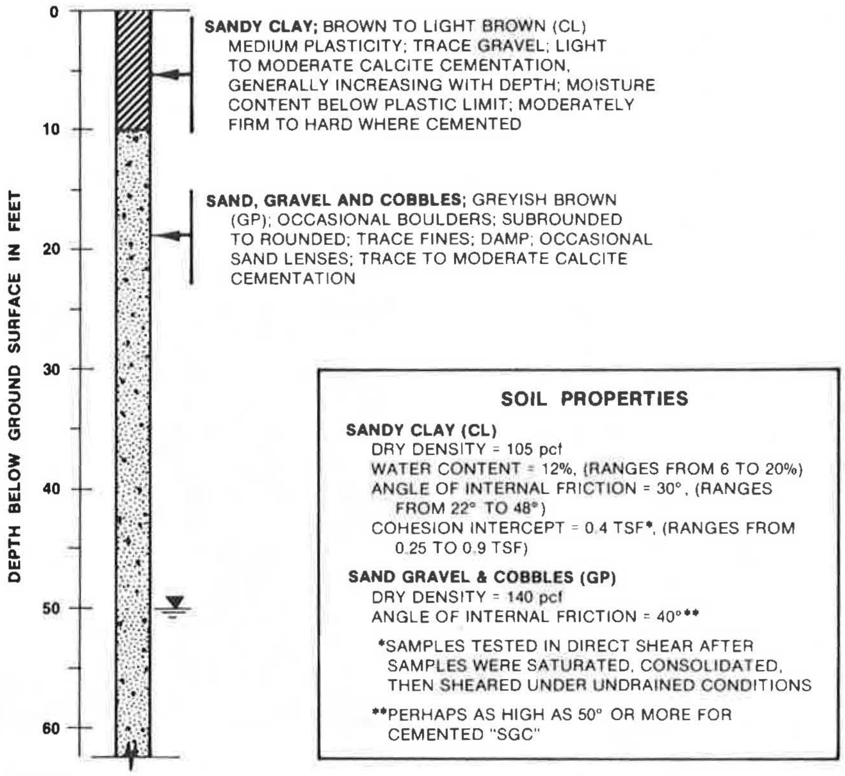

FIGURE 2 Generalized profile and material properties.

FIGURE 3 Inspection or large-diameter exploratory boring.

The SGC, when uncemented and free of apparent moisture, would exhibit behavior of almost cohesionless granular soil through which tunneling could be carried out only with complete protection of the face tops and sides of the excavation. Where the material was dense and cemented, tunneling was expected to be performed with little difficulty and with negligibly small settlements, except where raveling or runs at the face occurred. Without adequate face support, such runs may occur with attendant large and irregular loss of ground. The consequence may be runs that completely invade the heading, leading to the formation of a sinkhole at the surface of the ground.

CONTRACT PROVISIONS AND DESIGN

The potential variability in ground stability and the necessity of restricting settlements throughout the project area to minimize tunneling effects on overlying utilities and surrounding foundations dictated that the contractor be allowed maximum flexibility in choosing techniques and methods. At the same time, sufficient performance criteria were necessary for the owner to adequately control the work. To provide this control and flexibility, the contract specifications were written as broadly as possible. Although all specifications contain some prescriptive language, these specifications also contained performance or minimum design criteria. All contractors were required to prequalify, both financially and technically, before building.

Contractor provisions included

- Required use of a shield and tunnel-boring machine;
- Alternative design of initial support systems subject to minimum design criteria: (a) sustain full overburden but no less than 6,000 lb/ft^2, (b) sustain a diametrical distortion of 0.5 percent, (c) include provision for all loads including construction-induced load, and (d) maximum rib spacing or segment length of 5 ft 0 in.;
- Contractor responsible for maintenance and protection of existing overlying utilities and foundations;
- Contractor responsible for settlements in excess of 1$^1/_2$ in.;

- Force account for roadway and sidewalk repair (owner assumed responsibility for damage to residential structures);
- Alternative designs for final lining subject to the minimum criteria;
- Liquidated damage of $4,000/per day;
- Lowering the groundwater table to obtain workable conditions;
- Detailed preconstruction survey of area; and
- Escrow bid documents.

The interrelationship among excavation equipment, sequence of construction, and personnel safety dictated that the contractor be responsible for the design of the initial support system and tunnel shield. The contract required that the tunneling shield have a well-proportioned hood and be equipped with poling plates, breast tables, boards, or similar mechanisms to ensure positive control of the face of the excavation at all times and minimize surface disturbance and related soil movements.

The contract documents contained two alternative initial support systems, a precast concrete segmental lining and structural steel rib and wood lagging. After the initial support system was installed, a final cast-in-place lining was to be installed.

CONSTRUCTION PROGRESS AND OBSERVED CONDITION

The three tunnels were bid as a single contract package to enable common use of tunnel excavator, forms, and other construction materials. The tunnel project was first advertised in February 1984. Bids were received on May 10, 1984. A tabulation of bids is given in Table 2.

TABLE 2 BID RESULTS

Amount ($)	Contractor
49,633,450.00	Shank-Artukovich-Ohbayashi-Gumi
51,827,900.00	J. F. Shea Co., Inc.
55,197,920.00	Schiavone Construction Co.
55,852,401.00	Kiewit Construction Company and Kiewit-Grow, a joint venture
57,347,170.00	Phillips and Jordan, Inc., Dick Corporation, and Paschen Contractors, Inc., a joint venture
61,077,000.00	S & M-Mancini-Greenfield, a joint venture
63,441,932.00	Loram-Mergentime, a joint venture
64,291,389.00	Harrison Western Corporation
64,587,933.07	S. A. Healy Company
64,895,160.00	Traylor Bros., Inc.
65,556,110.00	Dillingham Construction, Inc., Ball, Ball & Brosamer, Inc., and John Mowlem & Company, PIC, a joint venture

The low bidder, Shank-Artukovich-Ohbayashi-Gumi, joint venture, was awarded the contract on June 1, 1984, and began sinking the construction shaft for the east tunnel in September 1984. In November 1984, three large-diameter wells were installed about the shaft to control local groundwater. An additional eight wells were installed along the alignment. Drawdowns of as much as 95 ft were observed. In general, groundwater was not a problem during tunnel excavation. Tunnels were excavated using a semimechanized shielded excavator, equipped with radial orange-peel doors and a bucket-type excavator, manufactured by the Hitachi-Zosen Corporation of Osaka, Japan (Figure 4). Critical to timely completion of the east tunnel was early delivery of the tunnel shield. The Japanese-manufactured shields were delivered in 6 months and were assembled on site. Generalized machine specifications are given in Table 3. The contractor proposed the use of a nonexpanded precast concrete segmental lining system. The segments for the 21-ft-diameter tunnel were 10 in. thick, and each ring was composed of four segments and a key. An 8-in.-thick

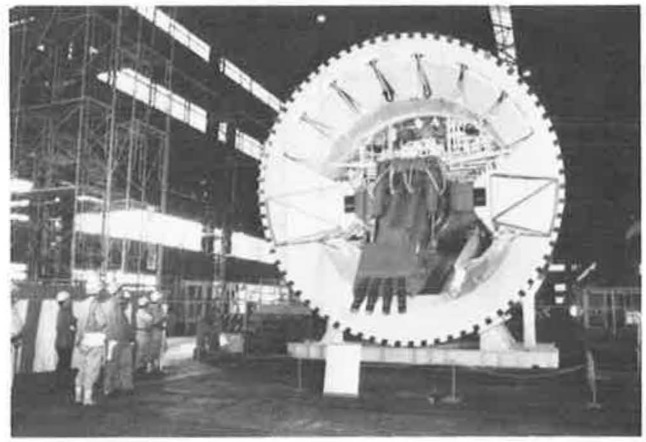

FIGURE 4 Shield 24 ft 10 in. in diameter.

TABLE 3 TUNNEL SHIELD AND INITIAL SUPPORT CHARACTERISTICS

Item	14 ft 0 in. Tunnel	21 ft 0 in. Tunnel
Nominal ring outside diameter (D_s)	16 ft 8 in.	24 ft 6 in.
Nominal segment thickness (h)	8 in.	10 in.
Width of segment (W)	4 ft 0 in.	4 ft 0 in.
Number of segments per ring	4 + key	4 + key
Approximate weight		
2 below spring line (each)	4.9 kips	9.1 kips
2 above spring line (each)	4.9 kips	9.1 kips
1 crown key	0.4 kips	0.7 kips
Reinforcement		
Longitudinal	W1.4 WWF, EF @ 12 in. on center	W2.9 WWF, EF @ 16 in.
Transverse	W3.5 WWF, EF @ 4 in. on center	W5 WWF, EF @ 3 in.
Area of steel (A_s)	0.105 in.^{2}/ft/face	0.2 in.^{2}/ft/face
Reinforcement ratio	0.012	0.018
Shield outside diameter (D_o)	16 ft 11.5 in.	24 ft 10 in.
Tail skin clearance	1 in.	1 in.
Tail skin thickness (t)	1 in.	3/4 in.
Ratio $D_o - 2t/D_s =$	1.0068	1.0099
Percent excess excavation	2.7	3.5
Shield propulsion system		
Number of jacks	18	18
Thrust force per jack	441 kips	221 kips
Face breasting system		
Number of jacks	7	7
Thrust force per jack	66 kips	66 kips

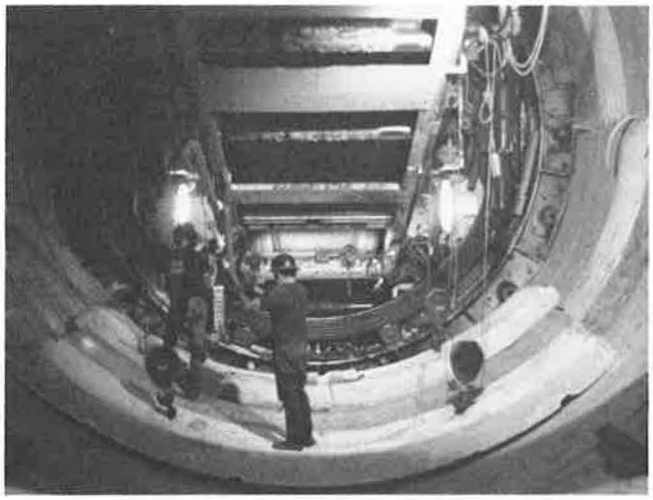

FIGURE 5 Segment erection.

precast segmental lining of similar geometry was proposed for the 14-ft-diameter tunnel. Figure 5 shows segment erection in the tail of the shield 24 ft 10 in. in diameter.

The contractor began tunneling on February 1, 1985, at a previously constructed 50-ft-diameter shaft at the east tunnel. Approximately 1 month later, tunneling was suspended when the contractor determined that the tunneling shield was not capable of advancing.

The material at the face was generally described as running sands, cobbles, and gravel. Persistent runs and chimneys to the surface had formed after each shove. The contractor was unable to control losses of ground at the face—it appeared that the machine was not capable of exerting any pressure on the face and that the excavator and breasting doors limited access to the face. The mining method used by the contractor generally contributed to this condition. He would excavate a central plug of ground and then shove ahead. This displacement allowed soil movement into the centrally excavated plug, initiating massive soil movements. To allow continued advancement, the radial face doors were relieved and allowed to rotate inward, fully relieving the entire soil face. Loss of this face support enabled ground movements to extend behind the region supported by the hood of the shield, resulting in subsequent surface subsidence and chimneys.

Subsequently, the contractor modified his tunneling equipment, as shown in Figure 6, to include the addition of poling plates, breasting boards, and a breasting table to his shield to increase the lead or hood of his machine. The poling plates and other face control mechanisms, when properly advanced, generally appeared to have improved the contractor's ability to minimize ground movements and stabilize the excavated face.

RATES OF PROGRESS

After the machine modifications, tunneling began in earnest on the east tunnel; the average rate of advance was approximately 75 ft/day. Two $9^1/_2$-hr production shifts and one 5-hr maintenance shift were employed. During tunneling, several 1,000-yd^3 or larger sinkholes developed above the tunnel and were filled before tunneling was resumed.

These large surface movements were tolerable because the area above the tunnel was cleared right-of-way. Holing through

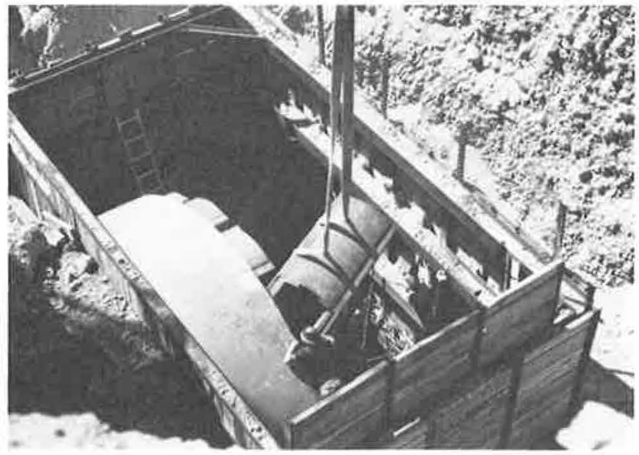

FIGURE 6 Installation of poling plates.

of the east tunnel was completed in March 1986, as shown in Figure 7. Shortly thereafter, concrete lining was begun. Concrete was placed by conventional tunneling techniques: telescoping metal forms were used and placement was done on a continuing three-shift basis—pour, set steel and forms, strip and move ahead (Figures 7–9). The tunneling shield was moved to the west tunnel and rebuilt. The shield and equipment were modified, including installation of larger cutting teeth on the poling plates and an increase in the overcutter. Tunneling progress on the west tunnel, now under way, is approximately

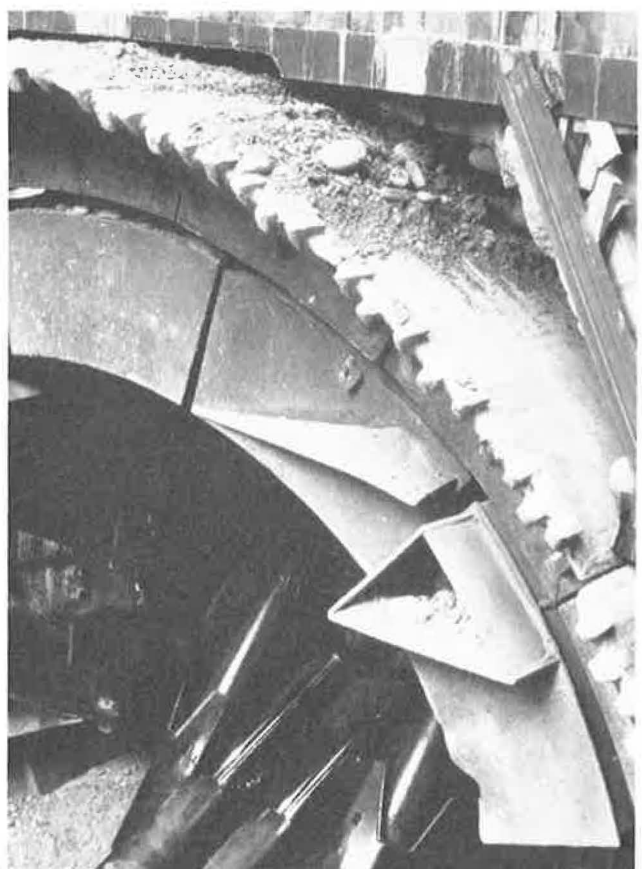

FIGURE 7 Holing through.

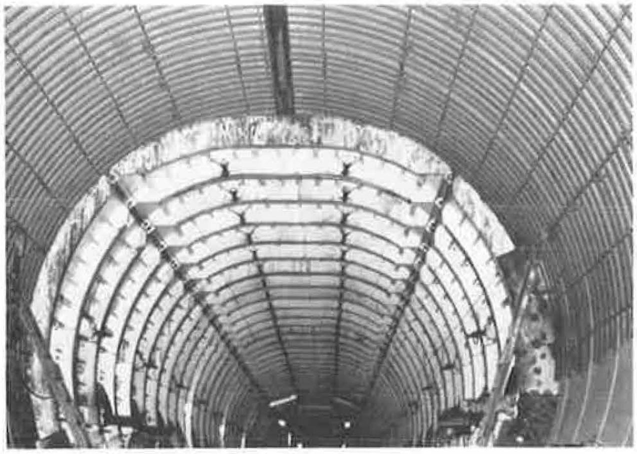

FIGURE 8 Final lining placement.

FIGURE 9 Finished tunnel.

140 linear feet per day. To date, progress on the best day was 228 linear feet.

MITIGATION OF GROUND LOSSES

The large ground losses created by construction of the east tunnel caused considerable concern. Because the west tunnel was to run beneath heavily traveled Central and Second Avenues in the middle of downtown Phoenix, no such losses could be tolerated. To minimize the induced settlements and provide a quick response mechanism if a large ground loss did occur, a compaction-consolidation grouting scheme was developed.

Compaction grout holes are drilled from the surface in advance of tunneling. As tunneling proceeds, and the shield passes each hole, as required, a pump crew feeds a low-slump soil and cement mixture into the ground above the tunnel envelope until refusal or until deflection of the segmental liner occurs. If a serious run occurs, large amounts of grout can be pumped through a series of neighboring holes. At several locations, chemical grouting was performed in advance of excavation to forestall ground losses. Ground settlements are generally kept to minimal levels, on the order of $^1/_4$ to $^1/_2$ in.

ACKNOWLEDGMENTS

This project was undertaken for the Highway Department of the Arizona Department of Transportation. Howard Needles Tammen & Bergendoff was the I-10 project design coordinator and designer of the tunnels. The contractor is a join venture of M. L. Shank Company, Denver, Colorado; John A. Artukovich Company, Azusa, California; and Ohbayashi-Gumi Ltd., Tokyo, Japan. Resident engineering services were provided by CRS Sirrine of Denver, Colorado. The assistance of R. S. Allmond, J. P. Whyte, J. Strid, and others of the resident engineering staff is gratefully acknowledged.

Insulation Against Ice in Railroad Tunnels

ERIK SANDEGREN

Many of the tunnels and rock cuttings in the Swedish railway system are old and badly affected by frost action and frozen groundwater. This ice is a serious problem because in bad cases it may intrude on the track structure from the walls, and in periods of thaw it may loosen and fall on the track. It is therefore necessary to remove the ice. This work is both expensive and dangerous, and high costs and risks might be avoided if the seepage of water could be reduced or the water prevented from freezing. Deep drainage, widening the tunnel section, caulking with sprayed concrete, grouting, and insulation used to be used for this purpose. Since 1979 an entirely new method of insulating with cellular plastic has been used. Details of this method are discussed: choice of material, joining, installation and dimensioning, and cost. An account is given of the work that has been completed and of the results that have been achieved. Conclusions that may be drawn from experience to date are presented.

In 1984 the Swedish State Railways was using 95 tunnels and a large number of rock cuttings of various ages. Frozen groundwater in both tunnels and cuttings and ice columns on tunnel roofs make it necessary to undertake major and costly work in many places. The ice also causes "frost shattering," which leads to uncontrolled rock falls and damage to the concrete reinforcement. The ice can cause, and sometimes has caused, damage to both staff and rolling stock. Drops of water on power lines, rails, and sleepers mean more maintenance work and reduce the useful life of the installations. Figures 1–3 show how severe the problem can be.

Cutting the ice away is both a time-consuming and a thankless task that involves high safety risks. Daily inspections, even on weekends, and patrols of three or four men cutting ice every day are expensive, particularly if so much ice is cut that it must be carried out of the tunnel or cutting.

The problem has been dealt with in many different ways. It is necessary to prevent the water from freezing before it reaches the drainage structures in the bottom of the tunnel. Many old methods, for example casings, fir twigs, mats, and electrical heating, now have been replaced by insulation with flexible cellular plastic.

REQUIREMENTS OF THE INSULATION MATERIAL

To insulate a cutting or a tunnel effectively against frost, a material that meets the following requirements is needed:

1. Very low heat conduction coefficient.
2. Water repellent to the extent that its heat insulation properties are scarcely modified over a period of time.
3. Resistant to weathering, water-soluble elements, and sunlight.
4. Strong enough to withstand the slipstream of the trains, wind, and minor mechanical damage.
5. Sufficiently malleable to be adapted to blasted rockfaces.

PROPERTIES

The insulation material used by the Swedish State Railways is a malleable, extruded, cellular polythene plastic (Ethafoam 220 from Dow Chemical Company). At first the white variety of the material was used but now the black version is used because its resistance to destruction on exposure to sunlight is significantly higher. It is possible that a similar material [Neopolen 1712, manufactured by Badische Anilin- und Sodafabrik (BASF)] may also be used. The properties of the materials are given in Table 1.

Sheets of Ethafoam that measure 2.75 × 0.60 m and are 50 mm thick are available in black. The rigidity provided by the 50-mm thickness makes this material suitable for insulation. One layer is used in the southern and middle parts of Sweden, and two layers are used in the north.

JOINING

As mentioned earlier, the sheet measures 2.75 × 0.60 m and thus only covers 1.65 m². If water is seeping through large areas, the sheets have to be joined to form a continuous piece with a surface area as large as 140 m². The sheets are welded together in mats 2.75 m wide (Figure 4). The length is limited by considerations of transport and handling. These mats are

FIGURE 1 Frozen groundwater in the Åmål Tunnel.

Swedish State Railways, Head Office, Geotechnique and Engineering Geology, S-105 50 Stockholm, Sweden.

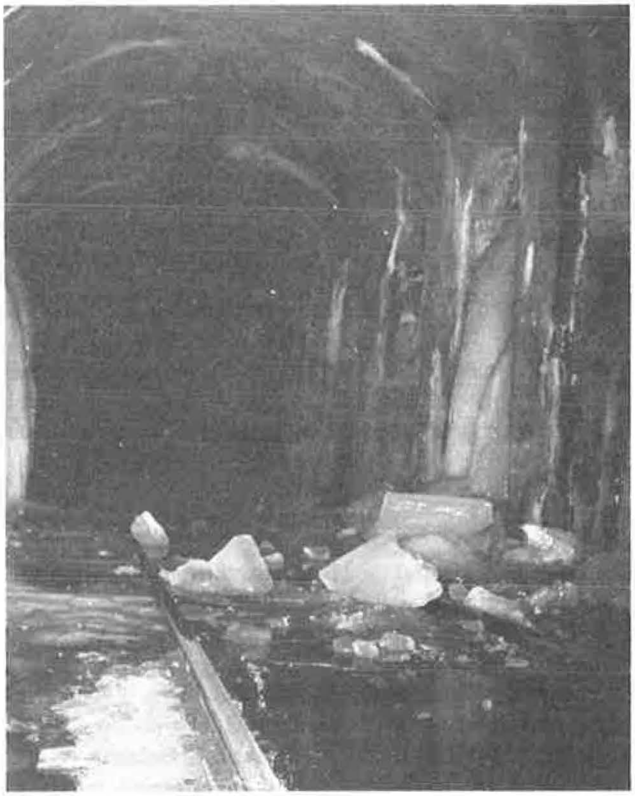

FIGURE 2 Ice fall in the Åmål Tunnel.

FIGURE 3 Concrete, cracked by ice, in the Liljeholmen Tunnel.

welded together on site with a hot-melt adhesive with a working temperature of approximately 170°C. Temperatures higher than 180°C destroy the mat. Meltex hot-melt adhesive equipment has been purchased and is shown in Figure 5.

TRANSPORTATION OF THE MATS

The prepared mats are transported from the bonding site to the mounting site by motor trolley (Figure 6), by sky-lift (Figure 7), or mounted on a truck with the mat hanging over the arm of the liftcage (Figure 8).

TABLE 1 MEAN VALUES ACCORDING TO DOW CHEMICAL AND BASF

Property (standard)	Unit	Ethafoam 220	Neopolen 1712
Density (DIN 53420)	kg/m^3	35	30
Load (DIN 53577) at			
10 percent compression	N/mm^2	0.035	
25 percent compression	N/mm^2	0.055	0.045
50 percent compression	N/mm^2	0.105	0.11
Deformation remaining after 24 hr recovery time (22 h, 50%, 23°C)	%	11	6
Tensile strength (DIN 53571)	N/mm^2	0.14	0.16
Elongation at fracture (DIN 53571)	%	90	55
Tearing resistance (DIN 54575)	N/mm^2	0.14	1.1
Water absorption over 24 hr (ASTM C-272)	% by volume	<0.5	<1.0
Steam permeability (DIN 52615)	μ-factor	>320	
Heat conduction coefficient at 25°C (DIN 52612)	W/mK	0.051	0.037

NOTE: DIN = Deutsches Institut für Normung.

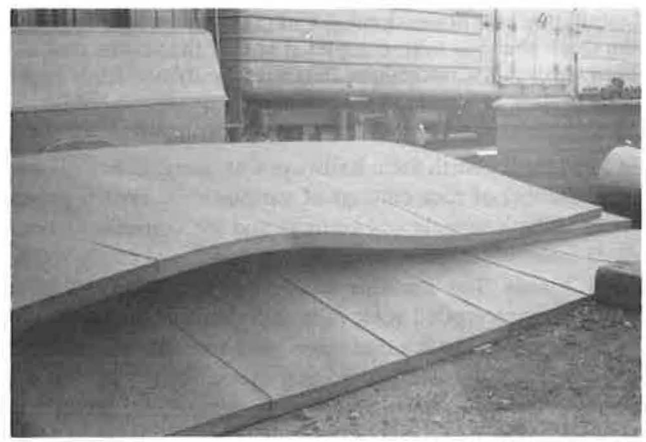

FIGURE 4 Ethafoam sheets glued together.

INSTALLATION

The mat is hung on the rock face by means of previously inserted bolts. The bolts have a fully threaded shank, are 12 mm in diameter, and are fixed to the rock with a chemical anchor (Figure 9). Drilling and setting one bolt takes only 3 or 4 min. A sky-lift is used for drilling and for hanging the mats (Figure 10). The mats are pressed manually against the wall. A more complete description of installation is given by Sandegren and Wallmark (*1*). Three installations are shown on Figures 11–13.

Costs

The 1982 costs of the material are given in Table 2.

WORK COMPLETED

Swedish Railways began insulating tunnels and cuttings with cellular plastic in 1979. Since then (end of 1986) 16 455 m of

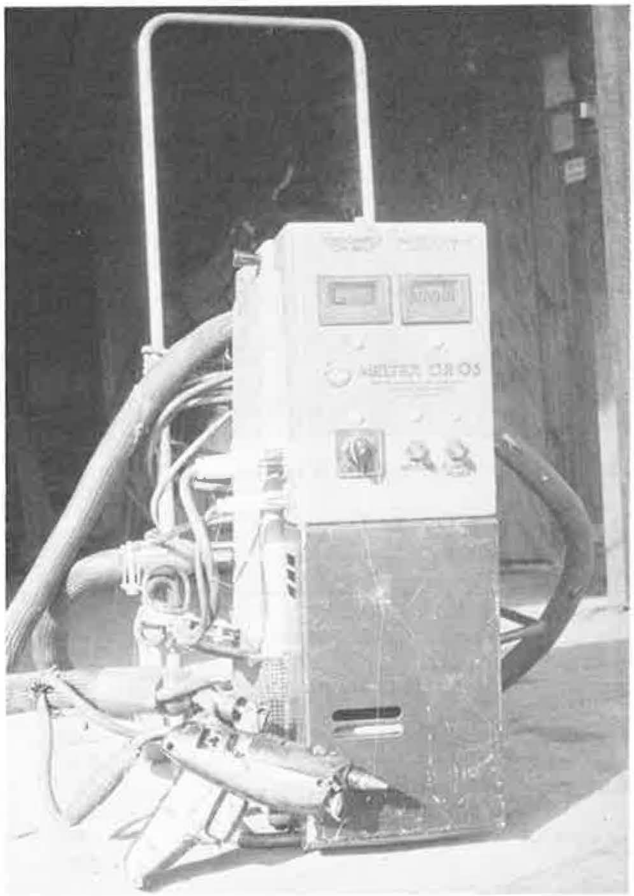

FIGURE 5 Equipment for bonding the sheets with hot-melt adhesive.

FIGURE 6 Transporting a mat with the aid of a trolley.

insulating mat have been installed. Table 3 gives completed and planned projects.

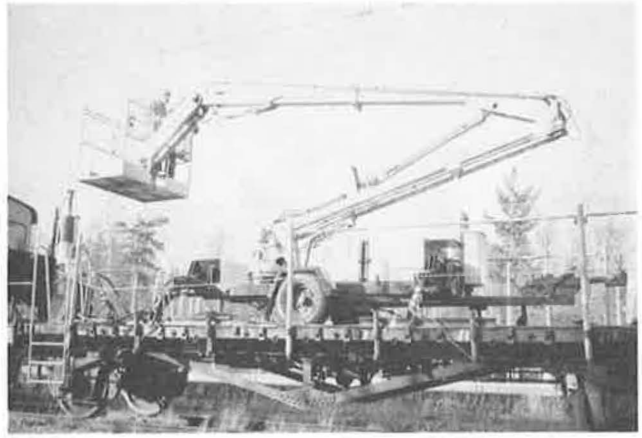

FIGURE 7 Sky-lift.

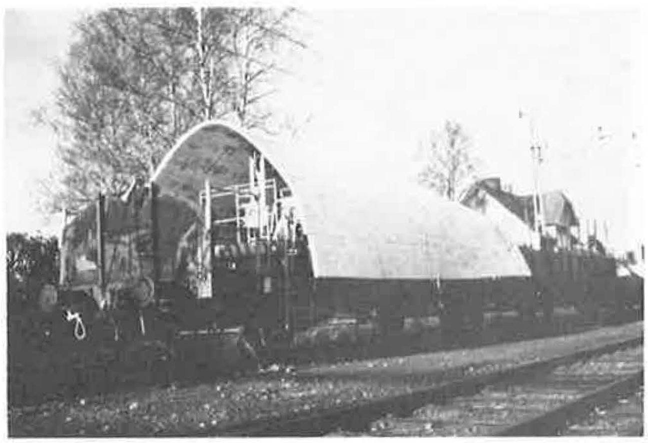

FIGURE 8 Transporting a mat with the help of a sky-lift mounted on a truck.

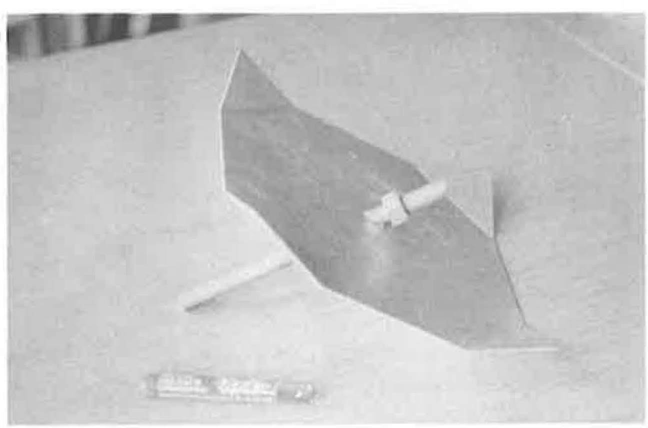

FIGURE 9 Chemical anchor, bolt, and washer plate.

RESULTS

By the winter of 1979–1980, Saetre (2) had shown by means of measurements that cellular plastic sheets provided effective insulation (Figures 14 and 15). However, when the idea was transferred to Sweden, the new material needed to be tried and a way of working with it, which was suited to the Swedish situation, had to be devised. Unexpected results, both negative and positive, were obtained before a viable method of installation was worked out.

Assessment of the projects that are now complete (Table 3) reveals that the results have exceeded expectations. However, a

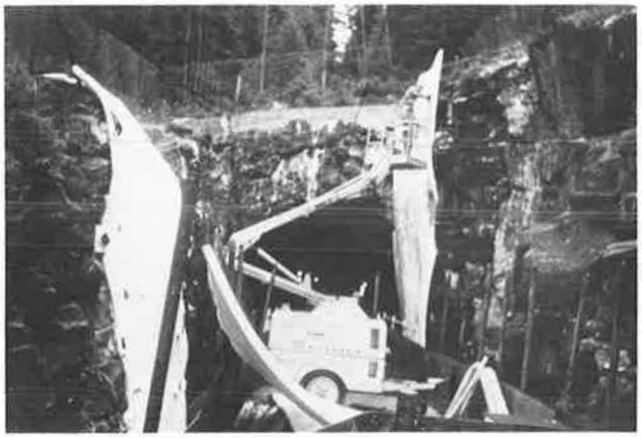

FIGURE 10 Installation of insulation with the aid of a sky-lift.

FIGURE 11 Installation in a cutting.

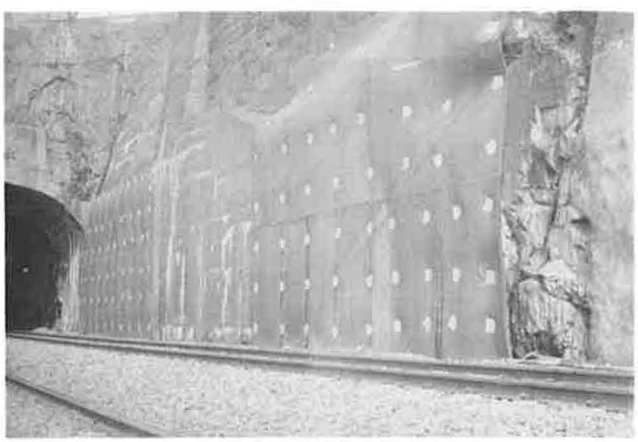

FIGURE 12 Installation in an opening.

number of things have been learned in the course of those projects.

At first 19-mm bolts fixed with a cotter key were used. In a few cases these bolts did not hold well, which meant that the mat hung down and caught on trains. Now 12-mm bolts, which are anchored with a chemical substance, are used. A great advantage of these slimmer bolts is that drilling can be done electrically.

FIGURE 13 Installation in a tunnel.

TABLE 2 COSTS

Material	SKr/Unit	SKr/m^2
Chemical anchor (HKA 3), each	6.70	3.35
Bolt, 0.33 m with washer plate and nut, hot galvanized, each	16.50	8.25
Ethafoam 220, 50-mm loose sheets, m^3	1240	62[a] 124[b]
Hot-melt adhesive, 5 kg	360	1.40[a] 2.80[b]
Welding mats to form sheeting factory		21[a] 42[b]
Total per m^2		96[a] 180[b]
Working costs		
Bonding work		7
Setting bolts and hanging mats (3 hr/day)		145
"Normal" case		106
Total cost		210–250[a] 300–340[b]

[a]50-mm insulation
[b]100-mm insulation

Careful sealing around the edges of the mat to prevent the circulation of air behind the mats also proved to be important. In places where there is now insulation, it has seldom been necessary to cut down ice. In those cases in which ice removal has been necessary, only icicles have needed to be cut from places where the insulation was not tightly mounted.

Usually the insulation has paid for itself in 4 or 5 years, but the cost of reduced "wear" on the rock wall and the track stock has not been taken into account.

CONCLUSIONS

The following conclusions can be drawn after installation of approximately 15 400 m^2 of insulation:

1. Flexible cellular plastic can be used successfully to insulate tunnels and cuttings against frost.
2. The material should be ultraviolet stabilized.
3. The edges of the insulation must be carefully sealed, so that cold air cannot penetrate behind the insulation from the sides.

TABLE 3 CUTTINGS AND TUNNELS INSULATED BY THE SWEDISH STATE RAILWAYS

Railway line	Place	m^2	T=tunnel C=cutting
1979			
Kiruna-Vassijaure	Nuolja	200	T
Kil-Ludvika	Loka	250	T/C
1980			
Kiruna-Vassijaure	Tornehamn	30	T
Ludvika-Borlänge	Rämshyttan	170	C
Bjärka-Säby-Västervik	Gamleby	40	T
Göteborg-Borås	Landvetter	440	T/C
1980-81			
Stockholm-Uppsala	Hagalund Ö	2450	T/C
Stockholm-Södertälje	Södermalm	80	T
Limmared-Värnamo	Gnosjö	300	T/C
1981			
Härnösand-Långsele	Kramfors	720	T/C *
Stockholm-Uppsala	Hagalund V	250	T
Stockholm-Södertälje	Västberga	10	T
Katrineholm-Åby	Graversfors	950	T *
Göteborg-Borås	Hindås	700	T/C
1982			
Älvsbyn-Boden	Laduberg	200	T
Stockholm-Södertälje	Liljeholmen	1500	T
Borås-Limmared	Aplared	1800	T/C
Ulricehamn-Limmared	Åsunden	400	T/C
Uddevalla-Munkedal	Kärra I	350	T/C
Göteborg-Huddevalla	Skeppsviken	170	T/C
1983			
Härnösand-Långsele	Kramfors	80	T **
Ludvika-Borlänge	Rämshyttan	30	C **
Limmared-Värnamo	Brandsmo	200	T/C
Borås-Limmared	Aplared	200	T/C **
1984			
Ställdalen-Ludvika	Grängesberg	35	C
Fagersta-Ludvika	Vad	200	C
Stockholm-Södertälje	Rönninge	250	T
Södertälje-Järna	Ström	70	T
Göteborg-Borås	Hindås	400	T/C **
Katrineholm-Åby	Graversfors	80	T **
1985			
Kristianstad-Karlskrona	Pengaberget	860	T
Halmstad-Getinge	Skogby	690	T
Halmstad-Getinge	Margretetorp	910	T/C
1986			
Halmstad-Getinge	Skogby Margretetorp	1100	T **
Hässleholm		260	C
Limmared-Värnamo	Gnosjö	80	T **
	Total 1986	16455 m	
Planned 1987 and later			
Göteborg-Borås	Hindås	200	T **
Göteborg-Uddevalla	Six tunnels		T/C
Halmstad-Getinge	Skogby Margretetorp	550	T **
Stockholm S-Hammarby-hamnen	Södersjukhuset	800	T
Mellerud-Kornsjö		500	C

* = three tunnels

** = adding work

FIGURE 14 Sealing along the edge of the mat with mineral wool.

4. The material is flammable, so great care must be taken when installing it to ensure that water seepage or ice formation cannot produce a spark-over from the overhead power line.

5. The cost of cutting ice is normally so great that the cost of installing insulation can be amortized over a few years, which makes the treatment economically viable.

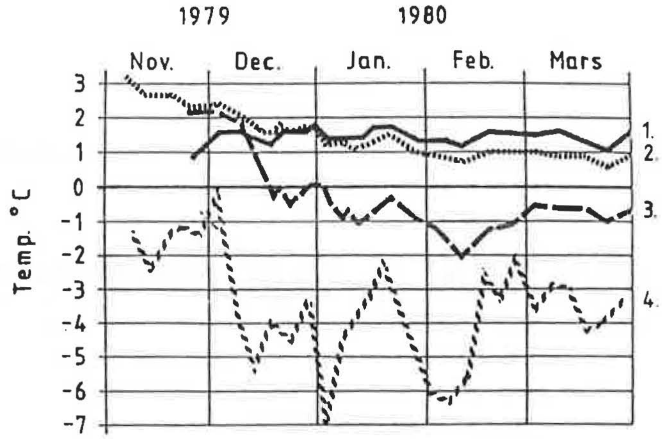

FIGURE 15 Measurements of the temperature in the Högberget Tunnel according to Saetre *(2)*: 1. = temperature on the rock face behind 70 mm insulation; 2. = temperature on the rock face behind 50 mm insulation; 3. = temperature 1.0 m into uninsulated rock; and 4. = 5-day mean temperature of tunnel air.

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2. K. Saetre. *Frostsikring av tunneler, isolering med Ethafoam.* Report GK 2531. Norges Statsbaner, Geoteknisk Kontor, Oslo, Norway, 1980.