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.

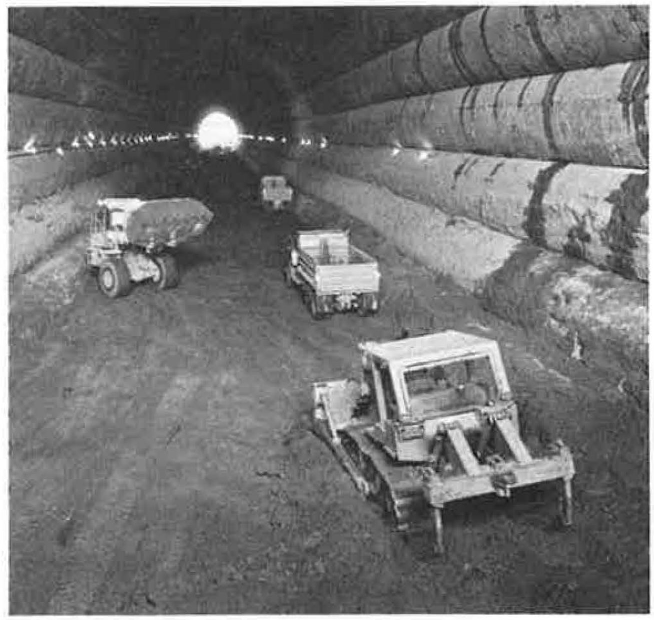

FIGURE 1 Tunnel mass excavation near completion (Richard S. Heyza/*Seattle Times*).

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 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 westbound 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

Howard Needles Tammen & Bergendoff, Seattle Western Regional Office, 600 108th Avenue, N.E., Suite 405, Bellevue, Wash. 98004.

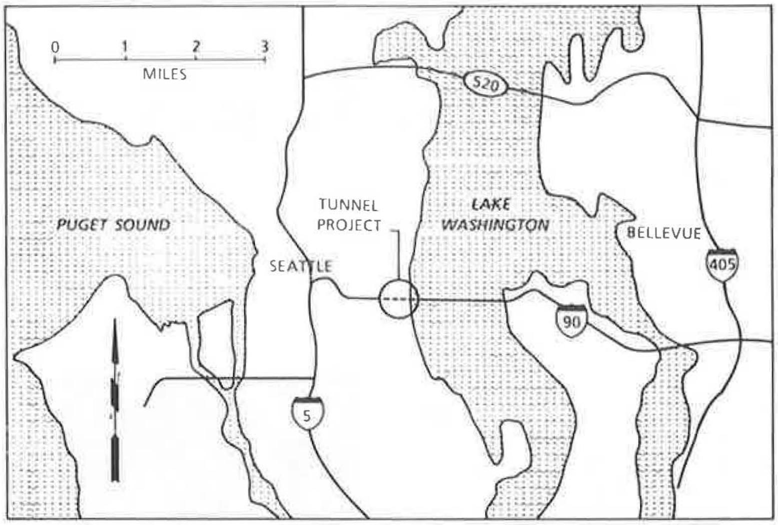

FIGURE 2 Location map.

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.

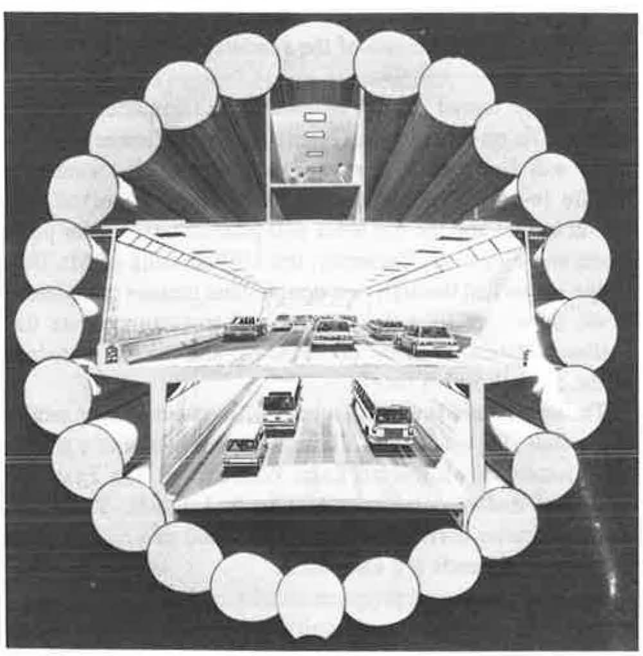

FIGURE 3 Traffic on three levels.

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

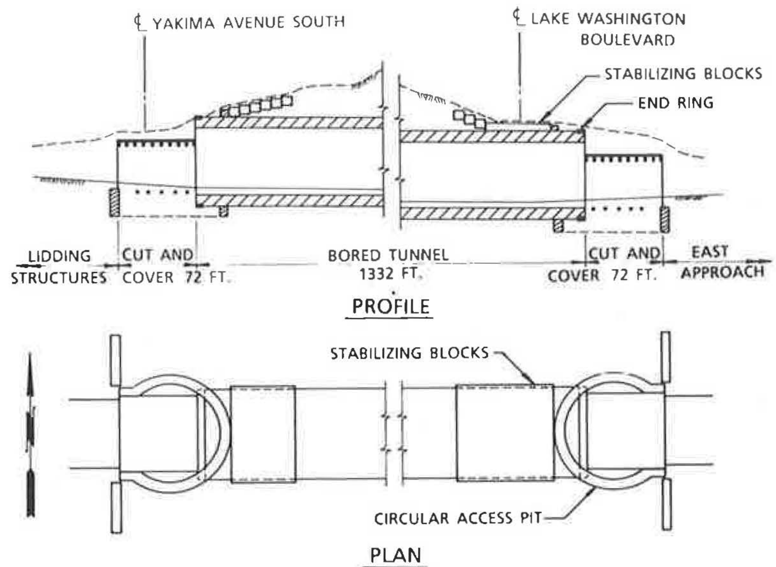

FIGURE 4 Structural arrangement.

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

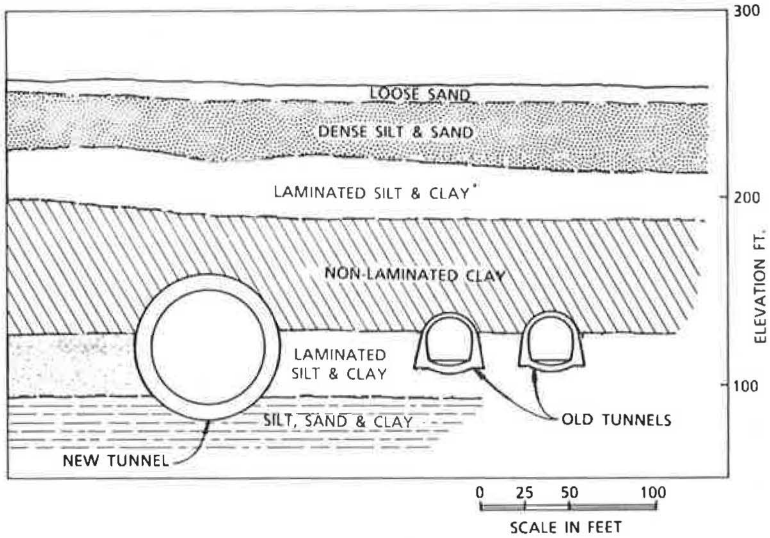

FIGURE 6 Typical subsurface section (looking east).

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.

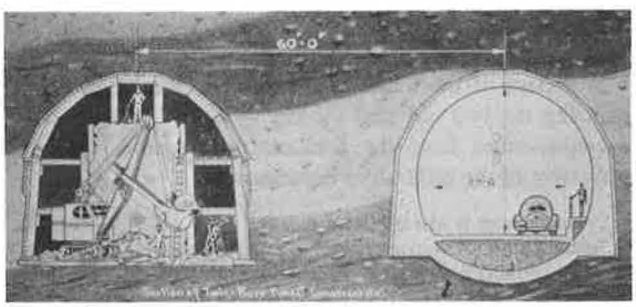

FIGURE 7 Construction of old tunnels.

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 westward 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.

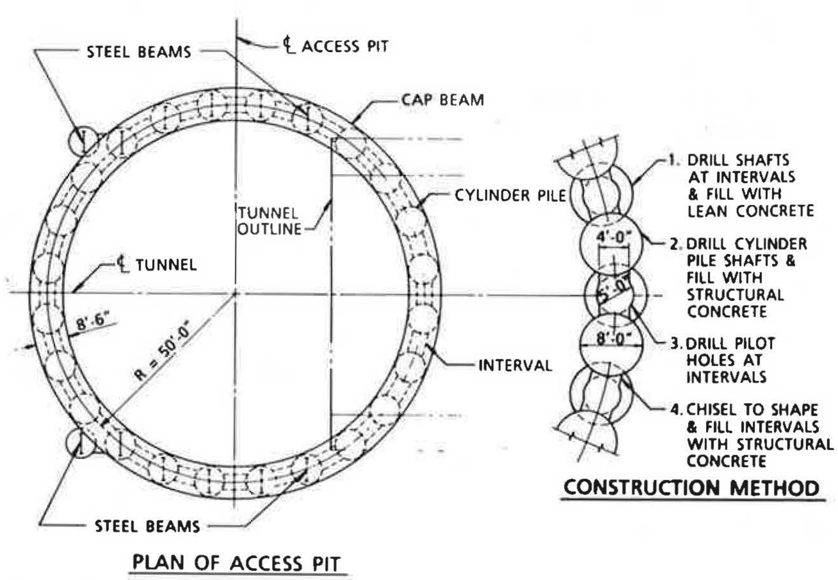

FIGURE 8 Access pit arrangement.

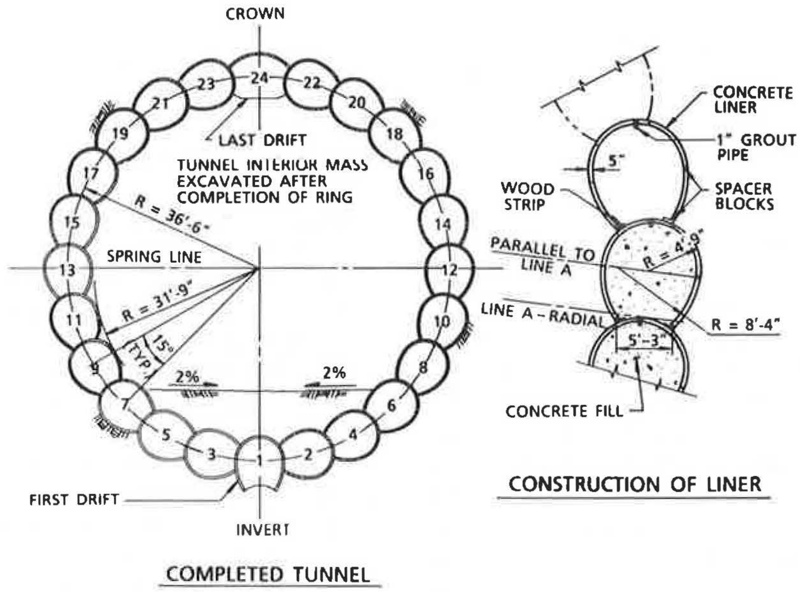

FIGURE 9 Stacked-drift tunnel liner.

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 1/2-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.