제 5 주제

미국 Seattle의 Mt. Baker Ridge Tunnel의 현장계측

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씨애틀 Baker 산 터널시공중의 지반 및 복공거동

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미국 씨애틀과 보스턴을 연결하는 I-90번 고속도로 (Freeway)의 씨애틀 기점에 위치한 Baker 산 터널은 토피 30m 정도의 연약지반층에 놓여있고, 산위에는 주택건물이 놓여있기 때문에, 계획, 조사, 실험, 설계에 20년 이상이 걸렸다. 이터널은 내경 19.4m의 원형단면으로서 연약지반상의 것으로는 세계최대이고, 길이 400여 m 인 도로터널이고, 이 터널옆에는 기존의 터널이 있다.

이 터널의 공사중 Disputes Review Board 에 터널 자료를 공급하고, 설계시의 제 가정치들의 타당성을 검증하기 위하여 광범위한 지반계측 계획을 수립하여 적용하였다.

주변 토질의 변위를 최소로 하면서 내경 19.4m의 토질심부를 굴착하기 위하여 일차로 지름 2.9m 의 적은 콘크리트 갱도 24개를 주 터널 주위에 시공한 후 변위를 계측하면서 심부를 굴착하였다.

시공중 계측되는 측정자료는 터널주위의 토질 안정,복공(liner) 의 처짐, 인접터널에 영향을 주는 하중과 충격등을 해석하는데 사용하고, 한편 시공중 굴착으로 인한 침하나 변위가 6cm를 초과하지 않도록 하는데 사용되었다.

GROUND AND LINER BEHAVIOR DURING CONSTRUCTION OF THE MT. BAKER RIDGE TUNNEL

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ABSTRACT

An extensive geotechnical monitoring program was used to assess ground and liner behavior, provide input to a Disputes Review Board, and verify design assumptions during construction of the Mt. Baker Ridge Highway Tunnel along I-90 in Seattle, Washington. The stacked-drift tunnel liner was constructed with twenty-four 2.9 m diameter concrete-filled drifts forming a semi-flexible compression ring, followed by excavation of the 19.4 m diameter soil core. Instrumentation data was used for monitoring hillside stability, liner deflections and loads, impacts on adjacent highway tunnels, for back analyzing soil properties, and for helping to limit construction-induced settlements to about 6 cm.

INTRODUCTION

The Mt. Baker Ridge Highway Tunnel, located in Seattle, Washington (Figure 1) was recently completed at a cost of $36.5 million, including $600,000 in resolved claims, for the Washington State Department of Transportation (WSDOT). The cost was $1.8 million below bid price due to low inflation. With an inside diameter of 19.4 m and an outside diameter of 25.2 m, this is the largest diameter soft-ground tunnel in the world. The tunnel is being constructed as an important part of the last remaining 11-km link of I-90 through Washington at a total cost of about $1.4 billion.

The tunnel was constructed using an innovative "stacked drift" method in which an articulated, or flexible, tunnel lining consisting of 24 concrete-filled drifts was first constructed to form a 406 m long horizontal compression ring followed by removal of the soil core.

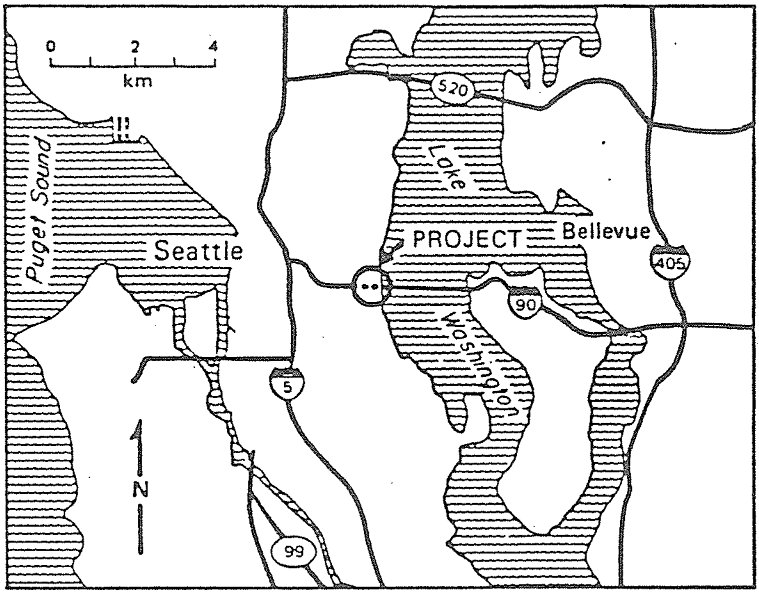

Figure 1. Project Location

Excavation began in January, 1983 and was completed in May, 1986. Currently, the interior roadway structure is under construction. When complete, the tunnel will provide five lanes for the I-90 freeway and a pedestrian/bicycle path. Three additional lanes will be provided by the existing two double-lane highway tunnels located a clear distance of 21.4 m to the south.

Construction of the tunnel was accompanied by a comprehensive geotechnical monitoring program. The program included monitoring of the magnitude and distribution of surface and subsurface settlements, displacements and stresses within the completed tunnel liner. The program was implemented due to the unprecedented size and method of construction of the tunnel and because of the very large settlements of over 0.3 m which occurred during construction of the neighboring two I-90 tunnels 45 years ago. This paper describes the geotechnical conditions, the design and construction approach, the geotechnical monitoring system and the results of the monitoring program which were used to assess and modify construction procedures and provide data for the Disputes Review Board.

Design of the tunnel was accomplished by Howard Needles Tammen and Bergendoff with geotechnical investigation, instrumentation, and underground engineering provided by Shannon & Wilson, Inc. WSDOT also established a Design Review Board comprised of Dr. Ralph Peck, Mr. Al Mathews, and Mr. Chuck Metcalf. A Disputes Review Board consisting of Joe Sperry, Ken Dodds, and Ron Heuer was established by the WSDOT and the contractor, the Guy F. Atkinson Company.

GEOTECHNICAL CONDITIONS

The tunnel was constructed primarily through glacially overridden hard lacustrine silts and clays as well as minor amounts of dense outwash sand, and till (Figure 2) deposited about 13,000 years ago. Soil properties used in design, as determined from laboratory and field tests are summarized in Table 1 and are discussed more fully by Robinson, et al. (1983).

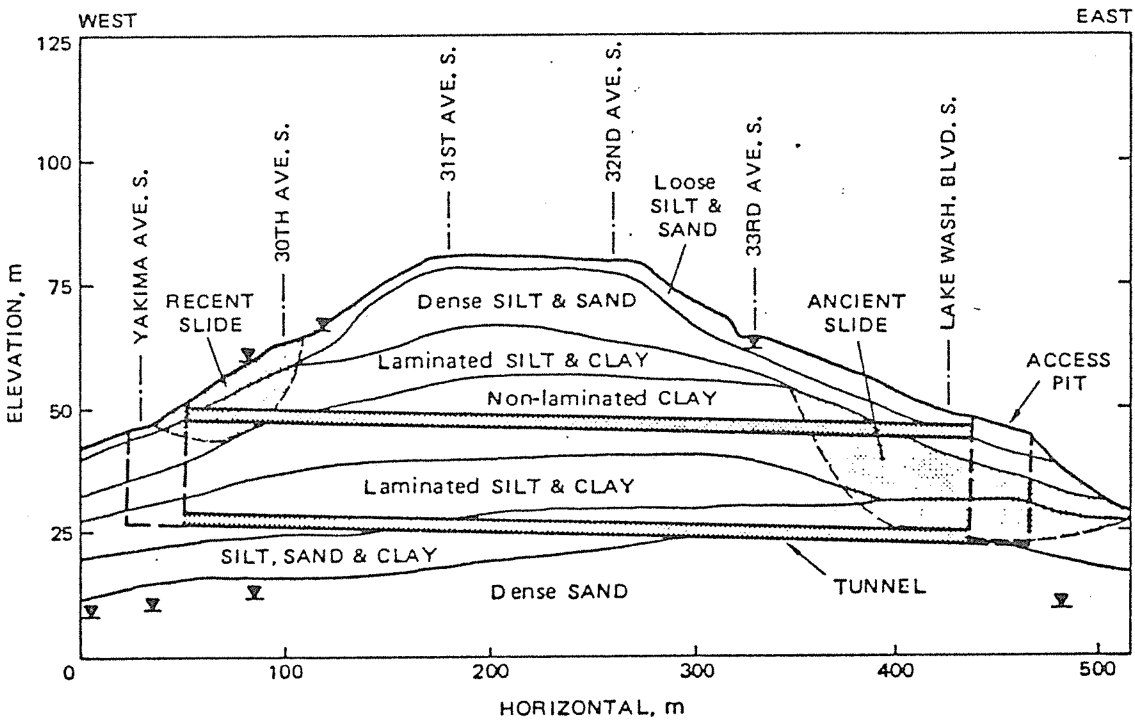

Figure 2. Longitudinal Subsurface Soil Profile

Table 1. SOIL PROPERTIES USED FOR DESIGN

Material Type	SPT "N"	Wet Unit Weight (kN/m^3)	w (%)	Atterberg Limits LL	Atterberg Limits PL	Shear Strength (MPa)	Pre-consol. Stress (MPa)	Elastic Modulus (MPa)
Dense Sand	81	20.1	11	--	--	--	--	140
Hard SILT & CLAY	35	18.4	32	60	29	0.14	1.17	140
Hard SILT	102	18.9	27	42	20	0.10	1.53	275
Very Dense SAND	114	18.7	14	--	--	--	--	345

The lower dense sand unit encountered near the east end of the tunnel was fairly dry and clean with a tendency to ravel in the heading, resulting in one large overbreak extending 6 m above Drift 1. The hard silt and clay soil units were fissured and slickensided, resulting in blocky behavior. The clay and silt units, and in particular the middle non-laminated clay unit, contained scattered cobbles and boulders up to 3 m in diameter. Boulders generally halted advance for one shift, until they were broken into manageable chunks.

The strength of the intact clays is high, with shear strengths up to 1 MPa, but the strength of the soil mass is significantly reduced by the numerous microfractures and slickensided joints. Estimated soil mass strength was used in design, as shown in Table 1.

Overconsolidation of the sands and clays has resulted in a relatively high deformation modulus. Modulus values had a significant range depending on the type of test. The unconfined compression tests tend to underestimate modulus so the modulus values derived from plate jack and field seismic velocity tests were used for design.

Glacial overriding also contributed to increased horizontal soil stresses which influence the distortions and stresses in the liner. Correlations with consolidation tests indicated K_o values of 0.6 to 1.0. At the time of the site explorations in the late 1960's, in situ methods for measuring K_o were not readily available. For design purposes, a K_o ranging from 0.5 to 2.0 was used.

DESIGN AND CONSTRUCTION APPROACH

Design of the Mt. Baker Ridge Tunnel as a single, large-diameter tunnel rather than as several smaller tunnels or even a braced-cut was mandated by public input to reduce effects on the overlying residential area and to reduce the required right-of-way. The implemented design took into consideration the extreme difficulty in constructing a large-diameter tunnel by conventional full-face methods. The design was developed to permit the use of conventional sized equipment to excavate a series of small-diameter tunnels, pre-forming a segmented, semi-flexible compression ring followed by subsequent excavation of the soil core (Figure 3). More details on design and construction are presented in Holloway and Kjerbol (1986).

Since the "stacked-drift" method required construction of the complete tunnel lining before excavation of the tunnel core, the lining must accommodate forces and deformations which are normally relieved or greatly reduced with more conventional tunnel construction methods. Conventional tunneling encourages in situ stress release, arching of the ground, and redistribution of the stresses into the surrounding soil. The flexibility of the designed liner allows it to adjust to non-uniform external soil pressure, thus minimizing the moment carrying requirements of the liner. The design

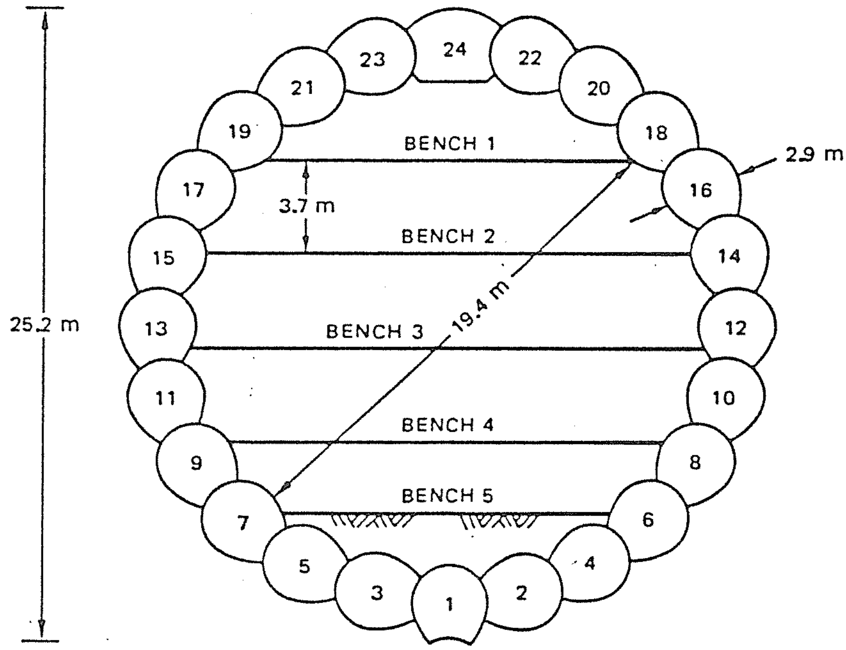

Figure 3. Sequence of Drift and Bench Excavations

also reduces ground disturbance, ground movements, and surface settlements.

Soil/liner interaction was evaluated using state-of-the-art FEM studies (Hendron, et al., 1971). Studies provided a range of design requirements for liner deformations (0.02 to 0.5%) and liner thrusts (14 to 54 MPa) under the effects of 3 to 30 m of soil cover, varying liner stiffness and soil modulus, and K_0 ranging from 0.5 to 2.0. Analyses indicated that the design was capable of sustaining the developed thrusts, eccentric loading and deformations even for extreme ranges of soil parameters.

Quarter scale model tests of two drifts were performed to assess joint strengths and deflections for a range of thrusts and eccentric loading. Tests indicated that the joints could sustain eccentricities equivalent to 1% distortion of the tunnel diameter under loads considerably higher than anticipated from the analyses. Simple 1/50 scale model tests on a complete flexible tunnel section were performed to assess the effects of varying shallow soil cover on deformations and ultimate collapse of the tunnel. These model tests demonstrated the stability of the tunnel liner under the equivalent of less than 3 m of granular soil cover.

The contractor was required to utilize tunnel construction methods that minimized ground deformations and resulting settlements due to individual drift construction, such that soil deformations measured 1 m above and beside each drift was less than 2.5 cm and 1.3 cm, respectively. The restrictions on maximum ground deformations required positive ground support at the heading, such as a shield and either an expanded or grouted, steel or concrete segment liner until the concrete backfill was placed.

The contractor was innovative in his selection of methods for constructing both the drifts and the joints between drifts and elected to construct the minimum 24 drifts, using a horseshoe-shaped tunneling shield. This shield shape was optimal for minimizing the overall thickness of the tunnel liner and maximizing the portion of the tunnel liner circumference that was constructed in any one excavation pass.

The joints between drifts had a width of 1.7 m to accommodate calculated maximum liner thrusts on the order of 14.5 MN/m of tunnel. To maintain liner flexibility, the joints between successive drifts were unreinforced, the only requirement being that all contacts were clean. Drifts were temporarily supported with expanded, precast concrete segments 12.7 cm thick and 1.2 m long with 5 segments per ring. Liner rings were seated against the exterior of the prior drift. The crown drift was odd-shaped and of non-uniform width and consequently was hand-excavated. The drift was supported with steel ribs and lagging braced against the adjacent concrete-filled drifts.

Each drift was backfilled with concrete immediately after it was cleaned out and prior to excavation of the next adjacent drift. The sequencing of drift excavation beginning at tunnel invert and progressing to tunnel crown (Figure 3) allowed the visual inspection of concrete quality at the joints between drifts, particularly at tunnel springline.

Immediately following concreting each drift, grout was injected through 25 cm pipes embedded in the concrete and attached to the crown of the precast liner. Grout pressures of up to 1 MPa (150 psi) were required, with average takes of two sacks of cement per 0.3 m of drift. During excavation of the core soils, grout was observed filling the voids between the mass concrete and the liner as well as voids between the liner and surrounding soil. Details of drift excavation, liner erection, and backfilling are provided by MacDonald (1985).

Following completion of the 24 drifts, the core soils were excavated in maximum of 4 m high lifts. This was intended to minimize the dangers of high unstable bench slopes and control the rate of deformation and stress buildup in the liner and surrounding soil.

INSTRUMENTATION PROGRAM

Due to the unprecedented size of the tunnel, the unusual method of construction, and the need to monitor impacts on overlying utilities and structures, a comprehensive geotechnical instrumentation and survey program was specified. The program was designed to monitor the behavior of individual drift excavations, tunnel liner, access pits, adjacent existing tunnels, and potentially unstable slopes and thereby limit damage to overlying utilities and structures, check design assumptions, and provide data for the resolution of disputes.

The instrumentation was selected and designed by Shannon and Wilson, Inc., to minimize interference with the contractor and maximize instrument survivability in active excavation areas. The plans and specifications included a $259,000 force account item for purchase of the instrument hardware and a bid item of $239,000 for instrument installations and assistance. The WSDOT monitored all instruments and survey points at an estimated cost of $675,000. Shannon and Wilson, Inc., installed the more sophisticated instrumentation systems and reduced and analyzed all instrument and some of the survey data at a cost of $645,000. Total cost of the instrumentation program was approximately $1,818,000, or 4.7% of the construction contract. Instrumentation program statistics are shown in Table 2.

Table 2. INSTRUMENTATION PROGRAM

Measurement Systems	Quantity of Installations	Sets of Readings	Resolution	Estimated Accuracy	% Still Working
Inclinometer Casings	62 (2,850 m)	37	±0.004 mm	±0.02 cm	94
Settlement Rings	1,772	67	±0.35 cm	±0.8 cm	94
Borehole Extensometers	5	20	±0.003 cm	±0.007 cm	100
Concrete Stress Meters	72	16	70 kPa	±170 kPa	94
Joint Meters (a)	20	49	±0.003 cm	±0.05 cm	80
(b)	81	37	±0.003 cm	±0.007 cm	95
Tape Extensometer Points	121	10	±0.002 cm	±0.025 cm	100
Strain Gages	192	43	±0.25 MPa	±0.5 MPa	96
Survey Points	309	21	±0.3 cm	±0.6 cm	83

The instrumentation program proved to be particularly useful in assessing the contractor's procedures. The data provided a basis for requiring early changes in his drift excavation and support methods and thereby minimizing ground deformation and consequent surface settlements and potential damage to surface structures. The data was also used by the Disputes Review Board, in assessing the validity of disputes and arriving at a reasonable resolution of claims.

Inclinometer

The inclinometer system consisted of grooved plastic casings grouted in up to 80 m deep borings arranged at 3 major (Figure 4) and 3 minor instrument lines. Measurements were taken with a biaxial inclinometer probe monitored with a portable RPP (Recorder-Processor-Printer) readout from the Slope Indicator Co. A total of 3 probes, 3 cables, and 2 readouts were utilized. One probe was irrepairably damaged by construction operations. The RPP readout boxes required several repairs, primarily due to heavy usage of the various controls. Likewise, the probes required periodic recalibration due to minor sensor drift and wear of the wheel assemblies.

Data errors of ±0.3 cm per 30 m of casing occurred when comparing data from different probes. When comparing data from the same probe, errors of ±0.05 cm per 30 m of casing were typical.

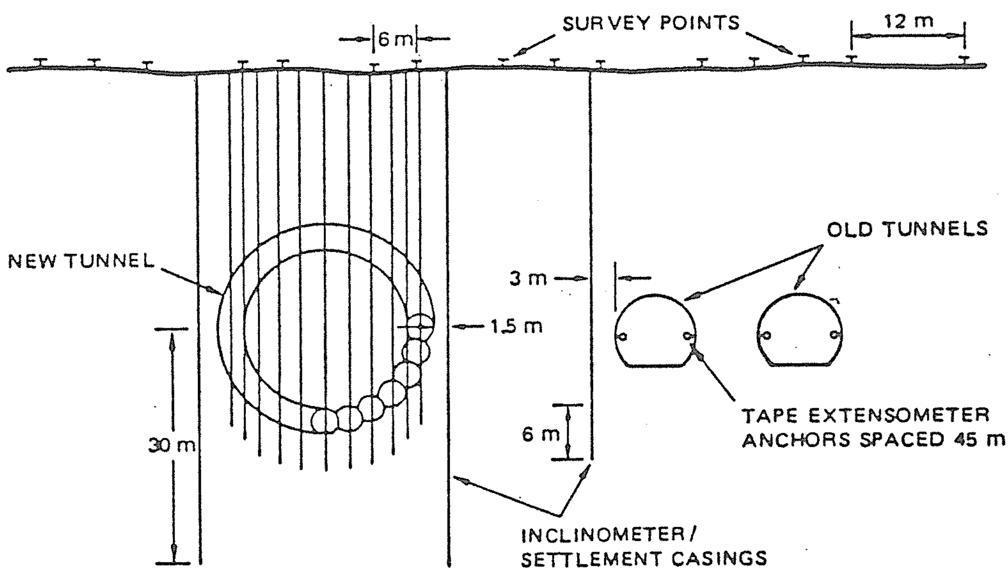

Figure 4. Typical Major Instrumentation Line

Subsurface Settlement

Fifty-eight of the inclinometer casings included stainless steel wire rings on 1-m sections of corrugated plastic pipe taped to the outside of the casings at 0.6 to 3 m intervals. The locations of these rings were monitored with a traversing inductance probe (Sondex) from the Slope Indicator Co. Four separate probes were required during the project due to high cable wear and the obliteration of foot markings on the cables. For increased speed and ease, measurements were generally made using the foot markings on the plastic cables, rather than with a steel reference tape.

With three probes, measurement errors of up to 1.3 cm per 30 m of casing occurred during the initial four-month life of each probe.

The errors were linear with depth and appeared to be related to hardening and/or shrinkage of the cables with time. After about four months, the shortening phenomena ceased and the length of the cables remained nearly constant. Fortunately, several casings located 20 m from the new tunnel were relatively stable and were used for reference readings to determine cable shrinkage. Periodic reference measurements were made with all probes, with and without a steel tape.

Borehole Extensometer

Five borehole sonic probe extensometers from Irad Gage Co. were installed in plastic inclinometer casing extending initially from ground surface, through and 15 m below Drift 1 (Figure 5). Each three-position extensometer was installed from Drift 1 using expanded Borros point anchors that pierced the platic casing.

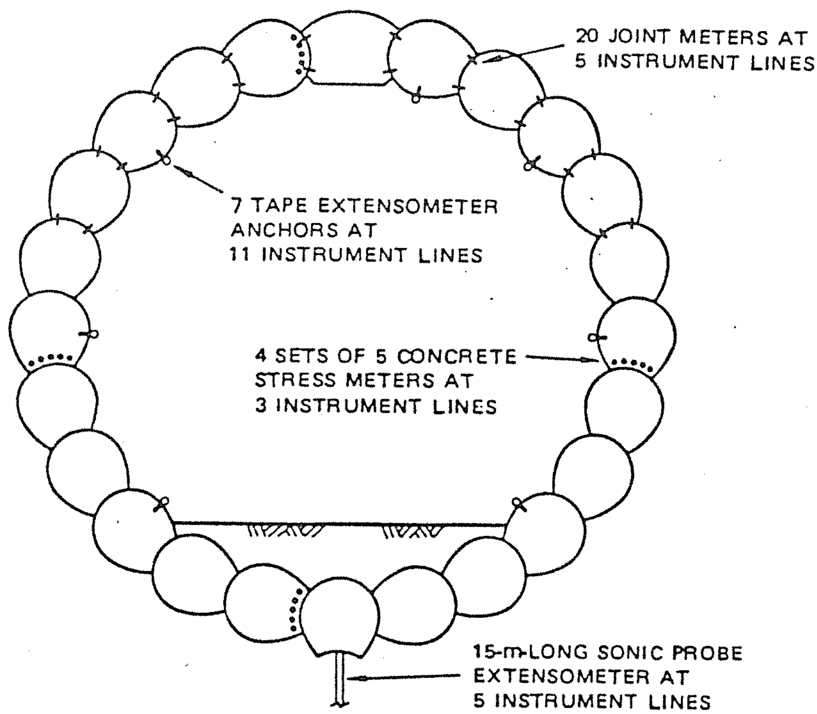

Figure 5. Typical Liner Instrumentation Layout

Concrete Stress Meters

Groups of five Carlson concrete stress meters were imbedded in the backfill concrete (Figure 5) at three stations located within 12 m of both portals, and near the center of the ridge. The mercury-filled, unbonded resistance gages were encased in 30 cm diameter cylinders of wet-cured concrete within three days prior to being imbedded in the backfill concrete of a drift. Groups of 5 cylinders were anchored in a period of 1 to 2 hours to the lining at the joint between drifts. The mass concrete was vibrated around the gages in the invert and springline drifts, but due to the presence of longitudinal reinforcement, was not vibrated around gages in Drift 23.

Laboratory tests on two concrete cylinder test gages indicated an accuracy of 70 kPa. Gage readings in the field were reproducible to $\pm$170 kPa. Gages in invert and springline drifts yielded reliable data; however, gages in Drift 23 provided very poor data.

Joint Meters

Joint meters were added to the instrumentation program (Figure 5) to monitor joint movements between drifts above springline. Due to large soil deformations, particularly above Drift 2, as well as smaller settlements above the rest of the lower drifts, there was concern that the core soils might continue to settle during construction of the upper drifts. Settlement of the upper drifts could open joints between drifts before the lining was completed. Joint meters were therefore installed at the joint in holes drilled through the liner and into the backfill concrete of each exposed drift.

The first 20 joint meters were Carlson unbonded resistance concrete joint meters with a range of 2.5 cm. These gages could tolerate only $\pm$0.25 cm of shear. About 20% of these gages were lost soon after they were installed due to water entering the leads where it had collected in instrument manholes at ground surface.

The remaining 81 joint meters were designed by Shannon and Wilson, Inc. to accommodate up to 1.2 cm of shear. The gages were made by Geokon, Inc., incorporating a sealed 5 cm range linear potentiometer covered with a 5 cm diameter, 25 cm long corrugated rubber hose attached to two plastic end anchors. The potentiometer leads were much less sensitive to water effects and only 5% were lost.

Tape Extensometer

Specially designed tape extensometer anchors, designed as a double-sided socket, were installed through the liner from inside the drifts (Figure 5) and then accurately surveyed. The intent was to subsequently accurately survey the other end of the socket when it was exposed during core excavation and thus determine absolute movement of the socket. Accuracy of the surveys in the drifts and core were not sufficient, however, for determining minor ground movements. Specially designed "keyhole-shaped" rings were attached to the sockets for measurements with a Slope Indicator Co. tape extensometer during excavation. The "keyhole" ring was large enough to place and remove a specially designed tape extensometer hook, using a 15 m long telescoping pole, manhandled from the bottom of the bench excavation. Due to windy conditions in the tunnel, repeatability and accuracy of this system were estimated to be on the order of 0.025 cm. Unfortunately, the enlarged "keyhole" rings also presented excellent targets for construction equipment excavating the bench soils. Four of the first eleven rings were damaged. Subsequent rings had a much lower damage rate and currently 100% of the rings are available for measurement.

Strain Gages

Weldable vibrating wire strain gages from Slope Indicator Co. were installed in cylinder piles at each of the access pits on 4 steel beams at 6 levels in groups of 4 gages each. Shannon and Wilson, Inc. installed all of the gages at the steel supplier's shop.

Surface Surveys

Standard surveying techniques were used for monitoring settlement as well as horizontal slope movements along tunnel centerline and 6 cross lines. The most significant contributor to survey error on this, as on most jobs, were minor benchmark shifts. Benchmark shifts of up to 1 cm were observed, requiring periodic correction of the settlement data.

Readouts and Data Collection

The inclinometer systems were monitored with an RPP readout. All other instrument systems were monitored with hand-held readouts and the data was recorded on electronic clipboards. Data was transmitted from the WSDOT field office to Shannon & Wilson for data reduction and evaluation using PDP/1134 and MacIntosh 512 computers. The raw tablulated data was generally submitted to the contractor within 16 hours of collection in the field. Reduced and analyzed data was tabulated in weekly and monthly reports and submitted to the WSDOT.

Monitoring will continue through 1990, but at a much reduced schedule since tunnel excavation is complete. Readings are planned bi-annually. The first set of 6-month readings since completing core excavation indicate very little change in the data.

RESULTS OF INSTRUMENTATION

Data Evaluation for Drift Construction

The largest soil deformations and surface settlements were expected to occur during drift construction. Since preservation of the soil strength was deemed critical to optimum soil/structure interaction, significant effort was devoted to monitoring soil deformations.

Ground Deformations. As noted earlier, the specifications included a requirement that the contractor select construction procedures which would limit vertical and horizontal deformations above and beside each drift excavation to less than 2.5 cm and 1.3 cm, respectively. During Drift 1 excavation, ground deformations were less than 2.5 cm, primarily because the ground was undisturbed and could accommodate, possibly with very little support, a single drift. The excavation of Drift 2, however, resulted in over 7.6 cm of ground deformation (Figure 6). Minor deformation occurred ahead of the shield, but 3.8 cm occurred over the shield, probably due to the 2.5 cm thick overcut and shield misalignment. At the tailskin of the shield, 3.2 cm occurred due to inadequate expansion of the segmented liner. An additional 1 cm occurred long-term due to gradual creep and collapse of soils around the liner. Horizontal deformations were less than 1 cm, indicating that displacements were mainly gravity induced.

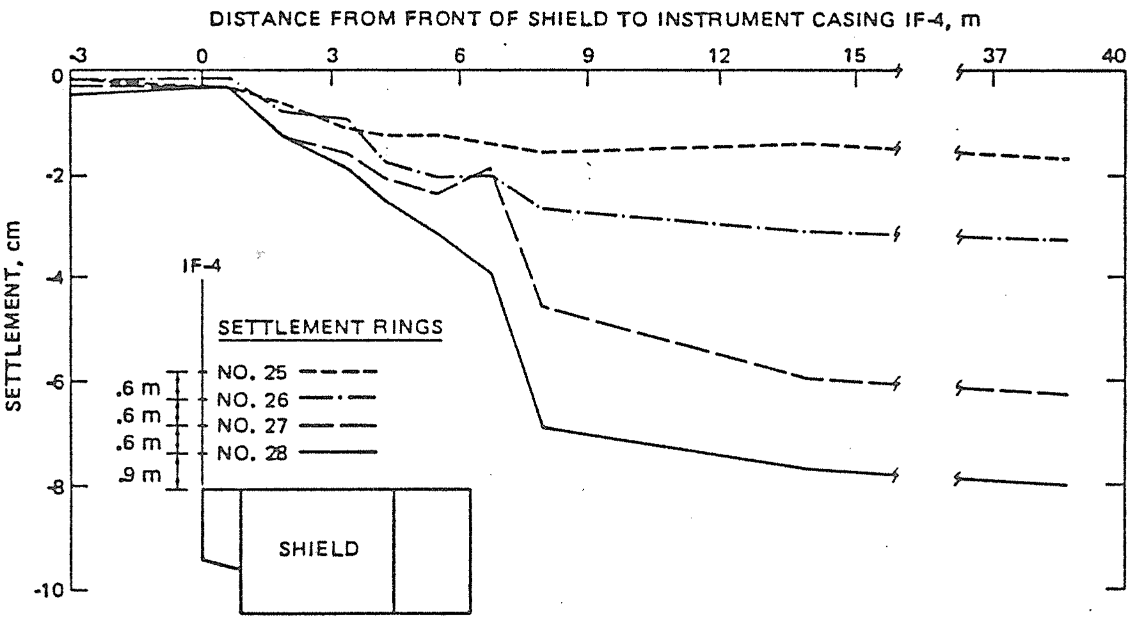

Figure 6. Subsurface Settlements Above Drift 2, F-line

Ground surface settlements derived by projecting the Drift 2 ground deformations to all 24 drifts would have resulted in 0.4 m of surface settlement. Since this was unacceptable, the WSDOT required the contractor to modify his construction methods. The instrumentation data indicated that a reduction in shield overcut thickness, improved expansion of the segments, and more thorough grouting to fill any voids around the liner and concrete backfill, would reduce ground deformations. After implementing these changes, successive drift excavations generally experienced less than 2.5 cm of ground deformation. Ground surface settlements derived by projecting 2.5 cm of settlement for each of the 24 drifts resulted in an estimated surface settlement of 9 cm, which is fairly close to the actual measured average settlement of 7.6 cm.

The deformation data from individual drifts provides useful information in evaluating ground losses. Cording and Hansmire (1975) relate ground loss volume (V_L) to displacement (d_v) and distance (y) of the measurement point above a tunnel of radius R:

$$V_L = d_v \times 2\,(R + y)$$

Calculated average ground loss per drift is 0.13 m^3/m, or approximately 2% of the drift volume which is comparable to ground losses for many tunnels in hard clay. Total cumulative ground losses calculated as a function of total excavated bore volume was 0.65%. The 0.65% ground loss correlates well with the measured 0.5% to 0.8% surface settlement trough volume indicating that only minor bulking has occurred. Thus, tight control of deformations directly above individual drifts proved to be a reasonable means for controlling surface settlements.

The inclinometer data proved less critical in assessing displacements around drifts, since horizontal displacements were generally much less than 1.3 cm adjacent to each drift. However, the data was useful in assessing total deformations around the bore and deformations adjacent to the old tunnels, as well as slopeward deformations near both access pits. Horizontal displacements toward the new bore totalled approximately 2.5 cm and occurred primarily along discrete bedding planes and slickensided surfaces (Figure 7). The horizontal displacements are relatively small when compared to cumulative vertical displacements, which range from 6.4 to 20.3 cm. Minor horizontal displacements toward the old tunnels were likely related to recompaction of disturbed soils around the old tunnels.

The inclinometers were most useful in evaluating slopeward

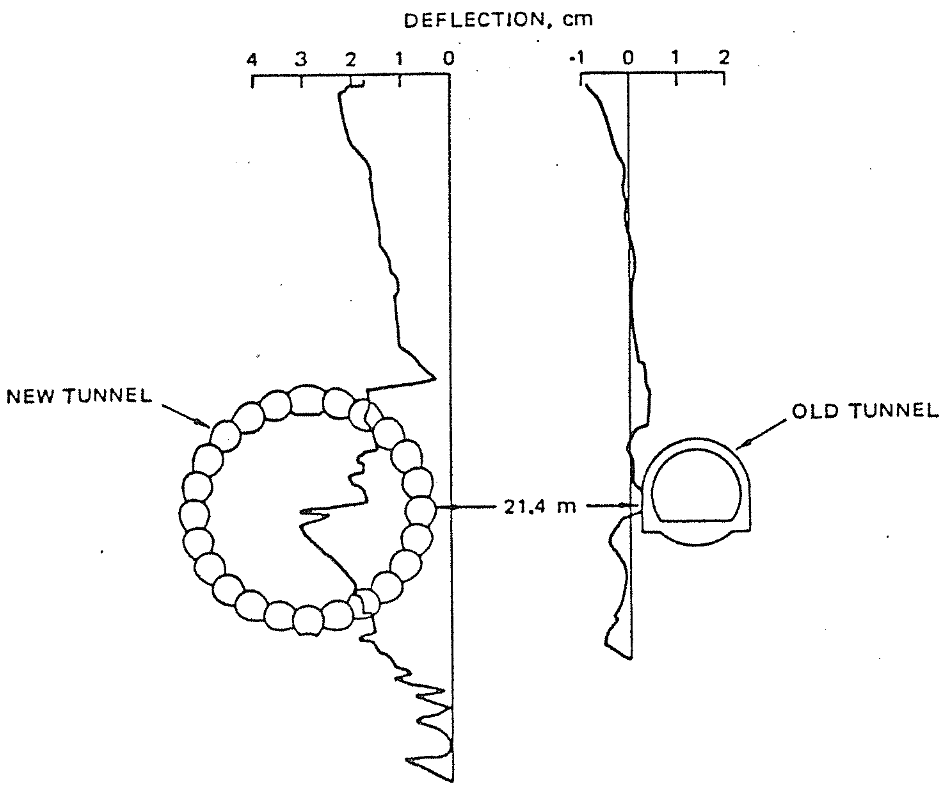

Figure 7. Total Cumulative Horizontal Displacements at the C-line

deformations adjacent to the west and east access pits. As discussed below, large settlements occurred near the west portal, likely related to disturbance of an old landslide. Large horizontal deformations of up to 5 cm also occurred near the west portal, but stopped as soon as the the compression ring liner was completed.

<u>Surface Settlements</u>. Settlements greater than 8 cm were related to remobilization of an old landslide area near the west portal. Elsewhere measured settlements (Figure 8) were about equal to predicted settlements of 5 to 10 cm determined by algebraically adding soil displacements measured around a deep 3 m diameter tunnel in London Clay (Attewell and Farmer, 1974). Transverse settlements near the center of the ridge are shown on Figure 9. Significant settlements occurred above the old tunnels due to recompaction of disturbed soils around these tunnels. Draw angles for the settlement trough range from 40° to 55° on the old tunnel side and 35° to 47° on the opposite side, indicating that the previously disturbed soils around the old tunnels had an impact on the geometry of the settlement trough. Maximum angular distortions were about 0.005, and average maximum angular distortions for all instrument lines was 0.003.

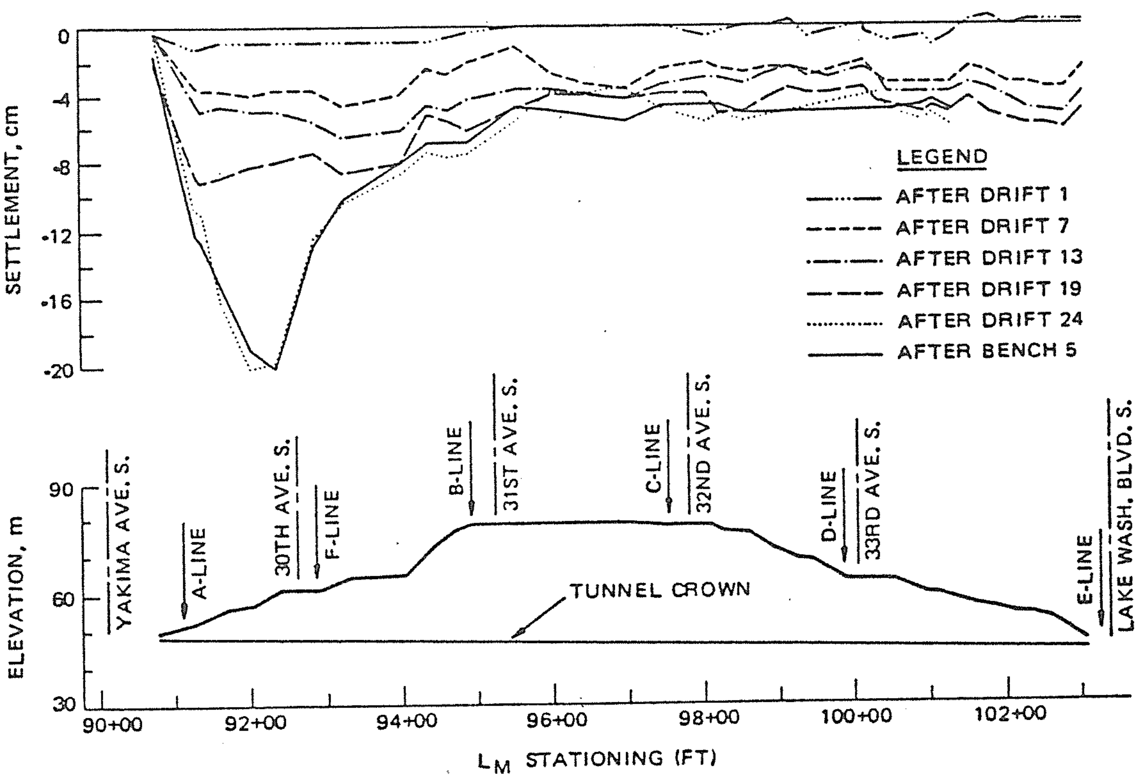

Figure 8. Longitudinal Settlements Along Tunnel Centerline

Settlement volumes ranged from about 2.6 m^3/m of tunnel at the east end, to as high 4.2 m^3/m near the western slide-prone end, i.e., from 0.5% to nearly 0.8% of the total tunnel volume. These settlement

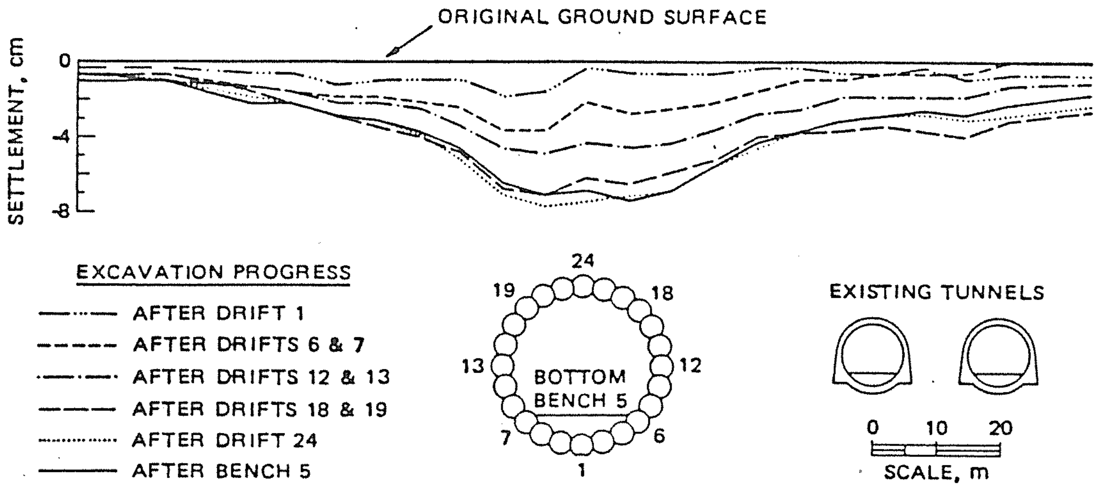

Figure 9. Transverse Settlements Near Center of the Ridge, B-line

volumes are significantly less than the typical 1% to 3% experienced by many tunnels in hard clays.

Figure 10 indicates that only minor settlements occurred through Drift 3 construction, probably due to arching of the soil loads. Following Drift 3, the soil arch apparently broke, resulting in a fairly constant rate of settlement as a function of drift construction until completion of the compression ring liner when settlements essentially stopped.

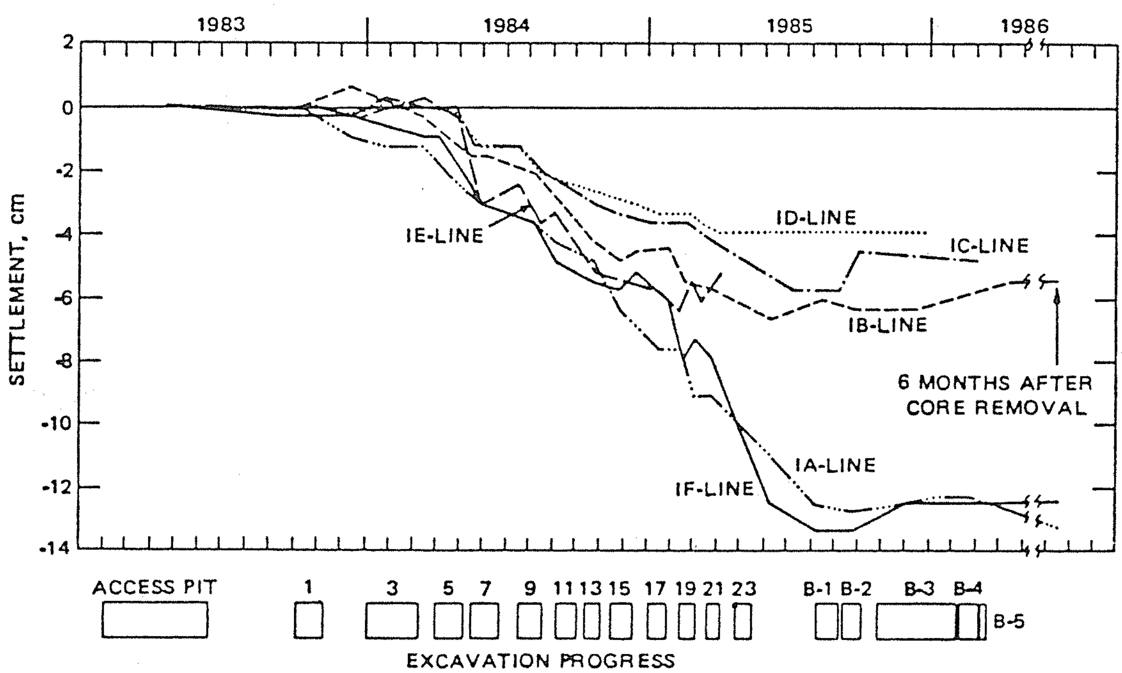

Figure 10. Settlement vs. Time Plot for Selected Points

In spite of these fairly large settlements at the west end of the tunnel, only minor damage occurred, including minor offsets of a sidewalk and minor foundation cracks in a WSDOT-owned house.

Data Evaluation for Soil Core Removal

Much of the installed instrumentation was designed to evaluate liner deformations and stresses, and surrounding soil movements during core excavation. As noted above, surface settlements and slopeward movements essentially ceased once the semi-flexible compression ring liner was in place. In fact, minor heave of 0.3 to 1 cm occurred during bench excavation which is about equal to projected heaves calculated by the FEM (Hendron, et al., 1971). Heaves are slightly less than those calculated by the FEM possibly indicating that actual modulus values are somewhat higher than assumed.

Concrete Stresses. Minor stress increases (Figure 11) occurred in the liner ring soon after stress meter installation and until completion of the ring. Significant stress increases occurred with the excavation of Benches 1 through 5. The maximum recorded stress was 8.9 MPa, considerably less than the concrete design stress of 23.4 MPa. The greatest stress buildup (Figure 12) was generally near the inside surface of the bore in the springline drifts and near the outside of the lining in the invert drifts, indicating an increase in the horizontal diameter of the liner. As noted earlier, the gages in Drift 23 functioned poorly and consequently measured loads in the tunnel crown are erroneously small.

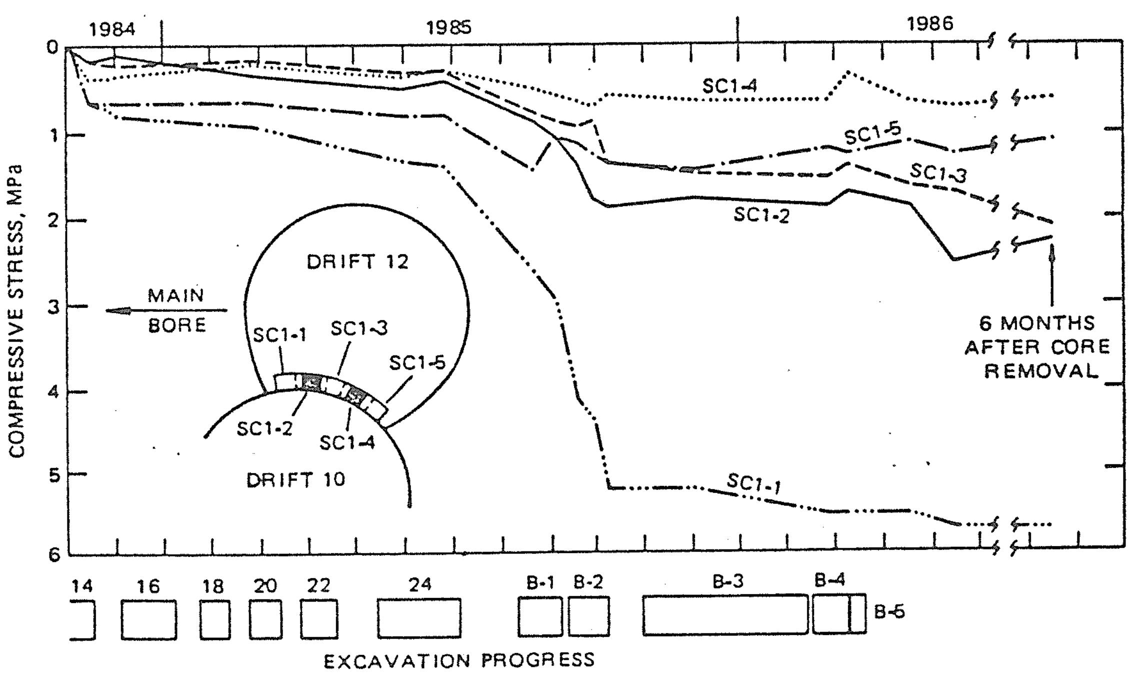

Figure 11. Measured Springline Stresses at the C-line

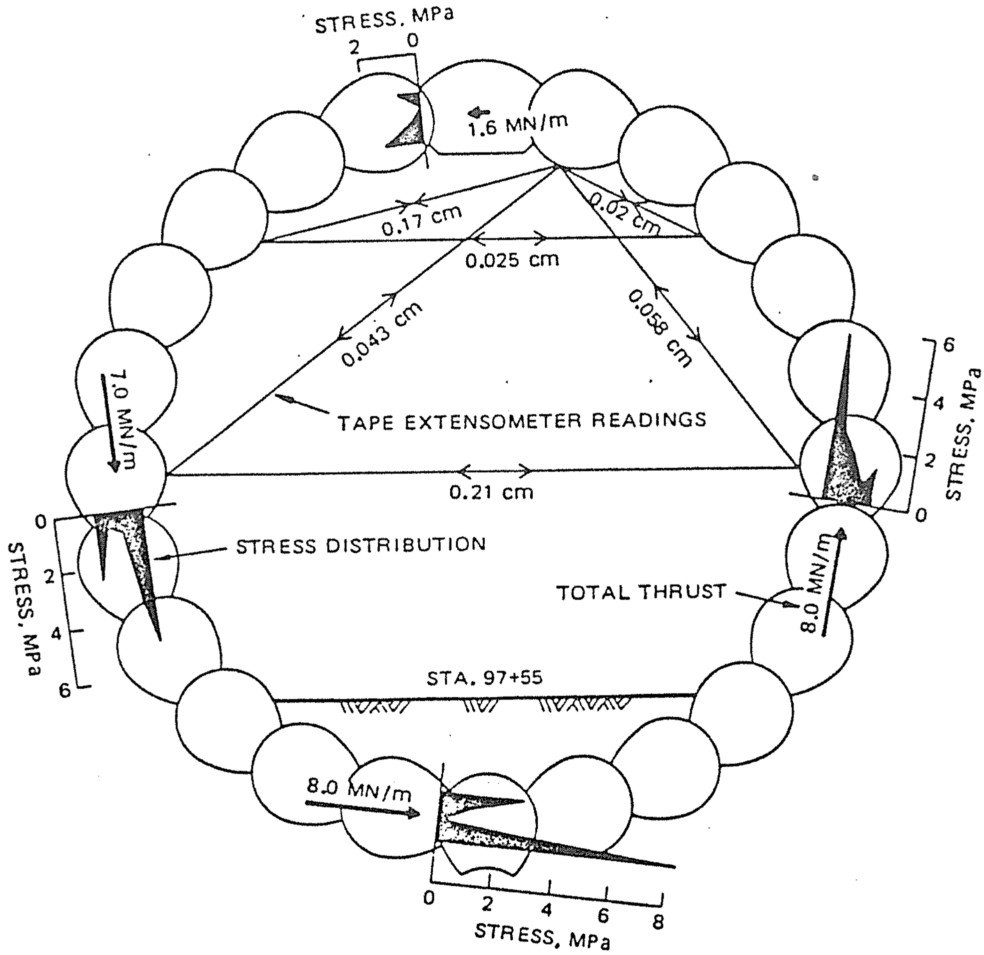

Figure 12. Liner Stresses and Deformations After Core Removal

Liner thrusts were predicted, using FEM analyses (Hendron et al., 1971) for K_0 of 0.5 and 2.0 (Figure 13). Analyses indicated that for those K_0 values there should be a wide variation between springline and invert thrusts for various depths (Z_0). We have also added the liner thrusts (T) for K_0=1, calculated as follows:

$$T = 1/2\ Z_0\ R\ (1 + K_0)$$

Measured thrusts vary by less than 10% from the calculated thrusts for K_0=1 indicating that K_0 is close to 1.0 and that the liner has accommodated most of the soil loads, as confirmed by the very minor load changes in the 6 months following core removal.

Liner Deformations. Tape extensometer and borehole extensometer data confirm the results of the concrete stress meter data and surface survey data. Tape extensometer anchors installed during core removal indicate approximately 0.2 cm or 0.01% diameter change (Figure 12). Horizontal deformations along the bore ranged from 0.08 to 0.5 cm, averaging 0.25 cm or 0.013%. The measured horizontal liner

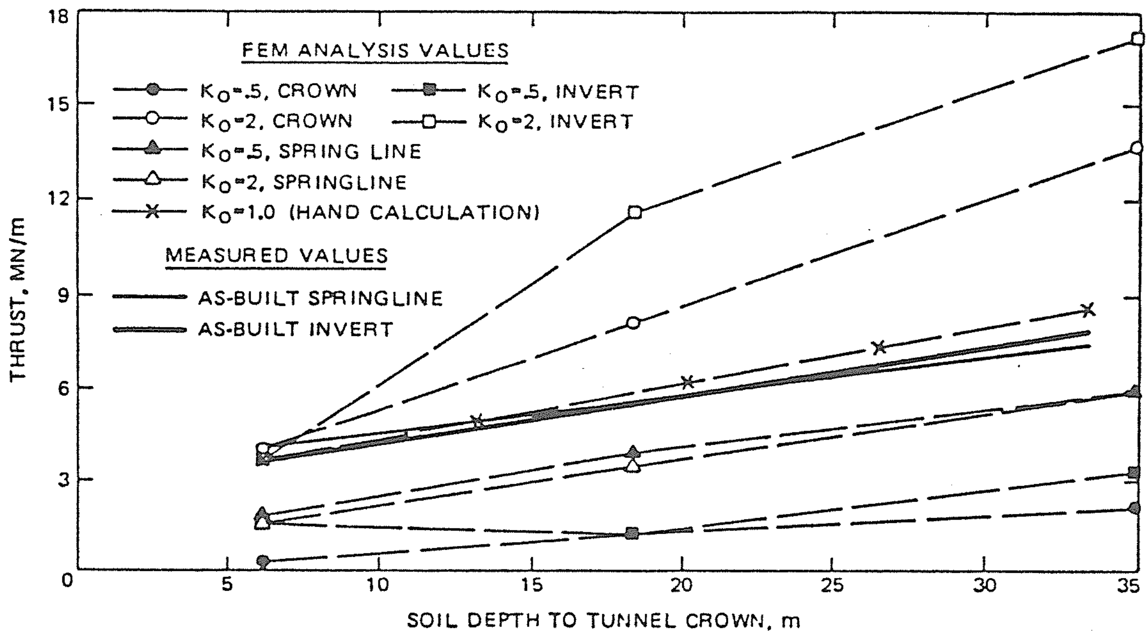

Figure 13. Measured and Design Liner Thrusts vs. Depth

deformations are extremely small when compared to FEM predicted deformations for K_0=0.5 to 2.0, and normal tunnel deformations of 0.1% to 0.5%. These small diameter deformations tend to confirm that K_0 is close to 1.0. The data also suggests that the zone of disturbed soils around the bore is relatively small and that the bore liner is in relatively intimate contact with the soil.

Borehole extensometers installed during Drift 1 excavation indicated approximately 0.5 cm of bore invert heave during core removal. Invert heaves of 1 to 3.5 cm were predicted with the FEM analyses. Since the deformations are less than anticipated, the ground mass modulus is probably greater than that used in the analyses. The small tunnel invert heave correlates well with the heave measured at ground surface during core excavation, indicating that the entire tunnel has heaved due to elastic rebound.

CONCLUSIONS

Mt. Baker Ridge Tunnel, the world's largest diameter soft-ground tunnel was successfully completed using conventionally sized tunneling equipment, while significantly reducing surface settlements over what might have occurred with a full-face tunnel. The tunnel was actually constructed at a lower cost than the bid price due to the low number of claims and lower than estimated escalation of material and labor prices.

In terms of performance, reliability, and survivability, the instrumentation program was an unqualified success. The instrumentation systems have suffered very low failure rates and continue to operate reliably after 4 years.

The successful implementation of a very comprehensive and fairly complex geotechnical instrumentation program has helped to minimize construction difficulties, damage to overlying structures, and the attendant increases in construction costs. In particular, the frequent monitoring of deformations above individual drifts permitted early warning of potentially large and disastrous deformations as well as the rational development of modifications to construction methods, thereby precluding the recurrence of similar large deformations during the excavation of subsequent drifts.

The instrumentation data has shown that this state-of-the-art tunnel lining system is performing better than predicted. Tunnel diameter changes of less than 0.013% are significantly less than the 0.1% to 0.5% associated with normal tunnel construction methods. Measured stresses in the concrete drift liner are only 1/2 to 1/3 of those predicted from the FEM parameter study. Consequently, it appears that the tunneling impacts and ground conditions are more favorable than those predicted from the investigations in the late 1960's.

The use of a stacked-drift method has proven very successful in limiting deformations near the adjacent tunnels to less than 1.3 cm and in maintaining surface settlements to generally less than 7.6 cm and consequently, eliminating any significant damage to structures and utilities.

The data should also be useful for evaluating similar designs for future large-diameter tunnels. The small ground deformations and settlements, and the favorable liner stresses all indicate that this is a viable approach for future large-diameter tunnels.