Evaluation of the Site Investigation and Construction Related Aspects of the Milwaukee's Crosstown Deep Tunnel

R. C. Ilsley
STS D'Appolonia, Ltd.

S. B. Fradkin
STS D'Appolonia, Ltd.

E. F. Shorey
CH2M HILL, Inc.

Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Paper No. 2.36

Evaluation of the Site Investigation and Construction Related Aspects of the Milwaukee Crosstown Deep Tunnel

R.C. Ilsley
Principal Engineering Geologist, STS D'Appolonia, Ltd., USA

S.B. Fradkin
Project Geologist, STS D'Appolonia, Ltd., USA

E.F. Shorey
Senior Engineering Geologist, CH2M Hill, Inc., USA

SYNOPSIS: The excavation of the 21,300 foot long, 32.33 foot diameter Crosstown Phase I Tunnel was completed by tunnel boring machine at an average rate of 108 feet per 3 shift day. No geologic conditions were encountered which were sufficiently adverse to impede the work. The geotechnical report combined with contract provisions prevented the unusual conditions encountered from becoming issues that delayed the work. It is probable that the monetary aspects related to these unusual conditions can be equitably resolved using the geotechnical data "base line" portrayed in the geotechnical report.

INTRODUCTION

The 21,300 foot long, 32.33 foot bored diameter Crosstown Phase I Interceptor is required for sewage storage and transport as part of the $2.2 billion Milwaukee Water Pollution Abatement Program (MWPAP). The tunnel with two 25 foot diameter construction shafts is located in Silurian Age dolomites of the Niagara Group. The tunnel was excavated down a 0.1 percent gradient with a Robbins tunnel boring machine. The excavation began in October 1985 and was completed in September 1986, the only significant delay being a 4 week period when a main bearing was replaced (Santacroce 1987). The work was done on a 3 shift 5 days a week basis.

The tunnel alignment is primarily beneath the Menomonee River Valley (Figure 1), which is also a pre-glacial valley, thereby resulting in up to 220 feet of glacial and post-glacial sediments with the water table level at about 5 feet below the existing ground surface, except at the west end of the alignment. Here the alignment moves out of the pre-glacial valley with correspondingly thinner soil cover resulting. The depth of the tunnel ranges between 275 and 345 feet. The bedrock cover over the tunnel crown is typically about 100 feet, but ranges from a minimum thickness of 18 feet to about 200 feet. The minimum thickness occurs in a depression of the bedrock surface within the pre-glacial valley as described above. The depressed area has been termed the "bedrock valley area". A series of 15 foot long probe holes into the tunnel crown in this area during construction did not encounter the bedrock surface.

EXPLORATION PLAN

The exploration was done in three phases. These phases may be broadly described as the study area exploration, corridor exploration and the potential problem or geological hazard exploration. Borehole spacing ranged from 200 feet centers in the potential problem areas to a maximum of 2,500 feet along the alignment.

The geotechnical report relied upon the vertical boreholes to recover core for logging, water pressure testing and for placement of piezometers. In addition, local tunnels (within 10 miles) and quarries were mapped and three pump tests performed. A laboratory test program to characterize rock material properties, including direct shear tests of discontinuities was performed. This information was combined to describe the rock mass characteristics. Angled boreholes were considered but given the perceived technical difficulties were not used. Also, the use of geophysical methods for fault location was not used because the potential offsets in formation boundaries were considered insignificant.

ROCK CONDITIONS

Lithology

The tunnel is in dolomite and is located primarily within the upper part of the Mayville Formation and the Waukesha Formation, mostly the latter (Figure 2). Along its length the tunnel penetrates almost the full thickness of the Waukesha Formation. The principal features used to subdivided the formations were: the presence of chert; zones of shale partings; and the presence and size of vugs. The lithologic descriptions of the formations developed from the geologic mapping of the excavated tunnel are consistent with the descriptions provided in the geotechnical report included with the contract documents. There are no differences of note. In addition, the location of the formation contacts encountered in the tunnel conformed closely to those predicted in the geotechnical report.

Rock Structure

Bedding - The bedding is defined by horizontal shale layers that range from very thin partings to layers about 2 inches in thickness. The thickness is generally variable along a single bedding plane. Ninety eight percent of the bedding planes logged from the core indicated shale fillings with an average thickness of approximately 0.05 inches and clay typically

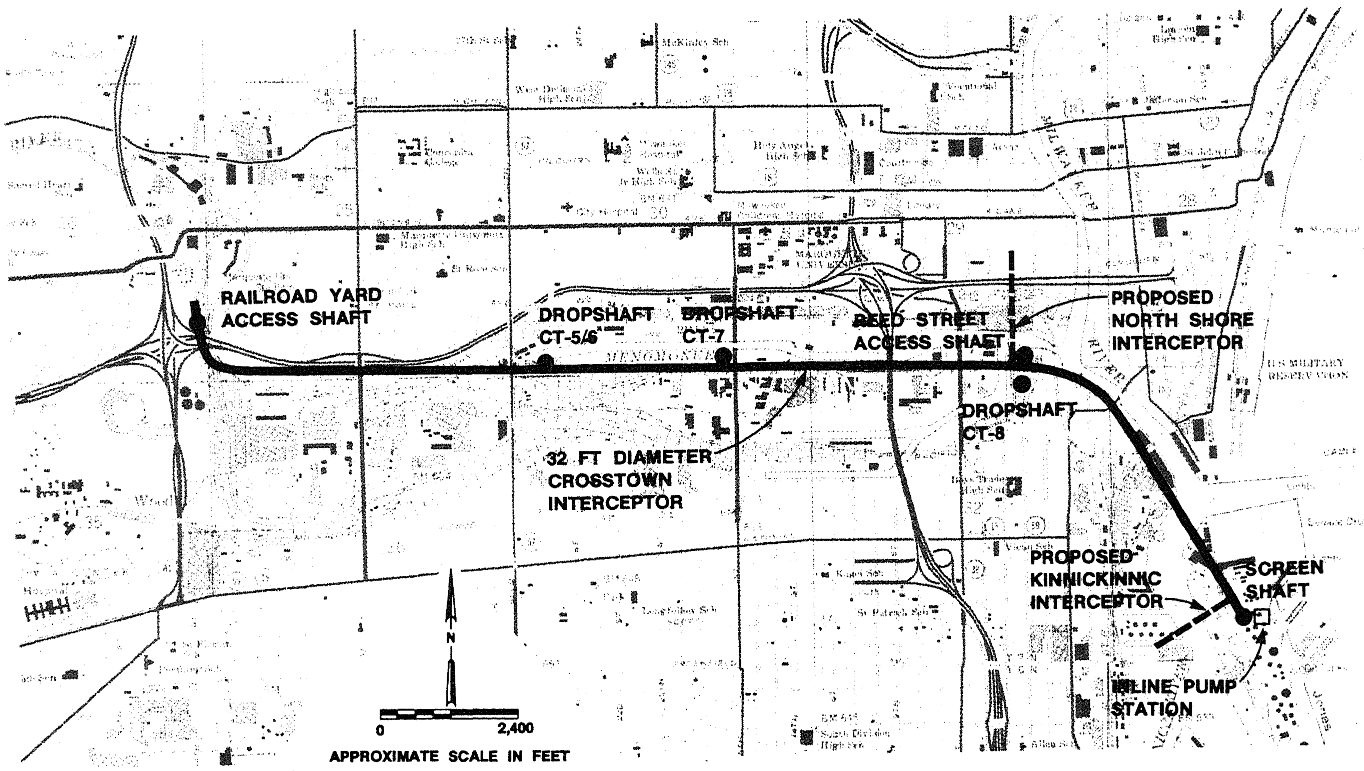

Figure 1. Vicinity Map: Crosstown Interceptor and Shafts.

0.25 inches. The gradient of the tunnel with respect to the dip of the beds was such that it cut downwards through the bedding planes at a very slight angle. Because of the circular tunnel shape, the limited slaking effects on the bedding plane fillings and the placement of rock reinforcement behind the TBM cutter head, no significant failure of thin bedding slabs was experienced, either short or long term.

Joints - For the site investigation, mapping of area tunnels and quarries during the exploration phase provided the orientation data for the joints (Ilsley '83). Logging of rock core provided the joint characteristics on a site specific basis. The principal part of the mapping effort was the detailed engineering geological mapping of an 8 foot diameter tunnel (excavated by TBM) in the Waukesha Formation, located about 10 miles from the project site. Determination of the preferred joint orientations in the Milwaukee area were made by plotting this data on stereograms. Joint spacing data were compiled from the tunnel maps and presented in tabular form in the geotechnical reports (Ilsley '84).

The geologic conditions in the excavated Crosstown Phase I tunnel were recorded on graphic and descriptive logging forms by engineering geologists.

The following general conclusions are based upon a comparison of the geotechnical report data and that from the geological mapping.

- The majority of joints are steeply dipping and belong to two major sets, one striking NW-SE and one striking NE-SW as predicted by the geotechnical report.
- The predominant joint set is the NW set; the table of joint spacings included in the geotechnical report inferred this was the case but not to the degree actually found.
- About 70 percent of the joints logged are either clay or shale filled. This is about three times more than indicated in the geotechnical report.

The most probable reason that the indicated percentage of clay or shale filled joints was greater was because the average joint aperture logged in the core was very small (in the range of 0.01 inch to 0.25 inch), whereas in the tunnel the significant joints logged had an average thickness of 1.25 inch. We would therefore recommend that when logging core for such facilities only discontinuities with an aperture greater than 0.10 inch be logged.

True spacings (measured perpendicular to joint strike) of the joints are tabulated for each joint set below for the whole length of tunnel.

TRUE JOINT SPACINGS

Joint Set(ft)	Minimum (ft.)	Maximum (ft.)	Median (ft,)	No. of Joints
NW	1 (6)	370 (>100)	34 (24)	198
NE	1 (9)	3,200 (>100)	50 (75)	105

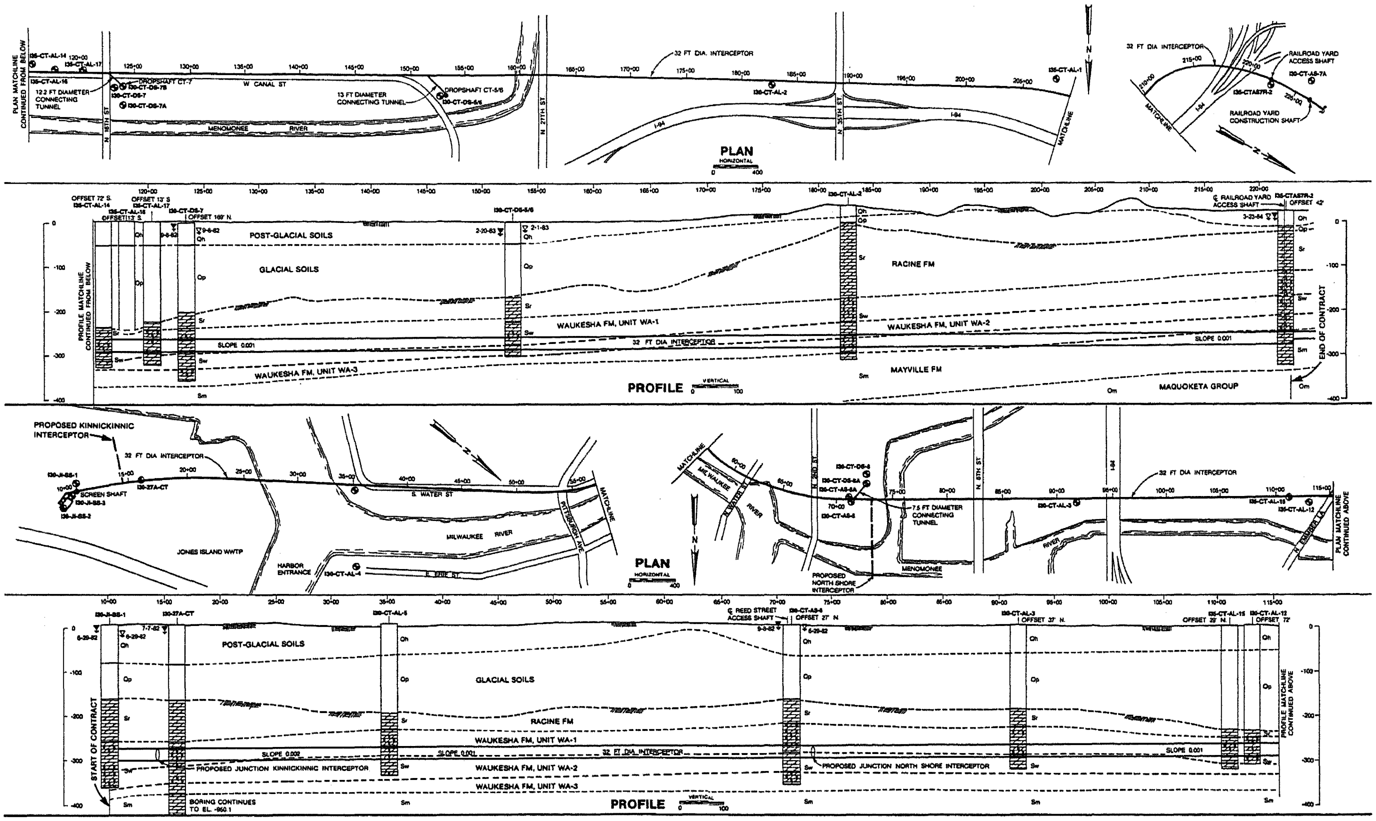

Figure 2. Plan and Geologic Profile.

The values in parentheses in the above table are those presented in the geotechnical report. The data presented in the table indicate that the observed joint spacings compare favorably with the values presented in the geotechnical report.

However, in the section of tunnel beneath the "bedrock valley" (station 108+00 to 121+00) the joint spacings were less than those presented in the geotechnical report.

Faults - The geotechnical report describes the type of fault that may be intersected as having a true width of 5 feet or less and having gouge or crushed rock zones. Also indicated is a possibility of encountering one or more faults of this size per mile of tunnel.

Most of the 126 faults mapped are features with relatively narrow width that are similar in character to the observed joints in the tunnel. About 80 percent of the observed faults have widths less than 0.1 foot; only about 5 percent have widths greater than 0.5 foot. In most cases the rock adjacent to the fault is undisturbed. About 80 percent of the observed faults have clay and/or shale fillings, occasionally with crushed rock (breccia) or secondary mineral deposition. The remaining faults typically have fillings of crushed or sheared rock, or mineral deposits.

All of the wider faults occur in the "bedrock valley" area between station 117+50 and 120+50. Five (5) faults occur in this section. They have maximum widths of 1 to 4 feet, but in all cases they narrow to about 1 inch or less in the tunnel crown. These features have apparent vertical displacements of 1 to 3 feet, but the total apparent vertical displacement across the zone is only about 2 feet. As the rock was continuous in the crown, no delays were experienced by the TBM.

Steel ribs and lagging were used to construct forms at the tunnel surface and across the faults which were then filled with grout.

Rock Strength

The average unconfined compressive strength for the Waukesha Formation, was 26,640 psi with a range of 6,750 to 45,100 psi. Unconfined compression tests on samples of weathered rock from boreholes within the "bedrock valley" area (station 108+00 to 121+00) gave values of 5,429 and 6,709 psi. The tunnel maps contain general qualitative descriptions of the rock fabric, including references to solutioning and weathering from station 110+50 to 119+50. This was the only tunnel section where the rock was weaker, however, the rate of TBM advance was somewhat greater than the average. It appears that the presence of chert did not affect the rate of TBM penetration or the disc-cutter replacement rate.

Initial Support

A 5 by 5 foot pattern of dowels with 9 dowels per station was required and paid for in the contract. This pattern of dowels functioned as both the minimum initial support and minimum final support. The dowels are 1.125 inch diameter rebar, encapsulated with polyester resin. The top five bolts are 10 feet long and the remainder 12 feet. The dowels were closed up to a 3 feet longitudinal spacing in the "bedrock valley" area where the rock cover was known to be thin. Other occasional spot reinforcement with dowels was used, sometimes with steel straps or wire mesh. These additional dowels were also paid for under the contract and numbered some 45,000 lineal feet (about 10 percent more than required for the 5 by 5 foot pattern). Steel straps and wire mesh were not paid for.

The top 5 dowels were placed directly behind the cutter head through slots cut in the trailing partial steel shield. The remaining 4 dowels were placed at a point some 75 feet behind the cutter head.

Final Support

The contract required about 700 feet of reinforced concrete lining at junctions with shafts and connecting tunnels. There was also a provision for 3,500 feet of lining to be placed at the engineer's direction. The evaluation of data collected during the tunnel mapping was the basis of determination of which parts of the tunnel should be lined. The general criteria used were:

- Greater than normal frequency of jointing.
- Increased number and/or thickness of clay, shale and/or gypsum fillings in joints and bedding planes.
- Juxtaposition of joints from different sets that may delineate wedges, blocks or slabs with potential for fall out.
- The presence of weathered zones which could be eroded or deteriorate with time.

In general, lining was recommended for tunnel sections in which joint and bedding characteristics, in terms of the criteria listed above, combine in a manner that gives potential for deterioration of the rock mass over time. Such deterioration is expected to be limited to minor spalling or raveling of the rock surface and to washout or solutioning of thicker joint fillings which could lead to new sources of groundwater inflow. The potential for washout and/or solutioning of isolated joints is recognized by specifying "dental treatment" for such features. This involves removing the joint filling to a depth proportional to the width and backfilling with shotcrete. A total of about 5,500 feet of engineer directed lining has been specified, some 25 percent of the total tunnel length.

GROUNDWATER CONDITIONS

Introduction

The dolomite bedrock through which the tunnel was excavated is a semi-confined aquifer. The underlying Maquoketa Group is a leaky aquitard and the overlying glacial soils provide recharge. Recharge also occurs to the west of Milwaukee where the dolomite outcrops or is close to the surface. The aquifer characteristics were estimated during the exploration phase by water pressure testing in the boreholes and three pump tests at different locations. The pump tests yielded flows of 34, 150, and 508 gpm and transmissivities of 470, 1300 and 5000 gpd/ft. The permeability coefficients measured ranged from $>1 \times 10^{-3}$ cm/sec to $< 1 \times 10^{-7}$ cm/sec with most values less than 1×10^{-5} cm/sec. Piezometric levels of the dolomite are generally within 10 to 20 feet of the ground surface. However, there is a 2 mile diameter cone of depression centered on the Red Star Yeast high capacity well which is offset about 600 feet to the north of station 165+00.

The soils aquifer along the tunnel alignment within the Menomonee River Valley, ranges from 150 to 220 feet thick. The soil stratigraphy is generally as follows; glacial soils (usually till) on bedrock, overlain by glacial lacustrine clays, outwash, alluvial soils, estuarine (marsh) deposits and fill. Observation wells open in the upper 30 feet or so show a water table generally about 5 feet below ground level. Piezometers placed in the alluvial zone indicate a semi-confined aquifer with piezometric levels a few feet higher than those placed at the soil/rock interface or within the bedrock.

It was recognized that dewatering of the alluvial layer might induce significant consolidation related settlements of the overlying estuarine deposits. Hence no dewatering at the Reed Street Access construction shaft was permitted. The possibility of connection between the dolomite bedrock and the upper alluvial aquifer was judged to be minimal and probably balanced by recharge from the river. However, provision was made in the contract to place a series of piezometers directly over the tunnel alignment, sensing the alluvial soil layer.

Water Inflows to the Tunnel

<u>Sources of Inflow</u> - The geologic mapping indicates that most inflows to the tunnel are from joints. Bedding planes are a minor inflow source. Also, some inflows are from zones of interconnected vugs.

Roughly one half of the northwest joints and one third of the northeast joints produced flowing water. Another approximate 15 percent of the joints in each set produced small amounts of water inflow that were described as dripping. Significant groundwater inflows into the tunnel did not begin until mid-March 1986 (Figure 3) at which time the TBM entered the "bedrock valley" area at about station 121+00.

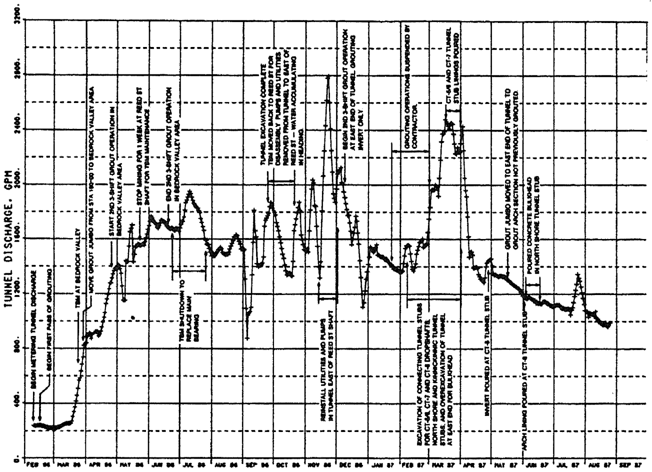

Figure 3. Estimated Inflows to Tunnel.

The next 1300 feet yielded total inflows of about 700 gpm and further tunnel excavation, raised this to a maximum of about 1,800 gpm. The geotechnical report indicated a total flow of about 1,500 gpm for the tunnel whereas the actual ungrouted flow (estimated from tunnel maps) was 2,300 gpm.

The "bedrock valley" area initially was, and remains, one of the wettest tunnel sections (Figure 4). The zone of wider faults (see "Joints and Faults") between station 117+50 and 120+50, the western end of the "bedrock valley" area, combined with the section from station 115+00 to 117+50, which includes four narrower faults, account for somewhat more than one third of the groundwater inflow estimated for the "bedrock valley" area.

Measurement of Inflow - Groundwater inflows have been indirectly measured using the amount of water being discharged by the contractor's pumping system (corrected for the amount of service water introduced) as an estimate of the amount of groundwater flowing into the tunnel. A plot of metered tunnel discharge with time is provided in Figure 3. The plot represents a 7-day term moving average through the data; i.e., the data point plotted for each day is the average of the discharge for that day plus the three preceding and three subsequent days.

Figure 3 contains a number of annotations of events in the tunnel which have likely affected the amount of water being discharged from the tunnel at specific points in time.

A review of Figure 3 indicates that the overall trend of the water inflows to the tunnel has been, and continues to be, downward from the peak inflows of June 1986. The current (late August 1987) water inflow rate of just less than 1,000 gpm is slightly more than half of the peak inflow rate of June 1986, and is less than half (about 45 percent) of the estimated initial ungrouted inflow rate for the whole tunnel of approximately 2,300 gpm.

Inflow monitoring within the tunnel has been conducted over nearly 80 percent of the tunnel length. Measurements were made between 20 January 1987 and 10 April 1987. The results are summarized in Table 1, and are compared graphically to the estimated initial inflows (based on the geologic mapping) in Figure 4.

A review of Figure 4 indicates that for a number of tunnel sections the groundwater inflows to the tunnel have been reduced significantly as a result of cementitious grouting. In a number of tunnel sections, significant inflow reductions have been achieved with one pass of grouting. However, for a number of sections in the west half of the tunnel, one pass of grouting has produced marginal results. An as yet unknown amount of grouting remains to be done in the invert in the western half of the tunnel (see "Grouting"), and it is expected that this additional grouting will result in further inflow reductions.

For a few sections, however, the dye dilution measurements appear to indicate that inflows have increased from the initial rates (Figure 4). The probable reason for these apparent increases is that the initial estimates were visual and were probably low. In general, these sections have relatively small inflows.

Contractual Provisions

Pumping of Inflows - Items in the contract bid provided graduated payments for pumping of the groundwater inflows beginning when flows exceeded 1,500,000 gpd. Measurement of these pumped flows by meters must take into account the water added by the contractor's operations. Also, the meters should be calibrated using a method such as the dye dilution method of flow measurement.

Grouting - Because the design aimed at minimizing the amount of tunnel to be lined, the control of groundwater by grouting was specified to be at the direction of the engineer. Bid items were provided for drilling grout holes, water pressure testing the grout hole and connecting the grout pump to the drill hole (connections), sacks of cement and cubic feet of grout placed. At the end of August, 1987 essentially two tunnel passes of cement grouting have been done. The quantities used are shown in Table 2. Although the contract has provision for chemical grouting (silica), none has been done at this time. The grouting plan was essentially feature grouting beginning with mixes based upon initial single water pressure test takes of the grout hole. Increases in grouting pressures signaled the use of progressively thicker mixes. The starting grout mix ratio was usually 10 to 1, water to cement by weight (Coon 1987) with 2 percent by weight of bentonite.

Piezometric Levels

Piezometric levels in soil and rock aquifers were drawn down as a result of groundwater inflows into the Crosstown Phase I tunnel and into Crosstown dropshaft excavations. The significant drawdowns occurred as the tunnel excavation passed into and through the "bedrock valley" area.

Numberous piezometers are located along the tunnel alignment. These piezometers have

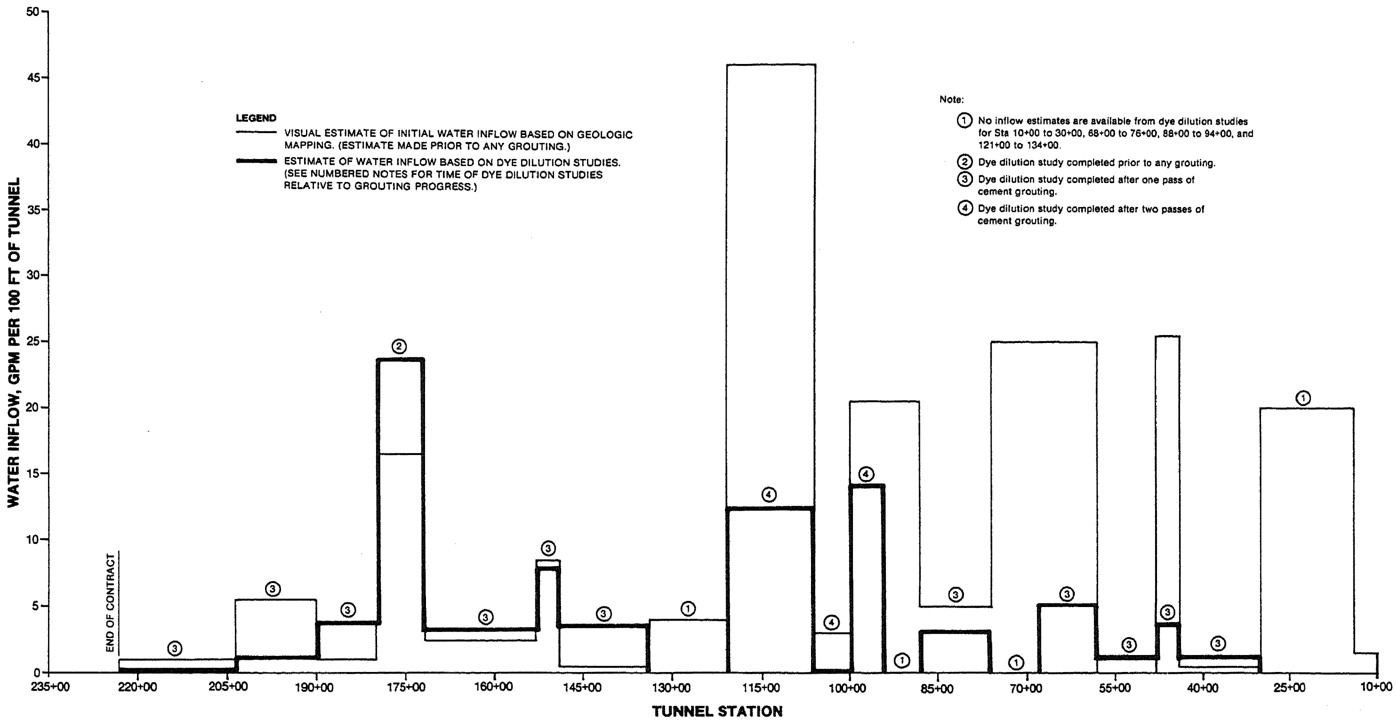

Figure 4. Water Inflows by Tunnel Section.

sensing zones in the alluvial, lacustrine and dolomite aquifers which underlie the estuarine deposits in the lower Menomonee River Valley. Piezometers with sensing zones within or above the estuarine deposits generally sense the water table aquifer; which was not drawn down. The measured piezometric levels in the alluvial soil aquifer are recovering from the low levels recorded in 1986. The recoveries as a percentage of the maximum drawdown are in the range of 20 to 40 percent. The measured piezometric levels in the dolomite aquifer are also recovering from the low levels recorded in 1986. Recoveries as a percentage of the maximum drawdown are in the range of 50 to 60 percent.

Table 1. Measured Water Inflows.
Note: W and D denote measurement by weir and dye dilution.

Stations	Test Method	Inflow (GPM)	Test Date
219+85 to 174+00	D	239	4-10-87
174+00 to 153+25	D	64	4-9-87
157+25 to 148+00	W	65	3-2-87
146+50 to 129+00	D	193	4-2-87
123+00 to 110+00	W	300	3-6-87
120+50 to 97+75	D	214	4-8-87
98+75 to 94+00	D	62	2-20-87
88+00 to 78+00	D	25	2-20-87
67+75 to 27+75	D	110	1-20-87

Table 2. Grout Quantities.
Notes: (1)Grouting suspended by contractor.
(2)Not a bid quantity.

Month	Station	No. of Holes	Lineal Ft. of Drilling	Bags of Cement	Cu. Ft. Grout
February, 1986	211+45 to 201+04	74	778	206	814
March	200+50 to 180+55	85	1,002	278	1,168
April	121+00 to 117+40	117	2,557	972	2,558
May	121+00 to 108+00	202	8,302	5,357	17,711
June	119+00 to 93+00	122	4,407	4,424	24,437
July	99+35 to 73+55	190	5,501	3,666	17,268
August	70+00 to 58+50	185	6,645	6,234	19,651
September	66+00 to 24+50	169	5,743	2,932	11,750
October	93+57 to 58+50	170	6,867	5,148	24,186
November	89+95 to 98+34	155	7,094	2,560	15,392
December	98+00 to 108+85 10+85 to 18+50	122	4,251	2,042	9,961
January, 1987	125+00 to 174+00 18+65 to 25+50	155	5,242	4,191	17,430
February 1)	—	—	—	—	—
March	149+00 to 157+50 108+00 to 111+00	189	7,877	3,963	14,840
April	111+00 to 120+00 173+00 to 180+00	279	10,224	3,183	10,592
May	10+30 to 17+00	57	2,506	2,410	7,445
June	17+00 to 24+00	180	8,093	5,551	21,019
July	24+00 to 29+00 45+00 to 60+00	163	7,554	3,131	12,768
August	60+00 to 70+00	205	9,177	2,310	13,464
Totals through 28 August 1987		**2,819**	**103,820**	**58,558**	**242,454**
Bid Quantities		2)	200,000	20,000	250,000

SUMMARY

A most successful aspect of the geotechnical exploration was the planned phasing of the work into the study area, corridor and problem or geological hazard exploration. This allowed the investigation to focus on the areas requiring further study and provided current geotechnical information for the facilities planning and design.

The spacing of exploratory boreholes along the alignment (maximum of about 2,500 feet) was adequate. The recognition of the potential for a "bedrock valley" came from topographic bedrock surface maps drawn using data from the first two phases. This prompted a final phase (problem area borings) of exploration which defined the extent of the depression, the increased joint frequency and the presence of weathering. However, no faults were intersected and none specifically inferred in this area because there were no apparent displacements across well defined formation contacts. The depression in the top of rock in itself indicated the probability of preferred erosion but without other corroborating factors it was judged that speculation on the presence of faults in this section was not appropriate. As it turned out, the thinning of the faults to joint dimensions through the crown enabled the TBM to continue excavation without delay. The volumes of water encountered in the "bedrock valley" were not predicted specifically in the geotechnical report. However, the contract provisions for pumping of this water and for grouting it off have prevented the inflows from being an issue.

The significant drawdown in the piezometric surface of the dolomite was related to the large inflows from the "bedrock valley" which in turn induced large drawdowns in the alluvial layer. This connection was unexpected as was the areal extent of the effect (about 2 miles in total length). Four recharge wells were setup and operated with some success in equalizing the levels. At the same time grouting efforts were doubled in the "bedrock valley" area. Recovery is underway in both the alluvial and dolomite aquifers. An optical survey of settlement points set up on sensitive structures within the affected area show settlements thus far to be generally less than 0.30 inch.

In hindsight, it would appear that having a shotcrete option for final lining support of the tunnel arch would have given more flexibility in choice of permanent support types. This would have been an intermediate choice between the pattern of dowels and a full circle, unreinforced concrete liner. Also, the specified nine bolt pattern could have safely been reduced to five, with the remaining four as optional depending upon the encountered rock conditions.

In conclusion it may be said that the geotechnical report was generally accurate when predicted and encountered conditions are compared. The phased approach to the exploration worked well. The important areas of initial support and groundwater inflows did not become issues because of provisions made elsewhere in the contract documents as previously described.

REFERENCES

Coon R.F., Shorey E.F., and Fradkin S.B., "Grouting for Groundwater Control, Milwaukee Inline Project" (1987), RETC Proceedings. Vol. 1 p. 435, New Orleans.

Ilsley R.C. and Costello M.J., "Discontinuity Characterization for Underground Openings in the Milwaukee Water Pollution Abatement Program" (1983), Underground Space, Vol. 7 pp. 214-220.

Ilsley R.C., Fradkin S.B. and McBee J.J., "Characterization of Rock Conditions for the Deep Tunnel Project in Milwaukee" (1984), Proceedings of 25th Rock Mechanics Symposium.

Santacroce P.U. and Meinholz J.H., "Tunnel Boring Machine Experience in the Milwaukee Program" (1987) RETC Proceedings, Vol. 2 p. 684.