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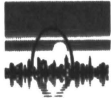


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SYNOPSIS Construction of the US22/SR7 interchange in Steubenville, OH resulted in the need to excavate the toe of the steep 350 ft. high slope overlooking the Ohio River. To maintain the stability of the slope, the Ohio Department of Transportation (ODOT) chose to construct a 4 tier, 130-ft. high, 2,200 ft. long tieback anchor retaining wall.

During the design phase, it became apparent that reductions in both the tieback loading and cost could be realized by lowering the groundwater levels in the hillside. A Pressure Relief Tunnel System (PRTS) was selected from several drainage options.

The PRTS consists of a 1,945 ft. long tunnel, excavated parallel to, and 200 ft. behind, the retaining wall, and a series 85 ft. long, sub-vertical drainage holes drilled upward from inside the tunnel.

This paper presents the design of the PRTS, an outline of the instrumentation program and a comparison of the observed and expected drawdowns.

INTRODUCTION

Construction of the US22/SR7 interchange in Steubenville, Ohio has recently been completed. The interchange project involved the construction of seven tie-back walls, three bridges, a tunnel and approximately four miles of roadway, all along 1.3 miles of centerline distance. The purpose is to connect with the new cable-stay bridge for US22 over the Ohio River and to divert the traffic on US22 around the City of Steubenville.

To construct the on-off ramp for the new bridge, it was necessary to excavate the bottom third of a steep 350 ft. high slope. A tie-back wall was chosen to support the excavation and to preserve Steubenville College which is at the top of the slope above the wall. This tie-back wall, called Wall No. 5, is one of the largest in the world. It is 130 ft. high and 2800 ft. long, and has 2140 tie-back anchors, with capacities up to 245 kips and lengths up to 150 ft. The layout of Wall No. 5 is shown in Figure 1.

GEOLOGIC SETTING

The project is in the Appalachian Plateau physiographic province, where the geologic sequence has thick, flat-lying beds of Pennsylvanian-age sandstones and shales with minor coal seams of the Connemaugh Series. The rocks have experienced little in the way of stress, even though they are about 300 million years old, so they have only infrequent fault and joint structure.

Natural slopes overlooking the Ohio River in the project area are steep, with natural slope angles of up to 45°. There is very little soil cover on the slopes. The more resistant, sandstone beds are exposed but the argillaceous units slake and are covered with a mantle of weathered chips of rock which can sustain tree growth.

A geologic section through the site at Wall No. 5 is shown in Figure 2. The sandstone units are generally strong and massive, while the shales vary from strong, massive, gray shales to weak red shales with some irregular slickensided surfaces. The coal units are thin and are associated with a variable underclay layers. A weak coal, red shale and underclay unit close to the base of Wall No. 5 is the most dominant factor in the design of the stabilization for the wall.

The rock structure at the site is the horizontal bedding and three sub-vertical joint sets. The most significant of these three is the stress relief joint set that strikes parallel to the Ohio River and dips at 70 degrees to 80 degrees towards the river.

Groundwater exits the face of the slope at the base of each of the sandstone units. The lower permeability shales and underclays act as aquitards. The water levels in piezometers installed in exploratory coreholes confirmed that there are at least two perched water tables in the stratigraphic section. The base water table is connected to the Ohio River.

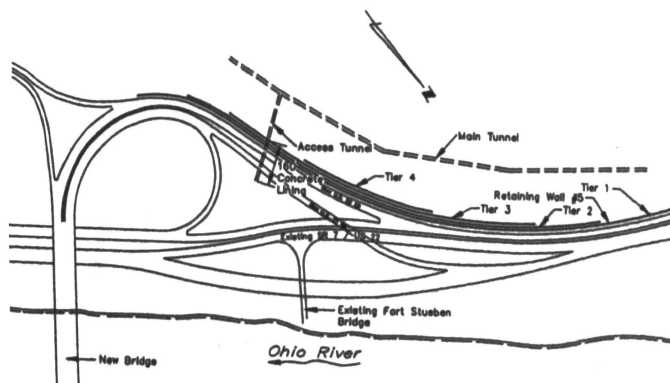


Fig. 1 Plan of Tieback Wall No. 5

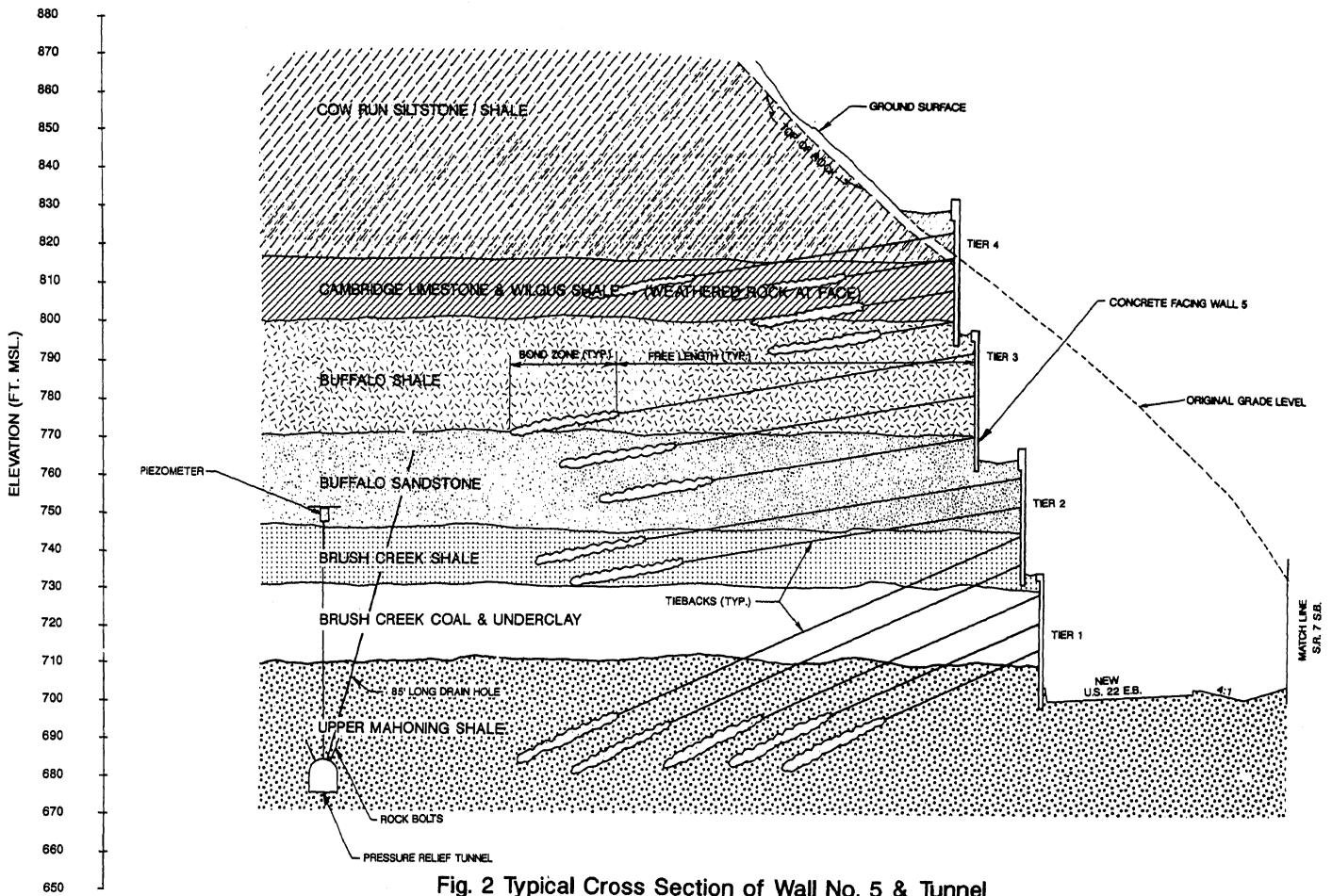
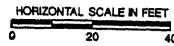


Fig. 2 Typical Cross Section of Wall No. 5 & Tunnel



GEOTECHNICAL ANALYSIS OF TIEBACK WALL

To analyze the stability of the wall and slope, a number of potential sliding mechanisms were examined using rock mass and joint strength parameters derived from laboratory testing of rock core, geotechnical mapping of rock exposures, and back analyses. The most feasible and critical potential mechanism is block sliding, where sliding would occur in the weak coal, underclay and red shale unit near the base of the wall with release surfaces provided by the stress relief joints. The stress relief joints were observed to be ubiquitous at the site, so the sliding block could be of any size. Other types of block sliding, as well as circular and curvilinear failure surfaces are not supported by the available geologic data, and when analyzed, they gave less critical factors of safety.

Loading on a typical sliding block for the limit equilibrium analyses is shown in Figure 3. The stability analyses focussed on identifying the amount of tie-back force that is required to prevent each block from sliding out of the hillside. The design tie-back force was calculated for each size of block to give factors of safety of 1.5 under long-term static conditions and 1.0 under earthquake conditions.

The most sensitive parameters in the analyses were found to be the hydrostatic loading from the groundwater and the shear strength of the weak zone at the base of the block.

As mentioned earlier, the shear strength of the weak zone was derived from laboratory tests and back analyses. Hydrostatic loadings on the release surface and the sliding surface were calculated from the piezometric pressures measured in the exploratory boreholes. The analyses clearly showed that the design tie-back requirements could be reduced by approximately one half if the base groundwater table could be lowered by the amount shown on Figure 3. This reduction in tie-back force represented a potential saving in tie-back anchors of approximately \$9 million. The surest and most effective method of lowering the groundwater level was by constructing a pressure relief tunnel with upward drain holes drilled from within the tunnel, as shown in Figure 2. The cost of the pressure relief tunnel system was estimated at \$2 million which represented a net saving to the project of \$7 million.

Extensive hydrogeologic modelling was undertaken to evaluate the optimum location of the tunnel and spacing of the drain holes.

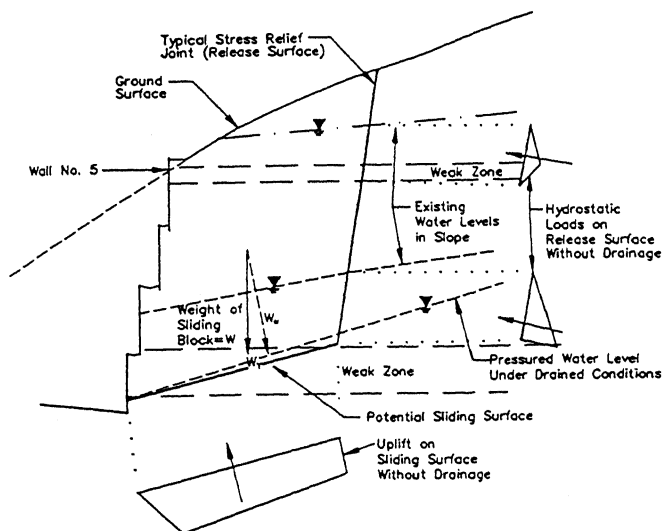


Fig. 3 Typical Loading on Sliding Block

SELECTED DESIGN OF WALL NO. 5

The selected design for Wall No. 5 is shown in Figure 2. It comprises:

- A 12 inch thick, reinforced concrete wall facing constructed in four tiers which are each approximately 30 ft. high. The wall was constructed from the top down, with support installed as the excavation progressed. The overburden and highly weathered rock were supported by soldier piles and timber lagging prior to concrete placement, while shotcrete and dowels were used to protect the argillaceous units in the fresh rock excavation;
- 2140 multi-strand, tendon tie-back anchors with variable lengths and capacities to provide appropriate supporting forces at each wall section. The largest anchors have capacities of 245 kips and free lengths of 115 ft. The anchors are inclined down at 10 degrees from the horizontal, except in the lower section of the wall the angle is set at 25 degrees to avoid anchoring in the weak zone; and
- A pressure relief tunnel system to reduce the groundwater loading on the tie-back wall.

PRESSURE RELIEF TUNNEL SYSTEM

The pressure relief tunnel system consists of an access tunnel, a main tunnel, drainage curtain and upward piezometers. The 326 ft. long access tunnel leads from the portal to the 1945 ft. long main tunnel which is 200 ft. behind the portal. The tunnels have been set at such a grade and elevation that they drain under gravity to the portal. The portal invert elevation is set such that water exiting the tunnel can flow under gravity into the surface water control system

for the highway. The tunnels need only be large enough for efficient tunnel construction and for drain hole drilling. Bidders were given the option of excavating the tunnel any size between 8 ft. wide by 9 ft. high and 10 ft. wide by 10 ft. high and any shape. The contractor selected an inverted U-shape, 8 ft. wide by 9 ft. high, as shown in Figure 4.

The tunnels are at fairly shallow depths below ground surface, so the stresses in the rock surrounding the tunnel are low relative to the strength of the siltstone. The stability of the tunnels is thus controlled by the structure of the rock (mainly bedding and stress relief joints) rather than overstressing of the rock. Rock support is provided by untensioned rock bolts in the tunnel crown and shotcrete around the full perimeter. The shotcrete was primarily required to prevent the rock from slaking. The 6 ft. long rock bolts are spaced on a 3 ft. by 4 ft. pattern and the headings were logged carefully so that additional support could be placed to stabilize particular rock blocks or zones.

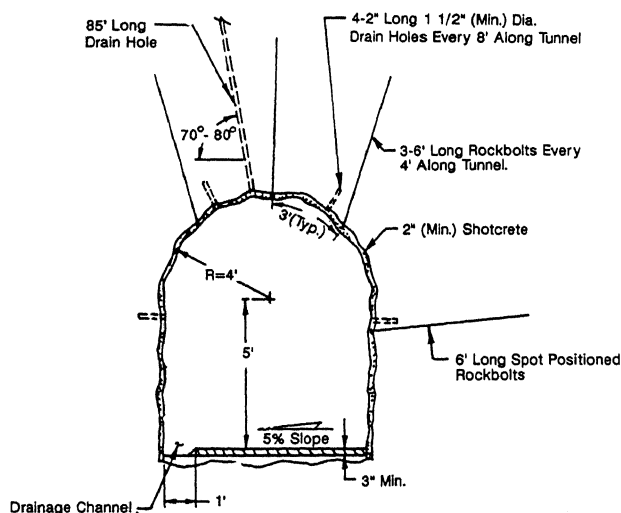


Fig. 4 Typical Tunnel Cross-Section

To confirm that the groundwater table is drawn down to the required level, the design incorporated piezometers installed in 32 to 59 ft. long up-holes 200 ft. on centers along the tunnel, as shown in Figure 5. Access on the hillside to drill from the surface was not possible. Each piezometer consists of a sensing zone at the top of the drill hole connected by a tube to a pressure gage at tunnel level; an isolation zone comprised of one packer just below the sensing zone with a grouted section between the packer and the tunnel crown; and connecting tubes for inflating the packer and de-airing and pressurizing the sensing zone. The piezometers were installed after tunnel excavation and before the start of drain hole drilling.

The drainage curtain consists of a line of drain holes drilled upwards and inclined at 80° to the horizontal towards the wall to intersect the stress relief joints. The hydrogeologic modeling indicated that primary holes spaced at 20 ft. along the tunnel axis would be sufficient

but provision was made in the contract for secondary drain holes to split-space the primary holes and tertiary holes to split-space the primary and secondary holes, if the piezometers do not show adequate lowering of the groundwater level.

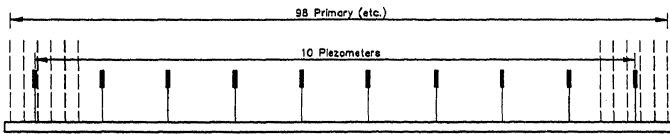


Fig. 5 Profile of Pressure Relief Tunnel

CONSTRUCTION OF THE TUNNELS

The tunnels were excavated by the drill-and-blast method, though the bidders were given the option of drill-and-blast, roadheader or TBM. The components of the tunneling cycle were:

- excavation;
- installation and testing of rockbolts; and
- shotcreting of the tunnel (after 100 ft. of advancement)

The following sections describe the details of the excavation cycle.

Excavation

Excavation consisted of drilling the face, loading and detonating the round and mucking the debris pile. Each round was drilled nominally 8 ft. long using jacklegs. One of the holes was drilled 10 ft. to probe ground conditions ahead of the next round. This probe hole was often drilled as part of the burn cut.

The day shift and night shift used variations of the same blast pattern. The basic pattern consisted of a burn cut, reliever holes and rib holes. The burn cut comprised two 3-inch diameter relief holes often drilled 10 ft. long, and four 1 3/4 inch diameter blast holes each 8 ft. long. These four blast holes were usually loaded with 3.9 pounds of explosive.

The two blast patterns differed in the number of reliever and rib holes and the sequence in which these holes were detonated. The night shift typically used 30 reliever and rib holes and 99 pounds of explosive per round. In contrast, the day shift only used 28 reliever and rib holes, but 109 pounds of explosive per round. Both shifts required approximately 1 hour to drill out a round, although occasional equipment breakdown would increase this time.

The muck pile was removed using a Wagner ST 3 1/2, a rubber tired, front end loading scooptram. The mucking portion of the excavation cycle generally required 1 hour to complete. However, towards the end of the project the tram time increased the mucking time to 2 hours per round. The increase was due to the greater distance from the portal and a change in the rock from siltstone to sandstone. The sandstone muck was larger, blockier and more difficult load into the scooptram.

Rockbolt Installation and Testing

The ground support system consisted of pattern rockbolts and shotcrete. The shotcrete was included to provide a weathering barrier for the siltstone rather than as a strength element. As expected, the rock was largely self-supporting, but rockbolts were installed to prevent bedding separation in the roof beam.

Most of the rockbolts were 6-feet long and all were epoxy coated. The rockbolts were installed in a 3 feet by 4 feet pattern with the 4 feet interval along the length of the tunnel. These bolts were aligned radially from the center of the invert (Figure 4). Four rockbolts were used across the tunnel at the start of the Access Tunnel where the heading was 10-foot wide tunnel while only three were installed in the 8-foot wide sections. The miners used the jackleg drills to install the rockbolts.

In many instances, the rockbolt pattern would not have supported potentially unstable blocks. In these situations such blocks were either removed by scaling or were stabilized by an adjustment in the rockbolt pattern. Those blocks that could not be stabilized by either of these procedures were stabilized by addition of supplemental rockbolts. Only 10 supplemental rockbolts were installed in the entire Pressure Relief Tunnel System.

Rockbolt tests were performed to provide an acceptable minimum level of quality control. Preconstruction tests were conducted in the portal wall to confirm the contractor's ability to install rockbolts and to determine the minimum length of quick set and boring required.

Once the excavation began, two of the first 5 rockbolts installed in the tunnel were tested. After this, a performance test was scheduled for once every 50 rockbolts. These tests were performed to ensure that the capacity of the rockbolts had not been reduced by either a change in the contractor's installation method, the geology, or the quality of the resin cartridges. The interval between performance tests was eventually increased to 150 rockbolts because of the good ground conditions and the fact that all the rockbolts tested passed the performance test.

Shotcrete Application and Testing

Each time the contractor advanced the tunnel heading approximately 100 ft., the excavation was halted and a 2-inch thick (minimum) layer of shotcrete was applied to the crown, haunches and walls. These areas were washed with a high pressure water hose before the shotcrete was applied.

The contractor used dry mix shotcrete consisting of 700 lbs. of cement to 800 lbs. of 3/8-inch diameter aggregate to 220 lbs. of C-33 sand. Accelerator was added (up to 5% by weight of cement) by hand to the shotcrete pot. The shotcrete was applied in a single layer working upward from the lower portions of the tunnel walls. The shotcrete thickness was monitored throughout its application.

Although the shotcrete was applied primarily to reduce weathering it could also contribute to rock support. Therefore a program of quality control testing was implemented. Three shotcrete core samples were taken from every 100 linear ft. along the tunnel axis. Approximately half of the total samples were taken from the crown. The remainder were taken from either wall. These samples were tested to determine their unconfined compressive strength.

PIEZOMETER INSTALLATION

Up-hole piezometers were installed from within the Main Tunnel on 200 ft. centers to monitor the drawdown of water levels as the drain holes were drilled (Figure 5). The data collected from the piezometers was used not only to demonstrate that the water levels had been drawn down to the design levels but, also to identify when the contractor could construct the lower half of Retaining Wall No. 5.

The ten piezometer holes were drilled with an air track drill rig using a 3 1/2-inch diameter bit. Before installing each of the piezometers, the packer system was leak tested. Any leaks were repaired. Also, the 2 3/4-inch diameter Aardvark packer was slipped into a 3-inch diameter steel tube and inflated to 60 psi for 24 hours. If there was a significant decrease in pressure, the connections were refitted and retightened.

Once the leak test was completed, the piezometer was assembled in the tunnel and installed. The assembly involved 4 steps. First, a threaded steel plug was inserted into the top of the packer. This plug formed a spacer at the top of the hole and set the size of the sensing zone. Second, 10 ft. long sections of mandrel pipe were attached to the back of the packer to push the packer to the tip of the hole. This tube was also used to inject grout into the hole below the packer. The grout entered the borehole through 1/2-inch diameter holes in this pipe. Third, a bulkhead was attached to the bottom of the pipe. The bulkhead consisted of an aluminum plate which was large enough to cover the borehole and to hold the piezometer system's valves and gauges. The final step involved the application of quickset mortar. This mortar was used to smooth the area surrounding the borehole collar and to help seal the bulkhead to the crown once the piezometer was installed.

The borehole was filled with grout in two steps. First, a 7 ft. long zone in the bottom of the hole was grouted. The remainder of the hole was grouted only after this initial grout set. By grouting the borehole in this manner, the chances of failing the rock in the crown of the tunnel were reduced significantly.

The grouting continued until the calculated void volume of grout was injected and a small rise in grout pressure above hydrostatic was noted.

The procedure for priming the sensing zone of the piezometer was carried out in two steps. First, the sensing zone was evacuated for 30 minutes. Second, the sensing zone was filled with de-aired water to a pressure that was approximately 10 to 20 psi above the piezometer and pipe pressure. This pressure was maintained while the secondary grouting was performed.

Once the secondary grouting was completed and the sensing zone had been primed, a falling head test was conducted to check that the piezometer system was operating properly. This test consisted of closing the valve that connected the sensing zone and the priming equipment and recording the pressure changes that occurred. Pressure readings were taken at approximately every 15 seconds for the first minute, thereafter at every minute for the next 4 minutes, then every 5 minutes for the next 25 minutes, and then every 1 hour for the next 23 hours.

Drain Holes

Once a piezometer was installed and activated, the contractor began drilling drain holes within 100 ft. of the piezometer. Primary drain holes, spaced 20 ft. apart, were drilled into the crown throughout the entire length of the Main Tunnel (Figure 6). Secondary drain holes were also drilled from Sta. 11+00 to Sta. 19+40. The secondary drain holes split spaced the primary drain holes, so that the spacing between drain holes was reduced to 10 ft. Secondary drain holes were drilled in this section of the Main Tunnel because of the relatively slow decrease in pressure indicated by the piezometers after the primary drain holes were drilled.

All of the drain holes were drilled between 70 to 80 degrees above the horizontal, such that they were inclined toward the free face of Retaining Wall No. 5. In this way the possibility of intercepting stress relief joints, which dip toward the river at 70 to 80 degrees, was enhanced.

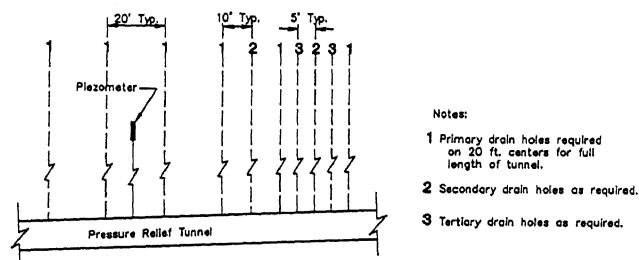


Fig. 6 Typical Profile of Piezometers & Drain Holes

Long-term Piezometer Response

After the falling head test was started, pressure gauge readings were taken twice a day for the first week and, once per week thereafter until the contractor disconnected the tunnel ventilation fan. These readings form the data base from which long-term piezometer response was determined.

Even before the piezometers were installed, the water levels had decreased due to the excavation of the tunnel. The primary drain holes further promoted drainage. In most of the tunnel, the primary drain holes were sufficient and no other steps had to be taken to lower the water level to the required design level. However, in the north end of the Main Tunnel, secondary drain holes were required.

Figure 7 shows the long-term response of piezometer TP-8. This response was typical of the other piezometers. The effect of the secondary drain holes, while delayed, can be seen clearly.

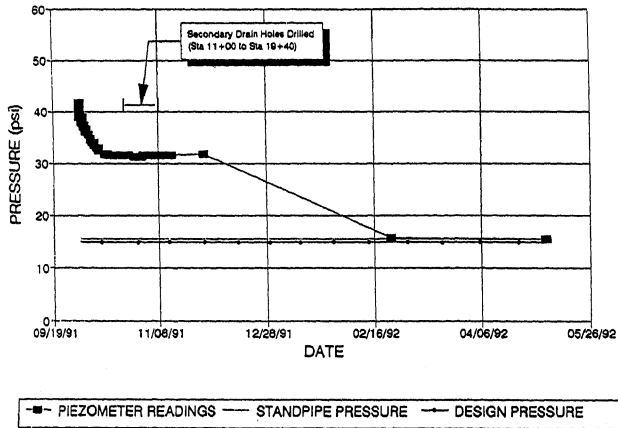


Fig. 7 Long Term Piezometer Readings
Piezometer TP-8

CONCLUSION

Based on the long-term piezometer responses, the depressurization tunnel has proven to be an effective means of reducing the hydrostatic pressure behind Retaining Wall No. 5. The reduction in the hydrostatic pressure has resulted in decreased tieback design forces which represent a net savings of \$7 million.