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## **Golder Associates**

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO  
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

STUDY OF WATER SEEPAGE  
THOROLD TUNNEL  
(WELLAND CANAL)  
THOROLD, ONTARIO

30M3-

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ABSTRACT

Golder Associates have been retained by the Ministry of Transportation and Communications to assess the causes of heaving of the asphaltic concrete in the floor of the Thorold Tunnel beneath the Welland Canal. The results of the investigation are reported and recommendations are given regarding remedial measures to correct the floor heave problem.

The investigation consisted of the coring of bedrock in two borings adjacent to the canal to assess the bedrock conditions; a hydrogeologic assessment of groundwater flows; inspections and sampling of the concrete in the tunnel floor; and a review of the design and construction details of the tunnel.

Based on the results of the study, we conclude that the majority of the seepage into the tunnel is coming through the construction joints. In our opinion, it would be difficult to effectively grout the construction joints to prevent seepage. The provision of a grout seal outside the tunnel is considered to be impracticable.

Therefore, we recommend that consideration be given to controlling seepage such that it prevents freezing and heaving beneath the asphaltic concrete. We propose that drainage slots be provided along the construction joints in the floor slab and that these slots be drained into the existing sewer line. Heating cables would be provided within the drainage slots to prevent freezing. Details of the floor slab drainage system are given on Figure 12.

We recommend that a test section of floor slab drainage be provided to establish the effectiveness of the system.

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## 1.0 INTRODUCTION

Golder Associates have been retained by the Ministry of Transportation and Communications to carry out a study of a pavement heaving problem within the Thorold Tunnel (Highway 58) beneath the Welland Canal (for location, see Figure 1). The problem consists of the growth of ice lenses beneath the asphaltic concrete and general icing conditions on the pavement surface, resulting in hazardous driving conditions.

Our terms of reference call for an investigation of the cause(s) of water seepage through the floor of the tunnel and to make recommendations with respect to measures required to correct the situation.

## 2.0 BACKGROUND OF TUNNEL

The following details of the tunnel design and construction are based on our discussions with Messrs. K. Selby, J. Lougheed and G. Green of the Ministry of Transportation and Communications, together with a review of the geotechnical investigation reports and construction drawings available from Burlington, Thorold and Downsview.

A plan and sections of the tunnel are shown on Figure 2. Geotechnical investigations and design criteria for the tunnel were provided by Acres Consulting Services Limited in 1964 and 1965. The tunnel construction was completed in 1968.

The investigations indicated that the tunnel alignment was underlain by some 25 ft. of overburden comprising predominantly silty clay, followed by about 60 ft. of dolomitic limestone (Goat Island, Gasport and Decew formations), and finally, calcareous shale of the Rochester formation. The hydraulic

conductivity of the dolomitic limestone varies from about  $9 \times 10^{-5}$  cm/sec. to  $5 \times 10^{-4}$  cm/sec., while the values in the shale range from  $5 \times 10^{-5}$  cm/sec. to  $1 \times 10^{-4}$  cm/sec.

The tunnel was constructed within the dolomitic limestone, extending down into the underlying shale. Construction was by the cut-and-cover method, with the work being carried out during the winter when the Welland Ship Canal could be drained.

The tunnel structure consists of a rectangular concrete box containing two traffic tubes - north and south tubes each having two lanes - separated by a 6.5 ft. wide utility corridor. The widths of the two traffic tubes are 30 ft. (south tube) and 34.2 ft. (north tube), with the clear height inside each tube being about 16 ft. Although there is some variation in thicknesses of the exterior members of the structure, the average thicknesses are as follows: floor slab, 5 ft.; walls, 4 ft.; and roof, 4.5 ft. Service buildings were provided above the tunnel at both ends.

Along the base of the tunnel, a 6 in. minimum thickness of concrete was placed over the rock surface. A 0.25 in. thick steel plate was then placed over the base concrete, beneath the floor slab, and extending 5.5 ft. up the outside of the tunnel walls to serve as a waterproof membrane. The steel plates were joined together by means of 8 in. wide "Armorline" sealing strips placed over the butt joints in the steel plates. Alternate shrinkage joints and construction joints, with 6 in. wide waterstops, were provided at 40 ft. spacing in the floor slab, walls and roof slab.

Waterproofing of the remainder of the structure was by means of 0.2 in. thick "Volclay" panels placed against the exterior face of the roof and walls, overlapping the steel plate along the lower portion of the walls. Protection for the "Volclay"



panels was provided by 1 in. thick fiberboard against the panels. A further 2 in. thickness of timber planking was provided (over the fiberboard) along the walls, while for the roof section, the fiberboard was overlain by 5 in. of concrete.

The space between the tunnel walls and the rock cut was back-filled with rockfill. Rockfill was also placed over the tunnel up to the level of the canal base for the section extending from the west wall of the canal to the East Service Building. Thus, the depth of canal water over the rockfill in this section is approximately 30 ft. Between the canal and the West Service Building, the rockfill extends up to approximately the original ground surface. Concrete bulkheads were provided between the tunnel structure and the sides of the rock cut at both service building locations to form barriers to water flow from the canal to the portal areas.

Sumps were provided beneath the tunnel floor at both portals and in the central section of the tunnel. The sumps at the portals collect surface water from gratings across the traffic lanes at the portals, subsurface water from outside the portal area, and water from the rock backfill. At both of these sumps, there are two 6 in. diameter pipes at each side of the tunnel to allow for drainage of the backfill. In addition, relief holes were provided from the base of these sumps. The central sump collects roadway surface water only by means of a system of catch basins and sewer pipe along the outer curb lines.

Seepage into the tunnel has been occurring essentially since the construction was completed. Some attempts have been made over the years to prevent the seepage by grouting of the joints from the inside of the tunnel. The grouting has not been fully successful.

### 3.0 INSPECTIONS AT SURFACE ABOVE TUNNEL

Several inspections were made of the surface conditions above the tunnel during the course of the investigation. Observations made during the inspections are discussed below.

An inspection of reported depressions in the fill above the tunnel along the alignment of the pondage canal was made on March 3, 1983. The rockfill in this section of the tunnel has been overlain by several feet of clayey silt, probably by sedimentation in the relatively calm backwater from the canal.

The depressions were on either side of the rock cut for the tunnel. Each depression was about 50 ft. square, thus they extended laterally from the edge of the rock cut to the near wall of the tunnel. The depths of the depressions were approximately 7 to 8 ft., with the bottom 2 ft. being covered with water at the time of the inspection.

Some coarse rockfill could be seen along the edge of the depression aligning approximately with the tunnel wall. The rockfill was infilled with clayey silt.

We understand from discussions with Mr. Lougheed that during previous inspections of these depressions several days prior to March 3rd when the water level was higher, a vortex of water flowing into the rockfill could be seen. We also understand that the depressions may be associated with excavations made adjacent to the tunnel, at the time of tunnel construction, to facilitate future twinning of the canal. It is possible that these excavations were never backfilled. However, it is also possible that some fines from the depressed areas may have been washed down into the rock backfill adjacent to the tunnel.

The depressions were being filled in during the inspection with clay being bulldozed in from the north side of the tunnel cut.

A series of small sinkholes were also noted within the clayey silt cover over the tunnel between the pondage canal and the East Service Building. These sinkholes varied from 2 ft. to 5 ft. in diameter, with the larger ones being up to 2.5 ft. deep. The sinkholes were located approximately along the alignments of the north and south walls of the tunnel. In the larger sinkholes along the north wall alignment, there were no exposures of rockfill. However, in several smaller sinkholes along the south wall alignment, open rockfill could be seen. We suggested to Mr. Lougheed that these sinkholes be filled with clay.

A further inspection was made of the canal base over the tunnel on March 7, 1983. The purpose of this inspection was to determine the type of material overlying the tunnel along the base of the canal, and specifically, to determine whether or not there is a clay seal at the canal base.

The canal base over the tunnel was covered with rockfill. Some silting of the rockfill has occurred. As well, a minor amount of clay had recently (during the construction of the canal wall) been spread over the north side of the tunnel. This was probably material that had been cleaned out from the new wall foundation area.

Based on our discussion with the contractor working on the canal wall, we understand that an 8 ft. deep sump had been dug below the canal base over the tunnel. We were told that the excavation for this sump was within rockfill for its full depth.

Thus, the inspection did not reveal any clay seal over the tunnel.

During the March 7th inspection, we also inspected, probed and sampled the clayey silt cover over the tunnel between the canal and the East Service Building. The thickness of the clayey silt varies from about 5 ft. midway between the main canal and the pondage canal, to about 1 ft. in some areas east of the pondage canal.

Hand probes put down through the base of two sinkholes along the alignment of the north wall of the tunnel indicate 12 to 18 in. of clay cover. Immediately adjacent to the sinkholes, the clay cover was 3 ft. thick.

A grain size analysis and Atterberg limit test results for a sample of the clayey silt are given on Figure 3.

#### 4.0 BEDROCK DRILLING AND IN SITU TESTING

##### 4.1 General

The field work for this portion of the investigation was carried out between February 14 and 23, 1983. A total of two borings (numbered 101 and 102) were drilled and the bedrock was continuously sampled at the locations shown on Figure 2 using a track-mounted power auger.

Both boreholes were advanced through the overburden and cased approximately 6 in. into the bedrock. The boreholes were then drilled with a 10 ft. long NQ size core barrel. Pneumatic packer tests were carried out as the drilling advanced and multi-level piezometers were installed for water sampling and water level monitoring. All core was logged in the field, placed in core boxes and transported to our laboratory in Mississauga for detailed logging.

Complete details of the bedrock strata encountered, piezo-meter installations, hydraulic conductivities, etc. are given on the Record of Borehole sheets following the text of this report. The locations of the boreholes were measured from the West Service Building and they were also referenced to the west wall of the Welland Canal. The borehole elevations were referenced to the finished ground floor slab of the West Service Building. Based on contract drawings at the site, the elevation of the floor slab is 588.50 ft., Geodetic.

#### 4.2 Pneumatic Packer Tests

Packer tests were carried out in both boreholes with NQ sized packers. The pneumatic packers enable testing to be conducted as the hole is advanced. At the end of each 10 ft. run of core, the drill rods were pulled back approximately 14 ft. and the packer equipment lowered down through the drill rods on a wireline. The packer equipment consists of two inflatable rubber seals, or packers, with a core of stainless steel rod down the centre for rigidity and passage of fluid into the section of borehole below the lower packer. The lowermost packer is designed to pass through the diamond drill bit. A stainless steel flange below the uppermost packer lodges in the core barrel just above the drill bit leaving the packer inside the drill rods.

Both packers are connected to the surface with a small diameter plastic line. Nitrogen is blown into the packers to inflate them and seal off the lower section of the borehole. Inflation pressure of the packers is of the order of 400 lb/sq.in. The upper packer seals the drill rods thus permitting water to flow only into the formation isolated by the packers. The lower packer isolates a test section between the bottom of the hole and the drill rods. Because each packer or seal, when inflated, is 3 ft. long, the isolated section of borehole is located

3 ft. below the drill bit. Therefore, by pulling the drill rods back 3.5 ft. after each 10 ft. core run, 0.5 ft. of overlap is obtained between successive test sections.

To investigate the hydraulic conductivity\* of the isolated sections of bedrock, falling head tests were performed. To conduct the test, the drill rods were filled with water and allowed to overflow until the start of the test. At the start of the test, the water level was permitted to drop. The rate of water level drop was recorded throughout each test. A record of the water level decline with time was used to compute hydraulic conductivities of the rock mass using Hvorslev's method. Because the packer configuration allows water to escape from the drill rods only through the centre of the packer assembly into the lower section of the borehole, the observed decline in water level is due to water being forced into the isolated section of rock. Thus, a hydraulic conductivity calculated for each interval along the borehole provides an indication of the relative soundness of the bedrock within the test interval if the stratum is relatively uniform. However, it must be noted that a relatively small, open joint within the 10 ft. length being packed can have a significant bearing on the average computed hydraulic conductivity.

#### 4.3 Piezometer Installations

Upon completion of each borehole, a multi-level piezometer was installed for the purpose of water level monitoring and groundwater sampling.

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\*Also referred to as coefficient of permeability.

The multi-level device consists of a bundle of 1/2 in. diameter riser tubes contained within a 2 in. PVC pipe that is plugged on the bottom. Each of the 1/2 in. diameter tubes is connected to a 90 degree elbow, which protrudes through the casing. Each tube extends to a different depth in the casing so that the water can be drawn from different depths in the borehole. The interval of borehole associated with each tube is isolated from above and below by packers that expand so that the annulus between the casing and the wall of the borehole is sealed over specified intervals and vertical movement of water within the borehole annulus is inhibited.

The packer devices are fitted along the casing to allow portions of the borehole to be sealed off. These packers comprise a rubber-like material (Dowell sealant) which is contained within a thin rubber membrane. The Dowell sealant expands on contact with water and thereby provides a tight seal against the walls of the borehole. The sealant is wetted by filling the inside of the PVC casing with water. A series of holes drilled in the casing beneath the packer allow the water to reach the sealant but the water is prevented from escaping into the annulus of the borehole by the rubber membrane over the sealant.

The sampling points in the two boreholes were located at intervals to coincide with the major bedrock strata as identified from an examination of the bedrock core recovered during drilling.

Groundwater samples were obtained from the multi-level devices using a gas-driven triple tube sampler. The sampler features three co-axial tubes; a small-diameter inner tube

attached to an inflatable mini-packer, a slightly larger-diameter riser tube through which the water sample was transmitted to ground surface, and an external gas-driven tube (in this case the 1/2 in. diameter polyethylene riser tube of the multi-level monitoring device) through which pressurized air is added to drive the water sample up the riser tube.

The groundwater samples were collected in a large flask at surface. Electrical conductance and pH for all samples were measured in the field immediately following sample collection. Two filtered (0.45  $\mu$ m) volumes were retained in 50 mL plastic syringes. Selected samples were forwarded to a commercial chemical laboratory (Barringer Magenta Ltd., Toronto) for the analysis of anions, cations and metals. The syringe sample for cation and metal analyses was acidified to pH <2 with nitric acid.

The results of analyses on these water samples are detailed in Appendix 4 and are discussed in Section 9 of this report.



## 5.0 CONCRETE CORING, INSPECTION AND TESTING

### 5.1 General

The tunnel floor surface was inspected and repair patches and wet spots were noted. A plan detailing the locations of the distressed areas was prepared as shown on Figure 4.

Based on the types and locations of the distressed areas, test areas were selected for removal of the asphaltic concrete and coring of the floor slab. The field work was carried out in two stages: the first series of tests consisting of 5 test areas in the north tube was completed during the period in which there was no water in the canal; the second series consisted of opening 5 test areas in the south tube after the canal was refilled. In addition, during this second stage, water tests were carried out to provide data on the flow of water in several core test locations in the north and south tubes.

All traffic control, cutting out and patching of test sections, and general assistance during the course of the field work was provided by the Ministry of Transportation and Communications, Thorold.

### 5.2 Floor Slab Inspection and Coring

In general, the following procedure was carried out for each test location after removal of a 2 ft. x 2 ft. area (approximate) of asphalt. The area was cleaned, photographed and inspected. Surface deterioration and location of weeping water was noted. Subsequently, a 3 in. diameter core sample, typically 4 ft. in length, was obtained.

After removal of the core, an expandable packer, with provision to couple a valve, was inserted into the hole for subsequent water testing. The test area was then backfilled with cold patch asphalt.

Following completion of testing, the packers were removed, the holes were grouted with Interplast rapid curing concrete, and the test areas were patched over.

The core samples were transported to our Mississauga laboratory for detailed examination and testing. The cores were photographed (see Figures 5 and 7 for photographs of typical core), selected prepared sections were examined microscopically, and compressive strength tests were carried out.

### 5.3 Water Tests

Falling or rising head tests were carried out in selected core test locations. All of the water tests were carried out after the canal was refilled. These tests consisted of re-opening the test locations, where necessary, attaching a shut-off valve to the packer and attaching a transparent flexible tube having a length of approximately 8 ft.

In all cases, the packer was sealed at a depth of about 6 in. below the top of the concrete. The tube was held vertically and the core hole and tube were filled with water. Readings of the rise or fall in water level were noted for selected intervals of time.

## 6.0 WATER MONITORING

The water monitoring program consisted of several components. These included the measurement of surface water levels in the canal, groundwater levels in the multi-level piezometers (Boreholes 101 and 102), recording operating times for pumps in each of the three sumps, and the collection of water samples from the canal, sumps, tunnel floor core holes and groundwater monitoring devices for chemical analyses.

### 6.1 Piezometric Levels

Water levels were measured in the canal and in the piezometers prior to, during and following the filling of the canal. These measurements were made so that the hydraulic gradients in the vicinity of the canal could be determined and the directions of groundwater flow established.

The piezometer installations were completed 2-1/2 weeks prior to the canal refilling. Periodic monitoring of the water levels prior to the canal refilling confirmed that stabilized conditions had been reached. The stabilized water levels ranged between elevations 539.5 ft. and 542.5 ft. in the piezometers installed within the dolostone formations, while the piezometric level within the underlying shale was at about elevation 511 ft.

The canal was refilled during the period 0215 hours, March 13, to 1030 hours, March 14, with the water level being raised from the base of the canal (approximately elevation 537 ft.) to elevation 568.6 ft. During this period, the water levels in the piezometers were monitored every 1 to 2 hours in conjunction with water level observations in the canal and in the tunnel. Plots of piezometer water elevations versus time

during the canal refilling are given on Figures 9 and 10. As well, the canal water levels during the refilling period are plotted.

As shown on Figures 9 and 10, the piezometers within the dolostone formations (above elevation 504 ft.) responded very rapidly to changes in the canal water level. The response times for the piezometers in Borehole 102 were more rapid than for the piezometers in Borehole 101; their distances from the west wall of the canal being about 75 ft. and 240 ft., respectively. When refilling of the canal was stopped at 1030 hours on March 14, the dolostone piezometers in Borehole 101 were within 2 ft. of the canal water level, while the maximum differential in Borehole 102 was 6.5 ft.

The piezometric level within the underlying shale formation was unaffected during the period of refilling of the canal.

A final set of piezometer levels was taken on March 18, 1983. At that time, we noted that the canal water level had been lowered to elevation 566.8 ft., that is, 1.8 ft. below the level recorded at 1500 hours on March 14th. The piezometric levels within the dolostone were generally between 1.7 ft. and 2.3 ft. lower than the corresponding levels on March 14th. However, the piezometric level within the Rochester shale was at elevation 523 ft., a rise of 12.6 ft. since March 14th.

## 6.2 Pumping Records

Records of the operating times for each pump in the tunnel sumps are available at the site. The records for times prior to emptying of the canal (late December, 1982) were reviewed. As well, detailed pumping records were maintained during the canal refilling and for a further 11 days following refilling.

The pumping records available prior to 1983 confirm that the water inflow into the west sump is considerably greater than that into the central and east sumps. During 1982, for example, Pumps 1, 2 and 3 in the west sump operated an average of 49 per cent of the time. The fourth pump in the west sump operated less than 1 per cent of the time. The pumps in the east sump operated about 1.5 per cent of the time, while those in the central sump operated less than 1 per cent of the time.

During the initial stages of the canal refilling, the pumping times were recorded hourly. After the canal water was essentially to its final level, the reading intervals were increased to 2 hours, then 4 hours, and finally to 24 hours for the final 6 days of monitoring. Records of the pumping times after the canal was refilled were maintained by the Ministry of Transportation and Communications staff at the site.

Since the operating times for the pumps in the central and east sumps, as well as for the fourth pump in the west sump, are minor in comparison to Pumps 1, 2 and 3 in the west sump, our comments regarding pumping will be confined to these latter three pumps.

Immediately prior to refilling the canal, the pumps were operating intermittently at an average (for the three pumps) of less than 5 per cent of the time. A more uniform pumping rate was established when the canal level had reached about elevation 550 ft., some 14 hours after refilling was begun. From there on, the pumping time increased gradually until an average operating time of 46 per cent was reached for the three pumps when the detailed monitoring was stopped on March 25th.

A plot of the percentage pumping time (average of the three pumps) versus time is given on Figure 11.

We understand from discussions with the St. Lawrence Seaway Authority that the canal water level was lowered to elevation 566.8 ft. on March 16, 1983. The water level was then raised to its normal operating level of 568.1 ft. on April 4, 1983.

### 6.3 Water Sampling

Twenty-one water samples were collected for geochemical analyses. Water samples were collected from four distinct areas in an attempt to establish the source of seepage water to the tunnel in terms of its chemical characteristics. The sample collection included:

1. Surface water samples from the canal prior to and following the filling of the canal. Three water samples were collected on March 9, 1983, from the main canal and backwater areas prior to the filling of the canal. With reference to Figure 2, these samples were collected from the base of the main canal (60+00), from a sink hole over the tunnel near the pondage canal (66+00), and from a pond in the backwater area adjacent to the East Service Building (71+00). One surface water sample was collected from the canal on March 18, 1983, following the filling of the canal (67+00).
2. Seepage samples from each of the three sumps prior to filling (March 9) and after filling (March 18) of the canal. The samples were collected from active seepage inflows where feasible; however, in the central sump, before and after filling, and in the east sump in the post-filling stage, no active seepage inflows were present and water samples were collected from the floor of the sumps.
3. Water samples from the inflows into the coreholes in the floor of the tunnel. Samples were collected from test

areas 1, 2, 3 and 5 (Figure 4) prior to the filling of the canal on March 9th.

4. Seven groundwater samples from the multi-level piezometers. Four samples were collected prior to the canal filling on March 10th; two of these were from Borehole 101 (piezometers 2 and 4) and two were from Borehole 102 (piezometers 1 and 3). Three groundwater samples were collected following the filling of the canal on March 18th. These were obtained from Borehole 101 (piezometer 2) and Borehole 102 (piezometers 1 and 3). Groundwater sampling procedures are described in Section 4.3.

#### 7.0 BEDROCK CONDITIONS

The rock strata encountered by the Thorold Tunnel consist of the Goat Island and Gasport Members of the Lockport Formation and the underlying Decew Formation and Rochester Formation. These formations comprise a horizontally bedded sequence of dolostone, dolomitic limestone and shale that correlate throughout the Thorold Tunnel site as shown in section on Figure 2.

Bedrock conditions at this site were previously investigated during the tunnel feasibility study prepared by H. G. Acres and Company Limited for the Department of Highways. The feasibility study included a total of 16 boreholes cored through the rock strata at this site. The core from each borehole was geotechnically logged and the various strata encountered were correlated between the boreholes.

The strata encountered in Boreholes 101 and 102 of this present investigation closely correlate with those of the previous Acres feasibility investigation. Boreholes 101 and 102 are shown in section on Figure 2.

Detailed geotechnical descriptions of the cores recovered from Boreholes 101 and 102 are provided on the Record of Borehole sheets following the text of this report.

## 7.1 Lockport Formation

### 7.1.1 Goat Island Member

The Goat Island Member is the upper member of the Lockport Formation. It forms the bedrock surface at the tunnel site and attains a thickness of 23 to 24 ft. The rock comprises fresh to faintly weathered, light brownish grey, fine grained, fossiliferous, medium to thickly bedded dolostone. Small solution cavities impart a vuggy texture to the rock and some cavities are gypsum coated or filled.

Bedding partings and joints within this member were noted to be moderately weathered, open features stained with iron hydroxides. These features have resulted in lower RQD values for the Goat Island Member compared to the underlying dolostone members. In Borehole 101, the RQD varied from 70 to 85 per cent and was lower in Borehole 102 where it varied from 50 to 80 per cent.

As noted on Figure 2, the majority of the bedrock excavation for the ship canal at this site was carried out within the Goat Island Member.

### 7.1.2 Gasport Member

The Gasport Member of the Lockport Formation is approximately 26 ft. in thickness at the site of the present borings. Geotechnical investigations by Acres indicate that this member



varies from 17 to 25 ft. in thickness across the site. It comprises fresh to faintly weathered, light to medium grey, fine to medium grained, stylonitic, thin to thickly bedded, fossiliferous, dolomitic limestone.

The upper half of the Gasport Member contains a dark grey, thinly bedded, fossiliferous, calcareous shale. This unit varies from 7 to 8 ft. in thickness where encountered in Boreholes 101 and 102, but locally thins to 4 ft. elsewhere at the tunnel site.

The RQD of the Gasport Member encountered in Boreholes 101 and 102 is high, varying from 80 to 90 per cent. As noted on the logs of Boreholes 101 and 102, the majority of the discontinuities within the rock are bedding plane separations.

## 7.2 Decew Formation

The Decew Formation is approximately 11 to 12.5 ft. thick where encountered in Boreholes 101 and 102 and previous investigations by Acres noted it to vary from 7 to 13 ft. in thickness. The strata comprises fresh, medium brownish grey, fine grained, medium to thickly bedded dolostone. This formation has a gradational contact with the underlying Rochester Formation shale.

The Decew strata is quite similar to the overlying Gasport Member. The RQD within the Decew was noted to vary from 80 to 90 per cent (see borehole logs 101 and 102). Bedding plane separations are the dominant discontinuities.

## 7.3 Rochester Formation

The Rochester Formation shale is more than 44 ft. thick at the Thorold Tunnel site based on previous borings put down

by Acres. Boreholes 101 and 102 were terminated 18 and 10 ft. respectively below the upper surface of this shale.

The shale is a fresh, medium to dark brownish grey, very fine grained, thinly to medium bedded, moderately fissile rock. The low point of the Thorold tunnel was entirely excavated within this Formation as shown on Figure 2.

The RQD of this strata was found to vary from 90 per cent in Borehole 101 to 50 per cent in Borehole 102, reflecting a higher number of bedding separations than the overlying dolomite.

#### 7.4 Bedrock Hydraulic Conductivity

The hydraulic conductivity of the bedrock encountered in Boreholes 101 and 102 was determined by carrying out falling head permeability tests at 10 ft. intervals using a single packer arrangement as the boreholes were advanced. The tests carried out in the boreholes are summarized in Appendix 1 and on the Record of Borehole sheets.

Following an examination of the core, it is considered that the permeability of the rock reflected by the hydraulic conductivity testing is due to open bedding planes and subvertical jointing. Several of those features were noted to be stained with iron hydroxides indicating the movement of groundwater.

Based on data from Boreholes 101 and 102, the hydraulic conductivity of the Goat Island Member dolostone is high, varying from  $2.6 \times 10^{-3}$  to  $7.8 \times 10^{-3}$  cm/sec, and averages  $5 \times 10^{-3}$  cm/sec. The highest values were obtained in the first 10 ft. section of bedrock in each borehole. The average of hydraulic conductivity reported in the Acres study for this member was  $5.2 \times 10^{-4}$  cm/sec.

The Gasport Member and Decew Formation dolostone has a somewhat lower hydraulic conductivity than the overlying Goat Island Member. The values vary from  $7.6 \times 10^{-5}$  to  $2.6 \times 10^{-3}$  cm/sec and average  $1.5 \times 10^{-3}$  cm/sec. These somewhat lower values are reflected by the lower RQD of the Gasport Member and Decew Formation compared to the overlying Goat Island Member. The average hydraulic conductivity for the Gasport Member and Decew Formation reported in the Acres study is  $1.8 \times 10^{-4}$  and  $9.0 \times 10^{-5}$  cm/sec, respectively.

A significant decrease in hydraulic conductivity was noted in the Rochester Formation shale. In Borehole 101, the values of hydraulic conductivity were approximately  $5.7 \times 10^{-5}$  cm/sec, showing a pronounced decrease from the overlying dolostone. This value is in agreement with the average hydraulic conductivity of  $5 \times 10^{-5}$  cm/sec determined for the Rochester shale by Acres. In Borehole 102, the value for the top 10 ft. of the shale encountered was  $1.6 \times 10^{-4}$  cm/sec.

## 8.0 FLOOR SLAB CONDITIONS

A plan showing the areas of distress in the tunnel floor slab, together with the locations of the test areas, is given on Figure 4. Figures 5 to 8, inclusive, show typical record photographs of the concrete surface after removal of the asphalt, as well as of the concrete cores. Descriptions of the test areas are given in Appendix 2 and the results of the water tests are given in Appendix 3.

### 8.1 General

It was noted during inspections of the tunnel floor that there was a discernible pattern to the location of problem areas.

In general, considerably more remedial work had been carried out at the east and west ends of each tube with comparatively little repair work in the centre section. In the main, the remedial works appeared to coincide with the sections of tunnel situated in the lower portions of the dolomitic limestone formations.

Visual inspection of each patched area in the tunnel floor showed that, in the majority of cases, the patching is associated with a construction joint. There are two types of patches. These are patches approximately 6 in. wide across the width of the tunnel over a construction joint, and isolated patches over or close to a construction joint. It was noted that the repair areas at the construction joints over the width of tunnel occur only in the north tube. It was also noted that the isolated patches occur with considerably greater frequency in the driving lanes of both tubes.

## 8.2 Inspection of Tunnel Floor Test Areas

Inspection of the concrete surface after removal of the asphalt showed that deterioration of the concrete surface was present in all cases except for test No. 4. This area was selected on the basis that no asphalt patching was present. At some locations, the exposed area was spalled along the length of the joint. At other areas, deterioration had occurred away from the joint and could be associated with a local surface crack, a honeycombed region or a grout hole.

At some of the test areas, the deterioration was extensive, resulting in exposure of the top reinforcing steel. There was no significant corrosion of the reinforcing steel at any of the test areas.

At most test areas where asphalt patching was present, the surface of the concrete was wet, with standing or weeping water noted.

The water in the north tube was generally observed to be weeping from the joints or defects in the surface, whereas the water in the south tube appeared to travel along the interface between the asphalt and concrete, and collect in the joints.

### 8.3 Inspection of Wall Joints

Visual inspection of the vertical wall joints showed that many joints were weeping. It was observed that these joints had caused problems over a lengthy period of time since some had been grouted to attempt a seal behind the water stop. Most joints had cementitious and/or elastomeric materials in their lower sections. It was also noted that deterioration in the form of spalling had occurred at many joints. Photographs of typical wall joint defects are given on Figure 8.

Based on the extent of remedial work in the tunnel floor and weeping water in the wall joints, it was observed that the asphalt patches in the driving lane could be associated with problem vertical joints. This was particularly the case in the south tube.

### 8.4 Concrete Cores

Inspection of the recovered cores showed that, in general, the concrete is dense and of an acceptable quality. However, some honeycombing was observed in isolated areas. Compressive strengths of core sections ranged between 32.1 and 46.5 MPa.

All of the cores from the north tube had a cold joint at approximately 11 to 22 in. below the surface. No similar discontinuity could be observed in the cores from the south tube.

Microscopic examination of sections of the cores showed that the concrete was not air entrained. No delamination was found immediately below the deteriorated top surface area except at the level of the steel where it was concluded that fracturing was produced by core recovery and was not related to concrete deterioration.

Microscopic examination of sections of core from the cold joint did not reveal any fracturing or scaling which could be related to freeze/thaw damage. It could not, therefore, be positively concluded that water permeated through this particular region.

#### 8.5 Water Test Results

The results of the water tests (for test procedure, see Section 5.3) showed that, in general, the hydraulic behaviour of the water in the concrete of the north tube was different from the south tube. In the north tube, positive water pressure was noted at both the east and west ends of the tunnel. In test area 1 (west end), the water level rose 2.3 m above the tunnel floor in 2.5 minutes. A pressure gauge was installed on the packer and a water pressure of about 15 lb/sq.in. was recorded. In test area 3 (east end), the rise in water level was less rapid (2 m in one hour) and a gauge water pressure of about 3 lb/sq.in. was recorded.

Falling head tests were carried out in test area 5 near the centre of the north tube and in test areas 6 and 8 in the south tube, since there was no positive pressure in these holes. The water test results are given in Appendix 3.

## 9.0 WATER ANALYSES

### 9.1 Water Chemistry

The results of the chemical analyses of water samples are tabulated in Appendix 4. Electrical conductance and pH data for all samples and cation, metal and anion analyses for 12 of the 21 samples are included.

Surface water samples were generally of low electrical conductance and at neutral pH. Concentrations of major anions and cations were also low, especially in the main channel before the canal was filled and, in general, following filling. An exception to these trends occurred in a sample from the small pond adjacent to the East Service Building prior to filling. The water, which was influenced by nearby coal piles, had somewhat elevated values of electrical conductance, and ion and metal concentrations.

The groundwater samples were generally characterized by moderate electrical conductance values and cation concentrations, and near neutral pH values. Concentrations of metals were generally low. Exceptions to these trends occurred in the deepest piezometer at Borehole 101 (Piezometer No. 1 in Rochester Formation) in both the pre- and post-filling stages. Testing on a sample from this piezometer indicates that the electrical conductance was exceptionally high (above 50,000  $\mu\text{S}/\text{cm}$ ) and concentrations of cations (Calcium, Sodium, Magnesium) were also high.

Seepage samples collected from the tunnel sumps had variable characteristics. In the east and central sumps, where water inflows were lowest and where some samples were obtained from the sump floor, electrical conductance and cation concentrations were moderately high. This was likely a consequence of the influence of road salt used to keep the roadway free of ice.

In the west sump, however, inflow of seepage water was quite substantial. In the pre-filling stage, seepage water was of similar characteristics to the shallow groundwater samples obtained from the piezometers and had moderate electrical conductance values and concentrations of anions, cations and metals. In the post-fill stage, seepage water to the west sump was of similar quality to the surface water in the canal. It had low concentrations of anions, cations and metals and low electrical conductance. The similarity in water quality substantiates the direct communication between the canal water and the tunnel backfill which is directly connected to the sump.

The chemistry of water samples obtained from the test areas in the tunnel floor was strongly influenced by contact with the concrete. The pH values were high ( $\approx 12$ ) and electrical conductance was also high. Because of the influence of the concrete, this water had characteristics much different from the source of seepage (canal and groundwater).

## 9.2 Groundwater Flow

Water levels within the canal and the piezometers indicate that groundwater flow in the area of the West Service Building was eastward from the fractured limestone bedrock towards the canal when the canal was dewatered. However, shortly after filling of the canal was initiated, groundwater flow directions reversed and were westward away from the canal.

Prior to the initiation of filling, the hydraulic gradient was approximately 2 per cent eastward from the West Service Building towards the canal. This gradient was calculated on the basis of water level measurements in the canal and in the second shallowest piezometer in each borehole (Piezometers 101-5 and 102-4) -- the groundwater level in the bedrock was not high enough to affect the shallowest piezometers prior to refilling the canal.



Less than five hours after the canal filling had started, the hydraulic gradient had changed to approximately 0.5 per cent westward from the canal towards the West Service Building and the west portal. Eight hours after the filling had begun, the hydraulic gradient had increased to approximately 1.3 per cent westward. As time progressed and the canal filled, the hydraulic gradient continued to increase gradually. Based on water levels in the shallowest piezometers at Boreholes 101 and 102, the hydraulic gradient increased to 1.8 per cent based on water level measurements made on March 18, 1983, four days after the canal had been filled.

Because of the increased hydraulic gradients, it is apparent that flow (seepage) of water from the canal through the rock-fill and the fractured bedrock towards the west portal of the tunnel increased as the water levels in the canal increased. The observed increase in inflow into the west sump following filling of the canal indicates that the concrete seal at the West Service Building is not very effective in restricting flow to the west portal, mainly as the top of seal is 5 ft. lower than the canal water level.

#### 10.0 CAUSE OF PROBLEM

The problem of pavement heaving and surface icing being encountered in the tunnel is associated with seepage into the tunnel. Based on our observations in the tunnel, together with the results of concrete coring in the floor of the tunnel, it is our opinion that the water seepage is primarily from the construction joints, with two principal sources.

Water is coming from the base of the tunnel and produces an upward hydraulic pressure of up to 15 lb/sq.in. at one of our test locations. Based on the results of the rising head and falling head tests, this problem appears to be confined to the east and west sections of the north tube. In these areas, deterioration tends to extend across the width of the tube.

The second source of ingress of water appears to be the vertical wall joints. In this case, there is no upward water pressure in the concrete, indeed, it was possible to get water to penetrate into the concrete as demonstrated by the falling head tests. This was the case in the centre of the north tube and for all tests carried out in the south tube.

It appears that the water from the vertical joints flows into the floor slab construction joint, and, in our opinion, is the reason why the majority of deteriorated areas are within the driving lane.

Laboratory examination of the concrete has shown that the general quality of the concrete is good, although isolated honeycombed areas and cold joints were found. However, a principal factor in the deterioration process is that the concrete is not air-entrained. This has resulted in severe scaling of the concrete surface and joints wherever the ingress of water has provided sufficient saturation of the concrete.

In addition to the deterioration of the concrete, it is thought that the constant moisture flow has resulted in premature asphalt deterioration at the problem areas.

In summary, our investigation has found that the water ingress is extensive and the following conclusions have been made.

1. Water is seeping from the vertical wall joints and flowing into the floor construction joints.
2. In the north tube, water is also coming from the base of the tunnel and is under considerable hydraulic pressure.
3. The concrete is not air-entrained and is susceptible to deterioration due to freeze/thaw action.

4. Previous attempts to eliminate the ingress of water at the construction joints in the floor and walls of the tunnel by grouting methods were not totally successful.

It is essential that these conclusions be considered in any future remedial work.

#### 11.0 RECOMMENDATIONS FOR REMEDIAL WORK

Two basic solutions to the pavement heaving and surface icing problem have been considered, namely, remedial works to prevent seepage into the tunnel, and remedial works to control seepage such that pavement heaving and surface icing are prevented.

##### 11.1 Seepage Prevention

Since the backfill outside the tunnel structure consists of coarse rockfill (5 ft. outside the tunnel walls and varying from about 25 ft. in thickness above the tunnel roof at the low point to nothing immediately west of the East Service Building, and about 35 ft. in thickness between the West Service Building and the canal), it is considered impracticable to provide an effective grout seal outside the tunnel.

Various methods of minimizing access of canal water to the tunnel could be used. For example, consideration could be given to sluicing sand into the rockfill when the canal is drained in order to plug up the flow paths through the rockfill. Some evidence of fine materials being washed into the rockfill was noted in the form of sink holes above the rockfill to the east of the main canal. A clay blanket could then be placed over the rockfill. These methods could be expected to minimize seepage; however, they would not prevent seepage and

would likely be expensive. The main benefit to this approach would be to reduce the amount of pumping time required annually.

A more positive method of seepage prevention would be by grouting the construction joints. However, the investigation has not defined locations where the water is penetrating the waterproofing membranes. The water seepage within the floor slab could conceivably be coming through imperfect seals in the steel plate membrane. Alternatively, it could be coming through thin zones in the "Volclay" membrane along the roof or walls, travelling down the joints to the floor slab. Additionally, previous attempts at preventing ingress of water by grouting have not successfully eliminated the problem. Therefore, even with an extensive grouting program, it is our opinion that elimination of water seepage is not adequately assured using this approach.

#### 11.2 Seepage Control

For the reasons given above, it is our opinion that remedial works to prevent seepage into the tunnel are likely to be expensive without assurance of success. Therefore, we have considered remedial works to control the seepage.

In our opinion, the most expedient means of dealing with the tunnel floor problem is by the provision of drainage facilities beneath the asphaltic concrete surface in conjunction with the existing sewer which presently connects the catch basins to the central sump. However, there are some unknown factors inherent in this solution, and therefore it is recommended that a test area be selected for this remedial treatment and that its behaviour be observed prior to its adoption for remedial works to the entire tunnel. The test area should be within the areas of concentrated problems at the east or west ends of the north tube and should include 6 to 8 consecutive construction joints.

#### 11.2.1 Detailed Remedial Treatment

Since the majority of the seepage is occurring along the construction joints, we suggest that drainage slots be cut along these joints in the floor slab after the asphalt has been stripped (see Figure 12). These drainage slots would be 6 to 8 in. in width and 18 to 20 in. in depth, extending for the full length of the floor joint in each tube. The base of the slot would be sloped down toward the outer curb and a 2 in. diameter hole would be drilled from the base of the slot to allow drainage into the existing sewer line.

A plastic pipe would be fitted into each hole connecting the drainage slot into the sewer. The section of pipe within the slot (extending full height of slot) would be perforated. The slot would then be filled with clean pea gravel.

After the asphaltic concrete has been stripped, the exposed surface of the concrete slab should be thoroughly inspected and any areas of seepage between the joints noted. Construction of the drainage slots should proceed until a full drainage connection is made at all selected locations. The effect of the provision of this drainage on the areas of seepage between the joints should be noted. If necessary, intermediary drainage should be provided between the joint areas.

Based on a freezing index of 500 degree days (F) for the Thorold area, and a coefficient of thermal conductivity of 2.1 Btu/ft.hr. °F for concrete, calculations indicate that the depth of frost penetration into the concrete can be of the order of 33 in. This compares with a predicted depth of penetration of 30 in. into soil based on correlations of

freezing index, field measurements and theoretical considerations by Brown<sup>1</sup> (1964). In order to prevent freezing within the drainage slots, we suggest that heating cables be provided in each slot. However, the effectiveness of insulation to prevent freezing in the test section should also be assessed. In this regard, we propose that selected slots be provided with styrofoam insulation as well as the heating cables. A steel plate would be fitted over each slot, with the heating cables attached to the underside of the plate. Consideration should also be given in final design to providing inspection/servicing access over the area of the perforated pipe. In order to monitor the effectiveness of the ice prevention systems, it is recommended that thermocouples be installed in the drainage slots. The temperature monitoring should proceed without benefit of heat from the heating cables. Heat would be applied only if the monitoring indicates that freezing is occurring.

After installation of all the drainage slots, any deterioration on the surface of the concrete should be removed and replaced with new concrete.

Observations from the test section should provide data with respect to the following points prior to finalizing the remedial works for the entire tunnel.

1. Does the drainage at the joints effect drainage of the slab between the joints?
2. Is the provision of heating cables adequate or necessary to prevent freezing at the drainage slots under all prevailing conditions?

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<sup>1</sup>Brown, W. G., 1964. Difficulties associated with predicting depth of freeze or thaw. Canadian Geotechnical Journal 1: 215-226.

3. Does the provision of drainage slots at the joints alter the load transfer between each slab?
4. In areas where seepage is from the vertical joints, is drainage across the entire width of the floor necessary?

#### 11.2.2 Other Remedial Works

In addition to the remedial work on the concrete base, it is recommended that the repair materials in the vertical joints be removed since they are serving no useful purpose. Their presence retains water in the joints, thus allowing saturation of the concrete surrounding the joint to take place, with resultant freeze/thaw damage.

GOLDER ASSOCIATES

*Alexander Brown*

*per* R. Grieve

*Leo R. Lahti*  
Leo R. Lahti, P. Eng.



RG/LRL/jm

APPENDIX 1

TABULATION OF PACKER  
TEST RESULTS



SUMMARY OF PERMEABILITY TESTS

## Boreholes 101 and 102

|              | Test<br>No. | Test<br>Interval<br>(ft.) | Hydraulic<br>Conductivity<br>(cm/s) |
|--------------|-------------|---------------------------|-------------------------------------|
| Borehole 101 | 1           | 26.3 - 31.7               | $5.5 \times 10^{-3}$                |
|              | 2           | 31.7 - 41.0               | $3.7 \times 10^{-3}$                |
|              | 3           | 40.5 - 51.0               | $2.6 \times 10^{-3}$                |
|              | 4           | 50.8 - 61.0               | $7.3 \times 10^{-4}$                |
|              | 5           | 60.8 - 71.0               | $2.1 \times 10^{-3}$                |
|              | 6           | 70.8 - 81.0               | $1.4 \times 10^{-3}$                |
|              | 7           | 80.8 - 91.0               | $5.5 \times 10^{-5}$                |
|              | 8           | 90.8 - 101.0              | $5.9 \times 10^{-5}$                |
| Borehole 102 | 1           | 15.3 - 21.1               | $7.8 \times 10^{-3}$                |
|              | 2           | 20.3 - 31.1               | $4.9 \times 10^{-3}$                |
|              | 3           | 30.3 - 41.1               | $2.3 \times 10^{-3}$                |
|              | 4           | 40.3 - 51.1               | $2.5 \times 10^{-3}$                |
|              | 5           | 50.3 - 61.1               | $7.6 \times 10^{-5}$                |
|              | 6           | 60.3 - 71.1               | $1.6 \times 10^{-4}$                |
|              | 7           | 70.3 - 81.1               | $1.6 \times 10^{-4}$                |

APPENDIX 2

DETAILS OF FLOOR SLAB  
TEST AREAS

DETAILS OF FLOOR SLAB TEST AREAS

The following are comments regarding water seepage and concrete conditions noted following removal of an approximately 2 ft. x 2 ft. square patch of asphaltic concrete at the test locations.

Test Area 1

1. Water weeping from honeycombed area approximately 1 ft. from the construction joint.
2. Three transverse reinforcing bars exposed in area of water flow.
3. No evidence of reinforcing steel corrosion.
4. Scaled concrete in vicinity of water flow.
5. Construction joint in good condition.
6. Following coring and evacuation of drillwater, core hole was observed for one hour. Approximately 7 in. of water inflow occurred during this period.
7. Area of honeycombing observed in extracted core at approximately 10 in. Cold joints noted at 21 in. and 30 in. Honeycombed region at 47 in.
8. Compressive strength of core section, 32.1 MPa.

Test Area 2

1. Water seeping from previous grout hole within test patch.
2. Surrounding area immediately around the grout hole severely scaled.
3. Concrete surface outside the scaled area in good condition.
4. No exposure of reinforcing steel.
5. Quality of extracted concrete core good.
6. Cold joint at 13 in.
7. Compressive strength of core section, 46.5 MPa.

Test Area 3

1. Asphalt patching extended across the width of the north tube.
2. Severe scaling along the exposed length of the joint.
3. Water lying in the joint.
4. Concrete surface adjacent to joint in fairly good condition.
5. No reinforcing steel exposed.
6. Core taken at the joint showed that scaling occurred to a depth of 8 in.

7. Core showed that quality of concrete generally dense.
8. Core taken close to joint showed cold joint at 16 in. and 24 in.
9. Burlap recovered at a depth of 25 to 31 in.

#### Test Area 4

1. No previous history of repair at this location.
2. Surface of concrete sound - no scaling.
3. No water present.
4. Extracted core showed good quality dense concrete.

#### Test Area 5

1. Asphalt wet - no previous history of repair.
2. Water present in previous grout hole location; however, water appeared to be leaking from small transverse crack within 6 in. of grout hole.
3. Surface of concrete good with no evidence of scaling.

#### Test Area 6

1. Slight moisture on concrete surface. No free water.
2. Six grout holes within test patch. One hole not completely grouted. No apparent flow of water from grout holes.
3. No indication of water seepage into core hole.

4. Moderate scaling on concrete surface and at joint.
5. Extracted core showed good quality dense concrete.
6. Compressive strength of core section, 38.1 MPa.

#### Test Area 7

1. Concrete surface damp. No indication of water seepage.
2. Three transverse rebars and two longitudinal rebars exposed.
3. Concrete around rebars easily dislodged with fingers.
4. Severe scaling of concrete surface; light scaling at construction joint.
5. Following coring and evacuation of drillwater, core hole was observed for 1.5 hours. No evidence of seepage into core hole.
6. Extracted core showed good quality dense concrete.

#### Test Area 8

1. Concrete surface damp. With exposure to air, concrete surface began to dry except along construction joint and at perimeter of test patch at asphalt/concrete surface.
2. Moderate scaling of concrete surface.
3. Light scaling at construction joint.
4. Extracted core showed good quality dense concrete.

Test Area 9

1. Free water noted in test patch.
2. After draining free water, seepage was noted from asphalt/concrete contact. Seepage water collected at construction joint.
3. Deterioration occurring in concrete surface to a depth of 1.5 in. along construction joint.
4. Surface of concrete covered with thin layer of silt extending 24 in. from curb.
5. Light scaling of concrete surface.

Test Area 10

1. Concrete saturated.
2. Deterioration occurring in concrete along construction joint over a width of 16 in. Concrete easily removed with fingers.
3. Severe scaling in surface of concrete close to the joint.
4. Light scaling in general concrete surface.

APPENDIX 3

RISING HEAD/FALLING HEAD TESTS



RIISING HEAD TESTSTEST HOLE C1 (North Tube)TEST HOLE C3 (North Tube)

| <u>Test #1</u>               |                               | <u>Test #2</u>               | <u>Test #1</u>               |                              |
|------------------------------|-------------------------------|------------------------------|------------------------------|------------------------------|
| <u>Time</u><br><u>(min.)</u> | <u>Height*</u><br><u>(mm)</u> | <u>Height</u><br><u>(mm)</u> | <u>Time</u><br><u>(min.)</u> | <u>Height</u><br><u>(mm)</u> |
| 0.00                         | 000                           | 000                          | 0.0                          | 150                          |
| 0.25                         | -                             | 420                          | 1.0                          | 315                          |
| 0.50                         | 556                           | 750                          | 1.5                          | 382                          |
| 0.75                         | 800                           | 1020                         | 2.0                          | 444                          |
| 1.00                         | 990                           | 1270                         | 2.5                          | 493                          |
| 1.25                         | 1150                          | 1500                         | 5.0                          | 723                          |
| 1.50                         | 1330                          | 1690                         | 8                            | 925                          |
| 1.75                         | 1470                          | 1880                         | 10                           | 1035                         |
| 2.00                         | 1600                          | 2040                         | 15                           | 1255                         |
| 2.25                         | 1700                          | 2180                         | 20                           | 1418                         |
| 2.50                         | 1820                          | 2300                         | 30                           | 1635                         |
| 2.75                         | 1940                          |                              | 40                           | 1780                         |
| 3.00                         | 2050                          |                              | 50                           | 1882                         |
| 3.25                         | 2160                          |                              | 60                           | 1960                         |
| 3.50                         | 2260                          |                              |                              |                              |
| 3.75                         | 2360                          |                              |                              |                              |

Note:

Water pressures measured in packers:

Core hole 1    ≈ 15 psi (gauge)

Core hole 3    ≈ 3 psi (gauge)

\*Height above road surface

FALLING HEAD TESTS

| <u>TEST HOLE C5</u><br><u>North Tube</u> |                              | <u>TEST HOLE C6</u><br><u>South Tube</u> |                              | <u>TEST HOLE C8</u><br><u>South Tube</u> |                              |
|--|------------------------------|--|------------------------------|--|------------------------------|
| <u>Time</u><br><u>(min.)</u>             | <u>Height</u><br><u>(mm)</u> | <u>Time</u><br><u>(min.)</u>             | <u>Height</u><br><u>(mm)</u> | <u>Time</u><br><u>(min.)</u>             | <u>Height</u><br><u>(mm)</u> |
| 0.00                                     | 2400                         | 0  | 2410                         | 0.0                                      | 2440                         |
| 0.25                                     | 1950                         | 1  | 2360                         | 0.5                                      | 2310                         |
| 0.50                                     | 1810                         | 2  | 2340                         | 1.0                                      | 2240                         |
| 0.75                                     | 1680                         | 3  | 2325                         | 1.5                                      | 2165                         |
| 1.00                                     | 1570                         | 4  | 2315                         | 2.0                                      | 2020                         |
| 1.25                                     | 1470                         | 5  | 2300                         | 2.5                                      | 1990                         |
| 1.50                                     | 1370                         | 6  | 2285                         | 3.0                                      | 1950                         |
| 1.75                                     | 1290                         | 7  | 2275                         | 3.5                                      | 1890                         |
| 2.00                                     | 1220                         | 8  | 2260                         | 4.0                                      | 1840                         |
| 2.25                                     | 1150                         | 9  | 2245                         | 4.5                                      | 1780                         |
| 2.50                                     | 1080                         | 10                                       | 2235                         | 5.0                                      | 1730                         |
| 3.00                                     | 970                          | 15                                       | 2180                         | 6.0                                      | 1620                         |
| 4.00                                     | 780                          | 20                                       | 2135                         | 7.0                                      | 1530                         |
| 5.00                                     | 630                          | 25                                       | 2085                         | 8.0                                      | 1440                         |
| 6.00                                     | 520                          | 30                                       | 2045                         | 9.0                                      | 1360                         |
| 7.00                                     | 440                          | 35                                       | 2000                         | 10.0                                     | 1290                         |
| 8.00                                     | 370                          | 40                                       | 1960                         | 15.0                                     | 970                          |
| 9.00                                     | 310                          | 45                                       | 1920                         | 20.0                                     | 715                          |
| 10.00                                    | 270                          | 50                                       | 1885                         | 25.0                                     | 530                          |
| 15.00                                    | 150                          | 60                                       | 1800                         | 30.0                                     | 385                          |
| 20.00                                    | 115                          | 70                                       | 1730                         | 35.0                                     | 275                          |
| 25.00                                    | 100                          | 80                                       | 1660                         | 40.0                                     | 195                          |
| 30.00                                    | 98                           | 90                                       | 1590                         | 45.0                                     | 135                          |
| 35.00                                    | 98                           |  |                              | 50.0                                     | 85                           |
|  |                              |  |                              | 55.0                                     | 50                           |

APPENDIX 4

WATER CHEMISTRY

WATER CHEMISTRY

| Sample                       | Temp.<br>(°C) | Electrical Conductance<br>(μS) <sup>1</sup> @ 25°C (μS/cm) |                    | pH   | Location                          |
|------------------------------|---------------|--|--------------------|------|-----------------------------------|
| Surface Water                |               |  |                    |      |                                   |
| * T1                         | 6             | 440  | 760                | 7.3  | Canal 60+00 (PRE) <sup>2</sup>    |
| T2                           | 7             | 700  | 1150               | 6.8  | Above tunnel 66+00 (PRE)          |
| * T3                         | 7             | 2300   | 3800               | 7.1  | East Service Building 71+00 (PRE) |
| * PCAN                       | 9             | 225  | 350                | 6.9  | Canal 67+00 (POST) <sup>3</sup>   |
| Sump Seepage                 |               |  |                    |      |                                   |
| * WS-2                       | 8             | 1000   | 1600               | 6.9  | West Sump (PRE)                   |
| CS                           | 6             | 25000  | 43000              | 7.1  | Central Sump (PRE)                |
| ES                           | 7             | 300  | 500                | 8.6  | East Sump (PRE)                   |
| * PWS                        | 10            | 200  | 300                | 7.1  | West Sump (POST)                  |
| * PCS                        | 8             | 5000   | 8000               | 7.1  | Central Sump (POST)               |
| * PES                        | 8             | 1200   | 1900               | 6.9  | East Sump (POST)                  |
| Test Areas (Floor of Tunnel) |               |  |                    |      |                                   |
| TP-1                         | 6.5           | 12000  | 20000              | 11.4 | Test Area 1 (PRE)                 |
| TP-2                         | 6             | 7500   | 13000              | 12.3 | Test Area 2 (PRE)                 |
| * TP-3                       | 6             | 7500   | 13000              | 12.3 | Test Area 3 (PRE)                 |
| TP-5                         | 6             | 22000  | 37000              | 12.5 | Test Area 5 (PRE)                 |
| Groundwater                  |               |  |                    |      |                                   |
| * MW-1-2                     | 6             | 50000 <sup>+</sup>   | 85000 <sup>+</sup> | 6.5  | Borehole 101, Piezometer 2 (PRE)  |
| * MW-1-4                     | 9             | 1500   | 2550               | 6.8  | 101-4 (PRE)                       |
| * MW-2-1                     | 6             | 900  | 1550               | 6.9  | 102-1 (PRE)                       |
| MW-2-3                       | 2             | 300  | 580                | 6.9  | 102-3 (PRE)                       |
| * PMW-1-2                    | 8             | 50000 <sup>+</sup>   | 80000 <sup>+</sup> | 6.5  | 101-2 (POST)                      |
| PMW-2-1                      | 8             | 5000   | 8000               | 7.1  | 102-1 (POST)                      |
| PMW-2-3                      | 8             | 600  | 960                | 7.1  | 102-3 (POST)                      |

\* Chemical analyses performed - see accompanying data sheets

<sup>1</sup> Conductance value at temperature measured in field (column 2)

<sup>2</sup> PRE - before canal filling

<sup>3</sup> POST - after canal filling

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| BARRINGER MAGENTA LIMITED                                       |        |        |       |        |        | FILE: Y380193<br>DATE: 14/04/83<br>MATRIX: AQ |        |                |        |
|---|--------|--------|-------|--------|--------|---|--------|----------------|--------|
| GOLDER ASSOCIATES.....(D.SMYTH).....REF:831-1038 (PREL.LISTING) |        |        |       |        |        |   |        | MO NO: 83-0193 |        |
| SAMPLE ID   | AG PPM | AL PPM | B PPM | BA PPM | BE PPM | CA PPM  | CD PPM | CO PPM         | CR PPM |
| T1-C  | .009   | .03    | .052  | .144   | <.0005 | 68.4  | .02    | <.05           | <.01   |
| T3-C  | .008   | <.01   | .261  | .074   | <.0005 | 206   | .02    | <.05           | .02    |
| WS-2C   | .009   | <.01   | .355  | .144   | <.0005 | 145   | .01    | <.05           | .01    |
| TP-3C   | .038   | <.01   | .022  | .713   | <.0005 | 641   | .02    | <.05           | <.01   |
| MW-1-2C   | .081   | <.01   | 2.08  | <.005  | <.0005 | 9680  | .10    | <.05           | <.01   |
| MW-1-4C   | .020   | <.01   | .237  | .054   | <.0005 | 287   | .02    | <.05           | .02    |
| MW-2-1C   | .069   | .46    | .182  | .076   | <.0005 | 74.3  | .03    | .27            | .18    |
| PCAN-C  | .031   | <.01   | .026  | .075   | <.0005 | 39.0  | <.01   | <.05           | <.01   |
| PMS-C   | .017   | <.01   | .033  | .074   | <.0005 | 47.4  | <.01   | <.05           | <.01   |
| PCS-C   | .019   | .04    | .122  | .109   | <.0005 | 114   | .02    | <.05           | <.01   |
| PES-C   | .017   | <.01   | .187  | .030   | <.0005 | 165   | .01    | <.05           | .01    |
| PMW-1-2C  | .050   | <.01   | 1.67  | <.005  | <.0005 | 10500   | .10    | <.05           | <.01   |
| SAMPLE ID   | CU PPM | FE PPM | K PPM | MG PPM | MN PPM | MO PPM  | NA PPM | NI PPM         | P PPM  |
| T1-C  | .052   | .08    | 3     | 17.1   | .02    | <.3   | 33     | <.05           | <.6    |
| T3-C  | <.008  | .22    | 7     | 77.4   | .04    | <.3   | 489    | .05            | <.6    |
| WS-2C   | .010   | .08    | 6     | 61.1   | .05    | <.3   | 99     | <.05           | <.6    |
| TP-3C   | .021   | .17    | 238   | .23    | <.01   | <.3   | 644    | .08            | <.6    |
| MW-1-2C   | .605   | .53    | 384   | 3900   | .22    | <.3   | 24800  | .37            | .9     |
| MW-1-4C   | .016   | .78    | 11    | 89.0   | .14    | <.3   | 405    | .05            | <.6    |
| MW-2-1C   | .217   | 1.26   | 9     | 16.1   | .05    | .6  | 64     | .10            | <.6    |
| PCAN-C  | <.008  | .06    | <1    | 8.75   | <.01   | <.3   | 12     | <.05           | <.6    |
| PMS-C   | <.008  | .05    | <1    | 11.0   | <.01   | <.3   | 12     | <.05           | <.6    |
| PCS-C   | .037   | .08    | 42    | 16.3   | .26    | <.3   | 1450   | <.05           | <.6    |
| PES-C   | <.008  | .10    | 4     | 58.9   | .01    | <.3   | 135    | <.05           | <.6    |
| PMW-1-2C  | .135   | .65    | 390   | 4110   | .21    | <.3   | 27300  | .36            | 2.6    |

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| BARRINGER MAGENTA LIMITED                                       |         |          |            |           |           | FILE: T380193<br>DATE: 14/04/83<br>MATRIX: AD |        |        |
|---|---------|----------|------------|-----------|-----------|---|--------|--------|
| GOLDER ASSOCIATES.....(D.SMYTH).....REF:831-1038 (PREL.LISTING) |         |          |            |           |           | MO NO: 83-0193                                |        |        |
| SAMPLE ID   | PB MG/L | SI PPM   | SR PPM     | TH PPM    | TI PPM    | V PPM   | ZN PPM | ZR PPM |
| T1-C  | <.05    | 2.31     | .459       | <.01      | <.005     | <.005   | .23    | <.05   |
| T3-C  | <.05    | 2.24     | 2.57       | <.01      | <.005     | <.005   | .12    | <.05   |
| WS-2C   | <.05    | 5.44     | 2.03       | <.01      | <.005     | <.005   | .18    | <.05   |
| TP-3C   | <.05    | 3.84     | 4.81       | <.01      | <.005     | <.005   | .17    | <.05   |
| MW-1-2C   | .25     | <.05     | 188        | <.01      | <.005     | <.005   | 1.81   | <.05   |
| MW-1-4C   | <.05    | 3.36     | 4.40       | <.01      | <.005     | <.005   | .13    | <.05   |
| MW-2-1C   | <.05    | 2.76     | .627       | .25       | .019      | .044  | .38    | <.05   |
| PCAN-C  | <.05    | 1.32     | .192       | <.01      | <.005     | <.005   | .25    | <.05   |
| PWS-C   | <.05    | 1.73     | .288       | <.01      | <.005     | <.005   | .25    | <.05   |
| PCS-C   | <.05    | 1.72     | .873       | <.01      | <.005     | <.005   | 1.69   | <.05   |
| PE3-C   | <.05    | 3.35     | 2.02       | <.01      | <.005     | <.005   | .17    | <.05   |
| PMW-1-2C  | .20     | <.05     | 202        | <.01      | <.005     | <.005   | 1.37   | <.05   |
|   |         |          |            |           |           |   |        |        |
| SAMPLE ID   | F- MG/L | CL- MG/L | P04-3 MG/L | N03- MG/L | S04= MG/L | BR- MG/L                                      |        |        |
| T1-C  | .61     | 58.5     | <.1        | 2.62      | 125       | <.1   |        |        |
| T3-C  | 4.16    | -----    | <.1        | <.05      | 662       | .9  |        |        |
| WS-2C   | 1.43    | 161      | <.1        | .70       | 321       | .6  |        |        |
| TP-3C   | .64     | -----    | <.1        | <.05      | 7.52      | 5.1   |        |        |
| MW-1-2C   | -----   | -----    | -----      | -----     | -----     | -----   |        |        |
| MW-1-4C   | -----   | -----    | -----      | -----     | -----     | -----   |        |        |
| MW-2-1C   | 2.86    | -----    | <.1        | <.05      | 433       | 4.7   |        |        |
| PCAN-C  | .26     | 28.3     | <.1        | 1.12      | 33.0      | <.1   |        |        |
| PWS-C   | .30     | 27.1     | <.1        | .43       | 48.2      | <.1   |        |        |
| PCS-C   | .30     | 27.1     | <.1        | .43       | 48.2      | <.1   |        |        |
| PE3-C PCS-C   | 8.59    | -----    | <.1        | <.05      | 321       | 1.4   |        |        |
| PMW-1-2C PCS-C  | 1.51    | 238      | <.1        | 1.10      | 455       | .6  |        |        |
| PMW-1-2C  |         |          |            |           |           |   |        |        |

**BARRINGER MAGENTA**

304 CARLINGVIEW DRIVE  
REXDALE, ONTARIO  
M9W 5G2  
(416) 875-3870

3750 - 18TH STREET  
SUITE 105  
CALGARY, ALBERTA  
T2E 6V2  
(403) 276-8701

FILE: T3-0193  
DATE: 14/04/83  
MATRIX: AQ

GOLDER ASSOCIATES,....(D.SMYTH)....REF:831-1038

NO NO: 83 0193

| SAMPLE ID | AG<br>MG/L | AL<br>MG/L | B<br>MG/L | BA<br>MG/L | BE<br>MG/L | CA<br>MG/L | CU<br>MG/L | CI<br>MG/L | CR<br>MG/L |
|-----------|------------|------------|-----------|------------|------------|------------|------------|------------|------------|
| T1        | .009       | .03        | .052      | .144       | <.0005     | 68.4       | .02        | <.05       | <.01       |
| T3        | .008       | <.01       | .261      | .074       | <.0005     | 206        | .02        | <.05       | .02        |
| WS-2      | .009       | <.01       | .355      | .144       | <.0005     | 145        | .01        | <.05       | .01        |
| TP-3      | .038       | <.01       | .022      | .713       | <.0005     | 641        | .02        | <.05       | <.01       |
| MW-1-2    | .081       | <.01       | 2.08      | <.005      | <.0005     | 9680       | .10        | <.05       | <.01       |
| MW-1-4    | .020       | <.01       | .237      | .054       | <.0005     | 287        | .02        | <.05       | .02        |
| MW-2-1    | .069       | .46        | .182      | .076       | <.0005     | 74.3       | .03        | .27        | .18        |
| PCAN      | .031       | <.01       | .026      | .075       | <.0005     | 39.0       | <.01       | <.05       | <.01       |
| PWS       | .017       | <.01       | .033      | .074       | <.0005     | 47.4       | <.01       | <.05       | <.01       |
| PCS       | .019       | .04        | .122      | .109       | <.0005     | 114        | .02        | <.05       | <.01       |
| PES       | .017       | <.01       | .187      | .030       | <.0005     | 165        | .01        | <.05       | .01        |
| PMW-1-2   | .050       | <.01       | 1.67      | <.005      | <.0005     | 10500      | .10        | <.05       | <.01       |
|           |            |            |           |            |            |            |            |            |            |
| SAMPLE ID | CU<br>MG/L | FE<br>MG/L | K<br>MG/L | MG<br>MG/L | MN<br>MG/L | MO<br>MG/L | NA<br>MG/L | NI<br>MG/L | P<br>MG/L  |
| T1        | .052       | .08        | 3         | 17.1       | .02        | <.3        | 33         | <.05       | <.6        |
| T3        | <.008      | .22        | 7         | 77.4       | .04        | <.3        | 489        | <.05       | <.6        |
| WS-2      | .010       | .08        | 6         | 61.1       | .05        | <.3        | 99         | <.05       | <.6        |
| TP-3      | .021       | .17        | 238       | .23        | <.01       | <.3        | 644        | .08        | <.6        |
| MW-1-2    | .605       | .53        | 384       | 3900       | .22        | <.3        | 24800      | .37        | .9         |
| MW-1-4    | .016       | .78        | 11        | 89.0       | .14        | <.3        | 405        | .05        | <.6        |
| MW-2-1    | .217       | 1.26       | 9         | 16.1       | .05        | .6         | 64         | .10        | <.6        |
| PCAN      | <.008      | .06        | <1        | 8.75       | <.01       | <.3        | 12         | <.05       | <.6        |
| PWS       | <.008      | .05        | <1        | 11.0       | <.01       | <.3        | 12         | <.05       | <.6        |
| PCS       | .037       | .08        | 42        | 16.3       | .26        | <.3        | 1450       | <.05       | <.6        |
| PES       | <.008      | .10        | 4         | 58.9       | .01        | <.3        | 135        | <.05       | <.6        |
| PMW-1-2   | .135       | .65        | 390       | 4110       | .21        | <.3        | 27300      | .36        | 2.6        |

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304 CARLINGVIEW DRIVE  
REXDALE, ONTARIO  
M9W 5G2  
(416) 875-3870

3750 - 18TH STREET  
SUITE 108  
CALGARY, ALBERTA  
T2E 6V2  
(403) 276-9701

FILE: T3-0193  
DATE: 14/04/83  
MATRIX: AQ

GOLDER ASSOCIATES.....(D.SMYTH).....REF:831-1038

WO NO: 83

| SAMPLE ID | PB<br>MG/L | SI<br>MG/L  | SR<br>MG/L    | TH<br>MG/L   | TI<br>MG/L   | V<br>MG/L   | ZN<br>MG/L | ZR<br>MG/L |
|-----------|------------|-------------|---------------|--------------|--------------|-------------|------------|------------|
| T1        | <.05       | 2.31        | .459          | <.01         | <.005        | <.005       | .23        | <.05       |
| T3        | <.05       | 2.24        | 2.57          | <.01         | <.005        | <.005       | .12        | <.05       |
| WS-2      | <.05       | 5.44        | 2.03          | <.01         | <.005        | <.005       | .18        | <.05       |
| TP-3      | <.05       | 3.84        | 4.81          | <.01         | <.005        | <.005       | .17        | <.05       |
| MW-1-2    | .25        | <.05        | 188           | <.01         | <.005        | <.005       | 1.81       | <.05       |
| MW-1-4    | <.05       | 3.36        | 4.40          | <.01         | <.005        | <.005       | .13        | <.05       |
| MW-2-1    | <.05       | 2.76        | .627          | .25          | .019         | .044        | .38        | <.05       |
| PCAN      | <.05       | 1.32        | .192          | <.01         | <.005        | <.005       | .25        | <.05       |
| PWS       | <.05       | 1.73        | .288          | <.01         | <.005        | <.005       | .25        | <.05       |
| PCS       | <.05       | 1.72        | .873          | <.01         | <.005        | <.005       | 1.69       | <.05       |
| PES       | <.05       | 3.35        | 2.02          | <.01         | <.005        | <.005       | .17        | <.05       |
| PMW-1-2   | <.05       | <.05        | 202           | <.01         | <.005        | <.005       | 1.37       | <.05       |
|           |            |             |               |              |              |             |            |            |
| SAMPLE ID | F-<br>MG/L | CL-<br>MG/L | PO4-3<br>MG/L | NO3-<br>MG/L | SO4=<br>MG/L | BR-<br>MG/L |            |            |
| T1        | .61        | 58.5        | <.1           | 2.62         | 125          | <.1         |            |            |
| T3        | 4.16       | 794         | <1.0          | <.50         | 662          | .9          |            |            |
| WS-2      | 1.43       | 161         | <.1           | .70          | 321          | .6          |            |            |
| TP-3      | .64        | 814         | <1.0          | <.05         | 7.52         | 5.1         |            |            |
| MW-1-2    | 291        | 100000      | <100          | <5.00        | 788          | 1030        |            |            |
| MW-1-4    | 3.94       | 1120        | <1.0          | <.50         | 442          | 9.7         |            |            |
| MW-2-1    | 2.86       | 573         | <1.0          | <.50         | 433          | 4.7         |            |            |
| PCAN      | .26        | 28.3        | <.1           | 1.12         | 33.0         | <.1         |            |            |
| PWS       | .30        | 27.1        | <.1           | .43          | 48.2         | <.1         |            |            |
| PCS       | 8.59       | 2350        | <1.0          | <.50         | 321          | 1.4         |            |            |
| PES       | 1.51       | 238         | 1.0           | <1.10        | 455          | .6          |            |            |
| PMW-1-2   | 163        | 68600       | <100          | <5.00        | 1170         | 710         |            |            |

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## LIST OF ABBREVIATIONS

The abbreviation commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

### I. SAMPLE TYPES

*AS* auger sample  
*CS* chunk sample  
*DO* drive open  
*DS* Denison type sample  
*FS* foil sample  
*RC* rock core  
*ST* slotted tube  
*TO* thin-walled, open  
*TP* thin-walled, piston  
*WS* wash sample

### II. PENETRATION RESISTANCES

#### Dynamic Penetration Resistance:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

#### Standard Penetration Resistance, *N*:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

*WH* sampler advanced by static weight—weight, hammer

*PH* sampler advanced by pressure—pressure, hydraulic

*PM* sampler advanced by pressure—pressure, manual

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

| <i>Relative Density</i> | <i>'N'</i><br><u>Blows/0.30m</u><br>or <u>Blows/ft.</u> |
|-------------------------|---|
| Very loose              | 0 to 4  |
| Loose                   | 4 to 10   |
| Compact                 | 10 to 30  |
| Dense                   | 30 to 50  |
| Very dense              | over 50   |

#### (b) Cohesive Soils

| <i>Consistency</i> | <u>kPa</u> | <i>'Cu'</i><br><u>psf.</u> |
|--------------------|------------|----------------------------|
| Very soft          | 0 to 12    | 0 to 250                   |
| Soft               | 12 to 25   | 250 to 500                 |
| Firm               | 25 to 50   | 500 to 1000                |
| Stiff              | 50 to 100  | 1000 to 2000               |
| Very stiff         | 100 to 200 | 2000 to 4000               |
| Hard               | over 200   | over 4000                  |

### IV. SOIL TESTS

*C* consolidation test  
*H* hydrometer analysis  
*M* sieve analysis  
*MH* combined analysis, sieve and hydrometer<sup>1</sup>  
*Q* undrained triaxial<sup>2</sup>  
*R* consolidated undrained triaxial<sup>2</sup>  
*S* drained triaxial  
*U* unconfined compression  
*V* field vane test

#### NOTES:

<sup>1</sup>Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

<sup>2</sup>Undrained triaxial tests in which pore pressures are measured are shown as  $\bar{Q}$  or  $\bar{R}$ .

# LIST OF SYMBOLS

## I. GENERAL

|                           |                                     |
|---------------------------|-------------------------------------|
| $\tau$                    | = 3.1416                            |
| $e$                       | = base of natural logarithms 2.7183 |
| $\log_e a$ or $\ln a$     | natural logarithm of $a$            |
| $\log_{10} a$ or $\log a$ | logarithm of $a$ to base 10         |
| $t$                       | time                                |
| $g$                       | acceleration due to gravity         |
| $V$                       | volume                              |
| $W$                       | weight                              |
| $M$                       | moment                              |
| $F$                       | factor of safety                    |

## II. STRESS AND STRAIN

|                 |  |
|-----------------|--|
| $u$             | pore pressure  |
| $\sigma$        | normal stress  |
| $\sigma'$       | normal effective stress ( $\bar{\sigma}$ is also used) |
| $\tau$          | shear stress   |
| $\epsilon$      | linear strain  |
| $\epsilon_{xy}$ | shear strain   |
| $\nu$           | Poisson's ratio ( $\mu$ is also used)                  |
| $E$             | modulus of linear deformation (Young's modulus)        |
| $G$             | modulus of shear deformation                           |
| $K$             | modulus of compressibility                             |
| $\eta$          | coefficient of viscosity                               |

## III. SOIL PROPERTIES

### (a) Unit weight

|            |   |
|------------|---|
| $\gamma$   | unit weight of soil (bulk density)                              |
| $\gamma_s$ | unit weight of solid particles                                  |
| $\gamma_w$ | unit weight of water  |
| $\gamma_d$ | unit dry weight of soil (dry density)                           |
| $\gamma'$  | unit weight of submerged soil                                   |
| $G_s$      | specific gravity of solid particles $G_s = \gamma_s / \gamma_w$ |
| $e$        | void ratio  |
| $n$        | porosity  |
| $w$        | water content   |
| $S_r$      | degree of saturation  |

### (b) Consistency

|           |  |
|-----------|--|
| $w_L$     | liquid limit   |
| $w_P$     | plastic limit  |
| $I_P$     | plasticity index   |
| $w_S$     | shrinkage limit  |
| $I_L$     | liquidity index = $(w - w_P) / I_P$                      |
| $I_C$     | consistency index = $(w_L - w) / I_P$                    |
| $e_{max}$ | void ratio in loosest state                              |
| $e_{min}$ | void ratio in densest state                              |
| $D_r$     | relative density = $(e_{max} - e) / (e_{max} - e_{min})$ |

### (c) Permeability

|     |                               |
|-----|-------------------------------|
| $h$ | hydraulic head or potential   |
| $q$ | rate of discharge             |
| $v$ | velocity of flow              |
| $i$ | hydraulic gradient            |
| $k$ | coefficient of permeability   |
| $j$ | seepage force per unit volume |

### (d) Consolidation (one-dimensional)

|       |  |
|-------|--|
| $m_v$ | coefficient of volume change<br>= $-\Delta e / (1 + e) \Delta \sigma'$ |
| $C_c$ | compression index = $-\Delta e / \Delta \log_{10} \sigma'$             |
| $c_v$ | coefficient of consolidation   |
| $T_v$ | time factor = $c_v t / d^2$ ( $d$ , drainage path)                     |
| $U$   | degree of consolidation  |

### (e) Shear strength

|          |   |
|----------|---|
| $\tau_f$ | shear strength                                      |
| $c'$     | effective cohesion                                  |
| $\phi'$  | effective angle of shearing resistance, or friction |
| $c_u$    | apparent cohesion*                                  |
| $\phi_u$ | apparent angle of shearing resistance, or friction  |
| $\mu$    | coefficient of friction                             |
| $S_r$    | sensitivity   |

\*For the case of a saturated cohesive soil,  $\phi_u = 0$  and the undrained shear strength  $\tau_f = c_u$  is taken as half the undrained compressive strength.

# RECORD OF BOREHOLE 101

SHEET 1 OF 2

LOCATION SEE FIGURE 2

BORING DATE FEB. 14-17 AND 23, 1983

DATUM GEODETIC

INCLINATION 90° AZIMUTH

SAMPLER / PENETRATION TEST HAMMER WEIGHT

DROP

| DEPTH<br>FEET                   | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV.<br>DEPTH<br>(ft.) | SAMPLES |      | WATER CONTENT, % |                  |                  |             | DISCONTINUITY DATA | HYDRAULIC<br>CONDUCTIVITY<br>k, cm/sec | ADDITIONAL<br>TESTING | INSTRUMENTATION |                              |                         |
|---------------------------------|-----------------|-------------|--------------|-------------------------|---------|------|------------------|------------------|------------------|-------------|--------------------|--|-----------------------|-----------------|------------------------------|-------------------------|
|                                 |                 |             |              |                         | NUMBER  | TYPE | BLOWS /FOOT      | RECOVERY         |                  | R.Q.D.<br>% |                    |  |                       |                 | FRACTURE<br>INDEX<br>PER FT. | DIP<br>WRT CORE<br>AXIS |
|                                 |                 |             |              |                         |         |      |                  | TOTAL<br>CORE, % | SOLID<br>CORE, % |             |                    |  |                       |                 |                              |                         |
|                                 |                 |             |              |                         |         |      |                  | W <sub>p</sub>   | W <sub>i</sub>   |             |                    |  |                       |                 |                              |                         |
| TYPE AND SURFACE<br>DESCRIPTION |                 |             |              |                         |         |      |                  |                  |                  |             |                    |  |                       |                 |                              |                         |

|    |           |   |  |       |   |    |    |  |  |  |  |  |  |  |
|----|-----------|---|--|-------|---|----|----|--|--|--|--|--|--|--|
| 0  |           | GROUND SURFACE  |  | 587.6 |   |    |    |  |  |  |  |  |  |  |
|    |           |   |  | 0.0   |   |    |    |  |  |  |  |  |  |  |
| 10 | NW CASING | OVERBURDEN  |  |       |   |    |    |  |  |  |  |  |  |  |
| 20 |           | BEDROCK SURFACE   |  | 564.6 |   |    |    |  |  |  |  |  |  |  |
|    |           |   |  | 23.0  | 1 | NQ | RC |  |  |  |  |  |  |  |
| 30 |           | LOCKPORT FORMATION<br>GOAT ISLAND MEMBER<br>FAINTLY WEATHERED TO FRESH<br>FAINTLY TO MODERATELY<br>WEATHERED ON JOINT SURFACES,<br>LIGHT BROWNISH GREY FINE<br>GRAINED MEDIUM TO THICKLY BEDDED<br>DOLOSTONE CONTAINS OCCASIONAL<br>VUGS AND STYOLITES AND MINOR<br>FOSSILS |  |       | 2 | "  | "  |  |  |  |  |  |  |  |
| 40 |           |   |  | 541.6 |   |    |    |  |  |  |  |  |  |  |
|    |           |   |  | 46.0  | 4 | "  | "  |  |  |  |  |  |  |  |
| 50 |           | LOCKPORT FORMATION<br>GASPORT MEMBER<br>FRESH, LIGHT GREY, FINE GRAINED,<br>MASSIVE TEXTURED, STYOLITIC, THICKLY<br>BEDDED DOLOMITIC LIMESTONE  |  | 535.6 |   |    |    |  |  |  |  |  |  |  |
|    |           |   |  | 52.0  |   |    |    |  |  |  |  |  |  |  |
| 60 |           | GASPORT MEMBER<br>FRESH, FAINTLY WEATHERED ON<br>JOINT SURFACES, DARK GREY, FINE<br>GRAINED MEDIUM TO THINLY<br>BEDDED, FOSSILIFEROUS, SHALEY,<br>CALCAREOUS DOLOSTONE  |  | 527.6 |   |    |    |  |  |  |  |  |  |  |
|    |           |   |  | 60.0  |   |    |    |  |  |  |  |  |  |  |
| 70 |           | GASPORT MEMBER<br>FRESH, MEDIUM GREY, FINE TO<br>MEDIUM GRAINED, THIN TO MEDIUM<br>BEDDED FOSSILIFEROUS DOLOMITIC<br>LIMESTONE, GRADATIONAL CONTACT<br>WITH OVERLYING AND UNDERLYING<br>BEDS  |  | 515.1 |   |    |    |  |  |  |  |  |  |  |
|    |           |   |  | 72.5  |   |    |    |  |  |  |  |  |  |  |
| 80 |           | DECEW FORMATION<br>FRESH, MEDIUM BROWNISH GREY,<br>FINE GRAINED, MEDIUM TO THICKLY<br>BEDDED, LAMINAR CROSS BEDDED<br>CALCAREOUS DOLOSTONE, GRADES<br>WITH UNDERLYING SHALE   |  |       | 7 | "  | "  |  |  |  |  |  |  |  |
|    |           |   |  |       |   |    |    |  |  |  |  |  |  |  |

CONTINUED ON SHEET 2

FAINTLY TO MODERATELY WEATHERED BEDDING SEPARATION (80-90° TCA) AND NEAR VERTICAL JOINTS (0-10° TCA)

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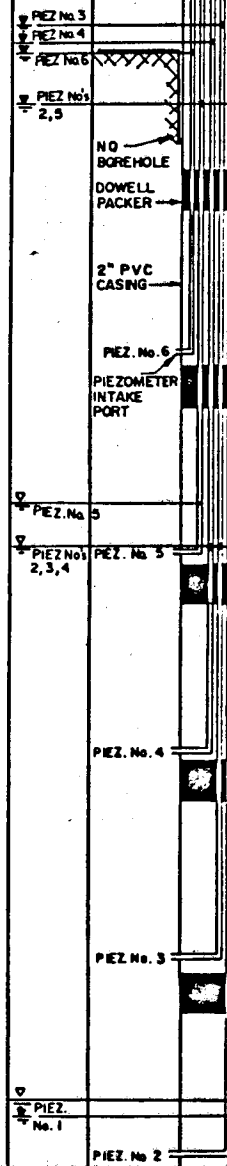
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FAINTLY TO  
MODERATELY  
WEATHERED BEDDING  
SEPARATION (80-90°  
TCA) AND NEAR  
VERTICAL JOINTS  
(0-10° TCA)



DEPTH SCALE (ALONG HOLE)  
1 INCH TO 10 FEET

Golder Associates

ROCK LOGGED *PS* DRAWN *CM*  
DATE FEB. 23/83 CHECKED *RB*

# RECORD OF BOREHOLE 101 cont'd

SHEET 2 OF 2

LOCATION SEE FIGURE 2

BORING DATE FEB. 13 - 17 AND 23, 1983

DATUM GEODETIC

INCLINATION 90° AZIMUTH

SAMPLER / PENETRATION TEST HAMMER WEIGHT

DROP

| DEPTH, FEET | DRILLING RECORD                 | DESCRIPTION   | SYMBOLIC LOG | ELEV. DEPTH (ft.) | SAMPLES |          | WATER CONTENT, % |               |          |                        | DISCONTINUITY DATA |                              | HYDRAULIC CONDUCTIVITY |                  |                  | ADDITIONAL TESTING | INSTRUMENTATION  |
|-------------|---------------------------------|---|--------------|-------------------|---------|----------|------------------|---------------|----------|------------------------|--------------------|------------------------------|------------------------|------------------|------------------|--------------------|--|
|             |                                 |   |              |                   | NUMBER  | TYPE     | RECOVERY         |               | R.Q.D. % | FRACTURE INDEX PER FT. | DIPS W/ CORE AXIS  | TYPE AND SURFACE DESCRIPTION | k, cm/sec.             |                  |                  |                    |  |
|             |                                 |   |              |                   |         |          | TOTAL CORE, %    | SOLID CORE, % |          |                        |                    |                              | H <sub>2</sub> O       | H <sub>2</sub> O | H <sub>2</sub> O |                    |  |
| 80          | ROTARY DRILLING<br>NO ROCK CORE | CONTINUED FROM SHEET 1<br>FOR DESCRIPTION SEE PREVIOUS SHEET  |              | 504.1             |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    |  |
|             |                                 | ROCHESTER FORMATION<br>FRESH, MEDIUM TO DARK BROWNISH<br>GREY, VERY FINE GRAINED, THINLY<br>TO MEDIUM BEDDED, MODERATELY<br>FRIABLE, CALCAREOUS SHALE |              | 83.5              | 8       | NQ<br>RC |                  |               |          |                        |                    |                              |                        |                  |                  |                    |  |
| 90          |                                 |   |              |                   | 9       | "        |                  |               |          |                        |                    |                              |                        |                  |                  |                    |  |
| 100         |                                 | END OF HOLE   |              | 486.6<br>101.0    |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZ. No. J  |
| 110         |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZOMETER WATER LEVELS<br>(MARCH 13, 1983<br>0030 hrs.) |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZOMETER 1<br>ELEV. 511.1<br>DEPTH 76.5                |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZOMETER 2<br>ELEV. 539.4<br>DEPTH 48.2                |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZOMETER 3<br>ELEV. 539.6<br>DEPTH 48.0                |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZOMETER 4<br>ELEV. 538.4<br>DEPTH 48.2                |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZOMETER 5<br>ELEV. 541.7<br>DEPTH 43.9                |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZOMETER 6<br>ELEV. DRY<br>DEPTH DRY                   |
| 120         |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | (MARCH 14, 1983<br>1500 hrs.)                            |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | PIEZ. ELEV. DEPTH  |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | 1 510.4 77.2   |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | 2 562.2 25.4   |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | 3 566.0 21.6   |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | 4 565.1 22.5   |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | 5 562.2 25.4   |
|             |                                 |   |              |                   |         |          |                  |               |          |                        |                    |                              |                        |                  |                  |                    | 6 564.6 23.0   |

DEPTH SCALE (ALONG HOLE)  
1 INCH TO 10 FEET

Golder Associates

ROCK LOGGED RA DRAWN DM  
DATE FEB. 25/83 CHECKED RA

**SHEET 1 OF 2**

BORING DATE FEB. 18, 21-23, 1983

**DATUM      GEODETIC**

SAMPLER / PENETRATION TEST HAMMER WEIGHT

**DROP** 

CONTINUED ON SHEET 2

## Golder Associates

ROCK LOGGED RB DRAWN DM  
DATE PER 25/83 CHECKED RB

# RECORD OF BOREHOLE 102 cont'd

SHEET 2 OF 2

LOCATION SEE FIGURE 2

BORING DATE FEB. 18, 21-23, 1983

DATUM GEODETIC

INCLINATION 90° AZIMUTH

SAMPLER / PENETRATION TEST HAMMER WEIGHT

DROP

| DEPTH<br>FEET | DRILLING RECORD | DESCRIPTION  | SYMBOLIC LOG | ELEV.<br>DEPTH<br>(ft.) | SAMPLES |      | WATER CONTENT, % |                  |                  | DISCONTINUITY DATA |                             | HYDRAULIC<br>CONDUCTIVITY<br>K, cm/sec | ADDITIONAL<br>TESTING<br>WATER<br>LEVELS<br>AND NOTES | INSTRUMENTATION |          |                                 |
|---------------|-----------------|--|--------------|-------------------------|---------|------|------------------|------------------|------------------|--------------------|-----------------------------|--|---|-----------------|----------|---------------------------------|
|               |                 |  |              |                         | NUMBER  | TYPE | BLOWS/FOOT       | RECOVERY         |                  | R.Q.D.<br>%        | FRACTURE<br>INDEX<br>PER FT |  |   |                 | DIP<br>° | TYPE AND SURFACE<br>DESCRIPTION |
|               |                 |  |              |                         |         |      |                  | TOTAL<br>CORE, % | SOLID<br>CORE, % |                    |                             |  |   |                 |          |                                 |
| 80            |                 | CONTINUED FROM SHEET 1<br>SEE SHEET 1 FOR DETAILS<br>END OF HOLE |              | 81.1                    |         |      |                  |                  |                  |                    |                             |  |   |                 |          |                                 |
| 90            |                 |  |              |                         |         |      |                  |                  |                  |                    |                             |  |   |                 |          |                                 |

PIEZOMETER WATER  
LEVELS  
(MARCH 13, 1983)  
0030 HRS.)  
PIEZOMETER 1  
ELEV. 540.7  
DEPTH 34.0  
PIEZOMETER 2  
ELEV. 540.7  
DEPTH 34.0  
PIEZOMETER 3  
ELEV. 540.8  
DEPTH 33.9  
PIEZOMETER 4  
ELEV. 542.6  
DEPTH 32.1  
PIEZOMETER 5  
ELEV. 548.2  
DEPTH 25.5  
(MARCH 14, 1983  
1500 HRS.)  
PIEZ. ELEV. DEPTH  
1 568.7 6.0  
2 568.7 6.0  
3 568.7 6.0  
4 567.0 7.7  
5 567.2 7.5

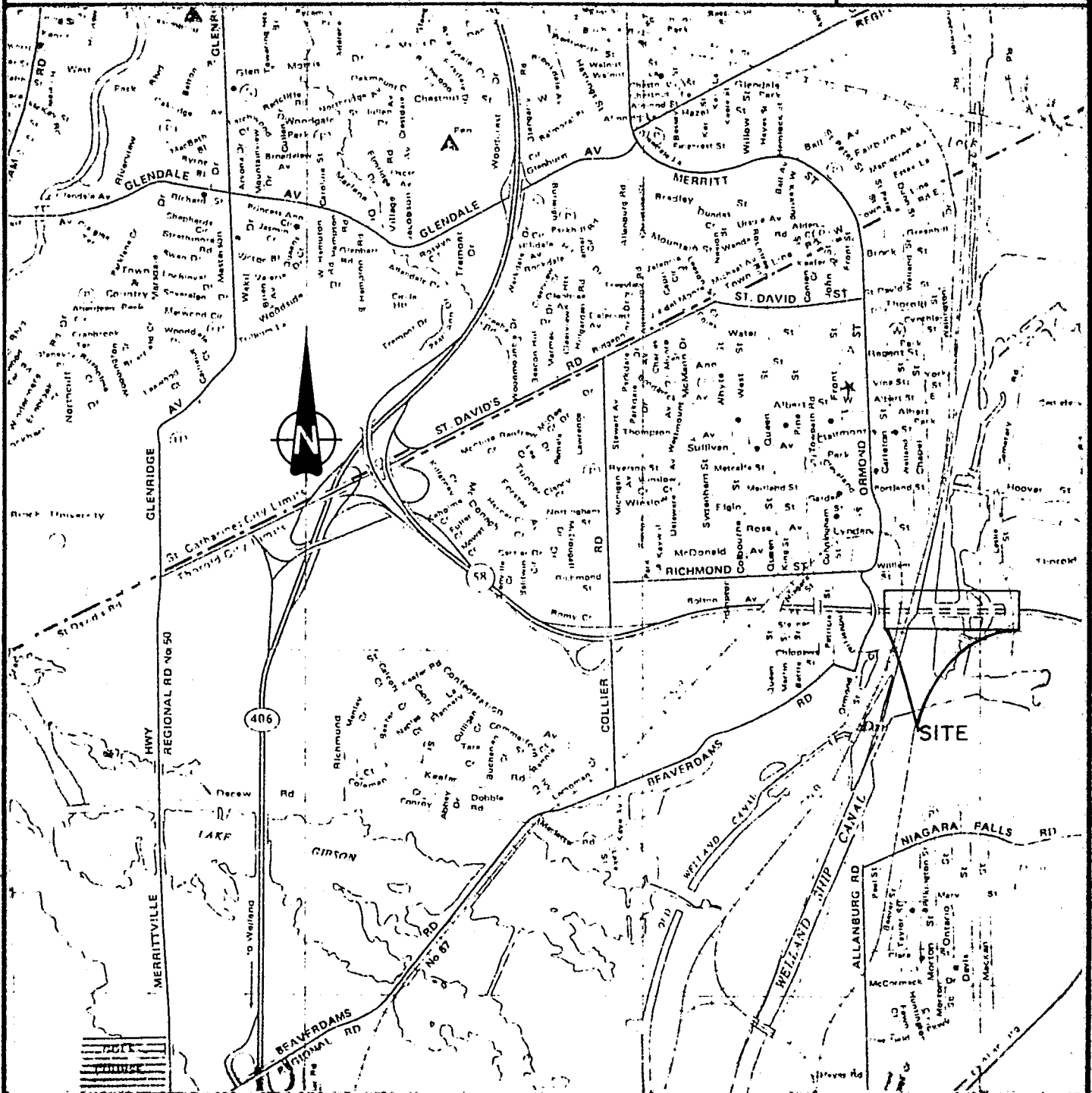
DEPTH SCALE (ALONG HOLE)  
1 INCH TO 10 FEET

Golder Associates

ROCK LOGGED RB DRAWN D.M.  
DATE FEB. 25/83 CHECKED RB

# SITE LOCATION PLAN

FIGURE 1



SCALE 1" to 2500' (approx.)

Date APRIL 15, 1983  
Project No. 83J-1038

**Golder Associates**

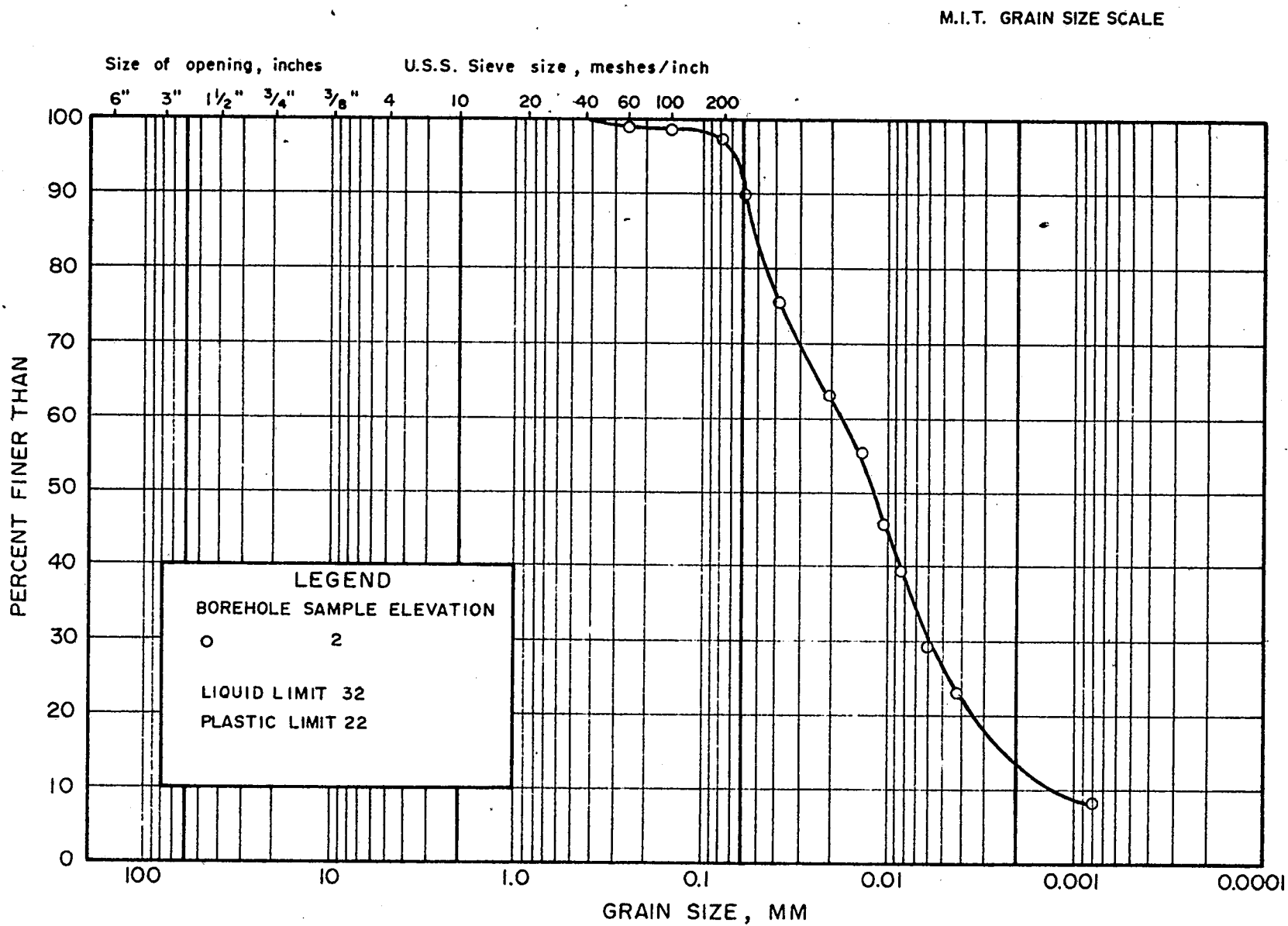
Drawn RWR  
Chkd. AM







Golder Associates



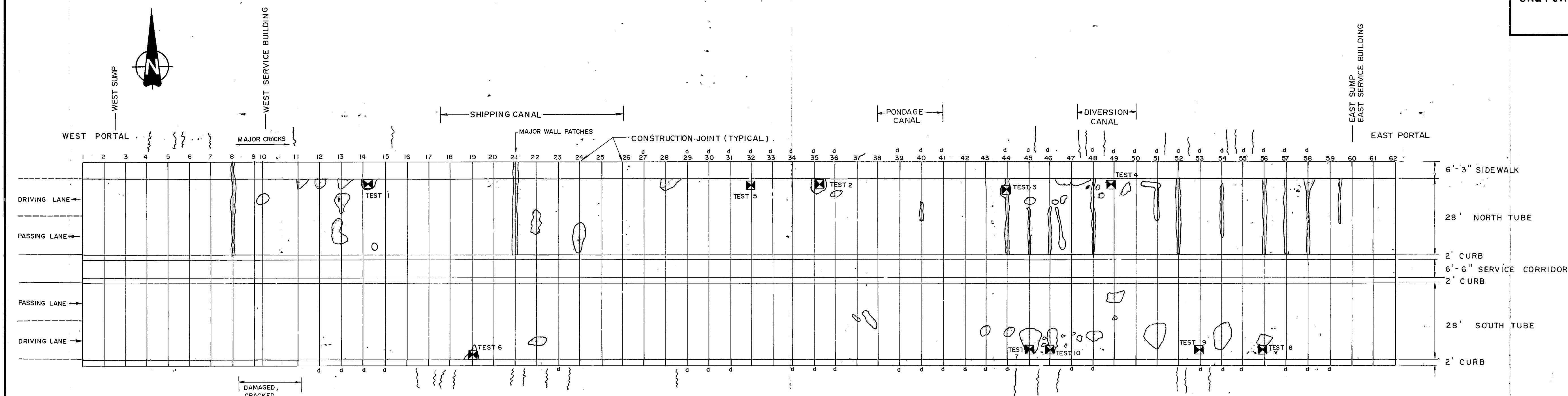
**GRAIN SIZE DISTRIBUTION**  
CLAYEY SILT OVER TUNNEL BETWEEN  
PONDAGE CANAL AND MAIN CANAL

FIGURE 3

| COBBLE<br>SIZE | COARSE      | MEDIUM | FINE | COARSE    | MEDIUM | FINE | SILT SIZE    |  | CLAY SIZE |  |
|----------------|-------------|--------|------|-----------|--------|------|--------------|--|-----------|--|
|                | GRAVEL SIZE |        |      | SAND SIZE |        |      | FINE GRAINED |  |           |  |

SKETCH SHOWING ASPHALT PATCHES  
AND TEST LOCATIONS  
TUNNEL FLOOR

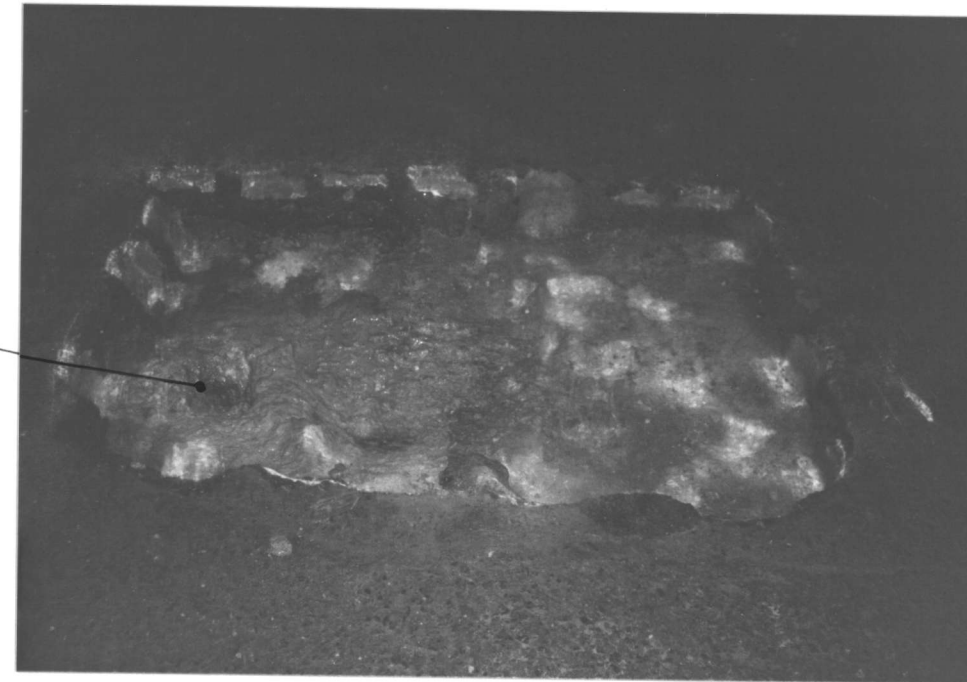
FIGURE 4



- LEGEND
- ⊗ LOCATION OF TEST AREA
  - ASPHALT PATCH
  - ~ CRACK IN TUNNEL WALL
  - d DRILL HOLES FROM PREVIOUS GROUTING

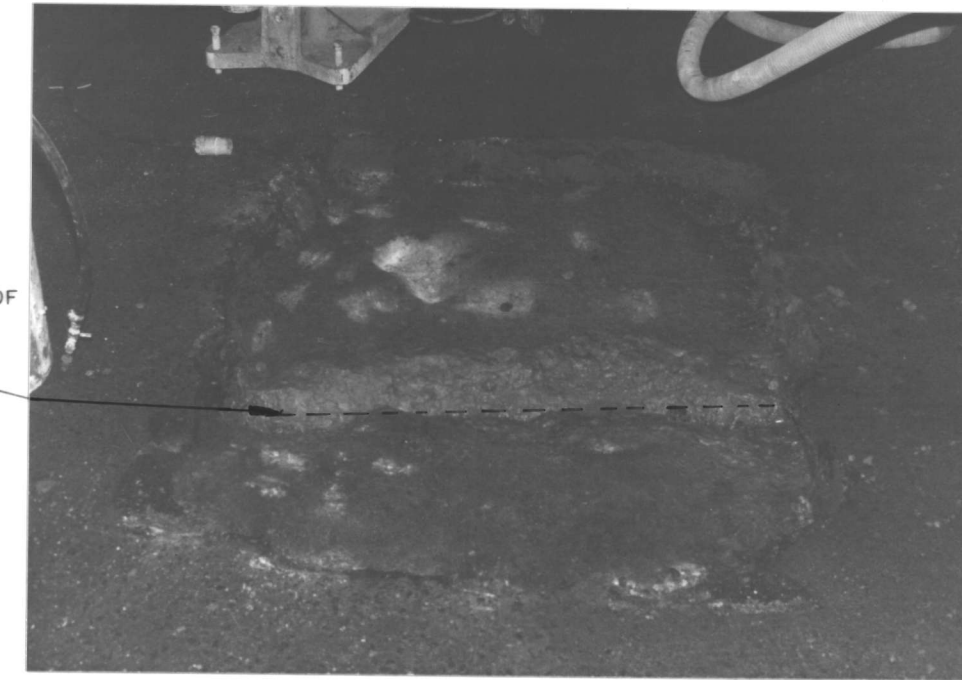
SCALE : HORIZONTAL 1" TO 100'  
WIDTH 1" TO 20'

WATER SEEPING  
FROM PREVIOUS  
GROUTHOLE



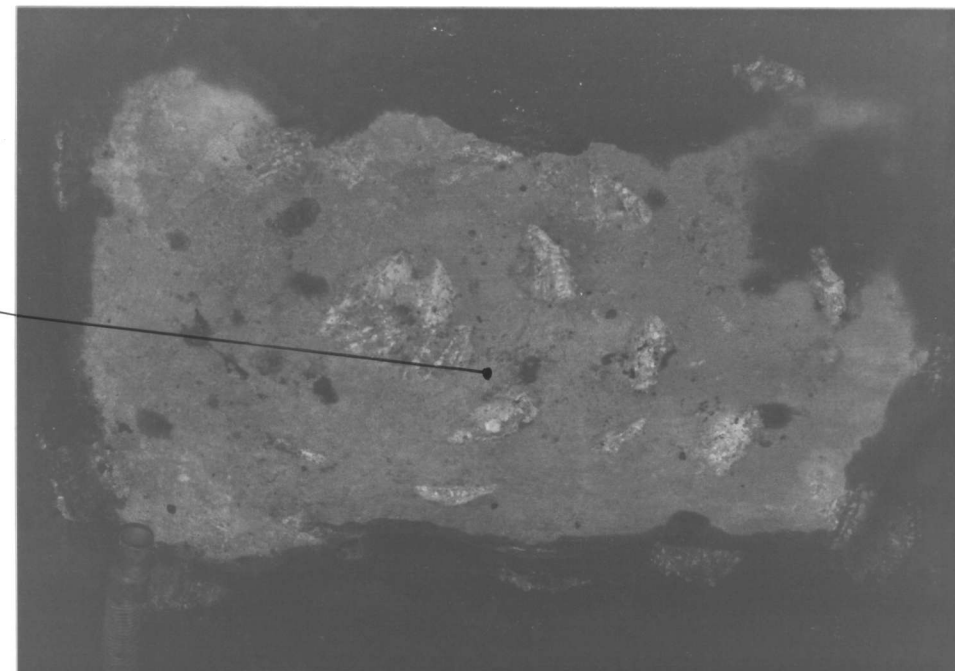
TEST LOCATION 2  
ASPHALT PATCHING CLOSE TO JOINT

DETERIORATION  
ALONG LENGTH OF  
JOINT



TEST LOCATION 3  
ASPHALT PATCHING ACROSS JOINT

NOTE SOUND  
SURFACE



TEST LOCATION 4  
NO REMEDIAL WORK AT THIS LOCATION

NOTE CONCRETE  
DETERIORATION AT  
JOINT.



TEST LOCATION 3  
CORE THROUGH JOINT

APPROXIMATE  
LOCATION OF JOINT

WATER SEEPING  
FROM SMALL  
HONEYCOMB AREA

CONCRETE SURFACE  
SCALED EXPOSING  
REINFORCING STEEL



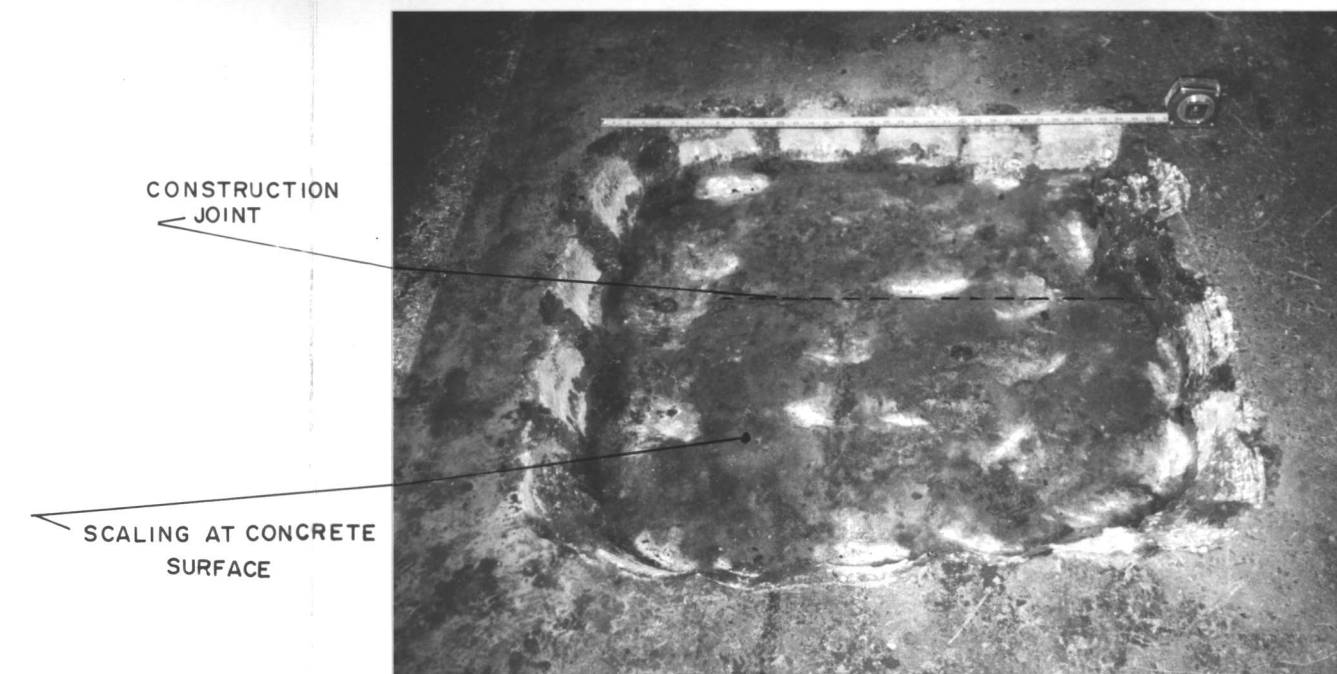
TEST LOCATION 1

Date... APRIL 14, 1983  
Project... 831-1038

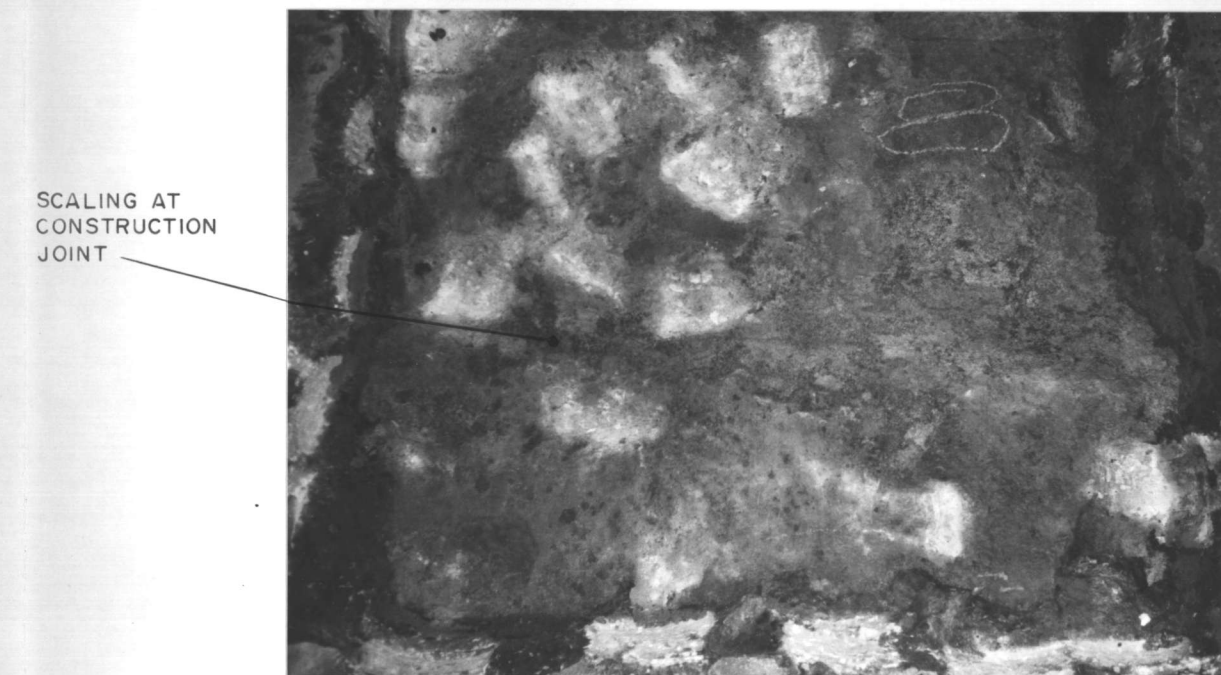
Golder Associates

Drawn... RWR  
Chkd... [signature]

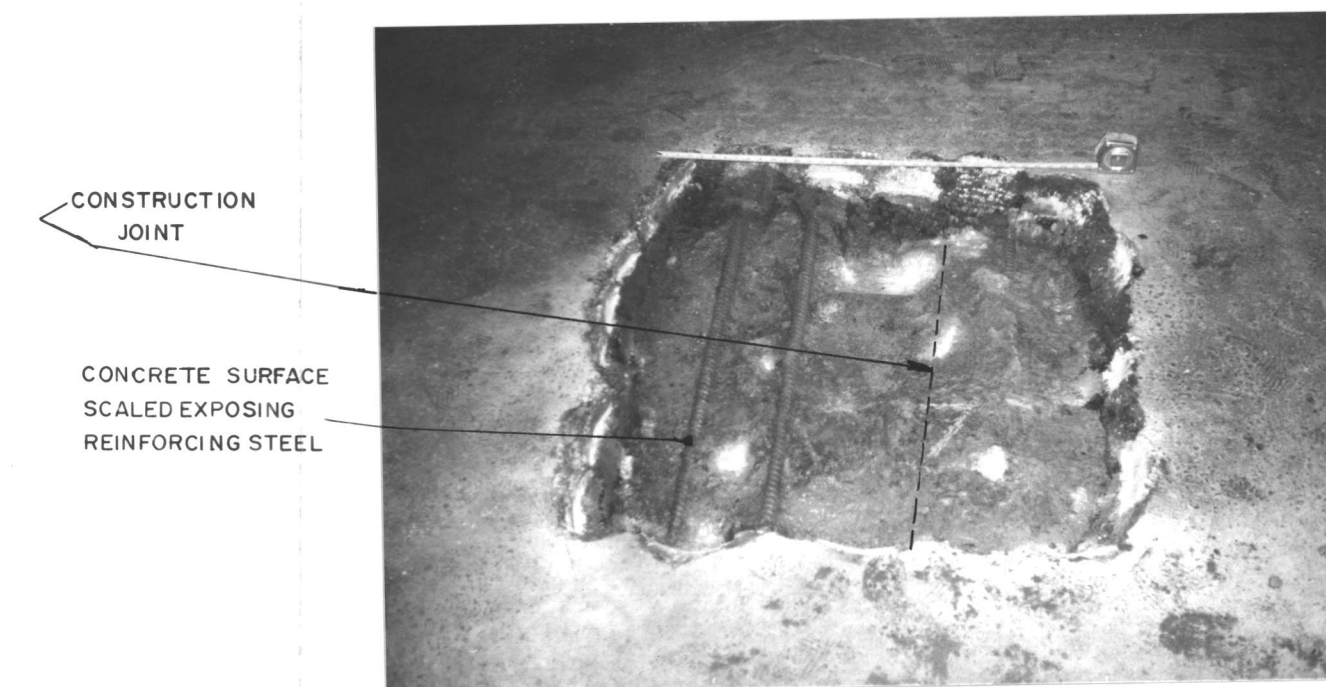




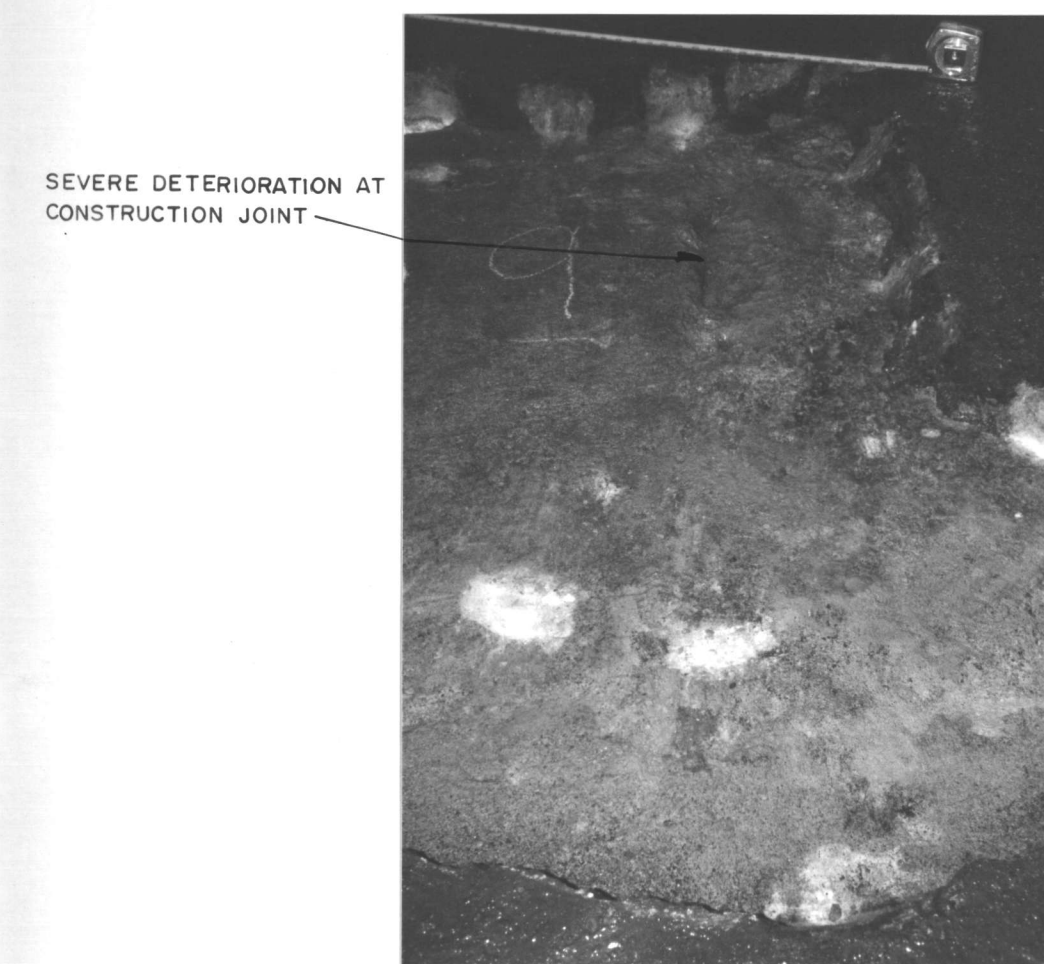
TEST LOCATION 6



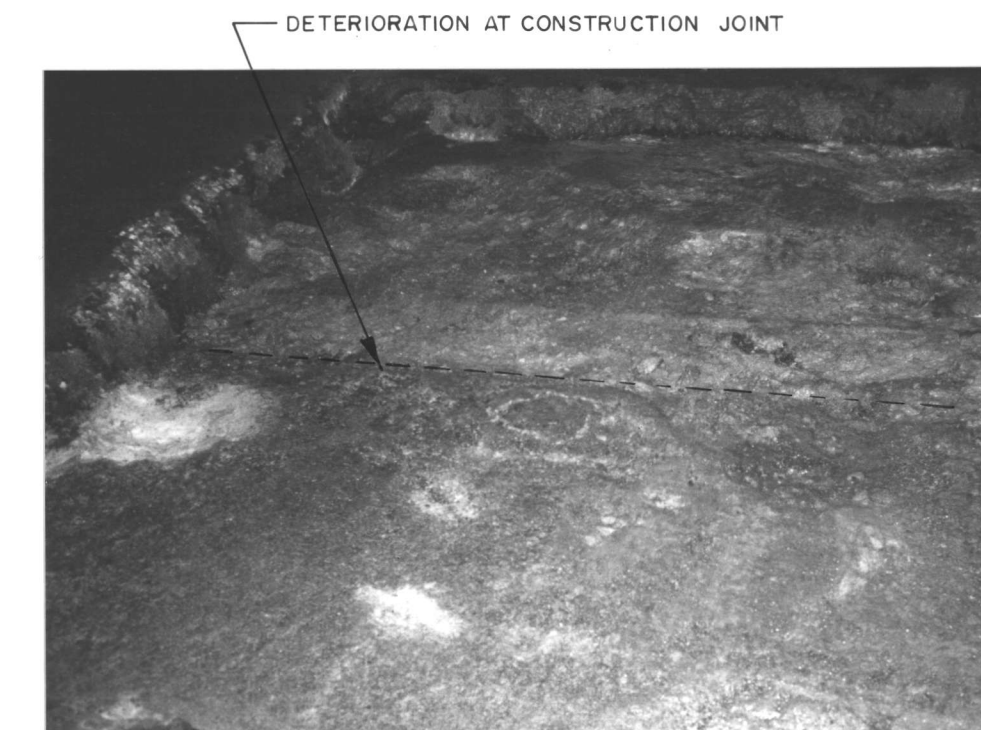
TEST LOCATION 8



TEST LOCATION 7

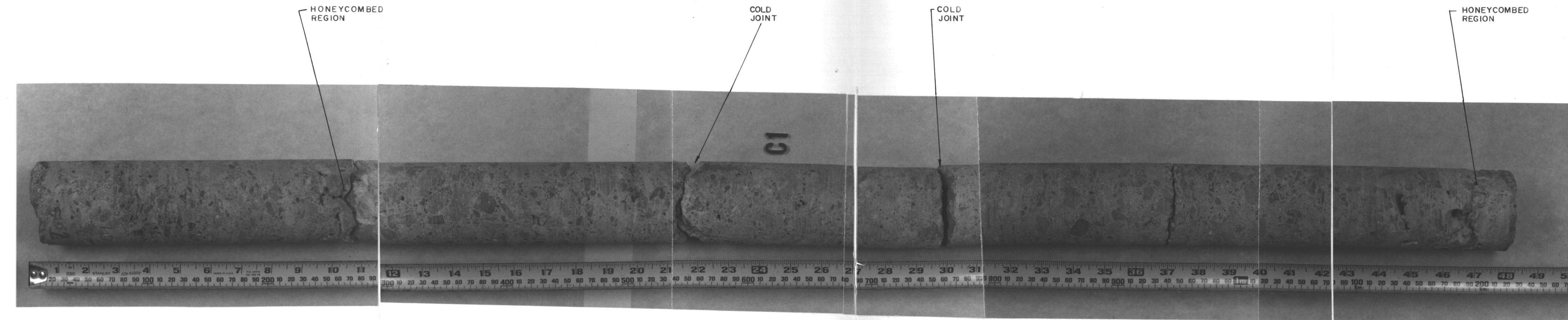


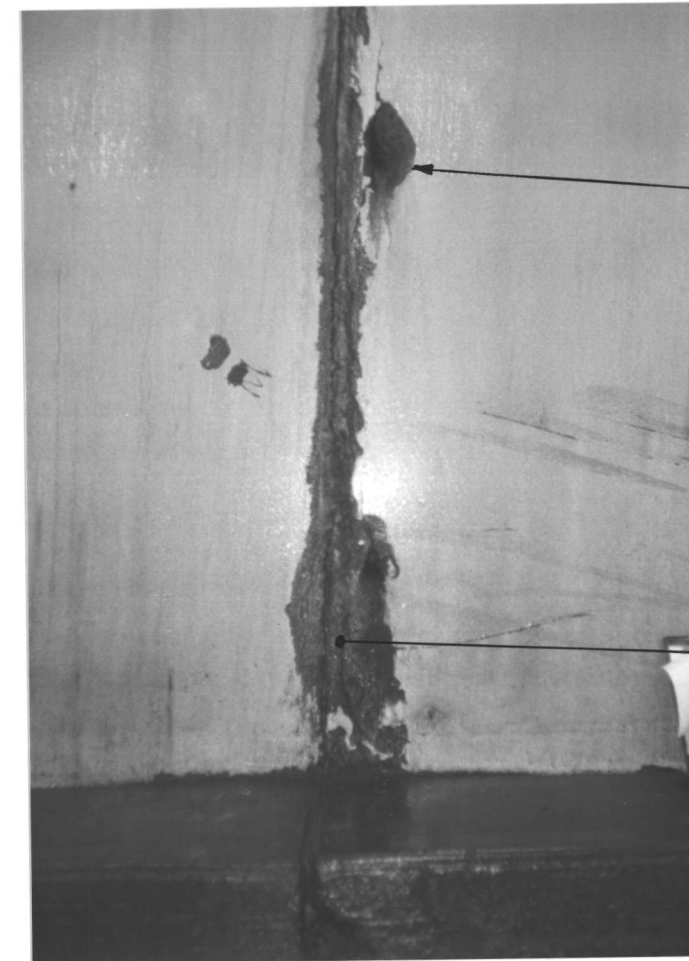
TEST LOCATION 9



TEST LOCATION 10







GROUT HOLE

LOWER JOINT FILLED  
TO PREVENT WATER  
SEEPAGE

TYPICAL DISTRESS AT JOINT



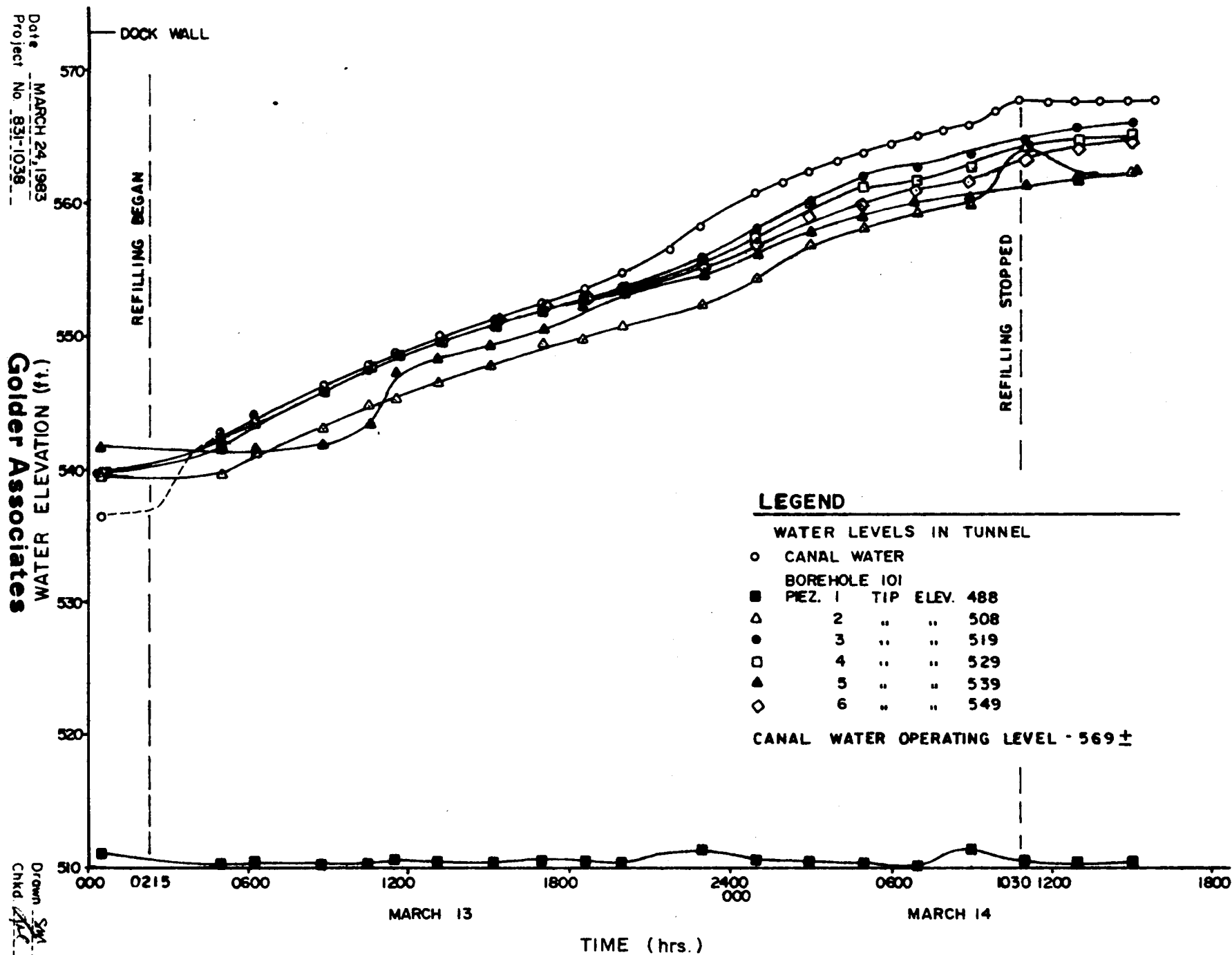
TYPICAL DISTRESS AT  
EDGES OF JOINT

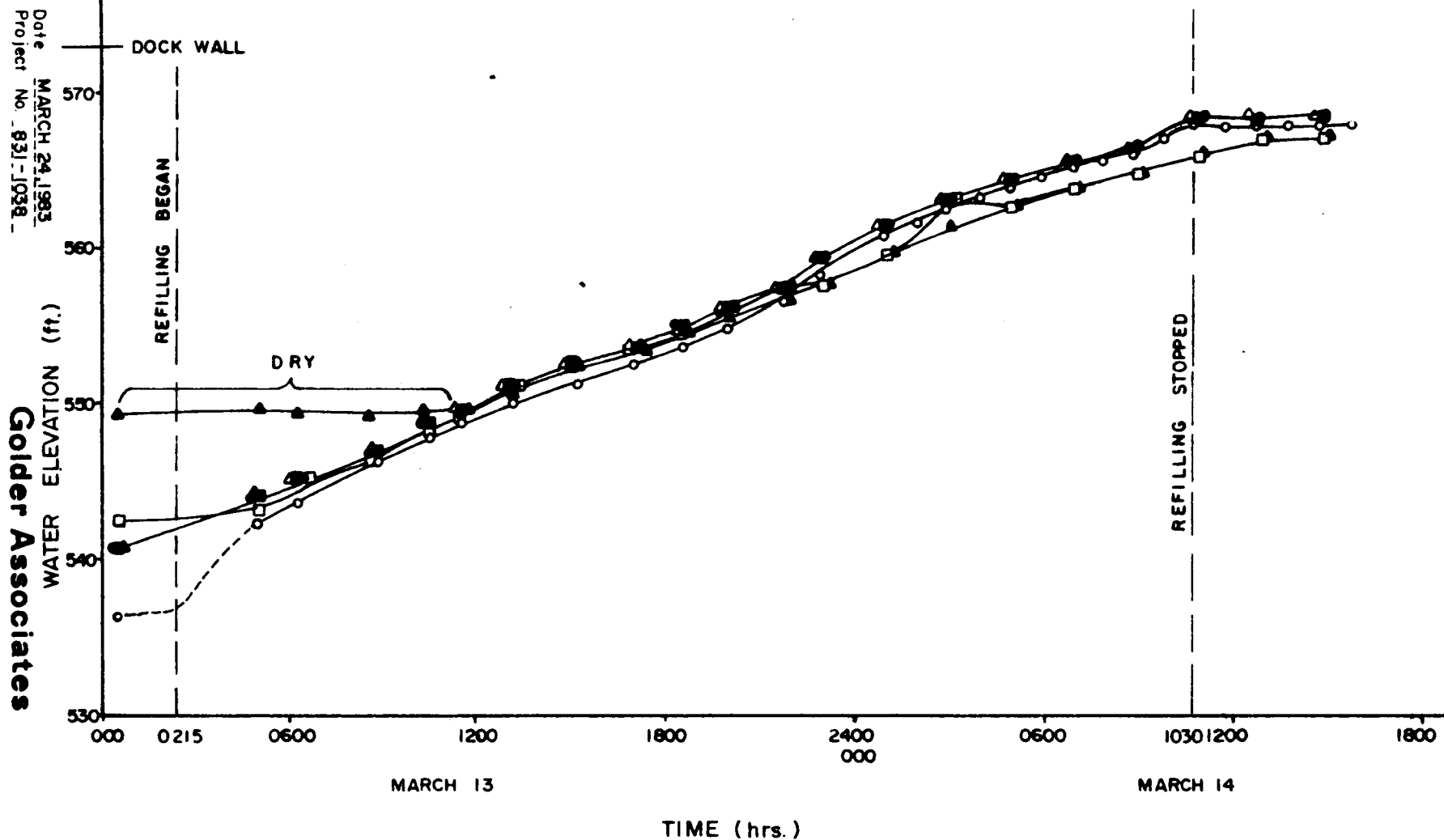


TYPICAL DISTRESS AT CURB

PLOT OF WATER LEVELS VS. TIME  
PIEZOMETER IOI  
DURING CANAL REFILLING MAR./83

FIGURE 9





Golden Associates

# LEGEND

## WATER LEVELS IN TUNNEL

- CANAL WATER
- BOREHOLE 102
- PIEZ. 1 TIP ELEV. 509
- △ 2 " " 519
- 3 " " 529
- 4 " " 539
- ▲ 5 " " 549

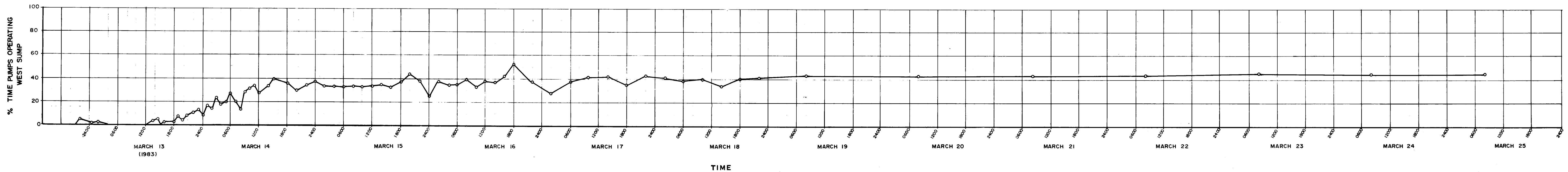
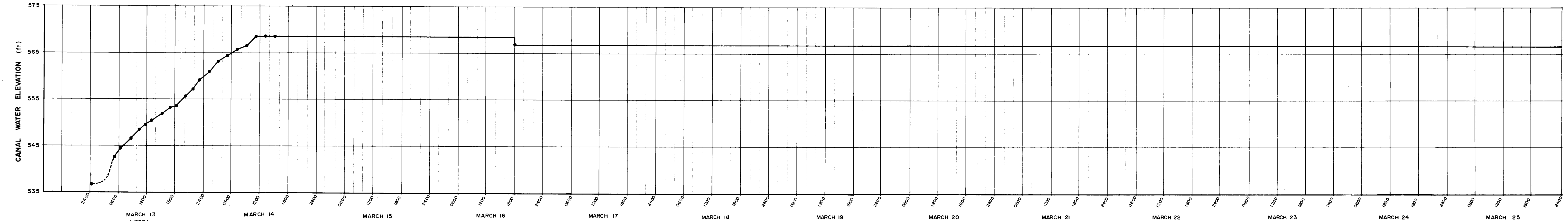
CANAL WATER OPERATING LEVEL - 569±

Drawn SM  
Chkd SM

PLOT OF WATER LEVELS VS. TIME  
PIEZOMETER 102  
DURING CANAL REFILLING, MAR./83

FIGURE 10



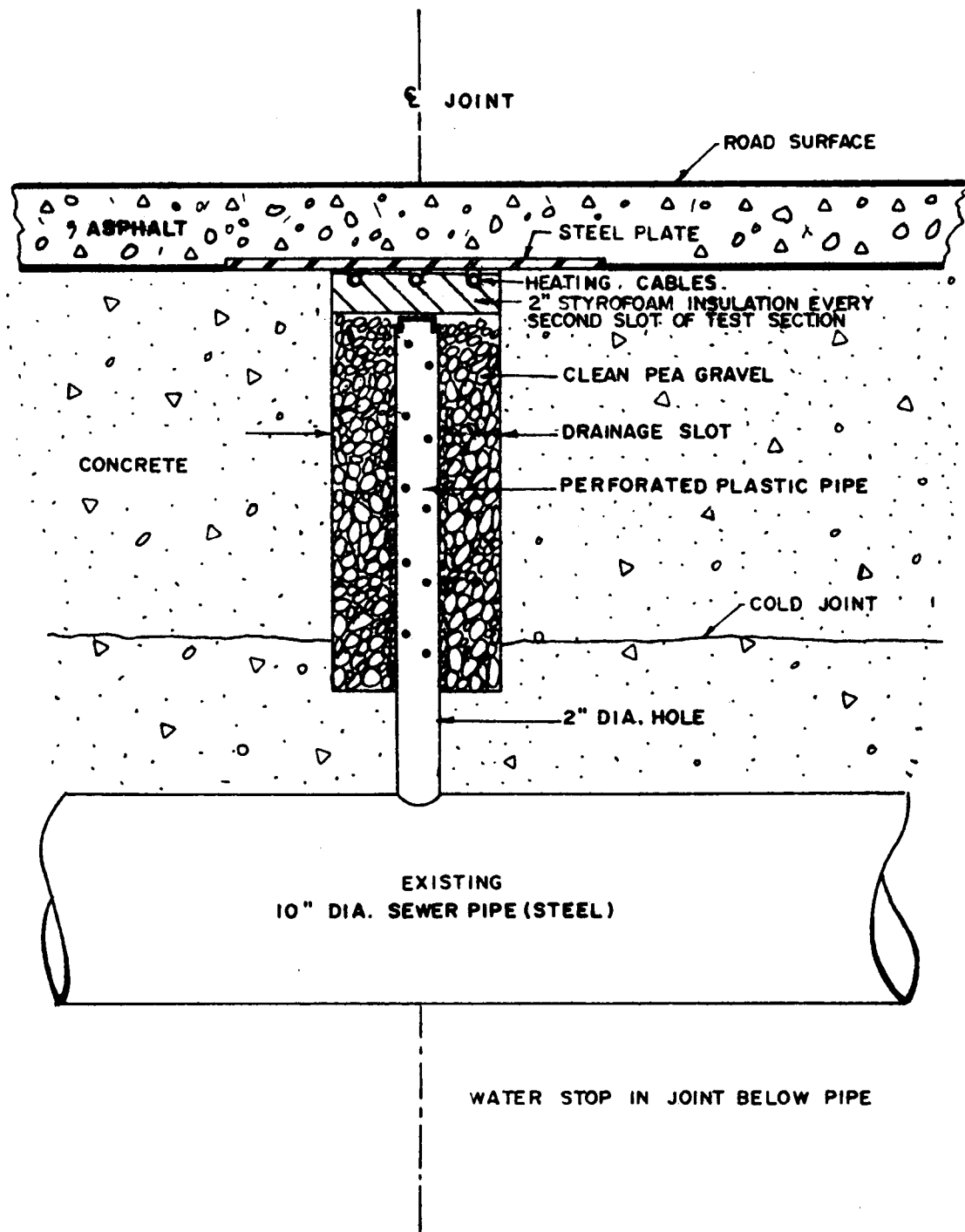


NOTES

- 1) CANAL REFILLED BETWEEN 0215 HOURS MARCH 13 AND 1030 HOURS MARCH 14.
- 2) WATER LEVEL IN CANAL AT ELEV. 568.6 ft. AT 1030 HOURS MARCH 14.
- 3) WATER LEVEL LOWERED TO ELEV. 566.8 ft. ON MARCH 16. (LATER RAISED TO NORMAL OPERATING LEVEL OF 568.1 ft. ON APRIL 4)
- 4) PUMP OPERATING TIMES ARE AVERAGE OF TOTAL TIMES FOR PUMPS 1, 2 AND 3.

# PROPOSED DRAINAGE SLOT FLOOR SLAB CONSTRUCTION JOINTS

FIGURE 12



SCALE : 1/8" TO 1"

Date APRIL 13, 1983  
Project No. 831-1038

**Golder Associates**

Drawn RWR  
Chkd. RL

# memorandum



To: G.C.E. Burkhardt  
Head, Structural Section  
5000 Yonge Street  
Willowdale, Ontario

Date: 83 07 12

From: Pavement & Foundation Design Section  
Room 315, Central Building  
Downsview

Re: Study of Water Seepage  
Thorold Tunnel under the Welland Canal,  
Hwy 58  
District 4, Hamilton

Golder Associates have now completed their final report following their investigation of the above-mentioned seepage problem. The report is essentially the same as the draft copy which we reviewed previously and discussed at our meeting on 83 06 09 in your office with yourself, M. Almer and K. Bassi. I am of the opinion that the factual data assembled and acquired by Golder fully justifies their conclusions that the seepage into the tunnel cannot be prevented or even minimized to an acceptable level without extremely expensive measures, the success of which cannot be assured. In consequence, it would appear that internal control of the seepage water is the only practical solution available to the Ministry. Copies of the final report have been forwarded to C. Robertson and K. Bassi, and I have retained one for our files. Three copies are attached to this memo for your use. Should you decide on any further distribution to interested parties, additional copies can be obtained directly from Golder. Please advise if I can be of any further help in this matter.

A handwritten signature in dark ink, appearing to read "K. G. Selby".

K. G. Selby, P. Eng.  
Senior Foundations Engineer

KGS:gm  
Encls.

cc: C. Robertson (1)  
K. Bassi (1)  
K. Selby (1)