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REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

FINAL FOUNDATION INVESTIGATION

VOLUME III

FOUNDATIONS AND EXCAVATIONS

PROPOSED CROSSING OF THE RE-ALIGNED WELLAND CANAL

MAIN STREET EAST TUNNEL

WELLAND

ONTARIO

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## ABSTRACT

This report forms Volume III of the final foundation investigation report and provides engineering recommendations for the foundations and earthworks required for the proposed Main Street East Tunnel crossing of the Re-aligned Welland Canal in Welland, Ontario.

Presented in the report are suggested methods of relieving the hydrostatic pressure in the bedrock aquifer which underlies the site. This pressure relief will be necessary to prevent hydrostatic uplift of the base of the construction excavations and permanent roadway approaches to the tunnel.

It is recommended that the permanent roadway approaches be cut at a side slope no steeper than 3.5 horizontal to 1 vertical and that in the deepest portions of the cut a 150 foot wide berm be provided at elevation 582, corresponding to the elevation of the proposed berms along the sides of the new canal.

The tunnel and retaining walls should in general be designed using an "at-rest" earth pressure coefficient,  $K_0$ , equal to 0.7. However, where roof loadings on the structures will induce stresses in the corners of the structures, the tunnel section should be designed using an "at-rest" earth pressure coefficient ranging between 0.5 and 1.0.

During excavation and dewatering at the site, rebound of the subsoil will occur. This rebound will generally be in the order of 0.5 to 1 inch. During construction of the tunnel structure and flooding of the canal differential settlement along the length of the tunnel is estimated to be about 0.5 inches.

Notes on the prevention of concrete deterioration by sulphate attack, the design of sumps and the roadway subgrade in the approach cuts are provided in the report.

## INTRODUCTION

The foundations and excavations at the site of the proposed Main Street East Tunnel crossing of the re-aligned Welland Canal in Welland, Ontario are discussed in this report which forms Volume III of the final foundation investigation report presented in 3 volumes, as follows:

- Volume I - Soil and Bedrock Conditions
- Volume II - Groundwater Conditions and Pumping Test Results
- Volume III - Foundations and Excavations

The proposed tunnel is to be constructed following the completion of the excavations for the Welland Canal and is to be formed in an open cut excavation. The proposed tunnel crossing is shown in plan on Figure 3-1 and a section taken along the centreline of the tunnel is given on Figure 3-2. As shown on these figures the tunnel crossing consists of four main sections.

- i) Roadway approach cut sections - The deep open cuts forming the approaches to the tunnel proper slope down along the roadway centreline from the existing ground surface (elevation 600 to 605) to about elevation 527 at a 6 percent grade. Where the depth of excavation is less than about 40 feet side slopes of 3 horizontal to 1 vertical are proposed. For a cut depth greater than 40 feet it is proposed to construct cut side slopes of not steeper than 3.5 horizontal to 1 vertical with 15 to 20 foot wide benches provided at about elevations 582 and 548.
- ii) Retaining wall sections - A 160 foot long retaining wall section is to be provided at each entrance to the tunnel proper. The retaining walls are to be constructed to a maximum height of about 63 feet and in the higher portions of the walls transverse

struts are to be provided. A monolithic concrete floor slab varying in thickness from about 2 feet to 5 feet is to be provided throughout the length of the retaining wall sections.

- iii) Flared sections - As shown on Figures 3-1 and 3-2 a 100 foot long section immediately behind each retaining wall section is to be roofed and covered to about elevation 582 with earth fill to provide service roads along each side of the re-aligned canal.
- iv) Tunnel section - The tunnel section of the proposed crossing is to be 524 feet in length and of twin-tube type construction. At the re-aligned canal centreline the proposed roadway grade is to be at elevation 508.8. The minimum excavation invert required for construction of the tunnel is about elevation 503.

Excavation for the proposed tunnel crossing is to be carried out in two stages. The first stage will follow completion of the canal excavation in the vicinity of the proposed tunnel crossing and consists of forming the excavations required for the construction of the tunnel and retaining wall sections of the crossing. The limits of the preliminary temporary excavations employing side slopes of 3 horizontal to 1 vertical are shown on Figures 3-1 and 3-2. The second excavation stage consists of forming the final roadway approach cut sections and will probably be carried out during the course of construction of the tunnel and retaining wall sections. Following completion of the tunnel and retaining wall sections the temporary excavations are to be backfilled to final ground levels.

Based on the soil and groundwater conditions established

at the proposed Main Street East Tunnel crossing of the re-aligned Welland Canal (Volumes I and II of the final foundation investigation report) there are two major engineering problems which critically affect the design of the proposed tunnel structure and associated roadway approach cuts.

- i) Control of hydrostatic pressure in the bedrock aquifer underlying the site.
- ii) Stability of high cut slopes.

The hydrostatic pressure in the aquifer must be temporarily relieved to permit construction of the structural portion of the proposed tunnel and must be permanently relieved to prevent hydrostatic uplift of the bottom of the deeper portions of the roadway approach cuts. The stability of the proposed cut slopes both during construction and throughout the design life of the tunnel determine the geometry of the cut slopes and the most economical length of the structural portion of the tunnel.

The control of hydrostatic pressure and stability of slopes together with other problems of design such as earth pressures, settlements and road pavements are discussed in detail in the following sections of this report.



CONTROL OF UPLIFT PRESSURES

The proposed tunnel site is underlain at a depth of about 105 to 120 feet by bedrock, the upper 5 feet of which is weathered and fractured and which forms a confined aquifer in hydraulic communication with some external groundwater source. The piezometric water level in the bedrock aquifer is at about elevation 576. Due to upward seepage, the piezometric water level in the bedrock is reflected in the relatively pervious till and lower silt deposit. This upward seepage is arrested by the overlying relatively impervious clayey silt and silty clay strata and hence the piezometric water levels in the clays and upper silts are independent of the water levels in the underlying silt, till and bedrock.

Although the till is generally silty some pervious gravelley zones were encountered within the till. These pervious zones are known to drain readily during pumping from the bedrock and must therefore be considered a source of free groundwater under the same piezometric pressure as the bedrock. For design, it is assumed that such pervious zones may extend throughout the complete depth of the till.

As the piezometric water level in the aquifer is independent of the water levels in the overlying clays and silts,

excavation in these clays and silts will have no effect on the piezometric water level in the bedrock, till and lower silt deposit. Thus, during excavation at the site a hydrostatic pressure equal to a piezometric water level at elevation 576 could exist beneath the bottom of the excavation. As excavation proceeds, this hydrostatic pressure would be sufficient to cause uplift of the excavation bottom when the hydrostatic uplift pressure exceeds the downward weight of soil, unless the piezometric pressure in the aquifer is relieved.

Various pressure relief systems to ensure basal stability of both temporary and permanent excavations have been studied. Possible pressure relief systems are presented on the figures and discussed in the following sections of this report.

#### A) Temporary Dewatering System

The temporary excavations required for the construction of the tunnel and retaining wall sections of the crossing will extend down to as low as about elevation 503 or to a depth of some 100 feet below the existing ground surface. Along about three-quarters of the tunnel length, the invert of the proposed excavation will be within the granular till which underlies the site.

The variation in factor of safety against hydrostatic uplift with depth of excavation (ie. excavation bottom elevation)

for the case of full piezometric pressure in the aquifer is presented on Figure 3-3. From this figure it can be seen that for a factor of safety of 1.3 against uplift\*, no control of the hydrostatic pressure in the aquifer is required for excavations above elevation 556. Below elevation 556 the factor of safety against hydrostatic uplift decreases and is effectively zero at elevation 520, the upper surface of the granular till.

Also shown on Figure 3-3 are the maximum piezometric water levels which can be allowed in the aquifer immediately beneath the proposed temporary excavation in order to have a factor of safety against hydrostatic uplift equal to 1.3 as the excavation depth increases.

For excavations within the granular till itself the piezometric water level must be maintained a minimum of 2 feet below the base of the excavation.

It is understood that the proposed excavation for the re-aligned Welland Canal is to be taken down to about elevation 538 with no specified control of the hydrostatic pressure in the aquifer. As this depth of excavation is some 18 feet below the minimum elevation at which we suggest groundwater control be

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\* Because of the variation in elevation of the upper surface of the till a factor of safety against uplift greater than unity must be used: 1.3 is recommended for design.

initiated no additional excavation should be allowed in the base of the canal until such time as the piezometric water level in the aquifer has been reduced to the recommended level given on Figure 3-3.

A performance specification defining the drawdown in piezometric water level(s) in the aquifer which must be achieved prior to excavation should be part of the contract specifications for the work. While contractors may, at their discretion, suggest specific means for complying with the specifications, approval of contractors' proposed dewatering scheme(s) by the Consulting Engineers is an essential condition of the contract.

Notes for the performance specification covering the relief of the hydrostatic pressure in the aquifer during the construction of the tunnel and retaining wall sections will be provided as an Addendum to this report.

i) Recommended Monitoring Devices

To enforce the performance specification it will be necessary to monitor the piezometric water level in the bedrock aquifer during excavation and throughout the one to two year period that the excavation will be open. To monitor the variations in the piezometric water level in the aquifer and to determine if the specified drawdown has been achieved by the dewatering system we recommend that a minimum of 6 piezometers be installed in the

bedrock. Suggested locations for these piezometers are shown on Figure 3-4.

The 2 piezometers located within the limits of the proposed Stage I excavation are essential since they form the principal monitoring system for the control of drawdown in the aquifer. Additional piezometers installed within the limits of the excavation would be desirable, but it is possible that they might be destroyed in view of the difficulty anticipated in carrying out general excavation operations in a site containing buried instrumentation devices. As an alternative to additional piezometers within the excavation, we suggest that 4 piezometers be installed outside the excavation as shown on Figure 3-4.

It is probable that the temporary dewatering scheme proposed by the contractor will consist of a system of pumped wells. For a known well system, theoretical drawdown contours within the radius of influence of the well system can be computed. Using the computed drawdown contours for the well system proposed by the contractor the drawdown at any point within the excavation can be inferred from the observed water levels in the 4 piezometers installed outside the excavation. The direct observation of the piezometers within the excavation should provide reasonable control for the temporary dewatering system.

In addition to the 6 piezometers installed in the bedrock, we suggest that 4 piezometers, installed in 2 boreholes, be sealed into the lower overburden deposits outside the area of both the tunnel and canal excavations. The purpose of the overburden piezometers is to monitor the amount of drawdown achieved in the till stratum during pumping and to confirm the piezometric conditions used for the design of the cut slopes. Suggested locations for these piezometers are shown on Figure 3-4.

As the piezometer installations will be in use over a relatively long period of time provision must be made for flushing the piezometers. The piezometer should consist of two plastic tubes which terminate in a porous tip such as the Casagrande or Bishop borehole piezometer. The plastic tubes should be protected with black iron pipe extending from the top of the upper seal to the ground surface. Where 2 piezometers are to be sealed into a single borehole, it may be necessary to advance the boring in Hx casing size to the depth of the upper piezometers.

Due to the high sulphate content of the groundwater in the bedrock/till aquifer (generally greater than 2,000 p.p.m.), we suggest that no bentonite seals be installed at this site as the swelling properties of bentonite may be affected by these high sulphate contents. All seals should be formed using "Ciment Fondu" and sand grouted in place. Shrinkage of the grout should be

prevented by the admixture of an expansive agent suitable for use with Ciment Fondu.

ii) Example of Possible Dewatering System

As recommended, the design of the temporary dewatering system installed at the site should be the responsibility of the excavating contractor and only the performance of the system should be specified. A possible dewatering system is presented in this report for illustration only. It is based on the results of the full scale pumping test carried out at the site, and indicates that the specified drawdown can be achieved by a system of pumped wells installed outside the limits of the proposed Stage I excavation.

Following completion of the re-aligned canal excavation the piezometric water level in the aquifer will be some 38 feet above the bottom of the canal prism (assuming no dewatering is carried out in conjunction with the canal excavation). The installation of pumped wells from the bottom of the canal against this unbalanced 38 foot head of water could be difficult and piping at the base of the well casing during installation could result in loosening of the till and dense silt at the proposed tunnel invert. To eliminate the problem of installing the wells against an artesian pressure, initial wells could be installed from the berm to be constructed along both sides of the canal at about elevation 582. Four 12-inch diameter pumped wells installed through the canal

berm and outside the Stage I excavation limits as shown on Figure 3-4 (well locations W1 to W4 inclusive) could in the area of the tunnel crossing draw the piezometric water level in the aquifer down to about 5 feet below the canal bottom. To achieve this drawdown each well would have to be pumped at a rate of about 80 I.G.P.M.

With wells numbered W1 to W4 installed and operating additional wells could be installed from the bottom of the canal prism. Six additional 12-inch diameter wells (numbered W5 to W10 inclusive) could draw the groundwater in the aquifer down to the piezometric levels indicated by the contours given on Figure 3-4. As shown on Figure 3-5 the maximum drawdown achieved by the 10 wells is at least 2 feet below the proposed tunnel invert.

The discharge from wells numbered W1 to W4 inclusive would be about 60 I.G.P.M. per well and the discharge from each of the 6 additional wells would be about 40 I.G.P.M. Thus the total quantity of water pumped by the well system would be about 480 I.G.P.M.

Whatever system of dewatering is used, simple metering devices should be installed so that the total flow and preferably the flow from each well is known.



B) Permanent Dewatering System \*

The proposed roadway approaches to the structural portions of the tunnel crossing are to be constructed in deep open cuts extending down to as low as elevation 527. At the deepest point, the west approach cut is to be within about 7 feet of the top of the "quasi-aquifer" (formed by the till and lower silt deposit) in which the piezometric water level in the weathered bedrock aquifer is reflected. Since the elevation of the upper surface of the "quasi-aquifer" varies across the site the thickness of the clay "plug" between the top of the till and the bottom of the approach cuts is not the same beneath the east and west tunnel approaches and does not increase in thickness at the uniform 6 percent roadway grade.

In the lower portions of both of the roadway approach cuts the weight of the clay "plug" will not be sufficient to prevent uplift under full hydrostatic pressure in the aquifer. Therefore, some permanent hydrostatic pressure relief will be required beneath both of the approach sections of the tunnel crossing. The maximum

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\* Factors additional to those discussed in this report and which could effect the permanent dewatering system are presently being considered and will be reported following additional field and laboratory studies.

allowable piezometric water levels in the aquifer for which the factor of safety against hydrostatic uplift will be 1.3 (the minimum factor of safety recommended for design) are shown on Figures 3-7 and 3-9. From these figures it can be seen that due to variations in the elevation of the upper surface of the "quasi-aquifer", the extent of hydrostatic pressure relief is not the same beneath the east and west roadway approach cuts.

The most feasible and economical method of achieving the required hydrostatic pressure relief is the installation of gravity relief wells. A gravity or artesian-flowing well installed in the bedrock aquifer from the bottom of the open cut excavation and discharging at or below the proposed final excavation invert would have the same effect on the groundwater regime in the aquifer as a deep pumped well installed from the present ground surface and pumped down to the discharge point of the gravity well.

A suggested permanent dewatering system identified as "Gravity Relief Well System A" is presented on Figure 3-6. This system consists of 5 eight inch diameter gravity relief wells installed along both the north and south ditch lines at each entrance to the structural portion of the tunnel crossing (i.e. a total of 20 wells). The wells are located in pairs at distances of about 3, 13, 33, 63 and 113 feet from the ends of each of the monolithic floor slabs proposed in the retaining wall sections of

the crossing. For design, the wells are assumed to discharge at a point 5 feet below the proposed final roadway grade. The theoretical piezometric groundwater surface contours resulting from the gravity relief well system are shown on Figure 3-6 and the theoretical piezometric water levels along the centreline of the tunnel crossing are presented on Figure 3-7.

As shown on Figure 3-7 the drawdown achieved by the suggested dewatering system is considerably greater than the drawdown required to prevent hydrostatic uplift of the bottom of the approach cuts. The piezometric water level profile due to the influence of "Gravity Relief Well System A" is however consistently below the proposed roadway grade and so prevents possible upward seepage of groundwater from the aquifer into the subbase material of the approach roadway pavement.

The total anticipated flow from the gravity relief well system is about 300 I.G.P.M. with the flow evenly distributed at each end of the tunnel. It is suggested that the water be led from the well heads to large sumps, which it is understood are to be provided at each tunnel portal.

An alternative permanent dewatering system identified as "Gravity Relief Well System B" is presented on Figure 3-8 together with the theoretical piezometric water level contours resulting from the gravity relief well system. The drawdown

piezometric water level profile along the centreline of the tunnel crossing is presented on Figure 3-9. This alternative dewatering system consists of a total of 18 eight inch diameter gravity relief wells with 8 wells installed at each tunnel entrance and 2 wells installed at the centre of the tunnel structure. The use of this relief well system is recommended only if hydrostatic pressure relief is required below the invert of the structural portions of the tunnel crossing as it requires an extra sump and pump at the centre of the tunnel. The alternative system has no advantage over the previously discussed "Gravity Relief Well System A" in the area of the open approach cuts.

The total anticipated discharge from the alternative relief well system is about 340 I.G.P.M. with about 135 I.G.P.M. coming from the 2 wells installed in the centre of the tunnel and the remainder of the discharge approximately evenly distributed between the tunnel entrances.

It is probable that adequate hydrostatic pressure relief will be achieved by either of the suggested relief well systems, but the drawdown shown on the figures is theoretical and based on a single pumping test. As it is known that the hydraulic characteristics of the aquifer are not perfectly uniform across the site it is possible that one or more of the wells will be located in a less permeable portion of the aquifer and will achieve less than the anticipated drawdown.

When the wells are put into operation, that is when the temporary dewatering system is stopped, the flow from the wells should be measured and additional wells should be installed if required. Provision for this should be made in the contract.

i) Recommended Monitoring Devices

Piezometers should be installed beneath the roadway approach cut sections of the crossing to monitor the piezometric groundwater level in the aquifer and to determine if adequate drawdown has been achieved by the permanent dewatering system installed at the site. Suggested locations for monitoring piezometers for "Gravity Relief Well Systems A and B" are presented on Figures 3-6 and 3-8 respectively. In addition to these piezometers, two piezometers installed within the limits of the Stage I excavation to monitor the effect of the temporary dewatering system could be incorporated into the permanent monitoring system.

The piezometers installed for permanent monitoring of piezometric water levels should be of a double lead flushing variety and should be sealed into the bedrock aquifer. To avoid forming a direct seepage path from the aquifer to the roadway subbase through granular backfill placed around the piezometer leads, the sealing agent should extend from about 1 foot below the upper surface of the bedrock to the top of the installation. As previously discussed, we suggest that only "Ciment Fondu"

and sand grout containing an expansive agent be used in forming seals at this site.

The piezometer leads should be led through buried pipes to convenient reading locations. Should the depth of earth cover in any direction around the buried pipes be less than 4 feet an equivalent amount of water proof thermal insulation should be provided to prevent freezing of the piezometer leads. A suggested permanent piezometer installation is shown on Figure 3-11.

The piezometric water levels may be determined by means of pressure gauges capable of recording negative pressures (such as a compound Bourdon gauge) and a piezometer flushing system should be incorporated into the installations. A suggested permanent piezometer recording and flushing installation and suggested piezometer tip configurations are presented on Figure 3-10.

The piezometers should be read regularly and the measurements should be transferred immediately to a wall graph in the engineer's office. Thus any gradual and unwelcome build up of pressure will be evident to all concerned.

For convenience and ease of recording the recording gauges and piezometer flushing equipment should be installed in instrumentation control stations located at each end of the proposed tunnel behind the high retaining walls which are to form

the tunnel entrances. The recording equipment for all of the permanent instrumentation installed at the proposed tunnel crossing can be located in these instrumentation control stations.

A simple control station consisting of a 10 foot long and 6 foot wide chamber could be formed by constructing a second small retaining wall parallel to and about 6 feet behind the main retaining walls. Suggested locations for each control station are shown on Figures 3-6 and 3-8 and a sketch of the control station location together with the piping for the piezometer leads is presented on Figure 3-12.

During the winter months it will probably be necessary to provide nominal heating equipment in these control stations to prevent the water in the piezometer leads from freezing.

#### ii) Construction of Gravity Relief Wells

The gravity relief wells installed at the site should be of the simplest construction possible and could consist merely of open holes in the bedrock. To prevent caving in the overburden deposits the holes should be cased from the top of the bedrock to the discharge point. No granular filters or gravel backfills should be used in the wells since they would decrease the efficiency of the wells and make maintenance of the wells difficult.

The anticipated discharge rate of any single well forming part of "Gravity Relief Well System A" is less than about

25 I.G.P.M. Although this discharge rate is within the capacity of smaller diameter wells we feel that an 8 inch diameter well is the smallest size well which can be economically and efficiently installed, developed, maintained and, if necessary, pumped. We therefore recommend that the gravity relief wells be at least 8 inches in diameter. To ensure complete penetration of the weathered bedrock aquifer the wells should be taken a minimum of 15 feet into the bedrock.

Chemical analyses additional to those presented in Volume II of the final foundation investigation report were carried out on samples of groundwater obtained from piezometers installed within the bedrock aquifer. The purpose of the analyses was to determine if the groundwater in the aquifer would actively corrode steel well casing and to provide an indication of the rate at which corrosion could occur. The results of the chemical analyses are presented in Appendix A of this volume of the report. A comparison of the measured values of sulphate ion concentration and pH for the groundwater samples obtained from the piezometers with the measured values for groundwater samples obtained from the pumped well (Volume II of the final foundation investigation report) indicates that the samples obtained from the piezometers are not representative of the groundwater in the aquifer. Therefore, estimates of the corrosive effects of the groundwater on steel well casing based on the chemical analyses presented in Appendix A



are not considered to be reliable.

Additional groundwater samples from the aquifer are to be obtained during the course of a second small scale pumping test which is to be carried out at the proposed tunnel site. The results of chemical analyses carried out on these samples should be representative of the groundwater in the aquifer and should form a basis for an estimation of the corrosive effect of the groundwater on steel well casing. Pending the results of chemical analyses on these additional samples, which will be presented as an addendum to this report, the groundwater in the aquifer must be considered corrosive and the gravity relief wells should initially be designed accordingly.

A permanent well installation which would resist corrosion could be achieved by installing stainless steel well casing rather than conventional casing. As the cost of stainless steel well casing would probably make its use uneconomical, we suggest that, as an alternative, the gravity relief well installation shown on Figure 3-11 be used.

The gravity relief well shown on Figure 3-11 may be installed by advancing a 12 inch diameter cased hole to the upper surface of the bedrock. An 8 inch diameter thick-walled well casing (wall thickness equal to 0.432 inches) fitted with a hardened steel drive shoe could then be centred in the 12 inch diameter

casing and seated in the bedrock. With the 8 inch diameter casing seated, the well may be advanced in 8 inch diameter size 15 feet into the bedrock. Following completion of the hole, 2 to 3 feet of stiff "Ciment Fondu" and sand mixture should be placed in the annular space between the 8 inch and 12 inch well casings. To prevent the Ciment Fondu and sand mixture from flowing into fissures in the upper weathered portion of the rock the stiffest mixture which can be pumped into the annular space between the casings should be used. An expansive agent should be added to the mixture to ensure that a tight seal forms around the bottom of the 8 inch diameter casing. After the initial plug has hardened the remainder of the annular space between the 8 and 12 inch casings should be filled with Ciment Fondu and sand grout containing an expansive additive.

The gravity relief well described above consists of a thick walled inner steel casing and outer sulphate resistant grout sleeve in a 12 inch steel casing. This will maintain the water tight integrity of the well if the inner casing is destroyed by corrosion.

Following installation of a gravity relief well, the well should be fully developed by surging and pumping and the efficiency of each well proved by means of a pumping test. Previously installed wells and the permanent piezometer installations could be used as observation wells to monitor the piezometric water

levels at various radial distances from the pumped well during these tests.

Should the pumping test indicate that a well does not have sufficient capacity to meet the requirements of the dewatering system

- i) additional surging of the well may increase the capacity of the well,
- ii) if attempts to develop the well prove unsuccessful or uneconomical a replacement well may be installed.

Over prolonged operating periods the efficiency of the wells may decrease. This decrease in efficiency will be reflected in a gradual increase in the piezometric pressure in the monitoring piezometer installations. Should the piezometric water level at any point along the Main Street East centreline approach the maximum allowable piezometric water level shown on Figures 3-7 and 3-9 the efficiency of the system should be re-developed by surging and pumping in the wells and possibly by installing new wells.

The discharge from the gravity relief wells should be led from the well heads to the large sumps which are to be constructed beneath each of the tunnel portals by means of buried and insulated pipes as shown on Figures 3-12. Since the elevation of the discharge pipes will determine the discharge elevations of the gravity wells the pipes should be installed 5 feet below

the roadway profile control line. Although the anticipated discharge to be carried by a single pipe is only about 75 I.G.P.M. the pipes should be considerably over-designed to reduce to a minimum the head losses due to flow. To prevent corrosion of the pipe walls we suggest that only transite pipe be used to carry well discharge. The discharge pipes should be insulated to prevent the formation of ice within the pipe. The insulation should be thermally equivalent to at least 4 feet of earth cover and should be waterproof. An insulating material such as foamed-in-place plastic would be acceptable.

To provide access to the well head for routine inspection and any necessary maintenance a manhole could be installed around each well head as shown on Figure 3-11. The manhole cover could be shaped to the configuration of the drainage ditch to prevent interference with surface flow in the ditches.

To prevent the buildup of hydrostatic pressure in the aquifer should the discharge pipes from the wells block, the well head configuration shown on Figure 3-11 is recommended. The well head consists of a "T" coupling leading to the discharge pipe and additional well casing extending 2 feet above the centreline of the discharge pipe. A steel cap of suitable weight, placed on the top of the open well casing would lift allowing the well water to escape into the manhole should the head of water in the well casing exceed

the discharge level of the well by more than 7 feet. The manhole cover should be designed to lift, allowing the well water to flow to the sumps through the open surface ditches should the hydrostatic pressure in the manhole exceed about 1 p.s.i.

C) Transition from Temporary to Permanent Dewatering System

Hydrostatic uplift of the bottom of the deeper portions of the roadway approach cuts will occur if the temporary dewatering system is shut down before the permanent gravity relief well system is installed. In addition, the central portions of the proposed tunnel section (D.H.O drawing number D-6273-P1, dated January, 1968) will tend to lift due to excess hydrostatic pressure if the temporary dewatering system is shut down either before the permanent dewatering system is installed or before the re-aligned canal is flooded to its design depth. Therefore, the permanent gravity relief well system and the permanent monitoring piezometers must be installed before the temporary dewatering is stopped.

The transition from the temporary to the permanent dewatering system should take place slowly and all monitoring piezometers should be read constantly during the transition period. Should the piezometric water level in any of the monitoring piezometers begin to exceed the maximum allowable piezometric water level (Figures 3-7 or 3-9) the total discharge of the temporary dewatering system should be increased. If 2 or 3 attempts to

complete the transition prove that the capacity of the permanent gravity relief well system is insufficient to give the required hydrostatic pressure relief, additional gravity relief wells may have to be provided at locations to be determined at that time.

STABILITY OF SLOPES

Excavation for the proposed tunnel crossing is to be carried out in two stages. Stage I, the temporary construction excavation will be formed following completion of the canal prism and will be required for the construction of the tunnel and retaining wall structures. The base of this excavation will extend from about station 21+20 to station 31+80 and the depth of the excavation invert will vary from about 80 feet below existing ground surface at the ends of the excavation to about 100 feet below existing ground surface at the centre of the cut. Along the length of the Stage I excavation, the invert is generally within dense glacial till although at the ends the invert is within the firm to stiff irregularly layered silty clay stratum encountered at depth by the borings put down during the preliminary and final foundation investigation. The greater part of the Stage I excavation is to be backfilled following completion of the tunnel and retaining wall structures.

The Stage II excavations or permanent roadway approach cuts will probably be excavated during construction and backfilling of the tunnel and retaining wall structures. The invert of these excavations will be at a maximum depth of about 80 feet below ground surface at the tunnel entrances and the invert will rise at a 6 percent grade to the existing ground surface. The invert of the permanent roadway approach cuts will generally be within the

stiff to very stiff clayey silt stratum but between about elevation 525 and elevation 560 the invert will intersect a thick but discontinuous silt to sandy silt stratum. The deeper portions of the excavations will be underlain by firm to stiff irregularly layered silty clay.

As the greater part of the excavations will be within or underlain by cohesive overburden deposits (clayey silt and silty clay strata) both the initial stability and long term stability of the cut slopes were studied. Stability analyses were carried out utilizing the computer of the Department of Highways, Ontario, supplemented by manual computations.

The initial stability of the cut slopes pertain to the end of construction case and assumes no changes in effective stress within the cohesive overburden deposits. This condition may be analysed using the total stress approach or the undrained shear strength of the cohesive strata ( $\phi = 0$  case). Based on the results of the in situ field vane testing and triaxial compression tests carried out on relatively undisturbed samples in the laboratory the following undrained shear strength parameters and unit weights for the various soil strata encountered during the preliminary and final foundation investigations were selected for design.



- Desiccated Crust
  - $C_u = 0$  (Tension crack)
  - $\phi = 0$
  - $\gamma = 135 \text{ lb/cu.ft.}$
- Clayey Silt Stratum (above elevation 550)
  - $C_u = 1,750 \text{ lb/sq.ft.}$
  - $\phi = 0$
  - $\gamma = 135 \text{ lb/cu.ft.}$
- Clayey Silt Stratum (below elevation 550)
  - $C_u = 1,500 \text{ lb/sq.ft.}$
  - $\phi = 0$
  - $\gamma = 135 \text{ lb/cu.ft.}$
- Upper Silt to Sandy Silt
  - $C_u = 0$
  - $\phi = 30^\circ$
  - $\gamma = 130 \text{ lb/cu.ft.}$
- Silty Clay Stratum
  - $C_u = 1,000 \text{ lb/sq.ft.}$
  - $\phi = 0$
  - $\gamma = 120 \text{ lb/cu.ft.}$
- Basal Till
  - $C_u = 0$
  - $\phi = 38^\circ$
  - $\gamma = 135 \text{ lb/cu.ft.}$

As the above undrained shear strength parameters are considered to be reasonably conservative averages of the measured shear strengths but considering the heights of slopes under study and the serious repercussions of a failure we suggest that the minimum acceptable computed factor of safety against deep-seated rotational instability based on a total stress approach should be 1.3.

In considering the intermediate or long term stability

of cuts made in cohesive overburden deposits such as are present at this site, larger errors can be introduced if pore pressure changes are not considered. Since, with time, the pore pressure conditions within the cut slopes will change due to horizontal seepage towards the cut faces and downward seepage into the lowered water table in the bedrock and till the stability of the permanent roadway approach cut slopes was analysed using the effective stress approach.

Based on the results of consolidated drained triaxial tests (S-tests) and consolidated undrained tests with pore pressure measurements ( $\bar{R}$ -tests) the following effective stress parameters were selected for the analyses of the long term stability of the cut slopes.

- Desiccated Crust (tension crack)
  - $c' = 0$
  - $\phi' = 0$
  - $\gamma = 135 \text{ lb/cu.ft.}$
- Clayey Silt Stratum
  - $c' = 0$
  - $\phi' = 31^\circ$
  - $\gamma = 135 \text{ lb/cu.ft.}$
- Upper & Lower Silt to Sandy Silt
  - $c' = 0$
  - $\phi' = 31^\circ$
  - $\gamma = 130 \text{ lb/cu.ft.}$
- Silty Clay Stratum
  - $c' = 200 \text{ lb/sq.ft.}$
  - $\phi' = 24^\circ$
  - $\gamma = 120 \text{ lb/cu.ft.}$

- Basal Till

$$c' = 0$$

$$\phi = 38^\circ$$

$$\gamma = 135 \text{ lb/cu.ft.}$$

Due to the difficulty of accurately estimating the pore pressures which will develop in the slope with time we suggest that the minimum acceptable computed factor of safety against a deep-seated rotational failure based on the above effective stress parameters and a reasonable estimate of the pore pressures should be 1.5.

#### A) Temporary Construction Excavations

Initial total stress stability analyses ( $\phi = 0$  condition) carried out along sections of the 3 horizontal to 1 vertical preliminary slopes for the temporary Stage I construction excavation indicate that the computed factors of safety based on the previously presented undrained shear strength parameters are less than the minimum recommended value (F.S. = 1.3). It will therefore be necessary either to flatten the preliminary slopes or, alternatively, to provide stabilizing berms. As any widening of the temporary Stage I construction excavation will result in not only additional excavation but also additional backfilling following completion of the tunnel structures it will be desirable to excavate the steepest slope or to construct the minimum berm configuration required to provide adequate stability of the slopes.

Large diameter undisturbed samples of the stiff irregularly layered silty clay stratum underlying the site are to be obtained in the area of the temporary construction excavation and additional laboratory triaxial testing is to be carried out on these samples to establish in more detail the undrained shear strength characteristics of the silty clay stratum. Total stress stability analyses ( $\phi = 0$  conditions) based on these more detailed undrained shear strengths are to be carried out and the stable slope configuration which will minimize excavation and backfilling will be established. The results of these stability analyses and recommended side slopes for the temporary construction excavation will be presented as an addendum to this report.

#### B) Permanent Roadway Approach Cuts

The results of preliminary total and effective stress stability analyses carried out at various cross sections along the proposed permanent roadway approach cuts indicate that, with time, the stability of the slopes increases. Thus, in general, the geometry of the cut slopes is governed by the end of construction case ( $\phi = 0$  or total stress stability analysis).

##### 1) Total Stress Stability Analyses

Computations were initially carried out to determine the factor of safety against the occurrence of deep-seated rotational failures of the side slopes of the permanent roadway approach

cuts proposed on the Department of Highways, Ontario Drawing No. D-6273-P1 "General Layout-Preliminary" revised January 3, 1968. These slopes were based on undrained shear strength data obtained during the course of the preliminary foundation investigation (our report 66134 dated May, 1967). The preliminary cut slopes are shown on the upper half of Figure 3-13.

The total stress stability analyses carried out for the preliminary slope geometry were based on the detailed soil stratigraphy and undrained shear strength parameters established during the course of the present foundation investigation. The stability analyses were carried out for sections taken perpendicular to the proposed slopes at stations 17+30, 31+75, 33+00 and 35+00 as these sections appear to be the most critical, based on the soil stratigraphy and slope geometry. The results of the total stress stability analyses carried out on the preliminary cut slopes are summarized on Figure 3-14.

As shown on Figure 3-14, the factor of safety against a deep-seated rotational failure occurring during or shortly after construction at stations 17+30, 31+75 and 33+00 is about unity or slightly greater. At these sections the proposed slopes will be underlain by the stratum of irregularly layered firm to stiff silty clay encountered at depth during the course of the final foundation investigation. The marginal stability of the proposed cut slopes

is primarily due to the relatively low undrained shear strength ( $C_u = 1,000$  lb/sq.ft.) of this stratum. The factor of safety against the occurrence of a deep-seated rotational failure of the cut slope at station 35+00 is about 1.2.

A sliding wedge stability analysis based on total stress parameters ( $\phi = 0$  case) was carried out for the preliminary cut slope at station 17+30. The results of the sliding wedge analysis are presented on Figure 3-15. As shown on this figure the minimum factor of safety occurs for a failure plane extending the full length of the slope and located at the base of the firm to stiff silty clay stratum. The minimum computed factor of safety based on a horizontal failure surface, is about 1.2 which is some 10 percent greater than the factor of safety computed on the basis of a deep-seated circular failure surface.

Based on the comparison between the sliding wedge and circular arc stability analyses at station 17+30 it appears that the permanent roadway approach cut slopes will tend to fail on a circular surface rather than a horizontal plane. For this reason the stability of the cut slopes was analysed using circular failure surfaces and the Modified-Bishop method of analysis.

As the computed factors of safety at the four sections of the preliminary cut slopes studied were less than the minimum acceptable factor of safety of 1.3, total stress stability analyses

based on deep-seated rotational failures were carried out to determine a slope configuration for which the computed factor of safety was acceptable. Based on the initially computed slope stabilities, the general slope geometry and the soil stratigraphy along the proposed excavation the stability of the cut slopes at about stations 17+30 and 33+00 appear to determine the geometry of the general excavation.

The stability of the cut slopes can be increased by either flattening the 3.5 horizontal to 1 vertical slopes proposed in the preliminary report or by constructing one or more berms to effectively flatten the slope. Preliminary calculations utilizing design charts indicate that the most feasible method of increasing the stability of the slopes will be to provide berms as this method of construction will minimize the additional volume of material to be excavated, particularly in the lower softer zones where excavation may be difficult.

Due to the geometry of the cut slopes and position of potential failure surfaces little benefit would be derived from constructing a berm in the lower portion of the slope. Therefore, a series of total stress stability analyses was carried out for various widths and elevations of berm constructed in the upper portion of the slope at a section through station 33+00, the least stable section along the proposed roadway approach cut. The results

of these computations which considered berm widths of 50, 100 and 150 feet at elevations 570, 576 and 582 are summarized on Figure 3-16.

As can be seen from Figure 3-16 little benefit is gained by constructing the berm below elevation 582, the elevation of the berms to be constructed along both sides of the re-aligned canal. As lowering the elevation of the berm will make gravity drainage of surface water from the berm into the re-aligned canal difficult we suggest that the berm be constructed at the same elevation as the canal berm. As shown on Figure 3-16 a 150 foot wide berm will be required at elevation 582 to give a computed factor of safety of 1.3 against the occurrence of a deep-seated rotational failure.

Consideration was also given to steepening the cut slope below elevation 556 (the elevation of a small lower berm required for the control of surface water run-off). Steepening of the lower portion of the slope should increase the weight of soil resisting movement along a potential circular failure surface and hence should reduce the width of berm required at elevation 582. The results of a series of total stress stability analyses for a 2.5 horizontal to 1 vertical cut slope below elevation 556 at station 33+00 are summarized on Figure 3-17. The results of these analyses indicate that steepening of the lower portion of



of the slope has little effect on the overall stability of the roadway approach cut slope and reduces the minimum required width of berm at elevation 582 by only a few feet. As steepening of the cut slopes from a slope of 3.5 horizontal to 1 vertical will create maintenance problems and considering the minimal benefit achieved by steepening the lower portion of the slopes we suggest that the permanent cuts be constructed at a side slope of 3.5 horizontal to 1 vertical and, at station 33+00, a 150 foot wide berm be provided at elevation 582.

A series of total stress stability analysis was carried out at a section taken perpendicular to the proposed cut slopes at station 17+30 to determine the minimum width of berm at elevation 582 required to give a computed factor of safety of 1.3. The results of these computations, which are summarized on Figure 3-18, indicate that the minimum acceptable width of berm required at station 17+30 is about 65 feet.

Based on the total stress stability analyses carried out for sections at stations 17+30 and 33+00, the preliminary cut slopes were revised to provide a stabilizing berm at elevation 582. The dimensions of the proposed berm were based on a minimum acceptable berm width of 65 feet at station 17+30 and a minimum acceptable berm width of 150 feet at station 33+00. The revised berms were made as similar as possible in all four quadrants of the roadway approach

cu' Some variation in the berm configuration was required as the Main Street East centreline and the re-aligned Welland Canal centreline are not perpendicular. The revised cut slopes for the southern half of the proposed tunnel crossing are shown in plan on Figure 3-13.

Total stress stability analyses were carried out for 8 typical cross-sections along the revised cut slopes to confirm that the computed factor of safety against the occurrence of a deep-seated rotational failure was equal to or greater than the minimum acceptable value of 1.3. The results of these analyses, which are summarized on Figure 3-19, indicate that, for the inferred soil stratigraphy and undrained shear strength parameters, the revised cut slopes are stable.

Three additional boreholes are to be put down at the proposed tunnel crossing to further delineate the soil stratigraphy along sections taken perpendicular to the proposed slopes at stations 17+30 and 33+00. In addition, large diameter undisturbed samples of the irregularly layered firm to stiff silty clay stratum underlying the deeper portions of the approach cuts are to be taken in 6 inch diameter borings to be put down in conjunction with the instrumentation of the tunnel crossing. Triaxial compression tests are to be carried out on these large diameter samples to determine the directional shear strength characteristics of the possibly anisotropically

consolidated silty clay. The results of the additional boreholes and laboratory testing together with any further revisions to the approach cut side slopes necessitated by the additional information will be presented as an addendum to this report.

ii) Effective Stress Stability Analyses

In determining the intermediate or long term stability of the slopes effective stress stability analyses were carried out for the revised cut slopes shown on Figure 3-13 and were based on the soil stratigraphy and effective stress parameters given in Volume I of this report. The analyses were carried out at sections taken perpendicular to the revised slopes at stations 15+50, 17+30, 31+75, 33+00 and 35+00 as these sections appear to be the most critical. For initial computations the two limiting pore water pressure conditions which could exist within the cohesive overburden deposits were considered. The most severe pore pressure condition which could exist within the cut slopes corresponds to a piezometric groundwater level at the face of the cut slope. This case was analysed to determine the minimum factor of safety for the slopes after several years. The minimum pore pressures which could exist within the cut slopes corresponds to the piezometric water level in the underlying aquifer due to the influence of the permanent gravity relief well system. This pore pressure condition assumes almost complete drainage of the cohesive overburden deposits and, although such drainage is unlikely,

corresponds to the maximum factor of safety for the slopes after several years. The results of the effective stress stability analyses are summarized on Figure 3-20.

As shown on Figure 3-20 the computed factors of safety for the sections analysed using the most severe pore pressure condition which could exist within the cut slopes are generally greater than the minimum acceptable factor of safety ( $F.S. = 1.5$ ). For the case of the lowered piezometric water level the computed factor of safety at all of the analysed sections is about 2 or greater.

As the long term stability of the cut slopes at stations 15+50, 17+30, 33+00 and 35+00 is considered adequate under the most severe pore pressure conditions no attempt was made to estimate the actual pore pressures along the failure surface. However, as the computed factor of safety, based on the most severe pore pressure conditions, for the slope at station 31+75 was less than 1.5 the pore pressures were estimated at this section.

The actual pore pressures which may be anticipated along a circular failure arc were estimated by assuming a parabolic transition from a perched groundwater level at the face of the cut slope to a lowered piezometric water level in the till. This lowered piezometric water level was assumed to be the same as the piezometric water level in the bedrock due to the influence of

Gravity Relief Well System "A" (Figure 3-6). Using this assumption, the pore pressure distribution within the cohesive overburden was established at several sections along the cut slope. A typical pore pressure distribution diagram is shown on Figure 3-21. From such pressure diagrams the pore pressure at several points along a trial failure surface could be determined. The pore pressures were then expressed as a piezometric head of water and, by interpolation, a unique phreatic surface could be established for each trial failure surface. A typical phreatic surface along the potential failure surface for the cut slope at station 31+75 is shown on Figure 3-21.

The factor of safety against the occurrence of a deep-seated rotational failure of the revised cut slope at station 31+75 was computed on the basis of effective stress parameters and a reasonable estimate of the pore pressure distribution along the potential failure arc. The results of this analysis indicated that, for the pore pressure distribution used, the computed factor of safety is 1.5, the minimum acceptable value.

Based on the stability analyses results discussed above it may be concluded that the intermediate or long term stability of the revised cut slopes presented on Figure 3-13 is adequate.

### iii) Recommended Monitoring Devices

Considering the height of the slopes along the proposed

roadway approach cuts and the seriousness of a slope failure we recommend that the roadway approach cut side slopes be instrumented to monitor any progressive movements which may occur in the slopes. Detection of the progressive movements could provide sufficient warning of the development of an incipient failure condition to allow remedial measures to be taken.

Any surficial movements of the slopes may be monitored by means of survey points located on the face of the slopes during excavation. These survey points could consist of 4 foot long wooden stakes or steel bars driven to a depth of 3 to 3.5 feet. The location and elevation of the survey points should be accurately determined at the time of installation and the locations and elevations monitored throughout the construction period to determine if any progressive surficial movement of the cut slope is occurring.

To monitor any progressive horizontal movements indicative of the development of a deep-seated failure within the permanent roadway approach cut slopes we suggest that 3 slope indicator casings be installed at the site. Should these installations indicate any horizontal movements as excavation proceeds we suggest that 2 additional casings be installed further down the cut slopes. These additional casings will substantiate the readings obtained in the original installations and will more closely

define the surface along which the movement is occurring. Suggested locations for the 3 initial casings and 2 additional casings are shown on Figure 3-13.

The slope indicator casing should be taken into bedrock to provide a fixed reference point for the readings. Should the portion of the casing within the bedrock be perforated these installations could also be used as standpipes to monitor the groundwater level in the bedrock at the casing locations.

Progressive downward movement of the ground surface behind the top of the cut slopes could be indicative of the development of a deep-seated rotational failure. We therefore suggest that ground settlement plates be installed immediately behind the tops of the permanent roadway approach cut slopes. These settlement plates could consist of 12 inch diameter piers founded on 2 foot square footings. The footings should be placed a minimum of 4 feet below ground surface to prevent any heaving due to frost action. The concrete piers should be capped at ground surface with a steel plate having a stainless steel raised ball or pin, the elevation of which can be accurately determined. The elevations of the settlement gauges should be accurately determined prior to the beginning of excavation of the approach cuts and should be monitored throughout the construction period.

EARTH PRESSURES ON STRUCTURES

As the tunnel and retaining wall structures are to be backfilled with the relatively impervious clayey silt to silty clay from the desiccated crust (to minimize seepage into the tunnel of surface water from the re-aligned canal and highly sulphated groundwater from the underlying aquifer) the lateral earth pressures which develop against the walls of the structures will originate in a well-compacted cohesive backfill. Further, as the walls of the structures are to be rigid the lateral earth pressures which develop will correspond to an "at-rest" condition. As the use of compacted clayey backfill behind rigid walls is not common a comprehensive literature search was carried out in an attempt to find case records giving measured values for the earth pressures which developed for such a condition. No records of measured values of lateral earth pressures against rigid walls backfilled with compacted clayey material were found.

Based on the engineering properties of the material in the desiccated crust ( $c'$ ,  $\phi'$ , Atterberg limits and compaction characteristics) together with experience for compacted granular backfill and in situ cohesive material behind rigid walls, we recommend that the walls be designed using an "at-rest" earth pressure coefficient,  $K_0$ , equal to 0.7. The triangular lateral earth pressure distribution thus computed should be redistributed uniformly over the full height of the wall.



The probable water level in the clayey backfill behind the flared and retaining wall sections of the crossing due to seepage from the canal is presented on Figure 3-22 together with the anticipated lateral earth pressure and hydrostatic pressure distribution at the highest point of the retaining wall section. Also shown on Figure 3-22 is the summarized results of a sliding wedge stability analysis carried out at the highest point along the retaining wall. This analysis gives the 'active pressure' which is the minimum value which can act on the wall. The 'pressure at rest', for which the wall must be designed is higher than this.

To reduce the hydrostatic pressure on the walls consideration was given to installing a granular drainage layer and underdrains behind the walls. To prevent fines from the clayey backfill being carried into the free-draining granular material it will be necessary to provide a filter layer between the clayey backfill and the drainage layer. Further, the drainage layer should not extend to the top of the retaining wall but should be capped by about 5 feet of compacted clay to prevent ingress of surface water. A suggested drainage layer and filter layer is presented on Figure 3-22 together with the possible upper seepage line from the canal into the drainage layer. The earth pressure distribution behind a retaining wall provided with a drainage layer and underdrains is also shown on Figure 3-22.

The concrete thickness in the upper portion of the walls is not sufficient to prevent frost penetration into the backfill behind the walls. Should frost-susceptible material be placed immediately behind the walls some ice pressures could develop against the walls. The provision of a non-frost susceptible and free-draining layer should prevent the formation of ice lenses.

It is understood from discussions with the Consulting Engineers that roof loadings in the tunnel and flared sections of the crossing will result in stress at the corners of the structural sections and that the magnitudes of these corner stresses vary as the lateral restraint due to earth pressures on the walls of the structure vary. It is therefore recommended that a range of lateral earth pressures corresponding to "at-rest" earth pressure coefficients,  $K_0$ , ranging between 0.5 and 1.0 be computed and the earth pressure resulting in the most severe stress conditions within the structure be used for design.

It is further understood that the roof loadings in the tunnel and flared sections of the crossing will result in lateral outward displacement of the walls of the structure. This outward displacement may amount to as much as  $3/16$  of an inch. It is not anticipated that this magnitude of displacement will be sufficient to mobilize high passive pressures within the clayey backfill material. It is however suggested that the tunnel walls be

designed to withstand a lateral earth pressure corresponding to an earth pressure coefficient of 1.0 (the upper limit of the range of values previously recommended for design) to accommodate any partial mobilization of passive pressure resulting from the minor outward displacement of the walls.

It should be noted that for a given "at-rest" earth pressure coefficient the magnitude of the earth pressures on the walls of the tunnel section of the crossing will vary with time due to the relative impermeability of the clayey backfill. This variation in earth pressure on the tunnel walls for a typical section of the tunnel is shown on Figure 3-23. As can be seen from this figure the water in the re-aligned canal immediately after flooding will act as a surcharge load and will appreciably increase the lateral pressure on the tunnel walls. With time however, downward seepage from the canal into the lowered piezometric water level in the underlying aquifer will result in considerably reduced lateral earth pressures but will create large hydrostatic pressures on the tunnel walls.

#### 1) Recommended Monitoring Devices

The 60 foot high retaining walls proposed at this site are relatively uncommon for clay soils but yet probably typical of retaining walls which will be required at other tunnel crossings in this area. Further, a literature search failed to locate any

recorded values of earth pressures due to compact clay backfill behind rigid walls. It is therefore suggested that lateral earth pressure cells be installed against at least one of the retaining wall sections to monitor the actual earth pressures developed.

A minimum of 4 pressure cells should be located at the base and quarter points of the retaining wall selected for instrumentation. Suggested locations for the cells are shown on Figure 3-22. Gloetzl Pressure Cells would probably be most suitable. To permit analysis of both total and effective earth pressures acting against the wall the compatible Gloetzl pore water pressure cell should also be installed at each location.

The leads from both the earth pressure cells and compatible pore water pressure cells could be taken to one of the instrumentation control stations located at the end of the retaining wall sections.

In conjunction with the earth pressure cells discussed above we suggest that at least 5 electrical strain gauges be installed within the proposed combination struts and sun shield adjacent to the instrumented retaining wall. These would allow the determination of strut loadings which would greatly assist in the accurate determination of the earth pressure distribution behind the retaining wall. Electrical leads from these strain gauges could be led to the instrumentation control station.

DEFORMATION UNDER TUNNEL STRUCTURES

The deformation under the proposed tunnel and retaining wall structures will consist of rebound and settlement. Settlement will occur as the effective vertical pressure on the tunnel invert is increased by construction or dewatering and rebound will occur as the effective vertical pressure is decreased by excavation or the recovery of reduced piezometric water levels in the soil beneath the tunnel invert. The effective vertical pressure is equal to the gross weight of the soil or the structure above the tunnel invert less the hydrostatic uplift pressure at the tunnel invert and is the pressure which will cause deformation of the soil. The effective vertical pressures at the centre of the tunnel structure (station 26+50), and at the tunnel portals (stations 24+00 and 29+00) during various stages of excavation, construction and dewatering are shown on Figure 3-24. The effective vertical pressures given on this figure were computed assuming the hydrostatic pressure in the aquifer acts directly as an uplift pressure against the proposed tunnel invert due to upper seepage through the granular till and lower silt deposits.

As shown on Figure 3-24 a considerable reduction in the effective vertical pressure (overburden pressure) will occur during excavation for the re-aligned canal and tunnel structure. Following excavation for the re-aligned canal however, there will

be an increase in effective overburden pressure resulting from the decrease in pore water pressure below the tunnel invert during temporary dewatering of the aquifer. The effective vertical pressure at the excavation invert will increase during construction and backfilling of the tunnel and retaining wall structures but will decrease as the transition from the temporary to the permanent dewatering system is effected. Immediately following flooding of the canal prism the canal water will act as a surcharge and the effective vertical pressure at the tunnel invert will increase. With time however, the surcharge effect of the canal water will be reduced by downward seepage from the canal into the underlying aquifer.

Some deformation, either settlement or rebound, will occur within the overburden and bedrock underlying the tunnel invert during each change in the effective vertical pressure acting at the invert. As the soil beneath the proposed tunnel and retaining wall structures consists generally of dense silts or sandy silt till the majority of this deformation will be elastic in nature and should occur during the excavation or construction periods. However, the entrances to the tunnel crossing and the central portion of the tunnel structure are underlain by as much as 15 feet of stiff to very stiff clayey silt and silty clay. In these areas settlements and rebounds may consist of some plastic deformation in addition to

the elastic deformation. For a static loading condition these plastic deformations may continue for a considerable period.

The anticipated deformation beneath the tunnel structure at stations 26+50 and 24+00 during the various stages of construction are shown on Figure 3-24. As shown on this figure the maximum anticipated rebound will occur when the Stage I excavation has been completed. The maximum anticipated settlement, which is in effect only a recovery of previous rebound, will occur following construction and backfilling of the tunnel structure but prior to the transition from the temporary to the permanent dewatering system. The final settlement will occur following the transition from the temporary to the permanent dewatering system and flooding of the re-aligned Welland Canal. If the proposed tunnel structure is backfilled with relatively impervious clay downward seepage of canal water will be minor and will not be sufficient to increase the piezometric water level in the aquifer. Thus the hydrostatic uplift pressure beneath the tunnel floor slab and hence the effective vertical pressure on the tunnel invert should remain constant and there should be no long term elastic deformations beneath the tunnel.

The variation in the maximum anticipated rebound (following completion of the Stage I excavation) and the final settlement (following flooding of the canal prism) along the

centreline of the proposed tunnel and retaining wall sections are shown on Figure 3-25. Also shown on this figure is the total vertical pressure, hydrostatic uplift pressure and effective vertical pressure acting on the proposed tunnel invert.

As shown on Figure 3-25 the maximum rebound along the centreline of the proposed tunnel varies in general from about 0.5 inches where the structures are to be founded on dense till to about 1.1 inches where the central portion of the actual tunnel structure is underlain by a lens of clayey silt. At the entrances to the tunnel crossing the rebound may be as much as about 2 inches due to the presences of some 15 feet of stiff silty clay immediately beneath the proposed invert.

Although the effective vertical pressure imposed on the subgrade by the structures and backfill may be as much as about 2 tons/sq.ft. in the vicinity of the flared sections of the crossing the pressure along the majority of the actual tunnel structure is about 1 ton/sq.ft. Outside the backfilled portions of the retaining wall structures the effective vertical pressure is negligible. Due to the relatively light loads imposed by the tunnel structure the corresponding elastic settlements will be minor and will probably be less than about 0.5 inch.

At the tunnel entrance the foundation stratum consists of stiff silty clay. In these areas however the imposed effective pressure should be less than about 0.25 ton/sq.ft. and the



settlement should be negligible. However, in the area of the tunnel entrances a portion of the rebound may be plastic in nature and could continue for a period of several years. These swelling or rebound pressures may be in the order of 0.5 ton/sq.ft. and the base slab of retaining structures should be designed to withstand this additional effective vertical pressure.

i) Recommended Monitoring Devices

Rebound gauges should be installed in the overburden and bedrock beneath the proposed tunnel invert to monitor the upward movement or rebound at tunnel invert elevation during excavation. This rebound will probably represent the upper limit of settlement beneath the proposed structures. As the proposed tunnel invert is to be located within several soil strata having different rebound characteristics (clayey silt, silty clay, silt and till) along the proposed tunnel centreline we suggest that 5 rebound gauges be installed in the overburden at various stations along the tunnel centreline. Suggested locations for these gauges are shown in profile on the proposed tunnel section and simplified soil stratigraphy profile given on Figure 3-25.

In addition, we suggest that 1 rebound gauge be installed in the sound bedrock at about station 29+00. This rebound gauge could serve as both a temporary bench mark when reading the overburden rebound gauges and as a gauge for measuring any possible rebound of the sound bedrock during excavation.

During excavation for the tunnel it will be undesirable to have steel casing installed within the excavation limits. Therefore, we suggest that following installation of a rebound gauge in a 6 inch diameter cased borehole, the casing be withdrawn and the hole be simultaneously backfilled with a heavy bentonite slurry to prevent sloughing from the sides of the hole. To facilitate locating the gauges during excavation we further suggest that the bentonite slurry be dyed a distinctive colour and thin walled waterproofed cardboard or transite pipe be installed in the borehole prior to pulling of the steel casing.

The installation of cardboard or transite pipe is suggested as the pipe will guide a sounding rod onto the rebound gauge but will not interfere with excavation operations.

The elevation of the rebound gauge may be determined by means of a conical shaped sounding tool pushed through the bentonite slurry and onto the rebound gauge by means of coupled 1/2 inch diameter black iron pipe. A calibrated steel or invar wire installed inside the black iron pipe should be used to measure the depth to the rebound gauge.

INSTRUMENTATION REQUIRED - SUMMARY

Due to the relatively deep nature of the cuts proposed at this site together with the presence of an artesian aquifer beneath the proposed excavations, we suggest that a fairly extensive instrumentation program be carried out to monitor hydrostatic pressures, rebound of the subsoil, surficial and internal slope movements and earth pressures. Following is a summary of a suggested instrumentation scheme:

1. Hydrostatic Pressure

a) Temporary Excavation - 6 piezometers sealed into the weathered bedrock with 2 of the piezometers located within the excavation limits. In addition, 4 piezometers installed in 2 boreholes should be sealed into the overburden deposits.

b) Permanent Roadway Approach Cuts - 5 piezometers sealed into the weathered bedrock beneath each approach cut. The piezometer leads should be connected to pressure gauges located in permanent instrumentation control stations.

2. Rebound of Subsoil at Tunnel Invert

- 5 overburden rebound gauges and 1 bedrock rebound gauge installed within the area of the excavation bottom.

3. Slope movements

a) Surficial - survey points located on the cut slopes.

b) Internal - 3 slope indicator casings installed prior to excavation at the site with 2 additional casings to be installed should the original installations indicate progressive movement.

- Ground settlement plates installed slightly behind the top of the cut slopes.

#### 4. Earth Pressures on Retaining Walls

- 4 Gloetzl earth pressure cells and 4 compatible pore pressure cells located at the quarter points of one retaining wall.
- 5 electrical strain gauges installed in the proposed combination struts and sun shield adjacent to the instrumented retaining wall.

As accurate elevations from a fixed datum will be required for the instrumentation scheme summarized above, it is suggested that a deep bench mark be established at the site prior to the beginning of any excavation. The bench mark could consist of "A" or "E" rod grouted into the bedrock and protected from overburden movements by means of casing extending from the upper surface of the bedrock to the top of the bench mark. The bench mark should be taken to a depth of at least 20 feet below the bedrock surface and should be located far enough away from the excavation to eliminate bedrock rebound effects. It is suggested that the bench mark be located near the observation platform to be constructed south-west of the canal and tunnel excavations as this area is both close enough to the site for survey purposes and provides protection for the bench mark during construction operations. To provide additional protection during construction operations, a manhole or catch basin such as the 42 inch or 48 inch diameter precast concrete catch basin and manhole shown on the Department of Highways, Ontario, Highway Standards No. DD 712 could be installed

around the top of the permanent bench mark installation. For frost protection purposes and additional safety, this manhole should extend a minimum of 4 feet below ground surface. A 6 inch diameter protective pipe should be installed around the bench mark and should extend a minimum of 10 feet below the manhole. As the perched groundwater level at the site is at or slightly above the existing ground surface provision should be made for draining the manhole.

It is understood that the proposed observation platform is to be of timber frame construction and is to be built on a 10 foot high compacted earth fill. We suggest that the bench mark be located within the fill but outside the area of the platform foundations. This would necessitate installing the bench mark after the earth fill is placed or the careful control of backfilling around the installation during filling operations. If the bench mark is installed prior to placing of the fill the pre-cast concrete catch basin and manhole should extend down to the original ground surface. This location for the bench mark has the advantage that the manhole can be easily drained by a small culvert leading to the face of the slope.

Should it be inconvenient to install the permanent bench mark within the compacted earth fill it could be located within or adjacent to the parking area adjoining the observation platform. These locations would however make gravity drainage of the protective manhole difficult.

MISCELLANEOUSA) Notes on the Prevention of Concrete  
Deterioration by Sulphate Attack

Chemical analysis carried out on samples of groundwater obtained from the till and bedrock aquifer indicate that the groundwater contains more than 2,000 p.p.m. sulphates. Such high sulphate concentrations in groundwater may cause deterioration of concrete containing ordinary Portland Cement. Experience has shown however that movement of groundwater is required for progressive deterioration since the initial sulphate attack will form a thin coating of deteriorated concrete which, unless washed away exposing fresh concrete to attack, will arrest the deterioration process.

As the cost of using sulphate resisting cement in the concrete will be high we suggest that the tunnel and retaining wall structures be designed to minimize the flow of water at the concrete faces and thus prevent serious progressive sulphate attack on concrete formed from ordinary Portland Cement. Since portions of the structures are to be founded on the till there will be groundwater movement beneath the structures and it will be necessary to protect the bottom of the monolithic floor slab. This protection could probably be accomplished by pouring a mat of concrete made with sulphate resisting cement over the entire bottom of the excavation. The mat could be covered by vinyl sheets

and/or asphalt layers and the tunnel built on these. The sides of the tunnel should be coated with asphalt and the construction excavation backfilled with impervious clay compacted in place. The clayey silt to silty clay from the desiccated crust is recommended for this backfill. The clay could either be compacted directly against the tunnel walls or, if desired, a 2 to 3 foot wide zone of "puddled" clay could be placed against the face of the wall. The moisture content of the compacted clay backfill should be chosen for minimum permeability rather than maximum density. In addition to employing impervious clay backfill at the site the outside face of the walls should have a batter of about 10 vertical to 1 horizontal, the wall being thicker at the base. This is most important as it means that settlement of the clay backfill will result in compaction against the outer face of the tunnel walls. The earth cover on the roof of the tunnel should also be impervious clay backfill and should be well compacted.

With compacted impervious soil around the walls of the tunnel there will be practically no flow of water. This means that the sulphates will form only a thin skin of soft paste on the concrete if at all, and experience suggests that no further deterioration will occur.

An alternative method of preventing the sulphates in the groundwater from reaching the concrete tunnel walls may be the use of pre-formed bentonite panels placed between the tunnel walls

and the compacted clay backfill. It is understood however, that the swelling properties of bentonite may be effected by high sulphate concentrations in groundwater. In addition, the swelling pressures induced on the tunnel wall may exceed anticipated earth pressures.\*

B) Notes on the Design of Sumps

It is understood that a sump is to be installed beneath each of the tunnel portals. As the main purpose of these sumps is to collect surface water runoff the capacity of the sumps will be large. The runoff collected in the sumps is to be pumped into the canal by means of large capacity sump pumps designed to operate only when the water in the sumps reaches a pre-determined depth.

In addition to collecting surface water runoff the sumps will be used to collect the discharge from the permanent gravity relief well system. The anticipated discharge entering each sump (i.e. from one-half of the well system) will be about 150 I.G.P.M. Although this flow rate is almost negligible when compared to the possible peak rate of surface water runoff the discharge will be continuous and hence some special provisions should be

---

\* The swelling properties of bentonite panels are presently being considered and will be reported following additional laboratory studies.



made for its control.

As the discharge from the wells will consist of groundwater from the bedrock aquifer it will contain sulphate concentrations of greater than 2,000 p.p.m. The sumps should therefore be constructed of concrete made using sulphate resistant cement or they should be provided with an impervious lining. Due to the possible corrosive effects of the groundwater steel plates are not recommended as a liner at this time. The discharge pipes from the wells should also be constructed of corrosion resistant material or should be readily exposed for easy replacement.

To avoid excessive operation and corrosion of the large capacity sump pumps designed to handle surface water runoff it may be economical to install additional small capacity sump pumps designed for continuous operation.

C) Notes on Roadway Subgrade in Approach Cuts

Although the gravity relief well system proposed for the tunnel site will reduce the piezometric water levels in the upper clays and silts it will not drain these upper deposits. Therefore, in the approach cuts the generally frost susceptible subgrade will be saturated and subjected to a readily available supply of groundwater. This could result in heaving of the pavement due to frost action. In addition some horizontal seepage of water from the subgrade into the subbase of the permanent road-

way pavement may occur unless special preventive measures are taken. In the extreme, this seepage could result in local hydrostatic uplift of the asphalt wearing surface of the pavement.

To prevent uplift of the roadway pavement and avoid as much as possible costly sub-excavation and backfilling with granular material we suggest that the roadway subgrade section shown on Figure 3-26 could be constructed in the approach cut portions of the tunnel crossing.

The seepage of groundwater into the granular material forming the pavement base may be prevented by installing a 6 inch drainage blanket of 3/4 inch crushed stone beneath the pavement. As the majority of water intercepted by the drainage blanket will flow down the 6% roadway grade transverse interceptor drains should be provided. These drains could consist of 6 inch diameter weeping tiles laid in 18 inch deep trenches as shown on Figure 3-26. The transverse drains should be connected to longitudinal drains installed beneath the curb lines but could, if desired, discharge into open ditches. Where the roadway subgrade consists of silts and sandy silts the transverse drains should be installed at 25 foot centres. If the subgrade consists of relatively impervious clayey silt the seepage will be minor and will generally be through thin interbedded silt layers. In these areas the spacing of transverse drains could be increased to 50 feet.

To prevent a loss of ground due to silt particles from the subgrade being carried into the drainage blanket by the groundwater a filter layer should be installed beneath the 3/4 inch crushed stone. The filter layer should be at least 6 inches thick and the gradation of the filter material should fall within the limits shown on Figure 3-27.

Frost penetration into the drainage blanket and into the frost-susceptible silty subgrade material could be prevented by installing a layer of thermal insulation beneath the roadway pavement. The insulation used should be both waterproof and sufficiently strong to support the design traffic loads. It is reported that foamed plastic panels have been used with considerable success as thermal insulation under roads in Ontario and Manitoba\*. We suggest that serious consideration be given to the use of such material at this site. As 1 inch of insulation is thermally equivalent to about 2 feet of soil the minimum thickness of insulation should be about 2 inches. To uniformly attenuate frost penetration beneath the pavement the insulation must be extended some distance into the shoulder of the road and care must be exercised in joint construction at slope changes.\*\*

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\* Young, F.D., 1965. "Experimental Foamed Plastic Base Course." Highway Research Record, No. 101, 1965: 1-10.

\*\* Straub, A.L., and Williams, W.G., 1967. "Use of Insulation to Uniformly Retard Frost Penetration under a Highway Pavement." Highway Research Record, No. 181, 1967: 77-93.

The installation of thermal insulation, drainage blanket and filter layer will require about 14 inches of sub-excavation in addition to excavation required for the actual pavement. Should the insulation be omitted it will be necessary to provide a minimum of 3 feet of earth cover over the drainage blanket to prevent freezing and frost heaving.

*J B Davis*

J. B. Davis, P. Eng.



*V. Milligan*

V. Milligan, P. Eng.

JBD:jg  
67106  
July 19, 1968.

## APPENDIX A

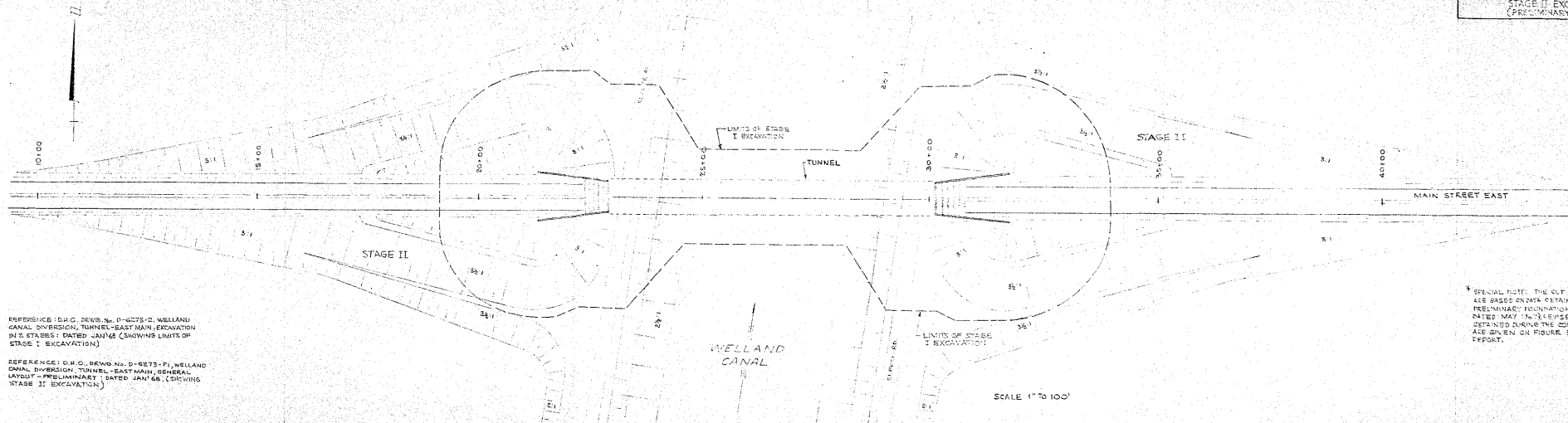
### Chemical Analyses of Groundwater

Detailed chemical analyses were carried out by the Chemical Section, Materials and Testing Division of the Department of Highways, Ontario on 3 samples of groundwater obtained from piezometers installed in the bedrock aquifer underlying the proposed tunnel site. The samples tested were taken from the piezometers installed within the bedrock in Boreholes T-4, T-125 and T-126 (piezometers 3, 5 and 8 respectively). The samples were obtained by inserting a small diameter plastic tube down the piezometer lead to a depth of about 25 feet below the stabilized water level. Water was expelled from the piezometer lead by pumping air under a pressure of about 30 lb/sq.in. down the small diameter tube.

The following table summarizes the results of the detailed chemical analyses. As discussed in the main body of this report, it is considered that the water samples tested are not representative of the groundwater in the aquifer.

TABLE I

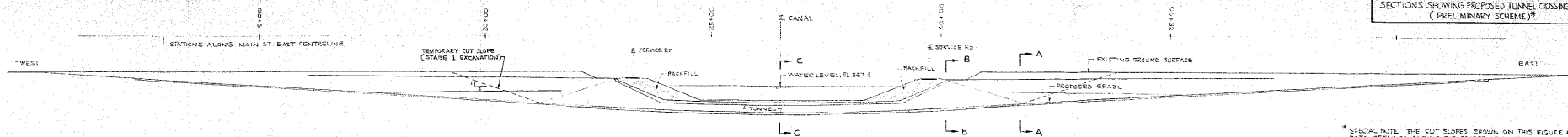
SAMPLE TESTED FOR	UNITS	PIEZOMETER 3 B.H. T-4	PIEZOMETER 5 B.H. T-125	PIEZOMETER 8 B.H. T-126
Sulphide	p.p.m.	16	Nil	Nil
Sulphate	p.p.m.	797	140	358
Chloride	p.p.m.	7	41	27
Total Hardness	p.p.m.	1209	256	433
Soluable Iron	p.p.m.	10	5	5
Calcium	p.p.m.	456	92	184
Magnesium	p.p.m.	159	37	42
Dissolved Solids	p.p.m.	2298	308	694
Total Alkalinity	g/litre	0.30	0.12	0.11
Conductivity	mho/cm	$1.652 \times 10^{-3}$	$0.350 \times 10^{-3}$	$0.733 \times 10^{-3}$
pH	-	7.7	8.2	8.2



REFERENCE: D.H.C. DWGS. No. D-6273-2, WELLAND  
CANAL DIVERSION, TUNNEL-EAST MAIN EXCAVATION  
IN 2 STAGES: DATED JAN'68 (SHOWING LIMITS OF  
STAGE I EXCAVATION)

REFERENCE: D.H.C. DWGS. No. D-6273-1, WELLAND  
CANAL DIVERSION, TUNNEL-EAST MAIN, GENERAL  
LAYOUT - PRELIMINARY: DATED JAN'68. (SHOWING  
STAGE II EXCAVATION)

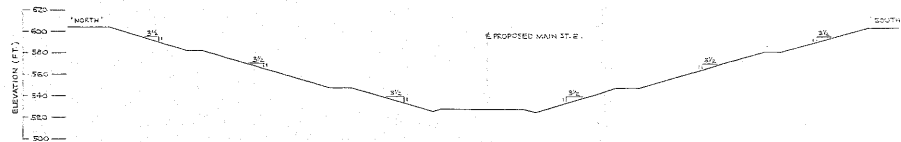
\* SPECIAL NOTE: THE CUT SLOPES SHOWN ON THIS FIGURE  
ARE BASED ON DATA OBTAINED DURING THE COURSE OF THE  
PRELIMINARY FOUNDATION INVESTIGATION (OUR REPORT 30154-  
DATED MAY 1967). REVISED CUT SLOPES BASED ON DATA  
OBTAINED DURING THE COURSE OF THE PRESENT INVESTIGATION  
ARE GIVEN ON FIGURE 3-13 AND IN ADDENDUM 'A' OF THIS  
REPORT.



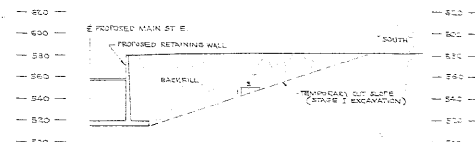
SECTION ALONG CENTRELINE OF PROPOSED WELLAND TUNNEL

SCALE 1" TO 100'

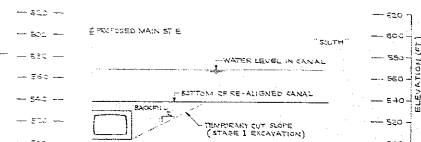
\* SPECIAL NOTE: THE CUT SLOPES SHOWN ON THIS FIGURE ARE BASED ON DATA OBTAINED DURING THE COURSE OF THE PRELIMINARY FOUNDATION INVESTIGATION (OUR REPORT 66184 DATED MAY, 1967). REVISED CUT SLOPES BASED ON DATA OBTAINED DURING THE COURSE OF THE PRESENT INVESTIGATION ARE GIVEN ON FIGURE 3-13 AND IN ADDENDUM 'A' OF THIS REPORT.

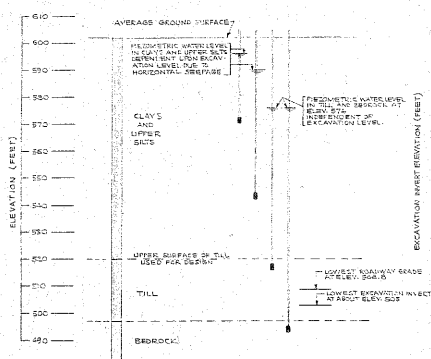


REFERENCE: D.H.O. DRWG. No D-6273-PI, WELLAND CANAL DIVERSION, TUNNEL-EAST MAIN, GENERAL LAYOUT-PRELIMINARY DATED JAN. 1968, SHOWING STAGE II EXCAVATION.

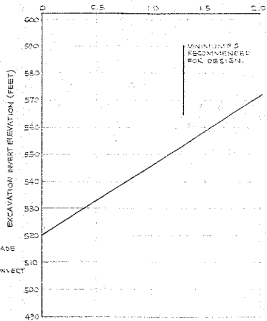


SCALE 1" TO 50'

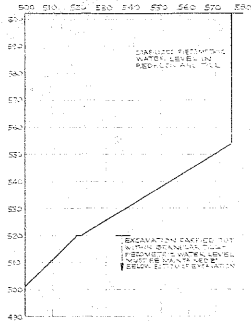




FACTOR OF SAFETY AGAINST HYDROSTATIC UPLIFT  
(ASSUMING NO RELIEF OF HYDROSTATIC HEADS IN BEDROCK & TILL)



MAXIMUM ALLOWABLE PIEZOMETRIC LEVEL IN TILL AND BEDROCK FOR FACTOR OF SAFETY AGAINST HYDROSTATIC UPLIFT IS



$$F = \frac{(2A \cdot T) + (S_u \cdot P \cdot T)}{H_p \cdot A \cdot \gamma_w}$$

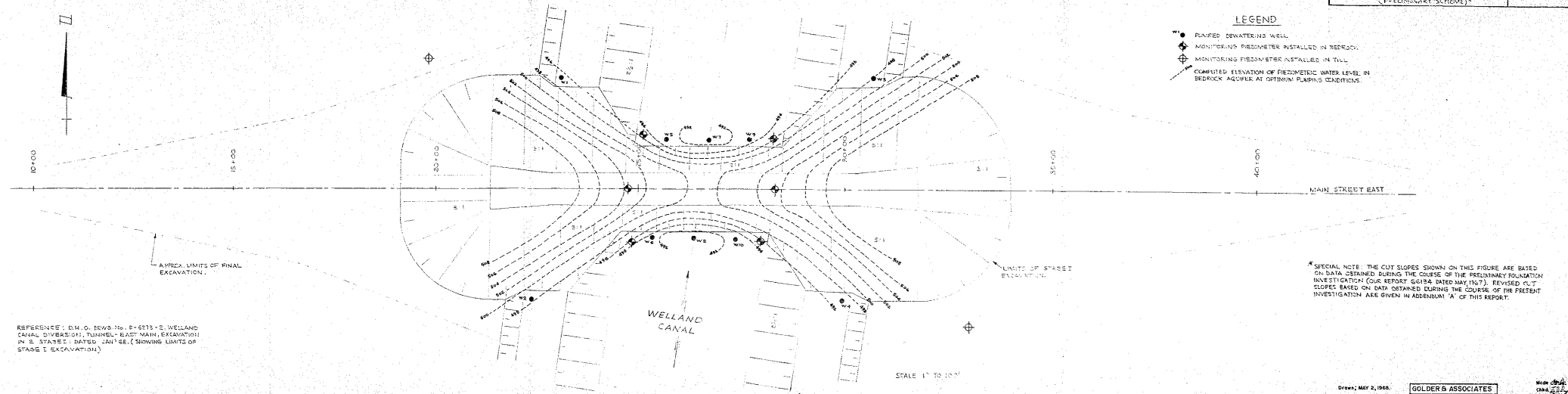
COMPUTATION OF FACTOR OF SAFETY  
AGAINST HYDROSTATIC UPLIFT

WHERE  $F$  = FACTOR OF SAFETY AGAINST HYDROSTATIC UPLIFT  
 $E$  = TOTAL UNIT WEIGHT OF SOIL (125 LB./CU. FT.)  
 $\gamma_w$  = UNIT WEIGHT OF WATER (62.4 LB./CU. FT.)  
 $A$  = BASE AREA OF EXCAVATION (SQ. FT.)  
 $T$  = THICKNESS OF CLAYS AND SILTS ABOVE TOP OF TILL (FT.)  
 $P$  = PERIMETER OF EXCAVATION (FT.)  
 $S_u$  = MOBILIZED SHEAR STRENGTH (LB./SQ. FT.)  
 $H_p$  = PIEZOMETRIC GROUNDWATER HEAD (FT.)

FOR LONG, WIDE EXCAVATION  $S_u \cdot P \cdot T$  IS  
NEGLECTABLE COMPARED TO  $2A \cdot T$

$$F = \frac{2A \cdot T}{H_p \cdot A \cdot \gamma_w}$$





REFERENCE: D.H.O. DRAWING NO. D-6775-P1, WELLAND CANAL  
DIVERSION TUNNEL - EAST MAIN, GENERAL LAYOUT - PRELIMINARY  
DATED JAN. 1968, SHOWING STAGE II EXCAVATION.

SECTION A-A  
STATION 31+72  
SECTION PERPENDICULAR TO PERMANENT CUT SLOPES

SCALE 1" TO 50'

SECTION B-B  
STATION 30+12  
SECTION THROUGH PROPOSED RETAINING WALL

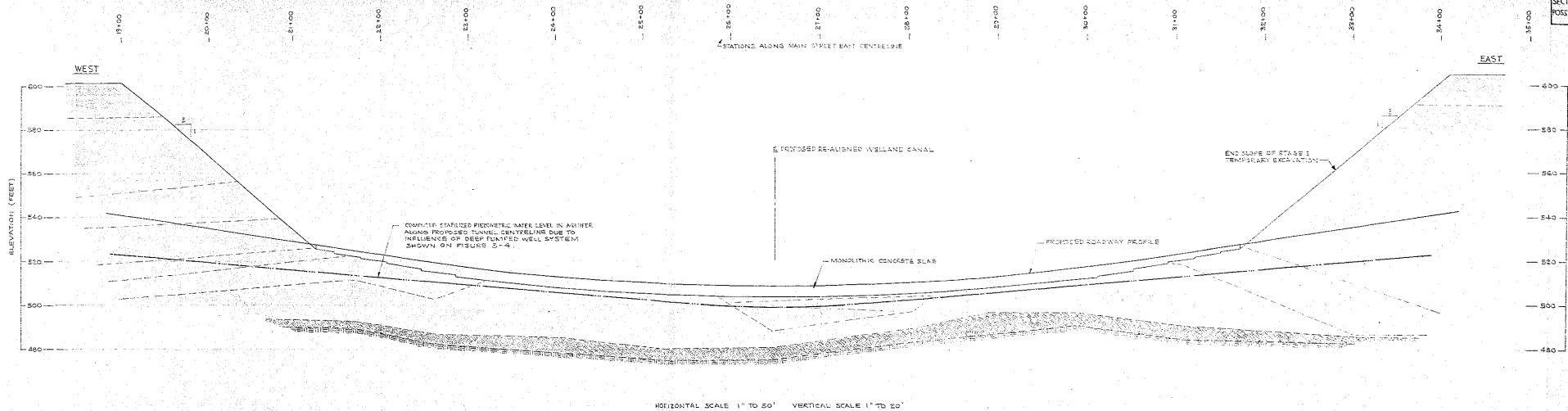
SECTION C-C  
STATION 30+50  
SECTION THROUGH PROPOSED TUNNEL STRUCTURE

DRAWN: MAY 1, 1968

GOLDER

SECTION ALONG PROPOSED TUNNEL CENTRELINE SHOWING  
POSSIBLE PERMETRIC WATER LEVEL DURING CONSTRUCTION

FIGURE 3-5



#### STRATIGRAPHY

- HARD MOTTELED BROWN CLAYEY SILT TO SILTY CLAY (CONSOLIDATED SILT)
- HARD TO STIFF REDDISH-BROWN CLAYEY SILT
- DENSE TO VERY DENSE REDDISH-BROWN SILT TO SANDY SILT
- STIFF TO VERY STIFF REDDISH-BROWN SILTY CLAY
- VERY DENSE REDDISH-BROWN SILTY SAND TO SANDY SILT, SOME GRAVELLY ZONES (TILL)
- WEATHERED AND FRACTURED BEDROCK (QUARTZITE)
- FAIRLY SOUND TO SOUND BEDROCK

DRAWN: MAY 1, 1968

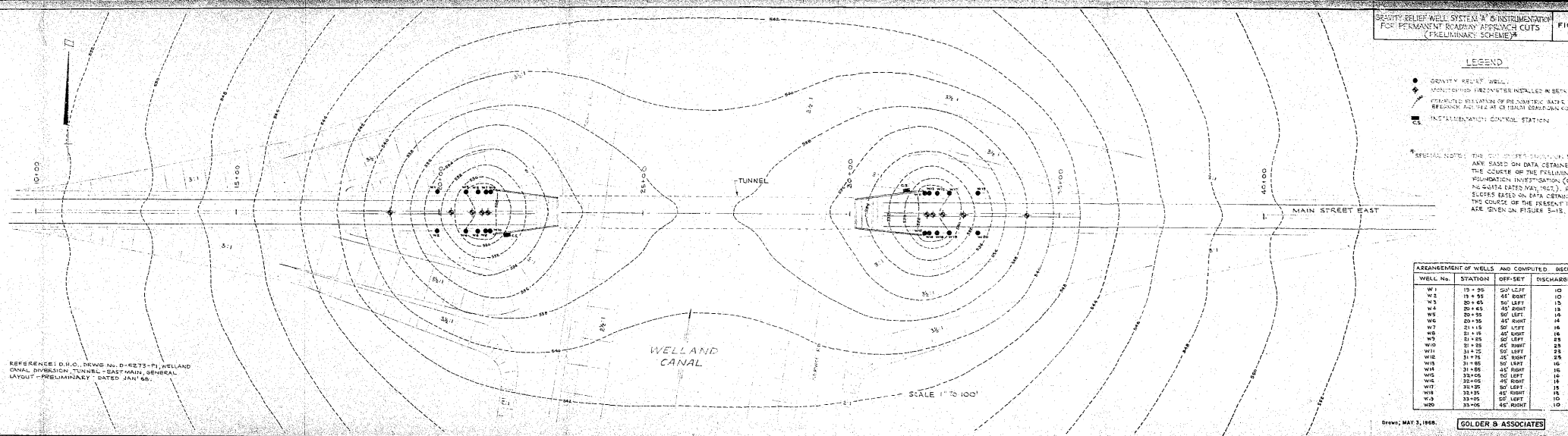
GOLDER & ASSOCIATES

MADE BY: G.A.

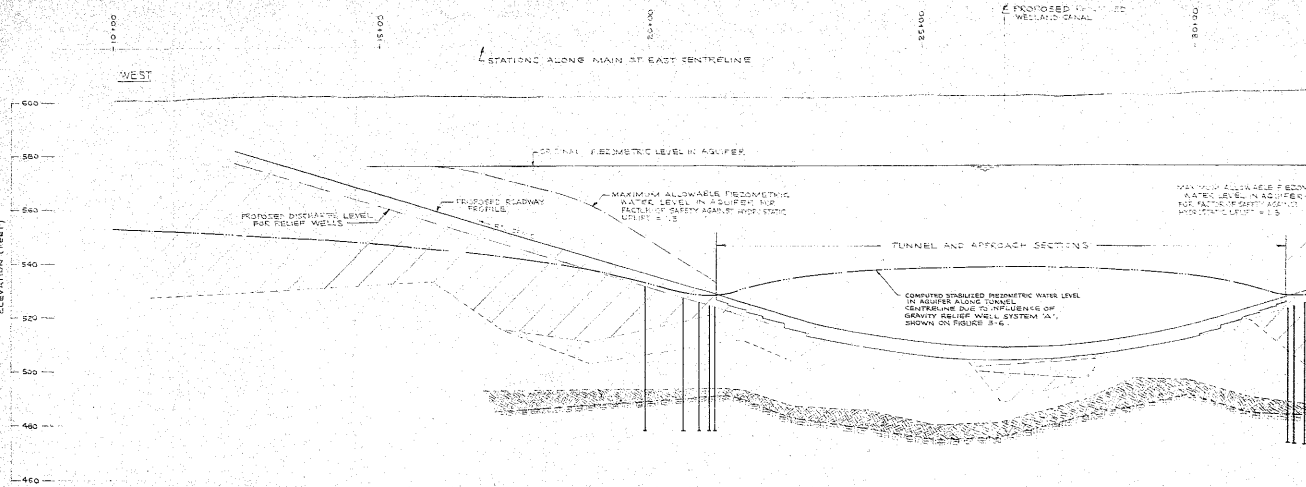
APP'D: J.C.

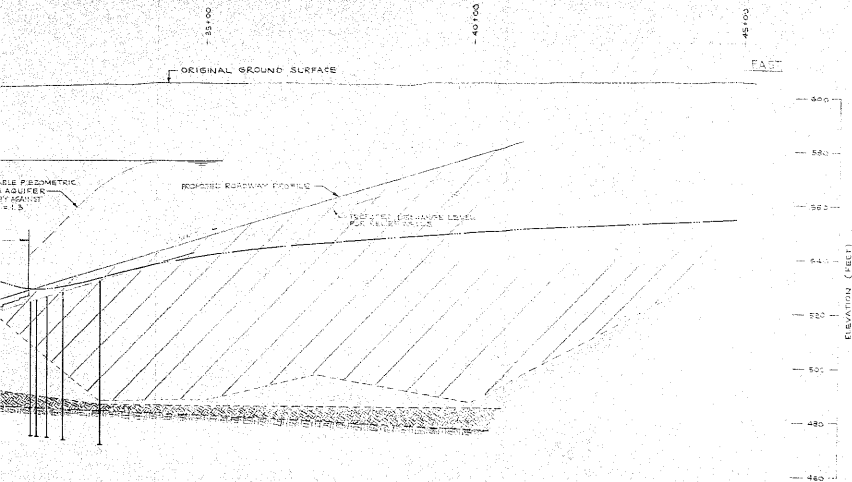
ALONG TUNNEL CENTRELINE  
STATION 31+72

FIGURE 3-7



ELEVATION (FEET)





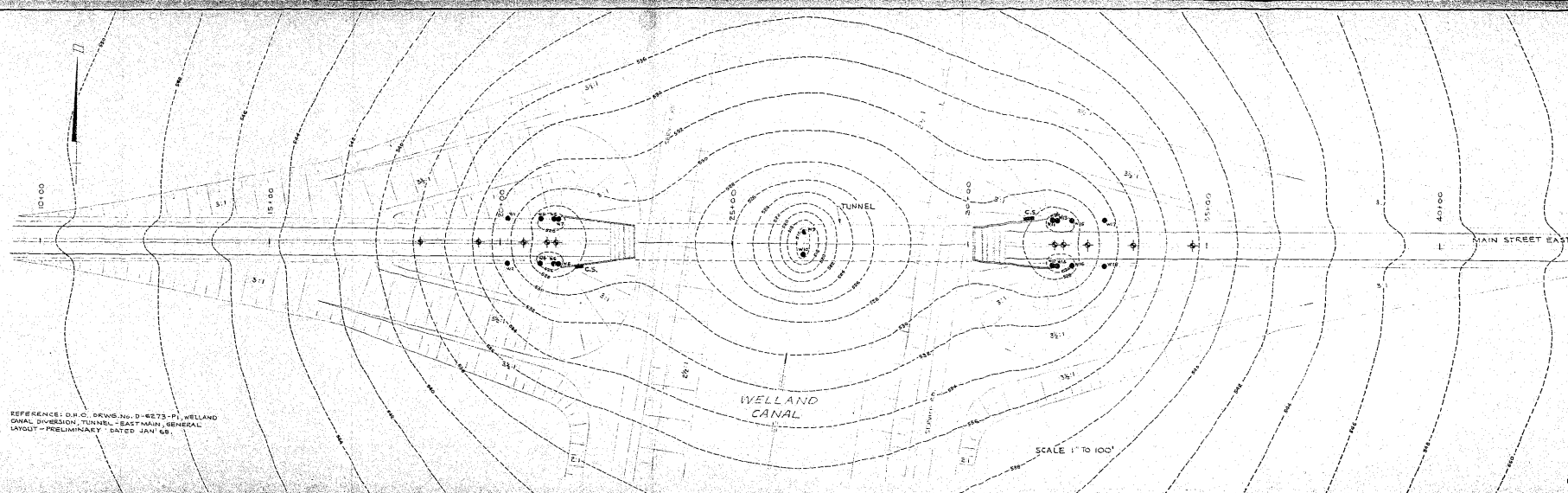
HORIZONTAL SCALE 1" TO 50' VERTICAL SCALE 1" TO 50'

Drawn MAY 1, 1958

GOLDER & ASSOCIATES

Scale 1" = 50'

REFERENCE: L.C. CANAL DIVERSION LAYOUT - PRELIMINARY



REFERENCE: D.H.O. DWS No. D-5273-P, WELLAND CANAL DIVERSION, TUNNEL - EAST MAIN, GENERAL LAYOUT - PRELIMINARY DATED JAN 68.

GRAVITY RELIEF WELL SYSTEM & INSTRUMENTATION FOR PERMANENT STATION APPROACH CUTS (PRELIMINARY PLAN) FIGURE 3-10

LEGEND

- GRAVITY RELIEF WELL
- MONITORING PRELIMINARY INSTALLATION TO BEDROCK
- COMPUTED PRELIMINARY OF ESTIMATED WATER LEVEL IN BLACKBOX ACQUISITION AT OPTIMUM DRAINAGE STATION
- INSTRUMENTATION CONTROL STATION

SPECIAL NOTE: THE CUT SLOPE DATA IN THIS FIGURE ARE BASED ON DATA OBTAINED DURING THE COURSE OF THE PRELIMINARY FOUNDATION INVESTIGATION. (OUR REPORT NO. 64154 DATED MAY 1957). REQUIRED CUT SLOPES BASED ON DATA OBTAINED DURING THE COURSE OF THE PRESENT INVESTIGATION ARE GIVEN ON FIGURE 3-15.

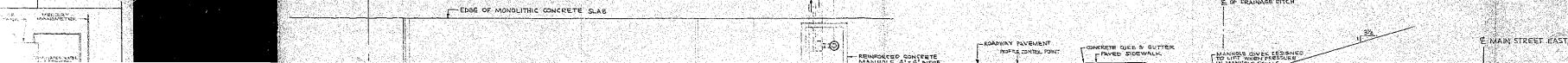
ARRANGEMENT OF WELLS AND COMPUTED DISCHARGES			
WELL No.	STATION	OFF - SET	DISCHARGE, RPM
W1	20+15	50' LEFT	7
W2	20+15	45' RIGHT	7
W3	20+45	50' LEFT	12
W4	20+45	45' RIGHT	12
W5	21+15	50' LEFT	13
W6	21+15	45' RIGHT	13
W7	21+45	50' LEFT	18
W8	21+45	45' RIGHT	18
W9	22+45	55' LEFT	48
W10	22+45	45' RIGHT	48
W11	23+15	50' LEFT	18
W12	23+15	45' RIGHT	18
W13	23+45	50' LEFT	18
W14	23+45	45' RIGHT	18
W15	24+15	50' LEFT	12
W16	24+15	45' RIGHT	12
W17	24+45	50' LEFT	7
W18	24+45	45' RIGHT	7

Drawn: APRIL 6, 1958

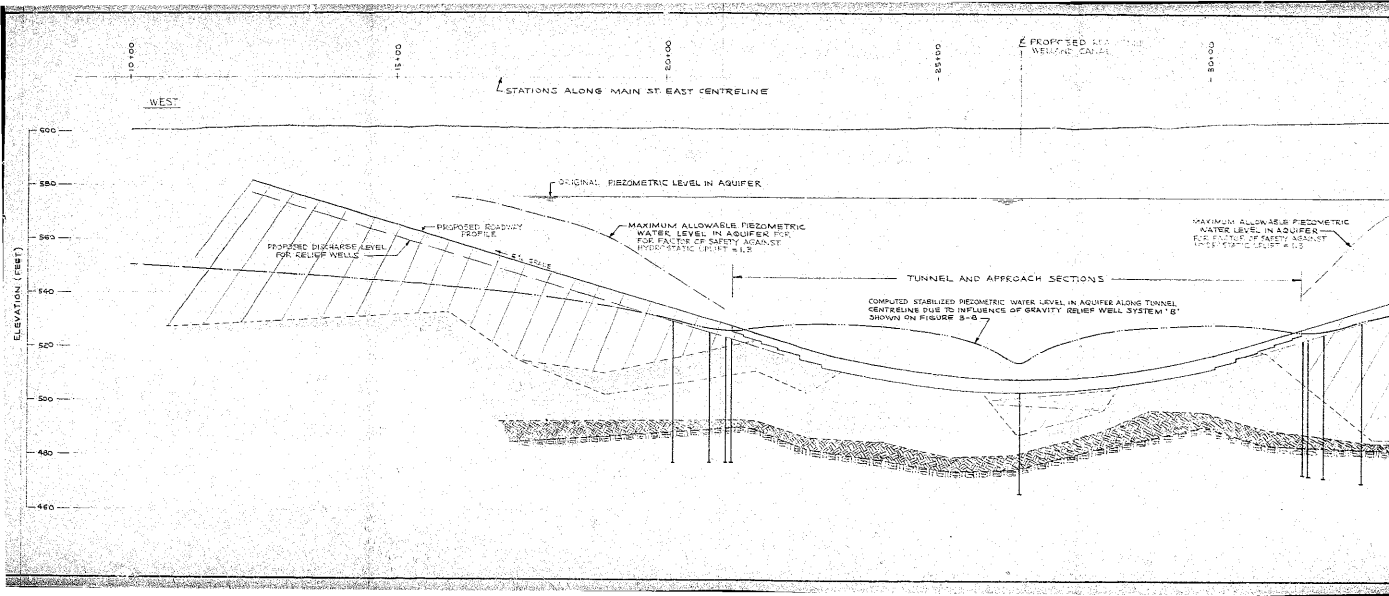
GOLDER & ASSOCIATES

Scale 1" = 50'

FIGURE 3-10

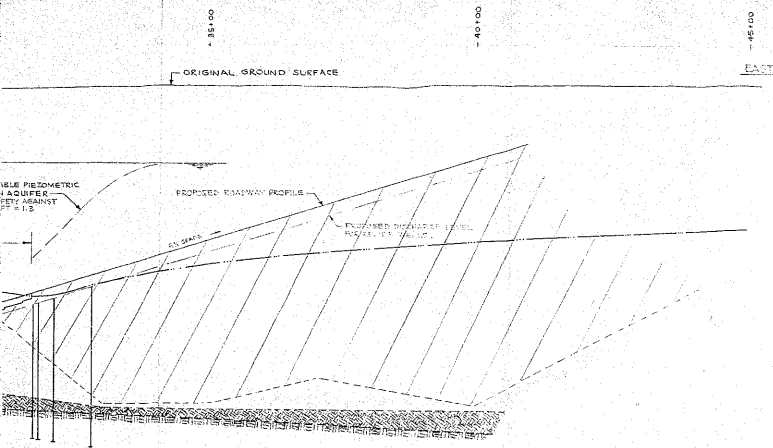


SUGGEST WELL SYS



PIEZOMETRIC WATER LEVEL ALONG TUNNEL CENTRELINE  
GRAVITY RELIEF WELL SYSTEM 'B'

FIGURE 3-9



LEGEND

GRAVITY RELIEF WELL

STRATIGRAPHY

- UPPER CLAYS AND SILTS
- LOWER SILT TO SANDY SILT
- VERY DENSE SILT TO SANDY SILT (FULL)
- WEATHERED & FRACTURED BEDROCK (GULF)
- FAIRLY SOUND TO SOUND BEDROCK

SCALE

HORIZONTAL SCALE 1" TO 100'  
VERTICAL SCALE 1" TO 20'

Drawn: MAY 6, 1966

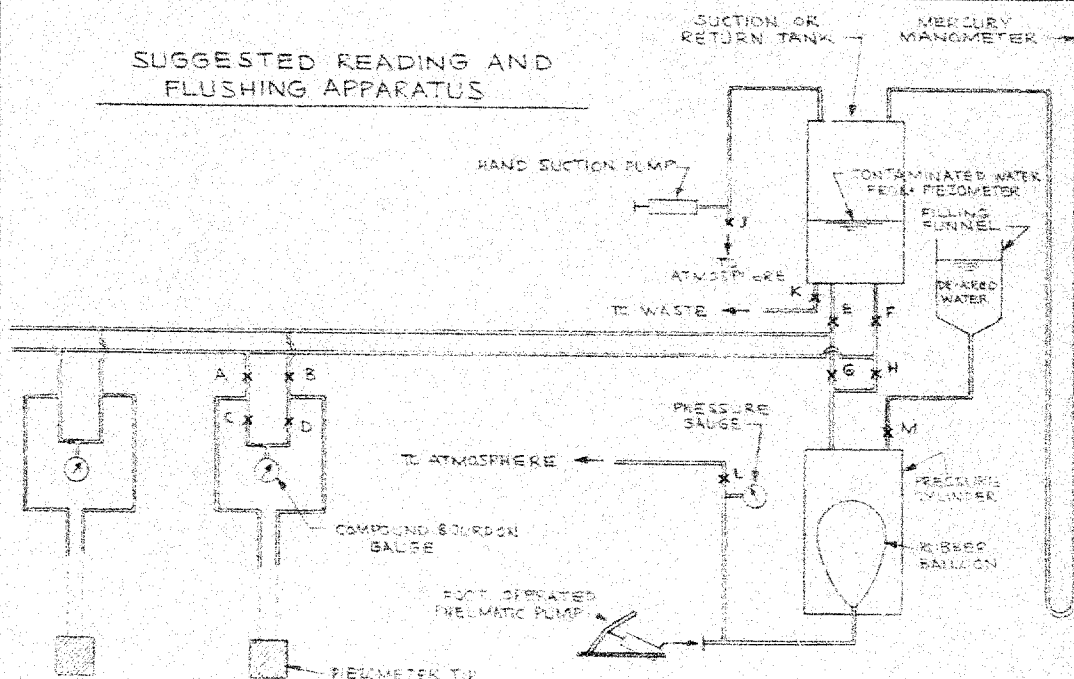
GOLDER & ASSOCIATES

Made: *[Signature]*  
Chd. *[Signature]*  
Appd. *[Signature]*

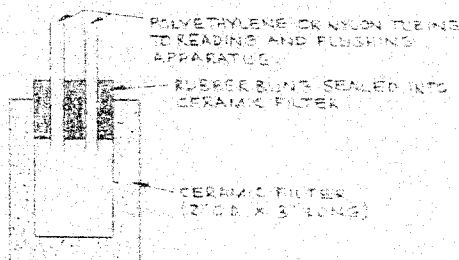


# SUGGESTED PERMANENT PIEZOMETER INSTALLATION

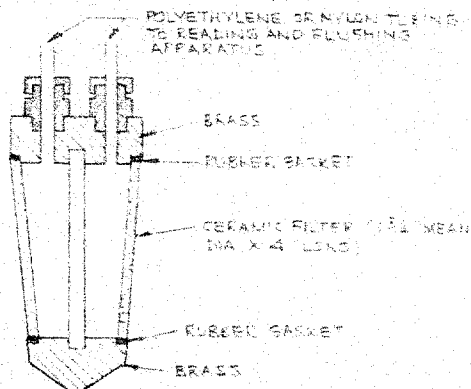
FIGURE 3-10



## SUGGESTED PIEZOMETER TIPS



### A. CASAGRANDE BOREHOLE PIEZOMETER



### B. BISHOP BOREHOLE PIEZOMETER

## OPERATING INSTRUCTIONS

### TO READ

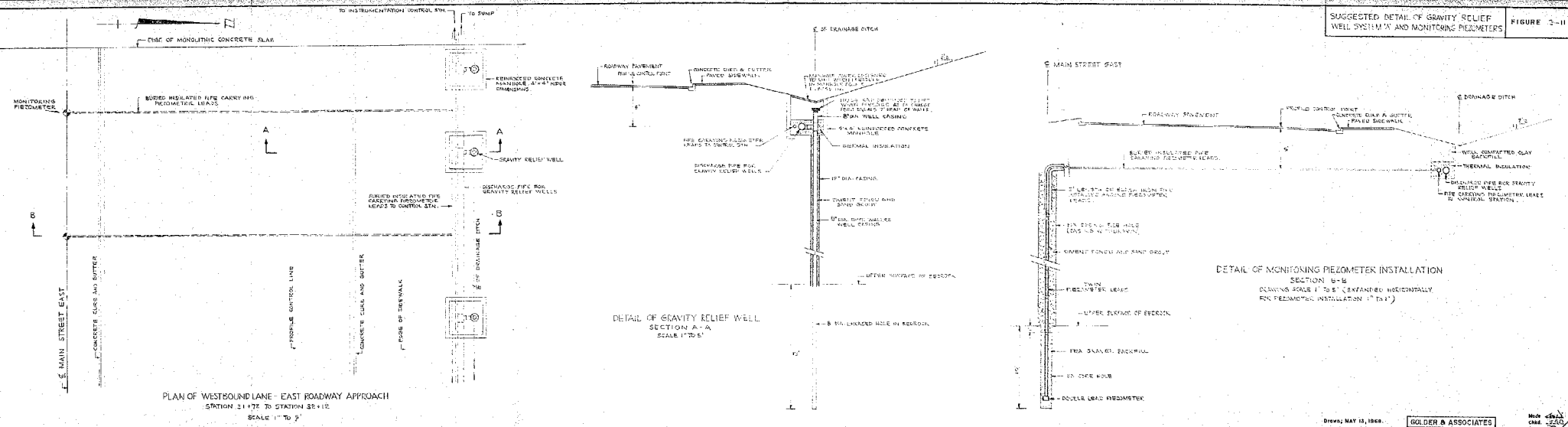
1. CLOSE VALVES A AND B - OPEN VALVE C OR D - READ PIEZOMETRIC PRESSURE ON COMPOUND BOURDON GAUGE

### TO FLUSH

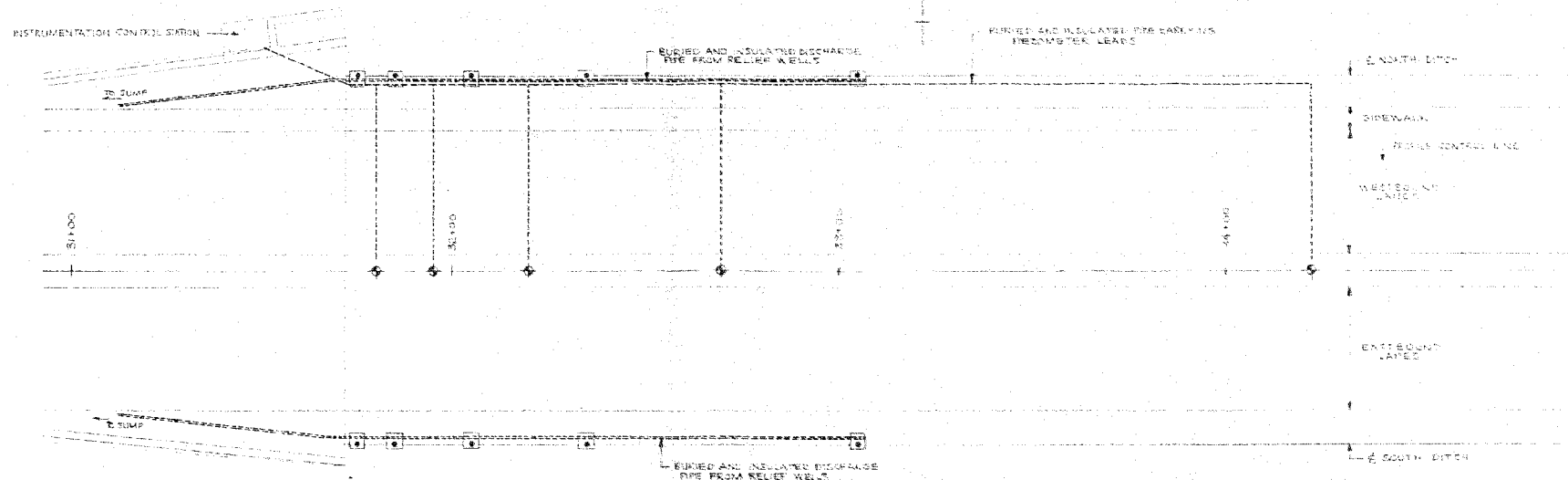
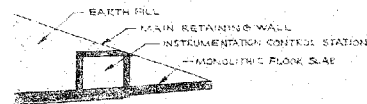
1. FILL PRESSURE CELL WITH DE-AIRIED WATER FROM FILLING FUNNEL - CLOSE VALVE M
2. CLOSE VALVES C AND D - OPEN VALVES A AND B
3. CLOSE VALVES F AND G - OPEN VALVES E AND H
4. APPLY A PNEUMATIC PRESSURE TO RUBBER BALLON. THIS FORCES DE-AIRIED WATER FROM PRESSURE CYLINDER THROUGH VALVES H AND A TO PIEZOMETER AND FLUSHES AIR OR CONTAMINATED WATER FROM THE PIEZOMETER AND LEADS THROUGH VALVES B AND E TO THE SUCTION OR RETURN TANK
5. IF PIEZOMETER IS SEVERAL FEET BELOW GAUGE LEVEL IT MAY BE NECESSARY TO CLOSE VALVE J AND APPLY A SMALL NEGATIVE PNEUMATIC PRESSURE ON THE RETURN TANK
6. TO REVERSE FLUSHING CLOSE VALVES E AND H AND OPEN VALVES F AND G
7. WHEN FLUSHING IS COMPLETED OPEN VALVE L TO RELEASE PRESSURE IN PRESSURE CYLINDER

GOLDER & ASSOCIATES

Made: *inc.*  
Chkd: *AB*  
Appd: *[Signature]*



# SIDE VIEW OF INSTRUMENTATION CONTROL STATION



SCALE 1" TO 30'

PLAN OF EAST ROADWAY APPROACH SHOWING  
PIPING LAYOUT FOR GRAVITY RELIEF WELL  
SYSTEM 'A' AND MONITORING PIEZOMETERS.

FIGURE 5-12

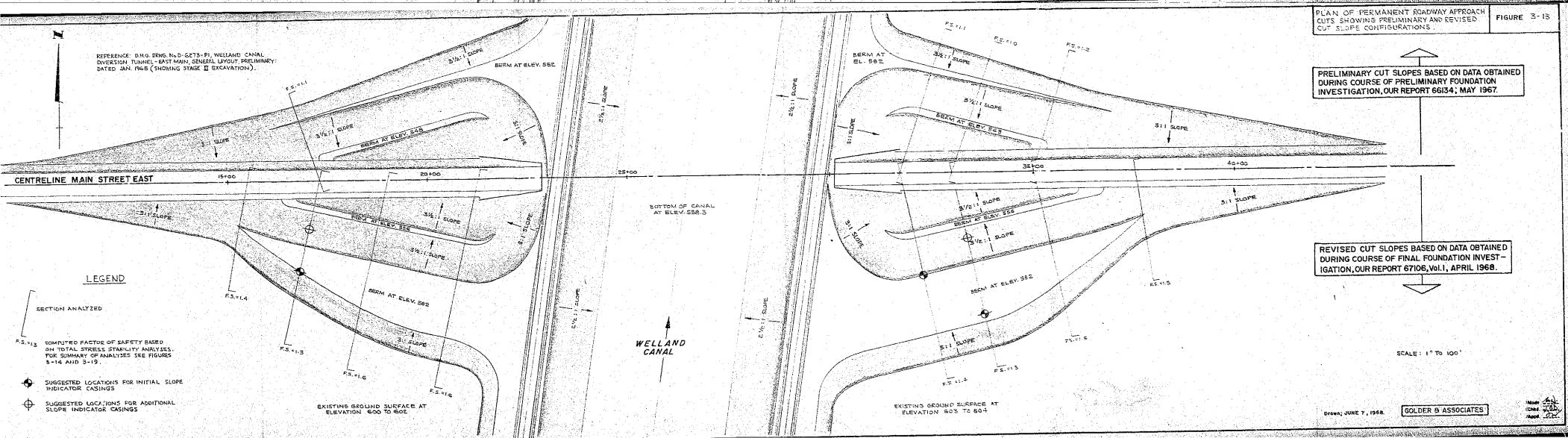
## LEGEND

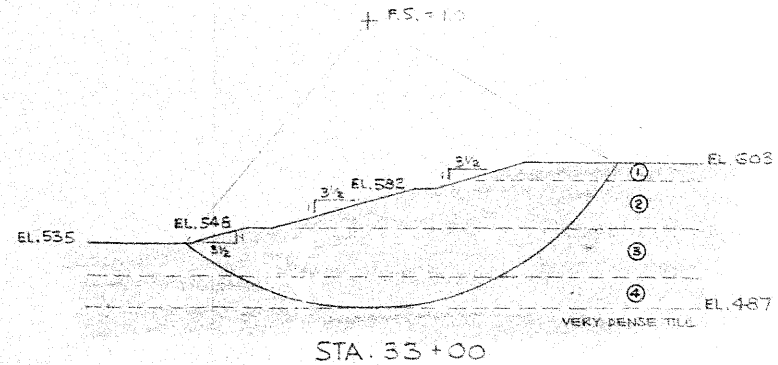
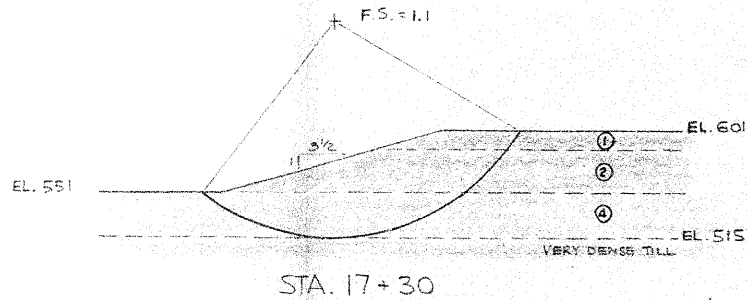
- CONCRETE MANHOLE AND GRAVITY RELIEF WELL
- MONITORING PIEZOMETER

Drawn: MAY 13, 1968

GOLDER & ASSOCIATES

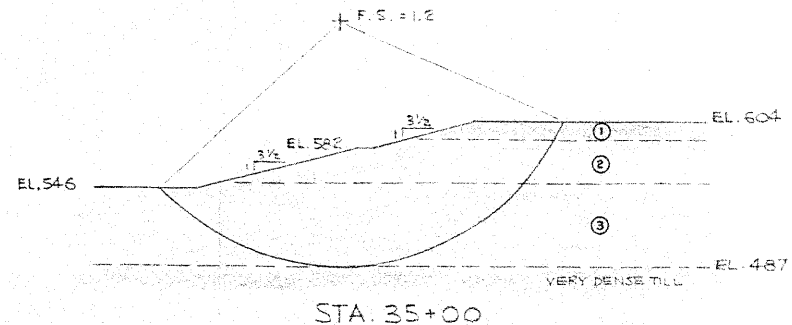
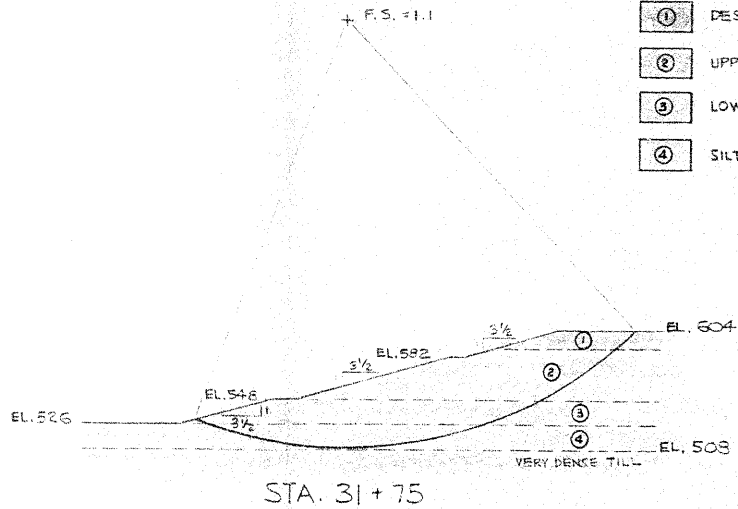
Mod: *[Signature]*  
Chk: *[Signature]*  
App: *[Signature]*





ENGINEERING PROPERTIES OF SOIL STRATA

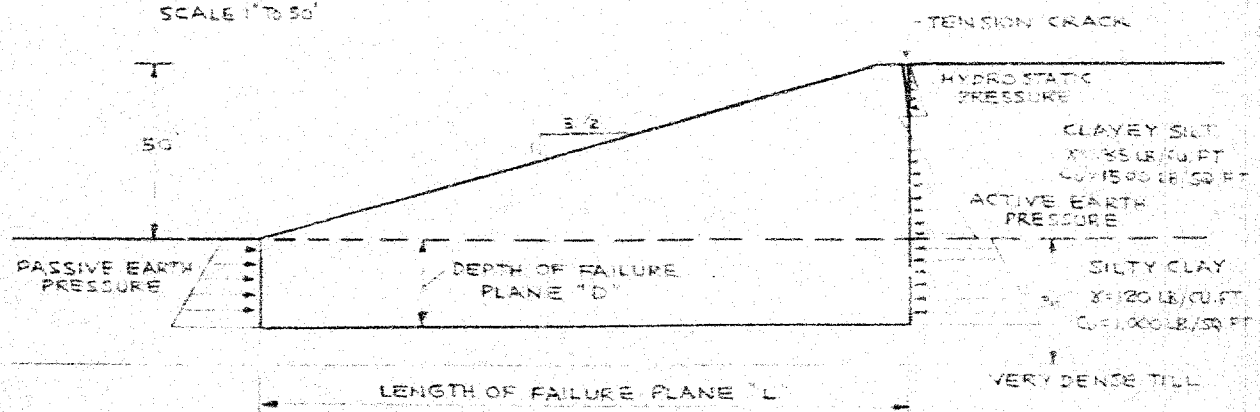
- ① DESICCATED CRUST ;  $C_u = 0$ ,  $\gamma = 135$  LB./CU.FT. (ALLOWANCE FOR TENSION CRACK)
- ② UPPER PORTION OF CLAYEY SILT ;  $C_u = 1,750$  LB./SQ.FT.,  $\gamma = 135$  LB./CU.FT.
- ③ LOWER PORTION OF CLAYEY SILT ;  $C_u = 1,500$  LB./SQ.FT.,  $\gamma = 135$  LB./CU.FT.
- ④ SILTY CLAY ;  $C_u = 1,000$  LB./SQ.FT.,  $\gamma = 120$  LB./CU.FT.



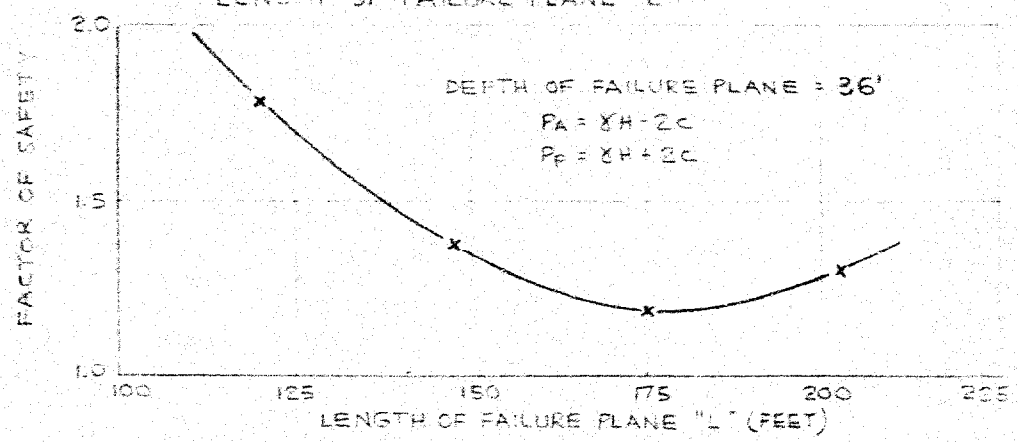
# SUMMARY OF SLIDING WEDGE STABILITY ANALYSES PRELIMINARY CUT SLOPE AT STATION 17+30

FIGURE 8-15

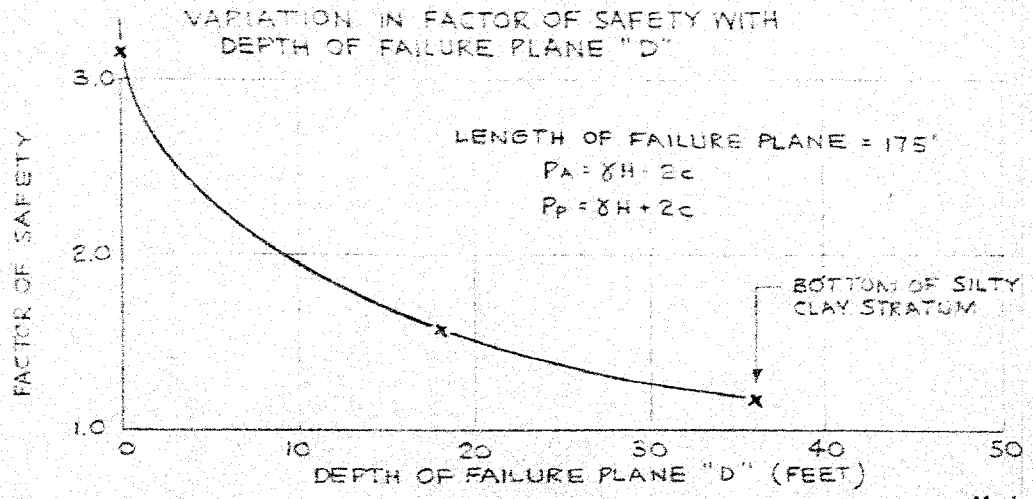
KEY SKETCH  
SCALE 1" TO 50'



VARIATION IN FACTOR OF SAFETY WITH  
LENGTH OF FAILURE PLANE "L"



VARIATION IN FACTOR OF SAFETY WITH  
DEPTH OF FAILURE PLANE "D"

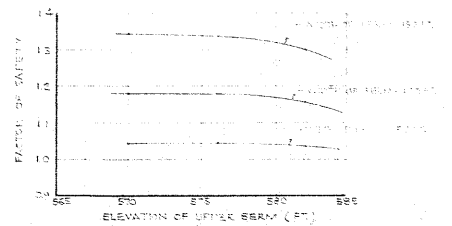
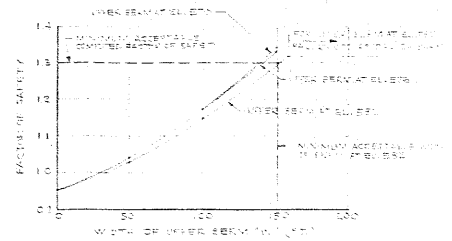
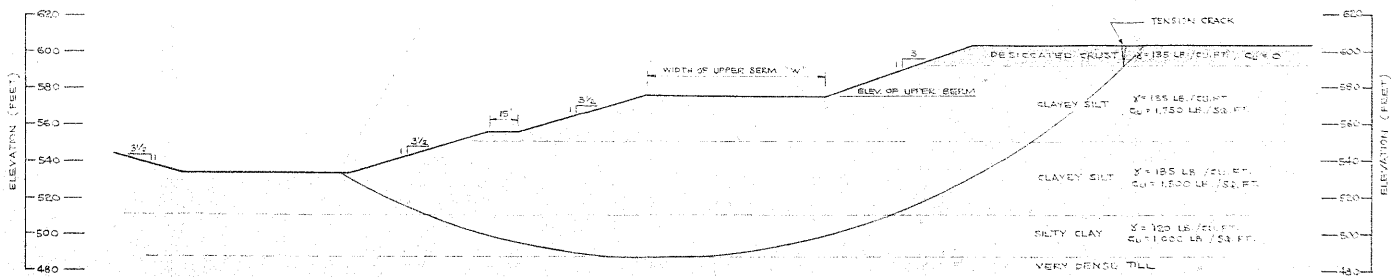


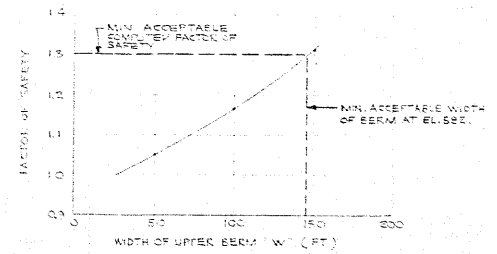
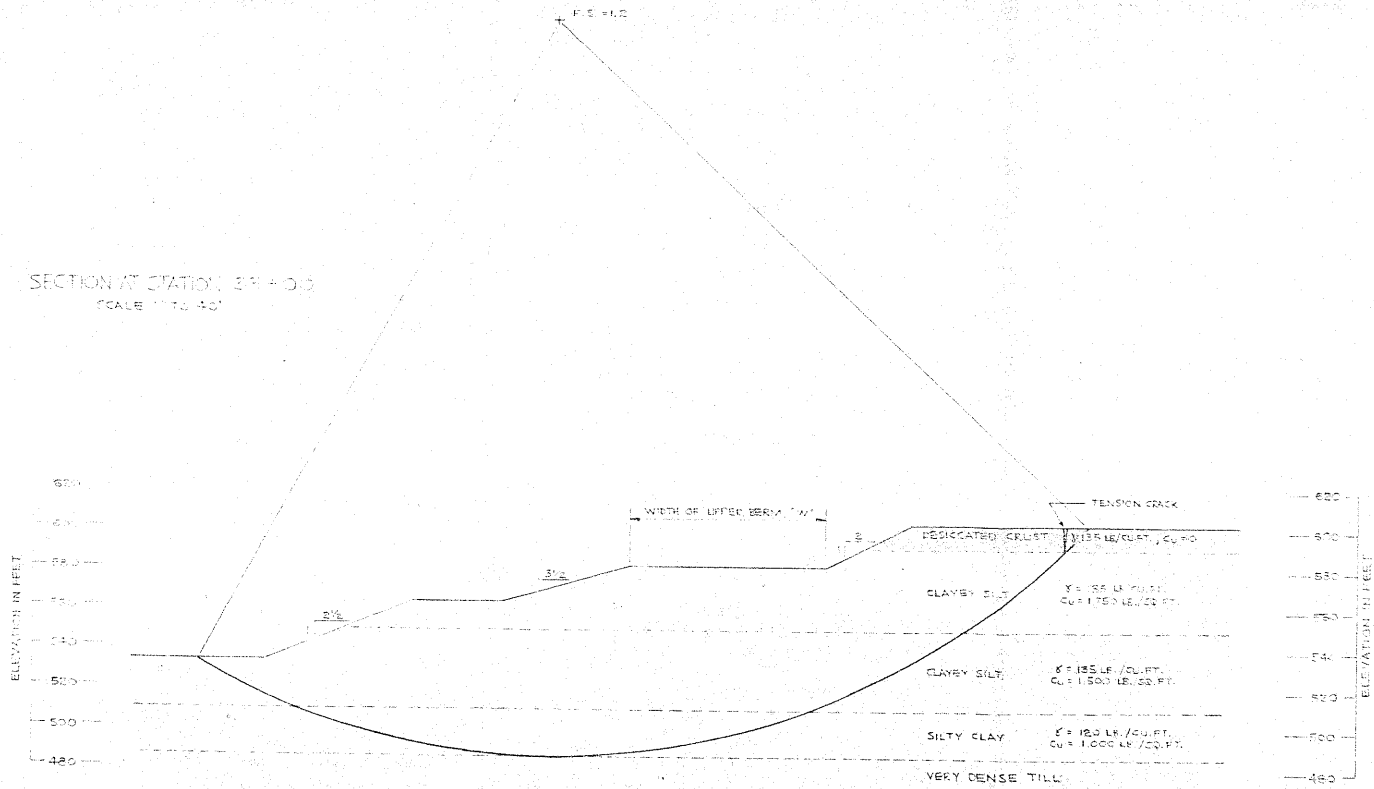
GOLDER & ASSOCIATES

Made *[Signature]*  
 Chkd. *[Signature]*  
 Appd. *[Signature]*

PROJECT No. 57106

SECTION AT STATION 33+00  
 SCALE 1" TO 40'



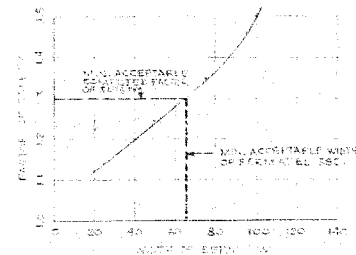
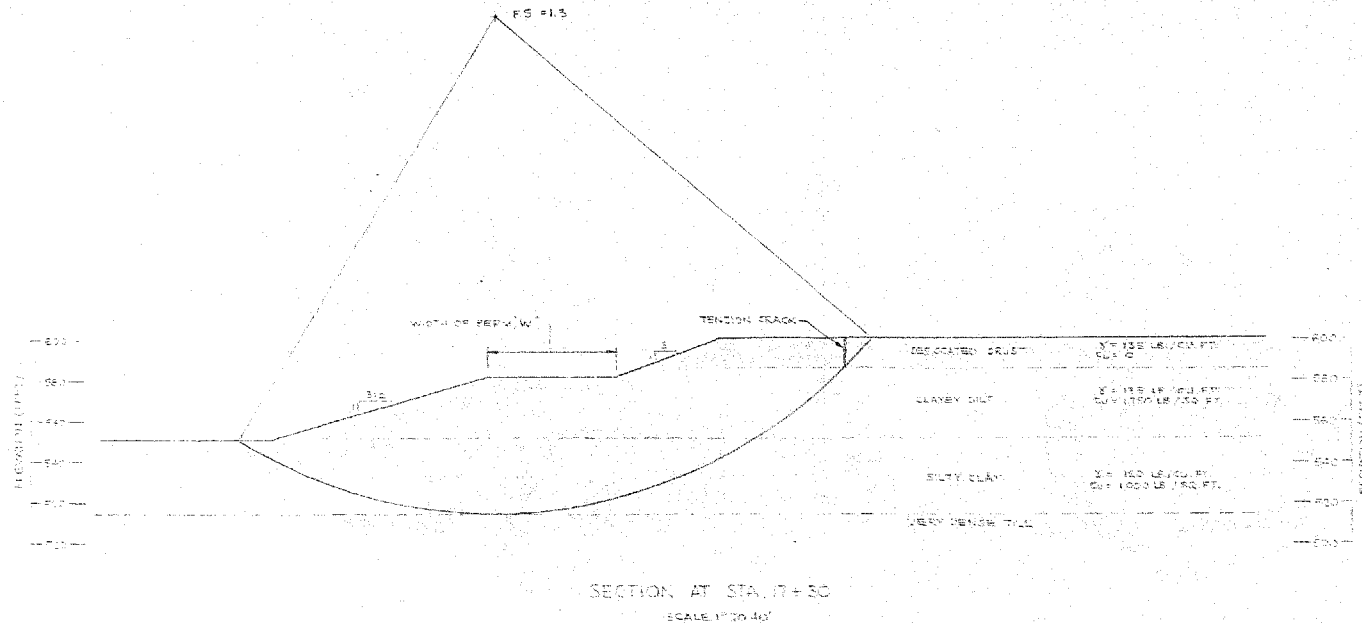


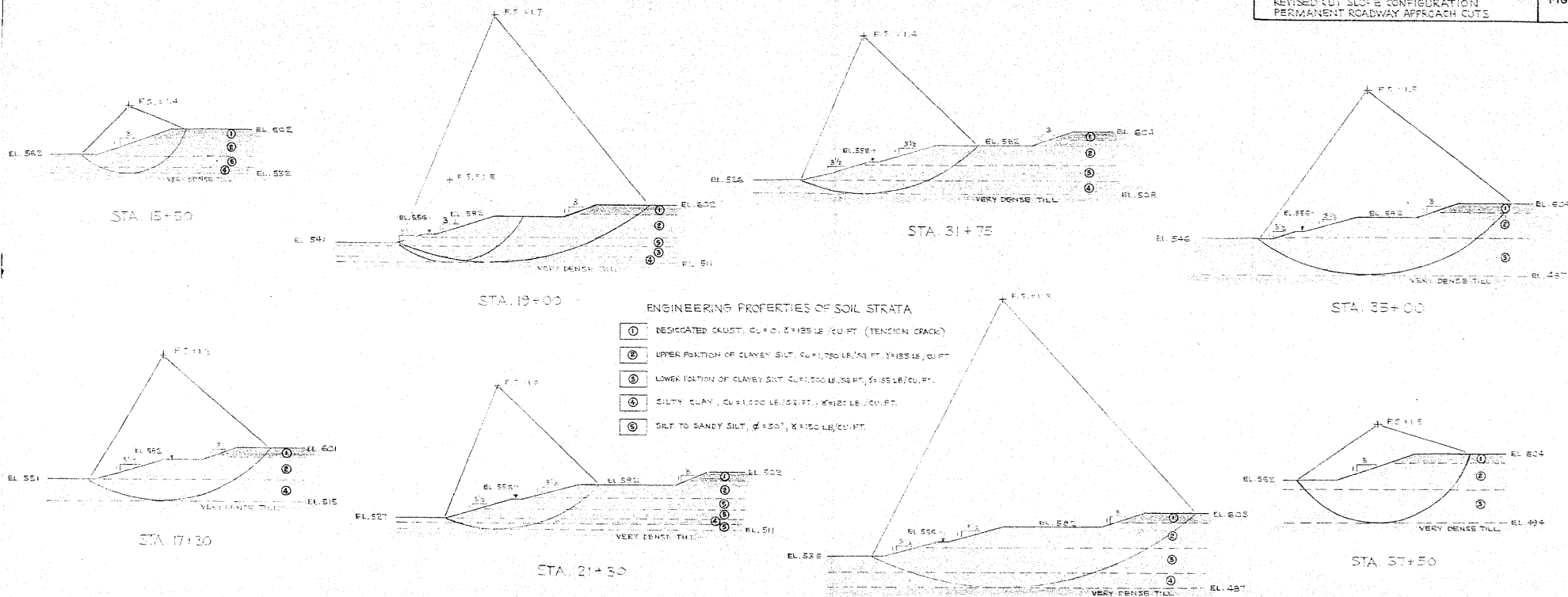
Drawn, JULY 5, 1966.

GOLDER & ASSOCIATES

Made *[Signature]*  
 Chkd. *[Signature]*  
 Appd. *[Signature]*







SCALE 1" TO 100'

Drawn: JULY 5, 1968.

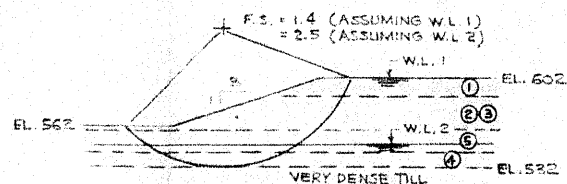
GOLDER & ASSOCIATES

Made by  
Chkd. by  
Appd. by

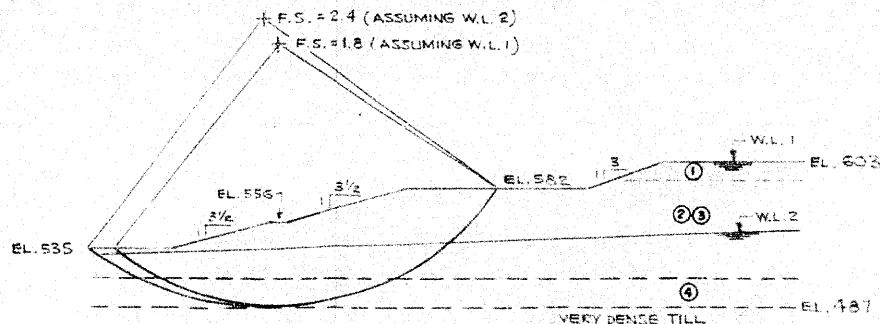
SCALE 1" TO 100'

SUMMARY OF EFFECTIVE STRESS STABILITY ANALYSES  
REVISED CUT SLOPE CONFIGURATION  
PERMANENT ROADWAY APPROACH CUTS

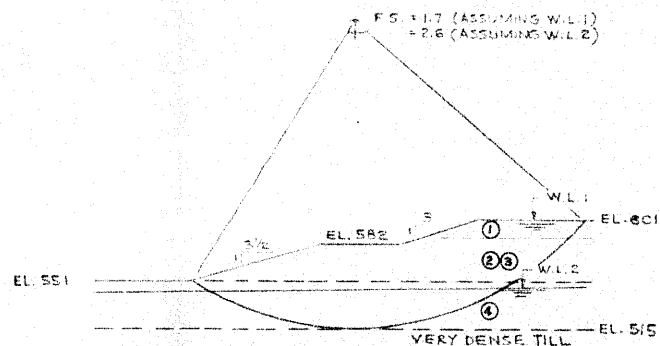
FIGURE 3-20



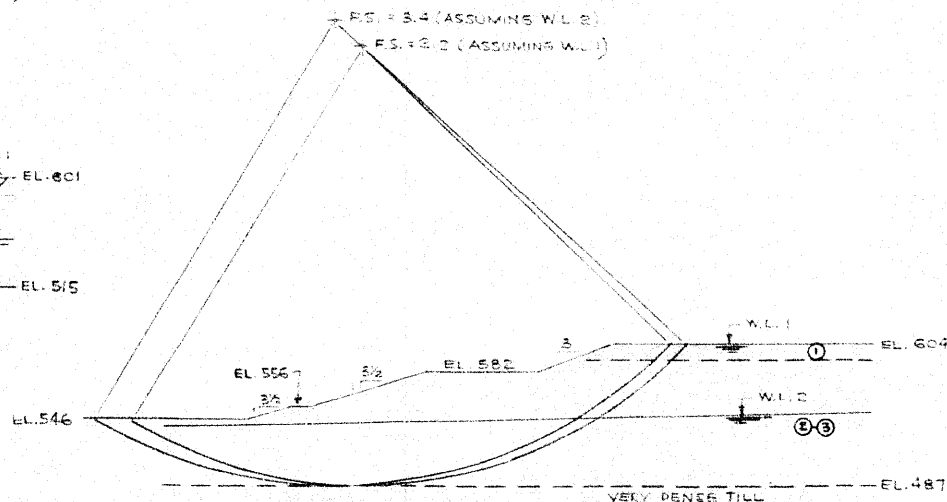
STA. 15+50



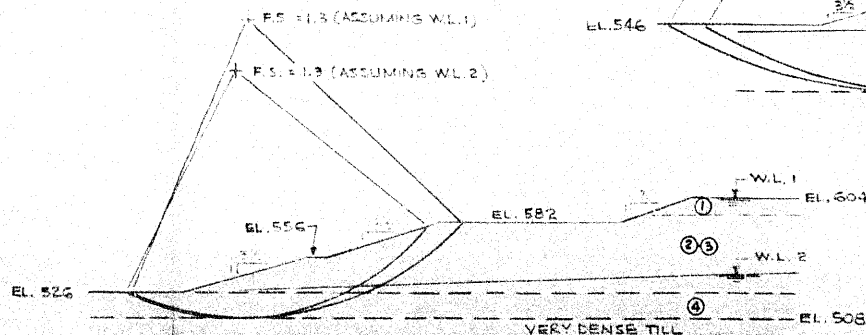
STA. 33+00



STA. 17+30



STA. 35+00



STA. 31+75

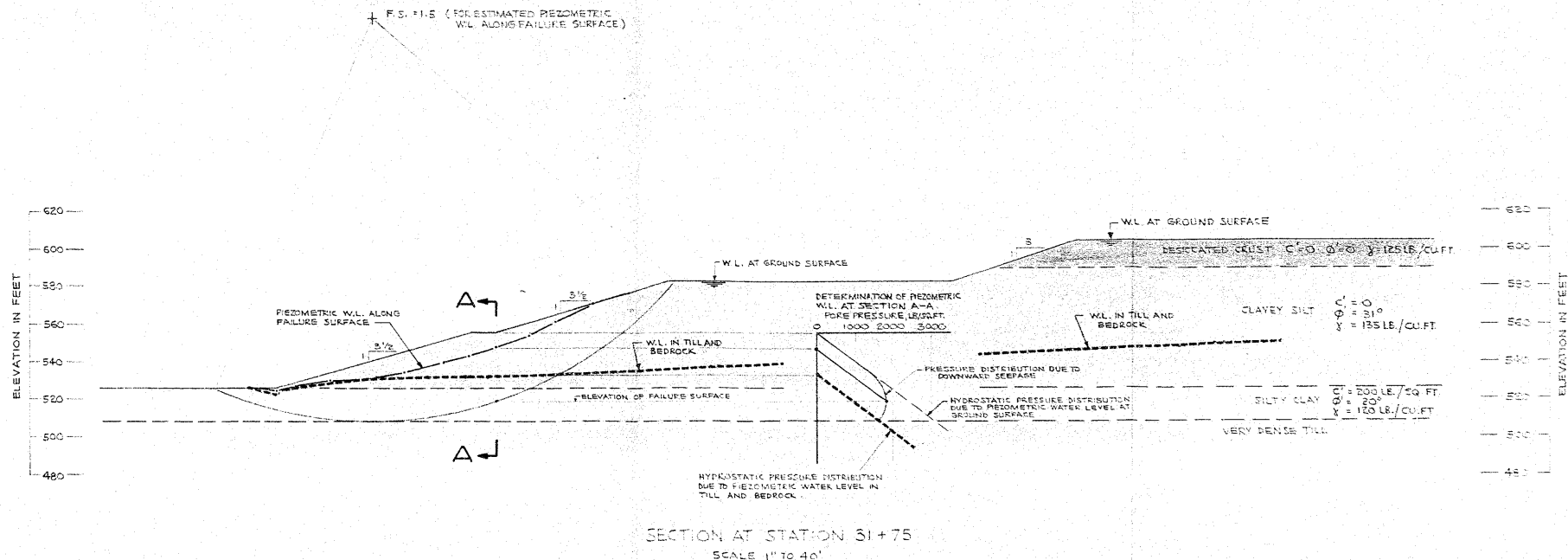
ENGINEERING PROPERTIES OF SOIL STRATA

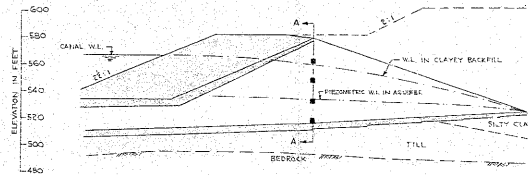
- ① DESICCATED CRUST,  $c' = 0$ ,  $\phi' = 0$ ,  $\gamma = 135 \text{ LB./CU. FT.}$  (TENSION CRACK)
- ②-③ CLAYEY SILT,  $c' = 0$ ,  $\phi' = 31^\circ$ ,  $\gamma = 135 \text{ LB./CU. FT.}$
- ④ SILTY CLAY,  $c' = 200 \text{ LB./SQ. FT.}$ ,  $\phi' = 24^\circ$ ,  $\gamma = 120 \text{ LB./CU. FT.}$
- ⑤ SILT TO SANDY SILT,  $c' = 0$ ,  $\phi' = 30^\circ$ ,  $\gamma = 130 \text{ LB./CU. FT.}$

Drawn: JULY 5, 1968.

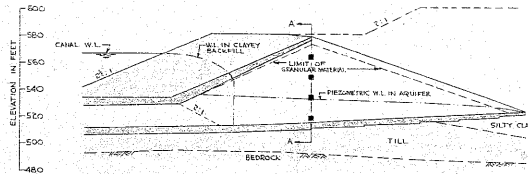
GOLDER & ASSOCIATES

Made by  
Chd. 7-68  
App. 1-68

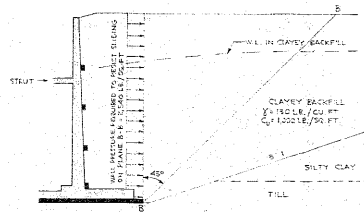




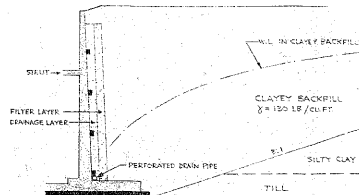
PIEZOMETRIC WATER LEVEL BEHIND WALLS  
(CLAYEY BACKFILL ADJACENT TO WALLS)  
SCALE: 1" TO 40'



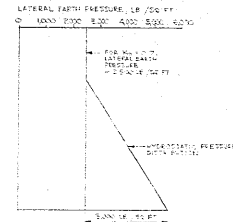
PIEZOMETRIC WATER LEVEL BEHIND WALLS  
(GRANULAR DRAINAGE LAYER PROVIDED)  
SCALE: 1" TO 40'



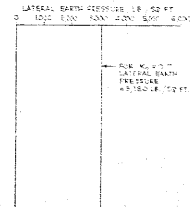
SECTION A-A  
(CLAYEY BACKFILL ADJACENT TO WALLS)  
SCALE: 1" TO 20'



SECTION A-A  
(GRANULAR DRAINAGE LAYER PROVIDED)  
SCALE: 1" TO 20'

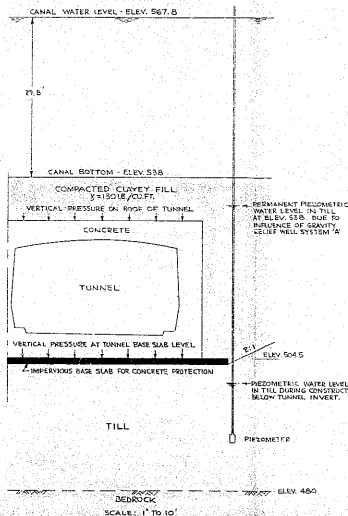


LATERAL EARTH PRESSURE ON WALLS  
(CLAYEY BACKFILL ADJACENT TO WALLS)  
SCALE: 1" TO 20'



LATERAL EARTH PRESSURE ON WALLS  
(GRANULAR DRAINAGE LAYER PROVIDED)  
SCALE: 1" TO 20'

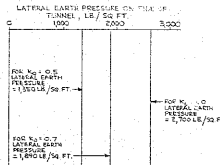
LEGEND  
■ EARTH PRESSURE CELLS



### EARTH PRESSURES DURING CONSTRUCTION

(TEMPORARY DRAWDOWN CONDITION  
CANAL NOT FLOODED)

#### VERTICAL PRESSURE ON TUNNEL ROOF = 1040 LB./SQ. FT.



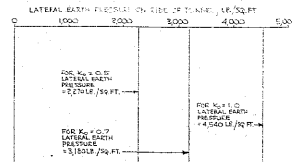
#### VERTICAL PRESSURE AT TUNNEL BASE SLAB LEVEL = 2850 LB./SQ. FT.

NOTE: LATERAL EARTH PRESSURE =  $K_0 \gamma H$   
 $K_0$  = EARTH PRESSURE COEFFICIENT  
 (AT REST CONDITION)  
 $\gamma$  = TOTAL UNIT WEIGHT OF FILL  
 = 130 LB./CU. FT.  
 $H$  = DEPTH BELOW CANAL BOTTOM

### EARTH PRESSURES AT END OF CONSTRUCTION

(PERMANENT DRAWDOWN CONDITION CANAL  
FLOODED... NO SEEPAGE OF WATER THROUGH  
CLAYEY BACKFILL.)

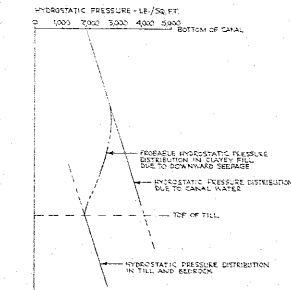
#### VERTICAL PRESSURE ON TUNNEL ROOF = 2090 LB./SQ. FT.



#### VERTICAL PRESSURE AT TUNNEL BASE SLAB LEVEL = 2910 LB./SQ. FT.

NOTE: LATERAL EARTH PRESSURE =  $K_0 (\gamma H + q)$   
 $K_0$  = EARTH PRESSURE COEFFICIENT  
 (AT REST CONDITION)  
 $\gamma$  = TOTAL UNIT WEIGHT OF FILL  
 = 130 LB./CU. FT.  
 $H$  = DEPTH BELOW CANAL BOTTOM  
 $q$  = SURCHARGE LOAD DUE TO CANAL WATER  
 =  $\delta_w H_w$

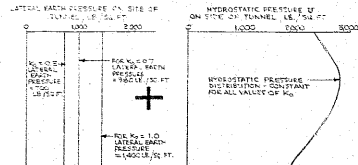
### HYDROSTATIC PRESSURE DISTRIBUTION BEHIND TUNNEL WALLS - LONG TERM CONDITION



### EARTH PRESSURES LONG TERM CONDITION

(PERMANENT DRAWDOWN CONDITION  
CANAL FLOODED... STABILIZED HYDROSTATIC  
PRESSURE DISTRIBUTION IN CLAYEY FILL)

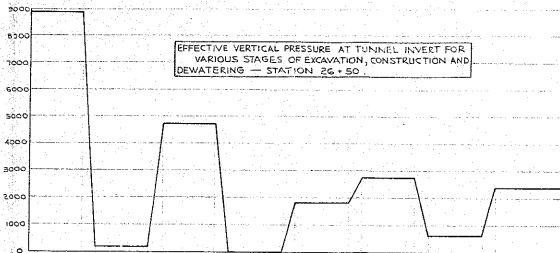
#### VERTICAL PRESSURE ON TUNNEL ROOF = 2280 LB./SQ. FT.



#### VERTICAL PRESSURE AT TUNNEL BASE SLAB LEVEL = 2910 LB./SQ. FT.

NOTE: LATERAL EARTH PRESSURE =  $K_0 \gamma H$   
 TOTAL LATERAL PRESSURE =  $K_0 \gamma H + U$   
 $K_0$  = EARTH PRESSURE COEFFICIENT  
 (AT REST CONDITION)  
 $\gamma$  = SUBMERGED UNIT WEIGHT OF FILL  
 $H$  = DEPTH BELOW CANAL BOTTOM  
 $U$  = HYDROSTATIC PRESSURE

EFFECTIVE PRESSURE UNDER TUNNEL STRUCTURE  
(LB./SQ. FT.) COMPRESSION



DEFORMATION UNDER TUNNEL STRUCTURE (INCHES)  
SETTLEMENT REBOUND

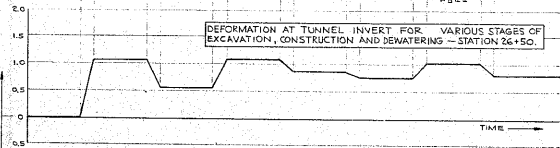
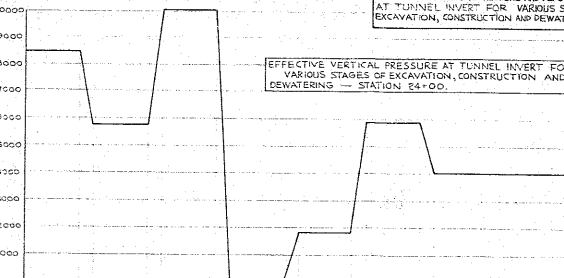


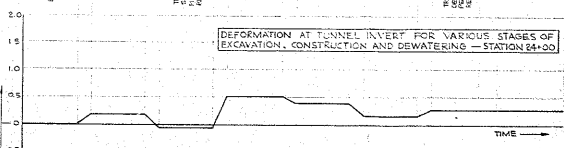
FIGURE 3-24

EFFECTIVE VERTICAL PRESSURE AND DEFORMATION AT TUNNEL INVERT FOR VARIOUS STAGES OF EXCAVATION, CONSTRUCTION AND DEWATERING.

EFFECTIVE PRESSURE UNDER TUNNEL STRUCTURE  
(LB./SQ. FT.) COMPRESSION



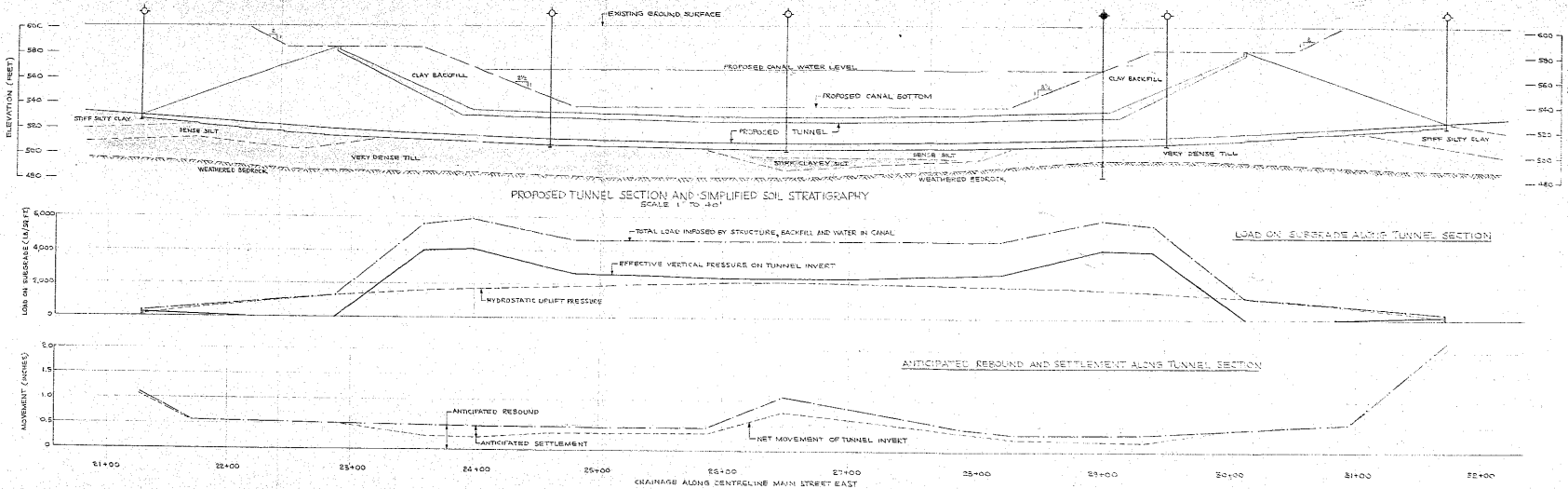
DEFORMATION UNDER TUNNEL STRUCTURE (INCHES)  
SETTLEMENT REBOUND



Drawn: JUNE 6, 1968.

GOLDER & ASSOCIATES

Made  
Chkd.  
Appd.



LEGEND

- PROPOSED OVERBURDEN REBOUND GAUGE
- ◆ PROPOSED BEDROCK REBOUND GAUGE

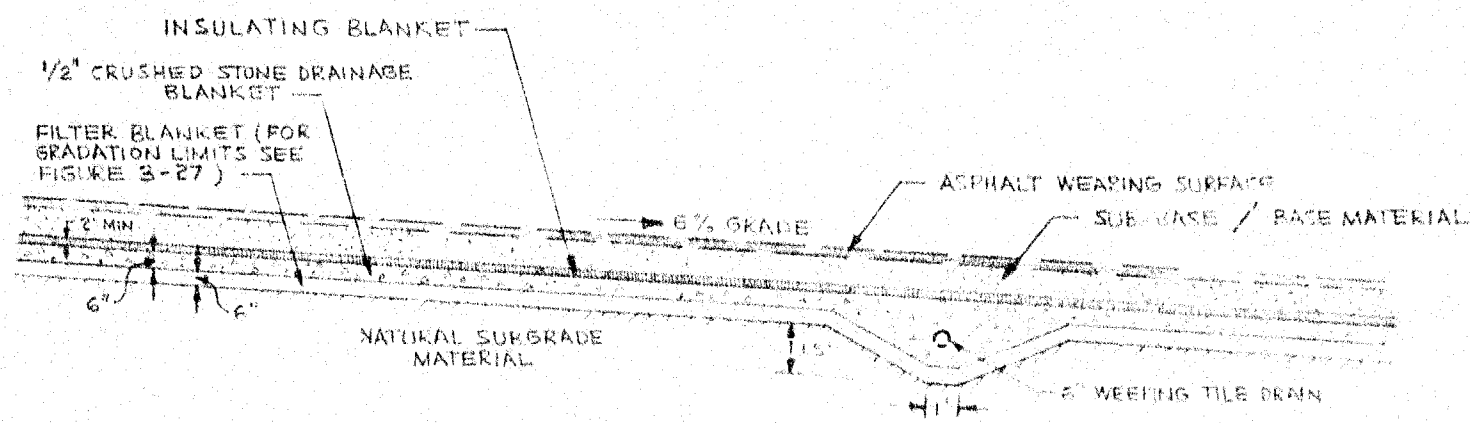
ASSUMPTIONS USED IN COMPUTATIONS

1. TOTAL UNIT WEIGHTS: WATER - 62.4 LB./CU. FT.  
SOIL - 135 LB./CU. FT.  
CONCRETE - 150 LB./CU. FT.
2. AVERAGE ORIGINAL GROUND SURFACE, EL. 602
3. STABILIZED PIEZOMETRIC WATER LEVEL IN BEDROCK, TILL AND LOWER SILT, EL. 576
4. CANAL BOTTOM, EL. 558
5. CANAL SEEMS, EL. 552
6. FINAL DEPTH OF WATER IN CANAL, 23 FEET
7. PIEZOMETRIC WATER LEVEL TEMPORARILY DRAWDOWN TO BELOW EXCAVATION INVERT (SEEPAGE CALCULATION)
8. PERMANENT GRAVITY RELIEF WELL SYSTEM 'A' INSTALLED AND OPERATING AS DESIGNED (SETTLEMENT CALCULATION)
9. TUNNEL AND RETAINING WALL STRUCTURES ARE AS SHOWN ON D.H.C. DRAWG. NO. D-8873-PI, WELLSLAND CANAL DIVERSION TUNNEL - EAST MAIN, GENERAL LAYOUT - PRELIMINARY, DATED, JAN. 1968.



LONGITUDINAL SECTION SHOWING SUGGESTED SUBGRADE  
TREATMENT BENEATH APPROACH ROADWAY PAVEMENT

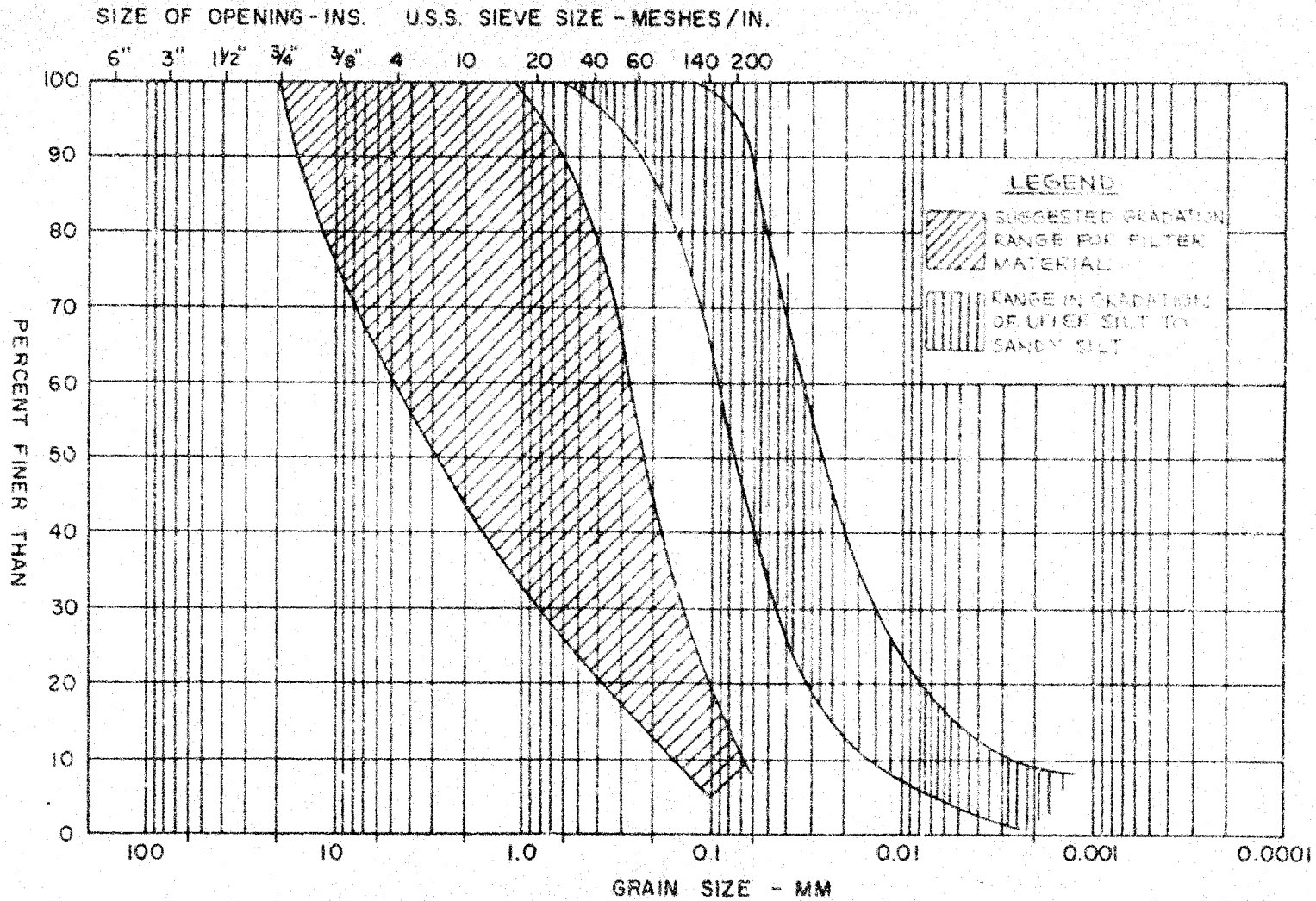
FIGURE 3-26



GOLDER & ASSOCIATES

Modr  
Chkd  
Appd

M.I.T. GRAIN SIZE SCALE



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			

GOLDER & ASSOCIATES

Mode  
Chkd  
Appd.

SUGGESTED GRADATION LIMITS OF FILTER MATERIAL  
BENEATH APPROX. COLUMN ELEMENT

FIGURE 3-27