February 6, 1967.

Memorandum to: Messrs. J. B. Wilkes,
J. Walter,
W. Bidell,
H. Greenland,
F. I. Newson,

Re: Feasibility Study Locating Hwy. 406 in the existing Welland Canal

The report prepared by H. Q. Golder & Associates dealing with this study, is attached. Mr. Milligan presented the data contained in this report, to the Department on September 22, 1966. No changes have been made to Mr. Milligan's presentation. The report, however, does confirm the discussion and the recommendations made at that time.

The report concludes basically that placing Hwy. 406 in the canal bed is entirely feasible, and that special attention must be given to ensure against slope failures at specific locations. The protection necessary in stabilizing the banks to ensure against failures, will generally involve the loading of the toe of the slope with fill prior to de-watering. The locations where this type of treatment may be required are outlined in the report.

If the final decision is made to use the existing Welland Canal for Hwy. 406, further investigation will be required and undertaken for detailed design purposes.

aR

A. Rutka,
Materials & Testing Engineer.

AR:pa
Encl.
c.c. Messrs. G. Hunter,
A. Stermac,
T. Kovich,
G. Wrong.

Note to Tony: I think it might be to our advantage to obtain some information on the canal bottom this winter if we can arrange it. Perhaps at your convenience you can discuss this with me with respect to finding out how it should be paid for in view of our limited budget.

67F212

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H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

FEASIBILITY STUDY

PORTION OF PROPOSED HIGHWAY 406 WITHIN WELLAND CANAL
STA. 800+00 TO STA. 1110+00

WELLAND ONTARIO

Distribution:

15 copies - Department of Highways, Ontario,
Toronto, Ontario

3 copies - H. Q. Golder & Associates Ltd.,
Toronto, Ontario.

January, 1967

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CONTENTS

ABSTRACT

A study to determine the feasibility of locating a portion of proposed Highway 406 in a section of the Welland Canal in the area of the City of Welland and which will not be used for shipping following completion of a proposed canal re-alignment is reported. The section involved is between stations 800+00 and 1100+00 (Canal chainage).

This report covers, in general terms, the subsurface conditions and engineering properties of the overburden in this area and discusses previous canal bank failures and the possibility of further slope failures in relation to the proposed highway. The report is based on studies of available reports on the soil conditions and on extant drawings of existing structures.

In general, from the consideration of problems of the stability of canal banks and foundations of structures, the construction of a section of Highway 406 within the drained excavation of the Welland Canal is feasible provided that protective measures, such as partial in-filling prior to drawdown of the canal, are carried out in areas of potential major instability. Such areas have been defined and are discussed in the report.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario (letter of authorization dated July 8, 1966) to examine the general feasibility of using the drained excavation of a section of the existing Welland Canal between Ramey's Bend and Bridge 12, now to be abandoned, as a route for Highway 406. The feasibility of the proposed highway is largely dependent on what bank stability problems there would be in dewatering the existing canal in this area.

This report covers, in general terms, the subsurface conditions and engineering properties of the overburden in this area and discusses previous canal bank failures and the possibility of further slope failures in relation to the proposed highway. The report is based on studies of available reports on the soil conditions and on extant drawings of existing structures.

Site visits have been made and surveys conducted of previous slope failures of the canal banks. Data supplied to us concerning the soil conditions and such failures, and consisting mainly of reports and drawings by the St. Lawrence Seaway Authority are referred to throughout the report and listed under "References". No borings or laboratory investigations or testing have been carried out for this study.

PROPOSED HIGHWAY AND NECESSITY FOR STUDY

It has been tentatively proposed (notes on meeting, D.H.O. Sept. 22, 1966) that, if feasible, Highway 406 within the canal would have four lanes, which together with a median strip would require a right-of-way of approximately 120 ft. width. The grade of the highway would be depressed below the surrounding terrain at almost elevation 560, thus facilitating the construction of exit and entrance ramps to the highway. At this grade the roadway would be some 20 ft. above the existing canal bottom.

It is known that, in the past, a number of failures of the existing canal banks have taken place. In some areas the banks have been trimmed to improve stability, but in many areas no protective measures have been taken and the slides are still in evidence. It can be expected that further bank failures will take place on dewatering of the canal - the effect of the dewatering constituting an unloading at the toe of the slopes. Since a significant part of the proposed route passes through the City of Welland, such bank failures would endanger existing bridge structures and property adjacent to the canal. It is therefore essential that the causes of the previous failures be investigated and examined in relation to the construction of the proposed highway and further, that areas of potential instability be defined.

SITE AND GEOLOGY

The portion of the Welland Canal considered in this study is between stations 800+00 and 1110+00; this portion of the canal is located within the Townships of Crowland and Thorold, County of Welland, and is outlined on Figure 1. Figures 14 to 27 inclusive show photographs of typical structures and the canal banks within the area studied.

The City of Welland is located approximately in the centre of the section. The canal is cut in a broad, relatively flat clay plain, which varies in surface elevation from about elevation 580 to 600. The bottom of the canal along this reach is at about elevation 540 i.e. the canal banks are approximately 40 to 60 ft. high. The width of the canal at the crest and at canal bottom is about 350 ft. and 200 ft., respectively. The water level in the canal is at about elevation 569 (some 29 to 30 ft. of water). The Welland River, which flows from west to east adjacent to the canal, is at a normal water level of about elevation 562 with the high water level being about elevation 569.

From available geological information (Chapman, Putnam, 1951) and inspection of the area, it is known that the overburden consists of thick deposits of silty clay overlying till, physiographically known as Haldimand Till. The till deposits which are of Wisconsin age, were buried by the silty clay, which is a lacustrine deposit laid down in glacial Lake Warren. The glacial lake phase was possibly interrupted by two major retreats of the ice front which results in distinctively different deposits - non-stratified relatively silty, homogeneous deposits laid down with the ice front fairly close, and heavily stratified very clayey deposits laid down when the ice front had retreated some distance. All of the deposits except the upper 30 to 40 ft. which are desiccated, are relatively soft and possibly only lightly over-consolidated. The total thickness of the overburden generally varies from 100 to 120 ft. in thickness.

The broad clay plain is bounded to the north by the Niagara Escarpment which steps down towards Lake Ontario. To the south the plain is bounded by the toe of the Onondaga cuesta.

The water shed in the area is provided by the Onondaga cuesta, which though quite low and lying close to the shore of Lake Erie, nevertheless forces the drainage to the north and east. In general the drainage in this primarily flat heavy clay area is quite poor.

The bedrock in the area is Palaeozoic, the beds of which dip slightly southward under Lake Erie. The bedrock, which generally varies in elevation from 460 to 500, is a massive dolomitic limestone interbedded with siltstone and calcureous shale of the

Salina formation, Devonian Period. This formation contains numerous gypsum inclusions from hairline thickness to as much as 12 inches.

SOIL CONDITIONS

Borings put down during previous investigations by others have provided information on the subsoil conditions along the canal (Refer to Tustin - 1966 a, b). The location of these boreholes and the inferred soil stratigraphy, as well as the engineering properties of the overburden are shown on Figure 2. Based on the stratigraphy and the reported engineering properties of the overburden, the soil conditions, between stations 800+00 and 1110+00, can be grouped into three main sections. A brief summary of the subsoil conditions within each section is presented below, with particular attention given to the upper cohesive strata of the overburden.

Section I (Sta. 800+00 to 880+00)

The crest of the canal bank in this section is at about elevation 582.

The surficial cover is a very stiff to stiff brown layered clayey silt and silty clay with a trace of sand and gravel. The thickness of this deposit, where encountered, ranges from 14 ft. to 20 ft. At borehole W-384 the surficial cover is a grey-brown sandy silt fill with some inclusions of organic matter.

Directly underlying the surficial deposits in this section is the predominant overburden stratum, a stiff grey-brown clayey silt till with occasional lenses of silty clay and which changes to a firm consistency with depth. The thickness of this stratum varies from 30 to 70 ft. extending down to as low as elevation 512. Grading analyses carried out on the silty till indicate that the clay content varies from about 20 to 40 per cent, with a silt content of up to about 65 per cent.

The engineering properties of the overburden along this section are summarized on Figure 8. The average liquid limit and plastic limits are 26 and 11 respectively, with an average water content of 19 per cent and indicate that the till is inorganic and of low to medium plasticity. The average unconfined compressive strength of the stratum, between elevations 520 and 560 (the critical zone based on slope stability considerations) is about 1.0 ton/sq.ft.

The clayey silt till is underlain by a dense to very dense sandy silt till which in turn is underlain by dolomitic limestone bedrock.

Section II (Sta. 890+00 to 960+00)

Underlying a thin surficial mantle of topsoil the predominant overburden stratum in this section is a grey-brown clayey silt till. The thickness of the silty till ranges from about 28 to 40 ft., extending down to as low as elevation 550. The upper mottled brown zone (some 10 to 14 ft. in thickness) has been desiccated into a hard crust and as such is highly fissured. Grading analyses carried out on samples from the deposit indicate that the composition of the till is similar to that of Section I.

The engineering properties of the overburden are summarized on Figure 9. The Atterberg limit tests carried out on the clayey silt till indicate that the till is inorganic and of low to medium plasticity with the corresponding liquidity indices varying from 0.3 to 0.6. The average measured unconfined compressive strength below the crust is about 0.8 tons/sq.ft.

Underlying the silty till, at a few of the boring locations, is a deposit of firm brown to grey stratified or varved silty clay with a trace of sand and gravel. The maximum thickness of this deposit is about 20 ft.

The varved silty clay is in turn underlain by a dense to very dense sandy silt till deposit. Bedrock was not proven at any of the borings put down in this section.

Section III (Sta. 980+00 to 1110+00)

The predominant overburden strata in this section are a red-brown to grey interbedded silty clay and/or stratified clay. The overall thickness of the strata varies from 60 ft. to 85 ft. The upper red-brown zone has been desiccated to a hard crust and as such is highly fissured. The thickness of individual layers within the stratified clay varies widely to a maximum of 3 inches approximately. Occasional partings and seams of silt and sand are also present within the stratified clay. Grading analyses carried out indicate that the clay content of the upper clay strata is higher in this section than in Sections I and II.

The engineering properties of the overburden are summarized on Figure 10. This figure shows that, in general, the clay strata are of high plasticity and the measured unconfined compressive strength below the upper crust and above elevation 520, while varying widely, has a mean value of about 0.6 ton/sq.ft., the lowest strength being in stratified clay at about 45 to 50 ft. depth.

Underlying the interbedded silty clay and stratified clay is a compact to very dense reddish-brown to grey silt and sand deposit. Directly underlying the silt and sand is a deposit of very dense grey silty sand till. This silty sand till deposit is continuous along the portion of the canal investigated. Dolomitic limestone bedrock of the Salina formation underlies the granular till; the surface elevation of the bedrock within this section ranges from about elevation 462 to 494.

GROUNDWATER CONDITIONS

The piezometric groundwater conditions were determined during the previous investigations by installing piezometers in the overburden (mainly along the west bank). The pertinent installations are shown on Figure 2; also included on this figure are the results of readings taken in these piezometers.

The readings taken in the piezometers indicate that the piezometric groundwater level within the bank of the canal is generally between 5 and 10 ft. above normal canal level (which is approximately at about elevation 569). The piezometers further indicate that, in general, the piezometric groundwater level increases in elevation to the west. Further, the previous investigations suggest that the groundwater seepage is from west to east (Peckover, Tanner, 1959) causing periodic wet spots within the west bank of the canal.

PREVIOUS CANAL BANK FAILURES

The location and lateral extent of known and observed canal bank failures are shown in plan on Figure 2. From visual observation of the apparent geometry of these failures, the slides have been divided into two categories, namely:

a) Shallow slope failures involving only some 10 ft. to 15 ft. depth of soil down the face of the slope; possibly caused by softening of the upper desiccated and fissured zone of the clay.

b) Deep rotational slides often causing movement at the base of the canal and affecting the whole mass of soil within the canal bank.

It has also been observed that:

i) The majority of the failures have taken place on the west or higher bank of the canal. Further, where the bank is high, the failures are of considerable lateral extent.

ii) Relatively few slope failures have taken place between sta. 800+00 and 880+00. The greatest number of failures have occurred between sta. 900+00 and 1050+00, particularly within Section III where the reported unconfined compressive strength is lowest (Fig. 10). Those failures which have taken place in Section II have occurred where the canal banks are relatively high (Refer to Fig. 9).

The failures have been plotted in terms of slope angle, (β), versus the height of slope, (H), on Figure 11. From this data an approximate limiting boundary condition can be assumed for various combinations of slope height and slope angle under the existing water level conditions. For example, for a slope angle, $\beta = 25°$, a height, H = 50 ft. would be unstable, whereas if the slope height is restricted to 40 ft. the bank would be stable. However, the important variable not considered in this plot is the influence of the overburden shear strength on the stability of the slopes. This point is examined further below.

<u>Stability in terms of reported undrained strength</u>

The slope failures have taken place in banks of clay or clay till. There is considerable evidence to show that the use of the undrained shear strength in the total stress, or $\emptyset = 0$ method of analysis, cannot be justified in analyzing the long term stability of slopes in fissured clays or in over-consolidated clays or clay tills; the error in estimating the factor of safety by this method can be extremely large. However, for slopes in normally consolidated clays, the errors in estimating the long term stability using the undrained shear strength and $\emptyset = 0$ analysis are not large, and generally on the safe side.

In the area of study (and particularly in Section III), below a depth of 30 ft. to 40 ft., the clay deposits are either normally consolidated or only lightly over-consolidated. Except for the upper 30 ft. which is fissured, the clay is intact. Consequently, as far as rotational slides are concerned (Type b) a preliminary evaluation of the stability of the canal banks can be made using the undrained shear strength of the clay and since, within the critical zone of sliding (40 ft. depth or greater), the clay is intact the undrained strength can be determined from unconfined compression tests. (In this case, the undrained shear strength, S_u, is taken as one-half the compressive strength.)

The stability of a slope using the total stress analysis can be expressed in terms of a dimensionless number, (N_s). (Taylor, 1948) defined as:

$$N_s = \frac{\gamma H}{S_u} \cdot F$$

where, γ = total unit weight of the soil (lb/cu.ft.)

H = height of the slope (ft.)

S_u = undrained shear strength (lb/sq.ft.)

F = factor of safety.

Computations carried out using the total stress analysis (Taylor, 1948) are summarized on the overlay to Figure 11. On this plot, the slope height was adjusted to consider submergence. It can be appreciated that a partially submerged slope of height, (H), is equivalent in terms of stability to a non-submerged slope of lesser height, the soil weight and shear strength being identical. Comparative stability analyses indicate that for 30 ft. of submergence (taken as the average depth of water in the canal) a canal bank slope of height, (H), with F = 1, is approximately equal to a non-submerged slope of height (H - 11 ft.). Consequently, this adjusted height has been used in the preparation of the overlay to Figure 11. Slope heights, with F = 1.0, have been computed for two values of shear strength, namely 500 lb/sq.ft. and 700 lb/sq.ft. It has further been assumed that the depth factor (the ratio of the slope height to the depth of a hard layer on which sliding would take place), n_d, is equal to 1.5*. It is apparent from the figure that the majority of the slope failures take place at an undrained shear strength between

*This figure is reasonable for an average canal bank height of 46 ft. Refer to stratigraphic details on Figures 8, 9 and 10.

500 lb/sq.ft. to 700 lb/sq.ft. These values agree quite well with the average reported measured undrained shear strength in Section III (Fig. 10), but are lower than the average in Sections I and II. (Fig's 8, 9).

In Figure 12, the canal bank failures are plotted in terms of the dimensionless parameter, (N_S), for $F = 1.0$, and the reported shear strength for the section in which the failure took place. It is apparent that for some of the slope failures, the in situ shear strength at failure must be less than that measured in unconfined compression tests. The reason for this may be due to the anisotropic nature of the stratified clay stratum, which, from previous studies (Report 6375, July, 1964), is known to have directional shear strength properties. For this material, the shear strength mobilized at failure is dependent on the orientation of preferred planes of apparent weakness. The average undrained shear strength along such "weak" layers was measured to be 500 lb/sq.ft.

Consequently, where the stratified clay is present, the banks can be assumed to be potentially unstable. From the stratigraphic section (Fig. 2), the area of stratified clay is generally within Section III and particularly between sta. 960+00 and 1050+00. North of this area, where the reported unconfined compressive strength is high (Section I), and south of this area where the banks are either flatter or have been partially unloaded by excavating material at the top of the banks, the slopes are relatively stable. The effects of dewatering the canal in these stable areas is briefly discussed below and more detailed examination given to the area of potential instability.

EFFECT OF DRAWDOWN OF THE CANAL ON EXISTING SLOPES

Stability in terms of undrained strength

The result of dewatering the canal is to remove a weight of water acting at the toe of the slopes. The total activating force causing sliding is therefore much greater than in the partially submerged case. The increase in activating force causes no increase in shearing strength however, if the rate of dewatering is sufficiently rapid that no water will escape from the pores of the soil. Consequently, the "undrained" shear strength of the soil can then be used in examining stability on drawdown of the canal.

In an examination of stability, the effect of dewatering can be considered as increasing the bank height, H, by the removal of 30 ft. of water. (This is comparable to rapidly increasing the height of a non-submerged slope by an equivalent height of soil of at least 11 ft.). From Figure 11, canal banks of average height $H = 45$ ft. and sloped at flatter than 20°, could be expected to suffer little instability on drawdown.

Where the bank material is strong with increasing depth and weathering has taken place on the upper or exposed surfaces of the clayey overburden, drawdown slope failures are generally relatively shallow since the strength is lowest in the zone of weathering or softening close to the surface. The majority of existing shallow slope failures observed (Type a) have taken place where the average slope angle was in excess of 25°. (The majority of cut or natural slopes along the canal are at an angle of from 25° to 35°.) Thus, shallow slope failures can be expected along the existing banks on drawdown. The significant point however is that, where there are no structures close to the edge or toe of the banks, (e.g. Section I), such failures are of minor consequence. Where they occur in such areas they can be treated following dewatering.

Where the bank material is significantly weaker with depth as in Section III the effect of drawdown (in effect increasing the height of the bank) is markedly more severe since deep failure surfaces are also critical (comparable to Type b). From the data presented in Fig's 11 and 12, the section of the canal where the weak stratified clay layer is present is obviously potentially unstable with respect to deep failures under the condition of rapid drawdown.

<u>Stability in terms of effective stress</u>

The use of the undrained shear strength, measured prior to drawdown, is only valid where the rate of dewatering is such that no appreciable drainage is able to take place in the vicinity of the incipient failure surface during the drawdown period. This can be illustrated in terms of effective stresses.

A summary of effective analyses of an existing typical section of the west bank in Section III (sta. 990+00) is given on Figure 13. The shear strength parameters assumed for the analyses were based on previous work in this area (Report 6375, July, 1964). The factor of safety, F, with respect to a deep failure through the lower "weak" layer is critical. F, prior to drawdown, is approximately 1.2*, making the assumptions concerning the shear parameters and piezometric levels given on the figure. On drawdown, if there is no dissipation of pore water pressures within the bank, the phreatic surface coinciding with the face of the slope, then F is reduced by over 33 per cent. If the rate of drawdown is slow, such that there is some drainage, the reduction in F is smaller. It is therefore important to examine what rate of dewatering of the canal can be considered slow enough to effect substantial drainage.

Effect of the rate of drawdown

Computations of stability for sta. 990+00 assuming the

*Considering the general assumptions made, the computed numerical value of F is not necessarily exact, but represents the general order of the stability of the slope.

lower phreatic surface shown in Figure 13 indicate that F, (after drawdown) is of the order of 0.9 to 1.0, deep failures through the "weak" clay layer being most critical. This value of F is about 20 per cent greater than F for no pore water dissipation within the slope.

The rate of dissipation of drawdown pore pressures can be estimated approximately by drawing a series of flow nets with a gradually lowering upper phreatic surface. Each flow net represents the condition existing in the bank at some definite time following drawdown of the canal water level. Using this procedure it is assumed that all the water drains out of the pores. A general procedure has been worked out by Reinius (1955) for earth dams, in which drawdown is related to a dimensionless number defined as

$$\frac{k}{n_s V}$$

where, k = coefficient of permeability of the soil, ft./day

V = rate of water level drawdown, ft./day

n_s = volume of water which can be drained out of a unit volume of soil, in per cent.

Using this approach and assuming dewatering complete in 2 years, or at a rate of 30/730 ft./day; for a clay slope ($k = 1 \times 10^{-7}$ cm./sec.) less than 20 per cent dissipation of pore water dissipation would have taken place in 2 years. However, this method applies to a dam with impounded water and is invalid for the natural banks at the site since water levels in the slope would be re-charged because of seepage from the west; the rate of pore pressure dissipation would therefore probably be slower than that computed above. While the effects of the layered character of the clay strata would increase the permeability in a horizontal direction, on the basis of our present knowledge of the permeability of the clay and the groundwater conditions, it is concluded that dewatering the canal even over a period of several years would not be of significant value in reducing the incidence of deep drawdown slope failures in areas where the "weak" layer is present (Section III).

AREAS NOT MARKEDLY AFFECTED BY DRAWDOWN OF THE CANAL

As discussed above, where the clay, although weathered at its surface, is relatively strong with depth (i.e. there are no "weak" layers present) any failures caused by drawdown of the canal would tend to be shallow. Further, if there are no structures or buildings adjacent to the canal the consequences of occasional deep failures during dewatering, should they occur, would not be of major consequence. Therefore, in the area north of the existing syphon structure (sta. 800+00 to 966+00), with the possible exception of that part of the east bank of the canal where it forms a dyke to the adjacent Welland River, no protective measures should be required prior to drawdown.

In the area between sta. 1050+00 and 1100+00 the soil conditions are relatively poor (Section III), but because of previous failures the height of the west bank has been reduced by excavation and the slopes flattened. (Fig. 14 - Photo 2). It is doubtful if major instability would result on dewatering at this bank. On the east bank no major structures or property is located close to the edge of the canal and it is likely that this general area need not be treated prior to drawdown.

The areas generally not requiring protective measures prior to drawdown are marked on Figure 1.

AREAS OF POTENTIAL MAJOR INSTABILITY

Within the downtown part of the City of Welland, the consequences of slope instability could be extremely serious since failures could cause the loss of valuable structures and property. It is also significant that within the same area the soil conditions with respect to slope stability are poor (Fig. 10) and the probability of deep failures on drawdown is high. This critical part of the proposed route is marked on Figure 1 and extends from about sta. 967+00 to 1050+00. Typical cross-sections of the canal are given on Figure 2 and foundation details of bridge structures and the syphon crossing of the Welland River, as determined from the available records, are shown on Figures 3 to 7 inclusive. Individual areas of potential major instability are discussed separately below.

Retaining walls and canal banks between sta. 967+00 and 1054+00

Retaining walls in this area are generally founded on timber piles of indeterminate length. Along the west bank of the canal runs a major transmission line of Ontario Hydro, generally within about 30 ft. of the top of the slope. A number of buildings (excluding the syphon approach works) are concentrated on the west bank of the canal and protective measures to prevent major slides will have to be taken; however, at certain locations the expense of protective measures may not be warranted:

At sta. 1001+50 on the east bank, the buildings of the United Steel Corporation Ltd. extend out to the crest of the slope. (Fig. 20 - Photo 14). However since these buildings are comparatively old and apparently unused, it is doubtful if the expense of protecting these structures is warranted. No

Between sta's 1007+98 (Bridge 14) and 1020+00 on the east bank there is a pile-supported retaining wall and loading dock which exhibits some signs of structural distress. (Fig. 18 - Photo 10, Fig. 19 - Photo 11). When shipping in the canal is terminated this loading dock will not be used and will probably have to be demolished. Consequently, protective measures here prior to dewatering could be restricted to demolition. YES

Another pile-supported loading dock is located in the east bank between sta. 1047+07 and 1054+00. The Union Carbide Ltd. buildings are located some 200 ft. back from the dock. (Fig. 14 - Photo 1). As discussed above, demolition of the dock will inevitably be required and other protective measures may not be justified, since the existing factory buildings are sufficiently far from the top of the slope to be unaffected by slope movements. YES

Along the west bank between sta. 1016+00 and 1033+00, the slope has been flattened and reduced in height to protect the existing transmission line and towers. The possibility of deep failures for a bank of H = 40 ft. and with an average slope of 23° to 24° seems small. (Refer to Fig. 11). Consequently, further protective measures here may not be justified.

Bridge structures and syphon

Inferred foundation details for bridge structures 13 to 16 inclusive and the syphon structure carrying the Welland River below the existing canal are given on Figures 3 to 7 inclusive.

The bridge structures are apparently founded on timber piles driven to practical refusal in the lower till or harder clay strata. The length of piles is not known. There is some evidence that the bridge abutments and piers have, at times, suffered movement. The most extreme example of this is at Bridge 16 (sta. 1045+07) where a concrete strut, 7 to 9 ft. in thickness, has been poured across a 200 ft. width of the canal between the centre piers of the bridge.

Since traffic must be carried over the bridges while the canal is being dewatered the safety of the structures must be assured and protective measures must be carried out prior to drawdown.

The syphon structure is of massive concrete and founded on timber piles apparently driven to refusal on rock. (Refer to Fig. 7). The possibility of drawdown of the canal causing movement of this structure seems small, but this is largely dependent on a structural analysis of the syphon.

The areas requiring protective or remedial measures prior to drawdown are marked on Figure 1.

SUGGESTED PROTECTIVE MEASURES

To provide protection in those specific areas above where major slope failures must be prevented, a practical measure would be to load the toe of the slopes or at the base of retaining walls or bridge structures, prior to drawdown, thus balancing the unloading at the toe by the removal of some 30 ft. of water. Since it has been proposed that the roadway grade would be about 20 ft. above the present canal bottom the fill, thus placed, would not be wasted since it would be used to form, in part, the subgrade for the highway and also in the construction of exit and approach ramps. As such protective fill would have to be dumped through water, its gradation should be as coarse as possible. Most of the material excavated in the Welland Canal re-alignment would be silty clay or clayey silt generally at a moisture content well above the plastic limit. This material would be unsuitable for dumping through water, particularly if it were to form the subgrade for a future highway. However a possible source of coarse fill is the spoil area located on the west bank of the canal running north from about sta. 866+00. The approximate location and extent of this area is shown on Figure 1.

Representative bulk samples of the material in this spoil area were obtained and it was found to consist mainly of boulders varying from 3 inches to 3 ft. in size bound in a lean matrix of silty clay. From a rough survey of the area it was estimated that about 400,000 to 500,000 cu. yd. of material were available for use as fill. If, to balance the unloading effect of some 30 ft. of water, about 15 ft. of material is dumped through water prior to drawdown between sta's 965+00 and 1033+00 approximately and also between sta's 1033+00 to 1045+00 approximately, then about 700,000 cu. yd. of material is required to cover the full width of the canal to a depth of 15 ft. If the height of dumped material is reduced to 12 ft. (which is possibly sufficient to prevent the majority of deep seated failures) and the fill concentrated more at the edges of the canal rather than covering the full width then the approximate amounts of 'fill required' to 'fill locally available' correspond. Consequently, it is important that the extent and quality of the spoil area be investigated in detail because of its availability and proximity to the site.

CONCLUSIONS AND RECOMMENDATIONS

In general, from the consideration of problems of the stability of canal banks and foundations of structures, the construction of a section of Highway 406 within the drained excavation of the Welland Canal is feasible provided that protective measures, such as partial in-filling prior to drawdown of the canal, are carried out in areas of potential major instability. Such areas have been defined and are discussed above.

In relation to the problems of earthworks and foundations no cost studies have been carried out. It is recommended that, accepting the general feasibility of the proposed highway, the following work be carried out prior to design:

i) The quantity of available material for in-filling must be defined. The local spoil area (sta. 800 to 860 approx.) should be surveyed and the quality of the material examined by test pitting. Sufficient testing should be carried out to determine the properties of the material when placed through water and later partially drained. It is estimated that about 25 to 30 test pits about 10 ft. to 15 ft. in depth and dug by back-hoe would be required.

ii) To date, no borings have been put down to determine the character of the canal bottom which, for example, if covered by soft or organic debris, may have serious effect on the placement of fill. It is suggested that borings at 2,000 ft. to 3,000 ft. centres be put down along the centre of the route and carried down to the till. The borings should be supplemented by dynamic cone penetration tests located between the borings. It is estimated that about 10 to 15 borings and 18 to 25 dynamic cone penetration tests would be required.

iii) The Welland River closely approaches the existing canal excavation in several areas. (Sta's 817-841, 908-917, 935-947). The water level in the river varies between elevation 562 and 569. Thus, if the excavation be completely drained, as much as 29 ft. head of water would act on the dyke separating the river from the excavation. It is essential that the stability of dyke sections be assured at all times. Since the banks in these specific areas are partially composed of fill (refer to Fig. 2) designed for a head of water acting in the opposite direction (Canal level - 569 to 570, low river level - 562), investigations should be carried out to determine the exact cross-section and quality of materials in the dyke and stability studies made for the drawdown and final conditions. It is estimated that about 10 borings, together with suitable piezometric installations would be required to determine the soil conditions in the dyke areas.

iv) The available boring information between sta's 960 and 1110 is scanty. (Refer to Fig. 2). It is recommended that further borings be put down in these locations for two purposes, viz. (a) to confirm the properties of the clay overburden as compared to those assumed in the report from existing data, and (b) to enable further studies to be made of the suggested protective measures. It is estimated that approximately 10 borings would be required. Cost studies should then be carried out of the most economical pattern and quantities of fill to be placed.

v) Some further investigation should also be carried out at the locations of the bridge structures and at the syphon and particular study given to the details and condition of the foundations. From the existing information and that obtained under (ii) and (iv) above it is probable that no more than two further borings per structure would be required.

vi) Available data on groundwater levels is limited. It is particularly important to determine the piezometric conditions in areas of potential major instability and where the Welland River is close to the canal. Piezometers could readily be installed during the investigations carried out under (ii) to (v) above and a programme of observation set up.

Since step (ii) can only be carried out when the existing canal is closed to shipping, it is recommended that it be completed during this winter. The remaining steps would only be taken when the character of the canal bottom has been defined.

B. T. Darch

B. T. Darch, P.Eng.

V. Milligan, P.Eng.

REGISTERED PROFESSIONAL ENGINEER V. MILLIGAN PROVINCE OF ONTARIO

BTD:VM:IMB
66093
January 31, 1967

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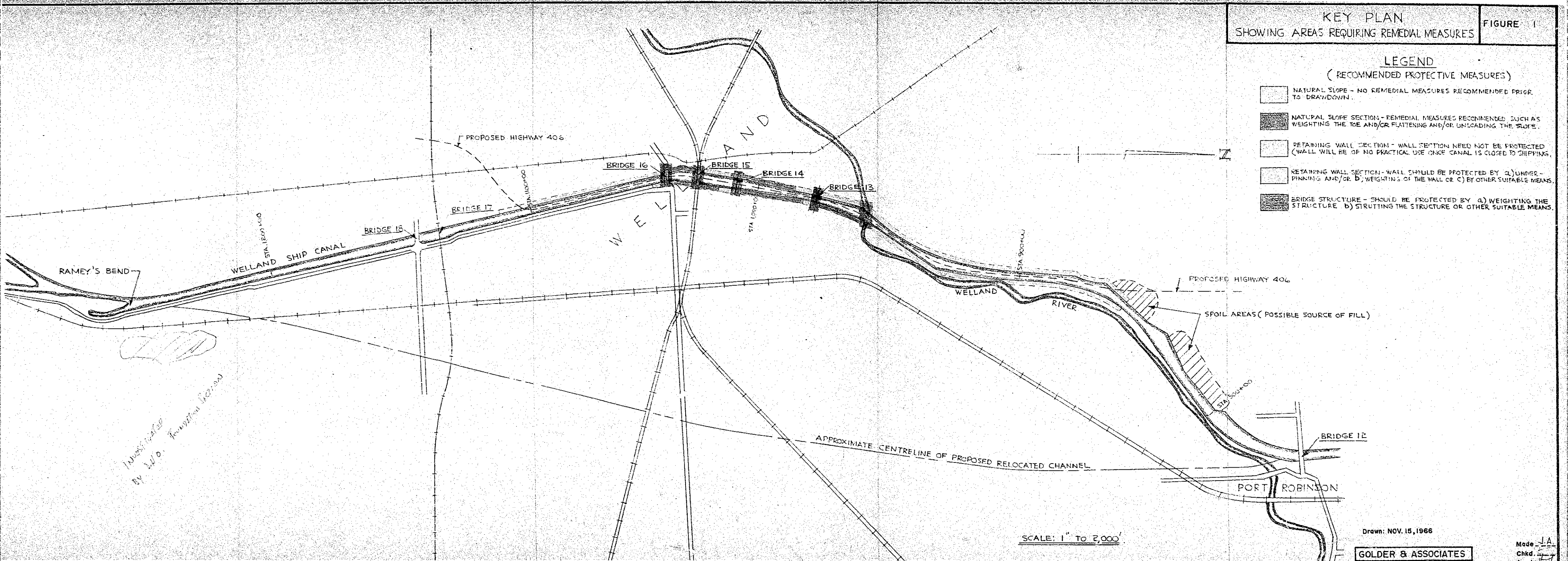
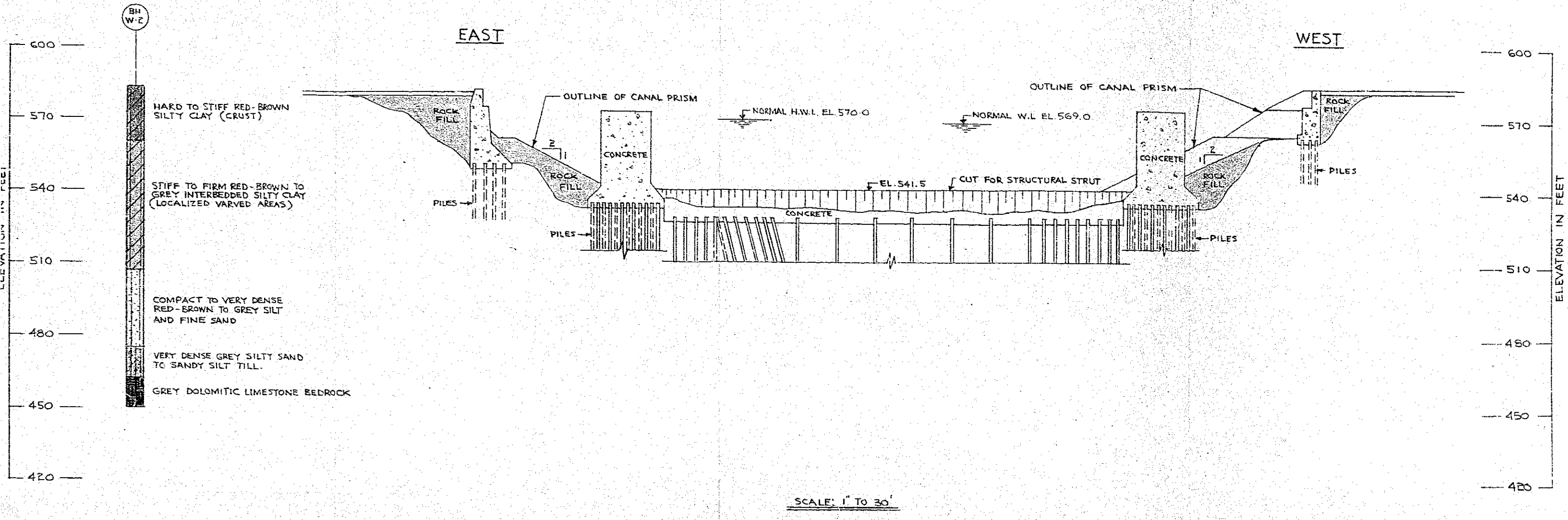
TUSTIN, T.G., 1966. (b) Report - "Preliminary Review of Geotechnical Properties of Overburden Deposits Bridge 13 to Bridge 18, Welland Canal". St. Lawrence Seaway Authority. Unpublished.

TUSTIN, T.G., 1966. (c) Report - "Preliminary Summary of Subsurface Conditions for channel relocation bridge 12 to Ramey's Bend, Welland Canal". St. Lawrence Seaway Authority. Unpublished.


KEY PLAN
SHOWING AREAS REQUIRING REMEDIAL MEASURES
FIGURE 1
LEGEND
(RECOMMENDED PROTECTIVE MEASURES)
NATURAL SLOPE - NO REMEDIAL MEASURES RECOMMENDED PRIOR TO DRAWDOWN.
NATURAL SLOPE SECTION - REMEDIAL MEASURES RECOMMENDED SUCH AS WEIGHTING THE TOE AND/OR FLATTENING AND/OR UNLOADING THE SLOPE.
RETAINING WALL SECTION - WALL SECTION NEED NOT BE PROTECTED (WALL WILL BE OF NO PRACTICAL USE ONCE CANAL IS CLOSED TO SHIPPING.
RETAINING WALL SECTION - WALL SHOULD BE PROTECTED BY a) UNDER-PINNING AND/OR b) WEIGHTING OF THE WALL OR c) BY OTHER SUITABLE MEANS.
BRIDGE STRUCTURE - SHOULD BE PROTECTED BY a) WEIGHTING THE STRUCTURE b) STRUTTING THE STRUCTURE OR OTHER SUITABLE MEANS.
N
PROPOSED HIGHWAY 406
BRIDGE 16
BRIDGE 15
BRIDGE 14
BRIDGE 13
BRIDGE 17
BRIDGE 18
BRIDGE 12
WELLAND
WELLAND SHIP CANAL
RAMEY'S BEND
STA 1200+00
STA 1100+00
STA 1000+00
STA 900+00
STA 800+00
WELLAND RIVER
PROPOSED HIGHWAY 406
SPOIL AREAS (POSSIBLE SOURCE OF FILL)
APPROXIMATE CENTRELINE OF PROPOSED RELOCATED CHANNEL
PORT ROBINSON
Investigated by W.O. Foundation Section
SCALE: 1" TO 2,000'
Drawn: NOV. 15, 1966
GOLDER & ASSOCIATES
Made J.A.
Chkd.

SECTION ALONG CENTRELINE OF BRIDGE Nº 16
(STA. 1045+07)
SHOWING FOUNDATION DETAILS

FIGURE 3



REFERENCES

1.) ST. LAWRENCE SEAWAY AUTHORITY, DRWG. Nº 7,612 SHOWING DETAILS OF BRIDGE Nº 16 - WELLAND SHIP CANAL; DATED AUG. 1, 1924.

2.) REFER TO FIGURE 15, PHOTOGRAPHS Nº 3 AND 4.

NOTE:
ALL PILES DRIVEN TO PRACTICAL REFUSAL IN CLAY OR TILL.

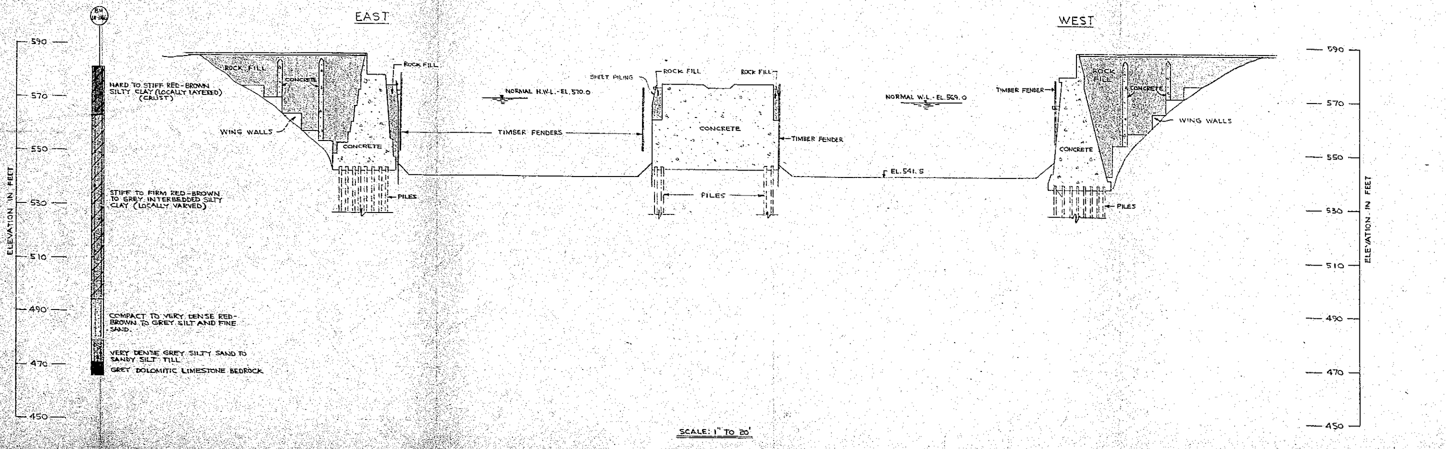
Drawn: OCT. 13, 1966

GOLDER & ASSOCIATES

Made J.A.
Chkd.
Appd.

SECTION ALONG CENTRELINE OF BRIDGE No 15
(STA. 1033+03)
SHOWING FOUNDATION DETAILS

FIGURE 4



REFERENCES

1.) ST. LAWRENCE SEAWAY AUTHORITY, DRWG. No 7,601 SHOWING DETAILS OF BRIDGE No 15 - WELLAND SHIP CANAL, DATED AUG. 1, 1924.

2.) REFER TO FIGURES 16 AND 17, PHOTOGRAPHS No 6 AND 7

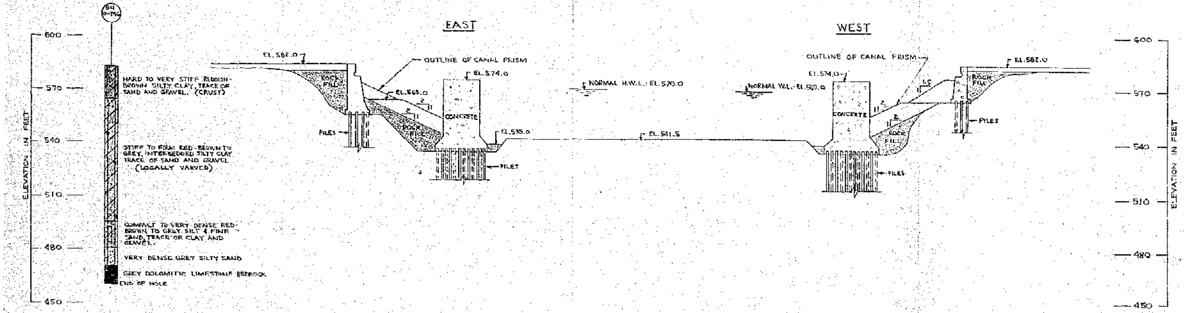
NOTE:
ALL PILES DRIVEN TO PRACTICAL REFUSAL IN CLAY OR TILL.

Drawn: OCT. 12, 1986

GOLDER & ASSOCIATES

SECTION ALONG CENTRELINE OF BRIDGE Nº 14 (STA. 1007+98) SHOWING FOUNDATION DETAILS

FIGURE 5



SCALE: 1" TO 30'

REFERENCES

1.) ST. LAWRENCE SEAWAY AUTHORITY, DRWG. Nº 7,650 SHOWING DETAILS OF BRIDGE Nº 14 WELLAND SHIP CANAL, DATED AUG. 1, 1924

2.) REFER TO FIGURE 20, PHOTOGRAPH Nº 13

NOTE:
ALL PILES DRIVEN TO PRACTICAL REFUSAL IN CLAY OR TILL.

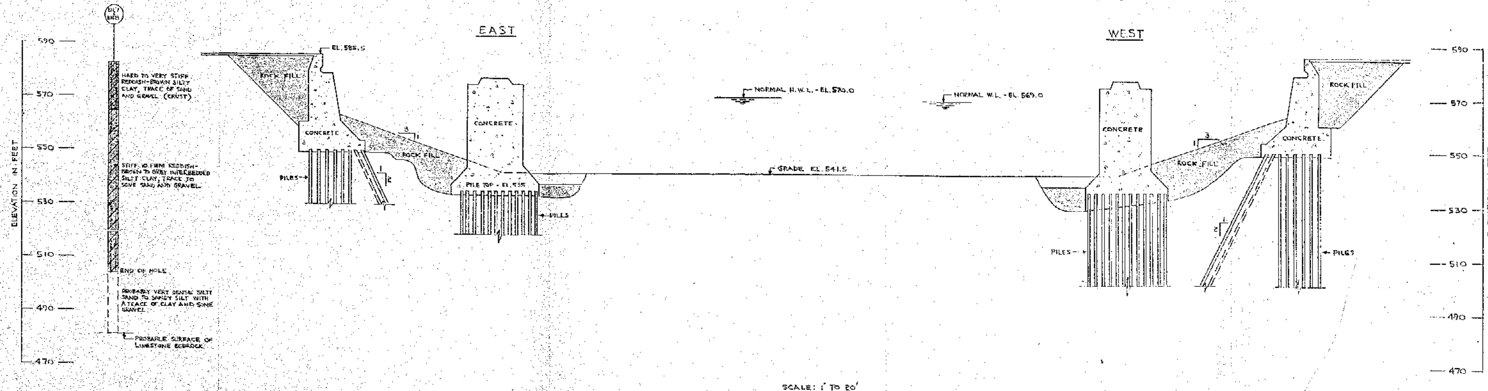
Drawn: OCT. 12, 1966

GOLDER & ASSOCIATES

Made. J.A.
Chkd.
Appd.

SECTION ALONG CENTRELINE OF BRIDGE Nº 13 (STA. 977 + 94) SHOWING FOUNDATION DETAILS

FIGURE 6



SCALE: 1" TO 20'

REFERENCES

1.) ST. LAWRENCE SEAWAY AUTHORITY, DRWG. Nº 7,644 SHOWING DETAILS OF BRIDGE Nº 13 - WELLAND SHIP CANAL, DATED DEC. 30, 1927.

2.) REFER TO FIGURE 22, PHOTOGRAPHS Nº 17 AND 18

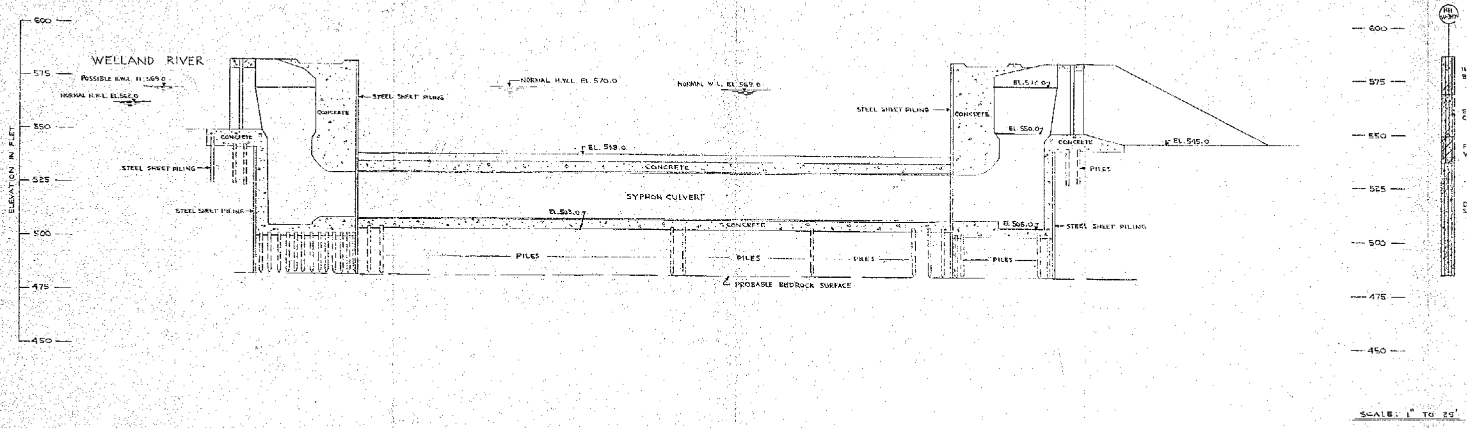
NOTE

ALL PILES DRIVEN TO PRACTICAL REFUSAL IN CLAY OR TILL.

Drawn: OCT. 11, 1966

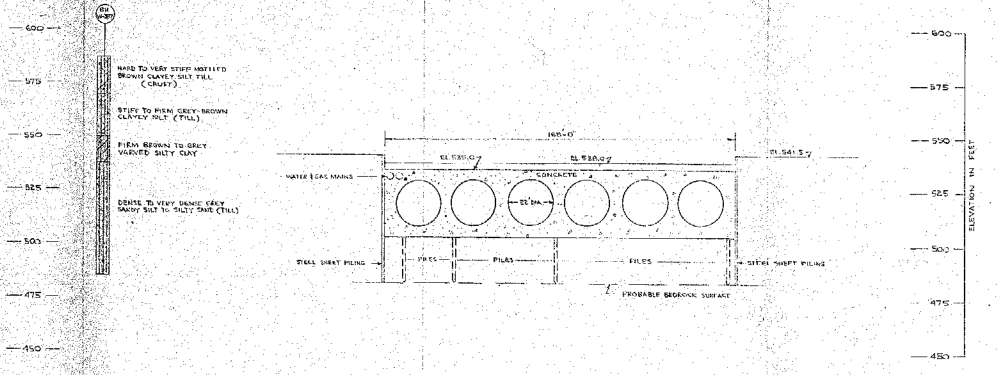
GOLDER & ASSOCIATES

Made
Chkd.
Appd.


WELLAND RIVER
ELEVATION IN FEET
600
575
550
525
500
475
450
POSSIBLE H.W.L. EL. 569.0
NORMAL H.W.L. EL. 562.0
CONCRETE
STEEL SHEET PILING
STEEL SHEET PILING
CONCRETE
STEEL SHEET PILING
NORMAL H.W.L. EL. 570.0
NORMAL W.L. EL. 569.0
EL. 538.0
CONCRETE
SYPHON CULVERT
EL. 503.0
CONCRETE
PILES
PILES
PILES
PROBABLE BEDROCK SURFACE
STEEL SHEET PILING
CONCRETE
EL. 572.0
EL. 550.0
CONCRETE
EL. 545.0
PILES
EL. 505.0
STEEL SHEET PILING
PILES
SCALE: 1" TO 25'

SECTION ALONG CENTRELINE OF SYPHON
(STA. 966+00)
SHOWING FOUNDATION DETAILS

FIGURE 7



SCALE: 1" TO 25'

REFERENCES

1.) ST LAWRENCE SEAWAY AUTHORITY, DRWG. Nº 6,558 SHOWING DETAILS OF SYPHON - WELLAND SHIP CANAL, DATED JUNE 1, 1925.

2.) REFER TO FIGURE 23, PHOTOGRAPHS Nº 19 AND 20.

NOTE:

ALL PILES DRIVEN TO PRACTICAL REFUSAL (PROBABLY TO BEDROCK SURFACE.

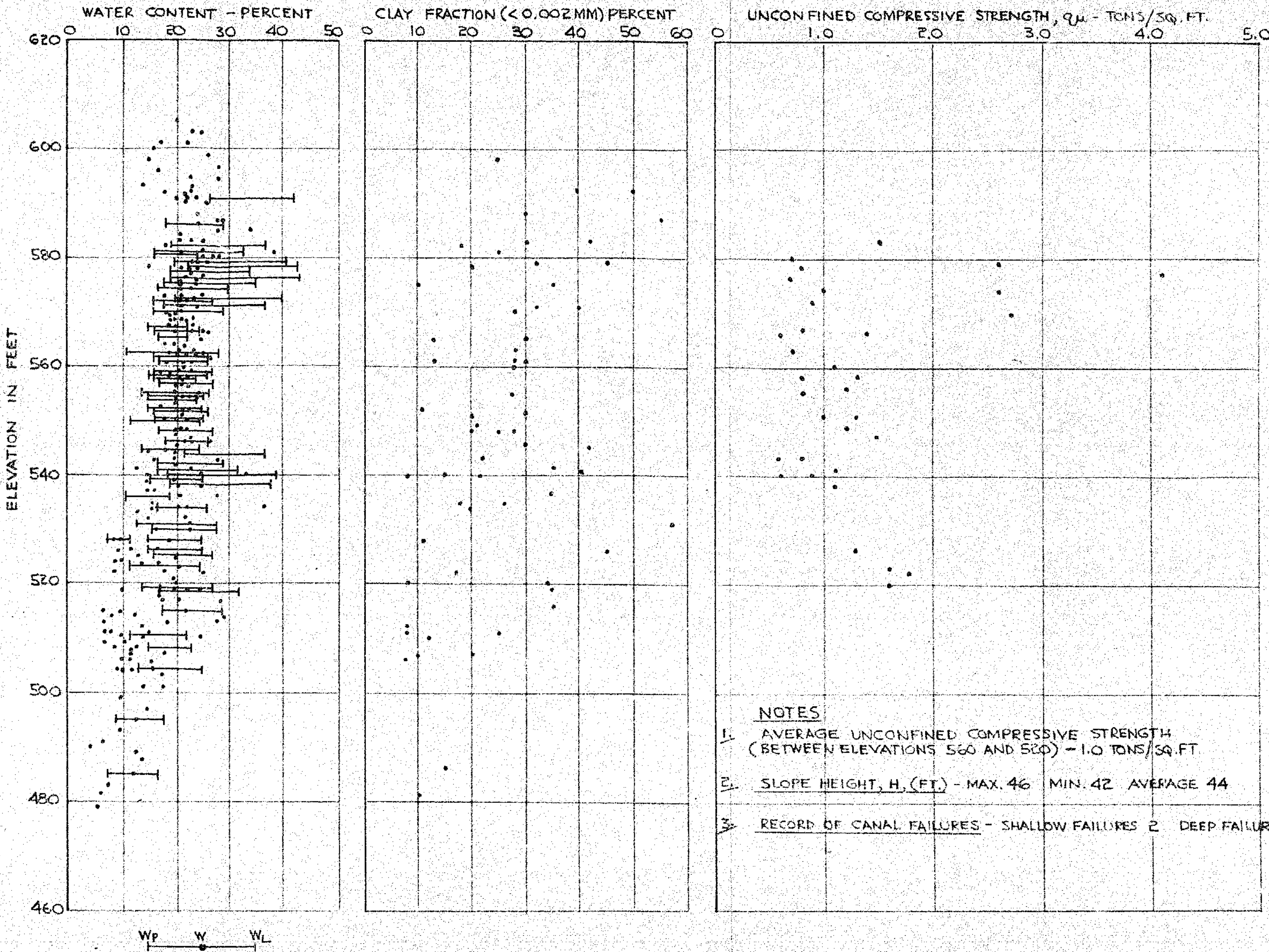
Drawn: OCT. 14, 1966

GOLDER & ASSOCIATES

Made J.A.
Chkd.
Appd.

SUMMARY PLOT OF ENGINEERING PROPERTIES
(STA. 800+00 TO 880+00)

FIGURE 8



NOTES

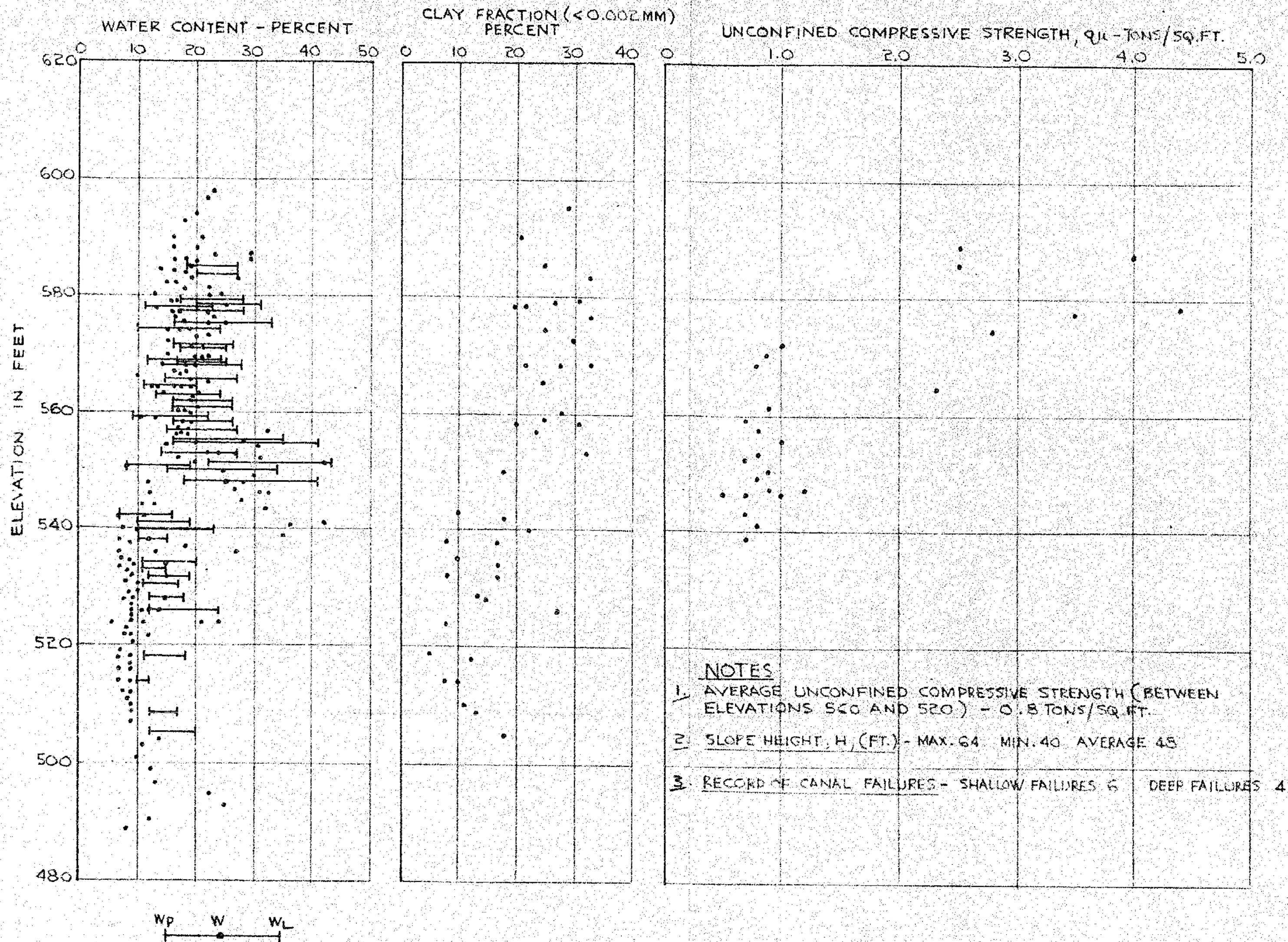
1. AVERAGE UNCONFINED COMPRESSIVE STRENGTH (BETWEEN ELEVATIONS 560 AND 520) - 1.0 TONS/SQ.FT.
2. SLOPE HEIGHT, H, (FT.) - MAX. 46 MIN. 42 AVERAGE 44
3. RECORD OF CANAL FAILURES - SHALLOW FAILURES 2 DEEP FAILURES 1

GOLDER & ASSOCIATES

Made J.A.
Chkd.
Appd.

SUMMARY PLOT OF ENGINEERING PROPERTIES
(STA. 890+00 TO STA. 960+00)

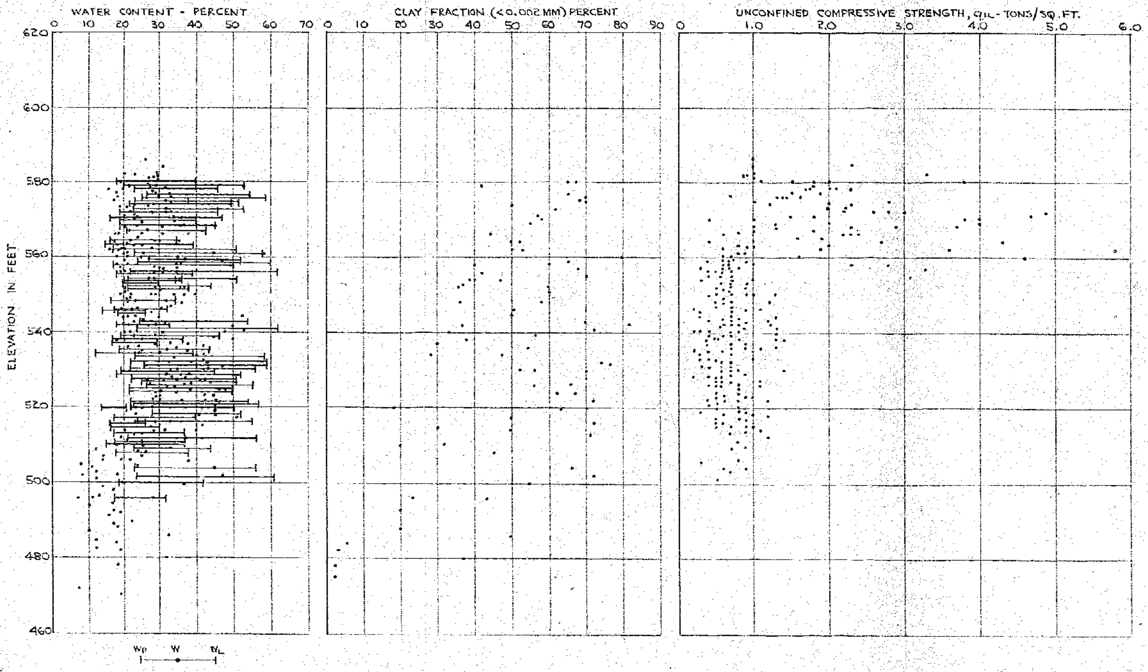
FIGURE 9

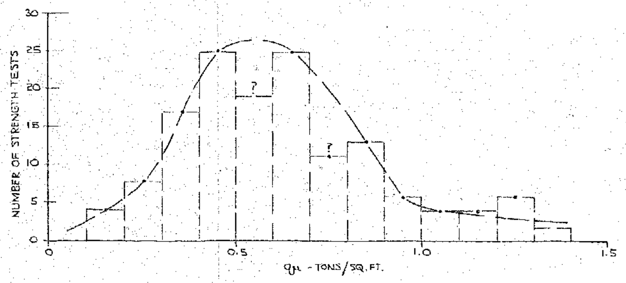


GOLDER & ASSOCIATES

Made I.A.
Chkd. B.T.D.
Appd. [illegible]

SUMMARY PLOT OF ENGINEERING PROPERTIES (STA. 980+00 TO STA. 1110+00)	FIGURE 10





FREQUENCY DISTRIBUTION CURVE

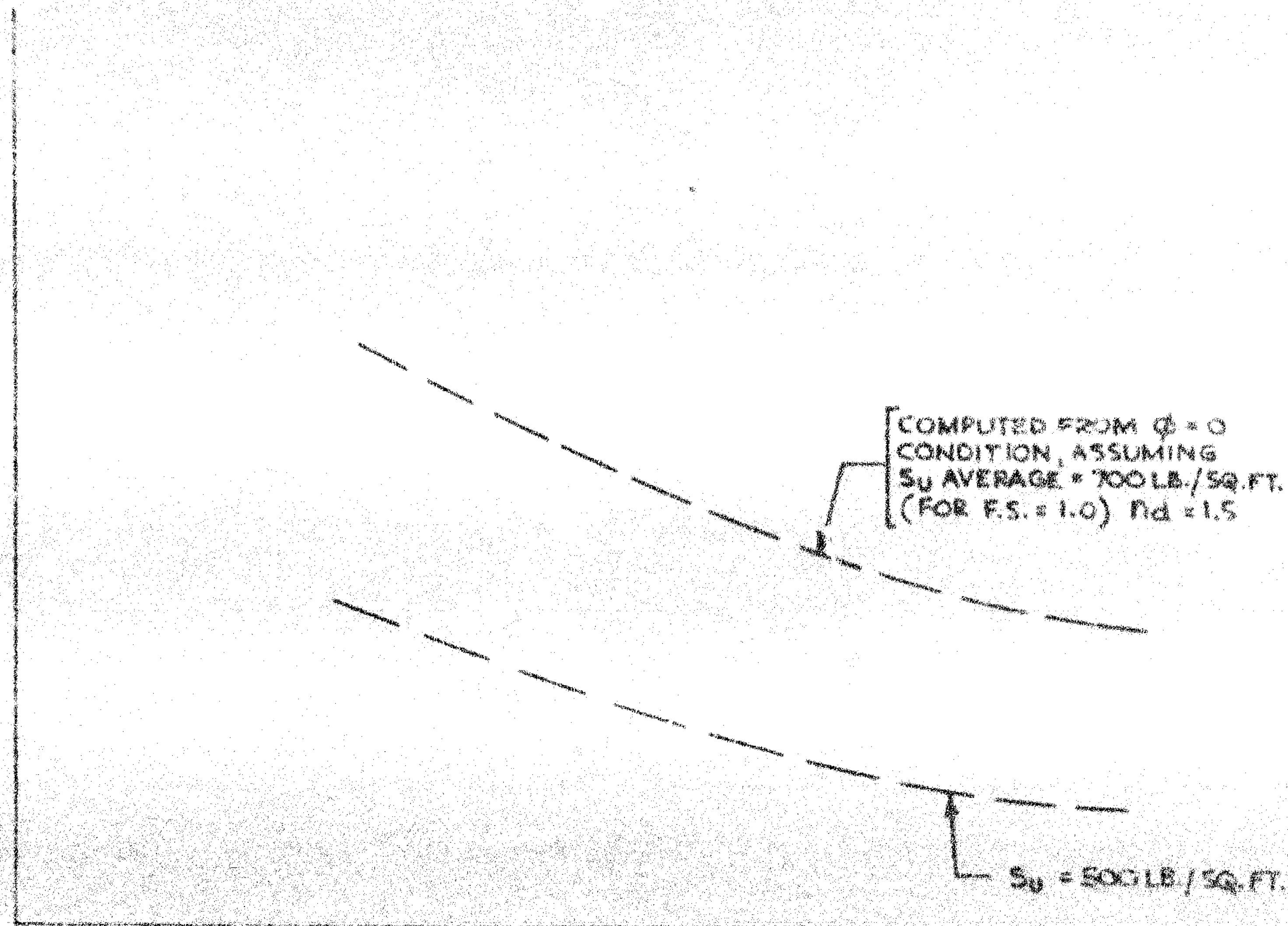
STATISTICAL SUMMARY OF MEASURED COMPRESSIVE STRENGTH, qu, BETWEEN ELEVATIONS 560 AND 520

NOTES

1. AVERAGE UNCONFINED COMPRESSIVE STRENGTH (BETWEEN ELEVATIONS 560 AND 520) - 0.6 TONS/SQ.FT.
2. SLOPE HEIGHT, H, (FT.) MAX. 55 MIN. 40 AVERAGE 45
3. RECORD OF CANAL FAILURES - SHALLOW FAILURES 8 DEEP FAILURES 8

GOLDER & ASSOCIATES

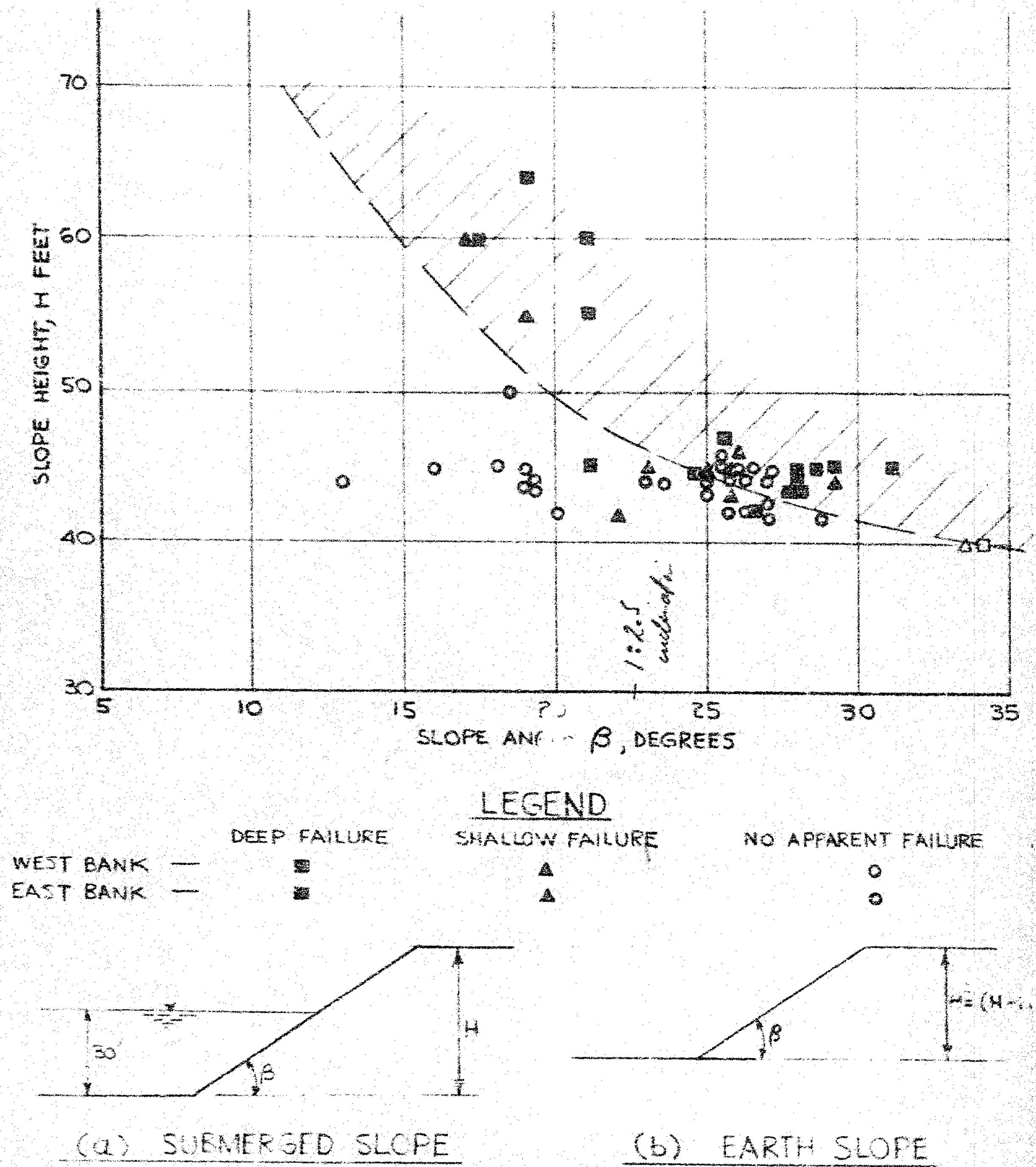
Made J.A.
Chkd. [illegible]
Appd. [illegible]


COMPUTED FROM ϕ = O
CONDITION, ASSUMING
S_u AVERAGE = 700 LB./SQ.FT.
(FOR F.S. = 1.0) n_d = 1.5
S_u = 500 LB./SQ.FT.

PROJECT No. 95023

SUMMARY OF CANAL FAILURES
(STA. 815+00 TO STA. 1,110+00)

FIGURE 11



Fa = Fb

REFERENCE
(STABILITY CHART TAYLOR (1948) FIG. 16-27)

GOLDER & ASSOCIATES

Made
Chkd.
Appd.

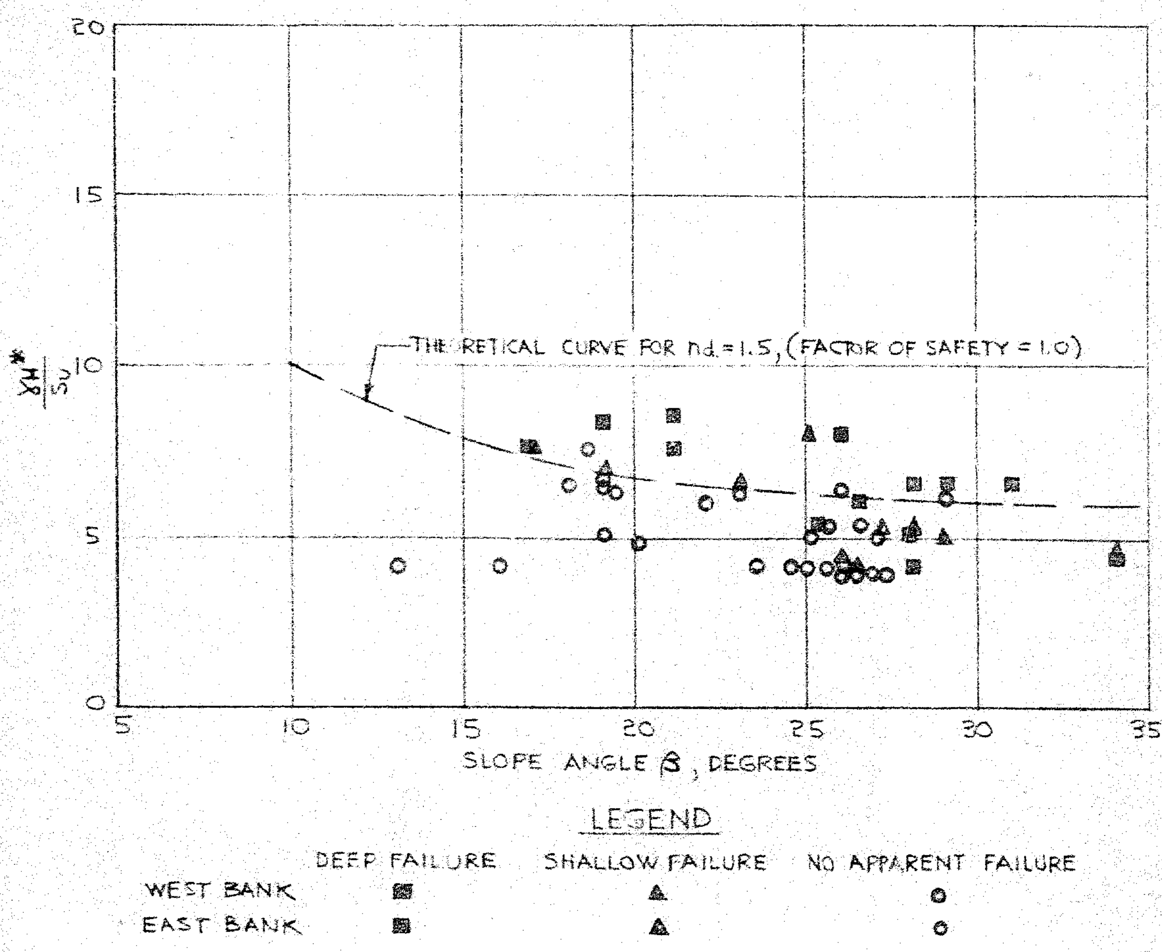
PROJECT No. 66083

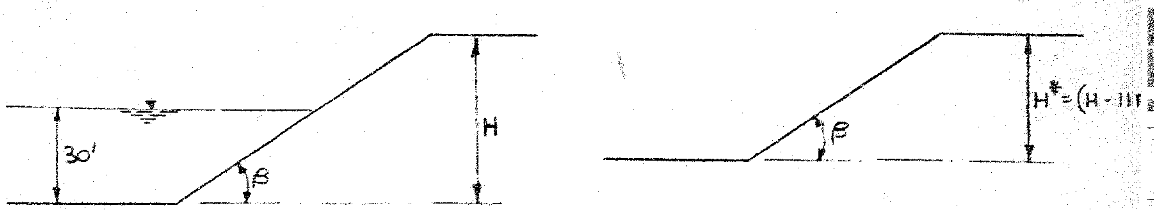
CANAL BANK FAILURES RELATED TO REPORTED UNDRAINED SHEAR STRENGTH
(STA. 815+00 TO STA. 1,110+00)

FIGURE 12

γ = TOTAL UNIT WEIGHT – LB./CU.FT.

S_U = UNDRAINED SHEAR STRENGTH $\frac{q_u}{2}$ LB./SQ.FT.





(a) SUBMERGED SLOPE (b) EARTH SLOPE

$F_a = F_b$

REFERENCE

(STABILITY CHART TAYLOR (1948) FIG. 16·27)

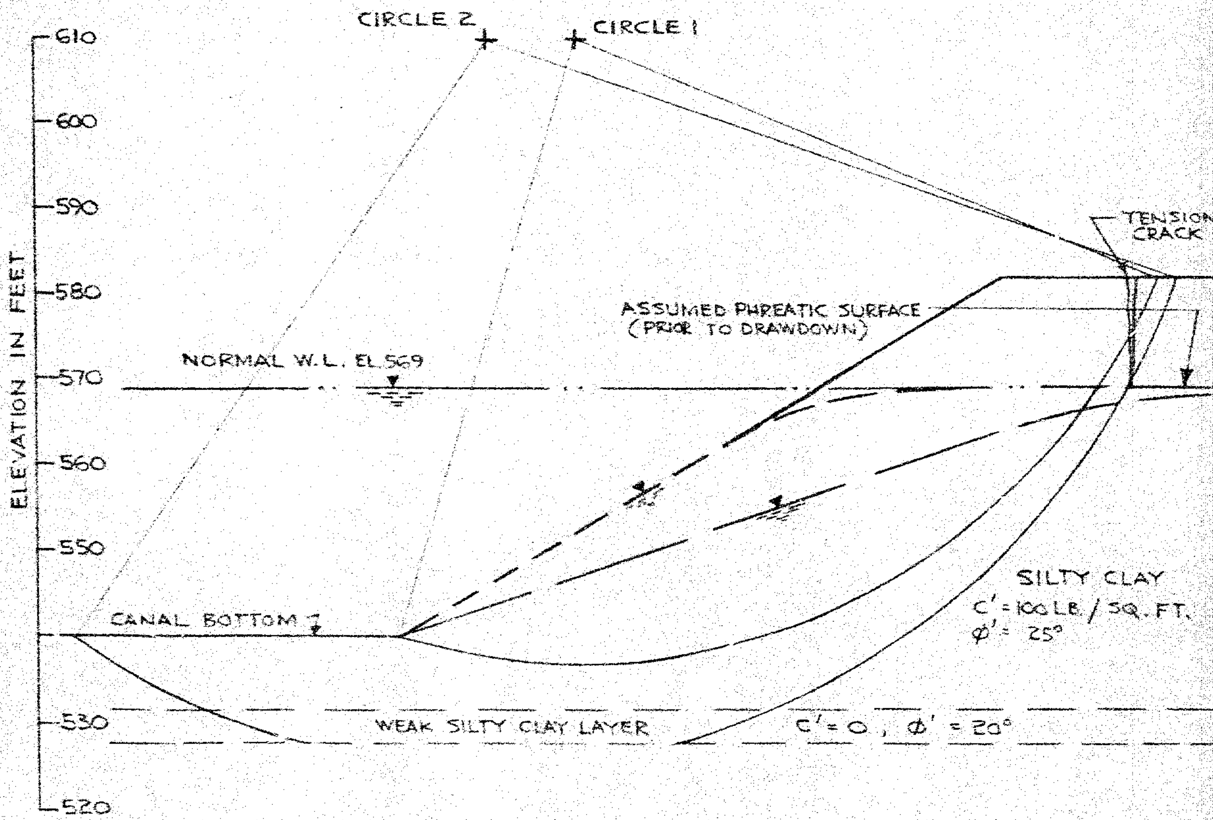
GOLDER & ASSOCIATES

Made J.A.
Chkd.
Appd.

PROJECT No. 662023

SUMMARY RESULTS OF EFFECTIVE STRESS STABILITY ANALYSES, WEST CANAL BANK CROSS-SECTION (STA. 990+00)

FIGURE 13



	CIRCLE No. 1 (THROUGH TOE OF SLOPE)	CIRCLE No. 2 (THROUGH WEAK LAYER)
EXISTING CONDITIONS (PRIOR TO DRAWDOWN)	1.3	1.2
AFTER RAPID DRAWDOWN (NO PORE WATER PRESSURE DISSIPATION)	0.8	0.8
AFTER SLOW DRAWDOWN (FULL PORE WATER PRESSURE DISSIPATION)	~1.0	0.9

GOLDER & ASSOCIATES

Made ...
Chkd. ...
Appd. ...

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 14

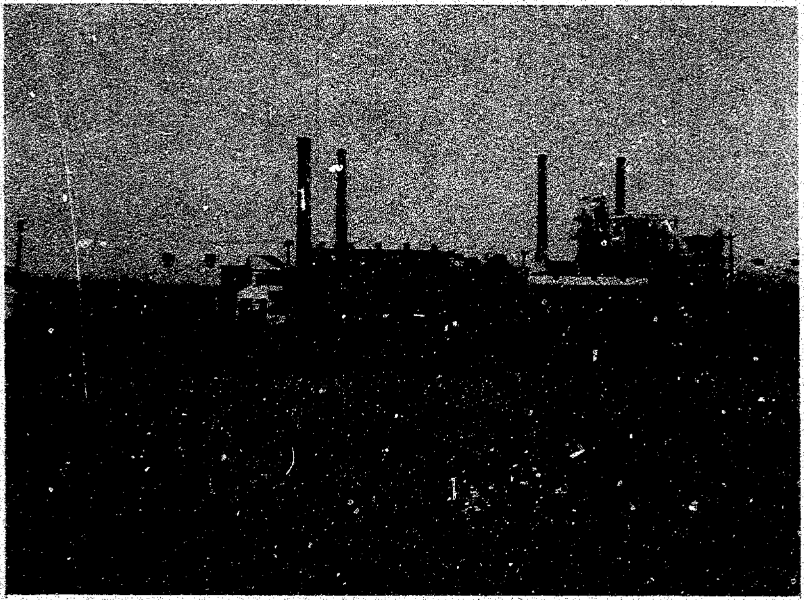


PHOTO 1

View of Union Carbide Ltd. Factory on east bank of canal at approximately STA.1054+00.

PHOTO 2

View of Transmission Tower on west bank of canal at approximately STA.1050+00 looking south.

PROJECT No. 66095

PHOTOGRAPHS OF SITE

FIGURE 14

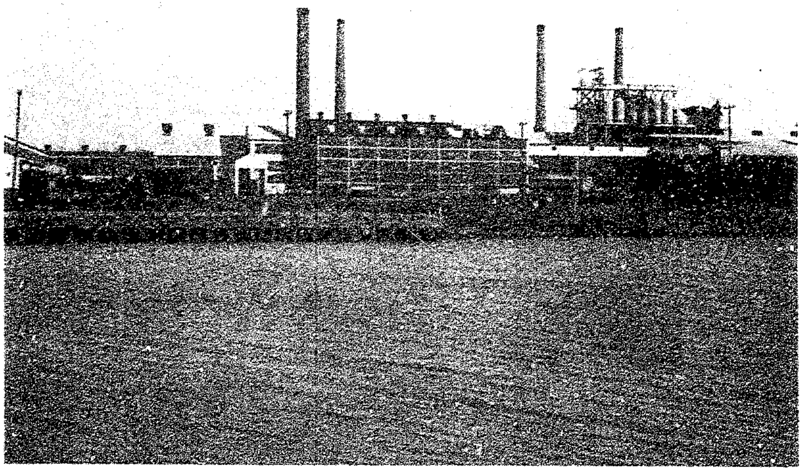


PHOTO 1

View of Union Carbide Ltd. Factory on east bank of canal at approximately STA.1054+00.

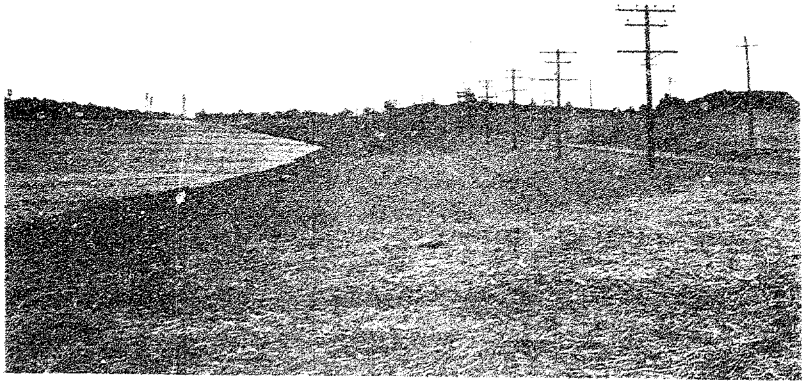


PHOTO 2

View of Transmission Tower on west bank of canal at approximately STA.1050+00 looking south.

GOLDER & ASSOCIATES

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 15

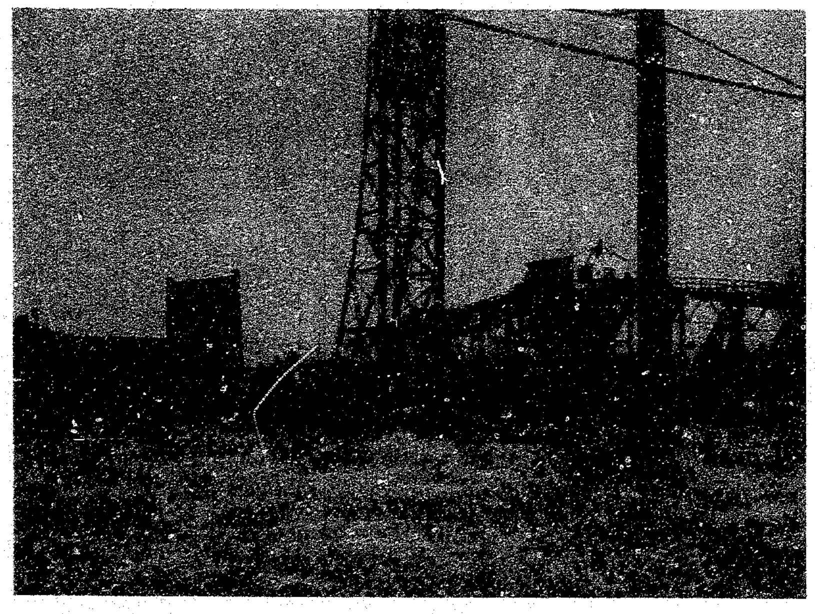


PHOTO 3

View of east abutment of Bridge No. 16 at approximately STA. 1045+07 looking south - east.

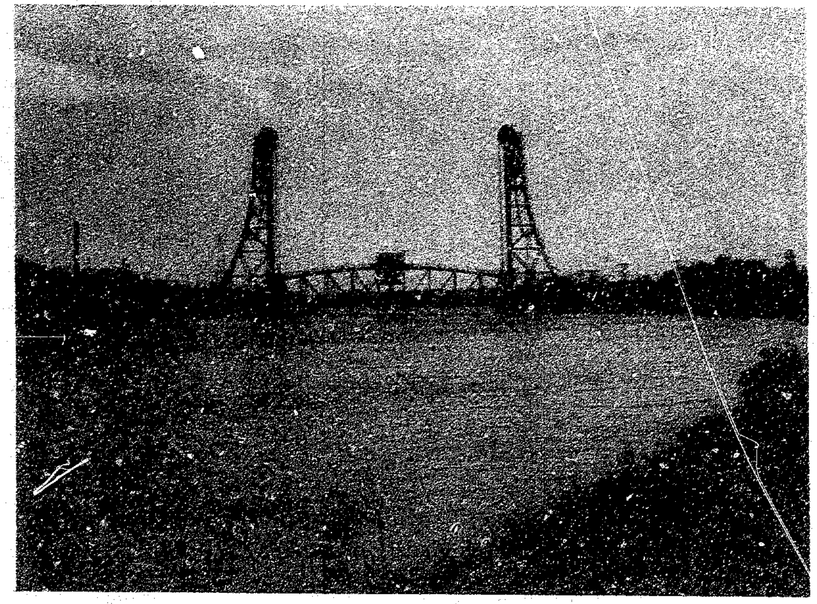


PHOTO 4

View of north side of Bridge No. 16 at approximately STA. 1045+07 looking south.

GOLDER & ASSOCIATES

PROJECT No.

PHOTOGRAPHS OF SITE

FIGURE 15

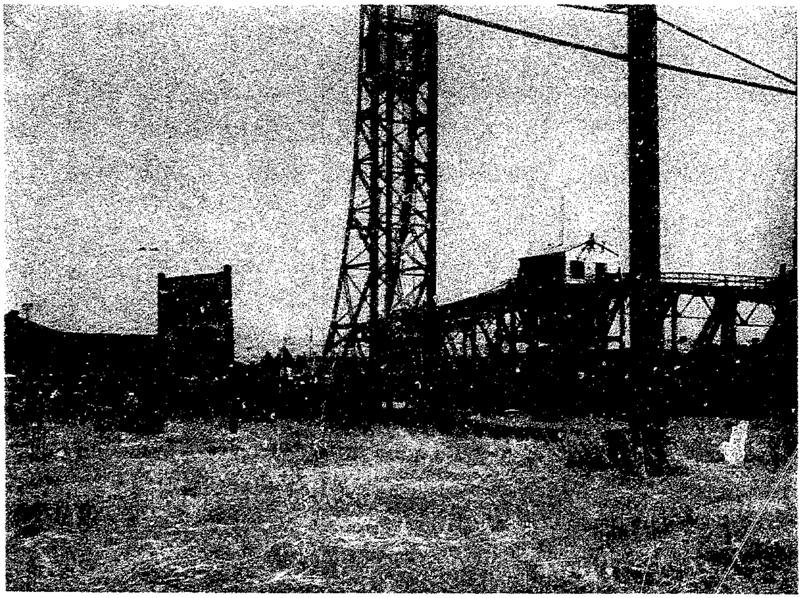


PHOTO 3

View of east abutment of Bridge No. 16 at approximately STA. 1045+07 looking south-east.

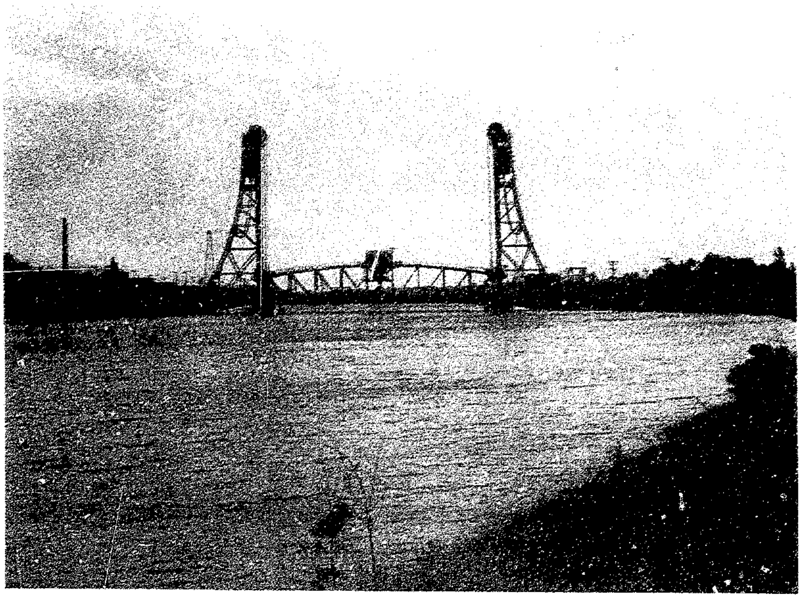


PHOTO 4

View of north side of Bridge No. 16 at approximately STA. 1045+07 looking south.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 16

PHOTO 5

View of shallow slope failure on west bank of canal at approximately STA.1040+00 looking south.

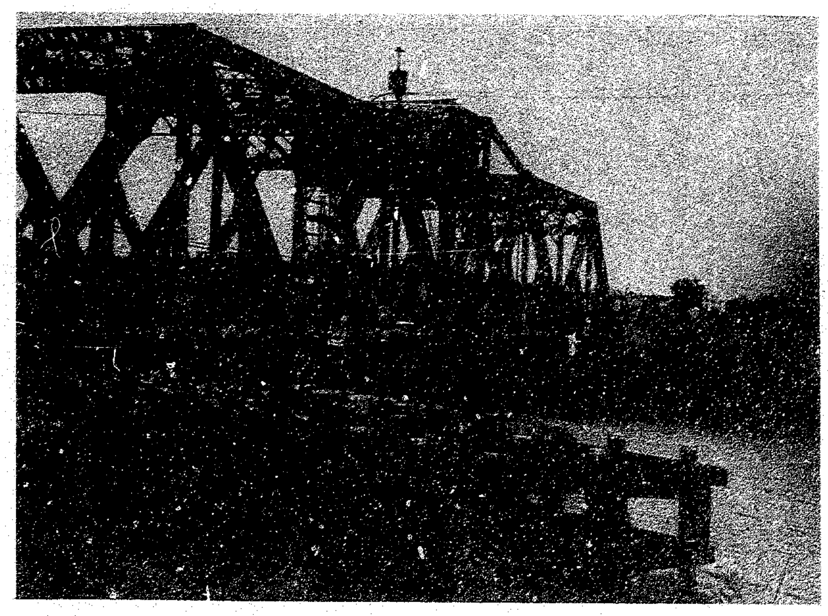


PHOTO 6

View of south side of Bridge No.15 at approximately STA.1033+03 looking north-east.

GOLDER & ASSOCIATES

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 16

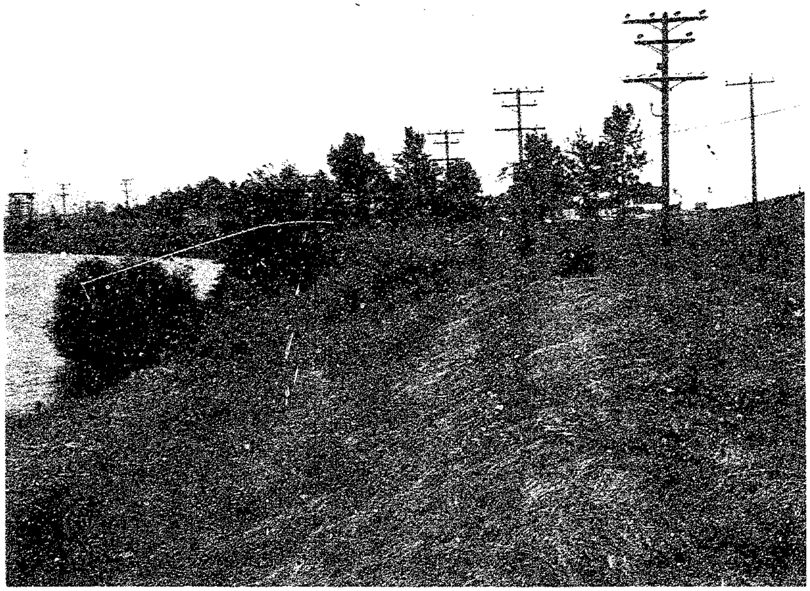


PHOTO 5

View of shallow slope failure on west bank of canal at approximately STA.1040+00 looking south.

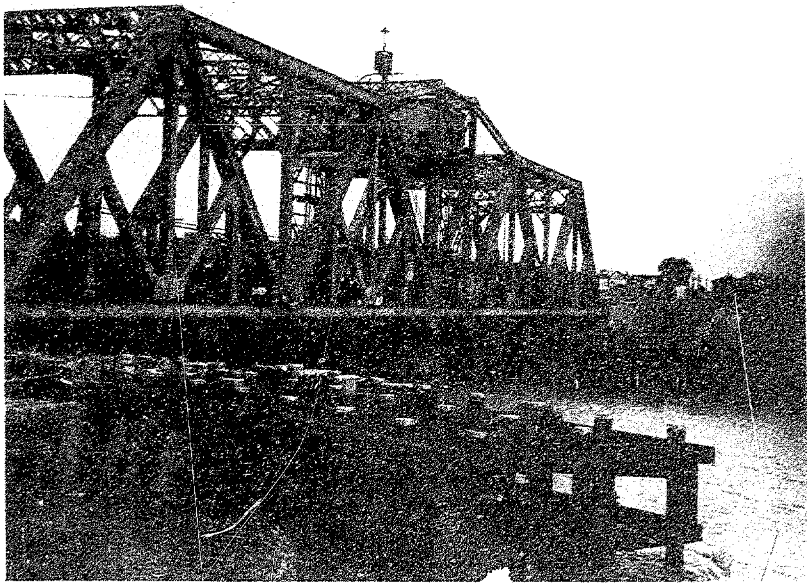


PHOTO 6

View of south side of Bridge No.15 at approximately STA.1033+03 looking north-east.

GOLDER & ASSOCIATES

PHOTOGRAPHS OF SITE

FIGURE 17

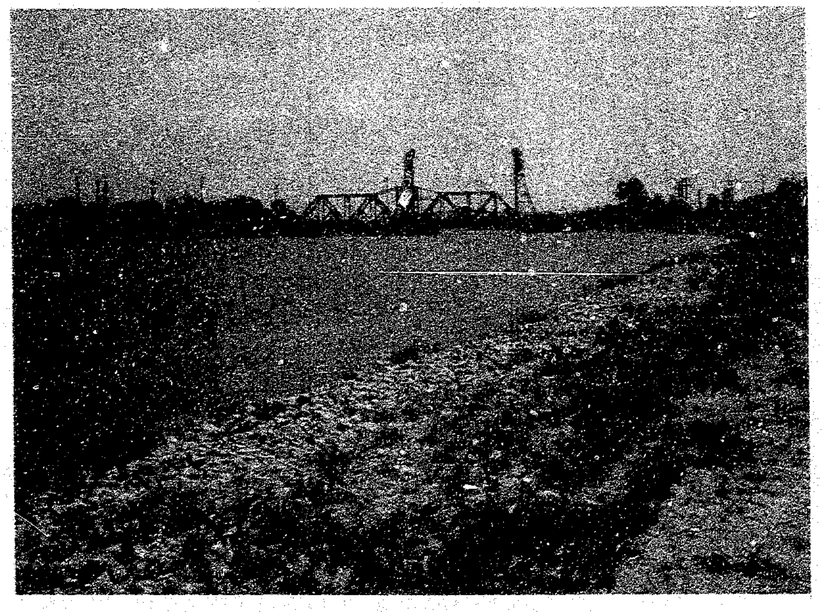


PHOTO 7

View of flattened west bank of canal at approximately STA. 1020+00 looking south, with Bridge No. 15 in background.

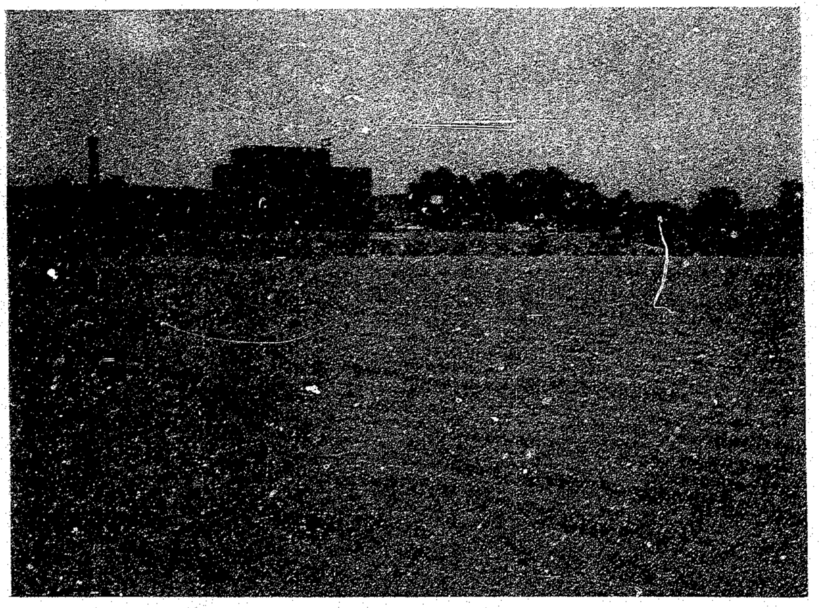


PHOTO 8

View of F.O. Boyle & Sons Building on east bank of canal at approximately STA. 1025+00.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 17

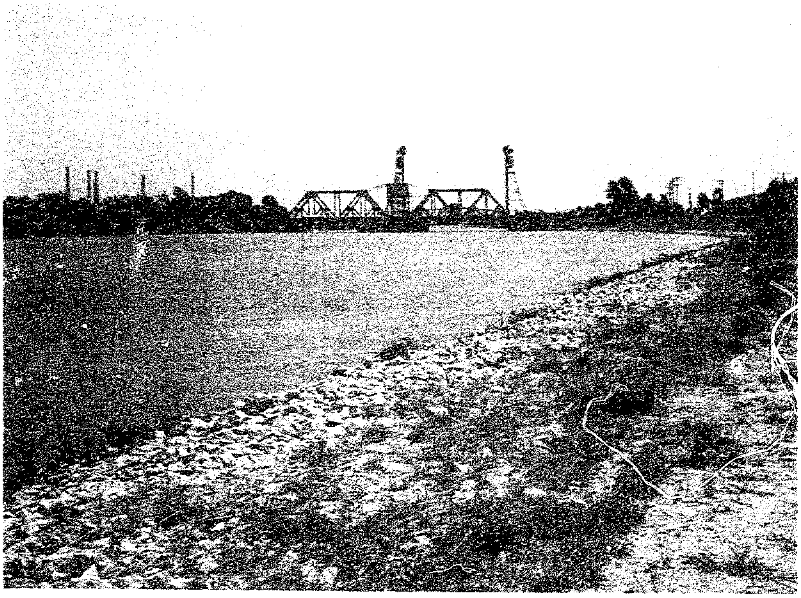


PHOTO 7

View of flattened west bank of canal at approximately STA. 1020+00 looking south, with Bridge No. 15 in background.

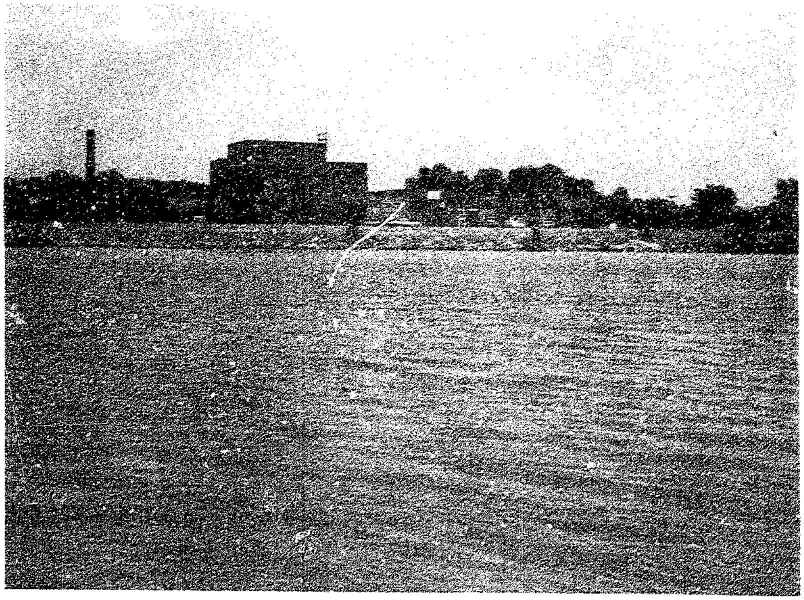


PHOTO 8

View of F.O. Boyle & Sons Building on east bank of canal at approximately STA. 1025+00.

GOLDER & ASSOCIATES

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 18

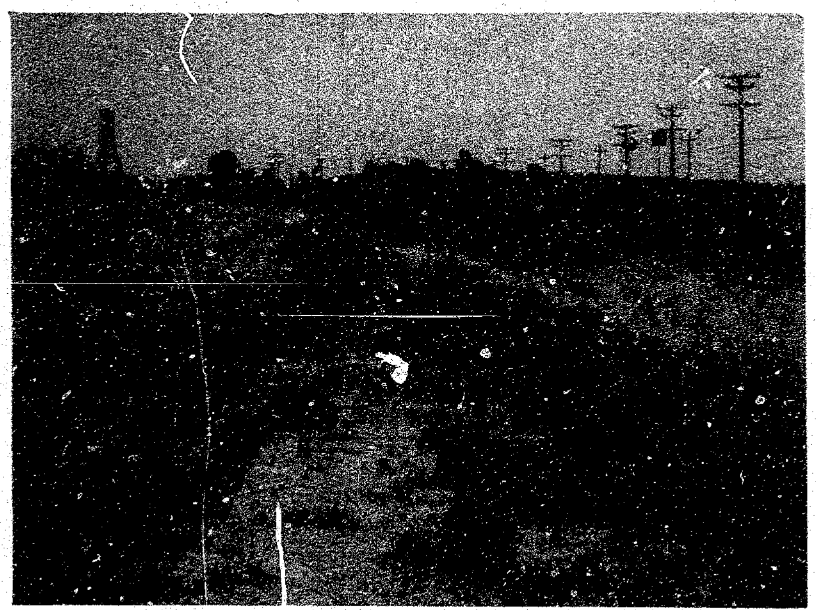


PHOTO 9

View of flattened west bank of canal between STA.1015+00 and Bridge No. 15.

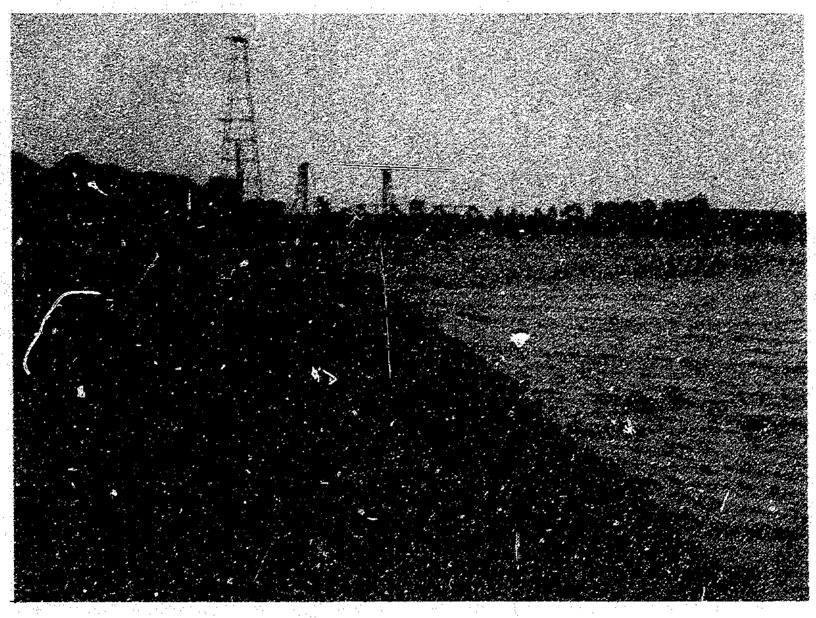


PHOTO 10

View of retaining wall on east bank of canal showing structural cracks in wall at approximately STA. 1012+00 looking south.

PROJECT No.

PHOTOGRAPHS OF SITE

FIGURE 18

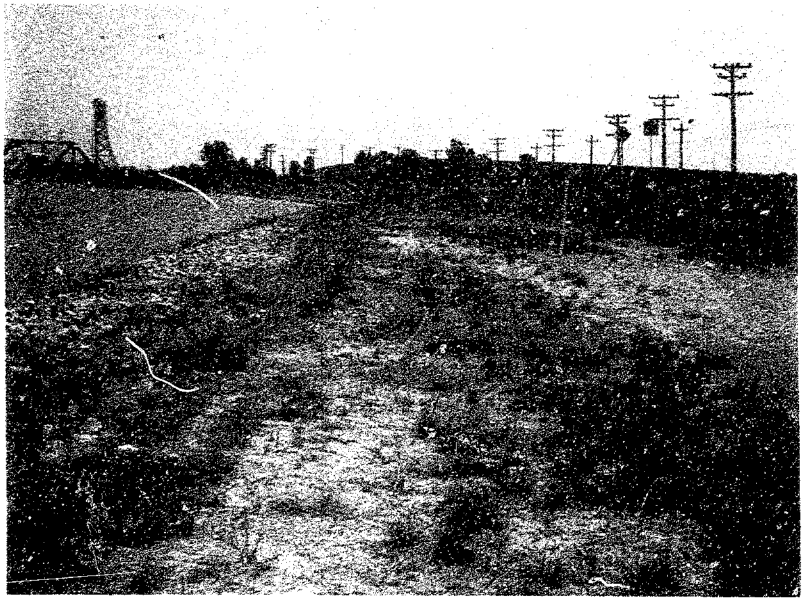


PHOTO 9

View of flattened west bank of canal between STA.1015+00 and Bridge No.15.

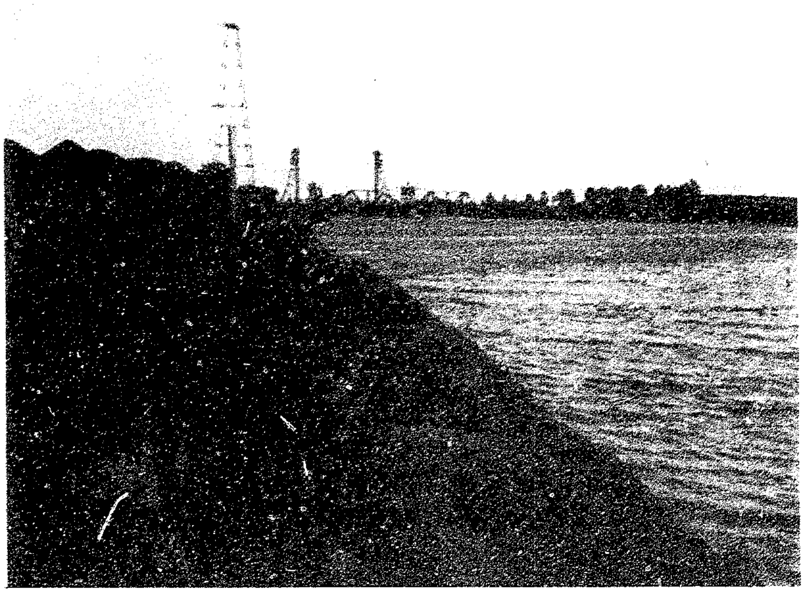


PHOTO 10

View of retaining wall on east bank of canal showing structural cracks in wall at approximately STA. 1012+00 looking south.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 19

PHOTO 11

View of coal loading dock and retaining wall on east bank of canal at approximately STA. 1010+00.

PHOTO 12

View of deep slope failure on west bank of canal at approximately STA. 1005+00 looking north-west

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 19

PHOTO 11

View of coal loading dock and retaining wall on east bank of canal at approximately STA. 1010+00.

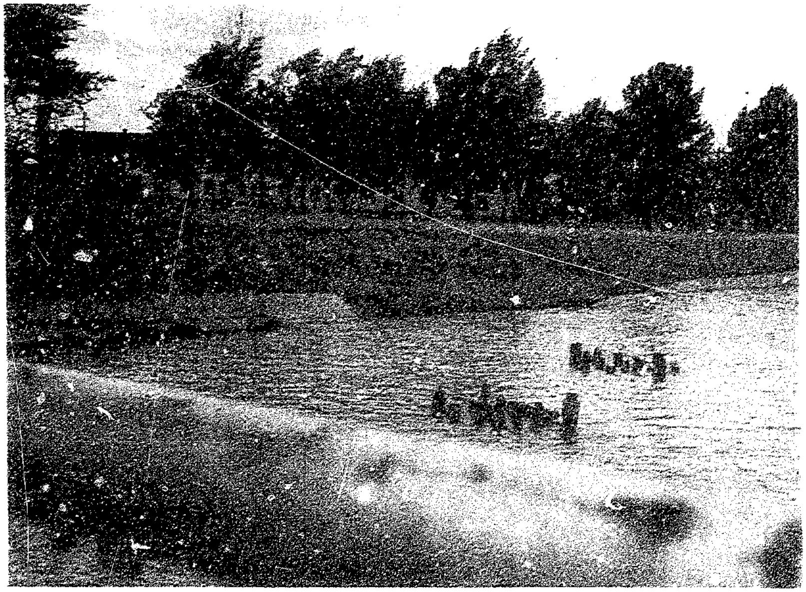


PHOTO 12

View of deep slope failure on west bank of canal at approximately STA. 1005+00 looking north-west

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 20

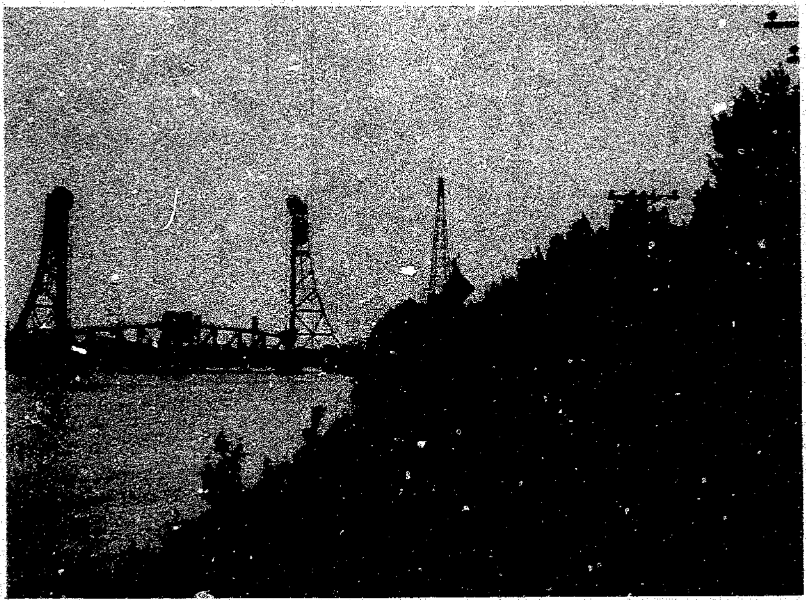


PHOTO 13

View of Transmission Tower at approximately STA.1006+80 and Bridge No. 14 looking south.

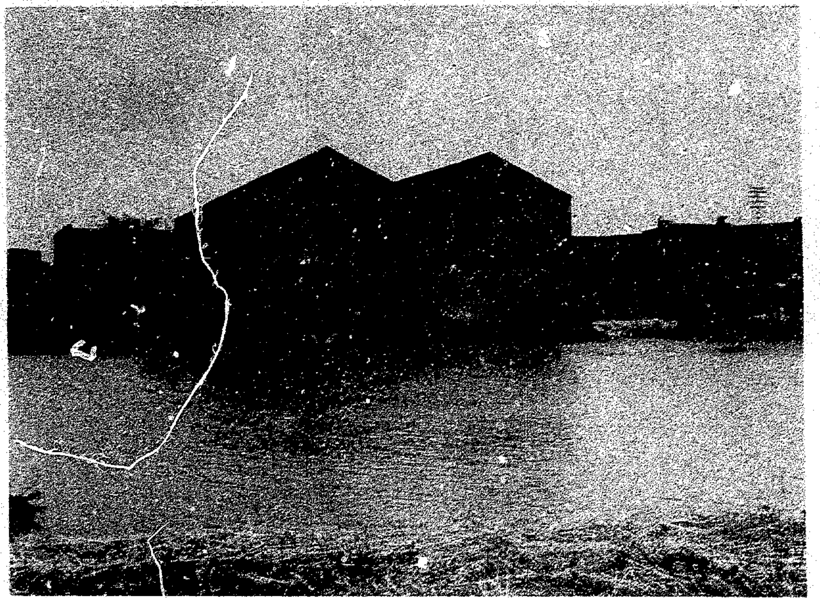


PHOTO 14

View of United Steel Corporation Ltd. Buildings on east bank of canal at approximately STA. 1001+50.

GOLDER & ASSOCIATES

PHOTOGRAPHS OF SITE

FIGURE 20

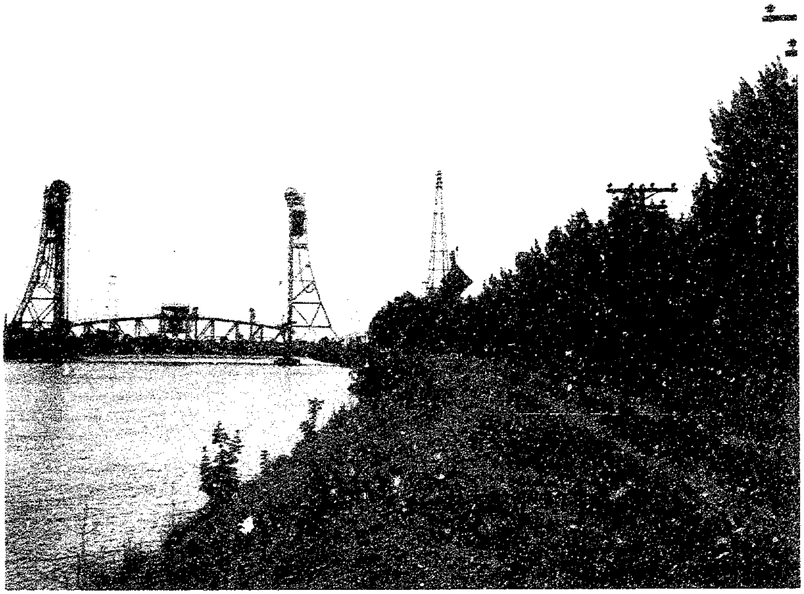


PHOTO 13

View of Transmission Tower at approximately STA.1006+80 and Bridge No. 14 looking south.

PHOTO 14

View of United Steel Corporation Ltd. Buildings on east bank of canal at approximately STA. 1001+50.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 21

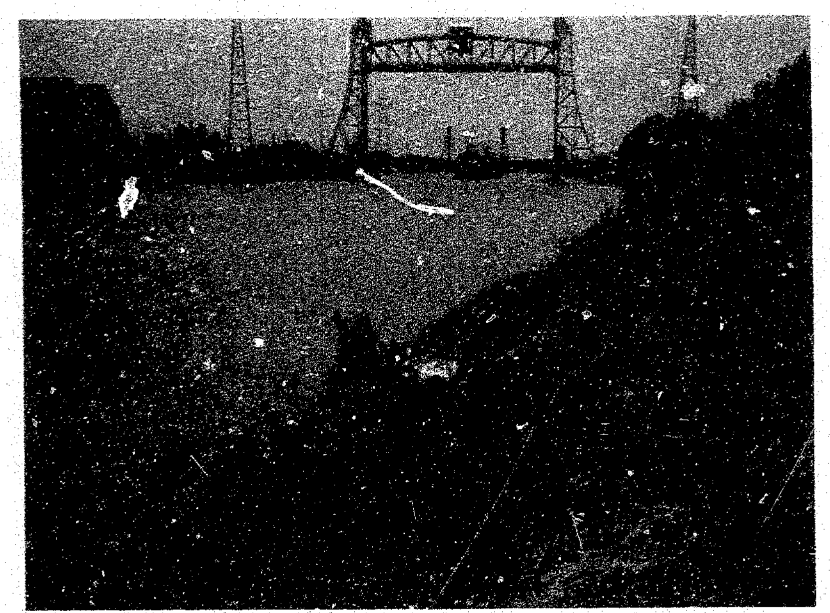


PHOTO 15

View of shallow slope failure on west bank of canal at approximately STA.993+00 looking south with Bridge No.14 in background.

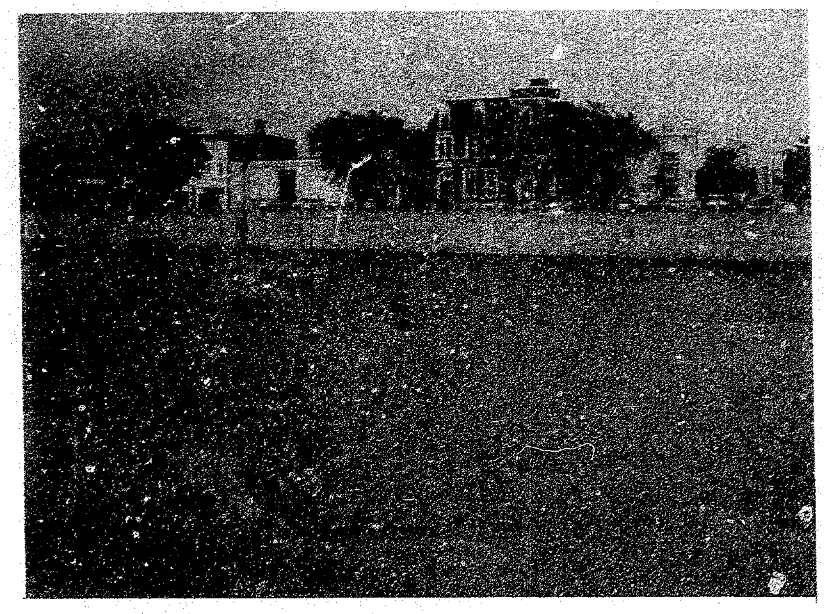


PHOTO 16

View of retaining wall on east side of canal at approximately STA.980+00.

GOLDER & ASSOCIATES

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 21

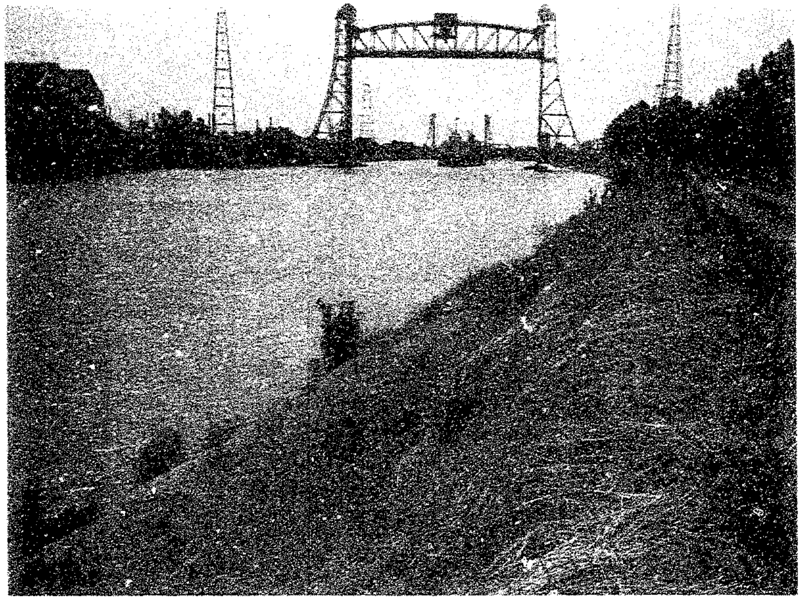


PHOTO 15

View of shallow slope failure on west bank of canal at approximately STA.993+00 looking south with Bridge No.14 in background.

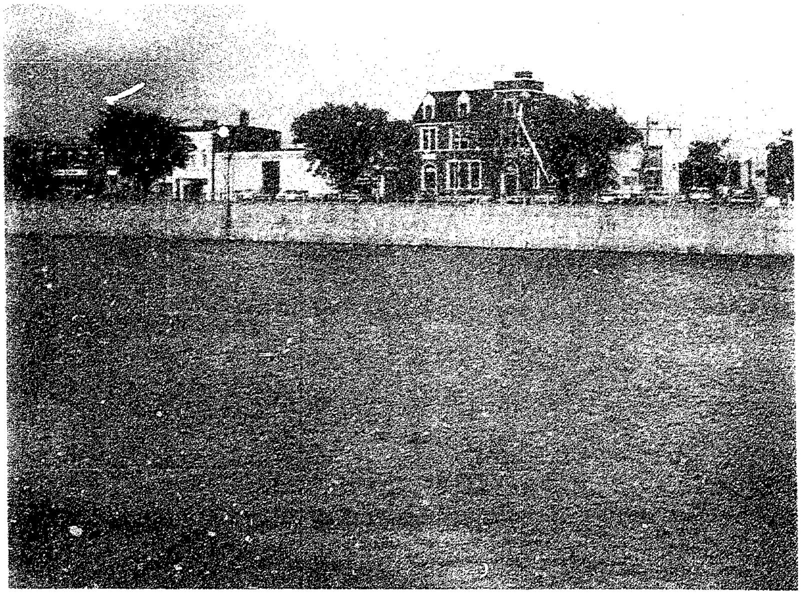


PHOTO 16

View of retaining wall on east side of canal at approximately STA.980+00.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 22

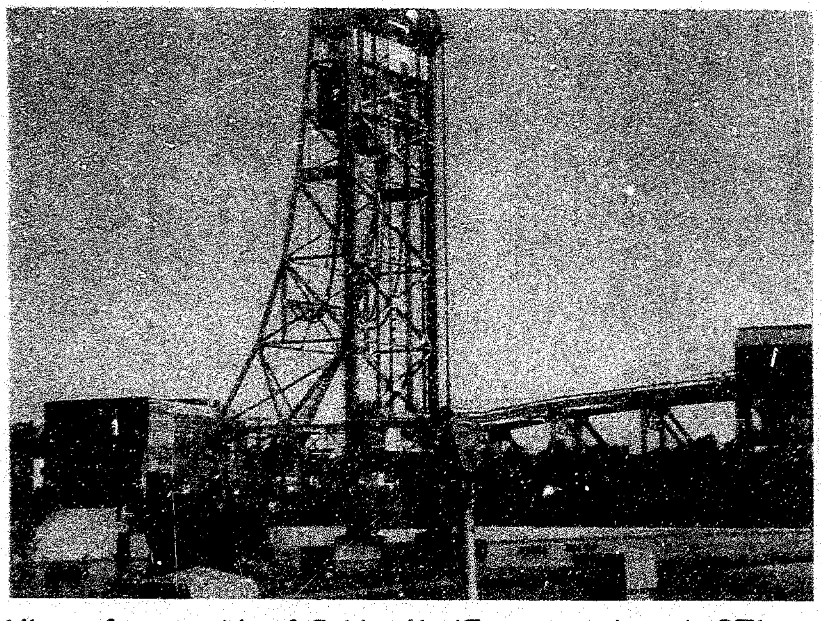


PHOTO 17

View of west side of Bridge No. 13 at approximately STA. 977+94 looking north-west.

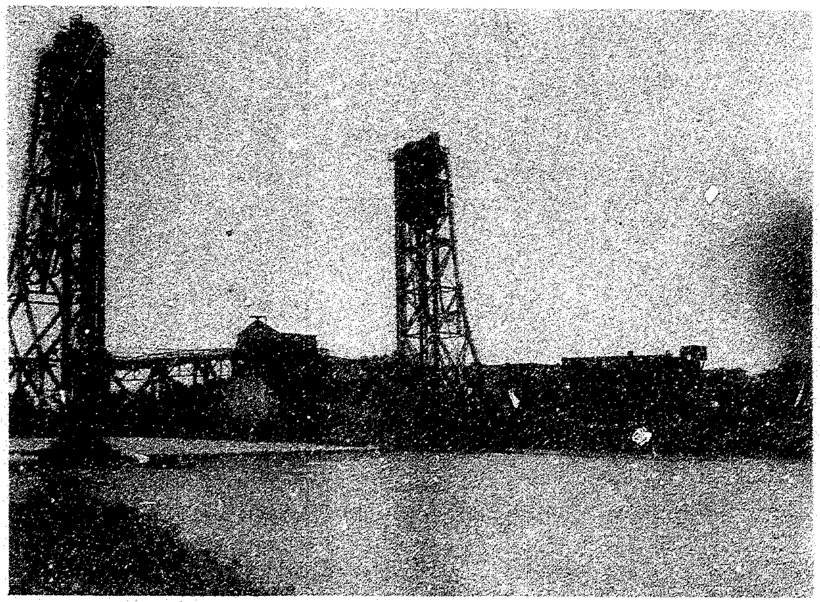


PHOTO 18

View of north side of of Bridge No. 13 and west retaining at approximately STA. 977+94 looking south-east.

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PHOTOGRAPHS OF SITE

FIGURE 22

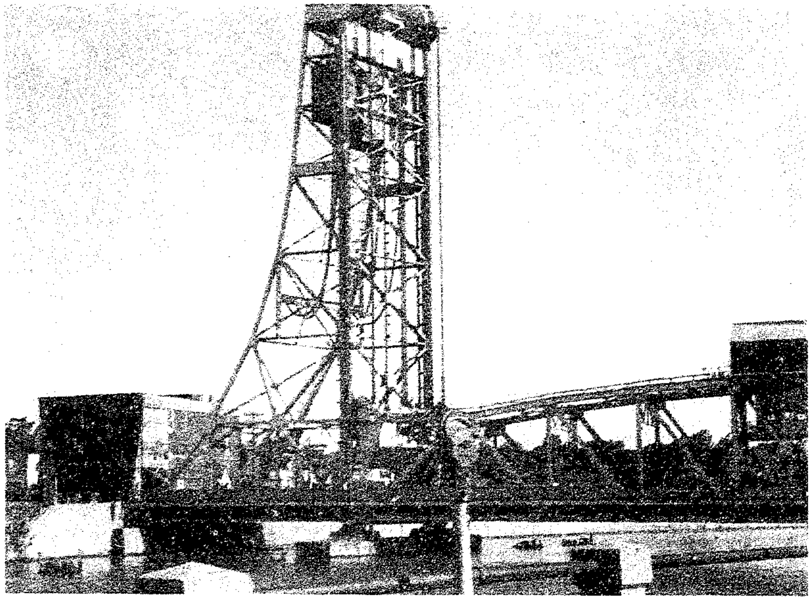

PHOTO 17

View of west side of Bridge No. 13 at approximately STA. 977+94 looking north-west.

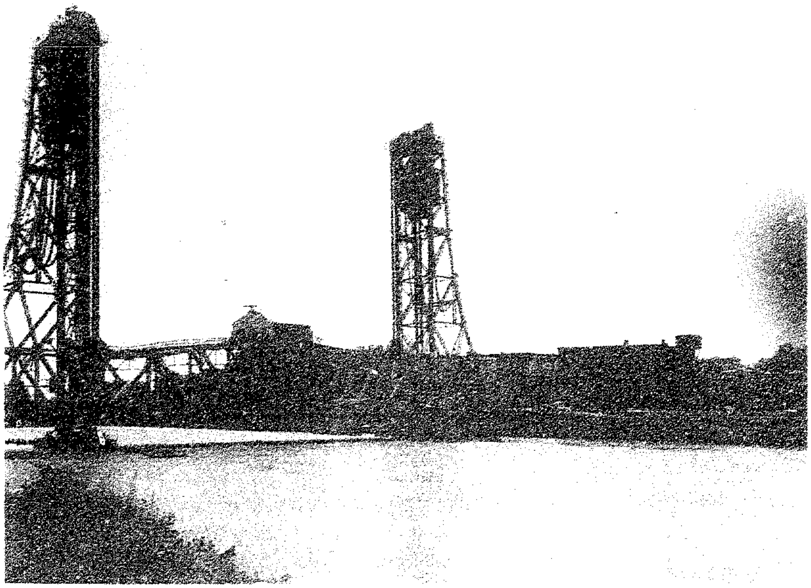

PHOTO 18

View of north side of of Bridge No. 13 and west retaining at approximately STA. 977+94 looking south-east.

PHOTOGRAPHS OF SITE

FIGURE 23

PROJECT No. 66093

PHOTO 19

View of Syphon at approximately STA.966+00 looking north.

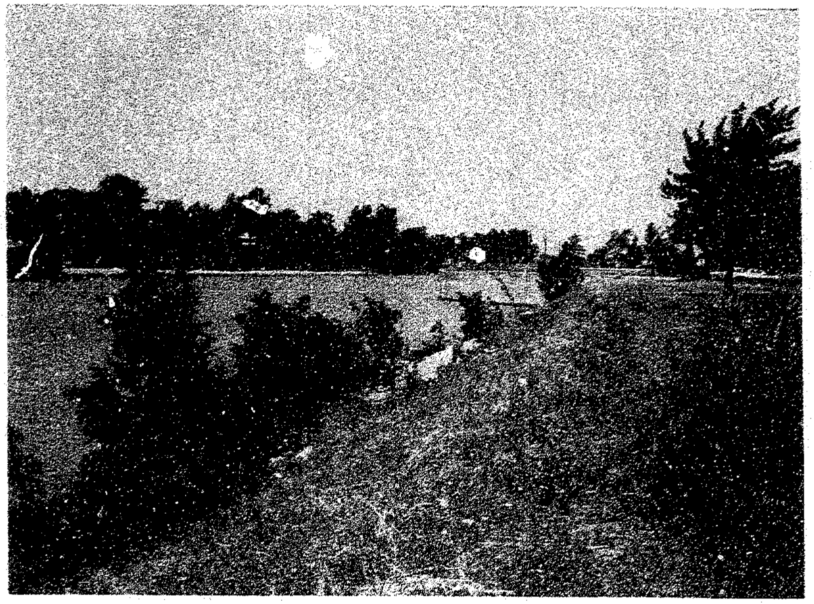

PHOTO 20

View of east branch of Syphon at approximately STA.966+00 looking north.

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PHOTOGRAPHS OF SITE

FIGURE 23

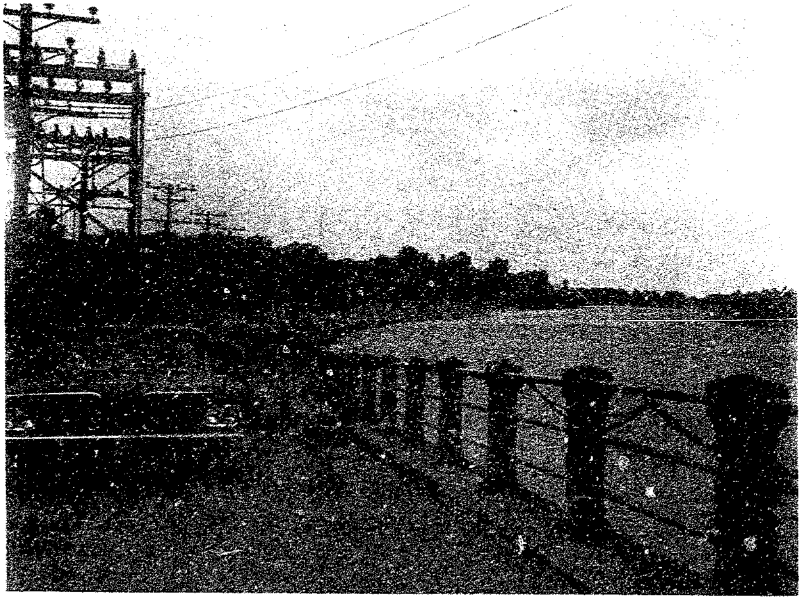


PHOTO 19

View of Syphon at approximately STA.966+00 looking north.

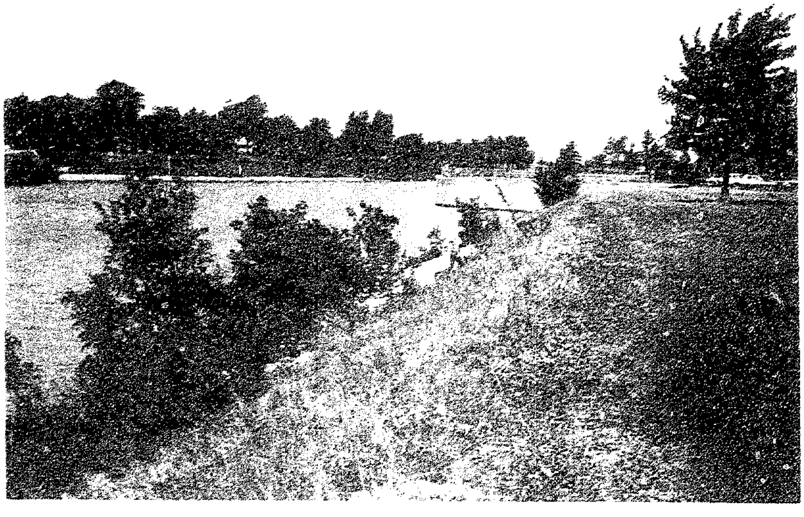


PHOTO 20

View of east branch of Syphon at approximately STA.966+00 looking north.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 24

PHOTO 21

View of east bank of canal showing dyke at approximately STA.939+00.

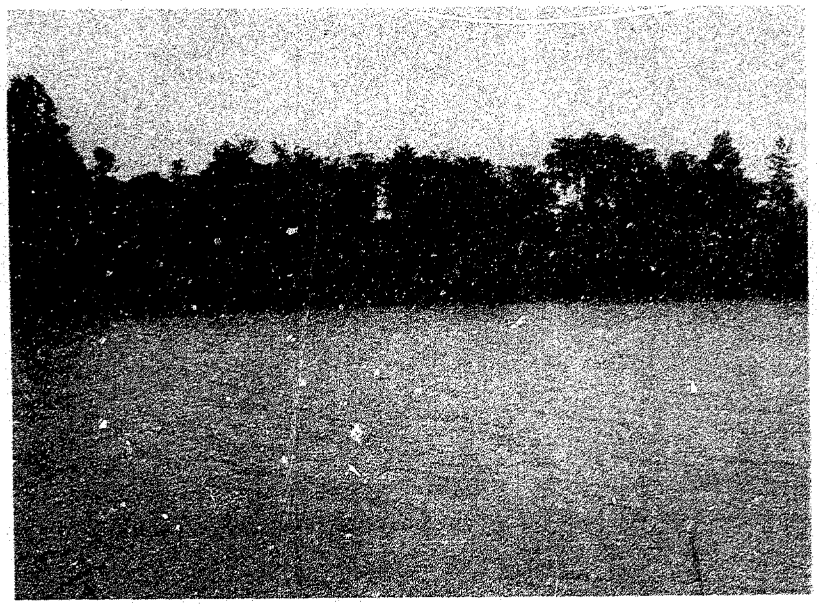


PHOTO 22

View of deep slope failure on west bank of canal at approximately STA.932+50.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 24

PHOTO 21

View of east bank of canal showing dyke at approximately STA. 939+00.

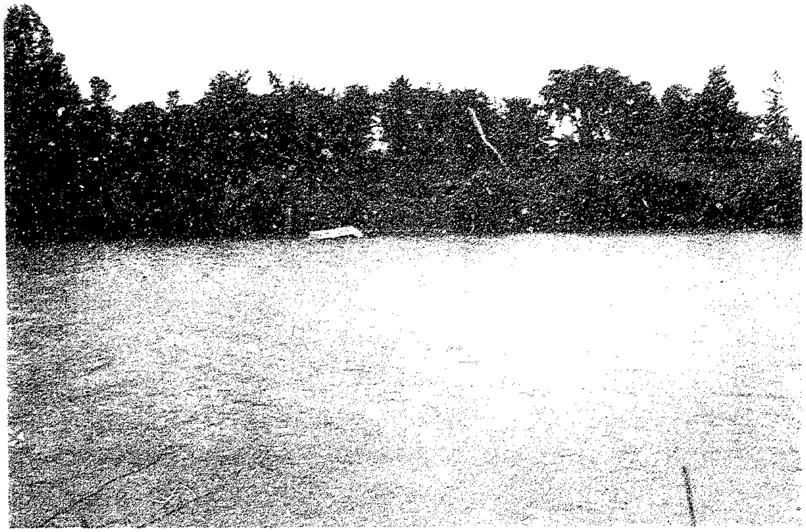


PHOTO 22

View of deep slope failure on west bank of canal at approximately STA. 932+50.

PROJECT No. ...

PHOTOGRAPHS OF SITE

FIGURE 25

PHOTO 23

View of shallow slope failure on west bank of canal at approximately STA. 914+00 looking south.

PHOTO 24

View of fill on east bank of canal at approximately STA.899+00

GOLDER & ASSOCIATES

PROJECT No. ___[illegible]

PHOTOGRAPHS OF SITE

FIGURE 25

PHOTO 23

View of shallow slope failure on west bank of canal at approximately STA. 914 + 00 looking south.

PHOTO 24

View of fill on east bank of canal at approximately STA. 899+00

PROJECT No.

PHOTOGRAPHS OF SITE

FIGURE 26

PHOTO 25

View of bermed west bank of canal at approximately STA.890+00

PHOTO 26

View of deep slope failure on east bank of canal at approximately STA.868+00.

PROJECT No. 66093

PHOTOGRAPHS OF SITE

FIGURE 26

PHOTO 25

View of bermed west bank of canal at approximately STA.890+00

PHOTO 26

View of deep slope failure on east bank of canal at approximately STA.868+00.

PHOTOGRAPHS OF SITE

FIGURE 27

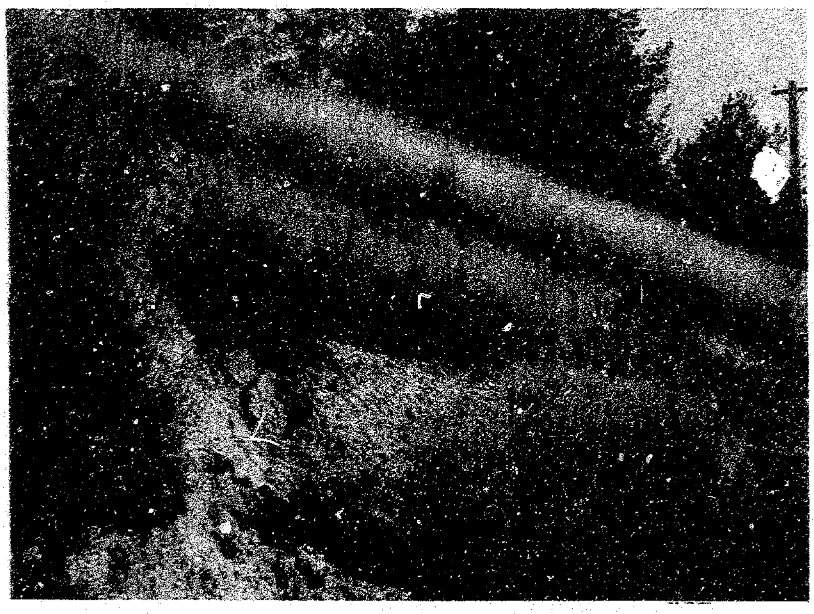


PHOTO 27

View of deep slope failure on east bank of canal at approximately STA.868+00.

PHOTO 28

View of spoil rock fill area on west bank of canal at approximately STA. 860+00.

PHOTOGRAPHS OF SITE

FIGURE 27

PHOTO 27

View of deep slope failure on east bank of canal at approximately STA.868+00.

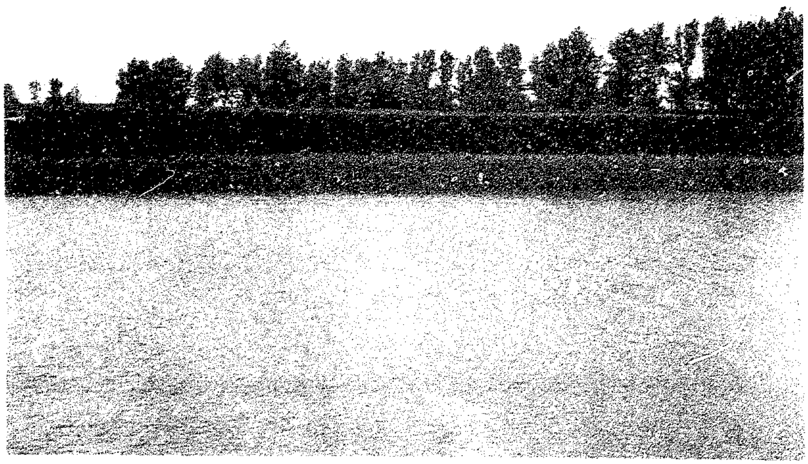
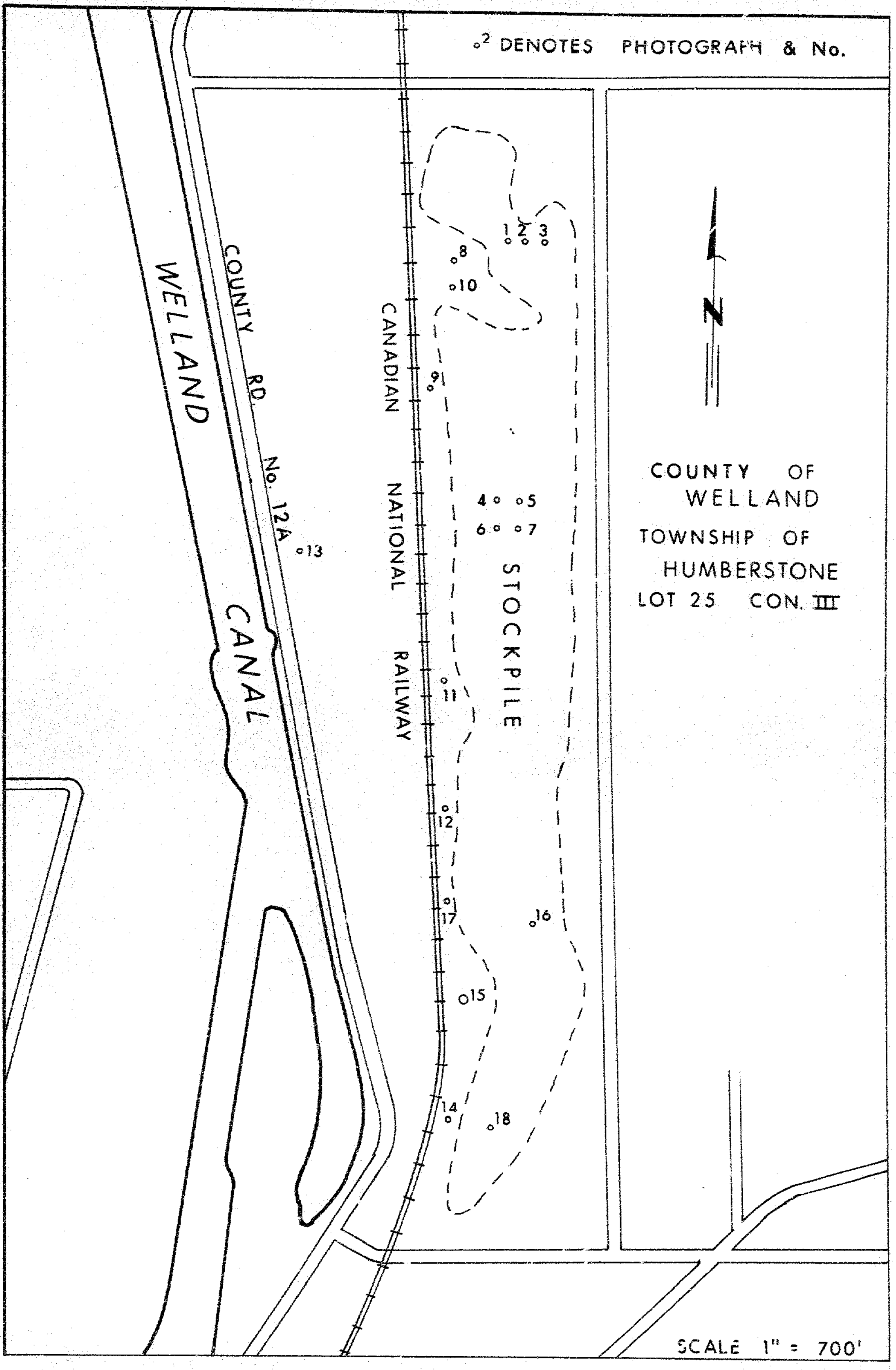


PHOTO 28

View of spoil rock fill area on west bank of canal at approximately STA. 860+00.

GOLDER & ASSOCIATES

PROJECT No.



∘2 DENOTES PHOTOGRAPH & No.
WELLAND CANAL
COUNTY RD. No. 12A
CANADIAN NATIONAL RAILWAY
STOCKPILE
1 2 3
8
10
9
4 5
6 7
13
11
12
17
16
15
14
18
N
COUNTY OF WELLAND
TOWNSHIP OF HUMBERSTONE
LOT 25 CON. III
SCALE 1" = 700'

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. B. R. Davis
Bridge Engineer
Bridge Division

Attn: Mr. S. McCombie

Our File Ref.

From: Foundation Section
Materials & Testing Div.
Room 107, Lab. Bldg.

Date: June 9, 1967

In reply to JUN - 9 1967

Subject: Soil Conditions within the Welland Canal

Sta. 850 + 00 to Sta. 1,100+00

Attached please find the report for the above mentioned site prepared and submitted by the consultant H. Q. Golder & Associates Ltd.

We have reviewed the report and found the contents satisfactory and self-explanatory. It is believed that this information is adequate at this stage of the project. Should the presently proposed scheme be adopted, some additional exploratory work will have to be carried out as suggested in the attached report. However, the proposed work is very limited and it is believed that it will not change the present concepts but will rather provide the information that should ultimately be required.

Should you have any queries in connection with this report, please feel free to contact this office.

AGStermac

A. G. Stermac
Principal Foundation Engineer

AGS:mt
Attach.

cc: Messrs.: B.R.Davis (2)
H.A.Tregaskes
D.W.Farren
G.K.Hunter (2)
H.Greenland
W.S.Melinyshyn
T.J.Kovich
B.A.Singh
St.Lawrence Seaway Authority

Foundation Files (2)
General Files

67 F212

(2) of (2)

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN
J. L. SEYCHUK

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
763-4103
767-9201

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS WITHIN CANAL

WELLAND CANAL

STA. 850+00 TO STA. 1,100+00

WELLAND ONTARIO

Distribution:

14 copies - Department of Highways, Ontario,
Toronto, Ontario.

3 copies - H.Q. Golder & Associates Ltd.,
Toronto, Ontario.

June, 1967 67028

TABLE OF CONTENTS

ABSTRACT

The results of an investigation to determine the subsoil conditions within the Welland Canal in the vicinity of the City of Welland, Ontario (between stations 850+00 and 1,100+00) is reported. The need for this investigation was discussed in our report 66093, dated January, 1967. This previous report presented the results of a study on the general feasibility of using the drained excavation of the section of the existing Welland Canal between Ramey's Bend and Bridge 12, now to be abandoned, as a route for Highway 406.

The water level in the canal along this reach is at about elevation 569 with the depth of water being about 30 to 33 feet; the top of the canal banks vary from elevation 580 to 600. In the northern section (north of about STA. 920+00) a stiff clayey silt up to 26 feet thick is encountered; this deposit is underlain in turn by layered silty clay (at some locations only) followed by a dense silty sand till. South of about STA. 930+00 the canal bottom is underlain by a firm to stiff interbedded silty clay up to 40 feet in thickness; in the southern portion of this section the silty clay is underlain by silt. Directly underlying the silty clay and/or the silt is the dense silty sand till encountered in the northern portion of the reach.

This investigation in general, confirms the stratigraphy along the west canal banks inferred from previous borings carried out by the St. Lawrence Seaway Authority, with the exception of the section between STA. 890+00 and 960+00. In this section the till surface is higher along the canal banks than beneath the canal, i.e. the subsoil seems to be dipping in an easterly direction.

INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario, (letter of authorization dated March 2, 1967) to carry out a soil investigation within the Welland Canal in the vicinity of the City of Welland, Ontario, namely between STA. 850+00 and STA. 1,100+00.

Our report No. 66093, dated January 1967, presented the results of a study on the general feasibility of using the drained excavation of the section of the existing Welland Canal between Ramey's Bend and Bridge 12, now to be abandoned as a route for Highway 406. In this report it was recommended that borings be put down within the canal section specified above. The purpose of this present investigation is twofold;

i) to determine the character of the canal bottom which, for example, if covered by soft or organic debris, may have a serious effect on the placement of fill.

and ii) to confirm the vertical and lateral extent of the overburden deposits inferred on Figure 2 in report No. 66093.

This report should be read in conjunction with the previous report.

PROCEDURE

The field work for this investigation was carried out between March 6 and 30, 1967. A total of 7 cased boreholes (No. C-1 to C-7) were put down in NX size, on about 5,000 foot centres, by 2 drum raft mounted diamond drill rigs. The borings, each of which was accompanied by a dynamic cone penetration test, were put down to depths ranging from 28 to 45 feet below canal bottom. In addition 20 dynamic cone penetration tests (No. C-8 to C-27) were put down between the borings on about 1,000 foot centres. The drilling equipment, including drillrigs, drum rafts, motor boats, anchors and steel cable, etc., were supplied and operated by the F.E. Johnston Drilling Company Limited.

The overburden was sampled with either a two inch diameter split spoon sampler or thin walled tube sampler. In situ field vane testing was also carried out within the cohesive strata. The field work was supervised throughout by an engineer from our staff.

A detailed log for each of the boreholes and dynamic cone penetration tests is given on the Record of Borehole sheets following the text of this report. The location of the borings and penetration tests, together with the inferred stratigraphy at borehole locations, are shown on Figures 1 and 2.

Samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of the laboratory testing are shown on the Record of Borehole sheets and on Figures 3 to 9, inclusive.

The borehole and penetration test locations were determined by our engineering personnel by reference to canal chainages marked on various concrete protective slabs and bridge structures flanking the canal. The elevation of the canal water level at the time of the investigation was provided by the St. Lawrence Seaway Authority. These elevations are referred to the same Geodetic datum as used in the previous investigation (report No. 66093). It is understood that a revised Geodetic datum will be introduced by the St. Lawrence Seaway Authority in the near future. Precautions, therefore, should be taken to ensure that the revised datum is not confused with the datum used in this report.

i) Conditions During Drilling Operations

At the beginning of the investigation ice sheets were located within the portion of the Welland Canal under investigation; the ice conditions were particularly severe in the northerly portion. Removal of ice was therefore required to position the rafts on the predetermined boring locations. Once the rafts were in place an effort was made to secure them with anchors placed within the canal bottom deposits. The flowing ice, however, made it impossible to steady the raft in this manner during the casing and sampling operations. Additional fixity was provided by securing the raft with steel cables running from the corners of the raft to shore; this anchorage proved satisfactory. During the first part of the week of March 13 the temperature dropped abruptly causing additional quantities of ice to be formed. This ice greatly hindered the drilling operations so much that it was necessary to terminate sampling operation on March 15 and again on March 18.

Following the above period the weather turned milder and the ice disappeared from the portion of the canal under investigation. The drilling programme was then completed.

SITE AND GEOLOGY

The portion of the Welland Canal under investigation is between Stations 850+00 to 1,100+00; this portion of the canal is located within the townships of Crowland and Thorold, County of Welland.

The city of Welland is located approximately in the centre of the section. The canal is cut in a broad, relatively flat clay plain, which varies in surface elevation from about elevation 580 to 600. The bottom of the canal along this reach was found to be at about elevation 536, i.e. the canal banks are approximately 44 to 64 feet high. The width of the canal at the crest and at canal bottom is about 350 ft. and 200 ft. respectively. The water level in the canal is about elevation 569 (some 31 to 34 ft. of water). The Welland River, which flows from west to east adjacent to the canal, is at a normal water level of about elevation 562 with the high water level being about 569.

From available geological information (Chapman, Putman, 1951) and inspection of the area, it is known that the overburden consists of thick deposits of silty clay and clayey silt overlying till, physiographically known as Haldiman Till.

The till which is of Wisconsin age, was covered by the silty clay, which is a lacustrine deposit laid down in glacial Lake Warren. The glacial lake phase was possibly interrupted by two major retreats of the ice front which resulted in distinctively different deposits, non-stratified relatively silty homogeneous deposits laid down with the ice front fairly close, and heavily stratified very clayey deposits laid down when the ice front had retreated some distance. All of the lacustrine deposits are relatively soft and possibly only lightly preconsolidated, except the upper 30 to 40 feet of the overburden outside the canal proper which is desiccated. The total thickness of the overburden generally varies from 100 to 120 feet.

The broad clay plain is bounded to the north by the Niagara Escarpment which steps down toward Lake Ontario. To the south the plain is bounded by the toe of the Onondaga cuesta.

The water shed in the area is controlled by the Onondaga cuesta, which though quite low lying close to the shore of Lake Erie, nevertheless forces the drainage to the north and east. In general the drainage in this primarily flat heavy clay area is quite poor.

The bedrock in the area is Palaeozoic. The beds dip slightly southward under Lake Erie. The bedrock, which generally varies in elevation from 460 to 500, is a massive dolomitic limestone of the Salina formation, Devonian period. There are numerous siltstone and calcareous shale interbeds within the bedrock. In addition the bedrock contains numerous gypsum inclusions from hairline thickness to as much as 12 inches.

SOIL CONDITIONS

The detailed soil stratigraphy encountered by the borings is given on the Record of Borehole sheets. The inferred stratigraphy along the portion of the canal investigated is also shown on Figure 1 (STA. 850+00 to 970+00) and on Figure 2 (STA. 970+00 to 1,100+00). The engineering properties of the subsoil are presented on Figures 3 to 9 inclusive.

The subsoil conditions encountered in this investigation can be sub-divided into two distinct zones, namely the northern section where the upper deposit is a clayey silt till and the southern section where the upper deposit is an interbedded silty clay. The two sections will be discussed separately below.

i) Northern Section (approximately STA.850+00 to 920+00)
(boring coverage BH's C-5 and C-6, Pen Test C-22 to C-27)

a) Clayey Silt Till

Directly underlying the canal bottom in this section is a deposit of relatively homogeneous reddish-brown clayey silt till with some sand and gravel throughout, the upper 1 or 2 feet of which are in a softened condition. The maximum encountered thickness of the deposit was 26 feet (BH C-6). Occasional sand and silt partings and seams up to 1/2 inch thick occur randomly throughout the deposit. Typical grading curves for the clayey silt till are shown on Figure 3. The total unit weight of the till, as determined from two laboratory tests, was found to be about 138 lb/cu. ft.

Atterberg limit tests were carried out on samples of the till. These results are shown on the Record of Borehole sheets and are summarized on Figure 8. The test results indicate that the liquid limit varies from 17 to 27 while the plasticity indices vary from 6 to 12; the corresponding natural water content ranges from about 2 percent below to 3 percent above the plastic limit. These results are typical of inorganic glacial clays and silts of low plasticity. Based on the laboratory testing the activity of the deposit was found to range from approximately 0.3 to 0.6 being generally about 0.4. Activities of this order of magnitude are indicative of inactive soils.

Standard cone penetration tests were carried out within the deposit; the results are summarized on the Record of Borehole sheets. The results indicate that the 'N' values range from about 10 blows/ft. to 33 blows/ft. being generally about 20 blows/ft. Based on the 'N' values it is estimated that the consistency of the deposit varies from stiff to very stiff. Two undrained triaxial tests carried out on samples of the till corroborate the range in consistency given above.

Underlying the clayey silt till at borehole C-5 is a deposit of stiff reddish-brown to grey layered silty clay with some sand and a trace of gravel; the thickness of this deposit is about 10 feet. A grading curve for a representative sample of the deposit is shown on Figure 5. The results of an Atterberg limit determination indicates that the silty clay is inorganic and of medium plasticity with the natural water content at about the liquid limit.

b) Sandy Silt Till

Underlying the layered silty clay or the clayey silt till, is a dense to very dense sandy silt till with some gravel and a trace of clay throughout. Both borings were terminated within this deposit. From previous experience in the area it is known that boulders occur within this deposit particularly with depth. Localized seams and layers of silt and sand up to 1 foot in thickness occur throughout. Grading curves for representative samples of the till (obtained using 1½ inch I.D. sampling equipment) are given on Figure 7. The in situ water content of the till deposit was found to be about 12 percent.

ii) Southern Section (approximately STA. 930+00 to 1,[illegible]00+00) (boring coverage BH's C-1, C-2, C-3, C-4, C-7 and Pen. Tests C-8 to C-21)

a) Interbedded Silty Clay

Directly underlying the canal bottom in this section is a stratum of reddish-brown to grey interbedded silty clay and/or stratified clay, the upper 1 to 2 feet of which is in a softened condition. The thickness of this stratum varies from 22 feet at borehole C-2 to 42 feet at borehole C-3. The lower portion of the stratum, at some of the boring locations, was noted to be varved. The thickness of individual layers within the stratified clay varied widely to a maximum of 3 inches approximately. Occasional partings and seams of silt and sand up to 1/2 inch thick are present throughout the stratum. Grading analyses for representative samples of the silty clay are shown on Figures 4 and 5.

Atterberg limit tests were carried out on samples of the interbedded silty clay. The results are shown on the Record of Borehole sheets and are also summarized on Figure 8. The test results are summarized in tabular form below:

Liquid Limit (w_L) Range (Average)	Plasticity Index (I_P) Range (Average)	Liquidity Index (I_L) Range (Average)
24 to 68 (44)	7 to 44 (24)	0.3 to 1.3 (0.7)

These results, which are typical of inorganic glacial clays, indicate that the silty clay stratum ranges from low to high plasticity generally being within the medium plastic range. The activity of the silty clay stratum varies from about 0.3 to 0.6 being typically about 0.35. Values of this order of magnitude are representative of inactive silty clays. The total unit weight of the silty clay was found to vary from about 115 lb/cu.ft. to 130 lb/cu.ft. being typically about 120 lb/cu.ft.

The undrained shear strength of the silty clay was measured by in situ vane testing in the field and by laboratory undrained triaxial tests. The results of these tests are plotted on the Record of Borehole sheets and are summarized on Figure 9 as a plot of shear strength vs. elevation. The undrained shear strength in the silty clay varies from about 700 lb/cu.ft. near the surface to 1,500 lb/cu.ft. with depth. In general the triaxial tests gave lower shear strength values than the in situ field vane tests. This is probably due to unavoidable disturbance during the sampling operations. The relatively high strains to failure during testing (greater than 15 percent) were a definite indication that some of the samples were disturbed to some degree. It is, therefore, concluded that the in situ vane test results provide a better indication of the in situ consistency of the stratum. Based on the above it is estimated that the consistency of the silty clay varies from firm to stiff.

The sensitivity of the silty clay, as measured by several field vane tests, is about 2 to 4; the stratum is thus moderately sensitive to disturbance.

The previous investigations in the vicinity indicated that the silty clay stratum is normally consolidated to lightly overconsolidated below about elevation 535 (canal bottom).

The laboratory test results indicate that the average plasticity index of the stratum is about 24; for this value the $^{Cu}/Po$ ratio is about 0.2 (Reference Skempton, 1957). Based on the undrained shear strength profile shown on Figure 9 and a $^{Cu}/Po$ of 0.2, it is estimated that the silty clay stratum (below canal bottom) could be preconsolidated in excess of existing overburden pressure by as much as 1 ton/sq.ft.

b) Silt

In the southern end of the portion of the canal under investigation (approximately south of STA. 1020+00) the silty clay stratum is underlain by a deposit of loose to compact reddish-brown silt with some sand and a trace of clay and gravel. The encountered thickness of this deposit at borehole C-1 and C-2 was 10 feet and 13 feet, respectively. Occasional layers of clayey silt up to 1/2 inch thick are located randomly throughout the deposit. Grading curves for two samples of the silt are shown on Figure 6.

c) Sandy Silt Till

Underlying the interbedded silty clay and/or the silt deposit is a dense to very dense reddish brown to grey brown silty sand and gravel till. This till sheet is the same deposit encountered in the northern portion of the canal section and as such is the only deposit that is continuous along the section investigated. The physical description and engineering properties of the deposit were discussed in detail above.

COMMENTS ON SOIL CONDITIONS

The borings put down did not encounter any undesirable surficial canal bottom deposits, such as deposits of organic origin. The upper 1 or 2 feet, although composed of the natural subsoil, is in a softened condition. It is considered however, that sub-excavation will not be required prior to placement of fill on the canal bottom.

In general the borings corroborate the stratigraphy inferred from the St. Lawrence Seaway Authority borings previously put down along the west bank of the canal. There is, however, one area where a significant deviation occurs as discussed below.

In report No. 66093 the section between STA. 890+00 and 960+00 was designated as section II; it was in this section that some discrepancies were noted. The present investigation indicated that the upper interbedded silty clay extended as far north as about STA. 930+00. The borings put down previously on the west canal bank indicate that the silty clay pinches out somewhere in the vicinity of STA. 960+00; it is encountered again, however, north of about STA. 930+00 (refer to Figure 2, report 66093). There is also a variation in the vertical extent of the deposits as encountered in the canal banks and beneath the canal. The boundary between the surficial deposits and the basal granular till occurs at a higher elevation on the canal banks. This would seem to indicate that the subsoil, particularly the till, dips steeply from the west to the east over the major portion of this section. It should be noted that the consistency and/or relative density of the deposits are within about the same range as presented in the previous report.

IMPLICATION OF PRESENT INVESTIGATION

As discussed in report 66093 the area of potential major instability of the canal banks, following drawdown, will be within that portion of the canal located in the downtown part of the city of Welland (STA. 967+00 to 1,050+00). This area is located within section III where the interbedded silty clay is present. The vertical and lateral extent as well as the engineering properties of this stratum previously assumed were corroborated by this investigation.

The area north of this section, namely sections I and II are not as critical with respect to stability since,

(a) the subsoil is more competent;
(b) no important structures are located immediately adjacent to the canal banks.

This investigation, however, pointed out the fact that some variation in the subsoil conditions occur in section II (STA. 890+00 to 960+00). In this section the till beneath the canal was encountered at a lower elevation than previously assumed. The cohesive surficial deposits beneath the canal, although not as competent as the till, do have an average consistency in the stiff range. Based on this consistency and taking into account point (b) above it is considered that the conclusions drawn in the previous report still apply, namely that the stability of the canal banks in this section following drawdown should not markedly be affected.

In summary it is considered that this investigation generally confirms the conclusions presented in the previous report regarding the feasibility of placing a highway within the confines of the drained canal. If the scheme is adopted it would be advisable to put down 2 additional borings between STA. 890+00 and 960+00 to further confirm the variation in the till elevation along this reach. This could be carried out as part of the additional boring programme outlined in section (iv) of the "Conclusions and Recommendation" section of report 66093.

for. B.T. Darch, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

V. Milligan, P. Eng.

BTD:VM:je
67028
June, 1967.

REFERENCES

CHAPMAN, L.J. and PUTNAM D.F., 1951 - The Physiography of Southern Ontario. University of Toronto Press, Toronto.

GOLDER, H.Q. & ASSOCIATES LTD., 1967 - Report - "Feasibility Study, Portion of Proposed Highway 406 within Welland Canal". Department of Highway, Ontario - Report No. 66093, January, 1967. Unpublished.

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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPES

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

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, blows/ft.*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	c_u, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer[1]
Q	undrained triaxial[2]
R	consolidated undrained triaxial[2]
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$

e = base of natural logarithms 2.7183

$\log_e a$ or $\ln a$, natural logarithm of a

$\log_{10} a$ or $\log a$, logarithm of a to base 10

t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

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

(b) *Consistency*

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

(c) *Permeability*

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

(d) *Consolidation (one-dimensional)*

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

(e) *Shear strength*

τ_f	shear strength	
c'	effective cohesion intercept	in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	effective angle of shearing resistance, or friction	
c_u	apparent cohesion*	in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
ϕ_u	apparent angle of shearing resistance, or friction	
μ	coefficient of friction	
S_t	sensitivity	

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

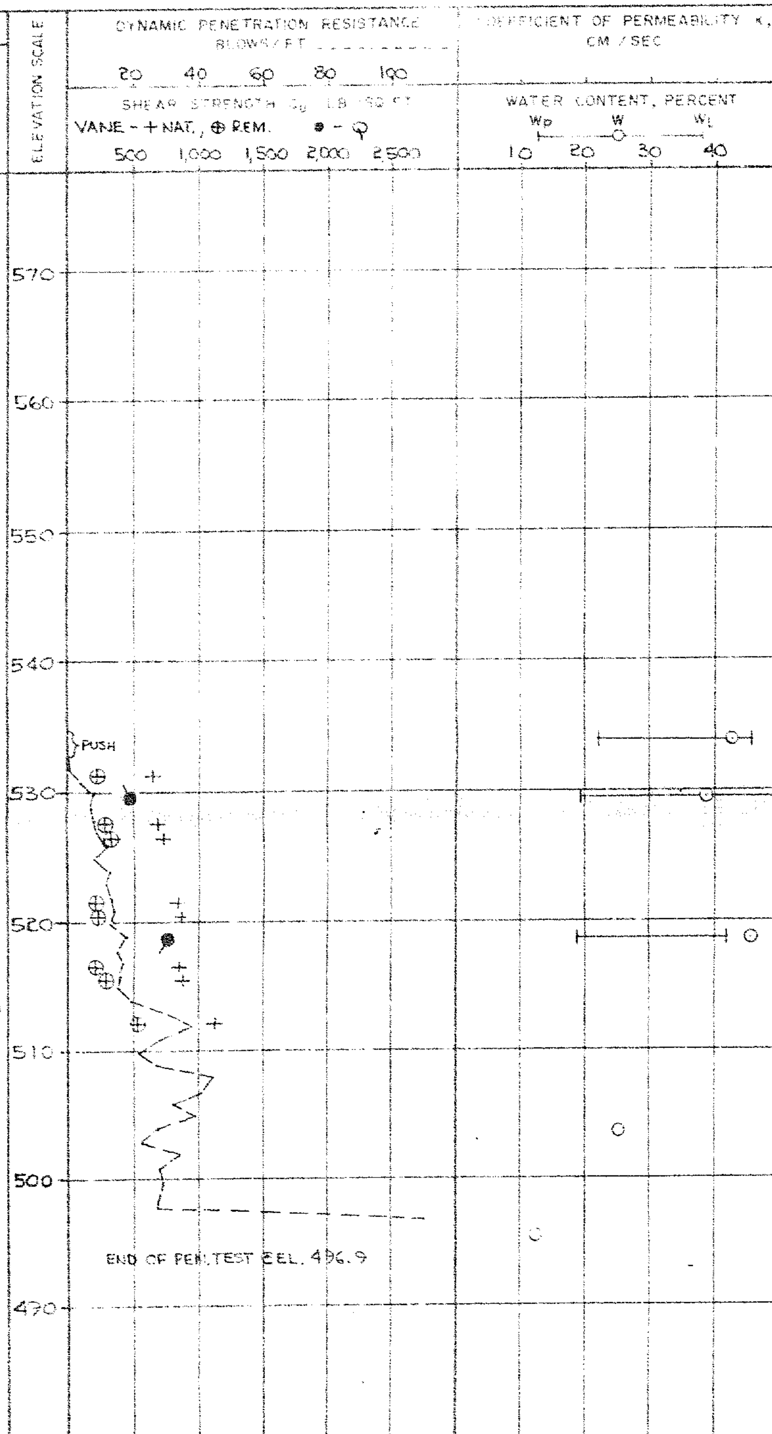
RECORD OF BOREHOLE C-1

LOCATION: STA. 1099 + 60 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 10, 11, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING — BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	ADDITIONAL LAB. TESTING
569.5	CANAL LEVEL				
0.0					
534.9	CANAL BOTTOM				
34.6	FIRM TO STIFF REDDISH - BROWN INTERBEDDED SILTY CLAY TRACE OF SAND AND GRAVEL. VARVED BELOW ABOUT EL. 514. (UPPER 1 TO 2 FEET IN A SOFTENED CONDITION)	1	2" D.O.	1	H
		2	2" T.O.	Pm	γ=117
		3	2" T.O.	Pm	
		4	2" T.O.	Pm	H, γ=115
		5	2" T.O.	Pm	
507.9		6	2" D.O.	8	
61.6	LOOSE REDDISH - BROWN SILT WITH SOME SAND AND A TRACE OF GRAVEL	7	2" D.O.	4	MH
498.3		8	2" D.O.	7	
71.2	VERY DENSE GREY-BROWN SILTY SAND AND GRAVEL WITH A TRACE TO SOME CLAY (TILL)	9	2" D.O.	52	MH
494.9 / 74.6	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED S.P.D.

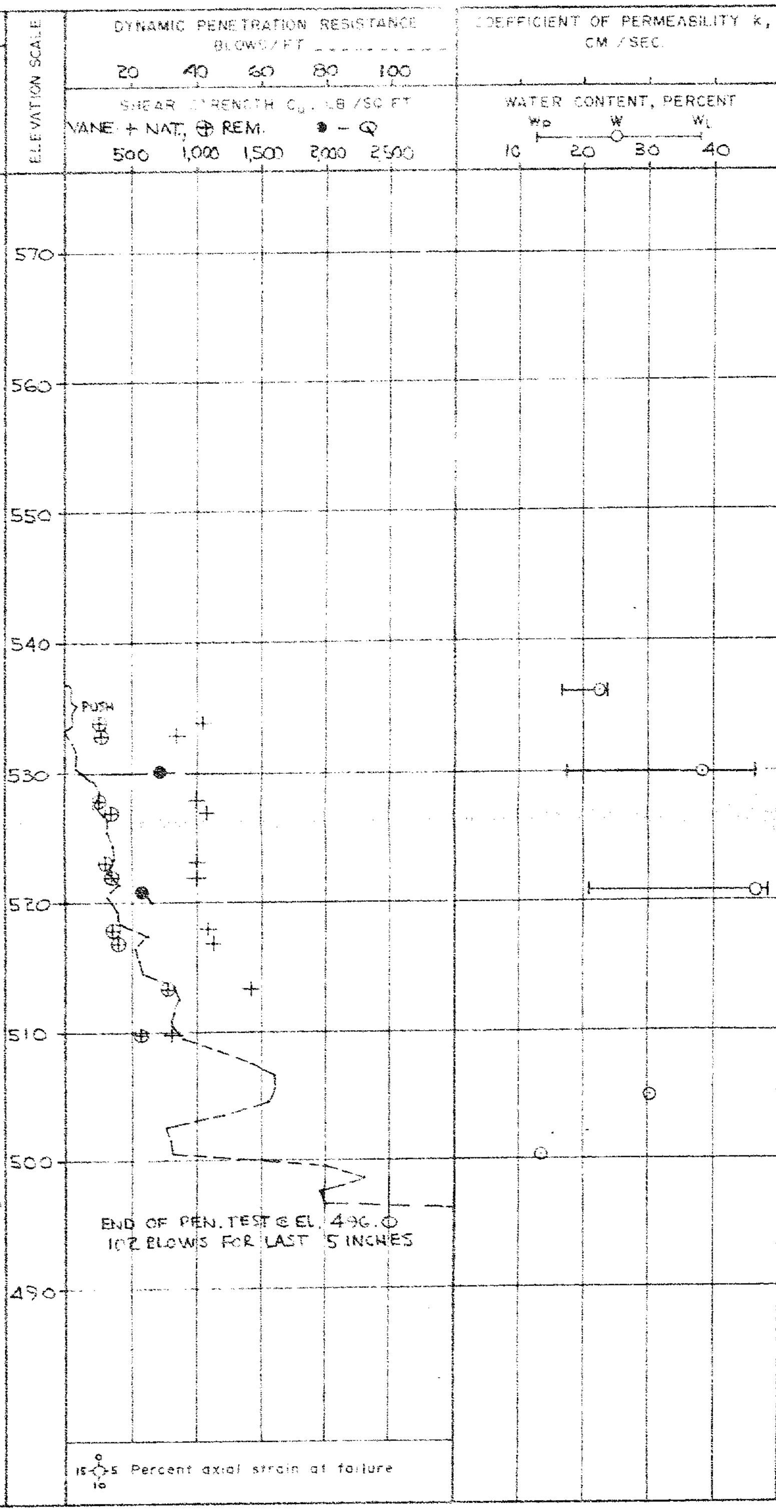
RECORD OF BOREHOLE C-2

LOCATION: STA. 1052+50 - ℄ CANAL, See Figure 1
BORING DATE: MARCH 13-14, 1967
DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING
BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES
PEN TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	ADDITIONAL LAB. TESTING
568.4	CANAL LEVEL				
0.0					
536.9	CANAL BOTTOM				
31.5	FIRM TO STIFF REDDISH-BROWN INTER-BEDDED SILTY CLAY TRACE OF SAND AND GRAVEL (UPPER 2 FEET IN A SOFTENED CONDITION)	1	2" D.O.	4	MH
		2	2" T.O.	Fm	γ=118
		3	2" T.O.	Pm	
		4	2" T.O.	Fm	γ=117
514.9		5	2" T.O.	Fm	
53.5	COMPACT REDDISH-BROWN SILT TO SILT AND SAND WITH A TRACE OF CLAY AND GRAVEL	6	2" D.O.	8	
		7	2" D.O.	19	H
502.0					
66.4	DENSE TO VERY DENSE REDDISH-BROWN SILTY SAND WITH SOME GRAVEL AND A TRACE OF CLAY (TILL)	8	2" D.O.	38	MH
496.0		9	2" D.O.	>100	
72.4	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED R.T.D.

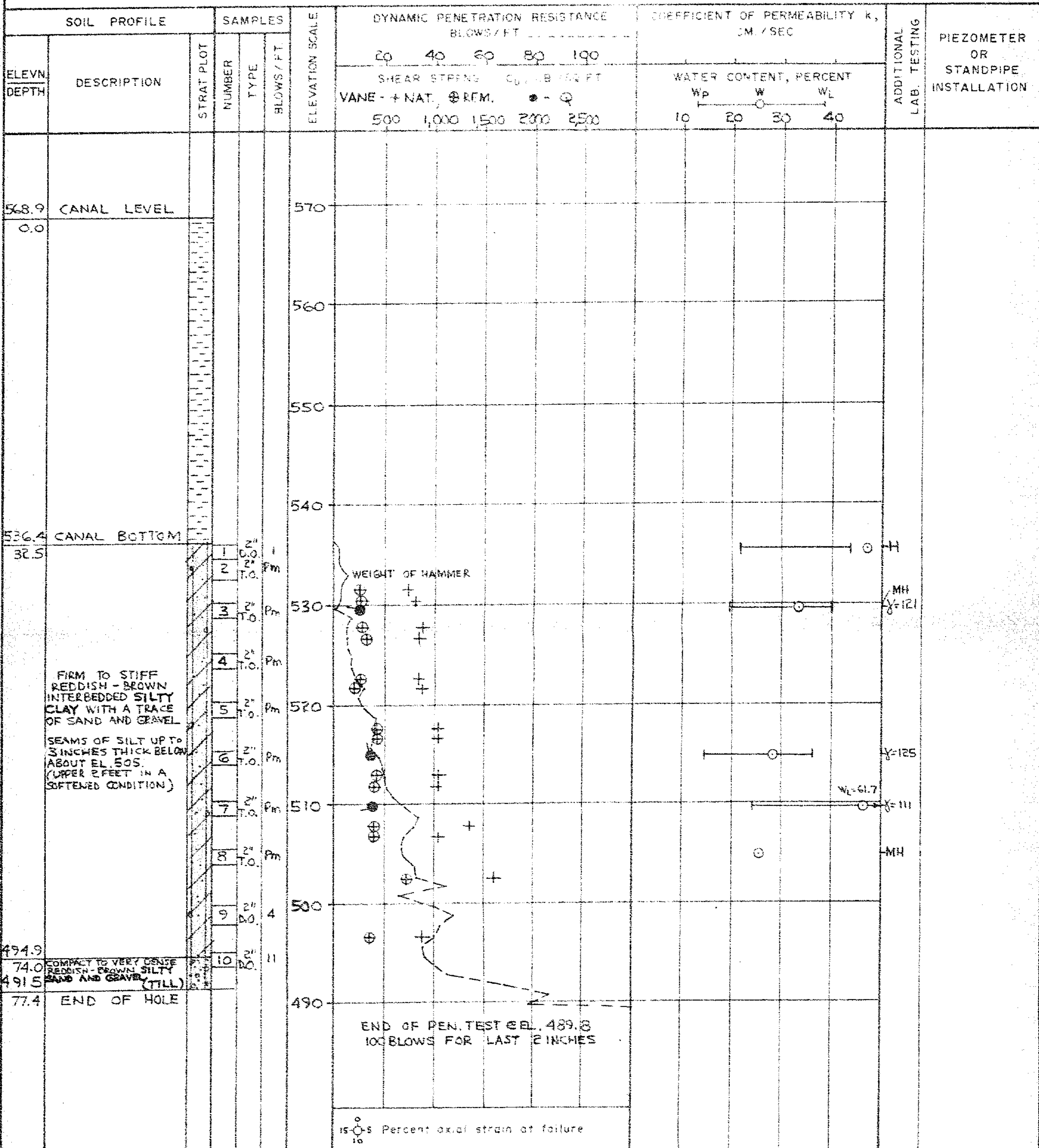
PROJECT No. 67028

RECORD OF BOREHOLE C-3

LOCATION STA. 1000 + 00 - ℄ CANAL See Figure 1 BORING DATE MARCH 16-17, 1967 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED [illegible]

PROJECT No. 67028

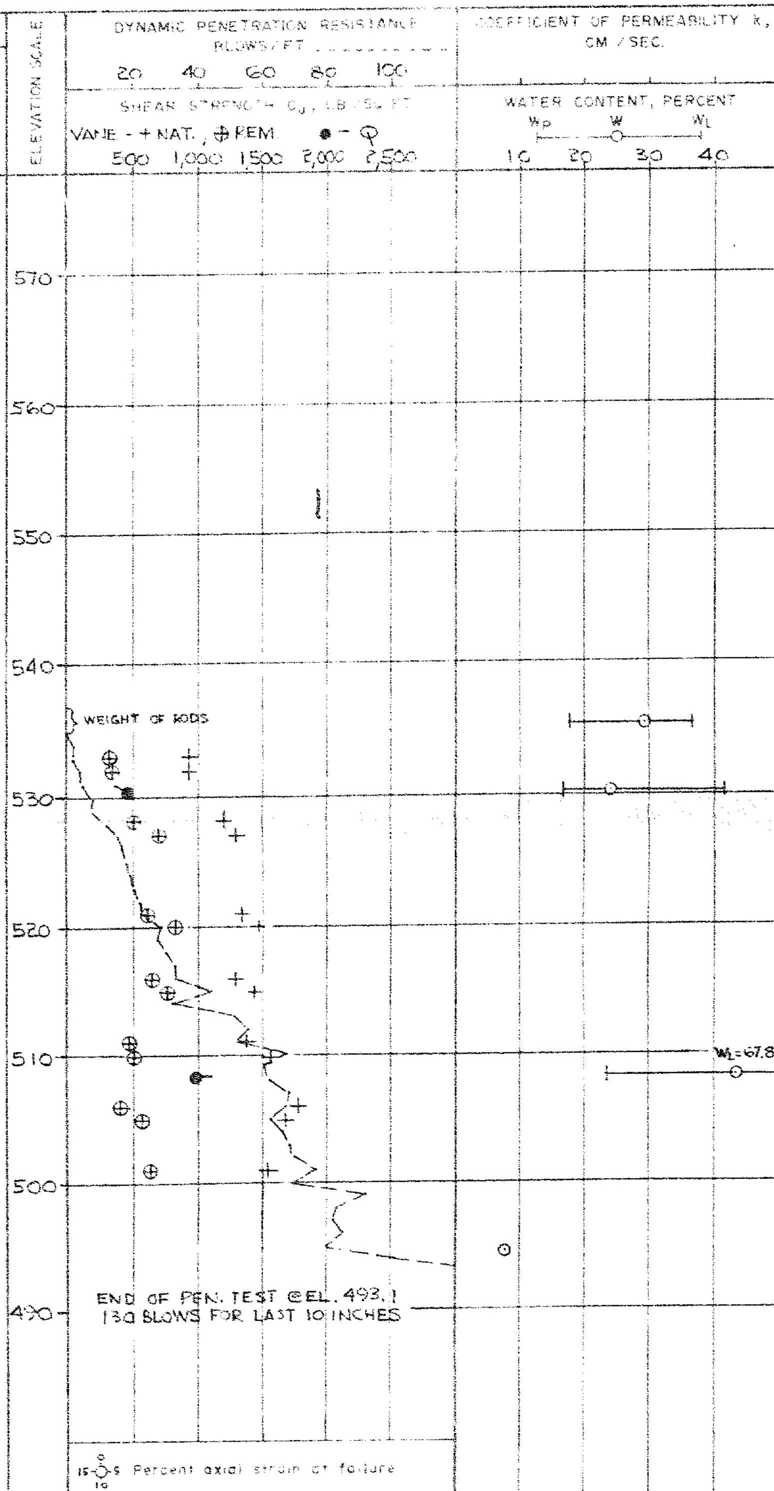
RECORD OF BOREHOLE C-4

LOCATION STA. 950+50 - ℄ CANAL See Figure 1 | BORING DATE MARCH 16-17, 1967 | DATUM GEODETIC

BOREHOLE TYPE WASH BORING | BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB DROP 30 INCHES | PEN. TEST HAMMER WEIGHT 140 LB DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FT	ADDITIONAL LAB. TESTING
568.9	CANAL LEVEL				
0.0					
536.8	CANAL BOTTOM				
32.1	FIRM TO STIFF REDDISH-BROWN & GREY INTERBEDDED SILTY CLAY TRACE TO SOME SAND AND GRAVEL. OCCASIONAL THIN SEAMS OF FINE SAND THROUGHOUT. VARVED BELOW ABOUT EL. 504 (UPPER 2 FEET IN SOFTENED CONDITION)	1	2" D.O.	Pm	MH
		2	2" T.O.	Pm	MH, γ=131
		3	2" T.O.	Pm	
		4	2" T.O.	Pm	
		5	2" T.O.	Pm	
		6	2" T.O.	Pm	γ=111
		7	2" T.O.	Pm	
498.5 / 70.4	COMPACT TO VERY DENSE REDDISH-BROWN SILTY SAND WITH SOME GRAVEL AND CLAY (TILL)	8	2" D.O.	19	
		9	2" D.O.	98	MH
493.9 / 75.0	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

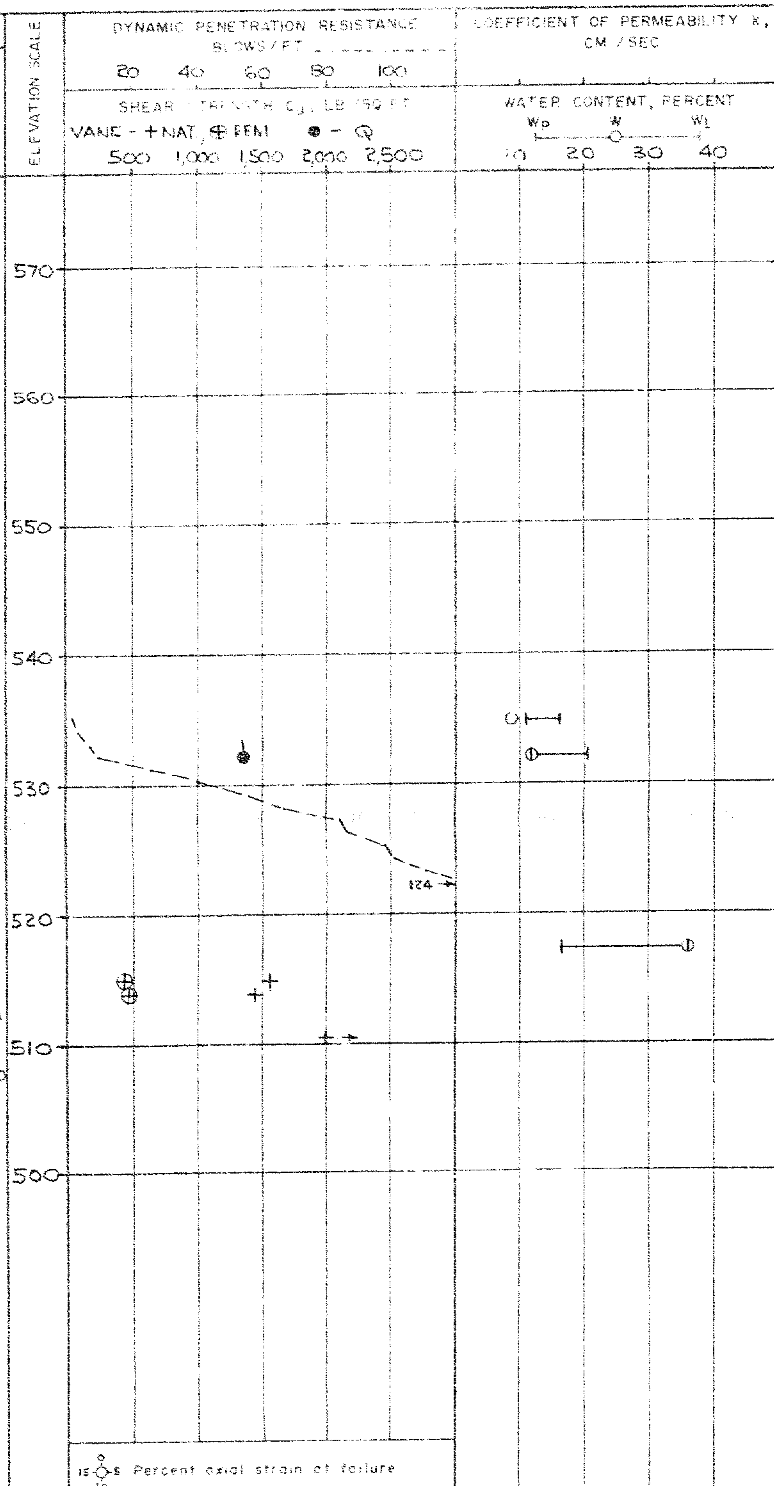
CHECKED R.R.

PROJECT No. 67028

RECORD OF BOREHOLE C-5

LOCATION: STA. 899+50 - ℄ CANAL, See Figure 1
BORING DATE: MARCH 13-14, 1967
DATUM: GEODETIC
BOREHOLE TYPE: WASH BORING
BOREHOLE DIAMETER: NX CASING
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	ADDITIONAL LAB. TESTING
568.4 / 0.0	CANAL LEVEL				
535.9 / 32.5	CANAL BOTTOM	1	2" D.O.	12	MH
	STIFF TO VERY STIFF REDDISH - BROWN **CLAYEY SILT** WITH SOME SAND AND GRAVEL (TILL)	2	2" T.O.	15	MH, γ=138
		3	2" D.O.	19	
		4	2" D.O.	12	
520.3 / 48.1	STIFF REDDISH - BROWN TO GREY LAYERED **SILTY CLAY** WITH SOME SAND AND A TRACE OF GRAVEL	5	2" D.O.	9	MH
		6	2" D.O.	Pm	
510.3 / 58.1	VERY DENSE GREY **SANDY SILT** WITH SOME CLAY AND GRAVEL (TILL)	7	2" D.O.	>100	
507.6 / 60.8	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED B.D.

RECORD OF BOREHOLE C-6

LOCATION STA. 850+50 - ℄ CANAL See Figure 1

BORING DATE MARCH 10-11, 1967 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	ADDITIONAL LAB. TESTING
569.5	CANAL LEVEL				
0.0					
537.3	CANAL BOTTOM				
32.2	STIFF TO VERY STIFF REDDISH-BROWN CLAYEY SILT WITH SOME SAND AND GRAVEL (TILL)	1	2" D.O.	33	MH
		2	2" D.O.	28	
		3	2" D.O.	29	MH
		4	2" D.O.	21	
		5	2" T.O.	25	γ=138
		6	2" D.O.	10	
511.4 / 58.1	COMPACT TO VERY DENSE GREY SILTY SAND AND GRAVEL TRACE OF CLAY (TILL)	7	2" D.O.	24	
		8	2" D.O.	49	MH
503.0 / 66.5	END OF HOLE				

DYNAMIC PENETRATION RESISTANCE BLOWS/FT: 20, 40, 60, 80, 100

SHEAR STRENGTH C_u LB./SQ.FT: VANE - + NAT, ⊕ REM. ● - Q: 500, 1,000, 1,500, 2,000, 2,500

COEFFICIENT OF PERMEABILITY K, CM./SEC.

WATER CONTENT PERCENT: W_P, W, W_L: 10, 20, 30, 40

PIEZOMETER OR STANDPIPE INSTALLATION

ELEVATION SCALE: 570, 560, 550, 540, 530, 520, 510, 500

168 →

END OF PEN. TEST @ EL. 528.5

15-○-5 Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.D.

PROJECT No. 67028

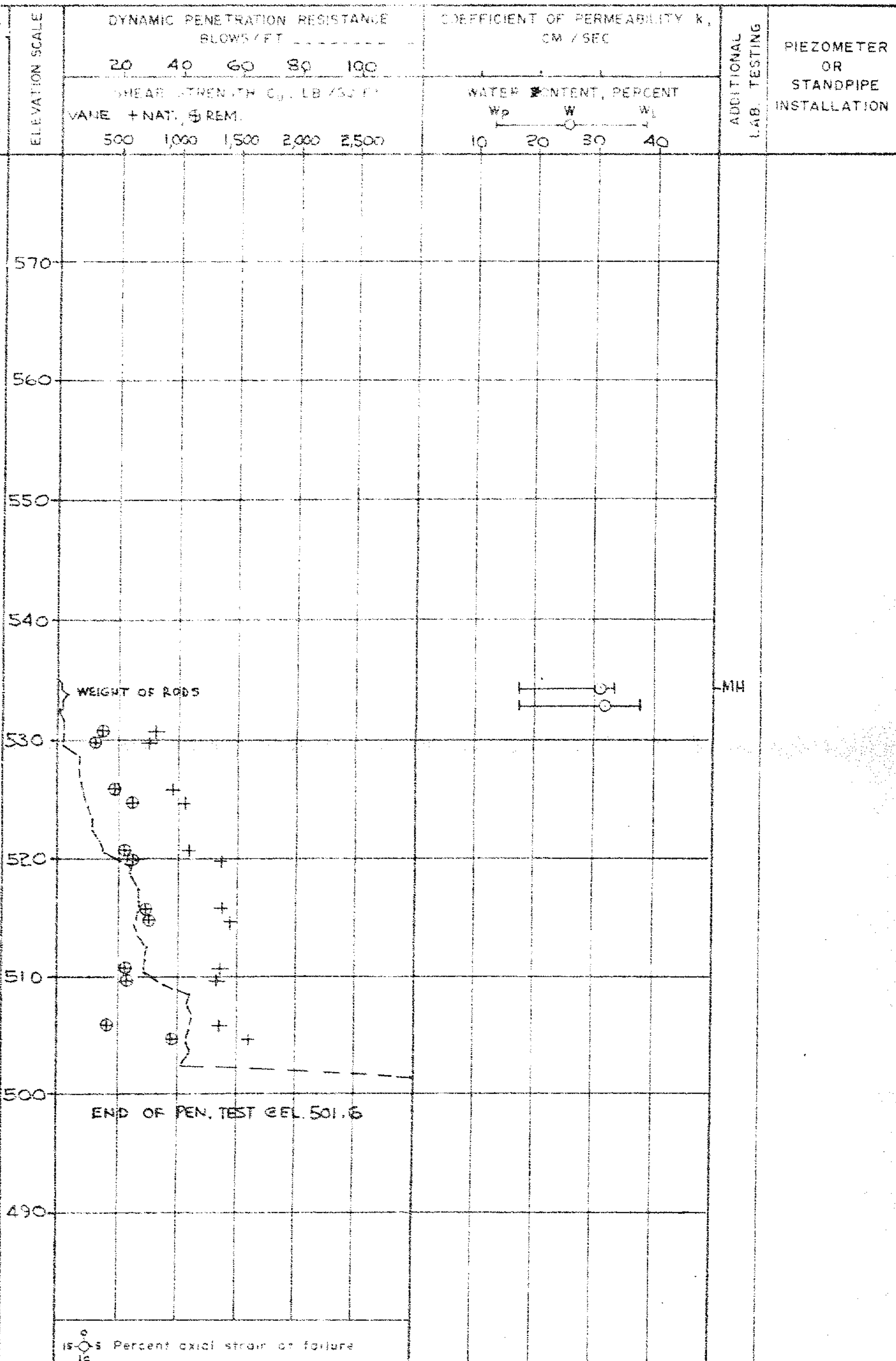
RECORD OF BOREHOLE C-7

LOCATION: STA. 975+00 ℄ CANAL, See Figure 1 — BORING DATE: MARCH 29, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING — BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.
569.6 / 0.0	CANAL LEVEL			
535.1 / 34.5	CANAL BOTTOM			
	FIRM TO STIFF REDDISH-BROWN (GREY LAYERING BELOW ABOUT EL. 509) INTERBEDDED SILTY CLAY WITH A TRACE OF SAND & GRAVEL (UPPER 3 FEET IN SOFTENED CONDITION)	1	2" T.O.	Pm
		2	2" T.O.	Pm
		3	2" D.O.	Pm
		4	2" T.O.	Pm
		5	2" D.O.	Pm
		6	2" T.O.	Pm
		7	2" D.O.	Pm
502.6 / 67.0		8	2" T.O.	Pm
	VERY DENSE GREY SILTY SAND WITH SOME GRAVEL AND A TRACE OF CLAY (TILL)	9	2" D.O.	>100
497.1 / 72.5	END OF HOLE	10	2" D.O.	>100



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED R.T.D.

PROJECT No. 61068

RECORD OF ~~BOREHOLE~~ PEN. TEST C- 8

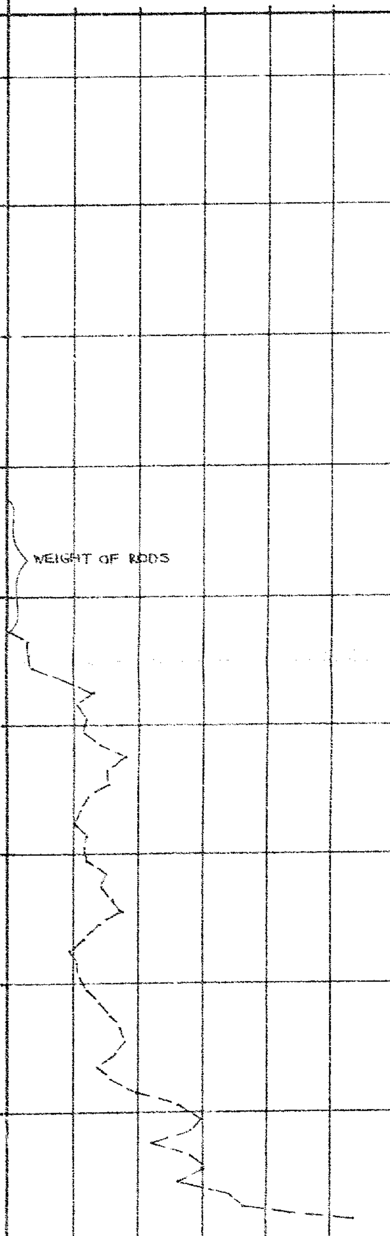
LOCATION: STA. 1096+00 - ℄ CANAL, See Figure
BORING DATE: MARCH 28, 1967
DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST
BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
569.5 / 0.0	CANAL LEVEL					570
						560
						550
537.5 / 32.0	CANAL BOTTOM					540
						530
	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 505)					520
						510
						500
80	PROBABLY SILTY SAND AND GRAVEL (TILL)					490
481.5 / 88.0	END OF PEN. TEST					480

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. ——————
20 40 60 80 100
SHEAR STRENGTH C_u, LB. / SQ. FT.



15-○-5 Percent axial strain at failure

COEFFICIENT OF PERMEABILITY k, CM. / SEC.

WATER CONTENT, PERCENT
W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED B.T.D.

RECORD OF ~~BOREHOLE~~ PEN. TEST C-9

LOCATION STA. 1060+00 - ℄ CANAL See Figure 1

BORING DATE MARCH 28, 1967

DATUM GEODETIC

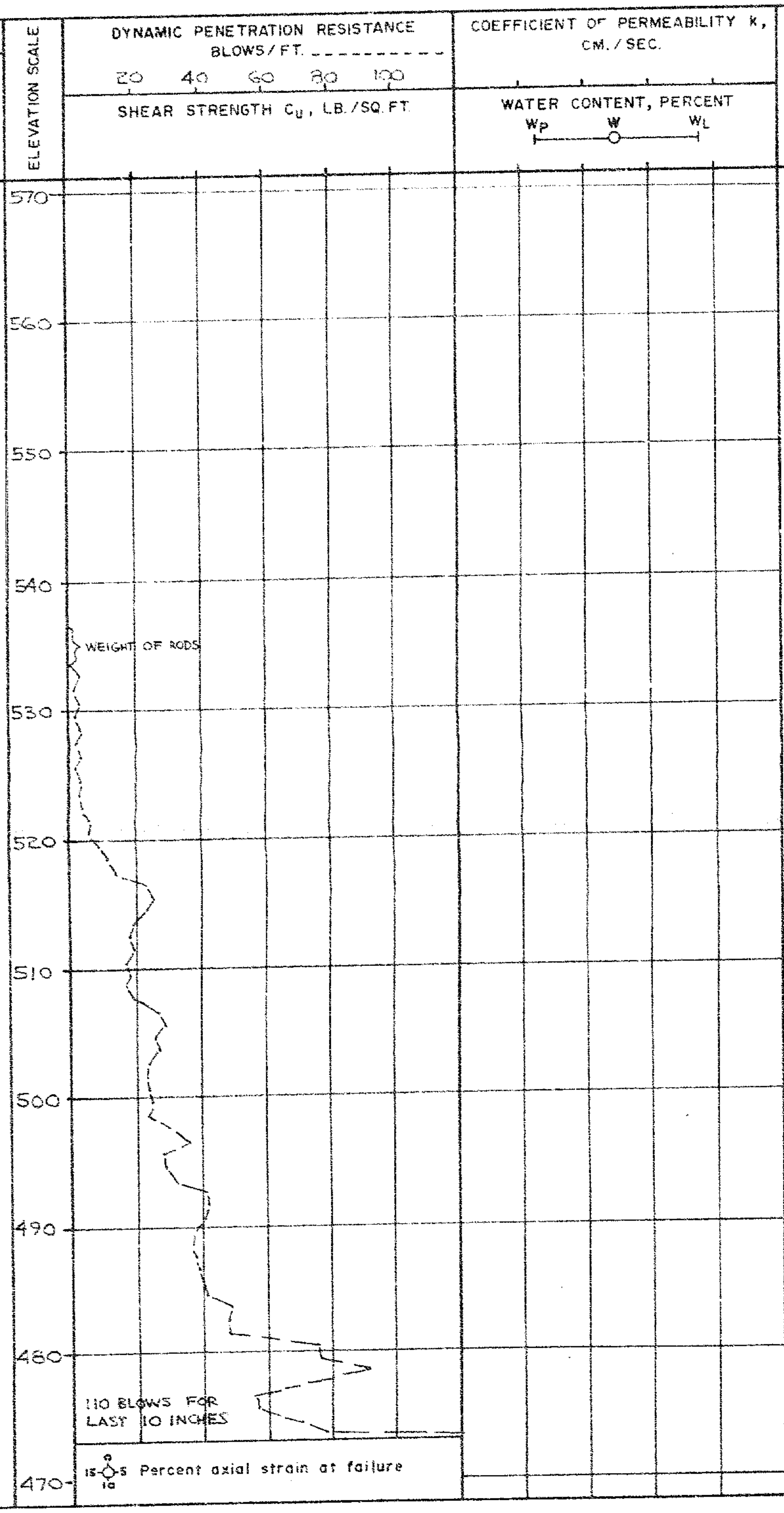
BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	TYPE (SAMPLES)	BLOWS / FT. (SAMPLES)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.5	CANAL LEVEL						
0.0							
536.5	CANAL BOTTOM						
33.0	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 505)						
80	PROBABLY SILTY SAND AND GRAVEL (TILL)						
472.7 / 96.8	END OF PEN. TEST						



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

RECORD OF ~~BOREHOLE~~ PEN. TEST C-10

LOCATION: STA. 1070+00 - ℄ CANAL, See Figure 1

BORING DATE: MARCH 27, 1967

DATUM: GEODETIC

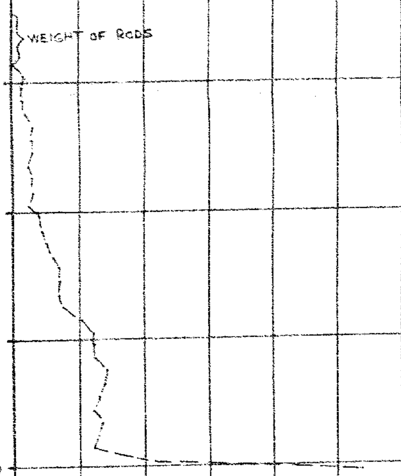
BOREHOLE TYPE: PENETRATION TEST

BOREHOLE DIAMETER: —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT. (20, 40, 60, 80, 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P, W, W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.5 / 0.0	CANAL LEVEL					570				
						560				
						550				
						540				
535.5 / 34.0	CANAL BOTTOM						WEIGHT OF RODS			
						530				
	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 513)					520				
						510				
498 / 497.5 / 70.0	END OF PEN. TEST					500				
	PROBABLY SILTY SAND AND GRAVEL (TILL)					490				



15 10 5 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED

PROJECT No. 67060

RECORD OF ~~BOREHOLE~~ PEN. TEST C-11

LOCATION STA. 1060+00 - ℄ CANAL See Figure 1

BORING DATE MARCH 27, 1967

DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

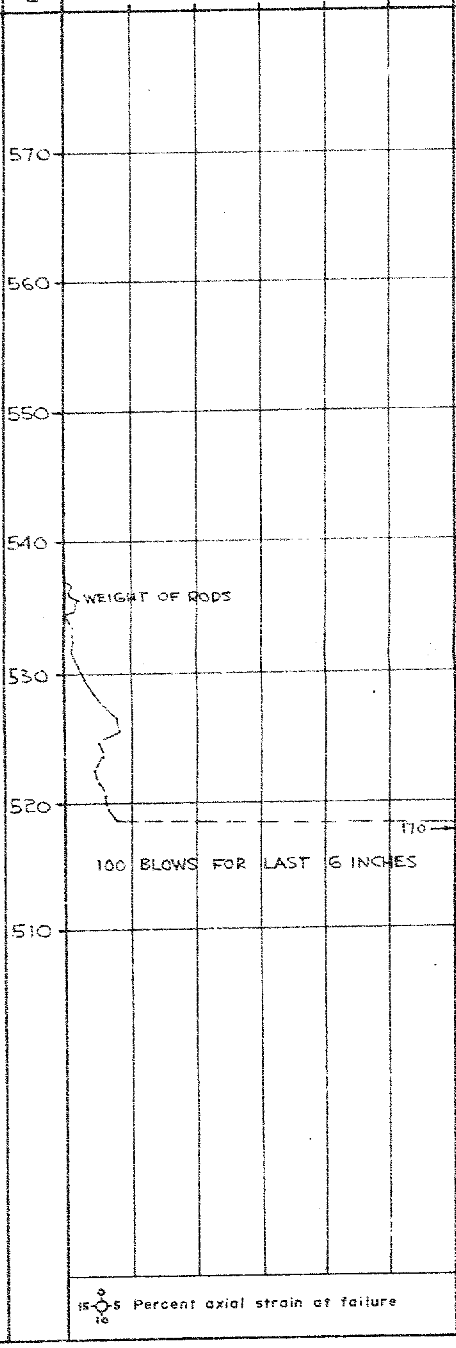
BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION (SOIL PROFILE)
569.5	CANAL LEVEL
0.0	
537.3	CANAL BOTTOM
32.2	PROBABLY INTERBEDDED SILTY CLAY
518.5	
517.0	END OF PEN. TEST
52.5	PROBABLY SILTY SAND AND GRAVEL (TILL)

Columns: SOIL PROFILE (ELEVN. DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, BLOWS / FT.); ELEVATION SCALE; DYNAMIC PENETRATION RESISTANCE BLOWS / FT. (20, 40, 60, 80, 100) / SHEAR STRENGTH C_u, LB / SQ. FT; COEFFICIENT OF PERMEABILITY k, CM. / SEC. / WATER CONTENT, PERCENT W_P W W_L; ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION



15 5 10 Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED [illegible]

PROJECT No. 67058

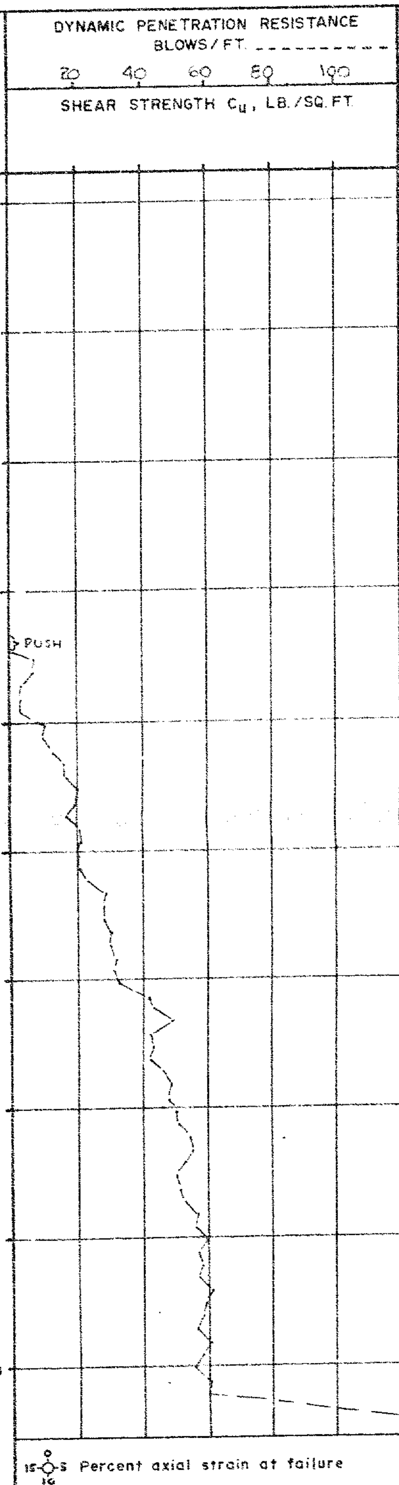
RECORD OF ~~BOREHOLE~~ PEN. TEST C-12

LOCATION: STA. 1040+50 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 23, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
569.5	CANAL LEVEL					570
0.0						560
						550
						540
536.5 / 33.0	CANAL BOTTOM					530
	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 500)					520
						510
						500
						490
85	PROBABLY SILTY SAND AND GRAVEL (TILL)					480
475.5 / 94.0	END OF PEN. TEST					470



COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

PROJECT No. 67028

RECORD OF ~~BOREHOLE~~ PEN. TEST C-13

LOCATION: STA. 1030+00 · ℄ CANAL, See Figure 1

BORING DATE: MARCH 22, 1967

DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST

BOREHOLE DIAMETER: ———

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FT.
568.5	CANAL LEVEL				
0.0					
537.5 / 32.0	CANAL BOTTOM				
	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 502)				
85	PROBABLY SILTY SAND AND GRAVEL (TILL)				
475.8 / 93.7	END OF PEN. TEST				

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. - - - - - - - - -

20 40 60 80 100

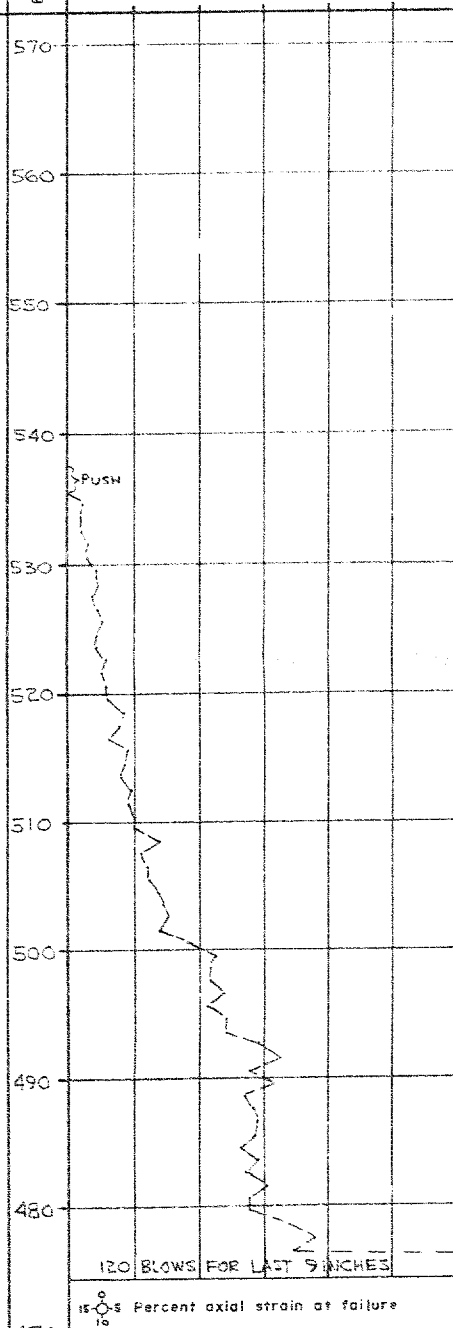
SHEAR STRENGTH C_u, LB./SQ.FT

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_p W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



15-○-5 Percent axial strain at failure (10)

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

PROJECT No. 67028

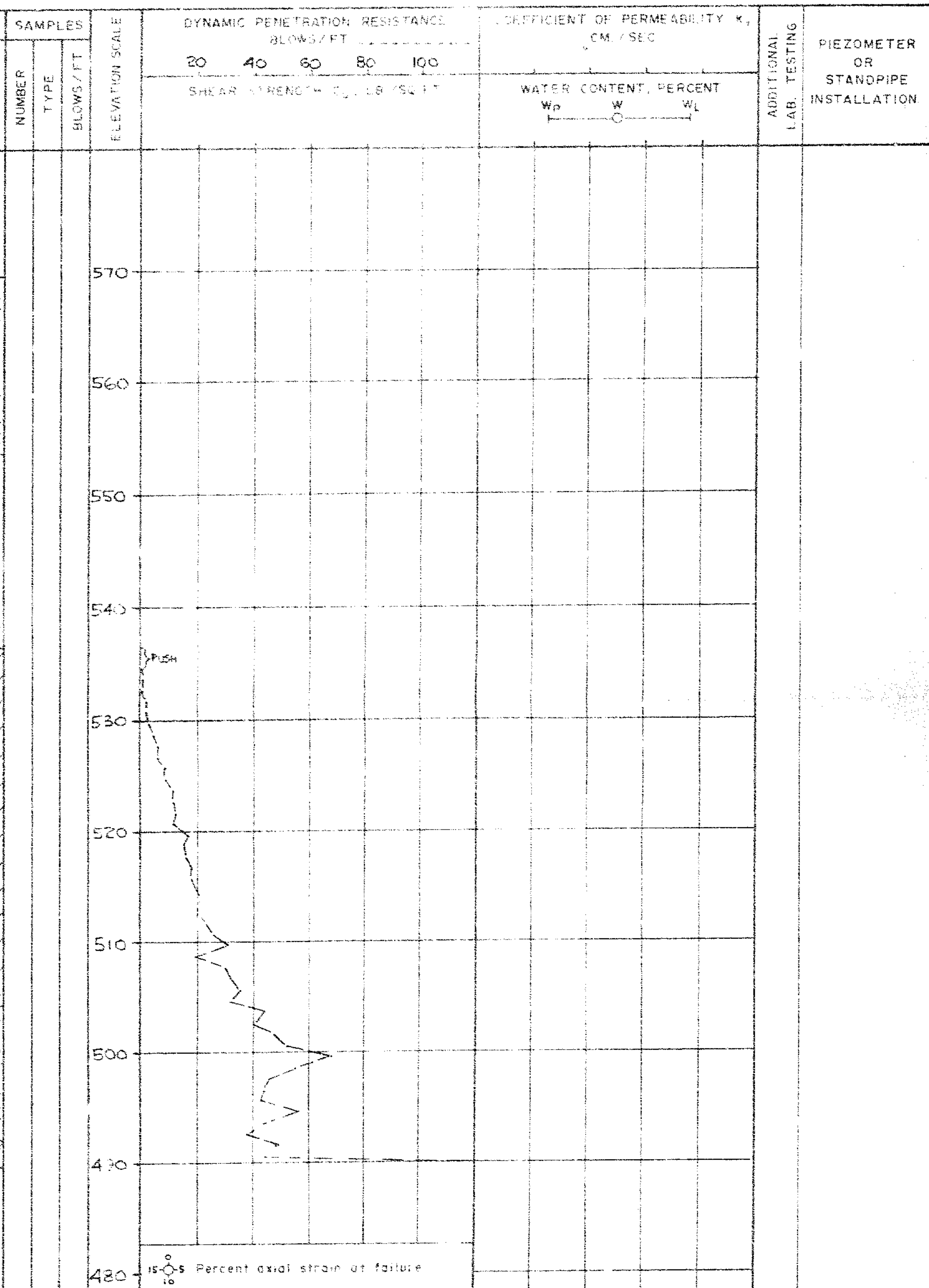
RECORD OF ~~BOREHOLE~~ PEN. TEST C-14

LOCATION: STA. 1020+00 - ℄ CANAL See Figure 1 | BORING DATE: MARCH 21, 1967 | DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST | BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES | PEN. TEST HAMMER WEIGHT 140 LB DROP 30 INCHES

ELEVN / DEPTH	SOIL PROFILE DESCRIPTION
569.7	CANAL LEVEL
0.0	
536.7	CANAL BOTTOM
33.0	PROBABLY INTERBEDDED SILTY CLAY
65	PROBABLY SILTY SAND AND GRAVEL (TILL)
490.0 / 79.7	END OF PEN. TEST



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

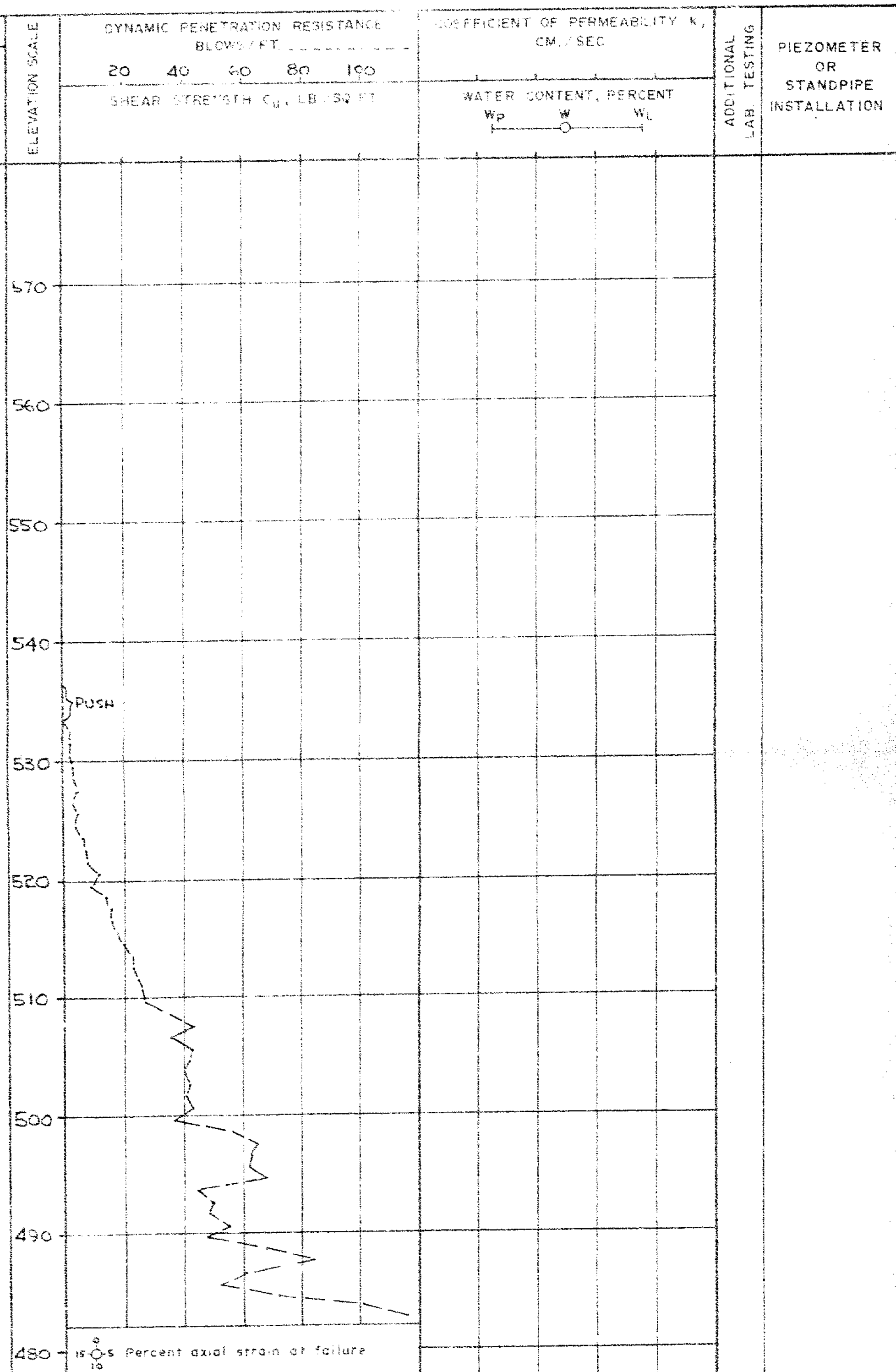
RECORD OF ~~BOREHOLE~~ PEN. TEST C-15

LOCATION STA. 1009+50 - ℄ CANAL See Figure 1 | BORING DATE MARCH 20, 1967 | DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST | BOREHOLE DIAMETER —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES | PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT
569.7	CANAL LEVEL				
0.0					
536.7	CANAL BOTTOM				
33.0	PROBABLY INTERBEDDED SILTY CLAY				
70	PROBABLY SILTY SAND AND GRAVEL (TILL)				
482.7 / 87.0	END OF PEN. TEST				



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED [illegible]

PROJECT NO. [illegible]

RECORD OF ~~BOREHOLE~~ PEN. TEST C-16

LOCATION: STA. 970+00 – ℄ CANAL, See Figure 1
BORING DATE: MARCH 15, 1967
DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST
BOREHOLE DIAMETER: ——

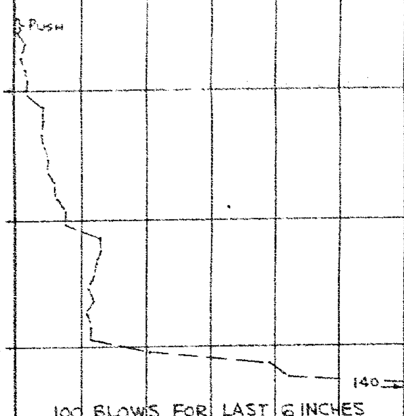
SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT.
568.7	CANAL LEVEL				
0.0					
535.7	CANAL BOTTOM				
33.0	PROBABLY INTERBEDDED SILTY CLAY				
59	PROBABLY SILTY SAND AND GRAVEL (TILL)				
506.2 / 62.5	END OF PEN TEST				

ELEVATION SCALE: 570, 560, 550, 540, 530, 520, 510, 500

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. - - - - - - - - - -
20 40 60 80 100

SHEAR STRENGTH C_u, LB. / SQ. FT.



15 –○– 5 Percent axial strain at failure
10

COEFFICIENT OF PERMEABILITY k, CM. / SEC.

WATER CONTENT, PERCENT
W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED S.T.D.

PROJECT No. [illegible]

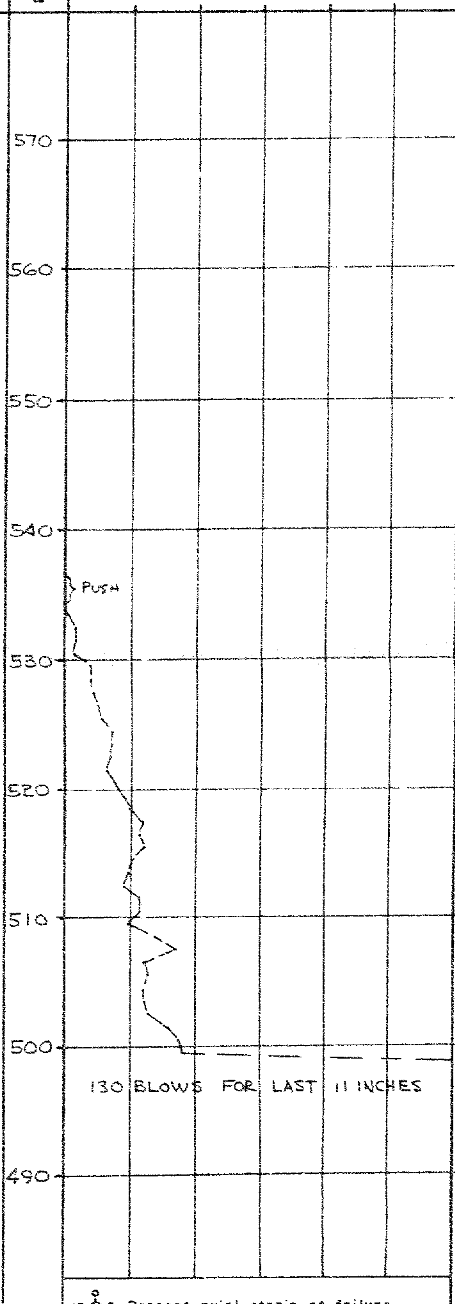
RECORD OF ~~BOREHOLE~~ PEN. TEST C-17

LOCATION: STA. 980+50 ℄ CANAL, See Figure 1 — BORING DATE: MARCH 20, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	BLOWS / FT.
569.7 / 0.0	CANAL LEVEL			
536.7 / 33.0	CANAL BOTTOM			
	PROBABLY INTERBEDDED SILTY CLAY			
500.0				
498.8 / 70.9	END OF PEN. TEST			
	PROBABLY SILTY SAND AND GRAVEL (TILL)			



15-○-5 10 Percent axial strain at failure

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.P.

PROJECT No. [illegible]

RECORD OF ~~BOREHOLE~~ PEN. TEST C-18

LOCATION: STA. 969+90 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 21, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT. PLOT	SAMPLES – NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
569.7 / 0.0	CANAL LEVEL					570
						560
						550
						540
534.7 / 35.0	CANAL BOTTOM					530
						520
	PROBABLY INTERBEDDED SILTY CLAY					510
						500
						490
485.3 / 84.4	END OF PEN. TEST					
	PROBABLY SILTY SAND AND GRAVEL (TILL)					480

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. - - - - - - - - - - -

20 40 60 80 100

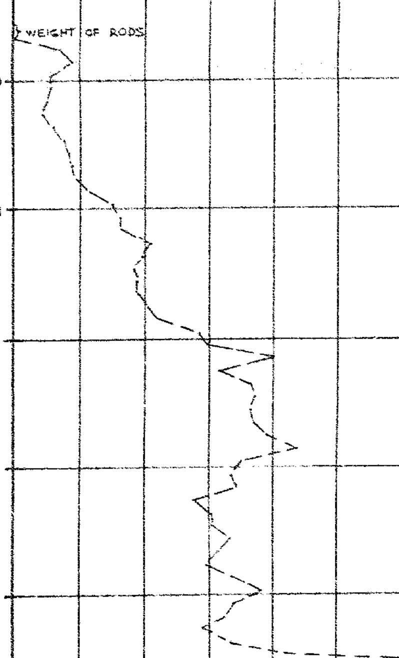
SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_p W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



15-○-5 Percent axial strain at failure (10)

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED B.T.D.

PROJECT No. 67459

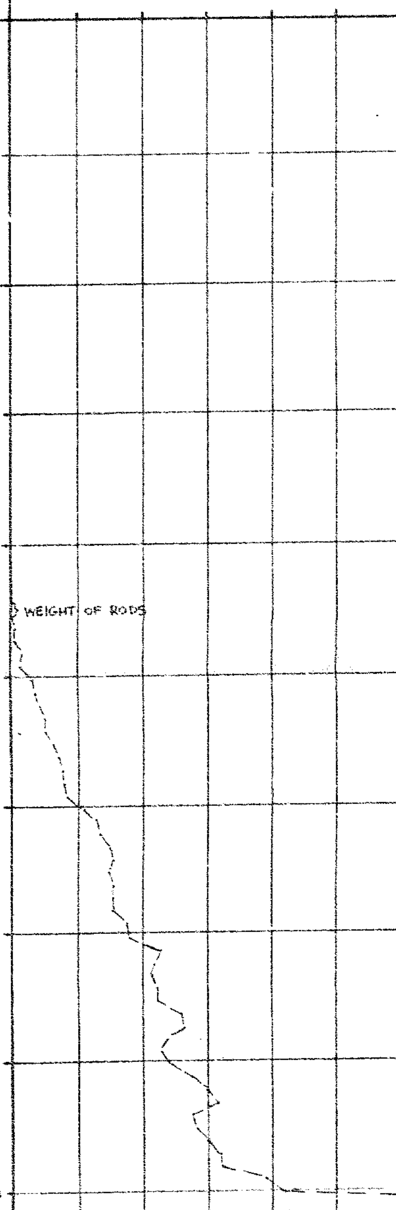
RECORD OF ~~BOREHOLE~~ PEN. TEST C-19

LOCATION: STA. 960+50 - ℄ CANAL, See Figure 1 | BORING DATE: MARCH 20, 1967 | DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST | BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES | PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20 40 60 80 100) / SHEAR STRENGTH C_u, LB/SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.7 / 0.0	CANAL LEVEL					570				
						560				
						550				
						540				
535.7 / 34.0	CANAL BOTTOM						WEIGHT OF RODS			
						530				
	PROBABLY INTERBEDDED SILTY CLAY					520				
						510				
						500				
78 / 489.4						490				
80.3	END OF PEN. TEST						120 BLOWS FOR LAST 4 INCHES			
	PROBABLY SILTY SAND AND GRAVEL (TILL)					480				



15 ○ 5 (10) Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED [illegible]

PROJECT No. 67060

RECORD OF ~~BOREHOLE~~ PEN. TEST C-20

LOCATION: STA. 941+60 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 22, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT. PLOT	SAMPLES – NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
569.5 / 0.0	CANAL LEVEL					570
						560
						550
						540
536.0 / 33.5	CANAL BOTTOM					
	PROBABLY INTERBEDDED SILTY CLAY					530
						520
						510
499.5						500
498.0 / 71.5	END OF PEN. TEST					
	PROBABLY SILTY SAND AND GRAVEL (TILL)					490

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. - - - - - - - - -

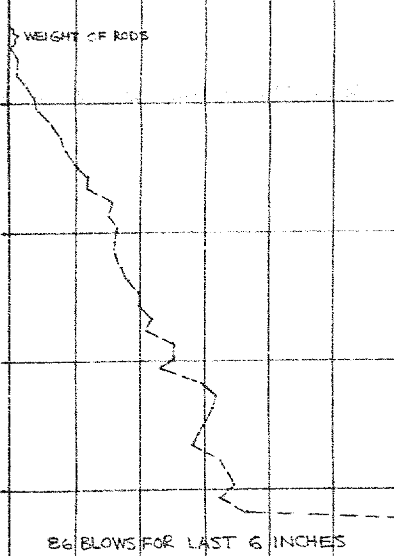
SHEAR STRENGTH C_u, LB. / SQ. FT.

COEFFICIENT OF PERMEABILITY k, CM. / SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



15-○-5 Percent axial strain at failure (10)

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

PROJECT No. 67060

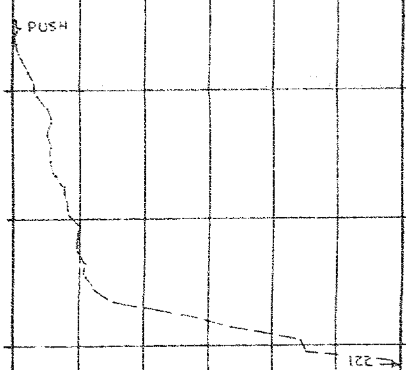
RECORD OF ~~BOREHOLE~~ PEN. TEST C-21

LOCATION: STA. 930+00 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 22, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20, 40, 60, 80, 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.					
569.5	CANAL LEVEL					570				
0.0						560				
						550				
						540				
535.5 / 34.0	CANAL BOTTOM						PUSH			
	PROBABLY INTERBEDDED SILTY CLAY					530				
						520				
55	PROBABLY SILTY SAND AND GRAVEL (TILL)					510	122 →			
508.3 / 61.2	END OF PEN. TEST						50 BLOWS FOR LAST 3 INCHES			
						500				



15 ─○─ 5 (10): Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED R.I.D.

PROJECT No. 67058

RECORD OF ~~BOREHOLE~~ PEN. TEST C-22

LOCATION STA. 920+00 - ℄ CANAL See Figure 1

BORING DATE MARCH 27, 1967

DATUM GEODETIC

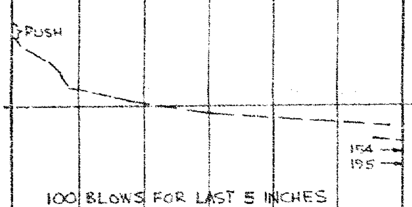
BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT. (20, 40, 60, 80, 100) / SHEAR STRENGTH C_u, LB./SQ.FT	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT.					
569.5 / 0.0	CANAL LEVEL					570				
						560				
						550				
						540				
536.6 / 32.9	CANAL BOTTOM									
	PROBABLY CLAYEY SILT (TILL)					530				
525.1 / 44.4	END OF PEN. TEST					520				



15 ○ 5 (0 above, 10 below) Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED [illegible]

PROJECT No. 61058

RECORD OF ~~BOREHOLE~~ PEN. TEST C-23

LOCATION: STA. 909+75 ℄ CANAL See Figure 1 — BORING DATE: MARCH 27, 1967 — DATUM: GEODETIC

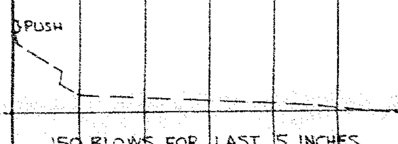
BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FT.	ELEVATION SCALE
569.5	CANAL LEVEL					570
0.0						560
						550
						540
537.3 32.2	CANAL BOTTOM					
	PROBABLY CLAYEY SILT (TILL)					
530.1 39.4	END OF PEN. TEST					530
						520

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. ---------- 20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.



COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

15 - 8 - 10 Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED RTD

RECORD OF ~~BOREHOLE~~ PEN. TEST C-24

LOCATION: STA. 890+00, See Figure 1

BORING DATE: MARCH 28, 1967

DATUM: GEODETIC

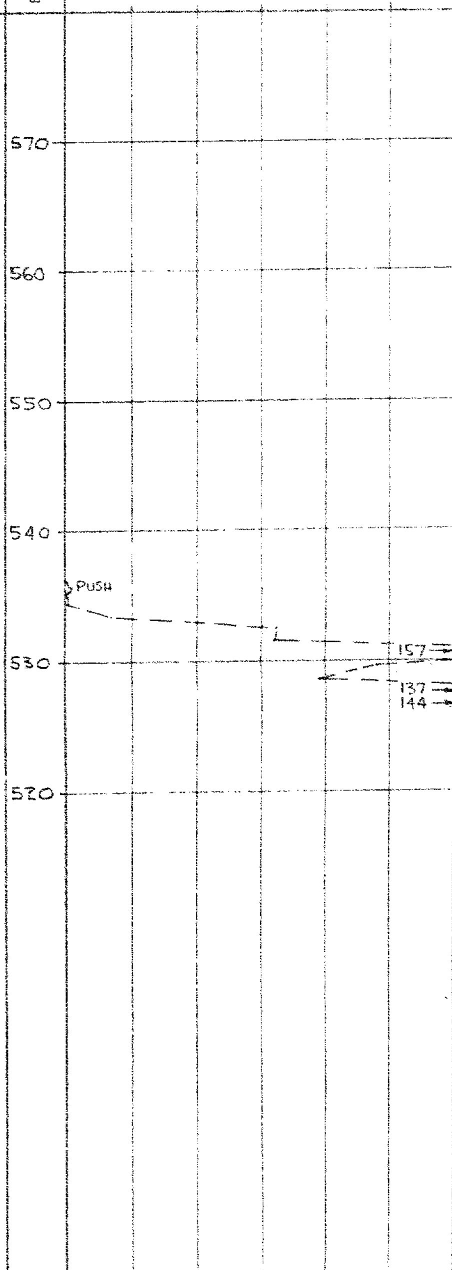
BOREHOLE TYPE: PENETRATION TEST

BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES								
ELEVN. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT (20, 40, 60, 80, 100); SHEAR STRENGTH C_u, LB / SQ FT	COEFFICIENT OF PERMEABILITY k, CM. / SEC; WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.5 / 0.0	CANAL LEVEL					570				
						560				
						550				
						540				
536.2 / 33.3	CANAL BOTTOM									
	PROBABLY CLAYEY SILT (TILL)					530				
526.5 / 43.0	END OF PEN. TEST					520				



15 ○ 5 10 Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.R.

PROJECT No. 6708

RECORD OF ~~BOREHOLE~~ PEN. TEST C-25

LOCATION: STA. 879+50 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 28, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT / SHEAR STRENGTH C_u, LB/SQ FT	COEFFICIENT OF PERMEABILITY k, CM/SEC / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.5 / 0.0	CANAL LEVEL					570				
						560				
						550				
538.1 / 31.4	CANAL BOTTOM					540	PUSH			
535.0 / 34.5	PROBABLY CLAYEY SILT (TILL)						200 BLOWS FOR LAST 6 INCHES			
	END OF PEN. TEST					530				

15 -○- 5 (10 below) Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.P.

PROJECT No. 67068

RECORD OF ~~BOREHOLE~~ PEN. TEST C-26

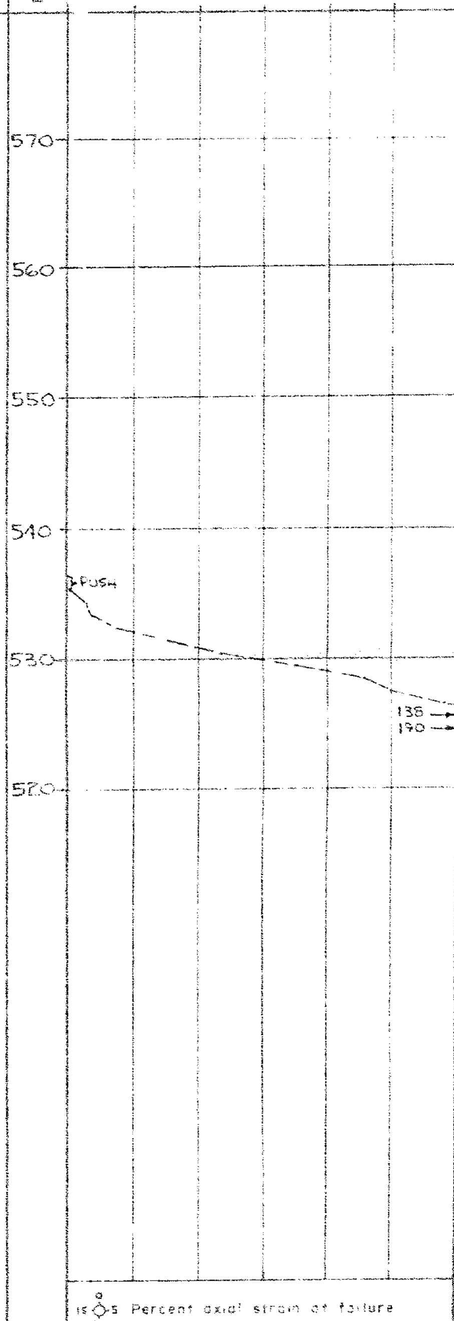
LOCATION: STA. 870 + 15 ℄ CANAL, See Figure 1 — BORING DATE: MARCH 28, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST — BOREHOLE DIAMETER: —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION
569.5	CANAL LEVEL
0.0	
536.5	CANAL BOTTOM
33.0	PROBABLY CLAYEY SILT (TILL)
524.5 / 45.0	END OF PEN. TEST

Column headings: SOIL PROFILE (ELEVN. DEPTH, DESCRIPTION, STRAT. PLOT); SAMPLES (NUMBER, TYPE, BLOWS / FT.); ELEVATION SCALE; DYNAMIC PENETRATION RESISTANCE BLOWS / FT.; SHEAR STRENGTH C_u, LB./SQ.FT.; COEFFICIENT OF PERMEABILITY k, CM./SEC.; WATER CONTENT, PERCENT W_P W W_L; ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

RECORD OF ~~BOREHOLE~~ PEN. TEST C-27

LOCATION: STA. 859+60 - ℄ CANAL, See Figure 1

BORING DATE: MARCH 29, 1967

DATUM: GEODETIC

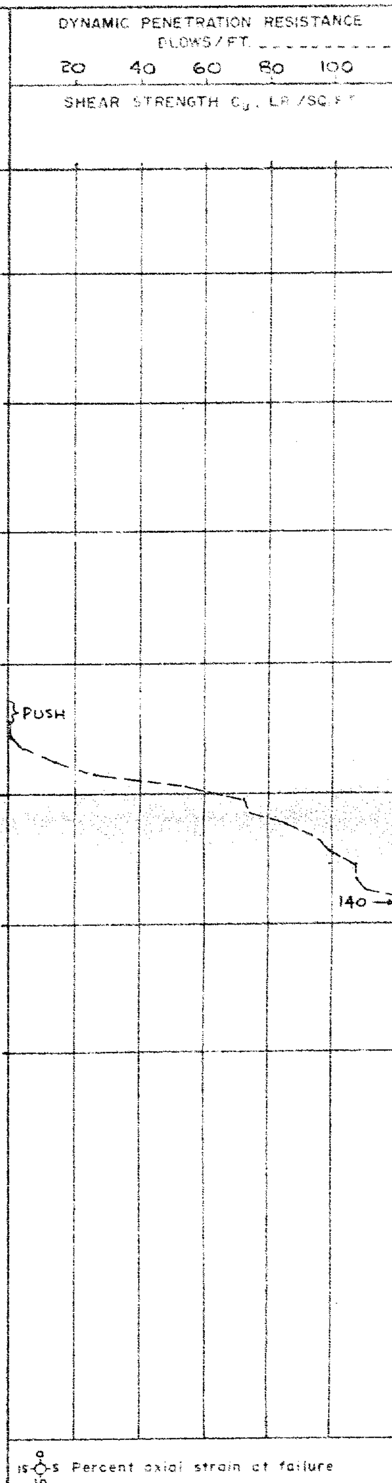
BOREHOLE TYPE: PENETRATION TEST

BOREHOLE DIAMETER: —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE — ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES — NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
						570
569.6	CANAL LEVEL					
0.0						
						560
						550
						540
537.1	CANAL BOTTOM					
32.5	PROBABLY CLAYEY SILT (TILL)					
						530
521.6						
48.0	END OF PEN. TEST					
						520
						510



COEFFICIENT OF PERMEABILITY k, CM. / SEC

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

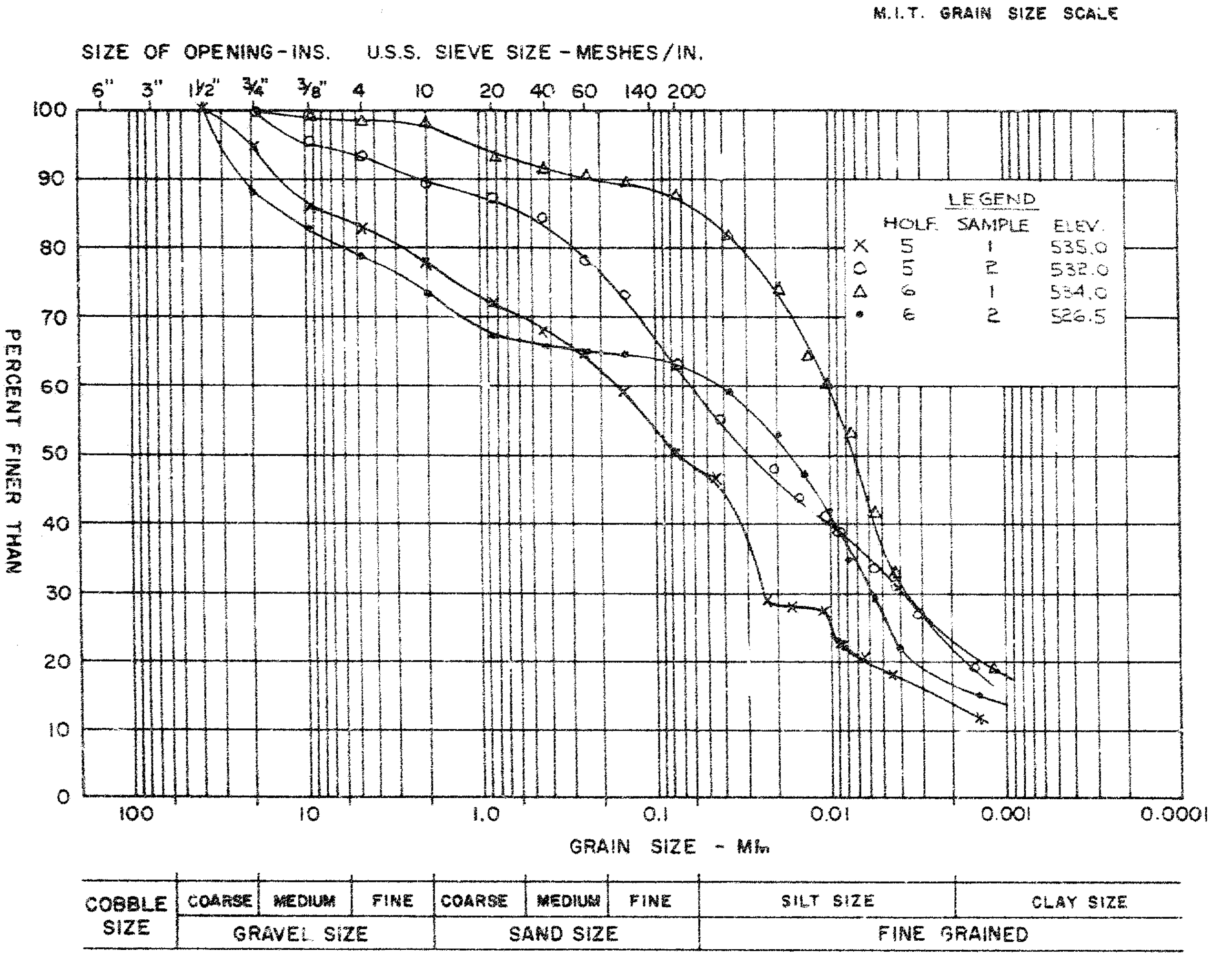
PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED BJD

PROJECT No. 67028



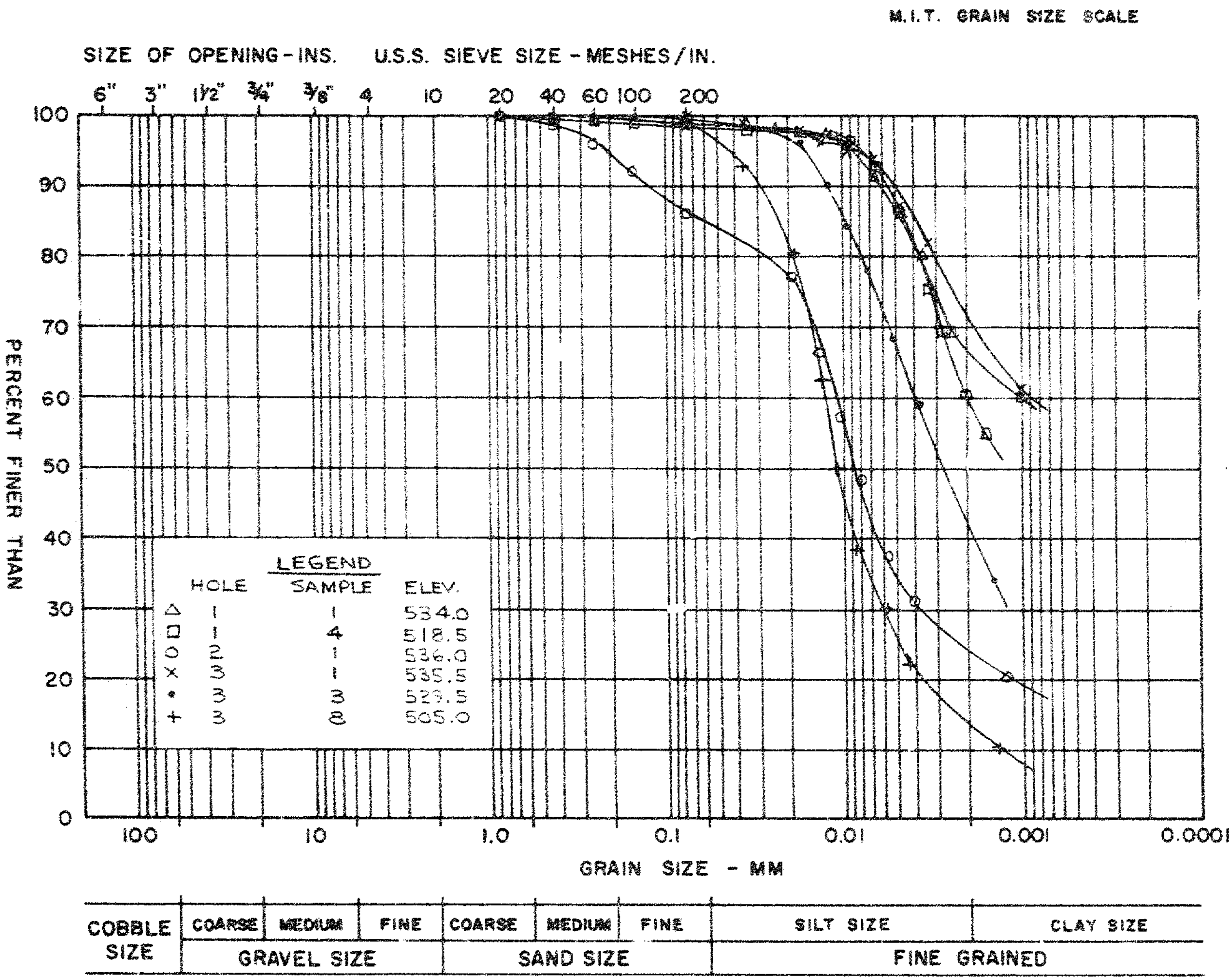
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

FIGURE 3

PROJECT No. 67028



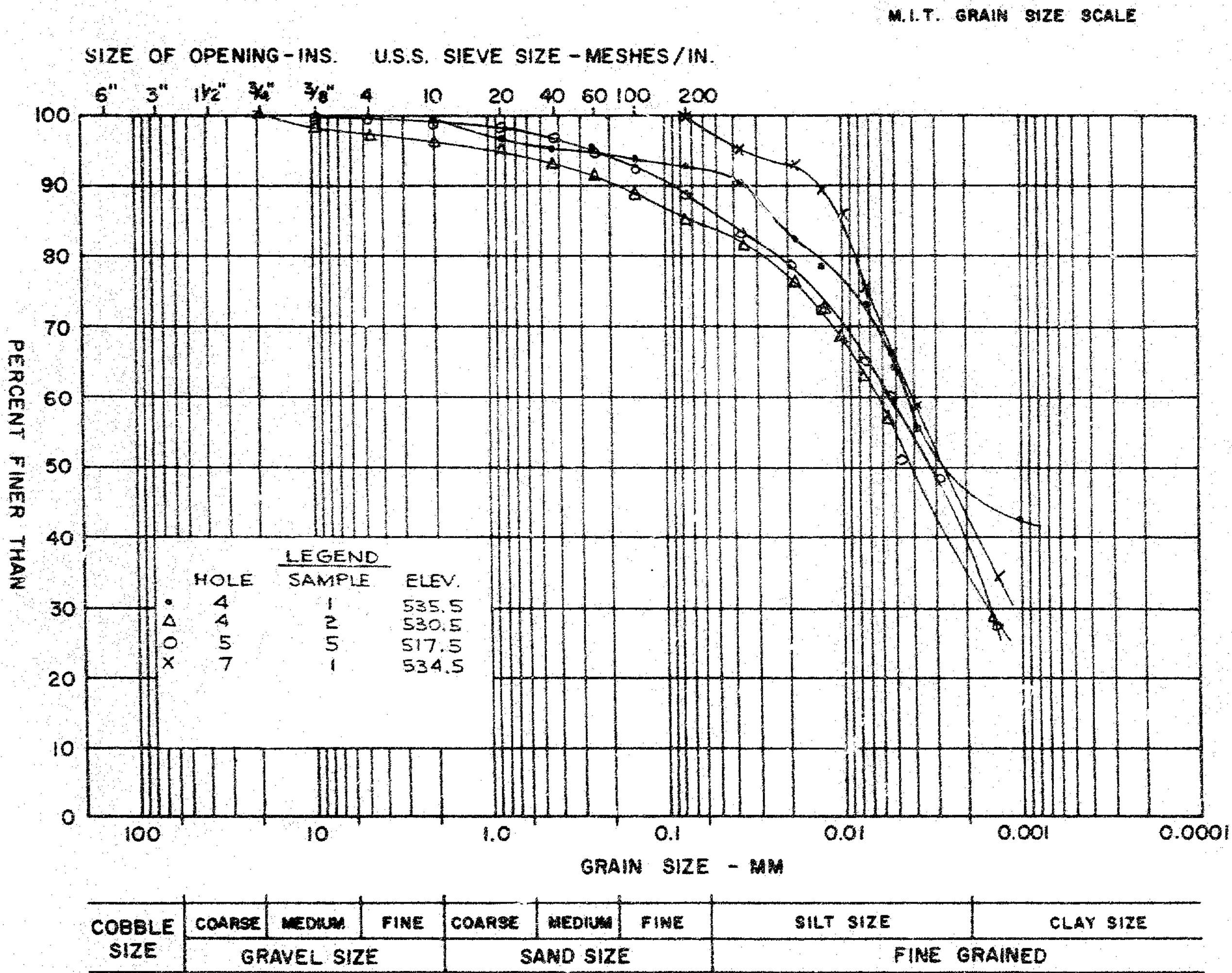
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

INTERBEDDED SILTY CLAY

FIGURE 4

PROJECT No. 67028

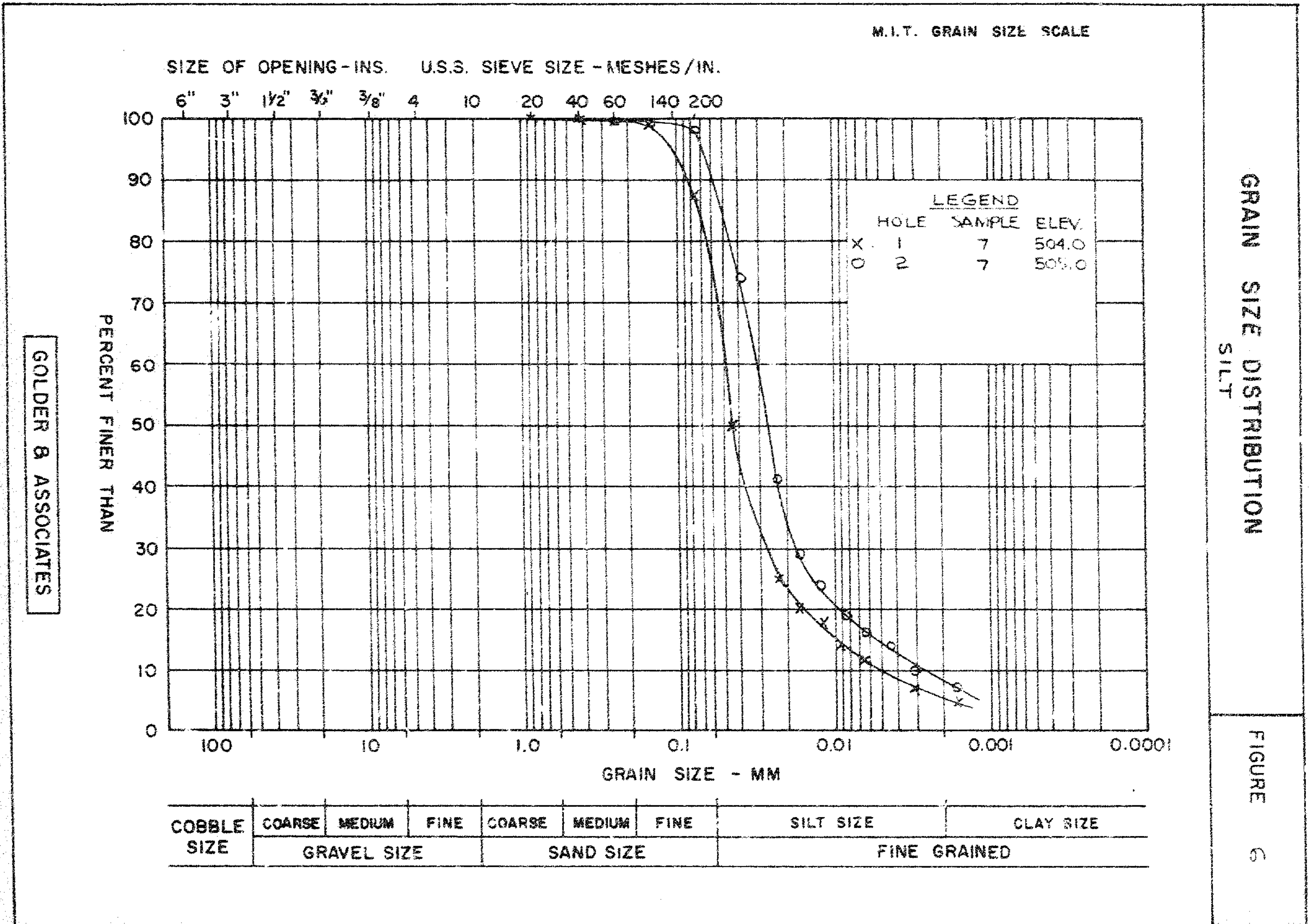


GOLDER & ASSOCIATES

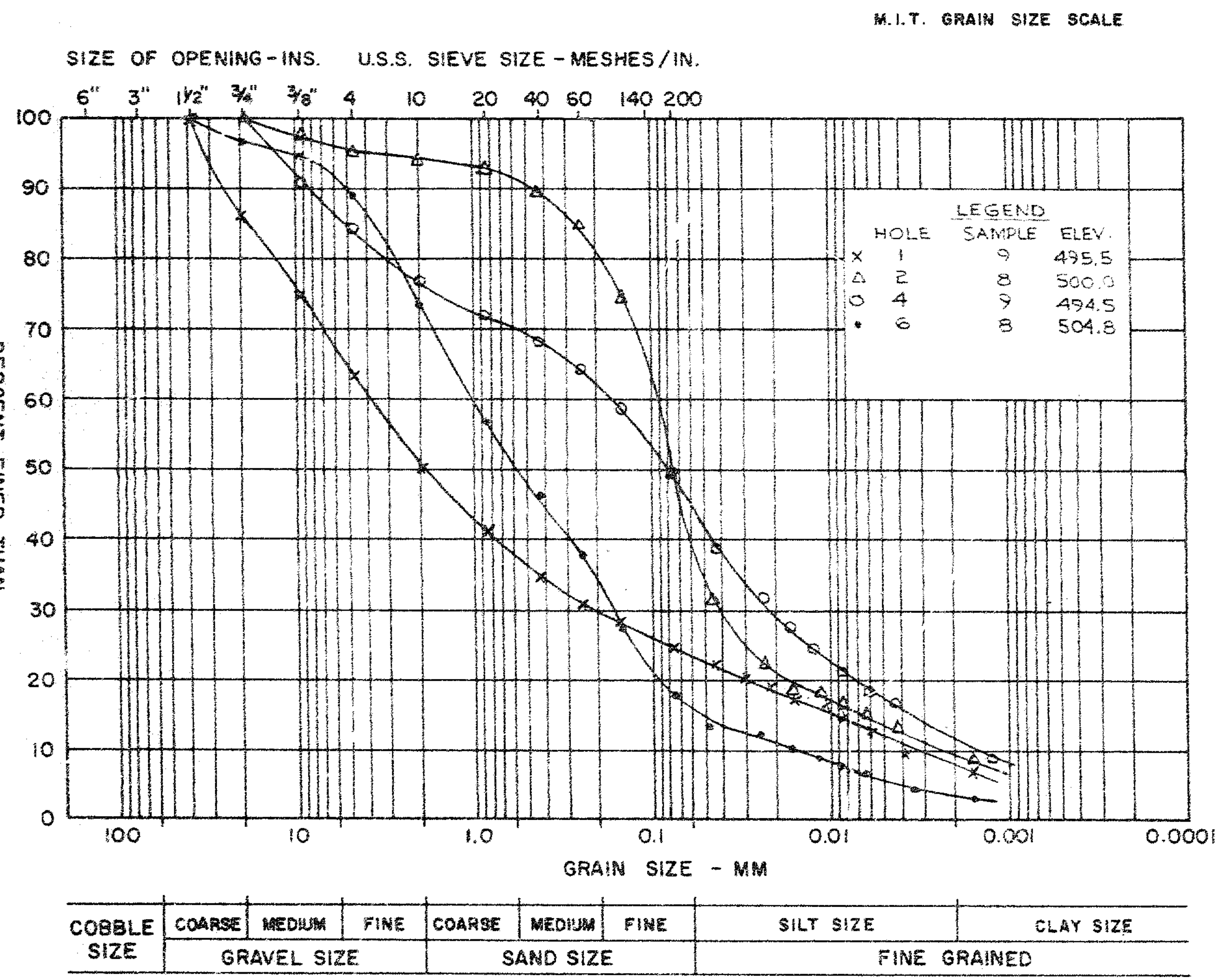
GRAIN SIZE DISTRIBUTION

INTERBEDDED SILTY CLAY

FIGURE 5


M.I.T. GRAIN SIZE SCALE
SIZE OF OPENING - INS.
U.S.S. SIEVE SIZE - MESHES/IN.
6" 3" 1½" ¾" ⅜" 4 10 20 40 60 140 200
PERCENT FINER THAN
100 90 80 70 60 50 40 30 20 10 0
100 10 1.0 0.1 0.01 0.001 0.0001
GRAIN SIZE - MM
LEGEND
HOLE SAMPLE ELEV.
X 1 7 504.0
O 2 7 505.0
COBBLE SIZE
COARSE MEDIUM FINE
GRAVEL SIZE
COARSE MEDIUM FINE
SAND SIZE
SILT SIZE
CLAY SIZE
FINE GRAINED
GOLDER & ASSOCIATES
GRAIN SIZE DISTRIBUTION
SILT
FIGURE 6

PROJECT No. 67028



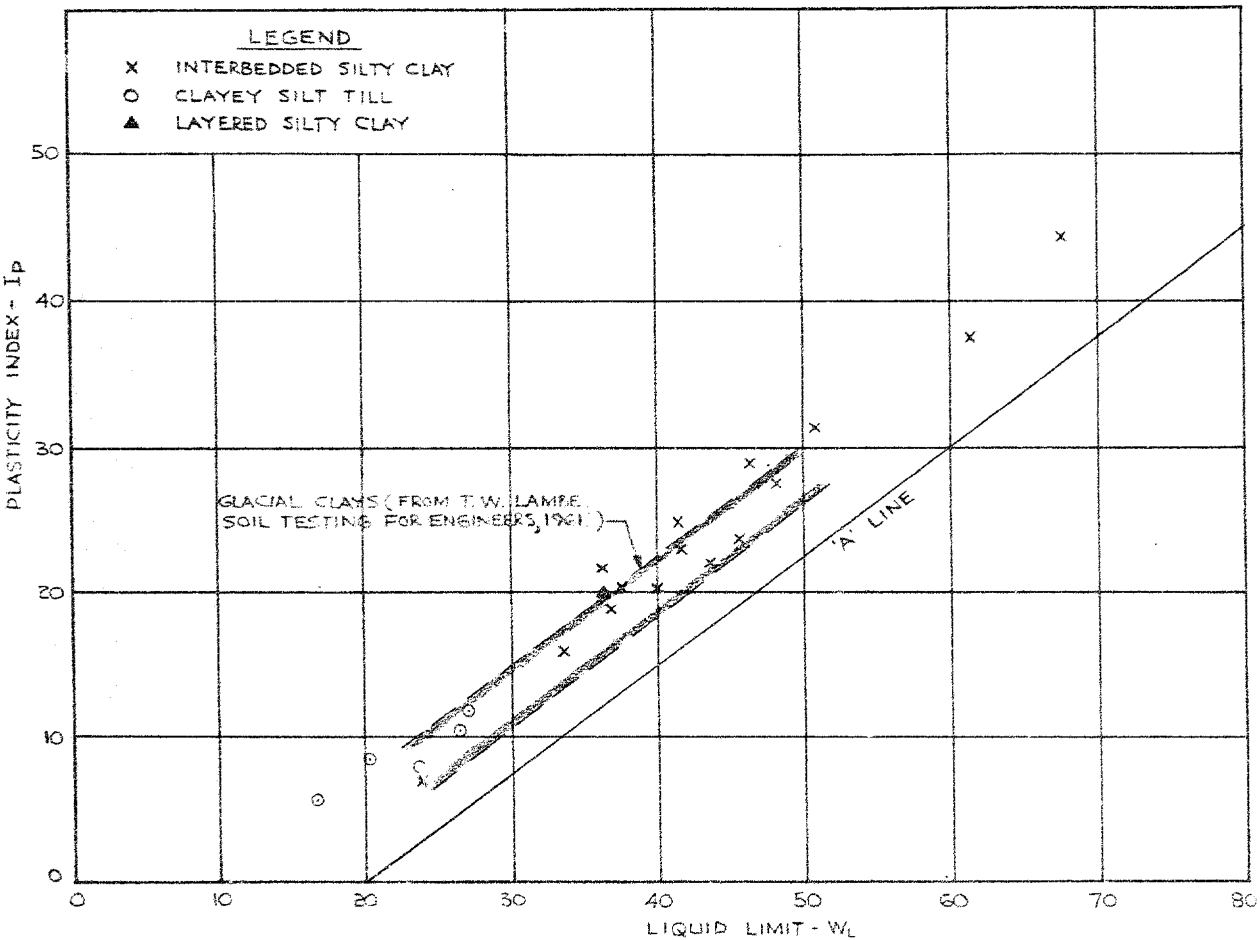
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

SILTY SAND TILL

FIGURE 7

PROJECT No. [illegible]



GOLDER & ASSOCIATES

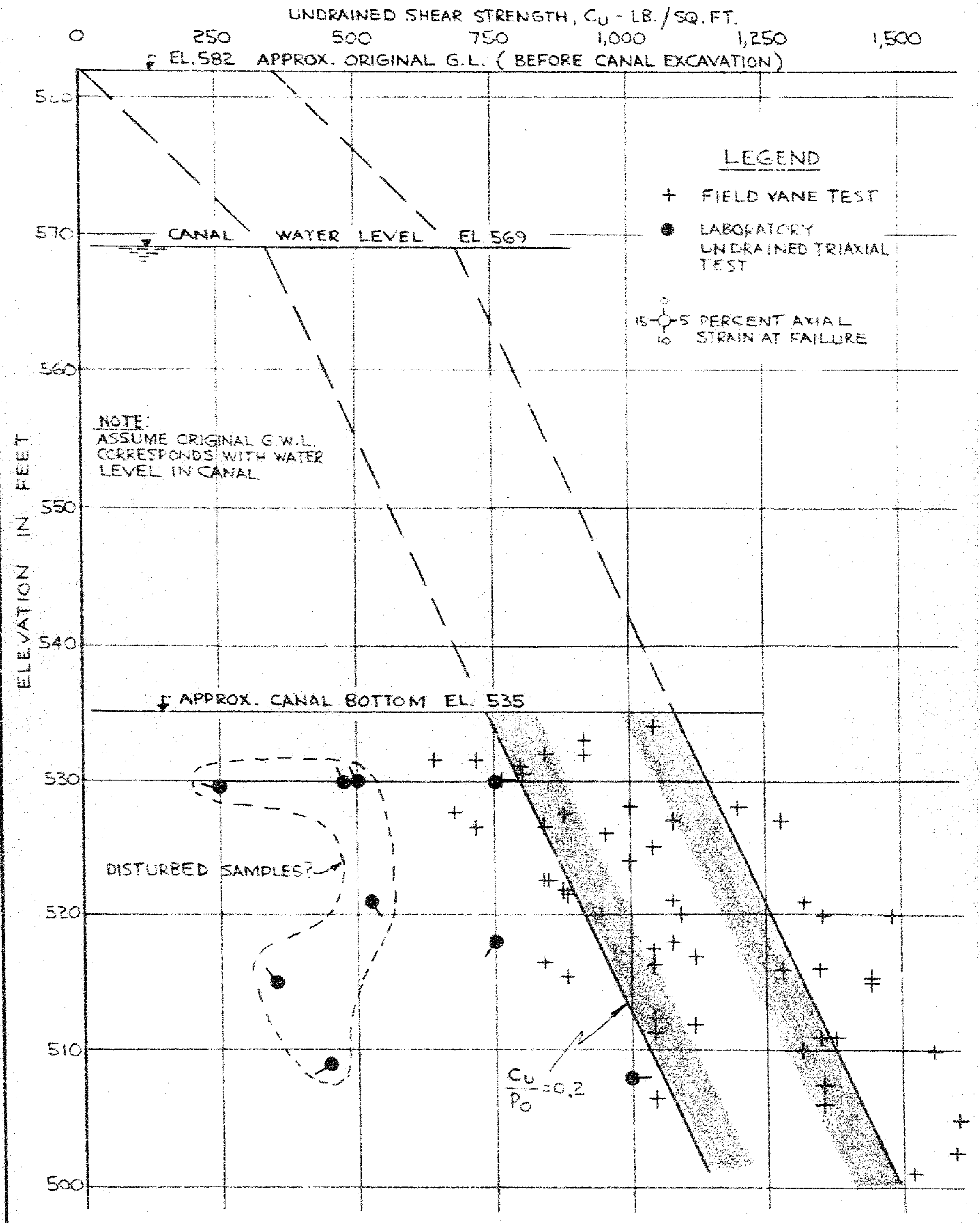
PLASTICITY CHART

FIGURE 8

PROJECT No. 67028

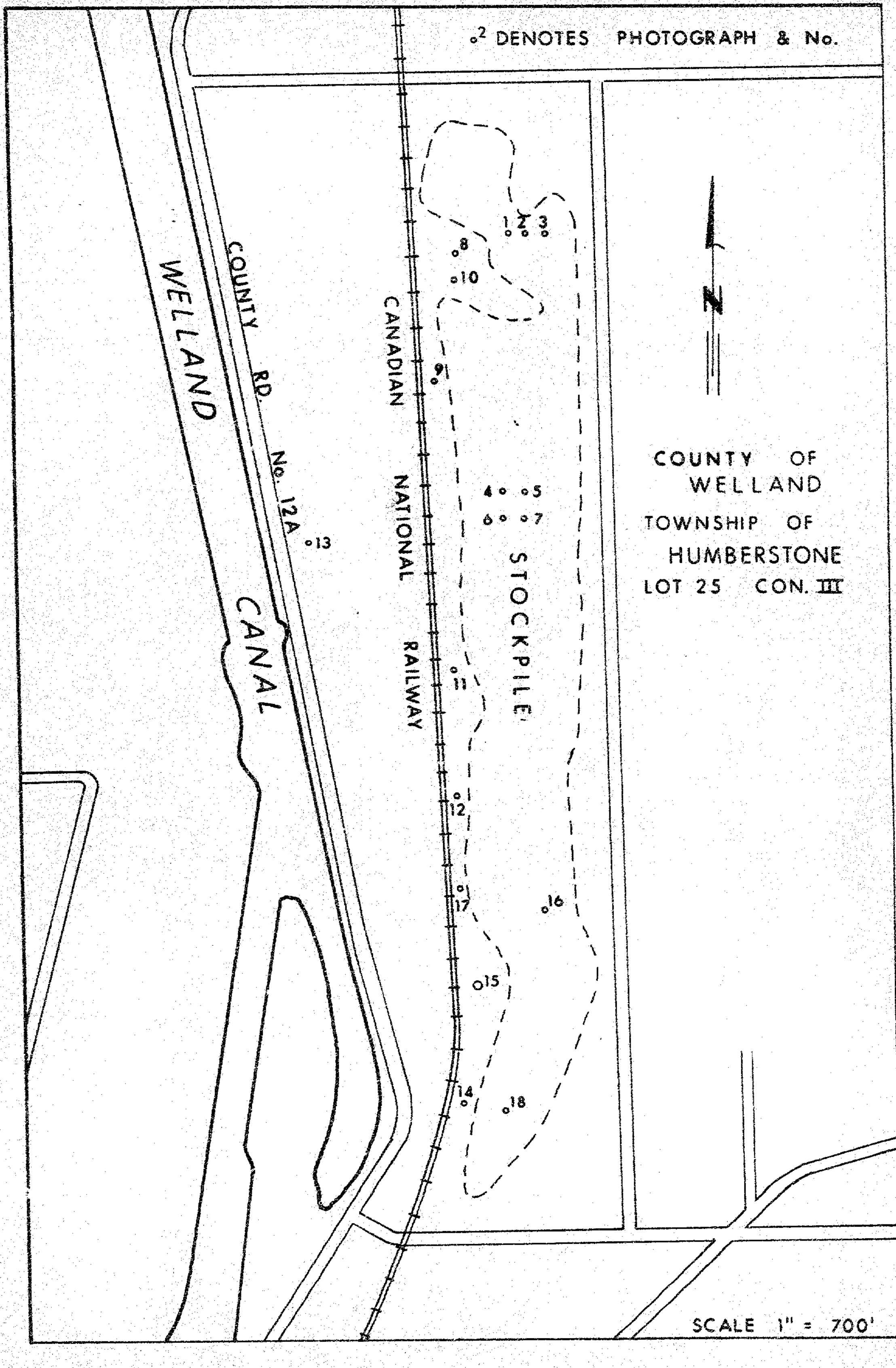
SUMMARY PLOT OF UNDRAINED SHEAR STRENGTH VS ELEVATION INTERBEDDED SILTY CLAY

FIGURE 9



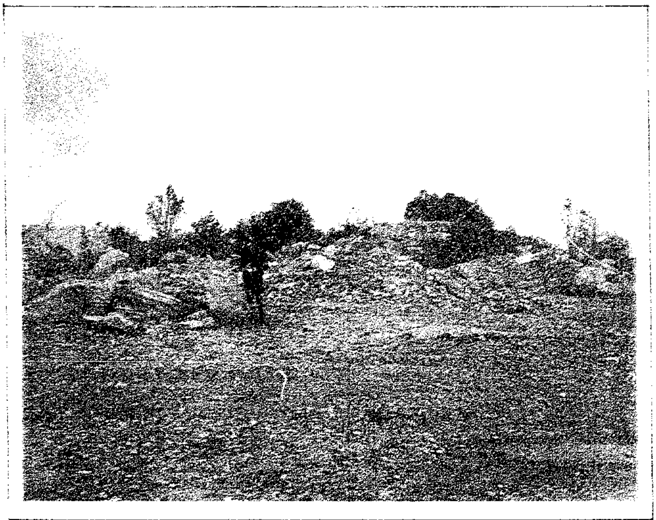
GOLDER & ASSOCIATES

Made J.A.
Chkd
Appd



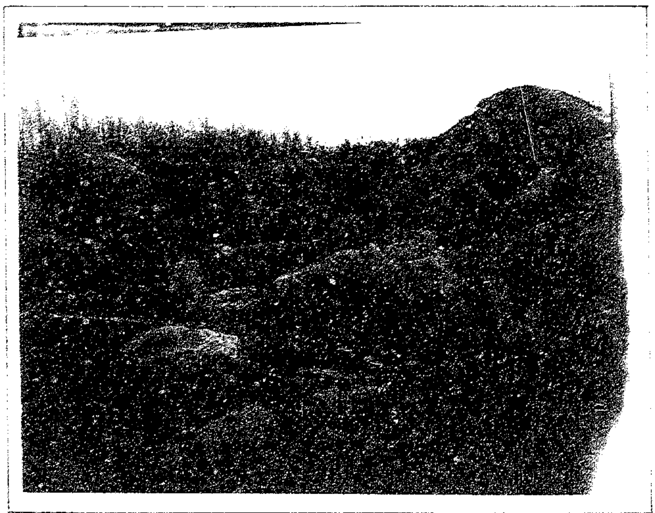
o2 DENOTES PHOTOGRAPH & No.
N
COUNTY OF WELLAND
TOWNSHIP OF HUMBERSTONE
LOT 25 CON. III
WELLAND CANAL
COUNTY RD. No. 12A
CANADIAN NATIONAL RAILWAY
STOCKPILE
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
SCALE 1" = 700'

1



2

3



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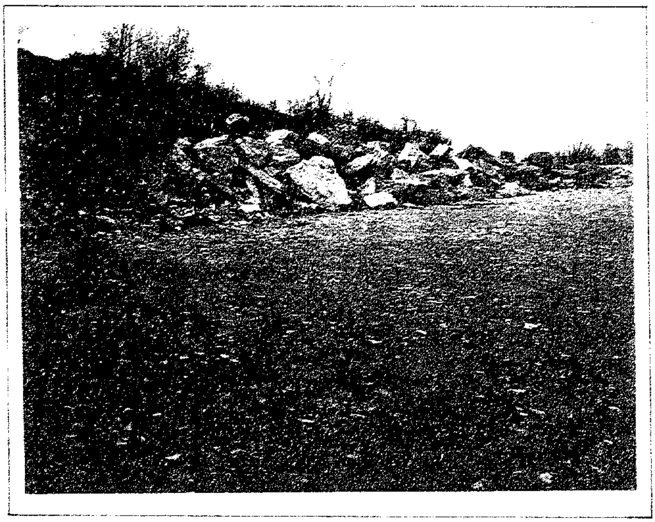
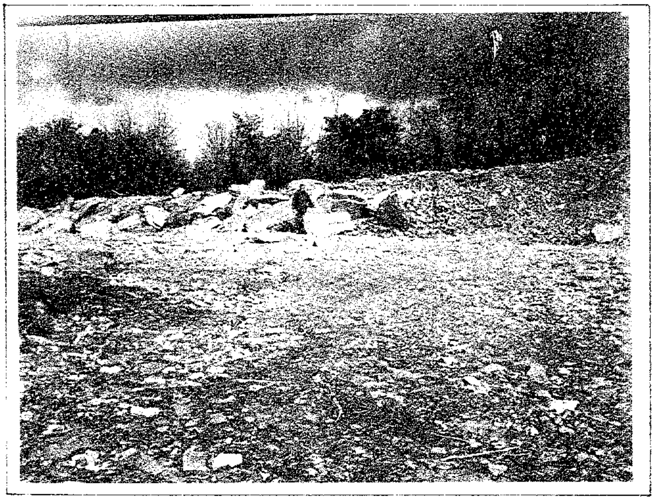
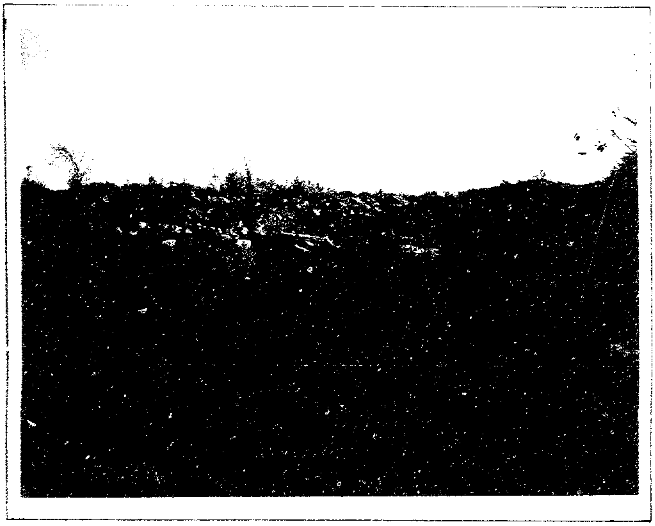
6



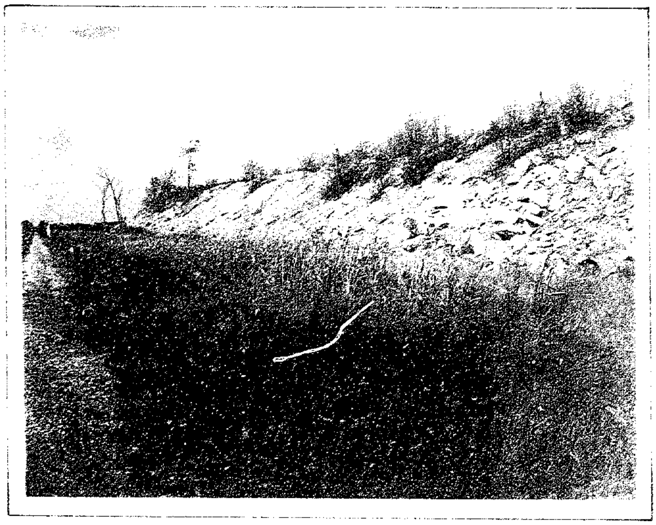
7



8



9



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16

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18

Problem Areas:

Sta 965+00 to Sta 1050+00 — General Limits.

i) Several, Retaining structures are situated between Sta 867+00 and 1054+00

ii) Bridges No's 13, 14, 15 & 16 are also situated within this limits in addition a syphon at Welland River crossing with existing Welland canal

Remidial Measures suggested.

a) Protection to all structures will be required as mentioned in [illegible]

b) Filling the entire width of the canal, ^to top^ for a ht of 15 ft from Sta 965+00 to Sta 1045+00 prior to drawdown will be required. This may amount to 700,000 c yds of material. Estimated that about 400,000 to 500,000 c yds of material is available on the west bank of the canal running north from Sta 866+00. By reducing the fill ht at the canal bottom to 12 ft, will reduce the quantity to 500,000 c yds. It is considered however sub-excavation will not be required prior to the placement of the fill in the canal.

Areas of no problem

Sta 800+00 to Sta 965+00 and Sta 1050+00 to Sta 1100+00

i) No protective measures during drawdown will be necessary between Sta 800+00 and Sta 965+00 except [illegible] part of East Bank where it forms a Dyke to adjacent Welland River.

ii) Subsoil conditions between Sta 1050+00 and 1100+00 are generally poor but banks are already flattened and therefore no special measures are required.

Suggested revision to Department of Highways, Ontario, Feasibilit
Report, East Main Street Crossing of the Welland Canal.

To go on Page 10 under Ground Water Conditions: Paragraph 2.

At the present time it appears that the bedrock/till aquifier is in direct communication with a large external groundwater source and is independent of the groundwater level in the overlying relatively impervious overburden. Lowering of the piezometric groundwater level in the aquifier may however, result in a minor downward migration of groundwater from the relatively impervious deposits. The effect, if any, of this downward movement of groundwater on the piezometric water level in the clayey material will be established after pumping tests are completed.

September 29, 1971

The Regional Municipality of Niagara
Public Works Department
P.O. Box 504
150 Berryman Avenue
St.Catharines, Ontario

Attn: Mr. K. Yu, P.Eng.
Design & Construction Engineer

RE: Golder Associates 1963 Soils
Report for Highway #406

Dear Sir:

Your letter of September 20, 1971 to Mr. G. K. Hunter regarding the above subject was sent to this Office for reply.

In 1963 the Consultant, H. Q. Golder and Associates Ltd., did carry out a field investigation along the proposed alignment of Hwy. 406 between the QEW and Westchester Avenue. However, after the completion of the field work the project was postponed and the Consultant was advised to discontinue his work. Consequently the collected information was never interpreted or analyzed. Only one copy containing the factual information was submitted to our Department. This report is available in our offices and you are welcome to peruse it at your convenience. Should you though wish to get a copy of this report we would advise you to contact the Consultant and make the necessary arrangements with him. We would not object to you getting this information provided it is for your use only and exclusively.

Sincerely yours,

AGStermac

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

AGS:mt

Kovich

TELEPHONE 685-1571

A STORMAC

THE REGIONAL MUNICIPALITY OF NIAGARA
PUBLIC WORKS DEPARTMENT
P.O. BOX 504
150 BERRYMAN AVENUE, ST. CATHARINES, ONTARIO

September 20th, 1971.

Department of Transportation
and Communications,
Central Region,
Downsview,
Ontario.

Attention: Mr. G. K. Hunter, P. Eng.
Manager of Regional
Systems Design.

REFERENCE: Golder Associates 1963 Soils
Report for Highway #406.

Dear Sir,

I understand that the Firm of Golder Associates was retained in 1963 by the then Department of Highways, Ontario to carry out a preliminary geo-technical investigation for various alternative routes of the proposed Highway #406. Some bore holes were put down in the vicinity of Regional Road #38 (Martindale Road) and Regional Road #77 (Wellandvale Road) immediately North and South of Wellandvale Road Bridge over Twelve Mile Creek respectively.

We are experiencing continuous maintenance problems with these two Regional Roads caused by slippage. In this regard we would appreciate receiving at your earliest convenience, two copies of the said Soils Report by Golder Associates and any additional soils information you may have in this area.

Yours very truly,

K. Yu, P. Eng.

Design & Construction
Engineer.

KY/pr

Foundations Section,
Design Services Branch,
Downsview, Ontario.

September 10, 1971.

H. G. Acres Ltd.,
1259 Dorchester Road,
Niagara Falls, Ontario.

Attention: Mr. A. L. McKechnie,
Executive Engineer.

Gentlemen:

Subject: Welland - Highway 406,
Functional Planning Study.

With respect to your letter of July 23, 1971 - P2223-00 - regarding the possibility of using slag on the above-mentioned project, we wish to submit the following information for your consideration.

Three different types of slag were investigated in our laboratory and the results of the various tests are summarized and attached. Based on these results we would conclude that the silico-manganese and the chrome slags are superior to the lime slag for the proposed use. Either of these could be used for backfilling the old canal and if properly placed and compacted should form a satisfactory foundation for the proposed expressway. There may still, however, be some reasons why these types of slag would not perform as desired and we would therefore suggest that you investigate further.

Yours truly,

AGS.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER.

AGS/ao
Attach.

cc: Foundations Files
Documents

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. A.G. Stermac,
Principal Foundation Engineer,
Foundations Office.

FROM: Chemical Section,
Materials and Testing Office.

ATTENTION:

DATE: August 10, 1971.

OUR FILE REF. 11-7-3

IN REPLY TO

SUBJECT: Slag Samples

Attached are the test results of three samples of slag analyzed at your request.

No heat loss could be determined for the silico-manganese and the chromium slags due to an increase in weight upon ignition. This weight increase was caused by oxidization of certain metallic compounds present in the slag, such as manganese and chromium. For the same reason the ignition loss of the lime slag is indicative only, as the actual figure was most probably somewhat higher (oxidation of the chromium in this sample would also have caused a certain gain in weight resulting in a somewhat lower total ignition weight loss).

Our laboratory does not believe that any "unfavourable" reactions would take place by using any of these slag samples with the water analyzed previously (report of July 21, 1971, Chemical File #11-7-5).

A.C.Suter,
Principal Chemical Engineer.

Per.

R. Sterk,
Chemical Engineer.

RS/mm
Attch.
c.c.file

Appendix

Test Results of Slag Samples

Type of Slag	Lime Slag	Silico Manganese Slag	Chrome Slag
Lab. No.	71-S-9989A	71-S-9990A	71-S-991A
% Silica (SiO_2)	25.44	35.56	35.52
% Alumina (Al_2O_3)	8.79	22.52	40.55
% Iron (Fe_2O_3)	0.81	ND	3.63
% Chromium (Cr_2O_3)	3.58	trace	5.38
% Calcium (CaO)	39.34	20.12	3.26
% Magnesia (MgO)	14.60	8.35	10.30
% Manganese (MnO)	NIL	11.10	trace
% Heat Loss at 900°C	7.47	-	-

NOTE: ND = Not Determined

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: A.G.Stermac,
Principal Foudation Engineer.

FROM: Rudy Sterk,
Chemical Engineer,
Chemical Section,
Materials and Testing Office.

ATTENTION:

DATE: July 21/71.

OUR FILE REF. 11-7-5

IN REPLY TO

SUBJECT: Water Sample
Welland Canal

Attached are the test results of a sample of water obtained from the Welland Canal and received on July 19/71. The sample was analyzed at your request to check on the presence of any compounds that may possibly react with the types of slag listed in the Appendix.

The test results showed that the water was quite"pure". For comparison, the chemical analyses of tap water from our Laboratory are also listed.

As can be noticed, the Welland Canal water showed quite similar concentrations for all the compounds tested. The slightly higher pH value was caused by the presence of carbonate ions (CO_3'').

The test results did not indicate that any"unfavourable" reactions would occur by the use of this water with any of the types slag listed.

A.C.Suter,
Principal Chemical Engineer.

Per.

R.Sterk,
Chemical Engineer.

RS/mb
c.c.files

APPENDIX

TEST RESULTS OF WELL WATER

Well Owner: ______________________________

Location: Welland Canal

Lab.No.	71-s-9769A	tap water
Sampling Date	July 19/71	-
ppm Chloride (Cl')	30	34
ppm Bi-carbonate (HCO_3')	100	110
ppm Carbonate (CO_3'')	6	trace
ppm Calcium (CaO)	42	46
ppm Magnesium (MgO)	27	15
ppm Sodium (Na)	ND	ND
ppm Iron (Fe)	0.1	NiL
ppm Total Hardness (as $CaCO_3$)	142	119
ppm Sulphate (SO_3)	15	20
pH	8.4	7.6
Appearance	clear	clear
Conductivity (mhos/cm)	1.801×10^{-4}	1.96×10^{-4}

Note: Total hardness of tap water in the Chemical Laboratory is approximately xxx ppm $CaCO_3$.

ND = Not Determined

July 14, 1971
P2223.00

HIGHWAY 406
FEASIBILITY STUDY

Table 1 - Slag Chemical Content

(1) - Lime Slag* (Grey - White)

SiO_2	25	-	28 per cent
Al_2O_3	5	-	6 per cent
CaO	55	-	57 per cent
MgO	8	-	9 per cent
FeO	2	-	5 per cent
Cr_2O_3	2	-	6 per cent

(2) - Silico Manganese Slag* (Green)

SiO_2	36	-	40 per cent
Al_2O_3	20	-	24 per cent
CaO	18	-	20 per cent
MgO	2	-	5 per cent
BaO	0	-	1 per cent
MnO	10	-	15 per cent

(3) - Chrome Slag* (Black)

MgO	25	-	27 per cent
CaO	3	-	4 per cent
SiO_2	34	-	36 per cent
Cr_2O_3	3	-	5 per cent
FeO	2	-	3 per cent
Al_2O_3	21	-	23 per cent

* Percentages indicates yearly average chemical content as supplied by Union Carbide.

Three samples of slag were tested in the laboratory and the results are tabulated as follows:

	Quality Tests % Loss		Physical Characteristics			
				Specific Gravity		
	$MgSO_4$	Los Angeles Abrasion	Absorption	Bulk (Dry)	Apparent	Grain Size
60067 Lime Slag	0.7	Too Soft	14.93	2.034	2.922	Crushed Material 2" Chunks to Dust
60068 Silico-Manganese Slag	0	17.9	0.77	3.100	3.176	Crushed Material 6" to 4" Chunks
60069 Chrome Slag	0	17.7	0.765	2.966	3.035	Crushed Material 3" Chunks (Some Dust)

Tony.

Of these three materials - I would surmise that the Silico-Manganese (2) and Chrome (3) Slags would be suitable materials i.e.

Low absorption.
Good Petrographic evaluation.
18% loss on L.A.
Presumably little reaction with $MgSO_4$. (Results to come)

The Lime Slag (1) is very poor - 15% absorption, no Petrographic required (all bad) and will almost certainly have an extremely high loss in $MgSO_4$ NO

Perhaps it could be used as fill material if there were no possibility of any freeze-thaw action occurring.

Rudy has the samples and will presumably pass the results on to you, and I will instruct the Aggregate Lab to pass the SO_4 results to you.

Alan.

July 23rd.

FORM OB-MT-84

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND TESTING OFFICE

COARSE AGGREGATE PETROGRAPHIC ANALYSIS

PIT NAME ______ LAB. NO. 71-B- 60069

DATE July 22/71 FRACTION 5/8 - 1/2 - 3/8 ANALYST D. SUMMERFIELD

TYPE NO.	TYPE	WEIGHT	%
1	CARBONATES (hard)		
2	CARBONATES (sandy, hard)		
3	SANDSTONE (hard)		
4	GNEISS (hard)		
5	QUARTZITE (coarse grained)		
6	GREYWACKE - ARKOSE		
7	VOLCANIC (slightly weathered)		
8	GRANITE - DIORITE		
9	TRAP		
	HARD SLAG	755	75.5
			75.5
20	CARBONATES (slightly weathered)		
21	CARBONATES (sandy, medium hard)		
22	SANDSTONE (medium hard)		
23	CARBONATES CRYSTALLINE (hard)		
24	CARBONATES CRYSTALLINE (slightly weathered)		
25	GNEISS (soft)		
26	CHERT - CHERTY CARBONATES		
27	GRANITE (brittle)		
28	VOLCANIC (soft)		
	HARD + SLIGHTLY PITTED SLAG	172	17.2
			17.2
40	CARBONATES (soft, slightly shaley)		
41	CARBONATES (sandy, soft)		
42	CARBONATES (deeply weathered)		
43	CARBONATES (shaley or clayey)		
44	CARBONATES (ochreous)		
45	CHERT - CHERTY CARBONATES (light leached)		
46	SANDSTONE (soft friable)		
47	QUARTZITE (fine grained)		
48	VOLCANIC (very soft, porous)		
49	CARBONATES CRYSTALLINE (soft)		
50	GNEISS (friable)		
51	GRANITE (friable)		
52	ENCRUSTATION		
53	CEMENTATIONS		
54	CEMENTATIONS (total)		
55	SCHIST (soft)		
	MODERATELY PITTED, POROUS SLAG	67	6.7
			6.7
60	OCHRE		
61	SHALE		
62	CLAY		
63	VOLCANIC (decomposed)		
	HEAVILY PITTED, FRIABLE, POROUS SLAG	6	0.6
	SLAG		
			0.6
	TOTALS	1000	100.0

ESTIMATED PERCENT CRUSHED	100
ESTIMATED PERCENT FLATS AND ELONGATED	15

BASIC PETROGRAPHIC NO. ______

JUL 23 1971

FORM OB-MT-294
DECEMBER, 1964

$MgSO_4$ TEST — COARSE AGGREGATE EXAMINATION

SAMPLE Nº 71-B 6006 ? PIT NAME ______ P.N. ______
RUN Nº 24 BASKET Nº 11 ANALYST ______ DATE AUG - 3 1971

$MgSO_4$ SOUNDNESS TEST 5 CYCLES

GRADE	SIEVE SIZE PASS	SIEVE SIZE RET.	LOSS DETERMINED ON SIEVE SIZE D.H.O.	LOSS DETERMINED ON SIEVE SIZE ASTM.	WEIGHT BEFORE gms.	WEIGHT AFTER gms.	GRAMS LOSS D.H.O.	GRAMS LOSS ASTM.	% LOSS D.H.O.	% LOSS ASTM.	GRADING	WEIGHTED % LOSS D.H.O.	WEIGHTED % LOSS ASTM.
B	3/4	3/8"	3/8"	5/16"	1000	1000 0 / 0			0.0	0.0	73.4		
C	3/8"	#4	#4	#5	300	298 2 / 0			0.7	0.0	26.6		
										TOTALS	100.0	0.0	0.0

	"B" PORTION TOTAL WT. BEFORE RESIEVING 1000 gms.				"C" PORTION TOTAL WT. BEFORE RESIEVING 300 gms.			
EXAMINATION OF SAMPLE BEFORE RESIEVING — EST. % AFFECTED								
EST. BREAKDOWN	LIGHT	MED.	SEVERE	ROCK TYPE	LIGHT	MED.	SEVERE	ROCK TYPE
BY SPLITTING								
CRUMBLING								
CRACKING								
FLAKING								
DISINTEGRATION								

EXAMINATION OF RETAINED PORTION OF SAMPLE AFTER RESIEVING

	RETAINED 3/8"	RETAINED #4
EST. % SEVERELY AFFECTED		
PRINCIPAL MATERIALS APPARENTLY RESPONSIBLE FOR LOSS		
MATERIALS RESPONSIBLE FOR HIGH P.N. BUT UNAFFECTED BY $MgSO_4$		

EXAMINATION OF SAMPLE

	PASS 3/8" RETAINED 5/16"				PASS #4 RETAINED #5			
EST. % OF SOUND MATERIAL								
PARTICLE SHAPE - EST%	0-10	10-20	20-30	OVER 30	0-10	10-20	20-30	OVER 30
FLAT & ELONGATED								
ANGULAR								
ROUNDED OR IRREGULAR								

REMARKS: ______

DEPARTMENT OF HIGHWAYS — ONTARIO

COARSE AGGREGATE

OVER

ABRASION TESTS

TYPE	EST. % CRUSH	GRADE	RET. Nº 12	PASS Nº 12	% LOSS
LOS ANGELES (500 REVS.)	100	B			17.7

ABSORPTION %	a	b	BULK S.G. (DRY)	a	b	BULK S.G. (SAT. SUR. DRY)	a	b	APPARENT S.G.	a	b
B			B			B			A		
A			C			C			C		
B−A			B−C			B−C			A−C		
$\frac{B-A}{A} \times 100$	0.860 %	0.730 %	$\frac{A}{B-C}$	2.961	2.970	$\frac{B}{B-C}$			$\frac{A}{A-C}$	3.033	3.036
AVG.	0.765 %		AVG.	2.966		AVG.			AVG.	3.035	

PERCENT COARSE AND FINE AGGREGATES		
LAB. CRUSHED	~~AS RECEIVED~~	LAB. CRUSHED
WT. TOTAL SAMPLE (DRY) W_1	48.5	
WT. SAMPLE RET. #4 (DRY) W_2	38.8	
WT. SAMPLE PASS #4 (DRY) W_3	9.7	
% COARSE AGGREGATE	80.0	
% FINE AGGREGATE	20.0	

UNIT WT.		
	LOOSE	lb/cu. ft.
	COMPACTED	lb/cu. ft.
CLAY LUMPS		%
LOSS BY WASHING PASS # 200		%
FLAT & ELONGATED PARTICLES		%
CRUSHED PARTICLES		%
PASS # 200 OR # 270 (TOTAL SAMPLE)		%

W_2 ~~AS RECEIVED~~ LAB. CRUSHED

DEPT. SIEVE DESIGNATION	INDIVIDUAL WEIGHT	CUMULATIVE WEIGHT	% RETAINED RET. # 4	% RETAINED TOTAL
4"				
3"				
2½"				
2"				
1½"				
1"				
⅞"	0.6		1.5	1.2
¾"	1.4		5.2	4.1
⅝"	4.9		17.8	14.2
½"	8.9		40.7	32.6
⅜"	12.7		73.4	58.8
# 3	6.5		90.2	72.2
# 4	3.8		100.0	80.0
TOTAL				

W_2 LAB. CRUSHED

DEPT. SIEVE DESIGNATION	INDIVIDUAL WEIGHT	CUMULATIVE WEIGHT	% RETAINED RET. # 4	% RETAINED TOTAL
2½"				
2"				
1½"				
1¼"				
1"				
⅞"				
¾"				
⅝"				
½"				
⅜"				
# 3				
# 4				
TOTAL				

FINE AGGREGATE

DEPT. SIEVE DESIG.	W_3 AS RECEIVED CUMULATIVE WEIGHT GM.	% RETAINED PASS # 4	% RETAINED TOTAL	W_3 LAB. CRUSHED CUMULATIVE WEIGHT GM.	% RETAINED PASS # 4	% RETAINED TOTAL
# 8						
# 16						
# 30						
# 50						
# 100						
# 200						
# 270						
PASS #200 #270 A						
PASS #200 #270 B						
DECANT						
TOTAL						

Mg SO4 SOUNDNESS TEST ______ CYCLES					
WEIGHT BEFORE GM.	WEIGHT AFTER GM.	GRAMS LOSS	PERCENT LOSS	GRADING	WEIGHTED % LOSS
			TOTALS		

LOSS BY ABRASION		%
UNIT WEIGHT	LOOSE	lb./cu.ft.
	COMPACTED	lb./cu.ft.
FINENESS MODULUS		
ORGANIC IMPURITIES		
PASS #200 OR #270 (PASS #4 FRACTION)		%

ABSORPTION %	a	°F	b	°F	BULK S.G. (DRY)	a	b	BULK S.G. (SAT. SUR. DRY)	a	b	APPARENT S.G.	a	b
B					V			V			V−W		
A					W			W			B−A		
S−A					V−W			V−W			(V−W)−(B−A)		
$\frac{B-A}{A} \times 100$		%		%	$\frac{A}{V-W}$			$\frac{B}{V-W}$			$\frac{A}{(V-W)-(B-A)}$		
AVG.				%	AVG.			AVG.			AVG.		

LABORATORY REMARKS ______________________________

N.B. Results to Al Hanks.

W. P. No.
CONT. No.
LAB. No. 71-B-60069
DATE REQ'D.

JUL 19 1971

COARSE AGGREGATE TEST			
GRADING	R		
$MgSO_4$	R		
P. N.	R		
L. A.	R		
ABS. & S.G.	R		
% CRUSHED			
% FLATS			
WASH PASS #200			
PROCTOR			

FINE AGGREGATE & SOIL TEST			
GRADING			
$MgSO_4$			
P. N.			
ABRASION			
ABS. & S.G.			
ORGANIC			
STRUCT. STR.			
SPEC. SIEVE			
HYDROMETER			
P.I.			
PROCTOR			
MOISTURE			
SOIL S.G.			

JUL 20 1971

OVER

July 14, 1971
P2223.00

HIGHWAY 406
FEASIBILITY STUDY

Table 1 - Slag Chemical Content

(1) - Lime Slag* (Grey - White) 60067 MISC 520

SiO_2	25	-	28 per cent
Al_2O_3	5	-	6 per cent
CaO	55	-	57 per cent
MgO	8	-	9 per cent
FeO	2	-	5 per cent
Cr_2O_3	2	-	6 per cent

(2) - Silico Manganese Slag* (Green) 60068 MISC 521

SiO_2	36	-	40 per cent
Al_2O_3	20	-	24 per cent
CaO	18	-	20 per cent
MgO	2	-	5 per cent
BaO	0	-	1 per cent
MnO	10	-	15 per cent

(3) - Chrome Slag* (Black) 60069 MISC 522

MgO	25	-	27 per cent
CaO	3	-	4 per cent
SiO_2	34	-	36 per cent
Cr_2O_3	3	-	5 per cent
FeO	2	-	3 per cent
Al_2O_3	21	-	23 per cent

* Percentages indicates yearly average chemical content as supplied by Union Carbide.

Unit Weights.

Quality tests ?

Compaction tests

Crush check — Timmins

Olson

FORM OSMT-144
REV. JULY 1969

OVER

SPECIMEN	MORTAR STRENGTH P.S.I.								
	7 DAY			14 DAY			28 DAY		
	STANDARD	SAMPLE	$\frac{\text{SAMPLE}}{\text{STANDARD}}$ %	STANDARD	SAMPLE	$\frac{\text{SAMPLE}}{\text{STANDARD}}$ %	STANDARD	SAMPLE	$\frac{\text{SAMPLE}}{\text{STANDARD}}$ %
1									
2									
3									
AVERAGE									

% LOSS BY WASHING OF COARSE AGGREGATE	
(a) % PASS # 4 (TOTAL SAMPLE)	
(b) % LOSS BY WASHING PASS # 200 (TOTAL SAMPLE)	
(c) PERMISSIBLE % PASS # 200 (PASS # 4 FRACTION)	
(d) PERMISSIBLE % PASS # 200 (RETAINED # 4 FRACTION)	
(e) CORRECTION FOR PASS # 4 FRACTION ($a \times \frac{c-d}{100}$)	
% LOSS BY WASHING PASS # 200 (RET. # 4 FRACTION) (b-e)	

MOISTURE CONTENT	a	b
WET WT. & DISH		
DRY WT. & DISH		
WT. OF MOISTURE		
WT. OF DISH		
DRY WT.		
% MOISTURE		

COMPACTION RESULTS

MAXIMUM WET DENSITY ____________ LB./CU. FT.
MAXIMUM DRY DENSITY ____________ LB./CU. FT.
OPTIMUM MOISTURE ____________ %
FIELD WET DENSITY ____________ LB./CU. FT.
FIELD DRY DENSITY ____________ LB./CU. FT.
FIELD MOISTURE ____________ %
% COMPACTION ____________

ATTERBERG LIMITS

LIQUID LIMIT ____________ %
PLASTIC LIMIT ____________ %
PLASTICITY INDEX ____________ %

GROUP INDEX

____

SPECIAL TESTS ____________

OFFICE ENGINEER'S RECOMMENDATIONS ____________

DATE ____________ SIGNATURE ____________

LEGEND:

A.S.C. — ACCEPTABLE FOR USE AS SAND CUSHION.
N.A.S.C. — NOT ACCEPTABLE FOR USE AS SAND CUSHION.
A.G. — ACCEPTABLE FOR USE AS GRANULAR BORROW.
N.A.G. — NOT ACCEPTABLE FOR USE AS GRANULAR BORROW.
M — SAMPLE IS ____________ FOR USE AS G.B.C. CLASS "B"
K — SAMPLE IS ____________ FOR USE AS G.B.C. CLASS "A"
X — ACCEPTABLE
N — NOT ACCEPTABLE
A — SAMPLE OF FINE AND COARSE AGGREGATE IS ____________
B — COARSE AGGREGATE PORTION OF SAMPLE IS ____________
C — FINE AGGREGATE PORTION OF SAMPLE IS ____________
D — SAMPLE OF COARSE AGGREGATE IS ____________
E — SAMPLE OF FINE AGGREGATE IS ____________

W.P. Nº. ____________

CONT. Nº. ____________

LAB. Nº. 71-B-60069

FORM OB-MT-84

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND TESTING OFFICE

COARSE AGGREGATE PETROGRAPHIC ANALYSIS

PIT NAME ______ LAB. NO. 71-B- 60068

DATE July 22/71 FRACTION 5/4 - 1/2 - 3/8 ANALYST D. SUMMERFIELD

TYPE NO.	TYPE	WEIGHT	%
1	CARBONATES (hard)		
2	CARBONATES (sandy, hard)		
3	SANDSTONE (hard)		
4	GNEISS (hard)		
5	QUARTZITE (coarse grained)		
6	GREYWACKE - ARKOSE		
7	VOLCANIC (slightly weathered)		
8	GRANITE - DIORITE		
9	TRAP		
	HARD SLAG	935	93.2
			93.2
20	CARBONATES (slightly weathered)		
21	CARBONATES (sandy, medium hard)		
22	SANDSTONE (medium hard)		
23	CARBONATES CRYSTALLINE (hard)		
24	CARBONATES CRYSTALLINE (slightly weathered)		
25	GNEISS (soft)		
26	CHERT - CHERTY CARBONATES		
27	GRANITE (brittle)		
28	VOLCANIC (soft)		
	HARD + SLIGHTLY PITTED SLAG	57	5.7
			5.7
40	CARBONATES (soft, slightly shaley)		
41	CARBONATES (sandy, soft)		
42	CARBONATES (deeply weathered)		
43	CARBONATES (shaley or clayey)		
44	CARBONATES (ochreous)		
45	CHERT - CHERTY CARBONATES (light leached)		
46	SANDSTONE (soft friable)		
47	QUARTZITE (fine grained)		
48	VOLCANIC (very soft, porous)		
49	CARBONATES CRYSTALLINE (soft)		
50	GNEISS (friable)		
51	GRANITE (friable)		
52	ENCRUSTATION		
53	CEMENTATIONS		
54	CEMENTATIONS (total)		
55	SCHIST (soft)		
	MODERATELY PITTED, POROUS SLAG	11	1.1
			1.1
60	OCHRE		
61	SHALE		
62	CLAY		
63	VOLCANIC (decomposed)		
	SLAG		
	TOTALS	1003	100.0

ESTIMATED PERCENT CRUSHED	100	
ESTIMATED PERCENT FLATS AND ELONGATED	15	BASIC PETROGRAPHIC NO. [illegible]

JUL 23 1971

FORM OB-MT-294
DECEMBER, 1964

DEPARTMENT OF HIGHWAYS ONTARIO

$MgSO_4$ TEST — COARSE AGGREGATE EXAMINATION

SAMPLE № 71-B 60068 PIT NAME ______ P.N. ______
RUN № 24 BASKET № 10 ANALYST ______ DATE AUG - 3 1971

$MgSO_4$ SOUNDNESS TEST 5 CYCLES													
GRADE	SIEVE SIZE		LOSS DETERMINED ON SIEVE SIZE		WEIGHT BEFORE gms.	WEIGHT AFTER gms.	GRAMS LOSS		% LOSS		GRADING	WEIGHTED % LOSS	
	PASS	RET.	D.H.O.	ASTM.			D.H.O.	ASTM.	D.H.O.	ASTM.		D.H.O.	ASTM.
B	3/4	3/8"	3/8"	5/16"	1000	995 / 4 / 1			0.5	0.1	74.8		
C	3/8"	#4	#4	#5	300	300 / 0 / 0			0.0	0.0	25.2		
										TOTALS	100.0	0.0	0.0

EXAMINATION OF SAMPLE BEFORE RESIEVING	"B" PORTION TOTAL WT. BEFORE RESIEVING 1000 gms				"C" PORTION TOTAL WT. BEFORE RESIEVING 300 gms.			
EST. % AFFECTED								
EST. BREAKDOWN	LIGHT	MED.	SEVERE	ROCK TYPE	LIGHT	MED.	SEVERE	ROCK TYPE
BY SPLITTING								
CRUMBLING								
CRACKING								
FLAKING								
DISINTEGRATION								

EXAMINATION OF RETAINED PORTION OF SAMPLE AFTER RESIEVING										
EST. % SEVERELY AFFECTED	RETAINED 3/8"					RETAINED #4				
PRINCIPAL MATERIALS APPARENTLY RESPONSIBLE FOR LOSS	RETAINED 3/8"					RETAINED #4				
MATERIALS RESPONSIBLE FOR HIGH P.N. BUT UNAFFECTED BY $MgSO_4$	RETAINED 3/8"					RETAINED #4				
EST. % OF SOUND MATERIAL	RETAINED 5/16"					RETAINED #5				
PARTICLE SHAPE - EST %	PASS 3/8" RETAINED 5/16"	0 – 10	10 – 20	20 – 30	OVER 30	PASS #4 RETAINED #5	0 – 10	10 – 20	20 – 30	OVER 30
FLAT & ELONGATED										
ANGULAR										
ROUNDED OR IRREGULAR										

REMARKS: ______

FORM OBMT-144
REV. JULY 1969

OVER

SPECIMEN	MORTAR		STRENGTH		P. S. I.				
	7 DAY			14 DAY			28 DAY		
	STANDARD	SAMPLE	SAMPLE/STANDARD %	STANDARD	SAMPLE	SAMPLE/STANDARD %	STANDARD	SAMPLE	SAMPLE/STANDARD %
1									
2									
3									
AVERAGE									

% LOSS BY WASHING OF COARSE AGGREGATE	
(a) % PASS # 4 (TOTAL SAMPLE)	
(b) % LOSS BY WASHING PASS # 200 (TOTAL SAMPLE)	
(c) PERMISSIBLE % PASS # 200 (PASS # 4 FRACTION)	
(d) PERMISSIBLE % PASS # 200 (RETAINED # 4 FRACTION)	
(e) CORRECTION FOR PASS # 4 FRACTION $(a \times \frac{c-d}{100})$	
% LOSS BY WASHING PASS # 200 (RET. # 4 FRACTION) (b-e)	

MOISTURE CONTENT	a	b
WET WT. & DISH		
DRY WT. & DISH		
WT. OF MOISTURE		
WT. OF DISH		
DRY WT.		
% MOISTURE		

COMPACTION RESULTS

MAXIMUM WET DENSITY ________ LB./CU. FT.
MAXIMUM DRY DENSITY ________ LB./CU. FT.
OPTIMUM MOISTURE ________ %
FIELD WET DENSITY ________ LB./CU. FT.
FIELD DRY DENSITY ________ LB./CU. FT.
FIELD MOISTURE ________ %
% COMPACTION ________

ATTERBERG LIMITS

LIQUID LIMIT ________ %
PLASTIC LIMIT ________ %
PLASTICITY INDEX ________ %

GROUP INDEX

____

SPECIAL TESTS ________

OFFICE ENGINEER'S RECOMMENDATIONS ________

DATE ________ SIGNATURE ________

LEGEND:

A.S.C. — ACCEPTABLE FOR USE AS SAND CUSHION.
N.A.S.C. — NOT ACCEPTABLE FOR USE AS SAND CUSHION
A.G. — ACCEPTABLE FOR USE AS GRANULAR BORROW.
N.A.G. — NOT ACCEPTABLE FOR USE AS GRANULAR BORROW.
M — SAMPLE IS ________ FOR USE AS G.B.C. CLASS "B"
K — SAMPLE IS ________ FOR USE AS G.B.C. CLASS "A"
X — ACCEPTABLE
N — NOT ACCEPTABLE
A — SAMPLE OF FINE AND COARSE AGGREGATE IS ________
B — COARSE AGGREGATE PORTION OF SAMPLE IS ________
C — FINE AGGREGATE PORTION OF SAMPLE IS ________
D — SAMPLE OF COARSE AGGREGATE IS ________
E — SAMPLE OF FINE AGGREGATE IS ________

W.P. Nº.

CONT. Nº.

LAB. Nº. 71-B-60068

COMPUTATION FORM GRANULAR MATERIALS

DEPARTMENT OF HIGHWAYS — ONTARIO

COARSE AGGREGATE

OVER

ABRASION TESTS

TYPE	EST. % CRUSH	GRADE	RET. Nº 12	PASS Nº 12	% LOSS
LOS ANGELES (500 REVS.)	100	B			17.9

ABSORPTION %	a	b	BULK S.G. (DRY)	a	b	BULK S.G.(SAT. SUR.DRY)	a	b	APPARENT S.G.	a	b
B			B			B			A		
A			C			C			C		
B−A			B−C			B−C			A−C		
$\frac{B-A}{A}$ x100	0.750%	0.790%	$\frac{A}{B-C}$	3.106	3.093	$\frac{B}{B-C}$			$\frac{A}{A-C}$	3.181	3.171
AVG.	0.770 %		AVG.	3.100		AVG.			AVG.	3.176	

PERCENT COARSE AND FINE AGGREGATES	~~AS RECEIVED~~ LAB. CRUSHED	LAB. CRUSHED
WT. TOTAL SAMPLE (DRY) W_1	37.0	
WT. SAMPLE RET. #4 (DRY) W_2	30.2	
WT. SAMPLE PASS #4 (DRY) W_3	6.8	
% COARSE AGGREGATE	81.6	
% FINE AGGREGATE	18.4	

UNIT WT.	
LOOSE	lb/cu. ft.
COMPACTED	lb/cu. ft.
CLAY LUMPS	%
LOSS BY WASHING PASS # 200	%
FLAT & ELONGATED PARTICLES	%
CRUSHED PARTICLES	%
PASS #200 OR #270 (TOTAL SAMPLE)	%

W_2 ~~AS RECEIVED~~ LAB. CRUSHED

DEPT. SIEVE DESIGNATION	INDIVIDUAL WEIGHT	CUMULATIVE WEIGHT	% RETAINED RET. # 4	% RETAINED TOTAL
4"				
3"				
2½"				
2"				
1½"				
1"				
⅞"	0.5		1.6	1.4
¾"	1.3		6.0	4.9
⅝"	4.3		20.2	16.5
½"	7.1		43.7	35.7
⅜"	9.4		74.8	61.1
#3	4.7		90.4	73.8
#4	2.9		100.0	81.6
TOTAL				

W_2 LAB. CRUSHED

DEPT. SIEVE DESIGNATION	INDIVIDUAL WEIGHT	CUMULATIVE WEIGHT	% RETAINED RET. # 4	% RETAINED TOTAL
2½"				
2"				
1½"				
1¼"				
1"				
⅞"				
¾"				
⅝"				
½"				
⅜"				
#3				
#4				
TOTAL				

FINE AGGREGATE

DEPT. SIEVE DESIG.	W_3 AS RECEIVED CUMULATIVE WEIGHT GM.	% RETAINED PASS #4	% RETAINED TOTAL	W_3 LAB. CRUSHED CUMULATIVE WEIGHT GM.	% RETAINED PASS # 4	% RETAINED TOTAL
# 8						
# 16						
# 30						
# 50						
#100						
#200						
#270						
PASS #200 #270 A						
PASS #200 #270 B						
DECANT						
TOTAL						

Mg SO4 SOUNDNESS TEST ______ CYCLES

WEIGHT BEFORE GM.	WEIGHT AFTER GM.	GRAMS LOSS	PERCENT LOSS	GRADING	WEIGHTED % LOSS
			TOTALS		

LOSS BY ABRASION	%
UNIT WEIGHT LOOSE	lb./cu.ft.
UNIT WEIGHT COMPACTED	lb./cu.ft.
FINENESS MODULUS	
ORGANIC IMPURITIES	
PASS #200 OR #270 (PASS #4 FRACTION)	%

ABSORPTION %	a °F	b °F	BULK S.G. (DRY)	a	b	BULK S.G. (SAT. SUR.DRY)	a	b	APPARENT S.G.	a	b
B			V			V			V−W		
A			W			W			B−A		
S−A			V−W			V−W			(V−W)−(B−A)		
$\frac{B-A}{A}$ x100	%	%	$\frac{A}{V-W}$			$\frac{B}{V-W}$			$\frac{A}{(V-W)-(B-A)}$		
AVG.		%	AVG.			AVG.			AVG.		

LABORATORY REMARKS ______

N.B. Results to Al Hanks.

W. P. No.

CONT. No.

LAB. No. 71-B-60068

DATE REQ'D.

JUL 19 1971

COARSE AGGREGATE				FINE AGGREGATE & SOIL			
TEST				TEST			
GRADING	R			GRADING			
$MgSO_4$	R			$MgSO_4$			
P. N.	R			P. N.			
L. A.	R			ABRASION			
ABS. & S.G.	R			ABS. & S.G.			
% CRUSHED				ORGANIC			
% FLATS				STRUCT. STR.			
WASH PASS #200				SPEC. SIEVE			
PROCTOR							
				HYDROMETER			
				P. I.			
				PROCTOR			
				MOISTURE			
				SOIL S.G.			

FORM OBMT-14-1

FORM OB-MT-294
DECEMBER, 1964

DEPARTMENT OF HIGHWAYS ONTARIO

$MgSO_4$ TEST — COARSE AGGREGATE EXAMINATION

SAMPLE Nº 71B 60067 PIT NAME ______ P.N. ______
RUN Nº 24 BASKET Nº 9 ANALYST ______ DATE AUG - 3 1971

$MgSO_4$ SOUNDNESS TEST 5 CYCLES

GRADE	SIEVE SIZE PASS	SIEVE SIZE RET.	LOSS DETERMINED ON SIEVE SIZE D.H.O.	LOSS DETERMINED ON SIEVE SIZE ASTM.	WEIGHT BEFORE gms.	WEIGHT AFTER gms.	GRAMS LOSS D.H.O.	GRAMS LOSS ASTM.	% LOSS D.H.O.	% LOSS ASTM.	GRADING	WEIGHTED % LOSS D.H.O.	WEIGHTED % LOSS ASTM.
	3/4												
B	1/2	3/8"	3/8"	5/16"	330	327/0/3			0.9	0.9			
C	3/8"	#4	#4	#5	300	298/0 2			0.7	0.7			
										TOTALS		0.7	0.7

EXAMINATION OF SAMPLE BEFORE RESIEVING	"B" PORTION TOTAL WT. BEFORE RESIEVING 330 gms.				"C" PORTION TOTAL WT. BEFORE RESIEVING 300 gms.			
EST. % AFFECTED								
EST. BREAKDOWN	LIGHT	MED.	SEVERE	ROCK TYPE	LIGHT	MED.	SEVERE	ROCK TYPE
BY SPLITTING								
CRUMBLING								
CRACKING								
FLAKING								
DISINTEGRATION								

EXAMINATION OF RETAINED PORTION OF SAMPLE AFTER RESIEVING

	RETAINED 3/8"	RETAINED #4
EST. % SEVERELY AFFECTED		
PRINCIPAL MATERIALS APPARENTLY RESPONSIBLE FOR LOSS		
MATERIALS RESPONSIBLE FOR HIGH P.N. BUT UNAFFECTED BY $MgSO_4$		

EXAMINATION OF SAMPLE

	RETAINED 5/16" PASS 3/8" 0-10	10-20	20-30	OVER 30	RETAINED #5 PASS #4 0-10	10-20	20-30	OVER 30
EST. % OF SOUND MATERIAL								
PARTICLE SHAPE-EST%	0-10	10-20	20-30	OVER 30	0-10	10-20	20-30	OVER 30
FLAT & ELONGATED								
ANGULAR								
ROUNDED OR IRREGULAR								

REMARKS:

FORM OSMT-144
REV. JULY 1969

OVER

SPECIMEN	MORTAR ____ STRENGTH P.S.I.								
	7 DAY			14 DAY			28 DAY		
	STANDARD	SAMPLE	SAMPLE/STANDARD %	STANDARD	SAMPLE	SAMPLE/STANDARD %	STANDARD	SAMPLE	SAMPLE/STANDARD %
1									
2									
3									
AVERAGE									

% LOSS BY WASHING OF COARSE AGGREGATE	
(a) % PASS # 4 (TOTAL SAMPLE)	
(b) % LOSS BY WASHING PASS # 200 (TOTAL SAMPLE)	
(c) PERMISSIBLE % PASS # 200 (PASS # 4 FRACTION)	
(d) PERMISSIBLE % PASS # 200 (RETAINED # 4 FRACTION)	
(e) CORRECTION FOR PASS # 4 FRACTION $(a \times \frac{c-d}{100})$	
% LOSS BY WASHING PASS # 200 (RET. # 4 FRACTION) (b−e)	

MOISTURE CONTENT	a	b
WET WT. & DISH		
DRY WT. & DISH		
WT. OF MOISTURE		
WT. OF DISH		
DRY WT.		
% MOISTURE		

COMPACTION RESULTS

MAXIMUM WET DENSITY ________ LB./CU. FT.
MAXIMUM DRY DENSITY ________ LB./CU. FT.
OPTIMUM MOISTURE ________ %
FIELD WET DENSITY ________ LB./CU. FT.
FIELD DRY DENSITY ________ LB./CU. FT.
FIELD MOISTURE ________ %
% COMPACTION ________

ATTERBERG LIMITS

LIQUID LIMIT ________ %
PLASTIC LIMIT ________ %
PLASTICITY INDEX ________ %

GROUP INDEX

____

SPECIAL TESTS ________

OFFICE ENGINEER'S RECOMMENDATIONS ________

DATE ________ SIGNATURE ________

LEGEND:

A.S.C. — ACCEPTABLE FOR USE AS SAND CUSHION.
N.A.S.C. — NOT ACCEPTABLE FOR USE AS SAND CUSHION.
A.G. — ACCEPTABLE FOR USE AS GRANULAR BORROW.
N.A.G. — NOT ACCEPTABLE FOR USE AS GRANULAR BORROW.
M — SAMPLE IS ________ FOR USE AS G.B.C. CLASS "B"
K — SAMPLE IS ________ FOR USE AS G.B.C. CLASS "A"
X — ACCEPTABLE
N — NOT ACCEPTABLE
A — SAMPLE OF FINE AND COARSE AGGREGATE IS ________
B — COARSE AGGREGATE PORTION OF SAMPLE IS ________
C — FINE AGGREGATE PORTION OF SAMPLE IS ________
D — SAMPLE OF COARSE AGGREGATE IS ________
E — SAMPLE OF FINE AGGREGATE IS ________

W.P. №. ______

CONT. №. ______

LAB. №. 71-B-60067

COMPUTATION FORM GRANULAR MATERIALS

DEPARTMENT OF HIGHWAYS — ONTARIO

COARSE AGGREGATE

OVER

ABRASION TESTS

TYPE	EST. % CRUSH	GRADE	RET. Nº 12	PASS Nº 12	% LOSS
LOS ANGELES (500 REVS.)					

ABSORPTION %	a	b	BULK S.G. (DRY)	a	b	BULK S.G. (SAT. SUR. DRY)	a	b	APPARENT S.G.	a	b
B			B			B			A		
A			C			C			C		
B−A			B−C			B−C			A−C		
$\frac{B-A}{A}$ x100	14.890%	14.970%	$\frac{A}{B-C}$	2.036	2.033	$\frac{B}{B-C}$			$\frac{A}{A-C}$	2.922	2.923
AVG.	14.930 %		AVG.	2.034		AVG.			AVG.	2.922	

PERCENT COARSE AND FINE AGGREGATES	[redacted]	LAB. CRUSHED
LAB. CRUSHED		
WT. TOTAL SAMPLE (DRY) W_1	22.8	
WT. SAMPLE RET. #4 (DRY) W_2	15.1	
WT. SAMPLE PASS #4 (DRY) W_3	7.7	
% COARSE AGGREGATE	66.2	
% FINE AGGREGATE	33.8	

UNIT WT.	LOOSE	lb/cu. ft.
	COMPACTED	lb/cu. ft.
CLAY LUMPS		%
LOSS BY WASHING PASS # 200		%
FLAT & ELONGATED PARTICLES		%
CRUSHED PARTICLES		%
PASS # 200 OR # 270 (TOTAL SAMPLE)		%

W_2 [redacted] LAB. CRUSHED

DEPT. SIEVE DESIGNATION	INDIVIDUAL WEIGHT	CUMULATIVE WEIGHT	% RETAINED RET. # 4	% RETAINED TOTAL
4"				
3"				
2½"				
2"				
1½"				
1"				
⅞"				
¾"	0.1		0.7	0.4
⅝"	0.2		2.0	1.3
½"	1.9		14.6	9.6
⅜"	5.7		52.3	34.6
#3	3.9		78.1	51.8
#4	3.3		100.0	66.2
TOTAL				

W_2 LAB. CRUSHED

DEPT. SIEVE DESIGNATION	INDIVIDUAL WEIGHT	CUMULATIVE WEIGHT	% RETAINED RET. # 4	% RETAINED TOTAL
2½"				
2"				
1½"				
1¼"				
1"				
⅞"				
¾"				
⅝"				
½"				
⅜"				
#3				
#4				
TOTAL				

FINE AGGREGATE

DEPT. SIEVE DESIG.	W_3 AS RECEIVED CUMULATIVE WEIGHT GM.	% RETAINED PASS #4	% RETAINED TOTAL	W_3 LAB. CRUSHED CUMULATIVE WEIGHT GM.	% RETAINED PASS # 4	% RETAINED TOTAL
# 8						
# 16						
# 30						
# 50						
# 100						
# 200						
# 270						
PASS #200 #270 A						
PASS #200 #270 B						
DECANT						
TOTAL						

$MgSO_4$ SOUNDNESS TEST ______ CYCLES

WEIGHT BEFORE GM.	WEIGHT AFTER GM.	GRAMS LOSS	PERCENT LOSS	GRADING	WEIGHTED % LOSS
			TOTALS		

LOSS BY ABRASION		%
UNIT WEIGHT	LOOSE	lb./cu.ft.
	COMPACTED	lb./cu.ft.
FINENESS MODULUS		
ORGANIC IMPURITIES		
PASS #200 OR #270 (PASS #4 FRACTION)		%

ABSORPTION %	a	°F	b	°F	BULK S.G. (DRY)	a	b	BULK S.G. (SAT. SUR. DRY)	a	b	APPARENT S.G.	a	b
B					V			V			V−W		
A					W			W			B−A		
S−A					V−W			V−W			(V−W)−(B−A)		
$\frac{B-A}{A}$ x100		%		%	$\frac{A}{V-W}$			$\frac{B}{V-W}$			$\frac{A}{(V-W)-(B-A)}$		
AVG.				%	AVG.			AVG.			AVG.		

LABORATORY REMARKS ____________________

N.B. Results to Al Hawks.

W.P. Nº. ______________

CONT. Nº. ______________

LAB. Nº. 71-B-60067

DATE REQ'D. ______________

JUL 19 1971

(NO PN^s per J Casey)

COARSE AGGREGATE				FINE AGGREGATE & SOIL			
TEST				TEST			
GRADING	R			GRADING			
MgSO4	R			MgSO4			
P.N. (No)	R			P.N.			
L.A.	R	N.S.M.		ABRASION			
ABS. & S.G.	R			ABS. & S.G.			
% CRUSHED				ORGANIC			
% FLATS				STRUCT. STR.			
WASH PASS #200				SPEC. SIEVE			
PROCTOR							
				HYDROMETER			
				P.I.			
				PROCTOR			
				MOISTURE			
				SOIL S.G.			

FORM OGMT-144

ACRES

July 23, 1971
P2223.00

Department of Transport &
Communications
Highway 401 & Keele Street
Toronto, Ontario

Attention: Mr. A. G. Stermac, P.Eng.
Principal Foundation Engineer
Materials & Testing Division

Gentlemen:

Welland - Highway 406
Functional Planning Study

Further to our recent conversations and the delivery of samples of Union Carbide slag and canal water to your office on July 13, this letter confirms that your department will carry out tests to establish the physical and chemical properties of these materials with a view to assessing their suitability as a fill for use in the disused section of the canal.

The filling procedure presently envisaged is the dumping of the bottom 15 feet of material through the water before any de-watering takes place.

Should you require any further samples we will be pleased to obtain these for you.

Yours very truly,

S. Maitland for

A. L. McKechnie
Executive Engineer

STM:gs

c. c. - Mr. R. W. Oddson, P.Eng.
- M. M. Dillon Limited

H. G. ACRES LIMITED
1259 Dorchester Road
Niagara Falls, Canada

Telephone 416-354-3831

TELEPHONE 685-1571

THE REGIONAL MUNICIPALITY OF NIAGARA
PUBLIC WORKS DEPARTMENT
P.O. BOX 504
1 ;0 BERRYMAN AVENUE, ST. CATHARINES, ONTARIO

March 25, 1971

Department of Highways,
Highway 401 & Keele Street,
Downsview, Ontario.

Attention: Mr. A. G. Stermac,
Principal Foundation Engineer,
Materials & Testing Division

Dear Sir,

Highway 406 - Functional Planning Study
Welland Canal Alignment - City of Welland

I have been directed to advise you that the Technical Advisory Committee for Highway 406, Welland Section, has authorised the Consultants, M.M. Dillon Limited, to proceed with the detailed investigation of three alternative highway corridors. As you are aware, study of the central corridor entails additional soil investigation along the banks of the present route of the Welland Canal and the Committee has further approved the Consultants' proposal in this matter subject however, to your approval of the precise nature and extent of the work to be undertaken at this time.

This letter should have been sent to you some time ago and I wish to apologize for any inconvenience or delay which may have been occasioned by my oversight.

Yours very truly,

C.H.R. Foster

CHRF/ss

C.H.R. Foster,
Secretary,
Technical Advisory Committee

ACRES

April 5, 1971
P2223.00

Department of Highways, Ontario
Highway 401 and Keele Street
Downsview, Ontario

Attention: Mr. A. G. Stermac
Principal Foundation Engineer
Materials Testing Division

Gentlemen:

Highway 406 - Functional Planning
Report - Welland Canal Alignment

At meeting 71-3 of the Technical Advisory Committee, Highway 406 - Welland, on Tuesday, February 23, 1971, H. G. Acres Limited was authorized to proceed with a soils investigation, subject to approval by Mr. A. G. Stermac. This letter confirms Mr. Stermac's approval for the program outlined below in accordance with the telephone conversation between Mr. Stermac and Mr. Conlon, on Monday, April 5, 1971.

As a result of Mr. Stermac's letter of March 24, 1971, and his discussion with Mr. R. J. Conlon on April 1, 1971, we reviewed our proposal for nine boreholes along the bank of the existing Welland Canal between Stations 850 + 00 and 1100 + 00. We were aware of the report on the DHO test shaft at Station 995 + 00, as pointed out by Mr. Stermac. In addition, there is a shaft and 5-inch diameter holes in the vicinity of the Townline Road/ Rail Tunnel, a 5-inch diameter hole at the south end of the Welland Canal Relocation, as well as 5-inch diameter sample data at the location of each of two old landslides just north of the DHO test shaft. The shear strengths obtained at these locations show a definite tendency for a decrease in shear strength to the south of the DHO test shaft, particularly in the Lower Stratified clay layer. From these trends it is anticipated that an increase in shear strength may exist in the northern part of the study area. It was our intention to investigate fully the variability of shear strength along the existing Welland Canal with the proposed nine 5-inch diameter boreholes.

H. G. ACRES LIMITED
Consulting Engineers
5259 Dorchester Road
Niagara Falls, Canada

Telephone 416-354-3831

Mr. A. G. Stermac, (continued)
April 5, 1971

However, in accordance with our discussion we agree that the investigation work should be divided in two; the first stage providing sufficient information for preliminary analysis of stability problems and preparation of approximate cost estimates for the central alignment; the second stage providing more detailed information at specific locations, if indicated to be necessary, after completion of Stage 1.

The first stage places considerable emphasis on utilizing the available information. The investigation work which we consider essential for completion of Stage 1 includes a minimum of three 3-inch diameter boreholes. Two of these boreholes will be located on the dike between the Welland River and the Welland Canal where no information is known to exist (borehole locations near Stations 930 + 00 and 950 +00). The third borehole will be located near Station 890 + 00 on the west bank of the Welland Canal and will serve to complete the stratigraphic picture in this area.

The 5-inch diameter holes will be eliminated from the Stage 1 program. However, we consider that some 5-inch diameter holes may still be required for Stage 2; the extent of the Stage 2 program being entirely dependent on the Stage 1 program, including the results of analyses determining the significance of variation in shear strength parameters for the stability of canal banks during partial or total dewatering.

Yours very truly,

A. L. McKechnie
Executive Engineer

ALMcK:gs

c.c. - Mr. J. R. Crosby
Mr. C. Foster

H. G. ACRES LIMITED

A. L. McKechnie
F2223.

MAR 8 1971

TECHNICAL ADVISORY COMMITTEE

HIGHWAY 406 - WELLAND

MEETING 71-3

Minutes of meeting held on Tuesday, 23 February 1971, at the Regional Municipality of Niagara Offices, 150 Berryman Avenue, St. Catharines, Ontario.

COMMITTEE MEMBERS IN ATTENDANCE

Representing Department of Highways, Ontario

Mr. C. R. Robertson	- District Engineer, District 4 Hamilton
Mr. J. J. Regan	- Construction Supervisor, District 4 Hamilton
Mr. G. K. Hunter	- Regional Road Design Engineer, Toronto
Mr. M. W. Robinson	- Regional Services Manager, Central Region
Mr. R. G. Burnfield (alternate for Mr. R. W. Oddson)	- Regional Functional Planning Engineer

Representing Regional Municipality of Niagara

Mr. M. Holenski (CHAIRMAN)	- Project Engineer
Mr. N. Dodd (alternate for Mr. A. Greaves)	
Mr. C. H. R. Foster (SECRETARY)	- Traffic Coordinator

Representing City of Welland

Mr. D. H. Landells	- City Engineer

Others in Attendance

Mrs. R. Doctorow	- Socialogist, Planning Department, D.H.O.

Representing Consultant

Mr. J. R. Crosby	- Project Director, M. M. Dillon
Mr. F. Z. Sobolak	- Project Manager, "
Mr. W. Gleis	- Recording Secretary, "
Mr. S. T. Maitland	- Project Engineer, H. G. Acres Ltd.

The Chairman called the meeting to order at 10.07 a.m.

1. Minutes of Meeting 71-2 were reviewed and the following corrections were suggested by the Consultant regarding Item 3. The second paragraph should be replaced by:

   "Mr. Stermac, Principal Foundation Engineer, D.H.O., confirmed the need for soil investigations as requested by Acres, in order to compile data required for establishing the economic viability of the route. He considered however that these investigations should be deferred pending a decision establishing the centre route as a valid alignment. The Committee agreed and decided not to proceed further with the soil investigations at this time.

   Mr. McKechnie expressed the view that even though the centre alignment was ruled out from a planning or political reason, it may be necessary to prepare capital cost estimates of the centre route in order to properly evaluate the economic viability of alternative routes."

   The above was suggested in a letter from Mr. McKechnie to Mr. Crosby and confirmed in a letter from Mr. Stermac to Mr. Crosby. Copies of the letters are attached hereto.

2. The Secretary reported no outstanding Committee correspondence.

3. Items held over from Meetings 70-1 and 71-2 and information arising from the N.R.H.C.C. meeting.

   (a) Mr. Foster distributed a new list of Committee members. Distribution of agendas and minutes of meetings will be made to Committee members only. Mr. Robertson suggested that telephone numbers of each individual should also be added to the list.

(b) Because of postponement of the last N.R.H.C.C. meeting, no decision has been reached relating to the procedures for submission, review, approval and payment of the Consultant's Monthly Progress Claims.

(c) No decision has been reached on the Technical Advisory Committee's Terms of Reference for the same reason as in (b) above.

4. The Consultant's Monthly Progress Report was reviewed and the following comments made:

(a) The Consultant noted that very little work has been done on item 4 because of delays in Committee decisions.

(b) Mr. Robinson enquired about item 8 and how soon will property evaluation be required. The Consultant advised that item 8 as well as 9 and 10 could be considered to be at least two months behind schedule.

(c) Mr. Burnfield expressed concern that the Consultant's monthly fees have fallen behind the estimated fees. The Consultant stated that the estimated total fee is still quite valid and as soon as decisions are made by the Committee regarding certain work items, the schedule and the resulting work and expenditures would be brought up to date.

5. The Consultant briefly reviewed the Committee report entitled: "Method for Evaluation of Freeway Location Alternatives". This report was compared with "Proposals for Evaluation of Alternative Transportation Schemes" which was developed by the inter-disciplinary committee of the Department of Highways. The two methods of evaluation are very similar in scope and both methods will be considered in establishing the evaluation procedure and the measures to be used in the evaluation.

(a) The report to the Committee entitled "Engagement of Proctor, Redfern, Bousfield & Bacon for the Planning and Environmental Aspects of this Project" was reviewed.

Considerable discussion took place regarding the acceptability of Proctor, Redfern, Bousfield & Bacon to participate in this study and the method of engaging them for this purpose. The Committee accepted the estimated cost of $21,700.00 furnished by Proctor, Redfern, Bousfield & Bacon as being within the limits estimated in the overall project cost.

It was moved by Mr. Foster, seconded by Mr. Robertson, that the Committee agree to M. M. Dillon Limited engaging Proctor, Redfern, Bousfield & Bacon for the purpose of evaluating the community effect items in the evaluation of freeway location alternatives.

CARRIED.

Mr. Burnfield requested that M. M. Dillon write to Mr. W. Bidell, Director of Planning at D.H.O. and request authorization to engage Proctor, Redfern, Bousfield & Bacon as associate-consultants for this study.

(b) Committee report entitled "Assignment of Responsibilities for the Evaluation of Alternative Freeway Locations. A table attached to the report listed various factors as proposed in the H.R.B. publication. On this table an attempt was made to identify the firm(s) or other agencies who will be responsible for the data and analyses. Certain factors require multiple input. In these cases, all those responsible have been identified.

The contents of the table were reviewed by the Committee and the following modifications suggested:

(i) Item 1(a) - should read "Functional Planning" instead of "Planning".

(ii) Item 1(b) - Right-of-way evaluation would be handled through the Property Sub-committee.

(iii) Item 7(a) - Property Values: Change in resale value - this will be handled through the Property Sub-committee.

(iv) Item 8(a) - Effect of tax base - this work to be handled through the Property Sub-committee.

(c) The Committee extensively discussed whether any of the corridors should be dropped at this time. The Chairman asked the Committee members and advisors as to their opinions on which of the 3 corridors could be eliminated. The results were that four were of the opinion that all 3 corridors should be evaluated further, three felt that corridors "B" or "C" could be dropped, and the remaining three made a choice of one alignment, either "A" or "C".

Mr. Landells informed the Committee that new sub-division plans have been registered in the area north and south of Fitch Street in the path of corridor "B" alignment. This development will have to be seriously considered in the evaluation of corridor "B".

As a result of the above discussion, the Committee agreed that it would be premature at this time to drop any of the 3 alignment corridors.

As a result of the above a decision was required regarding authorization for the consultant to proceed with additional soil investigations which are required for the evaluation of corridor "A" alignment.

It was moved by Mr. Robertson, seconded by Mr. Burnfield that the Consultant be authorized to proceed with soil investigations proposed by him. This authorization is subject to an approval by Mr. A. Stermac, Principal Foundation Engineer, D.H.O. The Secretary was directed to communicate with Mr. Stermac in this regards.

CARRIED.

6. The Committee report entitled "Correspondence and the Resolutions of City of Welland Council" was reviewed and considerable discussion took place regarding the opposition of City Council to locating Highway 406 within the canal bed. It appears that Council is very much concerned with the method of disposing of the old canal and the assurance that the present supply of canal water to industries and the City water treatment plant is maintained. If the supply of canal water is interfered with in any way, this interference should be rectified at no cost to the City of Welland. The City Council is also opposed to the St. Lawrence Seaway Authority's proposal to leaving the water in the old canal and using mechanical and chemical means to treat the water to prevent it from stagnating.

Mr. Burnfield suggested that the Committee should meet with representatives of the St. Lawrence Seaway Authority in order to determine the most up-to-date status of the disposition of the old canal. It would be advisable to hold this meeting as a joint meeting of the Technical Advisory Committee for the Highway 406-Welland project ar the Highway 3 & 406-Port Colborne project. Mr. Burnfield will request Mr. Oddson to arrange for such a meeting.

Mr. Burnfield also informed the Committee that the Province was approached by the St. Lawrence Seaway Authority about disposing of some of the lands on either side of both the old and new canal and the use of this land as parks. This matter is now being considered by the Ontario Parks Integration Board. Mr. Burnfield will arrange a meeting with Mr. C. R. Tilt, the Secretary of the Board. The meeting will be attended by Mr. Oddson, Mr. Holenski, Mr. Landells and Mr. Crosby.

7. The Consultant reported that Mr. Oddson requested from the D.H.O. Photogrammetry Division an updated photomosaic and 200 scale contoured plans sufficient to cover the area which would be affected by either of the three corridor alignments. A photomosaic based on 1970 flying is already available. This photomosaic will be extended and updated this Spring when new photography becomes available.

8. No other business.

9. The next meeting is scheduled for Tuesday, April 6, 1971, at 10:00 a.m. and will be held at 150 Berryman Ave., St. Catharines.

10. Meeting adjourned at 4:00 p.m.

Enclosures:

Letter from A. L. McKechnie 5/2/71
Letter from A. G. Stermac 9/2/71

Distribution:

All Committee members
H. G. Acres Ltd. (2)
M. M. Dillon Limited (3)
Committee Secretary - 2 extra copies

Project No. 6513-01
W.P. 100-68

M. M. DILLON LIMITED
4 March 1971

RECEIVED
FEB 3 - 71
M. M. DILLON LTD.
TORONTO OFFICE

FILE	6513-01	
JRC	C	
FZS	FS	

February ~~5, 1971~~
P2223.00

M. M. Dillon, Limited
P.O. Box 219
Station K
Toronto 315, Ontario

Attention: Mr. John R. Crosby, P. Eng.
Project Director

Dear John:

Hwy. 406 - Functional Report
Minutes of Meeting - January 19,1971

This will confirm my telephone call of this morning expressing my disagreement with the wording of Item No. 3, Soils Investigation - Welland Canal Route - 2nd paragraph, of the above minutes. Perhaps the following alternative wording could be suggested to the T.A.C. No. 2 Welland, for inclusion in a revision to be sent to all previous recipients of these minutes:

"Mr. Stermac, Principal Foundation Engineer, D.H.O., confirmed the need for soil investigations as requested by Acres, in order to compile data required for establishing the economic viability of the route. He considered however that these investigations should be deferred pending a decision establishing the centre route as a valid alignment. The committee agreed and decided not to proceed further with the soil investigations at this time.

Mr. McKechnie expressed the view that even though the centre alignment was ruled out from a planning or political reason, it may be necessary to prepare capital cost estimates of the centre route in order to properly evaluate the economic viability of alternative routes."

Please feel free to modify the above as you see fit, as your notes of the meeting may be fuller than mine on this point.

H. G. ACRES LIMITED
Consulting Engineers
1259 Dorchester Road
Niagara Falls, Canada

Telephone 416-354-3831

M.M. Dillon, Limited (cont'd)
February 5, 1971

I also talked with Tony Stermac this morning and he confirmed that the minutes misquoted him. He too will be writing to the Committee on this point and I am sending Tony a copy of this letter and the minutes in question.

Yours very truly,

A. L. McKechnie
Executive Engineer

ALMcK:gs

c.c. - Mr. A. G. Stermac

H. G. ACRES LIMITED

HON. GEORGE E. GOMME
MINISTER

A. T. C. McNAB
DEPUTY MINISTER

Hwy. 401 & Keele St.,
Downsview 464, Ontario.
Tel. 248-3282
(Area Code 416)

DEPARTMENT OF HIGHWAYS
Materials and Testing Office

February 9, 1971

FILE	6513-01	
JRC	C	

M. M. Dillon, Limited,
Consulting Engineers,
P.O. Box 219,
Station 'K',
Toronto 315, Ontario.

Attention: Mr. John R. Crosby, P. Eng.,
Project Director

Re: Hwy. 406 - Functional Report
Minutes of Meeting - January 19, 1971

Dear John:

I am in receipt of a copy of the letter Archie McKechnie wrote to you on February 5, 1971, regarding the 2nd paragraph of Item No. 3, Soils Investigation - Welland Canal Route. Since I did not receive the minutes of the meeting of January 19, 1971, I don't think I can write to the Committee, but would like to take this opportunity to advise you that I concur with Archie's statement. I must have been misunderstood, and therefore, misquoted. Archie's proposed revision records, in essence, what I have said and I would certainly be in agreement that the wording of the minutes be changed as proposed.

Sincerely yours,

Tony Stermac

A. G. Stermac
Principal Foundation Engineer

AGS/MdeF

cc: Mr. A. L. McKechnie,
Executive Engineer,
H. G. Acres & Co. Ltd.

RECEIVED
FEB 10 71
M. M. DILLON LTD.
TORONTO OFFICE

HON. GEORGE E. GOMME
MINISTER
A. T. C. McNAB
DEPUTY MINISTER

Foundations Files (Rm.110)

Your File Ref. P2223.00

Hwy. 401 & Keele St.,
Downsview 464, Ontario.

Tel. 248-3282
(Area Code 416)

DEPARTMENT OF HIGHWAYS
Materials and Testing Office

March 24, 1971

Mr. A. L. McKechnie,
Executive Engineer,
H. G. Acres Limited,
Consulting Engineers,
1259 Dorchester Road,
Niagara Falls, Canada.

Re: Highway 406 - Functional Planning
Report - Welland Canal Alignment

Dear Mr. McKechnie:

This is to confirm receipt of your letter of March 18, 1971 regarding the above mentioned subject.

I would like to draw your attention to the following report prepared for the Department of Highways by the Consultant, H. Q. Golder and Associates Ltd. in July, 1964.

Trial Shaft at Proposed Tunnel Site
Execution, Sampling and Test Results
Welland Ontario

The shaft was put down at the approximate Station 995+00. The report contains a great deal of information that is, I believe, directly applicable to your work. If my memory serves me right, one copy of this report should be in the possession of your Mr. R. Conlon.

It seems to me that the testing of the shaft samples has adequately defined the properties of all layers that were encountered (the upper and the lower stratified zone). These properties, I believe, could be assigned to all locations where such two zones are either known to exist, or would be encountered through the additional work to be carried out.

..... 2

Re: Highway 406 - Functional Planning
Report - Welland Canal Alignment ...

In the light of the foregoing, I would suggest that you reconsider

(a) the necessity of having to put down nine additional holes, and

(b) the necessity to recover 5-inch diameter samples.

Should you wish to discuss this matter further, please feel free to contact me.

Yours very truly

A. G. Stermac
Principal Foundation Engineer

AGS/MdeF

cc: Mr. J. R. Crosby, P. Eng.

Foundations Files

Gen. Files

ACRES

March 18, 1971
P2223.00

Department of Highways, Ontario
Highway 401 and Keele Street
Downsview, Ontario

Attention: Mr. A. G. Stermac
Principal Foundation Engineer
Materials & Testing Division

Gentlemen:

Highway 406 - Functional Planning Report - Welland Canal Alignment

Further to our recent meeting in St. Catharines, the technical advisory committee has agreed to proceed with the detailed investigation of three corridors for the new highway, which includes the central alignment.

The scope of the study necessitates that the stability of the existing canal banks during either partial or complete unwatering be analyzed.

In order to carry out this work, we must have a more complete picture of the regional geology and, in particular, the existence and extent of stratified clays. Our previous experience at the Townline Road/Rail Tunnel has shown the importance of these upper and lower stratified zones on stability analyses.

As a result, we have reviewed the available information related to stratigraphy and shear strengths in the area to determine whether additional drilling is required. Much of the available information on overall stratigraphy is contained in the Golder and Associates reports.

1 - Feasibility Study of Portion of Proposed Highway 406 with Welland Canal, January 1967.

2 - Soil Conditions within Welland Canal, June 1967.

Based upon the proposed central alignment for Highway 406, the canal bank stability must be reviewed between Stations 850+00 and 1100+00 (Welland Canal Chainage). Within this area, additional information is considered necessary in the following zones and for the following reasons:

H. G. ACRES LIMITED
Consulting Engineers
1259 Dorchester Road
Niagara Falls, Canada

Telephone 416-354-3831

Mr. A. G. Stermac, (cont'd)
March 18, 1971

1 - East bank of Welland Canal

Stability of the dike between the Welland Canal and the Welland River should be investigated, and, at present, there is no information within this dike. Three boreholes are proposed and would be located near the following canal chainages:

910 + 00
935 + 00
950 + 00

2 - West bank of Welland Canal North of the Welland River Siphon

Three additional boreholes are required in this area to check for the existence of stratified clays, both in the canal bank and at the depth. These boreholes would be located near the following canal chainages:

890 + 00
935 + 00
950 + 00

3 - West bank of Welland Canal South of the Welland River Siphon

There are several unstable sections of bank to the south of the Welland River Siphon and it is necessary that the three additional boreholes be drilled in this area to obtain good undisturbed samples for testing purposes. These boreholes would be located near the following canal chainages:

970 + 00
1010 + 00
1070 + 00

As shown on the attached sketch, we are recommending that a total of 9 additional boreholes be drilled. These holes should be large enough to allow the recovery of 5-inch diameter Shelby tube samples. The reason for requesting the large samples is to enable trimming of triaxial samples at 45 degrees to the bedding plane in stratified clays, and allow determination of the shear strengths parallel to the bedding plane.

We enclose two copies of sketch showing the approximate location of the boreholes.

If you wish to discuss further, any of the foregoing, please give us a call.

Yours very truly,

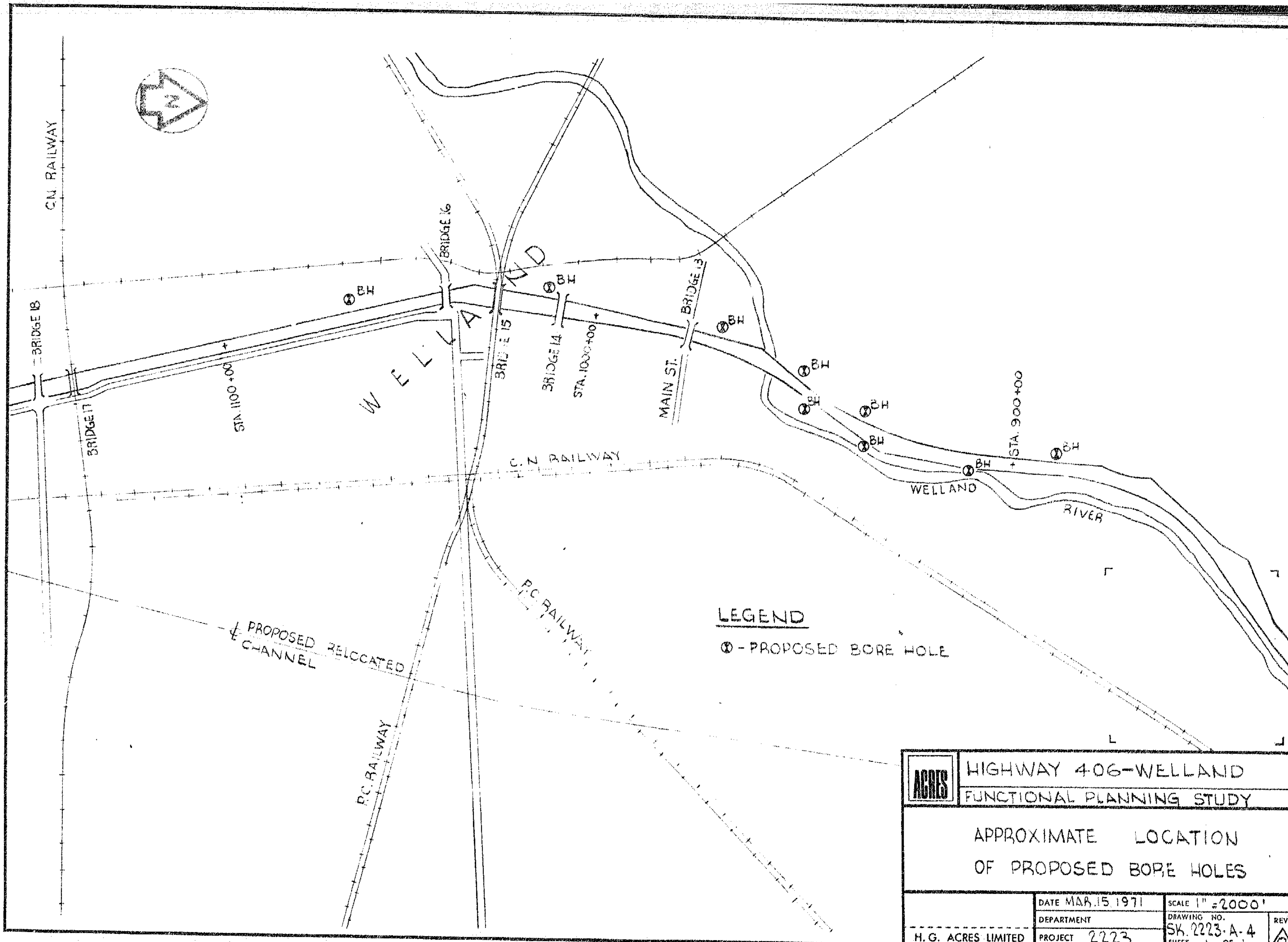
Maitland for

A. L. McKechnie
Executive Engineer

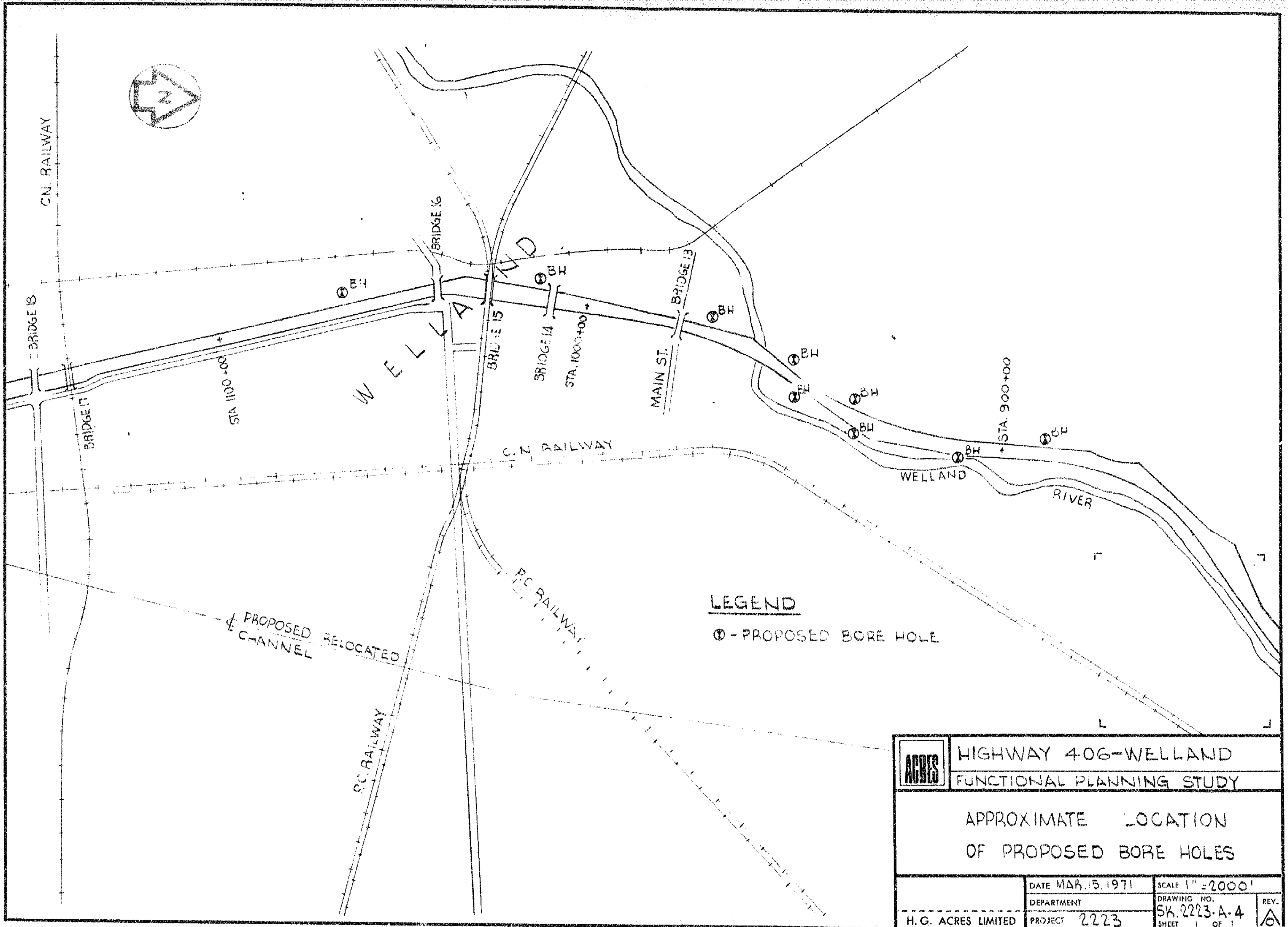
STM:gs
encls.

c.c. - Mr. J. R. Crosby, P.Eng.
- Mr. C. Foster

H. G. ACRES LIMITED



DRAWING No. SK-2223-A-4 SHEET OF REV. 0



ACRES

HIGHWAY 406–WELLAND

FUNCTIONAL PLANNING STUDY

APPROXIMATE LOCATION OF PROPOSED BORE HOLES

H. G. ACRES LIMITED	DATE MAR. 15. 1971	SCALE 1" = 2000'
	DEPARTMENT	DRAWING NO. SK. 2223-A-4 REV. 0
	PROJECT 2223	SHEET 1 OF 1

DRAWING No. SK-2223-A-4 SHEET OF REV. 0

TO: HWY. 406 - WELLAND, SUBCOMMITTEE NO. 2

RE: METHOD FOR EVALUATION OF FREEWAY LOCATION ALTERNATIVES

We were requested by the Chairman to prepare an evaluation form for alternative alignment proposals within the City of Welland.

Numerous methods and evaluation procedures are available. We have selected the method proposed by Oglesby, Bishop and Willeke in their paper, "A Method for Decision Among Freeway Location Alternatives Based on User and Community Consequences" (Highway Research Record Number 305, Washington, D.C., 1970).

To familiarize the Committee with the approach proposed by authors, we attach hereto Tables 1, 2 and 3 which were excerpted from the paper. Tables 1 and 2 tabulate the factors to be evaluated and the suggested units for measurment. Table 3 presents the results of a survey undertaken by the authors in an attempt to determine the importance of each factor in route location, in the opinion of sample groups of highway engineers and planners, community officials and citizens.

We also attach Figure 1 taken from the article, which identifies the "community factor profile" which may be used to rate the alternative highway proposals. It is to be noted that in order to reduce the complexity of the diagram and, in turn, of the decision-making process, the full set of community factors should be reduced whenever it is possible to do so. Two guidelines are suggested for accomplishing this: (a) eliminate all factors that are not relevant or important to the particular decision; and (b) eliminate all factors whose values are substantially the same for all alternatives. These tests must be acceptable to all parties involved in the study.

A table listing all evaluation factors is also attached. We have made a preliminary assessment as to the relative merits or demerits attributable to each of the three considered proposals. It should also be noted that a fourth alternative is considered -- assuming that no freeway is built. Due to the time limitation and the current stage of the project, only certain factors could be assessed and the evaluation can only be considered a "guesstimate".

As the study proceeds a more meaningful evaluation may be made of the factors which were indicated as unknown. We hope it will be possible to reduce the number of factors by eliminating those that are not relevant or important to the particular decisions, and eliminate those whose values are substantially the same for all alternatives.

Action Items

1. Review and approve the proposed evaluation methodology for use on this project.

2. Eliminate, if possible, all factors that are not relevant or important.

3. Eliminate all factors whose values are substantially the same for all alternatives.

4. Allocate the work required to quantify the remaining factors to:

- the Consultant
- the Sub-Committee
- others.

File 6513-01 M.M. DILLON LIMITED
15 January 1971

EVALUATION OF DIRECT AND INDIRECT EFFECTS ON THE LOCATION OF HIGHWAY 406 ALTERNATIVES IN THE CITY OF WELLAND

FACTORS	CORRIDOR A	B	C	NO FREEWAY	REMARKS
<u>Quantifiable Market Values</u>					
1. Cost of Highway					
(a) Planning	High	Moderate	Lowest	High	Guesstimate only - Need $ Cost
(b) Right-of-way	Moderate	High	Moderate	Unknown	" " " "
(c) Construction	High	Moderate	High	Probably High	" " " "
(d) Maintenance	High	Moderate	High	Moderate	" " " "
(e) Operation	Moderate	Moderate	Moderate	Lowest	" "
2. Costs (Benefits) to Highway User					
(a) Vehicle Oper. Cost, inc. Congestion Cost	Lowest	Moderate	Moderate	High	" " " "
(b) Travel time saving Commercial	Highest	Moderate	Moderate	None	" " " "
(c) Economic cost of Accidents	Lowest	Moderate	Moderate	Highest	" " " "
<u>Quantifiable Non-Market Values</u>					
3. Costs (Benefits) to Highway User - Travel Time Savings, Non-Commercial	Maximum	Moderate	Moderate	Minimum	" " Need Time Saving
<u>Non-Quantifiable Non-Market Values</u>					
4. Costs (Benefits) to Highway User					
(a) Motorist Safety	Highest	High	High	Lowest	Guesstimate only
(b) Motorist Comfort and Convenience	Good	Good	Good	Poor	-
(c) Aesthetics from Driver Viewpoint	Good	Good	Good	Poor	-

FACTOR	CORRIDOR A	CORRIDOR B	CORRIDOR C	NO FREEWAY	REMARKS
Community Effects					
5. Local Transportation Effects					
(a) Traffic service to community by freeway-highway capacity, O-D of trips, major traffic generators	Highest	Moderate	Lowest	N/A	To be quantified
(b) Effect on local transportation: City Street circulation and public transit	Unknown	Unknown	Unknown	Unknown	" "
(c) Access to Regional facilities: Recreation, education, culture, business and employment	Unknown	Unknown	Unknown	Unknown	" "
(d) Highway Design Standards: grades, alignment and interchange location	Good	Good	Good	N/A	" "
6. Community Planning and Environment					
(a) Land Use: development, changes in use, multiple use separation of uses	Unknown	Unknown	Unknown	Unknown	" "
(b) Aesthetic impact of freeway on community: depressed or elevated, landscaping, structures	"	"	"	"	" "
(c) Noise	"	"	"	"	" "
(d) Air pollution	Less	Less	Less	Most	

FACTOR	CORRIDOR A	CORRIDOR B	CORRIDOR C	NO FREEWAY	REMARKS
7. <u>Neighbourhood and Social Structure</u>					
(a) Property Values: Change in resale value	Unknown	Unknown	Unknown	Unknown	To be Quantified
(b) Neighbourhood impacts: displacement and relocation of people, environmental qualities, neighbourhood cohesiveness and stability	Unknown	Unknown	Unknown	Unknown	" "
(c) Parks and recreational facilities	"	"	"	"	" "
(d) Cultural and religious institutions	"	"	"	"	" "
(e) Historical sites and unique areas	"	"	"	"	" "
(f) School system: attendance boundaries, school environment	"	"	"	"	" "
8. <u>Community Economic and Fiscal Structure</u>					
(a) Effect on tax base: net change in assessed value of property on tax rolls	"	"	"	"	" "
(b) Community services: police and fire protection, utility services, water and garbage services	"	"	"	"	" "
(c) Commercial activity: wholesale, retail	"	"	"	"	" "
(d) Employment: creation of jobs, displacement of jobs	"	"	"	"	" "

TABLE 1

DIRECT EFFECTS OF FREEWAY CONSTRUCTION AND USE

Factor	Description	Units	Time Period, Years
Quantifiable market values			
1. Cost of highway			
a. Planning		Dollars	N. A.
b. Right-of-way	Capital cost and annual cost of planning, constructing, maintaining, and operating the freeway	Dollars	20 to 40
c. Construction		Dollars	20
d. Maintenance		Dollars	Annual
e. Operation		Dollars	Annual
2. Costs (benefits) to highway user			
a. Vehicle operating cost, including congestion costs	Net increase (decrease) in costs of vehicle operation per year	Dollars	Annual
b. Travel time savings, commercial	Net increase (decrease) in travel time multiplied by dollar value of commercial travel time	Dollars	Annual
c. Motorist safety, economic cost of accidents	Net change in expected number of accidents multiplied by average cost per accident	Dollars	Annual
Quantifiable nonmarket values			
3. Costs (benefits) to highway user			
Travel time savings, noncommercial	Minutes saved per vehicle trip	Minutes or hours	Annual
Nonquantifiable nonmarket values			
4. Costs (benefits) to highway user			
a. Motorist safety	Accident costs of pain, suffering, and deprivation	?	Annual
b. Motorist comfort and convenience	Discomfort, inconvenience, and strain of driving	?	Annual
c. Aesthetics from driver viewpoint	Benefit of pleasing views and scenery from the road	?	Annual

TABLE 2

COMMUNITY EFFECTS OF FREEWAY LOCATION AND USE

Factor	Measures and Suggested Measures		Time Period	
	Description	Units	Long Run	Short Run
Local Transportation Effects				
Traffic service to community by freeway-highway capacity, O-D of trips, major traffic generators	1. Percent reduction of through traffic on city streets [(vehicles before - vehicles after)/vehicles before]	Percent	x	
	2. Distance of freeway access from major traffic generators (e. g., academic, business, cultural, administrative centers) or as measured by road user or transportation costs	Miles	x	
	3. Corridor miles compatible with present or future public transportation development	Miles	x	
Effect on local transportation: city street circulation and public transit	1. Costs (savings) for improvement to city streets to provide for projected traffic volumes if freeway is not built	Dollars	x	
	2. Net change in parking space available as result of freeway	No. spaces	x	
	3. Number of interchanges with the community less streets closed	Number	x	
Access to regional facilities: recreation, education, culture, business, and employment	1. Travel time savings to regional activity centers [(minutes per vehicle) x (vehicles per day)] for each facility	Minutes per day	x	
	2. Number of trips to community generated from outside	Vehicles per day	x	x
Highway design standards: grades, alignment, and interchange location	1. Miles less than x percent grade	Miles	x	
	2. Miles of curvature less than y radius	Miles	x	
	3. Average distance between interchanges	Miles	x	
Community Planning and Environment				
Land use: land development, changes in use, multiple use, separation of uses	1. Land for potential development to which access is created	Acres	x	
	2. Miles of freeway separating incompatible land use minus miles dividing compatible uses	Miles	x	
	3. Miles adjacent to or through land undergoing change in use	Miles per acre	x	
Aesthetic impact of freeway on community: depressed or elevated, landscaping, structures	1. Miles depressed in residential areas plus miles elevated in commercial areas less miles at grade	Miles	x	
	2. Additional costs of aesthetic improvement in structures and landscaping	Dollars	x	
Noise	1. Increase in dB level weighted by miles residential, and numbers of schools, churches, and similar buildings adjacent to freeway	dB (weighted)	x	
	2. Additional cost of noise barriers in noise problem areas	Dollars	x	
Air pollution	1. Net change in noxious exhaust emissions for projected traffic with and without the freeway	Percent	x	
Neighborhood and Social Structure				
Property values: changes in resale values	1. Increase or decrease (net) over normal trend in property value classified by type of use and distance from freeway	Dollars	x	
Neighborhood impacts: displacement and relocation of people, environmental qualities, neighborhood cohesiveness and stability	1. Number of housing units displaced (or) number displaced as percent of community's total stock	Number Percent	x x	x x
	2. Number of people displaced (or) number displaced as percent of community's population	Number Percent	x x	x x
	3. Net loss of housing—units taken less vacant replacement housing in same price range with comparable financing less new construction planned on vacant land with financing	No. units	x	
	4. Cohesive neighborhoods severed by freeway (as determined by mapping neighborhood boundaries and social characteristics)	No. people	x	x
	5. Neighborhood stability (14, pp. 33-42)	Index No.	x	x
Parks and recreational facilities	1. Acres of parks lost (gained) as percent of total available acres	Percent	x	
	2. Cost of park replacement less compensation	Dollars	x	
	3. Number of parks affected	Number	x	x
Cultural and religious institutions	1. Number of churches taken (or) total attendance affected	No. churches No. people		x x
	2. Additional cost of relocation, excess over taking price	Dollars	x	
	3. Improved access or location for new church facilities	Minutes	x	
Historical sites and unique areas	1. Number of historical areas lost (total affected less those relocated)	Number	x	
	2. Value of monument measured by annual visits per year	Visits per year	x	
School system: attendance boundaries, school environment	1. Net loss (gain) in tax base for school system	Dollars	x	x
	2. Number of schools totally or partially taken (or affected)	Number		x
	3. Number of school attendance areas with access to school seriously impaired where boundaries cannot be adjusted	No. pupils	x	x
	4. Increase (decrease) in cost of providing school services because of changes in busing	Dollars	x	x
	5. Net additional cost to the community of relocating schools affected by freeway (plus) cost of noise reduction in schools adjacent to freeway	Dollars	x	
Community Economic and Fiscal Structure				
Effect on tax base: Net change in assessed value of property on tax rolls	1. Loss of assessed valuation in right-of-way as percent of community total	Percent		x
	2. Loss of assessed valuation in right-of-way less increase of land values (assessed) caused by freeway impact	Dollars	x	x
	3. Net loss (gain) in tax revenue caused by freeway impact	Dollars	x	x
Community services: police and fire protection, utility services, water and garbage services	1. Net increase (decrease) in costs of providing fire and police protection and water, sewerage, and garbage service	Dollars	x	
Commercial activity: wholesale, retail	1. Net increase (decrease) over normal trend in gross wholesale and retail sales	Dollars	x	
	2. Net number of businesses located (displaced) by freeway	Number		x
Employment: creation of jobs, displacement of jobs	1. Net number of jobs located (displaced) as a result of freeway	Number		x
	2. Net gain (loss) in gross earnings from jobs located or displaced by the freeway	Dollars	x	x
	3. Net increase (decrease) in job opportunities caused by expanded commuting area less jobs available to outside commuting	Number	x	x

TABLE 3

IMPORTANCE OF FACTORS IN ROUTE LOCATION TO 54 HIGHWAY ENGINEERS AND PLANNERS, 160 COMMUNITY OFFICIALS, AND 123 CITIZENS

Factor	Percent of Highway Engineers and Planners			Percent of Community Officials			Percent of Citizens		
	Major	Minor	No	Major	Minor	No	Major	Minor	No
Direct costs and benefits of freeway									
Cost of highway	95	4	1	86	12	1	81	16	1
Motorist safety and comfort	85	13	2	84	12	2	87	8	1
Travel time savings	52	43	5	55	40	2	61	31	7
Vehicle operating cost	41	54	5	29	48	21	19	50	29
Local transportation effects									
Traffic service to city	96	4	0	89	8	1	77	20	1
Local transportation	74	20	6	91	7	2	63	31	4
Regional access	50	45	5	65	31	2	55	37	7
Highway design standards	93	5	2	87	11	1	81	16	1
Community planning and environment									
Land use plans	65	32	3	79	17	2	52	37	8
Aesthetics of freeway	69	26	5	76	21	1	42	51	2
Noise	24	67	9	67	28	3	51	42	4
Air pollution	13	52	35	58	33	6	72	22	2
Neighborhood and social structure									
Property values	65	28	7	72	22	2	59	33	6
Neighborhood impact	54	41	5	59	33	5	40	41	17
Parks and recreation	82	18	0	58	34	6	37	51	8
Cultural and religious centers	54	43	3	36	57	4	17	61	18
Historical and unique areas	69	30	1	64	32	2	37	48	11
School system	56	37	7	51	43	5	43	42	13
Community economic and fiscal structure									
Effect on tax base	30	61	9	49	42	7	48	41	7
Community services	32	65	3	71	25	2	61	34	3
Commercial activity	37	50	13	56	39	3	47	43	8
Employment	41	45	14	58	32	8	60	35	2

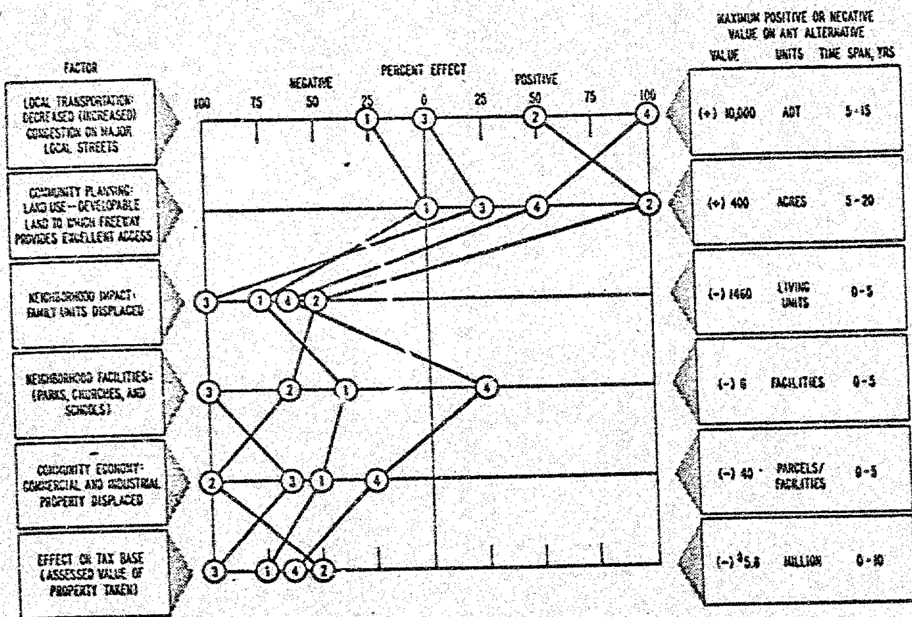


Figure 1. Community factor profile: Numbers in circles indicate the 4 alternatives.

cc: Mr. G. Stermac

Hwy. 401 & Keele St.,
Downsview 464, Ontario.

Tel. 248-3282
(Area Code 416)

February 9, 1971

Materials and Testing Office

M. M. Dillon, Limited,
Consulting Engineers,
P.O. Box 219,
Station 'K',
Toronto 315, Ontario.

Attention: Mr. John R. Crosby, P. Eng.,
Project Director

Re: Hwy. 406 - Functional Report
Minutes of Meeting - January 19, 1971

Dear John:

I am in receipt of a copy of the letter Archie McKechnie wrote to you on February 5, 1971, regarding the 2nd paragraph of Item No. 3, Soils Investigation - Welland Canal Route. Since I did not receive the minutes of the meeting of January 19, 1971, I don't think I can write to the Committee, but would like to take this opportunity to advise you that I concur with Archie's statement. I must have been misunderstood, and therefore, misquoted. Archie's proposed revision records, in essence, what I have said and I would certainly be in agreement that the wording of the minutes be changed as proposed.

Sincerely yours,

AGS

A. G. Stermac
Principal Foundation Engineer

AGS/MdeF

cc: Mr. A. L. McKechnie,
Executive Engineer,
H. G. Acres & Co. Ltd.

February 5, 1971
P2223.00

M. M. Dillon, Limited
P.O. Box 219
Station K
Toronto 315, Ontario

Attention: Mr. John R. Crosby, P. Eng.
Project Director

Dear John:

Hwy. 406 - Functional Report
Minutes of Meeting - January 19, 1971

This will confirm my telephone call of this morning expressing my disagreement with the wording of Item No. 3, Soils Investigation - Welland Canal Route - 2nd paragraph, of the above minutes. Perhaps the following alternative wording could be suggested to the T.A.C. No. 2 Welland, for inclusion in a revision to be sent to all previous recipients of these minutes:

"Mr. Stermac, Principal Foundation Engineer, D.H.O., confirmed the need for soil investigations as requested by Acres, in order to compile data required for establishing the economic viability of the route. He considered however that these investigations should be deferred pending a decision establishing the centre route as a valid alignment. The committee agreed and decided not to proceed further with the soil investigations at this time.

Mr. McKechnie expressed the view that even though the centre alignment was ruled out from a planning or political reason, it may be necessary to prepare capital cost estimates of the centre route in order to properly evaluate the economic viability of alternative routes."

Please feel free to modify the above as you see fit, as your notes of the meeting may be fuller than mine on this point.

H. G. ACRES LIMITED

M.M. Dillon, Limited (cont'd)
February 5, 1971

I also talked with Tony Stermac this morning and he confirmed that the minutes misquoted him. He too will be writing to the Committee on this point and I am sending Tony a copy of this letter and the minutes in question.

Yours very truly,

A. L. McKechnie
Executive Engineer

ALMcK:gs

c.c. - Mr. A. G. Stermac

NIAGARA REGIONAL HIGHWAYS CO-ORDINATING COMMITTEE

TECHNICAL ADVISORY COMMITTEE NO. 2 WELLAND

MINUTES OF SECOND MEETING

DATE: January, 19, 1971 PLACE: 150 Berryman Avenue,

PRESENT: Members: Mrs. Doctorow, Messrs. Holenski (Chairman) Robertson, Landells, Saltarelli, Oddson, Robinson, Regan, Hunter, Foster

Guests: Messrs. Korgemagi, Stermac, Dodd

Consultants: Messrs. Crosby, Sobolak, Maitland, McKechnie

1. Minutes of First Meeting

Adopted as presented.

2. Committee Name

The Chairman outlined the decisions made by the N.R.H.C. Committee with respect to committee names, organization and procedures. Henceforth this committee will be known as the Technical Advisory Committee for Highway 406 (Welland Section).

3. Soils Investigation - Welland Canal Route

The Consultants again raised the question of further investigation of soil conditions along the Welland Canal Route pointing out that they are already in possession of all the available data but need further information as to the precise depth at specific locations of a stratified layer which has been found at several points along the route but appears to vary in depth. The exact depth and extent of this layer will have considerable bearing on the design and cost of structures.

A DHO representative agreed that this was a problem, but pointed out that sufficient work had been done to determine that the central route was in fact economically feasible. He stated that any further work would be for the purpose of final design and suggested that no action be taken until such time as the highway route has been decided upon.

The Committee agreed and re-affirmed its previous decision not to proceed at this time.

4. Median Allowance for Rapid Transit

The Chairman advised the Committee that the N.R.H.C.C. had approved the provision of additional median width in the expressway for future rapid transit.

5. Consultant's Presentation - Alternative Alignments

The Consultant's made a further presentation to the Committee with respect to the background of the study, the examination of the alternative corridors and the projected traffic assignments to each. Most of this information is contained in the memorandums entitled "Interim Report on Alternative Corridors through Welland"; and "Traffic Assignments for three Alternative Alignments", forwarded to Committee members early in December, 1970.

It was agreed that the Committee members would give further consideration to this matter and that it be discussed at the next meeting.

6. Planning Aspects

The Committee discussed the question of participation by outside planning agencies, in particular the City of Welland's Staff and the Consultant's engaged by the City to prepare their Official Plan.

It is apparent that the planning information required for the study fall into two categories:

(i) Land use and other general information developed by the City's Consultants in preparing the Official Plan.

(ii) Additional information specifically relating to the alternative corridors and the implications of each. This aspect will have to be dealt with as part of the route studies.

With respect to item (i) it was agreed that the Welland representatives would discuss this matter with their City Council and endeavour to obtain authorization for their Consultants (Proctor & Redfern) to supply the necessary information to this Committee or to M.M.Dillon Limited.

.../3

With respect to item (ii) the Consultants recommended that in view of that firm's background and experience in the area that M. M. Dillon approach Proctor and Redfern to see if they would be interested in carrying out the planning studies required for the purposes of this study.

The Committee agreed with this recommendation and instructed the consultant to discuss the matter with Proctor & Redfern, and determine the scope of the work, costs and personnel to be assigned. The consultants will report on this matter at the next meeting.

7. Evaluation of Freeway Alternatives

The Consultants presented a memorandum entitled "Method for Evaluation of Freeway Location Alternatives", which outlines a proposed procedure for use in this study and itemizes the factors to be analyzed and taken into account in determining the best highway route.

It was agreed that this matter would be considered further after the appointment of consultants to carry out the planning work.

8. Photo Grammetric Mapping

The Consultants informed the Committee that while adequate mapping is available for the proposed central corridor, there is little or none for the two alternative corridors.

The DHO representatives agreed to request their Photogrammetric Section to include the required areas in their Spring aerial mapping program.

The next meeting of the Committee will be held at 150 Berryman Avenue, St. Catharines, at 10:00 a.m. Tuesday, February 23rd and will be an all-day session if required.

Mr. A. G. Stermac

Mr. C. R. Robertson
District Engineer
Hamilton

Materials and Testing Office

December 23, 1970

Highway 406 - Welland Subcommittee - Minutes of Meeting No. 1

Thank you for sending me a copy of the minutes of this meeting.

With respect to the soils information along the route of the present canal, I would like to advise that a considerable amount of information is already available. Several years ago, we retained the services of H. Q. Golder and Associates who studied the St. Lawrence Seaway boring information, the failures along the present canal, and in addition carried out some additional borings. They had shown the soil along the route of the present canal can be divided into three areas for design considerations.

This report is fairly extensive, and I am sure that you have a copy of it in your district office. I would, therefore, concur with the committee that no work be undertaken at this time. The work that we did was carried out to determine the feasability of constructing Highway 406 in the present canal, and the conclusions were that this was certainly feasible. If the Technical Advisory Committee would like to be brought up to date with the consultants information that we have, please let me know and I can arrange for the presentation.

I might add that the Department and the St. Lawrence Seaway Authority under the Chairmanship of Roy Burnfield have a technical committee which is looking into the problems of constructing Highway 406 in the present canal.

I think that it would be useful to have someone from the Functional Planning Office present what has happened so far to this Technical Advisory Committee.

A. Rutka

A. Rutka
Materials and Testing Engineer

AR/re
c.c. Messrs. T. J. Kovich
A. G. Stermac

TECHNICAL ADVISORY COMMITTEE

HIGHWAY 406 - WELLAND SUBCOMMITTEE

MINUTES OF MEETING NO. 1

C Robertson

Location: Regional Municipality Headquarters

Date: December 8, 1970

Time: 10:00 a.m.

Members Present: Hunter, Dobson, Doctorow, Robertson, Regan, Black, Martin (for Greaves); Holenski (Chairman), Landells, Foster

Consultants: Crosby, Sobolak, McKechnie, Maitland, Wade

1. Membership and Procedures

(a) The question of voting memberships, committee procedures and requirements for a quorum were discussed and the following decided:

Voting Members

As per the list attached to the minutes of the 2nd meeting of the Technical Advisory Committee.

Voting Procedure

One vote per organization (DHO, Region and Area Municipalities)

Quorum

One voting member from each organization will constitute a quorum of this subcommittee.

(b) Agendas will be prepared by the consultants and mailed to all members.

(d) Distribution of Minutes - All members of subcommittee, chairman and secretary of Technical Advisory Committee one copy each. Consultants: M. M. Dillon - two copies and H. G. Acres - two copies.

& HAT		NOTE & FILE	
MAYOR		DISCUSS WITH ME	
MUNIC.		PLEASE ANSWER	
SERVICES		NOTE & RETURN TO ME	
ACCOUNTS		INVESTIGATE & REPORT	
FILE		TAKE APPROPRIATE ACTION	
SIGNS & PERMITS		SHOW ME REPLY BEFORE MAILING	

(d) Subcommittees

It was pointed out that as the study progresses there will be a need to form subcommittees to handle such items as property acquisition, finance, etc.

After considerable discussion, it was agreed that this matter be taken up with the full Technical Advisory Committee for clarification of the subcommittees' terms of reference and authority.

2. Monthly Progress Claims

Mr. Crosby briefly reviewed the facts respecting the consultants' management team and inter-company engineering agreements which were previously outlined to the Technical Advisory Committee. He pointed out that the existing agreement between the DHO and M. M. Dillon provides for monthly progress payments and enquired as to the future processing of claims.

DHO representatives stated that there is not, as yet, an agreement between the Region and the DHO covering this study and that for the present the Consultant should continue to submit claims directly to the DHO. Subject to the direction of the Technical Advisory Committee a billing procedure will be set up as early as possible in 1971.

3. Study Progress to Date

The Committee was shown a display map indicating three possible alternative highway routes through the City of Welland developed by the Consultants. Mr. McKechnie explained the principal features of each alternative and the advantages and disadvantages of each. He stated that from the point of view of traffic demand (and hence service to the community) the consultants' advocated the choice of a route through the centre of the City, along the alignment of the present Welland Canal. He did however state that before such a route could be decided upon, it would be necessary to clarify the position of both the St. Lawrence Seaway Authority and the City of Welland with respect to the future disposition of the abandoned canal channel, particularly in regard to the question of dewatering and the maintenance of the City's water supply facilities.

Mr. Sobolak presented a summary of the proposed design criteria for the freeway and requested that the Committee approve of these criteria in order that the consultant may proceed with the preparation of construction estimates. Regional and Municipal representatives questioned the inclusion of an allowance for a future right-of-way for rapid transit in the median strip, which has the affect of substantially increasing the right-of-way widths and property requirements.

(4) DHO representatives indicated that there had been a change in DHO policy and it was now standard procedure to make this provision.

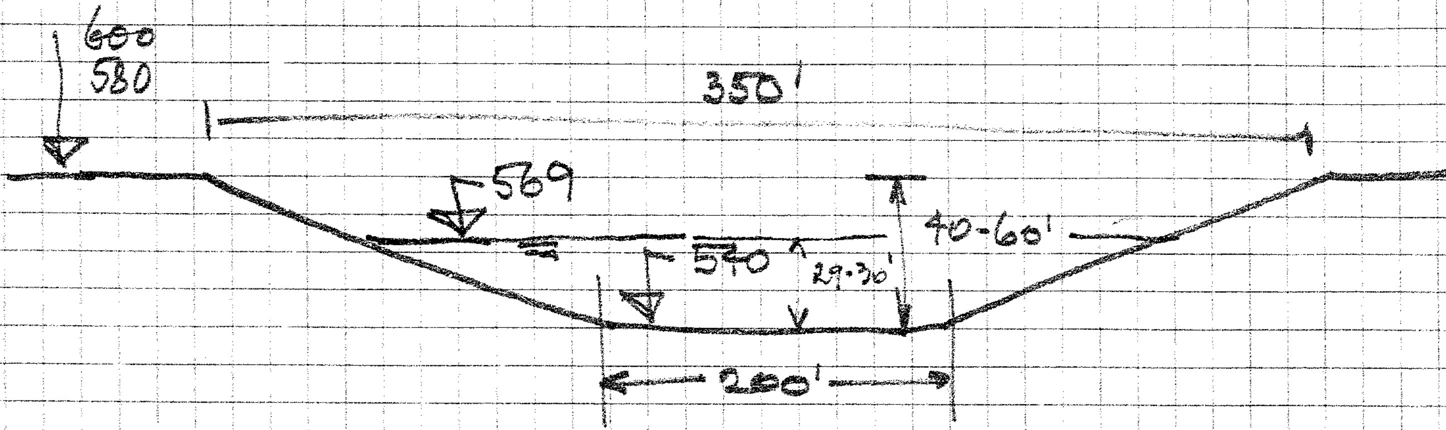
After considerable discussion, it was agreed that the consultants should proceed on the basis of the criteria submitted and, subject to future review.

4. Soils Investigation - Welland Canal Route

Mr. McKechnie informed the Committee that there were indications that there may be a problem with soil conditions along the Welland Canal route. Some investigations have already been carried out by other agencies but the consultants are of the opinion that further work is required. He requested authority to proceed with ten (10) boreholes at an estimated total cost of $15,000.

(3) The Committee directed that in view of the many uncertainties surrounding this particular route that no work should be undertaken at this time.

The Committee adjourned at 1:00 p.m. with the next meeting to be held at 10:00 a.m. on January 19, 1971 at 150 Berryman Avenue.


600
580
350'
569
540
29-30'
40-60'
200'

CANAL BOTTOM EL. 540.0
WATER LEVEL IN CANAL 569.0

GROUND ELEVATION VARIES 580 - 600

SECTION I (STA 800+00 TO 880+00)
VERY FEW FAILURES
CANAL BANKS EL ~ 582.0 Fig 8

SECTION II (STA 890+00 TO 960+00)
ONLY WHERE BANKS ARE HIGH
Fig 9

SECTION III (STA 980+00 TO 1110+00)
MANY FAILURES
Fig 10.

PIEZOMETRIC LEVEL WITHIN OVERBURDEN IS GENERALLY 5-10 FT ABOVE NORMAL CANAL LEVEL

NOTE:
CLOSING OF CANAL BY RLWY (NORTH SIDE) AND DAM AT TOWNLINE RD ON THE SOUTH END WILL BE COMPLETED BY END OF 1971.

CANAL BANK FAILURES

(a) SHALLOW
(b) DEEP

MAJORITY ON WEST (HIGH) SIDE OF CANAL

IN SECTION III (PARTICULARLY) CLAYS BELOW DEPTH OF 30-40 FT ARE EITHER NORMALLY CONSOLIDATED OR ONLY SLIGHTLY OVERCONSOLIDATED

IN SECTION III THERE ARE DEPOSITS OF STRATIFIED CLAYS. [DIRECTIONAL SHEAR STRENGTH] $C_u \cong 500$ PSF.
STA 960+00 TO 1050+00 POTENTIALLY UNSTABLE

DRAWDOWN

CANAL BANKS H=45 FT AND SLOPED 20° OR FLATTER ARE EXPECTED TO BE STABLE DURING DRAWDOWN

AREAS OF NO PROBLEM

STA 800+00 TO 966+00

NO PROTECTIVE MEASURES DURING DRAWDOWN ARE NECESSARY [EXCEPT POSSIBLY PART OF EAST BANK WHERE IT FORMS A DYKE TO ADJACENT WETLANDS.]

STA. 1050+00 TO 1100+00

SUBSOIL CONDITIONS POOR BUT BANKS ALREADY FLATTENED.

PROBLEM AREAS

STA 967+00 TO 1050+00 —— GENERAL

DETAILS

STA 967+00 TO 1054+00

RETAINING WALLS

BRIDGES Nº 13, 14, 15 & 16

SYPHON

ALL BRIDGES NEED TO BE PROTECTED !

DUMPING MATERIAL PRIOR TO DEWATERING

STA 965+00 TO 1033+00

STA 1033+00 TO 1045+00

file 022

ACRES

March 30, 1970
P2223.00

Department of Highways, Ontario
Foundations Office
Room 107, Laboratory Building
Downsview, Ontario

Attention: Mr. A. G. Stermac
Principal Foundations Engineer

Gentlemen:

Highway 406 - Functional Planning Report

We enclose herewith a copy of the notes from the meeting held in the Department of Highways Offices for your information.

Yours very truly,

S. T. Maitland

S. T. Maitland
Project Engineer

:gt
Encl.

H. G. ACRES LIMITED
Consulting Engineers
1259 Dorchester Road
Niagara Falls, Canada

Telephone 416-354-3831

March 30, 1970
P2223.00

HIGHWAY 406 - FUNCTIONAL PLANNING REPORT

Notes from a Meeting held in the
Department of Highways Downsview Offices

Present:	Messrs.	A. G. Stermac)	Principal Foundations Engineer, Materials and Testing Division, Department of Highways
		R. M. Isaacs) S. T. Maitland)	H. G. Acres Limited

Purpose of the meeting: To discuss the need for additional boreholes in the area of the old Welland Canal north of the city.

Requirement for Extra Drillholes and Testing

Acres tabled a drawing indicating the stratigraphy as interpretated from existing Geotechnical Reports and indicated that in the area north of East Main Street the location and/or existence of the stratified clays was in doubt. Mr. Stermac agreed that for the final report it was necessary to undertake some drilling in this area, but that he would prefer that this drilling be delayed until after the April 2 meeting with the Department of Highways, Ontario, at which an overall presentation of three alternative routes would be given by M. M. Dillon Limited.

The number of drillholes involved is six north of East Main Street and three between East Main Street and Townline Road.

Execution of the Work

Mr. Stermac indicated that he would be satisfied if Acres control the contract for this drilling work, as the Department of Highways had not the facilities for doing this work and would have to let a Contract themselves. He indicated that Acres should also do the testing and interpretation of the results which should then be forwarded to him for review and approval.

Feasibility

From a general discussion, it was agreed that the feasibility of either filling the canal completely or locating depressed highway within it was not in doubt although the cost would be high compared with normal highway construction. A great variety of solutions to the problem of routing a highway through Welland in the central alignment exist, and the most economical is not necessarily the best or the most acceptable to the local authority.

STM:gt

S. T. Maitland

SUBJECT: HIGHWAY 406 FUNCTIONAL REPORT

RE: PRELIMINARY EVALUATION OF ALTERNATIVE CORRIDORS WITHIN THE CITY OF WELLAND

INTRODUCTION

We have made a very brief examination of three proposed alternative alignments of Highway No. 406 in the City of Welland. The material used in this review was:

(i) the 1964 Traffic Study (Acres Research and Planning Limited), and

(ii) aerial photomosaics of the City (May 1969).

The alternative corridors, identified as "A", "B" and "C", are shown on the existing 1964 Land Use Plan as well as the 1990 Land Use Forecast and Development Plan.

EVALUATION

The three corridors have varying degrees of impact on the existing and future land use patterns, and conversely, the land use - expressway relationship is variable not only between corridors but throughout the length of each corridor.

The 1964 Land Use Plan shows that none of the corridors involve a major disruption of existing urban development. However, considering the corridors within the context of the 1990 Land Use Forecast and Development Plan, it can be said that while there will be come slight impact on the land use pattern within corridors "A" and "C", there would be considerable impact on the land use pattern in the northwest sector of the City which is to be developed as one large residential community comprising several neighbourhoods. It is impossible to determine the extent of any physical impact on the present and future land use patterns in the absence of specific information concerning the urban structural form of the area, together with its sociological and demographical characteristics.

The 1964 Traffic Study states that the 1972 and 1990 population forecasts are based on an annual growth rate of 1-3/4 percent and that the 1972 figure was increased to allow for an "influx" of approximately 2,500 persons. The 1969 assessed population figure supplied by the Department of Municipal Affairs, sets the population figure at 43,706 which is somewhat higher than the 1964 forecasting method would have indicated. The 1964 labour force figure was similarly projected, however, it is felt that these figures may also be slightly low. Thus, it appears that the rate of development within the City may actually be exceeding the earlier forecasts.

A review of the figures contained in the 1964 Traffic Study indicates that the population in the year 1990 will be distributed almost equally on both sides of the old canal; with about 52% in the eastern half and about 48% in the western half. Since the eastern area is somewhat smaller than the western area, this suggests that the eastern area is slated to develop at a higher density. This will almost certainly be true in the area peripheral to the C.B.D.

Although the 1990 population is forecasted to be distributed as mentioned, the labour force distribution is such that about 83% will be working east of the old canal with about 17% in the western portion. It would seem probable, therefore, that the home/work travel pattern will be predominately in a north-south direction on the east side and in an east-west direction for those living on the west side. This of course raises the question of the degree to which each of the Highway 406 corridors will function in the capacity of a local distributor route. It would seem that corridor "C" would not provide this service nearly as well as either corridor "A" or "B". Furthermore, urbanization of the eastern part of the City will be almost complete around 1990, with the Seaway Authority holdings forming an effective barrier to any further extension easterly. At this point in time, the western part of the City will still be able to accommodate more growth and the centroid of the total population will ultimately shift to somewhere west of the old canal. Just when the City becomes completely urbanized, and what kind of a density distribution pattern might exist, cannot be determined on the basis of present information.

It would appear that the C.B.D. will continue to be more or less central to the population area for the next 20 to 30 years. Again, insufficient information is currently available from which adequate analysis can be made of any effect on the C.B.D. It is fair to say, however, that corridor "A" affords the most convenient and direct access for the regional shopper destined to the City's C.B.D. In 1964, the regional movement to and from the two C.B.D. zones, during the three hour survey period, amounted to 17.8% of the total movements generated by this area. Since not all movements are served by the freeway, this does not provide convincing evidence suggesting that corridor "A" is essential to the C.B.D. However, in a theoretical sense, the construction of a freeway within Corridor "A" should enhance the regional attractiveness of the Welland C.B.D. while the development of a freeway within the alternative corridors would do little to serve this function.

The foregoing discussion points up the fact that population, employment and land use elements should be reviewed and up-dated to provide a more adequate input to the decision making process of selecting the most beneficial freeway corridor. In addition to the updating of this base data, it is felt that certain "areas of concern" exist which deserve careful examination as an integral part of the total process. Answers to questions in these areas are extremely difficult to quantify and require a sophisticated approach to the making of various judgement decisions. Some questions in these areas are as follows:

1. Physical

- What effect does the expressway have on the future urban fabric of the area?

- Does the alignment fracture a neighbourhood, a group of neighbourhoods, or school service areas?

- Is it possible to create future viable neighbourhood units in proximity to the corridor?

- Can the expressway "fit in" with existing development with a reasonable degree of urban architectural compatability?

- What will be the view of the adjacent areas and of the City from the expressway?

- What will be the view of the expressway from adjacent areas?

- Does the freeway provide an interesting and enjoyable experience for the driver as he progresses through the rural-urban-rural settings?

- Is the expressway "in scale" with its immediate surroundings, i.e. does it dominate by its width, height or support system?

- What is the effect on its environment if elevated, or depressed?

- What considerations regarding noise deterents?

- Can the expressway create an opportunity for constructive change, i. e. the development of well-landscaped lineal parks with useable open space either along side of the required right-of-way or beneath the elevated portions?

2. Commerce and Trade

- To what extent will the business, cultural and tourist trade benefit from the alternative corridors being considered?

3. Sociological

- What is the dominant social attitude towards expressways in Welland?

4. Socio-Economic

- What benefits acrue through cost-benefit analysis, i.e. home/work driver time, accident reduction, property values, etc.?

As stated earlier, the above questions require the application of valued judgements and therefore the answers are not fully quantifiable. It is, however, possible to establish a system of weights as a basis for evaluating and comparing answers in the various areas of concern. These answers would then be used as "support material" to be analyzed in conjunction with the usual land use and transportation data.

CONCLUSIONS AND RECOMMENDATIONS

It is apparent that the conclusions reached can only be considered as tentative ones. Although certain potential advantages and disadvantages for each corridor have been suggested, the depth of analysis does not justify the development of final conclusions - this is unfortunate.

During recent years, most areas of North America have become highly reactionary to expressway proposals put forth by road building agencies. The change in thinking is probably best summed up by the December 1969 Report of the "Organization for Economic Có-operation and Development", as excerpted below:

> *Today the best design for a transportation system is no longer necessarily the one which results in the lowest*

capital costs or in lower user costs, or which produces the biggest reduction in travel time. Rather, it is that design which yields the highest social return on the investment and which reconciles most effectively the conflicting interests of the individuals and various groups in the community affected by the proposed project.

The Memorandum of Agreement does not give direction to undertake any of the analyses which today are viewed as important input to the decision making process. We therefore recommend that the Regional Functional Planning Engineer consider the desirability of expanding the study terms of reference to include:

(i) the socio-economic impact of the project;

(ii) the physical effects of the project within the City of Welland; and

(iii) the means by which a satisfactory degree of essential local dialogue may be achieved.

HIGHWAY 406 FUNCTIONAL REPORT
(Merritt Road to Port Colborne)

AGENDA FOR PROJECT MANAGEMENT MEETING #9

To be held at 10:00 a.m., Wednesday, 18 March 1970, at the Central Regional Office.

The following items are proposed for discussion or review:

1. Discuss request for additional soils investigation within Corridor "A" (central route)

    a) Consultant also requests a copy of soils investigations carried out recently for the Lincoln Street crossing of the Welland River (this information should be of value in relation to Corridor "B" proposal). Damas & Smith

2. Review Consultant's Monthly Progress Report for the month ending 28 February 1970.

3. Review revised Tentative Design Criteria for the Highway 406 project (revised 17 March 1970).

4. Discuss current lack of progress in detailed analysis of Corridor "A" within the Welland Canal. The specific information required for this analysis was listed under item 5 of Project Management Meeting #8.

5. Bridge inventory data and traffic capacity calculations have not been completed due to lack of information from the St. Lawrence Seaway Authority essential to this review.

6. Presentation of preliminary alignment proposals within Corridors "A", "B" and "C" (Dwg. 6513-01-TP-9, 3 of 3).

    a) Review general corridors - traffic service, interchanges, etc.

    b) Review the attached report: Preliminary Evaluation of Alternative Corridors Within the City of Welland.

7. Consider tentative date for next meeting and information to be developed for this meeting.

8. Other business.

6513-01
17 March 1970

M. M. DILLON LIMITED

fiev ass

Functional Planning Section,
Central Region,
Downsview 464, Ontario.
July 9th, 1969.

Telephone: 248-3581

Mr. Y. Leblanc, Eng.,
The St. Lawrence Seaway Authority,
P.O. Box 200, St.-Laurent,
Montreal 379, Quebec.

Re: Highway 406 - City of Welland,
Your File: 50-17-150C

Dear Sir:

This is to acknowledge your letter of June 26th, 1969, in which you advised that you have now available the Water Quality Treatment Study for the future decommissioned section of the Welland Canal, and this information is now available for future meetings of the Technical Committee.

At the present time, the Department is considering assigning the preparation of a functional report to a consultant and, therefore, no design work is now underway by my office.

As soon as a consultant has been selected and is commencing work, I will call a meeting of the Technical Committee in order that this information, and other information that has been prepared by my own staff, can be discussed and passed on to the consultant.

Yours very truly,

R. G. Burnfield,
Regional Functional Planning Engineer.

RGB/amn.

c. c.

Members of Committee.- namely:-

(S.L.S.A.) M.H. Rehman.
(S.L.S.A.) Y. Leblanc and F.V. Jackson;
(D.H.O.) H. Tregaskes, F.I. Hewson,
D. Garner, A. Rutka, A. Stermac,
L. Schwabl (Chairman:R. G. Burnfield)

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: FILE

From: L. Schwabl,
Project Planning Engineer.

Attention:

Date: August 2nd, 1968.

Our File Ref.

In reply to

Subject:

Minutes of the 1st Meeting,
Highway 406, Welland Technical
Committee, Dist. #4, Hamilton.

Meeting held July 31st, 1968 from 10:00 a.m. to 12:00 noon in the Downsview Boardroom E-1.

Present were:
R. G. Burnfield, Chairman,
H. A. Tregaskes, D.H.O.
Y. LeBlanc, S.L.S.A.,
F. Jackson, S.L.S.A.,
M. H. Rehman, S.L.S.A.,
F. I. Hewson, D.H.O.,
D. Garner, D.H.O.,
A. Rutka, D.H.O.,
A. Stermac, D.H.O.,
L. Schwabl, D.H.O.

Terms of Reference:

To study the feasibility, determine possible problems, recommend economical solutions, prepare preliminary design schemes, and estimate construction cost for:

A new Highway 406 - presently proposed to run in a north - south direction, utilizing much of the existing ship canal bed where it passes through the built-up area of the City of Welland.

I. Introduction by the Chairman:

1. Review of the proposed new route of the ship canal and the so-required changes of the road network.

2. Brief presentation of the presently proposed Highway 406 between the Q. E. W. (St. Catharines) and Highway 3 (Pt. Colborne).

3. Brief review of the traffic demand within the City of Welland as prepared by H. G. Acres and its recommendation to place a new Expressway along the canal.

Summary of discussion followed the introduction:

1. The presently proposed alignment for Highway 406 consists of four lanes divided, with access facilities at Thorold Stone Road, and Lincoln Street, underpasses, existing lift bridges at Main, Lincoln and Ontario Streets and the New York Central Railway. The canal bed will be utilized by this highway between approximately Quaker Road and the railway crossing, a distance of about 3 1/2 miles.

   The vertical alignment is proposed to be under the existing structures at a depth to allow proper clearances.

2. Water Supply:

   Seaway Authority reported that some 68 industries, private property owners and public utilities use, at present, the canal water. Although there is no written lease agreement with the users, the Authority feels obligated as a goodwill gesture to have this service continued after the relocation of the ship canal.

   The section of the canal along which Highway 406 will be placed, will logically be dewatered and filled with earth material to about half of its present depth. To supply water to the users, it was suggested:

   (a) to lay a pipe from the remaining (but unused) canal south of Lincoln St. downstream to the users of the southern half, and
   (b) to build a small stream from the remaining canal north of Quaker Road in a southerly diversion up to the Welland River crossing for the users located on the northern half.

   The drain of the proposed highway would then be an "internal" design problem.

Much was said about how such facilities could be provided, but they are considered at present as a secondary problem.

II. Existing Structures:

Both the Seaway Authority and the Department's Bridge Office believe that the existing lift bridges are sound and can remain, providing that some minor repairs only are carried out.

Particular attention would have to be given to a proper stabilizing of the abutments, again a secondary problem at present.

III. Material:

A fair amount of soil investigation had been carried out already with the conclusion that no particular problems are expected.

Conclusion of the Meeting:

1. Functional Planning will prepare design schemes for Highway 406 within the City of Welland taking into account a westerly shift of the most northern portion of the present proposal.

2. Copies of the revised proposal should be forwarded to the:
   Seaway Authority,
   Materials & Testing Division, and
   Bridge Design Section,
   for studying purposes.

   The St. Lawrence Seaway Authority are to use this plan to develop a system with costs, for supplying water to the present users, or alternatives.

   The Materials & Testing Division are to determine locations of materials to be placed in the Canal, and to develop a method for construction.

3. The representatives will meet again with comments and recommendations concerning the designs. Problems will then be reviewed and further investigations, to be undertaken, assigned.

R. G. Burnfield,
Regional Functional Planning Engr.

L. Scnwabl,
Project Planning Engr.

LS/mw

Department of Highways Ontario
Copy for the information of

A. Stermac

file with report
by H Q Golder
on Hwy 406
AS

Functional Planning Division,
Downsview, Ontario,
December 2nd, 1968.

Telephone: 248-3581

The St. Lawrence Seaway Authority,
P.O. Box 200, St. Laurent,
Montreal 308, Quebec.

Attention: Mr. Y. Leblanc

Re: Highway 406, Welland Technical Committee, Dist. #4, Hamilton.

Dear Sir:

This is to acknowledge your letter of November 22nd, 1968 enquiring as to progress of studies for the use of the existing Welland Canal for the route of Highway 406.

Last July 31st, a meeting of the Technical Committee was held in order to obtain as much information as possible from the various divisions of the Department of Highways and the St. Lawrence Seaway Authority in order to determine functional designs for the proposed freeway. I understand that the information has been received, but due to other commitments, designs have not as yet been prepared. It is expected that sufficient work will be complete within the next few weeks so that a meeting can be arranged for sometime during the month of January, 1969. You will be notified precisely of the date of the next meeting when we are more certain of having the information available.

Yours very truly,

R. G. Burnfield,
Regional Functional Planning Engineer.

RGB/mw

c.c. M. Rohman, M. Paquin, F. E. Jackson - S.L.S.A.,
A. Rutka, A. Stermac, H. Tregaskes, G. K. Hunter, L. Schwabl, - D.H.O.

THE ST. LAWRENCE SEAWAY AUTHORITY
ADMINISTRATION DE LA VOIE MARITIME DU SAINT-LAURENT

P.O. Box 200, St-Laurent
Montreal 308, Quebec.
November 22, 1968

File: 50-17-150C

Department of Highways of Ontario,
Downsview,
Ontario.

Attention: Mr. R.G. Burnfield,
Regional Functional Planning Engineer

Gentlemen:

RE: Highway 406, Welland Technical Committee, Dist. #4, Hamilton

Since our meeting of July 31st, 1968, we have not been further informed of your progress on the planning of the section of Highway 406 using the future decommissioned Welland Canal.

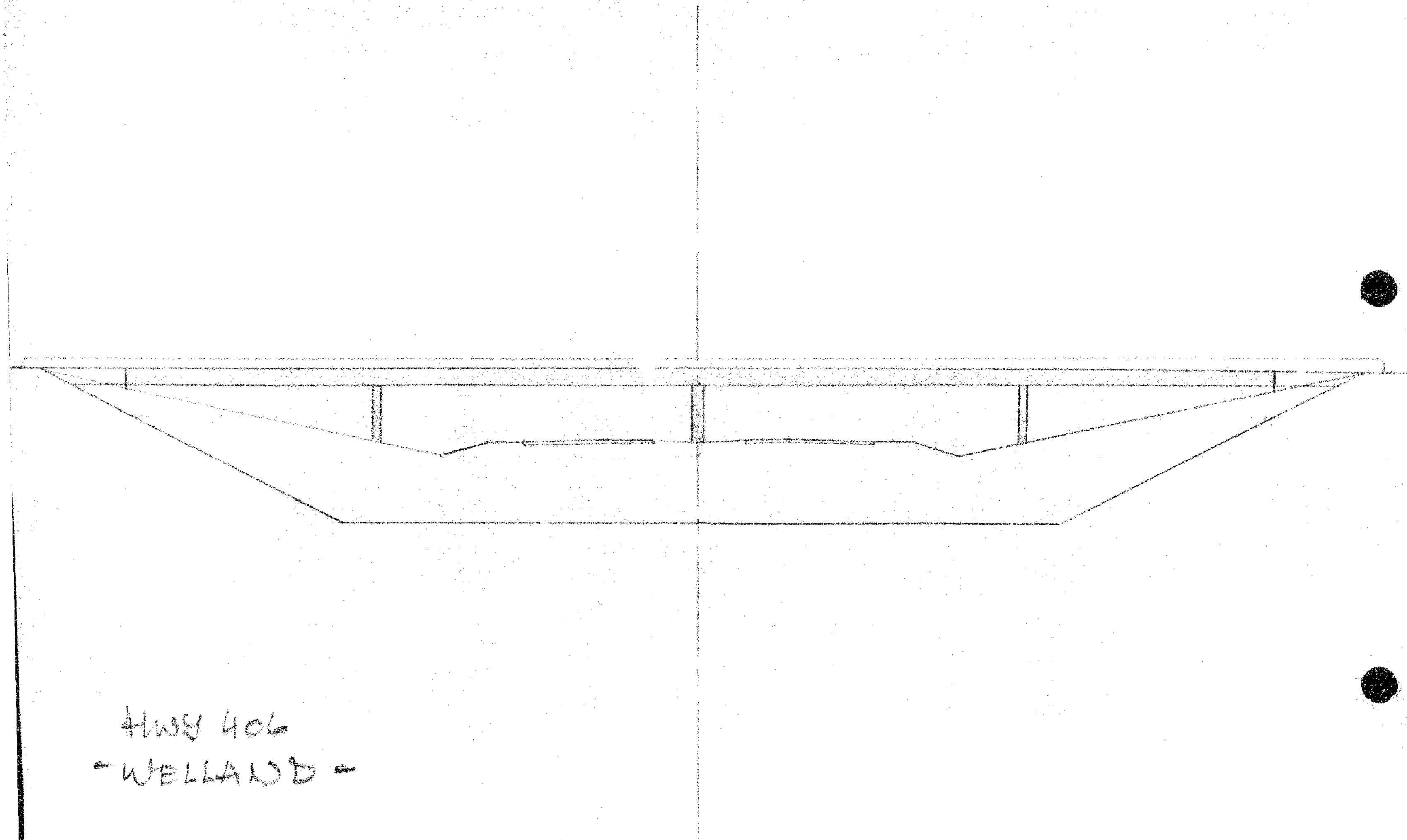
At this time we are specially interested in the timing of your project because the size and location of some structures may thereby be affected in their design. Hence we will appreciate your informing us of your progress to date.

Yours very truly,

THE ST. LAWRENCE SEAWAY AUTHORITY

Y. Leblanc, Eng.
Planning Division

YL/jd



HWY 406
- WELLAND -

Scale = 1" = 40'

Mr. A. Stermac

Telephone: 248-3266
Area Code: 416

Materials & Testing Division,
Downsview, Ontario,
October 18, 1967.

Dr. H. Q. Golder,
H. Q. Golder & Associates Ltd.,
3151 Wharton Way,
Cooksville, Ontario.

Dear Sir:

Re: Welland Canal

This will confirm the arrangements made after our meeting of October 17, 1967, with Mr. Milligan and you, to retain your firm to study the feasibility of utilizing the dredged material for subsequent backfilling of the existing canal, and also the alternative of utilizing the excavated material, which would be above optimum moisture content, for backfilling the canal after it has been in a stockpile for several years.

We would be pleased to receive your comments on these items. Charges for your services will be in accordance with your letter of August 4, 1967.

Yours truly,

A. Rutka,
Materials & Testing Engineer.

AR:pa

cc: A. Stermac
W. Fry

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM Telephone: 248-3415.

To: Mr. R.G. Burnfield.
Reg. Funct. Planning Eng.,
Functional Planning Branch,
Admin. Bldg.

Attn: Mr. W. Friedmann

From: G.K. Hunter,
Toronto Regional Road Design.

Date: July 26, 1967.

Our File Ref.

In reply to

Subject:

RE: Feasibility Study Portion of Proposed Highway 406 Within Welland Canal - Golder & Associates. District 4, Hamilton.

I am enclosing correspondence of June 20, 1967 from Mr. J. Blevins with regard to the above.

I would suggest that these points be considered before any final decision is made on the use of Highway 406 within the Welland Canal.

G.K. Hunter

G.K. Hunter,
Regional Road Design Engineer.

GKH/bb

c.c. D.W. Farren,
H. Greenland,
B. Davis,
T. Stermac, ✓
A. Rutka.

DEPARTMENT OF [illegible] ONTARIO

MEMORANDUM

Telephone: 248-3446

To: Mr. G.K. Hunter,
Reg. Road Design Engineer.

From: J.H. Blevins,
Toronto Regional Road Design.

Date: June 20, 1967.

Our File Ref.

In reply to

Subject: Re: Feasibility Study Portion of Proposed Highway 406 Within Welland Canal - Golder & Associates.

The above report has been reviewed and while I am not in a position nor do I wish to question the data or recommendations, it would appear that some of the anticipated problems could be avoided by reconsidering the original concept. I am not familiar with either the background or history leading up to the study and, therefore, some of the assumptions and proposals contained herein may be completely out of line. However, on reviewing the report, there appear to be certain aspects of the project that warrant further consideration.

1) It is assumed that the Highway alignment was located in the abandoned channel for reasons of right-of-way as well as the possibility of utilizing the old channel as a depressed freeway. However, the anticipated dewatering problems appear to render the old channel more of a liability than an asset.

2) It is assumed that the new relocated channel will be bigger and better than the old and that there will be more excavated material than there is old channel to be filled, resulting in a surplus of material available. The excavated material appears to be of questionable quality, particularly if deposited in water. However, I would point out that equally questionable material has been used for this purpose in the past. Furthermore, it was not a question of having a relatively large quantity available on the site but rather one of having to go to considerable expense to obtain it. In addition, this material was used to backfill swamps with little or no lateral support into an existing earth channel as is the case at hand.

Continued ..... /2

Mr. G.K. Hunter - Re: Feasibility Study.

3) It is assumed that the proposed highway grade of 540 was established to salvage the existing structures in Welland. However, there is no need to retain this grade throughout the entire project. In short, except at the bridges, the grade could be establis ed at any elevation desirable, even to the point of constructing a fill section.

4) While the report indicates that complete dewatering will cause slope failures, there is no specified minimum depth of water at which the slopes will become critical. In short, through the section influenced by the bridges, it may be possible to reduce the water level to a point that it would be feasible to backfill up to the water level with an acceptable material and then complete the fill in the dry with native material or it might be possible to complete the backfilling with native material, provided that the top 10 feet or so are placed in the dry. It may even prove feasible to use some type of soil stabilization to improve the bearing capacity of the native material when placed in water.

5) In view of the anticipated abundance of surplus material in the area, it might, as a last resort, be worthwhile to investigate the possibility of reversing the grades of the expressway and the canal crossings. This would require replacing the various structures, however, they would all be underpasses with relatively short spans. The necessary fills could be accommodated within the 350-foot right-of-way available.

JHBlevins GB.

J.H. Blevins
Sr. Expressway Design Engineer

JHB/GB

Mr. A. Stermac

Telephone: 248-3266

Materials & Testing Division,
Downsview, Ontario,
March 2, 1967.

H. Q. Golder & Associates,
2444 Bloor Street West,
Toronto 9, Ontario.

Attention Mr. V. Milligan

Re: Welland Canal
Feasibility Study

Dear Sir:

Further to our telephone conversation of February 28, 1967, I wish to advise that I have now made arrangements with Mr. W. A. O'Neil of the St. Lawrence Seaway Authority, permitting the Department to undertake an investigation of the materials along the canal bottom. A copy of my letter to Mr. O'Neill in this regard, is attached.

Please consider this additional work to be performed by your firm, as an extension of the work assigned to your company on the feasibility of locating Highway 406 in the canal, in accordance with my letter of July 8, 1966.

Yours truly,

AR

A. Rutka,
Materials & Testing Engineer.

AR:pa
Attach.

c.c. Messrs. A. Stermac,
H. Konings.

Mr. A. Stermac

Telephone: 248-3266

c.c. Messrs. J. Walter,
T. Hewson,
A. Stermac,
T. Kovich,
G. Tustin - S.L.S.A.
H. Q. Golder & Assoc.

Materials & Testing Division,
Downsview, Ontario,
March 2, 1967.

Mr. W. A. O'Neil,
Project Director,
Construction Office,
St. Lawrence Seaway,
St. Catharines, Ontario.

Dear Mr. O'Neil:

This will confirm our conversation of March 1, 1967, regarding the shallow borings and probings we wish to place in the existing canal through Welland, for the purposes of deciding the amount of soft material in the canal bottom. This work will commence immediately, and we will be finished with it before the canal is required for shipping purposes. H. Q. Golder & Associates has been retained by the Department for this investigation.

During our conversation you indicated there was a possibility that you may be placing a shaft to obtain soil samples, and to observe the soil conditions on the new canal alignment. We are very much interested in this shaft. If it is anywhere in the vicinity of the proposed tunnels, we might like to go down further than what you plan.

We did place a shaft down 85' in the City of Welland along the existing canal, a couple of years ago. Our report on this was given to George Tustin, but if you would like any further specific information regarding our test shaft, please let me know. Also, I recently sent George a copy of a soils report prepared by H. Q. Golder & Associates, regarding the feasibility of using the existing canal for our Hwy. 406. I have an extra copy of it, which I am sending to you under separate cover. I am sure that your Mr. Earl Nesbit could make use of it. Perhaps Mr. Nesbit could keep me advised of the progress of your shaft planning.

Yours truly,

A. Rutka

AR:pa

A. Rutka,
Materials & Testing Engineer.

Mr. A. Stermac

Materials and Testing Division

Downsview, Ontario
July 15, 1966

H. Q. Golder and Associates
2444 Bloor Street West
Toronto 9, Ontario

ATTENTION: Mr. V. Milligan

Gentlemen:

Re: Welland Canal Study

I am submitting a report on "A Study on a Slide of July 3, 1939", prepared by Mr. Kwan, January, 1966. This was sent to the Department by the St. Lawrence Seaway Authority, and it may be useful in your study of the overall stability problems connected with the present Welland Canal.

I have been in touch with Mr. Bill O'Neill, Regional Engineer for the Seaway Authority, St. Catharines, and he advises that he will send me the foundation plans for all of the structures from the siphon at the Welland River to Bridge #17 at the Welland Junction.

As soon as I receive these plans, I will pass them on to you.

Yours very truly,

aR

A. Rutka
Materials & Testing Engineer

Encl.
AR/re
c.c. Messrs. J. Walter
W. Bidell
A. Stermac
T. Kovich

Mr. T. Stermac

Materials and Testing Division

Downsview, Ontario
July 8, 1966

H. Q. Golder and Associates
2444 Bloor Street West
Toronto 9, Ontario

Gentlemen:

Re: Investigation of the Welland Canal, City of Welland

Further to our discussion and visit to the site on July 7, 1966, please consider this letter your authority to proceed with the investigation of the present Welland Canal.

A brief statement of the problem as we see it is as follows:

The Department is considering the location of Highway 406 in the present Welland Canal when it is drained and abandoned for shipping purposes. Since the canal was constructed, many minor and major slope failures have taken place, without any change in water level. The slopes would be expected to be less stable during the emptying phase. If some failures occur, extensive property damage might result. It is also proposed to raise the road bed above the present canal floor by as much as ten feet.

The depth of the soft clay at the bottom of the canal is not known, but it can be assumed that it should be removed to construct a stable road bed. The removal of the soft clay might also affect the slope stability.

The critical period of slope stability would be during the draw-down period, and it would last during construction. New slopes can be designed to ensure long-term stability.

Our suggested approach to this investigation is in accordance with our discussion and is as follows:

1. Compile and study all available information regarding geology, soil stratification, and soil properties, as well as construction and canal performance records.

The following information was supplied in this connection:

(a) Preliminary Review of Geotechnical Properties of Overburden Deposits, Bridge 13 to Bridge 18, Welland Canal - by T. G. Tustin, January, 1964.

(b) Preliminary Review of Subsurface Conditions, Bridge 12 to Bridge 13, Welland Canal - by T. G. Tustin, April, 1966.

(c) Preliminary Summary of Subsurface Conditions for Channel Relocation, Bridge 12 to Ramey's Bend, Welland Canal - by T. G. Tustin, February, 1966.

(d) Cross-Sections from Sta. 750+00 to Sta. 1280+00 - Welland Canal.

2. Pin point the locations where slope failures cannot be tolerated under any circumstances(bridges, transmission towers, etc.).

3. After your review of all records and your selection of the potential critical problem areas, a meeting should be held with the Department to advise of your findings and to determine the extent of any subsequent field and laboratory work, in connection with the treatments that should be employed to ensure the construction of the highway in a safe and economical manner.

If you require additional plans from either the Department or the St. Lawrence Seaway authority, please contact me and I will get them for you.

Charges for this work will be in accordance with your Schedule of Rates that we have in our files with your firm dated October 1, 1965.

Yours truly,

A. Rutka

A. Rutka
Materials & Testing Engineer

AR/re
c.c. Messrs. J. Walter
H. Greenland
T. Stermac
T. J. Kovich
G. Wrong
W. Fry
W. Bidell

cc: A. G. Stermac cc: Gen. File

Hwy. 25-406
Re: Location in the present Welland Canal

Mr. A. Rutka,
Materials and Testing Engr.,
Room 102, Lab. Bldg.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Room 107, Lab. Bldg.

June 22, 1966

Hwy. 406 - Location in the Present Welland Canal.

It is proposed to locate part of Hwy. 406 in the Welland canal once it is drained and abandoned for shipping purposes.

There are numerous problems connected with the above proposal that will require further investigation and work.

Probably the most important information required for the feasibility study is the information regarding the subsoil conditions and the performance of the canal slopes during the emptying phase as well as during construction and final stage of the highway.

It can be stated that very valuable information regarding the stretch of the present Welland canal under consideration for placement of the proposed Hwy. 406 does exist and is available.

This information is mentioned below, under 1, 2, 3 and 5. Information mentioned under 4, deals with the relocation of the canal and is, therefore, of only indirect importance.

In addition to the below mentioned information made available by the St. Lawrence Seaway Authority, there is also the very comprehensive report prepared by H. Q. Golder and Associates Ltd. regarding the soil investigation for the Welland tunnel at Welland.

The information contained in the two reports (mentioned under 2 and 3) prepared by Mr. T. S. Tustin of the S.L.S.A., is factual and deals primarily with the physical properties of the various soil strata encountered along the stretch of the canal between Bridge 12 and Bridge 18 (approx. Sta. 765 to 1143). No attempt was made to give these results any engineering interpretation. There are also some suggestions regarding additional work to be carried out in order to clarify or further substantiate certain statements.

cont'd. /2 ...

The mentioned information, however, was obtained for reasons and purposes different from that now under consideration. It will therefore be necessary to study the available information in the light of this new problem and supplement it wherever and whenever necessary.

It is a known fact that numerous slides of the canal banks have occurred in the past. This is well documented in the report mentioned under 1. The eventual draw-down will temporarily aggravate the condition and additional slides and failures are to be expected during this period. Since the highway will be on fill within the canal excavation, conditions will be greatly improved and the danger of long-term slides will be eliminated.

Positive protection has to be assured in places where failures cannot be tolerated under any conditions. These are bridges, important buildings, transmission towers, etc. It will be necessary to compile an inventory of these sites and provide recommendations for preventive treatment.

The proposed study may result in the finding that relatively inexpensive measures could provide conditions that would assure the stability of the banks along their entire length during draw-down and highway embankment construction. The excavation and removal of the top of the slope resulting in an overall flattening and shortening of the canal banks, should be given serious consideration.

It is important to know in what condition the canal bottom will be found when the canal is drained. As mentioned earlier, the highway will be on fill to be placed on the canal floor. If soft material is deposited on the bottom, it will have to be removed prior to fill placement. Specific recommendations pertaining to this problem will have to be made.

It is believed that because of the highway fill, the long-term stability problem of the slopes does not exist. However, specific recommendations should be made regarding the final slopes, especially if any temporary measures to assure stability were applied during the draw-down or construction periods.

cont'd. /3 ...

AVAILABLE INFORMATION:

1. Report on Slope Movements - Welland Canal -
By F. L. Peckover and R. G. Tanner,
March, 1959.

2. Preliminary Review of Geotechnical Properties of Overburden
Deposits, Bridge 13 to Bridge 18, Welland Canal -
By T. G. Tustin,
January, 1964.

3. Preliminary Review of Subsurface Conditions,
Bridge 12 to Bridge 13, Welland Canal -
By T. G. Tustin,
April, 1966.

4. Preliminary Summary of Subsurface Conditions for Channel
Relocation, Bridge 12 to Ramey's Bend, Welland Canal -
By T. G. Tustin,
February, 1966.

5. Cross-Sections from Sta. 750+00 to Sta. 1280+00 -
Welland Canal (2 sets).

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cont'd. /4 ...

WORK TO BE CARRIED OUT - TERMS OF REFERENCE:

1. Compile and study all available information regarding geology, soil stratification and soil properties, as well as construction and canal performance records.

2. Establish the usefulness of above mentioned compiled information with regard to the intended placement of the proposed Hwy. 406 within a portion of the abandoned drained present Welland canal.

3. Compile an inventory of sites where slope failures due to canal water draw-down cannot be tolerated under any circumstances (bridges, transmission towers, etc.).

4. Study and establish on the basis of available information the probability of slope failures along the remaining portions of the canal to be used for highway construction. Specify types of failures to be expected.

5. Prepare and submit proposal of additional field, laboratory and office work necessary to supplement available information in order to:

   (a) recommend necessary positive measures to assure adequate stability of structures mentioned under (3) during draw-down and highway construction;

   (b) recommend permanent measures at sites where temporary measures were applied;

   (c) recommend depth of soft material to be removed or treated at the canal bottom after draw-down and before fill placement;

   (d) recommend alternatives to prevent failure at locations listed under (4);

   (e) recommend measures to be taken if failures at locations listed under (4) are permitted to occur.

   (f) recommend the final height and shape of slopes which will assure adequate safety and require minimum maintenance.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF

cc: Mr. F. I. Newsom
Foundations Office
Gen Files

A. Steinac

Materials and Testing Division

Downsview, Ontario,
June 13, 1966.

Mr. G. Tustin,
St. Lawrence Seaway Authority,
5250 Ferrier Street,
Montreal 9, Quebec.

Dear George:

I mentioned that I would be sending to you the foundation reports we had in connection with our investigations along the present Welland Canal. These are being sent to you under separate cover. I understand that you already have the foundation report prepared by H. G. Acres at the Thorold site, and I am therefore not sending you a copy of it.

I have just received all of the reports you sent me on June 10, 1966. These reports will be of great help to us, and I would like to thank you very much for them. We will review all of your reports and then decide if any further field work should be done. I will pass on to you any further information we receive in connection with our investigation.

Yours sincerely,

aR

A. Rutka,
Materials & Testing Engineer.

AR/pa

Hwy. 406 in Old Welland Canal

Meeting at D.H.O., 8 June, 1966

Present:

J. Walter	A. Rutka	B.R. Davis
W. Bidell	A.G. Stermac	F.I. Hewson
	T. Kovich	

The numerous slope failures along the canal during and since its construction indicate that there will be many such failures when the canal is dewatered. Most of these can be cleared up easily during construction.

If the profile is a minimum of 10' above the canal bed (as seems probable), no deep failures should occur after construction. At some points, such as bridges, pipe crossings, etc. no failures can be tolerated at any time. Further soil testing and analysis are necessary before such critical areas can be determined.

The City's attitude towards using the Old Canal is still unknown.

The treatment required on the canal bottom cannot be determined until the close of navigation. This may be a problem.

There are some reservations about the material from the Seaway excavation. It may be too wet to place properly, but segregated stockpiles may make it suitable.

Once construction can be started, the project must be completed promptly, especially the crossings of the Old Canal.

The following decisions were made:

1. Mr. Bidell will sound out local feeling about the use of the Old Canal. He hopes to have this by September.

2. Mr. Stermac will attempt to predict the extent of slips which might affect roads, structures, services, etc. on the canal banks based on factual data available from previous investigations in this area carried out by various organizations. Supplementary drilling and testing will be carried out as required.

3. Mr. Rutka will check on methods of determining the stratigraphy under the canal itself.

4. Mr. Davis will investigate the cost of piling the pavement in case suitable fill cannot be arranged.

F.I. Hewson

F.I. Hewson,
Senior Bridge Liaison Engineer

FIH:rd

June 10, 1966

Mr. A. Stermac

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Mr. T. Kovich.

From: Mr. A. Rutka,

Date: June 2, 1966.

Our File Ref.

In reply to

Subject: Welland Canal

The Department is considering the possibility of placing Hwy. 406 in the present canal, after the new canal has been constructed. A meeting will be held on June 8, 1966, to,

1. Determine the stability of the present canal bank with and without water.

2. Arrive at the means of constructing a stable roadway.

It will not be possible to answer both of these questions in detail until a soils investigation has been completed.

I spoke to Mr. George Tustin of the St. Lawrence Seaway Authority, on June 1st, and he advises that the soil from Welland southerly consists primarily of soft clays, and Welland northerly, primarily silty sands. He envisages some slope problems when the water has been drawn down, and with all of the bridge piers. He will be sending me some cross sections and also a rough soils report, which he has prepared for his own purposes. I expect to have this within the next few days.

In any case, would you please arrange to be at the meeting on June 8th, where this subject will be discussed further.

A. Rutka

A. Rutka,
Materials & Testing Engineer.

AR/pa
c.c. G. Wrong.

Note to Tony: George Tustin would like to have all of the foundation reports connected with the canal. Would you please gather up all of the reports that we have so far with respect to the Carlton Street, Thorold and Welland Tunnels, and I will send them on to him. A.R.

(30L14-16)

30L14-16

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

30L14-16
GEOCRES No.

To: Mr. B. R. Davis
Bridge Engineer
Bridge Division

Attn: Mr. S. McCombie

Our File Ref.

From: Foundation Section
Materials & TestingDiv.
Room 107, Lab. Bldg.

Date: June 9, 1967

JUN - 9 1967

In reply to

Subject: Soil Conditions within the Welland Canal
Sta. 850 ≠ 00 to Sta. 1,100≠00

Attached please find the report for the above mentioned site prepared and submitted by the consultant H. Q. Golder & Associates Ltd.

We have reviewed the report and found the contents satisfactory and self-explanatory. It is believed that this information is adequate at this stage of the project. Should the presently proposed scheme be adopted, some additional exploratory work will have to be carried out as suggested in the attached report. However, the proposed work is very limited and it is believed that it will not change the present concepts but will rather provide the information that should ultimately be required.

Should you have any queries in connection with this report, please feel free to contact this office.

AGStermac

A. G. Stermac
Principal Foundation Engineer

AGS:mt
Attach.

cc: Messrs.: B.R.Davis (2)
H.A.Tregaskes
D.W.Farren
G.K.Hunter (2)
H.Greenland
W.S.Melinyshyn
T.J.Kovich
B.A.Singh
St.Lawrence Seaway Authority

Foundation Files (2)
General Files

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN
J. L. SEYCHUK

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
763-4103
767-9201

30 L14-16
GEOCRES No.

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS WITHIN CANAL

WELLAND CANAL

STA. 850+00 TO STA. 1,100+00

WELLAND ONTARIO

Distribution:

14 copies - Department of Highways, Ontario,
Toronto, Ontario.

3 copies - H.Q. Golder & Associates Ltd.,
Toronto, Ontario.

June, 1967 67028

TABLE OF CONTENTS

ABSTRACT

The results of an investigation to determine the subsoil conditions within the Welland Canal in the vicinity of the City of Welland, Ontario (between stations 850+00 and 1,100+00) is reported. The need for this investigation was discussed in our report 66093, dated January, 1967. This previous report presented the results of a study on the general feasibility of using the drained excavation of the section of the existing Welland Canal between Ramey's Bend and Bridge 12, now to be abandoned, as a route for Highway 406.

The water level in the canal along this reach is at about elevation 569 with the depth of water being about 30 to 33 feet; the top of the canal banks vary from elevation 580 to 600. In the northern section (north of about STA. 920+00) a stiff clayey silt up to 26 feet thick is encountered; this deposit is underlain in turn by layered silty clay (at some locations only) followed by a dense silty sand till. South of about STA. 930+00 the canal bottom is underlain by a firm to stiff interbedded silty clay up to 40 feet in thickness; in the southern portion of this section the silty clay is underlain by silt. Directly underlying the silty clay and/or the silt is the dense silty sand till encountered in the northern portion of the reach.

This investigation in general, confirms the stratigraphy along the west canal banks inferred from previous borings carried out by the St. Lawrence Seaway Authority, with the exception of the section between STA. 890+00 and 960+00. In this section the till surface is higher along the canal banks than beneath the canal, i.e. the subsoil seems to be dipping in an easterly direction.

INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario, (letter of authorization dated March 2, 1967) to carry out a soil investigation within the Welland Canal in the vicinity of the City of Welland, Ontario, namely between STA. 850+00 and STA. 1,100+00.

Our report No. 66093, dated January 1967, presented the results of a study on the general feasibility of using the drained excavation of the section of the existing Welland Canal between Ramey's Bend and Bridge 12, now to be abandoned as a route for Highway 406. In this report it was recommended that borings be put down within the canal section specified above. The purpose of this present investigation is twofold;

i) to determine the character of the canal bottom which, for example, if covered by soft or organic debris, may have a serious effect on the placement of fill.

and ii) to confirm the vertical and lateral extent of the overburden deposits inferred on Figure 2 in report No. 66093.

This report should be read in conjunction with the previous report.

PROCEDURE

The field work for this investigation was carried out between March 6 and 30, 1967. A total of 7 cased boreholes (No. C-1 to C-7) were put down in NX size, on about 5,000 foot centres, by 2 drum raft mounted diamond drill rigs. The borings, each of which was accompanied by a dynamic cone penetration test, were put down to depths ranging from 28 to 45 feet below canal bottom. In addition 20 dynamic cone penetration tests (No. C-8 to C-27) were put down between the borings on about 1,000 foot centres. The drilling equipment, including drillrigs, drum rafts, motor boats, anchors and steel cable, etc., were supplied and operated by the F.E. Johnston Drilling Company Limited.

The overburden was sampled with either a two inch diameter split spoon sampler or thin walled tube sampler. In situ field vane testing was also carried out within the cohesive strata. The field work was supervised throughout by an engineer from our staff.

A detailed log for each of the boreholes and dynamic cone penetration tests is given on the Record of Borehole sheets following the text of this report. The location of the borings and penetration tests, together with the inferred stratigraphy at borehole locations, are shown on Figures 1 and 2.

Samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of the laboratory testing are shown on the Record of Borehole sheets and on Figures 3 to 9, inclusive.

The borehole and penetration test locations were determined by our engineering personnel by reference to canal chainages marked on various concrete protective slabs and bridge structures flanking the canal. The elevation of the canal water level at the time of the investigation was provided by the St. Lawrence Seaway Authority. These elevations are referred to the same Geodetic datum as used in the previous investigation (report No. 66093). It is understood that a revised Geodetic datum will be introduced by the St. Lawrence Seaway Authority in the near future. Precautions, therefore, should be taken to ensure that the revised datum is not confused with the datum used in this report.

i) Conditions During Drilling Operations

At the beginning of the investigation ice sheets were located within the portion of the Welland Canal under investigation; the ice conditions were particularly severe in the northerly portion. Removal of ice was therefore required to position the rafts on the predetermined boring locations. Once the rafts were in place an effort was made to secure them with anchors placed within the canal bottom deposits. The flowing ice, however, made it impossible to steady the raft in this manner during the casing and sampling operations. Additional fixity was provided by securing the raft with steel cables running from the corners of the raft to shore; this anchorage proved satisfactory. During the first part of the week of March 13 the temperature dropped abruptly causing additional quantities of ice to be formed. This ice greatly hindered the drilling operations so much that it was necessary to terminate sampling operation on March 15 and again on March 18.

Following the above period the weather turned milder and the ice disappeared from the portion of the canal under investigation. The drilling programme was then completed.

SITE AND GEOLOGY

The portion of the Welland Canal under investigation is between Stations 850+00 to 1,100+00; this portion of the canal is located within the townships of Crowland and Thorold, County of Welland.

The city of Welland is located approximately in the centre of the section. The canal is cut in a broad, relatively flat clay plain, which varies in surface elevation from about elevation 580 to 600. The bottom of the canal along this reach was found to be at about elevation 536, i.e. the canal banks are approximately 44 to 64 feet high. The width of the canal at the crest and at canal bottom is about 350 ft. and 200 ft. respectively. The water level in the canal is about elevation 569 (some 31 to 34 ft. of water). The Welland River, which flows from west to east adjacent to the canal, is at a normal water level of about elevation 562 with the high water level being about 569.

From available geological information (Chapman, Putman, 1951) and inspection of the area, it is known that the overburden consists of thick deposits of silty clay and clayey silt overlying till, physiographically known as Haldiman Till.

The till which is of Wisconsin age, was covered by the silty clay, which is a lacustrine deposit laid down in glacial Lake Warren. The glacial lake phase was possibly interrupted by two major retreats of the ice front which resulted in distinctively different deposits, non-stratified relatively silty homogeneous deposits laid down with the ice front fairly close, and heavily stratified very clayey deposits laid down when the ice front had retreated some distance. All of the lacustrine deposits are relatively soft and possibly only lightly preconsolidated, except the upper 30 to 40 feet of the overburden outside the canal proper which is desiccated. The total thickness of the overburden generally varies from 100 to 120 feet.

The broad clay plain is bounded to the north by the Niagara Escarpment which steps down toward Lake Ontario. To the south the plain is bounded by the toe of the Onondaga cuesta.

The water shed in the area is controlled by the Onondaga cuesta, which though quite low lying close to the shore of Lake Erie, nevertheless forces the drainage to the north and east. In general the drainage in this primarily flat heavy clay area is quite poor.

The bedrock in the area is Palaeozoic. The beds dip slightly southward under Lake Erie. The bedrock, which generally varies in elevation from 460 to 500, is a massive dolomitic limestone of the Salina formation, Devonian period. There are numerous siltstone and calcareous shale interbeds within the bedrock. In addition the bedrock contains numerous gypsum inclusions from hairline thickness to as much as 12 inches.

SOIL CONDITIONS

The detailed soil stratigraphy encountered by the borings is given on the Record of Borehole sheets. The inferred stratigraphy along the portion of the canal investigated is also shown on Figure 1 (STA. 850+00 to 970+00) and on Figure 2 (STA. 970+00 to 1,100+00). The engineering properties of the subsoil are presented on Figures 3 to 9 inclusive.

The subsoil conditions encountered in this investigation can be sub-divided into two distinct zones, namely the northern section where the upper deposit is a clayey silt till and the southern section where the upper deposit is an interbedded silty clay. The two sections will be discussed separately below.

i) <u>Northern Section</u> (approximately STA.850+00 to 920+00) (boring coverage BH's C-5 and C-6, Pen Test C-22 to C-27)

a) Clayey Silt Till

Directly underlying the canal bottom in this section is a deposit of relatively homogeneous reddish-brown clayey silt till with some sand and gravel throughout,the upper 1 or 2 feet of which are in a softened condition. The maximum encountered thickness of the deposit was 26 feet (BH C-6). Occasional sand and silt partings and seams up to 1/2 inch thick occur randomly throughout the deposit. Typical grading curves for the clayey silt till are shown on Figure 3. The total unit weight of the till, as determined from two laboratory tests, was found to be about 138 lb/cu. ft.

Atterberg limit tests were carried out on samples of the till. These results are shown on the Record of Borehole sheets and are summarized on Figure 8. The test results indicate that the liquid limit varies from 17 to 27 while the plasticity indices vary from 6 to 12; the corresponding natural water content ranges from about 2 percent below to 3 percent above the plastic limit. These results are typical of inorganic glacial clays and silts of low plasticity. Based on the laboratory testing the activity of the deposit was found to range from approximately 0.3 to 0.6 being generally about 0.4. Activities of this order of magnitude are indicative of inactive soils.

Standard cone penetration tests were carried out within the deposit; the results are summarized on the Record of Borehole sheets. The results indicate that the 'N' values range from about 10 blows/ft. to 33 blows/ft. being generally about 20 blows/ft. Based on the 'N' values it is estimated that the consistency of the deposit varies from stiff to very stiff. Two undrained triaxial tests carried out on samples of the till corroborate the range in consistency given above.

Underlying the clayey silt till at borehole C-5 is a deposit of stiff reddish-brown to grey layered silty clay with some sand and a trace of gravel; the thickness of this deposit is about 10 feet. A grading curve for a representative sample of the deposit is shown on Figure 5. The results of an Atterberg limit determination indicates that the silty clay is inorganic and of medium plasticity with the natural water content at about the liquid limit.

b) Sandy Silt Till

Underlying the layered silty clay or the clayey silt till, is a dense to very dense sandy silt till with some gravel and a trace of clay throughout. Both borings were terminated within this deposit. From previous experience in the area it is known that boulders occur within this deposit particularly with depth. Localized seams and layers of silt and sand up to 1 foot in thickness occur throughout. Grading curves for representative samples of the till (obtained using 1½ inch I.D. sampling equipment) are given on Figure 7. The in situ water content of the till deposit was found to be about 12 percent.

ii) Southern Section (approximately STA. 930+00 to 1,100+00) (boring coverage BH's C-1, C-2, C-3, C-4, C-7 and Pen. Tests C-8 to C-21)

a) Interbedded Silty Clay

Directly underlying the canal bottom in this section is a stratum of reddish-brown to grey interbedded silty clay and/or stratified clay, the upper 1 to 2 feet of which is in a softened condition. The thickness of this stratum varies from 22 feet at borehole C-2 to 42 feet at borehole C-3. The lower portion of the stratum,at some of the boring locations, was noted to be varved. The thickness of individual layers within the stratified clay varied widely to a maximum of 3 inches approximately. Occasional partings and seams of silt and sand up to 1/2 inch thick are present throughout the stratum. Grading analyses for representative samples of the silty clay are shown on Figures 4 and 5.

Atterberg limit tests were carried out on samples of the interbedded silty clay. The results are shown on the Record of Borehole sheets and are also summarized on Figure 8. The test results are summarized in tabular form below:

Liquid Limit (W_L) Range (Average)	Plasticity Index (I_P) Range (Average)	Liquidity Index (I_L) Range (Average)
24 to 68 (44)	7 to 44 (24)	0.3 to 1.3 (0.7)

These results, which are typical of inorganic glacial clays, indicate that the silty clay stratum ranges from low to high plasticity generally being within the medium plastic range. The activity of the silty clay stratum varies from about 0.3 to 0.6 being typically about 0.35. Values of this order of magnitude are representative of inactive silty clays. The total unit weight of the silty clay was found to vary from about 115 lb/cu.ft. to 130 lb/cu.ft. being typically about 120 lb/cu.ft.

The undrained shear strength of the silty clay was measured by in situ vane testing in the field and by laboratory undrained triaxial tests. The results of these tests are plotted on the Record of Borehole sheets and are summarized on Figure 9 as a plot of shear strength vs. elevation. The undrained shear strength in the silty clay varies from about 700 lb/cu.ft. near the surface to 1,500 lb/cu.ft. with depth. In general the triaxial tests gave lower shear strength values than the in situ field vane tests. This is probably due to unavoidable disturbance during the sampling operations. The relatively high strains to failure during testing (greater than 15 percent) were a definite indication that some of the samples were disturbed to some degree. It is, therefore, concluded that the in situ vane test results provide a better indication of the in situ consistency of the stratum. Based on the above it is estimated that the consistency of the silty clay varies from firm to stiff.

The sensitivity of the silty clay, as measured by several field vane tests, is about 2 to 4; the stratum is thus moderately sensitive to disturbance.

The previous investigations in the vicinity indicated that the silty clay stratum is normally consolidated to lightly overconsolidated below about elevation 535 (canal bottom).

The laboratory test results indicate that the average plasticity index of the stratum is about 24; for this value the C_u/P_o ratio is about 0.2 (Reference Skempton, 1957). Based on the undrained shear strength profile shown on Figure 9 and a C_u/P_o of 0.2, it is estimated that the silty clay stratum (below canal bottom) could be preconsolidated in excess of existing overburden pressure by as much as 1 ton/sq.ft.

b) Silt

In the southern end of the portion of the canal under investigation (approximately south of STA. 1020+00) the silty clay stratum is underlain by a deposit of loose to compact reddish-brown silt with some sand and a trace of clay and gravel. The encountered thickness of this deposit at borehole C-1 and C-2 was 10 feet and 13 feet, respectively. Occasional layers of clayey silt up to 1/2 inch thick are located randomly throughout the deposit. Grading curves for two samples of the silt are shown on Figure 6.

c) Sandy Silt Till

Underlying the interbedded silty clay and/or the silt deposit is a dense to very dense reddish brown to grey brown silty sand and gravel till. This till sheet is the same deposit encountered in the northern portion of the canal section and as such is the only deposit that is continuous along the section investigated. The physical description and engineering properties of the deposit were discussed in detail above.

COMMENTS ON SOIL CONDITIONS

The borings put down did not encounter any undesirable surficial canal bottom deposits, such as deposits of organic origin. The upper 1 or 2 feet, although composed of the natural subsoil, is in a softened condition. It is considered however, that sub-excavation will not be required prior to placement of fill on the canal bottom.

In general the borings corroborate the stratigraphy inferred from the St. Lawrence Seaway Authority borings previously put down along the west bank of the canal. There is, however, one area where a significant deviation occurs as discussed below.

In report No. 66093 the section between STA. 890+00 and 960+00 was designated as section II; it was in this section that some discrepancies were noted. The present investigation indicated that the upper interbedded silty clay extended as far north as about STA. 930+00. The borings put down previously on the west canal bank indicate that the silty clay pinches out somewhere in the vicinity of STA. 960+00; it is encountered again, however, north of about STA. 930+00 (refer to Figure 2, report 66093). There is also a variation in the vertical extent of the deposits as encountered in the canal banks and beneath the canal. The boundary between the surficial deposits and the basal granular till occurs at a higher elevation on the canal banks. This would seem to indicate that the subsoil, particularly the till, dips steeply from the west to the east over the major portion of this section. It should be noted that the consistency and/or relative density of the deposits are within about the same range as presented in the previous report.

IMPLICATION OF PRESENT INVESTIGATION

As discussed in report 66093 the area of potential major instability of the canal banks, following drawdown, will be within that portion of the canal located in the downtown part of the city of Welland (STA. 967+00 to 1,050+00). This area is located within section III where the interbedded silty clay is present. The vertical and lateral extent as well as the engineering properties of this stratum previously assumed were corroborated by this investigation.

The area north of this section, namely sections I and II are not as critical with respect to stability since,

(a) the subsoil is more competent;
(b) no important structures are located immediately adjacent to the canal banks.

This investigation, however, pointed out the fact that some variation in the subsoil conditions occur in section II (STA. 890+00 to 960+00). In this section the till beneath the canal was encountered at a lower elevation than previously assumed. The cohesive surficial deposits beneath the canal, although not as competent as the till, do have an average consistency in the stiff range. Based on this consistency and taking into account point (b) above it is considered that the conclusions drawn in the previous report still apply, namely that the stability of the canal banks in this section following drawdown should not markedly be affected.

In summary it is considered that this investigation generally confirms the conclusions presented in the previous report regarding the feasibility of placing a highway within the confines of the drained canal. If the scheme is adopted it would be advisable to put down 2 additional borings between STA. 890+00 and 960+00 to further confirm the variation in the till elevation along this reach. This could be carried out as part of the additional boring programme outlined in section (iv) of the "Conclusions and Recommendation" section of report 66093.

for. B.T. Darch, P. Eng.

V. Milligan, P. Eng.

BTD:VM:je
67028
June, 1967.

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CHAPMAN, L.J. and PUTNAM D.F., 1951 - The Physiography of Southern Ontario. University of Toronto Press, Toronto.

GOLDER, H.Q. & ASSOCIATES LTD., 1967 - Report - "Feasibility Study, Portion of Proposed Highway 406 within Welland Canal". Department of Highway, Ontario - Report No. 66093, January, 1967. Unpublished.

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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPES

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

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, N: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, blows/ft.*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	c_u, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer[1]
Q	undrained triaxial[2]
R	consolidated undrained triaxial[2]
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$

e = base of natural logarithms 2.7183

$\log_e a$ or $\ln a$, natural logarithm of a

$\log_{10} a$ or $\log a$, logarithm of a to base 10

- t time
- g acceleration due to gravity
- V volume
- W weight
- M moment
- F factor of safety

II. STRESS AND STRAIN

- u pore pressure
- σ normal stress
- σ' normal effective stress ($\bar{\sigma}$ is also used)
- τ shear stress
- ϵ linear strain
- ϵ_{xy} shear strain
- ν Poisson's ratio (μ is also used)
- E modulus of linear deformation (Young's modulus)
- G modulus of shear deformation
- K modulus of compressibility
- η coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

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

(b) *Consistency*

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

(c) *Permeability*

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

(d) *Consolidation (one-dimensional)*

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

(e) *Shear strength*

- τ_f shear strength
- c' effective cohesion intercept; ϕ' effective angle of shearing resistance, or friction — in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$
- c_u apparent cohesion*; ϕ_u apparent angle of shearing resistance, or friction — in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
- μ coefficient of friction
- S_t sensitivity

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

PROJECT No. 67026

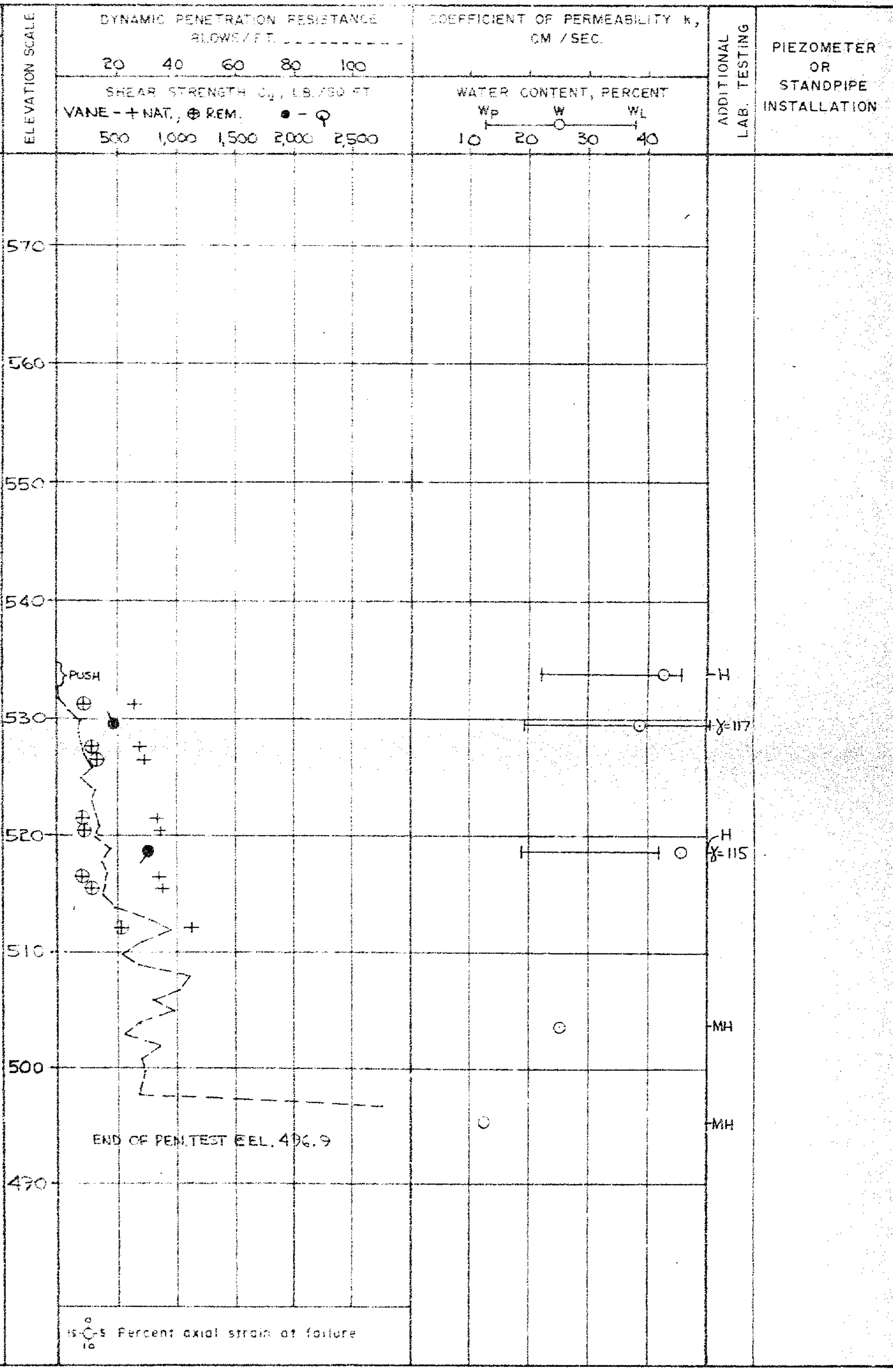
RECORD OF BOREHOLE C-1

LOCATION: STA. 1099+60 – ℄ CANAL, See Figure 1 | BORING DATE: MARCH 10,11, 1967 | DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING | BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES | PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.
569.5 / 0.0	CANAL LEVEL			
534.9	CANAL BOTTOM			
534.6	FIRM TO STIFF REDDISH - BROWN INTERBEDDED **SILTY CLAY**. TRACE OF SAND AND GRAVEL. VARVED BELOW ABOUT EL. 514. (UPPER 1 TO 2 FEET IN A SOFTENED CONDITION)	1	2" D.O.	1
		2	2" T.O.	Pm
		3	2" T.O.	Pm
		4	2" T.O.	Pm
		5	2" T.O.	Pm
507.9		6	2" D.O.	8
507.9 / 61.6	LOOSE REDDISH - BROWN **SILT** WITH SOME SAND AND A TRACE OF GRAVEL.	7	2" D.O.	4
498.3		8	2" D.O.	7
498.3 / 71.2	VERY DENSE GREY-BROWN **SILTY SAND AND GRAVEL** WITH A TRACE TO SOME CLAY (TILL)	9	2" D.O.	52
494.9 / 74.6	END OF HOLE			



15-○-5 Percent axial strain at failure
10

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED R.T.D.

PROJECT No. 67028

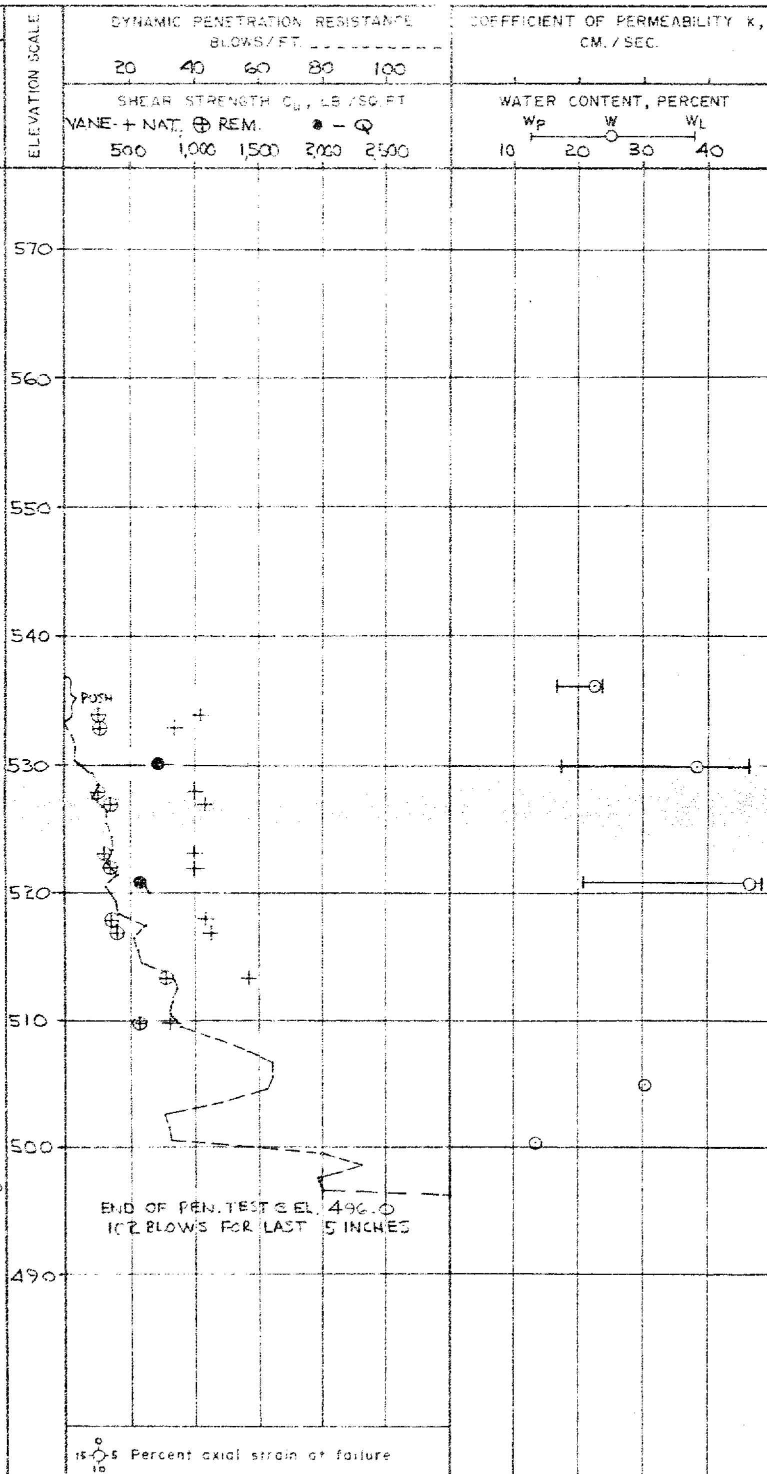
RECORD OF BOREHOLE C-2

LOCATION: STA. 1052+50 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 13-14, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING — BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	BLOWS / FT.	ADDITIONAL LAB. TESTING
568.4	CANAL LEVEL				
0.0					
536.9	CANAL BOTTOM				
31.5	FIRM TO STIFF REDDISH-BROWN INTER-BEDDED SILTY CLAY TRACE OF SAND AND GRAVEL (UPPER 2 FEET IN A SOFTENED CONDITION)	1	2" D.O.	4	MH
		2	2" T.O.	Fm	γ=118
		3	2" T.O.	Pm	
		4	2" T.O.	Pm	γ=117
514.9		5	2" T.O.	Fm	
53.5	COMPACT REDDISH-BROWN SILT TO SILT AND SAND WITH A TRACE OF CLAY AND GRAVEL	6	2" D.O.	8	
		7	2" D.O.	19	H
502.0					
66.4	DENSE TO VERY DENSE REDDISH-BROWN SILTY SAND WITH SOME GRAVEL AND A TRACE OF CLAY (TILL)	8	2" D.O.	38	MH
496.0		9	2" D.O.	>100	
72.4	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

PROJECT No. [illegible]

RECORD OF BOREHOLE C-3

LOCATION: STA. 1000+00 - ℄ CANAL, See Figure 1

BORING DATE: MARCH 16-17, 1967

DATUM: GEODETIC

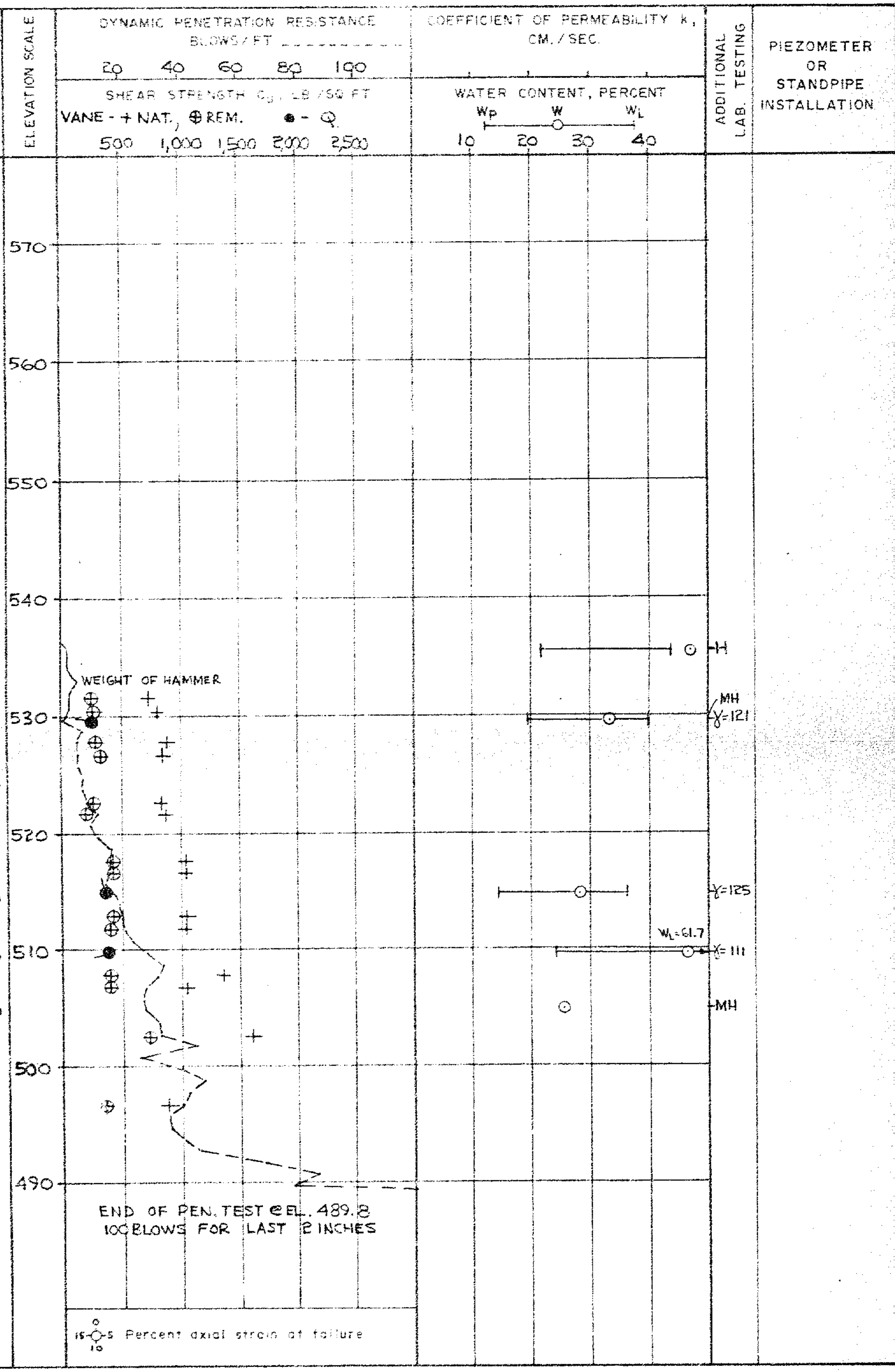
BOREHOLE TYPE: WASH BORING

BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	BLOWS / FT.
568.9	CANAL LEVEL			
0.0				
536.4 / 32.5	CANAL BOTTOM	1	2" D.O.	1
	FIRM TO STIFF REDDISH - BROWN INTERBEDDED SILTY CLAY WITH A TRACE OF SAND AND GRAVEL. SEAMS OF SILT UP TO 3 INCHES THICK BELOW ABOUT EL. 505. (UPPER 2 FEET IN A SOFTENED CONDITION)	2	2" T.O.	Pm
		3	2" T.O.	Pm
		4	2" T.O.	Pm
		5	2" T.O.	Pm
		6	2" T.O.	Pm
		7	2" T.O.	Pm
		8	2" T.O.	Pm
		9	2" D.O.	4
494.9 / 74.0	COMPACT TO VERY DENSE REDDISH - BROWN SILTY SAND AND GRAVEL (TILL)	10	2" D.O.	11
491.5 / 77.4	END OF HOLE			



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

PROJECT No. [illegible]

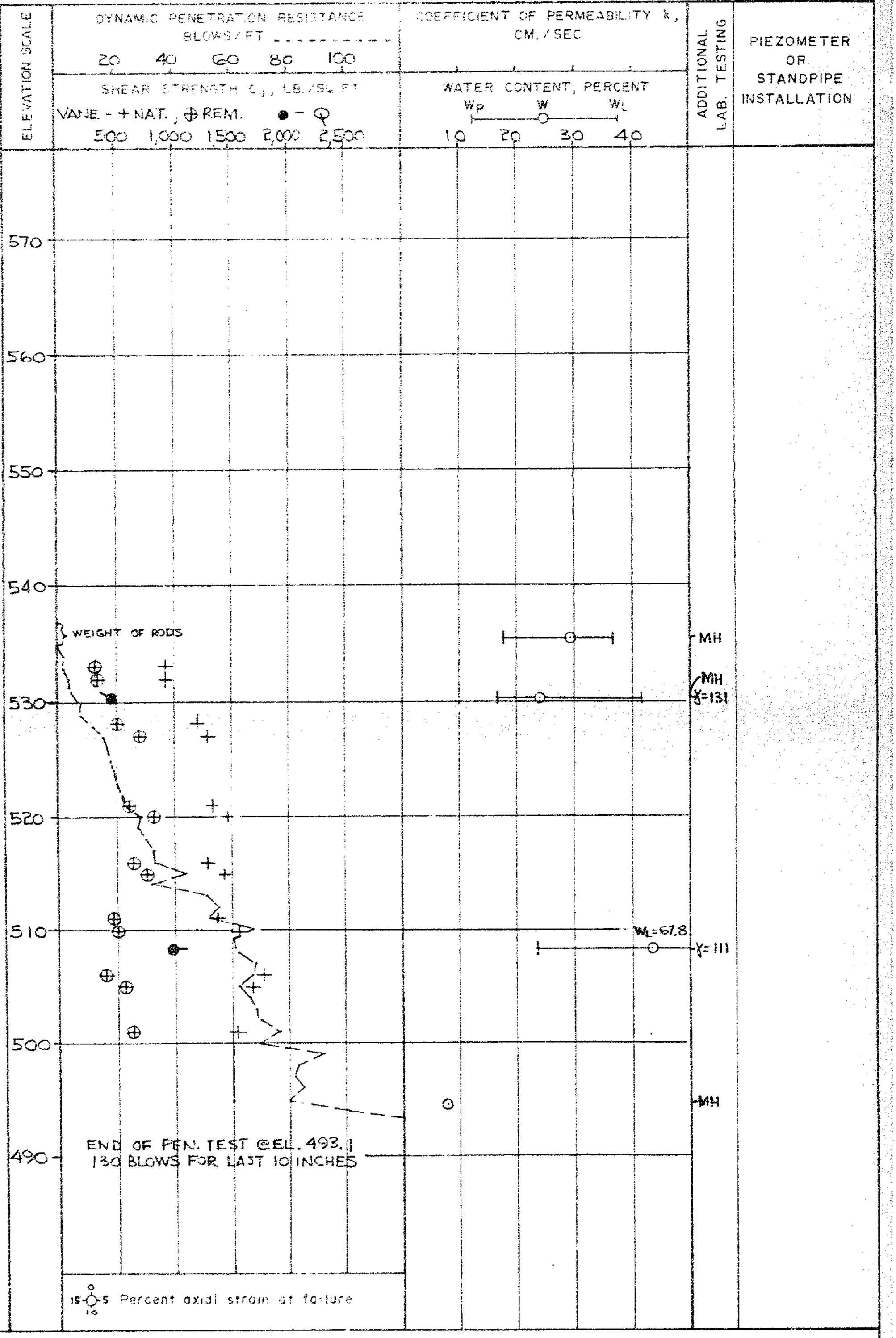
RECORD OF BOREHOLE C-4

LOCATION STA. 950+50 - ℄ CANAL See Figure 1 BORING DATE MARCH 16-17, 1967 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.
568.9	CANAL LEVEL			
0.0				
536.8	CANAL BOTTOM			
32.1	FIRM TO STIFF REDDISH-BROWN & GREY INTERBEDDED SILTY CLAY TRACE TO SOME SAND AND GRAVEL. OCCASIONAL THIN SEAMS OF FINE SAND THROUGHOUT. VARVED BELOW ABOUT EL 504 (UPPER 2 FEET IN SOFTENED CONDITION)	1	2" D.O.	Pm
		2	2" T.O.	Pm
		3	2" T.O.	Pm
		4	2" T.O.	Pm
		5	2" T.O.	Pm
		6	2" T.O.	Pm
		7	2" T.O.	Pm
498.5 / 70.4	COMPACT TO VERY DENSE REDDISH-BROWN SILTY SAND WITH SOME GRAVEL AND CLAY (TILL)	8	2" D.O.	19
		9	2" D.O.	98
493.9 / 75.0	END OF HOLE			



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED E.D.

PROJECT NO. [illegible]

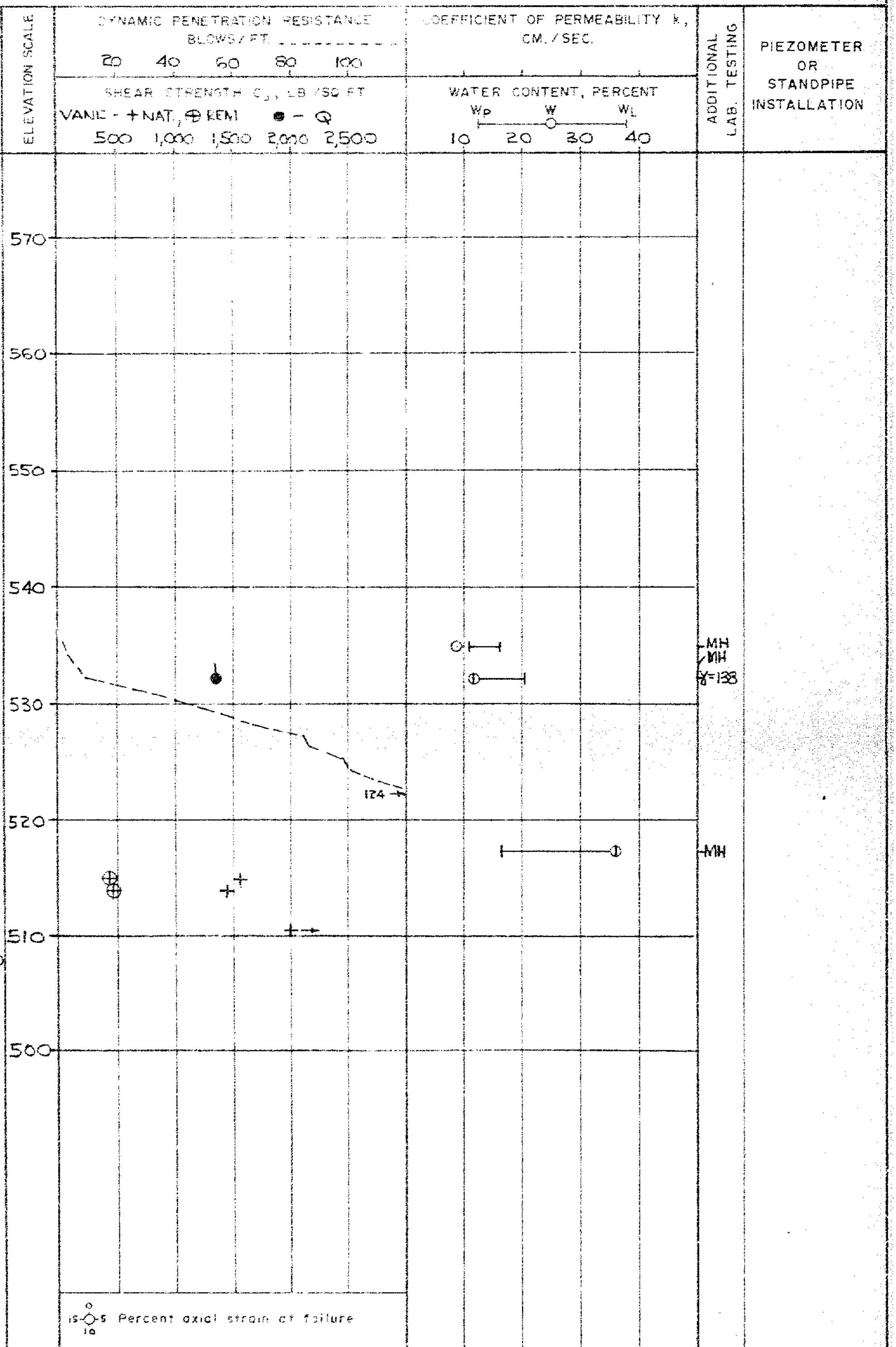
RECORD OF BOREHOLE C-5

LOCATION: STA. 879+50 - ℄ CANAL, See Figure 1 BORING DATE: MARCH 13-14, 1967 DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FT
565.4 / 0.0	CANAL LEVEL				
535.9	CANAL BOTTOM				
37.5	STIFF TO VERY STIFF REDDISH - BROWN **CLAYEY SILT** WITH SOME SAND AND GRAVEL (TILL)		1	2" D.O.	12
			2	2" T.O.	15
			3	2" D.O.	19
			4	2" D.O.	12
520.3 / 48.1	STIFF REDDISH - BROWN TO GREY LAYERED **SILTY CLAY** WITH SOME SAND AND A TRACE OF GRAVEL		5	2" D.O.	9
			6	2" D.O.	Pm
510.3 / 58.1	VERY DENSE GREY **SANDY SILT** WITH SOME CLAY AND GRAVEL (TILL)		7	2" D.O.	>100
507.6 / 60.8	END OF HOLE				



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED [illegible]

PROJECT No. 67028

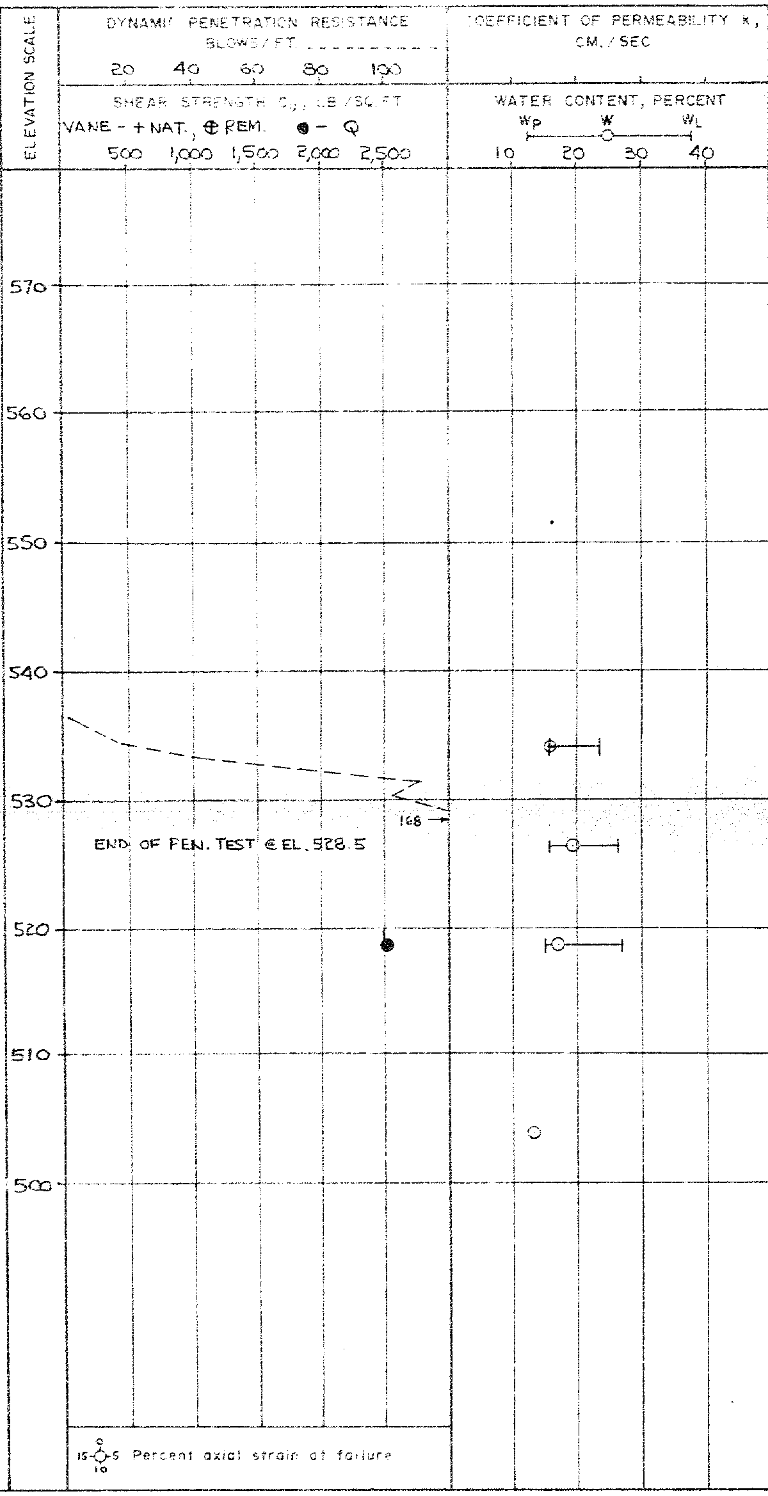
RECORD OF BOREHOLE C-6

LOCATION: STA. 850+50 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 10-11, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING — BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	ADDITIONAL LAB. TESTING
569.5	CANAL LEVEL				
0.0					
537.3	CANAL BOTTOM				
32.2	STIFF TO VERY STIFF REDDISH-BROWN CLAYEY SILT WITH SOME SAND AND GRAVEL (TILL)	1	2" D.O.	33	MH
		2	2" D.O.	28	
		3	2" D.O.	29	MH
		4	2" D.O.	21	
		5	2" T.O.	25	γ=138
		6	2" D.O.	10	
511.4 / 58.1	COMPACT TO VERY DENSE GREY SILTY SAND AND GRAVEL TRACE OF CLAY (TILL)	7	2" D.O.	24	
		8	2" D.O.	49	MH
503.0 / 66.5	END OF HOLE				



PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED R.T.D.

PROJECT No. [illegible]

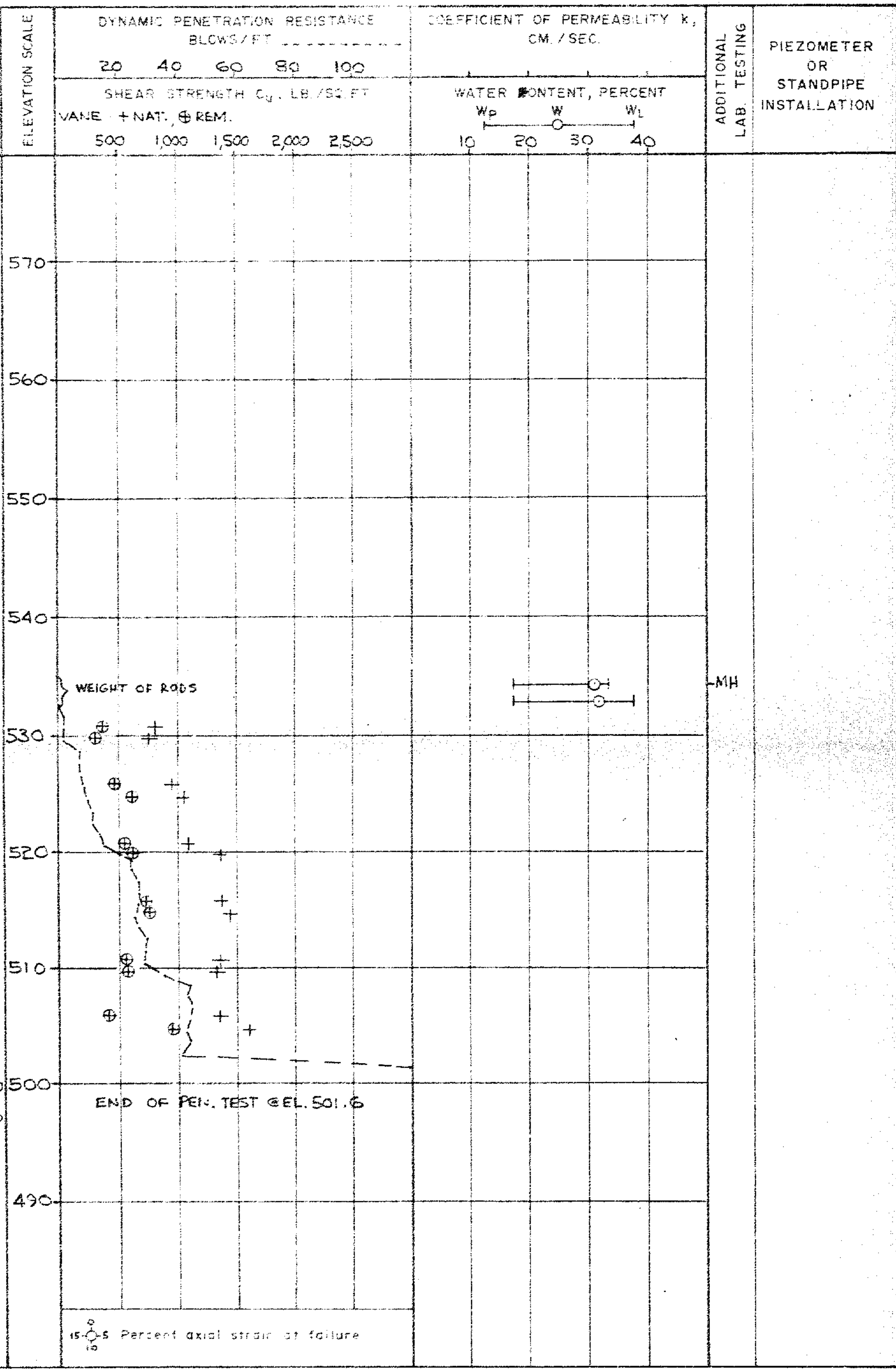
RECORD OF BOREHOLE C-7

LOCATION: STA. 975+00 - ℄ CANAL, See Figure 1 — BORING DATE: MARCH 29, 1967 — DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING — BOREHOLE DIAMETER: NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FT.
569.6	CANAL LEVEL			
0.0				
535.1	CANAL BOTTOM			
34.5	FIRM TO STIFF REDDISH-BROWN (GREY LAYERING BELOW ABOUT EL. 509) INTERBEDDED SILTY CLAY, WITH A TRACE OF SAND & GRAVEL (UPPER 3 FEET IN SOFTENED CONDITION)	1	2" T.O.	Pm
		2	2" T.O.	Pm
		3	2" D.O.	Pm
		4	2" T.O.	Pm
		5	2" D.O.	Pm
		6	2" T.O.	Pm
		7	2" D.O.	Pm
502.6		8	2" T.O.	Pm
67.0	VERY DENSE GREY SILTY SAND WITH SOME GRAVEL AND A TRACE OF CLAY (TILL)	9	2" D.O.	>100
497.1		10	2" D.O.	>100
72.5	END OF HOLE			



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.D.

PROJECT No. [illegible]

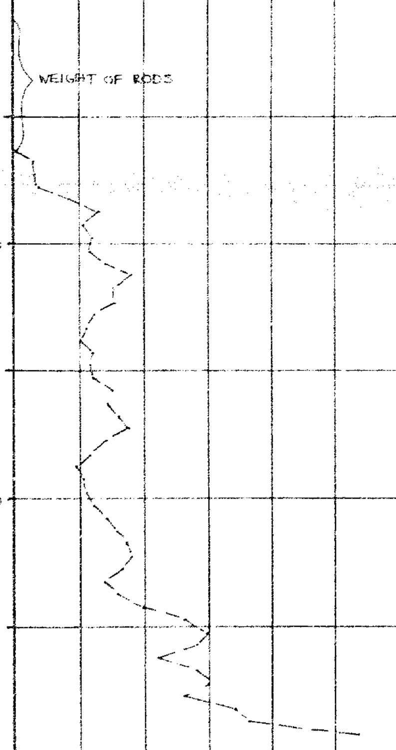
RECORD OF ~~BOREHOLE~~ PEN. TEST C- 8

LOCATION STA. 1090+00 - ℄ CANAL See Figure BORING DATE MARCH 28 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20 40 60 80 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_p W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.					
565.5	CANAL LEVEL					570				
0.0						560				
						550				
537.5	CANAL BOTTOM					540				
32.0	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 505)					530, 520, 510, 500, 490				
80	PROBABLY SILTY SAND AND GRAVEL (TILL)									
481.5										
88.0	END OF PEN. TEST					480				



15 –○– 5 Percent axial strain at failure (10 below)

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED K.T.D.

PROJECT NO.

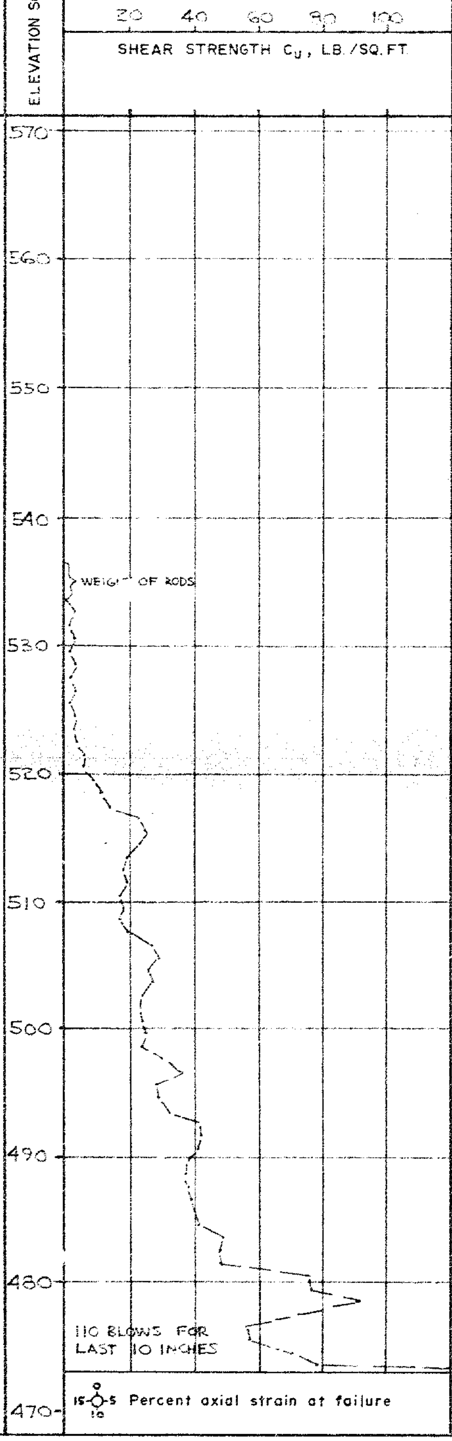
RECORD OF ~~BOREHOLE~~ PEN. TEST C-9

LOCATION: STA. 1080+00 - ℄ CANAL See Figure 1 | BORING DATE: MARCH 28, 1967 | DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST | BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES | PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	SOIL PROFILE – DESCRIPTION	STRAT. PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – BLOWS / FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P, W, W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.5	CANAL LEVEL					570				
0.0						560				
						550				
						540				
536.5	CANAL BOTTOM									
33.0	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 505)					530, 520, 510, 500, 490				
80	PROBABLY SILTY SAND AND GRAVEL (TILL)					480				
472.7 / 96.8	END OF PEN. TEST					470				



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED R.T.D.

PROJECT No. [illegible]

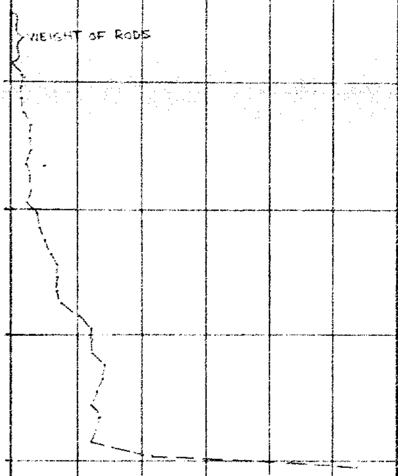
RECORD OF ~~BOREHOLE~~ PEN. TEST C-10

LOCATION STA. 1070+00 - ℄ CANAL See Figure 1 — BORING DATE MARCH 27, 1967 — DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST — BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20 40 60 80 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P — W — W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.					
569.5	CANAL LEVEL					570				
0.0						560				
						550				
						540				
535.5	CANAL BOTTOM									
34.0	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 513)					530				
						520				
						510				
499.5						500				
70.0	END OF PEN. TEST									
	PROBABLY SILTY SAND AND GRAVEL (TILL)					490				



15 ◇ 5 / 10 / 0 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED [illegible]

PROJECT No. 67060

RECORD OF ~~BOREHOLE~~ PEN. TEST C-11

LOCATION STA 1060+00 - ℄ CANAL See Figure 1

BORING DATE MARCH 27, 1967

DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE
ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT.	
569.5	CANAL LEVEL					570
0.0						560
						550
537.3	CANAL BOTTOM					540
32.2	PROBABLY INTERBEDDED SILTY CLAY					530
518.5						520
517.0						
52.5	END OF PEN. TEST					
	PROBABLY SILTY SAND AND GRAVEL (TILL)					510

DYNAMIC PENETRATION RESISTANCE BLOWS / FT.

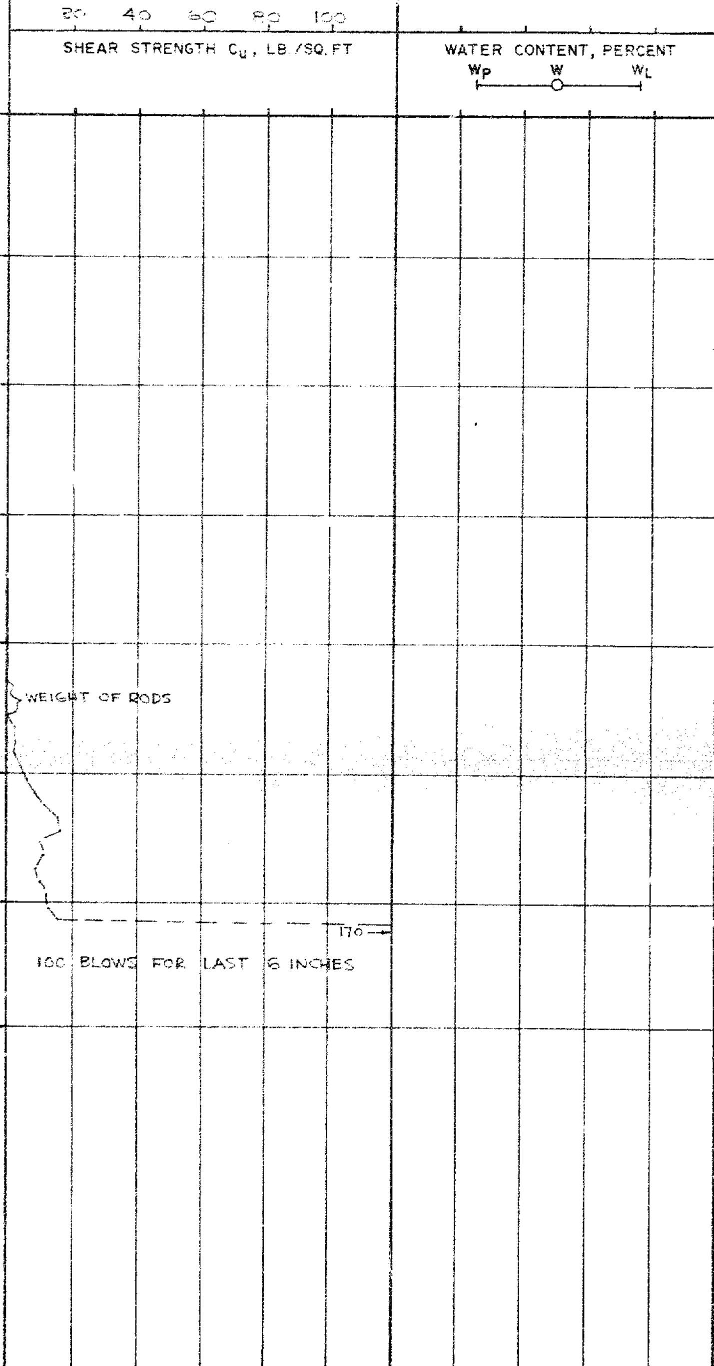
SHEAR STRENGTH C_u, LB./SQ.FT

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



15 5 10 Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED

PROJECT No. 67068

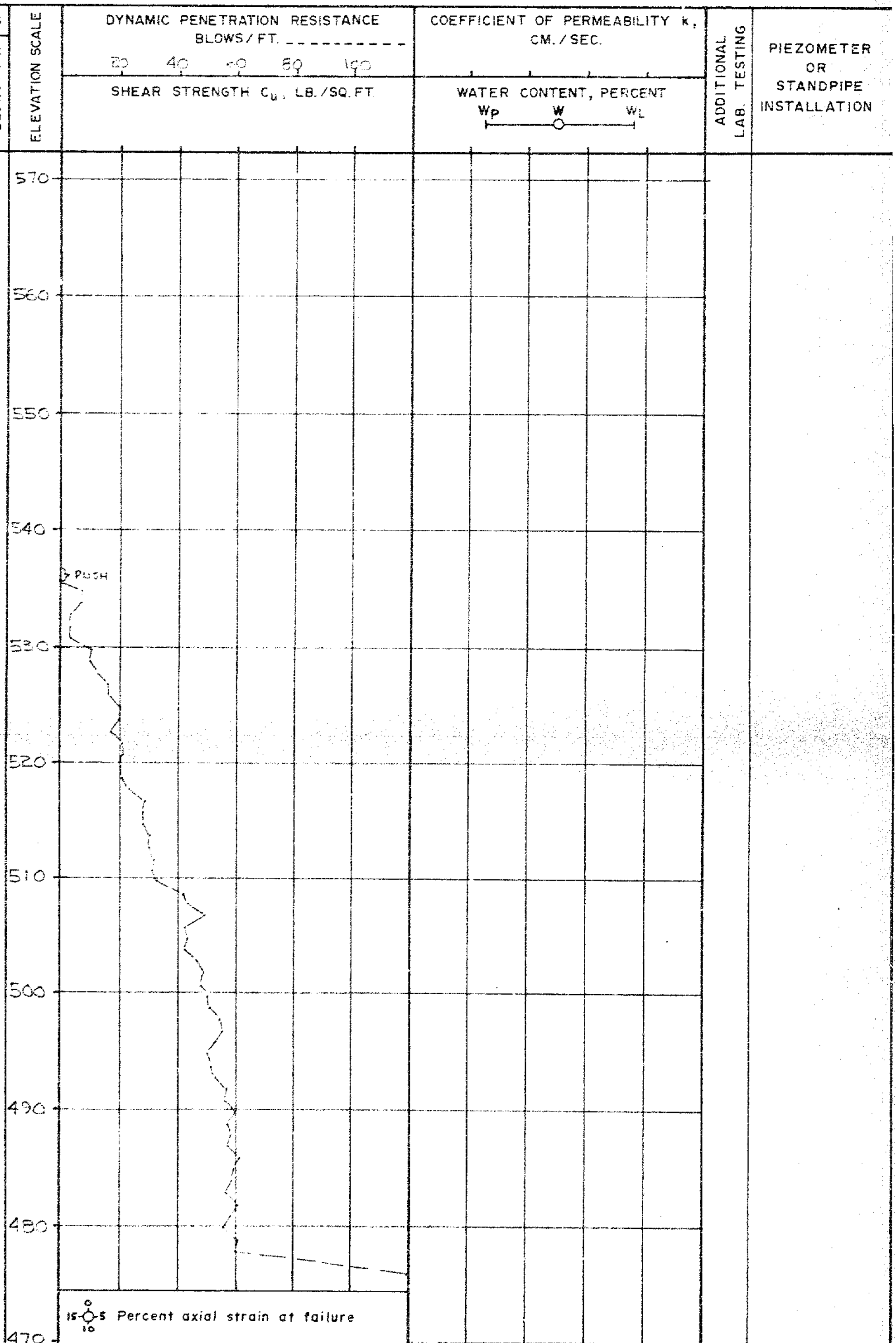
RECORD OF ~~BOREHOLE~~ PEN. TEST C-12

LOCATION: STA. 1040+50 ℄ CANAL, See Figure 1
BORING DATE: MARCH 23, 1967
DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST
BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT.
569.5	CANAL LEVEL				
0.0					
536.5	CANAL BOTTOM				
33.0	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 500)				
85	PROBABLY SILTY SAND AND GRAVEL (TILL)				
475.5					
94.0	END OF PEN. TEST				



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED E.T.D.

PROJECT No. [illegible]

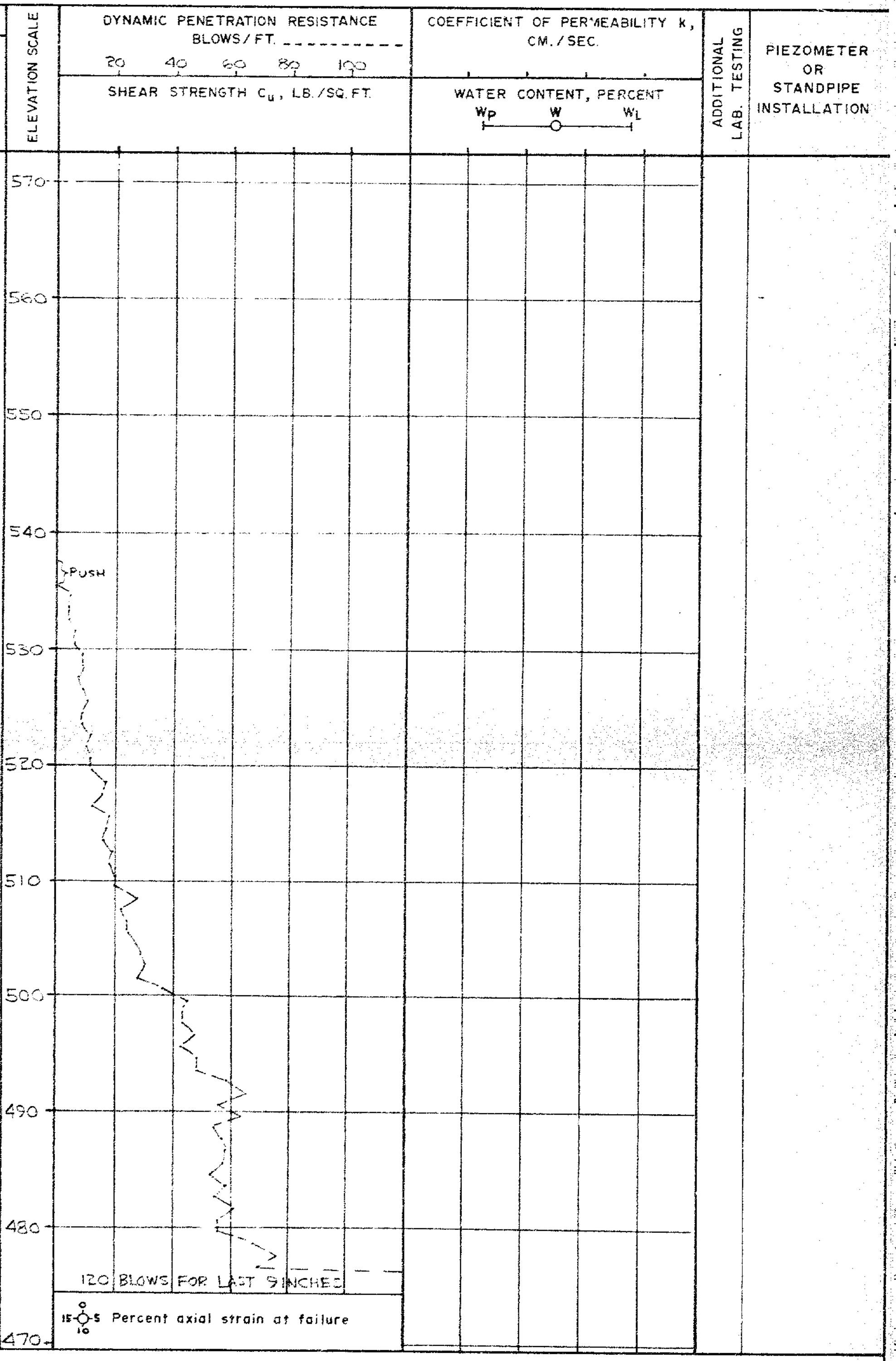
RECORD OF ~~BOREHOLE~~ PEN. TEST C-13

LOCATION STA. 1030+00 - ℄ CANAL See Figure 1 BORING DATE MARCH 22, 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES		
ELEVN. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.
569.5	CANAL LEVEL				
0.0					
537.5 / 32.0	CANAL BOTTOM				
	PROBABLY INTERBEDDED SILTY CLAY (PROBABLY CHANGING TO SILT BELOW ABOUT EL. 502)				
85	PROBABLY SILTY SAND AND GRAVEL (TILL)				
475.8 / 93.7	END OF PEN. TEST				



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED R.T.D.

PROJECT No. [illegible]

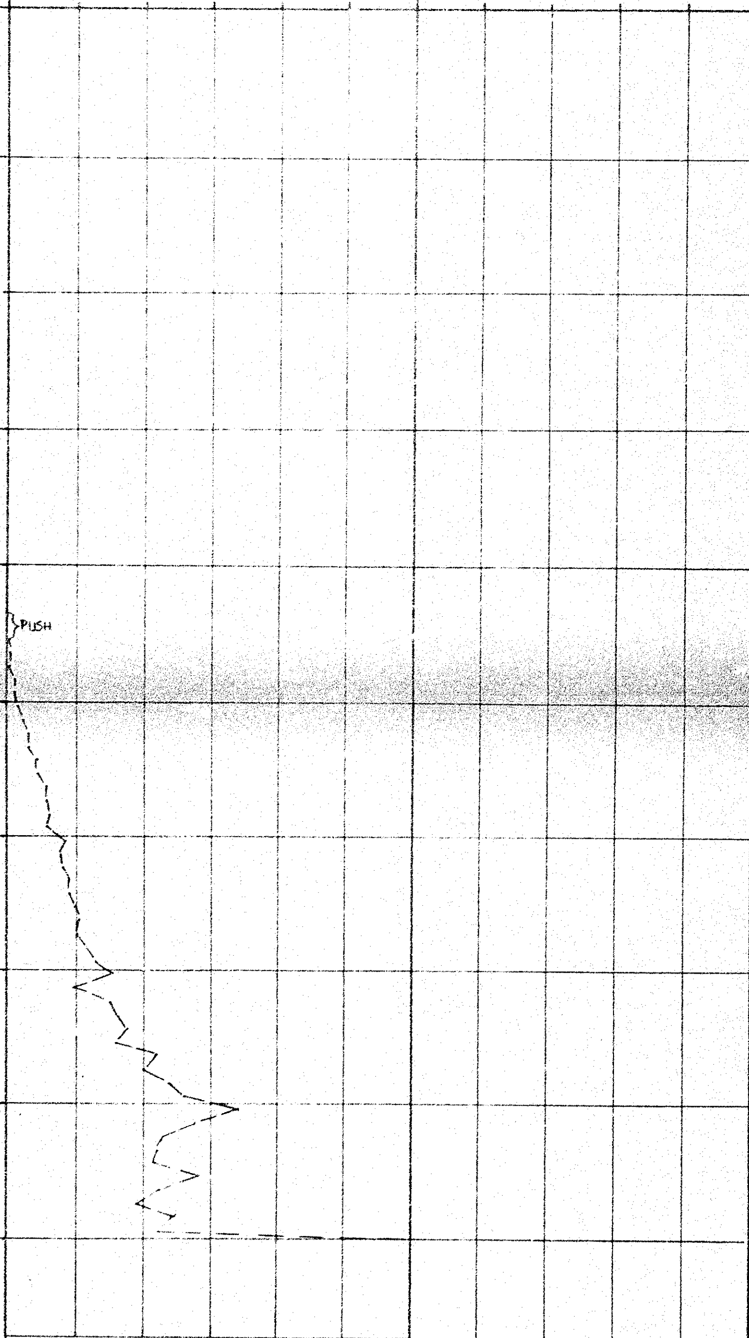
RECORD OF ~~BOREHOLE~~ PEN. TEST C-14

LOCATION STA. 1020+00 - ℄ CANAL See Figure 1 BORING DATE MARCH 21, 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20, 40, 60, 80, 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY K, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT.					
569.7 / 0.0	CANAL LEVEL					570				
						560				
						550				
						540				
536.7 / 33.0	CANAL BOTTOM						PUSH			
	PROBABLY INTERBEDDED SILTY CLAY					530				
						520				
						510				
65	PROBABLY SILTY SAND AND GRAVEL (TILL)					500				
490.0 / 79.7	END OF PEN. TEST					490				
						480				



15 ○ 5 / 10 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED R.T.D.

PROJECT No. [illegible]

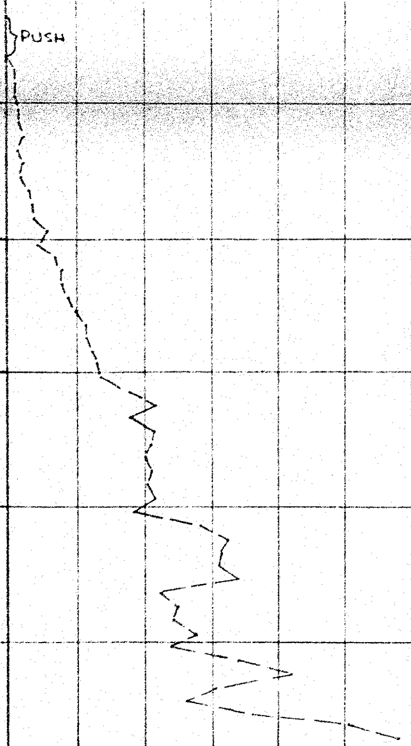
RECORD OF ~~BOREHOLE~~ PEN. TEST C-15

LOCATION STA. 1009+50 - ℄ CANAL See Figure 1 BORING DATE MARCH 20, 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 20 40 60 80 100 / SHEAR STRENGTH C_u, LB./SQ. FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT.					
569.7	CANAL LEVEL					570				
0.0						560				
						550				
						540				
536.7	CANAL BOTTOM									
33.0	PROBABLY INTERBEDDED SILTY CLAY					530	PUSH			
						520				
						510				
						500				
70	PROBABLY SILTY SAND AND GRAVEL (TILL)									
						490				
482.7										
87.0	END OF PEN. TEST					480				



15 -○- 5 (10) Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED B.T.L.

PROJECT No. [illegible]

RECORD OF ~~BOREHOLE~~ PEN. TEST C-16

LOCATION See Figure 1 STA 990+00 - ℄ CANAL

BORING DATE MARCH 18 1967

DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER ———

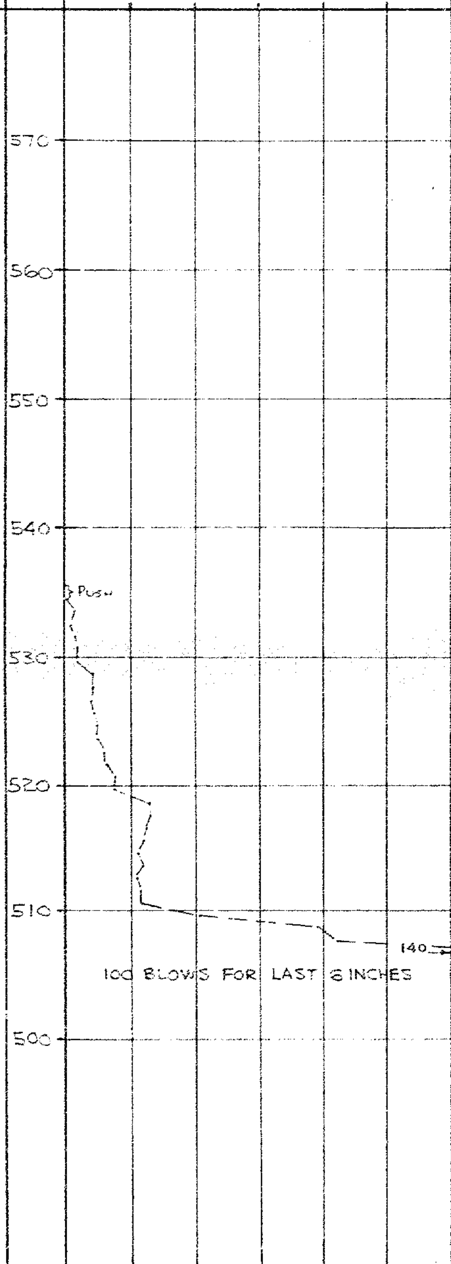
SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FT.	ELEVATION SCALE
						570
568.7 / 0.0	CANAL LEVEL					
						560
						550
						540
535.7 / 33.0	CANAL BOTTOM					
						530
	PROBABLY INTERBEDDED SILTY CLAY					
						520
						510
59 / 506.2	PROBABLY SILTY SAND AND GRAVEL (TILL)					
62.5	END OF PEN TEST					
						500

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. 20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.



15 ○ 5 Percent axial strain at failure

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED [illegible]

PROJECT NO.

RECORD OF ~~BOREHOLE~~ PEN. TEST C-17

LOCATION: STA. 580+50 ℄ CANAL, See Figure 1 | BORING DATE: MARCH 20, 1967 | DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST | BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES | PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

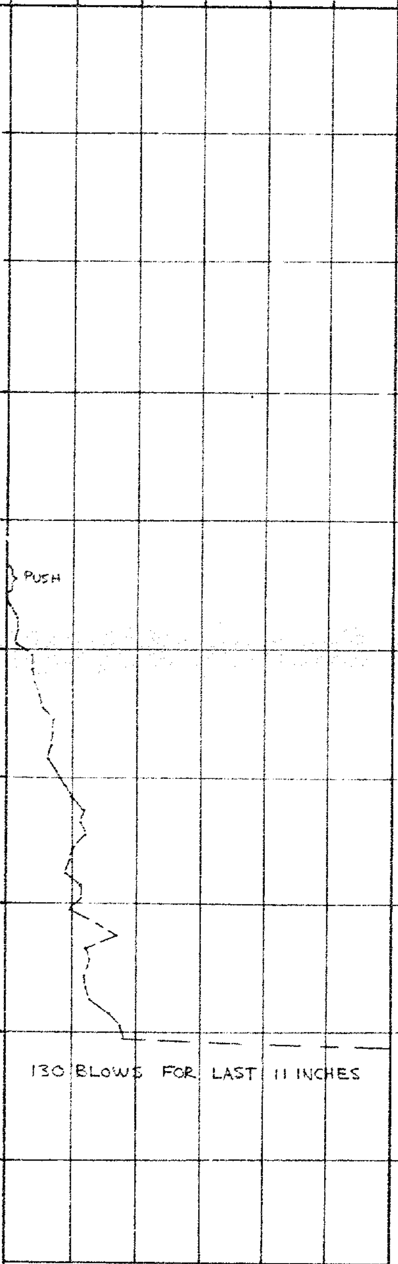
SOIL PROFILE: ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
569.7 / 0.0	CANAL LEVEL					570
						560
						550
						540
536.7 / 33.0	CANAL BOTTOM					
	PROBABLY INTERBEDDED SILTY CLAY					530
						520
						510
500.0						500
498.8 / 70.9	END OF PEN. TEST					
	PROBABLY SILTY SAND AND GRAVEL (TILL)					490

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. - - - - - - - -
20 40 60 80 100
SHEAR STRENGTH C_u, LB./SQ.FT

COEFFICIENT OF PERMEABILITY k, CM./SEC.
WATER CONTENT, PERCENT
W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



15-○-5 Percent axial strain at failure (10)

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED

PROJECT No. 67028

RECORD OF ~~BOREHOLE~~ PEN. TEST C-18

LOCATION: STA. 969+90 - ℄ CANAL, See Figure 1

BORING DATE: MARCH 21, 1967

DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST

BOREHOLE DIAMETER: ----

SAMPLER HAMMER WEIGHT ---- LB. DROP ---- INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
569.7 / 0.0	CANAL LEVEL					570
						560
						550
						540
534.7 / 35.0	CANAL BOTTOM					530
	PROBABLY INTERBEDDED SILTY CLAY					520
						510
						500
						490
85.3						
84.4	END OF PEN. TEST					
	PROBABLY SILTY SAND AND GRAVEL (TILL)					480

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. ----------

20 40 60 80 100

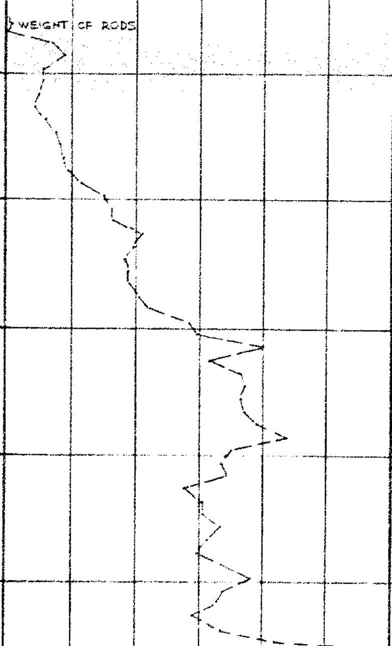
SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



15 ○ 5 (10) Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED R.T.R.

PROJECT No. 6790 EO

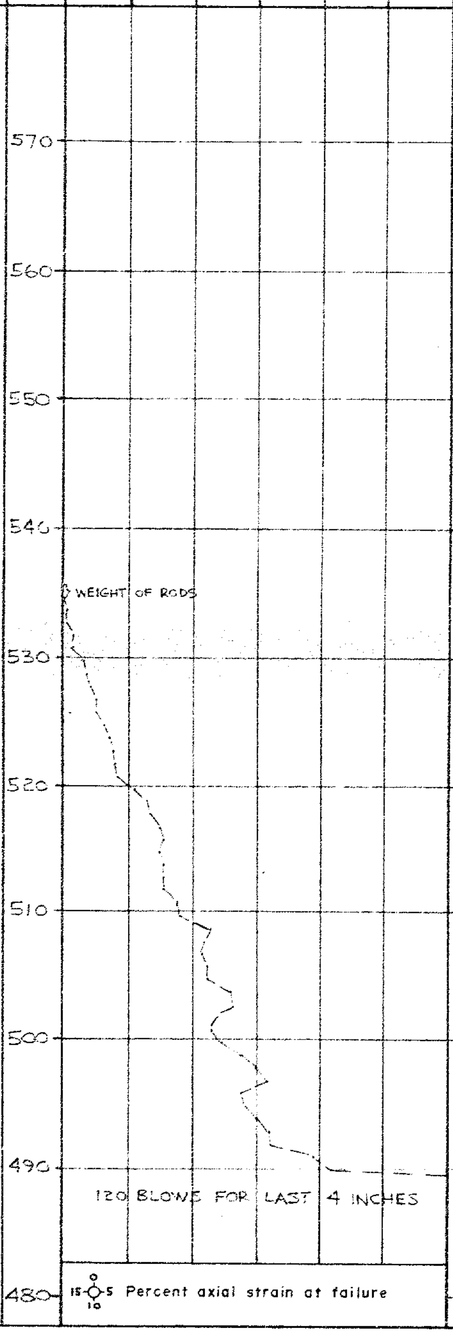
RECORD OF ~~BOREHOLE~~ PEN. TEST C-19

LOCATION STA. 960+50 - ℄ CANAL See Figure 1 BORING DATE MARCH 20, 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 20 40 60 80 100 / SHEAR STRENGTH C_u, LB./SQ.FT	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FT.					
569.7 / 0.0	CANAL LEVEL									
535.7 / 34.0	CANAL BOTTOM									
	PROBABLY INTERBEDDED SILTY CLAY									
78 / 89.4										
80.3	END OF PEN. TEST									
	PROBABLY SILTY SAND AND GRAVEL (TILL)									



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED R.T.D.

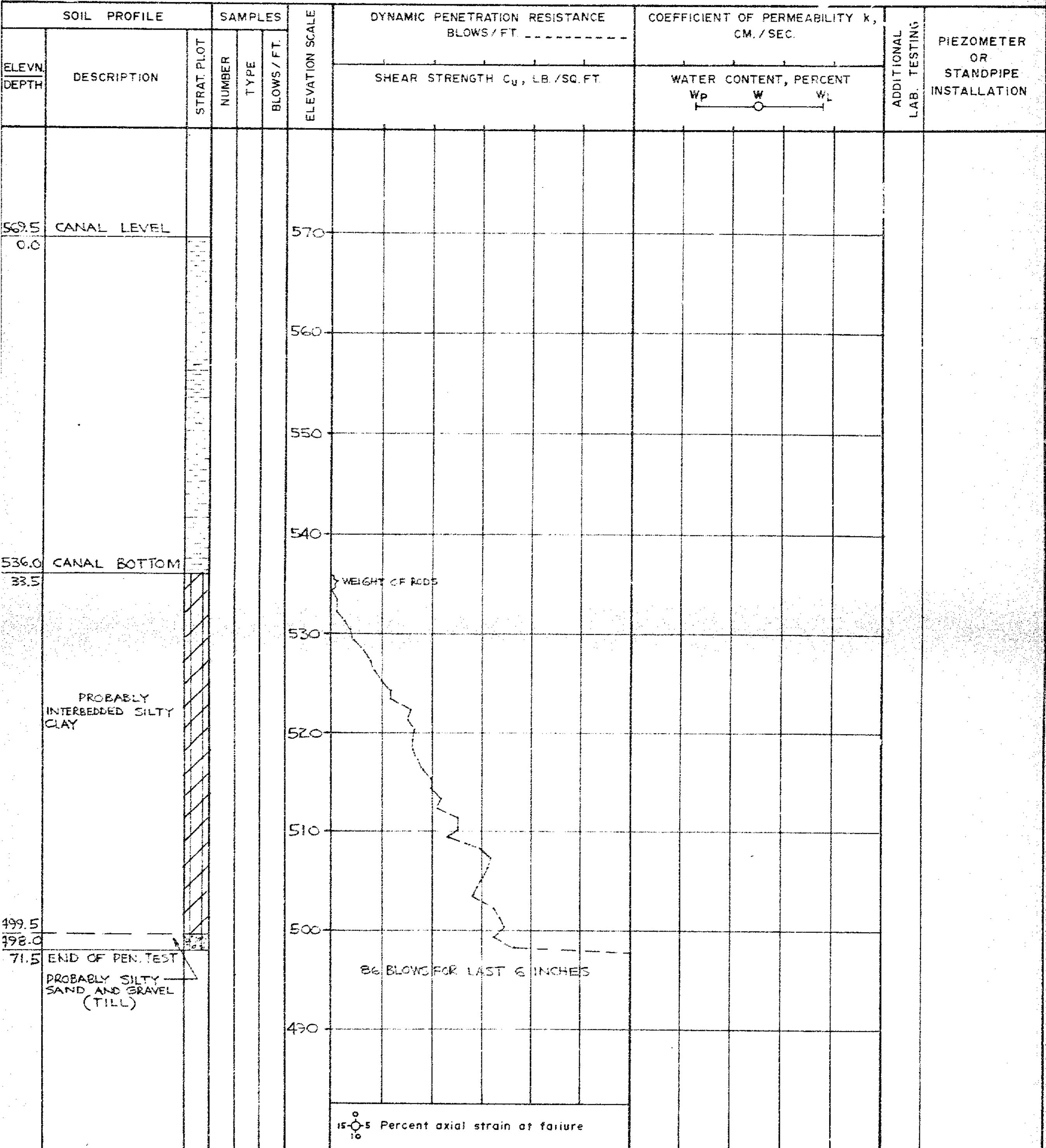
PROJECT No. 67058

RECORD OF ~~BOREHOLE~~ PEN. TEST C-20

LOCATION STA. 941+60 - ℄ CANAL See Figure 1 BORING DATE MARCH 22, 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ———

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED R.T.R.

PROJECT No.

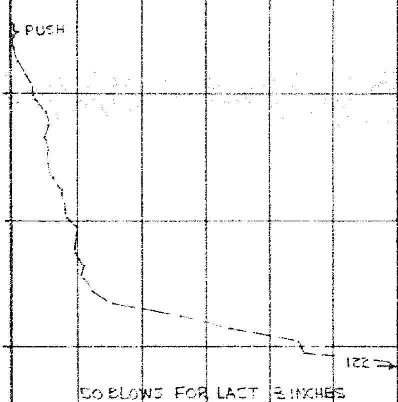
RECORD OF ~~BOREHOLE~~ PEN. TEST C-21

LOCATION: See Figure 1 — STA. 930+00 - ℄ CANAL
BORING DATE: MARCH 22 1967
DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST
BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20 40 60 80 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT Wp W WL	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT					
569.5 / 0.0	CANAL LEVEL					570				
						560				
						550				
						540				
535.5 / 34.0	CANAL BOTTOM						PUSH			
	PROBABLY INTERBEDDED SILTY CLAY					530				
						520				
55	PROBABLY SILTY SAND AND GRAVEL (TILL)					510	122 →			
508.3 / 61.2	END OF PEN. TEST						50 BLOWS FOR LAST 3 INCHES			
						500				



15 ○ 5 (0, 10) Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED R.J.D.

PROJECT No. [illegible]

RECORD OF ~~BOREHOLE~~ PEN. TEST C-22

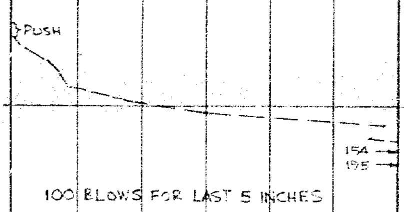
LOCATION: STA. 920+00 - ℄ CANAL, See Figure 1
BORING DATE: MARCH 27, 1967
DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST
BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	BLOWS / FT	ELEVATION SCALE
569.5 / 0.0	CANAL LEVEL					570
						560
						550
						540
536.6 / 32.9	CANAL BOTTOM					
	PROBABLY CLAYEY SILT (TILL)					530
525.1 / 44.4	END OF PEN. TEST					
						520

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. – 20, 40, 60, 80, 100
SHEAR STRENGTH C_u, LB./SQ. FT



COEFFICIENT OF PERMEABILITY k, CM./SEC.
WATER CONTENT, PERCENT – W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

15 -○- 5 (0, 10) Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED [illegible]

PROJECT No. [illegible]

RECORD OF ~~BOREHOLE~~ PEN. TEST C-23

LOCATION: See Figure 1 — STA. 905+75 ℄ CANAL

BORING DATE: MARCH 27, 1967

DATUM: GEODETIC

BOREHOLE TYPE: PENETRATION TEST

BOREHOLE DIAMETER: ——

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT. (20, 40, 60, 80, 100) / SHEAR STRENGTH C_u, LB./SQ.FT	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P, W, W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.5	CANAL LEVEL					570				
0.0						560				
						550				
						540				
537.3 / 32.2	CANAL BOTTOM						PUSH			
	PROBABLY CLAYEY SILT (TILL)									
530.1 / 39.4	END OF PEN. TEST					530	150 BLOWS FOR LAST 5 INCHES			
						520				

15-○-5 (0 above, 10 below) Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED [illegible]

PROJECT No 67058

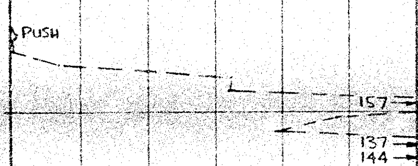
RECORD OF ~~BOREHOLE~~ PEN. TEST C-24

LOCATION STA. 890+00 See Figure 1 BORING DATE MARCH 28, 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT 20 40 60 80 100 / SHEAR STRENGTH C_u, LB./SQ.FT	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.					
569.5	CANAL LEVEL					570				
0.0						560				
						550				
						540				
536.2	CANAL BOTTOM									
33.3	PROBABLY CLAYEY SILT (TILL)					530				
526.5										
43.0	END OF PEN. TEST					520				



15 ○ 5 Percent axial strain at failure
10

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.T.R.

PROJECT No. [illegible]

RECORD OF ~~BOREHOLE~~ PEN. TEST C-25

LOCATION STA. 879 + 50 - ℄ CANAL See Figure 1 — BORING DATE MARCH 28, 1967 — DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST — BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS / FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT. / SHEAR STRENGTH C_u, LB./SQ. FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
569.5	CANAL LEVEL					570				
0.0						560				
						550				
						540				
538.1	CANAL BOTTOM						PUSH			
31.4 / 535.0	PROBABLY CLAYEY SILT (TILL)						200 BLOWS FOR LAST 6 INCHES			
34.5	END OF PEN. TEST					530				

15-○-5 (top 0, bottom 10) Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED [illegible]

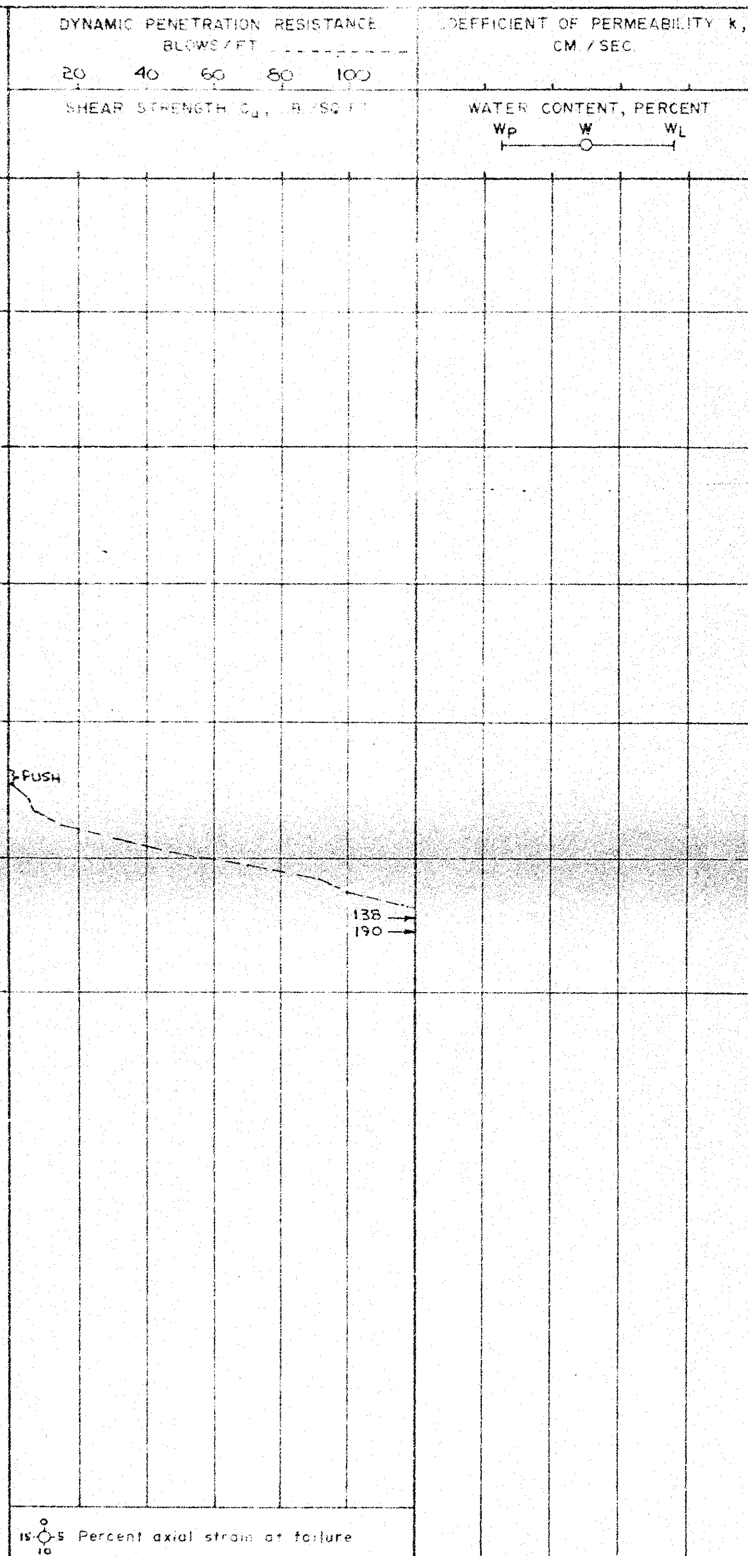
RECORD OF PEN. TEST ~~BOREHOLE~~ C-26

LOCATION STA. 870 + 15 - ℄ CANAL See Figure 1 BORING DATE MARCH 28, 1967 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT	ELEVATION SCALE
569.5	CANAL LEVEL					570
0.0						560
						550
						540
536.5 33.0	CANAL BOTTOM					
	PROBABLY CLAYEY SILT (TILL)					530
524.5 45.0	END OF PEN. TEST					520



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED B.T.D.

PROJECT No. 67058

RECORD OF ~~BOREHOLE~~ PEN. TEST C-27

LOCATION STA. 859+60 - ℄ CANAL See Figure 1 BORING DATE MARCH 29, 1967 DATUM GEODETIC

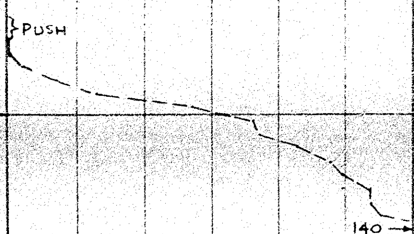
BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

SAMPLER HAMMER WEIGHT —— LB. DROP —— INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FT	ELEVATION SCALE
569.6	CANAL LEVEL					570
0.0						560
						550
						540
537.1	CANAL BOTTOM					
32.5	PROBABLY CLAYEY SILT (TILL)					530
521.6 / 48.0	END OF PEN. TEST					520
						510

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 20 40 60 80 100

SHEAR STRENGTH C_u LB./SQ.FT.



COEFFICIENT OF PERMEABILITY k, CM./SEC

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

15-O-5 Percent axial strain at failure (10)

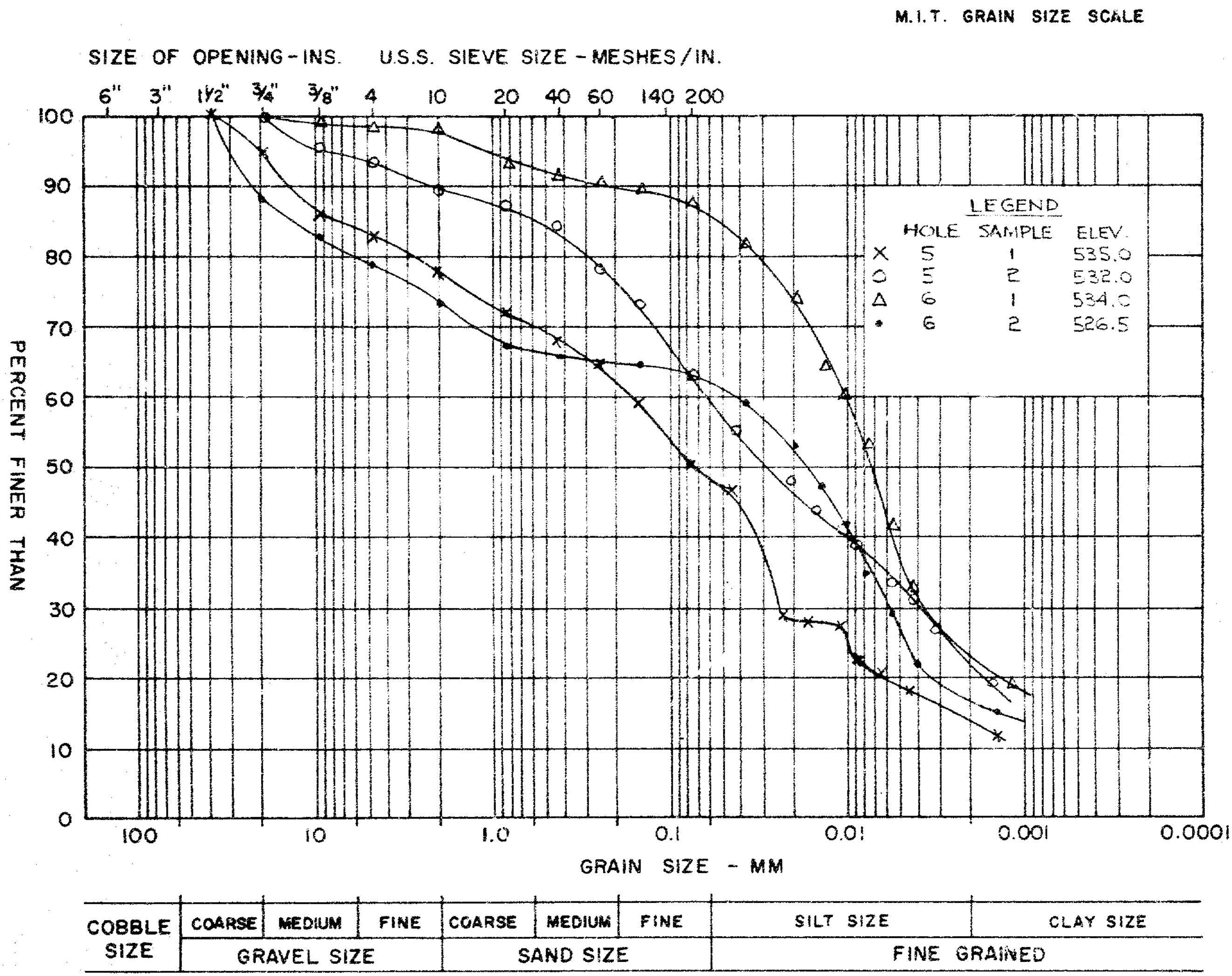
VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED B.J.D.

PROJECT No. 67029



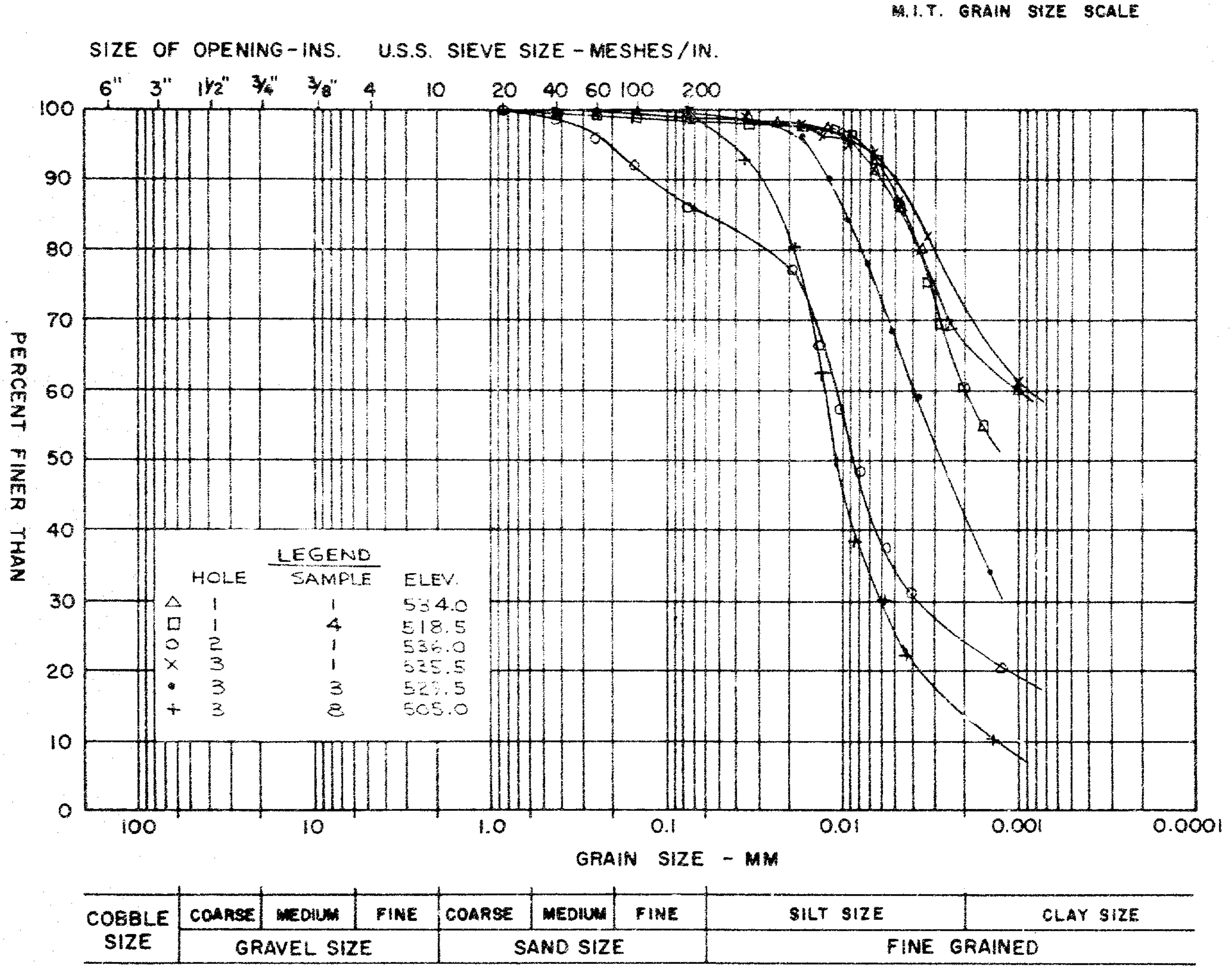
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

FIGURE 3

PROJECT No. 67028



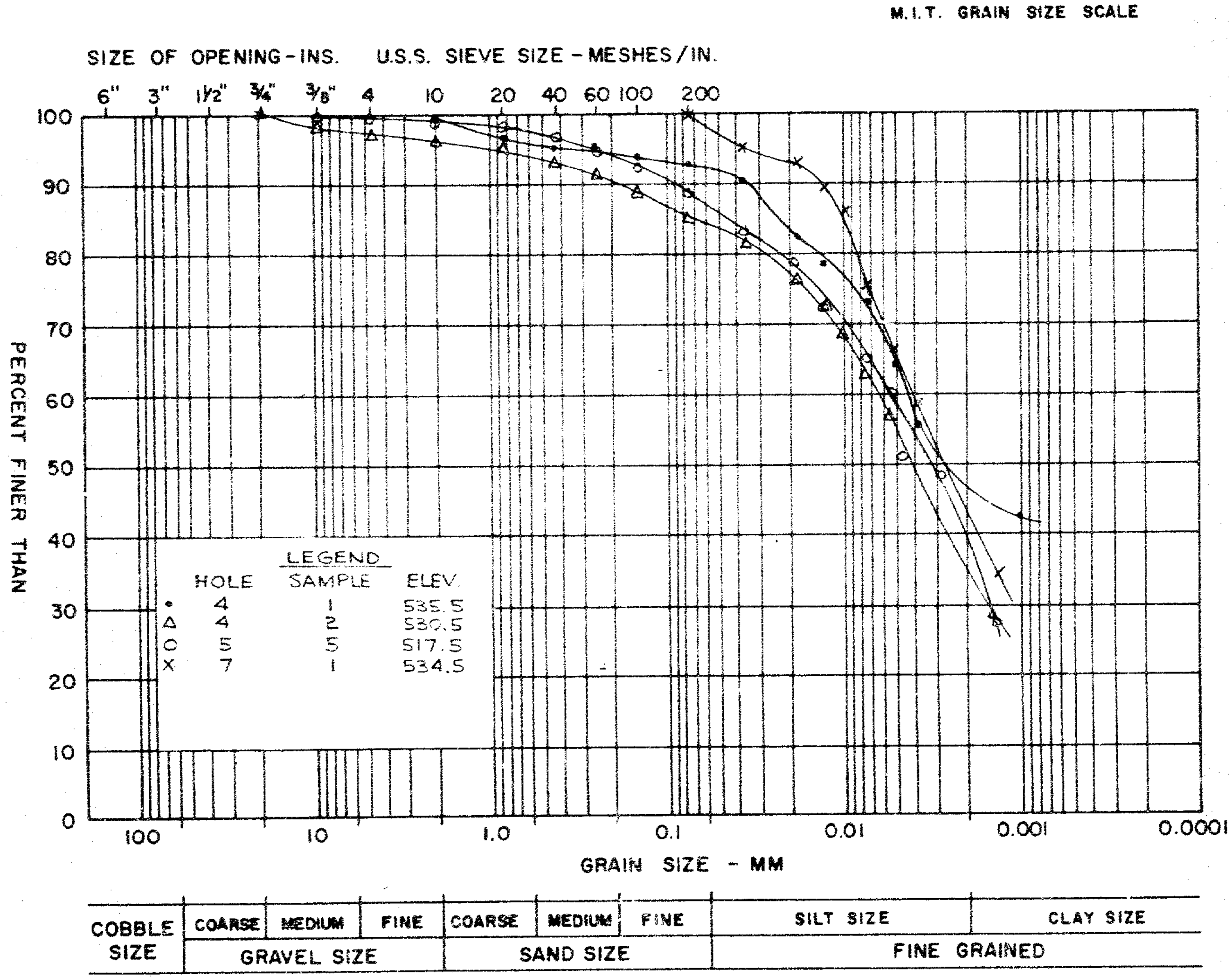
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

INTERBEDDED SILTY CLAY

FIGURE 4

PROJECT No. 67028



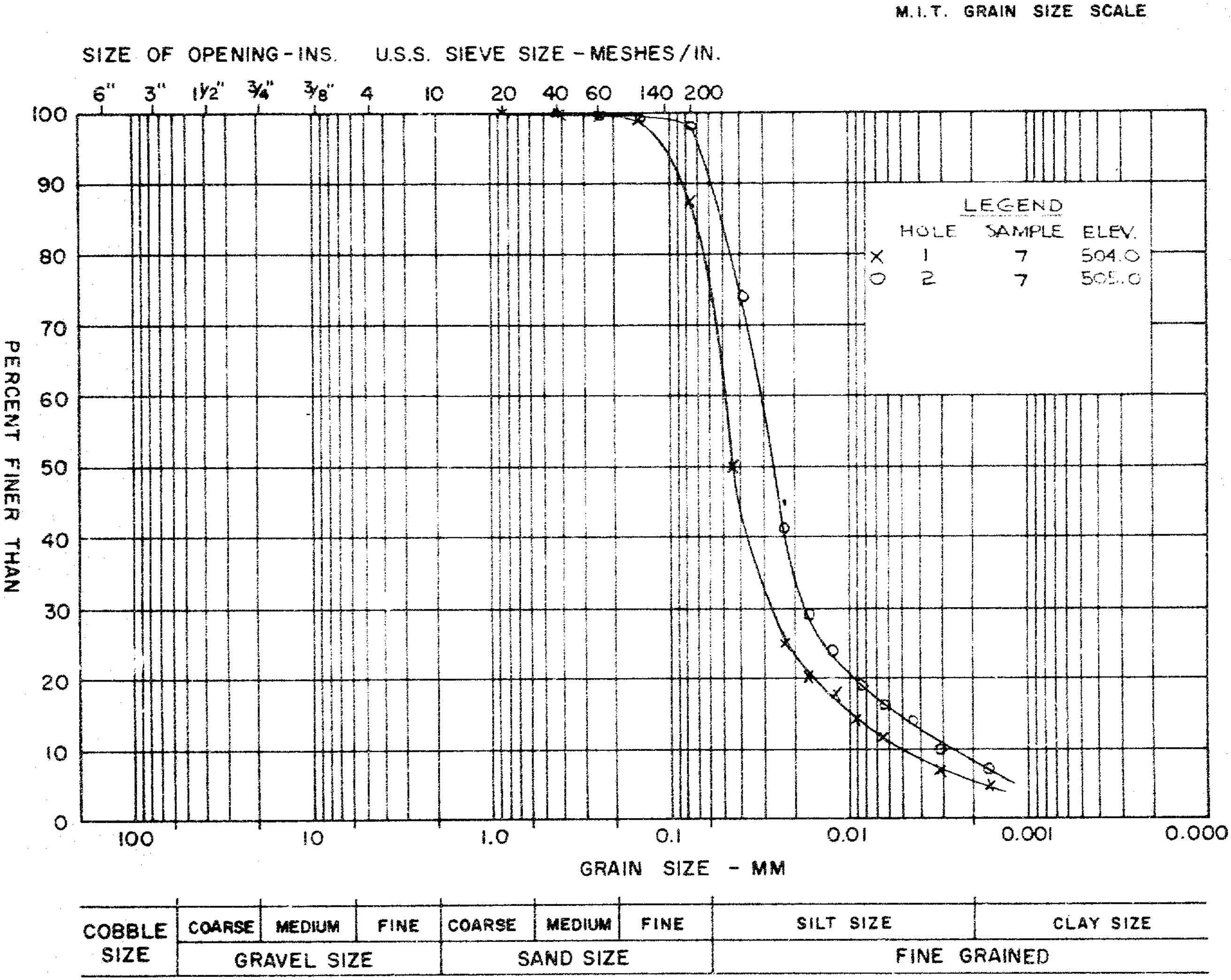
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

INTERBEDDED SILTY CLAY

FIGURE 5

PROJECT No. 67028



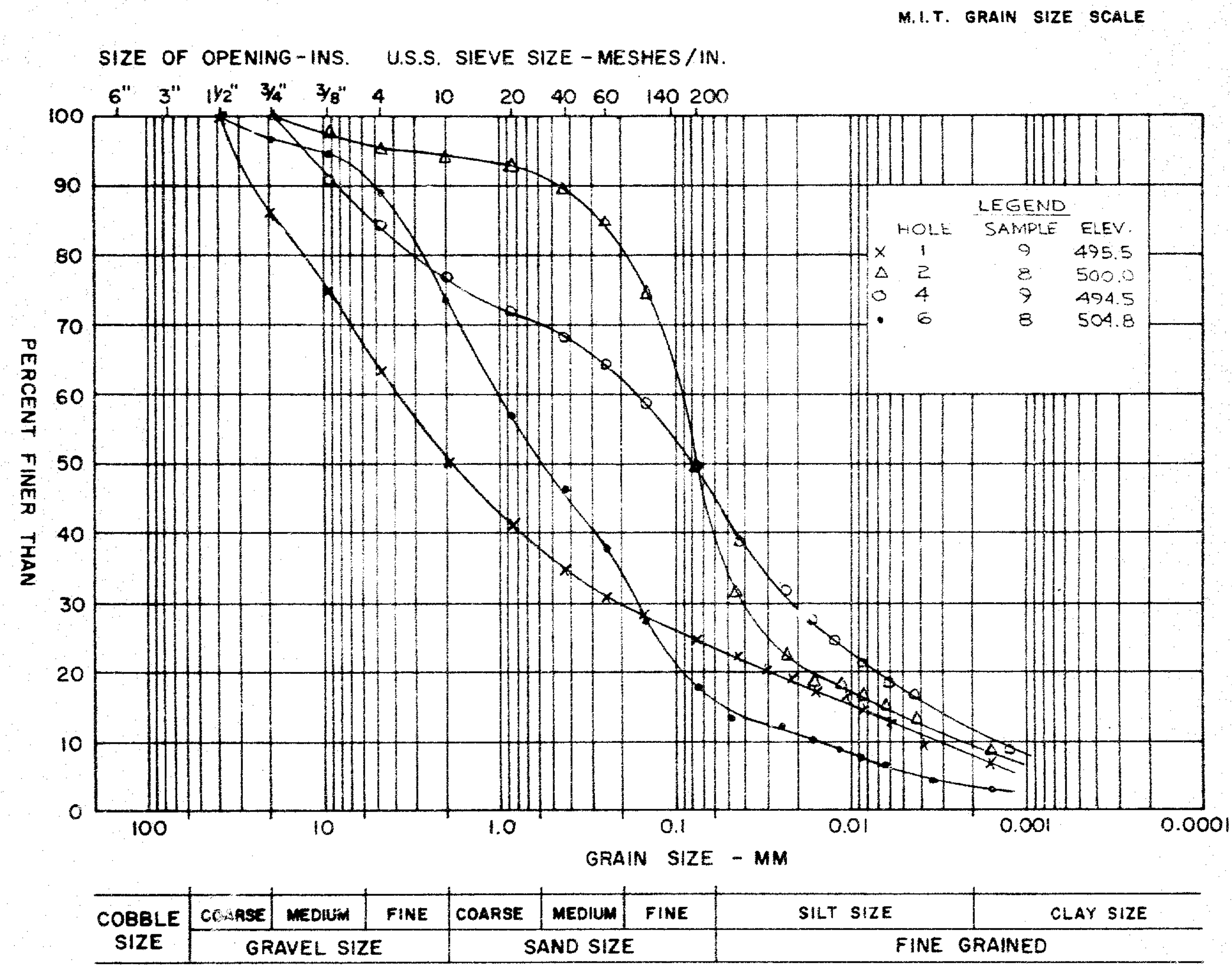
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

SILT

FIGURE 6

PROJECT No. 67028



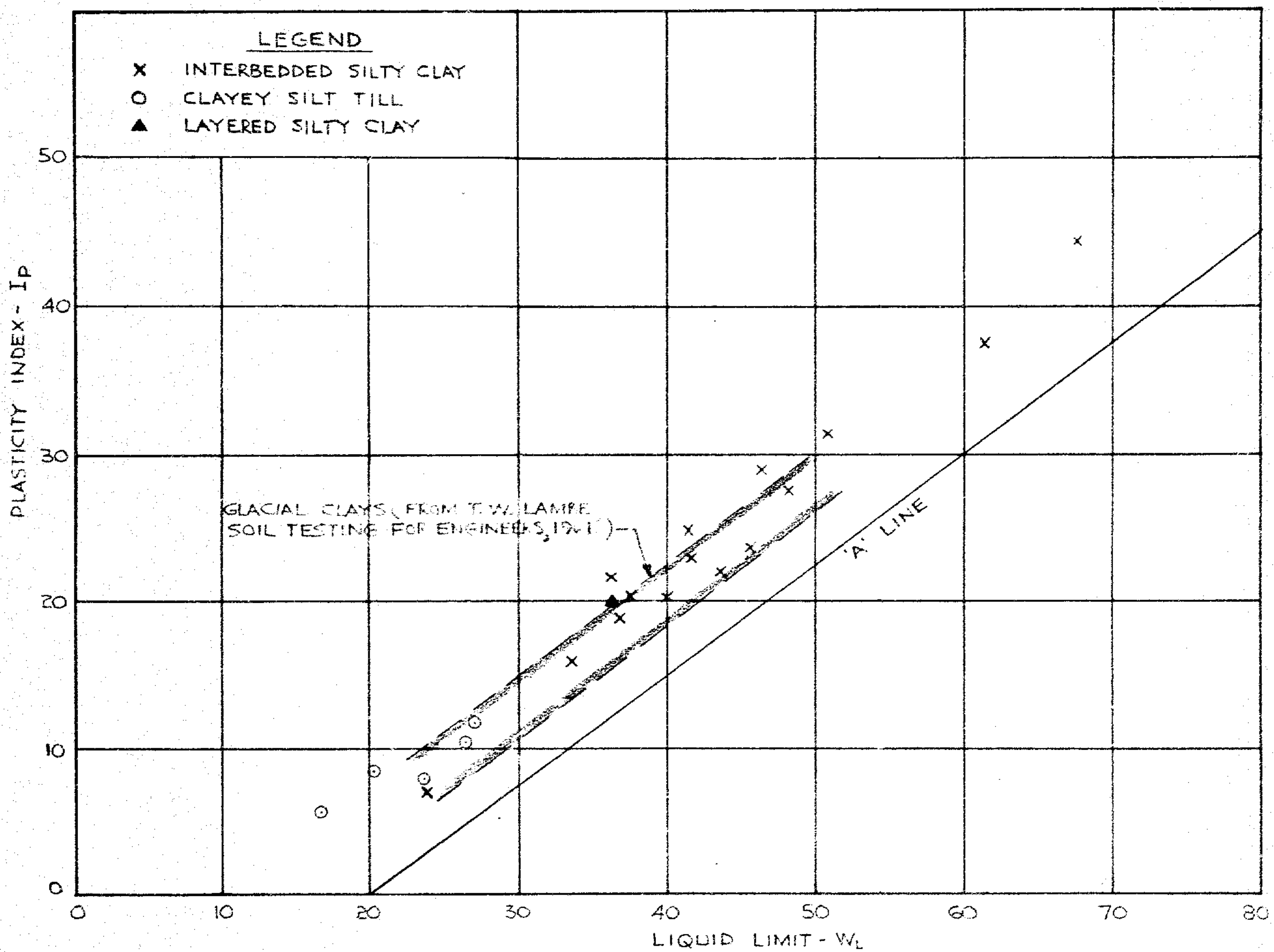
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

SILTY SAND TILL

FIGURE 7

PROJECT No. 67058



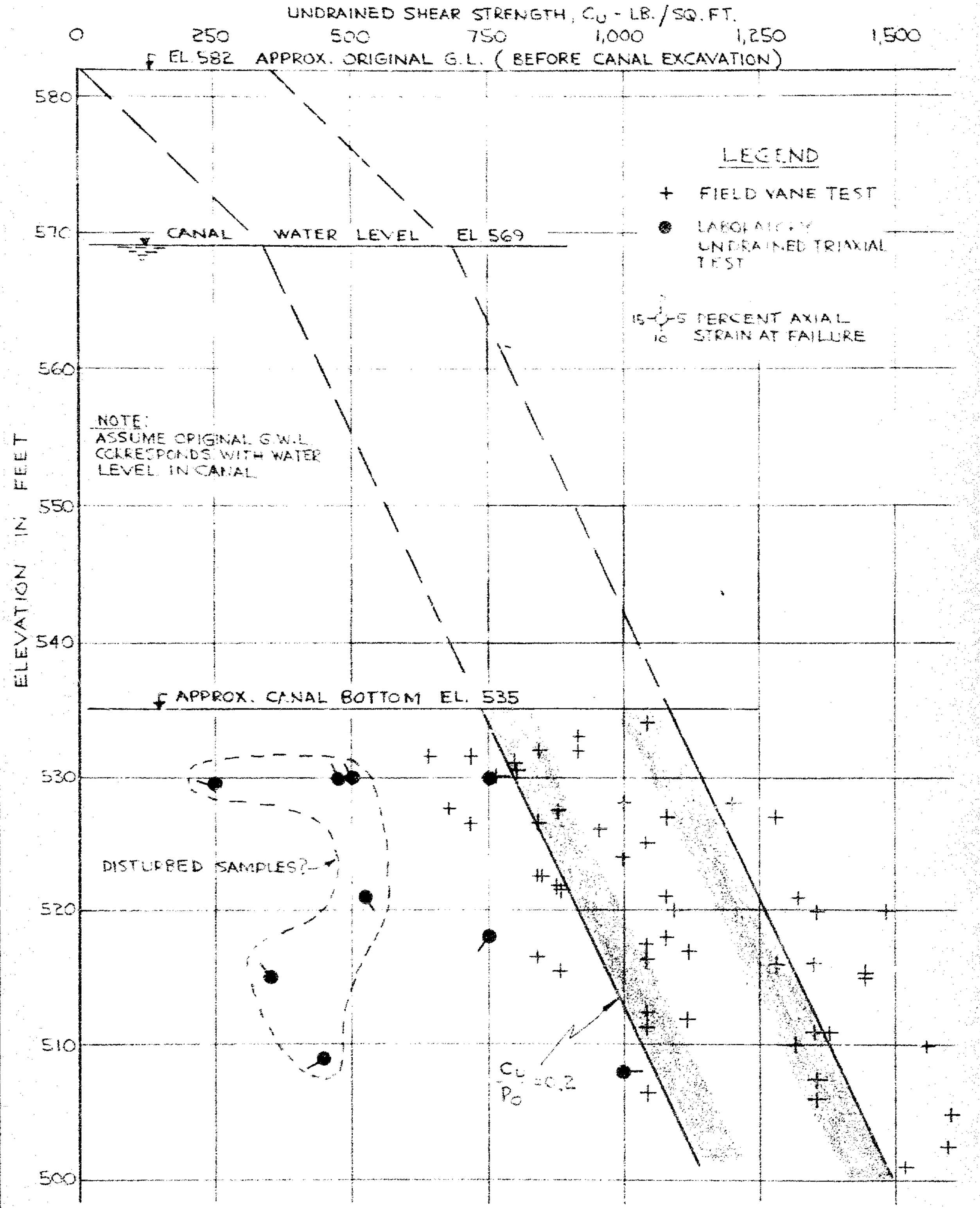
GOLDER & ASSOCIATES

PLASTICITY CHART

FIGURE 8

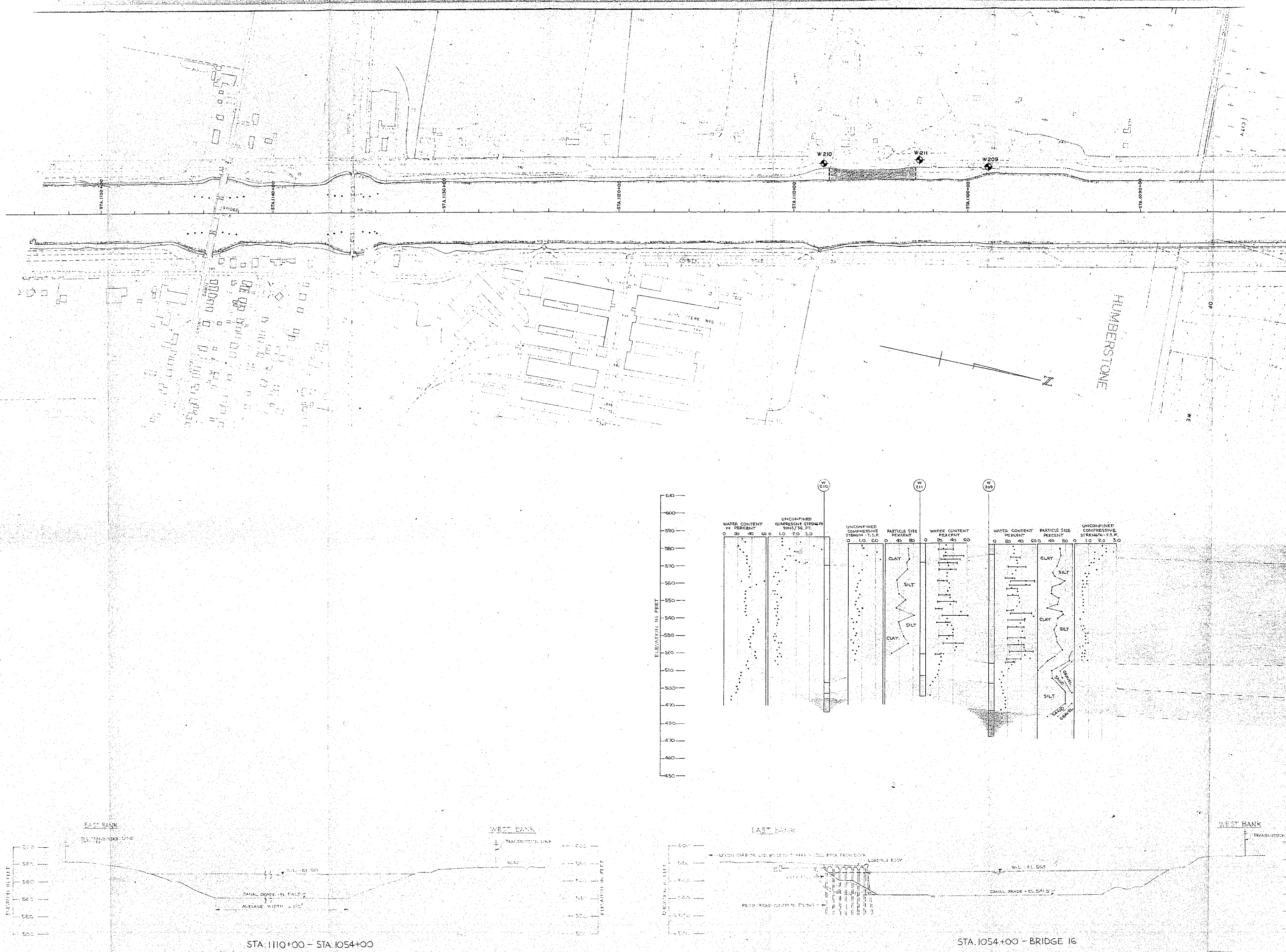
SUMMARY PLOT
OF UNDRAINED SHEAR STRENGTH VS ELEVATION
INTERBEDDED SILTY CLAY

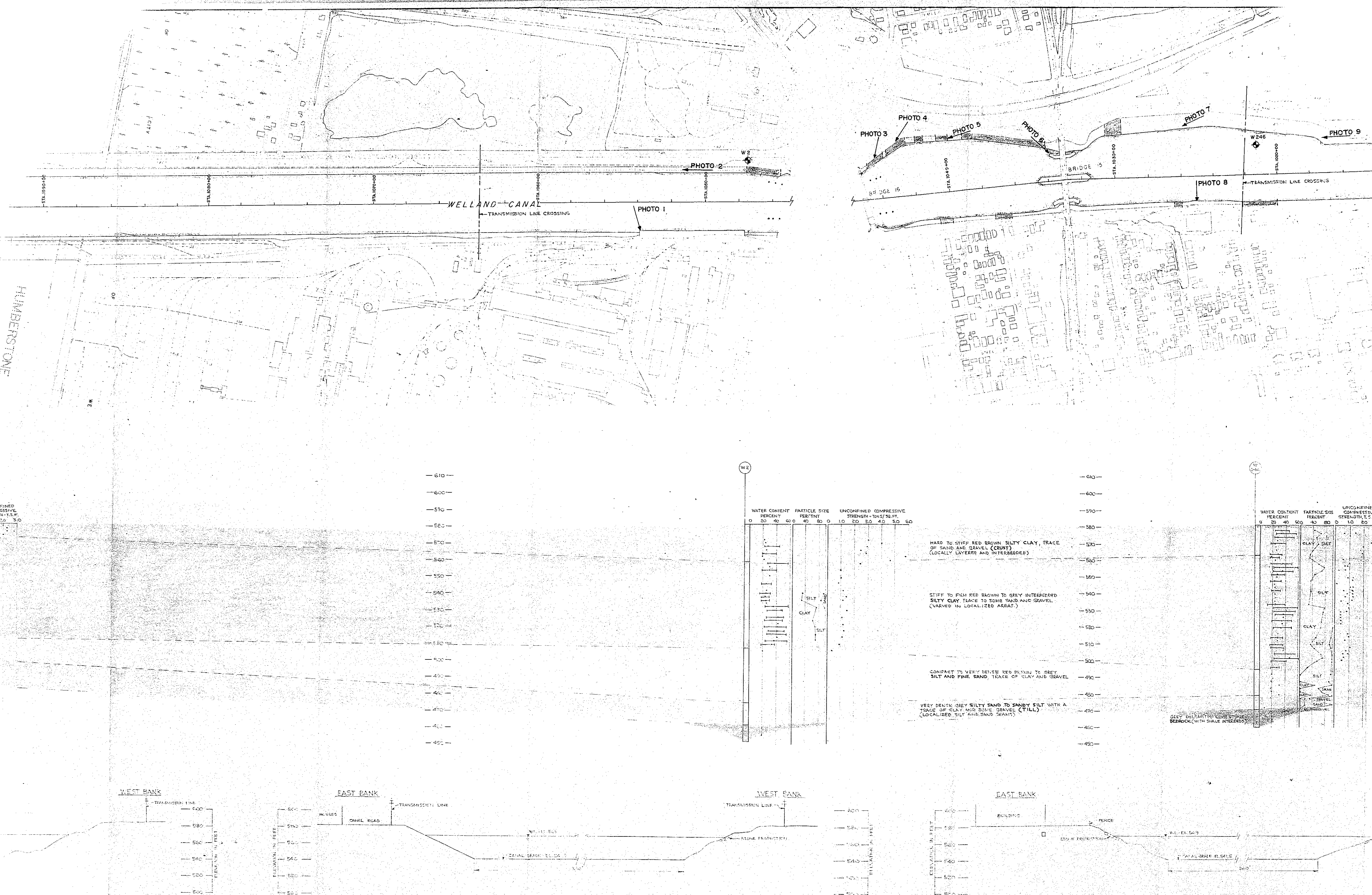
FIGURE 9



GOLDER & ASSOCIATES

Made J.A.
Chkd.
Appd.

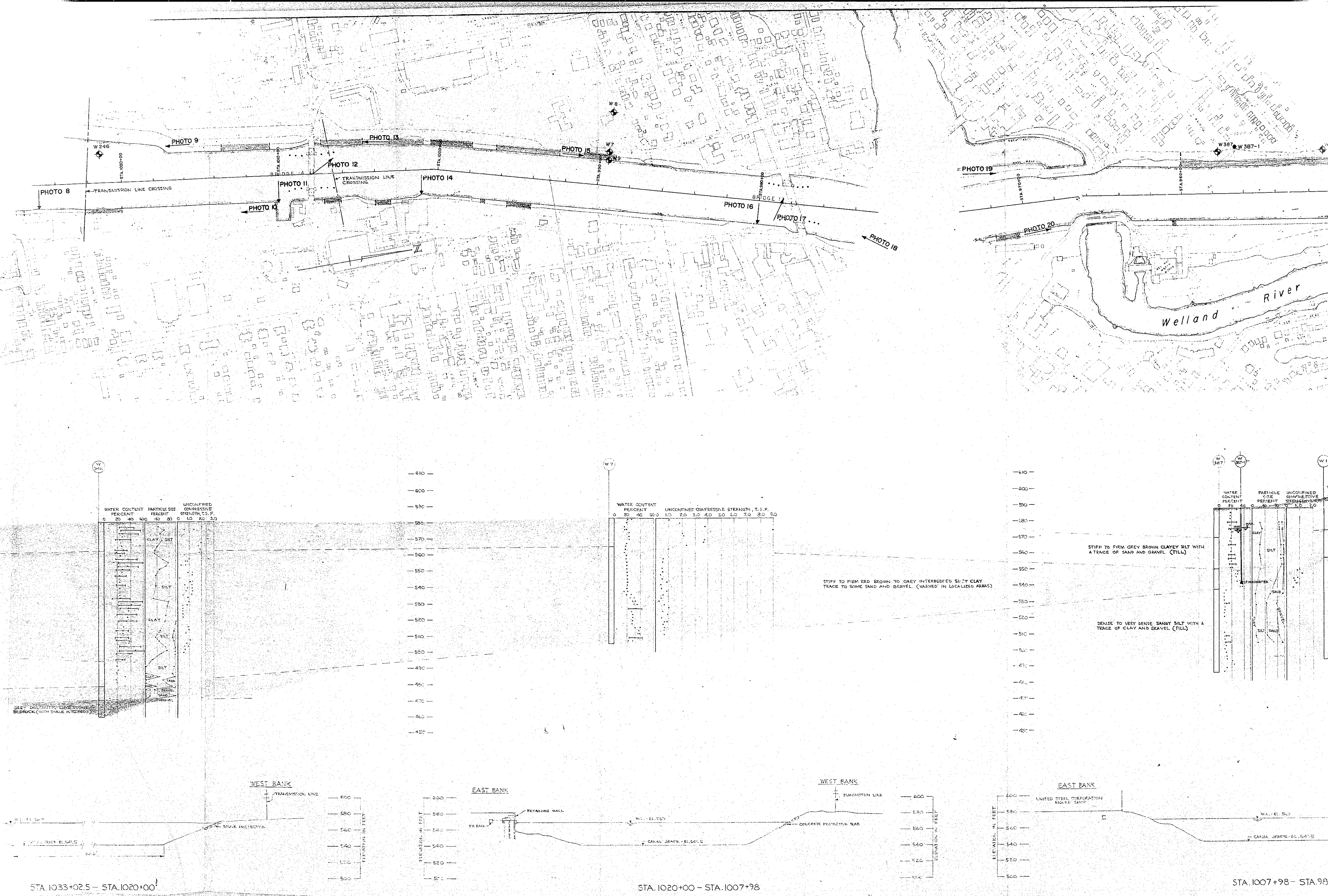

W 210
W 211
W 209
STA.1150+00
STA.1140+00
STA.1130+00
STA.1120+00
STA.1110+00
STA.1100+00
STA.1090+00
HUMBERSTONE
N
ELEVATION IN FEET
WATER CONTENT IN PERCENT
UNCONFINED COMPRESSIVE STRENGTH TONS/SQ. FT.
UNCONFINED COMPRESSIVE STRENGTH - T.S.F.
PARTICLE SIZE PERCENT
CLAY
SILT
SAND
GRAVEL
EAST BANK
WEST BANK
CANAL GRADE
AVERAGE WIDTH
STA. 1110+00 - STA. 1054+00
STA. 1054+00 - BRIDGE 16



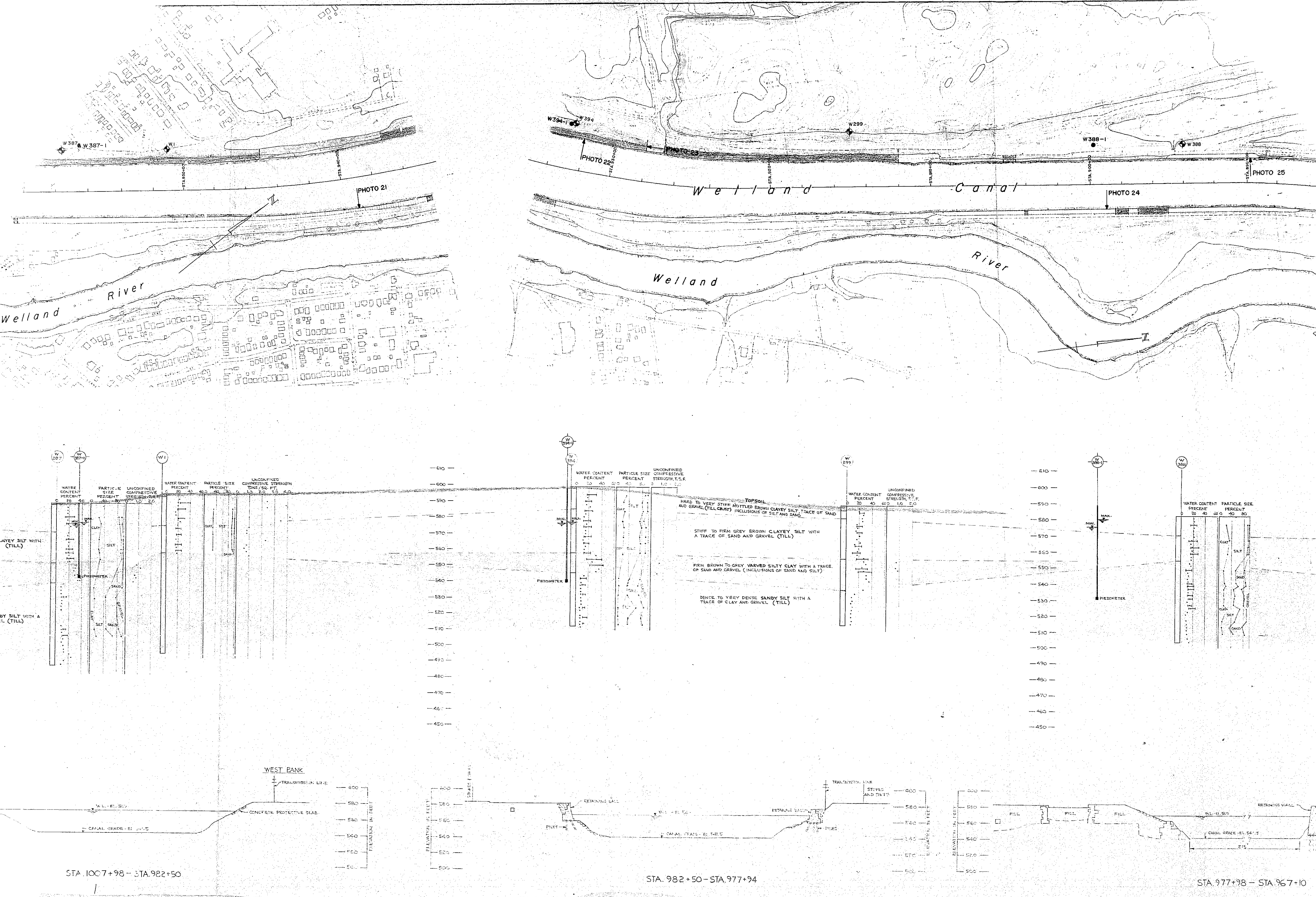
BRIDGE 16 – STA.1033+02.5

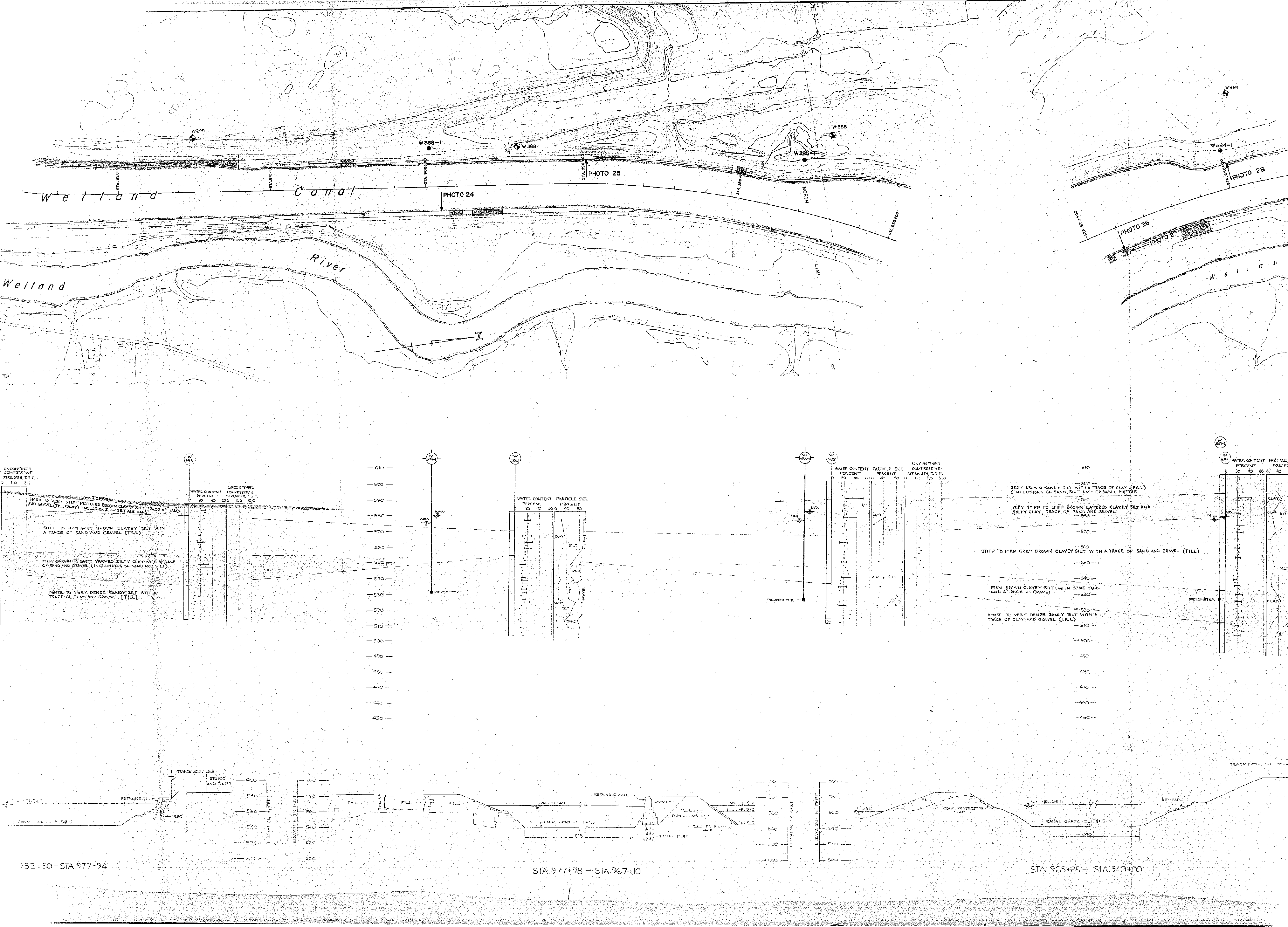
STA. 1033+02.5 – STA. 1020+00

SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS



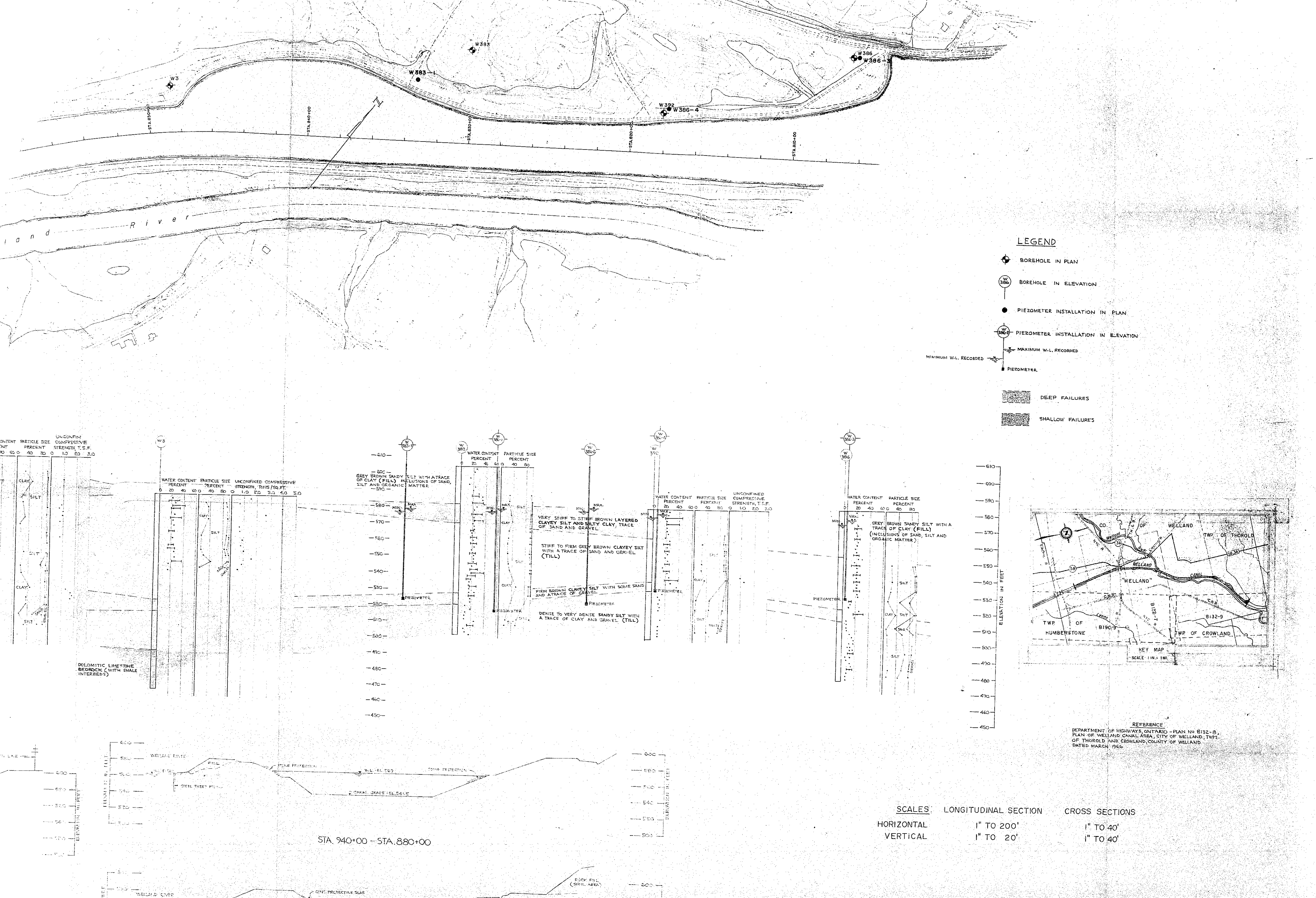
SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS


W387
W 387-1
W1
PHOTO 21
Welland River
W394-1
W394
PHOTO 22
PHOTO 23
W299
W388-1
W388
PHOTO 24
PHOTO 25
Welland Canal
Welland River
WATER CONTENT PERCENT
PARTICLE SIZE PERCENT
UNCONFINED COMPRESSIVE STRENGTH, T.S.F.
TOPSOIL
HARD TO VERY STIFF MOTTLED BROWN CLAYEY SILT, TRACE OF SAND AND GRAVEL (TILL CRUST) INCLUSIONS OF SILT AND SAND
STIFF TO FIRM GREY BROWN CLAYEY SILT WITH A TRACE OF SAND AND GRAVEL (TILL)
FIRM BROWN TO GREY VARVED SILTY CLAY WITH A TRACE OF SAND AND GRAVEL (INCLUSIONS OF SAND AND SILT)
DENSE TO VERY DENSE SANDY SILT WITH A TRACE OF CLAY AND GRAVEL (TILL)
PIEZOMETER
MIN.
MAX.
CLAY
SILT
SAND
GRAVEL
WEST BANK
TRANSMISSION LINE
CONCRETE PROTECTIVE SLAB
CANAL GRADE
RETAINING WALL
PILES
FILL
ELEVATION IN FEET
STA. 1007+98 – STA. 982+50
STA. 982+50 – STA. 977+94
STA. 977+98 – STA. 967+10

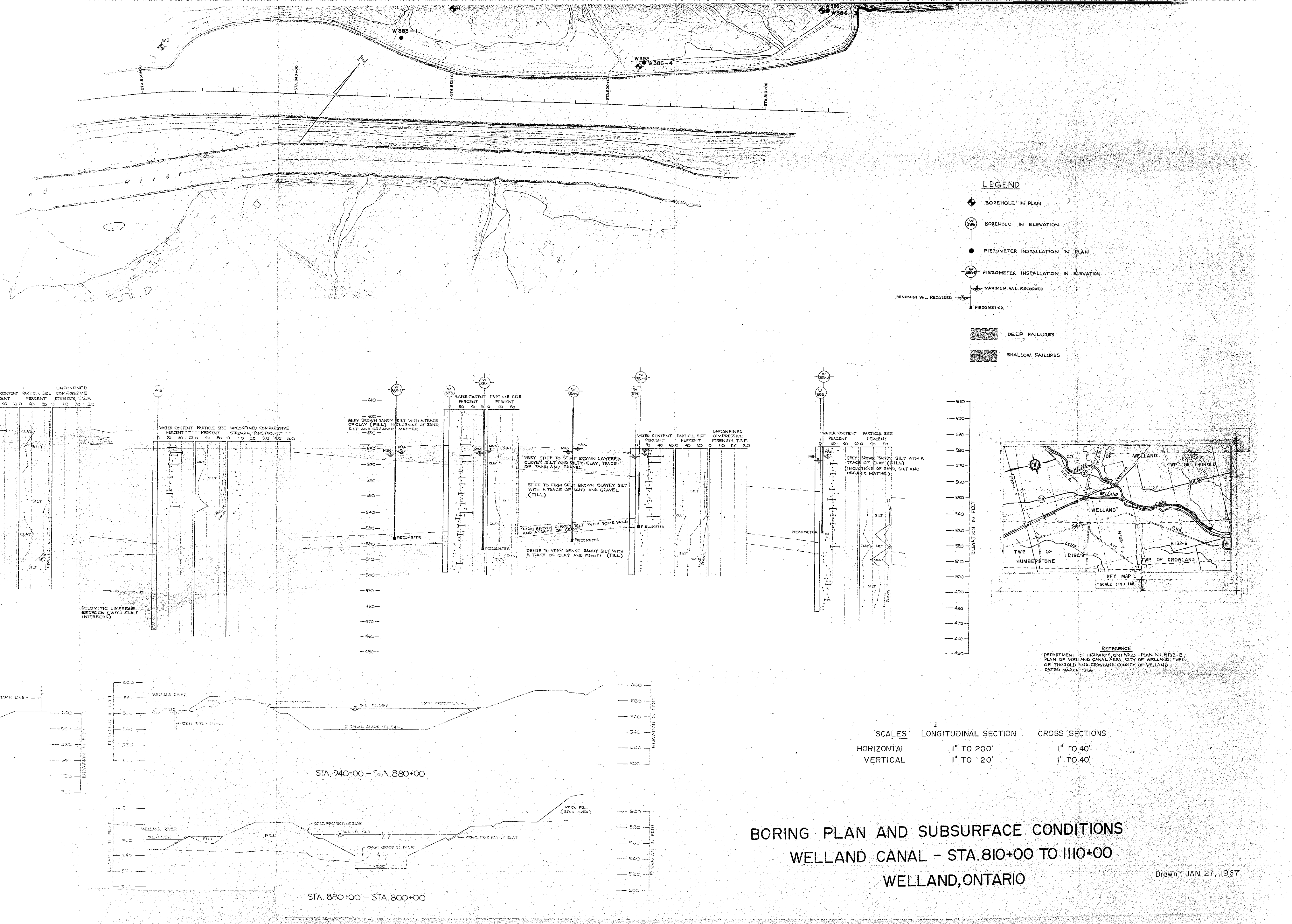


Welland Canal
Welland River
W299
W388-1
W388
W385
W385-1
W384
W384-1
PHOTO 24
PHOTO 25
PHOTO 26
PHOTO 27
PHOTO 28
NORTH
LIMIT
UNCONFINED COMPRESSIVE STRENGTH, T.S.F.
WATER CONTENT PERCENT
PARTICLE SIZE PERCENT
TOPSOIL
HARD TO VERY STIFF MOTTLED BROWN CLAYEY SILT, TRACE OF SAND AND GRAVEL (TILL CRUST) INCLUSIONS OF SILT AND SAND
STIFF TO FIRM GREY BROWN CLAYEY SILT WITH A TRACE OF SAND AND GRAVEL (TILL)
FIRM BROWN TO GREY VARVED SILTY CLAY WITH A TRACE OF SAND AND GRAVEL (INCLUSIONS OF SAND AND SILT)
DENSE TO VERY DENSE SANDY SILT WITH A TRACE OF CLAY AND GRAVEL (TILL)
PIEZOMETER
MAX.
MIN.
CLAY
SILT
SAND
GRAVEL
GREY BROWN SANDY SILT WITH A TRACE OF CLAY (FILL) (INCLUSIONS OF SAND, SILT AND ORGANIC MATTER
VERY STIFF TO STIFF BROWN LAYERED CLAYEY SILT AND SILTY CLAY, TRACE OF SAND AND GRAVEL
STIFF TO FIRM GREY BROWN CLAYEY SILT WITH A TRACE OF SAND AND GRAVEL (TILL)
FIRM BROWN CLAYEY SILT WITH SOME SAND AND A TRACE OF GRAVEL
DENSE TO VERY DENSE SANDY SILT WITH A TRACE OF CLAY AND GRAVEL (TILL)
610 600 590 580 570 560 550 540 530 520 510 500 490 480 470 460 450
TRANSMISSION LINE
STONES AND PIERS
RETAINING WALL
PILES
CANAL GRADE - EL. 541.5
ELEVATION IN FEET
FILL
ROCK FILL
RELATIVELY IMPERVIOUS FILL
TIMBER PILES
CONC. PROTECTIVE SLAB
RIP-RAP
EL. 562
~340'
982+50 – STA. 977+94
STA. 977+98 – STA. 967+10
STA. 965+25 – STA. 940+00

FIGURE

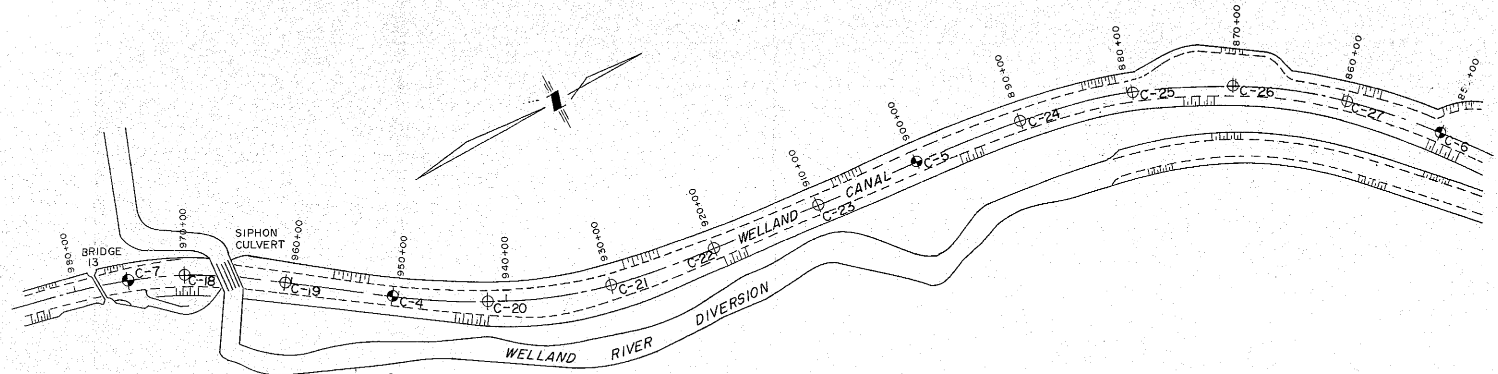


BORING PLAN AND SUBSURFACE CONDITIONS



BORING PLAN AND SUBSURFACE CONDITIONS
WELLAND CANAL - STA. 810+00 TO 1110+00
WELLAND, ONTARIO

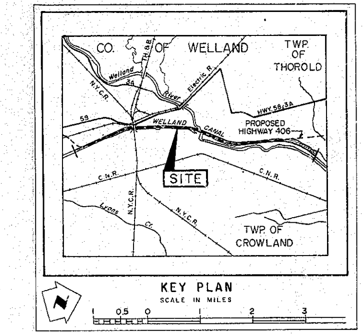
Drawn: JAN. 27, 1967



PLAN

SCALE

500 250 0 500 1,000FT.



KEY PLAN

SCALE IN MILES

1 0.5 0 1 2 3

LEGEND

Bore Hole

Cone Penetration Hole

Bore & Cone Penetration Hole

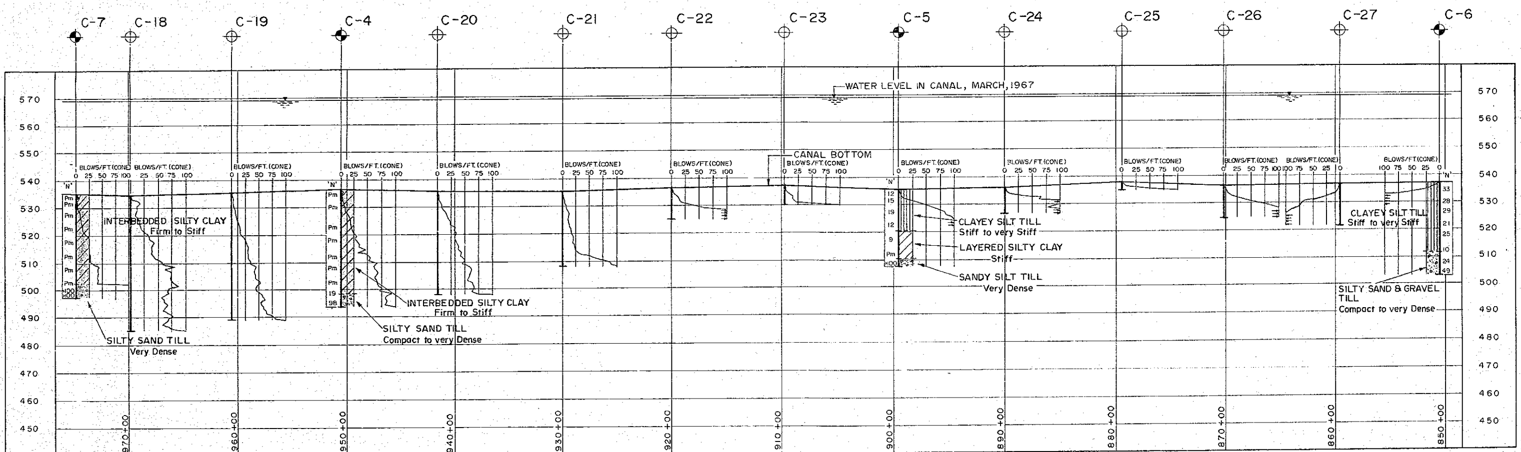
Water Levels established at time of field investigation.

ELEVATIONS REFER TO CANAL BOTTOM AT BOREHOLE LOCATIONS

NO.	ELEVATION	STATION	OFFSET
C-4	536.8	950+50	℄
C-5	535.9	899+50	"
C-6	537.3	850+50	"
C-7	535.1	975+00	"
C-18	534.7	969+90	"
C-19	535.7	960+50	"
C-20	536.0	941+60	"
C-21	535.5	930+00	"
C-22	536.6	920+00	"
C-23	537.3	909+75	"
C-24	536.2	890+00	"
C-25	538.1	879+50	"
C-26	536.5	870+15	"
C-27	537.1	859+60	"

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



SECTION ALONG CENTRELINE OF CANAL

SCALE

500 250 0 500 1,000FT.

PRINT RECORD		
NO.	FOR	DATE

REVISIONS		
DATE	BY	DESCRIPTION

H. Q. GOLDER & ASSOCIATES LTD.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION - FOUNDATION SECTION

WELLAND CANAL

STA. 850+00 TO STA. 980+00

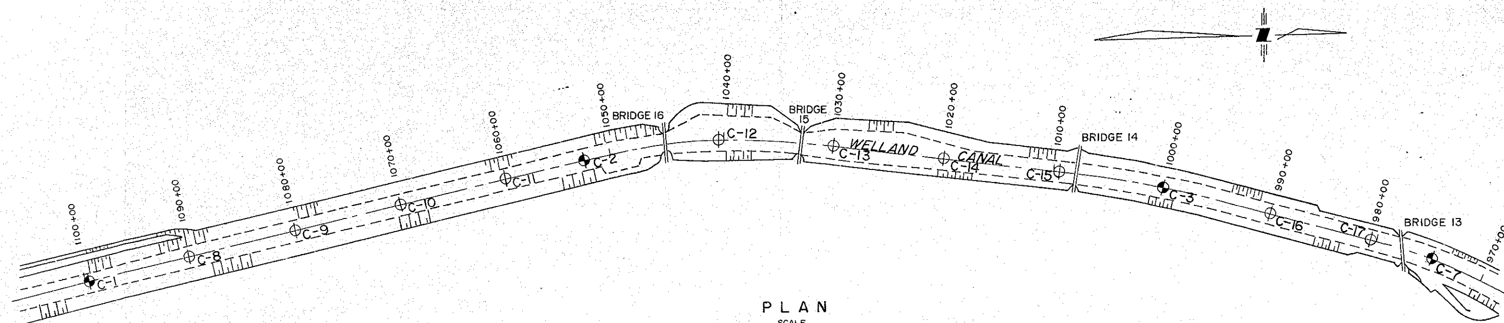
KING'S HIGHWAY NO. —— DIST. NO. 4

CO. WELLAND

TWP. CROWLAND & THOROLD LOT —— CON. ——

BORING PLAN AND SOIL STRATIGRAPHY

SUBM'D. B.T.D.	CHECKED B.T.D.	W.P. NO. ——	DRAWING NO. 1
DRAWN J.A.	CHECKED V.M.	JOB NO. 67028	
DATE MAY 24, 1967		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	



PLAN

SCALE

500 250 0 500 1000 FT.

SEE FIGURE No. 1

KEY PLAN

SCALE IN MILES

LEGEND

Bore Hole

Cone Penetration Hole

Bore & Cone Penetration Hole

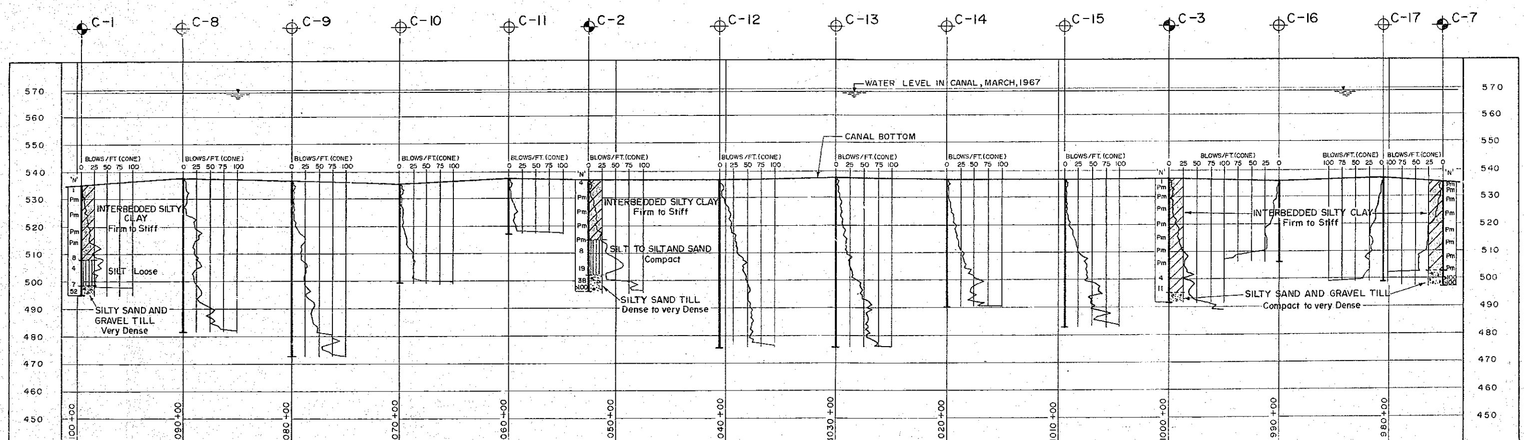
Water Levels established at time of field investigation.

ELEVATIONS REFER TO CANAL BOTTOM AT BOREHOLE LOCATIONS

NO.	ELEVATION	STATION	OFFSET
C-1	534.9	1099+60	℄
C-2	536.9	1052+50	"
C-3	536.4	1000+00	"
C-7	535.1	975+00	"
C-8	537.5	1090+00	"
C-9	536.5	1080+00	"
C-10	535.5	1070+00	"
C-11	537.3	1060+00	"
C-12	536.5	1040+50	"
C-13	537.5	1030+00	"
C-14	536.7	1020+00	"
C-15	536.7	1009+50	"
C-16	535.7	990+00	"
C-17	536.7	980+50	"

– NOTE –

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



SECTION ALONG CENTRELINE OF CANAL

SCALE

500 250 0 500 1,000 FT.

PRINT RECORD		
NO.	FOR	DATE

REVISIONS		
DATE	BY	DESCRIPTION

H. Q. GOLDER & ASSOCIATES LTD.

DEPARTMENT OF HIGHWAYS – ONTARIO

MATERIALS & TESTING DIVISION – FOUNDATION SECTION

WELLAND CANAL

STA. 970+00 TO STA. 1100+00

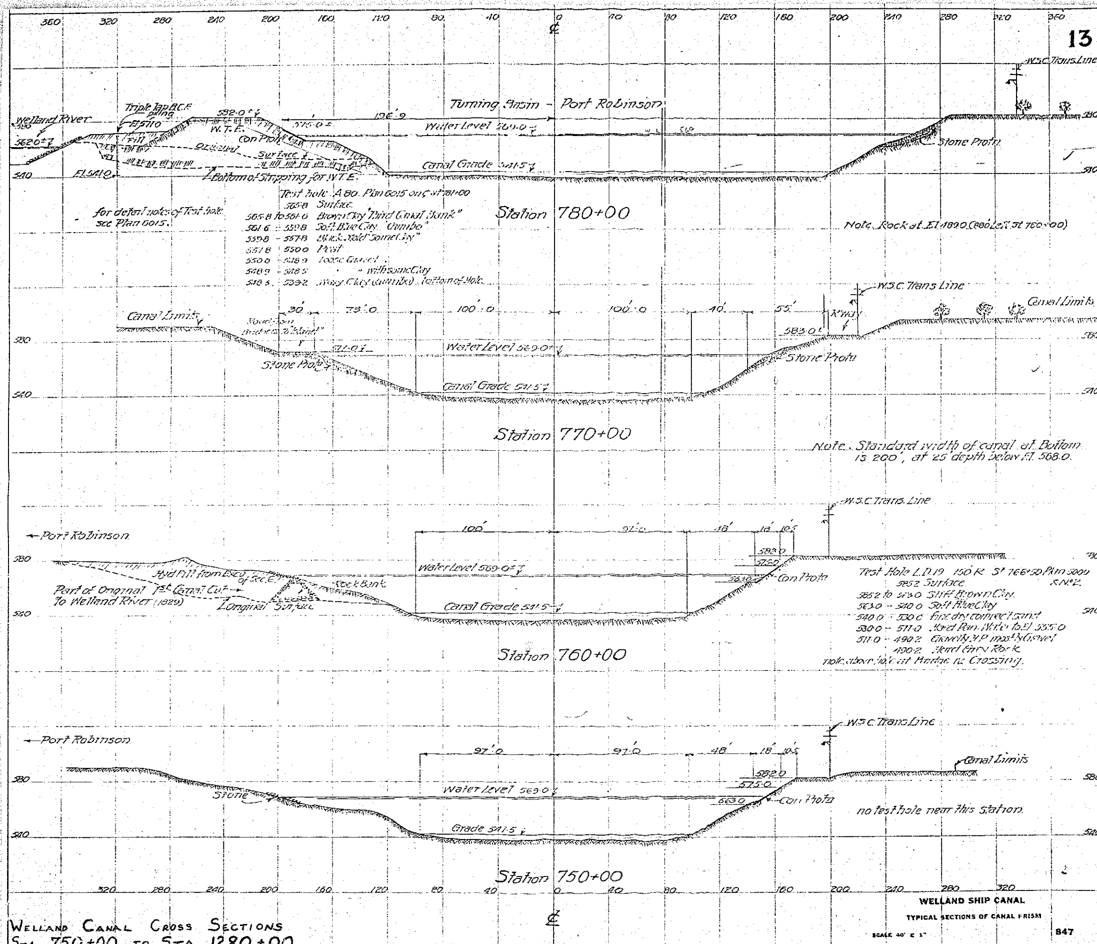
KING'S HIGHWAY NO. ——— DIST. NO. 4

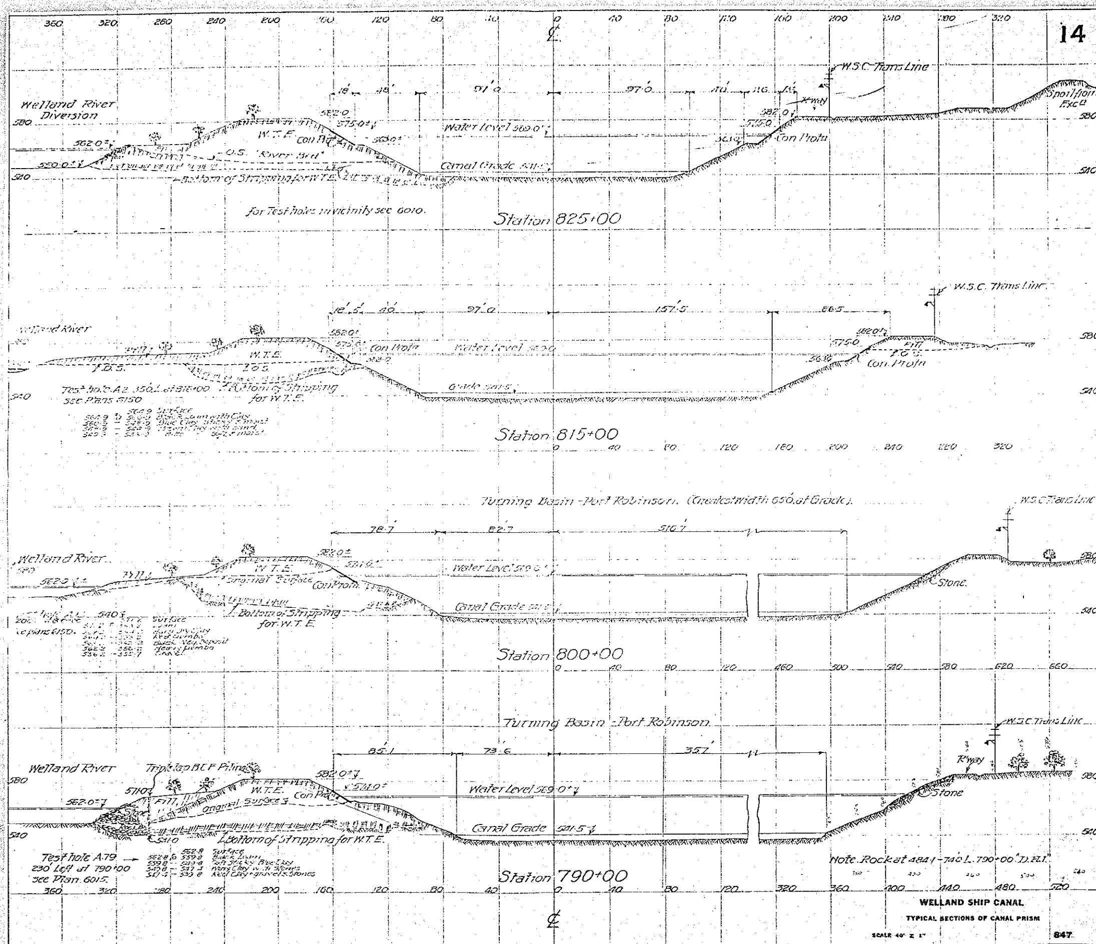
CO. WELLAND

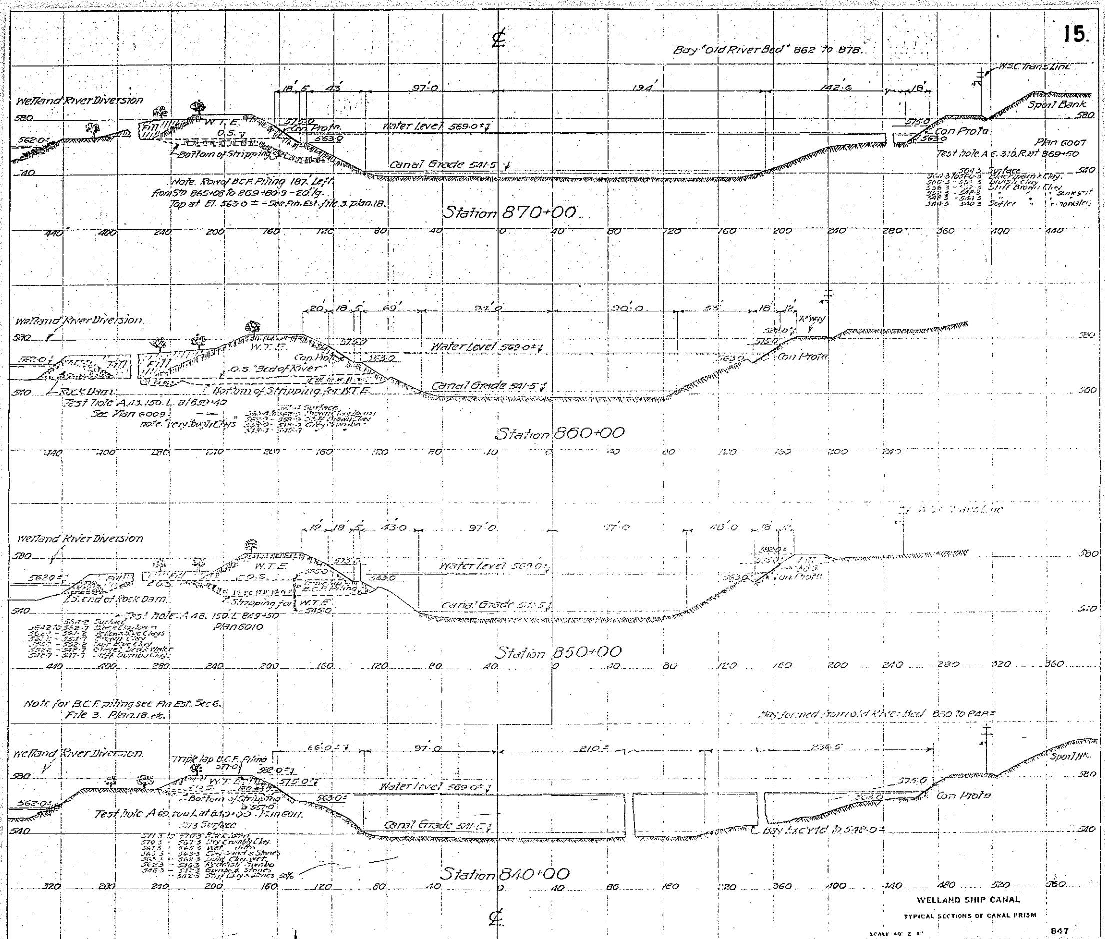
TWP. CROWLAND & THOROLD LOT ——— CON. ———

BORING PLAN AND SOIL STRATIGRAPHY

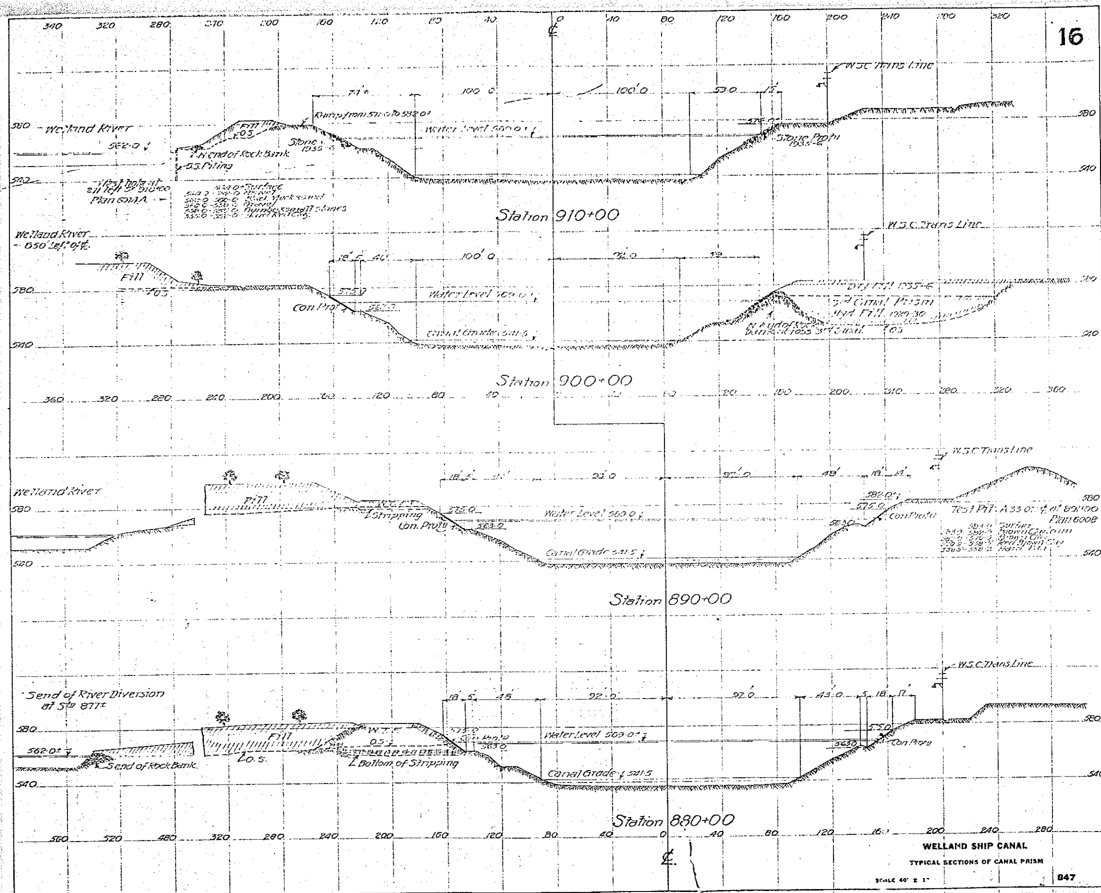
SUBM'D. B.T.D.	CHECKED B.T.D.	W.P. NO. ———	DRAWING NO. 2
DRAWN J.A.	CHECKED V.M.	JOB NO. 67028	
DATE MAY 19, 1967		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	


13
Turning Basin - Port Robinson
Welland River
Triple Tap R.C.F. piling
W.T.F.
Con Prot.
Water Level 569.0
Canal Grade 541.5
Stone Prot.
W.S.C. Trans Line
Bottom of Stripping for W.T.E.
Test hole A 80. Plan 6015 on ℄ st 781+00
for detail notes of Test hole see Plan 6015.
Station 780+00
Note. Rock at El. 489.0 (80' Left St 760+00)
Canal Limits
Stone Prot.
Canal Grade 541.5
Station 770+00
Note. Standard width of canal at Bottom is 200', at 25' depth below El. 568.0.
← Port Robinson
Hyd Fill from Exc of Sec E
Rock Bank
Part of Original 1st Canal Cut - To Welland River (1829)
Longitud Surface
Water Level 569.0
Canal Grade 541.5
Con Prot.
Station 760+00
Test Hole L.D.19 150 R St 766+50 Plan 5009
← Port Robinson
Stone
Water Level 569.0
Grade 541.5
Con Prot.
Canal Limits
no test hole near this station.
Station 750+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847
Welland Canal Cross Sections
Sta. 750+00 to Sta. 1280+00

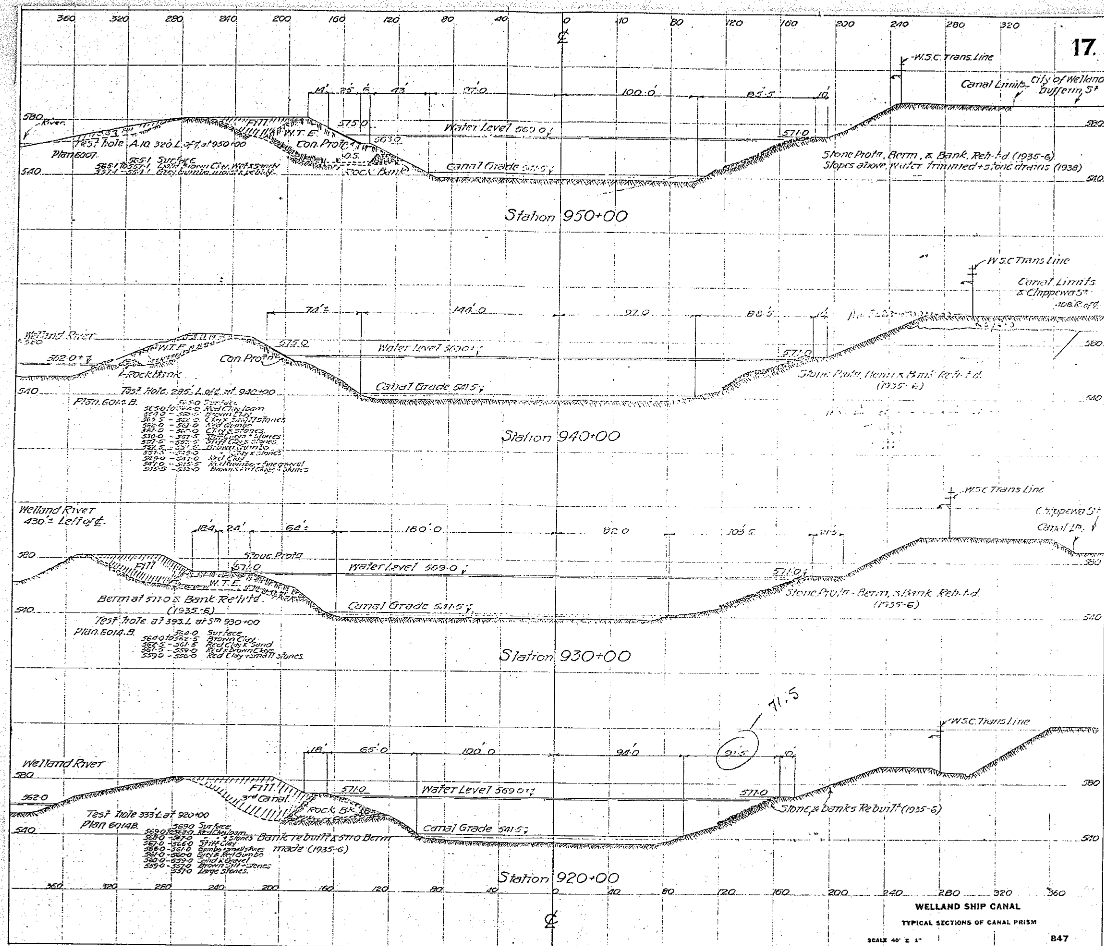

14
Welland River Diversion
W.S.C. Trans Line
Spoil from Exc^n
Water Level 569.0
Canal Grade
Con Profn
W.T.E.
"River Bed"
Bottom of Stripping for W.T.E.
For Test holes in vicinity see 6010.
Station 825+00
Welland River
Station 815+00
Turning Basin - Port Robinson. (Greatest width 650, at Grade).
Original Surface
Station 800+00
Turning Basin - Port Robinson.
Triple lap B.C.F. Piling
Test hole A79
230' Left of 790+00
see Plan 6015.
Note: Rock at 484 - 740 L. 790+00 D.H.
Station 790+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847



15.
Bay "Old River Bed" 862 to 878.
Welland River Diversion
W.T.E.
Con Prot.
Water Level 569.0±
Canal Grade 541.5
Bottom of Stripping
Spoil Bank
Plan 6007
Test hole A.E. 316 R at 869+50
Note. Row of B.C.F. Piling 18' Left from Sta. 865+08 to 869+80.9 - 20' lg. Top at El. 563.0± - See Fin. Est. file 3. plan 18.
Station 870+00
O.S. "Bed of River"
Rock Dam
Test hole A.13. 150 L at 859+40
See Plan 6009
Station 860+00
End of Rock Dam
Test hole A.48. 150 L 849+50
Plan 6010
Station 850+00
Note for B.C.F. piling see Fin Est. Sec 6. File 3. Plan 18 etc.
Bay formed from old River Bed 830 to 848±
Triple lap B.C.F. Piling
Test hole A 69. 100 L at 840+00 Plan 6011.
Bay Excv'td to 548.0±
Station 840+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847

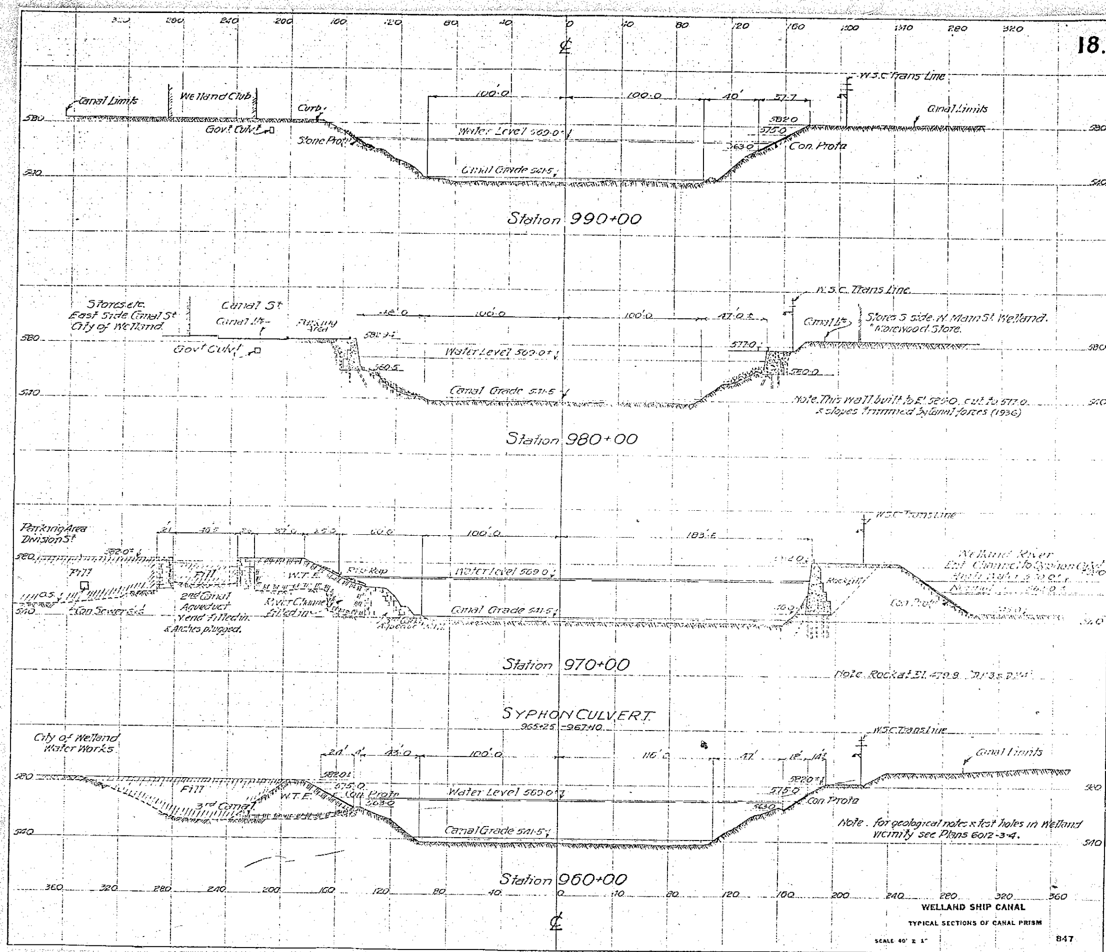


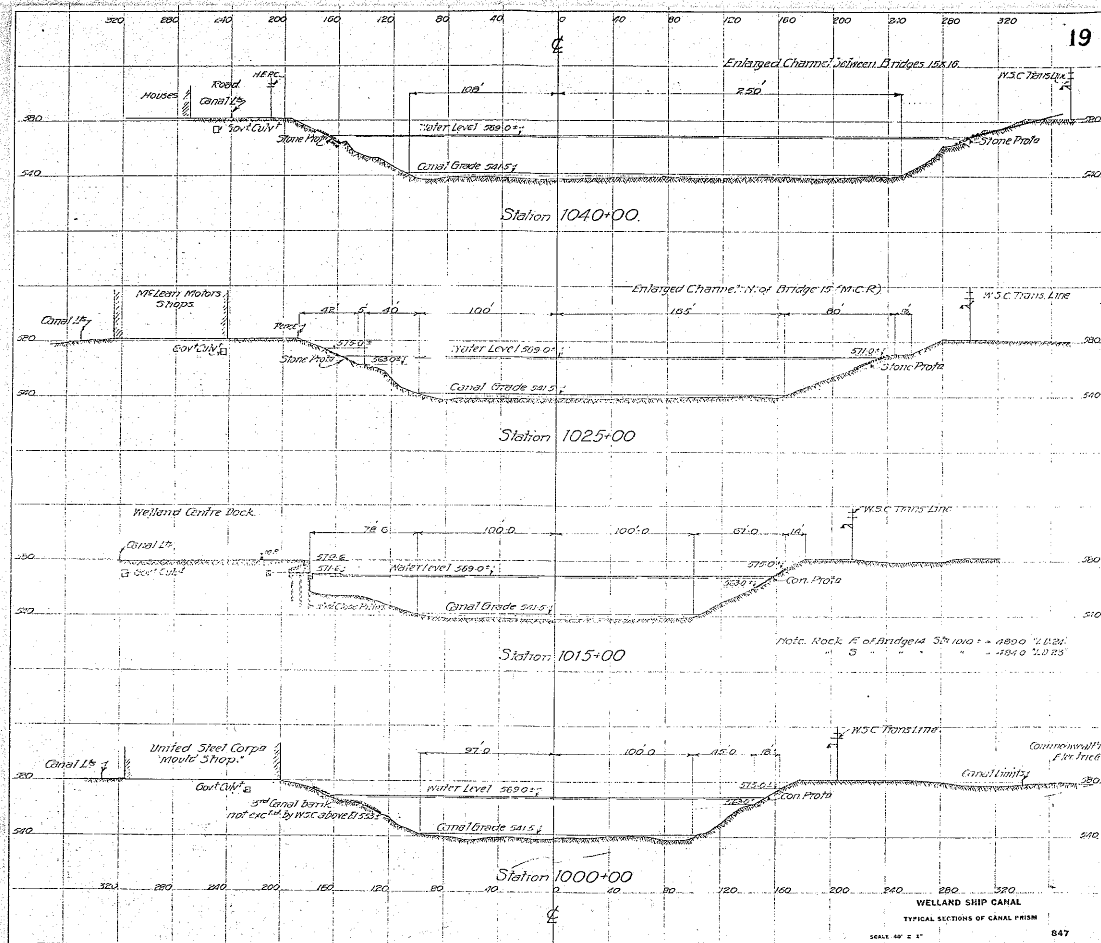
16
Welland River
562.0
End of Rock Bank
O.S. Piling
Pump from 571.0 to 582.0
Water Level 569.0
Stone Protn 1935-6
W.S.C. Trans Line
Station 910+00
Welland River
650' Left of ℄
Fill
Con. Protn
Water Level 569.0
Canal Grade 541.5
3rd Canal Prism
Station 900+00
Welland River
Fill
Stripping
Con. Protn
Water Level 569.0
Canal Grade 541.5
Test Pit A 33 of ℄ at 891+00
Plan 6008
Station 890+00
S. end of River Diversion at Sta 877±
S. end of Rock Bank
Fill
Bottom of Stripping
Water Level 569.0
Canal Grade 541.5
Con. Protn
Station 880+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847



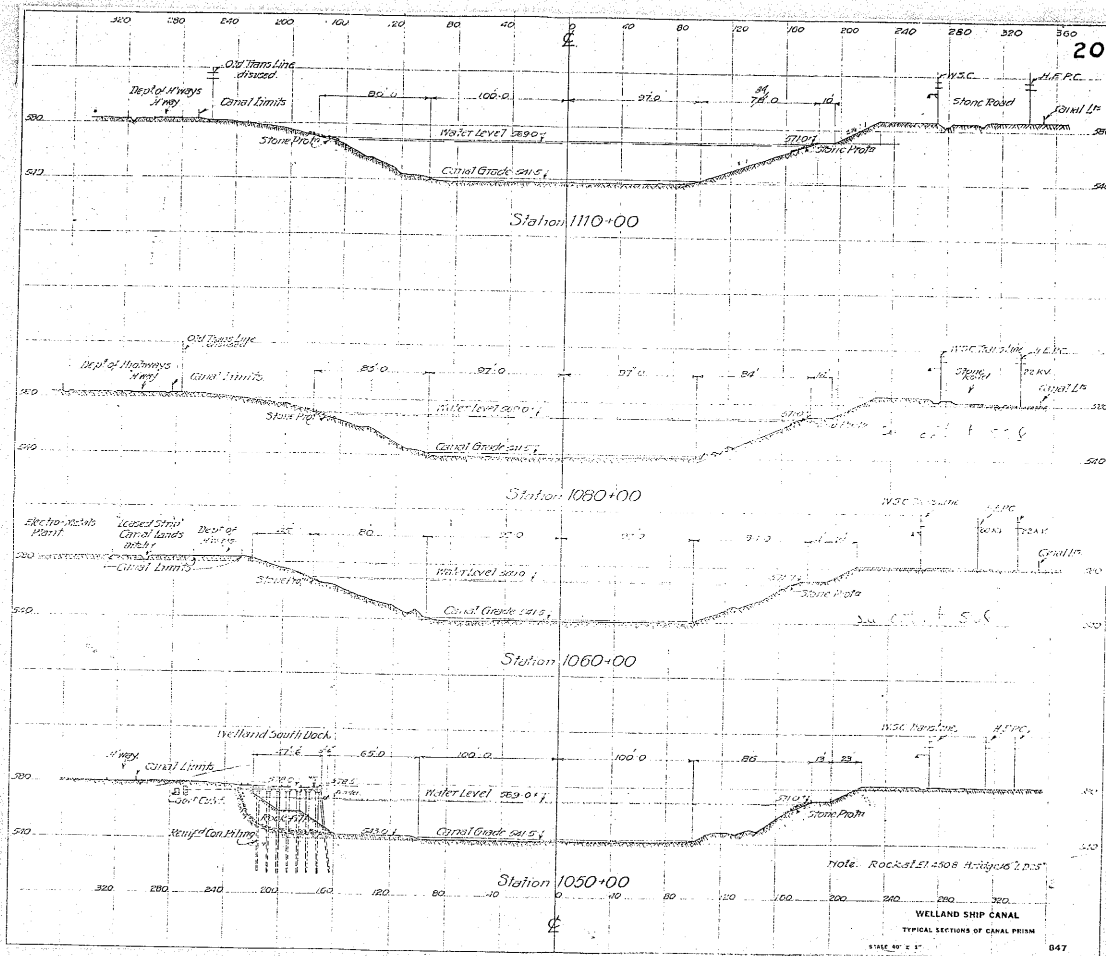
SOME DEFECTS IN NEGATIVE DUE

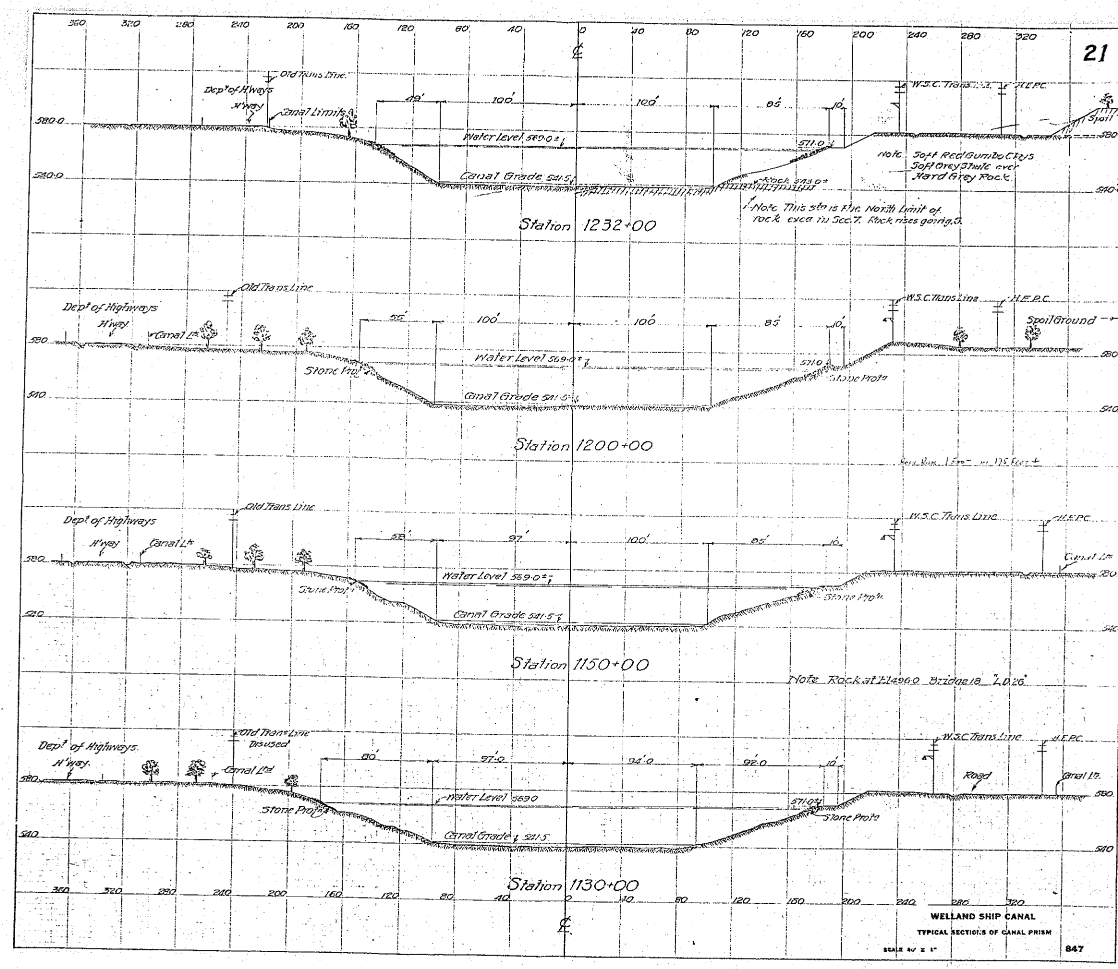
TO CONDITION OF ORIGINAL DOCUMENTS

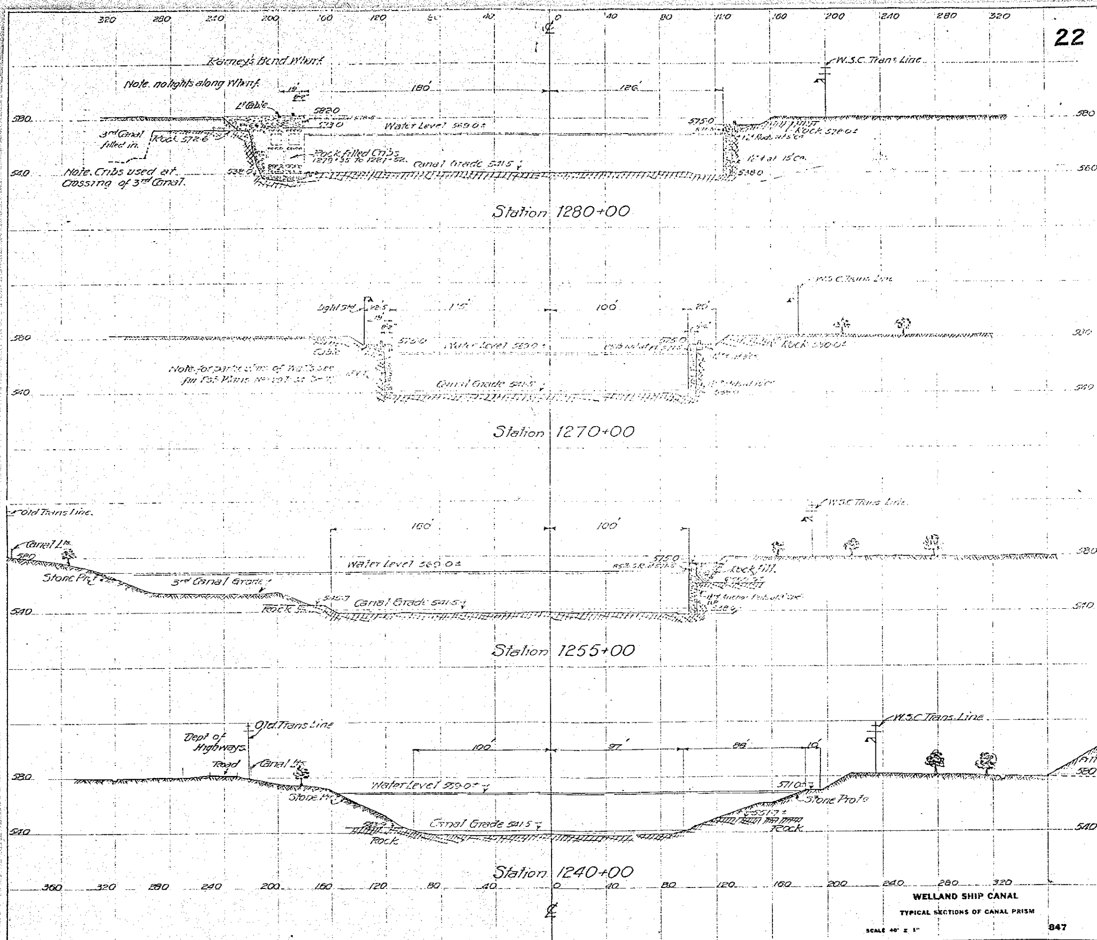

Canal Limits
Welland Club
Curb
Gov.t Culv.t
Stone Prot.
W.S.C. Trans Line
Con. Prota
Water Level 569.0
Canal Grade 541.5
Station 990+00
Stores etc.
East Side Canal St
City of Welland
Canal St
Parking Area
Gov.t Culv.t
Water Level 569.0
Canal Grade 541.5
W.S.C. Trans Line
Stores S side N. Main St Welland
Station 980+00
Parking Area
Division St
Fill
2nd Canal
Aqueduct
& Arches plugged.
River Channel
W.T.E.
Rip Rap
Water Level 569.0
Canal Grade 541.5
W.S.C. Trans Line
Welland River
Con. Prot.
Station 970+00
SYPHON CULVERT
965+25 - 967+10
City of Welland
Water Works
Fill
3rd Canal
W.T.E.
Con. Prota
Water Level 569.0
Canal Grade 541.5
W.S.C. Trans Line
Canal Limits
Con. Prota
Note. for geological notes & test holes in Welland vicinity see Plans 6012-3-4.
Station 960+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847

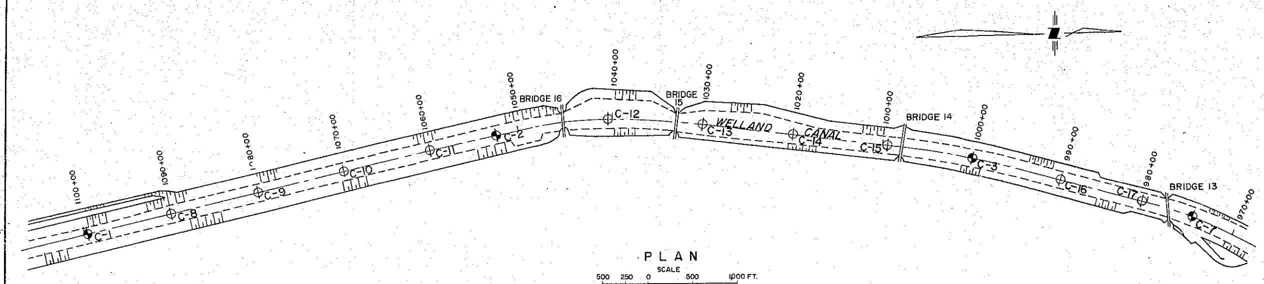


19
Enlarged Channel between Bridges 15 & 16.
Houses
Road
Canal Lt.
H.E.P.C.
Gov't Culv't
Stone Prot'n
Water Level 569.0±
Canal Grade 541.5±
W.S.C. Trans. Line
Station 1040+00.
Enlarged Channel N. of Bridge 15 (M.C.R.)
McLean Motors Shops
Canal Lt.
Gov't Culv't
Water Level 569.0±
Canal Grade 541.5±
Stone Prot'n
Station 1025+00
Welland Centre Dock
Canal Lt.
Gov't Culv't
Water Level 569.0±
Canal Grade 541.5±
Con. Prot'n
W.S.C. Trans. Line
Station 1015+00
United Steel Corp'n "Mould Shop."
Canal Lt.
Gov't Culv't
3rd Canal bank not exc'td by W.S.C. above El. 553±
Water Level 569.0±
Canal Grade 541.5±
Con. Prot'n
W.S.C. Trans. Line
Commonwealth Electric
Canal Limits
Station 1000+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
SCALE 40' = 1"
847


20
Old Trans Line disused
Dept of H'ways H'way
Canal Limits
W.S.C.
H.E.P.C.
Stone Road
Canal Lts
Water Level 569.0
Stone Profe
Canal Grade 541.5
Station 1110+00
Old Trans Line disused
Dept of Highways H'way
Canal Limits
Water Level 569.0
Canal Grade 541.5
Station 1080+00
Electro-Metals Plant
"Leased Strip" Canal Lands
Canal Limits
Water Level 569.0
Canal Grade 541.5
Station 1060+00
Welland South Dock
H'way
Canal Limits
Rock Fill
Reinf'd Con. Piling
Water Level 569.0
Canal Grade 541.5
Stone Profe
Note. Rock at El. 550.8 Bridge 16 L.D.S.
Station 1050+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847


21
Station 1232+00
Water Level 569.0±
Canal Grade 541.5
Canal Limits
Old Trans Line
Dept of H'ways
H'way
W.S.C. Trans.
H.E.P.C.
Spoil
Note. Soft Red Gumbo Clays
Soft Grey Shale over
Hard Grey Rock.
Note This sta is the North Limit of
rock exca in Sec.7. Rock rises going S.
Station 1200+00
Dept of Highways
H'way
Canal Lt
Old Trans Line
W.S.C. Trans Line
H.E.P.C.
Spoil Ground
Stone Prot
Water Level 569.0±
Canal Grade 541.5
Station 1150+00
Note Rock at El.496.0 Bridge 18 "L.D.26"
Station 1130+00
Old Trans Line
Disused
Road
Water Level 569.0
Canal Grade 541.5
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847


22
Barney's Bend Wharf
Note. no lights along Wharf
W.S.C. Trans Line
Water Level 569.0±
Canal Grade 541.5±
3rd Canal filled in
Rock filled Cribs
Note. Cribs used at Crossing of 3rd Canal.
Station 1280+00
Station 1270+00
Old Trans Line
Canal Lt.
Stone Pr.
3rd Canal Grade
Rock Fill
Station 1255+00
Dept of Highways Road
Old Trans Line
Stone Prot.
Rock
Station 1240+00
WELLAND SHIP CANAL
TYPICAL SECTIONS OF CANAL PRISM
847



PLAN

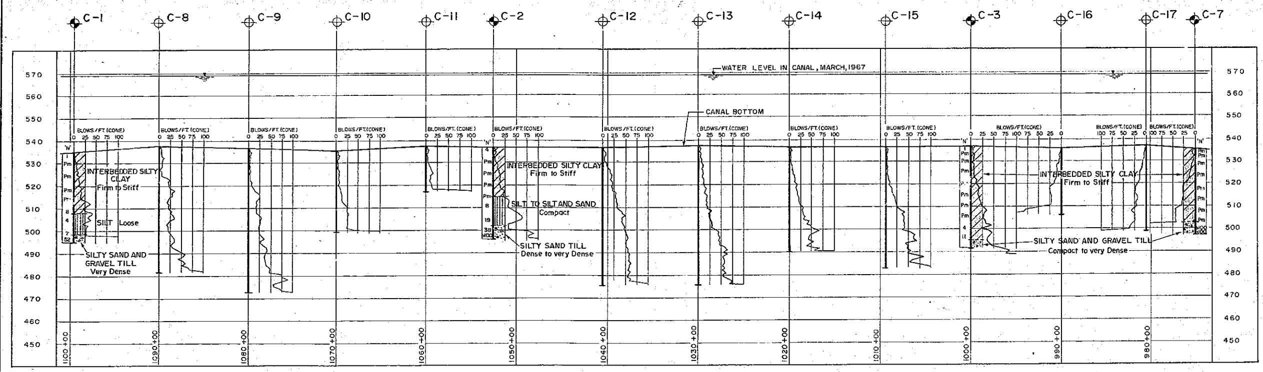
SCALE

500 250 0 500 1000 FT.

SEE FIGURE No. I

KEY PLAN

SCALE IN MILES



SECTION ALONG CENTRELINE OF CANAL

SCALE

500 250 0 500 1,000 FT.

30L14-16

GEOCRES No.

PRINT RECORD

NO.	FOR	DATE

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation.

ELEVATIONS REFER TO CANAL BOTTOM AT BOREHOLE LOCATIONS

NO.	ELEVATION	STATION	OFFSET
C-1	534.9	1099+60	℄
C-2	536.9	1052+50	"
C-3	536.4	1000+00	"
C-7	535.1	975+00	"
C-8	537.5	1090+00	"
C-9	536.5	1080+00	"
C-10	535.5	1070+00	"
C-11	537.3	1060+00	"
C-12	536.5	1040+50	"
C-13	537.5	1030+00	"
C-14	536.7	1020+00	"
C-15	536.7	1009+50	"
C-16	535.7	990+00	"
C-17	536.7	980+50	"

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

30L14-16

GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION

H. Q. GOLDER & ASSOCIATES LTD.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION - FOUNDATION SECTION

WELLAND CANAL

STA. 970+00 TO STA. 1100+00

KING'S HIGHWAY NO. ——— DIST. NO. 4

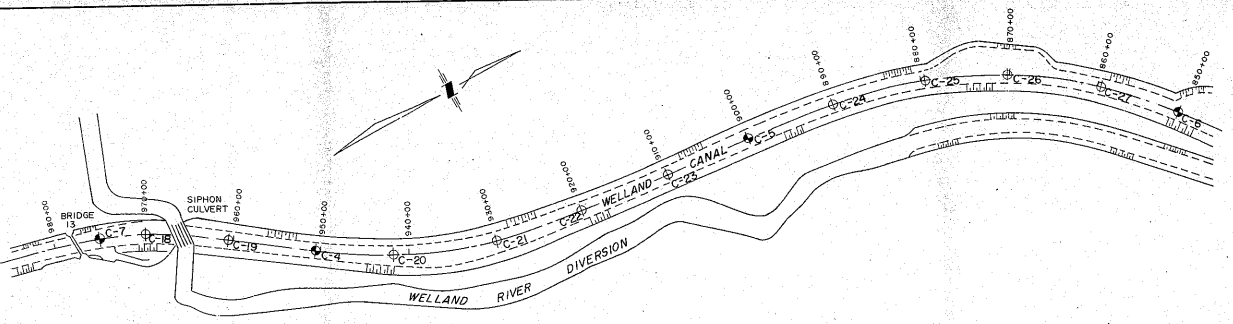
CO. WELLAND

TWP. CROWLAND & THOROLD LOT —— CON. ——

BORING PLAN AND SOIL STRATIGRAPHY

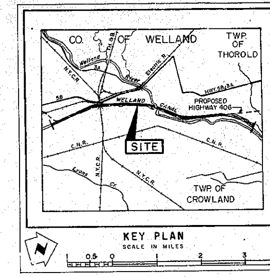
SUBM'D. B.T.D.	CHECKED B.T.D.	W.P. NO. ——	DRAWING NO. 2
DRAWN J.A.	CHECKED V.M.	JOB NO. 67028	
DATE MAY 19, 1967		SITE NO.	BRIDGE DRAWING NO.
APPROVED ________ PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS



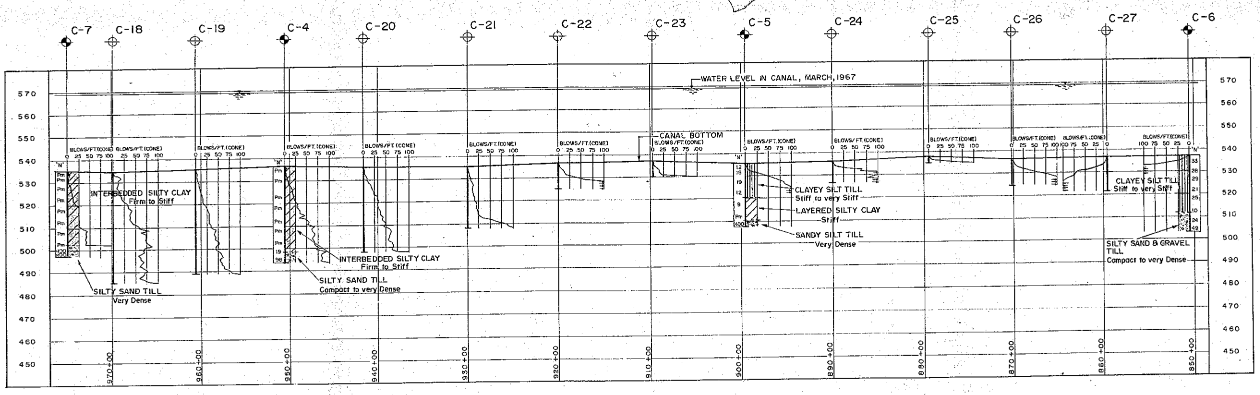
PLAN

SCALE 500 250 0 500 1,000FT.



KEY PLAN

SCALE IN MILES 0.5 0 1 2 3



SECTION ALONG CENTRELINE OF CANAL

SCALE 500 250 0 500 1,000FT.

30L14-16
GEOCRES No.

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation.

ELEVATIONS REFER TO CANAL BOTTOM AT BOREHOLE LOCATIONS

NO.	ELEVATION	STATION	OFFSET
C-4	536.8	950+50	℄
C-5	535.9	899+50	"
C-6	537.3	850+50	"
C-7	535.1	975+00	"
C-18	534.7	969+90	"
C-19	535.7	960+50	"
C-20	536.0	941+60	"
C-21	535.5	930+00	"
C-22	536.6	920+00	"
C-23	537.3	909+75	"
C-24	536.2	890+00	"
C-25	538.1	879+50	"
C-26	536.5	870+15	"
C-27	537.1	859+60	"

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

30L14-16
GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION

PRINT RECORD

NO.	FOR	DATE

H. Q. GOLDER & ASSOCIATES LTD.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

WELLAND CANAL

STA. 850+00 TO STA. 980+00

KING'S HIGHWAY NO. —— DIST. NO. 4
CO. WELLAND
TWP. CROWLAND & THOROLD LOT —— CON. ——

BORING PLAN AND SOIL STRATIGRAPHY

SUBM'D B.T.D.	CHECKED B.T.D.	W.P. NO. ——	DRAWING NO. 1
DRAWN J.A.	CHECKED V.M.	JOB NO. 67028	
DATE MAY 24, 1967		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS