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LOCATION MISSISSAUGI RIVER BANK

STABILIZATION, REMEDIAL MEASURES

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



## **Golder Associates**

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT  
TO

MINISTRY OF TRANSPORTATION AND  
COMMUNICATIONS

REVIEW OF RIVER BANK INSTABILITY  
MISSISSAGI RIVER  
IRON BRIDGE TO MISSISSAGI CHUTES  
WP 153-83-00                      DISTRICT 18  
PHASE II

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### SUMMARY

The results of an investigation of the causes of instability of the banks along part of the Mississagi River adjacent to Highway 17 are reported.

Erosion of the river banks along the Mississagi River downstream of Iron Bridge has become particularly evident since about 1960. There is ample local evidence that the erosion rate has markedly increased since that date, which marks the commencement of operation of Red Rock Falls Generating Station. A review of daily water level records prior to 1960 indicates consistently high water levels in the Spring of each year, markedly lesser flows over the rest of the year, particularly in the Summer and gradual changes in water level between these extremes. Comparison of these records with those after generation commenced show the same general pattern over the year, but with the significant difference that comparative daily levels are much more erratic in pattern and the changes in water level are abrupt; variations in water level of 2 m to 3 m take place in a relatively short period, say of several days. The largest and most frequent changes occur in the winter months where, prior to regulation, the least variation in river water level took place.

The pattern of abrupt daily changes in water level throughout the year, except for periods of high flood flows, is typical of river conditions downstream of hydro-electric dams. It leads to several effects:

- i) river velocities can change from below to well above critical scour values on a daily basis, thus resulting in increased erosion, both of steep banks in erodible materials and of material resulting from shallow slumps or failures;
- ii) following a large drop in river water level, high piezometric levels exist in the cohesive materials at the toe of the slope causing shallow rotational failures of the bank;
- iii) repeated fluctuations aggravate erosion of the slope toe and undermining of the loose granular materials comprising the upper part of the slope thus leading to surficial sliding of these materials.

At present, the river banks are standing at relatively steep angles and are only marginally stable under "normal" river flow conditions. Stability analyses show that the factor of safety against failure can be decreased by up to 25 per cent with a 1 m lowering of river water level. The rapid river water level changes recorded are sufficient to induce failures.

To only a limited extent, certain of the bank erosion observed since 1960 is inevitable as it is related to Spring flooding conditions, irrespective of whether or not river flows are regulated by the dam; however, to a much greater extent, the fluctuations in discharge downstream of the dam have induced increased erosion and scour and Hydro's operations are apparently responsible for this.

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- Unregulated River

- Regulated River

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## INTRODUCTION

This report presents the results of a subsurface investigation carried out along a section of the Mississagi River adjacent to Highway 17 and provides a discussion on the causes of instability of the river banks. The area of interest is shown on Figure 1 and extends from about 4 km south of the Town of Iron Bridge to Mississagi Chutes some 15 km downstream - Ministry of Transportation and Communications chainages 11+850 (Thompson Township) to 14+750 (Cobden Township).

Previous reports on this instability have been prepared by Ontario Hydro (Hydro) and by the Ministry of Transportation and Communications (MTC). These reports, together with other available data, were reviewed as part of our preliminary study, (Report No. 841-1356, dated January, 1985) which presented our preliminary opinion on the causes of instability of the Mississagi River banks. These reports, listed in Appendix A, should be read in conjunction with this report.

Our preliminary report outlined steps to be taken in order to present a more detailed assessment of the river bank conditions existing at present, and to provide a more accurate appraisal of the variation in river flow conditions. These steps have been completed as listed below:

- o a visual inspection of the river banks was carried out to identify types and location of instability;
- o boreholes were put down at four representative sites to establish the subsurface conditions. Piezometers were installed in all boreholes to determine piezometric levels within the various strata. The piezometers were monitored during the course of the field work and at the time of a subsequent site visit;



- o water depths and river flow velocities were measured to establish detailed profiles at six locations;
- o a backwater analysis was carried out to establish the hydraulic impact of river water level fluctuations along the length of the river within the study area;
- o an historical aerial photography interpretation was completed.

#### CHRONOLOGY OF PAST INSTABILITY

In the late 1950's, Hydro constructed the Red Rock Falls Generating Station on the Mississagi River, some 7 km upstream from the Town of Iron Bridge. This facility was placed in service in 1960 and generally operated as a peaking station until 1980. Throughout the 1960's and 1970's, Ontario Hydro received complaints from residents in the local area about widely varying river levels and an apparent increase in the rate of erosion of the river banks. Water level fluctuations of 1.8 m daily and as much as 3.4 m on one occasion had been recorded, (i).

As a result of flooding and severe erosion caused by an exceptional spring freshet flow in 1979, the residents of Iron Bridge demanded action from Hydro and in 1980, Hydro modified operations at Red Rock Falls Generating Station to limit daily fluctuations to 0.6 m with occasional higher fluctuations caused by the operation of other power stations upstream. From 1977 to 1982, Hydro undertook engineering and environmental studies to determine the extent and causes of the erosion problem, to assess their responsibility and to determine possible means of reducing erosion. The results of those studies are given in their report entitled "Mississagi River Erosion Control Program - Environmental Assessment" (i).

Air photos show Highway 17 essentially in its present alignment since at least 1947 (xiii). Contract documents prepared in 1960 and 1984 (ix) for work on Highway 17 show that the highway construction has not encroached on the river since Red Rock Falls Generating Station was built.

#### SUBSURFACE CONDITIONS

A subsurface investigation and inspection of the Mississagi River banks was carried out between May 21 and 25, 1985. A series of boreholes were extended at four sites which, based on our preliminary assessment of the available information, were considered to reflect the range of slope and subsurface conditions within the study area. River bed profiles and velocity measurements were made at each of these sites, as well as at two additional stations. The site locations are shown on the site plan, Figure 1, and on the air photo mosaics, Figures 2 to 5, inclusive.

The detailed subsurface conditions encountered in each of the borings, together with the results of laboratory tests carried out on representative samples, are given on the attached Record of Borehole sheets and on Figures 6 to 8. Cross-sections showing the stratigraphy, river bank profile and piezometer installations at each site are shown on the air photo mosaics, Figures 3 to 5, inclusive.

In general, the soils comprising the river banks consist of layered/interlayered sands, silts, organic silts and clayey silts. Grain size distribution curves for these strata are shown on Figures 6 to 8, inclusive.

The sands and silts generally form the upper part of the river banks. These granular materials are in a very loose to loose state of packing with 'N' values\* ranging from

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\* 'N' Values - Standard Penetration Resistance  
Refer to List of Abbreviations and Symbols

$K = 100 (0.012)^2$   
 $= 1.44 \times 10^{-2} \text{ m/sec}$   
 $4 \times 10^{-3} \text{ m/sec}$

## Groundwater Measurements

## Golder Associates

At the time of a site visit in July, 1985, the river water level was at about elevation 178 m (about 1 m lower than in May) as shown on Figure 9, and a corresponding drop was measured in the piezometric level in the sands and silts. Where piezometers were sealed into the clayey silt stratum, the decrease in piezometric level ranged from 0 to 0.3 m. [Although the piezometer in Borehole 9, Station 12+680, was installed within silts, due to the distance from the river edge (about 14 m), the water level in the piezometer decreased by only 0.1 m.]

#### DESCRIPTION OF RIVER BANK CONDITIONS

The river banks within the study area range in height from about 3 m to 7 m (exposed heights generally 1 to 6.5 m) and are standing at slope angles ranging from about 25 to 45° to the horizontal. In general, the soils comprising the river banks consist of layered/interlayered sands, silts, organic silts and clayey silts. Except where recent (this year) failures have occurred, the slopes are generally grassed and, at some locations, are vegetated with mature trees.

Air photograph interpretation carried out using photographs taken in 1980 is included as Appendix C of this report. A substantial proportion of the length of river bank studied, approximately 25 per cent, has been classified as containing existing failures of Types No. 1a and 1b as defined in Appendix C.

An historical aerial photograph interpretation was carried out for the years 1947, 1957, 1970 and 1980 and is included in Appendix D of this report. Six reaches of the river were chosen for the study as being representative of the varying conditions. Generally, about 10 per cent of each stretch of the river bank has visible signs of failure in 1947 and 1957. In the year 1970, the percentage of failed bank had increased to about 30 to 50 per cent and had remained

constant or decreased in the year 1980.

A field mapping study of the river banks was carried out by Ontario Hydro in 1980 (vii). Cross-sections are provided showing exposed bank materials, slope height and gradient generalized over short reaches of the river and erosion for each reach is classified as minor, moderate and high. Overall, the study placed about 25 per cent of the river bank in a high erosion category (refer to Appendix E).

The present study consisted, in part, of a bank inspection carried out via boat. The slopes were classified according to the height of exposed bank and to the type of erosion, or, failure mechanism most prevalent along sections of the river bank\*. These field observations, shown on Figures 3 to 5, inclusive, generally confirmed the air photo interpretations and highlighted those areas showing a high frequency of past and present instability. The stratigraphy established at the borehole locations is generally consistent with the Hydro study on exposed river bank soils. [However, the presence of the organic silty materials near the toe of the slopes was not noted.]

Both old and recent, surficial sliding and rotational failures were noted all along the river. Erosion of the slope materials at the toe of the exposed slope is ubiquitous. The scars of this toe erosion or scour were particularly evident during the site visit in July, 1985, because the river water level was about 1 m lower (elevation 178 m) than previously.

#### REVIEW OF DATA ON RIVER WATER LEVELS

River level and discharge records have been kept at various points on the Mississagi River for most years since 1915. Figure 1 shows the locations of the various gauging stations, and their reference numbers. Federal authorities have

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\* The classification system shown on Figures 3, 4, 5 throughout the report and is defined in Figure 25.

records at Iron Bridge (02CC002) for the years 1915-1919; at Mississagi Bridge (02CC004) for the years 1920-1950; at Mississagi Chutes (02CC008) for the years 1961 to present. Since 1960, Ontario Hydro has maintained a station (02CC009) at Red Rock Falls Generating Station which reports to the federal authorities. In addition, Ontario Hydro has operated independent gauging stations at Iron Bridge and at Mississagi Bridge on a semi-regular basis since 1979.

The gauging stations installed since about 1960 are automatic and record a continuous plot of water levels. Manual observations were kept on a once daily basis at stations operated prior to about 1960.

Daily water level readings for the years 1927, 1928, 1937, 1947, 1970 and 1979 have been presented on Figures 10 to 14. Maximum daily flow conditions were recorded in 1928; the years 1927, 1937 and 1947 were chosen at random to represent typical years prior to operation of the generating station (Figure 10a); the year 1979 was selected because it contained severe spring freshet flow; the years 1970 and 1979 also reflect typical daily fluctuations caused by the method of operating the generating station prior to 1980 (Figure 10b).

When the daily mean water levels, prior to river regulation (Figure 10a), are compared with the daily median water levels for 1970 and 1979 (Figure 10b)\*, it is apparent that the graphs for the 1970's are jagged, while the graphs for 1927, 1928, 1937 and 1947 are relatively smooth. This is also evident in comparing Figures 11a to 14a with Figures 11b to 14b.

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\* It should be noted that Figure 10a represents single daily readings while Figure 10b represents daily median water levels (the arithmetic average of the highest and lowest of twenty-four hourly water level observations). Consequently, the numbers plotted in the figures are not identical, but are sufficiently similar to allow meaningful comparison.

Further, in comparing Figures 11a to 14a with Figures 11b to 14b, it is apparent that the only time of year when the graph of the 1970 and 1979 river levels approaches the characteristic smoothness of the unregulated river is in the months when the river is at flood stage, beyond the effect of regulation. This is also demonstrated in the plots of recorded daily high and low water levels shown on Figures 15 to 18, inclusive.

In the unregulated river regime, as is typified by years PRIOR to the dam and as shown in Figure 10a, day to day changes of more than 0.2 m are relatively infrequent (see Figure 19). In contrast, in the regulated river, (AFTER dam construction), water level changes within a 24 hour period of up to 0.5 m are commonplace. It should be noted, as shown in Figures 20 and 21, that the largest and most frequent changes in 1970 and 1979 occurred in the winter months where, prior to regulation (as in 1927, 1928, 1937, 1947) the least variation in river water level took place.

#### RIVER CHANNEL CHARACTERISTICS AND WATER VELOCITIES

Figures 22 and 23 show cross-sections of Mississagi River at various locations within the study area.

The profile shown in Figure 22 (near Mississagi Chutes) was obtained from measurements made by the staff of Water Survey of Canada on April 22, 1970. The velocity contours have been produced from 50 spot velocity measurements taken during the profiling.

The river profiles and velocity contours shown on Figure 23 were derived from measurements taken during the course of the field work for the present study. The number of spot velocity measurements varied (depending on depth and breadth of the

river) from 22 to 47 per site. The mean velocities estimated at each site ranged from about 0.4 to 0.6 m/sec for the day of measurement. The highest mean velocity occurred at Station 17+930 (Thompson) while the lowest was at Station 13+500 (Cobden).

The cross-sections and velocity measurements were incorporated in the computer program HEC-2 which was used to simulate the hydraulic conditions of the river. The model was calibrated using both the calculated total channel flows and using the stage-discharge relationships obtained from Water Survey of Canada for Mississagi Chutes and Dean Lake Bridge at the downstream end and near the upstream end of the study reach, respectively. The analysis confirmed that the river geometry is such that changes in flow produce corresponding changes in river water level along the full reach of the study area. It was also possible to establish a relationship of river velocity with respect to river water elevation at each of the profile sites. The relationships are shown on Figure 23 and show that relatively small changes in river level greatly affect the river water velocity.

#### Significance of Water Velocities

Generally speaking, a river reaches a degree of equilibrium with time such that during periods of low flow there is little or no erosion; however, at periods of high flood flows, major erosion of the river banks takes place. At such times, water levels are high and may remain so for some time. It is considered that for non-regulated rivers, most of "all significant bank erosion occurs during major flood events" (Hey et al, 1982). In such cases, the discharge duration of floods may be more important than their magnitude. Other important factors related to erosion are - river channel velocity, channel geometry and soil type or texture.



Empirical relationships between soil median particle size and the "critical erosion velocity"\* are plotted in Figure 24. These relationships, while simplistic to some degree, are reasonably comparable and provide a probable lower bound to the probable critical erosion velocity for cohesionless soils. The lower bound of critical velocity for the fine sands to coarse silts which cover significant areas of the river banks can be estimated from Figure 21 at less than 0.5 m/s. This corresponds to a water level at approximate elevation 179 m. At lower water levels, river velocities are below the estimated critical value, but a rise of 1 m can produce velocities critical for the erosion of even coarse sand sizes or larger.

For cohesive soils, the critical velocities tend to be significantly higher. For example, studies for the Red River floodway around Winnipeg indicated that the critical erosion velocity for firm plastic Lake Agassiz clay, ( $s_u$ , about 50 kPa), was in excess of 1.25 m/sec.

Critical velocities also tend to be higher as the water sediment load increases; thus, for the same conditions of depth and geometry, "clean" water scours at lower velocities than sediment laden water. This is particularly important for the stretch of river below Red Rock Falls dam. The reservoir formed by the dam tends to retain the sediment load carried by the river to that point. The water discharged or spilled over the dam is therefore relatively clear and tends to pick up a load corresponding to that which it had above the reservoir; consequently erosion of the river banks follows. This mechanism explains a large measure of the erosion of river beds and banks below dams. In this context, Lane (1934) gives several examples including that of the Wisconsin River similar to, but wider than Mississagi

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\* "Critical erosion velocity" is that minimum river current velocity just capable of initiating the movement of grains of a given size or diameter.

River (discharge, 240 cumecs; average river slope  $< 0.3$  m per kilometre) where there was marked retrogression of the river bed downstream of a dam amounting to over 2 m in 18 years, the river bed material being medium to coarse sand. There is extensive experience in the literature for other rivers which indicates that such retrogression and bank erosion, developed through operation of regulating dams, can extend for considerable distances downstream amounting to tens of kilometers.

#### RELEVANT CAUSES OF RIVER BANK INSTABILITY

There are three types of instability identified along the banks of the Mississagi River; namely toe erosion, surficial sliding and rotational or block failure; these are illustrated schematically on Figure 25 and photographically on Plates I to IV.

Erosion of the materials at the toe of the slope (I) and removal by water flow causes undercutting and steepening of the slope. If the materials comprising the slope consist of sands and/or silts, this undermining process can cause surficial sliding (II) in which the top/surface layers of soil slide in thin sheets down the banks. Where weak cohesive strata are present at or near the toe of the slope, this steepening process along with removal of the stabilizing material at the toe can effect rotational or block failures (III) which occurs along or through the weaker stratum.

This natural toe erosion process is common to many rivers. An oversteepened slope created by such undercutting can remain stable for some time provided that toe erosion does not persist. With fluctuation in water levels, instability results; such fluctuations are important factors in erosion and consequent river bank instability.

### Significance of Water Level Fluctuations

Continual fluctuations of river water levels over the course of a day or a few days aggravates the toe erosion process markedly more than normal river flow velocities. During normal river flows, slumping of the bank materials during or immediately following flood periods can occur, removing the protective vegetation cover. During the summer, relatively constant water levels allow re-vegetation, therefore minimizing the erosion process during that period of time.

Fluctuations in river levels produce corresponding changes in the piezometric levels in the river banks. The time required for the piezometric level to stabilize with respect to the river level change is dependent on the permeability of the soil. For the river banks in the study area, where granular materials exist, permeabilities vary between about  $10^{-5}$  m/s for clean fine to medium sands and  $10^{-7}$  m/s or less for the layered sands and silts. Rapid decreases in river water level tend to induce "sudden" drawdown failures or in the less pervious deposits of layered sands and silts. Stability analyses carried out for these slopes indicate that a 1 m drop in river water level decreases the factor of safety against failure for the slope by about 25 per cent (Figure 26).

Changes in piezometric levels in cohesive strata (organic silt, and clayey silts) will occur over a longer period of time. This is shown by the water level measurements obtained in July, 1985, after lowering of the river water level by about 1 m from that during the field investigation. Water level records for Mississagi Chutes over that period of time\* were obtained from Water Survey of Canada. These

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\* Note: Water level records stamped "Provisional Records (subject to Revisions)" but are considered adequate for the purpose intended.

records indicate that the river water level lowering occurred gradually over about one month (June 7 to July 9, 1985).

Stability analyses (Sarma method) were carried out for typical slope profiles where sands overlie clayey silts at the toe of the slope. The piezometric level in the clayey silts assumed in the analysis is shown on Figure 26 and for this condition, the results indicate that a 1 m lowering of the river water level could decrease the factor of safety by about 12 per cent.

At present, the river banks are standing at relatively steep angles and are only marginally stable under "normal" river flow conditions. It is considered that the recorded rapid river water level changes are more than sufficient to induce failure.

#### CONCLUSIONS

Erosion of the river banks along the Mississagi River downstream of Iron Bridge has become particularly evident since about 1960. There is ample local evidence that the erosion rate has markedly increased since that date, which marks the commencement of operation of Red Rock Falls Generating Station.

A review of daily mean water level records for the years 1927, 1928, 1937 and 1947, considered to be representative of conditions prior to 1960, indicates consistently high water levels in the Spring of each year coinciding with break-up and Spring flooding, and markedly lesser flows over the rest of the year, particularly in the Summer. Changes in water level between these extremes are relatively gradual with time (see Figure 10a).

Comparison of these records with those of 1970 and 1979, show the same general pattern over the year, but with the significant difference that comparative daily river water levels are much more erratic in pattern and the changes in water level are abrupt; variations in water level of 2 m to 3 m take place in a relatively short period, say of several days (see Figure 10b).

This pattern is clearly indicated by a study of recorded daily high and low water levels (see Figures 15 to 18). It can be seen that marked differences in water level in one day take place, NOT in the Spring flood period, but in what are normally considered low flow, or low water level periods. Indeed, the greatest daily fluctuations in 1979 were those recorded in the Winter period(s) (see Figures 15 and 18).

This pattern of abrupt changes in water level within a day and occurring throughout the year, except for periods of high flood flows, is typical of river conditions downstream of hydro-electric dams. It leads to several effects:

- i) river velocities can change from below to well above critical scour values on a daily basis, thus resulting in increased erosion, both of steep banks in erodible materials and of material resulting from shallow slumps or failures;
- ii) following a large drop in river water level, high piezometric levels exist in the cohesive materials at the toe of the slope causing shallow rotational failures of the bank;

- iii) repeated fluctuations aggravate erosion of the slope toe and undermining of the loose granular materials comprising the upper part of the slope thus leading to surficial sliding of these materials.

To a limited extent, some of the bank erosion observed since 1960 is inevitable as it is related to Spring flooding conditions, irrespective of whether or not river flows are regulated by the dam; however, to a much greater extent, the fluctuations in discharge downstream of the dam have induced increased erosion and scour. When the river was unregulated and in its natural state, erosion only took place in the Spring; throughout the rest of the year, no erosion occurred and at periods, some deposition would take place. Slopes eroded had time to re-vegetate and thus were protected to some degree for the next Spring flooding. Since 1960, with erratic fluctuations of water levels and consequent fluctuations in current velocity throughout the year, erosion takes place on a continuous basis and the river banks thus affected have no opportunity to develop some degree of protection through re-deposition or re-vegetation.

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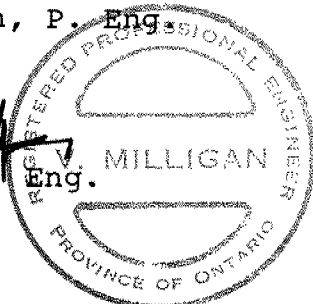




PLATE I SHOWING TOE EROSION AND FORMATION OF  
BACK SCARPS

( MTC CHAINAGE 17 + 800 )



PLATE II SHOWING SURFICIAL SLIDING

( MTC CHAINAGE 19 + 290 )



PLATE III      SHOWING ROTATIONAL FAILURES  
( MTC CHAINAGE 21+330 )



PLATE IV      SHOWING A COMBINATION OF SURFACIAL  
SLIDING AND SHALLOW ROTATIONAL FAILURE  
( MTC CHAINAGE 19+900 )



## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPES

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

### II. PENETRATION RESISTANCES

#### Dynamic Penetration Resistance:

The number of blows by a 63.6 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

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

The number of blows by a 63.6 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

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

*PH* sampler advanced by pressure—pressure, hydraulic

*PM* sampler advanced by pressure—pressure, manual

### III. SOIL DESCRIPTION

(a) <i>Cohesionless Soils</i>	' <i>N</i> ' <u>Blows/0.30m</u> or <u>Blows/ft.</u>
Relative Density	
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	<u>kPa</u>	' <i>Cu</i> ' <u>psf.</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

### IV. SOIL TESTS

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

#### NOTES:

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

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

## LIST OF SYMBOLS

### I. GENERAL

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

### II. STRESS AND STRAIN

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

### III. SOIL PROPERTIES

#### (a) Unit weight

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

#### (b) Consistency

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

#### (c) Permeability

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

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

$m_v$	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
$C_c$	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
$c_c$	coefficient of consolidation
$T_v$	time factor = $c_v / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) Shear strength

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

in terms of effective stress

$$\tau_f = c' + \sigma' \tan \phi'$$

in terms of total stress

$$\tau_f = c_u + \sigma \tan \phi_u$$

\*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 Nos 1, 2 & 3

METRIC

W P 153-83-00 LOCATION STA.13+509.9, STA.13+507.9, STA.13+506.4 (Cobden) ORIGINATED BY ASP  
 DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Solid Stem COMPILED BY GP  
 DATUM GEODETIC DATE MAY 21, 1985 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
181.48	Ground Level																
0.0	Topsoil																
181.02																	
0.46	Sand, fine to coarse.		1	50 mm D.O.	4												
			2	"	5		180										-0 99 1 -
	Loose Brown																
179.10			3	"	1												
2.38	Clayey Silt, organic finely laminated. Occasional sand seam.		4	"	1												
177.82	Firm Grey						178										
3.66	Sand, fine to medium, trace clay and silt.		5	"	4												-0 85 11 4
			6	"	6												
175.84	Loose Grey						176										
5.64	Silt, trace sand and clay.																
174.93	Compact Grey		7	"	11												
6.55	End of Borehole																
Borehole 1 Piezometer Installation Sta.13+509.9  Borehole 2 Piezometer Installation Sta.13+507.9  Borehole 3 Piezometer Installation Sta.13+506.4  WATER LEVEL $\nabla$ May 25, 1985 $\nabla$ July 9, 1985																	

+3, x5: Numbers refer to  
Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10



# RECORD OF BOREHOLE Nos 4,5 & 6

METRIC

W P 153-83-00 LOCATION STA.21+336.6, STA.21+346.6, STA.21+345.3 (Thompson) ORIGINATED BY ASP  
DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Hollow Stem COMPILED BY GP  
DATUM GEODETIC DATE MAY 21, 1985 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
180.82	Ground Level																GR SA SI CL
0.0	Topsoil																
180.35																	
0.47	Layered Sand - medium to coarse and Silty fine sand, occasional organics.		1	50 mm D.O.	2		180										k=9x10 <sup>-8</sup> m/sec
178.84	Very Loose Brown to Grey		2	"	1												
1.98	Clayey Silt, trace sand																0 2 68 30
178.42	Firm Grey																
2.40	Silty Sand		3	"	1												
177.92	Very Loose Grey						178										
2.90	Layered Clayey Silt - organic, trace sand		4	"	1												
	Silt - organic and Silty clay - occasional pocket of peat																
	Firm Grey		5	"	2												0 2 60 38
176.05	Sand - medium to coarse																
175.76	Compact Grey		6	"	2/		176										
5.06	End of Borehole																
								Borehole 4 Piezometer Installation					Sta. 21+336.6				
								Borehole 5 Piezometer Installation					Sta. 21+346.6				
								Borehole 6 Piezometer Installation					Sta. 21+345.3				
								WATER LEVEL May 25, 1985									
								July 9, 1985									

# RECORD OF BOREHOLE Nos 7 & 8

METRIC

W P 153-83-00 LOCATION STA.16+492.6, STA.16+487.3 (Thompson) ORIGINATED BY ASP  
 DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Hollow Stem COMPILED BY GP  
 DATUM GEODETIC DATE MAY 22, 1985 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
183.76	Ground Level																
0.0	Topsoil	33															
183.29																	
0.47	Layered Silty fine sand, Sandy silt, & Sand, fine to coarse (seams to 12 mm thick)		1	50 mm D.O.	4												
182.39	Loose Brown																
1.37			2	"	3		182										
	Sand - fine to medium		3	"	2												
			4	"	3												
	Very loose Brown						180										
179.80																	
3.96	Layered Sandy silt, Silt-organic, Silty sand Occasional woody peat seam.		5	"	2												
178.58	Very loose Grey																
5.18							178										
	Clayey silt - organic, trace sand																
177.21	Firm Grey		6	"	1												
6.55	End of Borehole																
Borehole 7 Piezometer Installation Sta.16+92.6 Borehole 8 Piezometer Installation Sta.16+487.3 WATER LEVEL $\nabla$ May 25, 1985 $\nabla$ July 9, 1985																	

+3, x5: Numbers refer to  
Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10

## METRIC

W P 153-83-00 LOCATION STA. 12+678 (Thompson) ORIGINATED BY ASP  
DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Hollow Stem COMPILED BY GP  
DATUM GEODETTIC DATE MAY 22, 1985 CHECKED BY \_\_\_\_\_

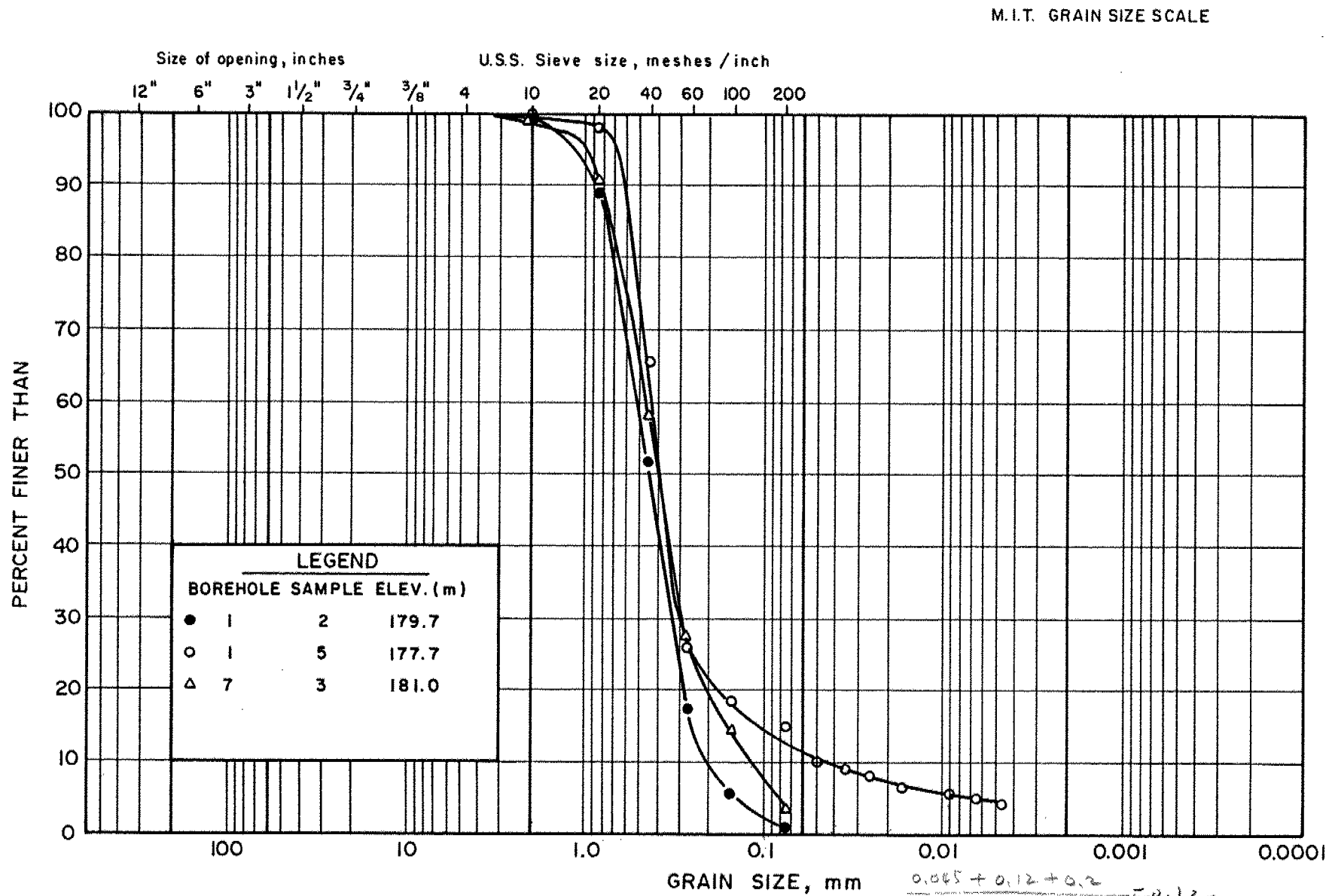
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						SHEAR STRENGTH	WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL						+ FIELD VANE x LAB VANE	
184.14	Ground Level														
0.0	Asphalt														
0.13	Fill - sand and gravel														
182.94															
1.20	Layered Sand - medium to coarse, and Silty Sand, with organic fine sandy silt seams 12 mm thick. Loose to very loose Brown		1	50 mm D.O.	5		182		○						
180.86															
3.28	Silty Sand Grey/ Brown		2	"	2					○					
180.48															
3.66	Silt - organic, trace clay and sand. Occasional sandy silt and silty sand seams 12 to 25 mm thick. Numerous woody peat zones.		3	"	1		180			○					
			4	"	1					○					
178.35	Firm Grey		5	"	1					○			0 3 87 10		
5.79	End of Borehole														
								WATER LEVEL	May 25, 1985						
									July 9, 1985						

+3, x5 : Numbers refer to Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10

# OVERSIZE DRAWING(S)

Golder Associates



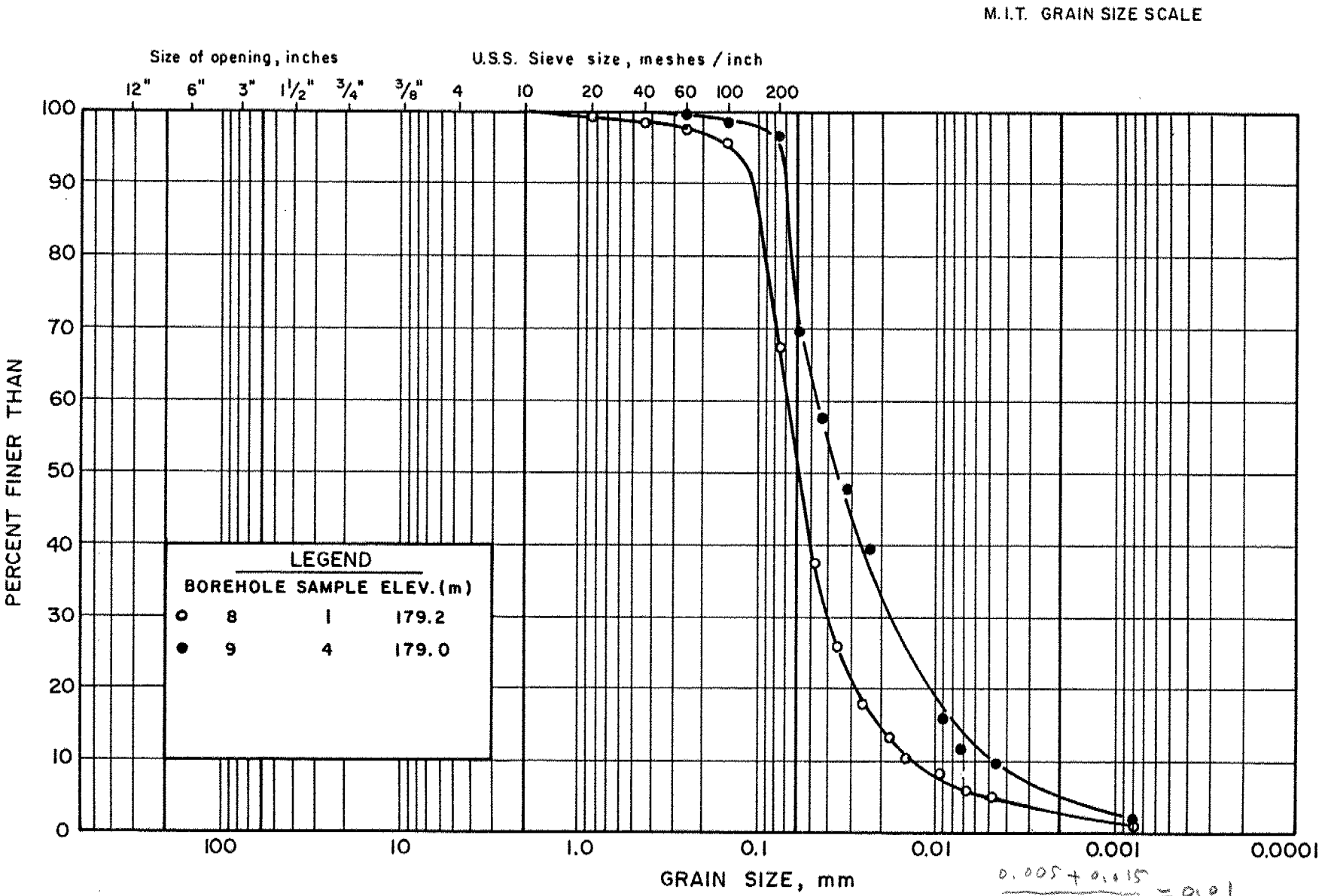
BOULDER SIZE	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
		GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION  
SAND

FIGURE 6



Golder Associates

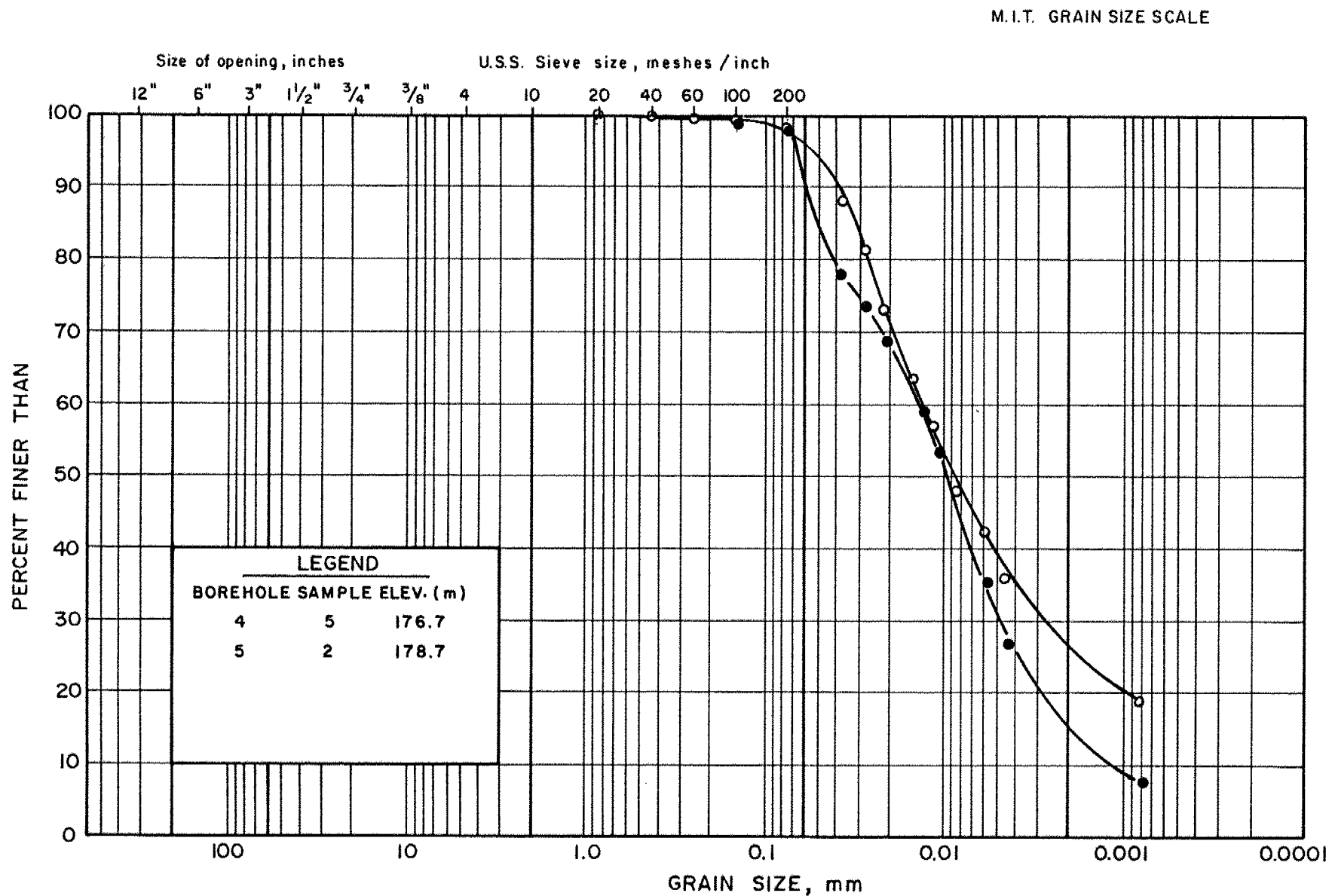


BOULDER SIZE	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
		GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION  
SILT TO FINE SANDY SILT

FIGURE 7

Golder Associates



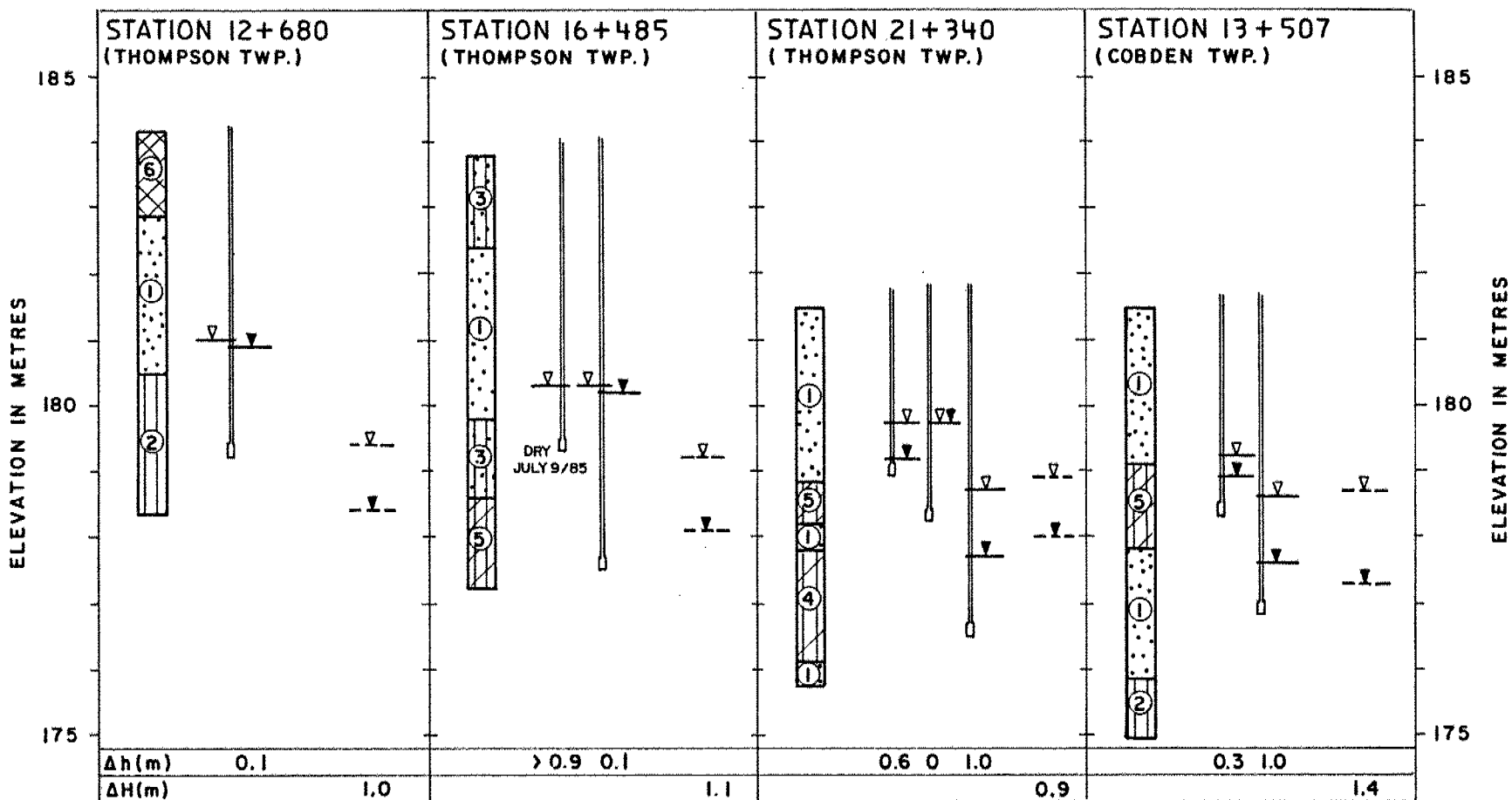
BOULDER SIZE	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
		GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT

FIGURE 8

# SUMMARY OF PIEZOMETER INSTALLATIONS AND WATER LEVEL MEASUREMENTS

FIGURE 9



## LEGEND



GENERALIZED  
STRATIGRAPHY

- ① SAND
- ② SILT
- ③ LAYERED SAND AND SILT
- ④ LAYERED SILT & CLAYEY SILT
- ⑤ CLAYEY SILT
- ⑥ FILL

## WATER LEVEL MEASUREMENT

▽ MAY 25, 1985  
▽ JULY 9, 1985

□ PIEZOMETER TIP

Δh - CHANGE IN PIEZOMETRIC  
LEVEL

## RIVER WATER LEVEL

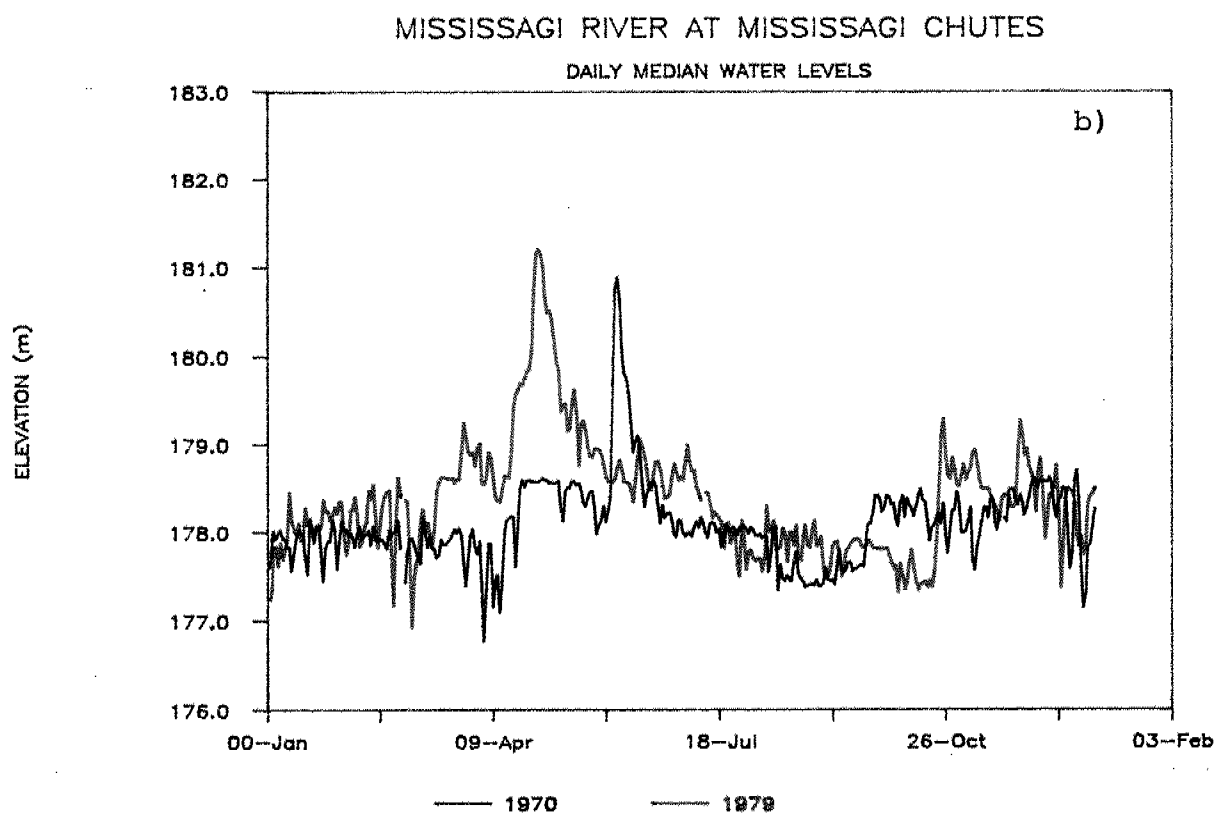
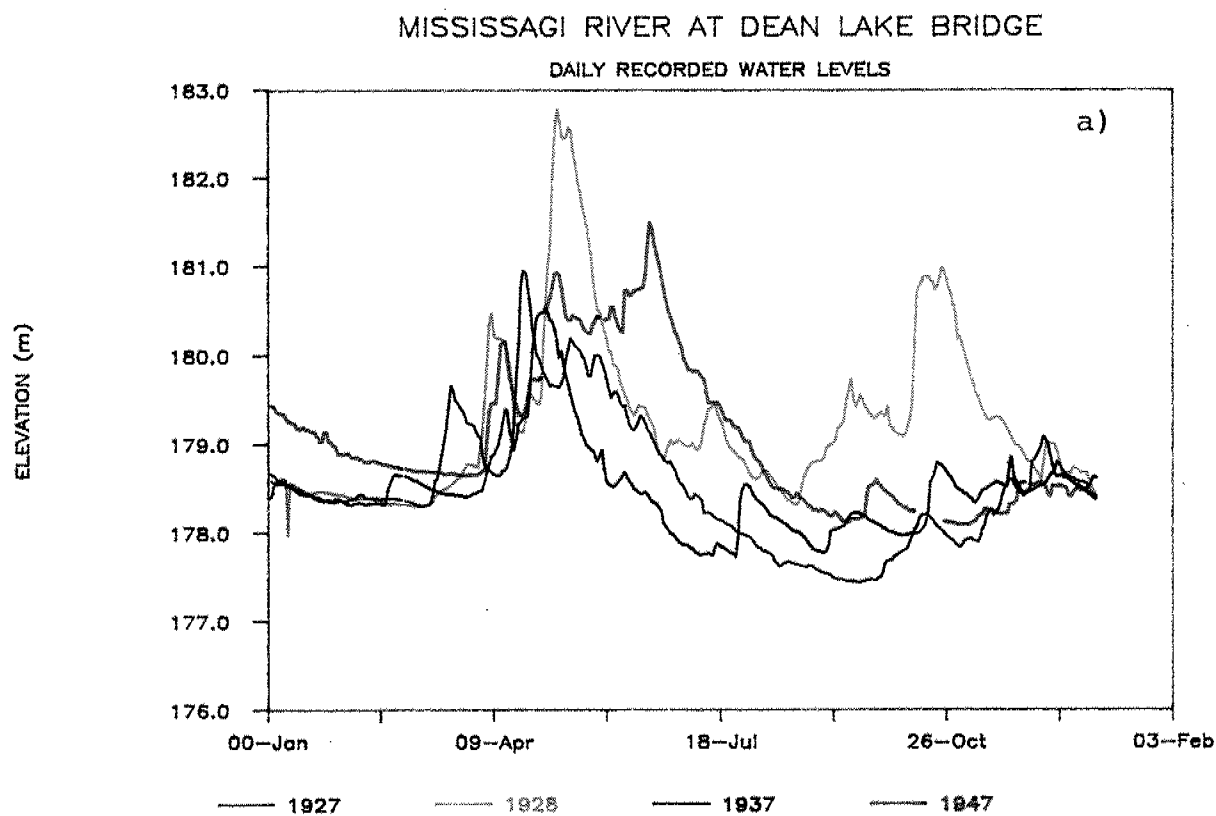
▽ MAY 25, 1985  
▽ JULY 15, 1985

ΔH - CHANGE IN RIVER  
WATER LEVEL

Date OCT 15, 1985  
Project 841-1356-1

**Golder Associates**

Drawn MHW  
Checked ASL

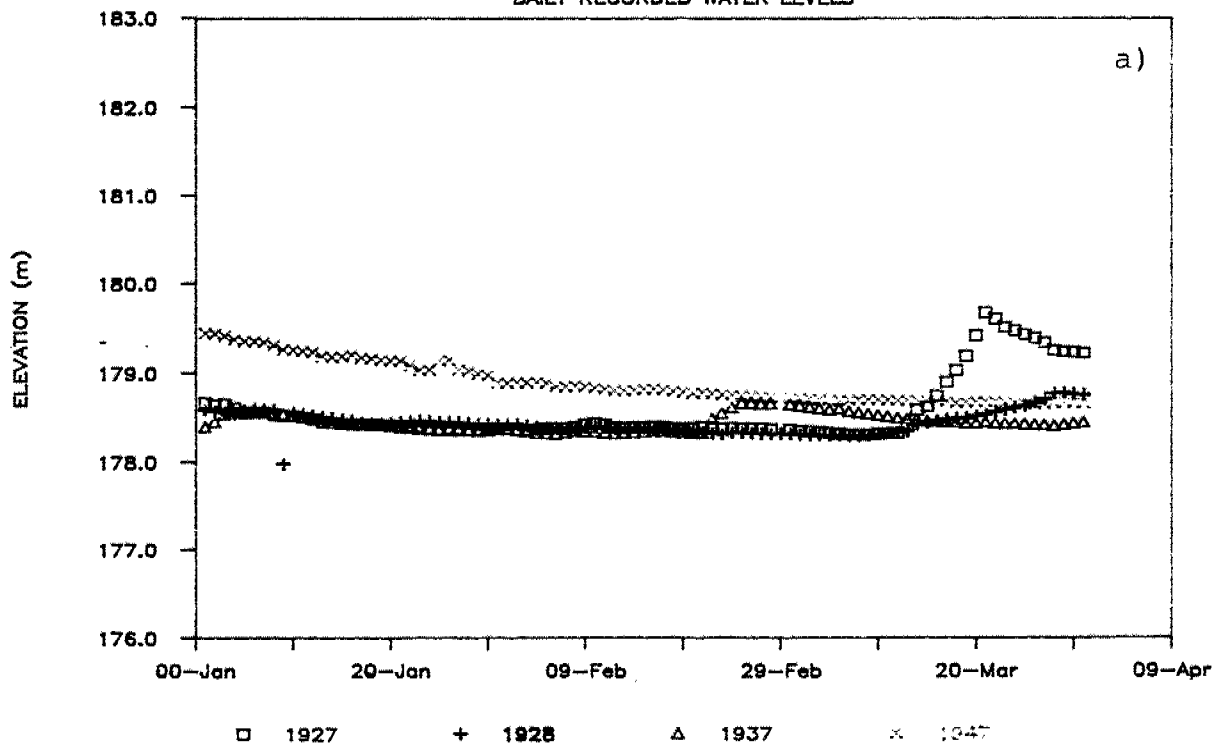


# DAILY WATER LEVEL ELEVATIONS ( FIRST QUADRANT )

FIGURE II

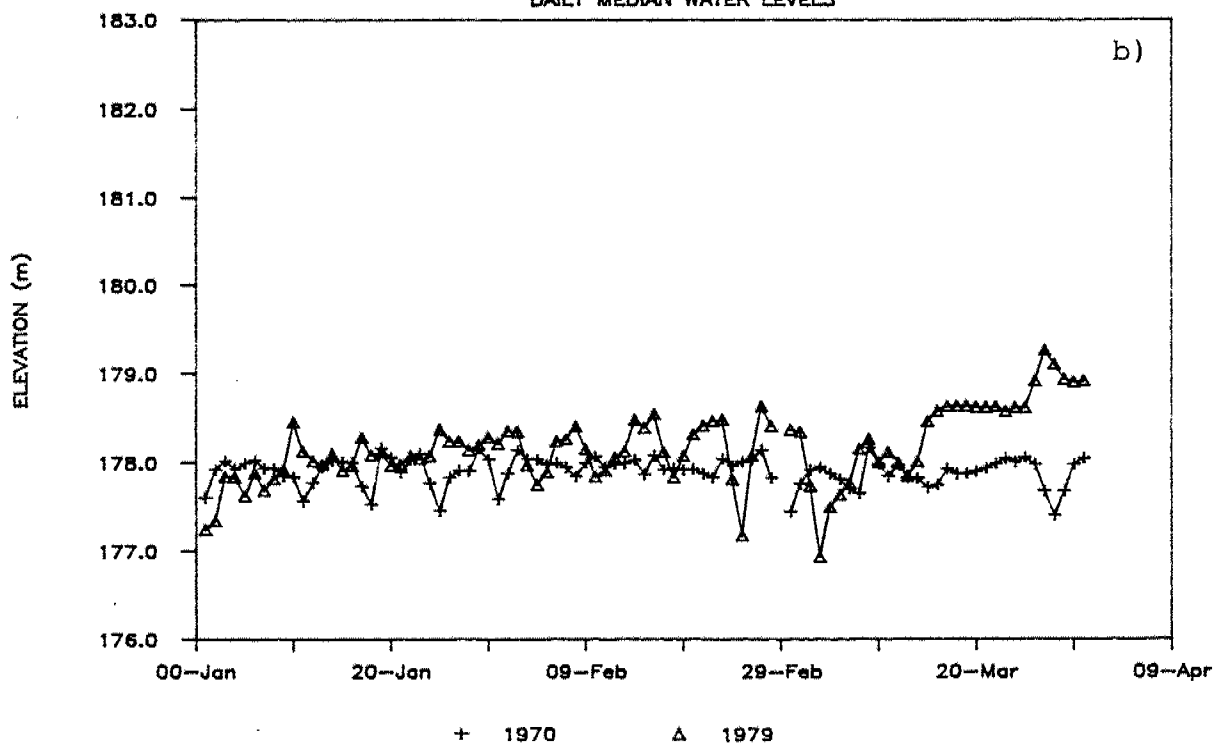
## MISSISSAGI RIVER AT DEAN LAKE BRIDGE

DAILY RECORDED WATER LEVELS



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

DAILY MEDIAN WATER LEVELS



Date OCT. 28, 1985  
Project 841-1356-1

**Golder Associates**

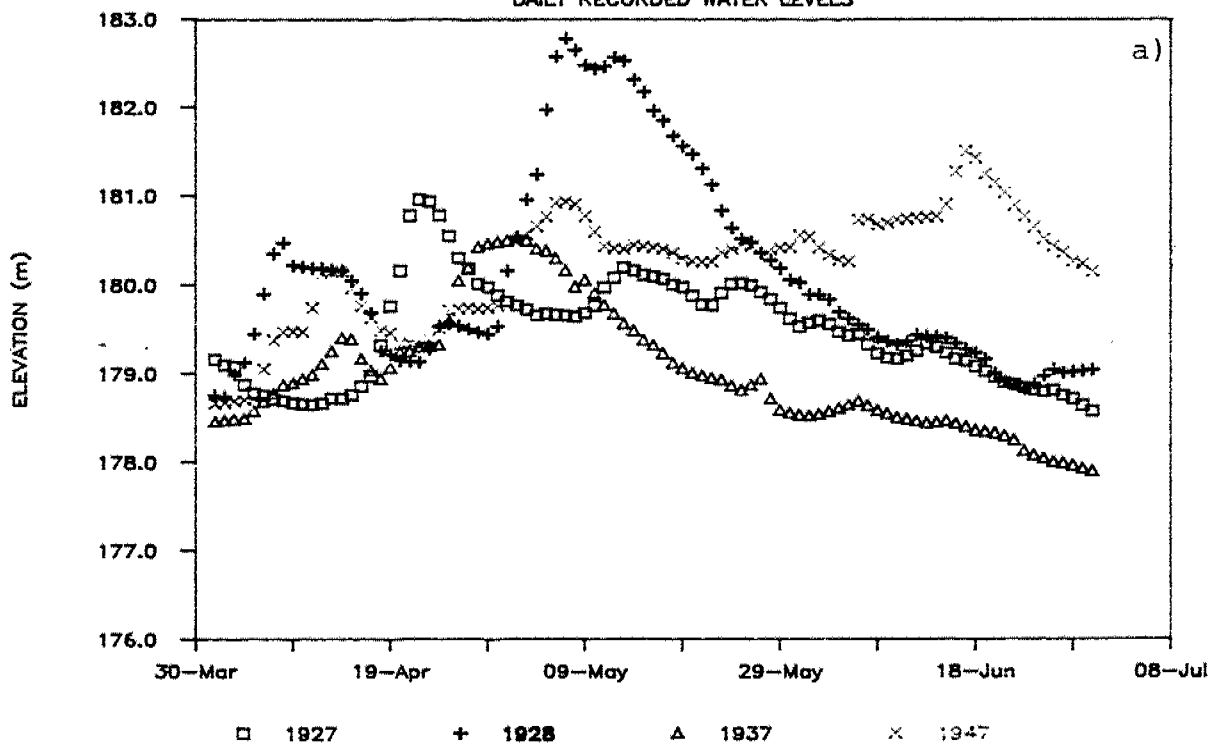
Data by ASP  
Checked by \_\_\_\_\_

# DAILY WATER LEVEL ELEVATIONS (SECOND QUADRANT)

FIGURE 12

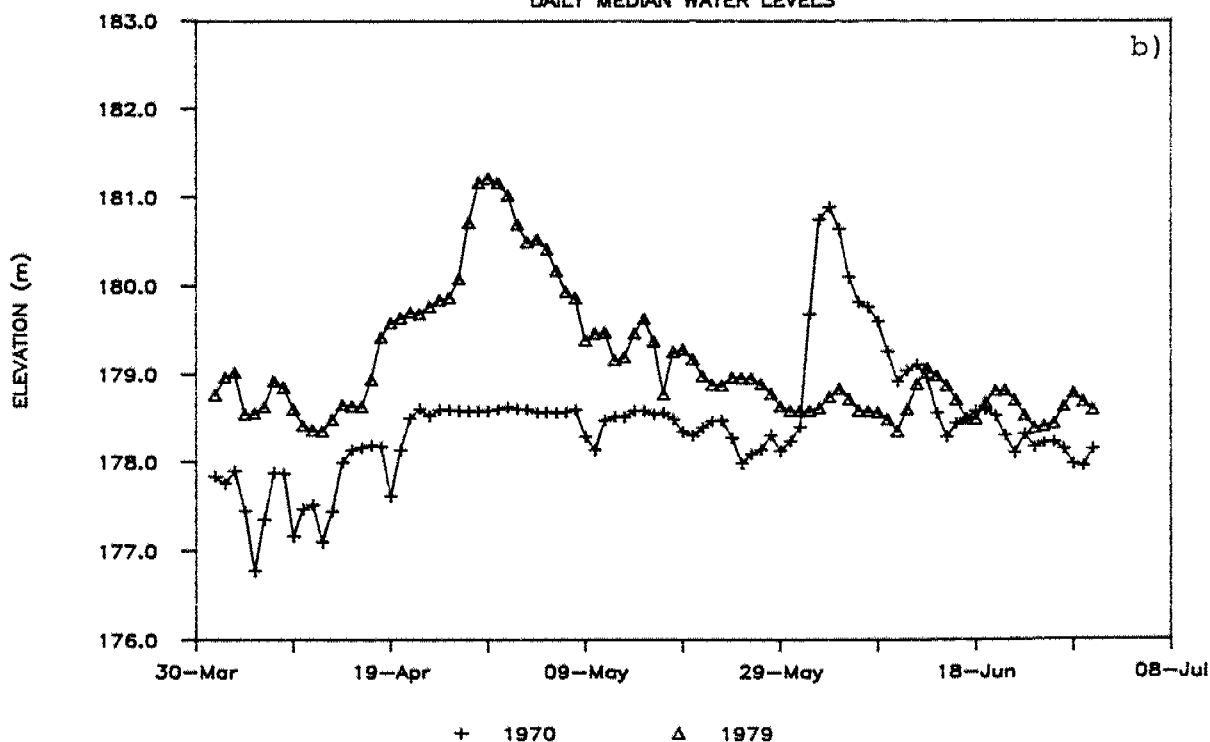
## MISSISSAGI RIVER AT DEAN LAKE BRIDGE

DAILY RECORDED WATER LEVELS



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

DAILY MEDIAN WATER LEVELS



Date OCT. 28, 1985  
Project 841-1356-1

**Golder Associates**

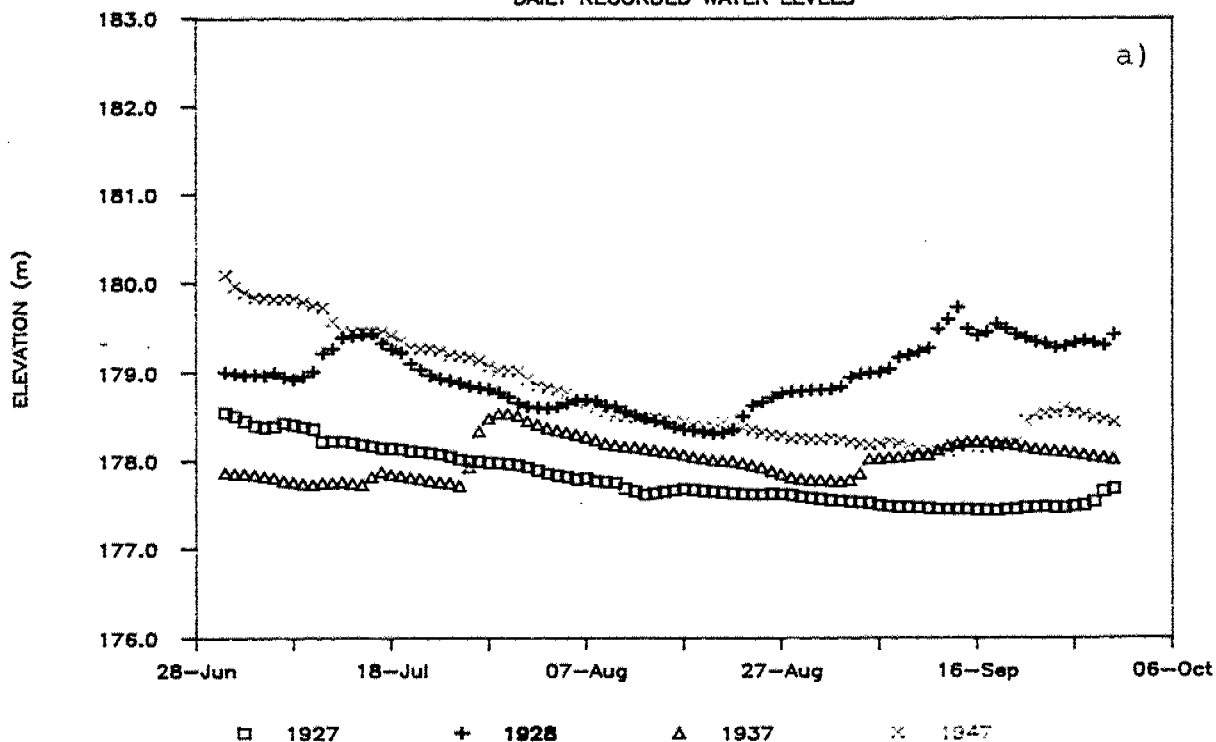
Data by ASP  
Checked by -----

# DAILY WATER LEVEL ELEVATIONS (THIRD QUADRANT)

FIGURE 13

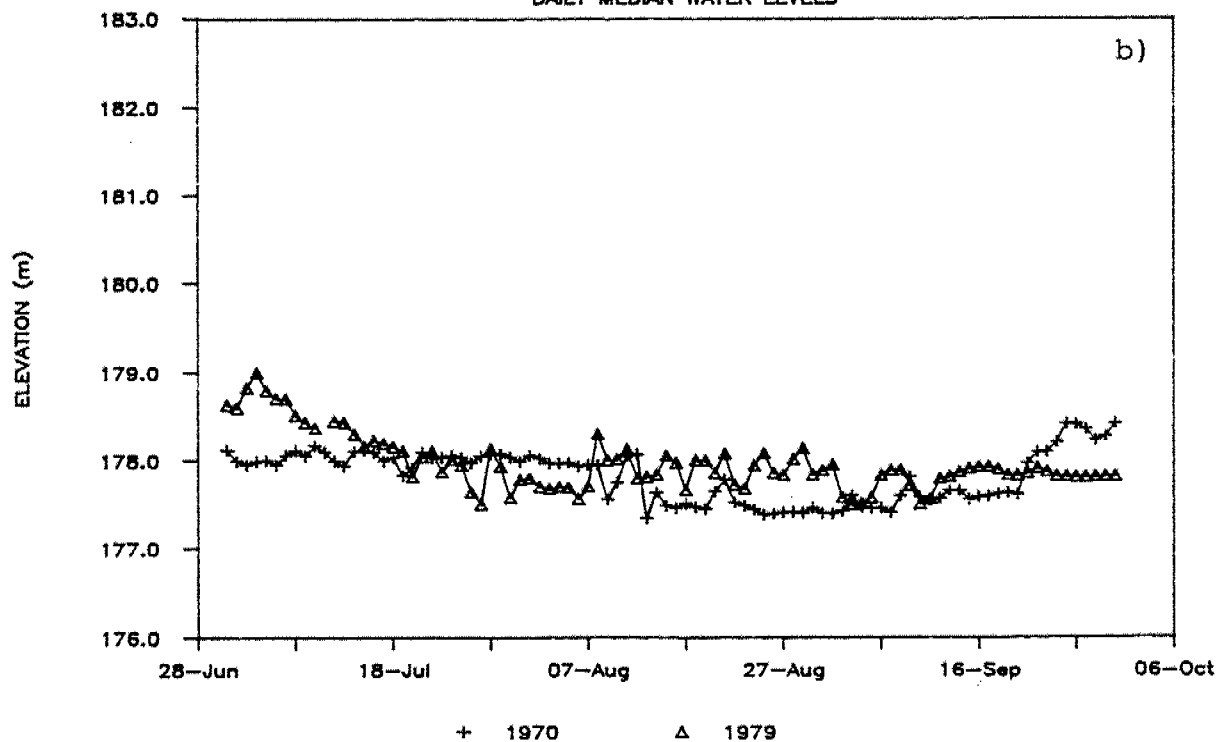
## MISSISSAGI RIVER AT DEAN LAKE BRIDGE

DAILY RECORDED WATER LEVELS



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

DAILY MEDIAN WATER LEVELS



Date OCT. 28, 1985  
Project 841-1356-1

**Golder Associates**

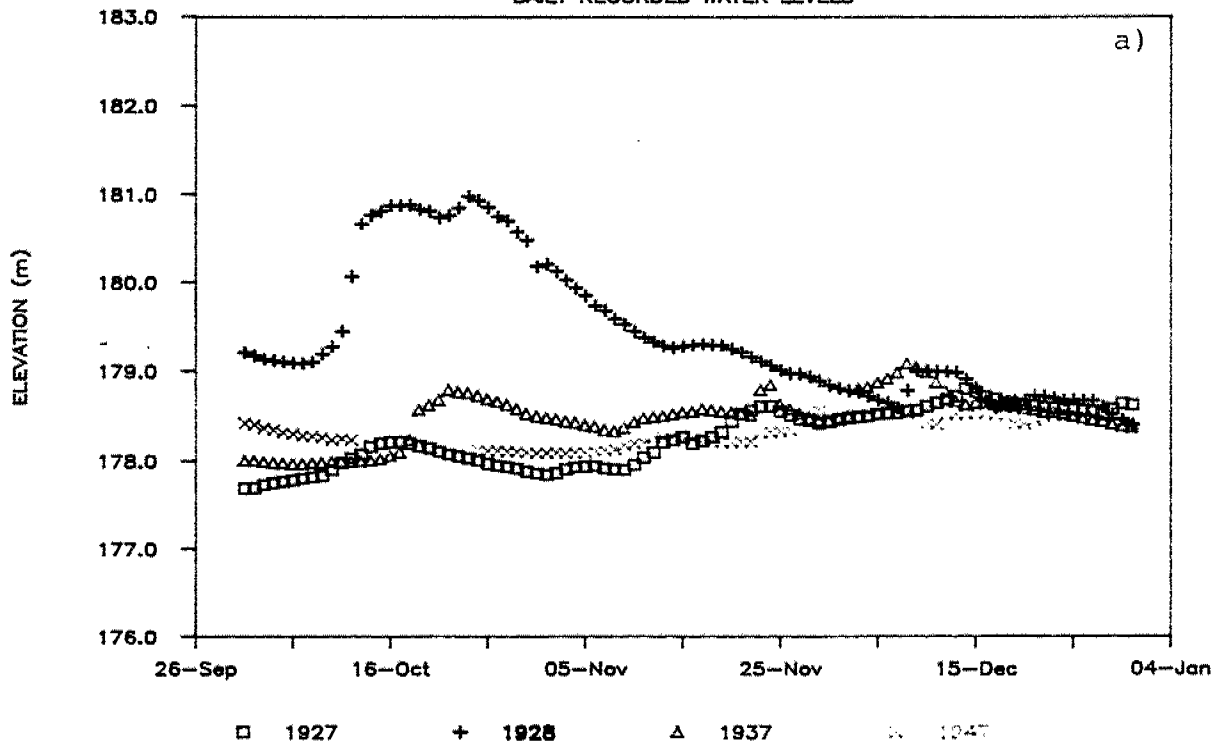
Data by ASP  
Checked by \_\_\_\_\_

# DAILY WATER LEVEL ELEVATIONS (FOURTH QUADRANT)

FIGURE 14

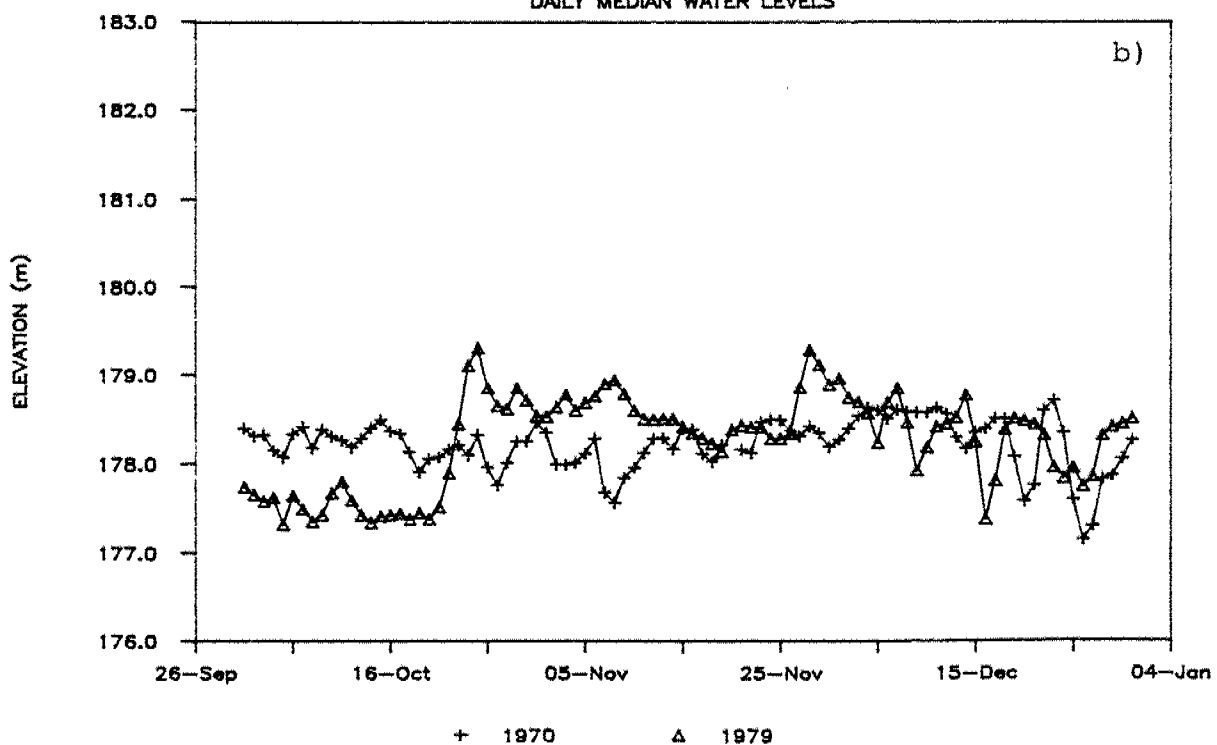
## MISSISSAGI RIVER AT DEAN LAKE BRIDGE

DAILY RECORDED WATER LEVELS



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

DAILY MEDIAN WATER LEVELS



Date OCT. 28, 1985  
Project 841-1356-1

**Golder Associates**

Data by ASP  
Checked by -----

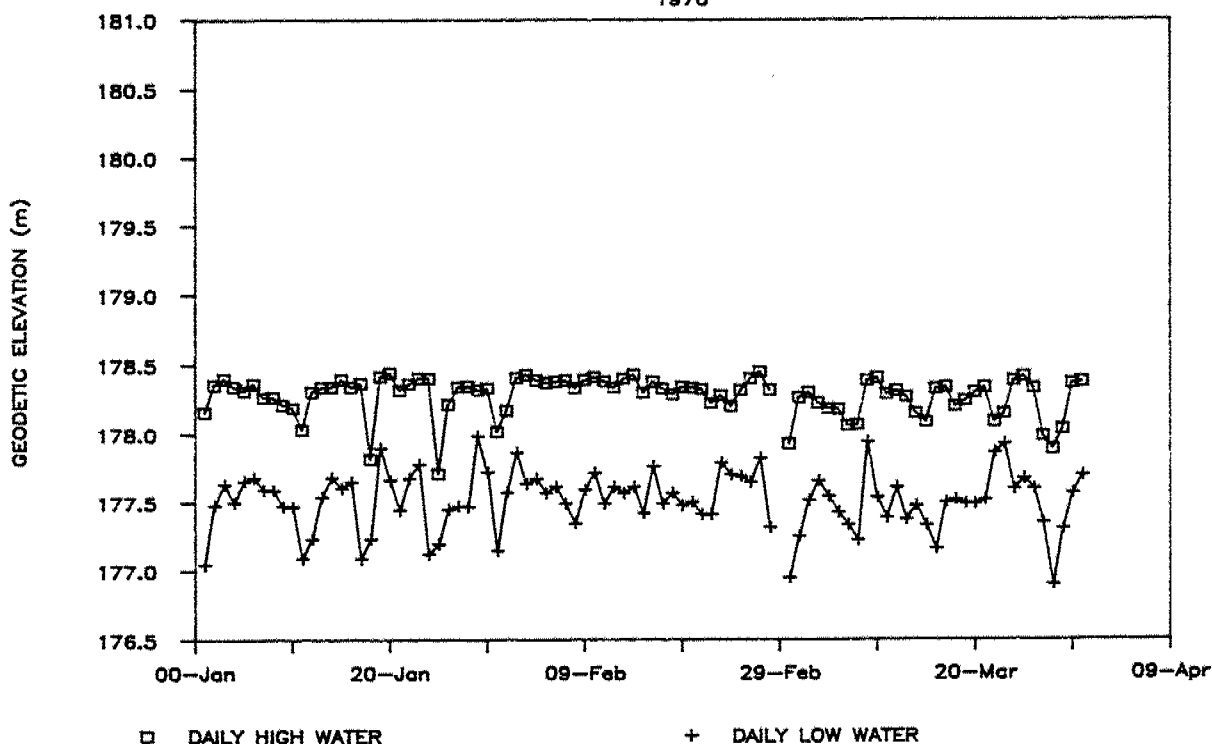


# DAILY HIGH AND LOW WATER LEVEL ELEVATIONS ( FIRST QUADRANT )

FIGURE 15

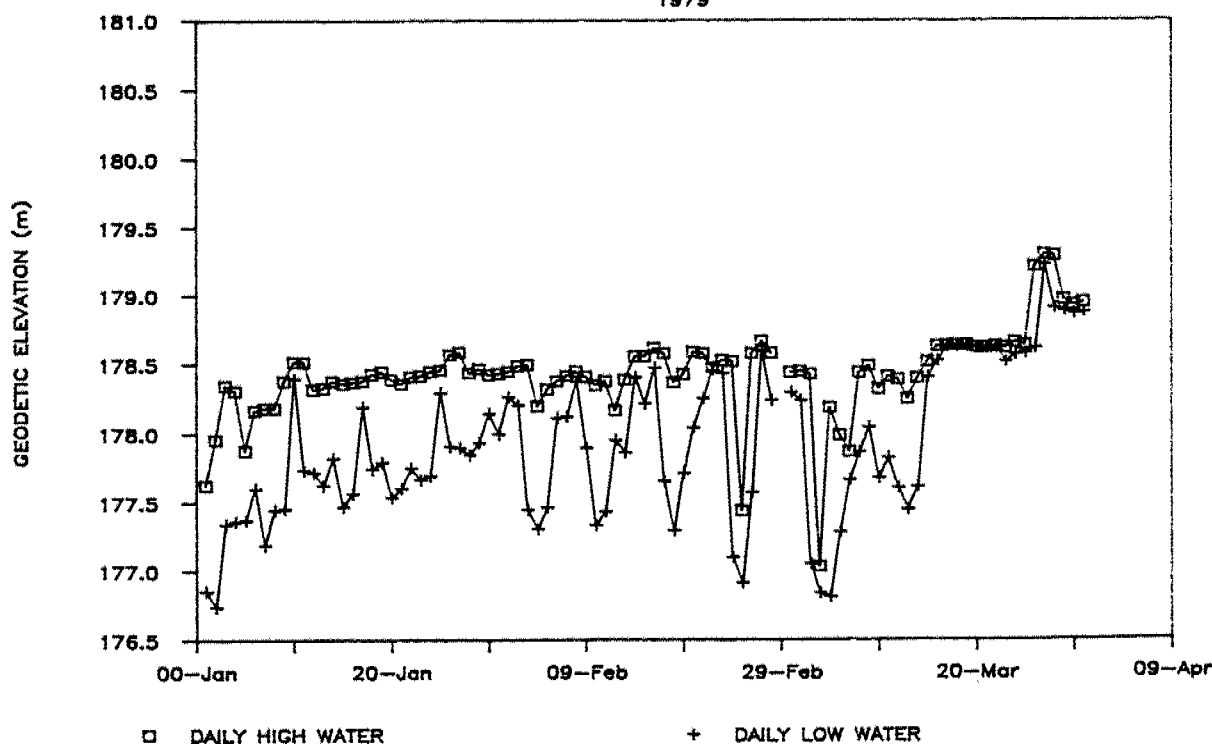
## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1970



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1979



Date OCT. 28, 1985  
Project 841-1356-1

**Golder Associates**

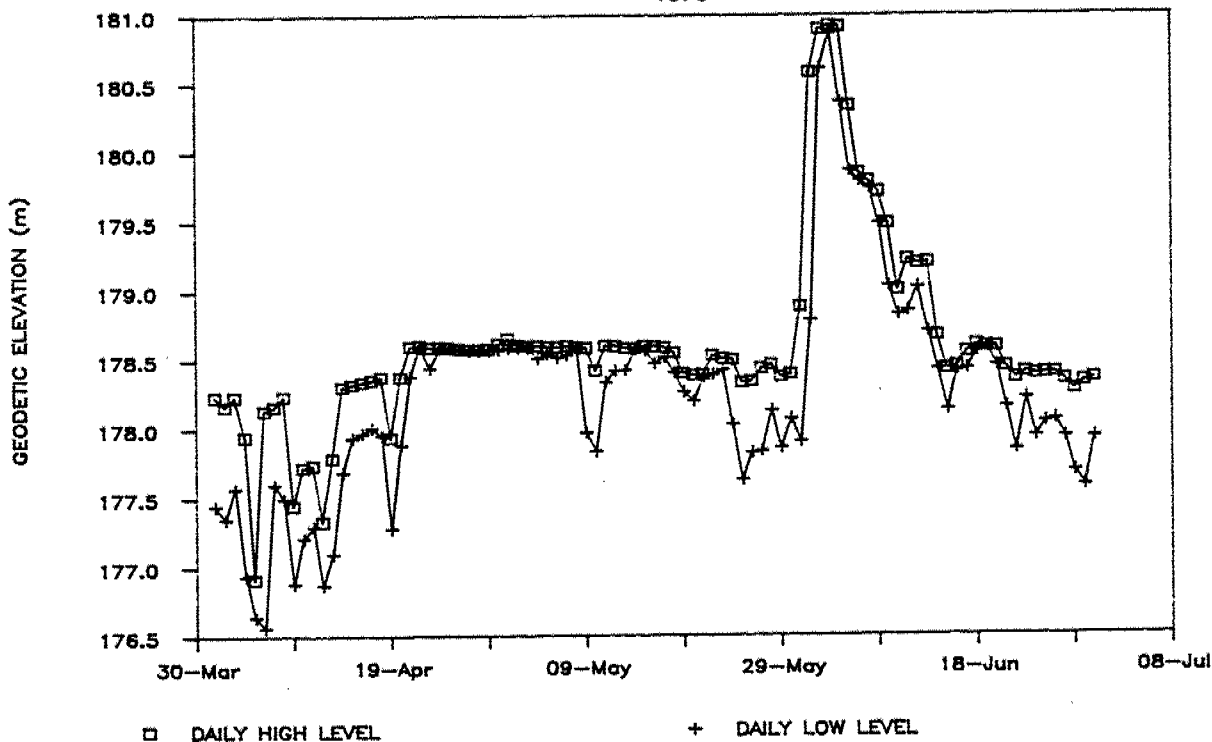
Data by ASP  
Checked by \_\_\_\_\_

# DAILY HIGH AND LOW WATER LEVEL ELEVATIONS ( SECOND QUADRANT )

FIGURE 16

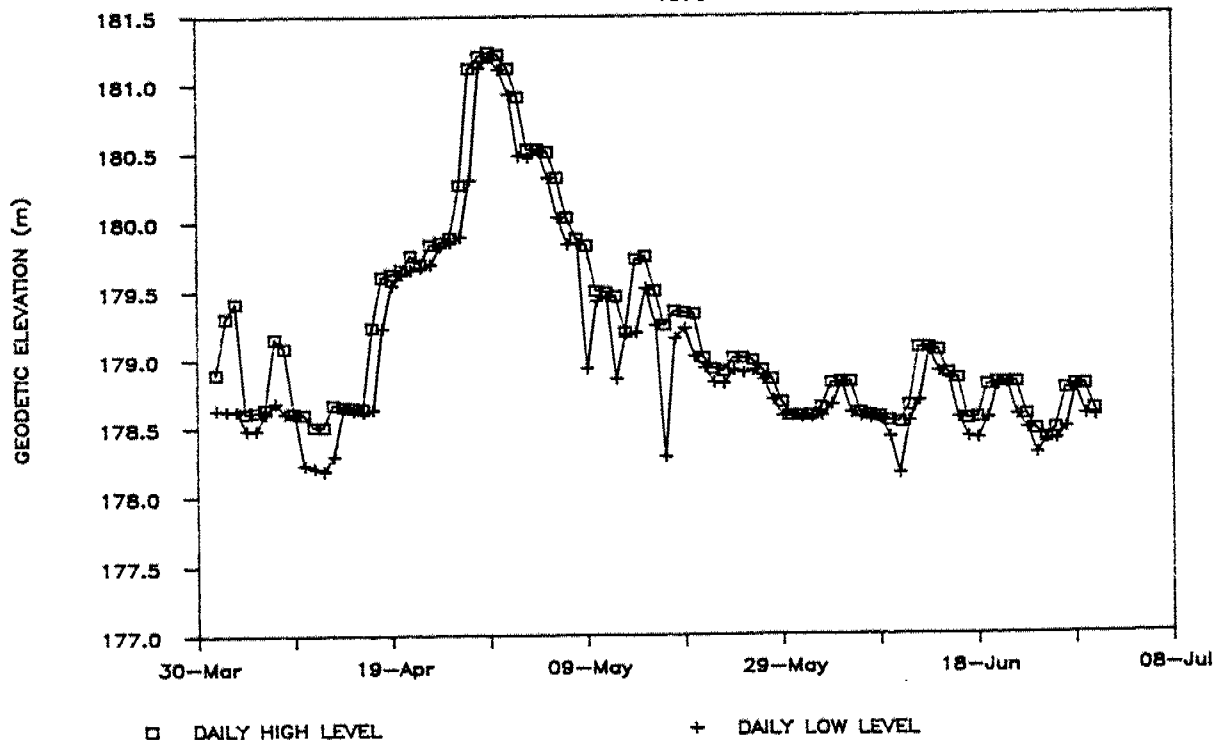
## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1970



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1979



Date OCT. 28, 1985  
Project 841-1356-1

**Golder Associates**

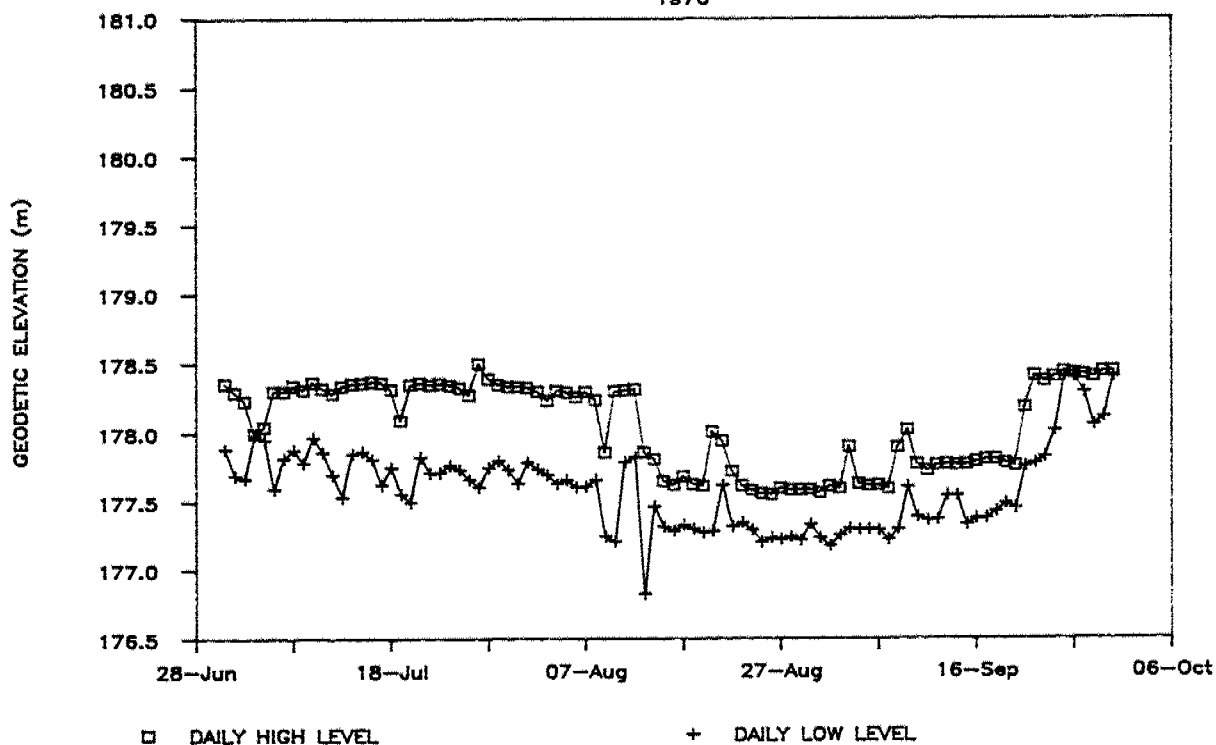
Data by ASP  
Checked by \_\_\_\_\_

# DAILY HIGH AND LOW WATER LEVEL ELEVATIONS (THIRD QUADRANT)

FIGURE 17

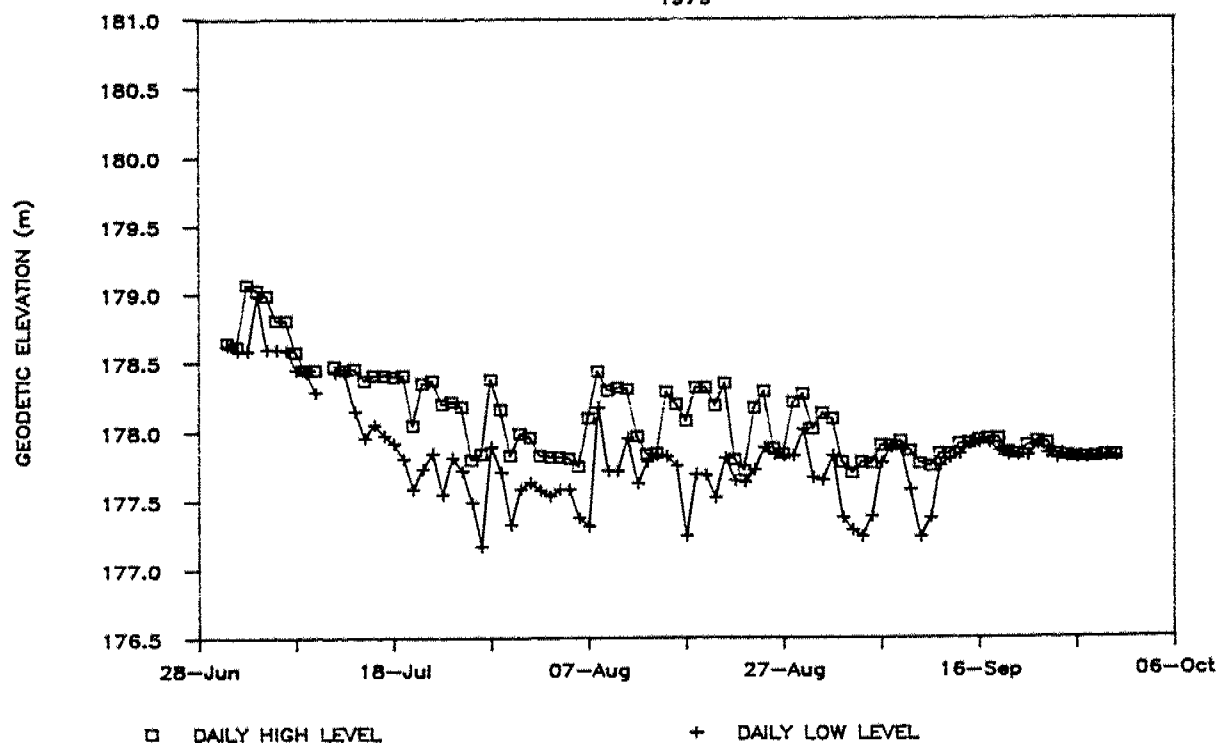
## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1970



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1979



Date OCT. 28, 1985  
Project 841-1356-1

**Golder Associates**

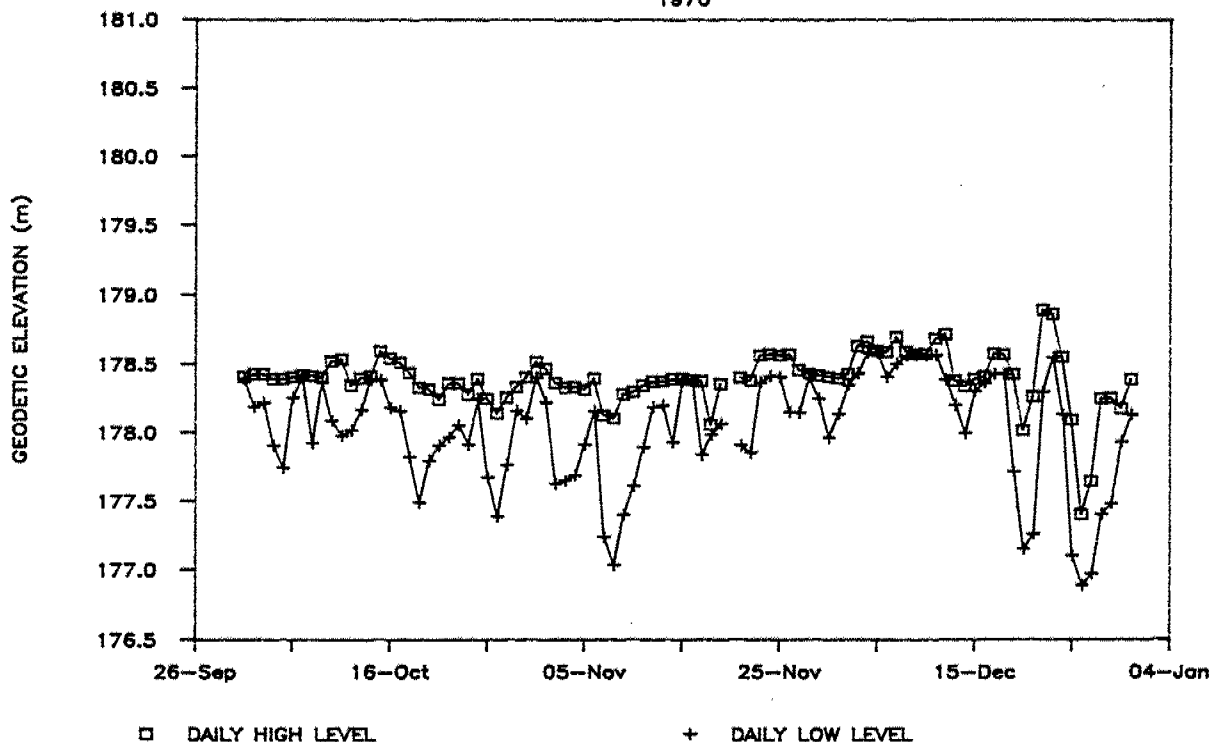
Data by ASP  
Checked by -----

# DAILY HIGH AND LOW WATER LEVEL ELEVATIONS (FOURTH QUADRANT)

FIGURE 18

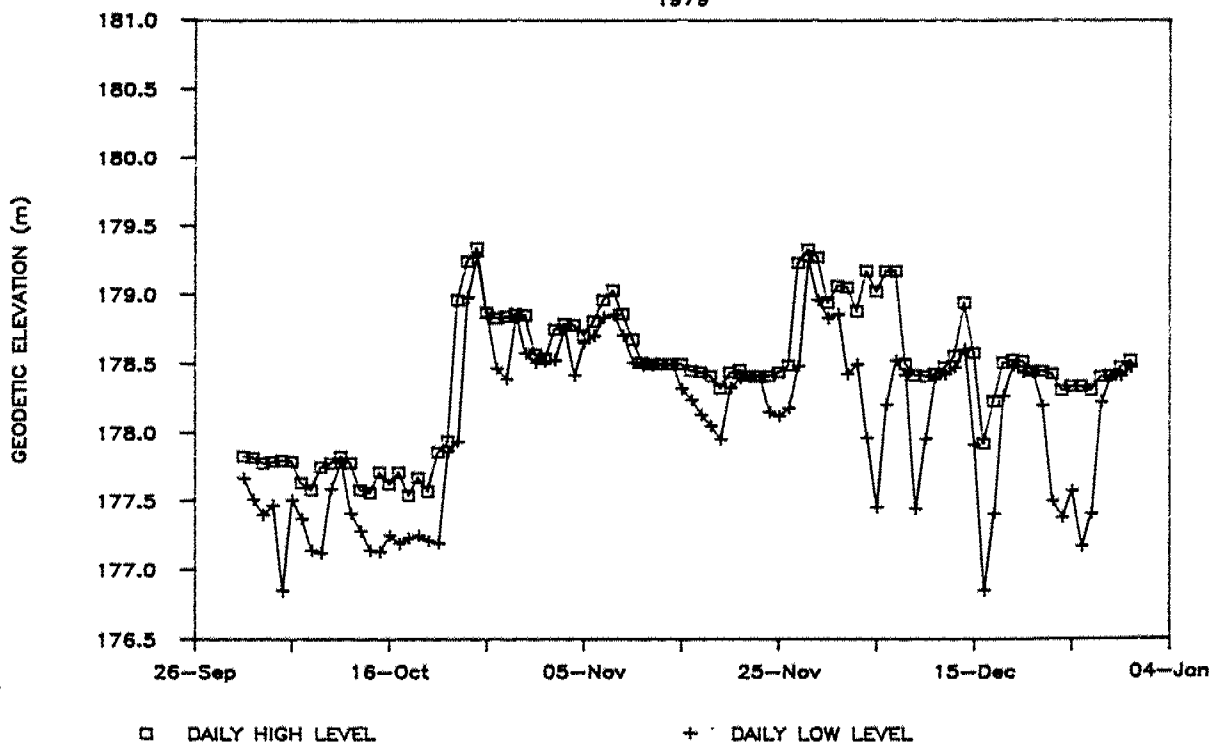
## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1970



## MISSISSAGI RIVER AT MISSISSAGI CHUTES

1979



Date OCT. 28, 1985  
Project 841-1356-1

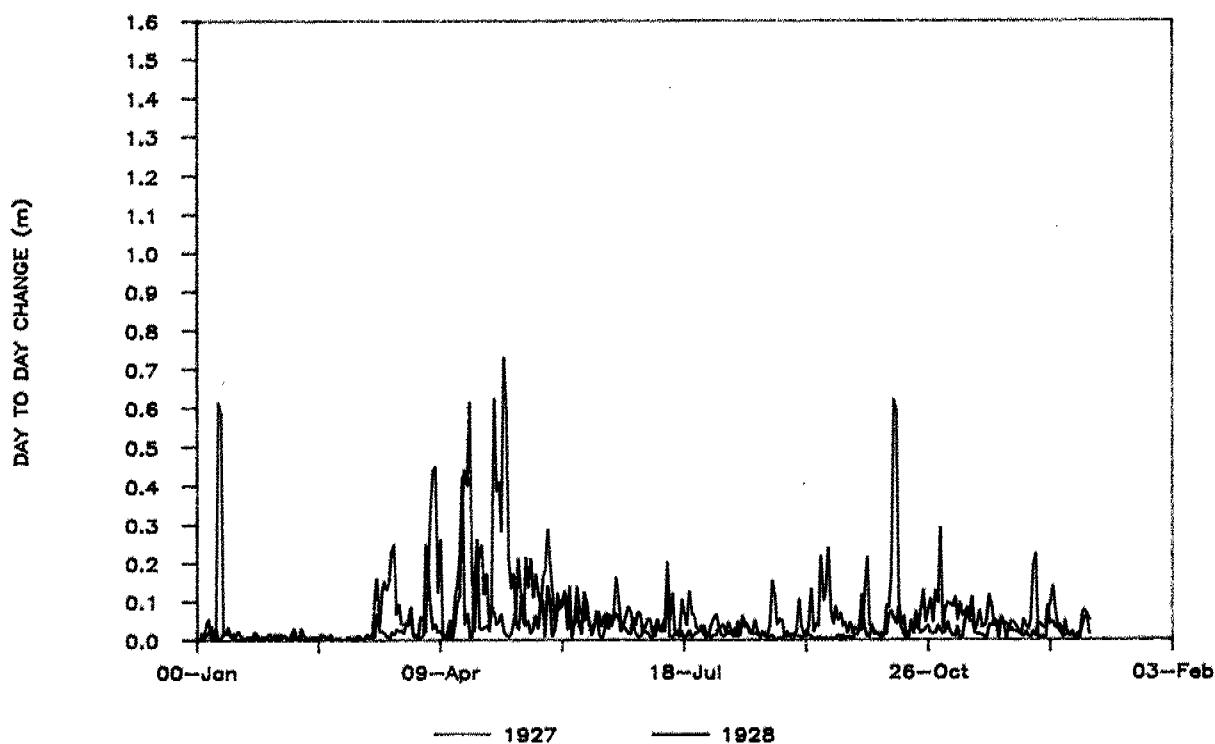
**Golder Associates**

Data by ASP  
Checked by -----

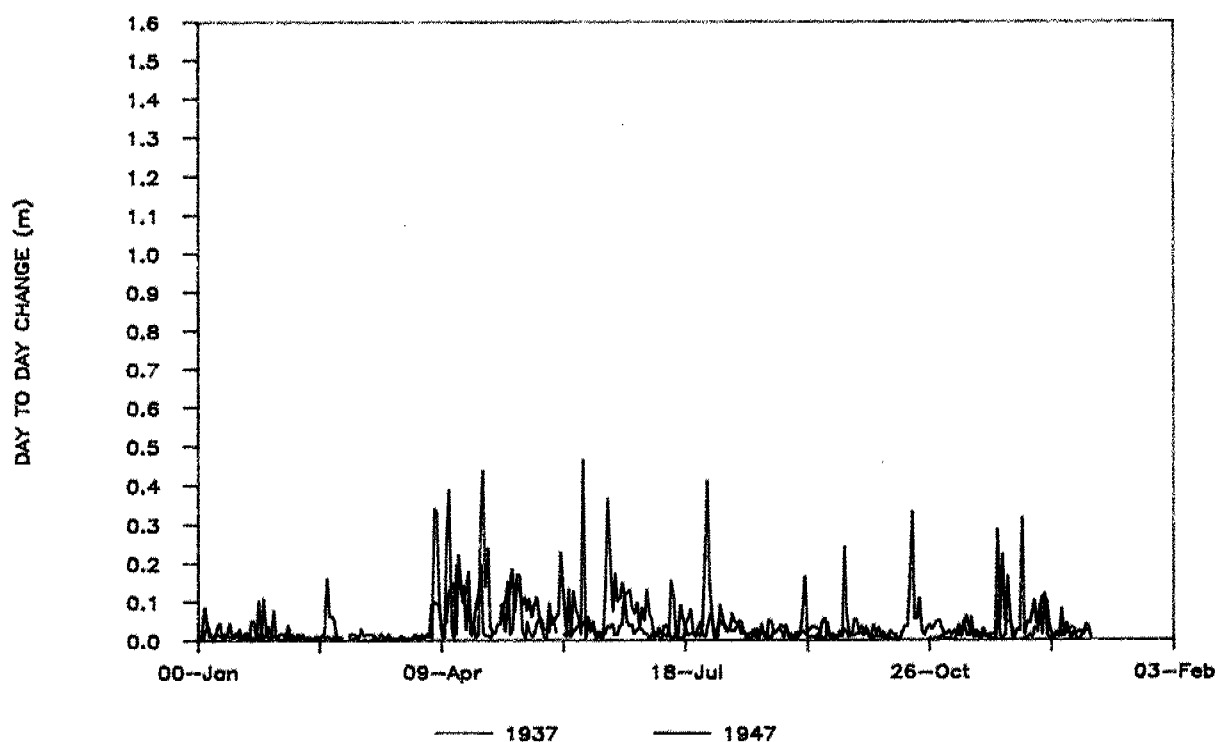
# DAY TO DAY WATER LEVEL FLUCTUATIONS - UNREGULATED RIVER

FIGURE 19

MISSISSAGI RIVER AT DEAN LAKE BRIDGE



MISSISSAGI RIVER AT DEAN LAKE BRIDGE



Date OCT. 28, 1985  
Project 841-1356-1

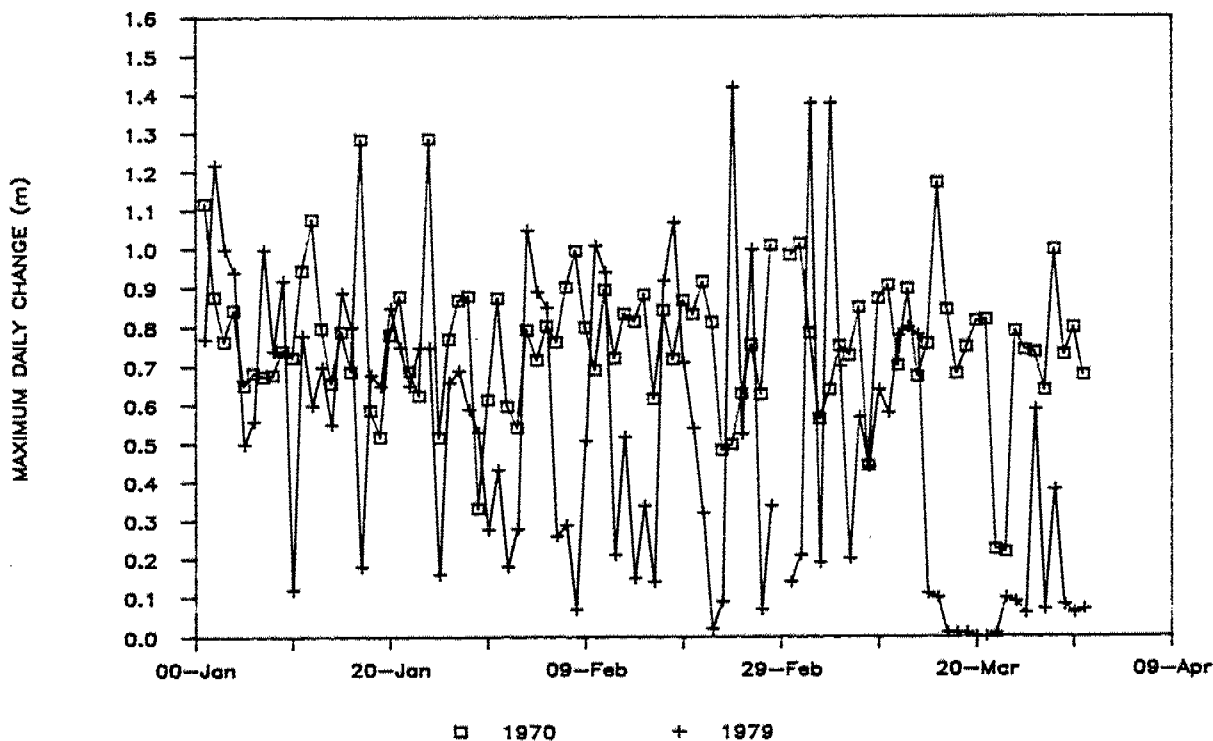
**Golder Associates**

Data by ASP  
Checked by -----

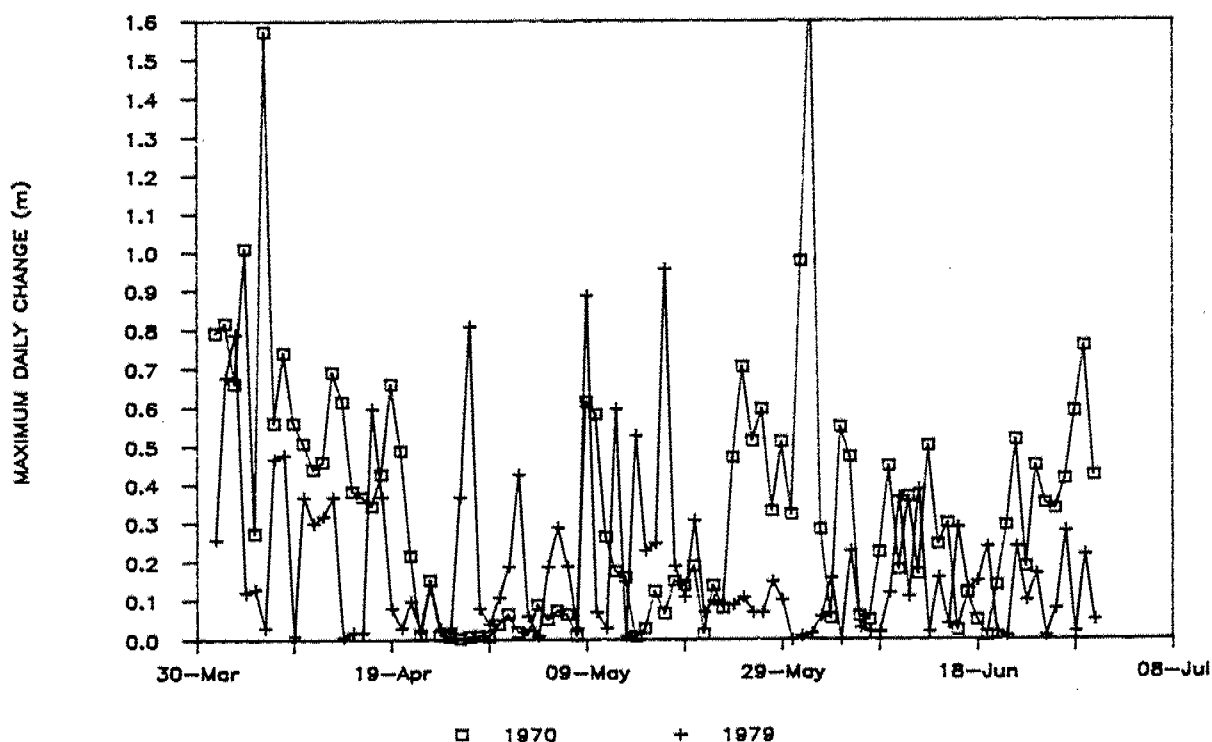
# DAILY WATER LEVEL FLUCTUATIONS-REGULATED RIVER ( FIRST AND SECOND QUADRANT )

FIGURE 20

MISSISSAGI RIVER AT MISSISSAGI CHUTES



MISSISSAGI RIVER AT MISSISSAGI CHUTES



Date OCT. 28, 1985  
Project 841-1356-1

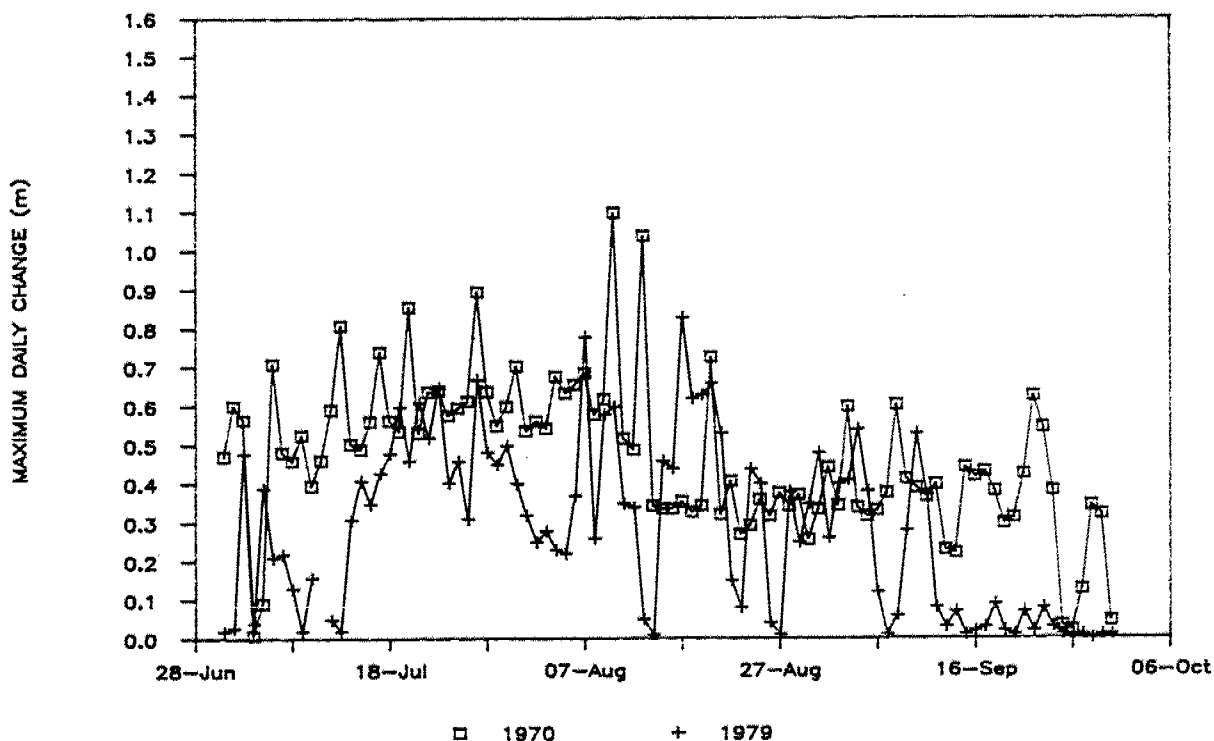
**Golder Associates**

Data by ASP  
Checked by -----

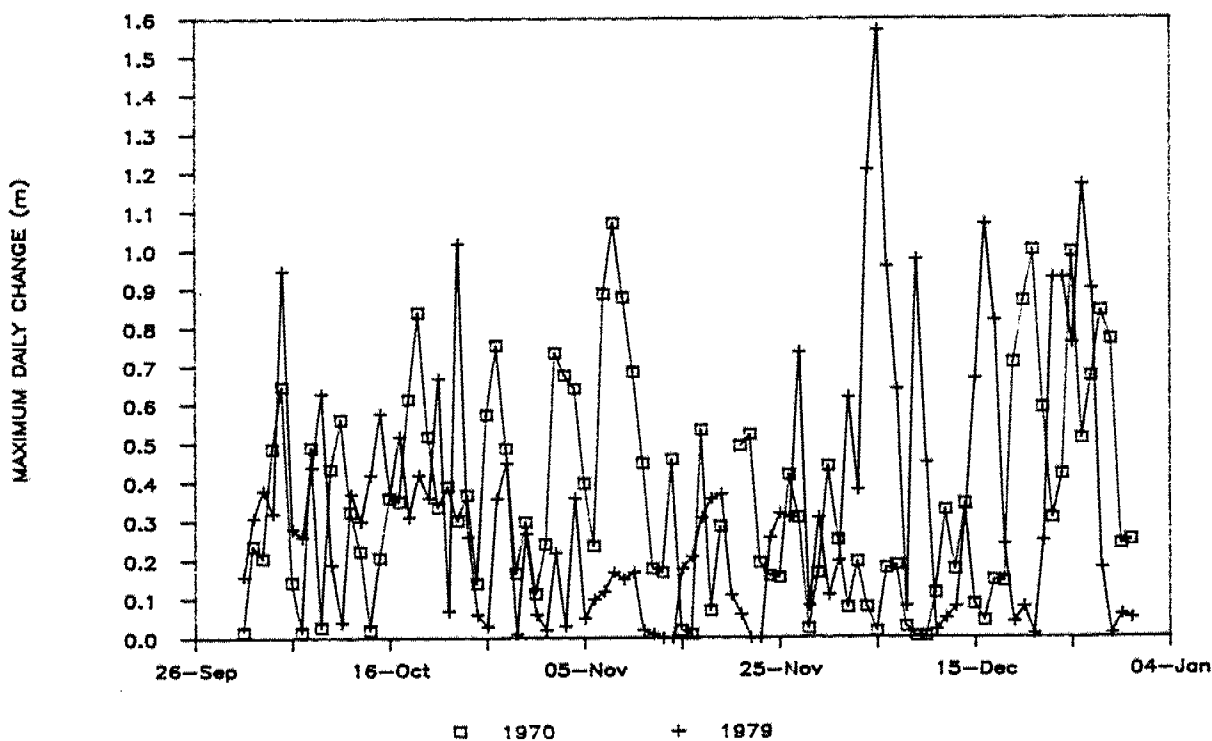
# DAILY WATER LEVEL FLUCTUATIONS-REGULATED RIVER (THIRD AND FOURTH QUADRANT)

FIGURE 21

## MISSISSAGI RIVER AT MISSISSAGI CHUTES



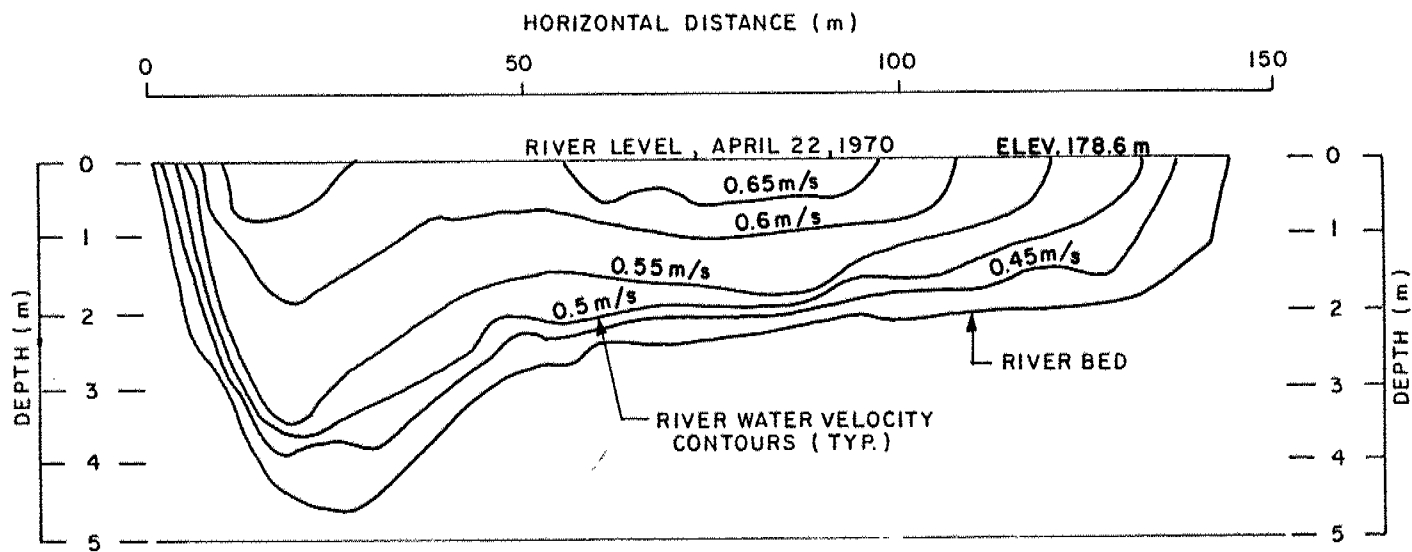
## MISSISSAGI RIVER AT MISSISSAGI CHUTES



Date OCT. 28, 1985  
Project 841-1356-J

**Golder Associates**

Data by ASP  
Checked by -----



MISSISSAGI RIVER CROSS-SECTION NEAR MISSISSAGI  
CHUTES (LOOKING DOWNSTREAM)

HORIZONTAL SCALE 1:1000 ; VERTICAL SCALE 1:100

NOTE: PROFILING BY WATER SURVEY OF CANADA  
LOCATION NOT PRECISELY DEFINED.

Date OCT. 22, 1985  
Project 841-1356-1

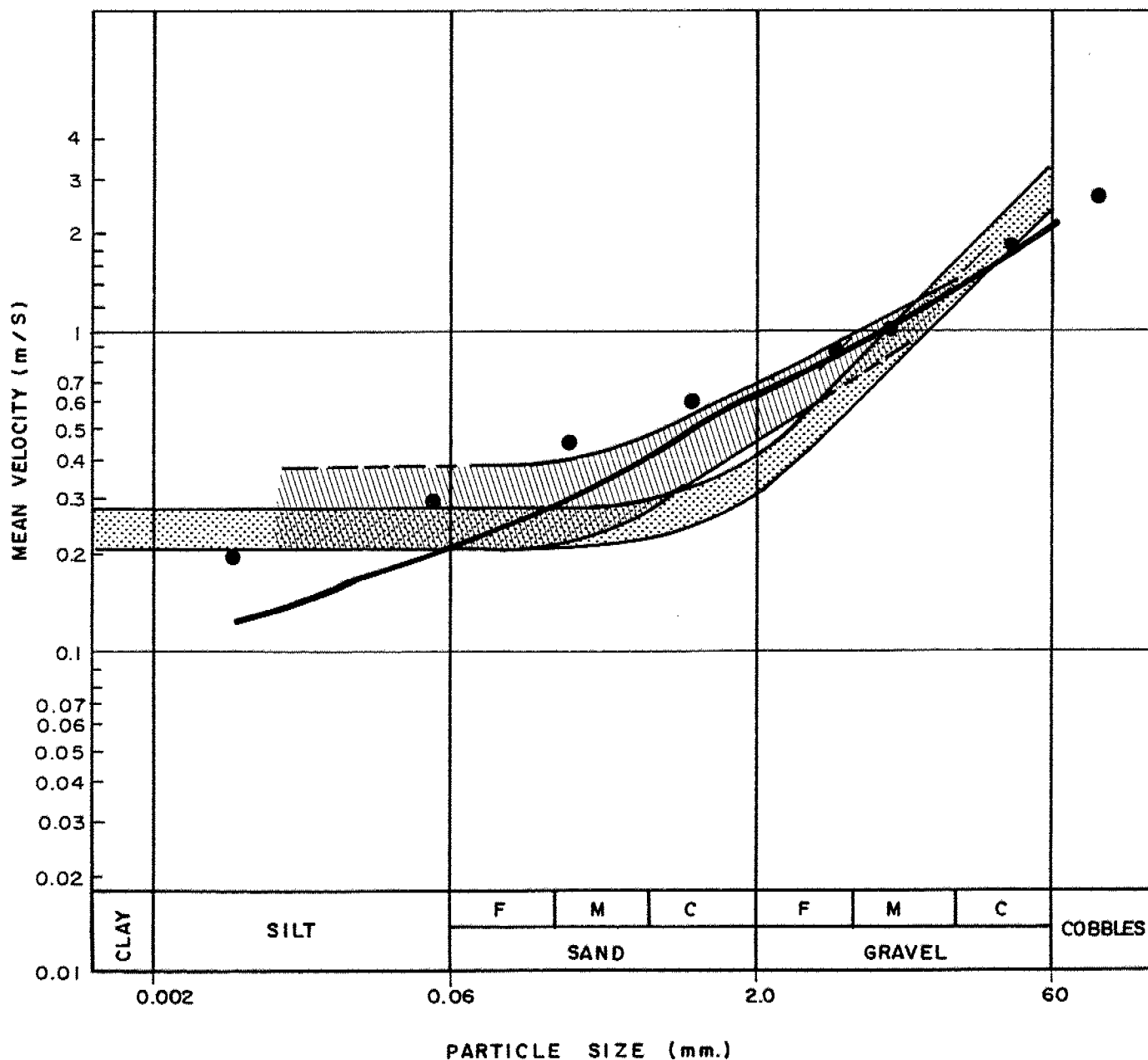
Golder Associates

DROWN MHW  
Chkd. ASR



# OVERSIZE DRAWING(S)

NOTE: VELOCITIES HAVE NOT BEEN CORRECTED  
FOR CHANNEL DEPTH OR FOR CHANNEL  
CURVATURE.



## LEGEND



SIMPLIFIED CONDITIONS FOR EROSION OF UNCONSOLIDATED  
SEDIMENTS (AFTER KENNEY 1975)



CRITICAL EROSION VELOCITY CESSATION OF MOVEMENT  
(AFTER DERBYSHIRE 1979)



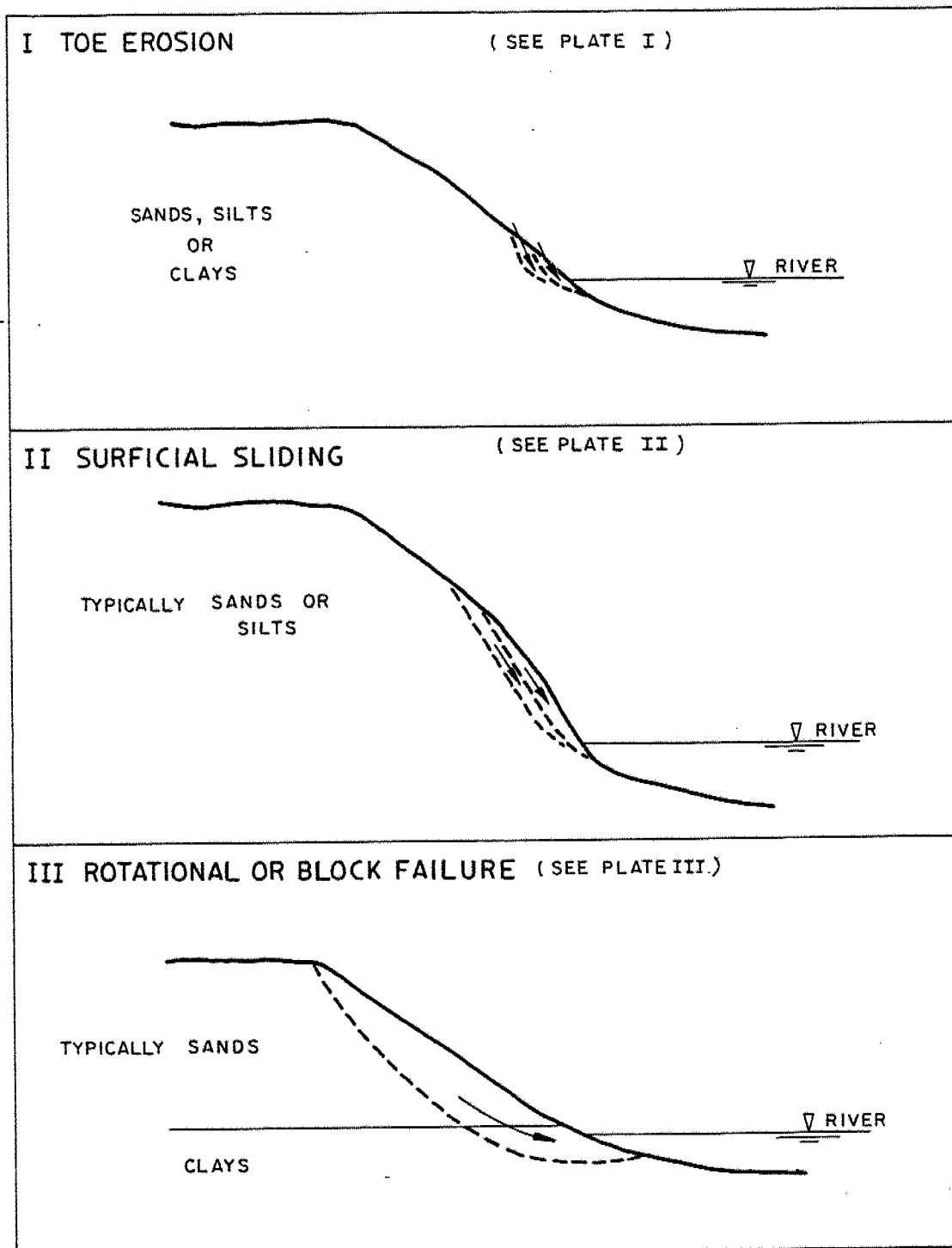
PERMISSIBLE VELOCITIES FOR NON COHESIVE SOILS  
(RUSSIAN WORK, REPRODUCED IN CHOW, 1959)



MAXIMUM PERMISSIBLE VELOCITIES (RUSSIAN WORK, REPRODUCED  
IN KINOKI, 1970).

SCHEMATICS SHOWING  
GENERALIZED FAILURE MECHANISMS

FIGURE 25



Date OCT. 15, 1985  
Project 841-1356-1

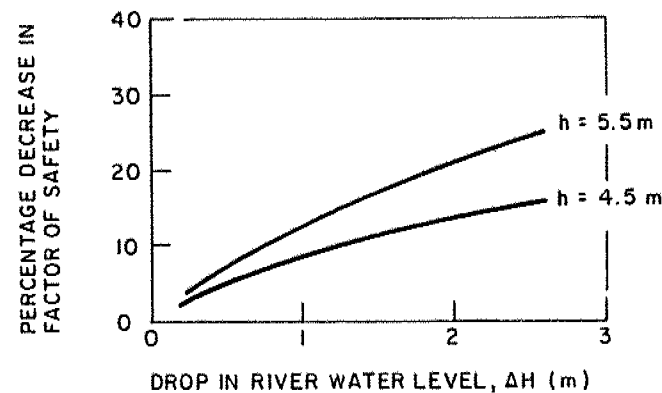
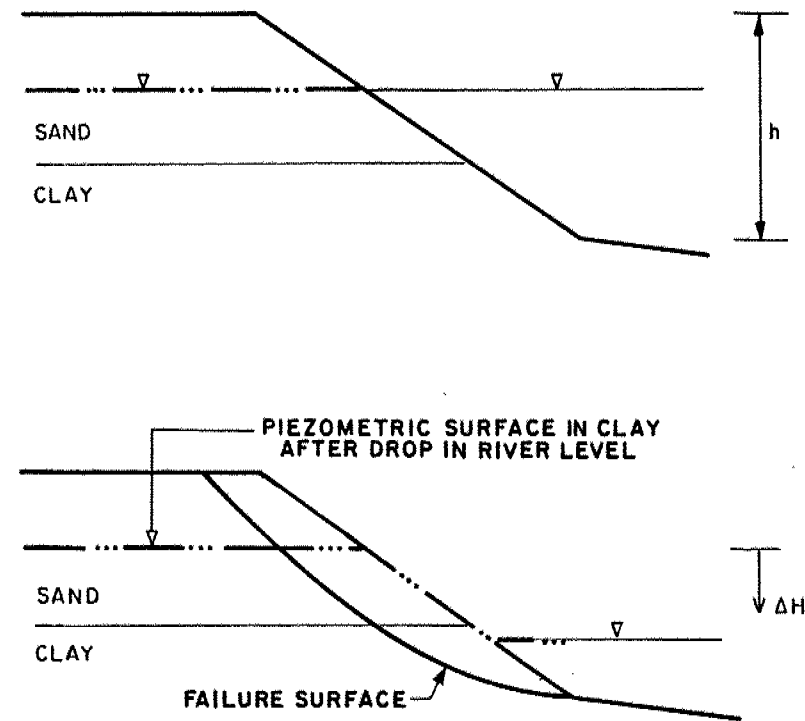
**Golder Associates**

Drawn MHW  
Chkd. BSL

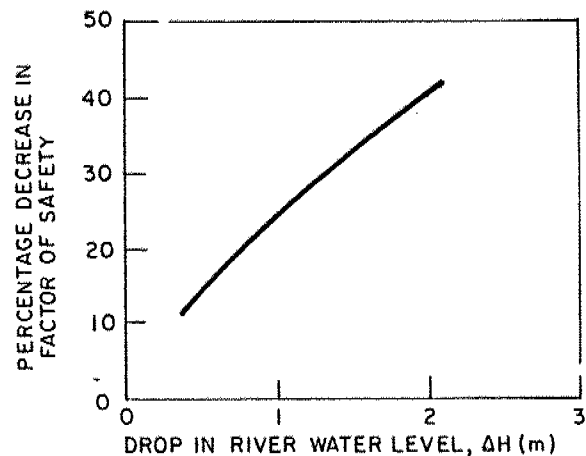
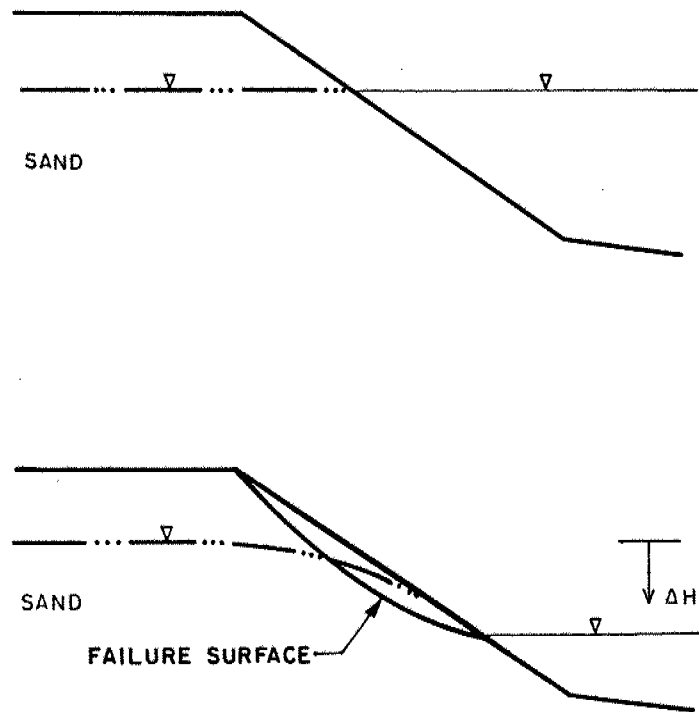
**SCHEMATIC SUMMARY OF STABILITY ANALYSES FOR SLOPES SUBJECT TO LARGE WATER LEVEL FLUCTUATIONS**

**FIGURE 26**

**TYPE III**



**TYPE II (OR III)**



Date OCT. 22, 1985  
Project 841-1356-1

**Golder Associates**

Drawn: MHW  
Checked: ASL

APPENDIX A  
LIST OF DATA REVIEWED

November, 1985

841-1356-1

Golder Associates

REPORTS

- i) Mississagi River Erosion Control Program, Environmental Assessment; Design and Development Division, Ontario Hydro, May 1982.
- ii) Memorandum from Engineering Materials Office regarding Slope Stability, Highway 17, 4.4 km East of Iron Bridge Structure, Easterly 15 km, District 18, Sault Ste. Marie; Ministry of Transportation and Communications of Ontario, February 20, 1984.
- iii) Remote Sensing Report on Bank Slope Stability - Mississagi River Adjacent to Highway 17, District of Algoma. Ministry of Transportation and Communications of Ontario, Thunder Bay, May 5, 1984.
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- vi) Remote Sensing Study, Bank Stabilization, Mississagi River, Iron Bridge-Mississagi Chutes; Ministry of Transportation and Communications, September 18, 1985.

MAPPING

- vii) Mississagi River - Red Rock Falls Generating Station, Erosion Studies River Bank Mapping, Sheets 4 through 9 of 9, Ontario Hydro Dwg. 992 DXH 10120 0120 to 992 DXH 10120 0125, mapped in 1980.
- viii) Mississagi River Bank Cross-sections from Approximately Iron Bridge to Mississagi Chutes. Surveyed in 1984 at 20 m Intervals of Chainage by the Ministry of Transportation and Communications of Ontario.
- ix) Highway 17 Contract Drawings for 1960 and 1984.
- x) Key Plan of Highway Chainages for Highway 17 near Iron Bridge. Ministry of Transportation and Communications of Ontario, 1984.
- xi) Topographical Mapping in 1:50,000 Scale. Energy, Mines and Resources Map No's. 41J6 and 41J3.
- xii) Northern Ontario Engineering Geology Terrain Study Data Base Map - Thessalon. Ontario Geological Survey Map No. 5007. 1:100,000.

- xiii) Air Photo Couplets for 1947.
- xiv) Air Photo Mosaics in 1:10,000 Scale for Years 1978, 1984.

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Station No. 02CC004: Dean Lake Bridge

- xv) 1927, 1928, 1937, 1947: Daily; Department of Water Power Branch, Ottawa
- xvi) 1983: Hourly; Ontario Hydro

Station No. 02CC008: Mississagi Chutes

- xvii) 1970, 1973, 1976, 1978, 1979, 1982: Historical Streamflow Summary; Inland Waters Directorate, Water Surveys of Canada
- vxiii) 1970, 1979, 1983: Hourly and Discharge; Inland Waters Directorate, Water Surveys of Canada

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  - Dean Lake Bridge - June 10, 1940
  - Mississagi Chutes - April 22, 1970
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APPENDIX B

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November, 1985

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APPENDIX C

REMOTE SENSING STUDY;  
1980 AIR PHOTOGRAPHY

November, 1985

841-1356-1

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TABLE 1EXPLANATION OF REMOTE SENSING STUDY CLASSIFICATIONS

<u>Type</u>	<u>Classification</u>	<u>Explanation</u>
2a	Recent	Visible signs of bank failure (failure scarps)
1b	Recent, probable	Some signs of erosion or failure are visible, but uncertainty exists due to perspective of photography and obstruction by vegetation
2	Potential, probable	Factors such as slope angle, proximity of thalweg and instability of adjacent slopes indicate erosion or failure could happen
3	Potential, possible	No indication of immediate erosion or failure.

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Berm - benched slope

# OVERSIZE DRAWING(S)

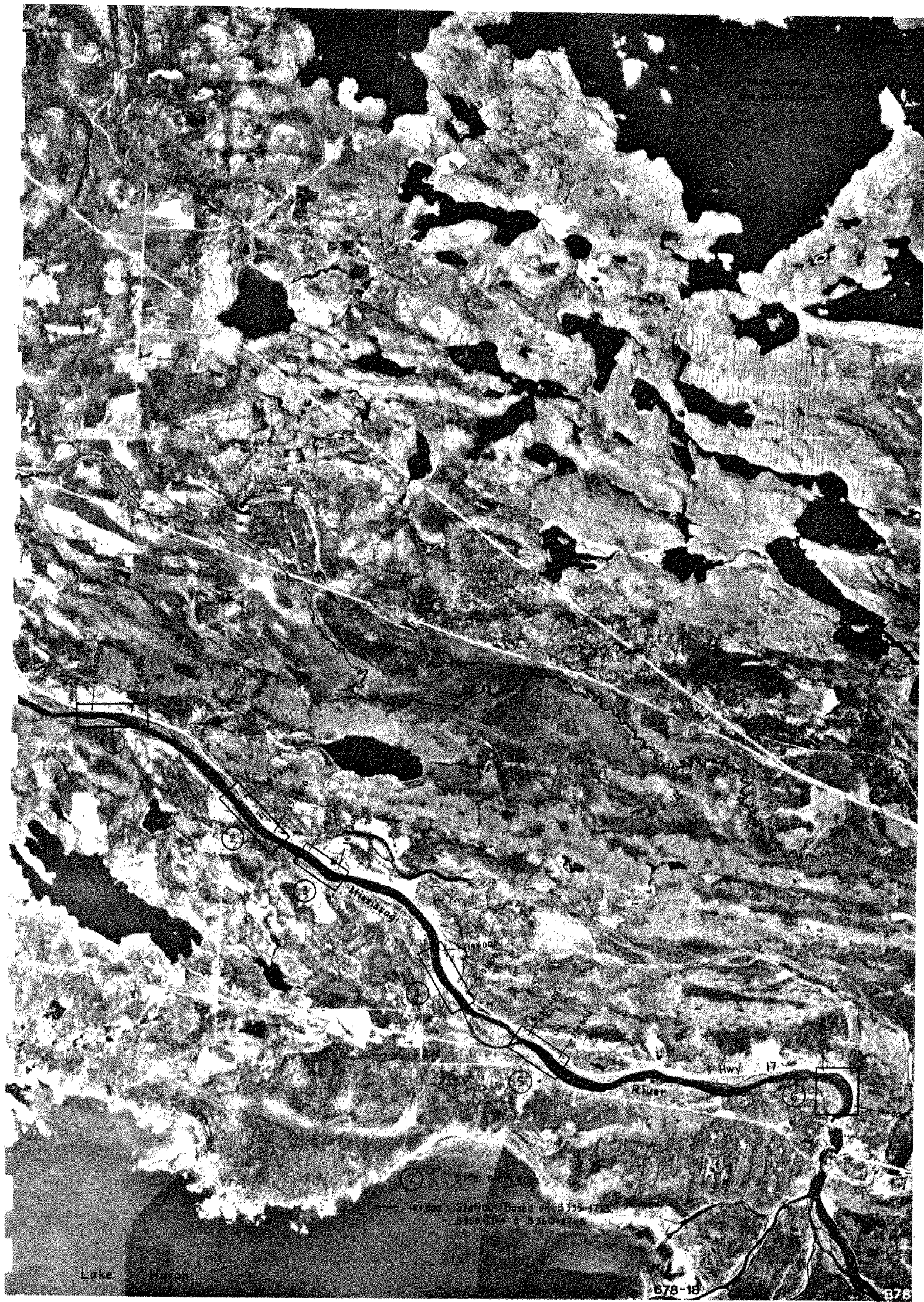
APPENDIX D

REMOTE SENSING STUDY;  
1947, 1957, 1970 and 1980 AIR PHOTOGRAPHY

November, 1985

841-1356-1

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
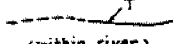

Site number  
14+200 Station Based on B 555-17.3  
B 555-17.3 A B 560-17.3

678-18


678

## Legend

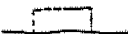
### River Geometry

 estimated	 shoreline
 (within river)	 thalweg

### Bank Erosion

	Type No.	Description
	1a	recent
	1b	recent, probable
	2	potential, probable
	3	potential, possible

### Bank Protection

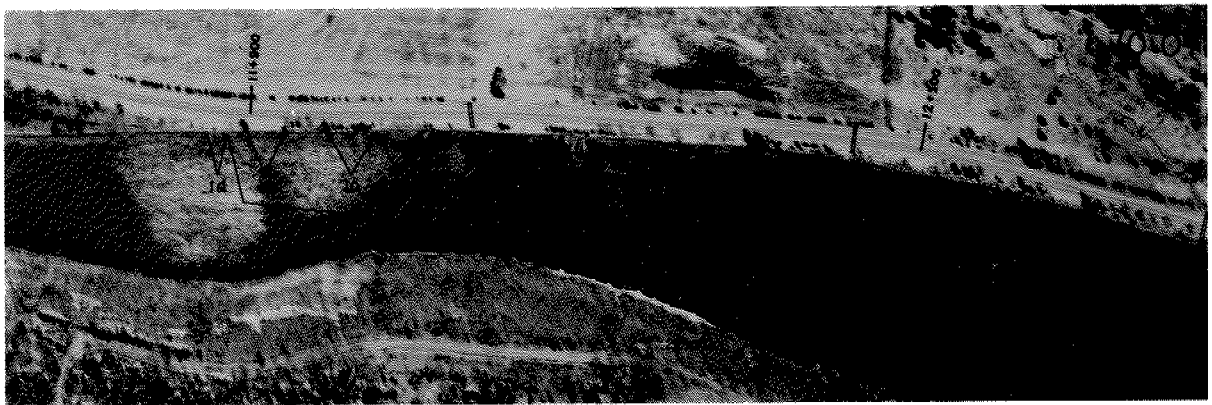
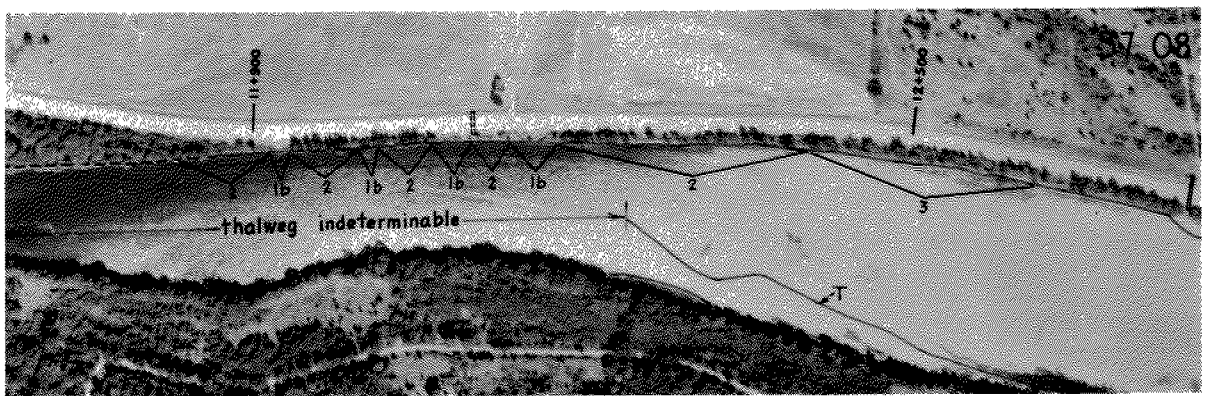
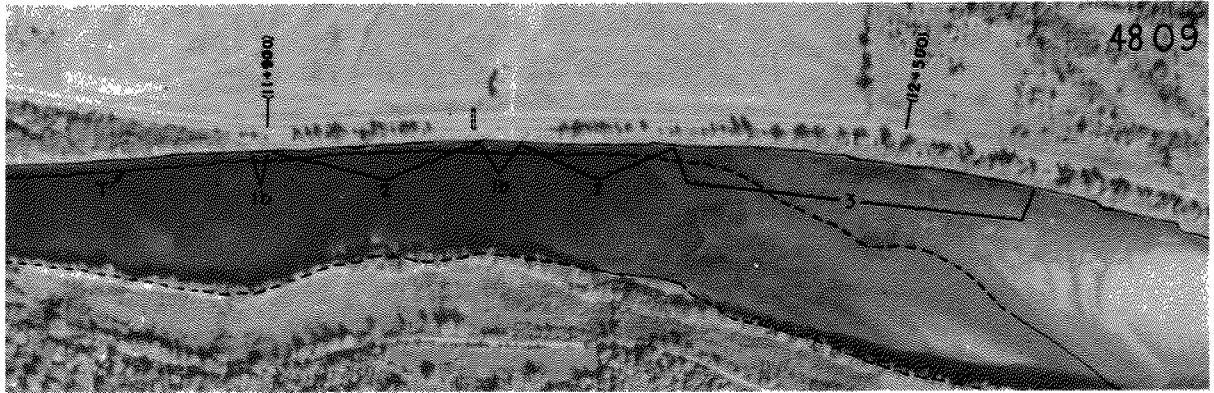
	existing bank protection
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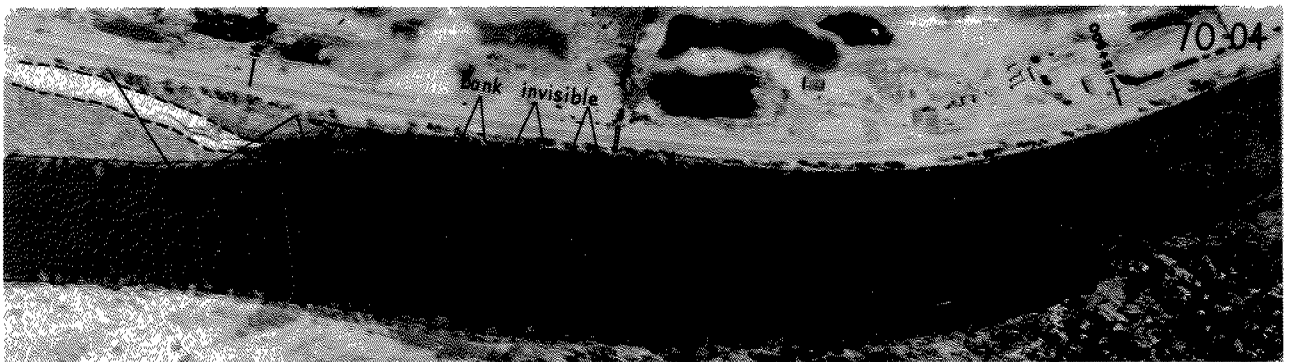
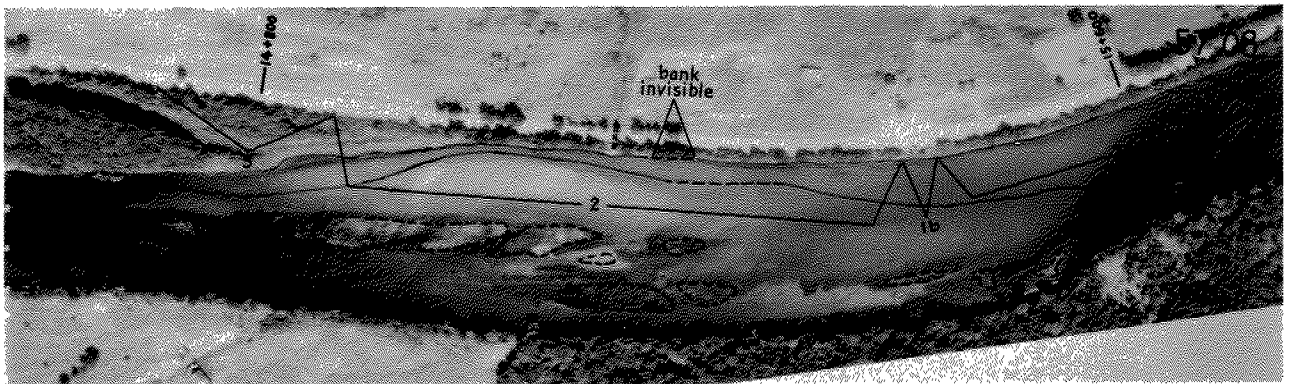
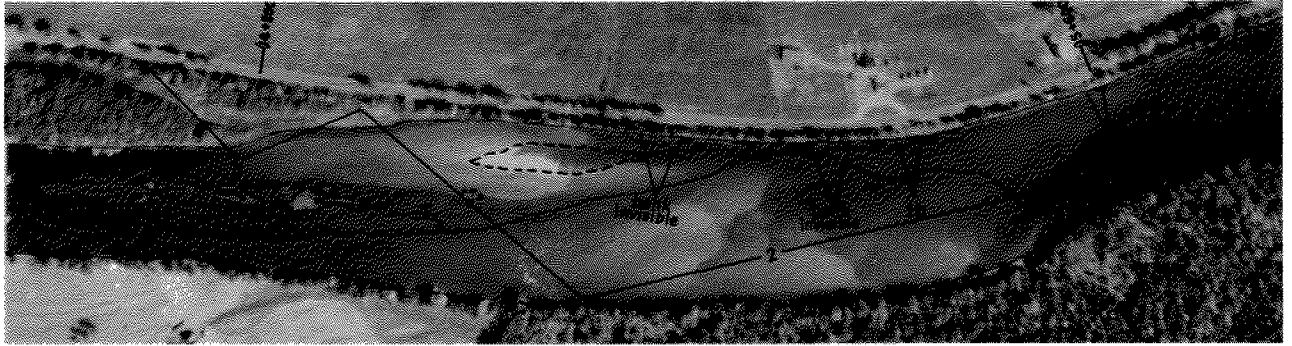
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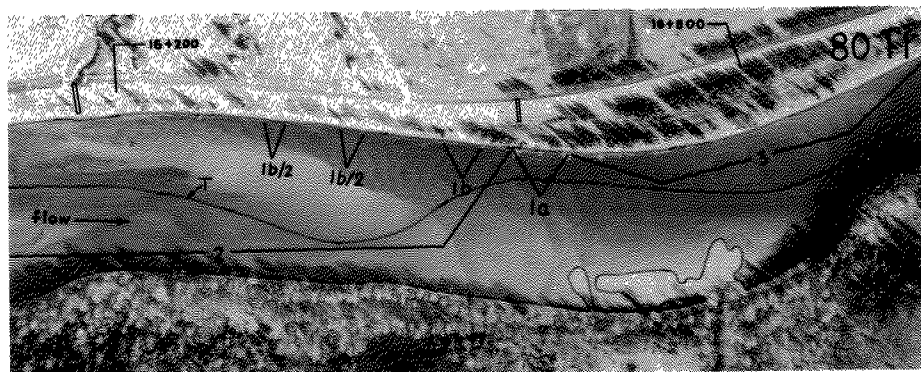
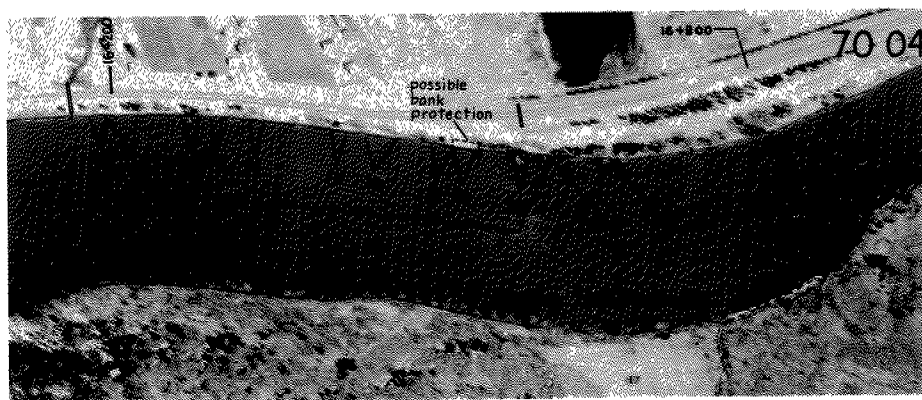
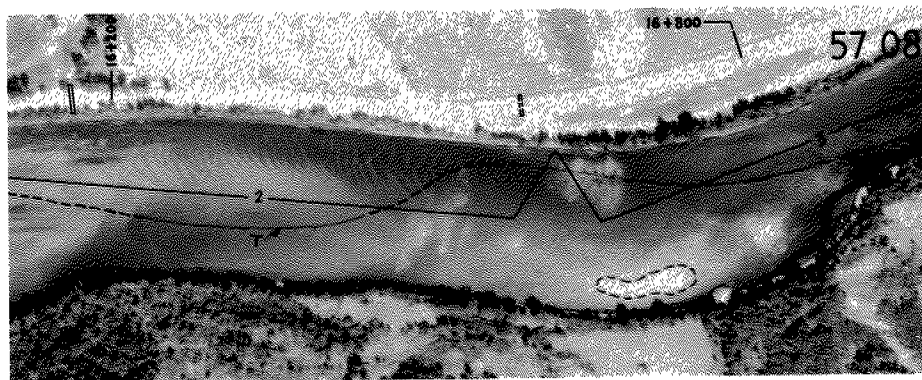
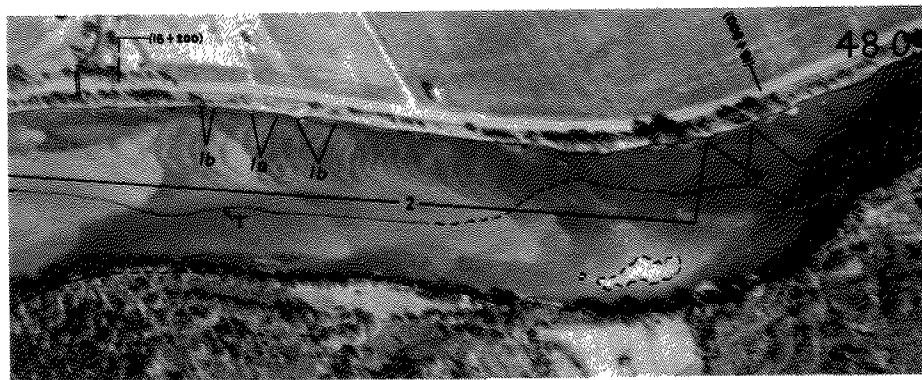
	culvert
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note: stations in 1948 enlargements have been extrapolated from 1982 MTC B-plans onto the old alignment and are in brackets.

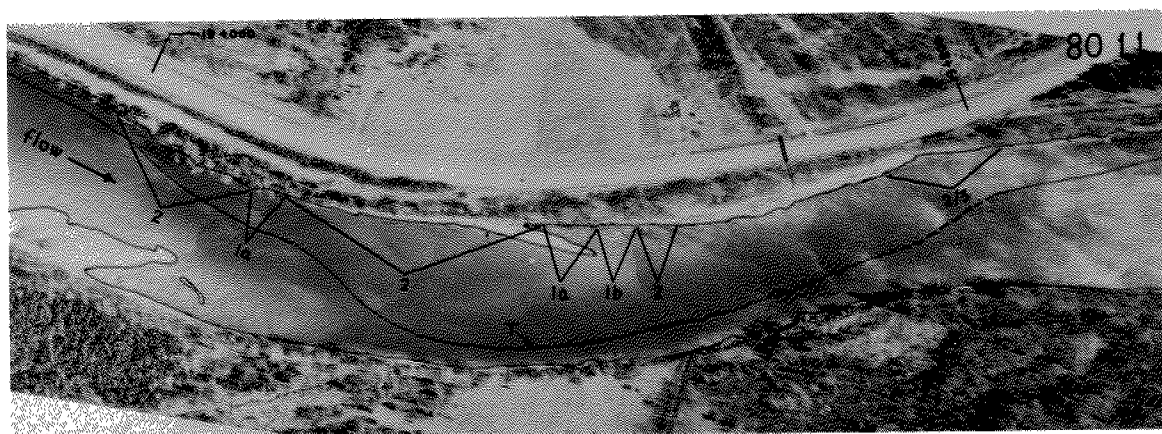
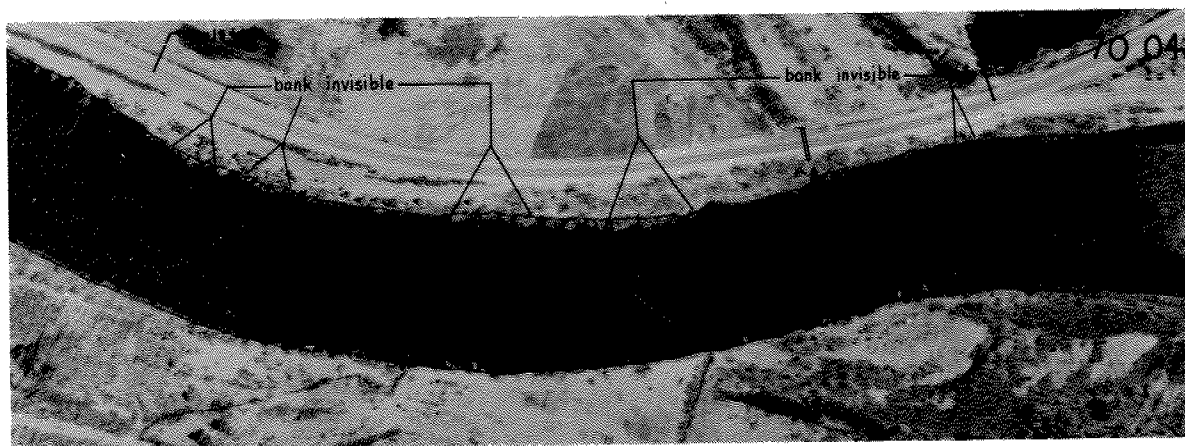
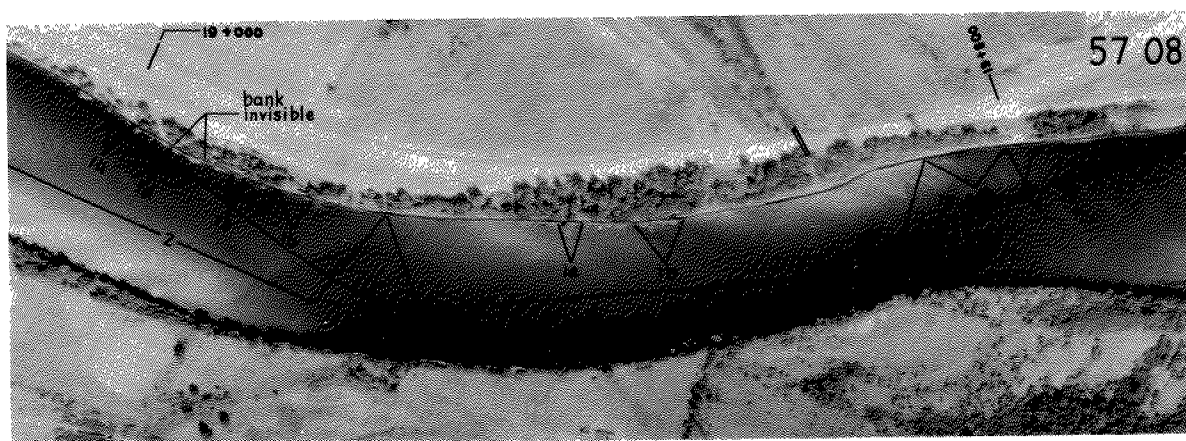


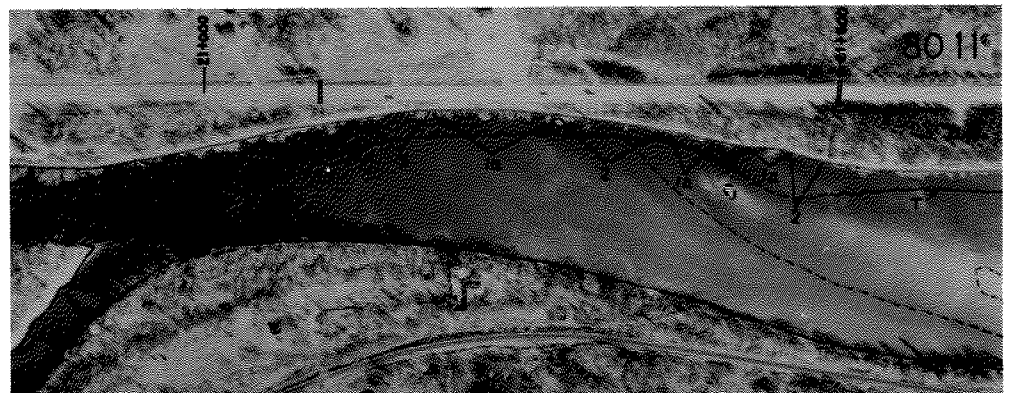
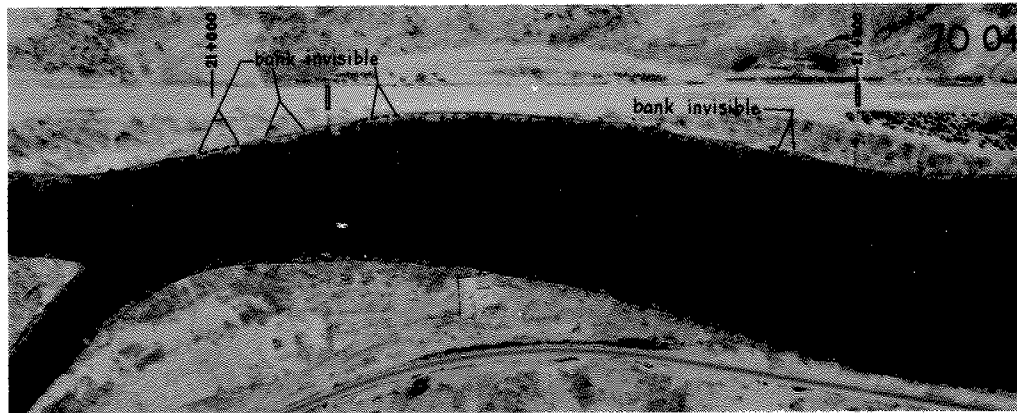
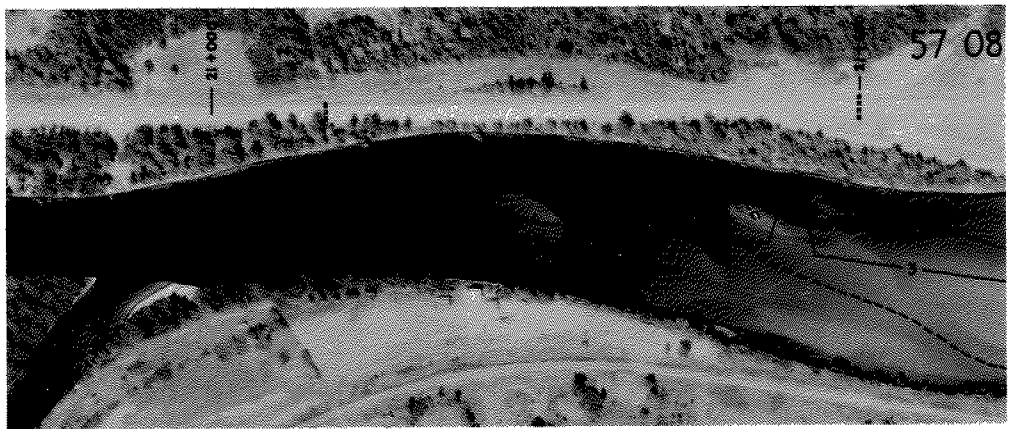
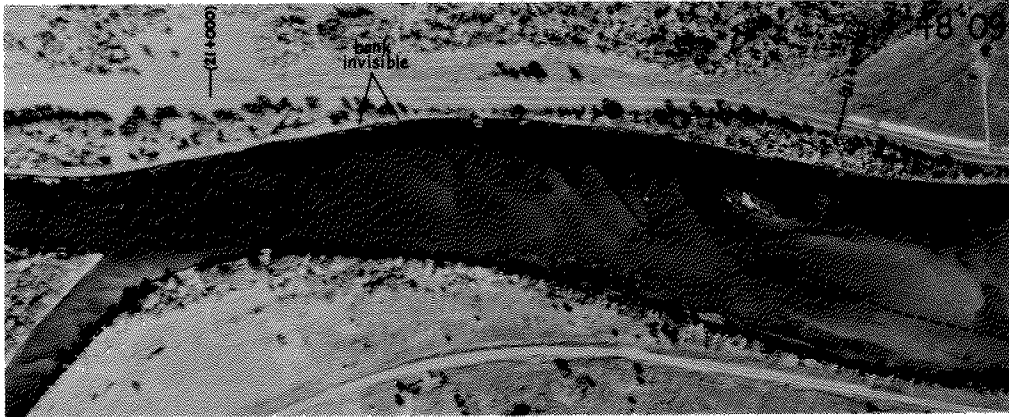




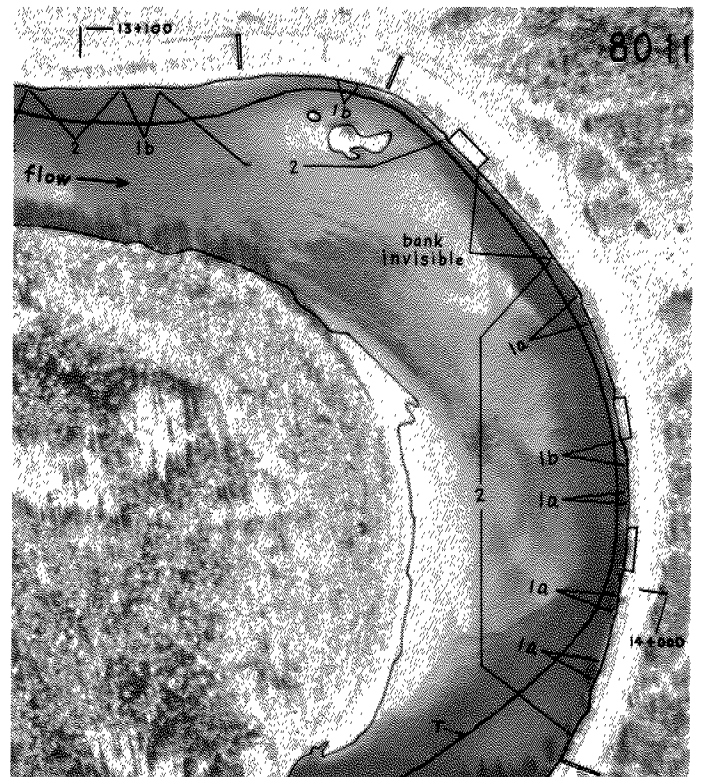
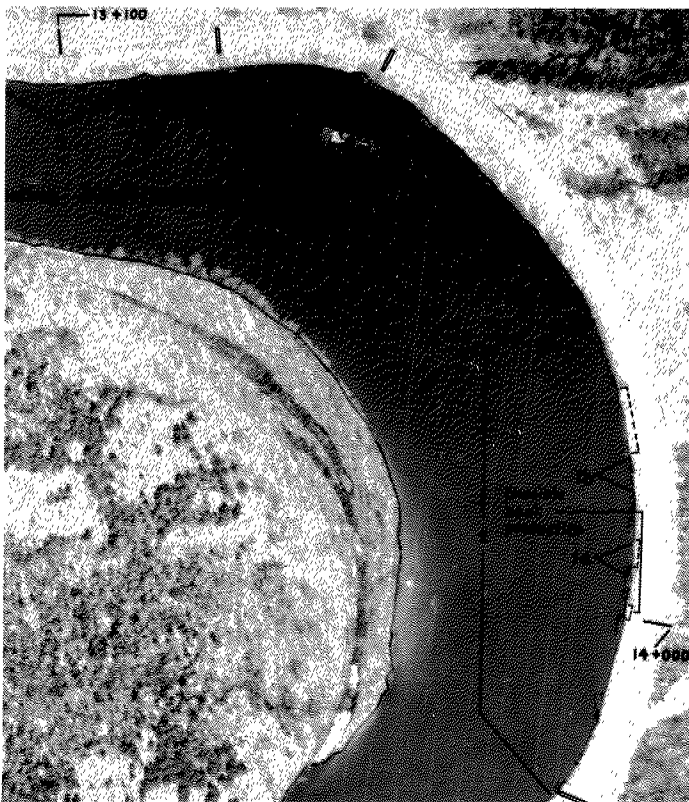
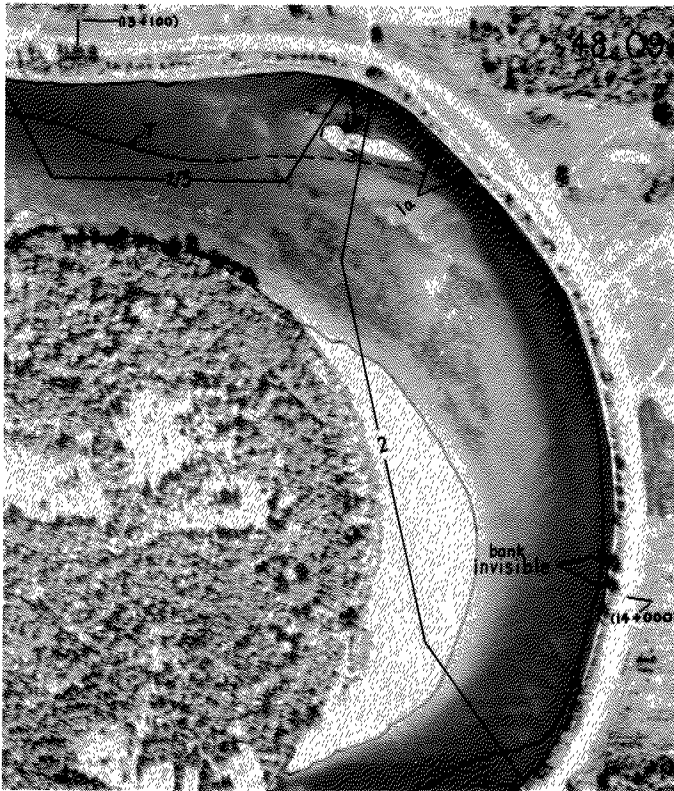












APPENDIX E  
RIVER BANK CONDITIONS  
COMPARISON STUDY

November, 1985

841-1356-1

Golder Associates

COMPARISON STUDY OF SOIL CONDITIONS AND EROSION NOTED ALONG THE  
MISSISSAGI RIVER BANK ADJACENT TO HWY. 17

M.T.C. AIR PHOTO INTERPRETATION (v) 1980 PHOTOGRAPHY REFER TO FIGURES C-1 THROUGH C-3					
			ONTARIO HYDRO 1980 (vii)		
SECTION	CHAINAGE	APPROXIMATE DESCRIPTION	EROSION * CATEGORY	SLOPE DESCRIPTION	EROSION
1	11+675 to 13+320 (Thompson)	Generally a potentially erodable area. - Frequent occurrences of recent erosion	2 (70%)  1a, 1b (30%)	- Bank height 2.5 m to 5 m; slope 35°; fine to medium sand overlying fine sandy silt - Seepage at sand/silt boundary - Trees, root mat, some M.T.C. bank protection	High
2	14+570 to 20+060 (Thompson)	- 0.4 km bank not visible  - Generally potential for erosion - Approximately 20-25% of bank shows indication of recent erosion  - Occurrences tend to be in form of frequent, short (<50 m) stretches	1a, 1b (15%) 2, 3 (85%)  1a, 1b (20%) 2 (80%)  1a, 1b (15%) 2 (65%) Bank Protection (5%)  1a, 1b (25%) 2 (75%)  2 (25%)  1a, 1b (50%) 2 (50%)	- 14+570 - 14+860 - Bank height 0.6 to 2.0 m; slope up to 40°; fine to medium sand overlying silt  - 14+860 - 16+685 - Bank height 4.5 to 6.0 m; slope 30-45°; fine sand overlying silt. Seepage at sand/silt interface. Shrubs and root mat on slope  - 16+685 - 19+050 - Bank height 3 to 5.5 m; slope 40°; fine sand, trace silt over clayey silt. Seepage at interface. Grass and shrubs on slope. Intermittent 30-50 m M.T.C. rock fill bank protection. Max. size 0.8 m diam., mean 0.2 m. Slope 32°.  - 19+050 - 19+495 - Bank height 4 to 4.5 m; slope 30 to 35°; fine to medium sand over clay silt. Seepage at interface. No note of vegetation  - 19+495 - 19+850 - Bank height 1 m; slope 10 to 20°; fine sand over silt. Shrubs and grass on slope.  - 19+850 - 20+060 - Bank height 2.5 to 3.0 m; slope 40°; fine sand over clayey silt. Seepage at interface	Minor near 14+570  Moderate  Minor to Moderate Minimum  High  Minor  Moderate
3	20+365 to 21+580 (Thompson)	- Alternating sections of potential erosion and areas of recent erosion  - 20+515 - 20+530 approx. have had some treatment	1a (25%) 2, 3 (75%)  Bank Protection  Bedrock 1b (50%) 3 (50%)  1a, 1b (30%) 2, 3 (70%)	- 20+365 - 20+560 - Bank height 2.5 to 3 m; slope 40°; fine sand over clayey silt. Seepage at interface. Trees and root mat noted  - 20+560 - 20+670 - Bank height 4.6 m; slope 42°; rock fill - max. size 1.2 m diam., mean 0.2 m  - 20+670 - 20+685 - rock outcrop  - 20+685 - 20+765 - Bank height 1.6 to 3.4 m; clayey silt. Cobbles and boulders in lower 0.6 to 1.5 m.  - 20+765 - 21+580 - Bank height 2 to 3 m; slope 35 to 40°; fine sand overlying clay silt. Seepage at interface. No vegetation shown	Moderate  Minor  Nil  Minor  Moderate
4	10+000 to 10+520 (Cobden)	- Generally potential erosion areas - Frequent recent erosion locations	1a, 1b (30%) 2 (70%)	- 10+000 - 10+520 - Bank height 2 m; slope 35 to 40°; fine sand over clayey silt. Seepage from bottom of sand layer	Minor
5	12+440 to 14+725 (Cobden)	- Generally a potential erosion area  - Some existing bank protection in place  - Approx. 400 m of recent failures in frequent occurrences of 50 m or less in length  - Thalweg generally follows shoreline	1a (55%) 2, 3 (45%)  2  1a, 1b (15%) Bank Protection (15%)  2, 3 (70%)  1a, 1b (20%) 2 (50%) Bank Protection (15%)	- 12+440 - 12+800 - Bank height 2 m; slope 35°; fine sand over clayey silt. Seepage at interface. No vegetation noted.  - 12+800 - 12+900 - Bedrock outcrop  - 12+900 - 13+250 - Bank height 2 to 2.5 m; slope 35°; fine sand over clayey silt. Seepage at interface. Trees and root mat on slope.  - 13+250 - 14+725 - Bank height 3.0 to 3.5 m; slope 35-40°; sand with some gravel (fill) over clayey silt. Seepage at interface.  - M.T.C. rock fill surface protection noted at several places	Minor  Nil  Minor  High

\* Percentages given reflect those portions of the river bank to which the erosion categories are applicable. The remainder within each reach encompass both bedrock slopes and those areas which could not be discernibly categorized due to air photograph quality.



TABLE E-2

## RIVER BANK CONDITIONS

GOLDER ASSOCIATES					GOLDER ASSOCIATES			
Interpretation of MTC River Bank Cross-sections, 1984 (viii)					River Bank Inspection, 1985			
SECTION AND CHAINAGE	SUB-CHAINAGE	PROFILE		APPARENT EROSION *	SUB-CHAINAGE	EXPOSED BANK HEIGHT	EROSION TYPE	COMMENT
		Total Slope Height (m)	Average Slope (degrees to horizontal)	(percentage of cross- sections)				
1. 11+675 to 13+320 (Thompson)	12+220 to 13+000	5 + 6.5	20 + 50	I 40% II/III 10%	-	M + H	I/III	
2. 14+570 to 20+060 (Thompson)	15+000 to 16+340  17+180 to 18+900	4 + 7  4.5 + 6	25 + 35  25 + 50	I 30% II 20% III 15%  I 25% II 20% III 10%	14+570 to 14+860 14+860 to 16+685 16+685 to 19+050 19+050 to 19+495  19+495 to 19+850  19+850 to 20+060	L + M  M + H  H  M + H  L  H	I/II  I/II/III  I/II/III  I/II  I  II	Intermittent bank protection (rip rap)
3. 20+365 to 21+580 (Thompson)	20+485 to 20+670  21+140 to 21+340	6.5  6 + 7	30  25 + 30	I/II 10%  I 20% II 60%	20+365 to 20+500 20+500 to 20+680  20+680 to 21+580	M + H   M	I   II	Bank protection (rip rap)
4. 10+000 to 10+520 (Cobden)	10+300 to 10+540	4.5	20	I 15%		M	I/II	
5. 12+440 to 14+725	12+860 to 13+600  13+600 to 14+160	3.5 + 6  5 + 7	20 + 45  25 + 45	I 45% II 10%  I 20% II 20% III 10%	12+440 to 12+800 12+800 to 12+950 12+950 to 13+400 13+400 to 14+725	L + M  M  L + M  M	I/III    I  II	Bank protection (rip rap)  Intermittent bank protection (rip rap)

\* Apparent type of erosion determined from detailed slope profiles surveyed by MTC. Percentages reflect the number of cross-sections which exhibited erosion.

APPENDIX F  
AREAS OF SEVERE EROSION

November, 1985

841-1356-1

Golder Associates

The Ministry of Transportation and Communications chainages for those sections of the Mississagi River banks adjacent to Highway 17 which exhibit a high proportion of slope failures are listed below. Within certain of these stretches, where the river bank crest is in close proximity of the highway, slope failures or tension cracks have encroached on and exposed the roadway base course. At these locations, the failure mechanisms within the 5 to 7 m high slopes include surficial sliding and/or rotational failures.

LIST OF SECTIONS OF RIVER BANKS EXHIBITING SEVERE EROSION

- o Station 12+200 - 12+400 (Thompson)
- o Station 15+500 - 16+000 (Thompson)
- o Station 17+100 - 18+500 (Thompson)
- o Station 19+000 - 20+000 (Thompson)
- o Station 21+100 - 21+800 (Thompson)
- o Station 12+300 - 12+800 (Cobden)
- o Station 13+300 - 13+800 (Cobden)

ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION

WP 280-85-01

DIST 18

HWY 17

STR SITE N/A

MISSISSAGI RIVER REMEDIAL MEASURES

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GEOCRES 41J-47

DATE ~~FORN~~ 10 1986

FOUNDATION INVESTIGATION REPORT  
For

W. P. 280-85-01, Site N/A

MISSISSAGI RIVER REMEDIAL MEASURES

Hwy. 17, District 18, Sault Ste. Marie

INTRODUCTION

This report summarizes the results of a foundation investigation required for the design of remedial measures along the Mississagi River between Iron Bridge and Mississagi.

The foundation investigation was initiated at the request of the Northwestern Region Geotechnical Section. Its purposes are to identify distressed areas of the river bank adjacent to Highway 17, to prioritize these areas with respect to the potential hazard to the highway, and to provide recommendations for remedial measures.

The field work for the investigation was carried out between 86 05 05 and 86 05 08, and consisted of inspections of the unstable river bank sections near Highway 17, from both the river and the highway.

The investigation also involved a review of:

-Relevant data from Phases I and II of the Golder Associates Report on the causes of river bank instability for W. P. 153-83-00 (1985). This data includes air photo analyses of the river banks, observations recorded during the site visit of 85 07 09, and subsurface information obtained in borehole #1 to #9.

-Surveyed cross-sections of Highway 17 at identified distressed areas (1985).

-The Mississagi River Erosion Control Program Environmental Assessment by Ontario Hydro (1984)

-The Soils Design Report by Northwestern Region Geotechnical Section (1984).

## SITE DESCRIPTION

The site is located along approximately 16 km of Highway 17 between the communities of Iron Bridge and Mississagi, in the Townships of Thompson and Cobden. The extent of the area investigated for this report is Sta. 11+000 Thompson Twp. to Sta. 14+200 Cobden Twp. Figure 1 is a map of the study area.

Within the study area, the river banks adjacent to Highway 17 generally range in height from 3 to 7 m with slopes as steep as  $45^{\circ}$  to the horizontal.

## SUBSURFACE CONDITIONS

During their investigation into the causes of river bank instability, Golder Associates advanced 9 boreholes in the study area. The locations of these boreholes are illustrated in Figure 1. Record of Borehole Sheets (BH #1 to #9) are provided in the Appendix along with a summary of Piezometer Installations and Water Level Measurements (Figure 2) and typical Grain Size Distribution Curves for various representative layers of overburden (Sand-Figure 3; Silt to Fine Sandy Silt-Figure 4; Clayey Silt - Figure 5).

Based on an evaluation of these boreholes and subsurface information provided in the Ontario Hydro Environmental Assessment Report and the Northwestern Region Soils Report, it is inferred that the overburden within the study area, although slightly variable from site to site, is generally noncohesive in nature with localized cohesive zones. It generally consists of layers of sands, silts, organic silts and clayey silts.

The boreholes were not advanced to bedrock, but bedrock does outcrop at various locations within the study area. The bedrock is primarily conglomerate, argillate, greywacke, quartzite and siltstone of the Middle Precambrian Gowganda Formation. It contains occasional east-west trending intrusions of Middle Precambrian Nipissing Diabase consisting of diabase, gabbro, metagabbro and granophyre. Faults cross the area in Cobden Township near stations 10+500 and 14+000.

During the field investigation by Golder Associates in May 1985, the groundwater elevation within the Highway 17 embankment was approximately 1 m above river water level, in the zone 5 m from the river's edge.

## DISCUSSION AND RECOMMENDATIONS

### Stability Analysis

The instabilities of the Highway 17 embankments adjacent to the Mississagi River are surficial failures. This assessment has been verified by the observations of site conditions and by the stability analysis of generalized embankment conditions using Bishops Simplified Effective Stress Method. The stability analysis illustrates that the critical failure surface occurs above the toe of the embankment, and is not a deep-seated failure. Refer to Figure 6 for an illustration of the embankment model and a summary of assumptions considered in the stability analysis.

The instabilities are caused primarily by undercutting of the embankment, aggravated by excess pore pressures in the embankment.

For stability analysis, the pore pressure  $u$  can be expressed in terms of the pore pressure ratio  $r_u$  defined by:  $r_u = \frac{u}{\gamma h}$  where  $h$  is the depth of a point below the surface and  $\gamma$  is the bulk density of the soil. This concept permits the modelling of seepage flow in effective stress slope stability analysis and has been incorporated in the solution for the Mississagi River banks.

As illustrated in Figure 7, the stability analysis of generalized embankment conditions treated with rock protection indicates that the surficial failures can be controlled by the remedial measures recommended in this report. Without rock protection the factor of safety was near unity. With the rock protection, the factor of safety was increased to over 1.2. Observations of similar treatments already constructed along Highway 17 verify the effectiveness of the suggested embankment treatment.

### Description of Remedial Sites

Most of the existing instabilities occur on outside river bends, where the erosive power of the river is at its maximum. This report considers remedial treatment for these existing problem areas.

However, except for rock outcrops and those areas that are already treated with embankment protection, all of the river embankments are potentially erodible. The construction of erosion control measures will cause changes in the river that may accelerate erosion in areas that are currently stable. Thus, additional embankment protection requirements may develop.

A total of 7 site (5080 m) that require remedial treatments have been identified. Approximate locations of these sites are illustrated on Figure 1. Specific details of these sites, including;

- survey chainage limits for remedial treatment
- distances between the shoulder of Highway 7 and the crest of the embankment
- approximate embankment heights, and
- slope angles

are presented in Table 1. Typical cross-sections of the Highway 17 embankments for each site are illustrated in Figures 8 to 14. Surveyed cross-sections of Highway 17 covering most of the sites are available. (Refer to rolls of cross-sections).

#### EMBANKMENT TREATMENT

##### Treatment Material

Ontario Hydro has constructed embankment protection, consisting of quarried stone slope protection in the Iron Bridge area. Also 4 unstable areas along Highway 17 have been treated with quarried stone by MTC.

During site visits for this project, the performance of the quarried stone was evaluated and considered to be an effective material for embankment protection at these locations. The following gradation requirement for the quarried stone, proposed by the Northwestern Region Geotechnical Section and similar to the material used by Ontario Hydro, is acceptable for the embankment protection.



Stone Size (mm)	Range (percent smaller)
200	100
150	60-100
106	50-70
75	25-45
37.5	10-25
16	5-15
9.5	0-10
4.75	0-5
75 $\mu$ mm	0-2

This gradation does not meet the theoretical requirements for a filter for the embankment materials. However, a comparison of existing treatments with and without a geotextile filter revealed no appreciable differences in performance. Therefore, in our opinion, a geotextile filter is not required for embankment protection.

#### Treatment Geometry

The recommended configuration for embankment treatment is illustrated in Figure 15. It has been designed, not only to control surficial failures, but also to improve overall slope stability.

The treatment consists of rock protection with the specified quarried rock material. The longitudinal limits for each site are defined by the survey chainages indicated in Table 1. Transversely, the treatment should extend from the specific elevation indicated in Table 1 to armour the slope and form an apron (minimum width of 4 m) along the river bed. The minimum slope for the rock protection is 1.5H:1V. The minimum thicknesses are 0.5 m on the slopes and 1.0 m along the river bed.

#### Slope Preparation

Except for the upper 1 m of the slope treatment, all vegetation should be cut to within 150 mm of the surface and cleared. Within the upper 1 m of slope treatment sound trees that do not interfere with construction should be left in place.

#### Construction

There are 2 approaches to the construction of the embankment treatment -from the highway using trucks, or from the river using barges.

Truck placement would involve the end dumping of quarried stone to form an access road along the toe of the treatment areas followed by reshaping of the stone to the recommended geometry. The disadvantages of this method include the potential disruption of Highway 17 traffic and the additional material required to construct the access road above the river level.

Barge placement would involve the use of shallow draft barges. A placing barge would be required to carry construction equipment. Two or three barges would be required to transport materials. The disadvantages of this method include the need for specialized barge operations and potential problems with insufficient water depth in the river.

#### Vegetation Treatment

Grass cover should be established in those areas of the embankment above the rock protection as soon as possible after placement of the quarried stone.

#### Shoulder and Granular Sub-Base Sealing

In their Soils Report, Northwestern Region has recommended Granular Sealing with RSI or SSI Emulsion on the shoulders and sideslopes to the sub-base level.

#### Priorities and Planning

The conditions of the existing embankments have been evaluated and prioritized according to the potential hazard to Highway 17. The rate of erosion, slope angle, height, and proximity to Highway 17 were considered in this evaluation.

The following schedule is recommended for the specific rehabilitation priorities indicated in Table 1:

Priority 1	1986
Priority 2	by 1987
Priority 3	by 1988

Quantities of materials required can be estimated from the surveyed cross-sections.

CONCLUSION

Some aspects of this report have been previously discussed in our memo dated 1984 02 20 to R. Girard who at that time was head of the Northwestern Region Geotechnical Section. However, we have incorporated a more detailed analysis of slope stabilization into this report, and have updated the extent of areas requiring remedial treatment.

This report was written by Mr. D. Dundas, Senior Foundations Engineer and reviewed by Mr. M. Devata, Chief Foundations Engineer (East).



*D. H. Dundas*

D. H. Dundas, P. Eng.

Senior Foundations Engineer

*M. Devata*

M. Devata, P. Eng.

Chief Foundations Engineer

(East)

## APPENDIX

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{v0}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

WP 280-85-01  
MISSISSAGI RIVER REMEDIAL MEASURES

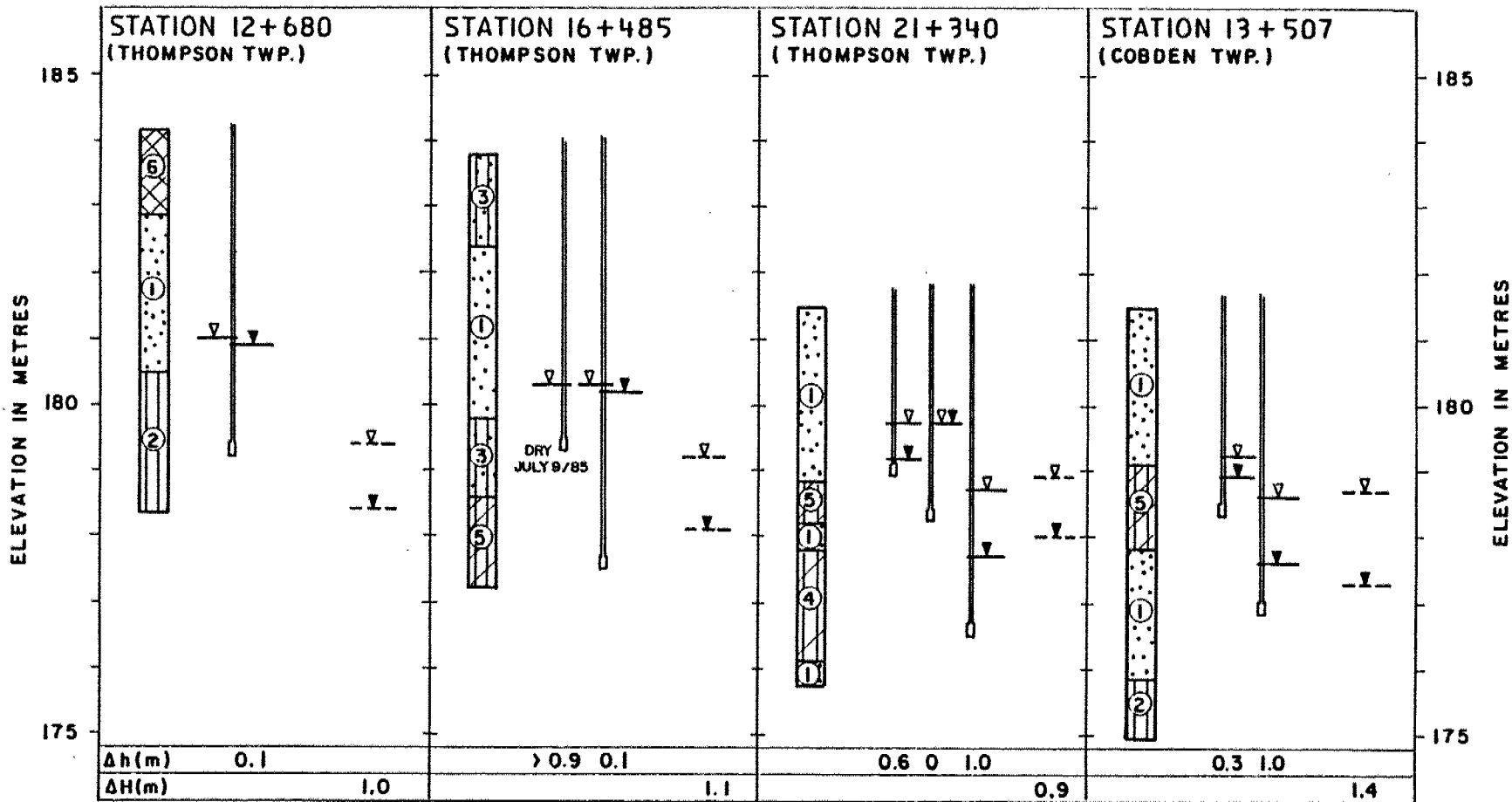
TABLE 1 SUMMARY OF REMEDIAL REQUIREMENTS

Site	Area	LOCATION		Distance Shoulder To Crest± (m)	Embankment Height± (m)	Existing Slope	Rock Protection Elevation (m)	Priority
		Township	Chainage± (m)					
1	A  B	THOMPSON	12+155 to 12+960	0 - 10	5-7	(1.8-2+):1	181	2
2		THOMPSON	15+425 to 16+550	0 - 4	5-7	(1.1-2+):1	181	2
3		THOMPSON	17+180 to 18+510	1 - 10+	4-6	(0.9-2) :1	180.5	1
		THOMPSON	18+510 to 18+800	10+	6-7	(1.5-2) :1	180.5	3
4		THOMPSON	21+150 to 21+350	7 - 10+	6	(1.1-2) :1	180	1
5		COBDEN	10+300 to 10+530	10+	5	(1.4-2+):1	179	3
6		COBDEN	13+320 to 13+610	6 - 10+	4-7	(1.1-2+):1	179	3
7		COBDEN	13+700 to 14+150	1 - 10	5-7	(1.2-2) :1	179	1

# OVERSIZE DRAWING(S)

# SUMMARY OF PIEZOMETER INSTALLATIONS AND WATER LEVEL MEASUREMENTS

FIGURE 2



## LEGEND



GENERALIZED  
STRATIGRAPHY

- ① SAND
- ② SILT
- ③ LAYERED SAND AND SILT
- ④ LAYERED SILT & CLAYEY SILT
- ⑤ CLAYEY SILT
- ⑥ FILL

## WATER LEVEL MEASUREMENT

▽ MAY 25, 1985  
▽ JULY 9, 1985

□ PIEZOMETER TIP

$\Delta h$  - CHANGE IN PIEZOMETRIC  
LEVEL

## RIVER WATER LEVEL

▽ MAY 25, 1985  
▽ JULY 15, 1985

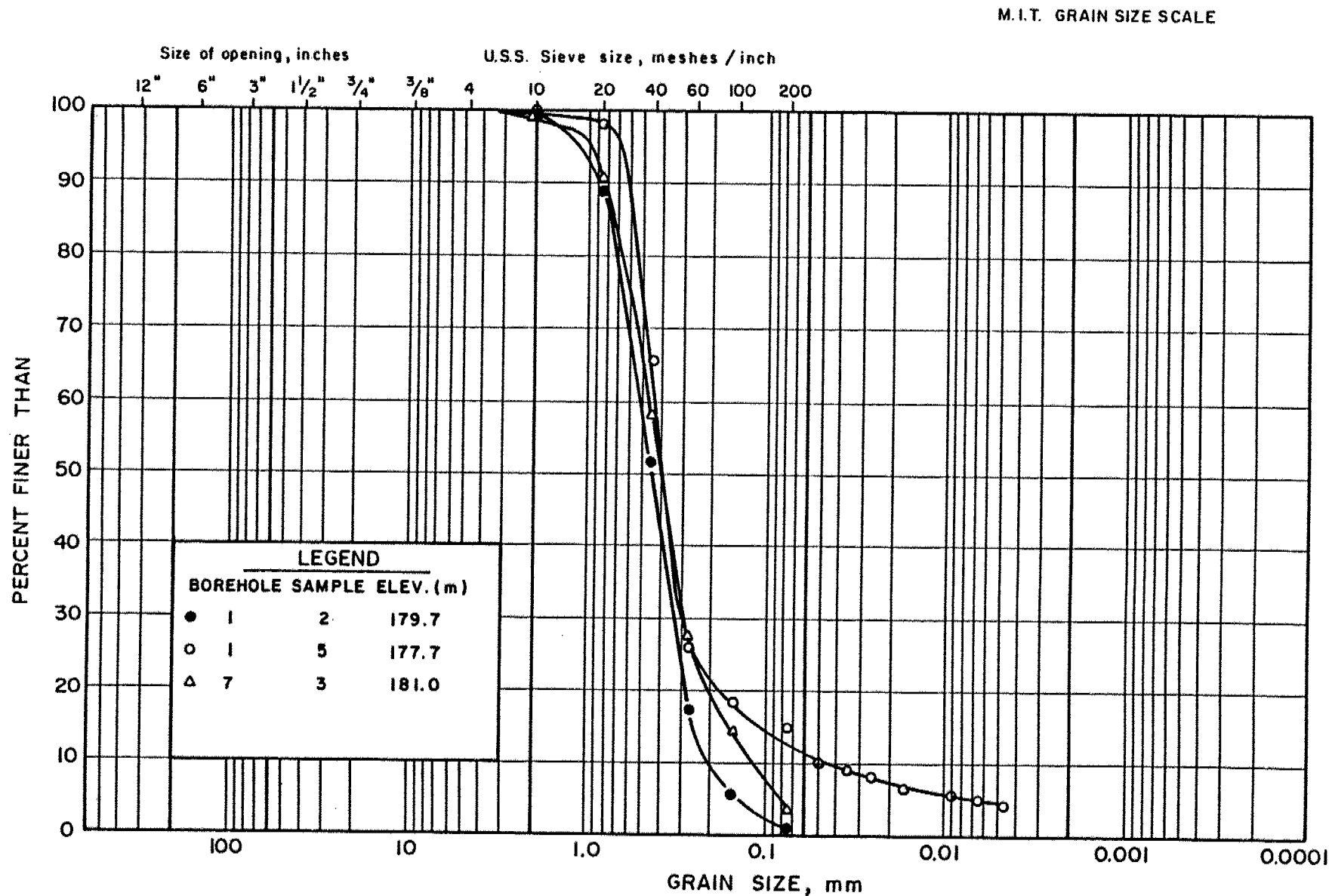
$\Delta H$  - CHANGE IN RIVER  
WATER LEVEL

Date OCT 15, 1985  
Project 841-1356-1

From Golder Associates  
Report for W P 153-83-00

Drawn MHW  
Chkd. ASE

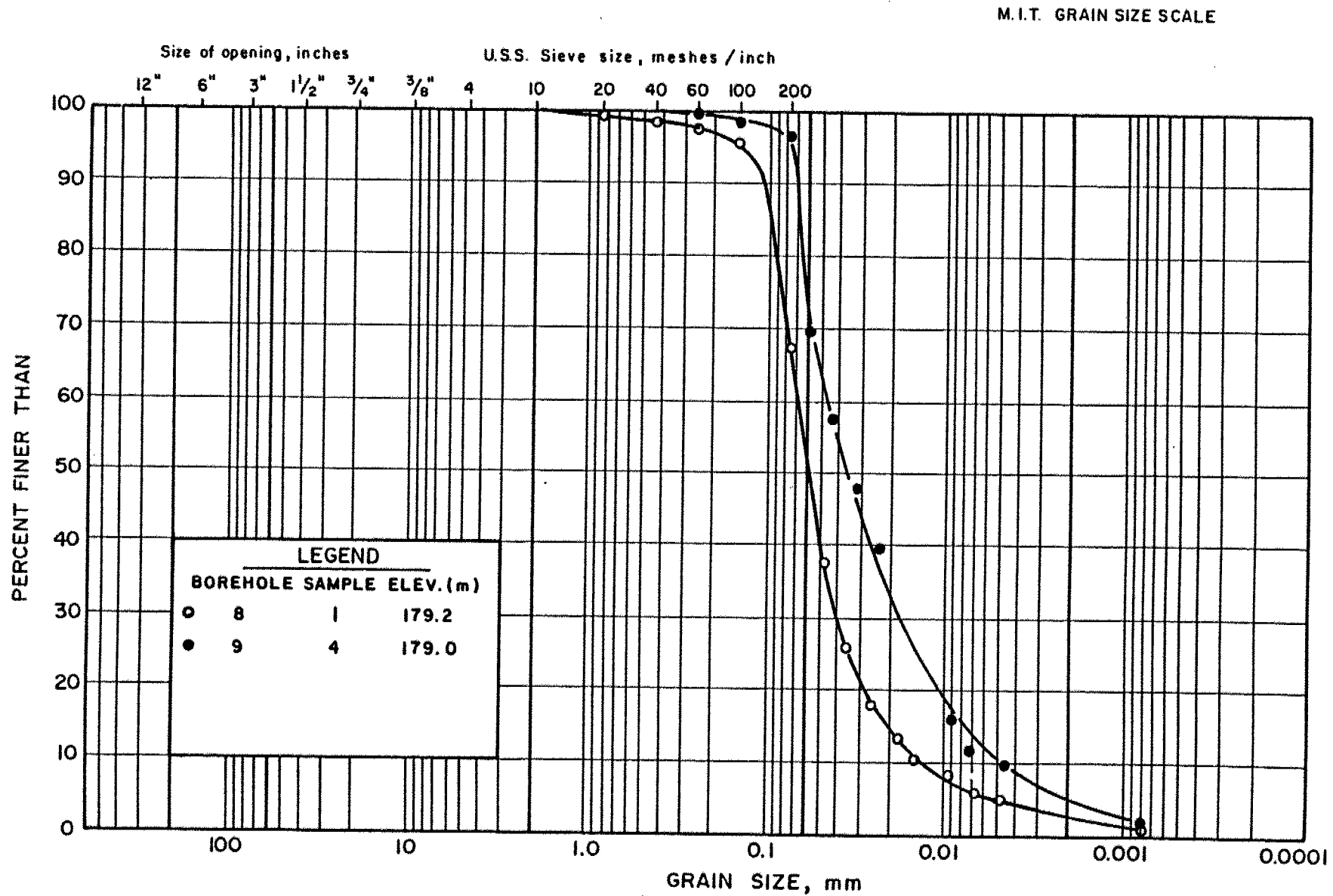




BOULDER SIZE	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
		GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION  
SAND

FIGURE 3



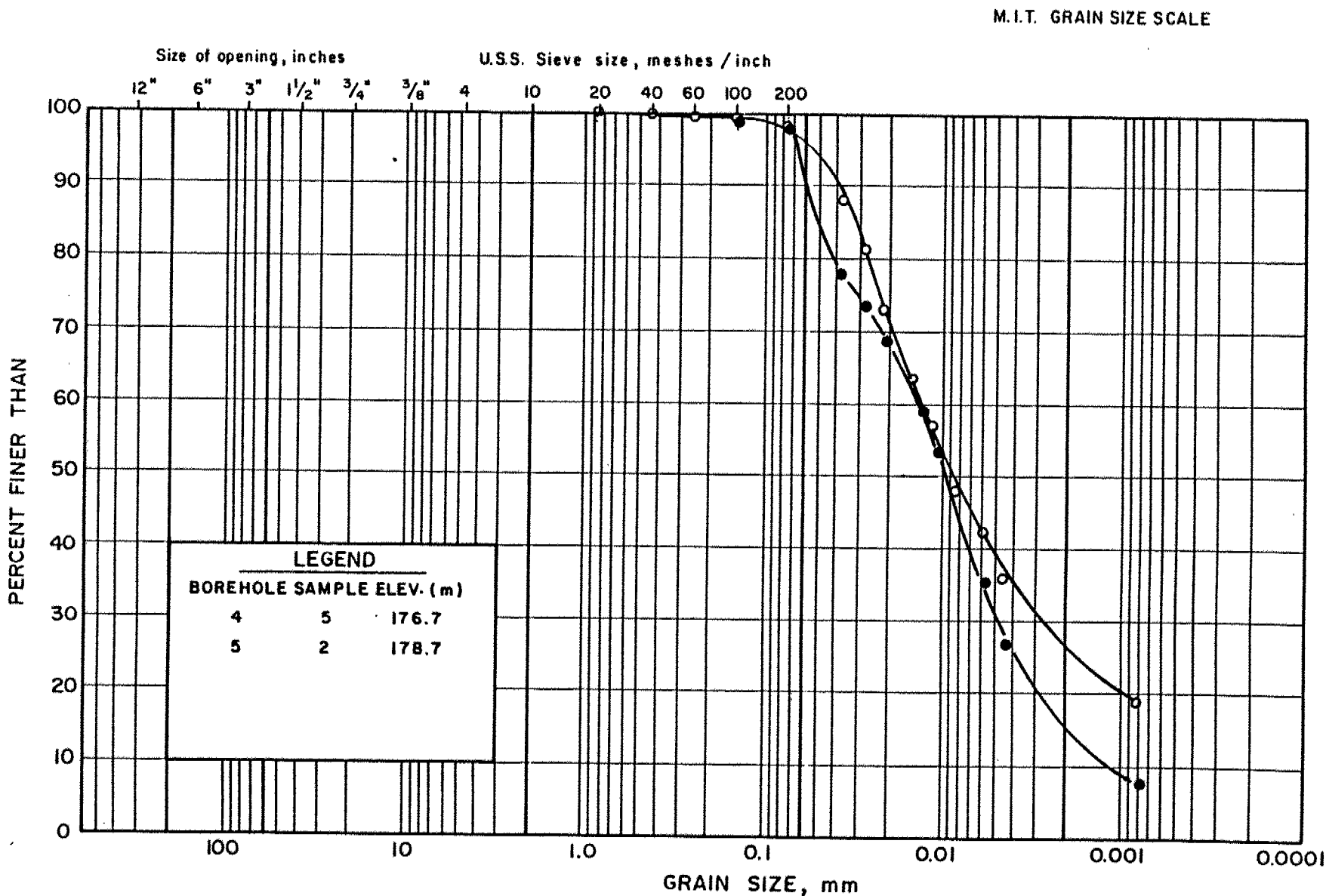
BOULDER SIZE	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
		GRAVEL SIZE			SAND SIZE			FINE GRAINED	

# GRAIN SIZE DISTRIBUTION SILT TO FINE SANDY SILT

FIGURE 4

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT

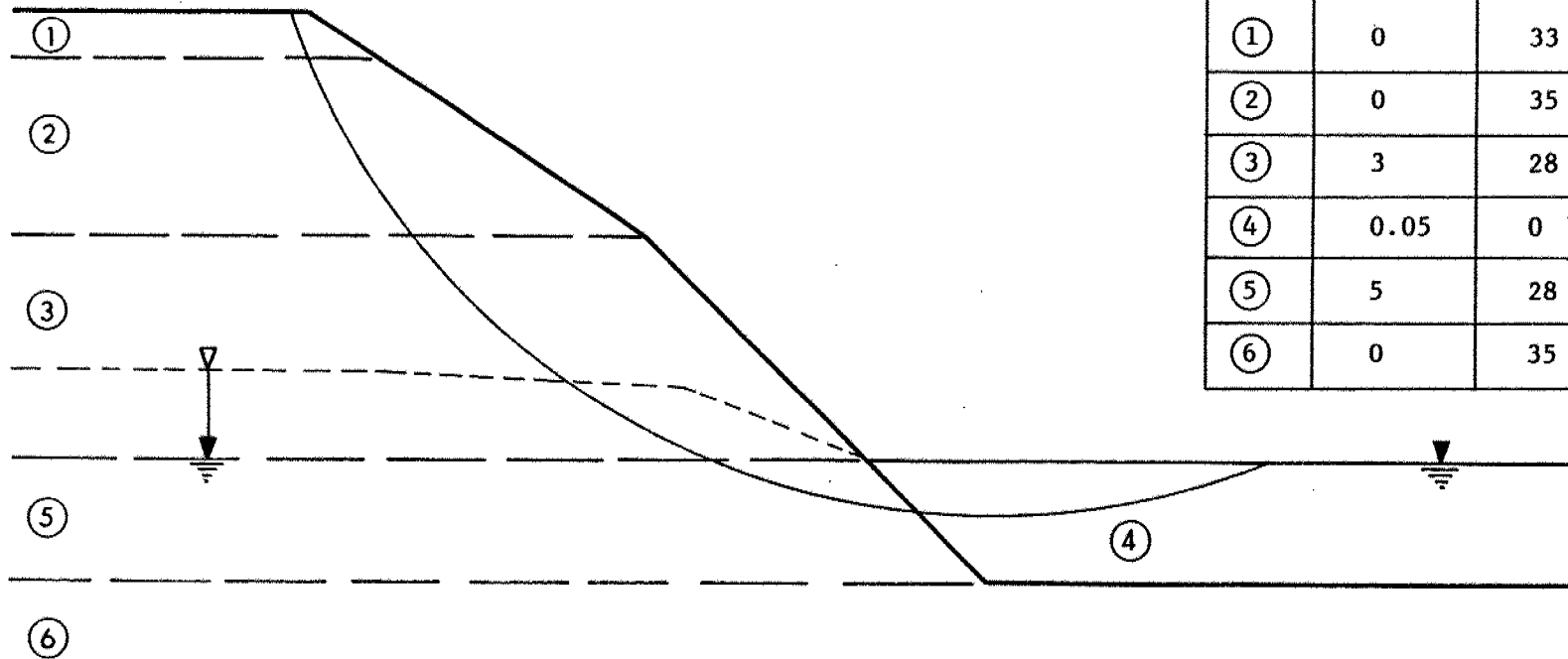
FIGURE 5



$$SF = 0.968 +$$

# ASSUMED SOIL PARAMETERS

Soil	Shear Strength $c'$ ( $kPa$ )	Angle of Int. Fric $\phi'$ ( $^{\circ}$ )	Bulk Density $\gamma$ ( $kN/m^3$ )	Submerged Density $\gamma'$ ( $kN/m^3$ )
①	0	33	18.8	8.8
②	0	35	18.8	8.8
③	3	28	18.1	8.1
④	0.05	0	10.0	0.0
⑤	5	28	18.1	8.1
⑥	0	35	18.8	8.8

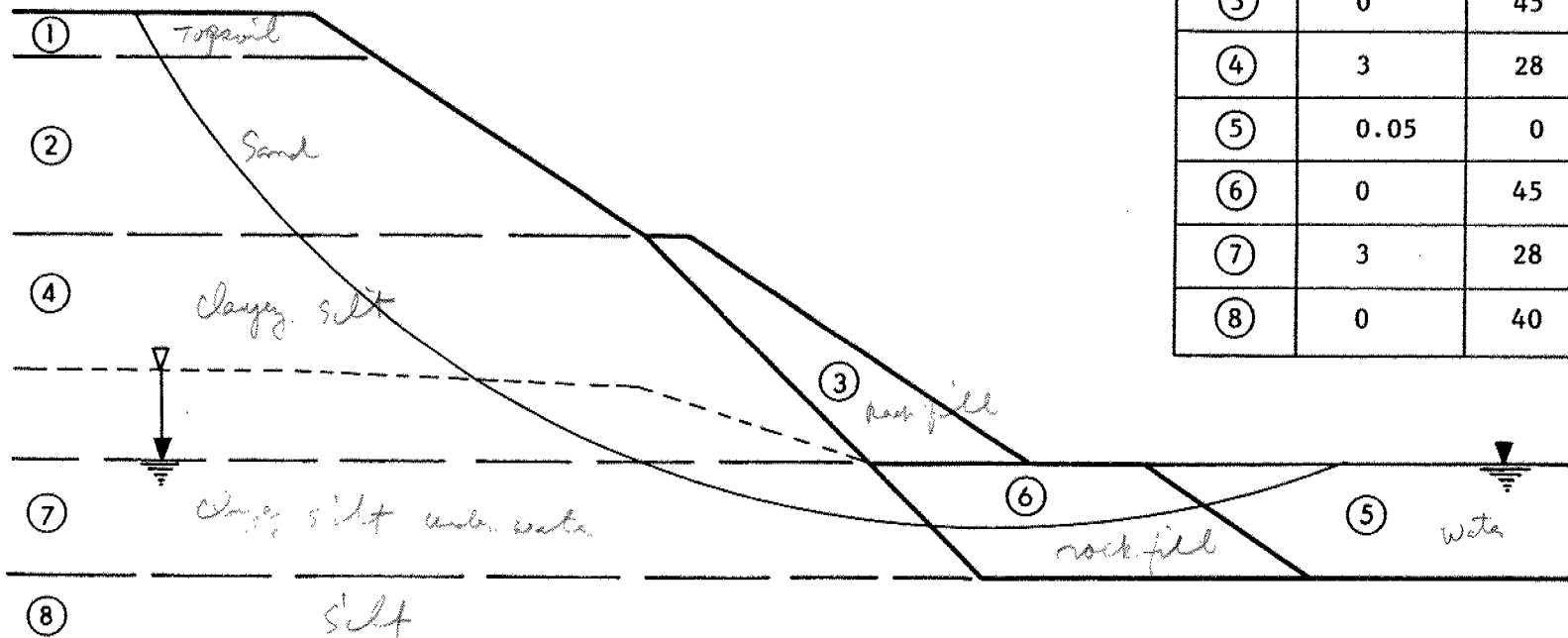


STABILITY ANALYSIS OF INITIAL SLOPE - GENERALIZED SLOPE CONDITIONS

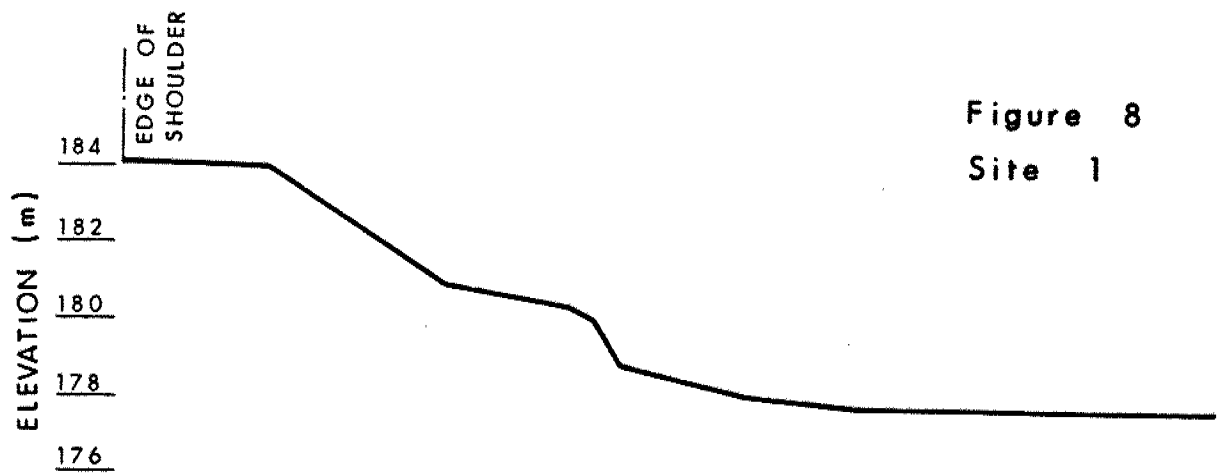
$$SF = 1.268 +$$

# ASSUMED SOIL PARAMETERS

Soil	Shear Strength $c$ ( $kPa$ )	Angle of Int. Fric $\phi$ ( $^{\circ}$ )	Bulk Density $\gamma$ ( $kN/m^3$ )	Submerged Density $\gamma'$ ( $kN/m^3$ )
①	0	33	18.8	8.8
②	0	35	18.8	8.8
③	0	45	18.1	8.1
④	3	28	18.1	8.1
⑤	0.05	0	10	0
⑥	0	45	18.1	8.1
⑦	3	28	18.1	8.1
⑧	0	40	18.8	8.8

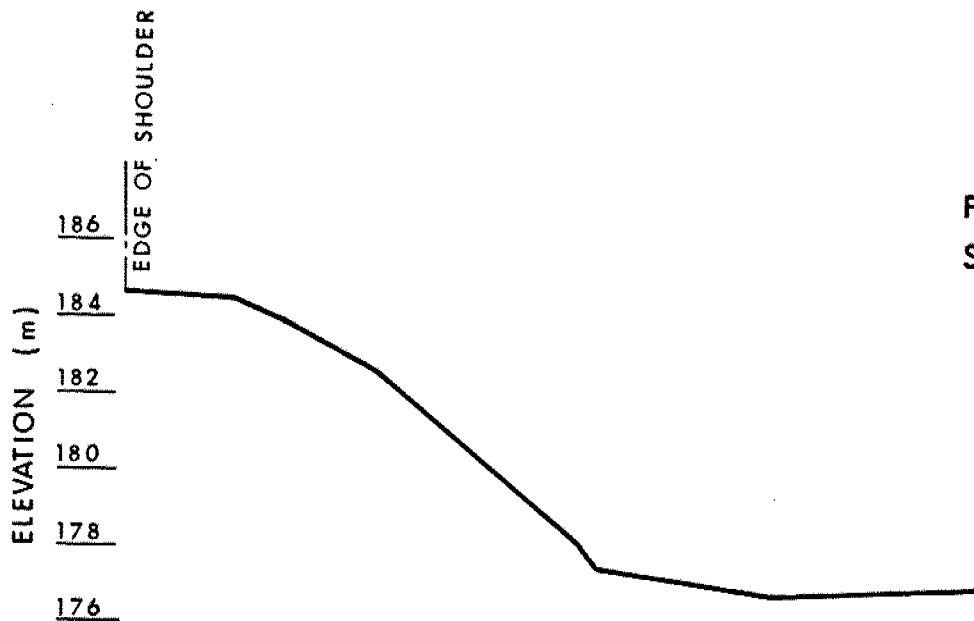


STABILITY ANALYSIS OF TREATED SLOPE - GENERALIZED SLOPE CONDITIONS



SCALE 1:200

TYPICAL SECTION STA 12+640  
THOMPSON TOWNSHIP

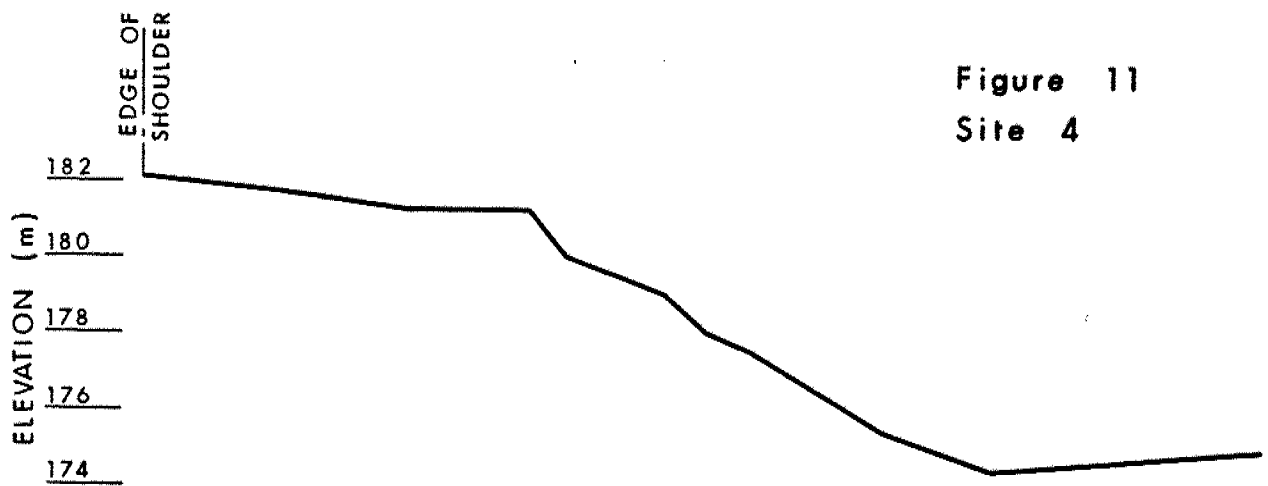


SCALE 1:200

TYPICAL SECTION STA 15+340  
THOMPSON TOWNSHIP



TYPICAL SECTION STA 18+000  
THOMPSON TOWNSHIP



TYPICAL SECTION STA 21+320  
THOMPSON TOWNSHIP

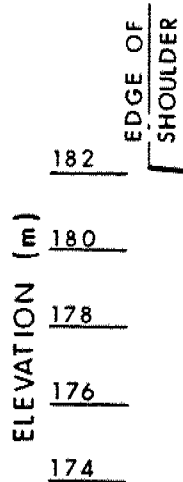


Figure 12  
Site 5

SCALE 1:200  
TYPICAL SECTION STA 10+360  
COBDEN TOWNSHIP

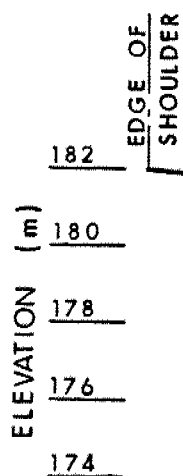


Figure 13  
Site 6

SCALE 1:200  
TYPICAL SECTION STA 13+520  
COBDEN TOWNSHIP

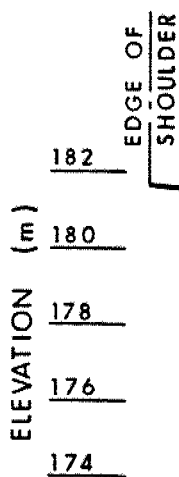
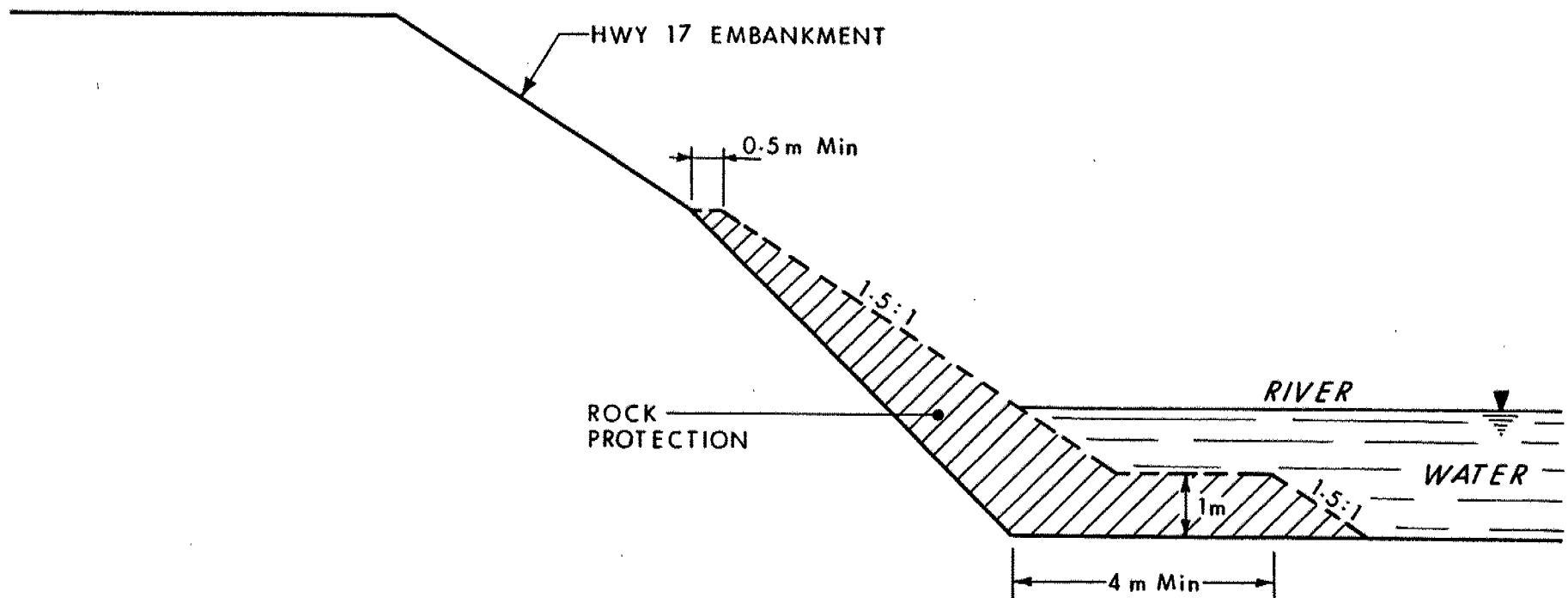


Figure 14  
Site 7

SCALE 1:200  
TYPICAL SECTION STA 13+960  
COBDEN TOWNSHIP





N T S

TREATMENT GEOMETRY GENERALIZED SLOPE CONDITIONS

Fig 15

WP 280-85-01

RECORD OF BOREHOLE Nos 1, 2 & 3

METRIC

W P 153-83-00 LOCATION STA.13+509.9, STA.13+507.9, STA.13+506.4 (Cobden) ORIGINATED BY ASP  
DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Solid Stem COMPILED BY GP  
DATUM GEODETIC DATE MAY 21, 1985 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
181.48	Ground Level																
0.0	Topsoil																
181.02																	
0.46	Sand, fine to coarse.		1	50 mm U.O.	4												
			2	"	5		180										
	Loose Brown																
179.10			3	"	1												
2.38	Clayey Silt, organic finely laminated. Occasional sand seam		4	"	1												
177.82	Firm Grey						178										
3.66	Sand, fine to medium, trace clay and silt.		5	"	4												
			6	"	6												
175.84	Loose Grey						176										
5.64	Silt, trace sand and clay.																
174.93	Compact Grey		7	"	11												
6.55	End of Borehole																
								Borehole 1 Piezometer Installation Sta.13+509.9									
								Borehole 2 Piezometer Installation Sta.13+507.9									
								Borehole 3 Piezometer Installation Sta.13+506.4									
								WATER LEVEL May 25, 1985									
								July 9, 1985									



RECORD OF BOREHOLE Nos 4, 5 & 6

METRIC

W P 153-83-00 LOCATION STA. 21+336.6, STA. 21+346.6, STA. 21+345.3 (Thompson) ORIGINATED BY ASP  
DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Hollow Stem COMPILED BY GP  
DATUM GEODETIC DATE MAY 21, 1985 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
180.82	Ground Level													GR SA SI CL
0.0	Topsoil													
180.35														
0.47	Layered Sand - medium to coarse and Silty fine sand, occa- sional organics.		1	50 mm D.O.	2		180							k=9x10 <sup>-8</sup> m/ sec
178.84	Very Loose Brown to Grey		2	"	1									
1.98	Clayey Silt, trace sand													
178.42	Firm Grey													0 2 68 30
2.40	Silty Sand		3	"	1									
177.92	Very Loose Grey						178							
2.90	Layered Clayey Silt - organic, trace sand		4	"	1									
	Silt - organic and Silty clay - occasional pocket of peat		5	"	2									
	Firm Grey													0 2 60 38
176.05	Sand - medium to coarse													
175.76	Compact Grey		6	"	2		176							
5.06	End of Borehole													
								Borehole 4 Piezometer Installation		Sta. 21+336.6				
								Borehole 5 Piezometer Installation		Sta. 21+346.6				
								Borehole 6 Piezometer Installation		Sta. 21+345.3				
								WATER LEVEL $\nabla$ May 25, 1985						
								$\nabla$ July 9, 1985						

## RECORD OF BOREHOLE Nos 7 &amp; 8

METRIC

W P 153-83-00 LOCATION STA.16+492.6, STA.16+487.3 (Thompson) ORIGINATED BY ASP  
DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Hollow Stem COMPILED BY GP  
DATUM GEODETIC DATE MAY 22, 1985 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40						60	80	100
								SHEAR STRENGTH									
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						WATER CONTENT (%)				
													20 40 60				
183.76	Ground Level													GR SA SI CL			
0.0	Topsoil																
183.29																	
0.47	Layered Silty fine sand, Sandy silt, & Sand, fine to coarse (seams to 12 mm thick)		1	50 H D.O	4												
182.39	Loose Brown																
1.37	Sand - fine to medium		2	"	3												
							182										
			3	"	2												
			4	"	3												
	Very loose Brown																
179.80							180										
3.96	Layered Sandy silt, Silt-organic Silty sand Occasional woody peat seam.																
			5	"	2												
178.58	Very loose Grey																
5.18	Clayey silt - organic, trace sand																
							178										
	Firm Grey		6	"	1												
177.21																	
6.55	End of Borehole																
							Borehole 7 Piezometer Installation Sta.16+92.6 Borehole 8 Piezometer Installation Sta.16+487.3 WATER LEVEL ▽ May 25, 1985 ▽ July 9, 1985										

+3, x5 : Numbers refer to Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10

## RECORD OF BOREHOLE No 9

METRIC

W P 153-83-00 LOCATION STA. 12+678 (Thompson) ORIGINATED BY ASP  
DIST 18 HWY 17 T.C.H. BOREHOLE TYPE Power Auger: Hollow Stem COMPILED BY GP  
DATUM GEODETIC DATE MAY 22, 1985 CHECKED BY \_\_\_\_\_

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
184.14	Ground Level												
0.0	Asphalt												
0.13	Fill - sand and gravel												
182.94													
1.20	Layered Sand - medium to coarse, and Silty Sand, with organic fine sandy silt seams 12 mm thick. Loose to very loose		1	50 mm D.O.	5								
180.86	Brown												
3.28	Silty Sand		2	"	2								
180.48	Grey/Brown												
3.66	Silt - organic, trace clay and sand. Occasional sandy silt and silty sand seams 12 to 25 mm thick. Numerous woody peat zones.		3	"	1								
			4	"	1								
178.35	Firm Grey		5	"	1								
5.79	End of Borehole												
						WATER LEVEL		May 25, 1985					
								July 9, 1985					

+3, x5 : Numbers refer to Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
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Ministry of  
Transportation and  
Communications

W.P. No. 153-83-00

DIST. No. 18 HWY. No. 17

LOCATION : Mississagi River Slope Protection

Iron Bridge to Mississagi Chutes

SOILS PROFILE No. \_\_\_\_\_

# SOILS DESIGN REPORT

ENGINEERING AND RIGHT-OF-WAY OFFICE  
NORTHWESTERN REGION

SOILS DESIGN REPORT

GEOTECHNICAL SECTION

NORTHWESTERN REGION

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## SUMMARY

1. Water level fluctuation, due to the routine peaking operations of the Red Rock Falls Generating Station, is accelerating the natural rate of erosion on the Mississagi River.
2. Hydro accepted responsibility for a portion of the erosion occurring on the Mississagi River from Red Rock Falls Generating Station to the Mississagi Chutes.
3. The most acceptable method of reducing erosion problems is a combination of quarried stone and vegetation. The quarried stone is placed on the lower portion of the bank where the water level fluctuates. Where the bank soils are exposed above the stone, vegetation in the form of grasses and legumes is to be planted.
4. Most of the construction will be done by truck, bulldozer and gradall. Only trees which have fallen, are in danger of falling or interfere with construction should be removed from the river bank. Wherever possible, sound trees should be left intact.
5. The performance of geotextile placed beneath the quarried stone in 1984 will be monitored in the future.
6. A hydrology consultant, divorced from the Ministry of Transportation and Communications as well as Ontario Hydro, should be contacted to:
  - a. carry out foundation investigations
  - b. determine the severity of the erosion problem
  - c. determine the degrees of responsibility of the various parties
  - d. make recommendations on a programme schedule for rehabilitation.

## 1.0 BACKGROUND

### 1.1 PAST HISTORY

Red Rock Generating Station, a hydroelectric generating station located on the Mississagi River, was placed into service in 1960 and has generally been operated as a peaking plant. Since that time complaints from river front property owners have been raised concerning the apparent increase in the rate of erosion, the damage to riverbank trees and vegetation and the inconvenience of daily water level fluctuations on recreational fishing and boating.

In the spring of 1979 a high freshet flow caused flooding in Iron Bridge and severe erosion on certain properties. The community reacted by establishing the Iron Bridge Flood and Erosion Control Committee and demanded action from Hydro. Since that time Hydro has held numerous public meetings, conducted extensive studies and have, or will be, rip raping 12.9 km of shoreline in Iron Bridge. Further rip rap work with shrubs and grass seeding both upstream and downstream of Iron Bridge are proposed. In addition most, if not all, of the landowners have signed an agreement with Hydro to set compensation for past damages caused by erosion. All of the proposed work by Hydro is on privately developed property. No work was to be carried out on the shoreline adjacent to Highway 17.

In the past, the peaking operations at the Red Rock Falls Generating Station have resulted in substantial daily water level fluctuations downstream of the plant. At Iron Bridge a daily water level change of 1.8 m had been experienced as a result of peak flow variations. It is understood as well, that as much as a 3.4 m fluctuation in a single day has resulted because of the high inflows resulting from the utilization of system reserve capacity at the upstream G. W. Rayner/Wells Generating Complex. In response to public comments, Hydro modified their operations in 1980 to limit this daily fluctuation to approximately 0.6 m.

Our records taken at the Mississagi Chutes in 1983 reveal that the daily fluctuation had been reduced to less than 0.6 m. The highest water level occurred on June 3 and the lowest water level occurred on November 6. The maximum difference in elevation between these dates was 3.6 m.

## 2.0 DESCRIPTION OF EROSION

The north Mississagi riverbank from Iron Bridge to the Mississagi Chutes is composed primarily of fine sands, silty sands and sandy silts with short stretches of exposed bedrock. The bank tends to be steep-sided and in places, is very close to Highway 17.

The erosion of the riverbank has resulted from numerous erosion processes which include:

1. erosion of soil along the toe of the bank (Figure 1) followed by undercutting of the bank with subsequent slope failure (Figure 2).
2. Sloughing of saturated banks due to rapid changes in the groundwater level (Figure 3).
3. erosion by seepage forces (Figure 4)

Other minor causes contributing to erosion are:

1. surface erosion due to runoff and ice action
2. alteration of the river bottom contours by current action thus altering the current velocity

The most critical agents of erosion are as follows:

1. fluctuation of the daily and long term flow
2. high freshet flows
3. groundwater fluctuation
4. natural daily flows

The daily fluctuation in the flow of the Mississagi River is one factor which can be directly attributed to the Ontario Hydro operations on this river. The continuous wetting and drying cycles cause a drawdown which accelerates the rate at which the fine sand and silt materials are removed from the toe of the bank. With the removal of the lower bank material, the upper bank is undermined and becomes more susceptible to further erosion, sloughing or complete failure because of the lack of foundation support.

The present erosion has resulted in numerous adverse effects such as:

1. loss of trees and other vegetation due to slippage into the river (Figure 5)
2. loss of support for guiderail causing potential traffic hazard (Figure 3 and Figure 6)
3. potential danger of complete undermining of Highway 17 (Figure 6 and Figure 7).

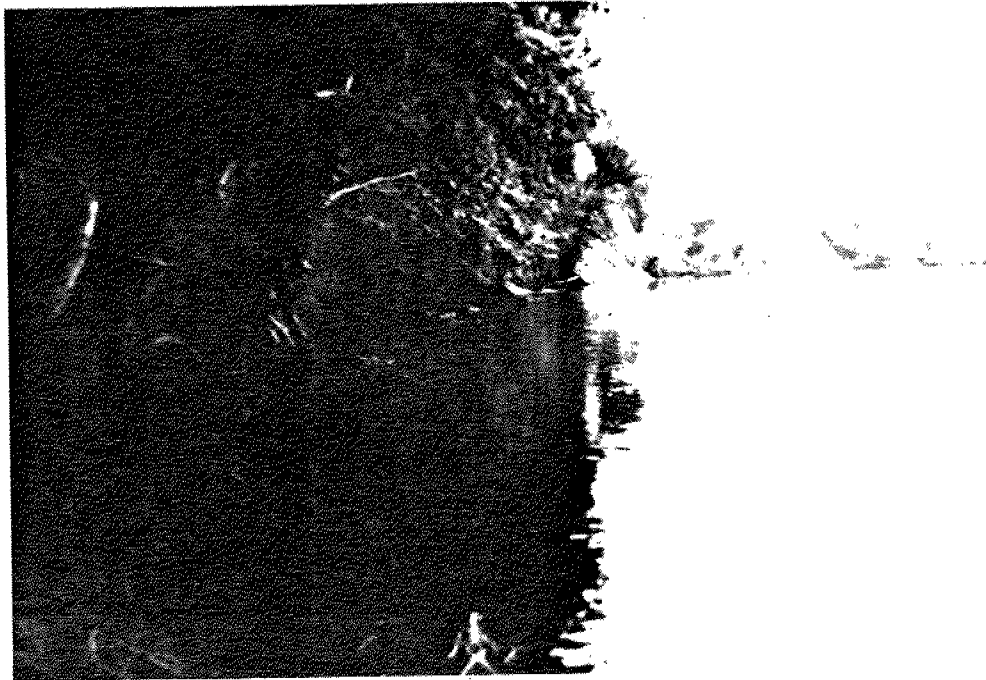


FIGURE 1



FIGURE 2



FIGURE 3

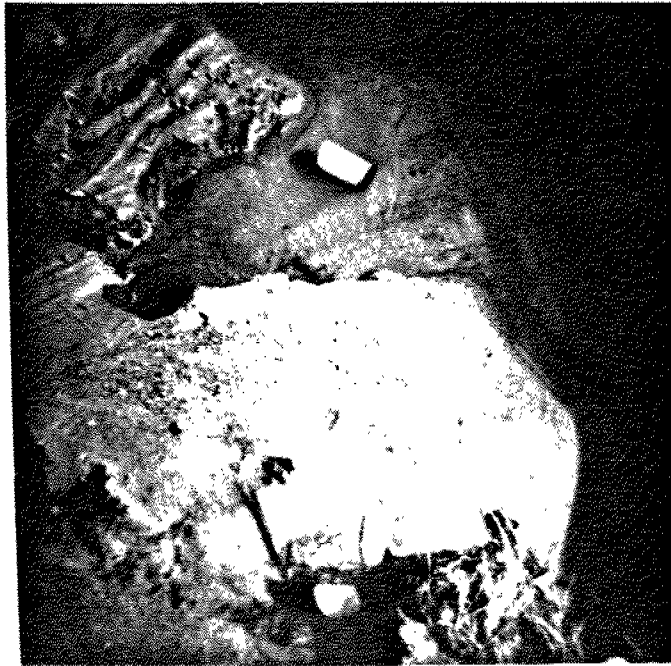


FIGURE 4



FIGURE 5



FIGURE 6



FIGURE 7

To date, four different areas of the riverbank from Iron Bridge to the Mississagi Chutes have been repaired using rip rap because of the serious erosion and the impending loss of the highway. These areas are as follows:

1. 15+060 to 15+380 Thompson Twp. - length - 320 m
  2. 11+850 to 12+125 Thompson Twp. - length - 275 m
  3. 15+380 to 15+500 Thompson Twp. - length - 120 m
  4. 20+500 to 20+670 Thompson Twp. - length - 170 m
- Total Length - 885 m

The first section was done in 1983 with no geotextile and the last three sections were done in the spring of 1984 with geotextile beneath the rip rap.

On the remaining sections from Iron Bridge to the Mississagi Chutes where the river is close to the highway there is a continuing erosion problem. If this erosion is allowed to continue uncontrolled, public safety may be endangered.

### 3.0 PURPOSES OF THIS REPORT

The purpose of this report is to identify and set priorities for additional lengths of riverbank to be stabilized with rip rap from Iron Bridge to the Mississagi Chutes.

Ontario Hydro has admitted some liability for the erosion of the Mississagi River banks caused by the fluctuation in water levels associated with the peaking operation of the hydroelectric generating stations on the river. To date, Hydro has admitted that up to 30 percent of the erosion is caused by daily peaking fluctuations. It is beyond the scope of this report to verify or deny this figure, but it should be pointed out that Hydro has contributed 100 percent for the private work in Iron Bridge.

### 4.0 M.T.C. INVESTIGATIONS

To date, the following investigations have been carried out:

1. Remote Sensing Report with aerial photographs
2. Geotechnical soil borings
3. Geotechnical & Foundation Section field investigation
4. Cross sections
5. Kepner Tregoe Analysis



A more detailed description of each of these follows.

#### 4.1 REMOTE SENSING

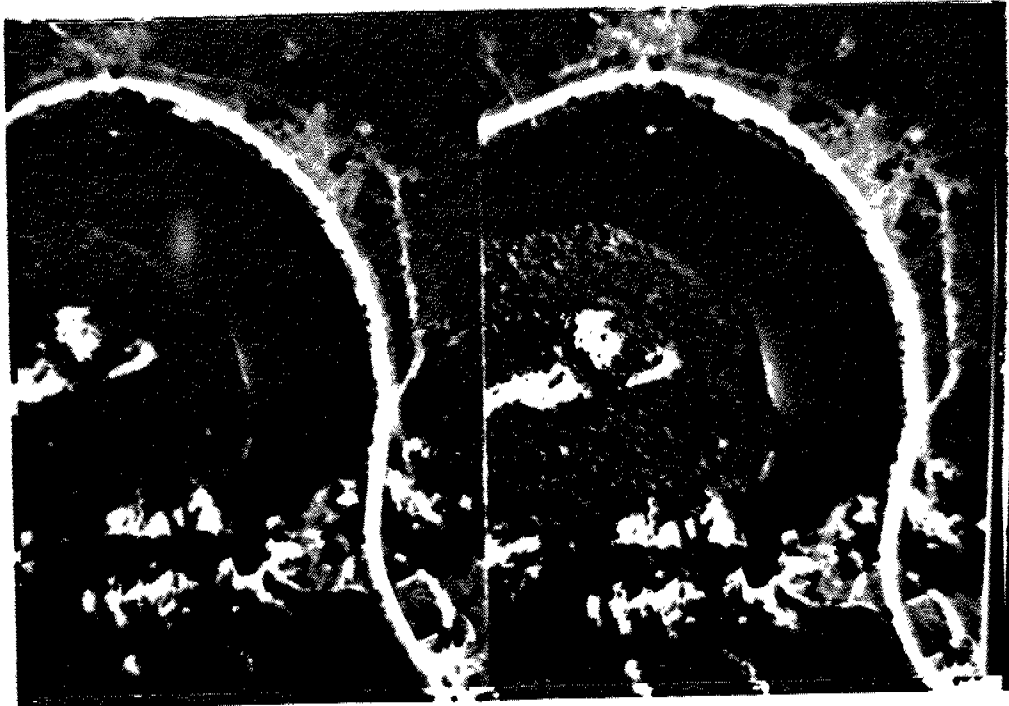
Stereo couplets for this area were prepared from aerial photographs flown in 1949, 1964, 1973 and 1981. Three critical locations where bank stability problems occur were identified along the Mississagi River adjacent to Highway 17. These locations are at Station 11+800 to 12+500 - Thompson Township, Station 18+000 to 18+700 - Thompson Township and Station 13+250 to 14+200 - Cobden Township. Figure 8 shows this last location with a stereo couplet taken in 1973.

The aerial photographs show relatively stable riverbanks in these areas in 1949 and 1964. In 1973 and 1981 the riverbanks show evidence of progressive riverbank instability.

It should be noted that the river elevation was lowest in 1949 at the time of photography and was increasingly higher when the photographs were taken in 1964 and 1973. In 1981, the river elevation was similar to that of 1973.

As mentioned previously, the first location from Station 11+850 to 12+125 in Thompson Township was treated with rip rap in 1984.

FIGURE 8



#### 4.2 SOIL BORING INVESTIGATION

A preliminary hand auger soil investigation of the river bank was carried out in July of 1984. This investigation was carried out to verify or refute the erosion potential of the slopes relative to that specified by the Geotechnical Engineering Department of Ontario Hydro.

In most cases the M.T.C. ratings did agree with Ontario Hydro but in a few locations there were extreme differences. Note in Figure 3 at Station 20+500 that there was severe sloughing in 1984. This area was stabilized in 1984 although Ontario Hydro stated that this location had minor erosion potential.

The logs of M.T.C. soils borings and laboratory test results may be found at the end of this report.

#### 4.3 FOUNDATION DESIGN SECTION

In January 1984 and August 1984, on-site visits were made with the Foundation Design Section from Head Office. On the first trip locations which were most critical for public safety were identified. Three lengths were chosen for stabilization in 1984. These were done as noted previously.

During the on-site review in August 1984 the remaining 6 km of shoreline were reviewed. Foundation Design Section has submitted a suggested schedule for treatment of various slopes. This may be found in Table I.

If required, the Foundation Design Section will undertake deep borings in predetermined areas to ascertain if any deep slope stability problems are anticipated.

#### 4.4 CROSS SECTIONS

In the spring of 1984 cross sections along 6185 m of Highway 17 were taken where the Mississagi River was close to the roadway. The sections were taken from 20 m left (of centre-line) to a point 15 m into the river (on the right) at 20 m intervals.

These sections were utilized to determine the angle of slope and the distance from the shore to the edge of pavement.

If required, future cross sections will be taken to monitor future settlement and sloughing of the shoreline.

TABLE I

LOCATIONS ON HIGHWAY 17 OF POTENTIAL SLOPE INSTABILITY

<u>STATION</u> -----	<u>SCHEDULING</u> -----
17+180 to 18+700 Thompson Twp.	Should be done in 2 to 4 years
21+140 to 21+340 Thompson Twp.	Should be done in 2 to 4 years
13+800 to 14+200 Cobden Twp.	Should be done in 2 to 4 years
12+125 to 13+000 Thompson Twp.	Probably will not require work within 5 years
15+500 to 16+340 Thompson Twp.	Probably will not require work within 5 years
13+000 to 13+300 Cobden Twp.	Probably will not require work within 5 years

#### 4.5 KEPNER TREGOR ANALYSIS

With over six kilometres of the Mississagi shoreline being close to the highway, it would be financially impossible and impractical to carry out a total slope stability contract. Subjective choices of different lengths and appropriate treatment schedules must be established. The Ministry, however, should not wait until there is severe sloughing with future potential hazards to the travelling public, before commencing some rehabilitation.

To determine a list of location priorities the Kepner Tregoe Decision Analysis technique was utilized. The criteria used to determine the potential hazard to the public caused by sloughing of the bank were:

1. Soil Erosion Potential
2. Past Rehabilitation Needs
3. Seepage Zones and Sloughing
4. Existing Tree Cover
5. River Current Velocity
6. Angle of Bank Slope
7. Distance of Shore to Edge of Pavement
8. River Vegetation

Each of these criteria was weighed for each location and in turn, weighed against the others for importance.

The results of this analysis may be found in Table II.

#### 5.0 REHABILITATION TECHNIQUES

It is known that much of the bank erosion is caused by the daily fluctuation. This fluctuation causes differential pore pressures in the bank and results in a drawdown effect. The fine sand and silt materials are then in turn removed from the toe area of the bank. Continued undermining of the material above the water surface results in the loss of support for the mid and upper bank soils which then slide into the water or are removed by high water flows.

To stabilize the bank, the toe of the slope should be reinforced to prevent progressive scour by the water. A portion of the bank surface above the normal operating water levels is submerged only temporarily during the annual spring freshet and infrequently during periods of Hydro's system reserve capacity flow. Vegetation cover would be adequate for protection against surface water runoff as well as the direct effects of these higher flows.

TABLE II

KEPNER TREGOE ANALYSIS  
W.P. 153-83-00  
IRON BRIDGE TO MISSISSAGI CHUTES  
HIGHWAY 17, DISTRICT 18

STATION	KEPNER TREGOE WEIGHTED TOTAL	KEPNER TREGOE PRIORITIES	LENGTH (m)
Thompson Twp.			
12+125 to 12+500	244	6	375
12+500 to 13+000	150	12	500
15+500 to 15+900	206	8	400
15+900 to 16+200	154	10	300
16+200 to 16+340	232	7	140
17+180 to 17+850	262	5	670
17+850 to 18+500	418	1	650
18+500 to 18+900	202	9	400
21+140 to 21+340	346	3	200
Cobden Twp.			
10+300 to 10+550	336	4	250
13+000 to 13+600	151	11	600
13+600 to 14+160	350	2	560
TOTAL			5,045 m

## 5.1 TOE PROTECTION

Rip Rap consisting of quarried stone is the only lining which meets all of the criteria for toe protection. The four locations that were stabilized in 1983 and 1984 have performed very well. At these sites, M.T.C. removed all of the trees and placed quarried stone right to the top of the slope. For future work this is considered to be rather superfluous as well as costly. Ontario Hydro on the other hand has placed quarried stone only to a level consistent with a flow of 250 m<sup>3</sup>/s where vegetation was intact or a level consistent with a flow of 350 m<sup>3</sup>/s where there was no vegetation.

This in turn means Ontario Hydro planned on placing rip rap from 1.7 m to 3.0 m in height above the low water mark.

As mentioned previously the difference between the low and high water levels in 1983 was 3.6 m.

It is recommended that on future work the slopes be stabilized for a height of 3.0 m above the low water mark. In terms of elevation, this means the rip rap would be placed to 180.5 at Dean Lake Bridge and 179.5 at the Mississagi Chutes.

## 5.2 RIP RAP

With the limited amount of very coarse gravel in this area it should be assumed that quarried material will be used. This quarried material used by Ontario Hydro and M.T.C. in 1983 and 1984 came from a quarry site just to the east of Iron Bridge. The operation of a quarry would be subject to approval by the M.N.R. under the Mining Act and applications for the necessary permits would be necessary.

The gradation requirements are as follows:

Stone Size (mm)	Range (percent smaller)
-----	-----
200	100
150	60 - 100
106	50 - 70
75	25 - 45
37.5	10 - 25
16	5 - 15
9.5	0 - 10
4.75	0 - 5
75 um	0 - 2

To date, this material appears to have worked quite adequately.

### 5.3 PLACEMENT OF RIP RAP

Trucks will be used to haul and dump the quarried stone. Specific access to the Mississagi River does not exist at present. The location of each access route should be selected in order to:

1. minimize the amount of vegetation and tree removal
2. minimize the disruption of existing guiderail
3. minimize the amount of grading

Access to the river will require placement of stone ramps on the bank. These ramps will be removed after construction is complete.

The trucks will end dump sufficient material, in the form of a shoreline road to provide the necessary protection. This material will then be reshaped to the final design specifications by equipment such as a gradall (Figure 9).

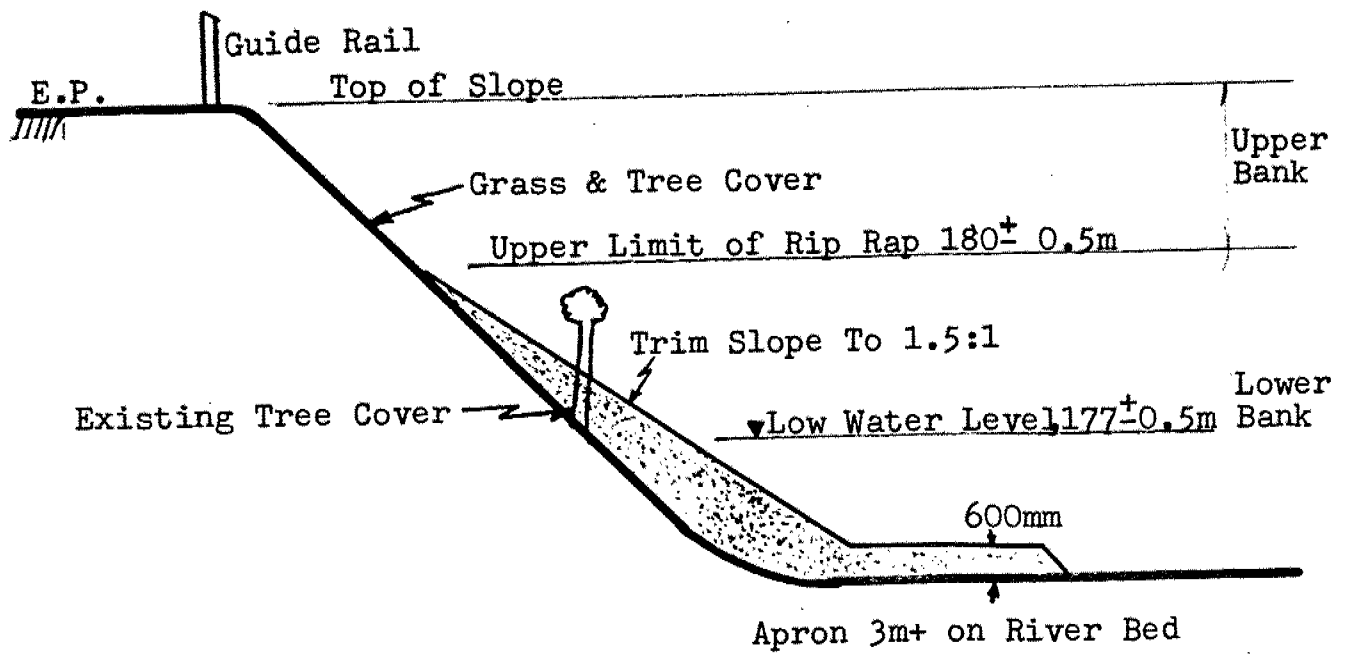
The exact amount of rip rap required at each location is not known at this time. Cross section quantities will be required. For design purposes assume the following criteria:

1. Allow for a minimum of a 3 m wide apron for the shoreline road.
2. Allow for a minimum slope and riverbed rip rap depth of 600 mm.
3. Allow for a maximum slope of 1.5:1.

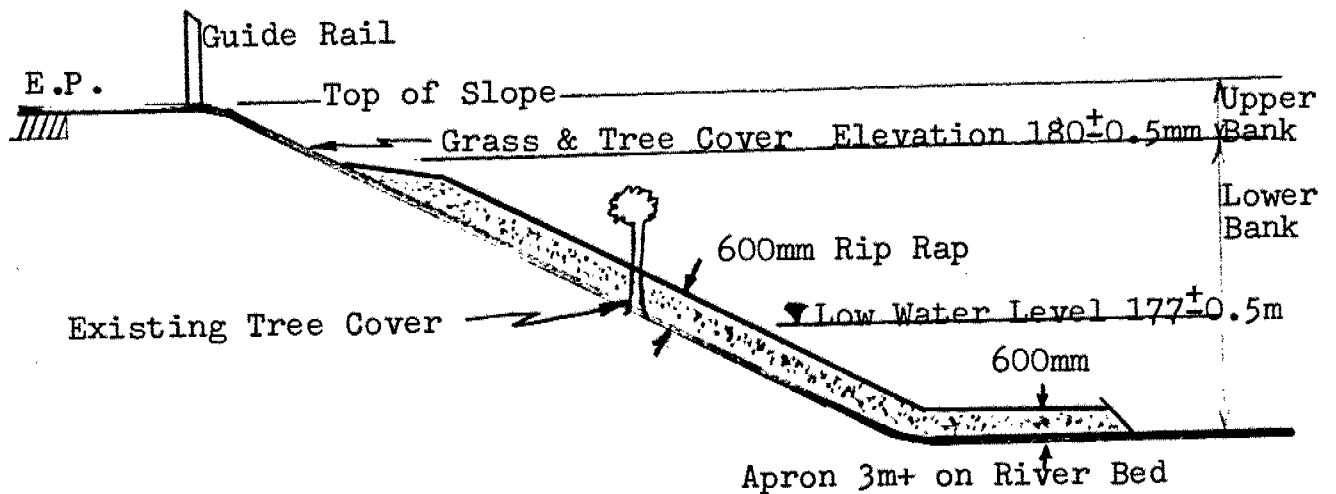
### 5.4 TREE REMOVAL

Wherever possible, sound trees along the river bank will be left intact. These trees form a valuable function in controlling erosion, and their unnecessary removal is counter-productive. The criteria for tree removal are:

1. trees which have fallen or are in the process of falling
2. dead trees
3. trees which interfere with construction and cannot be adequately pruned.



STEEP SLOPE  
(1.5:1 or Steeper)



FLAT SLOPE  
(Flatter than 1.5:1)

FIGURE 9  
Typical Bank  
Protection Treatment



All sound stumps will be left in place and trimmed back flush with or below the top of the bank protection. If the roots are loose, the stump should be removed.

All submerged or partially submerged logs which interfere with the construction activities or reduce the safety of the program should be removed.

### 5.5 GRADING

Very little reshaping of the bank should be carried out prior to placing the rip rap. Any grading would cause an undesirable disturbance to existing stable natural vegetation. Reshaping of the existing slopes should be confined to sloughed areas and ramps.

### 5.6 GEOTEXTILES

The real need for using a geotextile beneath the rip rap has not been proven. Under normal conditions in highway construction, a geotextile would be used to stop the fine grained silt materials from leaching out of the back slope because of the drawdown. The vegetation and topsoil on the existing slope may stop this leaching. In 1983, 320 m of slope was stabilized with no geotextile. In 1984 an additional 120 m of slope immediately adjacent to the first section was stabilized with geotextile. An investigation next spring after the high freshet should be carried out to observe if any real need for the geotextile can be determined.

If required the geotextile should consist of a non-woven, Class II, polyester or nylon material with an E.O.S.  $\leq 150$   $\mu\text{m}$ .

### 5.7 SHRUBS AND GRASSES

The high erosive forces on the river do not allow vegetation to become established where a frequent flow occurs. This is where rip rap is recommended. Where erosive energy is not sufficiently high to require rip rap, vegetation can be used to stabilize soil, to slow runoff, dissipate its erosive energy and filter sediment from runoff.

Ontario Hydro is planning on placing shrubs on the mid to upper reaches of the privately developed properties for a

height of approximately 1.5 m. It should be noted, however, that their height of rip rap protection is not as high as what is recommended in this report.

In the upper reaches above the rip rap protection, the slope will only be submerged by short duration, high water levels in the spring freshet. Normal hydraulic seeding and mulching procedures using binder and fertilizer should be sufficient for our needs.

Ontario Hydro was of the opinion that their shrub and grass seeding program should be carried out a year after the quarried stone had been placed. This would allow sufficient time for the rip rap and upper reaches of the bank to stabilize. In our case it is the undersigned's opinion that since only grass seeding is recommended it should be carried out as soon as possible after rehabilitation of the lower bank.

#### 5.8 SHOULDER & GRANULAR SUB-BASE SEALING

To minimize the chances of surface erosion due to runoff from our pavement, Granular Sealing with RSI or SSI Emulsion should be carried out on the shoulder and sideslope to the sub-base level. This is especially true in the passing lane areas where the superelevation slants toward the Mississagi River.

#### 6.0 RECOMMENDED PROGRAMME

With the potentially serious nature of this problem with respect to public safety a programme schedule for the rehabilitation of the Mississagi River slopes should be set up. Past history has shown that some of these banks are in a very unstable condition. Scouring of the toe of slope has caused undermining, sloughing and complete failure. At present these detrimental forces are still at work in other areas. Tension cracks on the sideslopes have been observed. These are an indication of future failure. By using the Kepner Tregoe Analysis in conjunction with the Foundation Design Section's recommendations, the following Programme Schedule should be initiated.

In 1986 rehabilitate the slopes from:

Station 17+850 - Station 18+500 Thompson Twp.	- 650 m
Station 13+600 - Station 14+160 Cobden Twp.	- 560 m
Station 21+140 - Station 21+340 Thompson Twp.	- 200 m
	-----
Total Length	1,410 m

In 1988 rehabilitate the slopes from:

Station 10+300 - Station 10+550	Cobden Twp	- 250 m
Station 17+180 - Station 17+850	Thompson Twp	- 670 m
Station 12+125 - Station 12+500	Thompson Twp	- 375 m
Station 16+200 - Station 16+340	Thompson Twp	- 140 m
		-----

Total Length 1,435 m

The remaining 2,200 m of riverbank slope close to Highway 17 should only require close monitoring for the next 5 years.

## 7.0 COSTS

It is understood that the rehabilitation work carried out in 1983 and 1984 cost approximately 300 dollars a linear metre. This work, however, was more extensive than the recommended method of rehabilitation in future projects. The cost of the quarried rock will be more expensive. This is only one portion of the total work required. For estimating purposes 350 present day dollars should be used for each linear metre of rehabilitation.

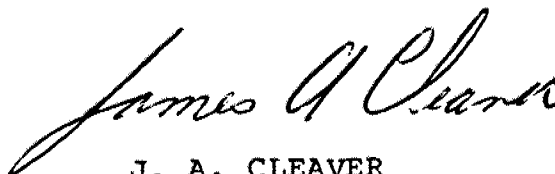
For scheduling purposes approximately 493,500 dollars would be required to carry out the work in 1986 and 502,250 dollars for the work in 1988.

## 8.0 ADDITIONAL WORK REQUIRED

Monitoring of these slopes will be required continuously by the District Maintenance Forces as well as the Regional Geotechnical Section. This monitoring programme could accelerate or delay any of the future rehabilitation work. The Foundation Design Section or its representative should undertake deep borings this winter in isolated areas to ascertain if any deep seated slope stability problems are anticipated.

Aerial photographs of this area should be flown again to provide the latest information on the progressive riverbank instability. The date for this photography work will be determined later. The scale of the black and white photographs is recommended to be 1/3000 which will allow detailed review.

A hydrology consultant, divorced from the Ministry of Transportation and Communications as well as Ontario Hydro, should be contracted to determine the degrees of responsibility of the various parties. Ontario Hydro has admitted some liability and should therefore contribute towards the cost of the rehabilitation works.

A handwritten signature in cursive script, reading "James A. Cleaver".

J. A. CLEAVER

Project Soils Engineer  
(For)

J. B. MACMASTER

Head, Geotechnical Section

Station 12+495 23.0-10.0 Rt C (D-0.7)

0 - 25 Tps  
25 - 400 Br F Gr  
400 - 2.1 Br Sa(y) Si  
2.1 - 5.1 Gry Sa(y) Si

SAMPLE NUMBER 84-RZZ-70 (2.1 - 5.1)

Gravel	0	%
Sand	26.5	%
Silt & Clay	73.5	%
FMC @ 5.0	36.6	%
PI	N.P.	
Group Symbol	ML	

Station 16+100 15.0-7.4 Rt C (D-0.6)

0 - 75 Tps & Rts  
75 - 400 Br F Gr  
400 - 500 Tps & Sa Mixed  
500 - 1.7 Br F Sa Tr Si Occ Gr  
1.7 - 5.2 Br Si(y) Sa (Dry-Moist)  
(Wet 4.0+)(Soft 5.2+)

SAMPLE NUMBER 84-RZZ-71 (1.7 - 5.2)

Gravel	0	%
Sand	63.1	%
Silt & Clay	36.9	%
FMC @ 4.0	22.5	%
PI	N.P.	
Group Symbol	ML - SM	

Station 17+500 13.0 Rt C (D-0.8)

0 - 300 Tps & Rts  
300 - 600 Lt Br Si(y) Sa  
600 - 1.6 Lt Br Sa with Si  
1.6 - 3.1 Red Br M Sa (Clean)  
3.1 - 4.2 Gry Si(y) Sa (Wet)  
4.2 Gry Si Tr Cl

SAMPLE NUMBER 84-RZZ-72 (4.2 - 4.6)

Gravel	0	%
Sand	0	%
Silt & Clay	100	%
FMC @ 4.6	52.6	%
PI	N.P.	
Group Symbol	ML	

Station 18+750 21.4-25.4 Rt C (D-1.6)

0 - 200 Tps & Rts  
200 - 2.6 Lt Br F Sa Tr Si Occ Gr  
2.6 - 3.8 Br M Sa Clean (Wet)  
3.8 - 4.2 Gry F Sa with Si (Wet, Comp)

SAMPLE NUMBER 84-RZZ-73 (200 - 2.6)

Gravel	1.1	%
Sand	97.5	%
Silt & Clay	1.4	%
FMC	3.8	%
Cu	2.4	
Cc	1.0	
Group Symbol	SP	

Station 21+203 12.5-14.0 Rt C (D-1.0)

0 - 100 Tps  
100 - 1.5 Lt Br Si(y) Sa  
1.5 - 3.5 Gry Si with Sa (Wet)

SAMPLE NUMBER 84-RZZ-74 (1.5 - 3.5)

Gravel	1.1	%
Sand	97.5	%
Silt & Clay	1.4	%
FMC	3.8	%
Cu	2.4	
Cc	1.0	
Group Symbol	SP	

Station 10+400 28.0-28.5 Rt C (D-2.0)

0 - 300 Tps  
300 - 1.0 Lt Br Si(y) Sa  
1.0 - 1.9 Lt Br Si with Sa  
1.9 - 2.3 Lt Br Sa(y) Si (Wet)

SAMPLE NUMBER 84-RZZ-75 (1.0 - 1.9)

Twp. Cobden

Gravel	0	%
Sand	37.2	%
Silt & Clay	62.8	%
FMC @ 1.4	24.1	%
PI	N.P.	
Group Symbol	ML	

Station 13+300 16.1-19.2 Rt C (D-1.3)

0 - 350 Tps  
350 - 800 Lt Br Si(y) Sa  
800 - 1.5 Lt Br Si with Sa (Moist)  
1.5 - 2.0 Lt Br Si Tr Sa (Moist)  
2.0 - 3.5 Gry Si Tr Sa & Cl

SAMPLE NUMBER 84-RZZ-76 (2.0 - 3.5)

Twp. Cobden

Gravel	0	%
Sand	14.2	%
Silt & Clay	85.8	%
FMC @ 2.5	38.4	%
LL	28.1	%
PL	21.2	%
PI	6.9	%
Group Symbol	CL - ML	

Station 13+600 22.8 Rt C (D-0.1)

0 - 250 Tps  
250 - 600 Lt Br F Sa with Si  
600 - 1.9 Lt Br F-M Sa Clean  
1.9 - 2.3 Br M-Co Sa Tr Si (Moist)  
2.3 - 3.8 Gry Si Tr Cl

SAMPLE NUMBER 84-RZZ-77 (2.3 - 3.8)

Twp. Cobden

Gravel	0	%
Sand	8.4	%
Silt & Clay	91.6	%
FMC @ 2.8	40.4	%
PI	N.P.	
Group Symbol	ML	

W.P. 153-83-00

TWP. COBDEN

Station 13+980 10.2-12.8 Rt C (D-0.3)

0 - 300 Tps  
300 - 1.8 Lt Br Si(y) Sa  
1.8 - 2.4 Gry Si with Sa  
2.4 - 3.9 Gry Si Tr Cl & Sa

SAMPLE NUMBER 84-RZZ-78 (2.4 - 3.9)

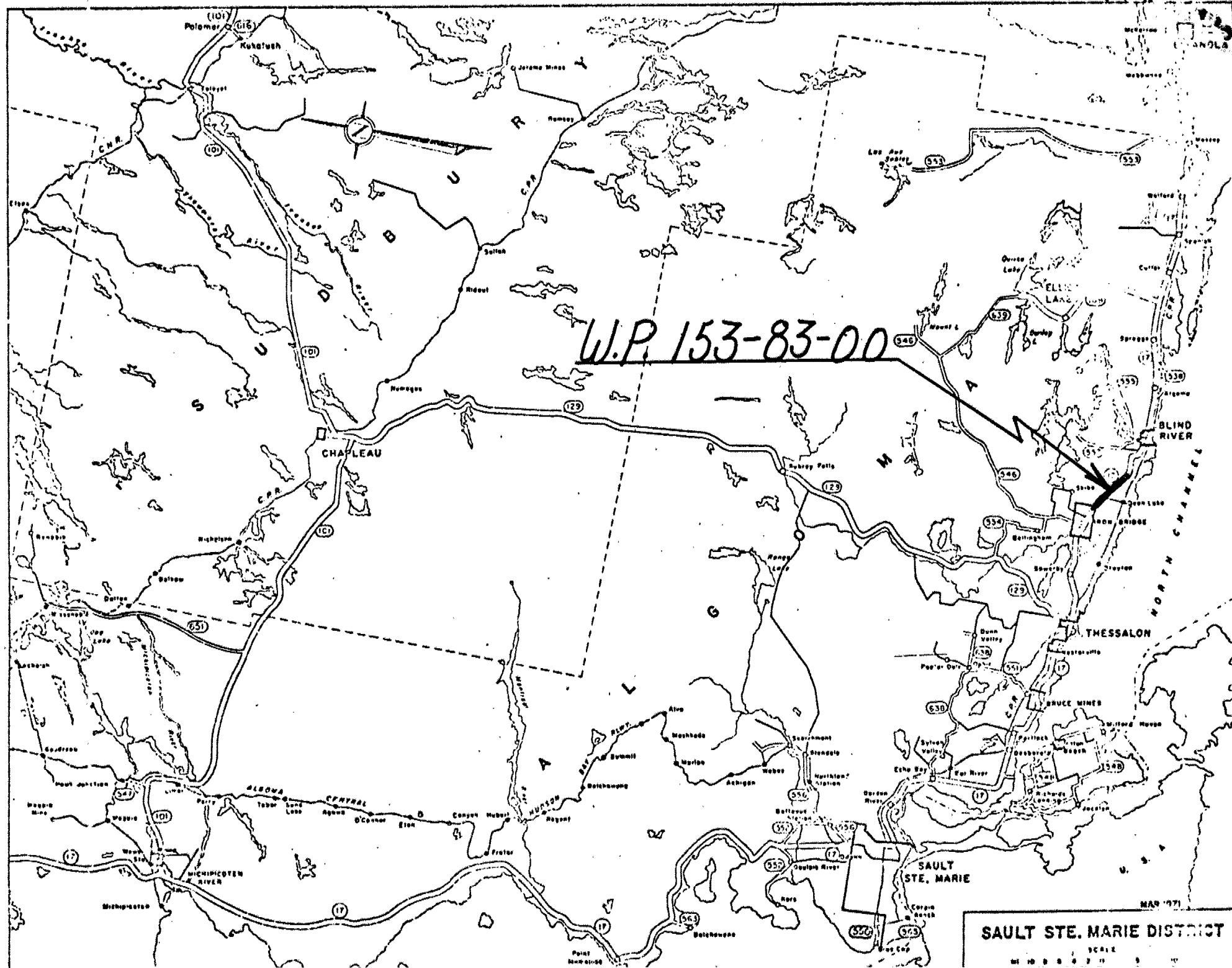
Gravel	0 %
Sand	0.4 %
Silt & Clay	99.6 %
FMC @ 3.0	41.3 %
PI	N.P.
Group Symbol	ML

Station 18+254 7.0-15.0 Rt C (D-0.5)

0 - 50 Tps  
50 - 450 Br F Gr  
450 - 1.6 Br Si with Sa  
1.6 - 3.8 Br M Sa Tr Si  
3.8 - 5.0 Gry Si Tr Sa

SAMPLE NUMBER 84-RZZ-79 (3.8 - 5.0)

Gravel	0 %
Sand	6.0 %
Silt & Clay	94.0 %
FMC @ 4.2	39.4 %
PI	N.P.
Group Symbol	ML

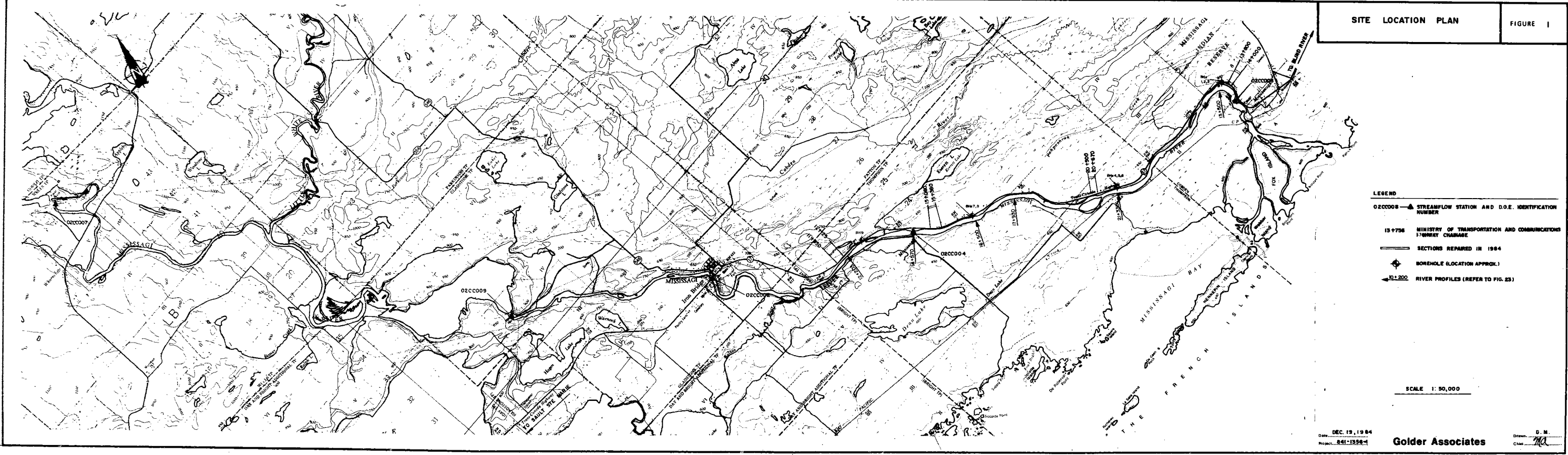


SAULT STE. MARIE DISTRICT

8849 '97

97481

1977



SITE LOCATION PLAN

FIGURE 1

- LEGEND
- 02CC008 — STREAMFLOW STATION AND D.O.E. IDENTIFICATION NUMBER
  - 15+756 MINISTRY OF TRANSPORTATION AND COMMUNICATIONS 1-WAY CHAIRMA
  - SECTIONS REPAIRED IN 1984
  - BOREHOLE (LOCATION APPROX.)
  - 10+200 RIVER PROFILES (REFER TO FIG. 25)

SCALE 1:50,000

DEC. 19, 1984  
PROJECT 841-1984

Golder Associates

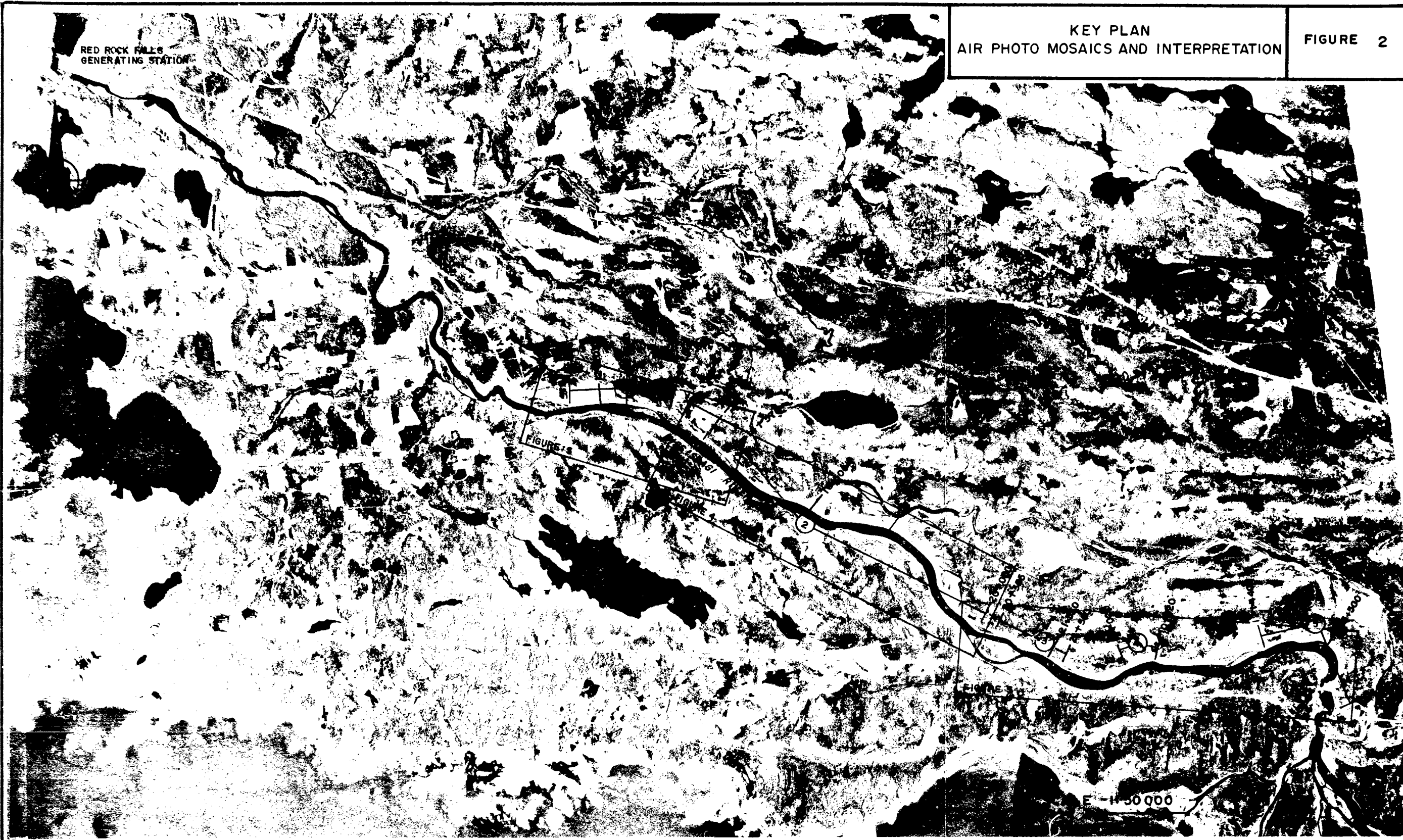
D. M.  
C.M.

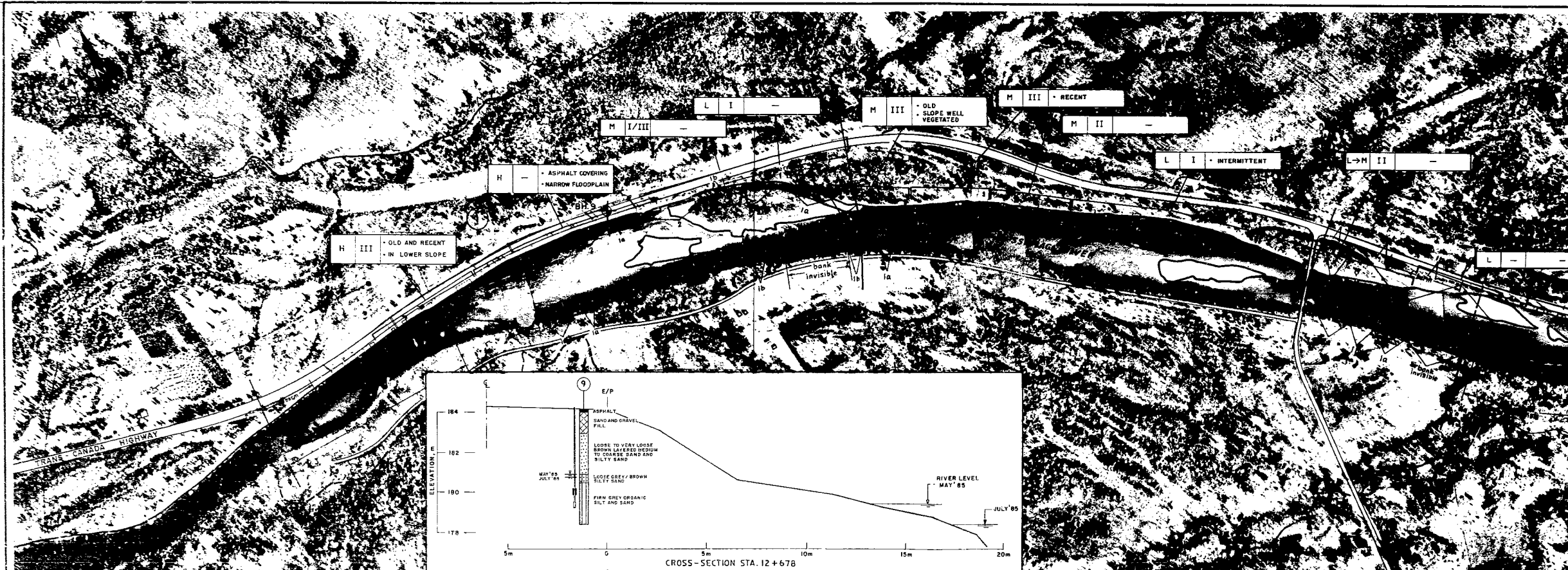


RED ROCK FALLS  
GENERATING STATION

KEY PLAN  
AIR PHOTO MOSAICS AND INTERPRETATION

FIGURE 2





### RIVER BANK CONDITIONS CHAINAGE 11+675 TO 15+000 (THOMPSON TOWNSHIP)

**FIGURE 3**

**LEGEND**

- BOREHOLE LOCATION IN PLAN
- BOREHOLE IN ELEVATION
- WATER LEVEL
- ▬ STRATA PLOT
- SEAL
- PIEZO

L I - INTERMITTENT	FIELD INSPECTION NOTES (APRIL, 1985)
REMARKS	
FAILURE MECHANISM	
I - TOR EROSION	
II - SURFICIAL SLIDING	
III - ROTATIONAL OR BLOCK FAILURE	
BANK HEIGHT (EXPOSED APRIL '85)	
L - < 1 m	
M - 1 to 3 m	
H - > 3 m	
REMOTE SENSING INTERPRETATION BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS (REFER TO APPENDIX C)	

SCALE 1:5,000 (APPROX.)

NOTES

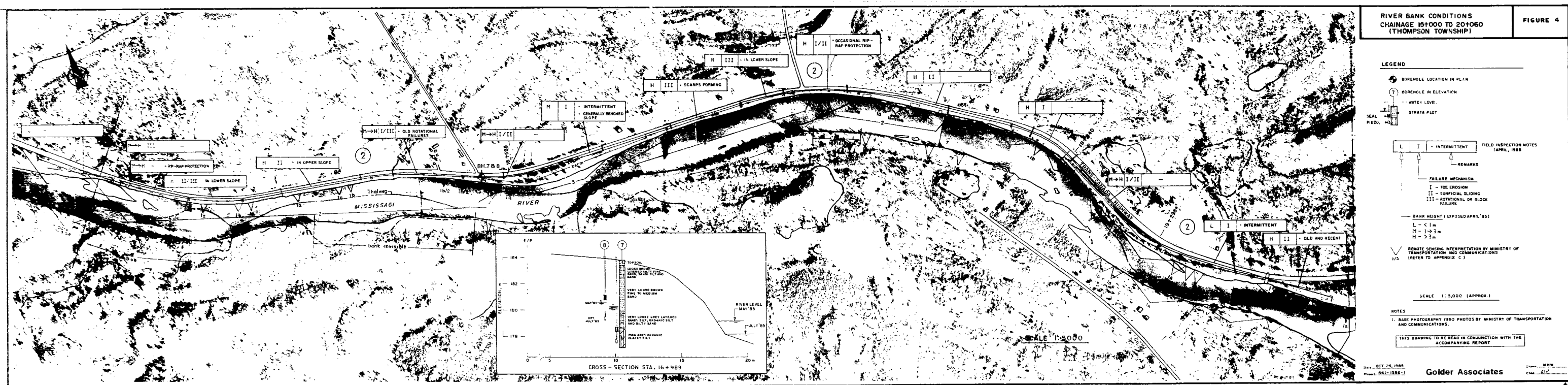
1. BASE PHOTOGRAPHY 1980 PHOTOS BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS.

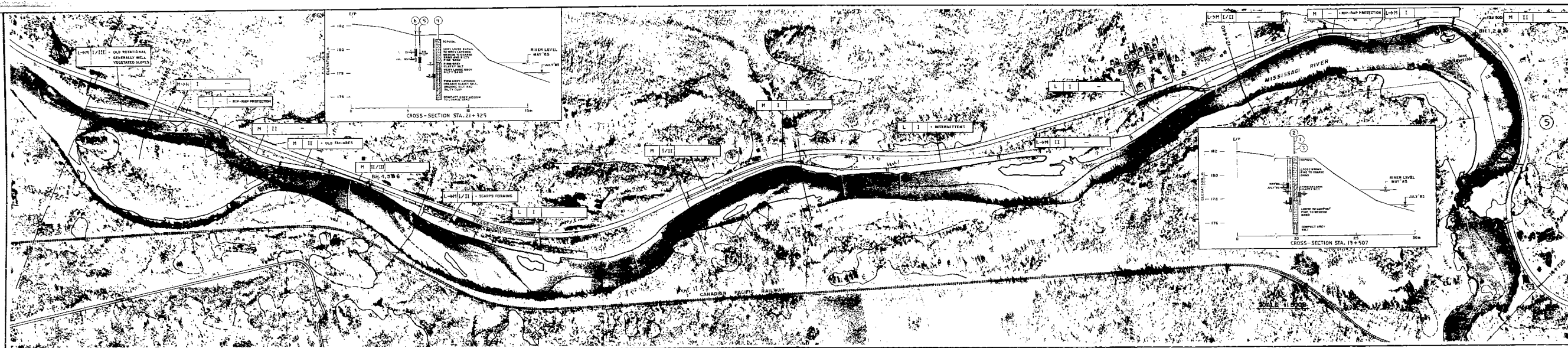
THIS DRAWING TO BE READ IN CONJUNCTION WITH THE  
ACCOMPANYING REPORT

Date: OCT. 25, 1985  
Project: 841-1356-1

**Golder Associates**

Drawn: N.H.W.  
Checked: R.S.P.





RIVER BANK CONDITIONS  
CH. 20+060 TO 22+788 (THOMPSON TWP.)  
CH. 10+000 TO 14+725 (COBLEN TWP.)

FIGURE 5

- LEGEND
- BOREHOLE LOCATION IN PLAN
  - BOREHOLE IN ELEVATION
  - WATER LEVEL
  - STRATY PLOT
  - SEAL
  - PIEZO
  - L I - INTERMITTENT FIELD INSPECTION NOTES (APRIL 1985)
  - REMARKS
  - FAILURE MECHANISM
  - SURFICIAL SLIDING
  - INTERNAL OR BLOCK FAILURE
  - BANK HEIGHT (EXPOSED IN L'90S)
  - L - 6.1m
  - H - 1.93m
  - H - 3.3m
  - ✓ REMOTE SENSING INTERPRETATION BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS (REFER TO APPENDIX C.1)

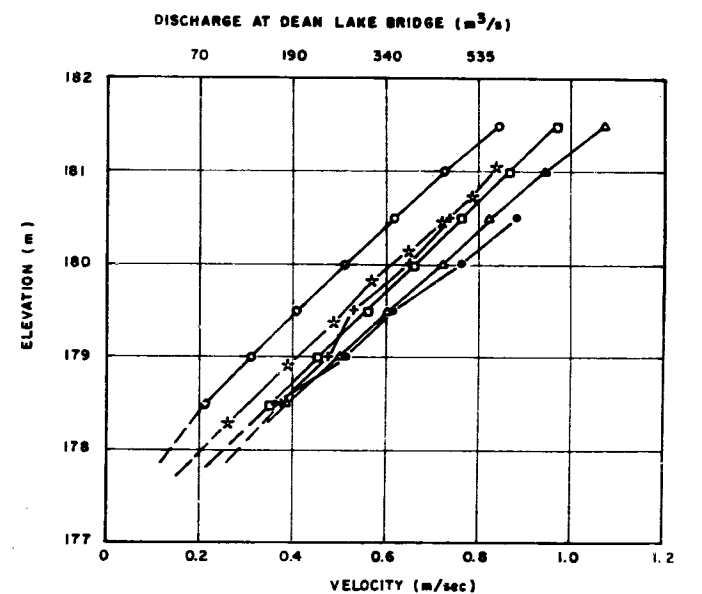
SCALE 1:5,000 (APPROX)

NOTES:  
1. BASE PHOTOS ARE 1980 PHOTOS BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS.  
THIS DRAWING IS IN CONNECTION WITH THE ACCOMPANYING REPORT.

DATE: OCT. 25, 1985  
DRAWN BY: [Signature]  
CHECKED BY: [Signature]  
Colder Associates

MISSISSAGI RIVER PROFILES  
CHAINAGE 12+203 (THOMPSON TWP.) TO  
CHAINAGE 13+500 (COBDEN TWP.)

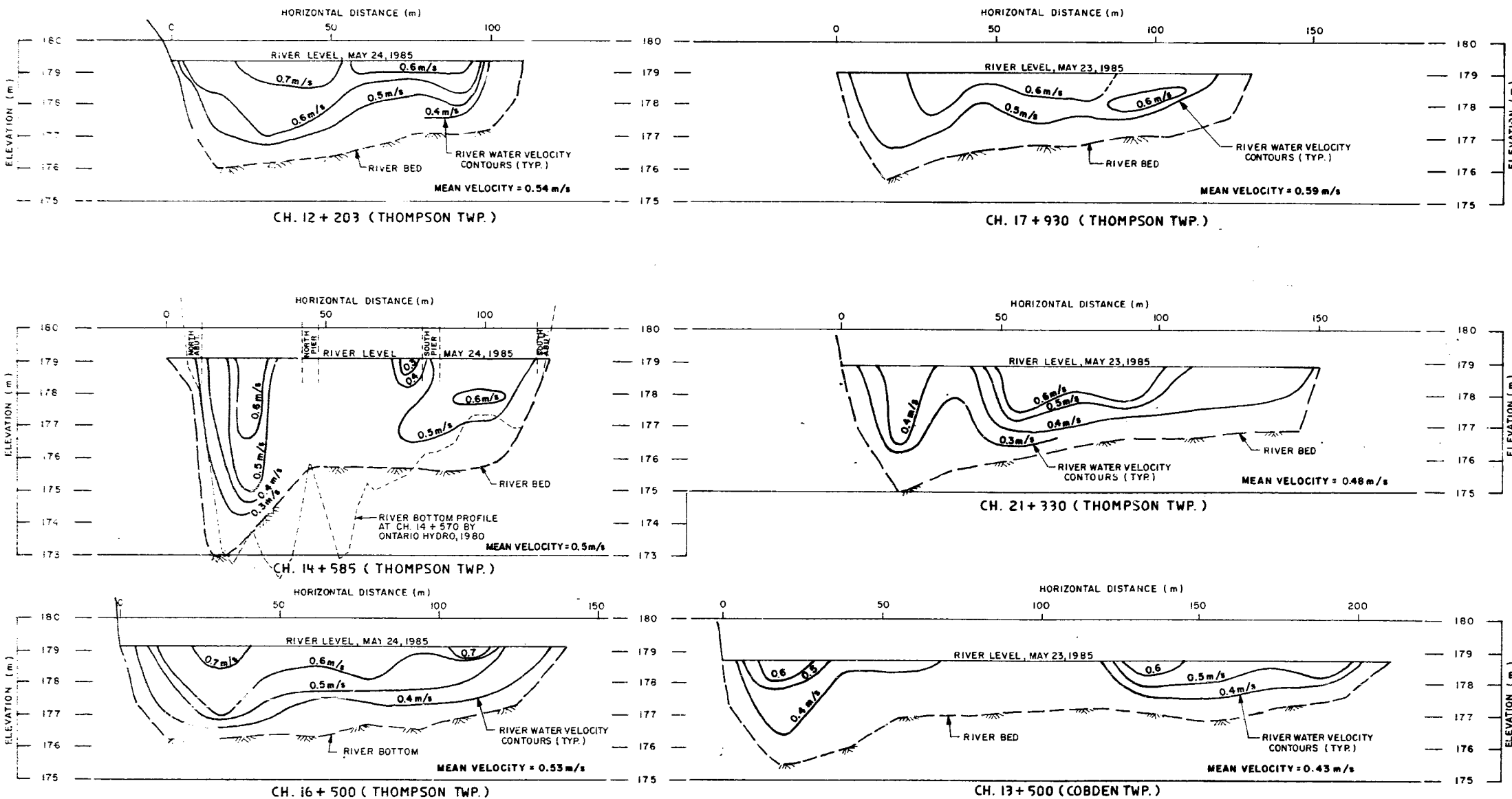
FIGURE 23



LEGEND

- △ CH. 12+203 (THOMPSON TWP.)
- CH. 14+570 " DEAN LAKE BRIDGE
- CH. 16+500 "
- ☆ CH. 21+330 "
- + CH. 13+500 (COBDEN TWP.)
- CH. 14+140 " MISSISSAGI CHUTES

CHANGE IN RIVER WATER VELOCITY WITH RIVER  
WATER ELEVATION



HORIZONTAL SCALE 1:1000; VERTICAL SCALE 1:100

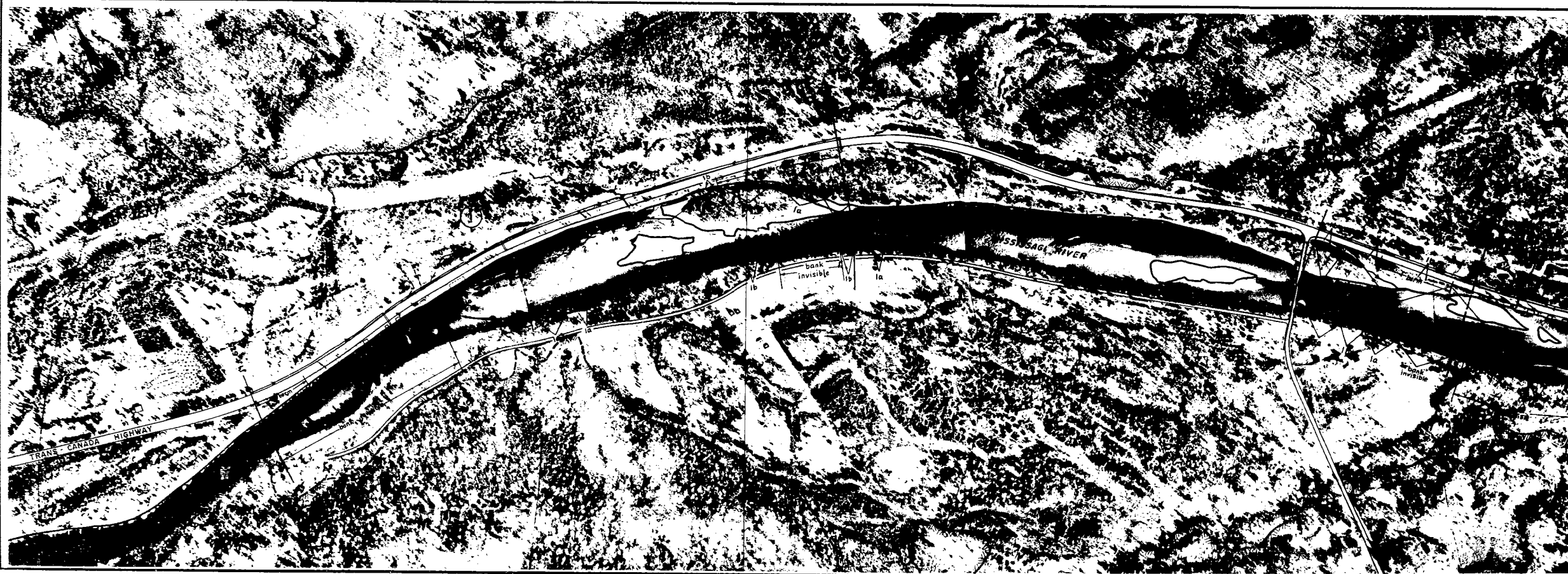
NOTE: ALL CROSS-SECTIONS HAVE BEEN DRAWN  
LOOKING DOWNSTREAM.

Date: OCT. 21, 1985  
Project: 851-1386-1

Golder Associates

Drawn: MNW  
Chkd: MSP





MISSISSAGI RIVER AIR PHOTO INTERPRETATION  
CHAINAGE 11+675 TO 15+000  
(THOMPSON TOWNSHIP)

FIGURE C-1

LEGEND

GENERAL TERRAIN INFORMATION

- APPROXIMATE BEDROCK OUTCROP OR DEPTH OF OVERBURDEN - 1 m.
- WATER ACCUMULATION (NATURAL OR MAN-MADE)
- INTERNALLY DRAINED AREA
- DRAINAGE CHANNEL ARROW INDICATES FLOW DIRECTION
- CULVERT

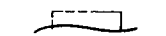
RIVER SEDIMENT

- SHORELINE AS OF 80/11/11
- THALWEG

EROSION OF BANK

TYPE No.	DESCRIPTION
1a	RECENT
1b	RECENT, PROBABLE
2	POTENTIAL, PROBABLE
3	POTENTIAL, POSSIBLE

EXISTING BANK PROTECTION



SCALE 1:5,000 (APPROX.)

NOTES

- 1) INTERPRETATION BY MR. L. TAN, P. ENG. MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
- 2) BASE PHOTOGRAPHY 1980 PHOTOS BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING REPORT

MISSISSAGI RIVER AIR PHOTO INTERPRETATION  
CHAINAGE 15+000 TO 20+060  
(THOMPSON TOWNSHIP)

FIGURE C-2

LEGEND

GENERAL TERRAIN INFORMATION

- BEDROCK OUTCROP OR DEPTH OF OVERBURDEN - 1 m.
- WATER ACCUMULATION (NATURAL OR MAN-MADE)
- INTERIALLY DRAINED AREA
- DRAINAGE CHANNEL ARROW INDICATES FLOW DIRECTION
- CULVERT

RIVER SEDIMENT

- SHOULDER AS OF 80/11/11
- THALWEG (WITHIN RIVER)

EROSION OF BANK

- | TYPE NO. | DESCRIPTION         |
|----------|---------------------|
| 1a       | RECENT              |
| 1b       | RECENT, PROBABLE    |
| 2        | POTENTIAL, PROBABLE |
| 3        | POTENTIAL, POSSIBLE |

EXISTING BANK PROTECTION

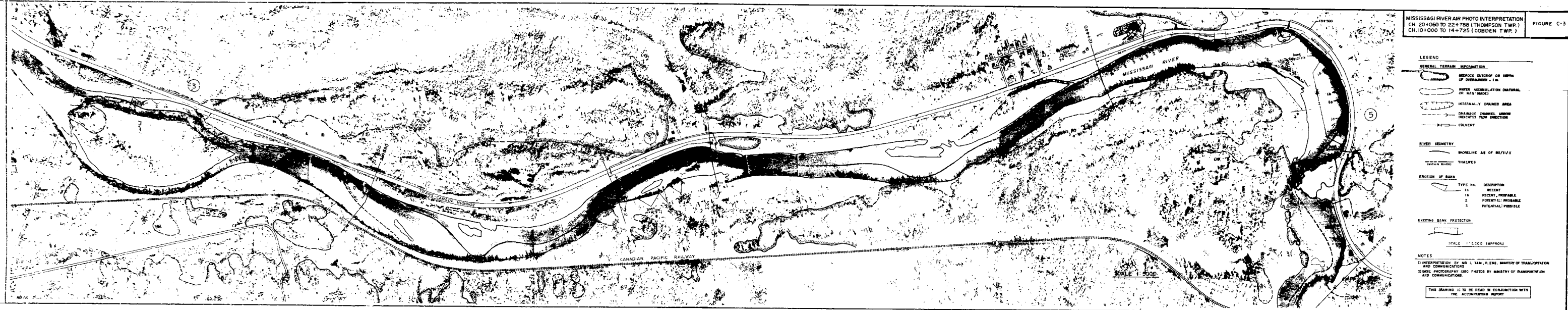
- SCALE 1:5,000 (APPROX.)

NOTES

- (1) INTERPRETATION BY MR. L. TAN, P.ENG. MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
- (2) BASE PHOTOGRAPHY 1980 PHOTOS BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING REPORT

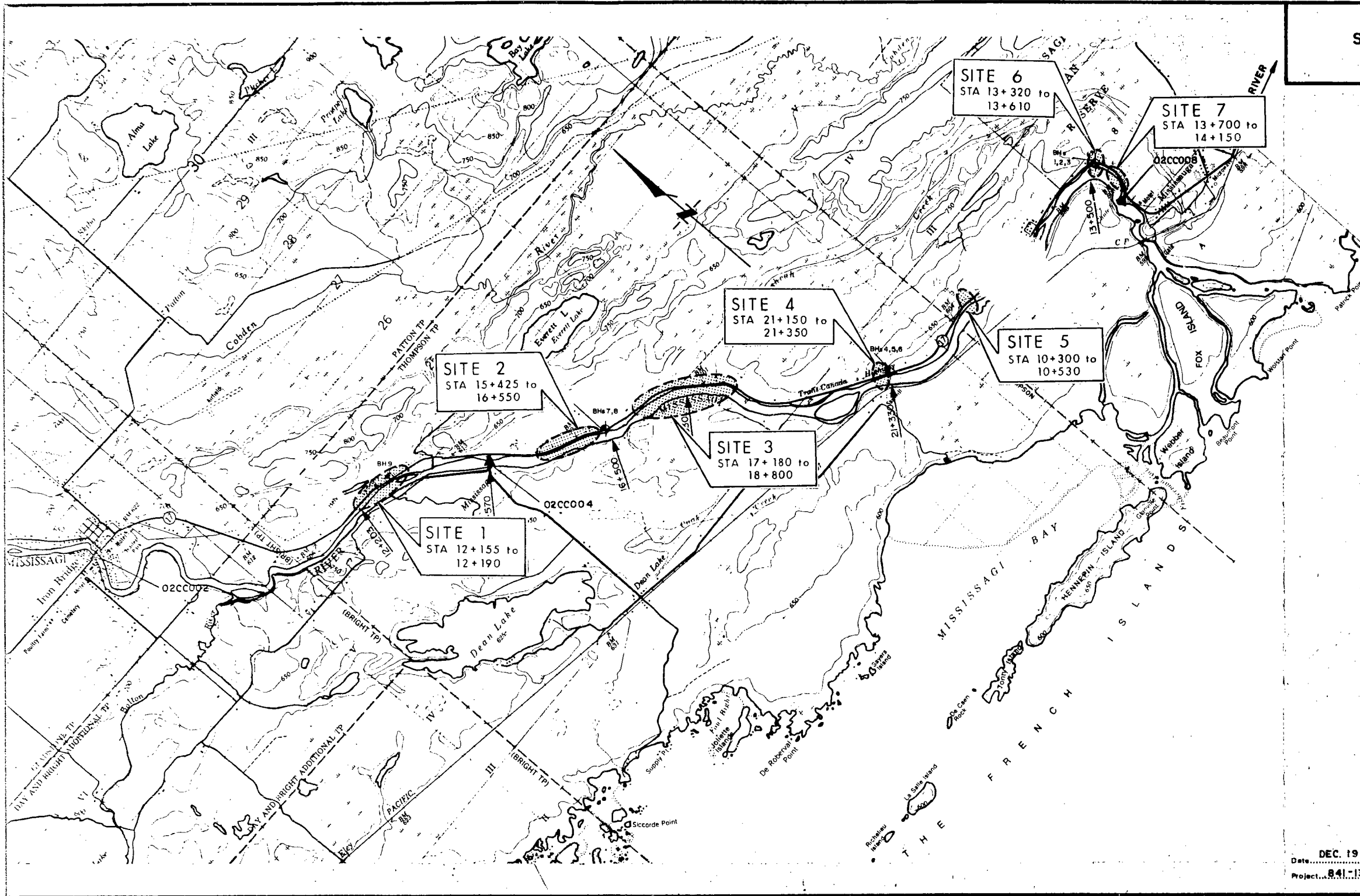






# SITE LOCATION PLAN

FIGURE 1



## LEGEND

02CC008 —▲ STREAMFLOW STATION AND D.O.E. IDENTIFICATION NUMBER

13+756 MINISTRY OF TRANSPORTATION AND COMMUNICATIONS HIGHWAY CHAINAGE

◈ BOREHOLE (LOCATION APPROX.)

◼ REMEDIAL SITES

SCALE 1: 50,000

Date DEC. 19, 1984  
Project 84-1356

From  
**Golder Associates**  
REPORT FOR WP 153-83-00

Drawn D. M.  
Chkd ma