

GEOCRES No:

52F-19

**ENGINEERING GEOLOGICAL INVESTIGATION
MTC TURTLE RIVER BRIDGE CROSSING
DISTRICT OF KENORA, ONTARIO, CANADA**

Prepared For:

**Ontario Ministry of Transportation and Communications,
Foundation Section,
Highway Engineering Division,
Central Building,
1201 Wilson Avenue,
Downsview, Ontario.
M3M 1J8**

**Our Reference Number R81136
MTC Reference No. WP 18.77.09
August, 1981**

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and Communications
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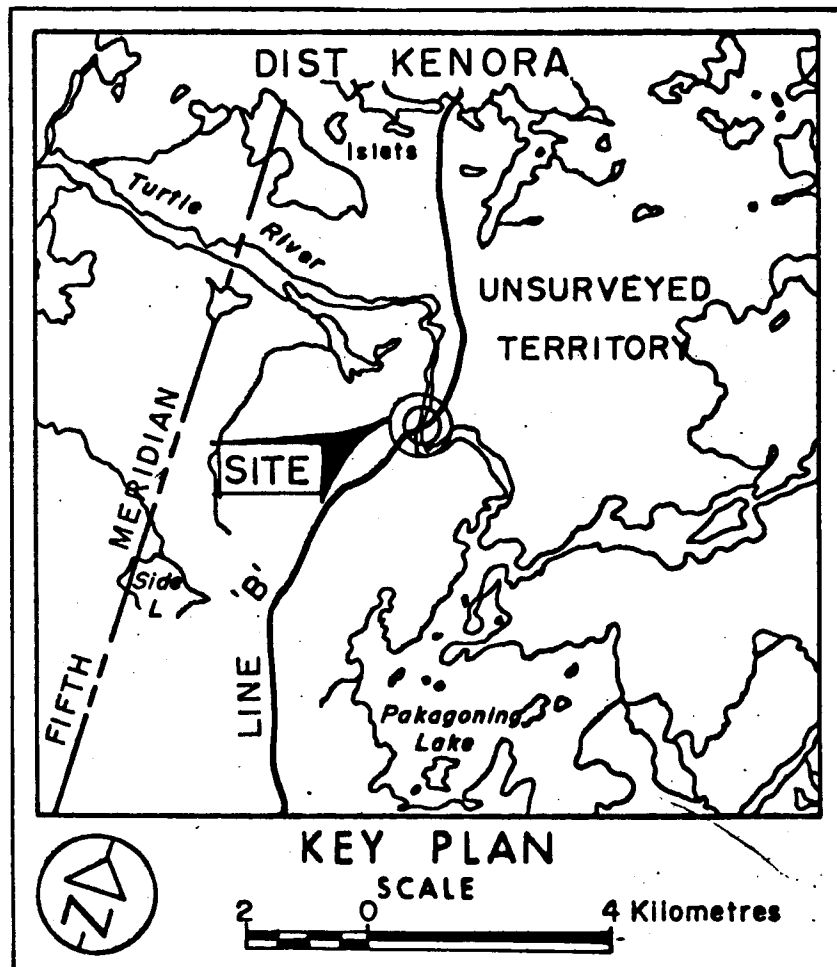
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Reference Number R81136

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1.0 INTRODUCTION

Robert Dodds Limited, Consulting Geological and Geotechnical Engineers, Thunder Bay, Ontario, was retained by the Foundation Section of the Ontario Ministry of Transportation and Communications (herein referred to as MTC) to carry out an engineering geological mapping and subsurface investigation at the proposed Turtle River Bridge crossing. The site is located approximately 66 air kilometers (41 miles) southwest of Ignace, Ontario (see Frontispiece).

The Ministry is planning to construct a single-span bridge to provide access from Highway #622 to Highway #17.

The investigation consisted of two phases:

1. Geological mapping of all bedrock exposures to determine the lithology, the qualitative rock quality designation, the degree of alteration and/or weathering, visual inspection of rock dowel anchorage, capability of the rock and the requirements for bedrock preparation (i.e. remedial measures) for abutments.

1.0 INTRODUCTION (cont'd)

2. A drilling program, utilizing a Winkie portable drilling unit, to determine the depth to refusal at abutment locations as well as determining the nature of the existing bedrock directly under the proposed footings. All boreholes and mapping were located on a borehole plan provided by MTC, Thunder Bay, Ontario.

Verbal authorization to proceed with the work was received from Mr. Murty Devata, P.Eng., of the Foundation Section of the MTC Ontario. Fieldwork was completed July 10, 1981.

2.0 SITE

2.1 Site Location

The proposed site is located on the Turtle River approximately 66 air kilometers (41 miles) southwest of Ignace, Ontario (see Frontispiece). The crossing is situated in a straight section at a point approximately 4 km (2 mi.) north of Pekagoning Lake.

Access to the site is possible only by air and water travel.

2.2 Site Description

The site is situated in a fairly straight river section just downstream from a greater than 90° change in river course. A site map is shown as Drawing No. 1 and is appended at the rear of this report.

Bedrock outcrops are visible over much of the river bank in the area of the western abutment. Outcrops are also visible both upstream and downstream from the eastern abutment location. Accumulation of boulders are prevalent, especially in the region of the eastern abutment. They are seen lining the river bank and amongst the overburden coverage on both east and west sides

2.0 SITE (cont'd)

2.2 Site Description (cont'd)

of the proposed crossing. The ground rises on both the east and west banks with apparent thin surficial cover.

Topography of the area is low to moderate with local variations and probably controlled by subsurface bedrock configuration.

At the time of the fieldwork program, the road alignment had already been slashed on both sides of the Turtle River. Tree cover in the area is predominantly jackpine with both birch and spruce.

3.0 FIELDWORK

The results of the geological mapping and location of the boreholes are shown on Drawing No. 1. Elevations of the boreholes were established by Robert Dodds Limited and tied into an established, temporary benchmark supplied by MTC personnel. The ensuing sections are a description of the bedrock and subsurface conditions.

3.1 Bedrock Geology

The mapping program delineated two major rock units. The first rock type consists of a granitic gneiss which is the dominant rock unit outcropping in the area. The second exposed unit is an intrusive syneo granite which is observed cross-cutting the granitic gneiss on the west bank of the proposed bridge site. In addition to the two rock units outcropping on surface, another unit was encountered within the drilling program. This unit can be classified as a biotite chlorite gneiss.

The rock units can be described as follows:

Type 1

Granitic Gneiss - This is the dominant exposed rock type of the proposed bridge site. It is exposed as

3.0 FIELDWORK (cont'd)

3.1 Bedrock Geology (cont'd)

rounded outcrop on both sides of the river. Outcrops appear to be exposed sections of rock ridges trending as marked on Drawing No. 1. The unit shows varying degrees of alteration dependant on the proximity to the intrusive (Type 2).

The outcrop surrounding the sample location 5 (see Drawing No. 1) was observed to contain structural features in the form of small scale folding and foliation as a result of the intrusion of Type 2, as well as the regional metamorphic event. These have little bearing on the proposed project, hence are mentioned in the interest of completeness but are not described in detail.

Type 1 can be described as fine to medium grained, varying in colour from greyish white to pinkish and displays some effects of alteration. A distinct lineation can be determined within the unit as well as well-developed to poorly -developed gneissic banding. Type 1 was cored over a short distance in Borehole No. 2. For a more detailed description, Borehole Logs are appended at the rear of this report and are discussed in greater detail in later sections.

3.0 FIELDWORK (cont'd)

3.1 Bedrock Geology (cont'd)

Franklin's⁽¹⁾ rock quality classification would categorize this unit as a high rating with an estimated RQD of 75% to 90%. The ultimate bearing capacity of the rock unit is estimated to be in the order of 9580 kN/m^2 (100 tons/ft²).

Type 2

Syeno Granite Intrusive - There is limited surface exposure of Type 2, it is localized dominantly on the western bank and occurs generally as a flat-lying sill structure of variable thickness (maximum .5 m - minimum .1 m, see Drawing No. 1). The intrusive is rich in potassium feldspar which accounts for the pink-orange tone of the unit. In some localized zones, the unit can be very coarse grained with well-developed potassium feldspar rhombhedrals. However, on the whole, the unit is classed as medium to coarse grained.

(1). FRANKLIN, J.A., EXERPT FROM "Methods of Geological Engineering" (Goodman, 1976) p 51 Rock rating based on bedding and/or fracture spacing related to rock strength.

3.0 FIELDWORK (cont'd)

3.1 Bedrock Geology (cont'd)

Type 2 was also encountered in coring Borehole No. 2. (See Borehole Logs appended at the end of this report, and Section 3.3, entitled Drilling Program). Franklins rock quality classification would catagorize this unit as a high rating with an estimated RQD of 75% to 90%. The ultimate bearing capacity of the rock unit is estimated to be in the order of 9580 kN/m^2 (100 tons/ft^2).

Type 3

Biotite Chlorite Gneiss - This unit was only encountered in the core from Borehole No. 2, a brief description is included here for completeness. The unit is medium grained, greenish-grey. Its colour results from concentration of biotite and chlorite.

3.0 FIELDWORK (cont'd)

3.2 Surficial Cover

Surficial cover, in the area of the proposed bridge site, consists primarily of boulders intermixed with variable interstitial material. The exact nature of this interstitial material is difficult to ascertain due to the high concentration of boulders which made regular soil sampling techniques difficult. However, examination of terrain mapping by the Ontario Ministry of Natural Resources (MNR) indicates that the site location is situated in a glacial fluvial outwash plain at the toe of a ground moraine. This would be in agreement to the accumulations of boulders. The interstitial materials are expected to be predominantly sands and gravels.

3.0 FIELDWORK (cont'd)

3.3 Drilling Program

The drilling program consisted of two boreholes advanced in the locations shown on Drawing No. 1. Due to the extreme overburden conditions, the boreholes were advanced with an AX core barrel. This method of advancement did not allow soil sampling or standard penetration tests to be incorporated in the drilling cycle.

Borehole No. 1 was advanced to a depth of 3.8 m (12.5 ft.). No bedrock was encountered at this location within the sampled interval. Numerous boulders up to .6 m (2 ft.) in diameter were encountered during advancement. These boulder conditions forced abandonment of Borehole No. 1 when a gravel seam was encountered which resulted in a loss of water return.

The location of Borehole No. 2 was changed due to the extreme conditions of Borehole No. 1 (see Drawing No. 1). Bedrock was encountered in Borehole No. 2 at a depth of 3.3 m (10.7 ft.) and was cored to a depth of 6.6 m (21.5 ft.). Rock Types 1 - 3 were encountered within the borehole. The condition of the core was fair, with a noticeable amount of machine breaks and

3.0 FIELDWORK (cont'd)3.3 Drilling Program (cont'd)

grinding. All three units were relatively free of fractures and no core was lost. Soundness of the bedrock was further indicated by good water return throughout the coring process. Drilling of Types 1 and 2 required heavy drill pressure. The extreme hardness of units 1 and 2 (high concentrations of quartz) resulted in a low diamond bit life. Type 3, on the other hand, required only moderate drill pressure and yielded better bit wear per foot drilled.

Drawing No. 2 shows a surface section surveyed from chainage 26+658 - 26+675. The positions of Boreholes, Nos. 1 and 2, are also located on this section. In addition to Drawing No. 2, Borehole Logs (Drawings, Nos. 3 and 4) are also appended at the rear of this report.

4.0 DISCUSSION

4.1 Proposal

The Ontario Ministry of Transportation and Communication is planning on constructing a road (Bending Lake Road) to joint the existing Highways, Nos. 622 and 17. The chosen alignment will necessitate the crossing of the Big Turtle River at the location indicated on the Frontispiece. The proposed bridge will be of a single span, with the leading edges of the two abutments shown to be located at stations 26+622 and 26+664.

4.2 General

The field investigation of the proposed location of the Turtle River crossing has indicated that sound bedrock exists to support foundation footings on the western abutment. It also indicates, however, that overburden coverage at the eastern abutment location is more extensive than was originally anticipated. Examination of eastern bedrock exposure suggests that the bedrock below the proposed footing is steeply dipping and on the leeward edge of a bedrock embankment.

4.0 DISCUSSION (cont'd)

4.2 General (cont'd)

It is suggested that this bedrock feature is a possible result of graben faulting which would also explain the configuration of the river course and the steeply dipping exposed bedrock interfaces surrounding the abutment locations. It is, therefore, recommended that in light of these considerations, the span of the bridge be increased slightly to place the foundation of the eastern leading edge of the proposed structure on more competent subsurface bedrock.

4.3 Foundations

The western abutment location will be founded on the bedrock surface. The bedrock at this location is interpreted to be a granitic gneiss. The ultimate bearing capacity of this rock type is estimated to be 9580 kN/m^2 (100 tons/ft^2). The bedrock at this location is already exposed.

The eastern abutment location should be founded on bedrock. An average of 3.4 m (11 ft.) of overburden will have to be stripped, based on the conditions encountered at Borehole No. 2, to expose the bedrock surface. The

4.0 DISCUSSION (cont'd)4.3 Foundations (cont'd)

bedrock is interpreted to be granitic gneiss/syeno granite. It is estimated that the ultimate bearing capacity for this abutment will be 9580 kN/m^2 (100 tons/ft^2).

The ensuing section describes foundation preparation of the abutment locations.

Earth pressures acting on the abutments can be calculated as shown on Drawing No. 5 (standard drawing). The effects of any point and surcharge loading should also be indicated in design and may be calculated as shown on Drawings, Nos. 1, 6 and 7. Rockfill material behind abutments should be a free draining, granular material. Water tending to collect behind the abutment should be drained. A typical drainage system is shown on Drawing No. 8 (standard). Weep holes through the abutment are not generally recommended due to a tendency for blockage through siltation or freezing.

4.0 DISCUSSION (cont'd)

4.4 Excavation Considerations

Excavation complications are anticipated on the eastern abutment if the footings are to be founded on the bedrock surface. The elevation of the bedrock interface will be below the water surface elevation of the Turtle River. Hence, dewatering and shoring of the excavation will be necessary to successfully complete the remedial measures outlined in this report. Sheet piling methods are not considered feasible as an excavation support in this location due to the predominant occurrence of boulders in the overburden coverage, which would make advancement difficult or impossible. Consideration could be given to steel H pile (soldier beams) incorporating timber lagging (transverse members) installed in conjunction with the rock fill around the abutments.

Pressure acting on shoring and bracing can be calculated as shown on Drawing No. 9.

4.0 DISCUSSION (cont'd)

4.5 Remedial Measures

4.5.1 Bedrock Preparation

Abutment Locations

Prior to pouring of the foundations, it will be necessary to remove all overburden including boulders, to expose the bedrock surfaces. Any loosened material should be removed and any debris in cracks should be removed to a depth of at least 100 mm (4 in.) below the bedrock surface. The final cleaning could be carried out either by brooming or by the use of compressed air and water. The natural slopes of the sites will facilitate cleaning by these methods.

The normal coefficient of friction between bedrock and concrete can be taken as 0.6, provided that the surface of the rock is roughened. At this site, it is assumed that the bedrock surface will be relatively smooth at the abutment locations and, therefore, a reduced coefficient of friction of 0.4 is recommended. Since the abutments will be founded on a sloping bedrock surface, no resistance to toe movement is provided. Therefore, rock anchors will be required to resist slippage.

4.0 DISCUSSION (cont'd)

4.5 Remedial Measures (cont'd)

4.5.2 Rock Anchors

Since at least the western abutment will be submerged during the high water levels of the Big Turtle River, the probability of the ingress of water along the bedrock interface dictates that the normal coefficient of friction between the footings and the bedrock could be reduced to zero. Rock anchors would then be required to resist the horizontal forces imposed by the abutments.

These anchors could consist of reinforcing #8 bar, inserted into pre-drilled holes in the bedrock, and filled with cement grout. The general practice is to drill holes into the bedrock approximately 3 to 4 times the diameter of the reinforcing rod to a depth of 40 to 60 times the diameters. The hole should then be cleaned of any debris and then filled with grout. Once filled with grout, the rods can be inserted forcing out the excess grout and thereby precluding the possibility of air voids existing within the rock hole. Approximately 6 in. (150 mm) of the reinforcing

4.0 DISCUSSION (cont'd)

4.5 Remedial Measures (cont'd)

4.5.2 Rock Anchors (cont'd)

rod should extend above the bedrock surface and cast into the footing. The grout could consist of cement, water and sand. The compressive strength of the grout should be in the order of 28 kN/m^2 (4,000 psi).

Design of the location of the rock anchors for the abutments should allow enough leeway as to permit changes of locations in the field for the following reasons:

- a) Rock anchors should not be placed along open fractures or joints,
- b) The rock anchors ideally should be drilled into the granitic gneiss or syeno granite.

Rock anchors may also be constructed by grouting in commercially available mechanical rock bolts. These rock bolts should be installed to a minimum depth of 1.2 m (4 ft.) into the bedrock and grouted with a minimum 700 kN/m^2 (100 psi) shear strength grout. Rated capacity for pullout of 75 mm

4.0 DISCUSSION (cont'd)

4.5 Remedial Measures (cont'd)

4.5.2 Rock Anchors (cont'd)

(3/4 in.) rock bolts utilizing mechanical anchorage only is 6.8 tonnes (15,000 lbs). The capacity can be doubled when strength of the grout is taken into consideration.

4.5.3 Abutment Approaches

The overburden material existing in the area of the abutment approaches is not expected to produce any stability problems. Preparation of approaches should consist of stripping the organic material from the surface. The underlying material is of adequate strength to act as a reliable subgrade for sand fill or rock fill for approach construction. The following are suggested side slopes for stable abutment approach construction given in a ratio of horizontal to vertical:

- | | |
|--------------------------|-----------|
| (1) sand and gravel fill | Slope 2:1 |
| (2) rock fill | Slope 1:1 |

4.0 DISCUSSION (cont'd)

4.5 Remedial Measures (cont'd)

4.5.4 Scour Protection

The river channel at the proposed bridge crossing is cut into bedrock. As the river flow will never achieve the threshold velocities for scouring of the channel surface, scour protection will not be required at the abutment locations.

5.0 ALTERNATE DESIGN CONSIDERATIONS

5.1 General

Preliminary design of the single-span bridge crossing of the Turtle River was based upon the assumption that bedrock was closer to surface on the eastern bank than the site investigation has proven.

On direct regard to this fact, it has been suggested that the span be increased to ensure a stable bedrock surface for the eastern abutment footing. Even with this increase in span, there are still expected excavation complications in the area of the eastern abutment due to the water level of the Turtle River.

Therefore, two alternative design possibilities are presented for consideration. These are described below.

5.2 Redesign of Eastern Abutment Footings

Redesign of the eastern abutment can be considered such that the footings are founded within the existing overburden material. A suggested elevation for the footing base is 392.5 m (1291.3 ft.).

5.0 ALTERNATE DESIGN CONSIDERATIONS (cont'd)

5.2 Redesign of Eastern Abutment Footings (cont'd)

Redesign of the abutment footings in this manner would greatly reduce the problems anticipated in excavating to bedrock but the footings would have to be enlarged to account for the lower bearing capacity of the overburden stratum in comparison to sound bedrock.

The recommended net allowable bearing pressure that abutments may apply to this overburden is 240 kN/m^2 (5000 psf).

Prior to pouring of concrete for the abutment footings, the exposed subgrade should be examined. Any loosened material or otherwise unacceptable material should be removed and replaced with compacted granular material or lean concrete.

5.0 ALTERNATE DESIGN CONSIDERATIONS (cont'd)

5.3 Realignment

Realignment of the right-of-way can be considered so that the crossing occurs slightly to the north of the present location (see Photo No. 1). With realignment both the eastern and the western abutment footings could be seated directly on exposed bedrock. This increase of span in this realignment would be of approximately the same amount as has already been proposed for the original location and excavation problems would be eliminated.

Realignment is considered to be the most viable alternative to the proposed design in view of the results of the completed site investigation.

6.0 CONCLUSIONS

1. Bedrock is exposed along the river channel at the location of the western abutment. The rock is interpreted as a granitic gneiss. It was concluded that the ultimate bearing capacity of this rock is $9580 \text{ kN/m}^2 (100 \text{ tons/ft}^2)$.
2. Overburden coverage at the eastern abutment location is deeper than originally anticipated. The coverage is at least 3.8 m (12.5 ft.) and probably deeper. The covering overburden at this location contains a high concentration of large boulders. The underlying bedrock is granitic gneiss and syeno granite. It was concluded that the ultimate bearing capacity of this rock is $9580 \text{ kN/m}^2 (100 \text{ tons/ft}^2)$.
3. Excavation problems are anticipated on the eastern abutment location along the original alignment due to the low relative elevation of the bedrock surface.
4. No additional scour protection than presently designed will be required at this site.

6.0 CONCLUSIONS (cont'd)

5. Alternative design should be considered such as founding the east abutment in the overburden soils or shifting the bridge location slightly to the north to allow both abutments to be founded directly on bedrock.

7.0 RECOMMENDATIONS

1. It is recommended that the span of the bridge be increased to ensure that adequate bedrock support is provided for the eastern abutment footings.
2. It is recommended that rock anchors be installed at the abutment locations to provide resistance to horizontal movement.
3. Design of the location of the rock anchor should allow for field alterations based on the following criteria:
 - a) Anchors should not be drilled along fractures or joints,
 - b) The anchors ideally should be drilled into the granite or syeno granite units.
4. Stripping of the overburden cover at the eastern abutment locations should provide for removal of large boulders to expose the bedrock surface.
5. Bedrock surface and/or joints should be cleared of all debris. The final cleaning of the surface can be carried out by either brooming or by compressed air.

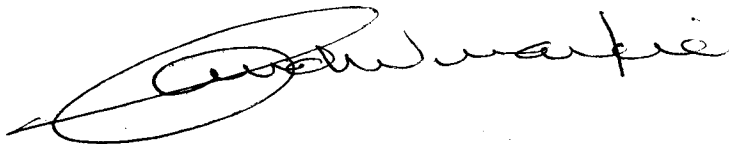
7.0 RECOMMENDATIONS (cont'd)

6. In view of the construction problems anticipated with the eastern abutment, it is recommended that consideration be given to alternate design proposals included in this report.

All the foregoing respectfully submitted.

Yours very truly,

ROBERT DODDS LIMITED,

A handwritten signature in cursive script, appearing to read "Andrew Mackie". The signature is written in dark ink and is positioned above the printed name.

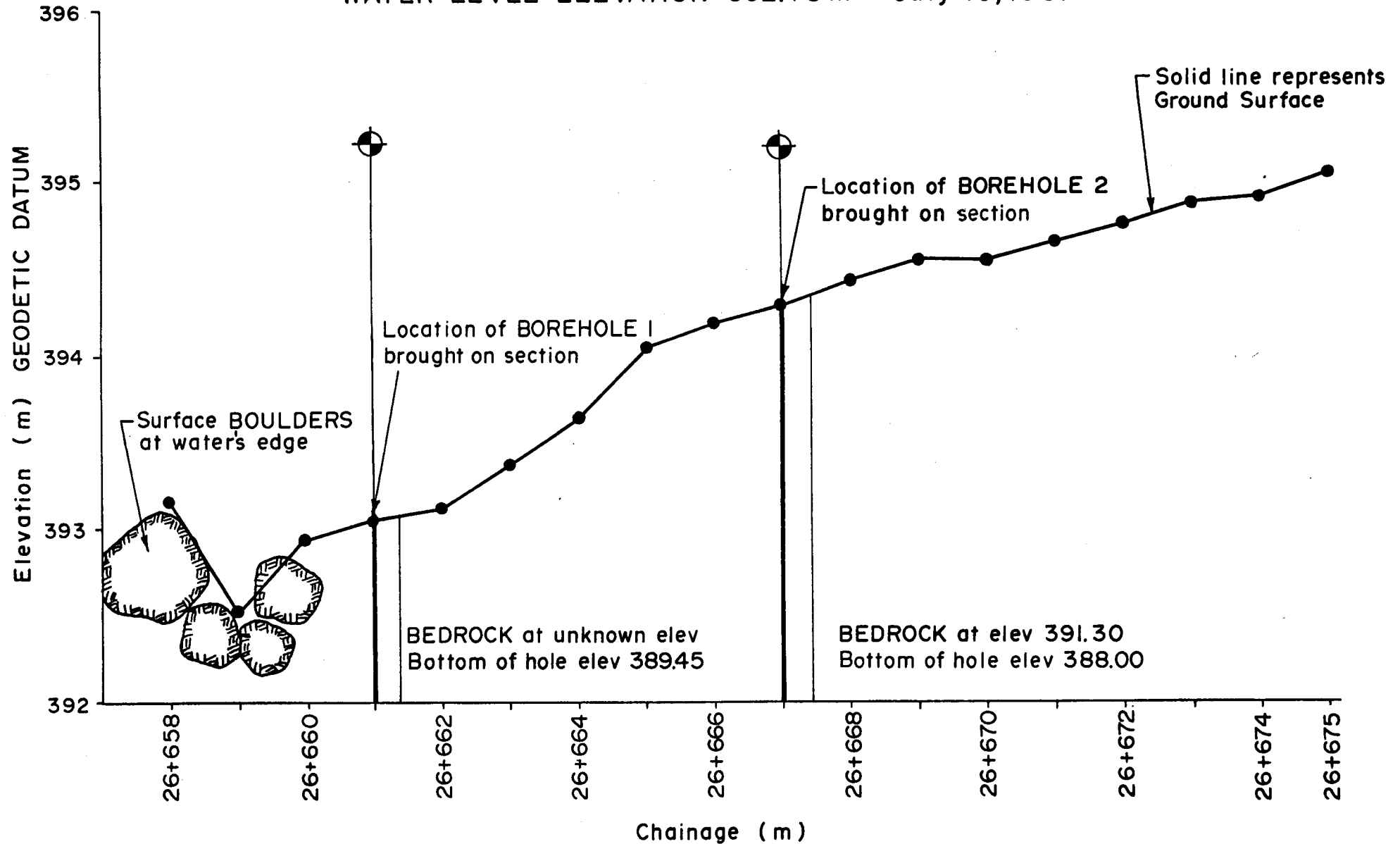
ANDREW MACKIE, BSc., Eng.

A handwritten signature in cursive script, appearing to read "Robert B. Dodds". The signature is written in dark ink and is positioned above the printed name.

ROBERT B. DODDS, Ph.D., P.Eng.

SECTION ALONG BRIDGE ALIGNMENT: CHAINAGE 26+658 - 26+675

WATER LEVEL ELEVATION 392.78 m July 10, 1981



Vertical Exaggeration: $\frac{5}{2} = 2.5$ Times



Drawing No. 3

METRIC

W P 18.77.09 LOCATION 26+661 o/s 4.5 m RT. ORIGINATED BY Am
DIST 19 HWY Bending Road BOREHOLE TYPE AXT Rock Core Cased COMPILED BY Am
Alignment 'B'
DATUM Geodetic DATE July 9, 1981 CHECKED BY [Signature]

[illegible]

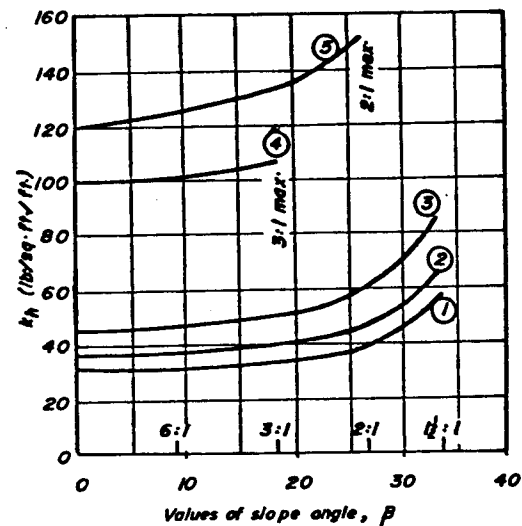
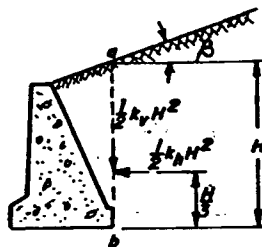
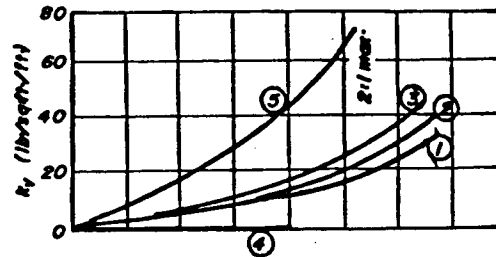
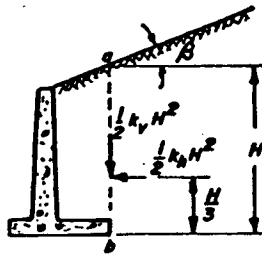


Drawing No. 4

METRIC

W P 18.77.09 LOCATION 26+667 o/s 4.5 m LT. ORIGINATED BY Am
 DIST 19 HWY Bending Road BOREHOLE TYPE AXT Rock Core Cased COMPILED BY Am
Alignment 'B'
 DATUM Geodetic DATE July 9 & 10, 1981 CHECKED BY [Signature]

[illegible]

**NOTES:**

Numerals on curves indicate soil types as described below

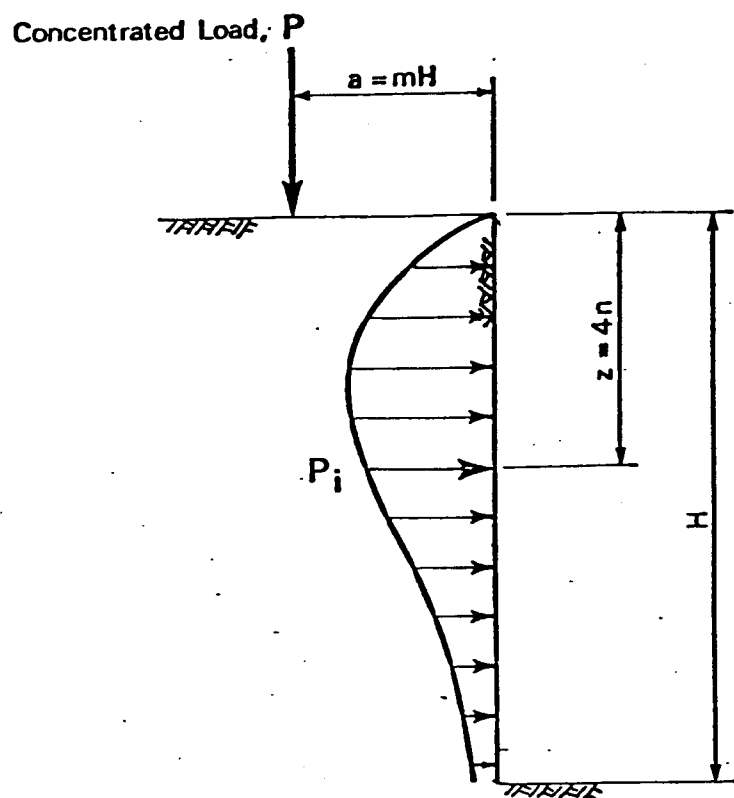
For materials of Type 5, computations should be based on a value of H four feet less than actual value

Types of backfill for retaining walls

- ① Coarse-grained soil without admixture of fine soil particles, very free-draining (clean sand, gravel or broken stone)
- ② Coarse-grained soil of low permeability due to admixture of particles of silt size
- ③ Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones
- ④ Soft or very soft clay; organic silt; or soft silty clay
- ⑤ Firm or stiff clay that may be placed in such a way that a negligible amount of water will enter the voids between the chunks during floods or heavy rains

EARTH PRESSURE CHARTS FOR RETAINING WALLS

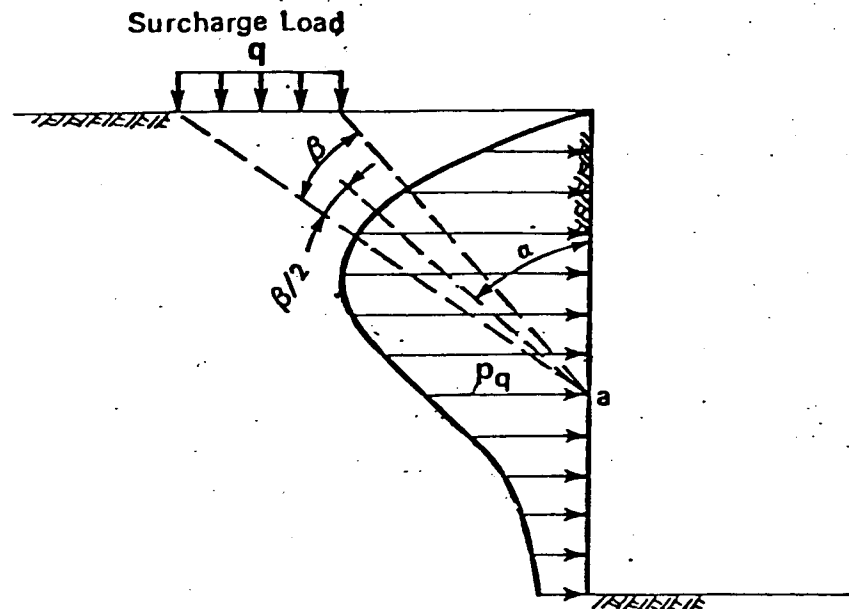
-less than 20 feet high-



$$P_i = 1.77 \frac{P}{H^2} \frac{m^2 n^2}{(m^2 + n^2)^3} \quad \text{for } m > 0.4$$

$$P_i = 0.25 \frac{P}{H^2} \frac{n^2}{(0.16 + n^2)^3} \quad \text{for } -m < 0.4$$

LATERAL PRESSURES DUE TO POINT LOADS



p_q = Horizontal Pressure at Point 'a'

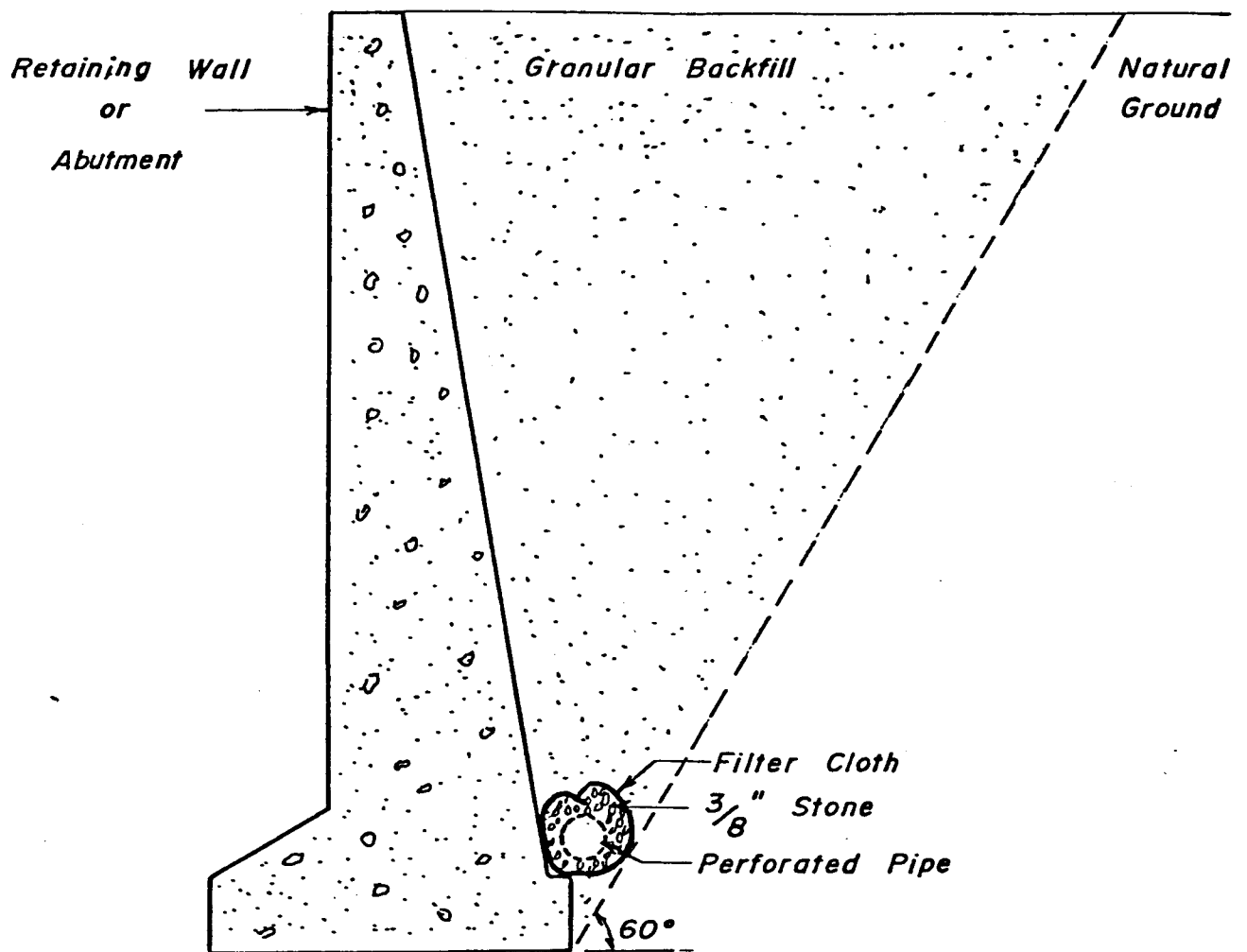
β = Angle at Visibility (Radians)

α = Angle between Vertical and Bisector at β (Radians)

q = Strip Load

$$p_q = \frac{2q}{\pi} (\beta + \sin \beta) \sin^2 \alpha + \frac{2q}{\pi} (\beta - \sin \alpha) \cos^2 \alpha$$

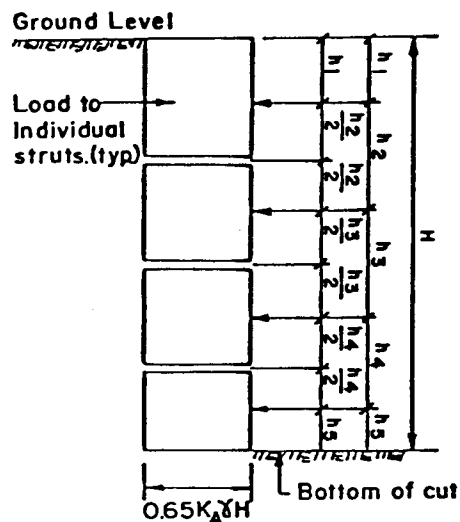
LATERAL PRESSURE DUE TO STRIP LOAD



BACKFILL DRAINAGE

PROBABLE DISTRIBUTION OF EARTH PRESSURES BEHIND BRACED CUTS

SANDS and COHESIONLESS SILTS



Recommended Values:

$$K_A = 0.33$$

$$\gamma (\text{bulk unit wt.}) = 1700 \text{ kg/m}^3 \text{ (110 pcf)}$$

$$\gamma (\text{submerged unit wt.}) = 960 \text{ kg/m}^3 \text{ (60 pcf)}$$

NOTES:

1. The apparent pressure diagrams as shown above do not bear any resemblance to the real distribution of earth pressure against the sheeting of the sides of a cut; they are merely an artifice for calculating values of the strut loads that will not be exceeded in any real strut in a similar open cut.
2. The system must be checked for partial excavation condition.
3. If surcharge loadings are present at or near the ground surface, these must be included in the lateral pressure calculation.
4. If the free water level is above the base of the excavation, the hydrostatic pressure must be added to the above pressure distribution for sands.
5. The above pressure distribution in sand should be used with caution for cuts of depth greater than 40 feet.

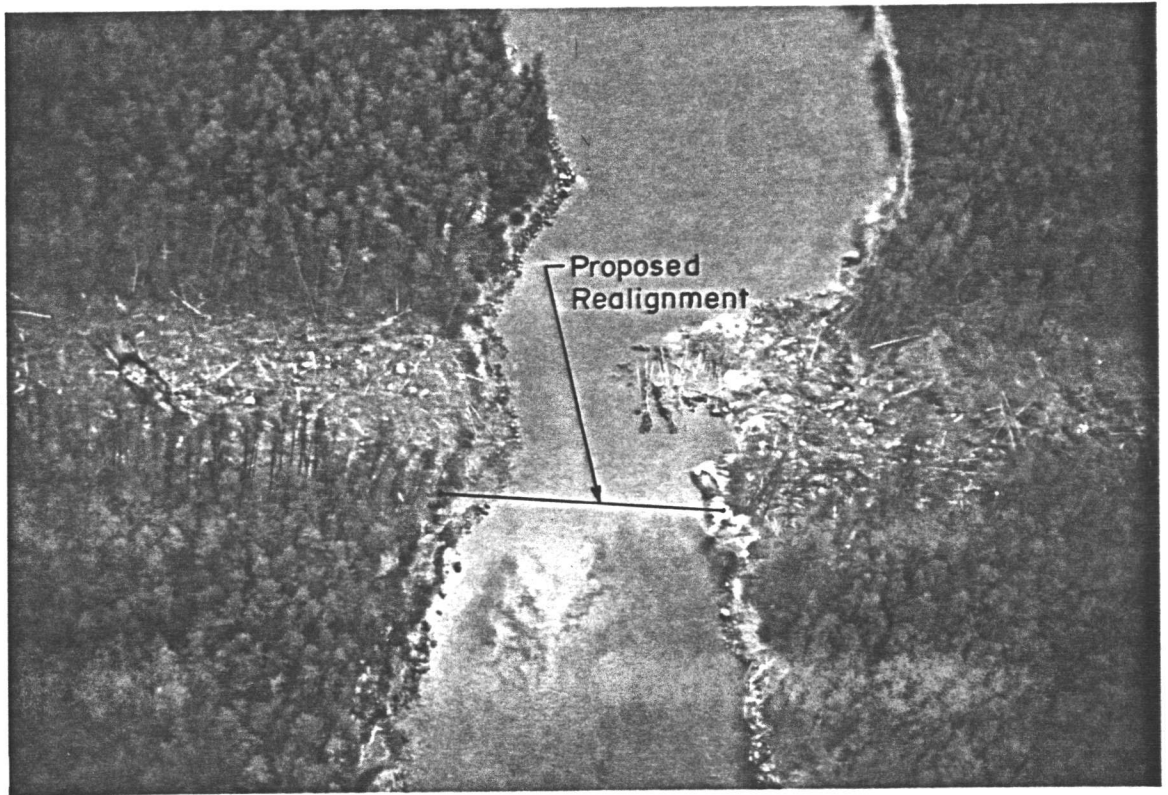


PHOTO NO. 1

Proposed Realignment of the Turtle River Crossing

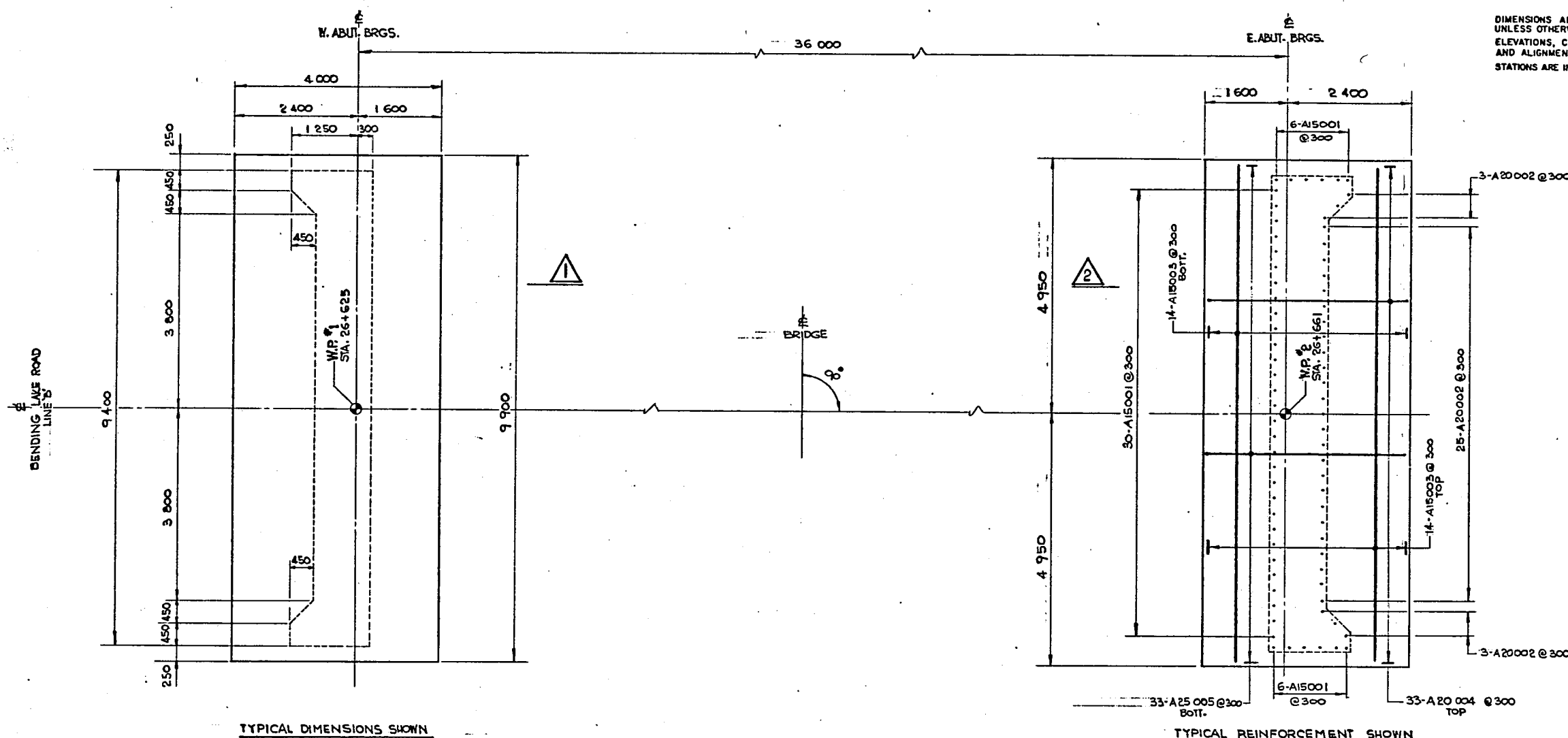
METRIC

DIMENSIONS ARE IN MILLIMETRES
UNLESS OTHERWISE SHOWN.
ELEVATIONS, COORDINATES, CURVE
AND ALIGNMENT DATA ARE IN METRES.
STATIONS ARE IN KILOMETRES + METRES.

DIST. 19
CONT No
WP No 18-77-09

TURTLE RIVER BRIDGE
AT BENDING LAKE ROAD
FOOTINGS

SHEET

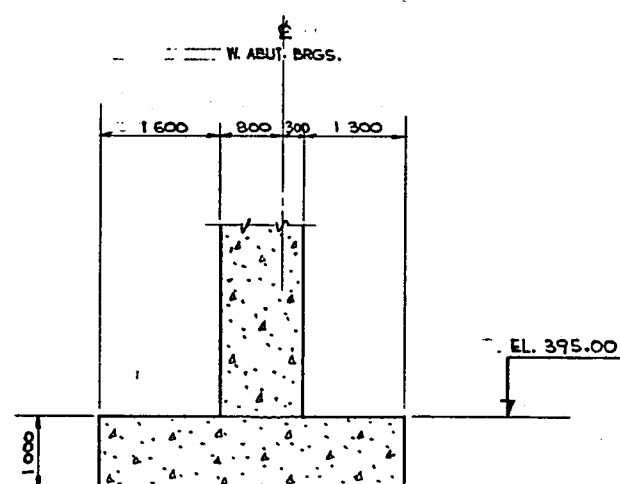


TYPICAL DIMENSIONS SHOWN

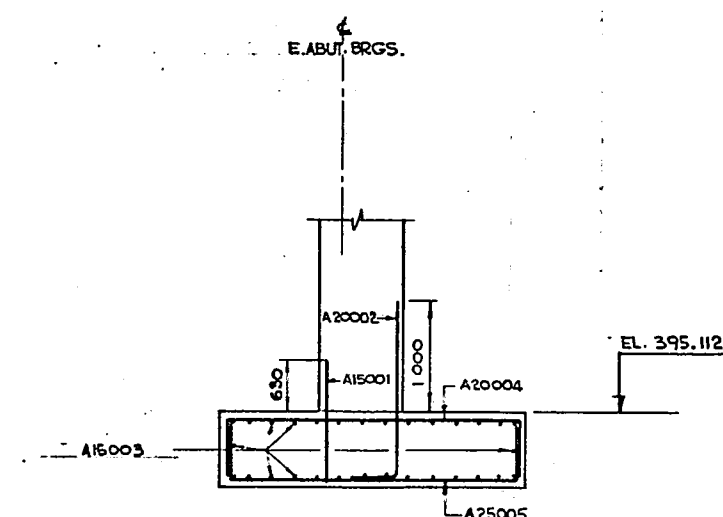
FOOTING LAYOUT

1:50

TYPICAL REINFORCEMENT SHOWN



1:50



2:50



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN			
CHECK			
DRAWING			

DATE DEC. 81
CHECK W.M.
CHECK A.W.
LOADS CHECKED C.M.
SITE No 41-121 DWG 3

METRIC

NOTE:
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

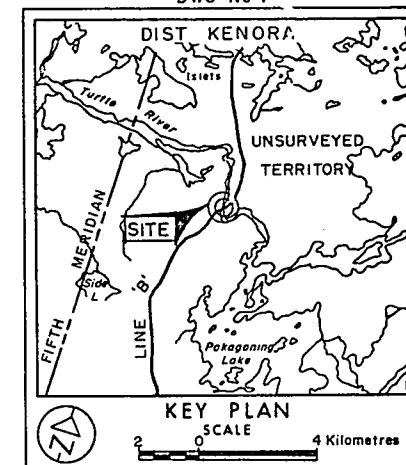
CONT No
WP No 18-77-09

TURTLE RIVER CROSSING
BORE HOLE LOCATIONS & SOIL STRATA



SHEET
32

ROBERT DODDS LIMITED
DWG No.



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation
- ▲ Location of Depth Sounding
- ▨ Bedrock Outcrop

No	ELEVATION	STATION	OFFSET
1	393.3	26+661	4.5 RT.
2	394.6	26+667	4.5 LT.

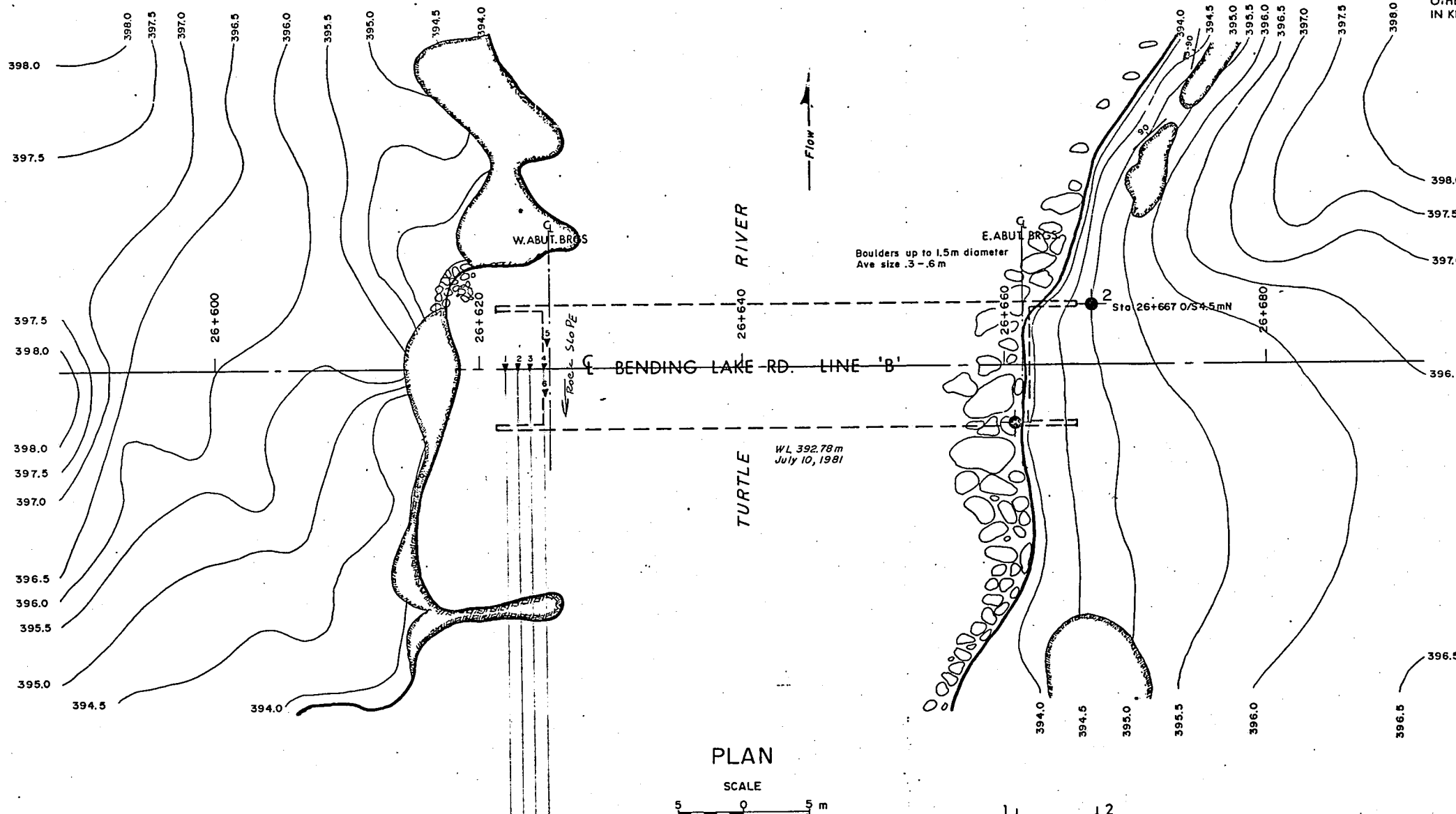
SOUNDINGS	ELEV. OF BEDROCK
1 391.46	
2 391.35	
3 391.37	
4 390.80	
5 391.29	
6 390.57	

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

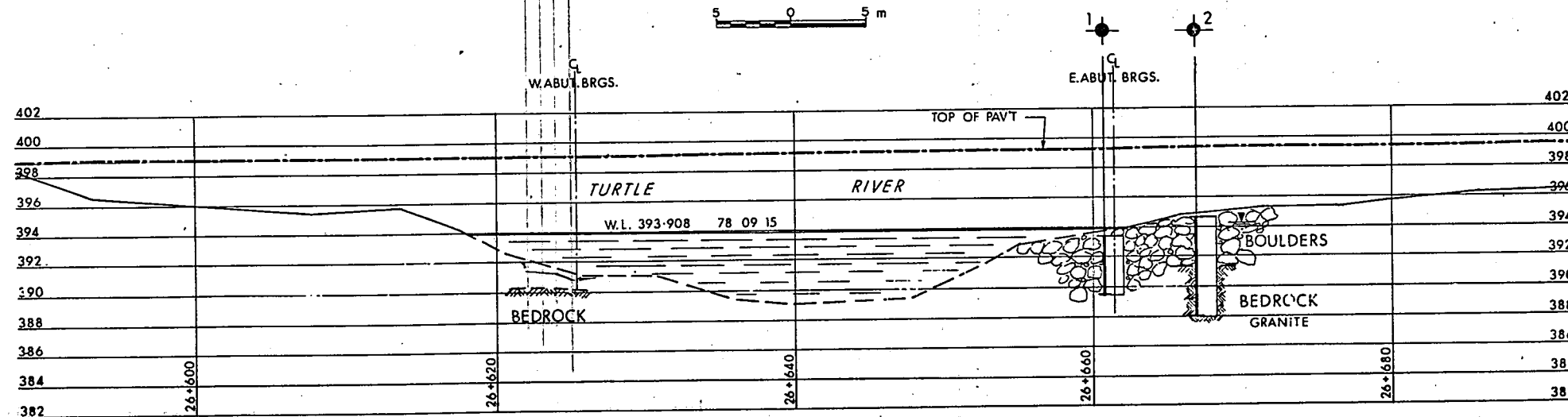
REVISIONS	DATE	BY	DESCRIPTION

Geocres No 52F-19	
HWY No BENDING LAKE RD	DIST 19
SUBM'D T. K. CHECKED	DATE 82 02 15 SITE 41-121
DRAWN	CHECKED APPROVED DWG 2



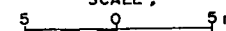
PLAN

SCALE



PROFILE LINE 'B'

SCALE



NOTE

The complete foundation investigation file for this project may be examined at the Engineering Materials Office, Downsview. Information contained in this file and any supplementary files is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 84-451



Ministry of
Transportation and
Communications

INDEX

<u>Page No.</u>	<u>Description</u>
1	Index
2	Abbreviations and Symbols
3-9	Foundation Investigation Report For: W.P. 18-77-09 Turtle River Crossing

NOTE: For purposes of the contract this report supersedes all other foundation reports prepared by or for the Ministry in connection with the above-mentioned project.

EXPLANATION OF TERMS USED IN REPORT

2

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 (RQD), 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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

For
Turtle River Structure
W.P. 18-77-09, Site 41-121
Bending Lake Road
District 19, Thunder Bay

INTRODUCTION:

This report summarizes the factual information obtained from a foundation investigation carried out by Robert Dodds Ltd., Consulting Geological and Geotechnical Engineers, Thunder Bay, Ontario. The investigation was completed on July 10, 1981 and consisted of 2 boreholes and six bedrock depth soundings. The boreholes were advanced by means of an AX core barrel to depths of 3.8 & 6.6 m, with bedrock being sampled in 1 borehole by obtaining 3.3 m of AX rock core. A geological mapping of all bedrock exposures was also completed.

SITE DESCRIPTION

The site of the proposed crossing is located approx. 66 air kilometers southwest of Ignace, Ontario, in the District of Thunder Bay. The crossing is situated in a straight section at a point approximately 4 km north of Pekagoming Lake.

The site is predominantly forested, with bedrock outcrops visible over much of the west river bank. Outcrops are also visible both upstream and downstream of the crossing on the east bank. Accumulation of boulders are prevalent on the east side of the river at the proposed crossing.

Topography of the area is low to moderate with local variations, probably controlled by subsurface bedrock configuration. The ground rises on both the east and west banks with apparent thin surficial cover.

SUBSURFACE CONDITIONS

General

Surficial materials in the area of the proposed crossing consist primarily of boulders intermixed with variable interstitial material. The exact nature of this interstitial material is difficult to ascertain due to the high concentration of boulders which made regular soil sampling techniques difficult. However, examination of terrain mapping by the Ontario

Ministry of Natural Resources indicates that the site location is situated in a glacial fluvial outwash plain at the toe of a ground moraine. This would be in agreement to the accumulations of boulders. The interstitial materials are expected to be predominantly sands and gravels.

Underlying the surficial material is granite bedrock.

The boundaries between the overburden and the bedrock are shown on the attached Record of Borehole Sheets. The elevations and locations of the boreholes are shown on Drawing No.2 along with an estimated stratigraphical profile based on borehole data.

The bedrock encountered is described in the following paragraphs.

Bedrock Geology

The mapping program delineated two major rock units. The first rock type consists of a granitic gneiss which is the dominant rock unit outcropping in the area. The second exposed unit is an intrusive synezoic granite which is observed cross-cutting the granitic gneiss on the west bank of the proposed crossing. In addition to the two rock units outcropping on surface, another unit was encountered within the drilling program. This unit can be classified as a biotite chlorite gneiss.

The rock units can be described as follows:

Type 1

Granitic Gneiss - This is the dominant exposed rock type at the proposed bridge site. It is exposed as rounded outcrop on both sides of the river. Outcrops appear to be exposed sections of rock ridges trending as marked on Drawing No.2. The unit shows varying degrees of alteration dependant on the proximity to the intrusive (Type 2).

The outcrop surrounding the sample location 5 (see Drawing No.2) was observed to contain structural features in the form of small scale folding and foliation as a result of the intrusion of Type 2, as well as the regional metamorphic event. These have little bearing on the proposed project, hence are mentioned in the interest of completeness but are not described in detail.

Type 1 can be described as fine to medium grained, varying in colour from greyish white to pinkish and displays some effects of alteration. A distinct lineation can be determined within the unit as well as well-developed to poorly-developed gneissic banding. Type 1 was cored over a short distance in Borehole No.2. A more detailed description follows in the later sections.

Franklin's rock quality classification would categorize this unit as a high rating with an estimated RQD of 75% to 90%.

Type 2

Syeno Granite Intrusive - There is limited surface exposure of Type 2, it is localized dominantly on the western bank and occurs generally as a flat-lying sill structure of variable thickness (maximum .5 m - minimum .1 m, see Drawing No.2). The intrusive is rich in potassium feldspar which accounts for the pink-orange tone of the unit. In some localized zones, the unit can be very coarse grained with well-developed potassium feldspar rhombhedrals. However, on the whole, the unit is classed as medium to coarse grained.

Type 2 was also encountered in coring Borehole No. 2. (See Borehole Logs appended at the end of this report). Franklins rock quality classification would categorize this unit as a high rating with an estimated RQD of 75% to 90%.

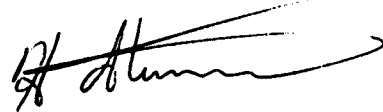
Type 3

Biotite Chlorite Gneiss - This unit was only encountered in the core from Borehole No. 2, a brief description is included here for completeness. The unit is medium grained, greenish-grey. Its colour results from concentration of biotite and chlorite.

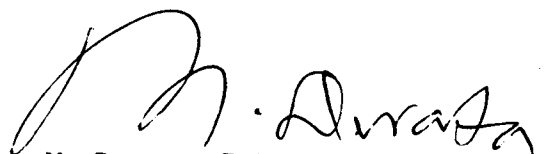
Sampled Bedrock

Bedrock was encountered in Borehole No. 2 at a depth of 3.3 m and was cored to a depth of 6.6 m. Rock Types 1-3 were encountered within the cored borehole. The condition of the core was fair, with a noticeable amount of

machine breaks and grinding. All three units were relatively free of fractures and no core was lost. Soundness of the bedrock was further indicated by good water return throughout the coring process. Drilling of Types 1 and 2 required heavy drill pressure. The extreme hardness of units 1 and 2 (high concentrations of quartz) resulted in a low diamond bit life. Type 3, on the other hand, required only moderate drill pressure and yielded better bit wear per foot drilled.



H. Sturm, P.Eng.
Project Foundation Engineer



M. Devata, P.Eng.
Chief Foundations Engineer
(East)

APPENDIX

RECORD OF BOREHOLE No 1

METRIC 8

W P 18.77.09

LOCATION 26+661 o/s 4.5 m RT

ORIGINATED BY Am

DIST 19 HWY Bending Road Alignment 'B'







BOREHOLE TYPE AXT Rock Core Cased

COMPILED BY Am

DATUM Geodetic

DATE July 9, 1981

CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	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						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
										WATER CONTENT (%)				
393.3														
393.2	Fibrous Organics					4	393.0							
	Zone of numerous boulders .3-.6 m dia. Determination of soil type between boulders was difficult as overburden had to be cored with an AXT corebarrel which did not allow regular sampling techniques.						392							
														
														
														
389.6							390							
389.5	Gravel .02-.04 m dia.													
3.8	END OF BOREHOLE													
							389							
							388							

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵ : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 2

METRIC 9

W P 18.77.09 LOCATION 26+667 o/s 4.5 m LT ORIGINATED BY Am
DIST 19 HWY Bending Road Alignment 'B' BOREHOLE TYPE AXT Rock Core Cased COMPILED BY Am
DATUM Geodetic DATE July 9 & 10, 1981 CHECKED

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	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																
								○ UNCONFINED + FIELD VANE					● QUICK TRIAXIAL x LAB VANE											
394.6																								
394.5	Fibrous Organics																							
0.1	Zone of numerous boulders .3 -.6 m in diameter. Determination of soil type between boulders was difficult as overburden had to be corebarrel which did not allow regular soil sampling techniques.																							
391.3																								
3.3	Altered granite contact zone, gray & white, medium grained		1	R.C. AXT	100%																			
391.0																								
3.6	Syeno granite intrusive. Pink, med. to coarse grained		2	R.C. AXT	100%																			
			3	R.C. AXT	100%																			
389.7			4	R.C. AXT	100%																			
4.9	Biotite chlorite gneiss, green with black gneissic banding fine to med. grained, locally intruded by syeno granite		5	R.C. AXT	100%																			
388.0																								
6.6	END OF BOREHOLE																							

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

memorandum



To: Mr. A. Radkowski,
Design Engineer
Structural Office

Date: 81 03 13

From: Pavement and Foundation Design Section

Re: Turtle River Bridge,
W.P. 18-77-09, Site 41-121
B.L.A.R., District 19, Thunder Bay

A review of the final structural plans for this structure leads to the following comment.

No foundation field investigation of this site was carried out due to the extreme difficulty of access and the fact that bedrock outcrops in the area of the crossing. The structure has accordingly been designed to be supported by spread footings on bedrock. A hand auger investigation of the area by the Northwestern Region Geotechnical Section indicates organic silt over bedrock or boulders in the area of the north abutment.

It is therefore proposed that when access to the site improves, a site visit should be made and the need for a full investigation re-evaluated.

It is understood that access to the site will be possible in the summer of 1981 following clearing of this right of way.

A handwritten signature in black ink, appearing to read "Peter J. Stuart".

Peter J. Stuart,
Foundation Engineer

for

M. Devata,
Senior Foundation Engineer

PJS:bcs

memorandum



To: Mr. P. Stuart
Foundation Engineer
Pavement & Foundation Design Section
Central Building, Downsview

Date: 1980 12 15

From: Geotechnical Section
Northwestern Region

RE: Work Project 18-77-09
Bending Lake Road - Turtle River Structure
Approximately 75.6 km north of Secondary Highway 622

During your last visit to Thunder Bay we discussed the above structure and what the soils borings indicated in the vicinity of the proposed structure.

Upon checking, I found that we had several borings on the south side of the river which indicate bedrock within 100 mm of the surface. In addition, the pedo sketch indicates a 2.5 m bedrock face along the south shore of the river.

On the north shore the pedo sketch indicates bedrock on the surface 20 m left of Station 26+700, approximately 52 m north of the north shore of the river. There are numerous boulders indicated along the north shoreline of the river. Borings indicate variable depths of overburden to bedrock and boulders. The following are boring logs near the shoreline on the north side of the river.

Station 26+660 C
0 - 510 Org M
510 NFP BR

Station 26+675 C
0 - 1.10 Org M & Blds
1.10 NFP BR

Station 26+690 C
0 - 150 Org M
150 NFP BR

Station 26+675 12.0 Lt C
0 - 200 Org M
200 NFP BR

Station 26+675 12.0 Rt C
0 - 200 Org M
200 NFP BR

From these borings it would appear that bedrock is near the surface in the entire area. I would suggest that when the locations of proposed footings are known, a field trip would probably preclude the need for a drill rig at this site. Please bear in mind that these borings are with a 50 mm diameter auger and that they could be on boulders rather than bedrock.

If there are any queries, please contact the undersigned.

H. Munford

H. Munford
Senior Soils Supervisor
FOR:
J. R. Girard
Head, Geotechnical Section

HM./blr



memorandum



To: M. Devata
Supervising Engineer
Pav't. and Foundation
Design Section
Engineering Materials Office
Room 315, Central Bldg.
Downsview.

Date: 79 08 20

FROM: Structural Section
Northwestern Region

Re: Turtle River Crossing - Site
#41-121, Bending Lake Road,
W.P. 18-77-09 - District 19
Thunder Bay.

Further to your memo 79 08 16, please be advised that a foundation investigation at the above noted site will not be required.

Exposed rock is evident on both sides of the river, some erratic layers of gravelly silt were encountered about 100' from the shore of the river. These layers were approximately 2' thick.
In order to give you a better idea as to the conditions, I am enclosing some photos taken during our trip.

If you require any further information kindly contact me.


W. Kulmatickas
Head, Structural Section

WK/eh
Enc.

c.c. B.T. Darch
G. Wrong

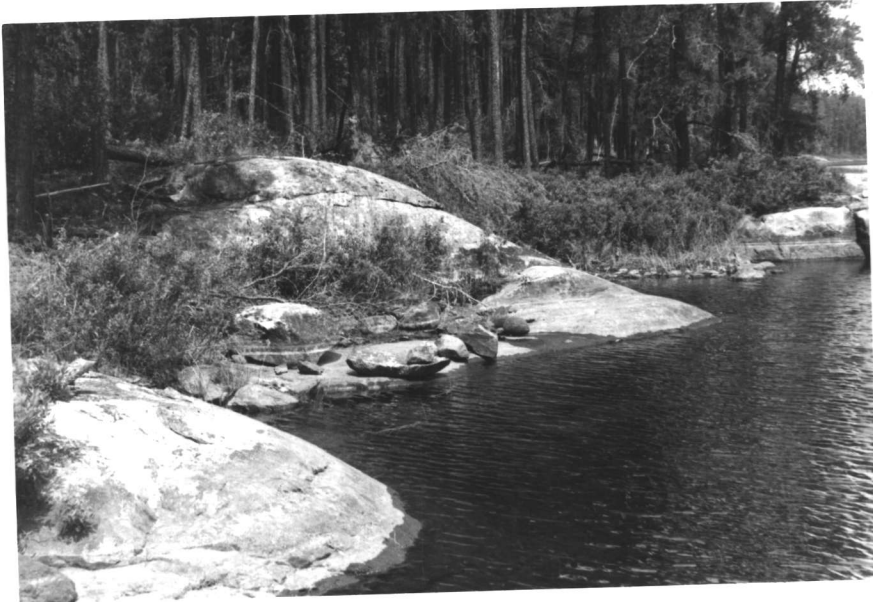
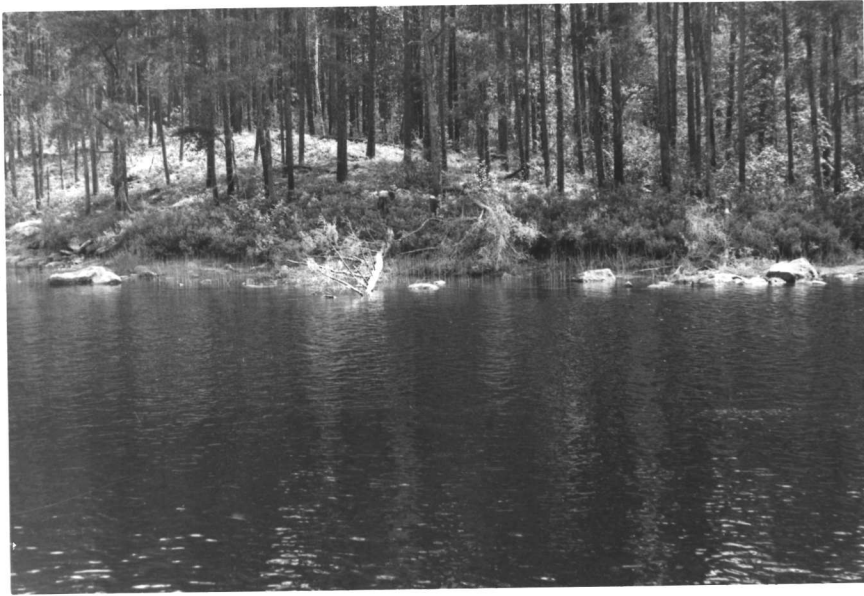
Copy to:
LEON SOSULA
FOR A. RADKOWSKI
N-INT SECTION
STRUCT. OFFICE
Nov. 20/79



W.P. 18-77-09

Little River Xing

1 of 3



W.P. 18-77-09
Turtle River Xing,
2 of 3



Turtle River Xing

WP. 18-77-09

3 of 3





Ministry of
Natural
Resources

Your file:

18-77-04.

Our file:

Regional Office
435 James Street South
P.O. Box 5000
Thunder Bay, Ontario
P7C 5G6

February 28, 1979

Mr. H. Munford
Senior Soils Supervisor
Geotechnical Section
Ministry of Transportation
and Communications
P.O. Box 1177
Thunder Bay, Ontario
P7C 4X9

Dear Mr. Munford:

Re: Proposed Bridge Over the Little Turtle River,
Bending Lake Road, District of Rainy River

Last fall we corresponded about a soils investigation at the MNR dam controlling Turtle Lake. In January of this year we retained the services of Robert Dodds Ltd. to undertake the soils investigation for the dam. Attached, for your information, is a copy of their report. Obviously we will require no further soils investigation at this site. Thanking you for your cooperation.

Yours truly,

R.B. Adamson
Regional Engineer
North Central Region

RBA:keg

Enc



	TAKE APPROPRIATE ACTION
SOILS	INVESTIGATE AND REPORT
	PLEASE ANSWER
	RETURN WITH COMMENTS
AGG.	DISCUSS WITH ME
	KEEP ME INFORMED
	NOTE AND RETURN
LAB.	NOTE AND FILE

*Should forward a
copy to Structural
Planning Dept.
COP*

See I copy to Structural

ROBERT DODDS Limited
CONSULTING GEOTECHNICAL ENGINEERS

GEOTECHNICAL INVESTIGATION
TURTLE LAKE DAM
Atikokan, Ontario

Our Ref. No. R79101
February, 1979

Prepared For:

Ministry of Natural Resources
Regional Office
435 James Street
P. O. Box 5000
Thunder Bay, Ontario

DISTRIBUTION: 3 copies- Ministry of Natural Resources
 1 copy - Robert Dodds Limited

/ed

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GRAIN SIZE DISTRIBUTION	4

1.0 INTRODUCTION

Robert Dodds Limited, Thunder Bay, was commissioned by the Ontario Ministry of Natural Resources to carry out a geotechnical investigation at the site of the Turtle Lake Dam near Atikokan, Ontario.

The purpose of the investigation was to determine the nature and extent of the subsurface material for preliminary design or feasibility assessment of a proposed replacement dam structure.

2.0 SITE DESCRIPTION

Turtle Lake Dam is located approximately 30 kilometers (48 miles) northwest of Atikokan, Ontario, along a single lane, gravel topped road known as the Clearwater West Lake Access Road. During the time of our field investigation, snow up to a depth of 1 m. (3 ft.) covered the majority of the site making an overall assessment difficult. Both the east and west flanks of the lake and river comprise hills of morainic material as is evidenced in exposures across the site and by large boulders up to diameters of 3.1 m. (10 ft.). Turtle River which flows south from the existing dam structure is also filled with very large boulders.

2.0 SITE DESCRIPTION CONT'D

The site is sparsely vegetated with predominantly small spruce, pine, alder, poplar, and birch trees.

The existing dam structure is of the rock and gravel- filled, timber crib design utilizing two sluice gates. During our field visit only the western gate was operating. Sheet- steel braced concrete walls for gravel fill retention were also observed at the sluice entrances.

Turtle River flows at a level about 1.5 m. (5 ft.) lower than its supply, Turtle Lake.

3.0 FIELD WORK

The field work was carried out between February 6th and February 14th, 1979. Originally, four borings were to have been advanced along the proposed dam centre line. Very difficult (ie. bouldery) ground conditions were encountered and subsequently only two boreholes were advanced. The ground conditions were judged to be consistent over the site based on embankment exposure and the inferred

3.0 FIELD WORK CONT'D

geological history of the area.

The boreholes were advanced by a Longyear 24 Drilling Unit utilizing washboring techniques and drilling of casing at the locations shown on Drawing No. 1. Borehole No. 1 was advanced to 12.89 (42.3 ft.) and Borehole No. 2 to 6.05 m. (19.8 ft.). Representative samples of soils and rock were obtained for identification and further testing. Standard penetration testing was performed in the granular material where possible to assess the relative density of the substrata.

In- situ permeability testing was performed at various depths within both boreholes. Results of the tests are shown on the Borehole Logs.

Borehole locations and collar elevations were established by Robert Dodds Limited field staff, and referenced to a local bench mark established on site.

4.0 SUBSURFACE CONDITIONS

Details of the subsurface conditions may be found in the Borehole logs appended as Drawings No. 2 and 3.

Generally, the subsurface strata comprise a clean, gravelly, well-graded sand with considerable cobble and boulder content. Borehole No. 1 proved 10.97 m. (36.1 ft.) of this surficial material overlying hard bedrock of metamorphic properties which has been intruded by a very hard granitic material. Borehole No. 2 intercepted a granitic boulder, 1.56 m. (5.1 ft.) in diameter at a depth of 0.76 m. (2.5 ft.). Granular material similar in nature to that encountered in Borehole No. 1 was found to a depth of 6.05 m. (19.8 ft.), at which point refusal to soil boring was encountered on a probable boulder.

The in- situ permeability tests show permeability values (k) of from $(7 \times 10^{-3} \text{ cm./sec.})$ to greater than $(9 \times 10^{-2} \text{ cm./sec.})$, indicating medium to high permeability of the subsurface material.

Stabilized groundwater levels were found to be 2.29 m. (7.5 ft.) and 2.32 m. (7.6 ft.) below ground surface in Borehole No. 1 and Borehole No. 2 respectively.

5.0 LABORATORY

Samples recovered from the drilling program were returned to our laboratory and re-examined for their textural classification. Four samples of the granular material were subjected to grain size analyses to correlate permeabilities determined in the field tests. Results of these tests are presented as Grading Curves appended as Drawing No. 4.

6.0 DISCUSSIONS

6.1 General

It is understood that the Ministry of Natural Resources intends to construct a replacement dam at the site investigated, for the purpose of flood control. At the time of writing this report, specific details of the structure are not known. The following discussions are, therefore, presented in general terms.

6.0 DISCUSSIONS CONT'D

6.2 Design Considerations

The structural treatment for the proposed dam is not known. However, it can be assumed that it will be one of either earth construction or concrete gravity wall type. Economics will likely dictate the method. The investigation has indicated that the subsurface stratum consists of a dense bouldery moraine deposit capable of supporting a dam and control structure.

One of the main concerns for this project may be the control of the seepage beneath the dam structure. Our investigation has revealed a subsurface material of relatively high permeability. The following subsections discuss seepage and possible seepage control measures.

6.2.1 Seepage

Laboratory testing carried out on selected samples has revealed that the granular material falls into the

6.0 DISCUSSIONS CONT'D

6.2.1 Seepage Cont'd

permeability range of $(10^{-3} \text{ cm./sec.})$ as shown on Drawing No. 4. This result is in compliance with permeability approximations based on calculations using the Hazen method and those determined in the in- situ field tests.

An average value of $k = (10^{-2} \text{ cm./sec.})$ may be used, and based on the following assumptions,

Hydraulic Head Retained	1.5 m. (5 ft.)
Crest Width	3.1 m. (10 ft.)
Dam Slope	2:1
Crest Length	46 m. (150 ft.)
Thickness of Previous Strata	8 m. (26 ft.)

an approximate seepage of $510 \text{ m}^3/\text{day}$ ($18,000 \text{ ft.}^3/\text{day}$) is calculated to take place beneath any relatively impervious dam structure.

6.0 DISCUSSIONS CONT'D

6.2.2 Seepage Control

Three common methods of reducing quantity of seepage can be considered for this project.

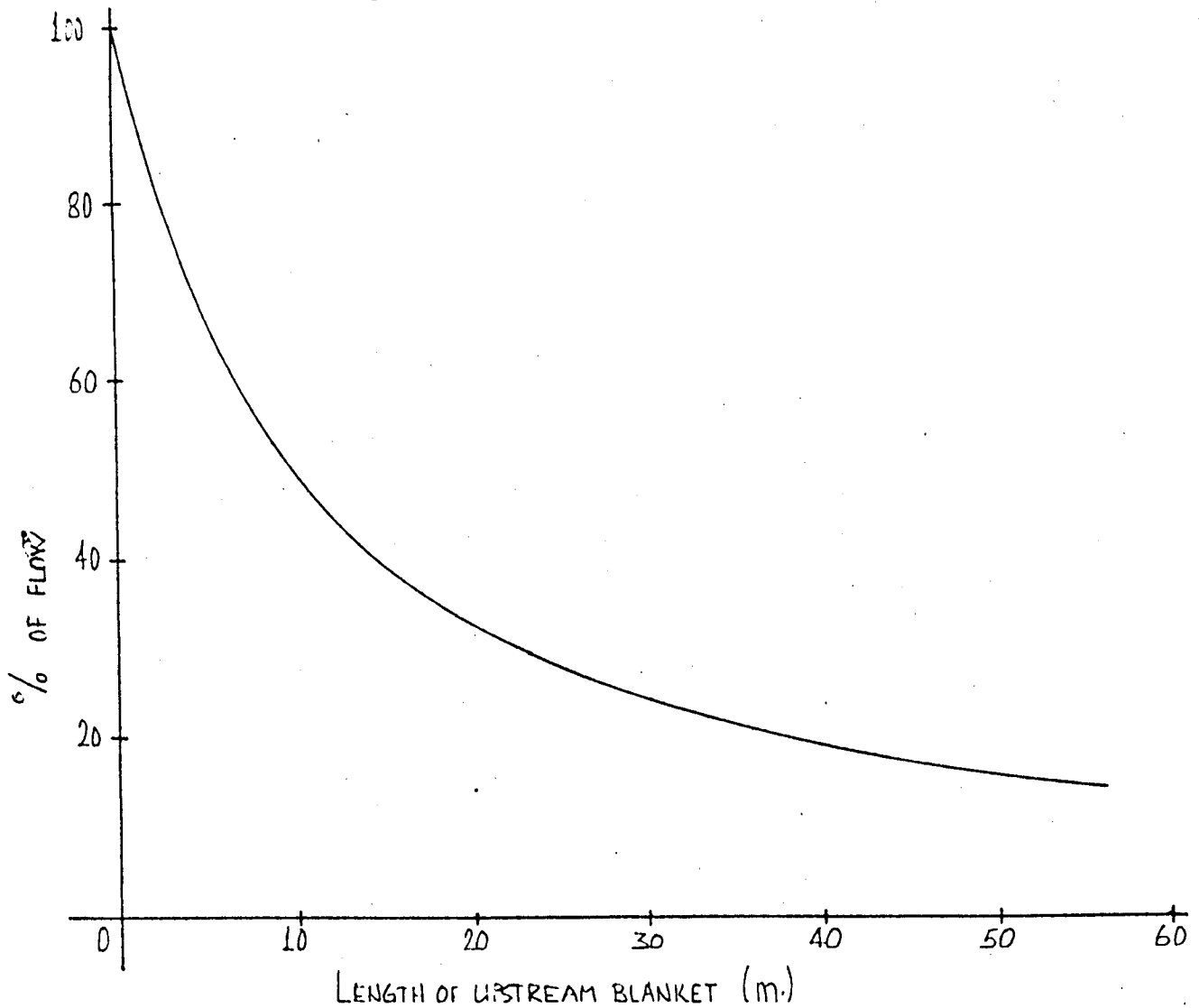
1. The permeability of the subsurface material can be decreased by the injection of grout mixtures to form what is termed a grout curtain.
2. The construction of a relatively impermeable cut- off may be undertaken to prevent the seepage of waters beneath the dam structure.
3. The construction of an impervious upstream blanket may be undertaken which essentially increases the length of the drainage path and thereby reduces the amount of seepage.

Grouting and cut- off construction are not considered feasible at this site. Installation of grout pipes or excavation into the bouldery material would be very difficult and costly. The only feasible method of reducing seepage would appear to be construction of an upstream blanket. The material in the blanket would have to be a clay or silt and some of it would have to be placed underwater.

6.0

DISCUSSIONS CONT'D6.2.2 Seepage Control Cont'd

For preliminary design purposes, an impervious upstream blanket construction of a minimum thickness of 0.61 m. (2 ft.) is calculated to reduce the seepage as shown below. The dam configuration discussed in Section 6.2.1 is assumed.

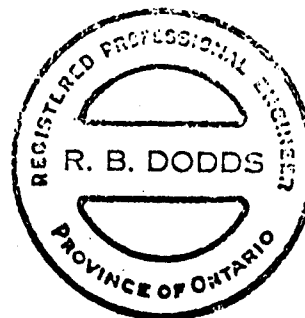


6.0 DISCUSSIONS CONT'D

6.3 Summary

The subsurface strata at the site appears to consist of dense bouldery sands and gravels capable of supporting a dam structure. This material is permeable and in order to achieve the required flow control it may be necessary to reduce seepage beneath the dam. The only feasible method of achieving this is with construction of an upstream blanket.

All the foregoing respectfully submitted,



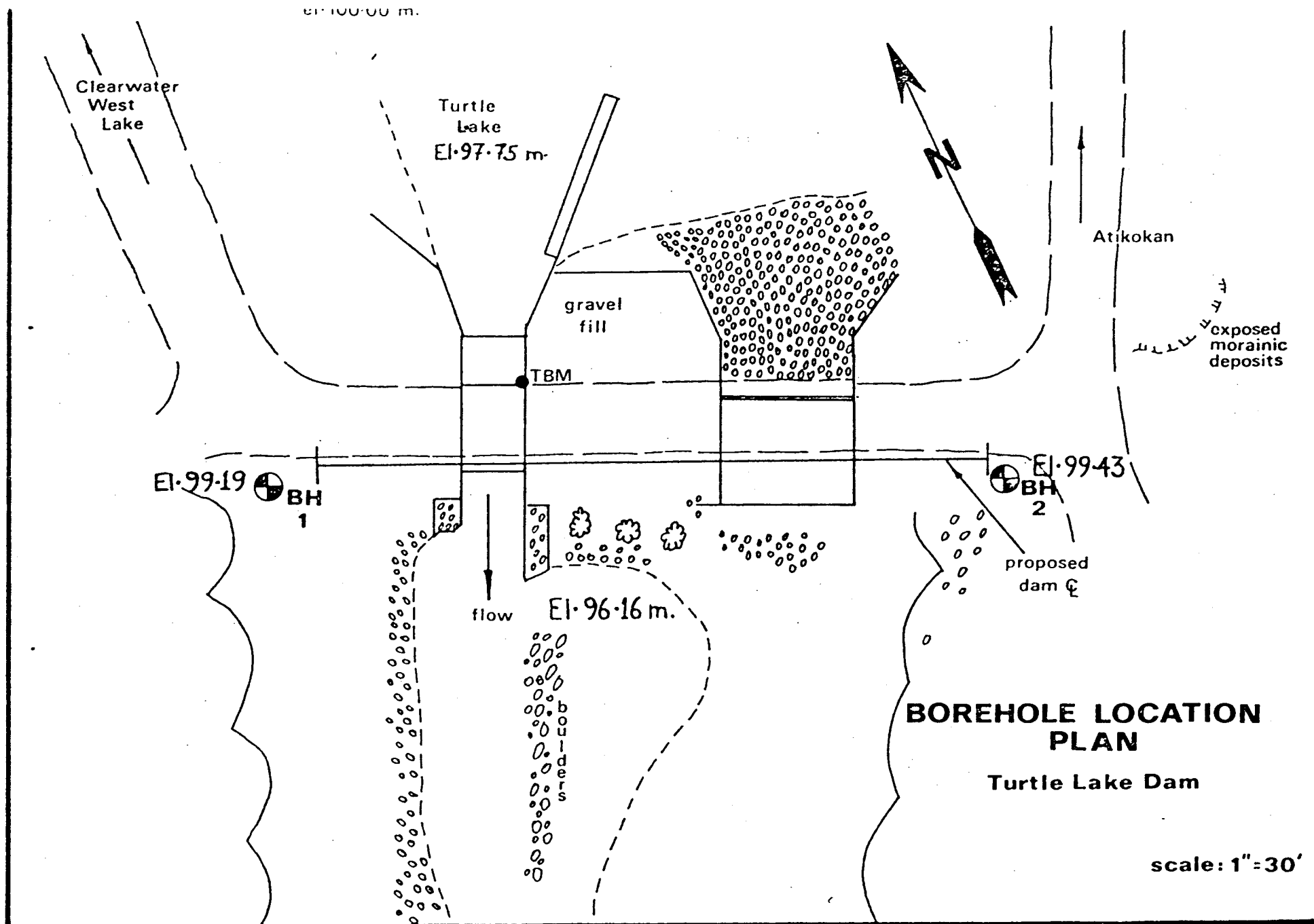
ROBERT DODDS LIMITED

A handwritten signature in cursive script, appearing to read "D. Georgiou".

DEMETRI N. GEORGIOU, B. Sc.

A handwritten signature in cursive script, appearing to read "Robert Dodds".

ROBERT B. DODDS, Ph. D., P. Eng.



DATA SHEET FOR BOREHOLE

DRAWING

(SHEET 1 OF 1)

Project No. R79101 (Your No. _____)
 Name TURTLE LAKE DAM
 Location ATIKOKAN, ONTARIO (48 km. NW)
 Date Location SEE DRAWING 1
 Date Drilled FEB. 1979 Hole _____
 Drilled by ROBERT DODDS LTD.
 Status LOCAL

Field
 2" O.D. Split Tube
 2" 3" I.D. Shelby Tube
 Rotary Core Sample
 Auger or Wash Sample
 2" Dia. Cone
 Field Vane
 Pressure Meter
 Sampler Pushed (pressure)
 Water Table (define apparent)

Laboratory
 Natural Moisture
 Plastic & Liquid Limit
 Lab Vane Test
 Torsion
 Unconfined Compression
 Undrained Triaxial at
 Overburden Pressure
 Strain at Failure
 Sensitivity

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Description Classification	Elevation meters (feet)	Depth meters (feet)	Penetration Resistance N 350 ft lbs blows/ft 20 40 60 80				Natural Water Content & Atterberg Limits (% dry weight)		Sample Type & Number	Unit weight Recovery %	
			Shear Strength				Sensitivity				
SAND - COMPACT TO DENSE, DRY BECOMING WET, GREY TO BROWN, USUALLY CLEAN, OCCASIONALLY TRACE SILT, FREQUENT GRAVEL AND COBBLE SIZES, OCCASIONAL BOULDER	99.19 325.4'										
	96.90 317.9'	2.29 7.5'							1	67%	
									2	67%	
									3	44%	
									4	44%	
									5	67%	
									6	72%	
ROCK - HARD, SOUND, GREEN-BLACK, BIOTITIC, SPARSE PYRITE MINERALIZATION, META- MORPHOSED BY VERY HARD, PINK GRANITIC INTRUSION	88.22 289.4'	10.97 36.0'							7	56%	
	86.30 283.1'	12.89 42.3'							8	89%	
	BOREHOLE TERMINATED								AWT CORE SIZE		

Notes

1. BOREHOLE ADVANCED BY LONGYEAR 24 DRILLING UNIT.
2. ELEVATIONS GIVEN ARE REFERENCED TO LOCAL BM ESTABLISHED ON SITE - DRAWING 1
3. PERMEABILITIES (k) SHOWN ABOVE DETERMINED IN-SITU

Borehole

1

DATA SHEET FOR BOREHOLE 2 DRAWING 3

(SHEET 1 OF 1)

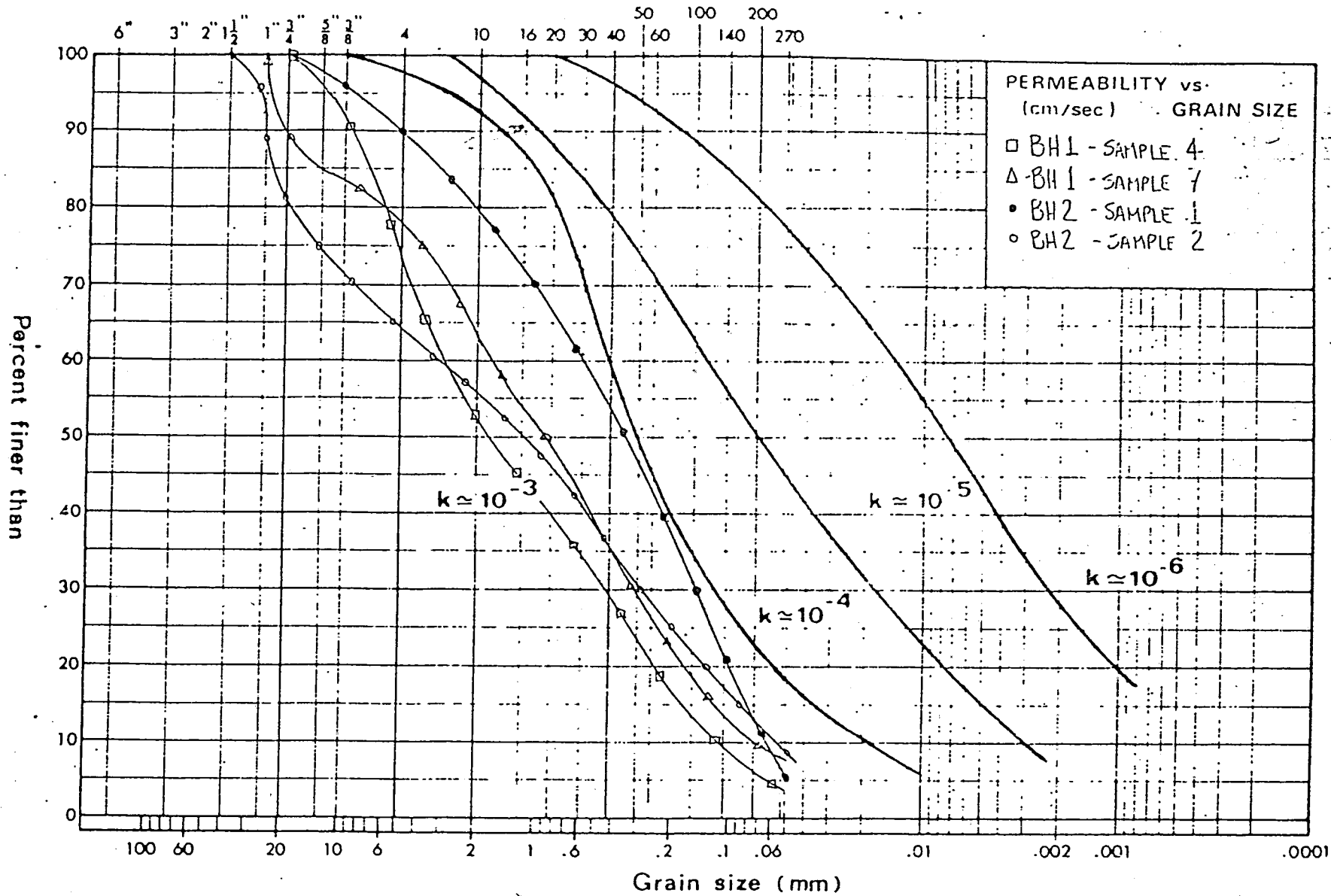
Well No. R79101 (Year No.)
 Location TURTLE LAKE DAM
ATIKOKAN, ONTARIO (48 km. NW)
 Location SEE DRAWING 1
 Drilled FEB. 1979 Hole
 Dr. by ROBERT DODDS LTD.
 Use LOCAL

Field
 2" OD Split Tube
 2" ID Shelby Tube
 Rotary Core Sample
 Auger or Wash Sample
 2" Dia Cone
 Field Vane
 Pressure Meter
 Sampler Pushed (pressure)
 Water Table (define, apparent)

Laboratory
 Natural Moisture
 Plastic & Liquid Limit
 Lab Vane Test
 Terzaghi
 Unconfined Compression
 Undrained Triaxial at
 Overburden Pressure
 Strain at Failure
 Sensitivity

Description Classification	Elevation metres (feet)	Depth metres (feet)	Penetration Resistance N 350 ft lb blows/ft				Natural Water Content & Atterberg Limits (% dry weight)				Sample Type & Number	Unit weight Recovery %
			20	40	60	80	Sensitivity					
SAND 99.43 326.2'	98.67	0.76										
BOULDER - VERY HARD, PINK, GRANITIC	323.7'	2.5'									1	97%
	97.11	2.32										
SAND - COMPACT TO DENSE, BROWN, CLEAN, BOULDERY	318.6'	7.6'										
	96.93	2.50										
	318.0'	8.2'									2	28%
											3	67%
	93.38	6.05										
	306.4'	19.8'	BOREHOLE TERMINATED REFUSAL ON PROBABLE BOULDER									

- Notes
1. BOREHOLE ADVANCED BY LONGYEAR 24 DRILLING UNIT.
 2. ELEVATIONS GIVEN ARE REFERENCED TO LOCAL BM ESTABLISHED ON SITE - DRAWING 1
 3. PERMEABILITY (k) SHOWN ABOVE DETERMINED IN-SITU.



COBBLE	coarse	medium	fine	coarse	medium	fine	SILT	CLAY
	GRAVEL			SAND			FINE GRAINED	

M. I. T. CLASSIFICATION

GRAIN SIZE DISTRIBUTION

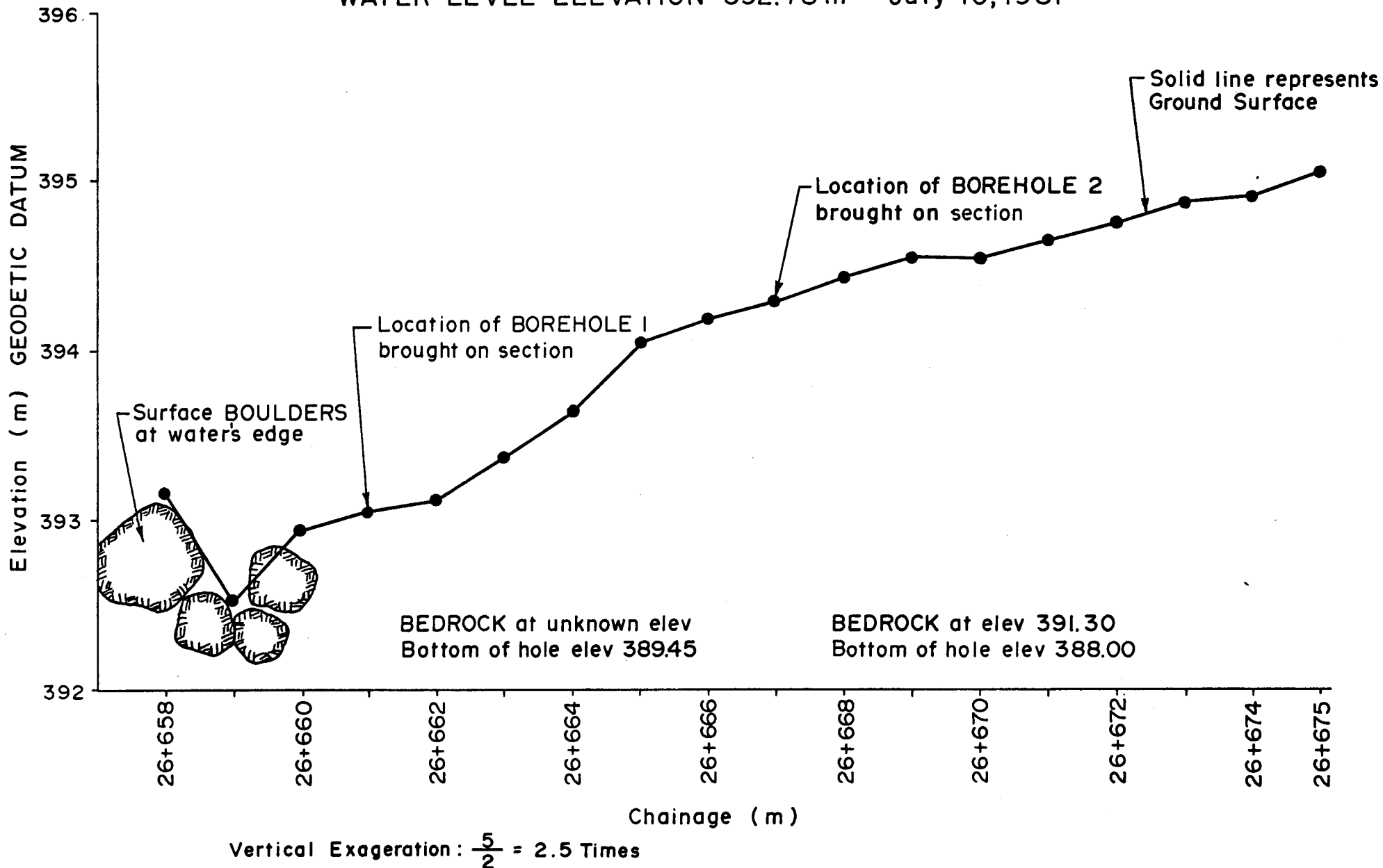
PROJECT: R79101
DWG. No. 4

SECTION ALONG BRIDGE ALIGNMENT: CHAINAGE 26+658 - 26+675

WATER LEVEL ELEVATION 392.78 m July 10, 1981

Ref. No. R81136

Drawing No. 2



memorandum



To: Mr. C.S. Grebski
Head, Operating Section
Structural Office

Date: 82 04 16

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

Re: Turtle River Bridge
41 miles S.W. of Ignace, Ontario
Bending Lake Road, Site #41-121
District #19, W.P. #18-77-09

We have reviewed the final bridge plan drawings for the above-mentioned structure site, and have no comments at this time.

A handwritten signature in dark ink, appearing to read "T. Kazmierowski".

T. Kazmierowski, P. Eng.
Foundations Engineer

TK/syc