55-F-228C

TRANS-CANADA HWY

MUSKOKA RIVER

B.A. 451

RACEY, MACCALLUM AND ASSOCIATES
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY
Consulting Engineers
AND ASSOCIATED STAFF

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REPORT NO. S-500-501/55/T-139-1

310 Odeon Building,
20 Carlton Street,
Toronto, Ontario.

19 September 1955.

55-F-228 C

The Department of Highways for Ontario,
c/o Lazarides, Lount and Partners,
79 Scollard Street,
Toronto, Ontario.

RE: FOUNDATION INVESTIGATION
MUSKOKA RIVER BRIDGE -
TRANS CANADA HIGHWAY.

Dear Sirs:

In accordance with your instructions, we have completed the foundation investigation at the above bridge site. The soil and rock samples have been studied, a number of soil samples were tested at the laboratory and we now wish to report on our findings as follows.

THE LOCATION OF THE SITE AND OF THE TEST HOLES

The site is located on the Trans-Canada Highway at the crossing of the Muskoka River, approximately four miles west of Bala, Muskoka. (See Enclosure No.1).

The location of the boreholes and of the twelve wash soundings to bedrock is shown on Enclosure No.2. In accordance with your proposals, marking off of the test holes was done by our engineer in the field, who also determined the elevations.

THE DRILLING WORK

The drilling equipment was moved from the highway to the site on August 23rd and the equipment subsequently set up on borehole No.1, which was completed on August 25th. Borehole No.2 was carried out from August 26th to August 29th and the twelve wash soundings were brought down from August 30th to September 1st. The equipment was subsequently moved out to the highway and returned to the warehouse.

The drilling equipment consisted of a diamond drill manufactured by CARL AYRE, four-inch drive-pipe, 2" and 3" thin-walled tube samplers and a standard 2" split barrel sampler. Diamond drilling was carried out with AXT diamond bits. The drive-pipe was driven with a 250 lbs. drive-hammer, dropped 30" and the split-tube sampler was driven with an energy equalling 4250 in.lbs.

Soil samples were obtained generally at 5 feet intervals, mainly taken with thin-walled tube samplers. Below a depth of approximately 18ft., sampling proved to be impossible, due to the occurrence of gravel in the sub-soil. Only one split-spoon sample was recovered in borehole No.1 from 20 to 21ft. depth, however, a trial to recover another sample was of no success. The samples from bedrock were obtained by core drilling with diamond bits.

The soil and rock samples will be stored on our premises for one-half year from now on, and shall be destroyed thereafter, if no instructions are received to the contrary.

DISCUSSION OF THE RESULTS

The soils profile, as revealed by the two boreholes, is described on the attached Engineering Data Sheets (Enclosures Nos.3 and 4).

Under a layer of approximately 4 ft. of sandy silt with lenses or layers of fine sand, the soil changes into a brown clayey sandy silt and siltclay in which, in borehole No.2, layers or lenses of fine sand and coarse silt were embedded. Between approximately 9 and 20 feet depth, both boreholes showed a grey, very soft clay, and siltclay which was stratified and, in borehole No.2, showed an unusual fissuration consisting of fine sandy seams. These may be fossil shrinkage cracks or may originate in faulting at a postglacial time, when the soils body was frozen. At a depth between 18 and 20 feet from the surface, the soil changed as found in sample No.5 of borehole No.1, into a gravelly coarse silt. However, due to the stratification of the soil and its geological origin, it is unlikely that the soil consists of coarse silt down to bedrock. Nevertheless, the gravelly character of the soil was found to continue down to bedrock.

The physical properties of the soils were checked in the laboratory and the clay, to a depth of approximately 9ft., proved to be in general of low plasticity and the underlying very soft grey clay of medium plasticity. The water contents in the clay were near the liquid limit and in some layers the water contents are far above the liquid limit. Also, the coarse silt at 20 ft.depth in borehole No.1 has a high water content. As the soils to a depth of 20 feet were very much similar to those previously investigated at the Moon River, where the sedimentary conditions were the same, only three unconfined compression tests were carried out on typical samples of the soil which showed, for the brown clayey, sandy silt and soft clay between 5 and 7 ft. depth in borehole No.1 an unconfined compressive strength of 0.36 tons per sq. ft. and a respective cohesion of 0.18 tons per sq.ft. A sample of the siltclay in borehole No.2 from 5.5 to 8 ft. depth, proved to be of higher strength, having an unconfined compressive strength of 0.76 tons per sq.ft. or a cohesion of 0.38 tons per sq.ft., these higher values obviously being caused by the inter-laid lenses or layers of fine sand or coarse silt. A characteristic sample of the very soft grey clay occurring between 9 and 20 feet depth, taken in borehole No.2 from 9.5 to 11.5 feet depth, showed an unconfined compressive strength of 0.22 tons per sq.ft. or a cohesion of 0.11 tons per sq.ft. It proved to be a very sensitive clay. These conditions compare very well with the values obtained from tests on samples of the deposits in the Moon River valley, where, to a depth of 7 feet, the unconfined compressive strength was up to 0.69 tons per sq.ft. and the corresponding cohesion 0.35 tons per sq.ft., and the underlying clay showed an unconfined compressive strength between 0.2 to 0.25 tons per sq.ft. or 0.1 to 0.12 tons per sq.ft. of cohesion.

The sample of gravelly coarse silt which underlies the very soft grey clay, showed a water content of 52% and the silt was in a loose state of compaction.

The depth of bedrock in borehole No.1 was 31ft. or elevation 615, in borehole No.2, 32 ft. or elevation 614. Soundings carried out along lines where the proposed piers would be located showed depths to bedrock which are entered on the sketch plan (Enclosure No.2). The surface of bedrock dips as shown on three sections on Enclosure No.2. Section A-A and Section B-B show that the dip of the surface of bedrock is in the opposite direction of the river flow. Section C-C follows the centre line of the proposed bridge and shows the generally increasing elevation of the surface of bedrock towards the south and the decreasing thickness of the river deposits from north to south. A third borehole was suggested on the south bank but was not carried out because sound bedrock, consisting of gneiss, is exposed.

Bedrock was found to be a grey solid gneiss, fine grained and rich in quartz, with pink streaks mainly consisting of feldspar.

CONCLUSIONS

The soil conditions at the site of the proposed Muskoka River Bridge are such that some stability considerations of the slope of the river deposits were considered necessary.

The south abutment and south pier of the proposed bridge will rest directly on or in bedrock, whereas the pier and abutment on the north side of the bridge will have to be supported by piles, which have to be anchored into bedrock to withstand any lateral pressure possibly exerted by the sloped river deposits on the northern embankment.

There is presently a slope at the north embankment of approximately 27 to 28 degrees towards the river and it was found that this slope in its natural state, even without any surcharge of approach fill, is unsafe. The critical height with the assumption of a presumably reasonable average of the physical properties of the soils would amount to up to 18 feet for the present slope, whereas the height is already 19 feet. A failure of the slope considering the gravelly lower 10 feet of the river deposits participating in a slide, would be a base failure tangent to the surface of bedrock. However, there is some probability that the lower ten feet of the gravelly soil of the valley deposits will offer sufficient shearing resistance to exclude the possibility of a base failure. The factor of safety against a base failure to develop cannot be determined accurately due to the lack of physical data of the soil properties of the lower 10 ft. Only one attempt was made to determine the factor of safety against a slope failure with slightly simplified assumptions and considering 8 ft. of granular fill to be superimposed on the surface of the existing ground level. As the factor of safety was found to be only 0.6 for this trial, it was not considered necessary to do further determinations of this nature since the instability of the slope was proved.

We feel, therefore, that it is not recommendable to begin any construction work on the bridge itself, before a stabilization of the approach has been reached. In the following, we wish to discuss some procedures to improve or change the present conditions at the site.

From the foregoing, dumping of the approach fill at once includes the highest probability of failure. But even dumping of fill in increments and determination of the degree of consolidation by measurement of pore water pressures in the soft grey clay, may result in failure due to the low stability of the natural embankment and the procedure would be quite time consuming. Further more, an excess surcharge would be necessary, which would have to be removed after consolidation. Another possibility to improve the conditions of the embankment would be blasting of explosive charges in trenches or boreholes, to cause a slide. A satisfactory removal of the soft soil layers however, may not be guaranteed. The displacement method applicable by blasting with a surcharge equal to the amount of soft soil to be replaced is not conceivable, since a slide would occur before the displacement fill has been dumped. Consolidation of the soft soil by sand drains or the Swedish cardboard drains, to be carried out without surcharge, would be at least very time consuming until sufficient consolidation for the dumping of approach fill would be reached.

We feel that a reduction of the angle of the slope towards the river, preferably to one in five or about 11 degrees, would stabilise the embankment with its surcharge of 8 feet of approach fill. This would necessitate however, that the northern abutment were to be moved about 40 ft. further north than presently proposed. We recommend to protect the slope with rip-rap as far as it is permanently or temporarily submerged, whereas the remaining part of the slope may be covered with sod, or seeded. For the side slopes of the approach fill, we recommend a slope of one in three.

We trust that the above information will be satisfactory. Please do not hesitate to get in touch with us, if any queries in this connection should arise.

Yours very truly,
RACEY, MacCALLUM AND ASSOCIATES LTD.

A. E. [signature]

for K. Tubbesing, P.Eng.

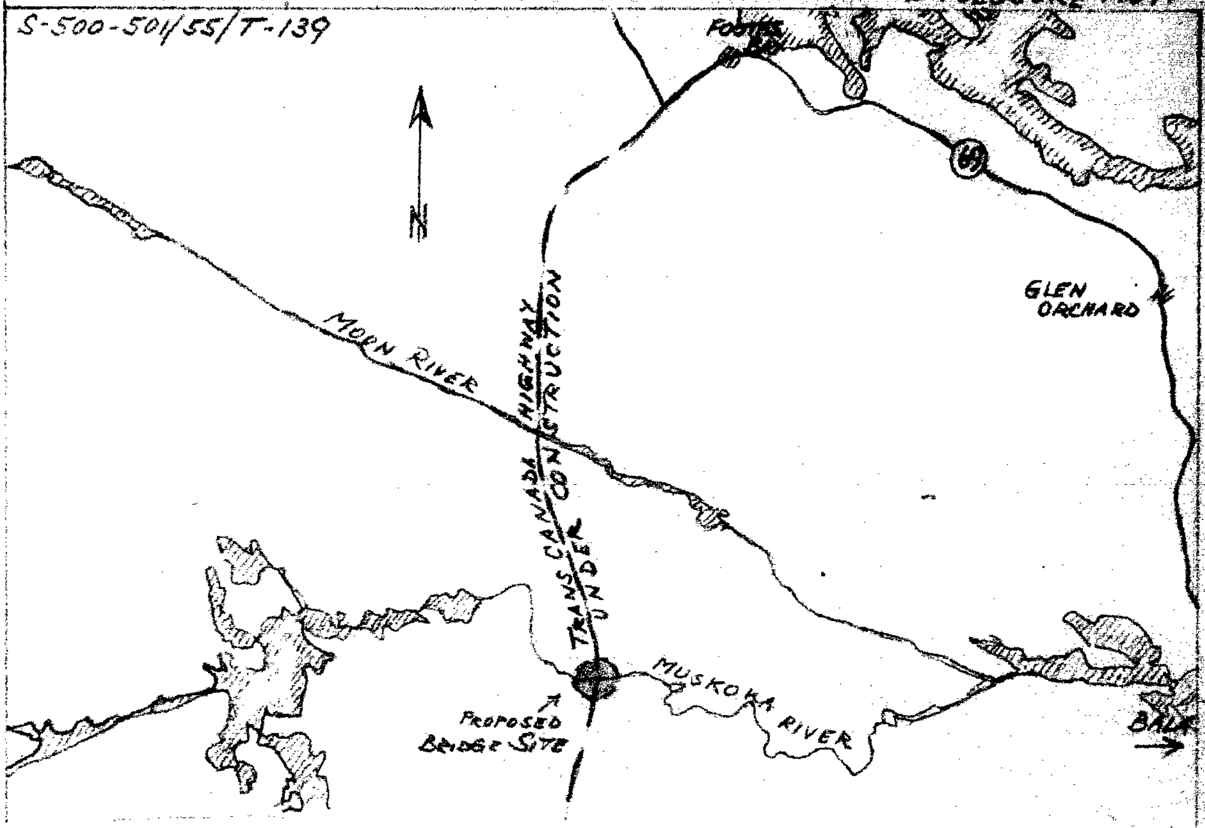
KT/AD

Original and 2 copies - Lazarides, Lount and Partners, Toronto.

copies 2 - Racey, MacCallum and Associates Ltd., Montreal, P.Q.
1 - Soils Engineer.

S-500-501/55/T-139

ENCLOSURE NO. 1

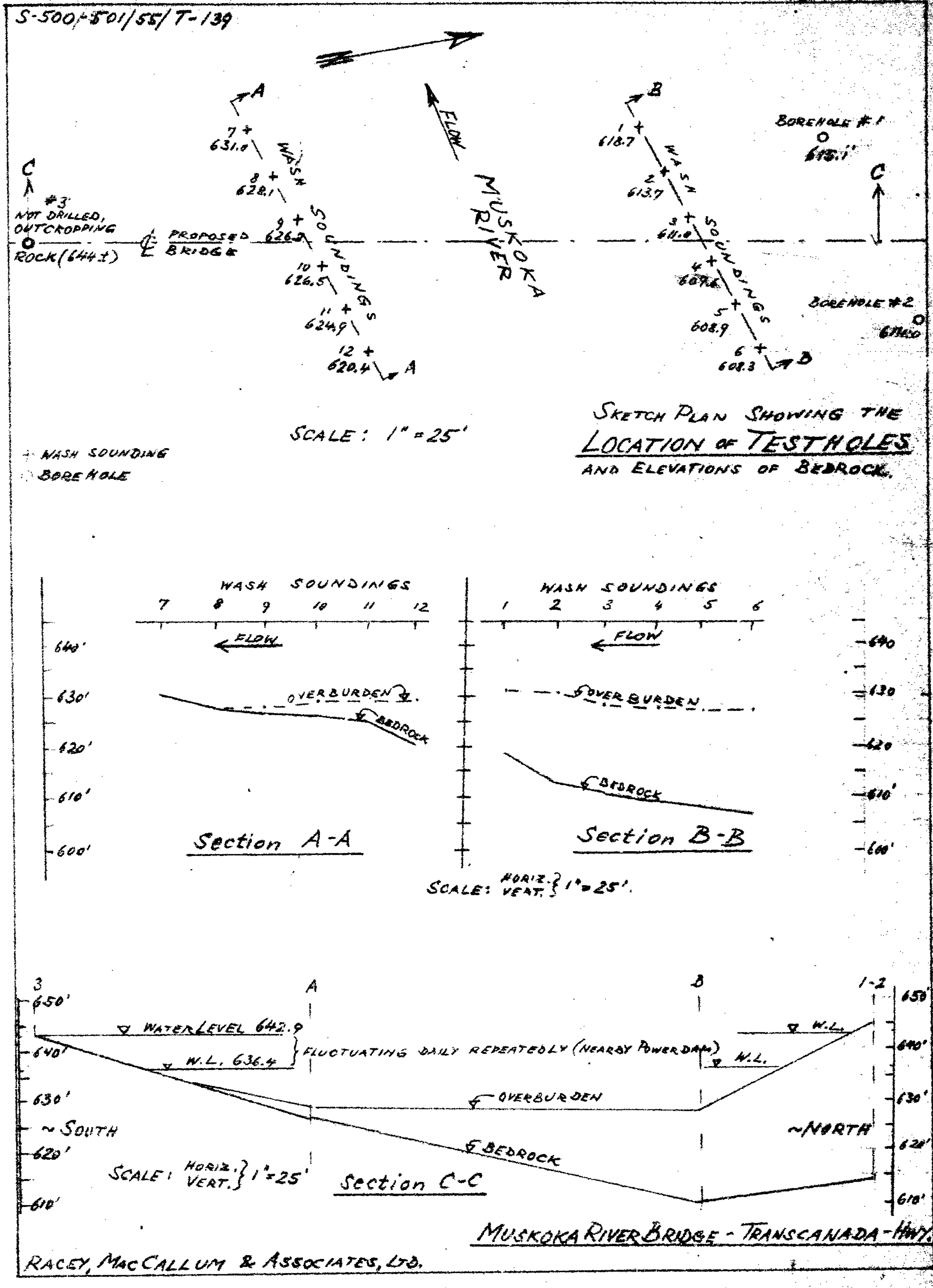


AFTER TOPOGRAP. MAP
SHEET MUSKOKA
SCALE 1" = 2 MILES

TOPOGRAPHICAL SKETCH
SHOWING THE
LOCATION OF THE
MUSKOKA RIVER
BRIDGE SITE
(TRANS CANADA HIGHWAY)

RACEY, MACCALLUM & ASSOCIATES, LTD.

ENCLOSURE No. 2



ENCLOSURE No. 3

Order No.: 5-500-501/55/F139 **RACEY, MacCALLUM AND ASSOCIATES** LIMITED

Foundation Engineering, Division

J. MARYKUCA — Driller

Hole Begun 23/8/55

S. SCOTT — Helper

Hole Ended 25/8/55 **Engineering Data Sheet for Borehole:** 1

Job Name: MUSKOKA RIVER BRIDGE – TRANSCANADA HWY.

B.F.W. — Checked by

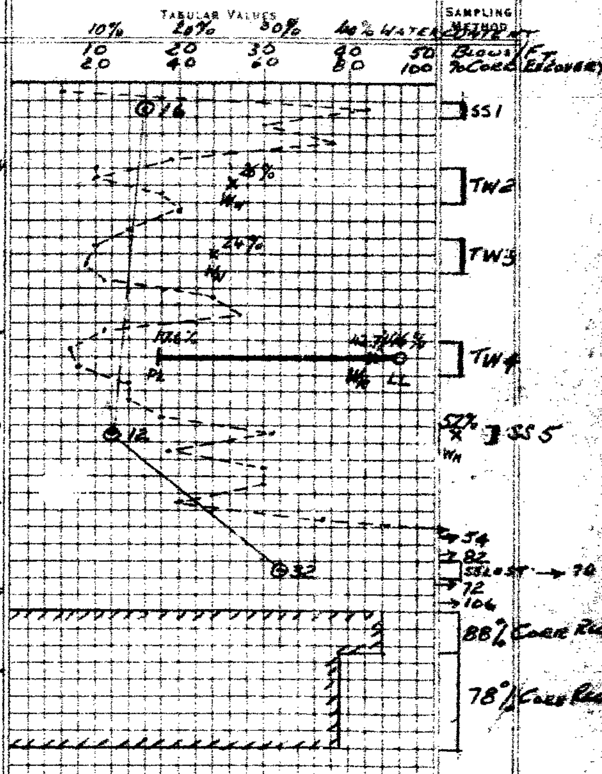
Job Located:

Hole Located: AS SHOWN ON ATTACHED SKETCH PLAN

Hole Elevation: 646' Datum: M.S.L.

29/8/55 — Day Month Year

DEPTH	EL.	THICK-NESS	SYMBOL	DESCRIPTION
0	646'			GROUND SURFACE — 0
				SANDY SILT with LENSES OR LAYERS OF FINE SAND
	(643.2	▽W.L. RIVER)		BROWN CLAYEY SANDY SILT AND SILTCLAY q_u = 0.36 t/sq.ft.
	(636.4	▽W.L. RIVER)		STRATIFIED, BROWNISH-GREY 10 SILTCLAY AND CLAY, SILTY, VERY SOFT, GREY NOT CALCAREOUS 15
				CHANGING TO 20 GRAVELLY COARSE SILT AND SILTCLAY(?) 25
31.0	615'			30 REFUSAL ON BEDROCK
				VERY SOLID GREY GNEISS, FINE GRAINED 35 WITH SOME STREAKS OF FELDSPAR
39.0	607'			END OF BOREHOLE 40



LEGEND:

- - - NUMBER OF BLOWS ON 4" DRIVE PIPE

⊙ " " " 2" SPLIT SPOON SAMPLER

x NATURAL WATER CONTENT (% OF DRY WT.)

PL —— LL ⊙ x W_N — PL PLASTIC LIMIT; LL LIQUID "; W_N NAT. WATER CONTENT

ENCLOSURE No.

Order No.: 5-500-50/55/E-139

RACEY, MacCALLUM AND ASSOCIATES LIMITED

Foundation Engineering Division

Hole Begun: 26/8/55

Hole Ended: 27/8/55

Engineering Data Sheet for Borehole: 2

J. MARYKUCA — Driller

S. SCOTT — Helper

B.F.W. — Checked by

Job Name: MUSKOKA RIVER BRIDGE – TRANSCANADA HWY.

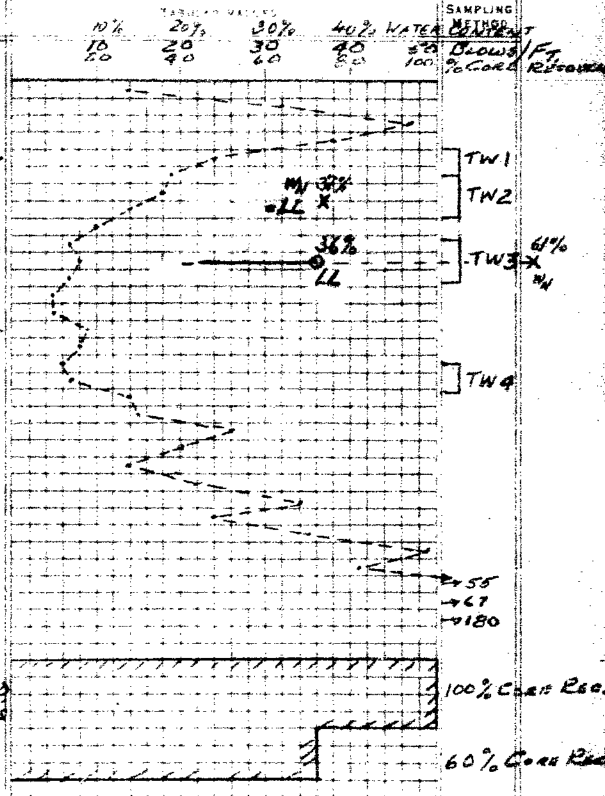
Job Located:

Hole Located: AS SHOWN ON ATTACHED SKETCH PLAN

Hole Elevation: 646.0 Datum: M.S.L.

29/8/55 (Day / Month / Year)

DEPTH	EL	THICKNESS	DESCRIPTION
0.0	646.0		GROUND SURFACE
			SANDY SILT with lenses or layers of fine sand
	643.2	HWL RIVER	
			Brownish SILT CLAY with layers or lenses of fine sand or coarse silt, qu = 0.76
	636.4	LWL RIVER	Very soft SILT CLAY, grey, fissured and with fine sandy seams, qu = 0.22 t/sq. ft.
			DITTO
~18'			CHANGE TO
			GRAVELLY SILT or SILT CLAY (?)
32.0	614.0		REFUSAL ON BEDROCK – DRILLED WITH ART BIT – LOST CORE
34.0			GREY, SOLID GNEISS, FINE GRAINED, RICH IN QUARTZ, WITH PINK STREAKS OF MAINLY FELDSPAR (UP ½" THICK).
41.0	605.0		END OF BOREHOLE



(FOR LEGEND SEE ENCLOSURE No. 3)

S-500-501/55/T-139

	W lbs	N lbs	T lbs	N tan ϕ
1	330	—	-100	—
2	3570	—	-400	—
3	8050	—	1,100	—
4	11960	—	3 400	—
5	16350	—	7,100	—
6	16260	—	10,000	—
7	9,400	—	4100	—
8	2,640	2 000	2 700	1,400

$\Sigma T = 27{,}900$ lbs.

$\Sigma N \tan\phi = 1400$ lbs

$L = 63'$ $c = 250$ psf

$$F.S. = \frac{\Sigma N \tan\phi + cL}{\Sigma T}$$

F.S. = 0.62

W = total weight of segment

T = tangential component of W

N = normal " " "

Scales: 1" = 10'
1" = 20,000 lbs.

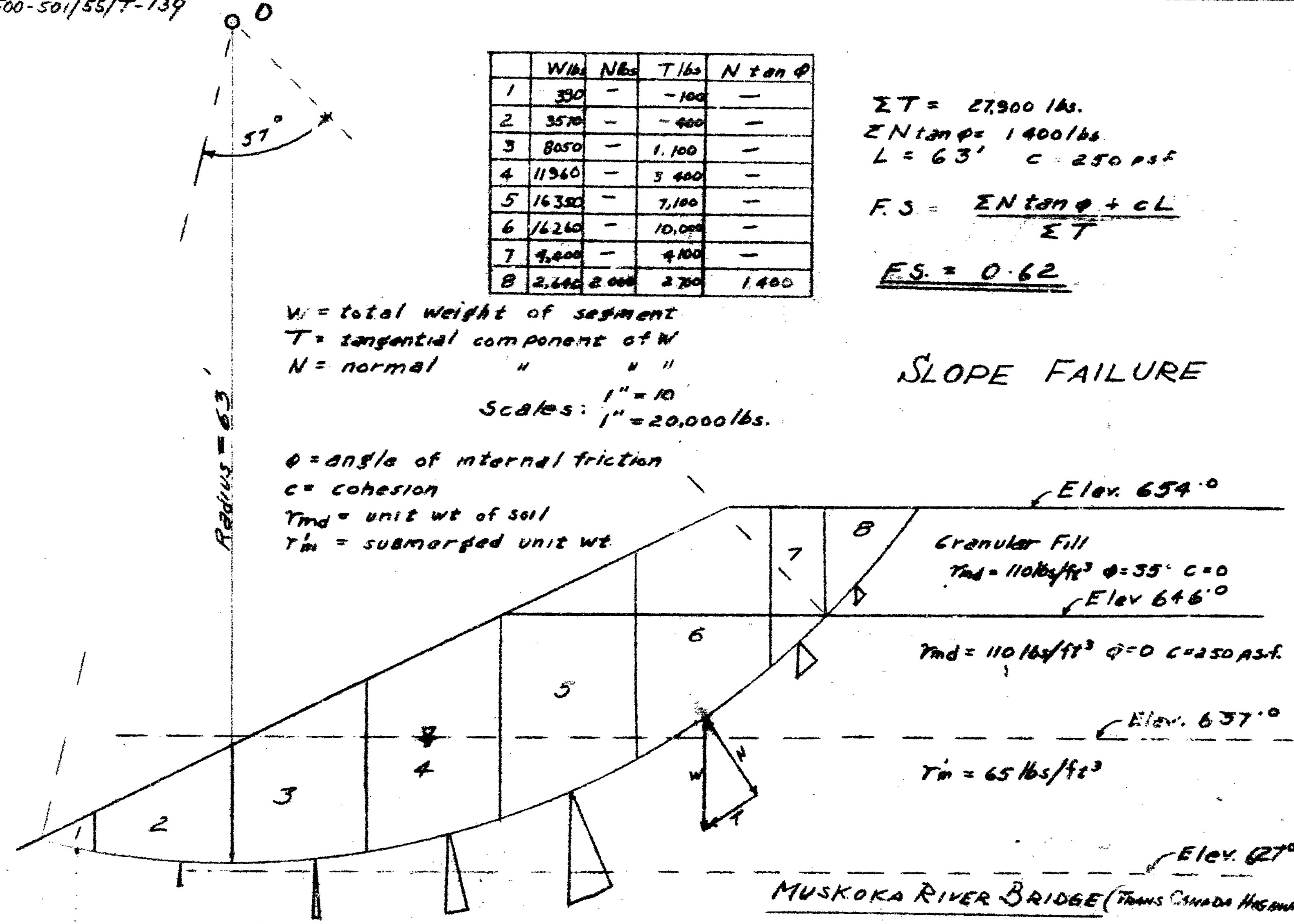
ϕ = angle of internal friction

c = cohesion

γ_{md} = unit wt of soil

γ'_{m} = submerged unit wt

SLOPE FAILURE



MUSKOKA RIVER BRIDGE (TRANS CANADA HIGHWAY)

RACEY, MACCALLUM & ASSOCIATES, LTD.

Sept. 9, 1955. B.F.W.

ENCLOSURE NO. 5