

#58-F-201C

Hwy #21

HEY ROCK CREEK

BRIDGE NEAR

GODERICH

BA 748

The Department of Highways of Ontario
280 Davenport Road,
Toronto - Ontario

RE. SUBSOIL INVESTIGATION FOR
THE PROPOSED HEYROCK CREEK BRIDGE
REPLACEMENT NEAR GODERICH, ONTARIO

58 F 201 C

Reference 105/F63

Dominion Soil Investigation Ltd.

4th June, 1958

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DOMINION SOIL INVESTIGATIONS LTD.

EMpire 6-6724

229 YONGE STREET — TORONTO 1, ONT.

Test Boring
Foundation Determination

Diamond Drilling
Soil Mechanics

June 4th, 1958

Report No. 105/F63

FOUNDATION INVESTIGATION FOR PROPOSED HEYROCK CREEK BRIDGE REPLACEMENT NEAR GODERICH, ONTARIO.

PURPOSE OF THE INVESTIGATION AND SCOPE OF THE REPORT

1. The investigation was undertaken to determine the foundation conditions at the above-mentioned site. The report covers the field work undertaken, the results of laboratory tests on selected samples, and recommendations on the allowable bearing capacity of the subsoil. Consideration is given to the probable lateral thrusts on the abutments and wing walls, together with a brief analysis of the geological development of Heyrock Creek and the consequent erosion protection required for the bridge.

LOCATION OF THE SITE AND BOREHOLES

2. Heyrock Creek Bridge is on Highway No. 21 approximately 10 miles North of Grand Bend. A sketch plan of the area illustrating the location of the site is shown on Enclosure No. 1.

The borehole locations are shown on the plan on Enclosure No. 2.

GEOLOGY AND HYDROLOGY OF THE AREA

3. The subsoil consists mainly of silty clay which has been subjected to
in
glacial loads and is/a stiff condition. The surface terrain is relatively flat except where cut by the numerous rivers and streams which flow into Lake Huron.

The rivers are very interesting since they appear to have the characteristic incised meanders of a rejuvenated river system. This rejuvenation is usually due to massive upheaval of the watershed area, which imparts added energy to the river. Some other reason must be found for the rejuvenation of Heyrock Creek.

About 150 years ago this area was very sparsely populated and probably almost completely covered with forests. This forest cover would slow the run-off into Heyrock Creek considerably. At that time the creek probably flowed slowly even in spring, and due to the relatively flat nature of the coastal strip meanders would be formed.

Since the first settlers arrived, almost all the forest cover has been removed, and since the subsoil is a relatively impervious clay, the run-off is very rapid. The consequent increase in the river's energy has resulted in the incised meanders.

It seems desirable to have some estimate of the rate at which the river is cutting into its bed. Based on the foregoing belief that 150 years ago the river was relatively quiet and meandered, it seems reasonable to assume that the bed was at that time of the order of 10 ft. below the surrounding countryside. At this time the river is 30 ft. below the surrounding countryside, which indicates a rate of erosion of 1 foot

every 7 years. This rate of erosion is confirmed by the destruction of the protective walls placed for the present bridge which was erected in 1924. It seems reasonable to assume that these walls were placed at least 3-4 ft. below the bottom of the river. These walls have been undermined on the upstream side and the bed at the centre of the creek is approximately $2\frac{1}{2}$ feet below the wall footings.

Some idea of the destructive power of the creek during spring floods is given by the picture at the end of this report and the fact that a piece of the protective wall about 2 ft. cube was found nearly a quarter mile downstream.

FIELD INVESTIGATION AND DESCRIPTION OF THE SUBSOIL

4. The field work was commenced on 21st May, 1958, and completed on 30th May. Four boreholes with associated cone penetration tests were carried out at the locations shown on Enclosure No. 2. It was not found to be possible to place all the boreholes at the points indicated on the Department plan, due to the presence of pieces of fallen protective wall or to the length of time that would have been required to construct an adequate drilling platform on the extremely steep slopes.

Drilling was carried out, using a conventional diamond drill equipped with an hydraulic head and adapted for soil sampling. Samples were obtained by means of the 2 in. O.D. standard split spoon and 2 in. I. D. thinwalled shelly tubes. A 2 in. diameter cone was driven beside each borehole. 'In situ' vane shear tests were carried out at depths intermediate between the normal 5 ft. sampling intervals. Penetration

of the standard split spoon and the 2 in. diameter cone was achieved by means of a 140 lb. hammer falling freely through a height of 30 ins. All the shelby tubes required pushing by means of the hydraulic head.

Borehole No. 1 indicated stiff to very stiff brownish silty clay containing sand pockets to a depth of 32 ft. or elevation 588 ft. where limestone boulders were encountered. Borehole No. 3, which was drilled from the bottom of the valley, indicated stiff brown clay containing some pea size gravel and sand pockets from elevation 602 ft. to 588 ft. where limestone boulders were encountered. Borehole 3 was diamond drilled through very stiff glacial till containing limestone boulders from elevation 588 ft. to elevation 577 ft.

Borehole No. 4 indicated stiff silty clay conditions from elevation 629 ft. to elevation 591 ft.

Borehole No. 2, however, was drilled through 10 ft. of loose silty sand containing some organics underlain by sandy silt containing some gravel to a depth of 15 ft. below ground surface.

At a depth of 16-17 ft. below ground surface, or elevation 614-615 ft., the stiff silty clay found in the other boreholes was encountered and was proved to elevation 583 ft. The silty sand and sandy silt found in the upper 15 ft. of the borehole is probably backfill behind the existing abutments.

RESULTS OF LABORATORY TESTS

5. The results of laboratory tests are shown on Enclosures 3 to 8. These tests indicate that the brown silty clay has a plasticity index aver-

aging approximately 10 and that the silty clay is in a stiff condition. The 'in situ' vane shear tests tend to give higher cohesive shear strengths than the unconfined compression tests. This is probably due to the presence of sand pockets since the soil is relatively insensitive.

DISCUSSION OF THE RESULTS

6. The silty clay below a depth of 5 ft. below the bottom of the river, or elevation 595 ft. has a cohesive shear strength in excess of 3,000 p.s.f. and has a relatively small plastic range. There does not, therefore, seem to be any problem concerning the availability of a suitable bearing strata. The main difficulties at the site are due to the very rapid spring runoff which has a considerable destructive force, the probable rate at which the river is cutting its bed, and the lateral stability of the abutments and wing walls.

Assuming that the economic life of the bridge is to be taken as 50 years, then at the present rate of erosion of the river bed the bottom of the river will be 5-7 ft. lower than at this time or at elevation 593-595 ft., and it therefore seems reasonable to place the bottom of the footings at elevation 590 ft. Assuming the worst conditions near the end of the economic life of the bridge, the footings should be assumed to be placed at the surface for shear failure considerations and an allowable bearing capacity of 3 t.s.f. should be assumed. At this intensity of loading the total settlement should not exceed 1 in.

The theoretical critical height of the clay through which Heyrock Creek is cutting its valley is in excess of 30 ft. However, the presence of the sand pockets and gravel in the silty clay makes

access of water into the exposed surfaces relatively easy. The presence of numerous surface slides in the area confirms this. The abutments and wing walls should, therefore, be designed on the assumption that lateral pressures will be exerted as time progresses owing to the inevitable deterioration in the properties of the clay on exposure. If a retaining wall is constructed with a clay backing, the pressure can be assessed on the assumption of active conditions only if its wall can yield continually without detriment and hence maintain the shearing stress. It is probable that the wingwalls, if designed so that they are not structurally integrated into the abutments, will be capable of considerable yielding. The forces on the walls are, therefore, likely to be intermediate between 'active' and 'at rest'.

A safety rule for active design in stiff clay is that the design value of p_a , the lateral pressure, should be not less than $0.25 p_v$ where p_v is the vertical pressure, or alternatively, not less than the horizontal pressure exerted by a fluid of density 30 p.s.f.¹. 'At rest' pressures should be assumed to be between 0.75 and $1.0 p_v$. A reasonable design value for the pressures on the yielding wing walls is, therefore, $0.50-0.60 p_v$ unless the Bridge Design Department has a policy of designing for full 'at rest' conditions for all clay backings.

Since the abutments will not be able to yield appreciably, they should be designed for 'at rest' conditions of $p_h = 0.75$ to $1.0 p_v$.

Reference:

1. F.D.C. Henry "The Design & Construction of Engineering Foundations," McGraw Hill, Toronto, Page 245.

Protection of the bridge from erosion behind the wing walls is essential, and will probably have to extend an appreciable distance up stream. The area in the region of Borehole No. 2 has been subjected to substantial erosion in the past few years, and a partially developed slide is indicated by the contour lines in this region. The protective walls of the existing bridge have been smashed completely on the north end of the upstream side of the bridge and a large piece of the south upstream section has also been ripped out. It appears that the river is changing its bed in a southerly direction and the main impact of the creek is directed at a point near borehole No. 2. It would seem advisable for the Hydrological department to see the site and pass an opinion on the probable direction and rate of future erosion.

It has been assumed that where possible the banks of the valley will be cut vertically and supported with temporary shoring, while the bridge is being constructed. Where this is done the temporary shoring may be designed to withstand active pressures, as mentioned previously, of 0.25 p_v. However, particularly on the south side of the bridge near borehole No. 2 where sand backfill was encountered, it will probably be necessary to place fresh backfill for the new bridge. Provision should be made to ensure that this backfill is adequately drained to prevent the possibility of high hydrostatic pressures developing behind the abutments.

CONCLUSIONS

7. 1. The estimated scour of the river over the next 50 years under normal conditions is 5-7 ft.
2. Foundations should be placed at elevation 590 ft. to allow for this future scour of the river bed.

3. An allowable bearing capacity of 3 t.s.f. may be assumed at elevation 590 ft., at which intensity of pressure the total settlement should not exceed 1 in.
4. Temporary shoring for the support of vertical clay faces may be designed from active pressure considerations and should be taken to be equal to the horizontal pressure exerted by a fluid with a unit weight of 30 p.s.f.
5. The abutments should be designed where the soil backing is clay, on the assumption that very little yield is likely and that the earth pressures will be those of the 'at rest' condition. Long term pressures should be assumed to be $0.75 - 1.0 p_v$.
6. Where the backfill is granular the abutment horizontal pressures may be assumed to be $0.5 p_v$.
7. The wing walls if not structurally integrated with its abutment may be designed for pressures intermediate between 'active' and 'at rest' if appreciable yield is allowable. Horizontal pressures should be assumed to be 0.5 to $0.6 p_v$, unless it is Department policy to design all clay backings for full 'at rest' conditions.
8. Adequate drainage from behind the abutments should be assured to prevent the development of hydrostatic pressures or softening of the natural clay.

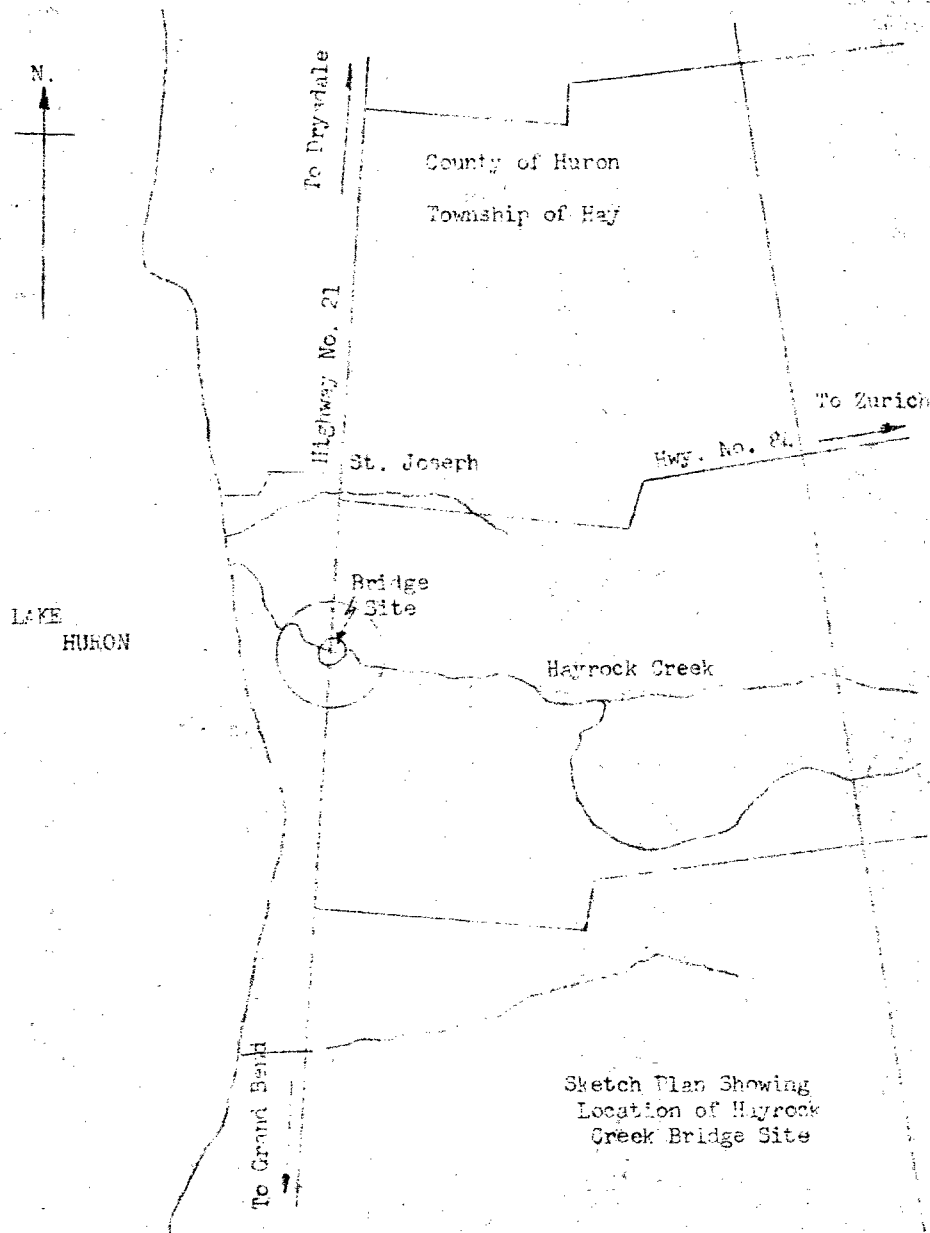
9. Protection against erosion behind the wing walls must be given, and it is suggested that the comments of the Hydrological Department be obtained concerning the probable rate at which the stream bed is moving in a southerly direction. Considerable attention should be given to the point of impact of the stream below borehole #2 where a minor slide is developing at this time.
10. The proposed life span of the bridge will probably have considerable bearing on the extent of the protective measures required.

Peter S. Martin Monk

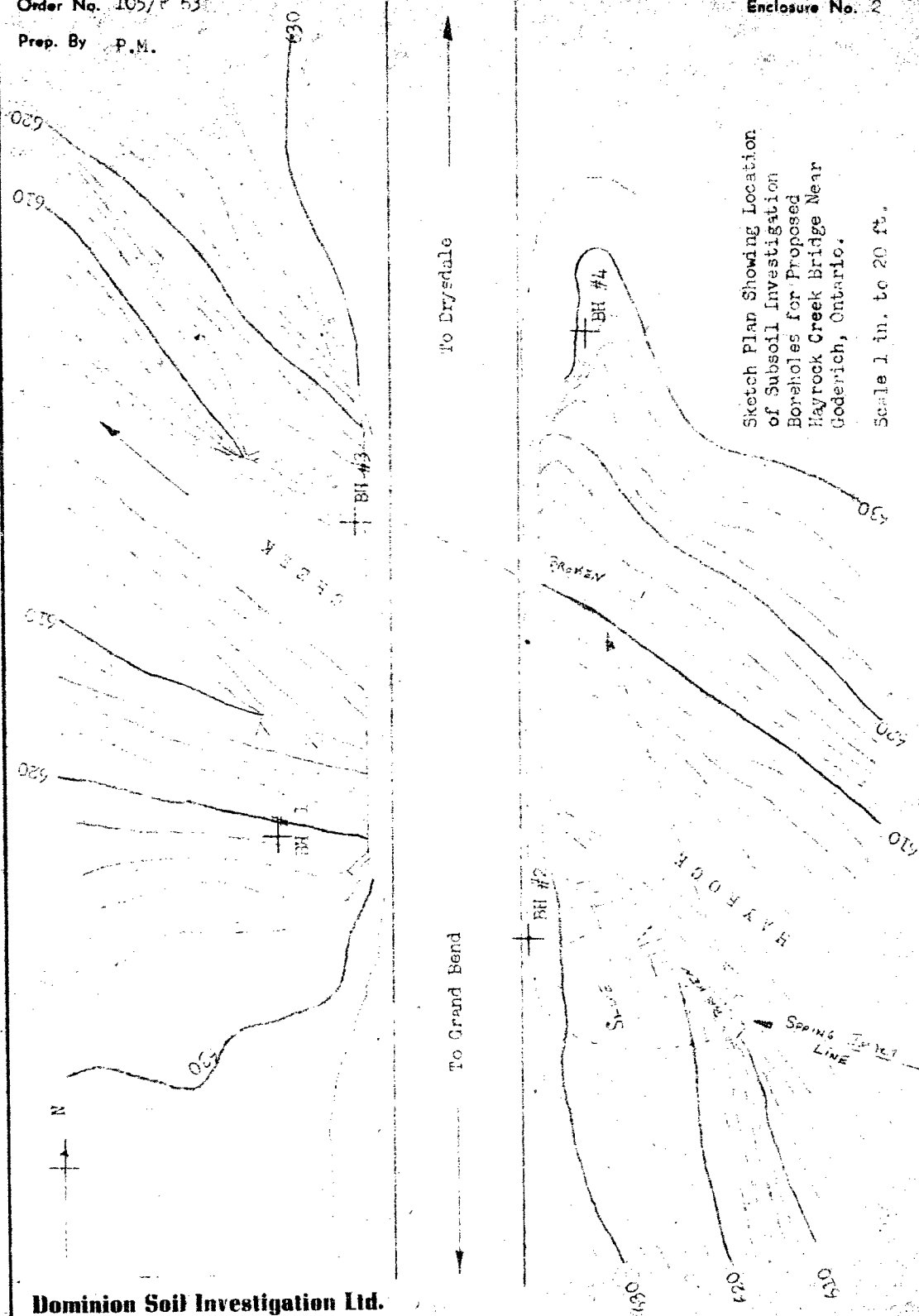
P.E.M. Monk, P. Eng.



Prep. By P.M.



Scale 1 in. to $\frac{1}{2}$ mile.



Sketch Plan Showing Location
of Subsoil Investigation
Boreholes for Proposed
Hayrock Creek Bridge Near
Godenrich, Ontario.

Scale 1 in. to 20 ft.

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: No. 1

Date: 27/5/58

Project: Proposed Bridge Replacement

Location: Heyrock Creek Nr. Goderich

Hole Location: See Enclosure No. 2

Hole Elevation and Datum: 619.9 Geo.

Field Supervisor: A.B. Prep.: P.M.

Driller: F.M.

Checked:

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

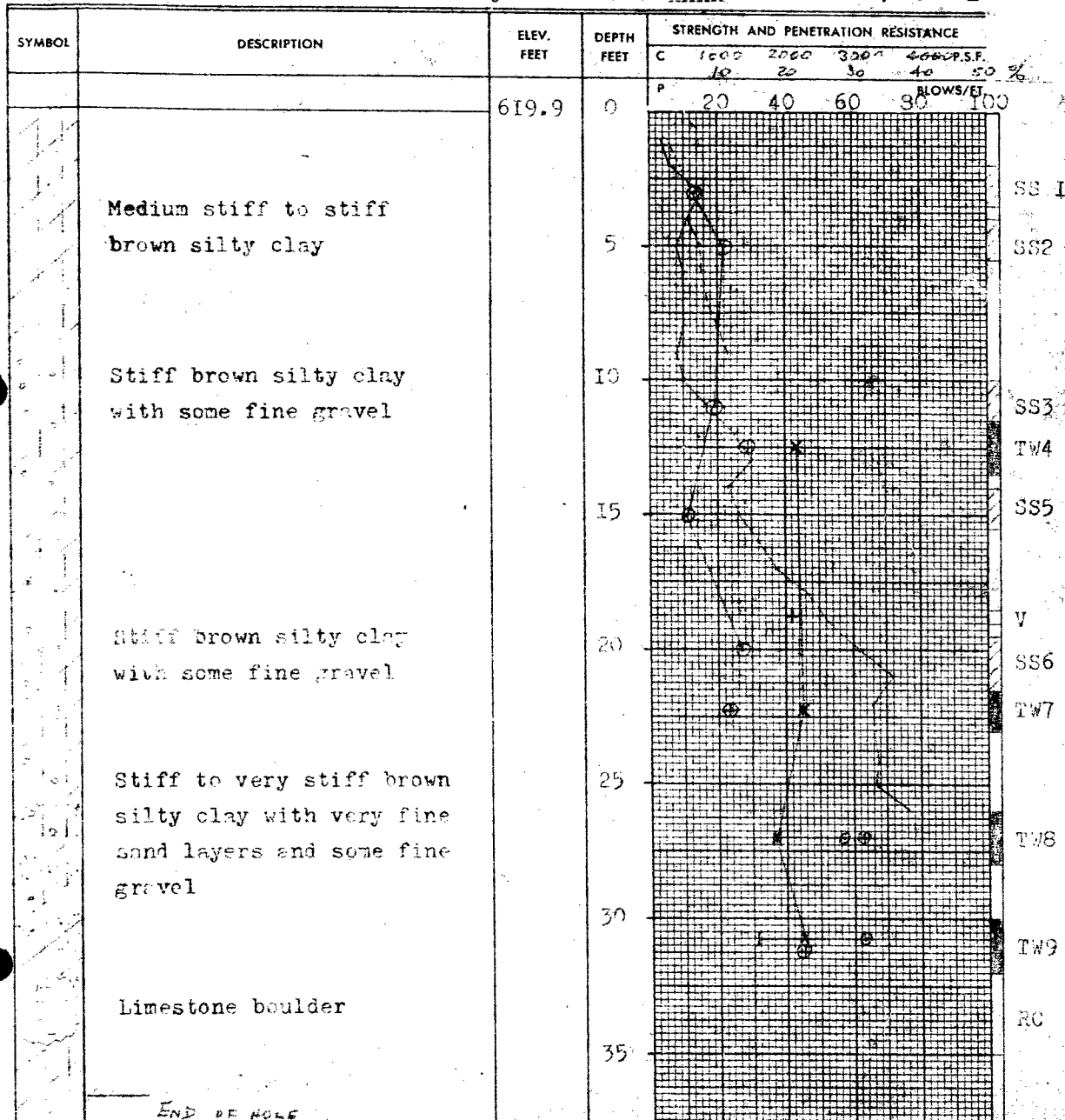
Casing

⊕
+³⊕
⊕
⊕

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: No. 2

Date: 3/6/58Project: Proposed Bridge ReplacementLocation: Hayrock Creek Nr. GoderichHole Location: See Enclosure No 2Hole Elevation and Datum: 631.2Field Supervisor: A.B. Prep.: H.M.Driller: C.S. Checked:**LEGEND**

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

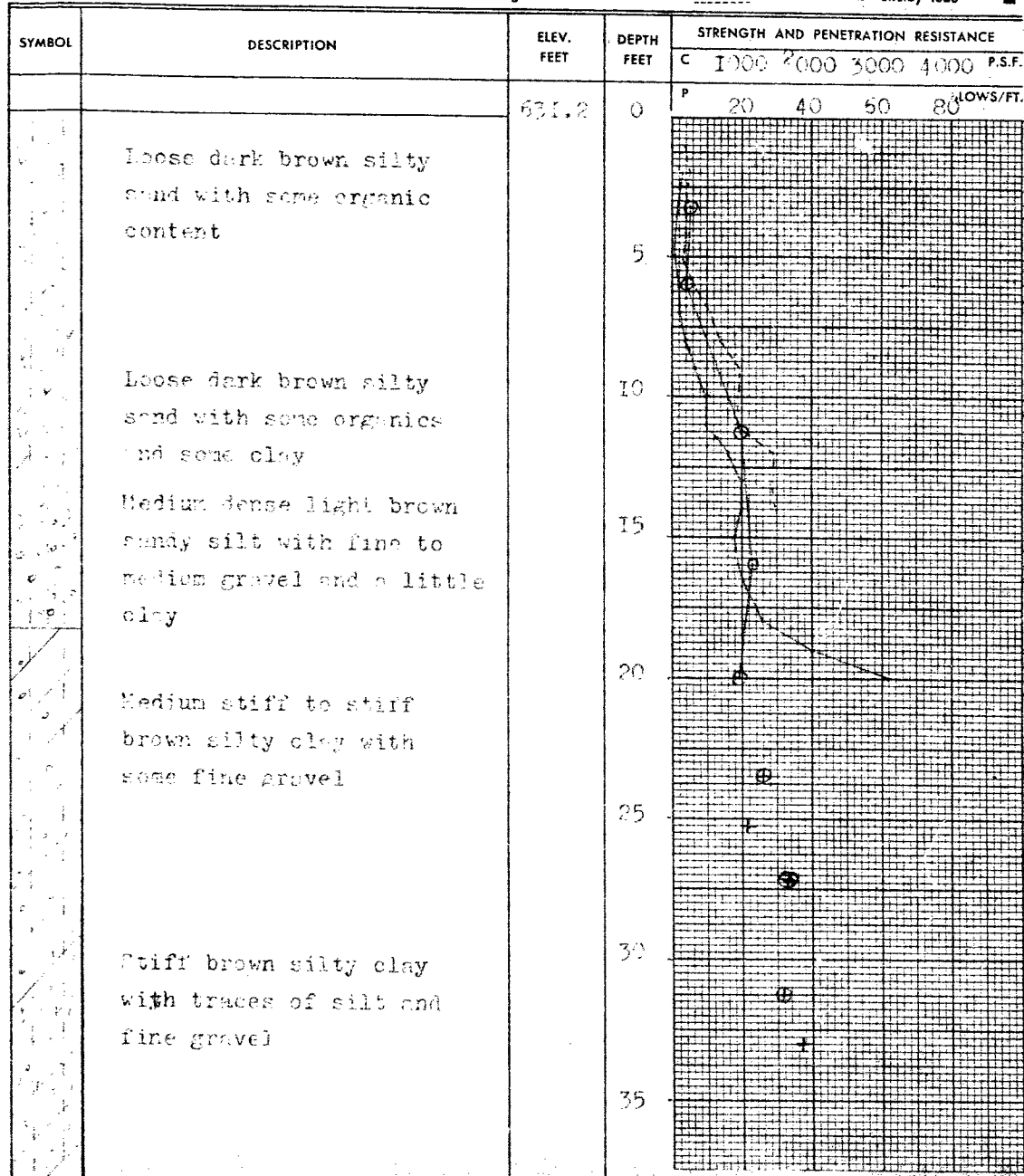
⊕
+³

⊕ ⊕

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: No. 2 Cont.

Date: 3/6/58

Project: Proposed Haycock Creek Bridge

Location: Near Loderich

Hole Location: See Enclosure No. 2

Hole Elevation and Datum: 631.2

Field Supervisor: A.B. Prep.: P.M.

Driller: C.S.

Checked:

LEGEND**Shear Strength (C)**Unconfined compression
Vane test and sensitivity (S)⊕
+s**Penetration Resistance (P)**

2" Split tube

2" Dia. Cone

Casing

⊕
⊕**Sampling Method**

2" Dia. split tube

⊕

2" Shelby tube

■

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE					BLOWS/FT.	
				C	1000	2000	3000	4000		
				P						
		596.2	35		20	40	60	80		TW 9 LOST
	Stiff silty clay with some sand									V
			40							TW 10 LOST
	Very stiff grey sandy silty clay with fine gravel									TW 11
			45							V
										SS I 2
	End of hole		50							V

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Engineering Data Sheet for Borehole: No. 3

Date: 27/5/58

Project: Proposed Bridge Replacement

Location: Heyrock Creek Nr. Goderich

Hole Location: See Enclosure No. 2

Hole Elevation and Datum: 602.0 Geo.

Field Supervisor: A.B. Prep.: P.M.

Driller: F.M. Checked:

LEGEND

Shear Strength (C)

Unconfined compression
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

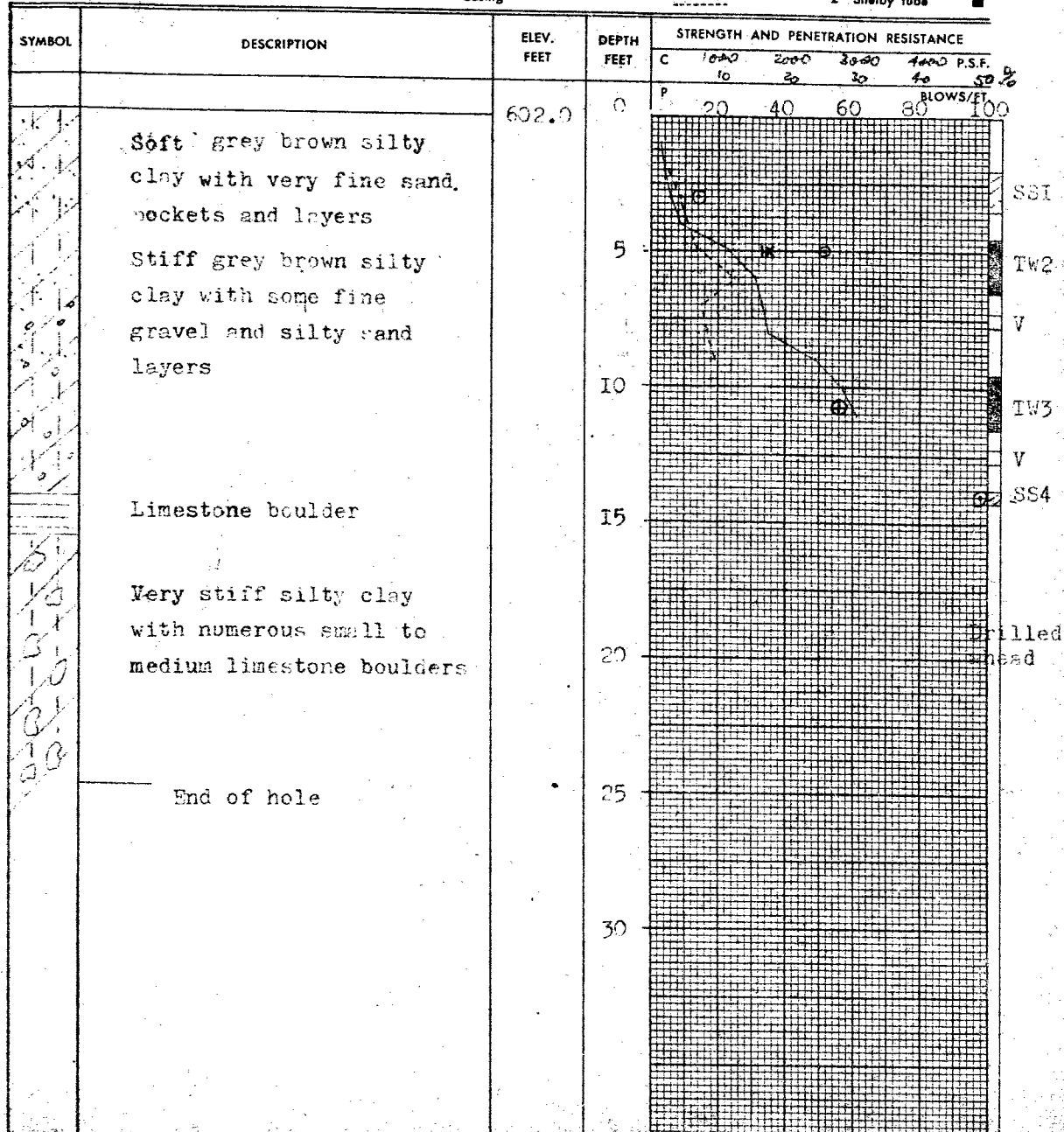
Casing



Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: No. 4

Date: 3/6/58

Project: Proposed Bridge Replacement LEGENDLocation: Heyrock Creek Nr. Goderich Shear Strength (C)

Hole Location: See Enclosure No. 2

Hole Elevation and Datum: 629 Geo.

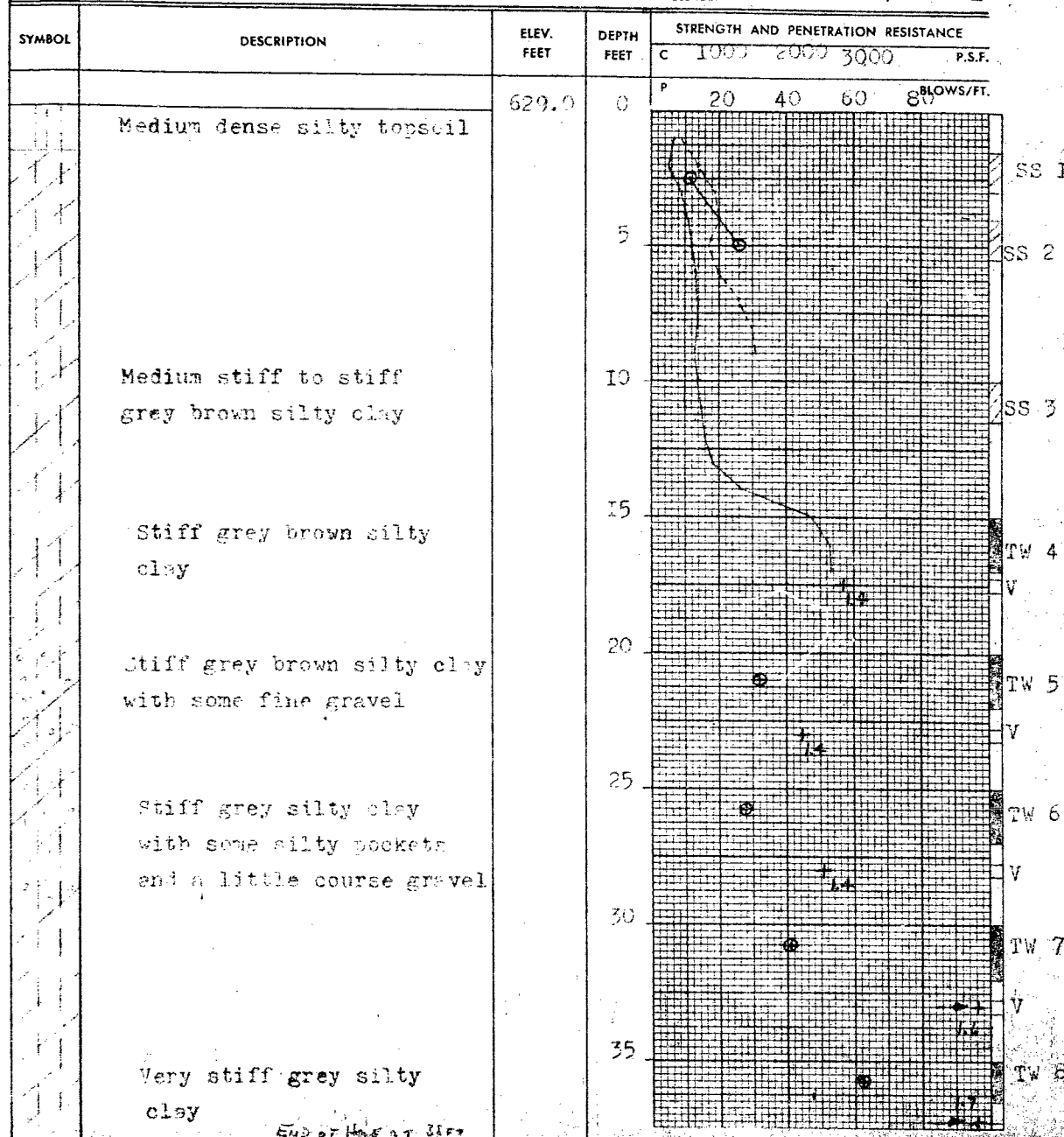
Field Supervisor: A.B. Prep.: P.M.

Driller: C.S. Checked:

Unconfined compression
Vane test and sensitivity (S)Penetration Resistance (P)2" Split tube
2" Dia. Cone
Casing⊕
1"Sampling Method

2" Dia. split tube

2" Shelby tube



Toronto 2, June 12th, 1958.

Memorandum to
Mr. F. C. Brownridge,
Materials & Research Engineer,
Downsview, Ontario.

Re: BA-748 - Heyrock Bridge
Highway 21, District # 3

BA-749 - Blanche River at Judge
Highway # 65, District # 14.

Attached please find above soil reports for your
file.

Encls.
JCM*DW.

J. C. McALLISTER
FOR S. McCOMBIE
BRIDGE PLANNING ENGINEER.