

G.I.-30 SEPT. 1976

GEOCRES No. 41K-35DIST. 18 REGION W.P. No. 903-72-01, 07CONT. No. 76-59W. O. No. STR. SITE No. 38S-285HWY. No. 17 W.B.LOCATION Shewfelt Creek & Hwy 17No of PAGES -=====
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

WATER SUPPLY INTERFERENCE INVESTIGATION
TOWNSHIP OF TARBUTT ADDITIONAL

Prepared for:-
THE MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

HYDROLOGY CONSULTANTS

October 24, 1986
Project: T 5958-HY

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WATER SUPPLY INTERFERENCE INVESTIGATION
TOWNSHIP OF TARBUTT ADDITIONAL

1.0 INTRODUCTION

On behalf of the Ministry of Transportation and Communications (MTC), a hydrogeologic investigation was conducted by Hydrology Consultants to evaluate a well interference complaint of Mrs. E. Paat, R.R. 1, Desbarats, in September, 1985. Based on a review of MTC reports, Ministry of Environment well information, and interviews with local residents, it was concluded that the Paat well may have been affected by highway and culvert construction activities at Shewfelt Creek, at a location roughly 400 m west of the Paat residence. It was concluded that further testing would be required to establish positively whether or not there had been well interference, and if so, how significant. (Figure 1)

In October, 1986, a testing program, involving the drilling and pumping of a well, was undertaken under the supervision of Hydrology Consultants in the vicinity of the reconstructed culvert near a previously observed free-flowing borehole attributed to be the possible cause of local water-supply depletion when the reconstruction work was in progress during the fall of 1976.



2.0 FIELD INVESTIGATION

On October 15, 1986, a 150 mm (6 inch) diameter well, designated as TW 1-86, was completed by Rennison Well Drilling Ltd., of Sault Ste. Marie. The well was installed about 15 metres (50 feet) northeast of the reported flowing borehole (BH-2), at the nearest location accessible to the rotary-drilling equipment (Figure 2).

The well intercepted the following geologic formations:

Fill	0 - 30.5 metres (10 feet)
Grey-red clay	- 5.49 metres (18 feet)
Clay, wood	- 6.40 metres (21 feet)
Red clay	- 24.69 metres (81 feet)
Gravel, boulders	- 25.76 metres (84.5 feet)
Bedrock	- 27.12 metres (89 feet)

Water-bearing formation was found in the deep granular overburden and at the bedrock contact. The static-level of the aquifer zone was about 0.3 metres (one foot) below grade, at an elevation of 188.2 metres (617.7 feet). Telescoping screen, comprising an upper 0.9 metre (3 foot) section of 1.02 mm (40 slot) screen and a lower 0.9 metre (3 foot) section of 0.76 mm (30 slot) screen, was set in the depth interval from 24.1 to 26.2 metres (79 to 86 feet). The well delivered up to 680 litres/minute (150 gallons/minute) during development for a four-hour period using air-lifting. During that time, minor drawdown effects were measured in the neighbouring wells.

On October 16, 1986, a controlled-discharge pumping test was commenced at a rate of 385 litres/minute (85 gallons/minute). However, a gradually steepening drawdown trend was observed, necessitating the reduction of the discharge rate to 205 litres/minute (45 gallons/minute) to ensure an uninterrupted withdrawal for the duration of the planned 24-hour aquifer test.

Prior to, during and following the pumping test, water levels were periodically measured in the domestic wells on neighbouring properties. Particulars and locations of the monitor wells are as follows:

E. Paat Residence: Three wells situated about 400 metres (1300 feet) east of the highway culvert.

1. The 0.9 metres (36-inch) diameter CSP hand-dug well, reportedly affected by the culvert-construction activities in October 1976. The well was measured to have a depth of 7.0 metres (22.8 feet) below ground level, and to contain about 0.6 metres (2.0 feet) of water. The static-level elevation was determined to be 188.9 metres (619.9 feet), by a MTC survey.
- 2) The 150 mm (6-inch) well drilled in October 1977 by R. Furkey to a reported depth of 11.6 metres (38 feet) inside the CSP well. The well was determined to have a static-level elevation of 188.8 metres (619.4 feet).
- 3) The 150 mm (6-inch) diameter well drilled in June, 1978 by A. Pozzebon to a reported depth of 14.3 metres (47 feet), and having a measured static-level of 189.0 metres (620.2 feet).

F. Ingle Residence: Two wells located about 80 metres (250 feet) and 120 metres (400 feet) west of the highway culvert.

- 1) The 0.9 metre (36-inch) diameter CSP hand-dug well, reportedly extending to a depth of 11 metres (36 feet), and having a measured static-level of 192.3 metres (630.8 feet).
- 2) The 150 mm (6 inch) diameter well drilled in November, 1975 to a reported depth of 26.8 metres (88 feet), and having a measured static-level of 188.5 metres (618.4 feet).



C. MacQuarrie Residence: One well located about 640 metres (2100 feet) east of the highway culvert.

The 150 mm (6 inch) diameter well drilled in October, 1978, to a reported depth of 12.2 metres (40 feet), and having a measured static-level of 189.8 metres (622.8 feet).

Water-level responses of the pumped well and the monitor wells are plotted in the attached semi-logarithmic graphs (Figures 3 to 6, inclusive), and are listed in Appendix A.

3.0 ANALYSIS

1. The measured static-level elevation in the pumped well of 188.2 metres (617.7 feet), was slightly higher than the reported static-level elevation of the flowing borehole of 187.8 metres (616 feet) in 1976. This suggests that the groundwater level within the aquifer has been restored to its original stage. In such case, the present water level in the Paat well should also be at its original stage. This indicates that the dug well may have contained only about one metre of water in 1976.
- 2) The pumping test demonstrated that the developed aquifer has a local transmissivity of about 105 m²/day (7000 gpd/foot), and, based upon the early response of the monitor wells, has a confined artesian storativity of about 5×10^{-5} . Strong negative boundary conditions were encountered after the initial hour of pumping, indicating that the aquifer is limited and receives inadequate recharge to balance the withdrawal rate of 385 litres/minute (85 gpm).
- 3) Hydraulic communication was confirmed between the pumped well and each of the monitor wells. The maximum drawdown effect, being 1.02 metres (3.34 feet), was noted in the Ingle drilled well, completed at the same depth as the pumped well. The least effect, being 0.05 metres (0.17 feet), was noted in the shallow Ingle dug well. The three Paat wells had similar drawdown trends, with the water-level lowering ranging from about 0.21 to 0.24 metres (0.7 to 0.8 feet) at the end of 24-hours of pumping. The MacQuarrie well exhibited a similar drawdown trend to the Paat wells, and experienced a total lowering of about 0.15 metres (0.5 feet), as expected considering its distance from the pumped well. All the monitored wells showed a recovery response when pumping was stopped.



- 4) The actual drawdown that occurred during the construction activities is deduced to be significantly less than observed during the recent pumping test. Conservatively, assuming that the free-flowing borehole had an efficiency similar to the drilled well, the flow rate would have to be about one-half the test rate, at the maximum available drawdown of about 4.3 metres (14 feet) to the river level. Being directly related to the withdrawal rate, the drawdown in the Paat dug well may have been about 0.12 metres (0.4 feet), 24 hours after the flow commenced in 1976.
- 5) Unless sufficient recharge is intercepted by the expanding cone under pumping of free-flow conditions, the drawdown will continue in the monitor wells beyond that observed during the 24-hour test at a rate dependent on the aquifer extent, permeability and recharge availability. Based on the observed aquifer response, the water-level may additionally lower by more than 0.3 metres (one foot) in the Paat well within one month at the deduced free-flow rate.

4.0 CONCLUSIONS

During the recent testing program, interference was observed at the Paat dug well allegedly affected by construction activities at the Highway 17 crossing of Shewfelt Creek in 1976. The water-level lowering was small, being about 0.25 metres (0.8 feet), after 24 hours of pumping at a rate of 205 litres/minute (45 gpm). Less drawdown would be initially expected when the previously plugged borehole was unintentionally opened during diversion of Shewfelt Creek in 1976.

However, slow continued drawdown could have occurred with prolonged discharge from the confined aquifer. Based on the observed drawdown trends, a total drawdown in the range of 0.3 to 0.6 metres (one to two feet) might have been observed one month after commencement of the free-flow. Normally, such interference would not be significant, but that amount of water-level lowering in a well of apparent limited available drawdown, could result in the interruption of the water supply.

It appears that free-flow from a borehole within the Shewfelt Creek diversion may have caused the water level in the Paat well to lower enough so that adequate water could not be obtained from that well. One other well of similar depth (McQuarrie) reportedly failed also. Wells with only a few feet of water in them would be sensitive to even slight interference.

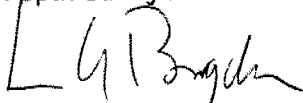
All water supplies were subsequently restored easily by drilling new wells deep enough to increase the available drawdown in the wells so the pumps can obtain the water required by their owners.



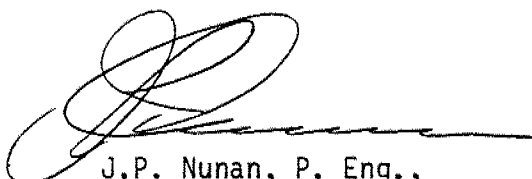
Today, the piezometer pressure in the aquifer has recovered to at least the level observed prior to the road and culvert work in 1976.

Respectfully Submitted,
HYDROLOGY CONSULTANTS

Prepared by:


L.G. Bryck, P. Eng.,
Senior Hydrogeologist

Reviewed by:


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LGB/ird

TARBUTT ADDITION

VI

Paat

MacQuarrie

Ingle

TW 1-86

Shewfelt Creek

V

Metres

0

1000

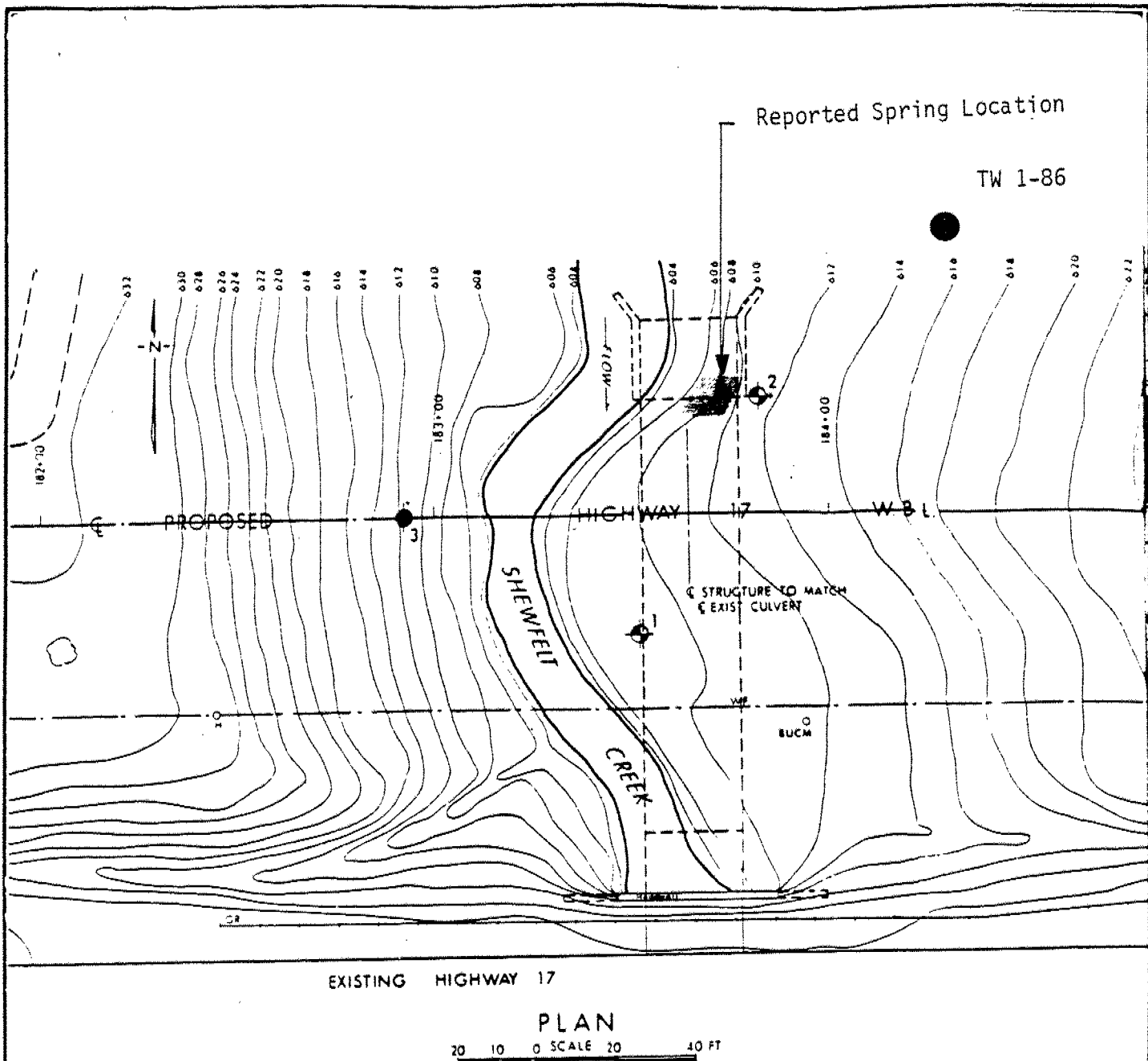


Trow Hydrology Consultants Ltd.

Test Well and
Monitoring Well Locations

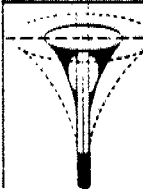
Proj No	T 5958-HY
Scale	As shown
Drawn by	
Appr by	
Revised	
Date	October, 1986

FIGURE 1



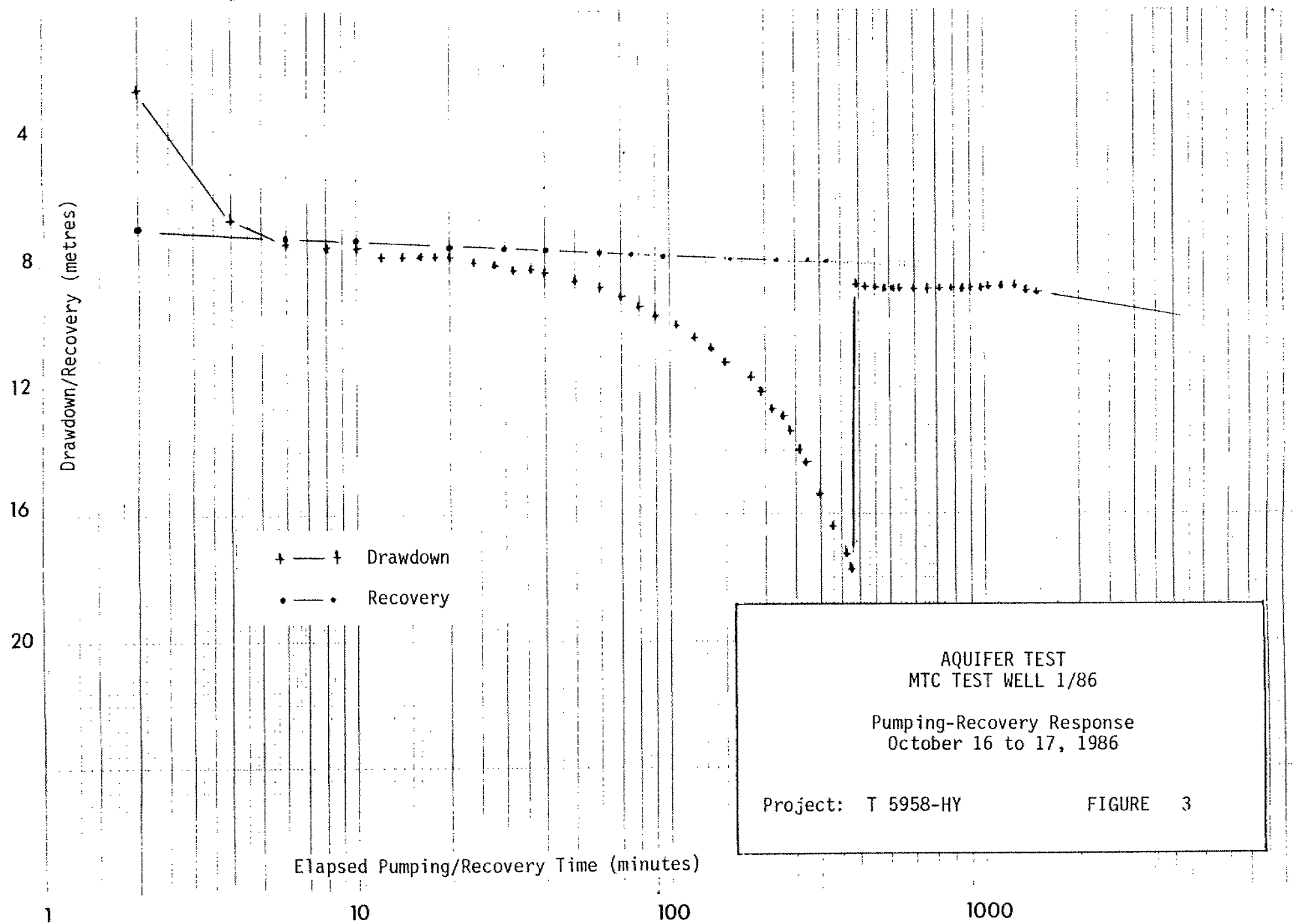
WATER SUPPLY INTERFERENCE STUDY TOWNSHIP OF TARBUTT ADDITIONAL

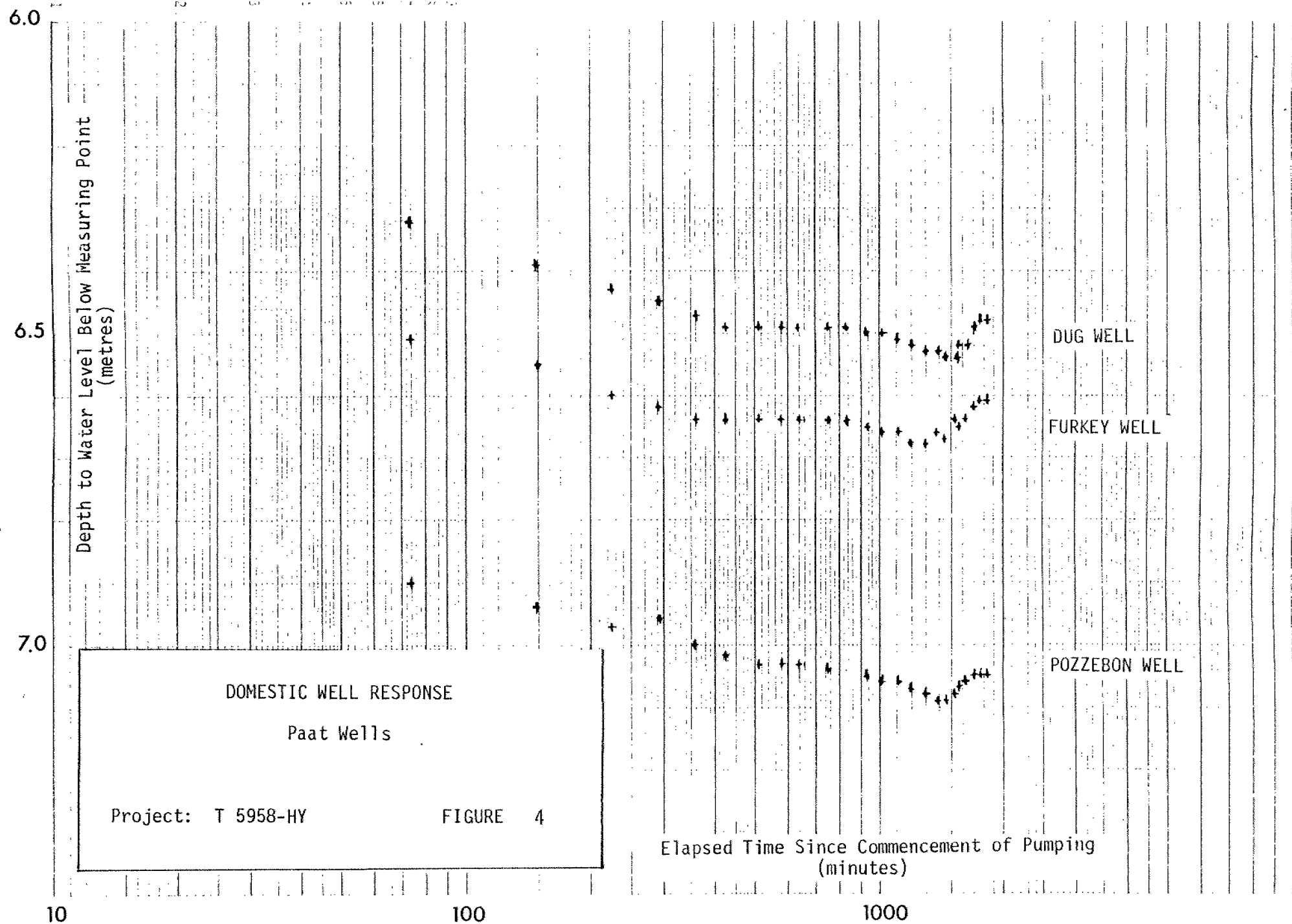
General Test Well and Spring Locations

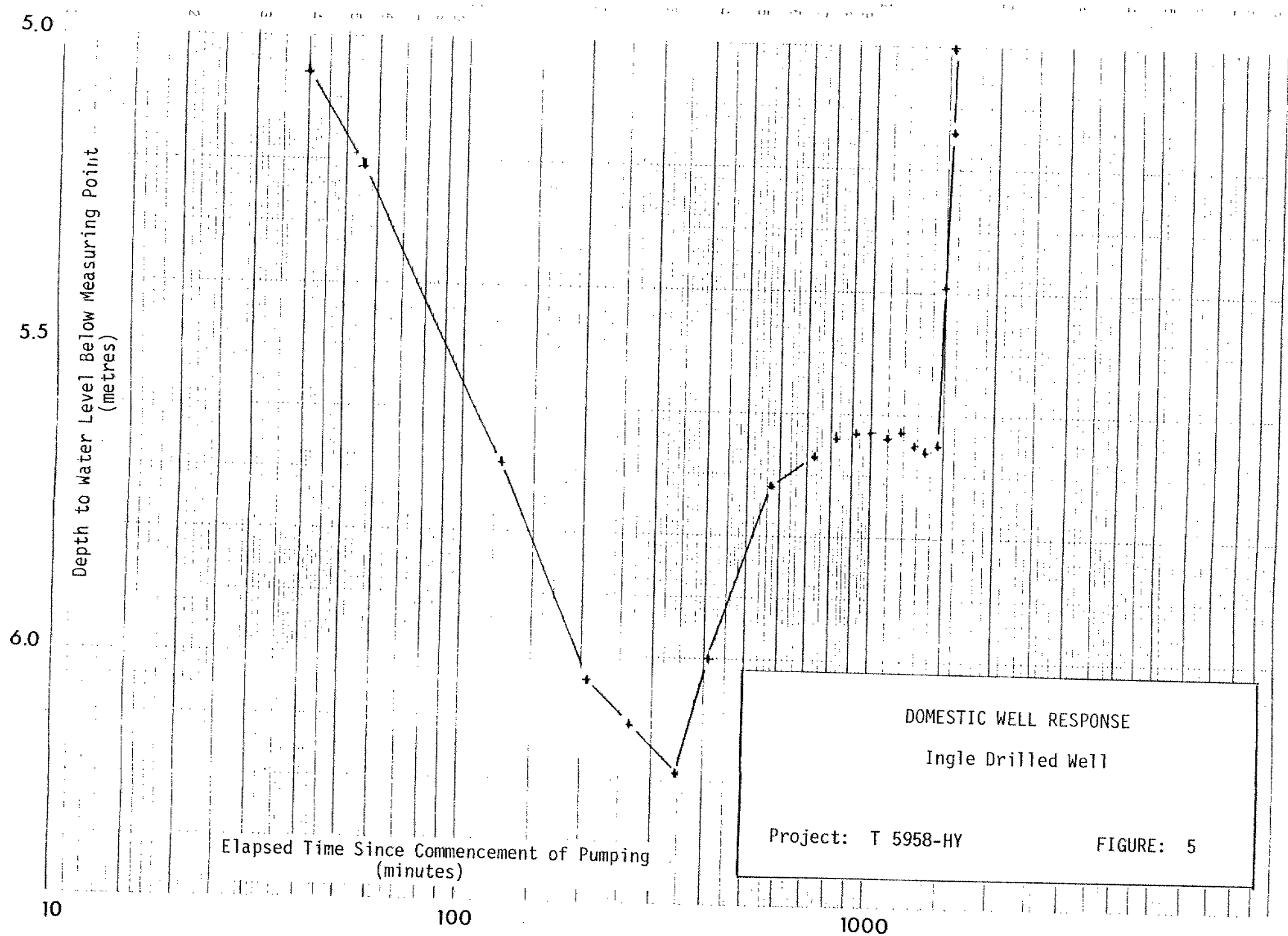


HYDROLOGY CONSULTANTS CONSULTING ENGINEERS AND GEOLOGISTS

Job No	T 5958-HY	Scale	As shown
Drawn by		Date	October, 1986
Appr. by		FIGURE 2	
Revised			







Depth to Water Level Below Measuring Point
(metres)

Elapsed Time Since Commencement of Pumping
(minutes)

DOMESTIC WELL RESPONSE
MacQuarrie Drilled Well

Project: T 5958-HY

FIGURE 6

10

100

1000

5.5

6.0



HYDROLOGY CONSULTANTS

PUMPING RECORD FOR PUMPED WELL 1-86 GAUGE HOLE

Municipality TARBUTT ADDITIONAL Project No. T 5958-HY Date Oct. 16-17, 1986
Distance from Pumped Well -- Static Level 0.79 m G.L. Elevation
Gauge Hole Type: Test Well x Producing Well Depth from Ground Level 26.2 m
Screened x Unscreened Rock (86 ft)
Height Measuring Point Above General Ground Level 189.05 m (620.23 ft)

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	-5	0.79	(Below top of casing)				
	-1	0.79					
	0						
	2	3.32					
	4	7.39					
	6	8.10	Submersible Pump Intake at 18.9 m (62 ft)				
	8	8.31					
	10	8.29					
	12	8.64	Pumping Rate = 385 L/min (85 gpm) until 375 minutes				
	14	8.51	and 205 L/min (45 gpm) thereafter for remainder				
	16	8.63	of test.				
	18	8.52					
	20	8.66					
	24	8.85					
	28	8.89					
	32	9.05					
	36	9.06					
	40	9.16					
	50	9.40					

Pumping Rate = L/min.

PUMPING RECORD FOR PUMPED WELL 1-86 GAUGE HOLE

Municipality _____ Project No. _____ Date _____

Distance from Pumped Well _____ Static Level _____ G.L. Elevation _____

Gauge Hole Type: Test Well ☒ Producing Well _____ Depth from Ground Level _____Screened ☒ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level _____

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	60	9.56					
	70	9.89					
	80	10.16					
	90	10.46					
	105	10.79					
	120	11.19					
	135	11.54					
	150	11.98					
	180	12.48					
	195	12.89					
	210	13.36					
	225	13.68					
	240	14.16					
	255	14.73					
	270	15.17					
	300	16.15					
	330	17.20					
	360	18.05					
	375	18.61					

Pumping Rate = _____ L/min.



HYDROLOGY CONSULTANTS

PUMPING RECORD FOR PUMPED WELL 1-86 GAUGE HOLE

Municipality _____ Project No. _____ Date _____

Distance from Pumped Well _____ Static Level _____ G.L. Elevation _____

Gauge Hole Type: Test Well ☒ Producing Well _____ Depth from Ground Level _____Screened ☒ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level _____

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	390	9.53			1380	9.80	
	420	9.59			1440	9.83	
	450	9.64			1474	9.81	
	480	9.64					
	510	9.60			Recovery commenced at		
	540	9.65			1485 minutes (12:45 pm, Oct.		
	600	9.70			17, 1985)		
	660	9.69					
	720	9.68					
	780	9.70					
	840	9.68					
	900	9.64					
	960	9.62					
	1020	9.59					
	1080	9.60					
	1140	3.59					
	1200	9.62					
	1260	9.58					
	1320	9.72					

Pumping Rate = _____ L/min.

RECOVERY RECORD FOR PUMPED WELL 1/86 GAUGE HOLE

Municipality _____ Project No. _____ Date _____

Distance from Pumped Well _____ Static Level _____ G.L. Elevation _____

Gauge Hole Type: Test Well ☒ Producing Well _____ Depth from Ground Level _____Screened ☒ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level _____

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	2	2.11			70	1.29	
	4	1.92			75	1.28	
	6	1.82			80	1.27	
	8	1.75			85	1.26	
	10	1.70			95	1.23	
	12	1.67			115	1.19	
	14	1.64			141	1.17	
	16	1.61			155	1.13	
	18	1.59			175	1.11	
	20	1.57			195	1.09	
	25	1.52			215	1.08	
	30	1.48			235	1.06	
	35	1.45			255	1.05	
	40	1.42			275	1.05	
	45	1.40			295	1.04	
	50	1.37			315	1.03	
	55	1.35					
	60	1.33					
	65	1.31					

Pumping Rate = _____ L/min.



HYDROLOGY CONSULTANTS

PAAT

RECORD FOR PUMPED WELL _____ GAUGE HOLE - Dug Well

Municipality Tarbutt Additional Project No. T 5958-HY Date Oct. 16-17/86Distance from Pumped Well 400 m± Static Level 6:30 m G.L. Elevation _____Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level 7 m (22.8 ft)Screened _____ Unscreened x Rock _____Height Measuring Point Above General Ground Level 195.24 m (640.55 ft)

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	-15	6.30	(Below top of CSP)				
	0	-	Test commenced at 12:00 noon on October 16, 1986				
	74	6.32					
	148	6.39					
	225	6.43					
	292	6.45					
	361	6.47					
	426	6.49					
	508	6.49					
	582	6.49					
	635	6.49					
	751	6.49					
	930	6.50					
	1018	6.50					
	1113	6.51					
	1199	6.52					
	1287	6.53					
	1375	6.53					
	1456	6.54					

Pumping Rate = _____ L/min.

HYDROLOGY CONSULTANTS

PAAT

RECORD FOR PUMPED WELL _____ GAUGE HOLE _____ - Bug Well

Municipality _____ Project No. _____ Date _____

Distance from Pumped Well _____ Static Level _____ G.L. Elevation _____

Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level _____

Screened _____ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level _____

[illegible]

Pumping Rate = _____ L/min.



HYDROLOGY CONSULTANTS

RECORD FOR PUMPED WELL _____ GAUGE HOLE _____ PAAT
- Furkey Well
Municipality TARBUTT ADDITIONAL Project No. T 5958-HY Date Oct. 16-17/86
Distance from Pumped Well 400 m± Static Level 6.45 m G.L. Elevation _____
Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level 11.6 m (35 f)
Screened _____ Unscreened x Rock _____
Height Measuring Point Above General Ground Level 195.24 m (640.55 ft)

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	-15	6.45	(Below top of CSP)				
	0	-	Test commenced at 12:00 noon on October 16, 1986				
	74	6.51					
	148	6.55					
	225	6.60					
	292	6.62					
	361	6.64					
	426	6.64					
	508	6.64					
	582	6.64					
	635	6.64					
	751	6.64					
	930	6.65					
	1018	6.66					
	1113	6.66					
	1199	6.68					
	1287	6.68					
	1375	6.66					
	1456	6.69					

Pumping Rate = _____ L/min.



RECORD FOR PUMPED WELL _____ GAUGE HOLE

Height Measuring Point Above General Ground Level

Pumping Rate = _____ L/min.



HYDROLOGY CONSULTANTS

PAAT

- Pozzebon Well

RECORD FOR PUMPED WELL

GAUGE HOLE

Municipality TARBUTT ADDITIONAL Project No. T 5958-HY Date Oct. 16-17/86Distance from Pumped Well 400 m± Static Level 6.87 m G.L. Elevation _____Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level 14.3 m (47 ft)Screened _____ Unscreened x Rock _____Height Measuring Point Above General Ground Level 195.24 m (640.55 ft)

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	-15	6.87	(Below top of casing)				
	0	-	Test commenced at 12:00 noon on October 16, 1986				
	74	6.90					
	148	6.94					
	225	6.97					
	292	6.96					
	361	7.01					
	426	7.03					
	508	7.03					
	582	7.03					
	635	7.03					
	751	7.04					
	930	7.05					
	1018	7.06					
	1113	7.06					
	1119	7.07					
	1287	7.08					
	1375	7.09					
	1456	7.09					

Pumping Rate = _____ L/min.

RECORD FOR PUMPED WELL

PAAT

GAUGE HOLE - Pozzebon Well

Municipality _____ Project No. _____ Date _____

Distance from Pumped Well _____ Static Level _____ G.L. Elevation _____

Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level _____

Screened _____ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level

[illegible]

Pumping Rate = _____ L/min.



HYDROLOGY CONSULTANTS

RECORD FOR PUMPED WELL

GAUGE HOLE

INGLE

- Drilled Well

Municipality TARBUTT ADDITIONAL Project No. T 5958-HY Date Oct. 16-17/86Distance from Pumped Well 120 m± Static Level 4.63 m G.L. Elevation _____Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level 26.8 m (88 f

Screened _____ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level 193.10 m (633.55 ft)

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	-65	4.74				
	-30	4.63	(Below top of measuring point)			
	0	-	Test commenced at 12:00 noon on October 16, 1986			
	40	5.06				
	55	5.21				
	126	5.69				
	208	6.04				
	264	6.11				
	342	6.19				
	409	6.00				
	490	5.76				
	567	5.72				
	640	6.38				
	734	5.67				
	817	5.64				
	912	5.63				
	1000	5.62				
	1100	5.64				
	1185	5.65				

Pumping Rate = _____ L/min.

INGLE

- Drilled Well

RECORD FOR PUMPED WELL

GAUGE HOLE

Municipality _____ Project No. _____ Date _____

Distance from Pumped Well _____ Static Level _____ G.L. Elevation _____

Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level _____

Screened _____ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level _____

[illegible]

Pumping Rate = _____ L/min.



HYDROLOGY CONSULTANTS

INGLE
- Dug Well

RECORD FOR PUMPED WELL

GAUGE HOLE

Municipality TARBUTT ADDITIONAL Project No. T 5958-HY Date Oct. 16-17/86
Distance from Pumped Well 80 m± Static Level 0.81 m G.L. Elevation _____
Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level 11 m (36 ft)
Screened _____ Unscreened _____ Rock _____
Height Measuring Point Above General Ground Level 193.08 m (633.46 ft)

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	-70	0.81	(Below top of CSP)				
	0	-	Test commenced at 12:00 noon on October 16, 1986				
	40	0.82					
	126	0.82					
	208	0.82					
	264	0.82					
	342	0.82					
	409	0.82					
	490	0.83					
	567	0.83					
	640	0.83					
	734	0.83					
	817	0.83					
	912	0.84					
	1000	0.84					
	1100	0.84					
	1185	0.84					
	1268	0.85					
	1356	0.85					

Pumping Rate = _____ L/min.



HYDROLOGY CONSULTANTS

MacQUARRIE
- Drilled Well

RECORD FOR PUMPED WELL _____ GAUGE HOLE _____

Municipality TARBUTT ADDITIONAL Project No. T 5958-HY Date Oct. 16-17/86
Distance from Pumped Well 640 m± Static Level 5.47 m G.L. Elevation _____
Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level 12.2 m (40 ft)
Screened _____ Unscreened _____ Rock _____
Height Measuring Point Above General Ground Level 195.29 m (640.72 ft)

Date & Time	Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)		Elapsed Time (min.)	Depth to Water (m)	Drawdown (m)
	-155	5.47	(Top of casing)				
	-25	5.52					
	0	-	Test commenced at 12:00 noon on October 16, 1986				
	62	5.56					
	135	5.53					
	194	5.56					
	282	5.58					
	349	5.59					
	416	5.61					
	500	5.61					
	573	5.62					
	646	5.62					
	741	5.67					
	824	5.62					
	922	5.63					
	1010	5.64					
	1100	5.64					
	1192	5.76					
	1280	5.65					

Pumping Rate = _____ L/min.

HYDROLOGY CONSULTANTS

MacQUARRIE
- Drilled Well

RECORD FOR PUMPED WELL

GAUGE HOLE

- Drilled Well

Municipality _____ Project No. _____ Date _____

Distance from Pumped Well _____ Static Level _____ G.L. Elevation _____

Gauge Hole Type: Test Well _____ Producing Well _____ Depth from Ground Level _____

Screened _____ Unscreened _____ Rock _____

Height Measuring Point Above General Ground Level _____

[illegible]

Pumping Rate = _____ L/min.

WATER SUPPLY
INTERFERENCE INVESTIGATION
TOWNSHIP OF TARBUTT ADDITIONAL

Prepared for:
THE MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

HYDROLOGY CONSULTANTS
(Division of Trow Ltd.)

September 20, 1985

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- Appendix A: MTC Borehole Records for BH-1, BH-2 and BH-3
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WATER SUPPLY INTERFERENCE INVESTIGATION TOWNSHIP OF TARBUTT ADDITIONAL

1.0 INTRODUCTION

At the request of the Ministry of Transportation and Communications(MTC), a hydrogeologic investigation was conducted by Hydrology Consultants to evaluate the well interference claim of Mrs. E. Paat, RR 1, Desbarats.

Construction activities related to crossing of Highway 17 (WBL) at Shewfelt Creek (Figure 1) were alleged to have caused the loss of an established farm water supply, with the subsequent consequences to a dairying operation.

The investigation comprised a review of the pertinent correspondence, construction drawings and foundation investigation reports filed with the MTC; regional water-well information published by the Ministry of the Environment; and a site visit to interview a MTC construction staff member and local residents in the vicinity of the crossing.

2.0 SUMMARY OF STATEMENTS/REPORTS/OBSERVATIONS

2.1 Mrs. E. Paat, Claimant

Mrs. E. Paat stated that her well went dry in October 1976, at which time construction of the road crossing was in progress. The well consists of 36-inch diameter CSP casing, and was hand dug by her husband. During the period of about 25 years following completion, the well yielded adequate supplies for domestic and dairying uses, regardless of climatic conditions. At no time, had water been hauled to supplement the well supply.

When the water supply problem occurred in October, 1976, water was hauled and put into the well, but, within a period of hours, the well failed again. Consequently, this practice was not continued and the cows were watered at the creek until winter conditions limited access to the creek. Thereafter, the dairy head was sold, and the remaining calves and dry cows were watered from a storage tank in the barn, which was periodically filled with hauled water.

In October 1977, Mr. R. Furkey drilled a 6-inch diameter cased well within the affected well to a total depth of 38 feet. A water supply was restored, but it had a high iron content.

Subsequently, in June 1978, the MTC arranged for the construction of another 6-inch diameter drilled well, which extended to a total depth of 47 feet. The water was better in quality than the Furkey well and has been used as the household source since completion.

During the site inspection, the unused 36-inch diameter CSP well was measured to have a static level of 19.8 feet and a total depth of 23 feet, thus about three feet of water in the well. The water level depth could not be measured in the drilled wells, being equipped with sanitary seals.

2.2 Mr. F. Ingle, Neighbour

Until June 1976, Mr. Ingle used a 36-inch diameter CSP well, hand dug to a depth of 30 feet about 15 years earlier. The dug well initially contained no water, but over the winter filled to near surface, and yielded sufficient supplies for the two-person household thereafter.

At the time of the highway widening, his residence was moved back on the property and a drilled well was provided by the MTC as a replacement source in June, 1975. The well was reportedly completed in the overburden at a depth of 86 feet and yielded silty water, which still requires filtering and periodic backwashing. No problems were experienced with regard to water-supply availability during the construction activities at Shewfelt Creek.

During the inspection (September 11, 1985), the static level in the dug well was measured to be 4.2 feet below ground level, providing an available drawdown of about 26 feet. The drilled well was equipped with a sanitary seal and was not accessible for measurement.

2.3 Mr. C. McQuarrie, Neighbour

Mr. McQuarrie advised that from 1974 to the fall of 1976, an adequate domestic supply was obtained from an 18-inch diameter concrete lined bored well extending to a depth of about 20 feet with a 2½ inch diameter well point extending deeper. During the fall of 1976, the supply failed and water was thereafter hauled for many months.

In November 1978, a new well was drilled by MTC to a depth of about 50 feet, which has since provided adequate domestic supplies.

During the inspection, the uncased dug well was measured to have a static level of 15.2 feet and a total depth of about 18 feet, providing an available drawdown of about 3 feet. The drilled well was not accessible for water-level measurement.

2.4 Mr. G. Gardiner, MTC

Mr. Gardiner reported that, on October 14, 1976, when the stream channel was excavated between the two rows of sheet pile to an elevation of about 600 feet, an artesian spring was intercepted about 34 feet north of the west lane centreline and within 5 feet of a sheet-pile wall (Figure 2). At that time, the sheet piling extended to a depth of about 15 feet from top of footings, and the east side footing excavation extended to an elevation of about 595 feet. After the channelling, driving of the H-piles to a depth of about 60 feet was commenced in the east side excavation. Excavation for the west side footings was completed on October 25, 1976 to a similar depth, and subsequently installation of the H-piles was completed.

The artesian spring in the river bottom continued to flow throughout the construction period, with the adjacent footing excavations being about 5 feet lower in elevation than the river bottom. Although not quantifiable, the artesian flow was reportedly substantial, with the strong upwelling of clear water in the otherwise muddy river channel.

In his recollection, Mr. Gardiner stated that the footing excavations remained essentially dry during the construction period and that no seepage was discernable around the "driven" H-piles.

Evidence of continued artesian flow could not be established during the site visit due to high turbidity of the river water.

2.5 Foundation Investigation Report, MTC, April 1975 Foundation Investigation Report, MTC, April 1976

These reports document the subsurface conditions intercepted by three boreholes installed in the immediate vicinity of the Shewfelt Creek crossing. Records of these boreholes are provided in Appendix A.

Boreholes No. 1 and No. 2, extended through a cohesive lacustrine deposit, largely comprising firm to stiff clay, with occasional silt seams, about 65 feet in thickness, and into an underlying dense bouldery sand and gravel formation, about 15 feet in thickness, overlying bedrock. An above-ground artesian pressure (Figure 3) was found in the granular zone. The recorded piezometric head was about 612 feet in BH No. 1 and 616 feet in BH No. 2. A flow of about 7 to 8 gallons per minute was observed from the boreholes. The sand and gravel zone was inferred to be a confined aquifer, being recharged from the surrounding terrain at higher elevation.

The boreholes were reportedly sealed with bentonite to contain the artesian flow.

3.0 INTERPRETED HYDROGEOLOGIC CONDITIONS

The limited water-well information, summarized in Appendix B, in the vicinity of the Shewfelt Creek-Highway 17 area indicates that a broad bedrock depression is present beneath the creek, typically containing basal granular sediments covered by a thick clay layer.

Continuity of the basal granular zone, occurring immediately above the bedrock surface, may be inferred from the construction area at the creek eastward to the Paat and McQuarrie residences.

There appears to be a natural groundwater gradient from the adjacent uplands towards the creek, with much of the infiltrating precipitation being transmitted through the basal granular zone. The clay layer is too tight to transmit water and acts as an aquiclude above the sand/gravel aquifer beneath the stream valley.

Generally, private wells in the vicinity of the Shewfelt stream crossing have been either drilled or dug through the surficial clay layer and into the underlying water bearing sand and gravel. Most wells are located near residences on the upland area where the static water levels are well below ground surface. It is only in the stream valley that flowing conditions would be encountered.

4.0 ANALYSIS OF DATA

As lateral inflow was precluded by the sheet piling extending to about 15 feet and the footing excavations extending to a depth about 5 feet below the river, the artesian spring encountered in the river bottom had to be supplied by a vertical conduit connected to the water zone beneath the clay layer.

The occurrence of a natural vertical opening extending to the confined deep granular zone is geologically improbable in a saturated clay deposit. However, the reported location of the artesian spring is close to the location of Borehole No. 2.

The deep boreholes were reportedly plugged. The plug may have been in the upper few feet, as the deep placement of bentonite would be extremely difficult with the persistent upward flow in the borehole. Information is unavailable regarding the depth, setting and thickness of the bentonite seals on the borehole records.

If only sealed at surface, the borehole in a cohesive soil could remain open, being filled with water from the upflow.

The interception of the artesian spring in the river bottom occurred following the excavation of 4 to 8 feet of earth from

the channel. This could have removed a surface seal and possibly re-establish the free flow seen when the borehole was open.

The artesian flow at stream level could have been more substantial than earlier estimated when the top of the casing was several feet higher. This would support the MTC observation of substantial flow from the artesian spring.

Hydraulic continuity is feasible between the water-bearing formation intercepted by the original Paat well and the granular zone intercepted by the deep MTC boreholes at the stream crossing. The water-bearing material at both locations appears to be similar.

Theis non-equilibrium calculations, based on confined artesian conditions and a moderate discharge of 5 to 10 gallons/minute, indicate that the artesian pressure, at the distance of the Paat well could be lowered by a few feet. For example, within one day at a discharge rate of 10 gallons per minute from an aquifer having a transmissivity of 1000 gpd/foot and a storativity of 10^{-5} , the water level could lower about 3.5 feet at a distance of 1300 feet. Additional drawdown would be incurred with prolonged pumping or if negative boundary conditions were encountered.

The reported occurrence of the water-supply interruption of the Paat residence is compatible with the apparent hydraulic conditions. With lowering of the groundwater level, any water put into the well would drain within a short-time interval, as was observed.

The low water stage measured in the Patt well suggests that drainage of the deep aquifer has continued in the vicinity of the

highway crossing. The present supply from the well appears limited, considering the available drawdown within the well bore.

There is uncertainty whether or not the groundwater level, as measured in the "Paat" well now is lower than it was prior to construction or if indeed it was lowered during, or as a consequence of the construction work. If the water level in the Paat well was lowered due to loss of water from the artesian aquifer during construction, it is unlikely that it has recovered.

The fact that the well now contains $2\frac{1}{2}$ to 3 feet of water begs the question, why did the supply fail?

5.0 CONCLUSIONS AND RECOMMENDATIONS

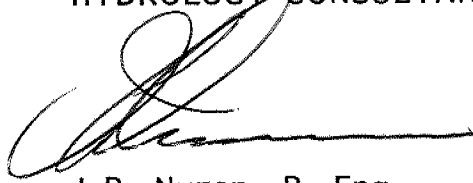
The Paat well could have been adversely affected by an observed artesian flow at the construction site, if the aquifer zone beneath the surficial clay layer extend between the construction site and the Paat well.

The currently available data are inadequate to ascertain whether or not significant interference occurred. Interference could be verified, or refuted, by installation of a temporary test well close to where the artesian flow was noted and conduction of a controlled discharge pumping test while measuring water level response in the Paat well. The test well could be plugged after completion of the pumping test. We see no other means of reaching a definitive position.

The presence of three feet of water in the Paat well now coupled with the report of Mrs. Paat, that hauled water, dumped into the well, disappeared quickly, suggests that the water level stage may have been naturally lower during the fall of 1976, and that this may have contributed to the failure of her well then.

We recommend that further testing is warranted. The cost of such work is estimated to be approximately \$5,000.

Respectfully Submitted,
HYDROLOGY CONSULTANTS



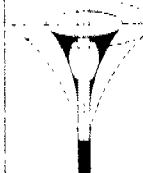
J.P. Nunan, P. Eng.
Principal Hydrogeologist

FIGURES



WATER SUPPLY INTERFERENCE STUDY
TOWNSHIP OF TARBUTT ADDITIONAL

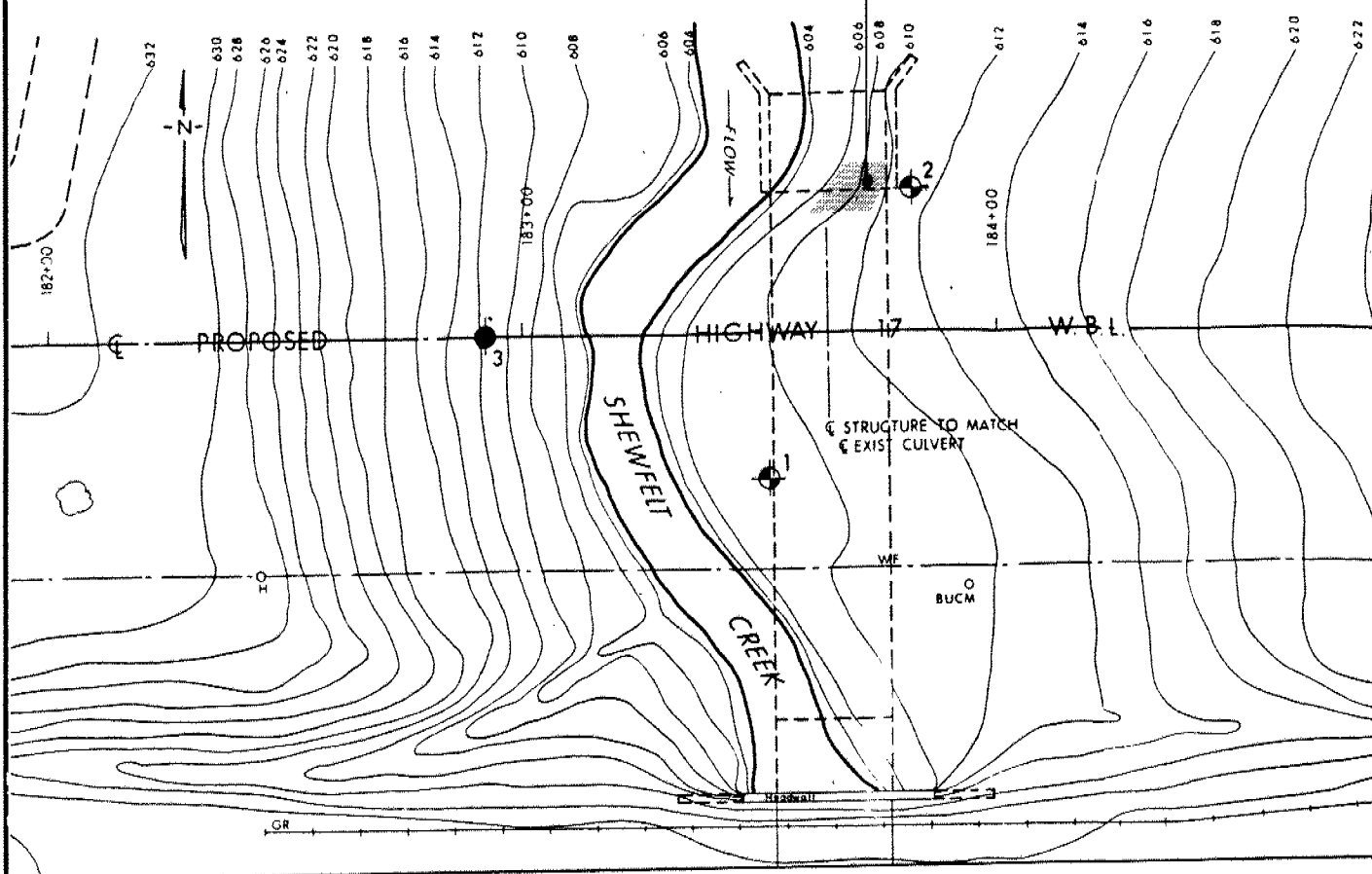
Study Location



HYDROLOGY CONSULTANTS
CONSULTING ENGINEERS AND GEOLOGISTS

Job NO	T 5958-HY	Scale	1:15,800±
Drawn. by		Date	September, 1985
Appr. by		FIGURE 1	
Revised			

Reported Spring
Location



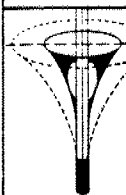
EXISTING HIGHWAY 17

PLAN

20 10 0 SCALE 20 40 FT

WATER SUPPLY INTERFERENCE STUDY TOWNSHIP OF TARBUTT ADDITIONAL

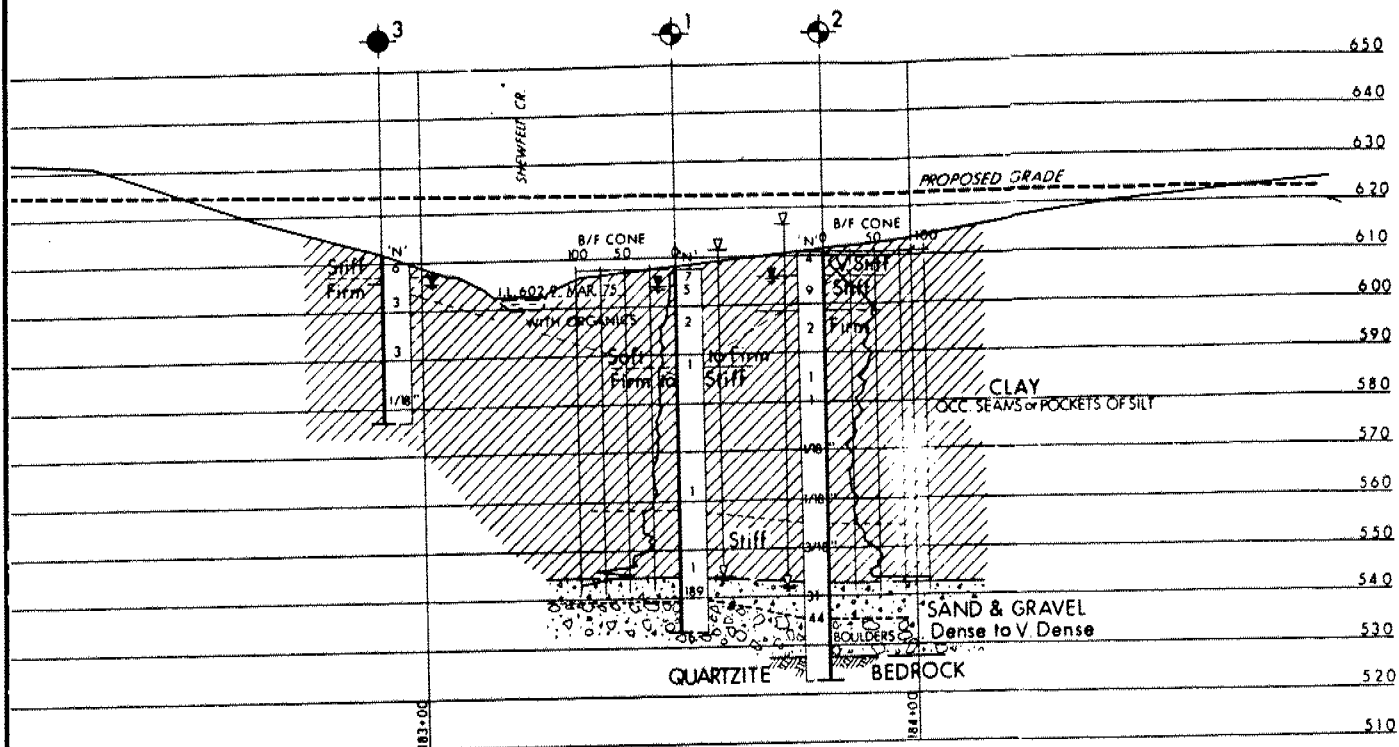
Plan At Highway Crossing



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CONSULTING ENGINEERS AND GEOLOGISTS

Job No	T 5958-HY	Scale	As shown
Drawn. by		Date	September, 1985
Appr. by			
Revised			

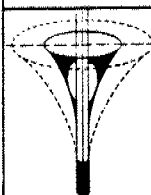
FIGURE 2



Q PROFILE
20 10 0 SCALE 20 40 FT

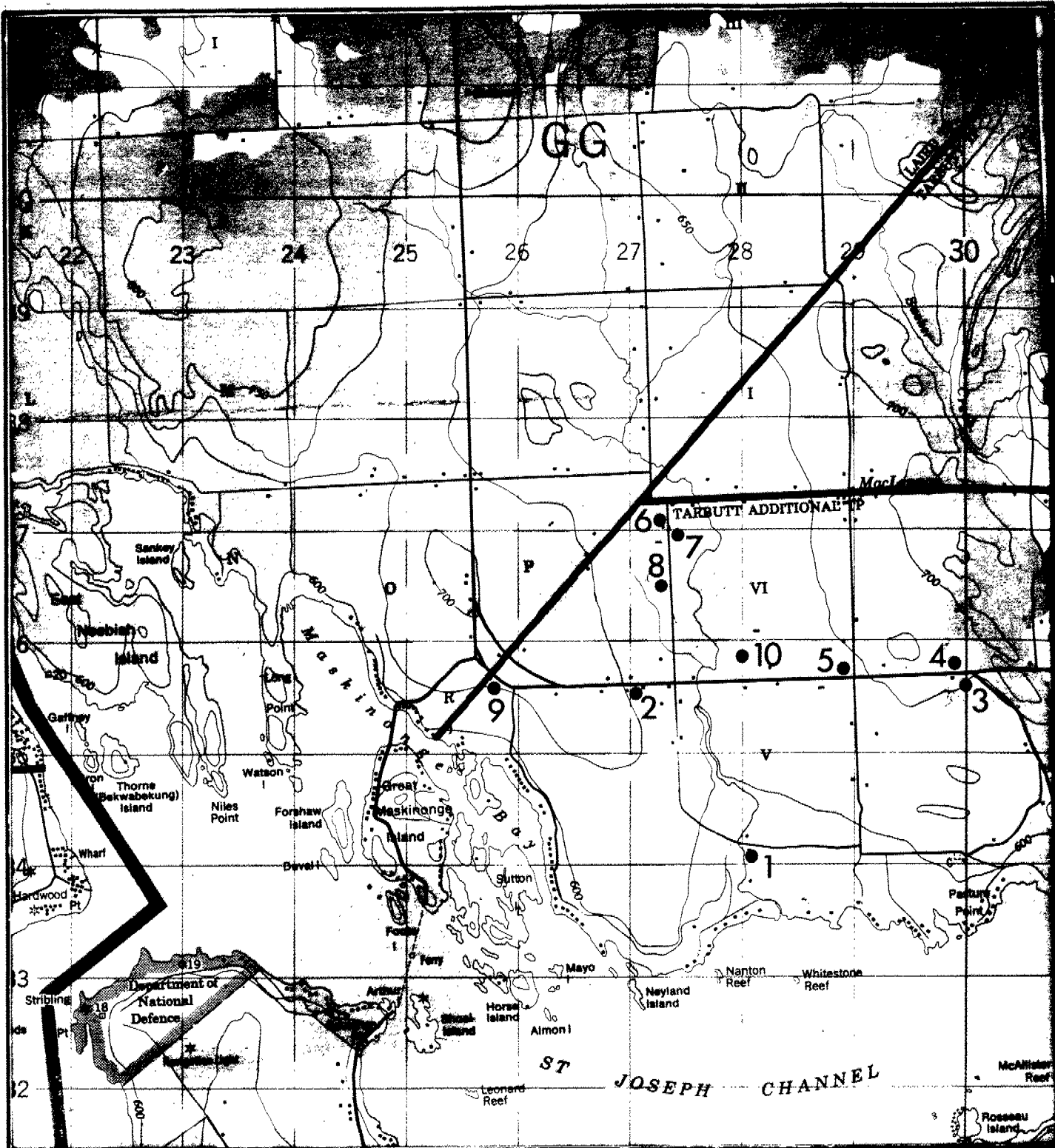
WATER SUPPLY INTERFERENCE STUDY TOWNSHIP OF TARBUTT ADDITIONAL

Geologic Section At Highway Crossing



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CONSULTING ENGINEERS AND GEOLOGISTS

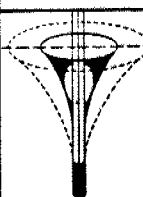
Job NO	T 5958-HY	Scale	As shown
Drawn. by		Date	September, 1985
Appr. by		FIGURE 3	
Revised			



● 1 Domestic Well

WATER SUPPLY INTERFERENCE STUDY TOWNSHIP OF TARBUTT ADDITIONAL

Domestic Well Locations
(Filed with Ministry of the Environment)



HYDROLOGY CONSULTANTS
CONSULTING ENGINEERS AND GEOLOGISTS

Job No	T 5958-HY	Scale	1:50,000
Drawn by		Date	September, 1985
Appr. by			
Revised			

FIGURE 4

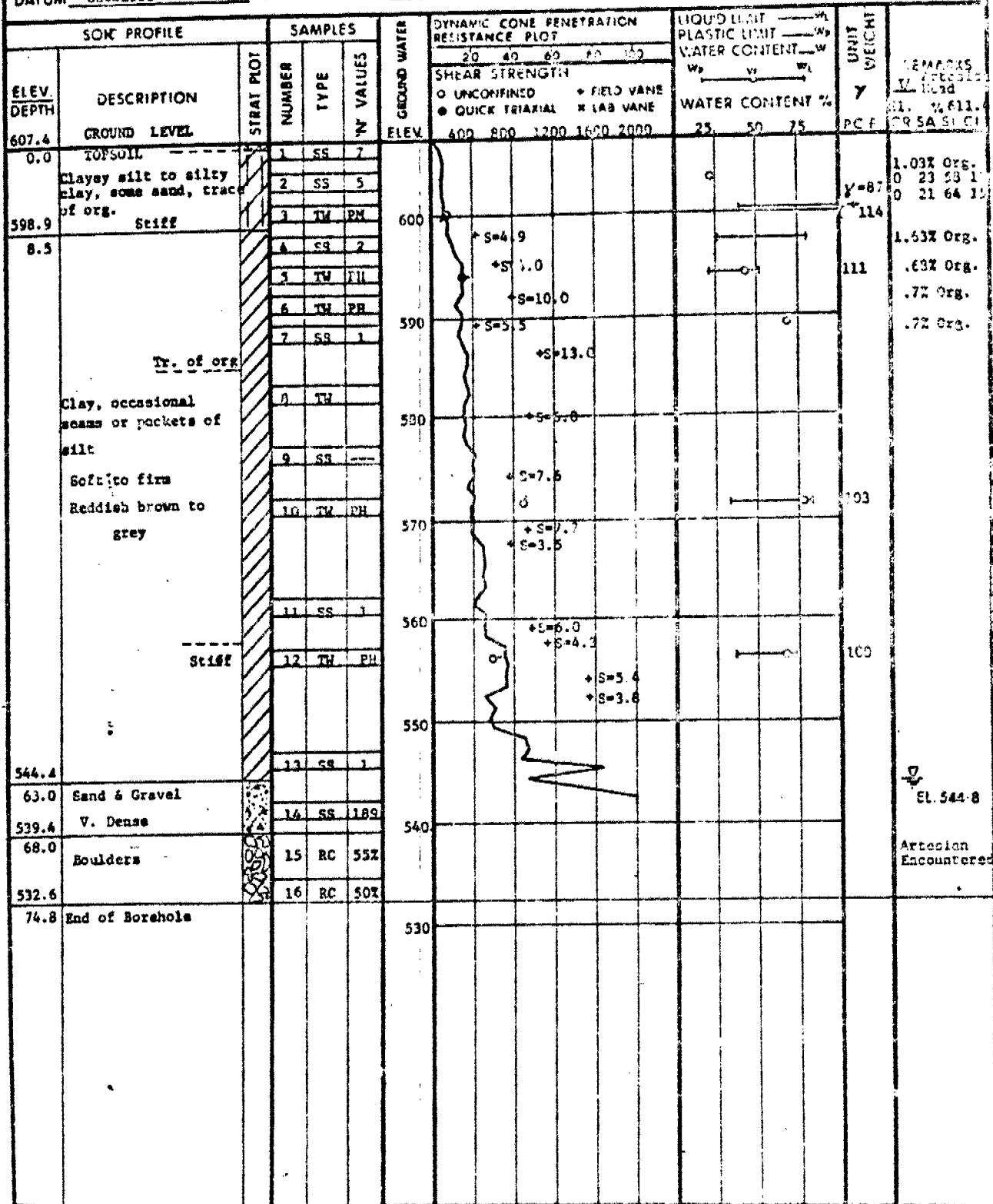
APPENDIX A

APPENDIX A

 MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
 ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 1

W.P. 903-72-07 LOCATION STA: 183+52 30' RT OF Q. U.B.L. OF RT. 17 ORIGINATED BY
 DIST. 18 HWY. 17 BORING DATE MARCH 17th and 18th, 1972 COMPILED BY B.A.
 DATUM GEODETIC BOREHOLE TYPE AUGER & CONE TEST CHECKED BY



OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
 ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 2

W.P. 903-72-07

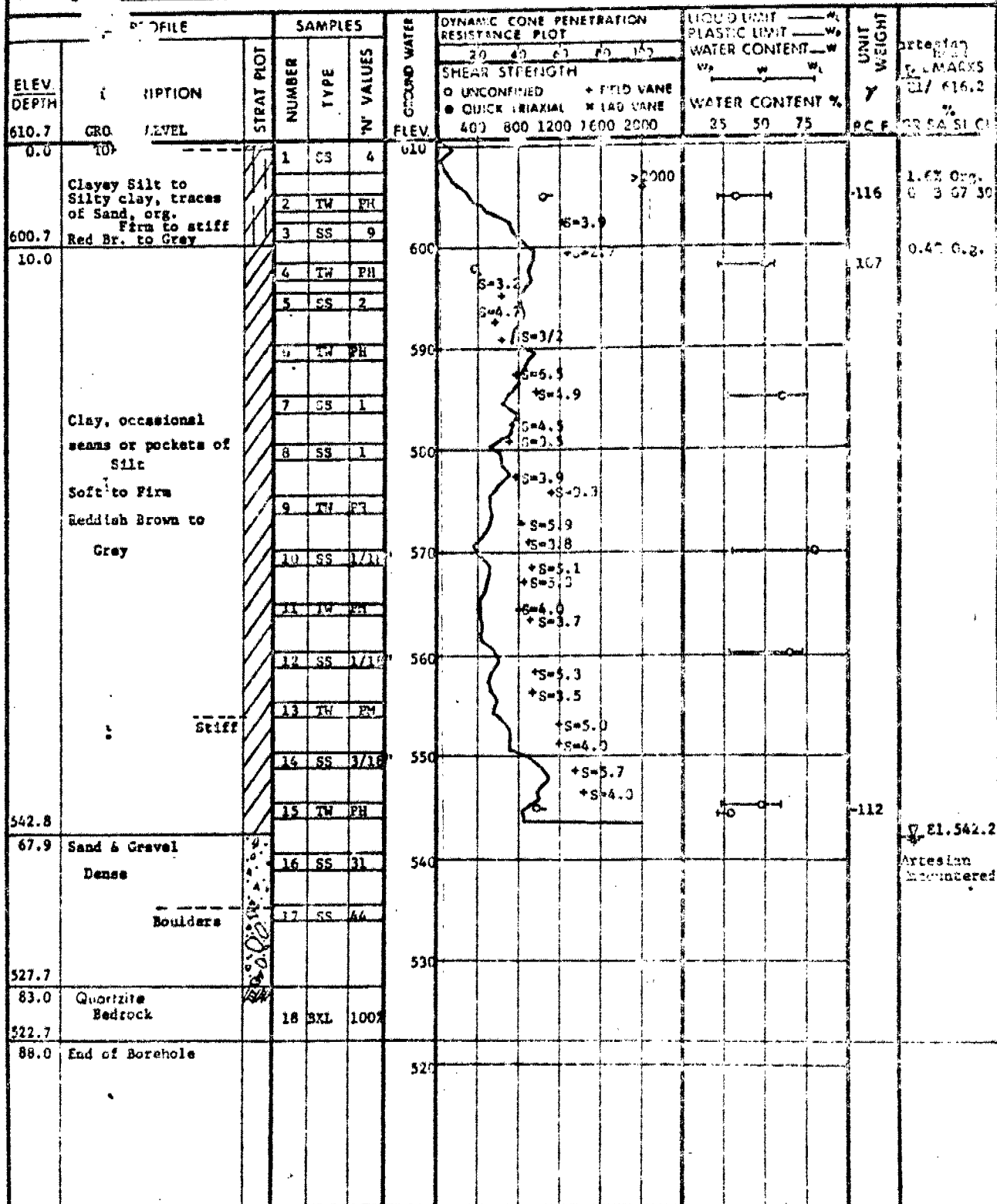
LOCATION STA. 181+82.50' LT. ON G. H.W. 1. 17

ORIGINATED BY

DIST. 18

HWY. 17

BORING DATE MARCH 18, 1975

COMPILED BY DATUM BOREHOLE TYPE CHECKED BY 

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH - CIVIL ENGINEERING OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

W.P. 903-72-07

LOCATION STA 35+02.5 H.B. 17

ORIGINATED BY C.M.V.

DIST. 18 HWY. 17

BORING DATE MAR 19, 1972

COMPILED BY C.M.V.

DATUM GEODETIC

BOREHOLE TYPE AUGER AND SPLIT WITH ONE AT EACH END

CHECKED BY C.M.V.

SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION TEST-STATUS PLOT	LIQUID LIMIT w_p		REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE VALUES		PLASTIC LIMIT w_p	WATER CONTENT w	
612.2	GROUND LEVEL						
0.0	Topsoil		1 SS 6				
	Clay, occasional Pockets of Silt		2 TM PH				
	Soft to Firm		3 SS 3				
	Reddish Brown to Gray		4 TM PH				
			5 SS 3				
			6 TM PH				
			7 SS 1/18"				
577.9	End of Borehole						
34.5							

20
15 \diamond 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX B

APPENDIX B

Water Well Information
Shewfelt Creek-Highway 17 Area
Township of Tarbutt Additional

<u>Well No.</u>	<u>MOE Number</u>	<u>Static Level (ft)</u>	<u>Pumping Level (ft)</u>	<u>Test Rate (gpm)</u>	<u>Test Duration (hrs)</u>	<u>Water Quality</u>	<u>Geologic Log (ft)</u>
1	1016	12	20	25	8	Fresh	Red clay 32; gravel 48; rock 54.
2	1018	_____ DRY _____					Silt 9; clay 27; boulder, clay and medium sand 80; quartz 89.
3	1012	6	20	3	4	Fresh	Clay 22; boulders and clay 28.
4.	1022	_____ DRY _____					Medium sand 29; granite 30.
5	1023	9	18	5	--	Fresh	Clay 31; gravel 32.
6	1024	Flowing	3	5	4	Fresh	Red clay 72; gravel 73.
7	1025	24	24	2	8	Fresh	Medium sand 5; red clay 40; white gravel 41.
8	1026	2	10	2	8	Fresh	Medium sand 7; red clay 66; gravel 67.
9	1027	2	22	2	5	Fresh	Gravel 8; rock 30.
10	E. Paat	22	25	50	15		Clay 10; boulders and granite 35; granite 47.

DOCUMENT MICROFILMING IDENTIFICATION

GEOCREs No. ~~24~~ 41K-35

DIST. 18 REGION

W.P. No. 903-72-07

CONT. No. 76-59

W. O. No.

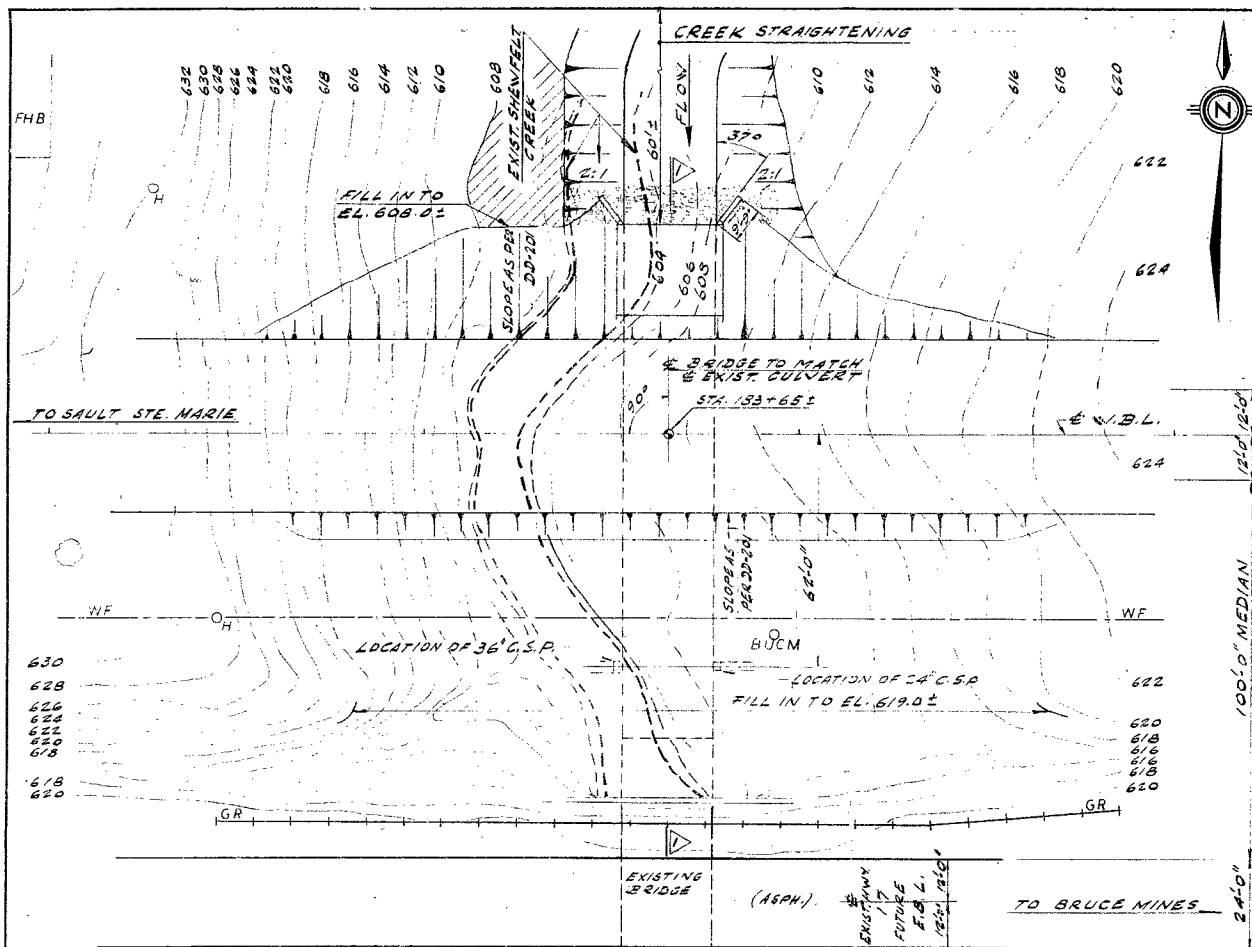
STR. SITE No. 385-285

HWY. No. 17

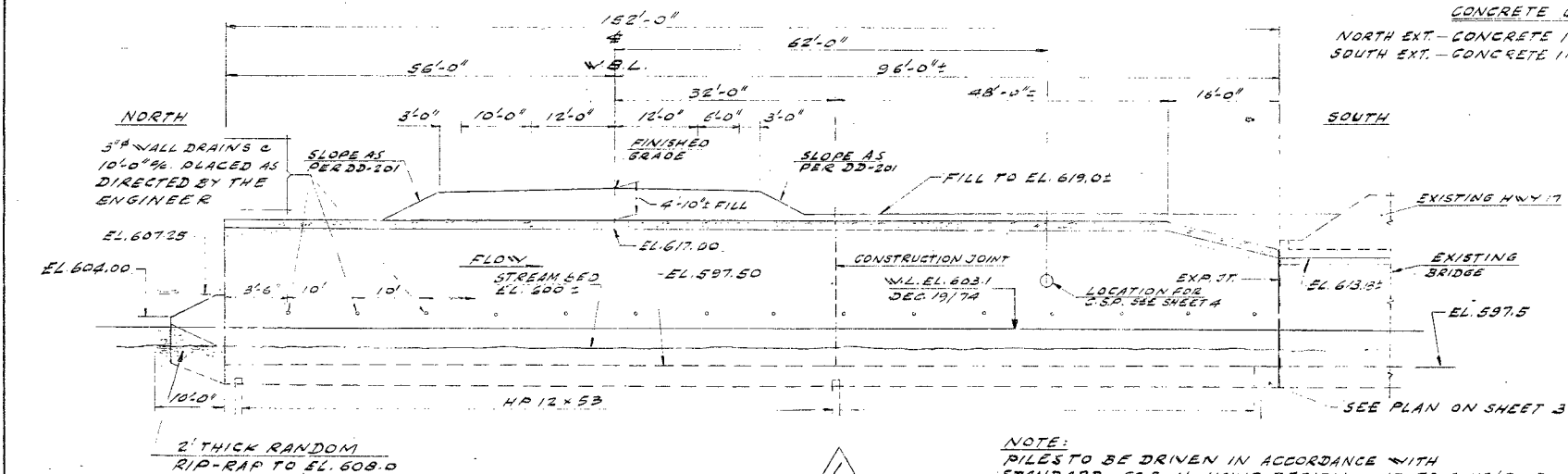
LOCATION Hwy 17 : Shewfelt
Creek Sault Ste. Marie

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 1

REMARKS: ② documents to be unfolded
before microfilm

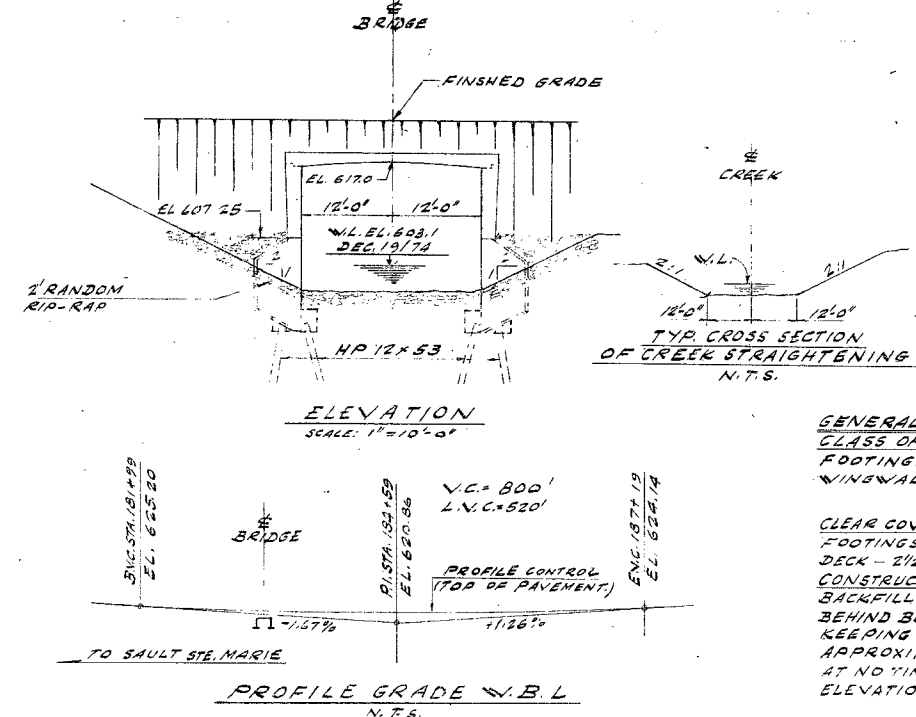


PLAN
SCALE 1"=20'-0"



NOTE:
PILES TO BE DRIVEN IN ACCORDANCE WITH
STANDARD 553-11 USING DESIGN LOAD TO TONS/PILE.

SCALE: 1"=10'-0"

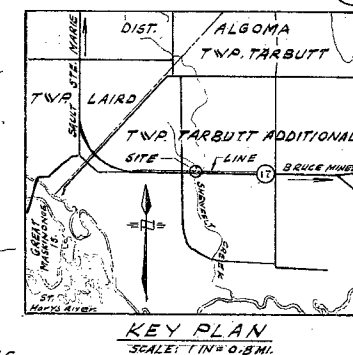


LIST OF DRAWINGS

- SHEET 1 GENERAL LAYOUT I
2 BORE HOLE LOCATIONS AND SOIL STRATA NORTH
3 FOOTING LAYOUT & REINFORCING EXTENSION
4 REINFORCING & DETAILS
SHEET 5 GENERAL LAYOUT II SOUTH
6 REINFORCING & DETAILS EXTENSION
7 STANDARD DETAILS

CONCRETE QUANTITIES

NORTH EXT.-CONCRETE IN BRIDGE - 622 CU. YD.
SOUTH EXT.-CONCRETE IN BRIDGE - 70 CU. YD.



GENERAL NOTES

CLASS OF CONCRETE:
FOOTINGS, ABUTMENTS, DECK AND
WINGWALLS - 3000 P.S.I.

CLEAR COVER TO REINFORCING STEEL:
FOOTINGS, ABUTMENTS & WINGWALLS - 3"
DECK - 2 1/2" TOP, 1 1/2" BOTTOM.
CONSTRUCTION NOTES:
BACKFILL SHALL BE PLACED SIMULTANEOUSLY
BEHIND BOTH ABUTMENTS AND WINGWALLS
KEEPING THE HEIGHT OF THE BACKFILL
APPROXIMATELY THE SAME.
AT NO TIME SHALL THE DIFFERENCE IN
ELEVATIONS BE GREATER THAN 2 FEET.

S.B.M. NO. 3127 EL. 613.043
TABLET IN S. FACE OF S. HEADWALL
152' RT 133+47

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

SHEWELT CREEK BRIDGE
1.5 MI. WEST OF HWY. 548
KING'S HIGHWAY No. 17 DIST. No. 78
60. DIST. ALGOMA
TWP. TARBUTT ADDITIONAL LOT 8 CON. 6

GENERAL LAYOUT I
APPROVED: [Signature] CONTRACT No. [Blank]
DESIGN: A.L. CHECK: W.V. W.P. No. 903-72-07
DRAWING: A.V. CHECK: A.L. DATE: June 75 LOADING: 1520-44 SITE No. 385-285 SHEET 1



FOR REDUCED PLAN
USE SCALE BELOW
10 1 2 3
3 INCHES ON ORIGINAL PLAN

41K-35
GEOCRE 15

RECEIVED
FEB 11 1976
SOL. MECHANICAL
ENGINEERING
CONSULTANTS



Memorandum

To: Mr. B. J. McKenna (2)
Regional Structural Planning Engr.
Northwestern Region
Thunder Bay

From: Soil Mechanics Section
Geotechnical Office
West Building, Downsview

Attention:

Date: April 15, 1975

Our File Ref. W.P. 903-72-07

In Reply to

APR 16 1975

Subject:

FOUNDATION INVESTIGATION REPORT for

Proposed Structure at the Crossing
of Hwy. 17 (W.B.L.) and Shewfelt Creek
Twp. of Tarbutt Additional, Lot 8, Con. 6
District No. 18, Sault Ste. Marie
W.P. 903-72-07 - Site No. 38S-285

Attached we are forwarding to you our detailed Foundation Investigation Report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

M. DEVATA
Supervising Engineer.

c.c. E. J. Orr
B. R. Davis
W. L. Lees
G. E. French
B. J. Giroux
R. Morgenroth
G. A. Wrong
P. Lewycky
McCormick, Rankin & Assoc. Ltd., Attn: J. Sutherns

Files
Record Services

J. Anderson)
N. G. Maluzinsky) memo only

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 - (4.3) CLAY, OCCASIONAL SEAMS OR POCKETS OF SILT
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FOUNDATION INVESTIGATION REPORT

for

Proposed Structure at the Crossing
of Hwy. 17 (W.B.L.) and Shewfelt Creek
Twp. of Tarbutt Additional, Lot 8, Con. 6
District No. 18, Sault Ste. Marie
W.P. 903-72-07 - Site No. 38S-285

1. INTRODUCTION

A request for a foundation investigation at the crossing of proposed W.B.L. Hwy. 17 and Shewfelt Creek was received by this Section in a memorandum dated February 26, 1975 from Mr. H. Dost, Regional Structural Planning Supervisor, Northwestern Region.

A field investigation was subsequently carried out by the Soil Mechanics Section to determine the subsoil and groundwater conditions at the site. This report contains the results of this investigation and our recommendations pertaining to the design of the proposed structure foundations, as well as the approach embankments.

2. DESCRIPTION OF SITE AND GEOLOGY

The site is located in the Twp. of Tarbutt Additional, approximately 1.5 miles west of the Hwy. 17 and Hwy. 548 junction. The general area is utilized basically for agricultural purposes. Shewfelt Creek in this area flows in a north-south direction draining into St. Joseph Channel. The west bank of the creek has slopes of about 3 to 1 and the slopes for the east bank are about 7 to 1 or flatter. The creek valley has a maximum depth of about 30 ft. with a total bank to bank width of about 300 ft. The depth of the water in the creek at the time of the field investigation was about 1.5 ft.

The existing single span, rigid frame, concrete structure, is 24 ft. wide, 14 ft. high and 56 ft. long. According to Drawing No.D2455-1, the present structure is supported on piles and was built around 1937.

The available Drawing does not indicate the type of piles, but it is believed that these piles are timber piles. The existing structure exhibits signs of deterioration, having a badly weathered south face and cracks throughout the exposed faces of the structure. At this stage it was not known whether the existing structure would be replaced by a new one or not. Approximately one mile downstream from this site, the remains of an old collapsed structure was observed at the time of the field investigation. This structure was accommodating traffic for a local rural road. Since the structure was not in service, the local road has been re-routed to avoid this crossing. The remains of this failed structure and debris collected by this might reduce the hydraulic efficiency of Shewfelt Creek in this area.

According to available geological information and observations made in this area, it is inferred that the overburden covering the valley floor is composed of a geologically recent deposit of highly compressible lacustrine clay of variable thickness. The clay is underlain by quartzite and greywacke bedrock of the Lorrain formation of the early Proterozoic Era.

3. FIELD AND LABORATORY WORK

This field investigation consisted of three sampled boreholes, two of which were accompanied by dynamic cone penetration tests. The boreholes were advanced using bombardier mounted auger drill rigs with hollow stems as well as solid augers. Disturbed samples were obtained using a 2-inch O.D. split-spoon sampler driven according to the specifications for the Standard Penetration Test, with a driving energy of 350 ft. lbs. per blow. The same method was used for the dynamic cone penetration tests. "Undisturbed" samples were recovered by using 2-inch I.D. Shelby tubes which were pushed hydraulically into the soil. Field vane tests were carried out to determine the in situ undrained shear strengths of the clay stratum.

Artesian conditions were encountered in the deeper borings, No. 1 & 2, and the flow was sealed off in the respective borings by means of a bentonite seal.

Samples were visually examined in the field and subsequently in the laboratory, and tests were carried out on representative samples to determine the following soil properties:

- Grain-size Distributions
- Atterberg Limits
- Natural Moisture Contents
- Unconfined Shear Strengths
- Consolidation Characteristics
- Bulk Densities
- Organic Contents

The results of field and laboratory tests are summarized in the Record of Borehole sheets, Plasticity Chart (Fig. 1), Grain-size distribution envelope (Fig. 2) and, a shear strength vs. elevation graph (Fig. 3).

All boreholes were surveyed in the field by personnel from District Construction Surveys, Sault Ste. Marie. The elevations are referenced to the Geodetic Datum.

4. SUBSOIL CONDITIONS

(4.1) General

In general, the subsoil at the site consists of a thin layer (6 to 8 inches) of topsoil. On the east bank of the creek, immediately below the topsoil is an 8 to 10 ft. firm to stiff stratum of clayey silt to clay with traces of sand and organics. Directly below this stratum or immediately below the topsoil, is the predominant deposit consisting of a reddish brown clay with occasional seams or pockets of silt. This soft to stiff cohesive stratum extends down to about elevation 542 to 544, some 63 to 68 ft. below ground surface. The clay stratum is underlain by a granular deposit of 12 to 15 ft. thickness, consisting of dense to very dense sand and gravel in the upper half of the deposit, and mainly boulders in the lower half of the deposit.

Bedrock was established only at one location and was found to be at elevation 527.7, some 83 ft. below existing ground surface.

The boundaries between the various deposits, as determined at the boring location, are shown on the accompanying Borehole sheets. The stratigraphical profile, inferred from this data, is shown on Dwg. No.9037207-A.

From ground surface downwards, the various soil types encountered are described as follows:

(4.2) Clayey Silt to Clay

This cohesive deposit was encountered in the borings carried out at the east banks (B.H.'s No. 1 & 2) immediately beneath a thin (6 to 8 inch) layer of topsoil. The thickness varies from 8.5 to 10 ft.

This cohesive material is very similar to the predominant deposit in this area except the stratum contains various amounts of dispersed organic material. The organic content of this material ranges from 0.4 to 1.6 percentage by weight. The 'N' values range from 4 to 9 blows/ft. The undrained shear strength measurements for the in situ field vane tests range from 1280 p.s.f. to greater than 2000 p.s.f. The consistency of this material is estimated to be firm to stiff. Physical properties of this deposit, as determined from field and laboratory tests, are shown on the Record of Borehole sheets, and also on Figs. 1, 2 & 3 found in the Appendix of the report.

(4.3) Clay, Occasional Seams or Pockets of Silt

This is the predominant deposit across the site and was encountered immediately below the topsoil at B.H. 3 and beneath a stratum of clayey silt to clay with organics at B.H. 1 and B.H. 2 in the area of the east bank of the creek. Within this deposit, random seams upto 1/8 inch thick or pockets of silt were also encountered. The thickness of the stratum ranges from 8 to 9.5 ft. for B.H. 1 & 2 respectively.

The engineering properties of this stratum, as determined by the field and laboratory testing, are as follows:

	<u>Range</u>	<u>Average</u>
Bulk Density (γ p.c.f.)	100 - 112	107
Liquid Limit (W_L %)	28 - 82	67
Plastic Limit (W_p %)	18 - 35	25
Natural Moisture Content (W %)	30 - 80	60
Undrained Shear Strength -	<u>Range</u>	<u>Sensitivity</u>
Field Vanes (p.s.f.)	440 - 1520	3 - 13 (Avg. 5.1)
Laboratory Tests (p.s.f.)	150 - 1030	

The Atterberg Limit tests, summarized above, are also plotted on the Plasticity Chart, Fig. 1. These results indicate that the clay is of high plasticity and, in general, inorganic except in the upper portion which contains random organic inclusions. The undrained shear strengths obtained from the laboratory testing gave, for most samples tested, consistently lower values than those obtained from the field vane tests. It is considered that this is primarily due to unavoidable sample disturbance caused by the shipping of the samples, field and laboratory handling, and subsequent testing. In view of this, it is our opinion that the results from the field vane tests are more reliable. A plot of shear strength vs. elevation is shown on Fig. 3

(4.4) Sand Gravel and Boulders

Directly below the cohesive stratum is a granular deposit consisting of sand and gravel. Numerous boulders were found in the lower portion of the deposit at B.H. 2. The thickness of the overall deposit is approximately 15 ft. as seen in B.H. 2. The upper boundary of the deposit ranges from elevation 542.8 to elevation 544.

Standard Penetration Tests of the sand gravel deposit gave 'N' values ranging from 31 to 189 blows per foot, indicating the relative density to be dense to very dense.

(4.5) Bedrock

Bedrock was proven by obtaining 5 ft. of BXL core only at B.H. 2, and found to be at elevation 527.7, 83 ft. below ground level. Rock core samples were examined and identified as sound quartzite without any evidence of weathering.

5. GROUNDWATER CONDITIONS

Groundwater level observations were carried out during the period of the investigation in the open boreholes. The observations are presented on the individual borelog sheets as well as on Drawing No.9037207-A.

An artesian water pressure head was encountered in B.H.'s 1 & 2, which were put down in the valley floor of Shewfelt Creek. When the hollow stem augers were advanced through the upper cohesive stratum into the granular zone, the water rose instantaneously and a flow of 7 to 8 gallons per minute was observed. This artesian condition was encountered at elevation 545 and 542 for B.H.'s 1 & 2 respectively. The artesian head stabilized itself at elevations of 612 and 616 for B.H.'s 1 and 2 respectively, which corresponds to respective heights of 4 ft. and 6 ft. above existing ground level. It is inferred that the sand and gravel stratum in this area is acting as a confined aquifer; this zone is probably being charged with groundwater from the surrounding terrain which is at a higher elevation.

At the time of the field investigation, the ice level elevation of Shewfelt Creek was measured to be 602.9. It is inferred that the groundwater level in the open boreholes, if they were terminated well above the artesian influence, would be close to the creek water level. In B.H. 3, no attempt was made to establish the stabilized water level.

6. DISCUSSION AND RECOMMENDATIONS

(6.1) General

It is proposed to widen the present two lane Hwy. 17 between Desbarats and Sault Ste. Marie to four lanes. The existing Hwy. 17, where it crosses Shewfelt Creek, will be designated as the E.B.L.

structure and in order to accommodate the future four lane traffic in this area, a new W.B.L. structure over Shewfelt Creek will be required some 120 ft. north of the existing bridge.

According to available information, two types of structure are to be considered, namely -

- i) rigid frame single span structure similar to the existing one at the E.B.L. structure;
- ii) corrugated structural plate pipe arch structure.

The proposed grade at the W.B.L. crossing of Hwy. 17 will be about elevation 624. Taking the creek bed at elevation 600, the maximum height of the embankment will be in the order of 24 ft. It is understood that the E.B.L. grade will also be revised and an additional 18 inch raise is proposed in this area.

The presence of an extensive deposit of soft and highly compressible clay at a relatively shallow depth below ground surface, will mean that the stability of the embankment, as well as the settlement induced in the foundation subsoil, will be critical as far as the feasibility of the proposed scheme is concerned. As the stability and settlement of the approach fills are the major problems, these will be discussed first.

(6.2) Stability and Settlement Considerations of the Approaches (W.B.L.)

Stability analyses, in terms of total stresses have been carried out, both in the longitudinal and transverse directions with the following assumptions:

Fill Material (Granular Type) -

Bulk Density	$\gamma = 130$ p.c.f.
Angle of Shearing Resistance	$\phi = 30^{\circ}$
Embankment Slopes	2:1

Foundation Subsoil -

<u>Elevation</u>	<u>γ (p.c.f.)</u>	<u>γ' (p.c.f.)</u>	<u>Cu (p.s.f.)</u>
600 to 590	100	38	600
590 to 570	100	38	700
570 to 560	100	38	800
Water Level Elevation - 600			

The stability computations carried out indicate that the embankment, whose maximum height is 20 ft., will be stable with respect to an overall deep seated rotational type of failure. A mid-height berm of 20 ft. length should be required for an embankment height of 24 ft. Smooth transitions between no berms and 20 ft. berm requirements should be affected as the height of fill varies from 20 to 24 ft. Alternatively, if light weight fill ($\gamma = 90$ p.c.f.), such as the locally available slag from the Algoma Steel Company, is used, embankments to the profile grade (elevation 624) can be constructed with 2:1 side slopes and without any berm.

The underlying highly compressible clay stratum will undergo excessive settlement due to consolidation, over a long-term period, under the weight of the approach embankments. Since the type of structure is not finalized at this stage, it will be difficult to ascertain the induce weight due to embankment loading. However, settlements estimated for typical fill heights are as follows:

<u>Height of Fill</u>	<u>Total Estimated Settlements</u>
14 ft. (granular fill, $\gamma = 130$ p.c.f.)	18 - 22 inches 25 years
20 ft. (lightweight fill, $\gamma = 90$ p.c.f.)	(50% in 1-2 yrs.)
20 ft. (granular fill, $\gamma = 130$ p.c.f.)	24 - 30 inches 25 years
	(50% in 1-2 yrs.)
24 ft. (granular fill, $\gamma = 130$ p.c.f.)	32 - 36 inches 25 years
	(50% in 1-2 yrs.)

It is considered that the estimated settlements may occur at a faster rate than that theoretically computed, because of the probable presence of the occasional permeable silt layers within the cohesive stratum, which would accelerate the drainage in the lateral direction. In view of this, it would therefore be advantageous to construct the embankments first and leave them in place for as long a period as possible, prior to constructing the structure. Consideration should be given to utilizing light weight fill and thus minimize the settlements in comparison to settlements induced by the embankments constructed of locally available earth material.

(6.3) Stability Considerations - Existing Hwy. 17 (E.B.L.)

A grade revision may be required in the vicinity of the present Shewfelt Creek structure crossing. According to available information from the Regional Systems Design Section, the existing fills within the structure confines will be excavated to some 5 to 6 ft. below the present grade, and replaced with lightweight fill material such as slag, to the revised profile grade (elevation 621.5). As previously mentioned, this requires only 18 inches increase in the height of the embankments in this area. Since the lightweight fill is only 90 p.c.f., the induced loads on the subsoil will be less than the existing condition. If the sequence of construction as mentioned previously is followed, no stability and no further settlement problems are anticipated.

(6.4) Structure Foundations

It is understood that two types of structure may be considered and therefore our comments for each type of structure is as follows:

(6.4.1) Rigid Frame Single Span Structure

A rigid frame single span structure could be supported on end bearing piles driven to refusal in the lower portion of the sand and gravel where boulders are present. For estimating purposes, it can be assumed that the pile tips will be located at about elevations 538 to 535. The allowable pile load would be dependent on the section chosen. The underlying cohesive subsoil will settle due to the embankment load; therefore, some negative skin friction loads may be imposed on the piles supporting the foundations. The negative skin friction component should be allowed for by employing a design value which is 85 percent of the allowable structural capacity of the pile section. For instance, 12BP74 steel H-piles should be designed for 80 tons/pile, rather than the 95 tons/pile usually employed. No bouldery or rock fill should be placed in areas where piles are to be driven. Pile caps for the abutments should be at sufficient depth below finished grade so as to ensure adequate frost protection.

The foundation excavations may extend below the creek water level. Since the cohesive subsoil is relatively impervious, no major dewatering problems are anticipated. Any seepage into the excavations could be controlled by using conventional techniques such as pumping from sumps, etc. If the excavations for the structure foundations are located within the confines of the existing channel, a temporary creek diversion may be necessary.

If the structure is designed as rigid frame, then a coefficient of earth pressure at rest (K_0) of 0.5 should be assumed for the granular material placed behind the wall when designing the wall sections. However, if some movement of the top of the wall is permitted, then a coefficient of active earth pressure (K_a) of 0.33 can be used. In all cases, the design should incorporate the full effect of the surcharge located above the walls.

In order to relieve the build up of excess hydrostatic pressure behind the walls, suitable drainage measures should be provided. Weep holes, located at the base of the walls, could be employed for this purpose.

Protective measures against scour, due to river erosion, should be provided. Recommendations pertaining to this aspect should be obtained from the Hydrology Section.

(6.4.2) Corrugated Structural Plate Pipe Arch Structure

As an alternative, a corrugated structural plate pipe arch structure could be employed at this site. No major complications are envisaged with regard to the placement of the structure, provided the bedding and backfilling for the pipe arch will be carried out in accordance with current M.T.C. practices. The pertinent Standard is No.DD808-B (Type 4A). It is recommended that the structural pipe arch be placed on a mat of granular material with a total thickness of 24 inches, in order to distribute the critical corner bearing pressure,

so that the stress increase induced in the cohesive foundation subsoil will not exceed the allowable bearing capacity of this stratum.

The pipe arch will settle differentially due to the consolidation of the underlying cohesive stratum by the embankment loading. In order to allow for this settlement, it is recommended that the structure be cambered. The camber required should be based on the embankment loading, as well as the type of fill material. This aspect can be further discussed once the type of scheme is finalised.

The excavation for the pipe arch may extend below the creek water level. Dewatering considerations will be similar to those discussed in subsection (6.4.1).

It should be noted that, if acceptable earth fill material is used for embankment construction in this area, the proposed pipe arch structure should be of sufficient length to accommodate any berm requirements in the transverse direction.

7. MISCELLANEOUS

The field work for this investigation was carried out during the period of March 17 1975 to March 22, 1975, under the supervision of Mr. M. MacLean, Project Engineer and Mr. C. McKercher, Student Field Technician.

The equipment used for subsoil sampling was owned and operated by Master Soil Investigation Ltd.

This report was written by Mr. C. McKercher, and Mr. H. Shah, Project Engineer, and was reviewed by Mr. M. Devata, Supervising Engineer.

H. Shah

H. SHAH
Project Engineer.

M. Devata

M. DEVATA
Supervising Engineer.

APPENDIX

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 1

W.P. 903-72-07

LOCATION STA: 183+52 30' RT OF C. W.B.L. OF HWY. 17

ORIGINATED BY MM

DIST. 18 HWY. 17

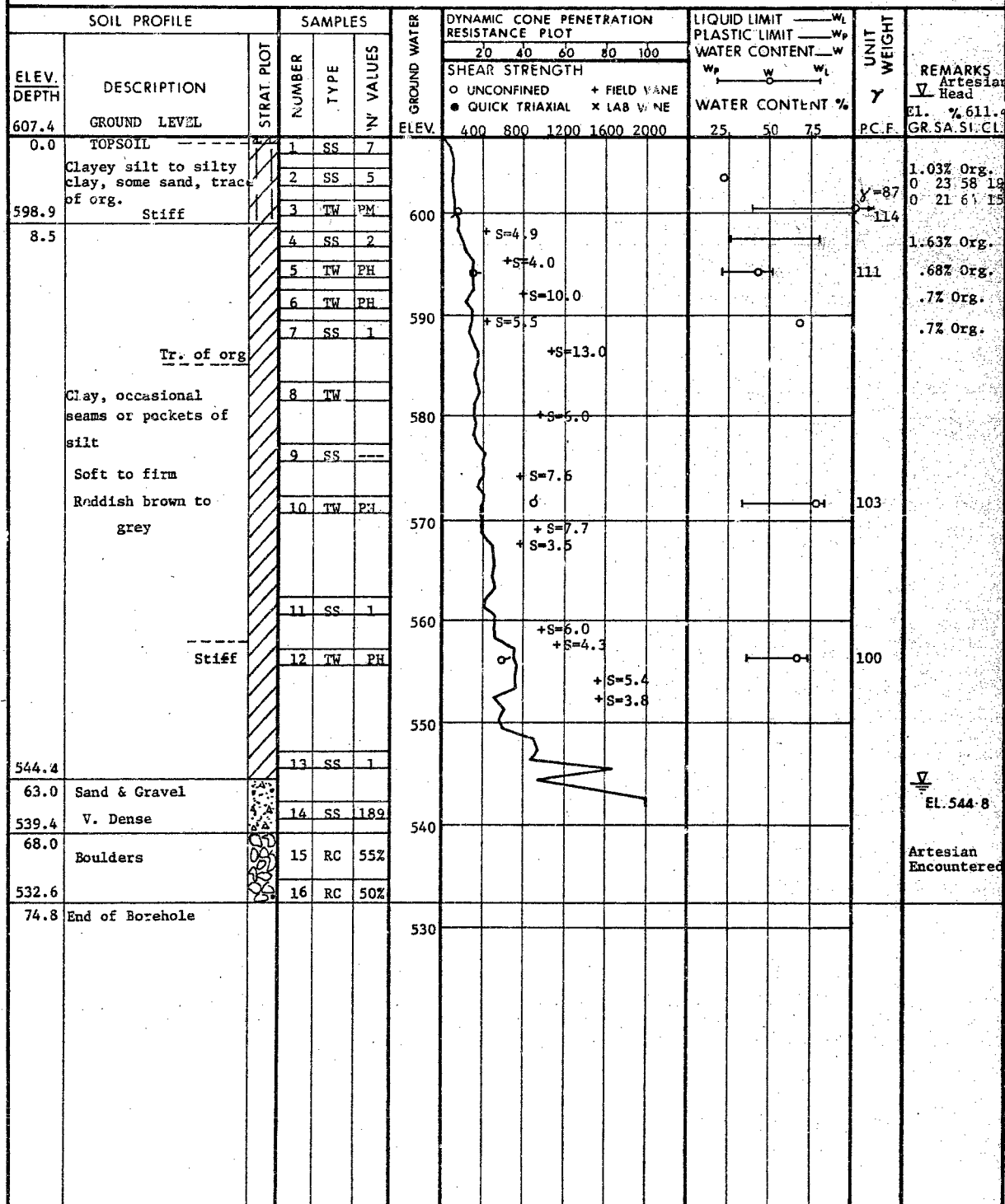
BORING DATE MARCH 17th and 18th, 1975

COMPILED BY S.O.

DATUM GEODETIC

BOREHOLE TYPE AUGER & CONE TEST

CHECKED BY So



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 2

W.P. 903-72-07 LOCATION STA. 183+82 30' LT OF C W.B.L. HWY. 17 ORIGINATED BY C.Mc
DIST 18 HWY. 17 BORING DATE MARCH 18, 1975 COMPILED BY S.O.
DATUM GEODETIC BOREHOLE TYPE AUGER AND SAMPLE WITH CME 55 MACHINE CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W _t			UNIT WEIGHT γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	W _p	W	W _L		
610.7	GROUND LEVEL															artesian head
0.0	TOPSOIL		1	SS	4	610										EL/ 616.2
	Clayey Silt to Silty clay, traces of Sand, org. Firm to stiff Red Br. to Grey		2	TW	PH											1.6% Org.
600.7			3	SS	9											0 3' 67 30
10.0			4	TW	PH											0.4% Org.
			5	SS	2											
			6	TW	PH											
	Clay, occasional seams or pockets of Silt		7	SS	1											
	Soft to Firm		8	SS	1											
	Reddish Brown to Grey		9	TW	PH											
			10	SS	1/18'											
			11	TW	PM											
			12	SS	1/18'											
			13	TW	PM											
		Stiff	14	SS	3/18'											
			15	TW	PH											
542.8			16	SS	31											El. 542.2
67.9	Sand & Gravel Dense		17	SS	44											Artesian Encountered
	Boulders															
527.7																
83.0	Quartzite Bedrock		18	BXL	100%											
522.7																
88.0	End of Borehole															

RECORD OF BOREHOLE NO 3

W.P. 903-72-07

LOCATION STA: 182+92 C W.B.L. HWY. 17

ORIGINATED BY C. McK.

DIST. 18 HWY. 17

BORING DATE MARCH 19, 1975

COMPILED BY C. McK.

DATUM GEODETIC

BOREHOLE TYPE AUGER AND SAMPLE WITH CME 45 MACHINE

CHECKED BY

SOIL PROFILE		STRAT. PLOT	SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE	N' VALUES		20	40	60	80	100	SHEAR STRENGTH					
												O UNCONFINED + FIELD VANE					
												● QUICK TRIAXIAL x LAB VANE					
						WATER CONTENT %					w_p w w_L						
612.2	GROUND LEVEL						400	800	1200	1600	2000	25	50	75			
0.0	Topsoil -----		1	SS	6												
	Clay, occasional Pockets of Silt		2	TW	PH												
	Soft to Firm Reddish Brown to Grey		3	SS	3												
			4	TW	PM												
			5	SS	3												
			6	TW	PM												
			7	SS	1/18"												
577.7																	
34.5	End of Borehole																

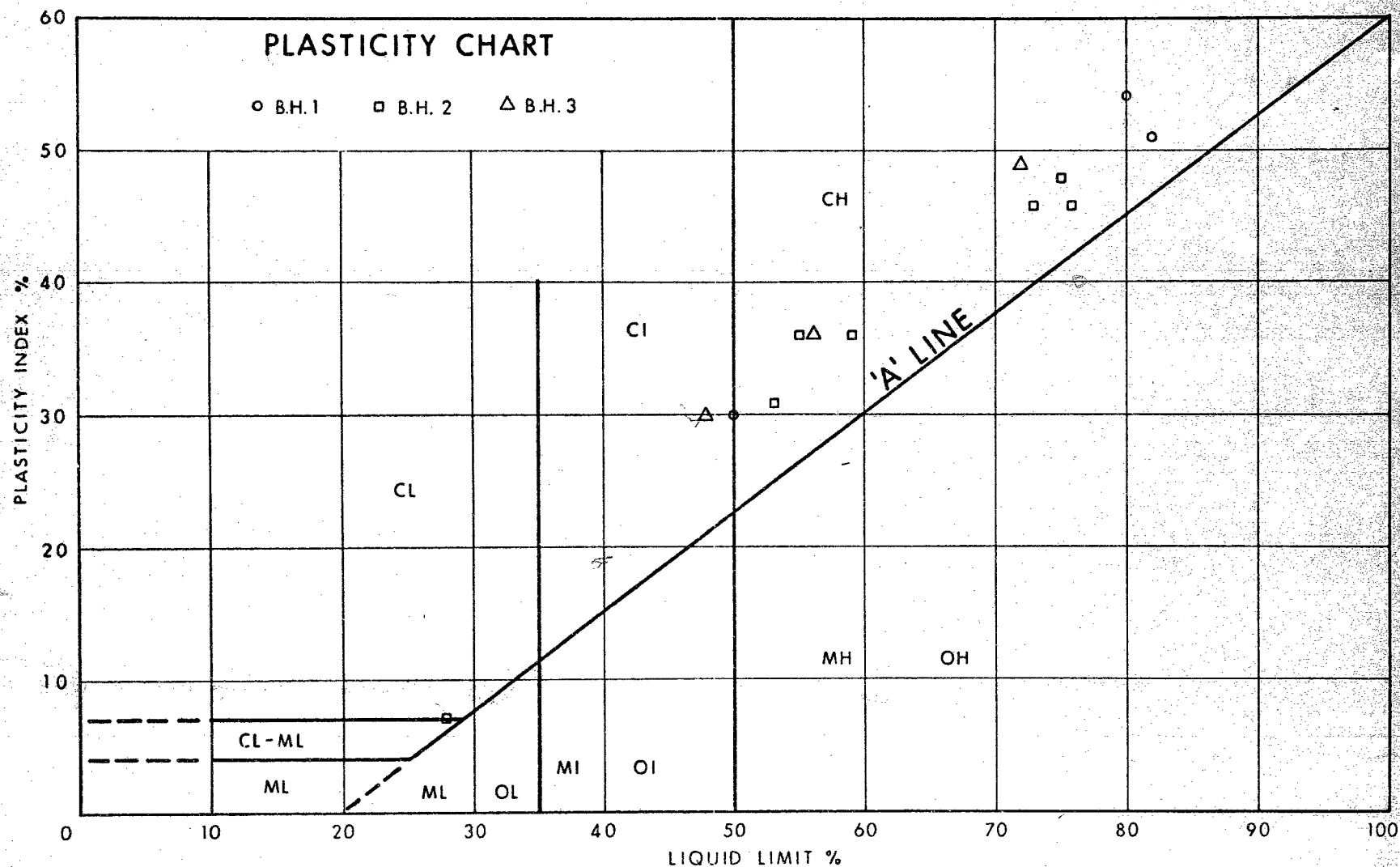


FIG. 1

GRAIN SIZE DISTRIBUTION

UNIFIED SOIL CLASSIFICATION SYSTEM

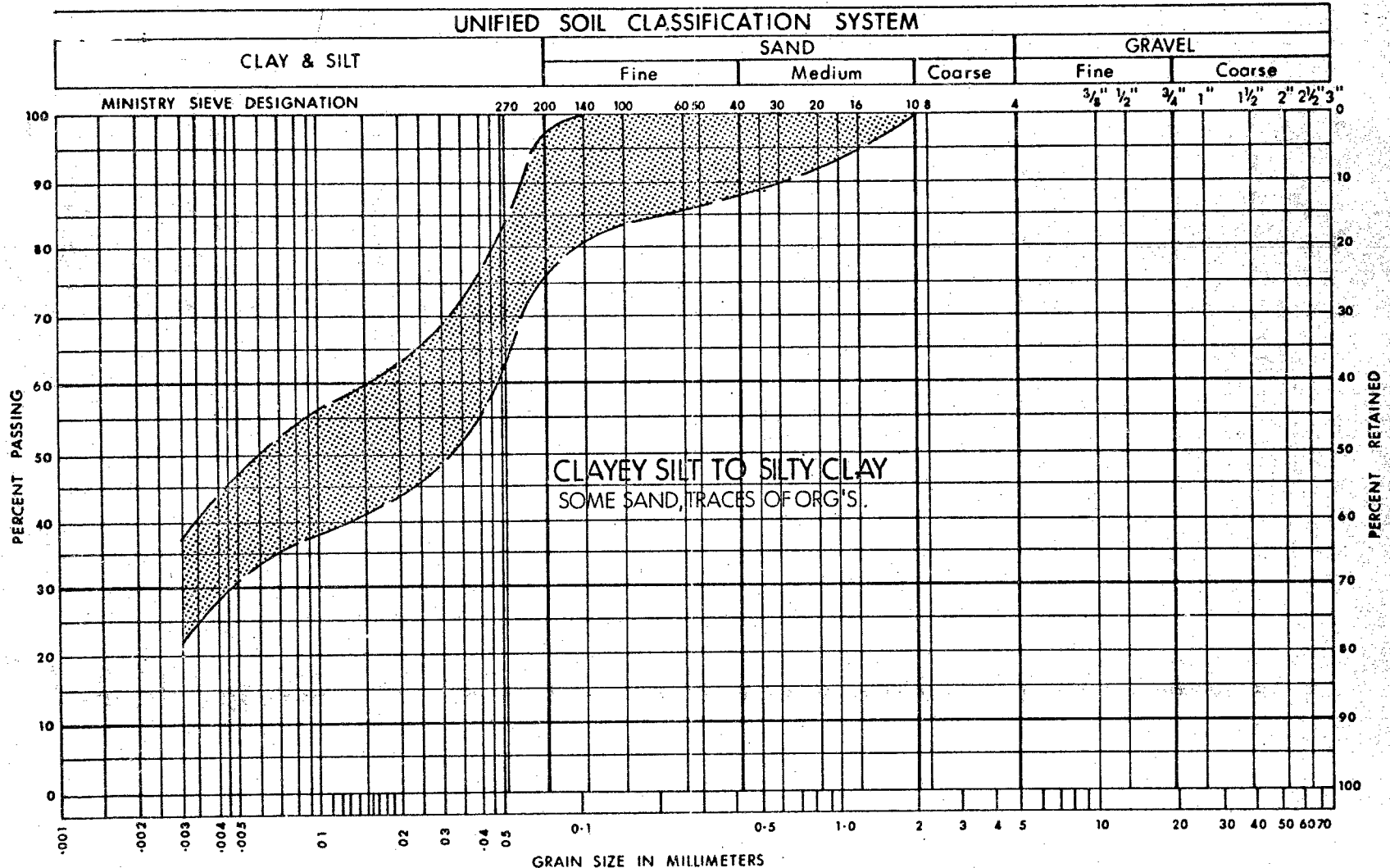


FIG. 2

SHEAR STRENGTH Vs ELEVATION

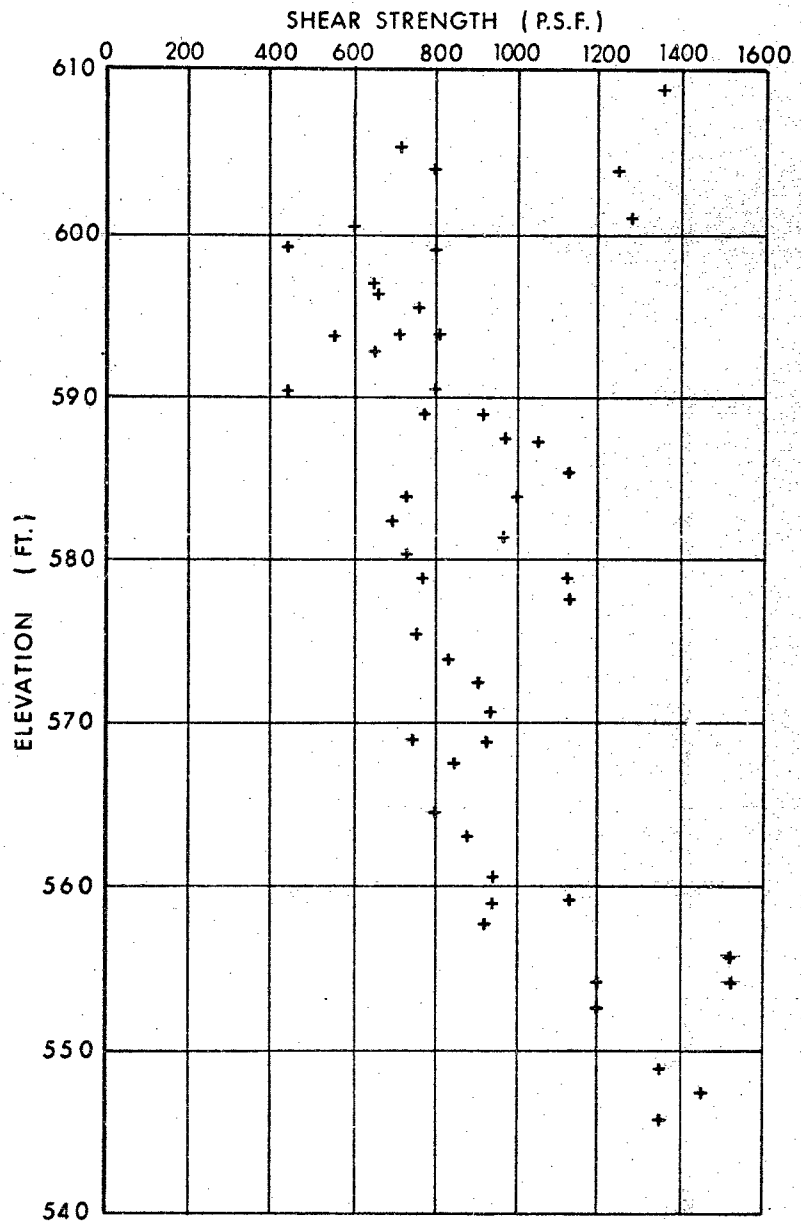


FIG. 3

W.P. 903-72-07

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTPENETRATION RESISTANCE

'N'-STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>c LB./SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
w_s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

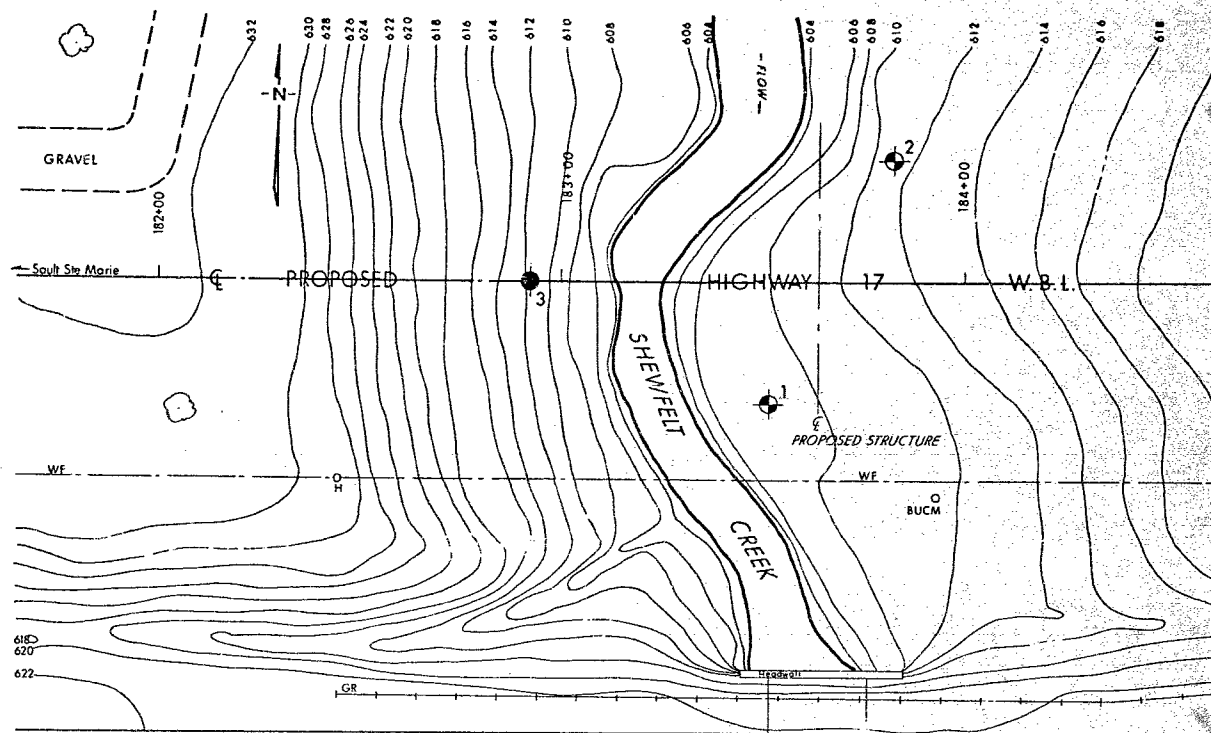
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

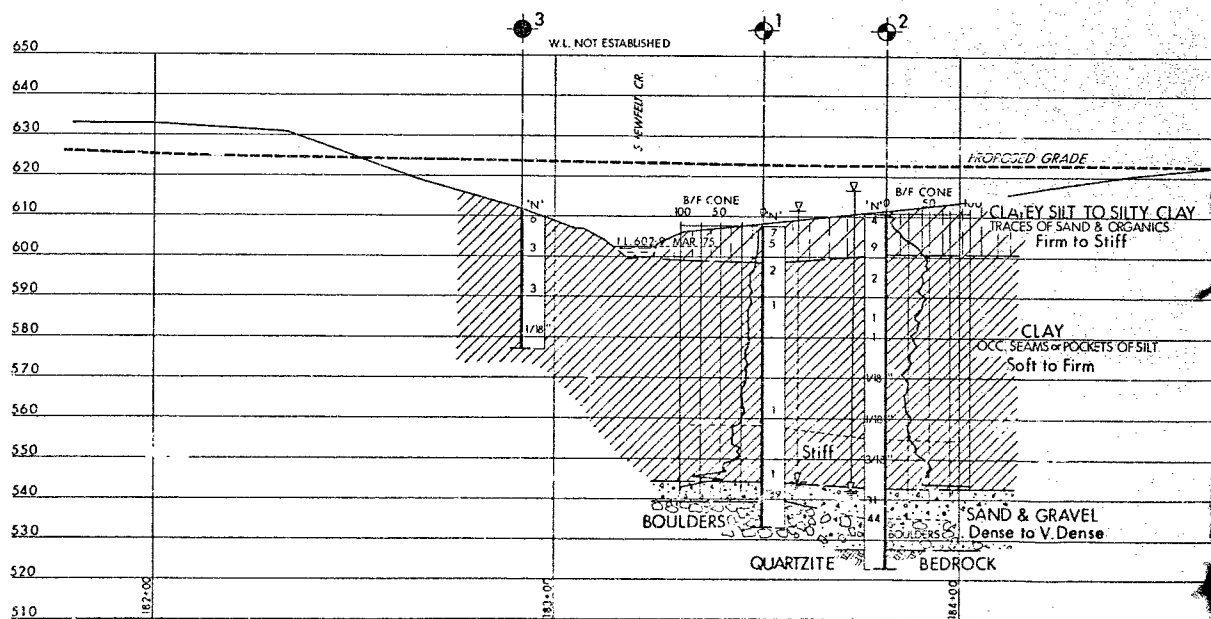
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



EXISTING HIGHWAY 17

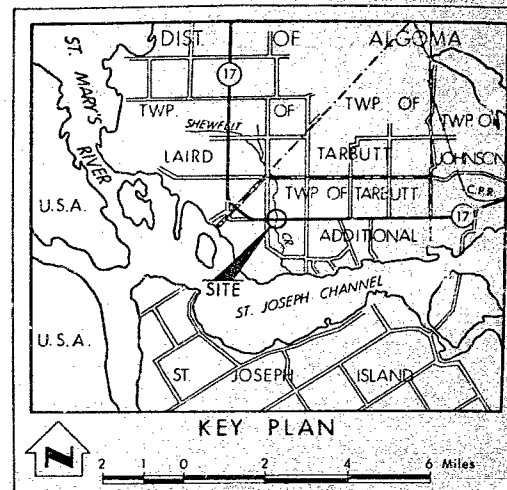
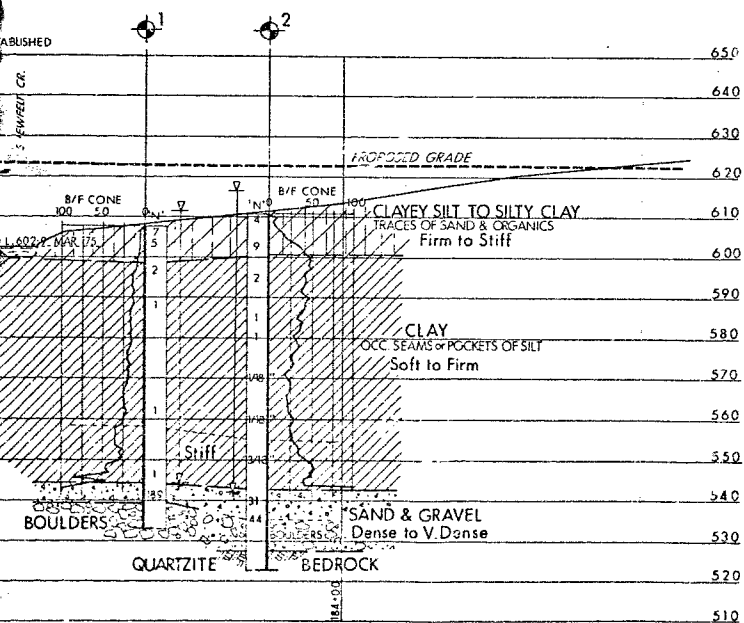
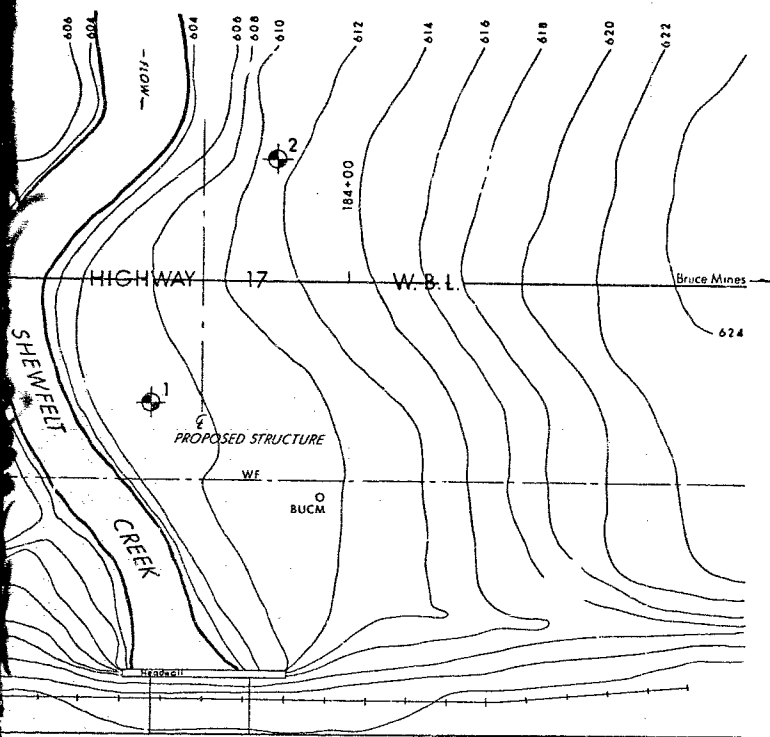
PLAN

20 10 0 SCALE 20 40 FT



Q PROFILE

20 10 0 SCALE 20 40 FT



LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Resistance Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation, MAR. 1975		
	Head Encountered		
	ARTESIAN CONDITION		
NO.	ELEVATION	STATION	OFFSET
1	607.4	183+52	30' RT.
2	610.7	183+82	30' LT.
3	612.2	182+92	CL

NOTE

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

NOTE FOR CONTRACT DOCUMENT

The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the Sault Ste Marie District Office.

DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
ENGINEERING SERVICES BRANCH—GEOTECHNICAL OFFICE—SOIL MECHANICS SECTION

SHEWELT CREEK

HIGHWAY NO 17 PROP. W.B.L. DIST. NO 18

DIST. OF ALGOMA

TWP TARBUTT ADDITIONAL LOT 8 CON 6

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD H S. CHECKED	WP NO 903-72-07	DRAWING NO.
DRAWN S O. CHECKED	WD NO	9037207-A
DATE 9 APR 1975	SITE NO 385-285	BRIDGE DRAWING NO.
APPROVED	CONT NO	



BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR Bot Construction DESIGN LOAD OF PILE 70 tons
HAMMER DETAILS: TYPE BIRMINGHAM DRIVING WEIGHT 3000 HEIGHT OF FALL OR ENERGY 25000
TYPE OF ANVIL OR CAP H-Linet (Steel) WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS H' PILE (STEEL) @ 53 lbs/ft BATTER: 1:4
PILE NO. 42 LOCATION N.E. FTg. DATE DRIVEN Oct 18/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
40'-1"	1		40'-1"	26		86'-7"	51	20		76	
	2			27			52			77	
	3			28			53			78	
	4			29			54			79	
	5			30			55			80	
	6			31			56			81	
	7			32			57			82	
	8			33			58			83	
	9			34			59			84	
	10			35		86'-7"	60	20		85	
	11			36			61	24		86	
	12			37			62	24		87	
	13			38			63	24		88	
	14			39			64	24		89	
	15		86'-7"	40			65	24		90	
	16			41			66	80		91	
	17			42			67	100		92	
	18			43			68			93	
	19			44			69			94	
	20			45			70			95	
	21			46			71			96	
	22			47			72			97	
	23			48			73			98	
	24			49			74			99	
40'-1"	25		86'-7"	50	20		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	10	6	5	4	4	4
MEASURED REBOUND IN INCHES	0.3	0.3	0.3	0.3	0.3	0.3
FINAL LENGTH OF PILE	86'-7" 67'-8" FINAL CUT OFF ELEVATION 595.5					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED R.H. Beeher
NAME (PRINT) R.H. BEEHER
DATE Oct 18/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

File Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BERMINGHAM FOR B&B CONST. DESIGN LOAD OF PILE 70 Tons
HAMMER DETAILS: TYPE DIESEL B225 BERMINGHAM WEIGHT 3000 HEIGHT OF FALL OR ENERGY 25000
TYPE OF ANVIL OR CAP HELMUT (STEEL) WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS STEEL 'H' @ 53 lbs Lin. FT. BATTER: 1:6
PILE NO. 2 LOCATION S.W. FTg. DATE DRIVEN Nov. 9/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
33'-8"	1		33'-8"	26	1	79'-2"	51	18		76	
	2			27	1		52	18		77	
	3			28	1		53	18		78	
	4			29	1		54	18		79	
	5		79'-2"	30	1		55	18		80	
	6			31	1		56	21		81	
	7			32	1		57	21		82	
	8			33	1		58	24		83	
	9			34	1		59	24		84	
	10			35	1		60	24		85	
	11		79'-2"	36	2		61	20		86	
	12			37	2		62	20		87	
	13			38	2		63	24		88	
	14			39	2		64	24		89	
	15			40	2		65	36		90	
	16			41	2		66	36		91	
	17			42	2		67			92	
	18			43	2		68			93	
	19			44	2		69			94	
	20			45	3		70			95	
	21			46	3		71			96	
	22			47	3		72			97	
	23			48	3		73			98	
	24			49	18		74			99	
33'-8"	25		79'-2"	50	18		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	11	9	8	3	3	3
MEASURED REBOUND IN INCHES	5/8	5/8	5/8	5/8	5/8	1/2
FINAL LENGTH OF PILE	67'-3"					FINAL CUT OFF ELEVATION 595.50

REPORT TO BE SENT TO: -

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED CLINT REBOWSON
Per R H Bremer
NAME (PRINT) R H BREMER
DATE Nov. 26/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BERMINGHAM FOR BOT CONST DESIGN LOAD OF PILE 70 Ton
HAMMER DETAILS: TYPE DIESEL B 225 BERMINGHAMER WEIGHT 3000 HEIGHT OF FALL OR ENERGY 25000
TYPE OF ANVIL OR CAP HELMUT (STEEL) WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS STEEL 'H' @ 53 lbs/ft BATTER: VERTICAL
PILE NO. 6 LOCATION S.E. FTg. DATE DRIVEN Nov. 3/76

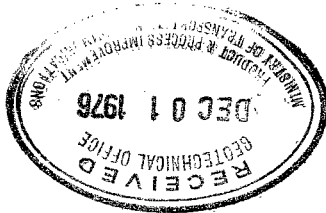
TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
60'-5"	1		60'-5"	26		60'-5"	51	16		76	
	2			27			52	16		77	
	3			28			53	16		78	
	4			29			54	16		79	
	5			30		60'-5"	55	12		80	
	6			31		72-10	56	12		81	
	7			32			57	12		82	
	8			33			58	12		83	
	9			34			59	12		84	
	10			35			60	12		85	
	11		60'-5"	36	2		61	12		86	
	12			37	2		62	12		87	
	13			38	2		63	12		88	
	14			39	2		64	16		89	
	15			40	2		65	49		90	
	16			41	2	72'-10"	66	175		91	
	17			42	4		67			92	
	18			43	4		68			93	
	19			44	4		69			94	
	20			45	4		70			95	
	21			46	4		71			96	
	22			47	4		72			97	
	23			48	8		73			98	
	24			49	3		74			99	
60'-5"	25		60'-5"	50	16		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	20	12	12	8	8	4
MEASURED REBOUND IN INCHES	5/8	5/8	5/8	5/8	5/8	5/8
FINAL LENGTH OF PILE	66'-2" FINAL CUT OFF ELEVATION 595.50					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED CLINT Robinson
Per R.H. Beemer
NAME (PRINT) R.H. BEEMER
DATE Nov. 26/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM



NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BERMINGHAM FOR POT CONST. DESIGN LOAD OF PILE 70 Tons
HAMMER DETAILS: TYPE BERMINGHAMER DIESEL B225 WEIGHT 3000 HEIGHT OF FALL OR ENERGY 25000
TYPE OF ANVIL OR CAP HELMET (STEEL) WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS STEEL 'H' PILE 53 lbs/ft BATTER: 1:6
PILE NO. 39 LOCATION N.W. F79. DATE DRIVEN Oct 28/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
40'-6"	1		40'-6"	26	1	86'-11"	51	5		76	
	2			27	1		52	5		77	
	3			28			53	13		78	
	4			29			54	13		79	
	5			30			55	13		80	
	6			31			56	14		81	
	7			32			57	14		82	
	8			33			58	20		83	
	9			34			59	20		84	
	10			35			60	20		85	
	11			36	2		61	20		86	
	12			37	2		62	20		87	
	13			38	2		63	28		88	
	14			39	2		64	28		89	
	15			40	2		65	28		90	
	16			41	2		66	28		91	
	17			42	2		67	28		92	
	18			43	2		68	28		93	
	19			44	2		69	28		94	
	20			45	4		70	28		95	
	21			46	4		71	156 276		96	
	22			47	4		72			97	
	23			48	4		73			98	
	24			49	4		74			99	
40'-6"	25		86'-11"	50	5		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	18	18	18	18	10	8
MEASURED REBOUND IN INCHES	3/4	3/4	3/4	3/4	3/4	3/4
FINAL LENGTH OF PILE	71'-11"					FINAL CUT OFF ELEVATION 595.50

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED R.H. Beemer
NAME (PRINT) R.H. BEEMER
DATE OCT. 28/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BERMINGHAM FOR BCT ENT DESIGN LOAD OF PILE 70 Ton
HAMMER DETAILS: TYPE BERMINGHAMER WEIGHT 3000 HEIGHT OF FALL OR ENERGY 1100
TYPE OF ANVIL OR CAP STEEL HELMET WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS STEEL 'H' PILE @ 53 lbs/ft BATTER: 1:6
PILE NO. 24 LOCATION N.W. FTG. DATE DRIVEN Nov. 2/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
40'-7"	1		40'-7"	26		61'-9"	51	10		76	
	2			27			52	10		77	
	3			28			53	10		78	
	4			29			54	10		79	
	5			30			55	10		80	
	6			31			56	10		81	
	7			32			57	10		82	
	8			33			58	16		83	
	9			34			59	22		84	
	10			35			60	48		85	
	11		61'-9"	36	4		61	60		86	
	12			37	4		62	60		87	
	13			38	4		63	60		88	
	14			39	4		64	80		89	
	15			40	4		65	84		90	
	16			41	4		66	88		91	
	17			42	4		67	90		92	
	18			43	4		68			93	
	19			44	4		69			94	
	20			45	6		70			95	
	21			46	6		71			96	
	22			47	6		72			97	
	23			48	6		73			98	
	24			49	6		74			99	
40'-7"	25		61'-9"	50	6		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION

BLOWS PER INCH

1 2 3 4 5 6
9 8 8 8 8 8

MEASURED REBOUND IN INCHES

FINAL LENGTH OF PILE

67'-10"

FINAL CUT OFF ELEVATION

595.50

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE

ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED

R. H. BEEHER

NAME (PRINT)

R. H. BEEHER

DATE

NOV. 2/76

ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BIRMINGHAM FOR BOT CONST DESIGN LOAD OF PILE 70 Ton
HAMMER DETAILS: TYPE BIRMINGHAM DIESEL B225 WEIGHT 3000 HEIGHT OF FALL OR ENERGY 25000
TYPE OF ANVIL OR CAP STEEL HELMET WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS STEEL 'H' PILE @ 53 lbs/ft BATTER: 1:6
PILE NO. 25 LOCATION N.W. FTG. DATE DRIVEN Nov. 7/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
40'-6"	1		40'-6"	26		86'-6"	51	14		76	
	2			27			52	14		77	
	3			28			53	14		78	
	4			29			54	25		79	
	5			30			55	25		80	
	6			31			56	18		81	
	7			32			57	18		82	
	8			33			58	18		83	
	9			34			59	18		84	
	10			35			60	20		85	
	11		86'-6"	36	3		61	20		86	
	12			37	3		62	20		87	
	13			38	3		63	24		88	
	14			39	3		64	100		89	
	15			40	5		65	200		90	
	16			41	5		66			91	
	17			42	5		67			92	
	18			43	5		68			93	
	19			44	5		69			94	
	20			45	10		70			95	
	21			46	10		71			96	
	22			47	10		72			97	
	23			48	10		73			98	
	24			49	10		74			99	
40'-6"	25		86'-6"	50	14		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	20	10	10	10	10	10
MEASURED REBOUND IN INCHES	1/2"	1/2"	1/2"	1/2"	1/2"	1/2"
FINAL LENGTH OF PILE	65'-4"			FINAL CUT OFF ELEVATION 595.50		

REPORT TO BE SENT TO: -

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED R.H. Beemer
NAME (PRINT) R.H. BEEMER
DATE Nov. 2/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BOT Construction DESIGN LOAD OF PILE 70 TONS
HAMMER DETAILS: TYPE BERMINGHAM HAMMER WEIGHT 3000 lbs HEIGHT OF FALL OR ENERGY 25000
TYPE OF ANVIL OR CAP HELMET (STEEL) WEIGHT OF ANVIL OR CAP 1100 lbs
PILE DETAILS 4" P.I.E. (STEEL) BATTER: 1:4
PILE NO. 39 LOCATION N.E. Fly DATE DRIVEN OCT. 16/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
40-7	1		47-7	26		87-2	51	20		76	
	2			27			52	20		77	
	3			28			53	20		78	
	4			29			54	20		79	
	5			30			55	20		80	
	6			31			56	20		81	
	7			32			57	20		82	
	8			33			58	20		83	
	9			34			59	20		84	
	10			35			60	20		85	
	11			36			61	20		86	
	12			37			62	20		87	
	13			38			63	20		88	
	14			39			64	24		89	
	15			40			65			90	
	16			41			66			91	
	17			42			67			92	
	18			43			68			93	
	19			44			69			94	
	20			45			70			95	
	21			46			71			96	
	22			47			72			97	
	23			48			73			98	
	24			49			74			99	
	25			50			75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	9	8	6	4	4	4
MEASURED REBOUND IN INCHES	.3	.3	.3	.3	.3	.3
FINAL LENGTH OF PILE	64'-2"					FINAL CUT OFF ELEVATION 595'-5"

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED R. H. Beemer
NAME (PRINT) R. H. BEEMER
DATE OCT. 16/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.





BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BET CONSTRUCTION DESIGN LOAD OF PILE 70 TONS
HAMMER DETAILS: TYPE DIESEL B225 WEIGHT 3000 HEIGHT OF FALL OR ENERGY 25000
TYPE OF ANVIL OR CAP HELMET (STEEL) WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS 'H' PILE STEEL @ 53 lbs/ft BATTER: 1:4
PILE NO. 25 LOCATION N.E. FTG. DATE DRIVEN OCT 25/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
40'-0"	1		40'-0"	26	1	59'-4"	51	16		76	
	2			27	1		52	16		77	
	3			28	1		53	16		78	
	4			29	1		54	16		79	
	5			30	1		55	16		80	
	6			31	1	80'-0"	56	16		81	
	7			32	1		57	16		82	
	8			33	1		58	16		83	
	9			34	1		59	16		84	
	10			35	1		60	16		85	
	11			36	2		61	16		86	
	12			37	2		62	18		87	
	13			38	2		63	18		88	
	14			39	2		64	12		89	
	15			40	3		65	12		90	
	16			41	3		66	12		91	
	17			42	3		67	12		92	
	18			43	3		68	14		93	
	19			44	3		69	14		94	
	20			45	5	80'-0"	70	106		95	
	21			46	5		71			96	
	22			47	5		72			97	
	23			48	5		73			98	
	24			49	5		74			99	
40'-0"	25		59'-4"	50	16		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	12	8	5	5	5	5
MEASURED REBOUND IN INCHES	0.3	0.3	0.3	0.3	0.3	0.3
FINAL LENGTH OF PILE	70'-0"					FINAL CUT OFF ELEVATION
						595.5

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED R.H. Beemer
NAME (PRINT) R.H. BEEMER
DATE OCT. 25/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM

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Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.





BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 76-59 STRUCTURE W.P. NO. 903-72-07
CONTRACTOR BOT Construction DESIGN LOAD OF PILE 70
HAMMER DETAILS: TYPE DIESEL BIRMINGHAMMER 3225 WEIGHT 3000 ^{lbs} HEIGHT OF FALL OR ENERGY 25000 ^{FT/LB}
TYPE OF ANVIL OR CAP STEEL WEIGHT OF ANVIL OR CAP 1100
PILE DETAILS H' STEEL @ 53 lbs/FT BATTER: 1:6
PILE NO. 6 LOCATION N.E. F Ty. RT of E DATE DRIVEN OCT-14/76

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
<u>40-7'</u>	1	<u>WEIGHT OF HAMMER</u>	<u>40-7'</u>	26	<u>WEIGHT OF HAMMER</u>	<u>87</u>	51	2		76	
	2			27			52	2		77	
	3			28			53	3		78	
	4			29			54	3		79	
	5			30			55	3		80	
	6			31			56	4		81	
	7			32			57	6		82	
	8			33			58	7		83	
	9			34			59	10		84	
	10			35			60	10		85	
	11			36			61	10		86	
	12		<u>WELD.</u>	37	<u>4</u>		62	10		87	
	13		<u>87-0"</u>	38	2		63	<u>12</u>		88	
	14			39	2	<u>87</u>	64	<u>240</u>		89	
	15			40	2		65			90	
	16			41	2		66			91	
	17			42	2		67			92	
	18			43	2		68			93	
	19			44	2		69			94	
	20			45	2		70			95	
	21			46	2		71			96	
	22			47	2		72			97	
	23			48	2		73			98	
	24			49	2		74			99	
	25			50	2		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	<u>20</u> <u>240</u>	<u>10</u> <u>120</u>	<u>12</u> <u>80</u>	<u>12</u> <u>80</u>	<u>5</u> <u>60</u>	<u>5</u> <u>60</u>
MEASURED REBOUND IN INCHES	.2	.2	.3	.3	.4	.4
FINAL LENGTH OF PILE	<u>64'</u>					FINAL CUT OFF ELEVATION <u>595.5</u>

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,
DOWNSVIEW, ONTARIO

SIGNED R. H. Beemer
NAME (PRINT) R. H. BEEMER
DATE OCT-14/76
ATTACH SKETCH OF PILE NUMBERING SYSTEM

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Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



Soil Mechanics Section
Geotechnical Office
West Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Tel: (416) 248-3282

March 17, 1975

Master Soil Investigation,
104 Kenhar Drive,
Weston, Ontario.
M9L 1N4

Dear Sirs:

This letter confirms our request by telephone of 10th March 1975, for the supply of a Type I Auger drilling machine (M.V. mounted), (Item No. 5.1.1), together with all necessary equipment, as per your Tender for Supply Contract S-74-2110, at Echo Bay, Ontario, on March 17th 1975.

Mobilization will be from our project W.P. 903-72-03.

Our Project Number is W.P. 903-72-07.

Yours truly,


M. DEVATA
Supervising Engineer.

c.c. W. W. Fry
(Attn: Mrs. M. Porter)
Files (?)
Record Services



Memorandum

To: Mr. B. J. McKenna,
Reg. Structural Planning Engineer,
Northwestern Region,
Thunder Bay.

From: Structural Office,
West Building, Downsview.

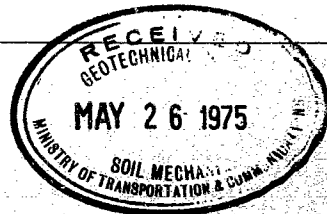
Attention:

Date: May 23, 1975.

Our File Ref.

In Reply to

Subject: Shewfelt Creek Structure,
W. P. 903-72-07, Site 38S-285,
Highway 17, District 18.



Attached herewith are prints of the Preliminary Bridge Plan Drawing 38S-285-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$250,000.00 which includes tender, materials, engineering and sundry construction.

We have sent a copy of the Preliminary Plan to the Hydrology Office for their comments.

Any comments or revisions you may have should be submitted at your earliest convenience.

CSG/cf
Atch.

C. S. Grebski,
Structural Design Engineer.

c.c. B. R. Davis
W. D. Birch
A. E. McKim
A. Radkowski
M. Stoyanoff
✓ C. Mirza
J. Harris
J. Anderson
N. Maluzynsky
S. Edwards

NO COMMENTS

FD M.A.

JUNE 4/75



Memorandum

To: Mr. C. Mirza,
Head, Soil Mechanics Section,
West Building, Downsview.

From: Structural Office,
West Building, Downsview.

Attention:

Date: June 13, 1975.

Our File Ref.

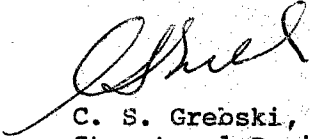
In Reply to

Subject:

Shewfelt Creek Structure,
W.P. # 903-72-07 Site # 38S-285
Highway # 17 District # 18

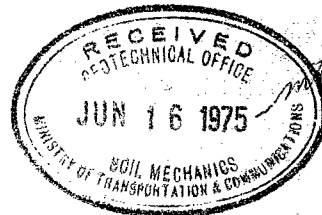
Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure. Kindly give us your comments at your earliest convenience.

CSG/cf
Attch.


C. S. Grebski,
Structural Design Engineer.

Refer to memo

H. Shah
June 19, 1975



Mr. C.S. Grebski
Structural Design Engineer
Structural Office
West Building, Downsview

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

June 19, 1975

SHEWFELT CREEK STRUCTURE
Highway #17 District #18
(Sault Ste. Marie)
W.P. #903-72-07 Site #38S-285

Our comments for the final bridge drawings Sheets 1 & 3 of the above mentioned structure are as follows. It is ascertained from the Structural Design Office that end bearing 12BP53 Steel H-piles which will be driven according to the Hiley Formula, have actually been designed to about 85 per cent of the structural capacity of the piles and takes into account the negative skin friction forces as mentioned in our Foundations Report. However, the design load of 70 tons/pile as shown on the drawings will be used in the field for controlling pile driving during construction. It is understood that the area between Stations 182+65± and 184+60± which is located in the vicinity of the structure between the present Hwy. 17 (future E.B.L.) and the proposed Hwy. 17, W.B.L. will be filled up to elevation 619. In view of this, no stability problems are anticipated for the proposed approaches. However, settlements will take place due to the induced embankment loadings. In order to minimize these settlements and consequently to reduce maintenance costs, consideration should be given for the use of light weight fill.

H. Shah
Project Engineer

For

M. Devata
Supervising Engineer

HS/MD/rjc

c.c. W.L. Lees
G.E. French
R. Morgenroth

Files ✓
Record Services



Ministry of
Transportation and
Communications

Memorandum

To: Mr. W.L. Lees,
Regional Manager,
Reg. Planning and Design,
Northwestern Region, Thunder Bay.

From: Structural Office,
West Building,
Downsview, Ontario.

Attention:

Date: February 9, 1976.

Our File Ref.

In Reply to

Subject:

W.P. 903-72-07, Site 38S-285
Shewfelt Creek Bridge
Hwy. 17, District 18

Please find enclosed four sets of prints of drawings 38S-285-1 to -7 for your use.

One print of drawing 38S-285-1 is being forwarded to the Systems Design Project Review Section.

One set of prints is also being forwarded to the following:

Estimating Section
Regional Structural Planning Engineer
Assistant Construction Engineer (Structures)
District Office
Structural Maintenance Engineer
Soil Mechanics Section
Hydrology Section

The D4 and Special Provisions were mailed to you previously.

NZ/ac
Encl.

[Signature]
N. Zoltay,
Structural Contract
Specifications Engineer.

c.c. J. Wear
B. Giroux
B. McKenna
A.E. McKim
G.E. French
W. Birch
C. Mirza
J. Harris
N.G. Maluzynsky
J. Anderson





Memorandum

To: Mr. W.L. Lees,
Regional Manager,
Reg. Planning and Design Office,
Northwestern Region, Thunder Bay.

From: Structural Office,
West Building,
Downsview, Ontario.

Attention:

Date: February 10, 1976.

Our File Ref.

In Reply to

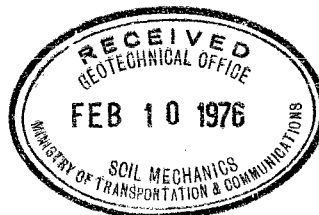
Subject: W.P. 903-72-07, Site 38S-285
Shewfelt Creek Bridge
Highway 17, District 18

Please delete tender item for Granular C Backfill to Bridge. The Regional Planning and Design Office will provide this tender item, as I was advised by Mr. A. Radkowski.

N. Zoltay,
Structural Contract
Specifications Engineer.

NZ/ac

c.c. A. Radkowski
J. Wear
B. Giroux
B. McKenna
A.E. McKim
G.E. French
W. Birch
C. Mirza ✓
J. Harris



Mr. B. McKenna
Regional Structural Planning Engineer
Northwestern Region, Thunder Bay

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

April 20, 1976

Shewfelt Creek Bridge
Hwy. 17, District 18, Sault Ste. Marie
W.P. 903-72-07

This is regarding your teletype of March 26, 1976 on the above subject. The statement quoted therein from the Foundation Report (page 2) is incorrect and should read as below:

"The existing structure exhibits signs of deterioration, having a badly weathered south face and cracks throughout the exposed faces of the structure. At this stage it was not known whether the existing structure would be replaced by a new one or not."

This statement is correct and there is no error by the Soil Mechanics Section.

The Engineer in the field observed that he was able to remove concrete from the south face and expose the reinforcing steel. He also observed a full height crack separating the north wing wall. These observations are further confirmed in your own teletype. The Foundation Investigation Report did not contain any comment regarding the structural soundness of the culvert. No field reconnaissance report was issued by your Office for this project which is customary and contains a description of the condition of the existing structure.

As per your memorandum you were requested in March, 1975 to determine if replacement of structure was required and a site visit was made by your Office on March 26, 1975. The foundation investigation was carried out during the period of March 17, 1975 to March 22, 1975, which preceded the site visit by your Office, and it is true that no decision was made by this time and could not have been made before March 26, 1976. This Section was never advised by you regarding the replacement, or otherwise, before the report was issued on April 15, 1976.

A. Prakash

A. Prakash
Senior Engineer

For: M. Devata
Supervising Engineer

cc: C.S. Grebski	W. Neilipovitz
G. French	Files
W. Lees	Record Services
R. Morgenroth	