G.I.-30 SEPT. 1976

GEOCRES No. ____________

DIST. ________ REGION ____________

W.P. No. ____________

CONT. No. ____________

W. O. No. 93-11015

STR. SITE No. 14-282

HWY. No. County Rd. 7

LOCATION Co. Rd. #7 Over Highland Creek, County of Lambton

No of PAGES - —

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. ____

REMARKS: ____________

M E M O R A N D U M

Ontario

To: A.A. Witecki
Municipal Engineer
Approvals Section
Structural Office
7th Floor, Atrium Tower

Date: July 23, 1993

Attn: Albert Lee

From: Foundation Design Section
Room 315, Central Building

Re: Proposed Grade Change
County Rd. 7 at Highland Creek
Structure Site No. 14-282, W.O. 93-11015
District 1, (Chatham)

We have reviewed the supplied geotechnical data and other information for this project. Our comments and queries are as follows.

1) Design Drawing #2 indicates that the existing profile grade will be increased by about 3 to 4 ft. The photographs taken at this site show horizontal movement of the existing guardrail, which indicates a somewhat unstable condition.

2) In our opinion, the additional load will aggravate this situation.

3) The construction of the proposed bin-type retaining walls on the existing slope will require extensive excavation which in turn may jeopardize the integrity of the travelled road.

4) The magnitude of the additional settlement?

5) Will the buried utilities withstand the additional settlement?

6) Will benching be required?

Should further information be required, please contact our office.

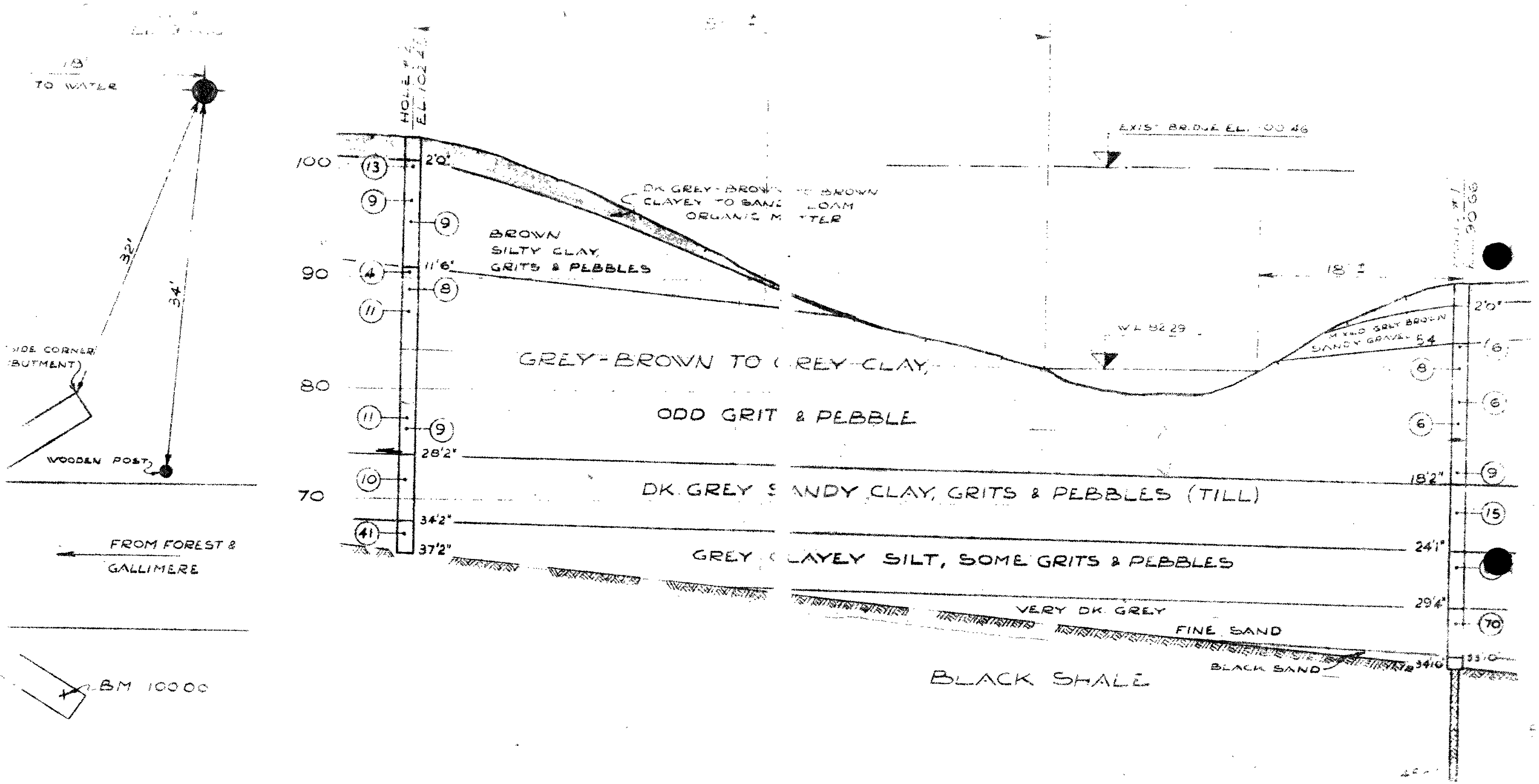
P. Payer

P. Payer, P. Eng.
Senior Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

PP/jb



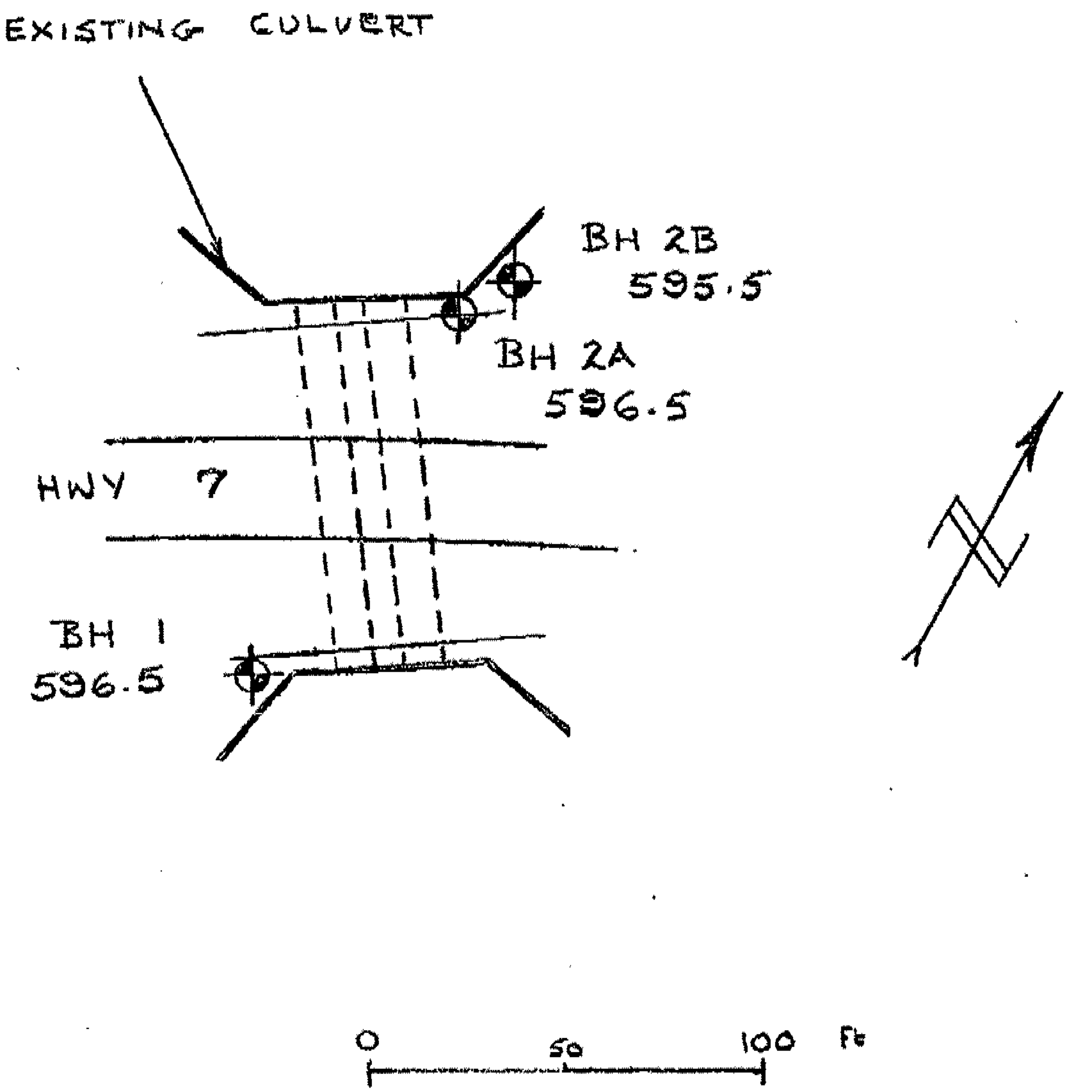
SECTION THROUGH HOLES 2 & 1

SCALE: HOR. & VERT. 1" = 10'0"

LEGEND

REFERENCE NO. 92 - 6 - P5 ENCL. NO. 1

BOREHOLE LOCATION PLAN



LOG OF BOREHOLE 1

ENCL.NO. 2

REF. No.: 92-6-P5	
CLIENT: LAMBTON COUNTY	
PROJECT NAME: HIGHLAND CREEK ROAD CROSSING IMPROVEMENT	Method: S.S. AUGER
LOCATION: HIGHLAND CREEK AND COUNTY ROAD 7, PLYMPTON TWP.	Diameter: 3 IN.
DATUM: LOCAL	Date: June 22nd, 1992

GROUND SURFACE 596.5 ft

PL %	NMC %	LL %	UNIT WT pcf	UNDR STRNG Field Vane psf	UNDR STRNG Lab. Compr psf	Sample No.	Type	N-Value	MATERIAL DESCRIPTION	DEPTH ft	ELEV. ft	WATER DATA	REMARKS
						1	SPT	31	sand and gravel with clay lumps, FILL — dense		596, 594		
						2	SPT	34		5	592, 590		
						3	SPT	36		10	588, 586		
						4	SPT	9	compact		584, 582		
						5	SPT	17		15	580, 578	▽ (approx. 580)	
						6	SPT	8	grey silty CLAY with embedded sand and gravel — stiff	20	576		
						7	SPT	16	very stiff	25	574, 572		
						8	SPT	19			570		

END OF BOREHOLE
BM ELEV. 595.02 ft. TOP OF CONCRETE HEADWALL.

Vertical Scale: 1:60

DOMINION SOIL INVESTIGATION
Sarnia, Ontario

SHEET 1 OF 1

Checked: CK
HOLE NO. 1

LOG OF BOREHOLE 2A

REF. No.:	92-6-P5		ENCL.NO. 3
CLIENT:	LAMBTON COUNTY		
PROJECT NAME:	HIGHLAND CREEK ROAD CROSSING IMPROVEMENT	Method:	S.S. AUGER
LOCATION:	HIGHLAND CREEK AND COUNTY ROAD 7, PLYMPTON TWP.	Diameter:	3 IN.
DATUM:	LOCAL	Date:	June 17th, 1992

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	DEPTH ft	ELEV. ft	WATER DATA	REMARKS
PL %	WMC %	LL %	UNIT WT	UNDR STRNG Field Vane	UNDR STRNG Lab. Compr	No.	Type	N-Value						
			pcf	psf	psf					GROUND SURFACE 596.5 ft				
												596		
						1	SPT	12		clay with embedded sand and gravel, FILL		594		
						2	SPT							
										END OF BOREHOLE (REFUSAL TO AUGERING ON LARGE GRAVEL) BM ELEV. 595.02 ft. TOP OF CONCRETE HEADWALL.				

Vertical Scale: 1:60

DOMINION SOIL INVESTIGATION
Sarnia, Ontario

SHEET 1 OF 1

Checked: CK
HOLE NO. 2A

LOG OF BOREHOLE 2B

ENCL.NO. 4

REF. No.:	92-6-P5
CLIENT:	LAMBTON COUNTY
PROJECT NAME:	HIGHLAND CREEK ROAD CROSSING IMPROVEMENT
LOCATION:	HIGHLAND CREEK AND COUNTY ROAD 7, PLYMPTON TWP.
DATUM:	LOCAL
Method:	S.S. AUGER
Diameter:	3 IN.
Date:	June 18th, 1992

PL %	WMC %	LL %	UNIT WT pcf	UNDR STRNG Field Vane psf	UNDR STRNG Lab. Compr psf	No.	Type	N-Value	MATERIAL DESCRIPTION	DEPTH ft	ELEV. ft	WATER DATA	REMARKS
									GROUND SURFACE 595.5 ft				
						1	SPT	16	clay with embedded sand and gravel, FILL; stiff		594		
						2	SPT	15		5	592 / 590		
						3	SPT	11			588		
						4	SPT	11		10	586 / 584		
						5	AS			15	582 / 580		
						6	SPT	16			578		
						7	SPT	12	silty CLAY with embedded sand gravel; grey; stiff	20	576 / 574		
						8	SPT	13			572		
						9	SPT	17	black; very stiff	25	570		

END OF BOREHOLE
BM ELEV. 595.02 FT. TOP OF CONCRETE HEADWALL.

Vertical Scale: 1:60

DOMINION SOIL INVESTIGATION
Sarnia, Ontario

SHEET 1 OF 1

Checked: CK
HOLE NO. 2B

DOMINION SOIL
CONSULTING ENGINEERS

DOMINION SOIL INVESTIGATION INC.
3953 RIBERDY ROAD
WINDSOR, ONTARIO N8W 3W5
TEL: (519) 969-7530
FAX: (519) 969-0160

Ref. No. 92-6-P5

February 12, 1993

County of Lambton
Public Works Department
Highway # 21, Box 3000
Wyoming, Ontario
N0N 1T0

<u>Attn: Mr. Rob Steiginga</u>

Re: Proposed Grade Change at the County Road 7 Bridge
Highland Creek - Lambton County, Ontario
<u>Stability Assessment of the Slope</u>

STRUCTURE SITE No. 14-282

Dear Sir:

Further to the request of MTO we reviewed the available information pertaining to the proposed development at Highland Creek Road crossing. We understand that a more detailed explanation of the conclusions arrived at in our soil report No. 92-6-P5 of September 4, 1992, is required, addressing the concerns regarding the stability condition of the proposed work.

We consider relevant for the stability assessment of the proposed development, described schematically in the available terms of reference, the following aspects.

- **<u>The General Stability</u>** of the final embankment with respect to a potential general shear failure.

In this respect we have looked to a conservative assumption in which the existing embankment and the proposed addition were assimilated with a permanent surcharge of approximately 2300 psf (for the existing embankment) plus 500 psf (carried on by the addition) acting over the foundation soil at the river bed level. No shear strength was assigned, at this stage, to the embankment fill. According to the above schematic, the risk of a general failure is augmented since the shear strength of the foundation soil only is supposed to overcome the imposed loads.

As presented within the original soil report prepared by E.M. Peto Associates Ltd. in 1961 and confirmed during our soil investigation of 1992, a minimum allowable pressure of 2100 psf is available at the river bed elevation. This allowable pressure incorporates a factor of safety of a minimum of 3. Consequently, the actual bearing capacity of the foundation soil is assessed to a minimum of 6300 psf and, therefore, the effective safety factor against the general failure under a total applied load of 2800 psf is in excess of 2.

- The Headwall Stability

A detailed evaluation of the lateral earth pressures on existing wall was already incorporated in our previous soil report. The recommended total thrust of 27.8 kips per running foot of wall was determined on a conservative basis, disregarding the major load relief provided by the presence of the two 20 foot diameter culverts. Therefore, for the wall analysis the above thrust can be considered acting on the wall wings only rather than along the total length of the wall. There is much less lateral load against the frontal portion of the headwall framing the culverts since the embankment weight in this portion appears to be carried directly be the corrugated culvert and transferred to the foundation soil.

A detailed analysis of the general stability and structural condition of the subject walls was beyond the scope of our work.

- The Structural Stability of Culverts

This aspect was addressed in our soil report due to the its direct implications with the general performance of the earth works at this project. The integrity of the culverts is crucial for the overall stability of the work. The existing headwalls will not sustain the induced embankment lateral pressures without the load relief provided by the culverts.

Should an investigation of the culvert condition reveal an unsatisfactory status, then an appropriate upgrading is mandatory.

- The Local Stability of the Existing Embankment

This aspect is analyzed under the assumption of a stable foundation and in conjunction with the backfill material characteristics, the design geometry, and the intended overload. In accordance with our findings at the test holes, the existing embankment appears capable to carry the proposed development if the general slope is not greater than 2.5 H: 1V and the local stressing underneath the proposed bin structures are proved to be acceptable. This last condition is to be verified in conjunction with the proposed detailed design.

Obviously, the intrinsic stability of the embankment is intimately related to the stability of the culverts and headwalls.

- The Intrinsic Stability of the Proposed Addition

This aspect was also addressed in our soil report in terms of the design loads for the proposed retaining elements (bin structures) the recommended geometry and slope surface treatment, as well as the material specifications.

We trust the above comments address your present requirements. We will be pleased to provide further assistance once the project details are available.

Yours very truly,
DOMINION SOIL INVESTIGATION INC.

D. Dimitriu, Ph.D.
Senior Engineer

DD/lb

xc: Ministry of Transportation
Attention: Mr. Albert Lee

DOMINION SOIL
CONSULTING ENGINEERS

DOMINION SOIL INVESTIGATION INC.
3953 RIBERDY ROAD
WINDSOR, ONTARIO N8W 3W5
TEL: (519) 969-7530
FAX: (519) 969-0160

FAXED

September 4, 1992
Report No. 2

RECEIVED SEP 14 1992

RECEIVED 1992 STRUCTURAL OFFICE M.T.C.

Ref. No. 92-6-P5

The Corporation of the County of Lambton
Public Works Department
Highway 21, Box 3000
Wyoming, Ontario
N0N 1T0
Fax 519-845-3817

STRUCTURE 14-282

Attention: Mr. Rob Steigigna

Re: Geotechnical Review No. 2 of the Proposed Grade Changes at the County Road 7 Bridge over Highland Creek - Lambton County, Ontario

Dear Sir:

On speaking with Mr. Murray Heinbuch of Altair Engineering, we were informed the headwall for the culverts should be considered as rigid and the wing walls as flexible. Consequently, the earth pressures will exert at-rest pressures on the headwall and active pressures on the wing walls. We have reviewed the relevant geotechnical report prepared in 1961, and have based our analysis on the contents of this report used in conjunction with the recent borehole data contained in our June 25, 1992 factual report issued to your office (our Ref. No. 92-6-P5).

The purpose of the geotechnical review is to identify the potential implication of raising the road grade while maintaining the road allowance width.

The review must consider:

1) an increase of lateral earth pressure on existing abutment-type reinforced concrete retaining walls;
2) the stability of a new slope rate geometry;
3) increased vertical pressures on the two csp culverts;

4) settlement of the culvert foundation soil (below invert);

5) settlement of the embankment fill (above obvert); and

6) requirements for slope face revetment.

The following is a summary of the relevant subsurface conditions contained on the Log of Borehole sheets submitted June 25, 1992 by Dominion Soil. Please refer to the borehole sheets for detailed information. Borehole 1 was completed on the south side of the road. Boreholes 2 and 2A were completed on the north side of the road.

The fill material within the south side of the constructed road allowance embankment is generally composed of sand and gravel with frequent occurrences of clay lumps. The fill on the north side of the road is composed primarily of clay. The material composition of the transition between these borehole data points is unknown. The compaction condition of the fill, however, was found to be "dense" to "compact" above El. 586± feet, and "loose" to "compact" below.

Water was encountered in Borehole 1 at El. 580.0 feet.

DISCUSSION

In order to maintain the horizontal width of the level section of roadway, the addition of 4± feet of fill will be required beneath the road. Fill on the side slopes will be required to effect a slope no greater than 2.5H:V. To prevent slope steepening, the County is proposing to construct an Arm Tec type flexible retaining wall set back 5 feet from the existing retaining wall.

1) Lateral Earth Pressure on Existing Wall

With the placement of a second retaining wall structure (regardless of type) and additional fill on the existing backfill to the existing concrete headwall, an increase in lateral pressure should be expected. Currently, the lateral earth pressure at any given depth below the top of the existing retaining wall can be calculated based on an upward backslope of fill at a 30 degree angle. Assuming the existing headwall performs as a rigid structure, then the pressures acting on the wall are considered horizontal at-rest pressures.

From the borehole data, we interpret an internal angle of friction of fill to be 35 degrees. The active pressure coefficient for the existing slope geometry is $K_A = 0.58$. The lateral earth pressure at any depth below the top of wall acting parallel to the slope can be calculated using the following formula.

$$Pa = 67\ Z$$

where:

Pa = active earth pressure (p.s.f.)

Z = depth below wall (feet)

The total active thrust on the vertical wall is 13.6 kips per running foot acting parallel to the sloping backfill. The resultant is located approximately 13.3 feet from the top of the wall.

With the new retaining wall located 10 feet from the existing retaining wall, the net increase in earth pressure on the existing wall is approximately 5% over existing conditions.

With the addition of the shallow retaining structure and the fill material at 5 feet from the wall, the existing wall capacity should be evaluated. The new lateral earth pressure acting at any depth below the top of the wall, acting parallel to the new slope can be calculated using the following formula:

$$Pa = 390 + 61\ Z.$$

The new total active thrust on the vertical wall is 27.8 kips per running foot parallel to the new sloping backfill. The resultant force is located approximately 10.5 feet from the top of the wall.

The above new forces should be used to check the stability of the existing retaining wall to receive the added loads.

2) Stability of New Slope Rate Geometry

The placement of the fill to the configuration shown on Drawing 2 of 2 (County of Lambton, County Road No. 7, Job No. 90130, dated April 1992) will not compromise the global embankment stability against deep seated soil shear. The factor of safety against deep seated shear failure is greater than 2, as determined using the data from the 1961 geotechnical report (E.M. Peto Job No. 6137, May 3, 1961).

3) Increased Vertical Pressure on Culvert

The manufacturer of the steel culvert should be contacted to assess the crushing resistance of the steel culvert. Although the composition of the fill over the culvert was found to vary, we conservatively estimate the bulk unit weight of the soil to be 130 p.s.f. As a result, the existing soil surcharge embankment pressure is a maximum of 2300 p.s.f if there is currently 18 feet of embankment cover. The addition of 4 feet of cover will increase the load on the culvert 500 p.s.f.

The ability of the culvert to receive these existing loads and the new loads is, in part, a function of the adequacy of bedding and surround materials. We cannot comment on the placement adequacy of the bedding and surround materials.

4) Settlement of the Culvert Foundation Soil

With the placement of a soil surcharge load of approximately 500 p.s.f, the native soil below the culvert invert will experience an increase in loading intensity. Based on the data contained in the 1961 geotechnical report, we can calculate the anticipated additional consolidation settlement.

Given the age of the current embankment in comparison to the expected rate of settlement generation listed in the 1961 geotechnical report, we expect all primary and secondary consolidation settlements are complete beneath the culvert.

We expect the placement of the proposed fill (4 feet to 6 feet) will result in additional time dependent consolidation beneath the culvert of 1 inch near the centre of the roadway, and about ¼ inch at the headwalls. The effect of the expected longitudinal bending of the culvert should be assessed by the culvert manufacturer.

5) Settlement of the Embankment Fill

The boreholes completed in June 1992 revealed a material composition inconsistency within the general embankment fill. The placement of this fill should, however, be considered as well compacted as inferred from the standard penetration resistance of the sampling tools.

The elastic compression and compaction settlement of the existing embankment fill will not be noticed during the filling operation. If the clayey fraction of the fill is significant, then there is a potential for some long term distortion of the pavement due to the consolidation of the clayey fill. If the remainder of the clayey fill is as compacted as that encountered by the boreholes, we expect consolidation settlement of the embankment fill to be negligible.

6) Requirements for Slope Face Revetment

The County is proposing to utilize an Arm Tec Bin-Type retaining wall. The applicability of this solution should be assessed only after the safety of the headwalls and culvert has been established. The maximum net allowable contact pressure at the base of the bin structure should not exceed 1500 p.s.f. If the bin structure is considered "flexible" (ie. allowed to rotate), then the structure will be required to retain active earth pressures; the magnitude of which can be calculated using the following formula.

$$Pa = 45 Z$$

Where:

Pa = the active pressure at depth Z (p.s.f.)

Z = the depth below the top of the Bin-Type structure (feet)

It must be noted, the above active pressure acts on the bin at the same angle of the slope.

The backfill material behind the bin structure must be composed of a free-draining granular fill (such as Granular "B Type I"). A system of positive drainage control (such as Big "O") should be placed behind the retaining structure at its base to eliminate any potential for base erosion, hydrostatic pressure, and frost problems.

The exposed slope should be suitably "dressed" to secure the fill from overland erosion.

ALTERNATIVES

If the assessed loading conditions on the headwall are too great or the Bin-Type retaining structure cannot support the stated active pressures, then a geogrid reinforced slope system can be constructed to significantly reduce the lateral earth pressures on the various structures. Please contact our office, we will be pleased to discuss the implications of this alternative.

CLOSURE

The limitations of this report, as discussed in detail in Appendix "A", constitute an integral part of this report. We recommend the Geotechnical Consultant to review design drawings and the intended method of construction prior to implementation in order to assure conformance with the geotechnical restrictions and assumptions.

We trust this report is complete within the terms of our reference. However, should questions arise concerning this report, do not hesitate to contact us.

Yours very truly,
DOMINION SOIL INVESTIGATION INC.

T. O'Dwyer, P.Eng.

xc Altair Engineering (Mr. Murry Heinbuch) Fax 519-542-6551
Dominion Soil Investigation Inc., Sarnia