



MERLEX ENGINEERING LTD.

CONSULTING GEOTECHNICAL ENGINEERS

FINAL

FOUNDATION INVESTIGATION AND DESIGN REPORT

McDougall Creek Culvert

Gabion Retaining System

W.P. 359-00-00

Highway 63
Station 20+367, Township of Poitras
M.T.O. District 54, Sudbury

Merlex Reference No. 00/04/00036

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Submitted to

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GEOCRES 31L-91

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1.0 INTRODUCTION

Merlex Engineering Ltd. has been retained by Earth Tech (Canada) Inc., on behalf of the Ministry of Transportation, to carry out a foundation investigation at McDougall Creek located in the Community of Thorne, Ontario. The purpose of the investigation was to delineate the subsurface conditions at both ends of the existing concrete box culvert, for the design of a retained soil system (gabion wall).

The terms of reference for this project are outlined in our proposal 00/04/00036 dated August 28, 2001. The work was carried out in accordance with the "Guidelines for Professional Engineers Providing Geotechnical Engineering Services" (1993) and the provisions in the RFP Terms of Reference.

2.0 SITE DESCRIPTION

The area of investigation, McDougall Creek culvert, is located within the Community of Thorne on Highway 63. This site is located within MTO District 54 approximately 65 km north of the City of North Bay. The proposed construction consists of widening the existing highway platform to include two 3.75 m driving lanes with 2.5 m wide shoulders and 0.5 m rounding on both sides. The project area is shown on the enclosed Key Location Plan and a site plan is shown on Figure 1 (Borehole Location and Soil Strata).

3.0 INVESTIGATION PROCEDURES

The fieldwork for this investigation was carried out during the period of September 19 and 20, 2001 and October 12, 2001. The initial fieldwork consisted of a total of 3 sampled boreholes in conjunction with 1 Dynamic Cone Penetration Test (DCPT). The second phase of the fieldwork consisted of 2 testpits. Locations are shown on Figure 1.

The borings were advanced with a bombardier mounted CME 45B diesel powered drill rig, equipped with 160 mm outside diameter (O.D.) continuous flight hollow stem augers. Soil samples were obtained at regular intervals of depth using a 50 mm O.D. split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. The boreholes were advanced to a maximum depth of 7.6 m (Borehole No. 2) or practical auger refusal, whichever occurred first, whereas the DCPT was advanced to refusal at a depth of 12.1 m (elevation 166.6) at Borehole No. 2. The groundwater conditions in the open boreholes were observed during the drilling operation and are described on the Record of Borehole Logs that follow the text.

The testpits were advanced with a CAT 322 track mounted excavator to the maximum depth of 6.2 m at Testpit B.

The field work was completed under the supervision of a senior member of our field engineering staff, who was responsible for locating the boreholes/testpits and DCPT, supervising the drilling, sampling and testing operations, logging the boreholes/testpits along with examining and preparing the samples for shipment to our North Bay Laboratory for further examination and select testing. Laboratory testing included determination of natural moisture content. The results of the laboratory testing are given on the individual Record of Borehole Logs.

The attached Borehole Location and Soil Strata information for this site is presented on Figure 1. The base drawing was provided by Earth Tech (Canada) Inc., who also provided all coordinates and reference elevations at the borehole locations. Stratigraphic information contained on the above noted figure is based on our evaluation of conditions encountered in the field.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the testpit locations are shown on the enclosed Record of Testpit Logs. The subsurface conditions encountered at the borehole locations, along with the laboratory test results, are presented on the Record of Borehole Logs. The stratification boundaries indicated on the Record of Testpit and Borehole Logs are inferred from non-continuous sampling, observations taken during hole advance, and the results of SPT and/or DCPT values. The boundaries between various strata shown on the Record of Borehole Logs generally represent transition from one soil type to another and should not be regarded as exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole/testpit locations.

In general, the area consists of sand (embankment) fill containing cobble/boulders overlying native cohesionless sand deposits with varying silt and gravel content with frequent cobbles and boulders. A summary of the subsurface conditions is as follows:

At Borehole Nos. 2 and 3, a surficial layer of silty organics approximately 50 mm thick was penetrated. At Borehole Nos. 1, 2 and 3, an embankment fill deposit consisting of fine to medium sand trace/some silt trace/some gravel with cobbles and boulders was penetrated from the surface to depths of approximately 2.4 m, 3.1 m and 1.4 m below existing grade respectively (elevations 177.5 m, 175.6 m, and 177.4 m). The natural moisture content of this deposit ranged from 6 to 15%. Based on the results of the SPT and DCPT, this deposit was in a loose state of relative density. Underlying this stratum, a fine to medium sand deposit with trace/some silt and trace/some gravel was penetrated. This deposit was found to contain occasional cobbles and boulders and the natural moisture content of the samples obtained was measured to range between 9 and 26%. The results of the SPT and DCPT indicate that this deposit is in a loose to dense (generally compact) state of relative density. Auger refusal was encountered in this deposit in Borehole Nos. 1, 2 and 3 at depths of 3.7 m, 7.6 m, and

2.9 m respectively (elevation 176.2 m, 171.1 m and 175.9 m). The DCPT carried out at Borehole No. 2 extended to a depth of approximately 12.1 m (elevation 166.6 m) at which point refusal to further penetration was encountered. The results of this test indicate that below 7.6 m, the deposit is generally in a compact state of relative density.

At Testpit A, underlying 500 mm of water, a deposit of cobble and boulder fill was penetrated to a depth of 1.3 m. This was underlain by a gravelly sand deposit with numerous cobbles. This deposit was penetrated to a depth of 2.8 m below the water surface where a grey silty fine sand with gravel was encountered. The testpit was terminated in this deposit at a depth of 3 m (elevation 175.1 m).

At Testpit B, a surficial 100 mm thick layer of silt with roots and organics was penetrated. Underlying this surficial layer, a fill deposit consisting of a gravelly sand with numerous cobbles and boulders was encountered to a depth of some 3.8 m. At this depth, a 300 mm thick seam of black silt with organics was encountered. Underlying this thin deposit, a grey silty sand with gravel and cobbles was penetrated. The testpit was terminated in this deposit at a depth of some 6.2 m below existing grade (elevation 171.6 m).

On October 12, 2001, during testpit excavations, the water level in McDougall Creek was measured at an elevation of some 178.1 m at Testpit A and recorded in Testpit B at elevation 176.3 m. Groundwater levels will fluctuate seasonally.

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5.0 DESIGN COMMENTS AND RECOMMENDATIONS

5.1 General

In order to widen the highway embankment at McDougall Creek, a gabion wall has been proposed. The wall will extend horizontally 12.0 m (typically) on both sides of the concrete box culvert, at both ends (see Figure 1).

The results of testpits indicate that loose cohesionless fill material extends to between elevations 174.0 m to 175.3 m near the existing culvert ends. As indicated by the boreholes, the cohesionless fill deposit appears to be shallower moving away from the culvert. Generally, a cohesionless sand deposit was encountered underlying the surficial fill deposits. These sand deposits were generally in a loose state of relative density becoming compact with depth.

5.2 Foundation

5.2.1 Bearing Resistance

The gabion wall will be founded on a compacted layer of Granular B Type II (see 5.2.2) underlain by cohesionless sands with varying silt and gravel content with numerous cobbles and boulders. Based on the SPT “N” and DCPT values from the boreholes, we have determined a factored geotechnical resistance at ULS of 130 kPa. A SLS resistance of 80 kPa reflects settlement consideration of 25 mm for footings established on properly prepared subgrade at elevation 177.0 m.

5.2.2 Stability Considerations

For design, a conventional gabion wall 3.0 m high with a 2.0 m wide base inclined at a 6 degree slope to the vertical was considered. The wall would be constructed of 1 m high baskets with the middle and top baskets 1.5 and 1.0 m wide respectively. Limit equilibrium stability analysis was carried out using the commercially available GawacWin 1.0 software. The factor of safety for overall stability coefficient of the slope was in the order of 1.32 with a water level at elevation 178.5 m.

The gabion wall retaining system is to be placed at each of the four quadrants with the front face of the retaining wall flush with the end of the box culverts (i.e. 9.5 m right of centerline and 9.0 m left of centerline). The base of the wall should be founded at elevation 177.0 m and using 1 m high baskets, the wall can be constructed to elevation 180.0 m. Installation of the gabion wall should conform to OPSS 512. The stone gradation for the front face element should be well graded and conform to OPSS 1004. To prevent migration of fines from the engineered fill behind the face element a strip of non-woven geotextile (such as MacTex Geotextile MX155 or equivalent) should be placed vertically along the back face of the retaining wall system.

At the time of this investigation the water level in the creek was measured at 177.8 m, however, during the summer of 2001 the creek water level had dropped to 176.5 m. The subgrade must be kept dry at all times during construction operations. Prior to placement of the first gabion lift, the exposed subgrade at elevation 177.0 m should be subexcavated to a minimum of 300 mm and proof rolled with a heavy vibratory compaction unit. Backfill below the gabion wall should be placed under the area of influence of the structure and consist of a 300 mm thick layer of OPSS Form 1010 Granular B Type II compacted in accordance with OPSS 501. The backfill behind the structure should consist of well graded free draining Granular B Type I and placed in accordance with OPSS 512.

5.2.3 Rock Fill

At the front of the gabion wall toe, a minimum height of 1.0 m of rock fill, sloped at a 2:1 (horizontal to vertical) must be provided. As a minimum, creek side backfill should consist of rock fill which meets hydraulic and fish habitat sizing requirements.

6.0 CLOSURE

Information provided in this report is valid only at the locations drilled. Any assumptions of continuity of soil stratigraphy between boreholes/testpits, as shown on the enclosed cross-sections, is intended as an aid for design purposes only and does not constitute a statement of existing conditions for contractual or construction purposes.

Details of the investigation, the material analysis and recommendation in this report are considered to be complete. However, should any questions arise, please do not hesitate to contact the undersigned.

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