



ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 83-88-02/03

DIST 17

HWY 69/17

STR SITE ~~46-203~~ 304

S.E. By-Pass Interchange Structures

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GEOCRES 411-133

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FOUNDATION INVESTIGATION REPORT

For

S.E. By-Pass Interchange Structures

W.P. 83-88-02/03 Site 46-203³⁰⁰

Hwy. 69/17, District 17, Sudbury

INTRODUCTION

This report summarizes the foundation investigation^a for two structures to carry Hwy. 17 over Hwy. 69 for the proposed S.E. By-Pass. The report is applicable to the two proposed structures, their immediate approaches, and related retaining structures.

SITE DESCRIPTION

The site is located at the proposed Hwy. 69/17 interchange, in the Regional Municipality of Sudbury, approximately 0.5 km south of the existing intersection of Hwy. 69 and the S.W. By-Pass.

The immediate area is exposed bedrock with occasional areas of shallow overburden. The topography is moderately steep and rugged with Hwy. 69 in a rock cut up to 6 to 8 m deep at the proposed structure locations. The east and west abutments of the proposed W.B. structure and the west abutment of the proposed E.B. structure are on exposed bedrock. The east abutment of the proposed E.B. structure is in an area of shallow overburden.

Just beyond the north-west quadrant of the site there is a commercial building, and beyond the immediate west approaches there are a few residential buildings. Otherwise the land is not in use.

INVESTIGATION PROCEDURES

A foundation investigation for this site was conducted on 89 11 10 and between 89 12 12 and 89 12 14. The investigation consisted of an examination of the exposed bedrock surfaces and cut faces, and the excavation of test pits at the east abutment of the proposed EB structure to classify the overburden material and determine the elevations of the bedrock surface. The contract drawings and Geotechnical Section boreholes

for previous highway widening in this area (Contract 78-79, W.P. 22-77-01) were also reviewed in order to approximate the depth to bedrock at the proposed pier locations.

SUBSURFACE CONDITIONS

Refer to Drawings No. 83-88-02/03 A&B for contours of the bedrock surface and details of bedrock elevations determined from the test pits at the east abutment of the proposed EB structure.

The east and west abutments of the proposed WB structure, and the west abutment of the proposed EB structure are on exposed bedrock. The bedrock at these locations is essentially flat.

At the east abutment of the proposed EB structure the bedrock surface dips N to S at approximately 15°, and is overlain by approximately 1 to 2.5 m of a heterogeneous mixture of sand, silt, cobbles and boulders.

From a review of documents from Contract 78-79, and assuming standard MTO construction procedures were followed, it is assumed that the bedrock surface at the pier locations for both proposed structures are under 0.5 to 0.9 m of road subgrade material, which in turn is underlain by approximately 0.3 to 0.5 m of shattered rock.

Groundwater was not encountered at the test pit at the SE quadrant. However, water was flowing in the Hwy. 69 ditches.

Bedrock

The bedrock is basically a thinly bedded metamorphic gabbro containing occasional slaty layers. It has essentially E-W trending features and near vertical bedding. There are numerous fractures - typically spaced at 1 to 2 m horizontal and 2 to 3 m vertical, that tend to make the rock blocky and brittle.

The east face is relatively stable probably due to superior blasting control. The drill traces are still visible, except for the most southerly 1/3 of the cut.

The west face is less stable with numerous overhanging blocky zones up to 1.2 m thick. This face has more, if minor, rock fall along the ditch line. There is some undercutting of the base, probably due to poor blasting control, especially in the area of Sta. 11+875 to Sta. 11+885 (Hwy. 69 chainage.).

DISCUSSION

It is proposed to construct two structures to carry the Hwy. 17 (SE By-Pass) over Hwy. 69. The WB structure has a proposed span geometry of 40 m - 40 m and will also carry Ramp NS-W. The EB structure has a proposed span geometry of 40 m (west) - 44 m (east) and will also carry Ramp SN-E. At the proposed crossings, Hwy. 69 is in an area of rock cut as deep as 8 m on the east side and 6 m on the west side.

Some cutting of the existing rock will be required in front of the west abutment of the proposed EB structure in order to establish the required ditch line for Hwy. 69. Minor cutting may be required in front of the east and west abutments of the proposed WB structure. No cutting will be required in front of the east abutment of the EB structure which will be in fill.

RECOMMENDATIONS

Bearing Capacity

All footings may be founded on sound bedrock. This implies that all loose or shattered rock under the plan limits of the footings should be removed and replaced with concrete. At the east abutment of the proposed EB structure, mass concrete may be used to bring the base of the footing to the ground surface. For design purposes, the following O.H.B.D.C. bearing capacities may be assumed:

Factored Bearing Capacity at U.L.S. = 10,000 kPa

Bearing Capacity at S.L.S. Type II will not govern

Alternatively, the east abutment of the proposed EB structure may be founded on a compacted Granular 'A' pad. The pad should extend to the bedrock surface, and a minimum of 1 m beyond the plan limits of the footing, from where it should slope down at 1H:1V or flatter. The granular material should be compacted to 100% proctor density. Please refer to

Figure 1 for a diagram illustrating the required geometry of the pad. For design purposes, the following O.H.B.D.C. bearing capacities may be assumed:

Factored Bearing Capacity at U.L.S. = 900 kPa

Bearing Capacity at S.L.S. Type II = 350 kPa

Consideration was given to founding the east abutment of the proposed EB structure on rock fill. However, due to concerns with excessive settlements within the rock fill, this option was not recommended for this otherwise unyielding structure.

The same recommendations as above pertain to any retaining walls.

Earth pressure

Backfill to abutments and retaining walls should consist of granular material in accordance with Ministry of Transportation Standard Special Provision #121 (83 10).

Alternatively backfill may consist of rock fill in which case special care will be required to avoid damaging the abutment or retaining structures. It would be preferable to place a 0.3 m± cushion of Granular 'A', or smaller rock fill (with diameter of less than 0.3 m), between the structure and the main mass of rock fill.

Computation of earth pressures should be in accordance with Section 6.6.1.2 of the O.H.B.D.C. Foundations on bedrock will be unyielding and the at-rest condition will govern earth pressure design. Foundations on Granular 'A' pads will be yielding and the active condition will govern earth pressure design. For design purposes, the following properties for backfill are recommended:

| Material | ϕ | γ | K _O | K _A |
|--------------|--------|------------------------|----------------|----------------|
| Granular 'A' | 35° | 22.8 kN/m ³ | 0.43 | 0.27 |
| Granular 'B' | 30° | 21.2 kN/m ³ | 0.5 | 0.33 |
| Rock Fill | 35° | 18.0 kN/m ³ | 0.43 | 0.27 |

Sliding Resistance

Sliding resistance between concrete and bedrock, or between concrete surfaces, should be calculated in accordance with Section 6-7.3.3.2 of the O.H.B.D.C. assuming an unfactored ϕ value of 30°.

If necessary, sliding resistance can be supplemented by dowelling into bedrock or between concrete surfaces.

For design purposes, the following O.H.B.D.C. capacities may be assumed for the bond between bedrock and grout:

Factored Bond Capacity at U.L.S. = 500 kPa
Bond Capacity at S.L.S. Type II will not govern design length.

However, the minimum dowel embedment should be 1 m.

Neither the structural strength of the dowel, nor the compressive strength of the grout should be exceeded. Also, the annular space around the dowel (to be grouted) should be in the order of 2 cm wide (between the dowel and the bedrock).

The capacity of dowels between concrete surfaces will be dependent on the strength of the concrete and the dowel.

Sliding resistance between concrete and Granular 'A' should be calculated in accordance with Section 6-7.3.3.2 of the O.H.B.D.C. assuming an unfactored ϕ value of 35°.

Slope Stability

Cut rock slopes will be stable with near vertical faces provided that blasting is carefully controlled. Controlled perimeter blasting should be considered in order to ensure a neat excavation line and thus minimize face instabilities and long-term maintenance problems. Line drilling, cushion blasting or preshearing techniques may be required depending on the

judgement of the contractor. In any case, the contractor should be advised that a neat excavation line is a requirement.

In order to ensure stability at the east and west abutments of the proposed WB structure and at the west abutment of the proposed EB structure, footings should be located far enough from the face of rock cuts to be outside a plane defined as 1H:2V from the toe of the cut slope. At the east abutment of the proposed EB structure, the base of the footing should be located outside a 2H:1V slope from the ditch line.

At the east and west abutments of the proposed WB structure and at the west abutment of the proposed EB structure the rock face within a perpendicular distance of 10 m from the footing should be rock bolted. Resin bonded rock bolts should be used in a 3 m by 3 m pattern, with a 4 m embedment length.

Earth fill slopes up to 8 m high will be stable at 2H:1V. Rock fill slopes up to 8 m high will be stable at 1.5H:1V.

The report was written by M. Datta and reviewed by M. Datta, Chief Engineer.

Dewatering

Since groundwater was not encountered, dewatering is not anticipated to be a problem. If seepage into the excavation at the east abutment of the proposed EB structure does occur, it can be dewatered by conventional sump pumping.

Settlement

No settlement of the structures is anticipated.

Minor settlements may occur within a rock fill approach to the east abutment of the EB structure. This settlement could be minimized by surcharging the embankment with 1 m of additional rock fill and preloading for a period of as little as a few days (as settlements will be caused by reorientation and crushing within the rock fill and will be almost instantaneous).

Alternatively settlements within the approaches could be minimized by using standard earth fill.

Frost Protection

Frost protection is not required for footings on sound bedrock.

For footings on compacted Granular 'A', earth cover of 2.0 m (or equivalent insulation) is required for frost protection.

MISCELLANEOUS

The field work for this project was carried out under the supervision of S. Holmes, Foundation Engineer and D. Dundas, Sr. Foundation Engineer.

The equipment used was hired by MTO District 17.

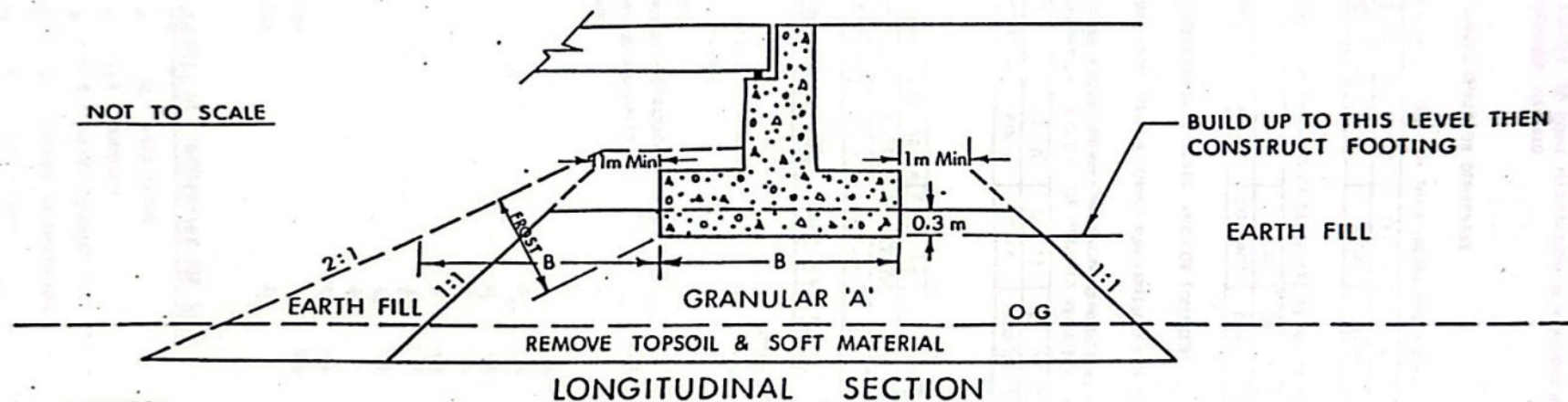
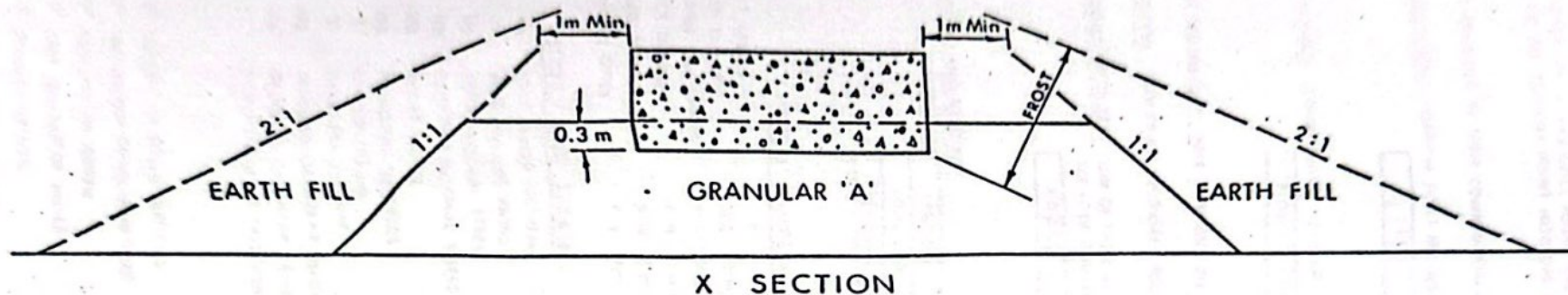
The report was written by D. Dundas and reviewed by M. Devata, Chief Foundation Engineer.



D. Dundas, P.Eng.
Sr. Foundation Engineer

M. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX



NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T C STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ontario

Ministry of
Transportation

**ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE**

FIG No 1

W P 83-88-02/03

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SHANKER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FROM A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION OBSERVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 1" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

| c_u (kPa) | 0 - 12 | 12 - 25 | 25 - 50 | 50 - 100 | 100 - 200 | > 200 |
|-------------|-----------|---------|---------|----------|------------|-------|
| | VERY SOFT | SOFT | FIRM | STIFF | VERY STIFF | HARD |

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

| N (BLOWS/0.3m) | 0 - 5 | 5 - 10 | 10 - 30 | 30 - 50 | > 50 |
|----------------|------------|--------|---------|---------|------------|
| | VERY LOOSE | LOOSE | COMPACT | DENSE | VERY DENSE |

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

| RQD (%) | 0 - 25 | 25 - 50 | 50 - 75 | 75 - 90 | 90 - 100 |
|---------|-----------|---------|---------|---------|-----------|
| | VERY POOR | POOR | FAIR | GOOD | EXCELLENT |

JOINTING AND BEDDING:

| SPACING | 50mm | 50 - 300mm | 0.3m - 1m | 1m - 3m | > 3m |
|----------|------------|------------|------------|---------|------------|
| JOINTING | VERY CLOSE | CLOSE | MOD. CLOSE | WIDE | VERY WIDE |
| BEDDING | VERY THIN | THIN | MEDIUM | THICK | VERY THICK |

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

| | | | |
|----|---------------------|----|---------------------------|
| SS | SPLIT SPOON | TP | THINWALL PISTON |
| WS | WASH SAMPLE | OS | OSTERBERG SAMPLE |
| ST | SLOTTED TUBE SAMPLE | RC | ROCK CORE |
| BS | BLOCK SAMPLE | PH | TW ADVANCED HYDRAULICALLY |
| CS | CHUNK SAMPLE | PM | TW ADVANCED MANUALLY |
| TW | THINWALL OPEN | FS | FOIL SAMPLE |

MECHANICAL PROPERTIES OF SOIL

| | | |
|----------------|-----------------------|--------------------------------------|
| m_v | kPa^{-1} | COEFFICIENT OF VOLUME CHANGE |
| C_c | 1 | COMPRESSION INDEX |
| C_s | 1 | SWELLING INDEX |
| C_a | 1 | RATE OF SECONDARY CONSOLIDATION |
| c_v | m^2/s | COEFFICIENT OF CONSOLIDATION |
| H | m | DRAINAGE PATH |
| T_v | 1 | TIME FACTOR |
| U | % | DEGREE OF CONSOLIDATION |
| σ'_{vo} | kPa | EFFECTIVE OVERBURDEN PRESSURE |
| σ'_p | kPa | PRECONSOLIDATION PRESSURE |
| τ_f | kPa | SHEAR STRENGTH |
| c' | kPa | EFFECTIVE COHESION INTERCEPT |
| ϕ' | -° | EFFECTIVE ANGLE OF INTERNAL FRICTION |
| c_u | kPa | APPARENT COHESION INTERCEPT |
| ϕ_u | -° | APPARENT ANGLE OF INTERNAL FRICTION |
| τ_R | kPa | RESIDUAL SHEAR STRENGTH |
| τ_r | kPa | REMOULDED SHEAR STRENGTH |
| S_t | 1 | SENSITIVITY = $\frac{c_u}{\tau_r}$ |

STRESS AND STRAIN

| | | |
|--------------------------------------|-----|-------------------------------|
| u_w | kPa | PORE WATER PRESSURE |
| r_u | 1 | PORE PRESSURE RATIO |
| σ | kPa | TOTAL NORMAL STRESS |
| σ' | kPa | EFFECTIVE NORMAL STRESS |
| τ | kPa | SHEAR STRESS |
| $\sigma_1, \sigma_2, \sigma_3$ | kPa | PRINCIPAL STRESSES |
| ϵ | % | LINEAR STRAIN |
| $\epsilon_1, \epsilon_2, \epsilon_3$ | % | PRINCIPAL STRAINS |
| E | kPa | MODULUS OF LINEAR DEFORMATION |
| G | kPa | MODULUS OF SHEAR DEFORMATION |
| μ | 1 | COEFFICIENT OF FRICTION |

PHYSICAL PROPERTIES OF SOIL

| | | | | | | | | |
|--------------|------------------------|--------------------------------|-------|------|---|------------|-----------------------|--|
| ρ_s | kg/m^3 | DENSITY OF SOLID PARTICLES | e | 1, % | VOID RATIO | e_{\min} | 1, % | VOID RATIO IN DENSEST STATE |
| γ_s | kN/m^3 | UNIT WEIGHT OF SOLID PARTICLES | n | 1, % | POROSITY | I_D | 1 | DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$ |
| ρ_w | kg/m^3 | DENSITY OF WATER | w | 1, % | WATER CONTENT | D | mm | GRAIN DIAMETER |
| γ_w | kN/m^3 | UNIT WEIGHT OF WATER | S_r | % | DEGREE OF SATURATION | D_n | mm | n PERCENT - DIAMETER |
| ρ | kg/m^3 | DENSITY OF SOIL | w_L | % | LIQUID LIMIT | C_u | 1 | UNIFORMITY COEFFICIENT |
| γ | kN/m^3 | UNIT WEIGHT OF SOIL | w_p | % | PLASTIC LIMIT | h | m | HYDRAULIC HEAD OR POTENTIAL |
| ρ_d | kg/m^3 | DENSITY OF DRY SOIL | w_s | % | SHRINKAGE LIMIT | q | m^3/s | RATE OF DISCHARGE |
| γ_d | kN/m^3 | UNIT WEIGHT OF DRY SOIL | I_p | % | PLASTICITY INDEX = $w_L - w_p$ | v | m/s | DISCHARGE VELOCITY |
| ρ_{sat} | kg/m^3 | DENSITY OF SATURATED SOIL | I_L | 1 | LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$ | i | 1 | HYDRAULIC GRADIENT |
| γ | kN/m^3 | UNIT WEIGHT OF SATURATED SOIL | | | | k | m/s | HYDRAULIC CONDUCTIVITY |

