



THURBER ENGINEERING LTD.



MEMORANDUM

To: Christopher Schueler, P.Eng.
AECOM

Date: March 9, 2017

From: Sydney Pang, P.Eng.
Alastair Gorman, P.Eng.
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-4406-20

FOUNDATION INVESTIGATION AND DESIGN PERRY'S CREEK BRIDGE (SITE 26-141) GEOCRES NO. 31D-654

PART 1 FACTUAL INFORMATION

1 INTRODUCTION

A preliminary memorandum dated February 18, 2015 presented a brief summary of the factual findings from a foundation review carried out for the existing Perry's Creek Bridge in the Municipality of Peterborough. It also presented preliminary geotechnical recommendations for use in assessment of the existing foundations at the site. Those recommendations were provided for planning, structure evaluation and preliminary design purposes only.

Since the issue of the preliminary memo, information provided by AECOM indicated that a decision has been made to include conversion to semi-integral abutments in the rehabilitation strategy. Accordingly, five (5) new boreholes have been advanced in the vicinities of the north and south abutments. The main purposes of these boreholes were to confirm bedrock elevations behind the abutments to determine if rock excavation is required, provide information for roadway protection design and to confirm the nature of the abutment backfill which the original design drawings show to be rock fill.

Foundation comments and recommendations from the preliminary memo are retained in the first part of this memo. The second part of this memo presents the investigation procedures and additional subsurface information depicted in the new boreholes, and provides foundation comments and recommendations pertinent to excavation requirements and roadway protection design for semi-integral abutment conversion. It is noted that this memo supersedes the preliminary memo from February 2015.

Information from AECOM indicates that the proposed rehabilitation works include replacing concrete deck slab and barrier walls; replacing westerly exterior girders; removing expansion joints and converting to semi-integral abutments; constructing new approach slabs with sleeper slabs; replacing bearings at abutments; reconstructing abutment front faces; patch repairing

precast girders and substructure. AECOM also advises that the increase in dead load on the abutments and pier footings as a result of the proposed rehabilitation works will be less than 10%.

The following reference numbers apply to this site:

- Current W.P. 4011-13-01
- Site No. 26-141
- Historic GEOCREs No. 31D-057
- Historic W.P. 8-62

2 SITE DESCRIPTION

The site is located to the north of Burleigh Falls, approximately 100 m south of the intersection of Highway 28 and County Road 36 in the Geographic Township of Harvey in the Municipality of Peterborough. Based on the description in Section 6.3.3 of the RFP, the existing bridge, constructed in 1967, is a 3 span (15m + 27m + 15m) pre-stressed pre-cast AASHTO concrete girder bridge. The overall deck width is 20m with an asphalt riding surface of 19m carrying one lane of traffic plus one auxiliary lane in each direction of Highway 28. A historic General Layout drawing confirms the bridge length but shows that the bridge width (perpendicular to centreline) varies from about 17 m at the south abutment to about 20 m at the north abutment. A retaining wall extends from the east side of the north abutment. Based on the archived drawings, the wall measures up to about 6 m in length and up to about 6 m in height between its top and the footing. The structure was last rehabilitated in 1992 which involved concrete overlay.

The natural terrain in the vicinity of the bridge is generally flat. The historic data indicates that the original grade in the vicinity of the bridge ranged between approximate Elevations 234.7 and 243.8m (770 to 800 ft.). Highway 28 was constructed to approximate Elevation 243.1m. A historic General Layout drawing indicates that the approach fills were constructed by placing rock fill with the forward slopes inclined at 1.5H : 1V, resulting in overall embankment heights of up to about 9m.

The mapping in the Physiography of Southern Ontario by Chapman and Putnam shows that the site lies within the physiographic region known as the Dummer Moraines. This is an area of jagged stony land running along the southern border of the Canadian Shield. The till is composed of coarse boulders and sandy materials. This area is characterized by the presence of limestone and Precambrian plutonic rocks with bedrock outcrop occurring throughout.

3 SUBSURFACE CONDITIONS FROM GEOCREs

A site investigation was completed by the Foundation Section, Materials and Testing Division of the Department of Highways Ontario (DHO) and reported on January 1966. No borehole log was available in the records. Information presented on a Bore Hole Locations and Soil Strata drawing indicates that a total of four boreholes were drilled and sampled. Three of the boreholes located at the creek banks were advanced to the order of 7 m (23 ft.) to 3.6 m (12 ft.) depths below the original ground surface. The borehole located on water was advanced to about 2.4 m (8 ft.) depth below creek level. Boreholes 3 and 4 at the south bank were close to the south pier of the bridge,

while Borehole 2 on water and Borehole 1 at the north bank were close to the north pier and the north abutment, respectively.

It was reported that the boreholes generally encountered a thin layer of topsoil overlying granite bedrock, except in Borehole 3 where the bedrock was reportedly covered by 1.1 m (3.5 ft.) of boulders. The upper 0.9 to 2.1 m (3 to 7 ft.) of bedrock was reportedly fractured and the recovery was generally poor. No RQD or core recovery values were reported. Approximate bedrock elevations inferred from the drawings are as follows:

| <u>Borehole</u> | <u>Top of Rock Elevation</u> |
|-----------------|------------------------------|
| 1 | 242.6 m (796 ft.) |
| 2 | 233.5 m (766 ft.) |
| 3 | 234.4 m (769 ft.) |
| 4 | 235.6 m (773 ft.) |

It is noted that the bedrock excavation at the west abutment location created by blasting was inspected by an MTO geologist during construction. A documented note dated March 1967 described the exposed rock as granite gneiss grading into meta-syenite and meta-pegmatite, and blocky due to well developed joint sets.

The water level in Perry's Creek was at Elevation 234.3 m (768.8 ft.) at the time of the investigation.

The available GEOCRE information is attached in Appendix A.

4 ADDITIONAL FOUNDATION INVESTIGATION

The selected rehabilitation strategy requires excavation of the approach fills behind the existing abutments for conversion to semi-integral abutments. Five (5) new boreholes were advanced to prove bedrock, and to confirm the nature and extent of the existing abutment fill.

The additional boreholes, numbered 15-P01 to 15-P05, were drilled and sampled between May 6 to 12, 2015 on Highway 28. Boreholes 15-P01 and 15-P02 were located behind the south abutment, and Boreholes 15-P03 to 15-P05 were located behind the north abutment. All boreholes were terminated within bedrock at depths ranging between 4.1 and 11.2 m below highway grade. The records of boreholes and a Borehole Locations and Soil Strata drawing are attached in Appendix D. The borehole locations were laid out in the field and utility clearances obtained prior to drilling. Full time traffic control was implemented during drilling by a professional traffic control firm.

A track-mounted D52 drill rig was used in conjunction with hollow stem augers to advance the boreholes through the abutment backfill. Fill samples were obtained at selected intervals using a 50 mm nominal diameter split spoon sampler in conjunction with the Standard Penetration Test (SPT). An NQ core barrel in conjunction with NW casing were used to advance the boreholes and obtain core samples through boulders, cobbles and coarser rock fill, and within the underlying bedrock.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes, processed the recovered soil samples and secured the rock cores in wooden core boxes for transport to Thurber's laboratory for further examination and testing. All recovered split spoon samples were subjected to visual identification and to natural moisture content determination. At least 25% of the recovered samples were subjected to grain size distribution analysis. Point load testing was carried out at selected depth intervals on intact bedrock cores for estimation of the unconfined compressive strengths. The results of the laboratory testing are summarized on the Record of Borehole sheets, the figures and tables included in Appendix D.

The boreholes were not surveyed and their locations on the Borehole Locations and Soil Strata drawing are shown based on field measurements referencing surface features. An approximate highway grade at Elevation 243.0 m has been provided by AECOM for completing the records of boreholes and the drawing.

5 SUBSURFACE CONDITIONS BEHIND ABUTMENTS

In general, the subsurface stratigraphy immediately behind the abutments consists of a pavement structure on concrete approach slab (where present) overlying fill containing a mixture of cobbles, boulders and possibly in a sand and gravel matrix, which is underlain by granite bedrock. The following provides a more detailed account of the stratigraphy depicted in the boreholes.

5.1 Asphalt and Concrete Approach Slab

A 150 mm thick layer of asphalt was encountered at ground surface in all five boreholes. In Boreholes 15-P02 to 15-P04, 225 to 250 mm thick concrete approach slabs were encountered immediately below the asphalt.

5.2 Pavement Granular

Gravelly sand to sandy gravel fill containing occasional cobbles ranging between 0.7 and 1.2 m in thickness was encountered below the asphalt and/or concrete approach slab in all five boreholes. The depth to the base of the pavement granular ranges from 1.1 to 1.6 m (Elevations 241.4 to 241.9 m). SPT 'N' values greater than 100 blows for less than 0.3 m penetration indicated a very dense condition, although some of them may be attributed to the presence of cobbles and/or boulders.

5.3 Fill

In Boreholes 15-P01 to 15-P04, abutment backfill consisting of a mixture of cobbles, boulders possibly in a sand and gravel matrix underlies the pavement granular, and extends to bedrock at between 6.4 and 9.7 m depths (Elevations 233.3 to 236.6 m) near the south abutment, and between 1.1 and 2.5 m depths (Elevations 240.5 to 241.9 m) near the north abutment. This fill was not encountered in Borehole 15-P05.

The boreholes were advanced through this fill using rotary core drilling techniques, in conjunction with SPT sampling between core runs. Cobble size rock cores ranging between 25 and 200 mm

in length were recovered in the four boreholes. In Boreholes 15-P01 and 15-P02, boulder size rock cores up to 500 mm in length were also recovered. Measured SPT 'N' values of 45 and 65 blows for 0.3 m penetration indicate dense to very dense conditions. 'N' values of greater than 100 blows for less than 0.3 m penetration infer the presence of cobbles, boulders or rock pieces.

5.4 Bedrock

Near the south abutment, Boreholes 15-P01 and 15-P02 encountered bedrock at 6.4 and 9.7 m depths (Elevations 236.6 and 233.3 m), respectively. Near the north abutment, Boreholes 15-P03 to 15-P05 encountered bedrock at between 1.1 and 2.5 m depths (Elevations 240.5 to 241.9 m).

The bedrock may be described as slightly to moderately weathered granite, whitish grey to pink in colour with sub-horizontal to sub-vertical joints. An exception is near the bottom of Run #4 in Borehole 15-P02 where the rock colour becomes black and, in conjunction with the surface texture, indicating what appears to be an intrusion of plutonic rock.

The Total Core Recovery (TCR) within the bedrock varies from 85% to 100%. The Rock Quality Designation (RQD) of the granite ranges from 53% to 95% indicating fair to excellent rock quality.

Unconfined compressive strengths inferred from point load tests ranged from 92 MPa to greater than 200 MPa indicating a strong to very strong rock.

5.5 Groundwater Conditions

All boreholes except Borehole 15P-02 (possibly drill water) were dry upon completion. Observations made during borehole drilling indicated that the abutment backfill was permeable since there was no drill water return as coring proceeded until bedrock was reached. Groundwater is not expected within the fill at this site.

6 SITE OBSERVATIONS

Foundation engineering staff from Thurber visited the site to observe conditions related to the geotechnical performance.

There were no obvious signs of settlement or distress at the foundation elements.

The original design drawings indicated that rockfill was to be used as abutment backfill. The approach embankments appeared to be stable, with no obvious signs of instability or bulging. Rip-rap boulders and cobbles surrounding the piers and the forward slope areas appear intact.

Photographs of the structure and the approaches are attached in Appendix B.

THURBER ENGINEERING LTD.



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Senior Associate, Senior Foundation Engineer



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Review Principal, Designated MTO Contact

7 EXISTING FOUNDATIONS

Based on the historic General Layout (GL) drawing for the structure, D-5881-1 dated January 1966, the abutments and piers are supported by spread footings founded on bedrock. The approaches consisted of rockfill at an inclination of 1.5H to 1V at the forward slopes. Retaining walls behind the north abutment were to be backfilled with rockfill.

Based on the GL and other drawings including Foundations (D-5881-3), South Abutment (D-5881-4), North Abutment (D-5881-5) and Retaining Wall (D-5881-7), both abutments and associated retaining walls are supported on stepped footings, and both piers are supported on circular footings in the water. Notes on these drawings state that “foundations to be placed against sound undisturbed ground” for the piers, and “step bottom of footing to follow profile of bedrock” for the abutments.

The footings for the abutments and piers are shown as being founded at the following approximate elevations:

| Foundation Element | Imperial (ft) | Metric (m) |
|---------------------------|--------------------------------|--------------------------------|
| South Abutment | 779.0 (west) to 768.5 (east) | 237.4 (west) to 234.2 (east) |
| South Pier | ≤ 767.5 | ≤ 233.9 |
| North Pier | ≤ 767.5 | ≤ 233.9 |
| North Abutment | 785.5 (west) to 773.0 (east) | 239.4 (west) to 235.6 (east) |
| North Retaining Wall | 773.0 (south) to 785.0 (north) | 235.6 (south) to 239.3 (north) |

The drawings do not show bearing pressures on bedrock used for designing these footings.

The DHO report recommended that the pier footings be founded on bedrock surface with a design load of 10 t.s.f. (Working Stress Design). This is equivalent to an allowable geotechnical resistance of approximately 1,000 kPa for bedrock. The report recommended that the abutments be supported by end-bearing piles driven to bedrock. However, the design drawings show that the structure is supported entirely on spread footings bearing on bedrock. It is assumed that all abutment, pier and retaining wall footings have been designed using the values discussed above.

8 ASSESSMENT OF EXISTING FOUNDATIONS

Based on the bedrock conditions shown to exist at this site and the information provided by the historical drawings and reports, a factored geotechnical resistance at ULS of 1,500 kPa for footings on bedrock has been calculated in accordance with the requirements of the CHBDC. The SLS condition does not apply to footings founded on bedrock. For sliding resistance at the footing and bedrock interface, an ultimate coefficient of friction, $\tan \delta$, of 0.65 may be used for evaluation. The above values can be used for carrying out an assessment of the existing structure and for preliminary design of any modifications that may be necessary.

The RFP listed semi-integral abutment conversion as a rehabilitation requirement. The historic drawings show that rockfill was proposed to be used as backfill to the abutments. Should this be

confirmed, conversion to semi-integral abutments may require replacement to some extent of the rockfill immediately behind the abutment walls with approved granular materials. Moreover, the abutments are skewed at 40° to the bridge centreline in accordance with the historic drawings. Structural and other assessments will be required to determine if this skew is acceptable for semi-integral abutment conversion.

Based on the rehabilitation works outlined earlier in this memo and the fact that the dead load on the abutments and pier footings are expected to increase by less than 10% after rehabilitation, it can be assumed that the bridge foundations will continue to perform satisfactorily provided that all structural requirements are met.

9 EXCAVATION AND BACKFILL

It is anticipated that temporary excavations in the order of 2 to 3 m will be required for semi-integral abutment conversion at this site. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purpose of OHSA, the pavement granular and the fill with cobbles and boulders are classified as Type 3 soils. All excavations must be carried out in a manner that avoids undermining or destabilising the existing bridge and retaining wall foundations, existing approach slopes and the adjacent highway.

At the south abutment, excavations will extend through the pavement granular into the upper portion of the fill with cobbles and boulders. At the north abutment, it is anticipated that the pavement granular and all the fill with cobbles and boulders will have to be removed within the footprint of the excavation.

Where space permits, temporary excavations may be formed with sideslopes not steeper than 1H : 1V. Flatter slopes may be required at locations where the fills are less competent than what is assumed during design or where water seepage affects surficial stability. Otherwise, roadway protection will be required to support the ground adjacent to these temporary excavations (see section below).

Backfill to the abutments should consist of Granular A or Granular B Type II materials meeting the gradation and relevant requirements stipulated in OPSS.PROV 1010. Compaction procedures and equipment to be used adjacent to the existing structures must be in accordance with the relevant OPSS.PROV 501 requirements.

10 ROADWAY PROTECTION

Roadway protection may be required during construction. An item titled "Protection System" as per OPSS.PROV 539 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 and the alignment of the shoring be specified on the contract drawings.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is soldier pile and lagging wall. It is anticipated that the soldier piles will need to be installed into the rock fill at the south abutment and into bedrock at the north abutment to develop the required toe resistance. Drilling through fill with cobbles and boulders and/or bedrock will be required to form the sockets.

A temporary soldier pile and lagging wall may be designed using the parameters given below:

| | | |
|------------|---|---|
| γ_s | = | 21 kN/m ³ (gravelly sand to sandy gravel fill) |
| γ_R | = | 23 kN/m ³ (fill with cobbles and boulders) |
| K_a | = | 0.33 (gravelly sand to sandy gravel fill) |
| | = | 0.24 (fill with cobbles and boulders) |
| K_p | = | 4.2 (fill with cobbles and boulders) |
| | = | 4.6 (bedrock) |

The designer of the roadway protection system should check whether the depth of pile is sufficient to provide base fixity. The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. All shoring systems must be designed by a Professional Engineer experienced in such designs.

11 CLOSURE

The factual subsurface information used for foundation review and assessment of the existing foundation conditions was taken from the DHO report titled "Foundation Investigation Report for Proposed Structure at the Crossing of Perry's Creek and Hwy. 28, (Line 'A'), District 7 (Port Hope), W.J. 65-F-129, W.P. 8-62, dated January 20, 1966. The factual subsurface information used for determining excavation and roadway protection design requirements was provided by the five additional boreholes drilled for this investigation.

This memorandum was prepared by Dr. Sydney Pang, P.Eng., and was reviewed by Mr. Alastair Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

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Review Principal, Designated MTO Contact



Attachments



Appendix A
GEOCRES Information

MEMORANDUM

CC: GEN. FILES

23-66-215
~~W.P. 8-62~~

TO: Mr. B. R. Davis,
Bridge Engineer,
Bridge Division.

FROM: Foundation Section,
Materials and Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. S. McCombie

DATE: January 20, 1966

OUR FILE REF.

IN REPLY TO

JAN 31 1966

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Structure at the Crossing
of Perry's Creek and Hwy. 28,
(Line 'A'), District 7 (Port Hope).

W.J. 65-F-129 -- W.P. 8-62

BA2253

Attached, we are forwarding to you, our foundation investigation report on the subsoil conditions existing at the above structure site. We believe that the factual data and recommendations contained therein, will be adequate for your design requirements. Should you require further information, please feel free to contact our Office.

AGS/MdeF
Attach.

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
G. K. Hunter (2)
D. P. Collins
T. J. Kovich
A. Watt
Foundations Office
Gen. Files

FOUNDATION INVESTIGATION REPORT
For
Proposed Structure at the Crossing
of Perry's Creek and Hwy. 28,
(Line 'A'), District 7 (Port Hope).
W.J. 65-F-129 -- W.P. 8-62

At the request of the Bridge Location Section (memo dated November 5, 1965, from Mr. J. B. Curtis), a foundation investigation was carried out at this site. Presented in this report are the results of this investigation, together with recommendations pertaining to the foundation for the proposed new structure.

The field investigation was confined to four sampled boreholes. It was not possible to drill more than one borehole in the vicinity of the proposed pier locations because of the extremely fast flowing water and generally rocky condition of the stream bed. It is believed, however, based on visual inspection of the site area, that the rock surface is within a few inches of the creek bed.

The investigation revealed that the site is generally covered by a thin layer of topsoil followed by granite bedrock, except in B.H. #3, the bedrock was covered by a 3.5-ft. thick deposit of boulders. The upper 3 to 7 ft. of bedrock in B.H.'s #3 and #4, is fractured and the recovery was generally poor. The locations of the boreholes and their elevations are shown on Dwg. 65-F-129A.

cont'd. /2

A three-span structure (35'-60'-35'), having a width of 52 ft., is contemplated at this location. It is recommended that the piers for the proposed structure be founded on bedrock surface with a design load of 10 t.s.f. A dewatering scheme will be necessary since the footings will be constructed below creek water level.

The proposed abutments may be constructed within the approach fills and supported on end-bearing piles driven to bedrock. Design loads to be used are dependent on the pile section selected.

No stability problems are anticipated for the proposed approach fills with standard 2:1 slopes.

The field work, performed during the period December 8 to 17, 1965, together with the preparation of this report, was undertaken by Mr. V. Korlu, Project Foundation Engineer. The investigation was carried out under the general supervision of Mr. M. Devata, Senior Foundation Engineer, who reviewed this report.

Equipment used was owned and operated by Canadian Longyear Ltd.

January 1966.

APPENDIX I.

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' : - 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>'N' BLOWS / FT.</u> | <u>c LB. / SQ. FT.</u> | <u>DENSENESS</u> | <u>'N' BLOWS / FT.</u> |
|--------------------|------------------------|------------------------|------------------|------------------------|
| VERY SOFT | 0 - 2 | 0 - 250 | VERY LOOSE | 0 - 4 |
| SOFT | 2 - 4 | 250 - 500 | LOOSE | 4 - 10 |
| FIRM | 4 - 8 | 500 - 1000 | COMPACT | 10 - 30 |
| STIFF | 8 - 15 | 1000 - 2000 | DENSE | 30 - 50 |
| VERY STIFF | 15 - 30 | 2000 - 4000 | VERY DENSE | > 50 |
| HARD | > 30 | > 4000 | | |

TYPE OF SAMPLE

| | | | |
|------|-----------------------|-------------------------------|-------------------|
| S.S. | SPLIT SPOON | T.W. | THINWALL OPEN |
| W.S. | WASHED SAMPLE | T.P. | THINWALL PISTON |
| S.B. | SCRAPER BUCKET SAMPLE | O.S. | OESTERBERG SAMPLE |
| A.S. | AUGER SAMPLE | F.S. | FOIL SAMPLE |
| C.S. | CHUNK SAMPLE | R.C. | ROCK CORE |
| S.T. | SLOTTED TUBE SAMPLE | | |
| | P.H. | SAMPLE ADVANCED HYDRAULICALLY | |
| | P.M. | SAMPLE ADVANCED MANUALLY | |

SOIL TESTS

| | | | |
|-----|---------------------------------|------|-----------------|
| Qu | UNCONFINED COMPRESSION | L.V. | LABORATORY VANE |
| Q | UNDRAINED TRIAXIAL | F.V. | FIELD VANE |
| Qcu | CONSOLIDATED UNDRAINED TRIAXIAL | C | CONSOLIDATION |
| Qd | DRAINED TRIAXIAL | S | SENSITIVITY |

ABBREVIATIONS 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 |
| 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 INTERCEPT |
| 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 |
| σ' | 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_o | 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 |

File this phase

1965 DEC 2 AM 11:23

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P

PTHP DOWN 3 DEC 2/65 11:00A VR

D P COLLINS, DIST ENGR

ATTN - MR D A WALLACE

MAINTENANCE ENGINEER

SUBJECT - PERRY'S CREEK BRIDGE, HWY 28, LINE A. DIST. 7

AT BURLEIGH FALLS

THIS IS TO ADVISE YOU THAT THE FOUNDATION INVESTIAGATION WORK

WILL COMMENCE ON MONDAY DEC 6/65

OUR ENGINEER IS MR V KORLU, WHO WILL BE SUPERVISING JOB

M DEVATA, SR FOUNDN ENGR FOR

MR A G STERMACMMMMMM PRIN FOUNDN ENGR MATLS & TESTING DIV

L

T

MEMORANDUM

65-F-129

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

FROM: Bridge Division,
Downsview, Ontario.

DATE: November 5, 1965.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.F. 8-62, Site #26-119,
Perry's Creek Bridge,
Hwy. 28 Line A - Dist. 7.

Herewith, one print of drawing E 4127 - 1 showing the probable location of footings for the above structure.

Please arrange for a foundation investigation of sufficient scope to enable us to proceed with the design.



JFW/sp

J. B. Curtis,
Regional Bridge Location Engineer.

cc. R. Forrest
A. Crowley

COMPLETION DATE
FEBRUARY 2, 1966

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: February 10, 1966.

Our File Ref.

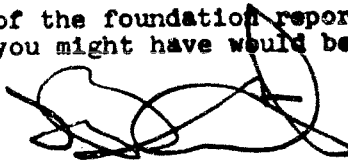
In Reply To

SUBJECT:

W.P. 7-62, Site 26-23,
Trent Canal at Burleigh Falls,
W.P. 8-62, Site 26-119,
Perry's Creek at Burleigh Falls,
Hwy. 28, District 7

Herewith one print each of the preliminary drawings
D 5804-1 and D 5881-P for the above structures.

The recommendations of the foundation reports have been
followed. Any comments you might have would be appreciated.



JFW/ag

J. B. Curtis,
Regional Bridge Location Engineer.

Encl.

W.P. 8-62

at ^{route} hard. Prior bedrock not
established. Drilling not
done because of inaccessibility
of location.

Feb 24. 1966

als

By phone to : John Curtis on Feb 24/66.

Downsview, Ontario,
January 14th, 1966.

Mr. J. K. Theil,
District Engineer,
Ontario Water Resources Commission,
801 Bay Street,
Toronto 5, Ontario.

Re: Hwy. 28, Line 'A', Prop. Rev. and
Hwy. 36, Line 'G', Prop. Rev. - Grading W.P. 90-62,
Structures: Trent Canal, W.P. 7-62 and
Perry's Creek, 8-62, *65-6-129*
Burlingame Falls, Dist. 7, Port Hope.

Dear Sir:

Please find attached one print each of plan B-302-19 and profile
C-302-21 and C-302-8.

It is the Department's intention to carry out reconstruction at the
above mentioned locations in the near future as shown on the accompanying
plan.

Please advise whether the proposals outlined will in any way affect
work proposed by the Ontario Water Resources Commission.

Yours very truly,

H. Smith,

For:

R. G. Burnfield,
Regional Functional Planning Engineer.

RGB/HS/WH/rw
Attach.

c.c. D. P. Collins, G. K. Hunter, J. Curtis.

Mr. D. P. Collins,
District Engineer,
Port Hope.

K. W. Ingham

March 23rd, 1967

Re: Foundations for Perry's Creek Bridge, Highway 28

The concern that has been expressed about the west abutment footing for the Perry's Creek Bridge would appear to resolve itself into whether or not it was possible to drill, blast and remove the bedrock to the required design elevations. In effect we are saying that if the bedrock is suitably sound then any excessive overbreak is due to improper blasting technique and is the sole responsibility of the contractor.

Some discussion of the condition of the surrounding bedrock is pertinent to the problem, and also the probable condition of the bedrock at the site before blasting.

The bedrock is predominantly granite gneiss, grading into meta-syenite and meta-pegmatite, with a pronounced vertical lineation - which trends approximately N.W., S.E. The rock is more or less banded with occasional zones rich in biotite - mica 0.05 to 0.1 ft. in thickness aligned parallel to the lineation. Although the bedrock is generally sound, i.e., only superficially weathered, the biotite zones are weathered to a depth of 5 to 6 ft. often with evidence of tree roots in the upper 2 to 3 ft. Thus they constitute planes of weakness, if only on a limited scale.

The structural continuity of the bedrock is more important than the rock properties in this case. It is properly described as being blocky, due to a well developed joint pattern. Two sets of vertical joints and a set of horizontal joints subdivide the rock substance into blocks 1 to 6 ft. in diameter. In addition at least one set of planes of preferred fracture is present dipping at an angle of 65° to 70° aligned approximately parallel to the pre-existing face.

Within 3 to 5 ft. of the original rock surface the fractures are somewhat enlarged and emphasized by surface weathering and some deposition from circulating ground water. In a few cases the inclined fractures have been considerably widened and are partly filled with detritus. A prominent example can be seen in the upper part of the rock face at the north end of the footing excavation and

/cont'd.....

traced down to the bottom of the excavation. Thus a large block of rock underlying the abutment footing is potentially unstable. Anchoring the footing by means of rock dowels may minimize any possible effects on the foundation.

Examination of the blast-holes indicates that they were drilled to a depth and spacing consistent with standard practice for crystalline rock of this type. Examination of the rock surrounding the blast-holes indicates an average loading.

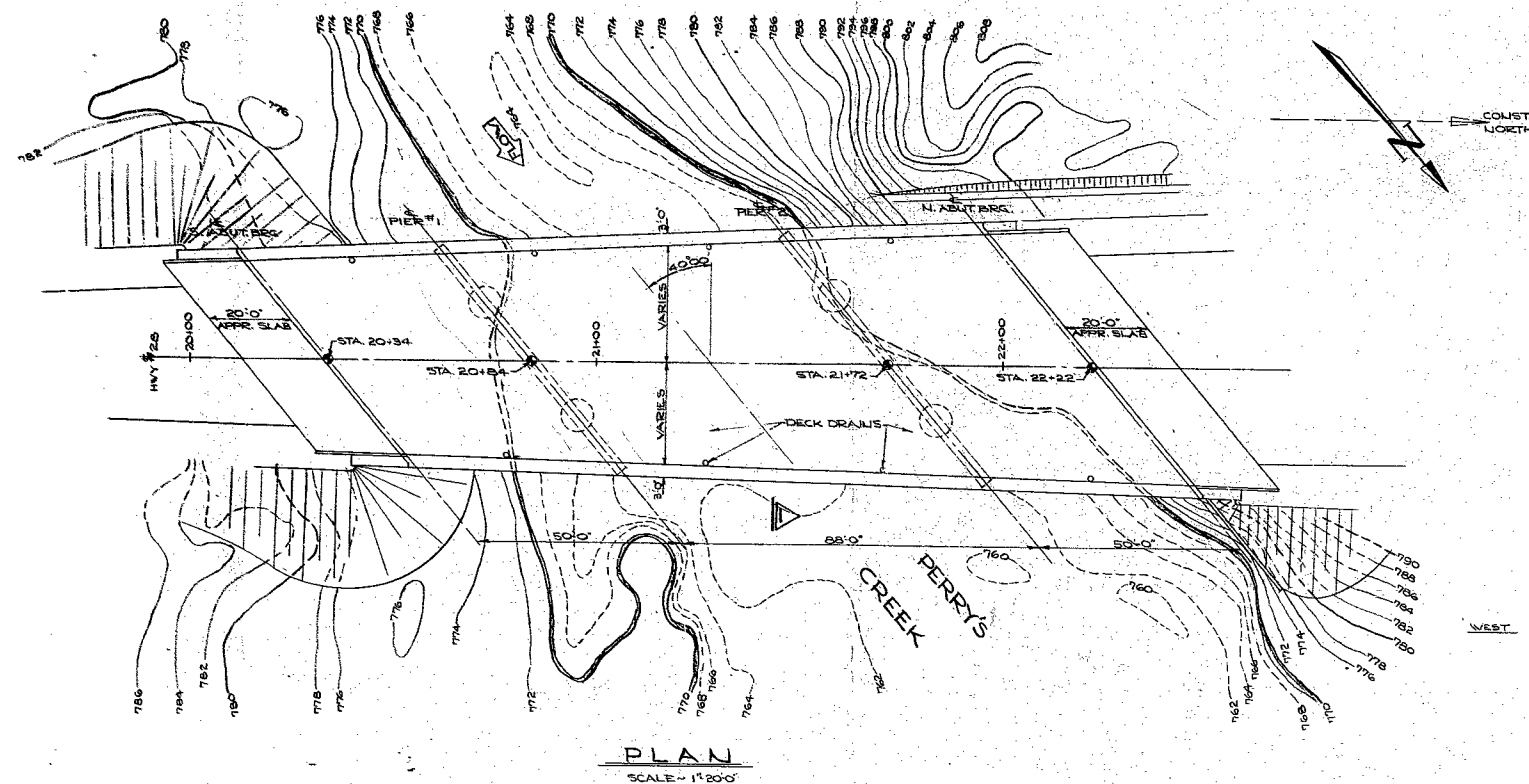
In my opinion the rock was not subjected to excessive blasting, the additional excavation being due more to the structural condition of rock than to extensive overbreak. However, under these conditions it would appear that the best technique would have been to just jolt the rock free with a much reduced charge on a tighter hole spacing - thereby permitting more control over the removal of rock from the excavation.

K. W. Ingham
K. W. Ingham,
Geologist.

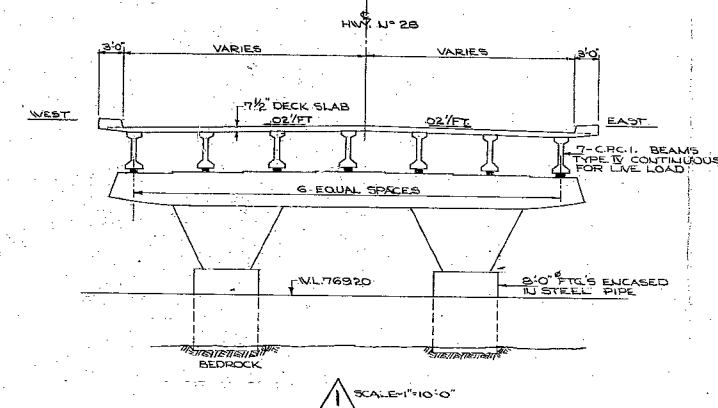
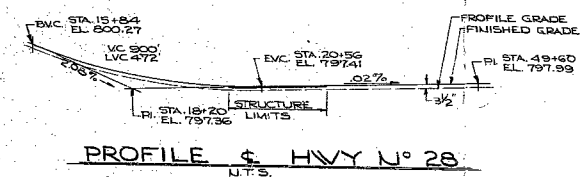
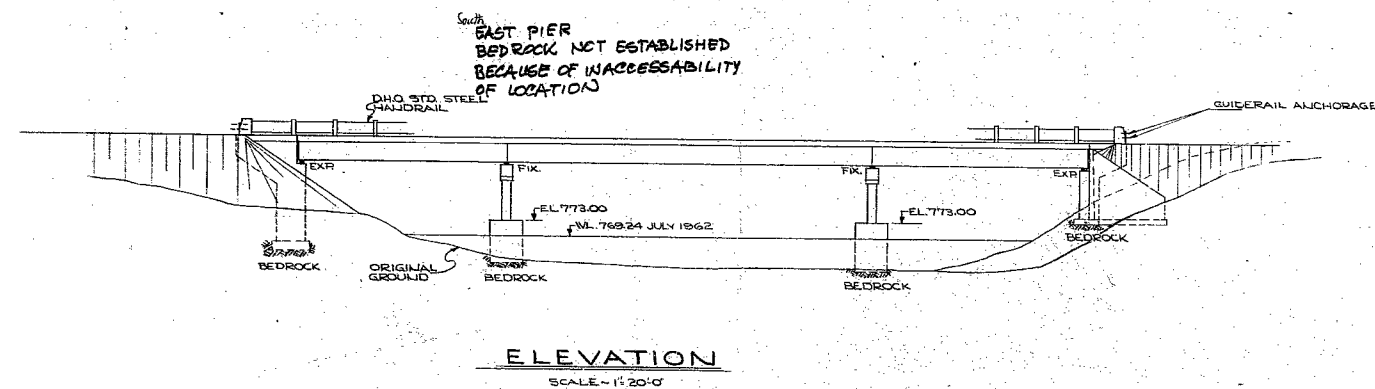
KWI jm

cc: R. Panter
J. McDougal
A. E. McKim
A. Selby

#65-F-129
W.P. 8-62
HWY 28
CROSSING
OF PERRY'S
CREEK



B.M. ELEV. 773.89
GEODETIC DATUM
S.I.B. IN GRANITE BOULDER.
51'0" LT. OF STA. 20+60

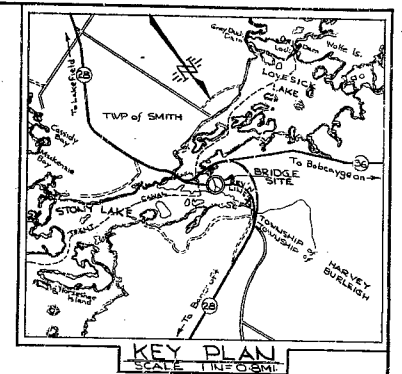


NOTES.

CLASS OF CONCRETE:
1 PRECAST MEMBERS 5000 RS./ DECK SLAB 4000 RS./
REMAINDER 3000 RS./

2 CLEAR COVER:
FOOTINGS, ABUTMENTS & PIERS - 3"
PRECAST MEMBERS - 1 1/2"
DECK SLAB - BOTTOM 1", TOP 2"

3. CONSTRUCTION NOTES:
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF $\pm 1/8$ ".

[illegible]

| | | | |
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| REVISIONS | | | |
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| | | | |
| | | | |
| | | | |
| | DATE | BY | DESCRIPTION |

| | | | | | | | | | | | | | | | | | |
|--|---------------------------------|-----------------------------|----------------------|------------------------|----------------------|-----------------------|--|---------------------|--|---|------------------------------|-----------------------------|--|----------------------|-----------|--|--|
| DEPARTMENT OF HIGHWAYS BRIDGE DIVISION | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| <u>PERRY'S CREEK BRIDGE</u> (AT BURLEIGH FALLS) | | | | | | | | | | | | | | | | | |
| KING'S HIGHWAY No. 28 CO. PETERBOROUGH J.W. HARVEY | DIST. No. 7 LOT 5 CON. I | | | | | | | | | | | | | | | | |
| <u>PRELIMINARY</u> | | | | | | | | | | | | | | | | | |
| <table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td colspan="2" style="padding: 5px;"> APPROVED _____ </td> <td style="padding: 5px;"> SITE No. 26-141 </td> <td style="padding: 5px;"> W.P. No. B-62 </td> </tr> <tr> <td colspan="2" style="padding: 5px;"> BRIDGE ENGINEER _____ </td> <td colspan="2" style="padding: 5px;"> CONTRACT Nos. _____ </td> </tr> <tr> <td style="padding: 5px;"> DESIGN W.L.L. _____ DRAWING D.H.B. _____ </td> <td style="padding: 5px;"> CHECK _____ LOADING _____ </td> <td colspan="2" style="padding: 5px;"> DRAWING No. D-5881-P </td> </tr> <tr> <td style="padding: 5px;"> DATE 4/1/1966 </td> <td style="padding: 5px;"> H2O - 516 </td> <td colspan="2"></td> </tr> </table> | | APPROVED _____ | | SITE No. 26-141 | W.P. No. B-62 | BRIDGE ENGINEER _____ | | CONTRACT Nos. _____ | | DESIGN W.L.L. _____ DRAWING D.H.B. _____ | CHECK _____ LOADING _____ | DRAWING No. D-5881-P | | DATE 4/1/1966 | H2O - 516 | | |
| APPROVED _____ | | SITE No. 26-141 | W.P. No. B-62 | | | | | | | | | | | | | | |
| BRIDGE ENGINEER _____ | | CONTRACT Nos. _____ | | | | | | | | | | | | | | | |
| DESIGN W.L.L. _____ DRAWING D.H.B. _____ | CHECK _____ LOADING _____ | DRAWING No. D-5881-P | | | | | | | | | | | | | | | |
| DATE 4/1/1966 | H2O - 516 | | | | | | | | | | | | | | | | |



Appendix B
Site Photographs



Photo 1: North Abutment (East Side) – facing south



Photo 2: North Pier Showing Rip-Rap– facing east



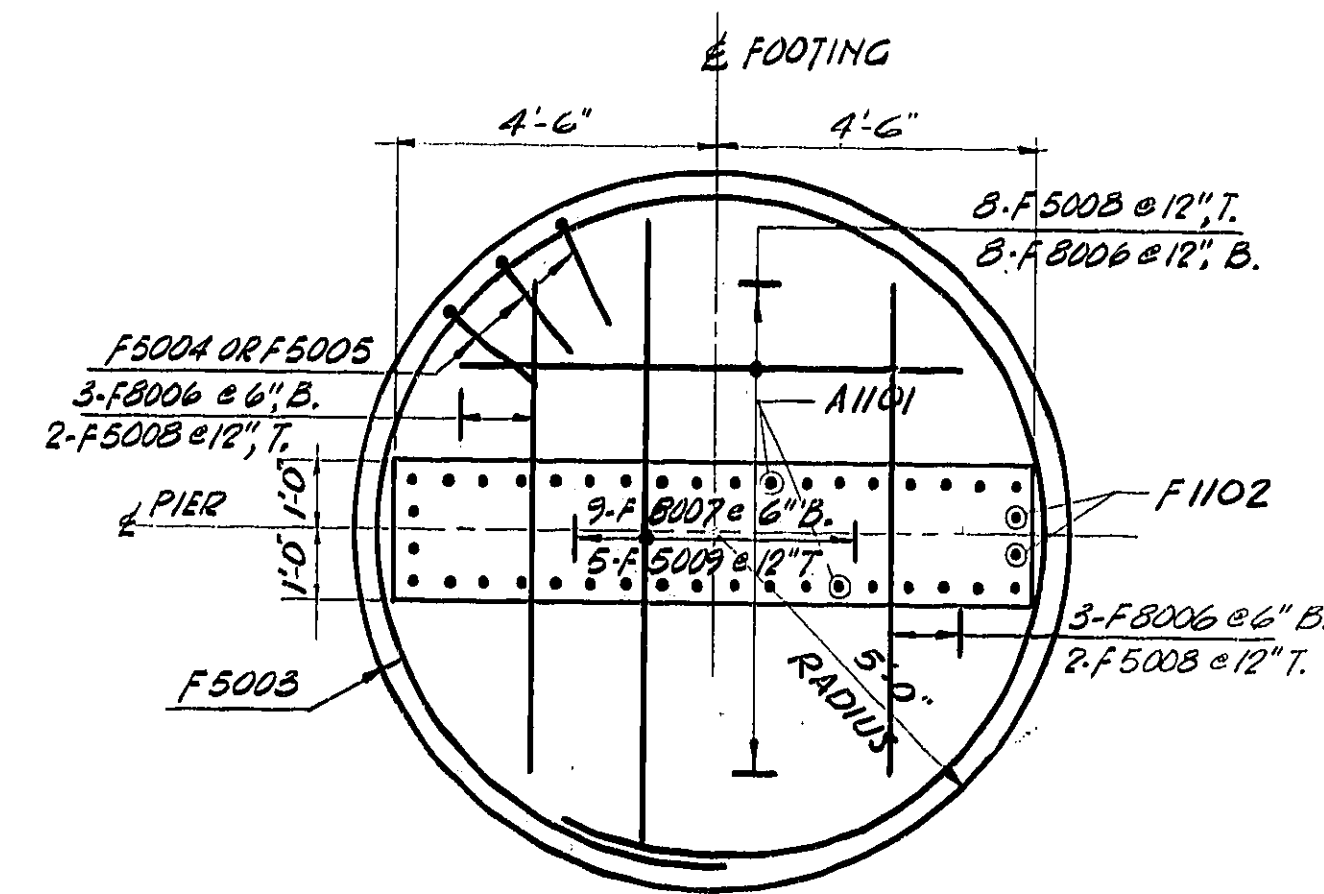
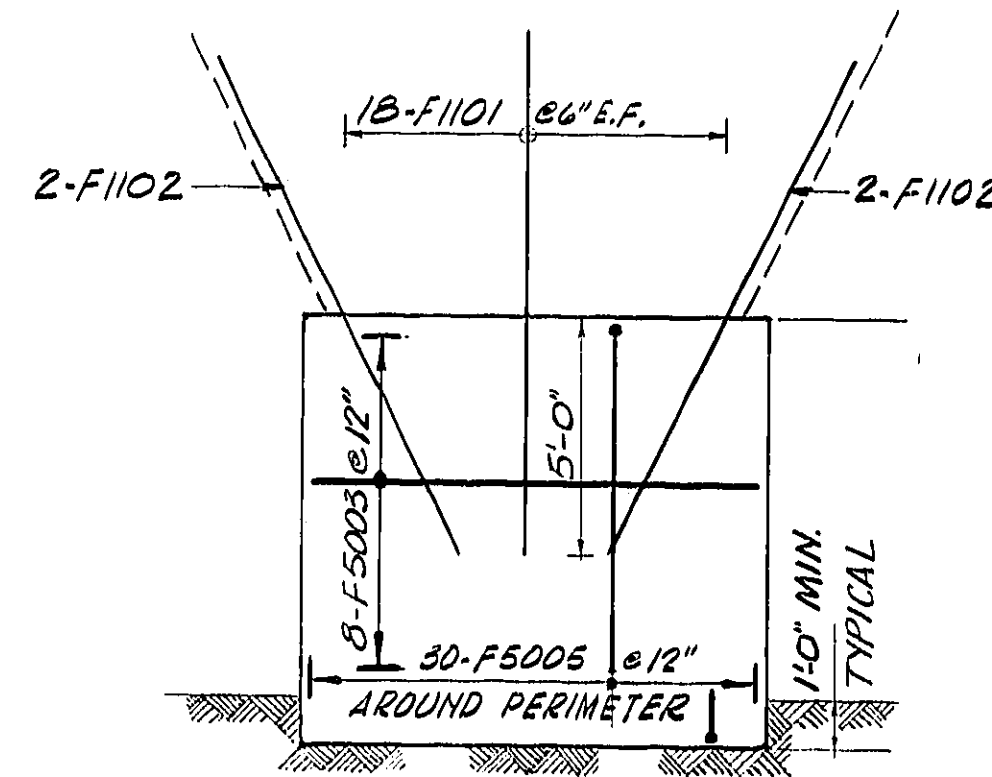
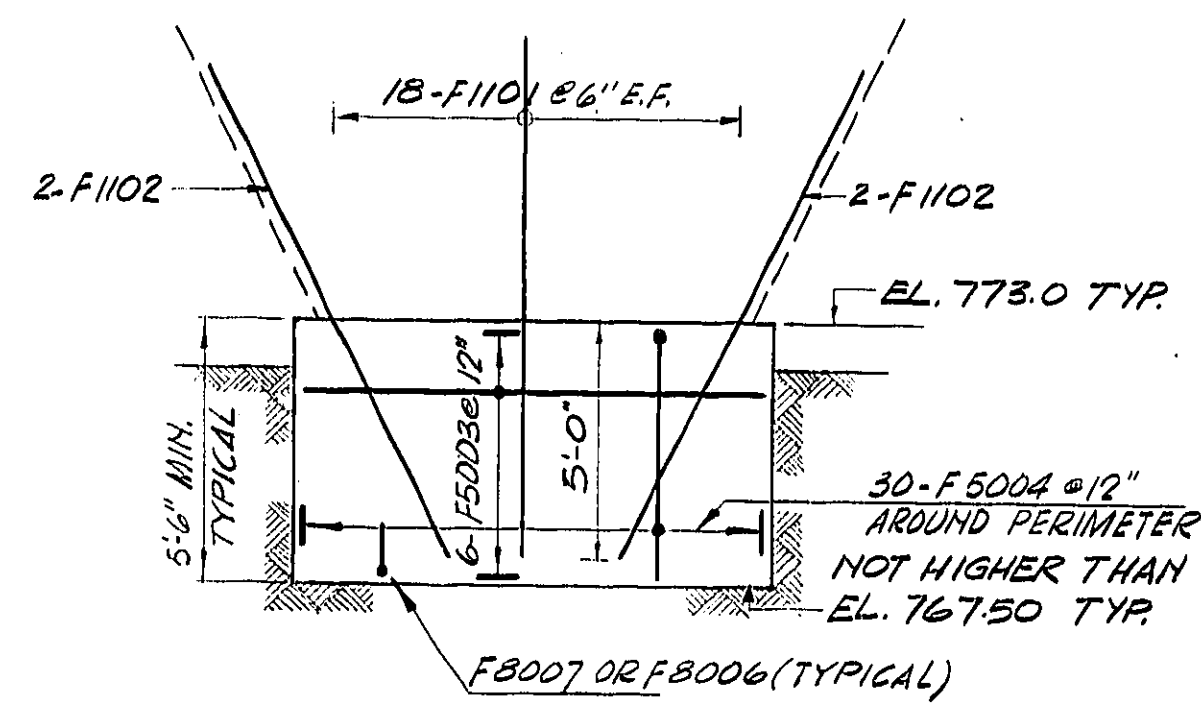
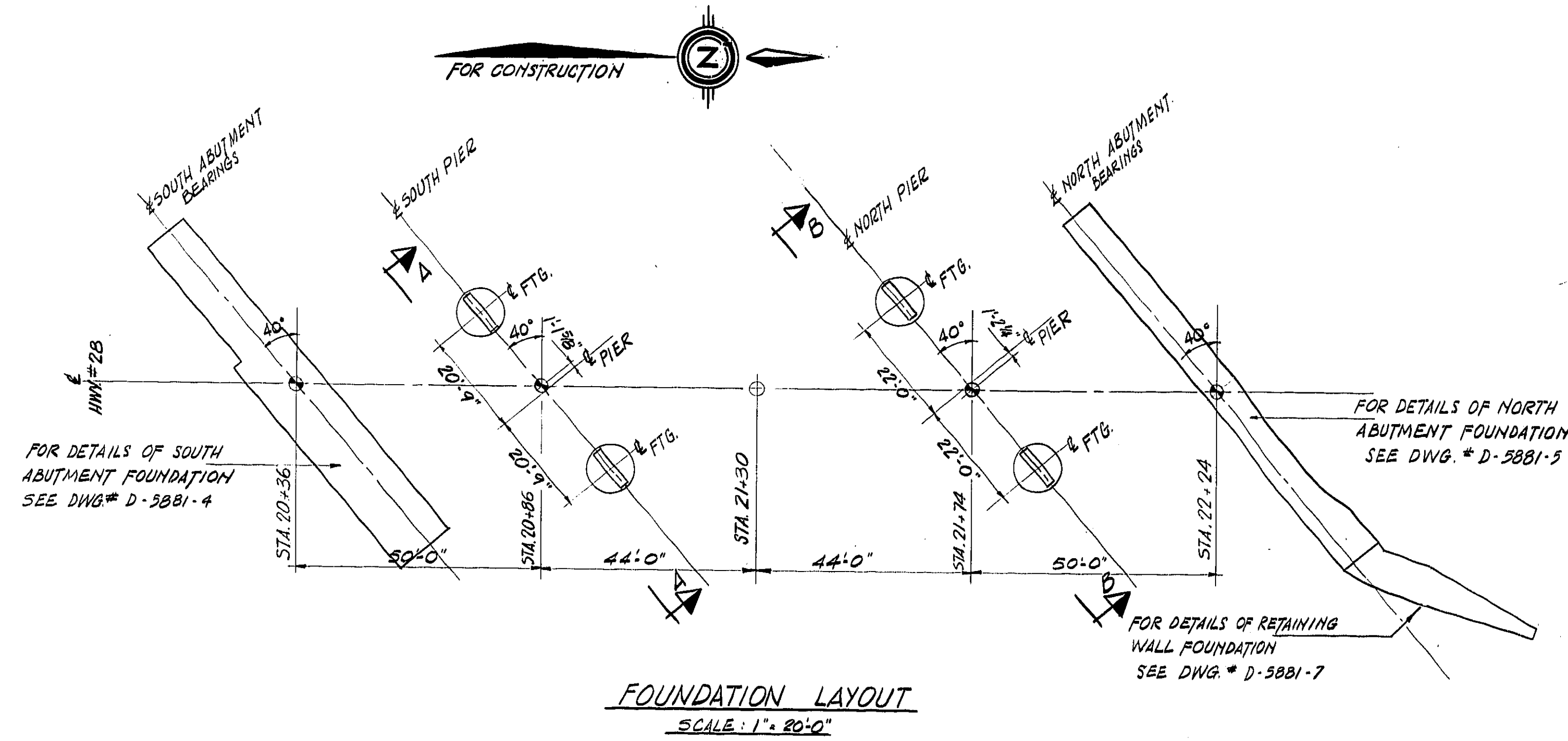
Photo 3: South Pier and South Forward Slope - facing south



Photo 4: North and South Approaches (West Side) - facing south



Appendix C
Archived Drawings



NOTE:
INCREASE DIAMETER OF FOOTING 0.4 FEET FOR EACH FOOT OF EXCAVATION BELOW EL. 763.0. THE INCREASED DIAMETER SHALL BE USED FOR THE FULL HEIGHT OF THE FOOTING.

FOUNDATION NOTES
1 - FOUNDATIONS TO BE PLACED AGAINST SOUND UNDISTURBED GROUND.

LEGEND:
T. - TOP
B. - BOTTOM
E.F. - EACH FACE

| PRINT RECORD | | |
|--------------|-----|---------|
| No. | FOR | DATE |
| 1 | AS | 28.3.66 |

| REVISIONS | DATE | BY | REV. AS-CONSTRUCTED. | DESCRIPTION |
|-----------|------|----|----------------------|-------------|
| | | | | |

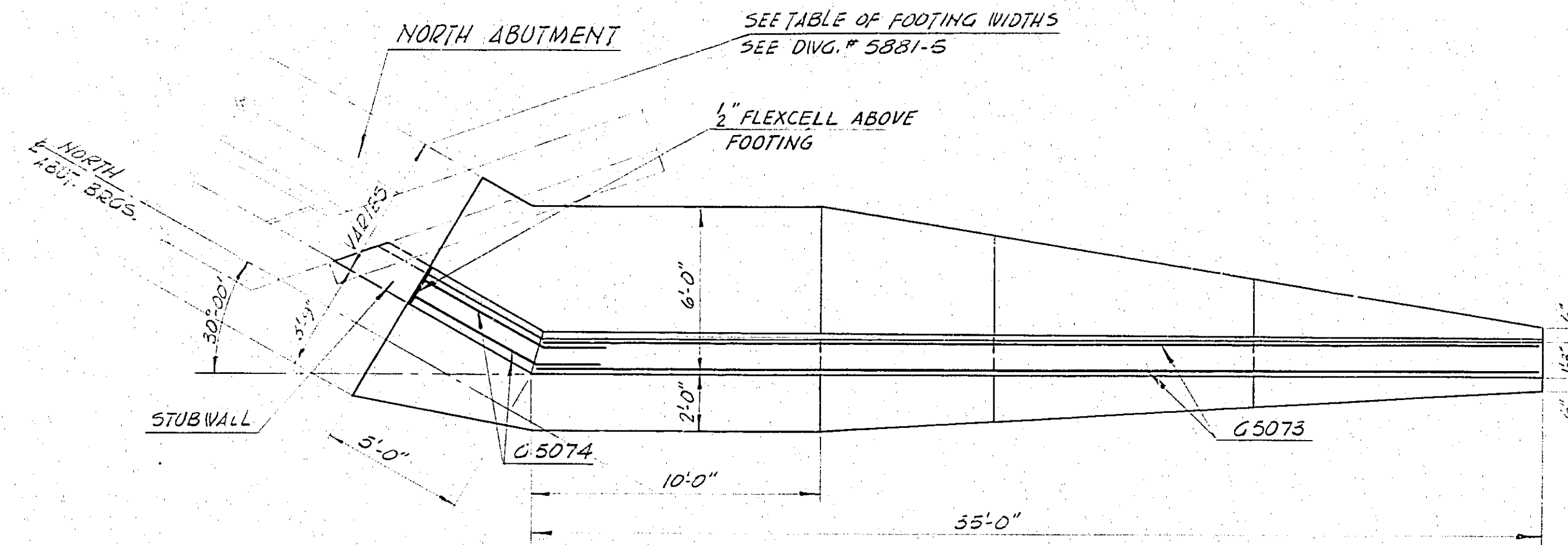
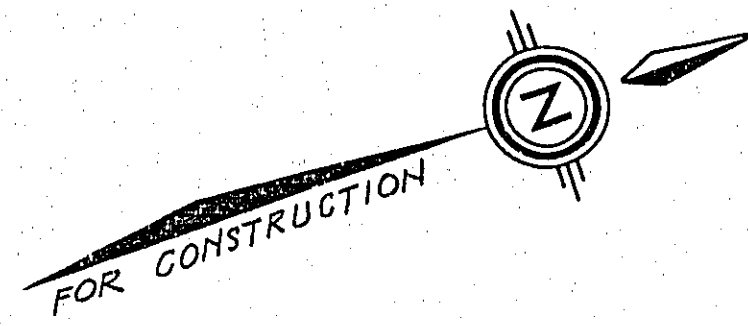
| | | | |
|---|-----------------|--------------|----------|
| DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION | | | |
| LAUGHLIN, WYLLIE & UFNAL CONSULTING ENGINEERS TORONTO | | | |
| PERRY'S CREEK BRIDGE (AT BURLEIGH FALLS) | | | |
| KING'S HIGHWAY No. 28 | DIST. No. 7 | | |
| CO. PETERBOROUGH | LOT 5 | | CON. 1 |
| FOUNDATIONS | | | |
| APPROVED | DESIGNER | SITE No. | W.P. No. |
| DESIGN K.V.S. | CHECK D.K.M.M. | 26-141 | 8-62 |
| DRAWING A.W.S. | CHECK D.K.M.M. | CONTRACT No. | 66215 |
| DATE FEB. 1966 | LOADING H20-316 | DRAWING No. | D-5881-3 |



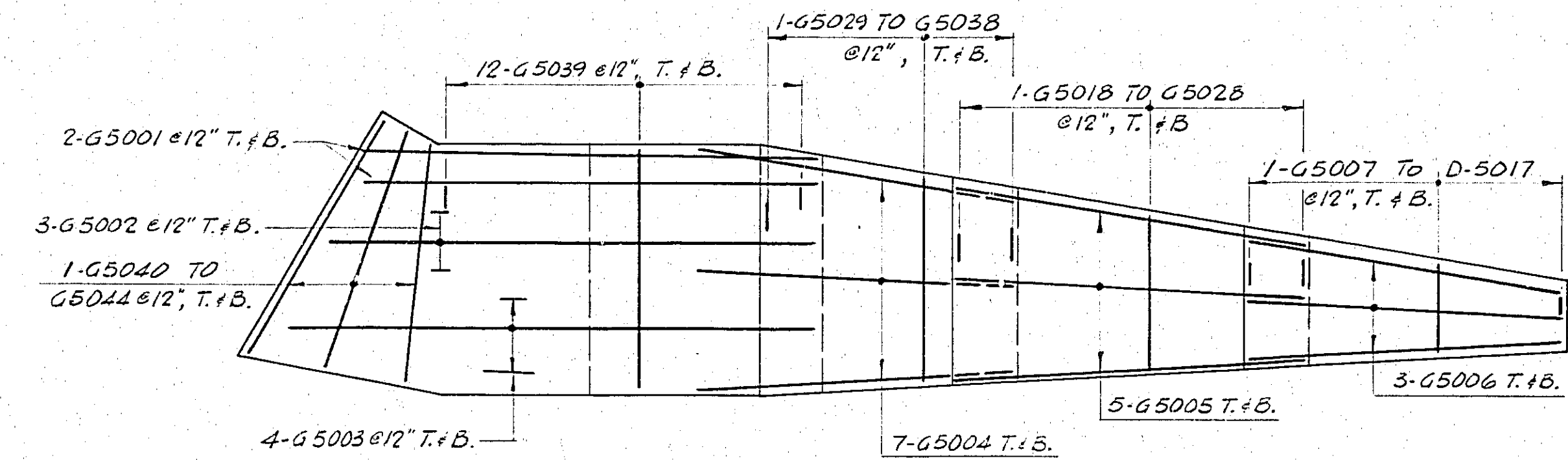
71609

302-13-3

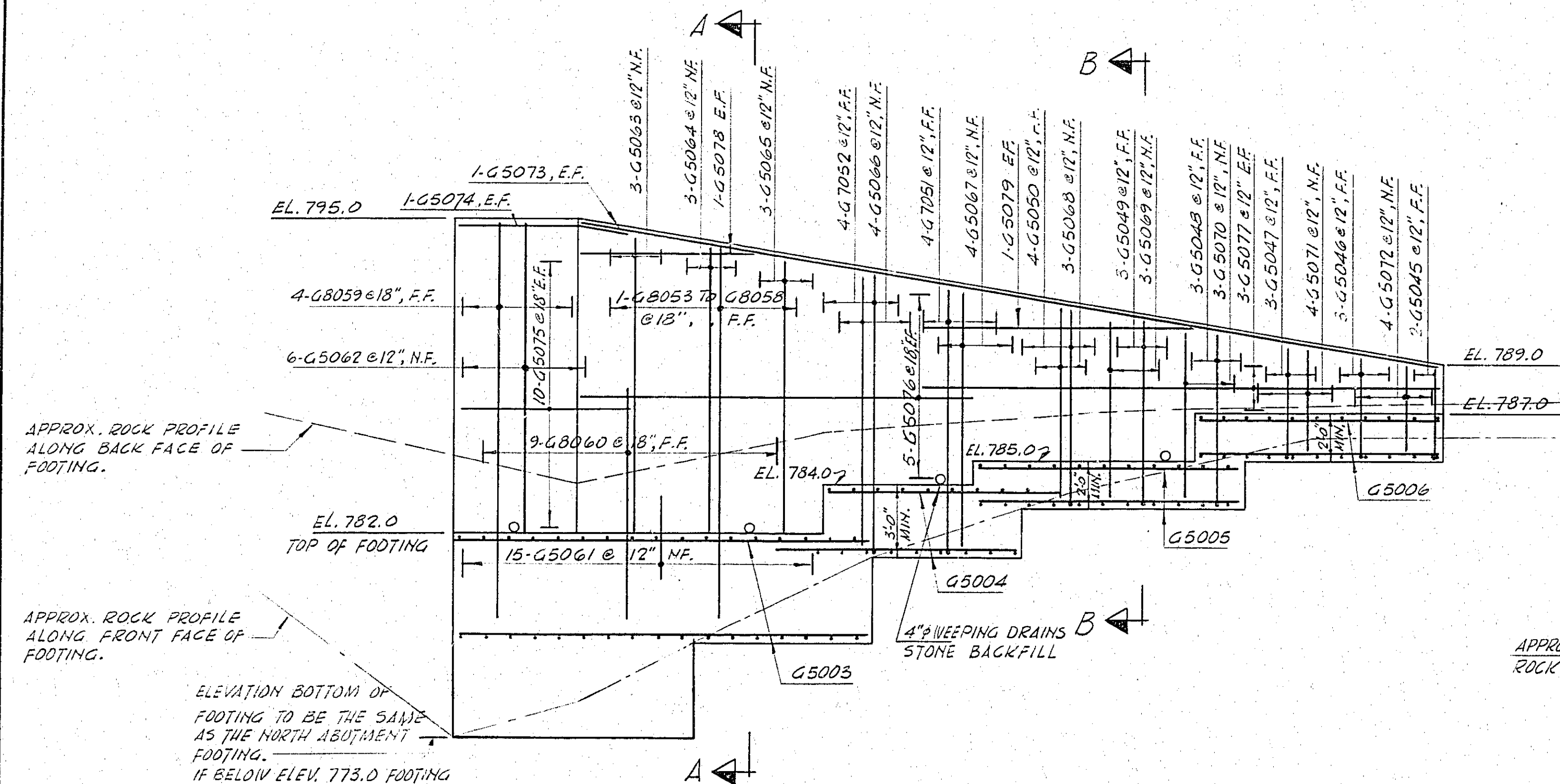
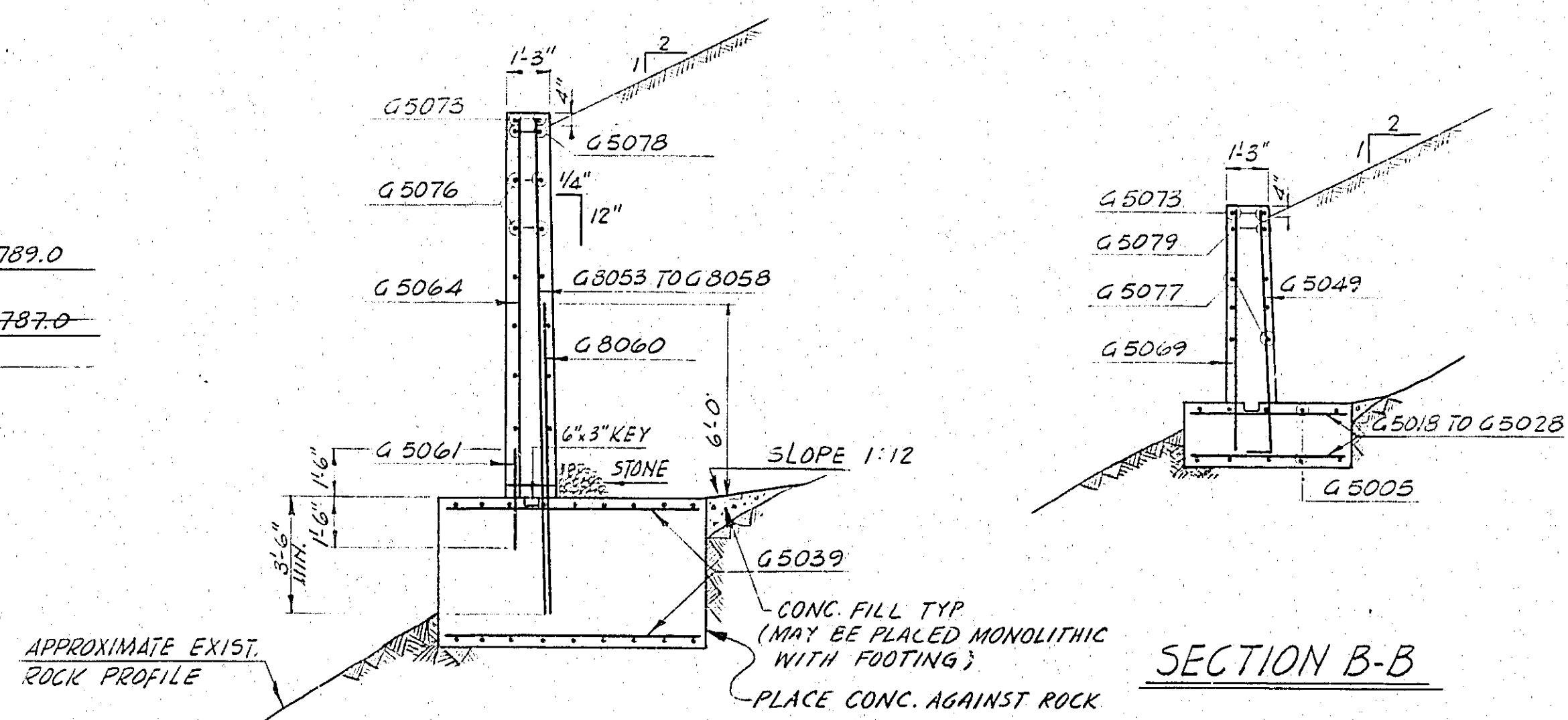
TWP# 302-141-3-4



PLAN



FOOTING PLAN

ELEVATION

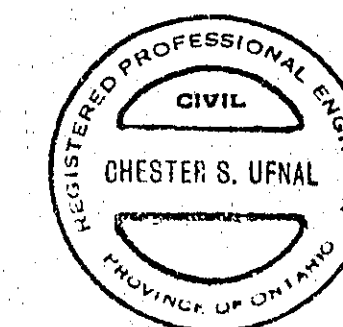
SECTION A-A

SECTION B-B

LEGEND:
T. - TOP
B. - BOTTOM
N.F. - NEAR FACE
F.F. - FAR FACE

[illegible]

| | | | |
|--|------------------------|-----------------------------|----------------------|
| DEPARTMENT OF HIGHWAYS ONTARIO | | | |
| BRIDGE DIVISION | | | |
| LAUGHLIN, WYLLIE & UFNAL CONSULTING ENGINEERS TORONTO | | | |
| <u>PERRYS CREEK BRIDGE</u> (AT BURLEIGH FALLS) | | | |
| KING'S HIGHWAY No. <u>28</u> | | DIST. No. <u>7</u> | |
| CO. <u>PETERBOROUGH</u> | | | |
| TWP. <u>HARVEY & BURLEIGH</u> | | LOT <u>5</u> | CON. <u>1</u> |
| <u>RETAINING WALL</u> | | | |
| APPROVED <u>[Signature]</u> | | SITE No. <u>20-101</u> | W.P. No. <u>8-62</u> |
| BRIDGE ENGINEER | | | |
| DESIGN <u>D.McH.</u> | CHECK <u>D.C.B.</u> | CONTRACT No. | <u>GG-215</u> |
| DRAWING <u>G.S.</u> | CHECK <u>D.K.McH.</u> | | |
| DATE <u>FEB 1946</u> | LOADING <u>W20-516</u> | DRAWING No. <u>D-5881-7</u> | |

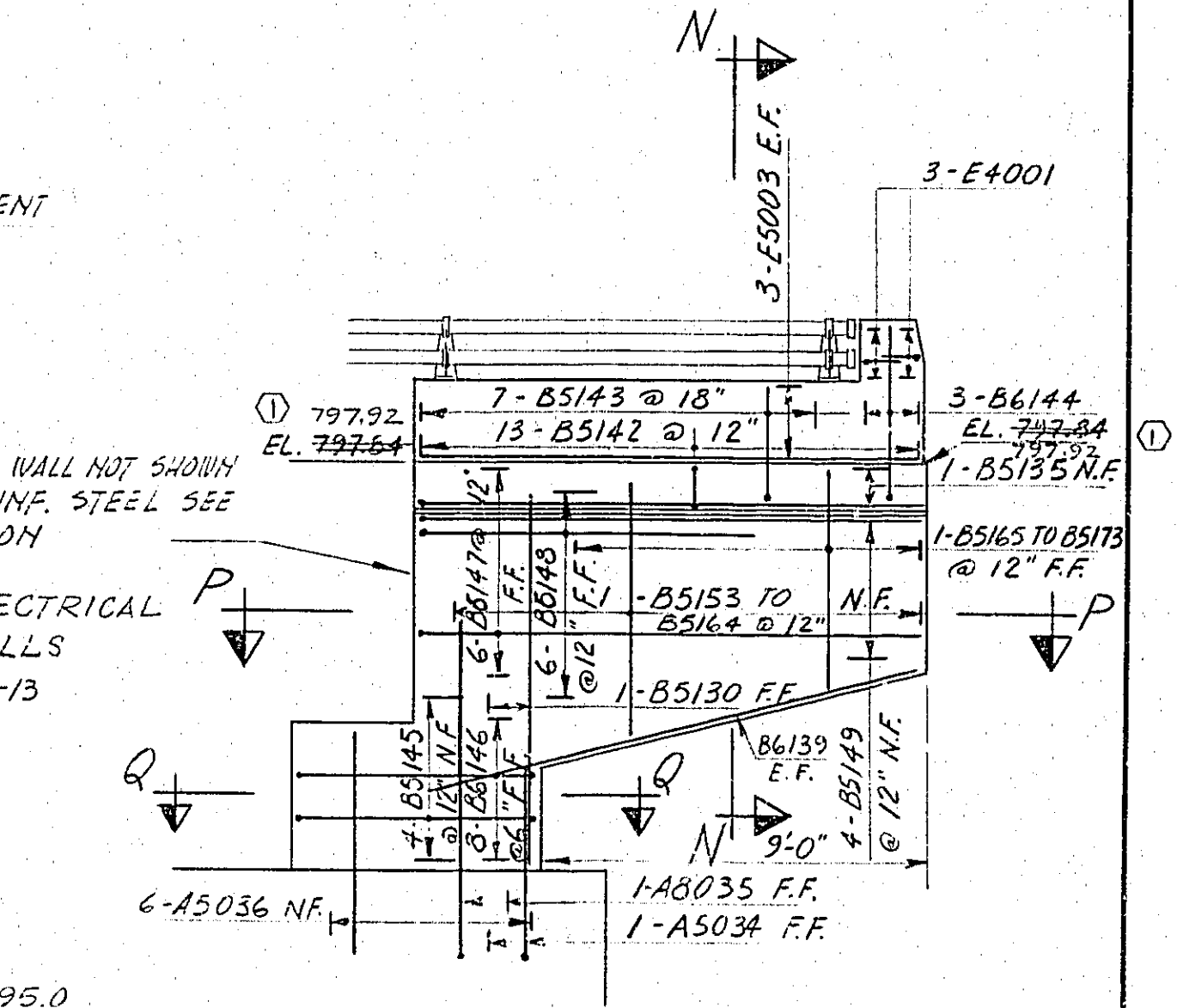
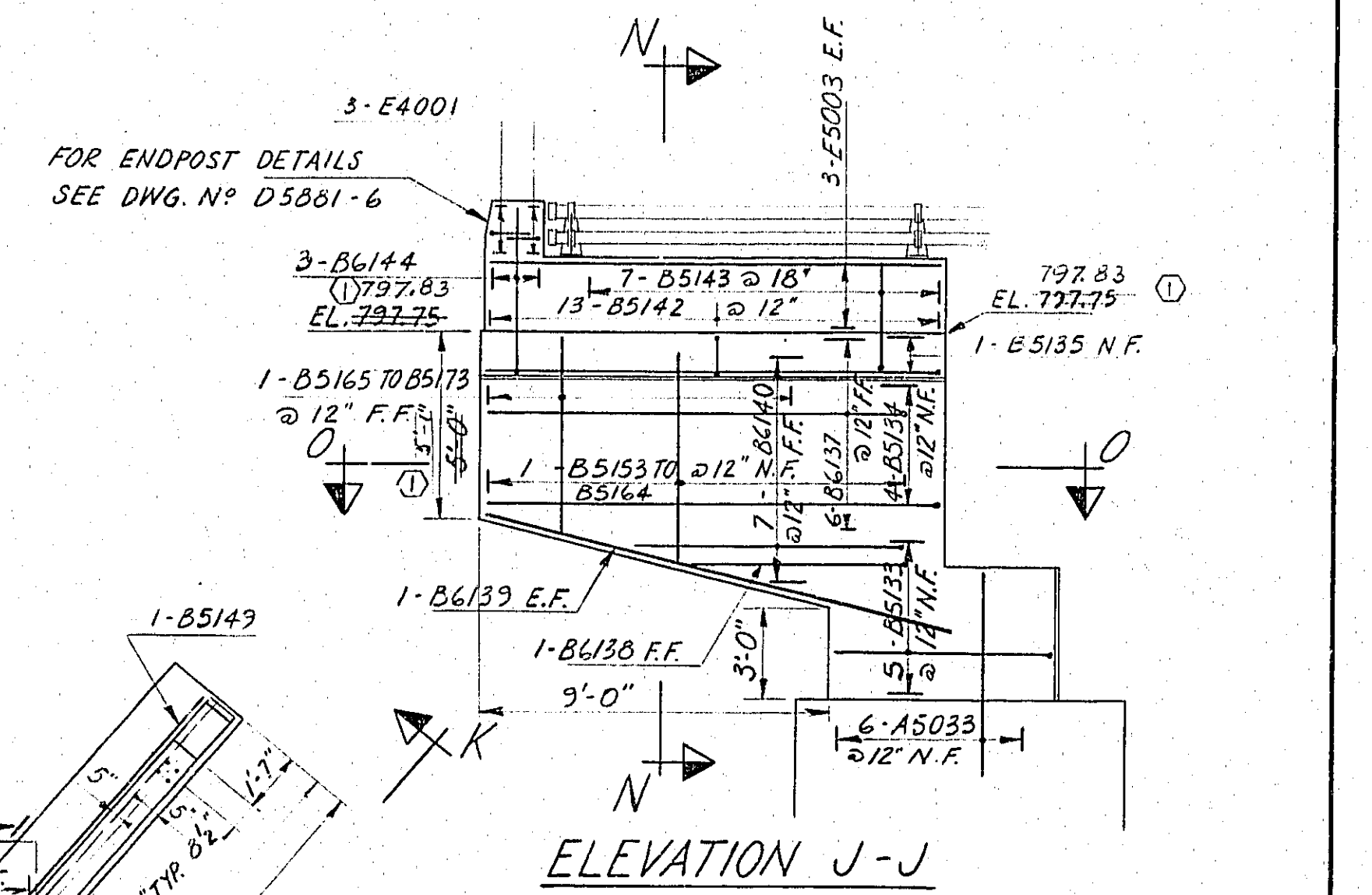
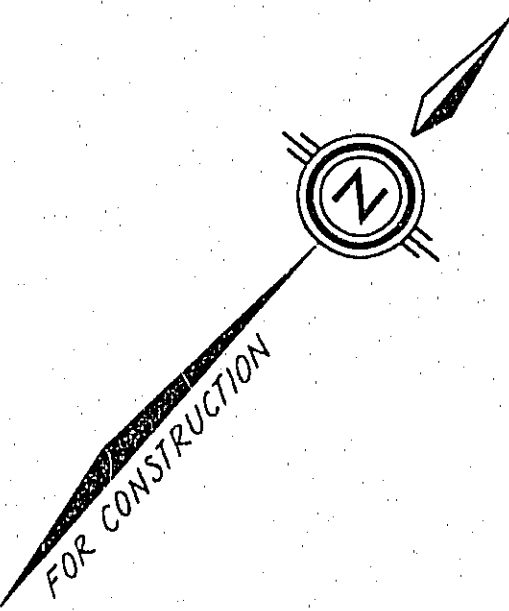


SCALE: 1/4" = 1'-0" UNLESS NOTED

| TABLE OF FOOTING WIDTHS | |
|-----------------------------|---------------------------|
| BOTTOM OF FOOTING ELEVATION | REQUIRED WIDTH OF FOOTING |
| 785.5 TO 779.0 | 7'-0" |
| 779.0 TO 777.0 | 8'-0" |
| 777.0 TO 773.0 | 9'-0" |

NOTE: IF THE ROCK IS UNSOUND AT ELEV. 773.0 FOOTING DESIGN TO BE REVISED BY THE ENGINEER.

NOTE: WINGWALLS PARALLEL TO EXTERIOR GIRDERS



NOTES:
- FOR WALL SECTIONS SEE DWG. NO. D5881-6
- FOR FOOTING LOCATION & FOUNDATION NOTES SEE DWG. NO. D5881-3

| REVISIONS | DATE | BY | DESCRIPTION |
|-----------|------|----|---|
| 17.10.68 | 425 | | 1) ELEV. & DIMENSIONS REVISED AS-CONSTRUCTED. |

| | | | |
|--|-----------------|-------------------------------------|---------------|
| DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION | | | |
| LAUGHLIN, WYLLIE & UFNAL CONSULTING ENGINEERS TORONTO | | | |
| PERRYS CREEK BRIDGE (AT BURLEIGH FALLS) | | | |
| KING'S HIGHWAY No. 28 | | DIST. No. 7 | |
| CO. PETERBOROUGH | | TWP. HARVEY & BURLEIGH LOT 5 CON. I | |
| NORTH ABUTMENT | | | |
| APPROVED | BRIDGE ENGINEER | SITE No. 26-141 | W.P. No. 8-62 |
| DESIGN D.K.M.M. | CHECK D.C.B. | CONTRACT No. | 66-215 |
| DRAWING M.R.K. | CHECK D.K.M.M. | DRAWING No. | D-5881-5 |
| DATE FEB 1964 | LOADING H20-S16 | | |

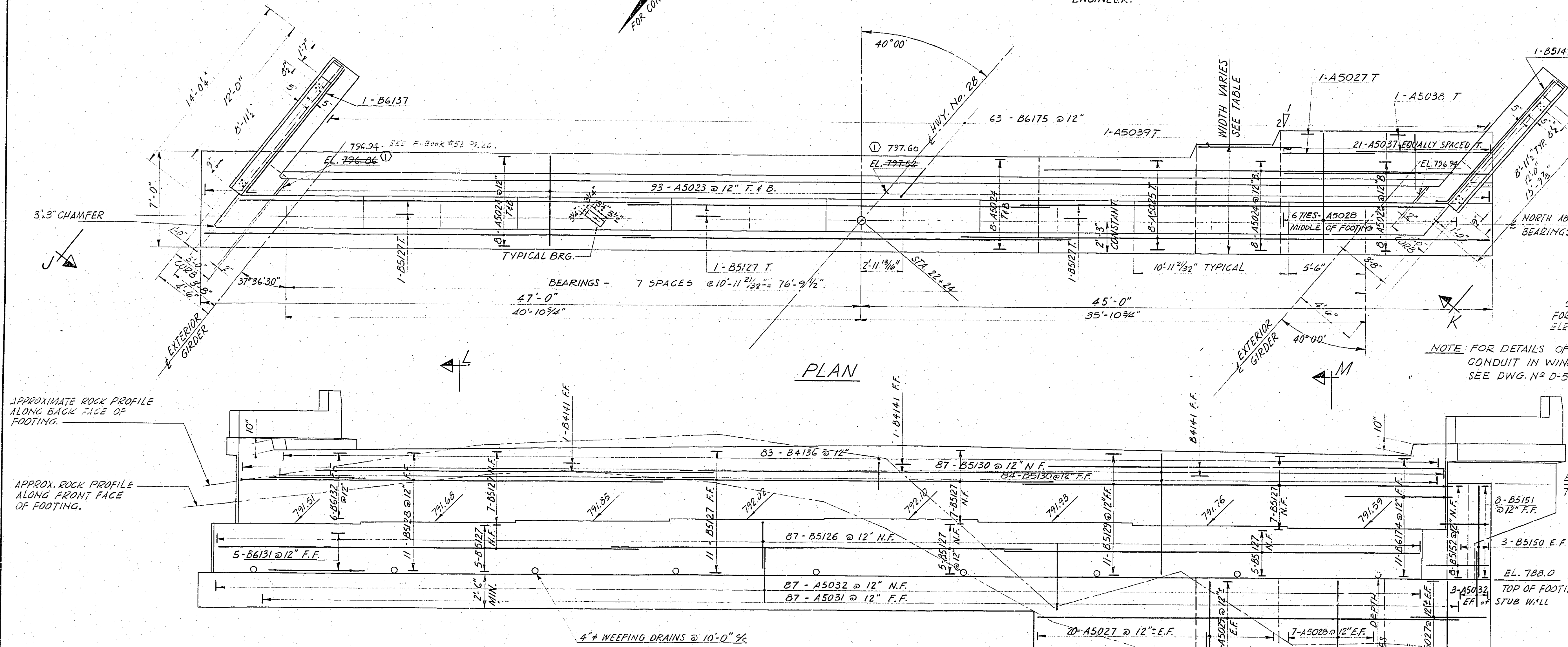
SCALE: 1/4" = 1'-0"

LEGEND
N.F. DENOTES NEAR FACE
F.F. " FAR FACE
E.F. " EACH FACE
T. " TOP
B. " BOTTOM

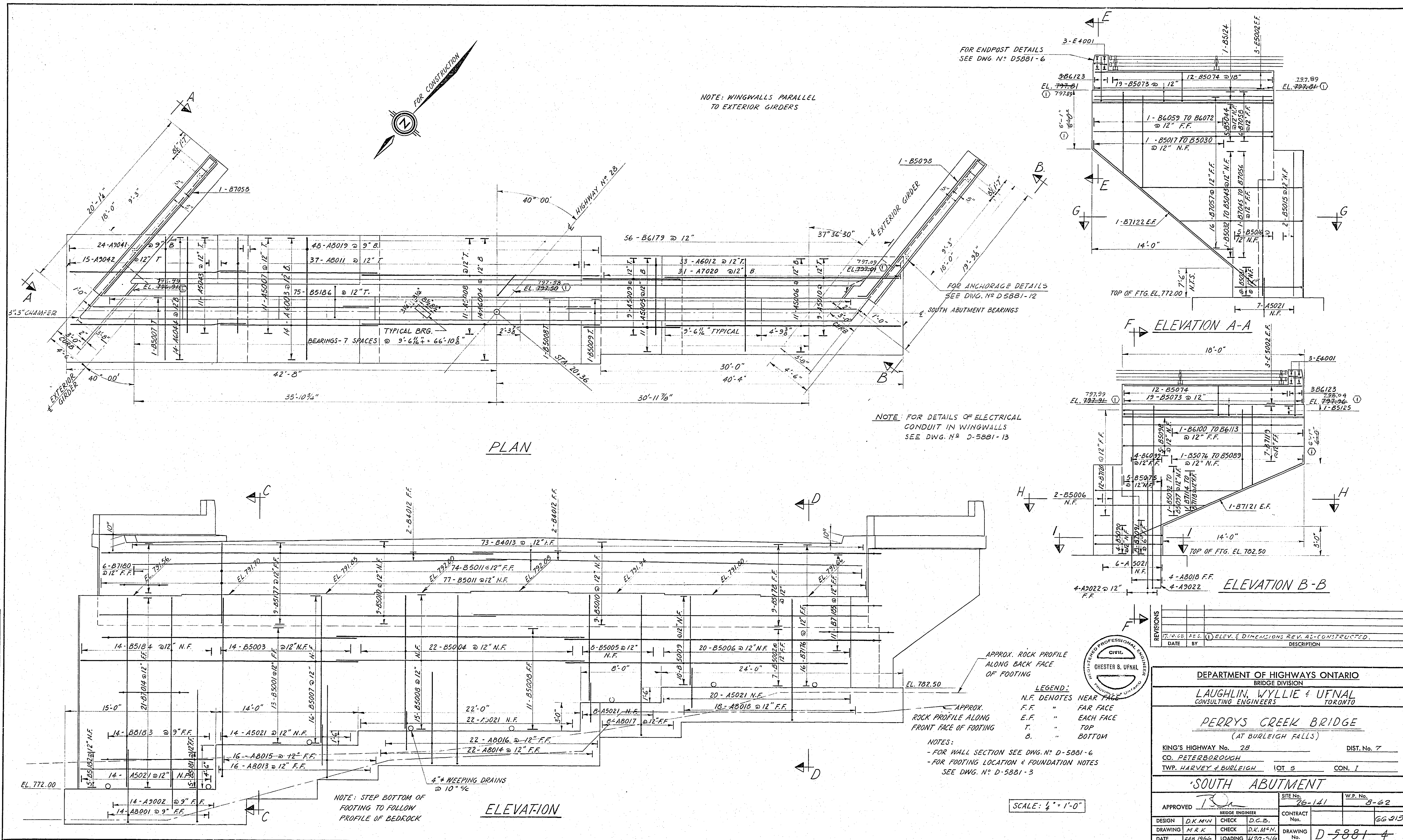
NOTE: STEP BOTTOM OF FOOTING TO FOLLOW PROFILE OF BEDROCK

PLAN

ELEVATION



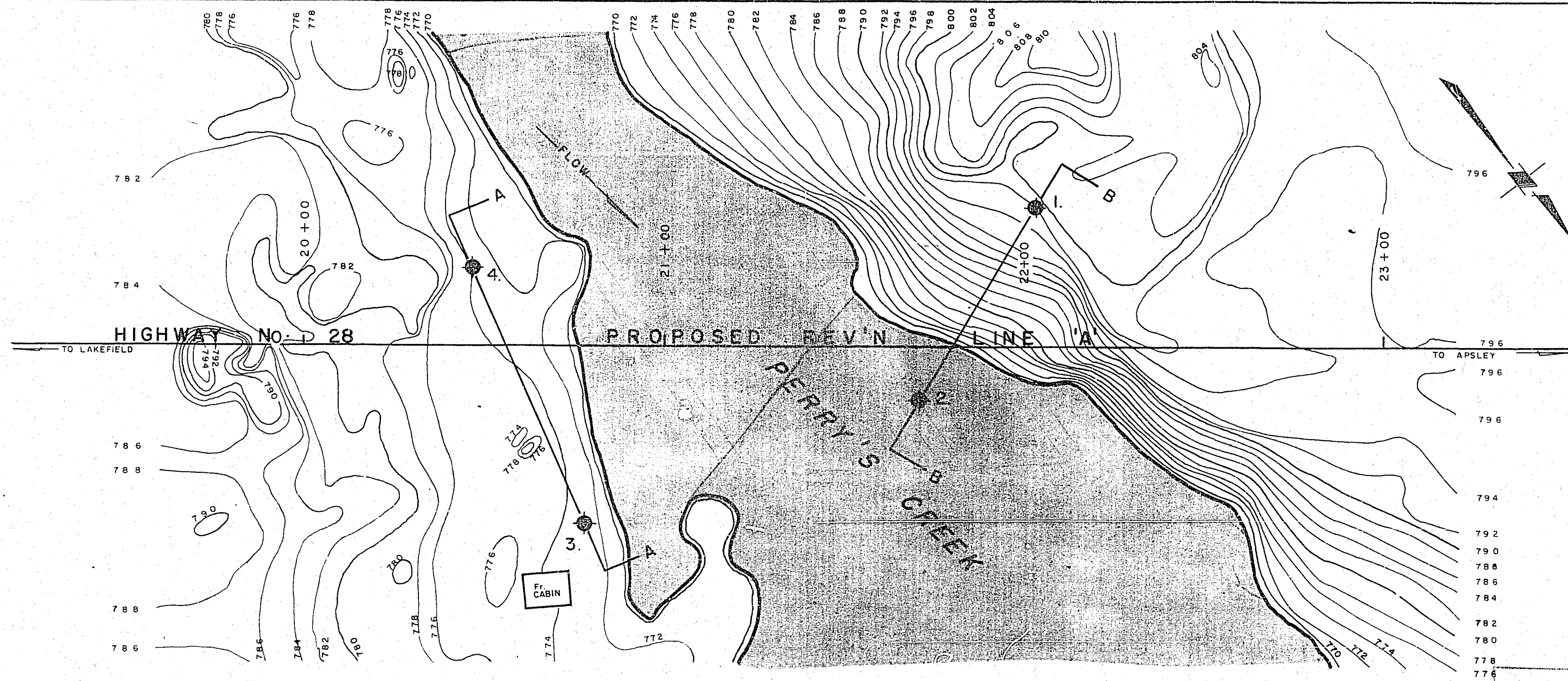
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|--------------|-----|---------|
| No. | FOR | DATE |
| 1 | DS | 12-2-62 |



| PRINT RECORD | | |
|--------------|-----|---------|
| No. | FOR | DATE |
| 1 | DS | 12.3.88 |

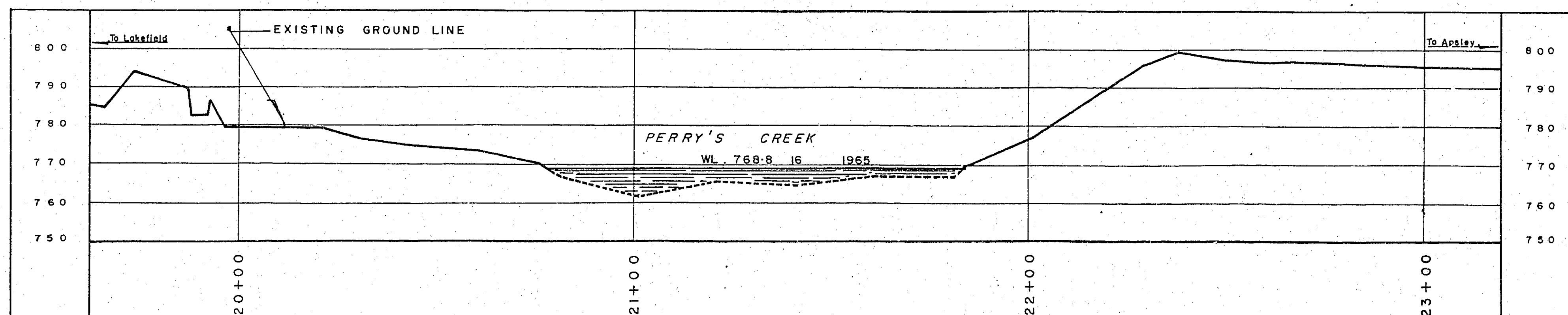
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|-----------|---------|---|
| NO. | DATE | DESCRIPTION |
| 1 | 12.3.88 | ELEV. & DIMENSIONS REV. AS CONSTRUCTED. |

| | |
|---|--------------------------------|
| DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION | |
| LAUGHLIN, WYLLIE & UFNAL CONSULTING ENGINEERS TORONTO | |
| PERRY'S CREEK BRIDGE (AT BURLEIGH FALLS) | |
| KING'S HIGHWAY No. 28 CO. PETERBOROUGH TWP. HARVEY & BURLEIGH | DIST. No. 7 LOT 5 CON. 1 |
| SOUTH ABUTMENT | |
| APPROVED: [Signature] | SITE No. 26-141 W.P. No. 8-62 |
| DESIGN: D.K.M.N. CHECK: D.G.B. | CONTRACT No. 66215 |
| DRAWING: M.R.K. CHECK: D.K.M.N. | DRAWING No. D-5881-4 |
| DATE: FEB 1966 | LOADING: W20-516 |



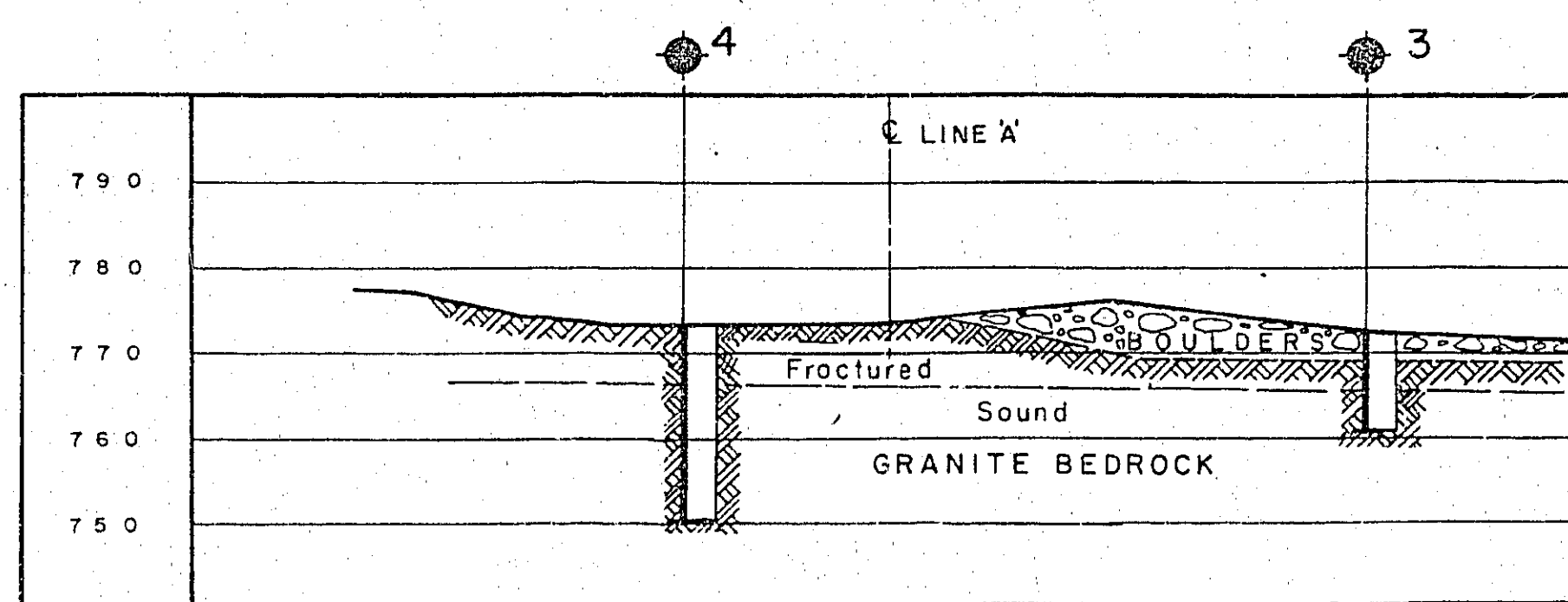
PLAN

SCALE IN FEET: 20 10 0 20 40 60



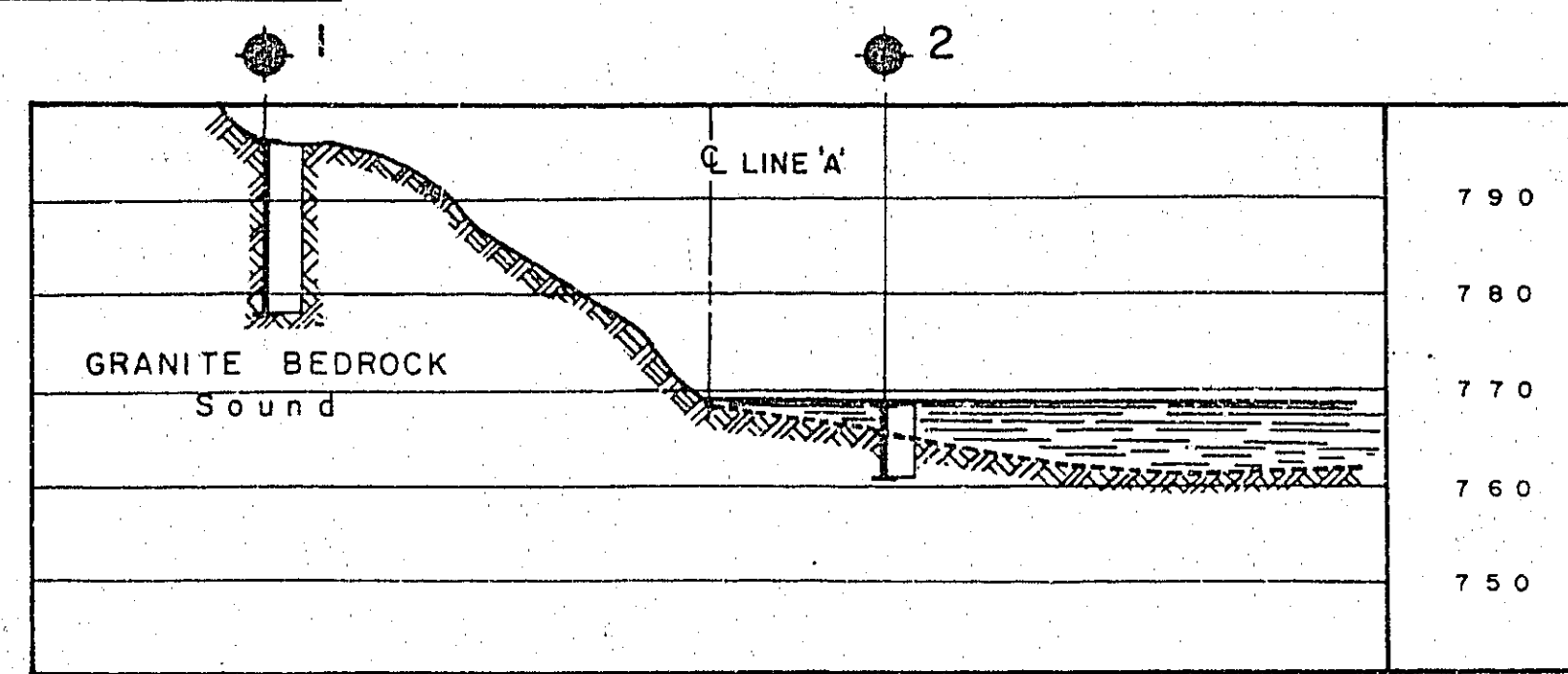
PROFILE

SCALE IN FEET: 20 10 0 20 40 60



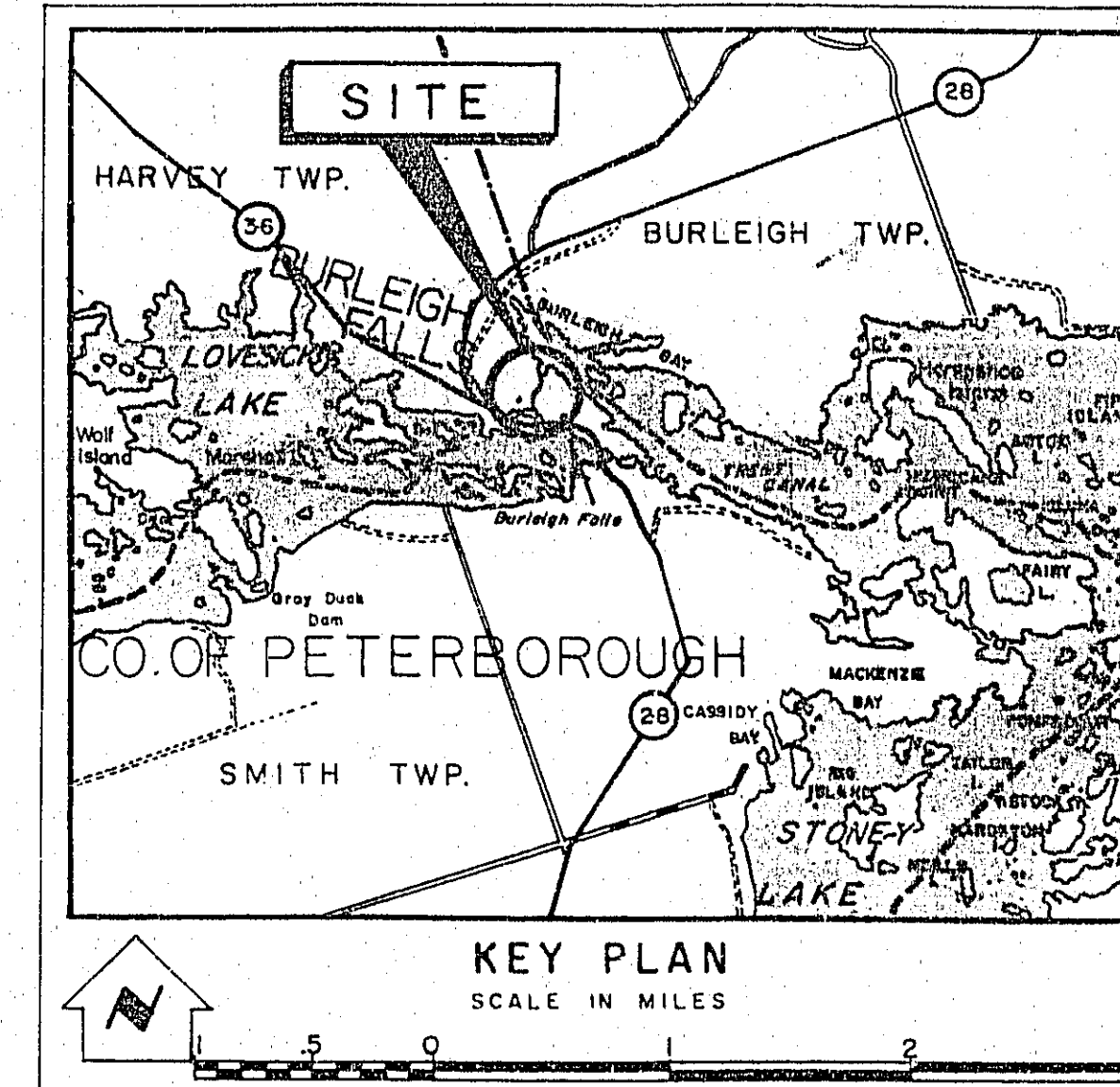
A — A

SCALE IN FEET: 20 10 0 20 40 60



B — B

SECTIONS



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation, DEC. 1965

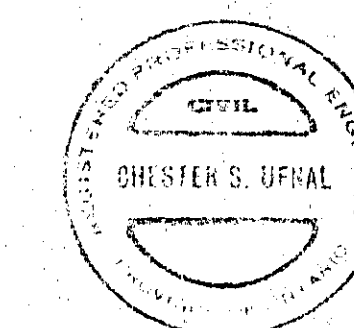
| NO. | ELEVATION | STATION | OFFSET |
|-----|-----------|---------|--------|
| 1. | 795.9 | 22 + 04 | 40' LT |
| 2. | 768.8 | 21 + 70 | 15' RT |
| 3. | 772.6 | 20 + 78 | 50' RT |
| 4. | 773.0 | 20 + 47 | 22' LT |

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

| PRINT RECORD | NO. | FOR | DATE |
|--------------|-----|-----|---------|
| | 1 | 65 | 21-1-66 |

| REVISIONS | DATE | BY | DESCRIPTION |
|-----------|----------|--------|--------------------|
| 1 | 17-10-66 | W.F.C. | REVISED 30-10-1966 |



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

PERRY'S CREEK

KING'S HIGHWAY NO. 28 PROPOSED REV'N LINE A DIST. NO. 7
CO. PETERBOROUGH BURLEIGH FALLS
TWP. OF HARVEY LOT 5 CON. 1

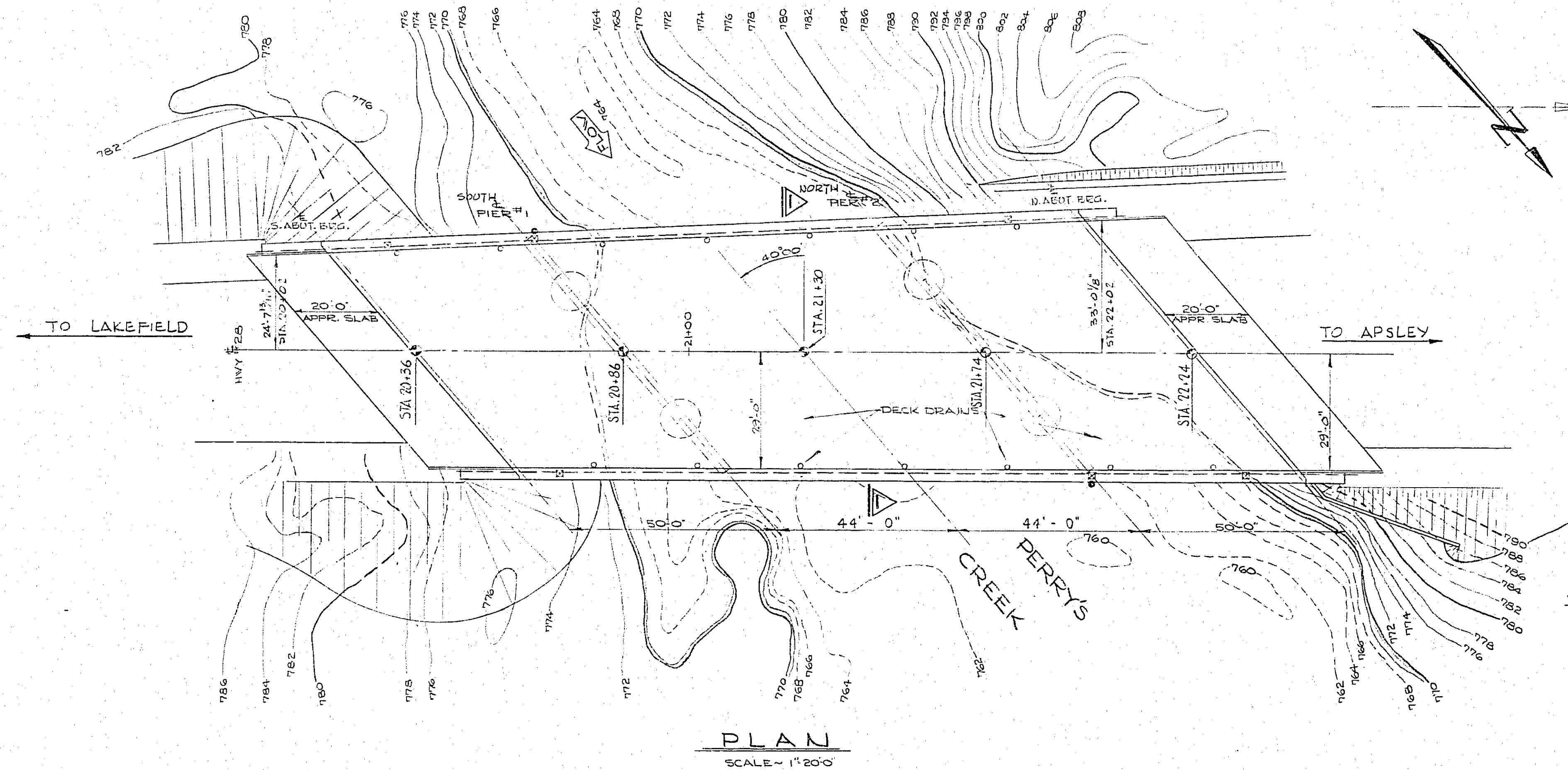
BORE HOLE LOCATIONS & SOIL STRATA

| | | | |
|-----------------------------|------------------|--------------------|--------------------|
| SUBM'D V.K. | CHECKED M.D. | W.P. NO. 8-62 | M.S.T. DRAWING NO. |
| DRAWN J.N. | CHECKED W.F.C. | JOB NO. 65-F-129 | 65-F-129 A |
| DATE 26 JAN. 1966 | SITE NO. 213-141 | BRIDGE DRAWING NO. | |
| APPROVED <i>[Signature]</i> | CONT. NO. 66-215 | | |

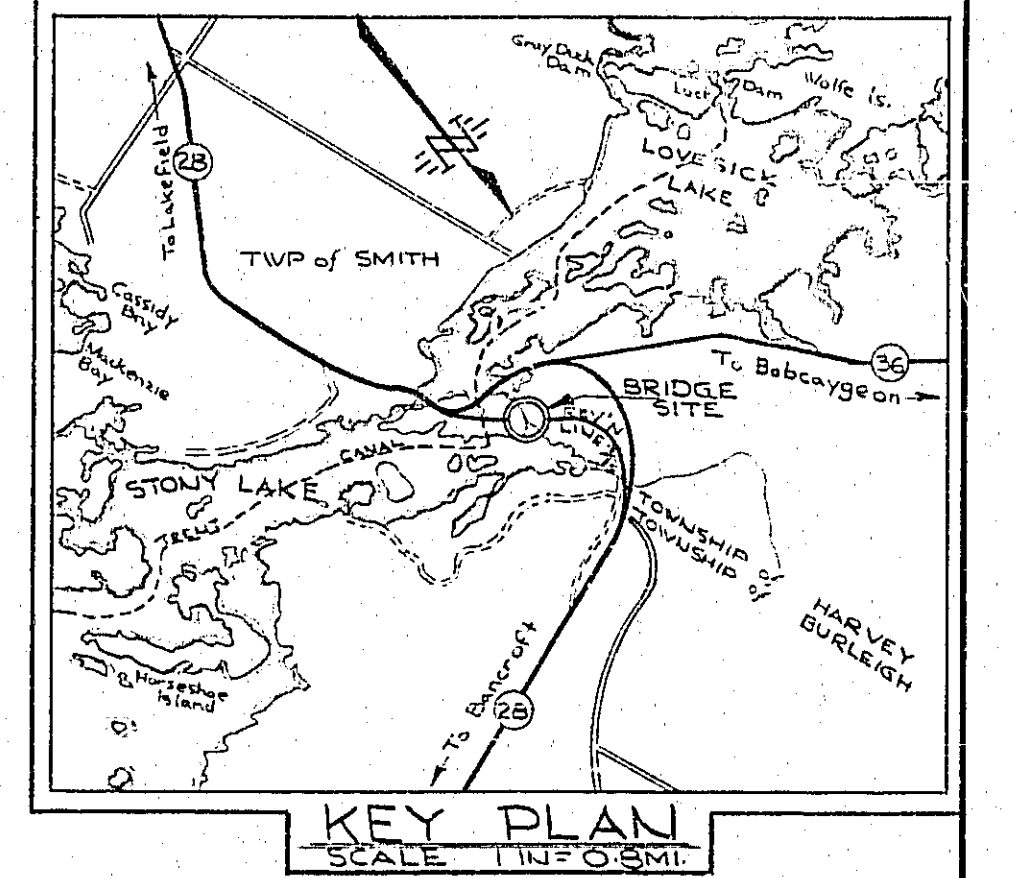
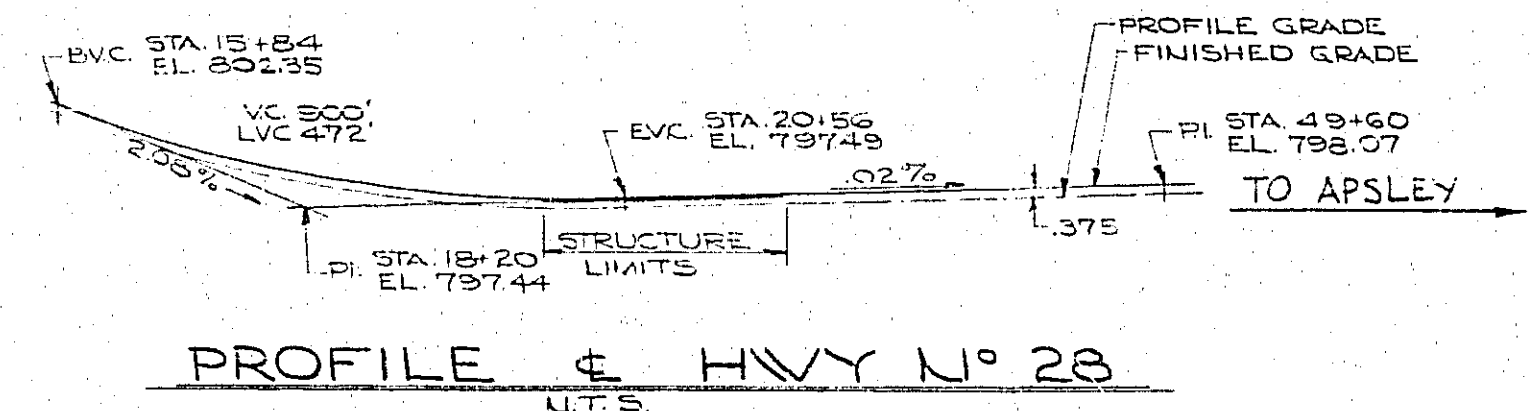
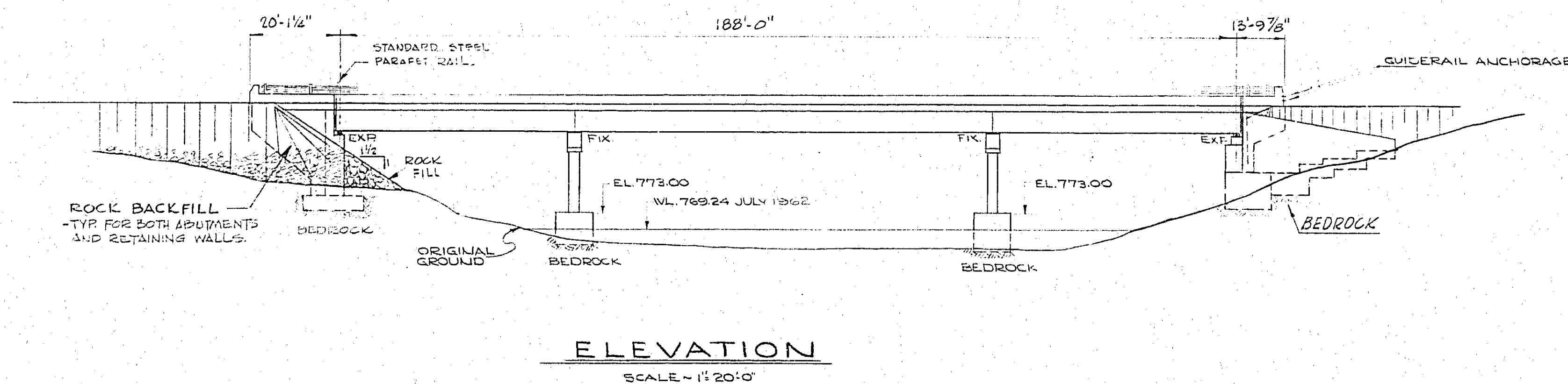
REF NO: E-4127-1

302-13-2

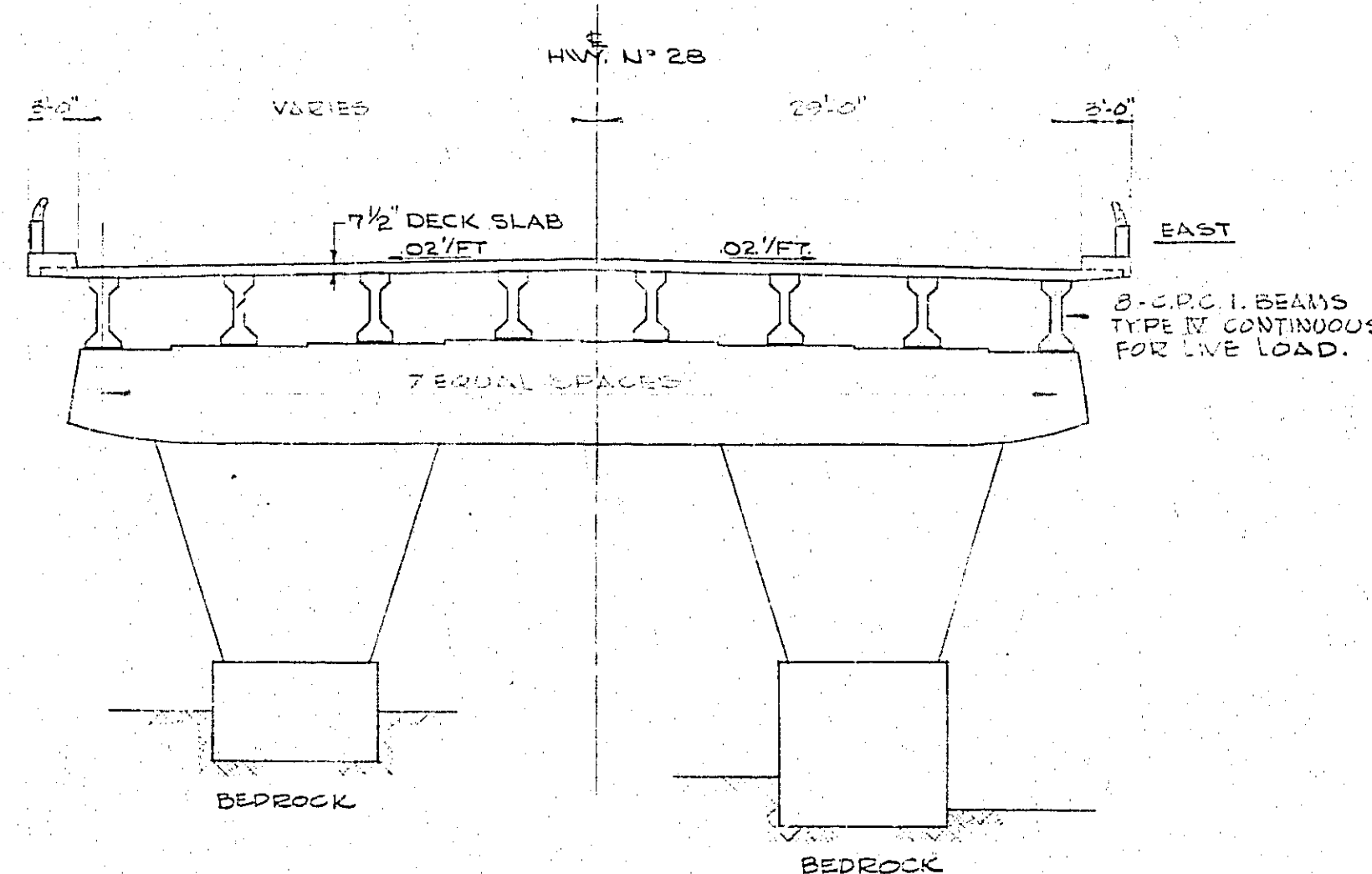
TWP=302-141-2-A



B.M. ELEV. 773.89
GEODETIC DATUM
S.I.B. IN GRANITE BOULDER
51'-0" LT. OF STA. 20+60



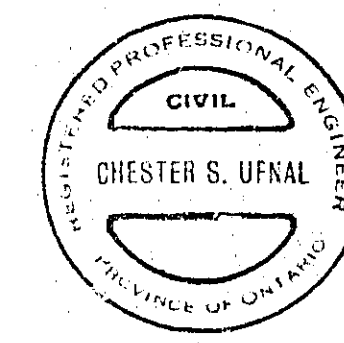
NOTES
CLASS OF CONCRETE:
1 PRECAST MEMBERS 5,000 P.S.I., DECK SLAB, DIAPHRAGMS AND PIERS (EXCEPT FOOTINGS) 4,000 P.S.I. - REMAINDER 3,000 P.S.I.
2 CLEAR COVER:
FOOTINGS ABUTMENTS & PIERS - 3" CURBS - 2"
PRECAST MEMBERS - 1 1/2"
DECK SLAB - BOTTOM 1", TOP 2"
3 CONSTRUCTION NOTES:
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF 1/8".



ELECTRICAL ITEMS:
LEGEND
• LIGHTING POLE LOCATION
x 8'-8" JUNCTION BOX (EVERDUR)
--- 3" PUCT (TRANSITE TYPE II OR EQUAL)

LIST OF DRAWINGS

- D-5881-1 GENERAL LAYOUT
- 2 BORE HOLE LOCATION & SOIL STRATA
- 3 FOUNDATIONS
- 4 SOUTH ABUTMENT
- 5 NORTH ABUTMENT
- 6 ABUTMENT SECTIONS
- 7 RETAINING WALL
- 8 PIERS
- 9 PRESTRESSED GIRDERS & BEARINGS
- 10 DECK AND DIAPHRAGM DETAILS
- 11 APPROACH SLABS
- 12 STANDARD STEEL PARAPET RAIL
- 13 BRIDGE ELECTRICAL DETAIL
- 14 MISCELLANEOUS DETAILS



| REVISIONS | DATE | BY | DESCRIPTION |
|-----------|----------|------|------------------------|
| 1 | 17.10.66 | A.E. | REVISED AS CONSTRUCTED |

| | | | |
|---|-------------------------|---------------------------------------|---------------|
| DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION | | | |
| LAUGHLIN WYLLIE & UPFAL CONSULTING ENGINEERS TORONTO | | | |
| PERRY'S CREEK BRIDGE (AT BURLINGHAM FALLS) | | | |
| KING'S HIGHWAY No. 28 | | DIST. No. 7 | |
| CO. PETERBOROUGH | | TWP. HARVEY & BURLINGHAM LOT 5 CON. I | |
| GENERAL LAYOUT | | | |
| APPROVED | BRIDGE ENGINEER | SITE No. 26-141 | W.P. No. 8-62 |
| DESIGN | K.N.S. CHECK D.K.M.C.N. | CONTRACT No. | 66-215 |
| DRAWING | A.W.S. CHECK D.C.B. | DRAWING No. | D-5881-1 |
| DATE | JUN. 1966 | LOADING | H20-S16 |

D-30243-1

26-141

D5881
1 to 14

| PRINT RECORD | No. | FOR | DATE |
|--------------|-----|-----|-------|
| | 1 | 22 | 23/62 |



Appendix D

Additional Investigation

Records of Boreholes
Laboratory Test Results

Borehole Locations Plan and Soil Strata Drawing

RECORD OF BOREHOLE No 15-P01

1 OF 1

METRIC

W.P. 4011-13-01 LOCATION Perry's Creek N 4 936 547.8 E 407 260.6 ORIGINATED BY ES
 HWY 28 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2015.05.07 - 2015.05.07 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
|---------------|--|------------|---------|------|--------------|----------------------------|-----------------|---|----|----|----|---|--|--|---|---|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | WATER CONTENT (%) | | | | | | |
| 243.0 | GROUND SURFACE | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | |
| 0.0 | ASPHALT:(150mm) | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | |
| 0.2 | SAND and GRAVEL, trace to some silt Very Dense Brown Moist (FILL) | | 1 | GS | | | | | | | | | | | | | | |
| 241.9 | Occasional inferred cobbles | | 1 | SS | 78/ 0.175 | | | 242 | | | | | | | | | | 42 48 10 (SI+CL) RUN #1 TCR=22% SCR=0% RQD=0% |
| 1.1 | FILL, mixture of cobbles and boulders, possibly in a sand and gravel matrix Cobbles | | 1 | RUN | | | | 241 | | | | | | | | | | |
| | Cobbles and boulders (up to 500mm) | | 2 | RUN | | | | 240 | | | | | | | | | | RUN #2 TCR=49% SCR=41% RQD=41% |
| | Cobbles and boulders (up to 400mm) | | 3 | RUN | | | | 239 | | | | | | | | | | RUN #3 TCR=83% SCR=76% RQD=76% |
| | Cobbles (25mm to 75mm) | | 4 | RUN | | | | 238 | | | | | | | | | | RUN #4 TCR=6% SCR=4% RQD=0% |
| 236.6 | SAND and GRAVEL, occasional cobbles Dense Brown Wet | | 3 | SS | 45 | | | 237 | | | | | | | | | | |
| 6.4 | GRANITE slightly to moderately weathered, whitish grey and pink, sub-horizontal to sub-vertical joints, strong to very strong | | 5 | RUN | | | | 236 | | | | | | | | | | RUN #5 TCR=84% SCR=80% RQD=79% |
| | Sub-vertical joint (25mm to 100mm) at 6.6m, 6.7m, 6.9m, 7.0m and 7.4m | | | | | | | | | | | | | | | | | |
| | Sub-vertical joint (25mm to 75mm) at 8.0m and 8.1m | | 6 | RUN | | | | 235 | | | | | | | | | | RUN #6 TCR=100% SCR=100% RQD=90% |
| 234.6 | END OF BOREHOLE AT 8.4m. BOREHOLE DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | | | | | | | | | | | | |
| 8.4 | | | | | | | | | | | | | | | | | | |

ONTMT4S 0620.GPJ 2015TEMPLATE(MTO).GDT 3/8/17

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-P02

1 OF 2

METRIC

W.P. 4011-13-01 LOCATION Perry's Creek N 4 936 550.9 E 407 256.7 ORIGINATED BY ES
HWY 28 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2015.05.12 - 2015.02.12 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|--------------|----------------------------|-----------------|---|--|--|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | |
| 243.0 | GROUND SURFACE | | | | | | | | | | | |
| 0.0 | ASPHALT:(150mm) | | | | | | | | | | | |
| 0.2 | CONCRETE:(225mm) | | | | | | | | | | | |
| 242.6 | | | | | | | | | | | | |
| 0.4 | Gravelly SAND , trace silt, occasional inferred cobbles Very Dense Brown Moist (FILL) | | 1 | SS | 65/ 0.250 | | 242 | | | | | 16 80 4 (SI+CL) |
| 241.7 | FILL , mixture of cobbles and boulders, possibly in a sand and gravel matrix Cobbles (25mm to 100mm) | | 2 | SS | 50/ 0.025 | | 241 | | | | | Core from 1.7m to 2.9m depths |
| 1.3 | | | | | | | 240 | | | | | |
| | Sand, trace to some gravel, trace silt Very Dense Brown Moist | | 3 | SS | 65 | | 239 | | | | | 8 87 5 (SI+CL) Core from 3.6m to 4.6m depths |
| | | | | | | | 238 | | | | | Core from 4.6m to 5.2m depths |
| | Cobbles (50mm to 150mm) | | | | | | 237 | | | | | RUN #1 TCR=53% SCR=32% RQD=25% |
| | Cobbles (175mm to 200mm) Boulders (up to 250mm) Gravel | | 1 | RUN | | | 236 | | | | | RUN #2 TCR=38% SCR=5% RQD=5% |
| | | | | | | | 235 | | | | | |
| | Cobbles (25mm to 75mm) Gravel | | 2 | RUN | | | 234 | | | | | RUN #3 TCR=9% SCR=0% RQD=0% |
| | Possible sand from 8.2m to 9.5m depth Cobbles and gravel | | 3 | RUN | | | | | | | | |
| 233.3 | | | | | | | | | | | | |
| 9.7 | GRANITE slightly to moderately weathered, whitish grey and pink | | | | | | | | | | | |

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-P02

2 OF 2

METRIC

W.P. 4011-13-01 LOCATION Perry's Creek N 4 936 550.9 E 407 256.7 ORIGINATED BY ES
 HWY 28 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2015.05.12 - 2015.02.12 CHECKED BY SKP






| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|--|--|--|--|--|------------------------------------|-------------------------------------|-----------------------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | | | | | | |
| | Continued From Previous Page | | | | | | | | | | | | | | | | |
| 231.8 | becoming black, sub-horizontal to sub-vertical joints, strong to very strong Sub-vertical joint (25mm to 50mm) at 9.8m, 10.0m, 10.7m 150mm at 10.4m 125mm at 11.0m Possible sand (50mm) at 11.0m | | 4 | RUN | | | 232 | | | | | | | | | 1 4 3 3 | RUN #4 TCR=100% SCR=100% RQD=95% |
| 11.2 | END OF BOREHOLE AT 11.2m. WATER LEVEL AT 8.4m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | | | | | | | | | | | |

RECORD OF BOREHOLE No 15-P03

1 OF 1

METRIC

W.P. 4011-13-01 LOCATION Perry's Creek N 4 936 581.8 E 407 210.2 ORIGINATED BY ES
 HWY 28 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2015.05.08 - 2015.05.08 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|---|---------|------|--------------|----------------------------|-----------------|---|--|--|---|--|----|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | WATER CONTENT (%) | | | | |
| 243.0 | GROUND SURFACE | | | | | | | | | | | | | | |
| 0.0 | ASPHALT:(150mm) |  | | | | | | | | | | | | | |
| 0.2 | CONCRETE:(225mm) |  | | | | | | | | | | | | | |
| 0.4 | Gravelly SAND Very Dense Brown Wet (FILL) |  | 1 | SS | 79/ 0.200 | | 242 | | | | | | | 34 60 6 (SI+CL) | |
| 1.1 | FILL, mixture of cobbles and boulders, possibly in a sand and gravel matrix Cobbles (75mm to 150mm) |  | 1 | RUN | | | 241 | | | | | | | RUN #1 TCR=40% SCR=23% RQD=23% | |
| 240.5 | | | | | | | | | | | | | FI | | |
| 2.5 | GRANITE slightly to moderately weathered, whitish grey and pink, sub-horizontal to sub-vertical joints, strong to very strong Sub-vertical joint (25mm) at 2.7m, 2.9m 200mm at 3.1m Sub-horizontal joint(25mm) at 2.7m, 3.2m and 3.3m Sub-vertical joint (25mm to 100mm) at 3.4m, 3.7m and 3.9m Sub-horizontal joint (25mm to 50mm) at 3.6m and 3.8m |  | 2 | RUN | | | 240 | | | | | | 4 | RUN #2 TCR=94% SCR=89% RQD=53% | |
| | | | 3 | RUN | | | | | | | | | 1 | | |
| 238.9 | | | | | | | | | | | | | 2 | | |
| 4.1 | END OF BOREHOLE AT 4.1m. BOREHOLE DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.2m, SAND TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | 239 | | | | | | 3 | RUN #3 TCR=100% SCR=100% RQD=73% | |
| | | | | | | | | | | | | | 1 | | |

+³, ×³: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-P04

1 OF 1

METRIC

W.P. 4011-13-01 LOCATION Perry's Creek N 4 936 594.9 E 407 207.4 ORIGINATED BY ES
HWY 28 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2015.05.11 - 2015.05.11 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|--------------|----------------------------|-----------------|---|--|--|--|--|------------------------------------|-------------------------------------|-----------------------------------|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | |
| 243.0 | GROUND SURFACE | | | | | | | | | | | | | | | | |
| 0.0 | ASPHALT:(150mm) | | | | | | | | | | | | | | | | |
| 0.2 | CONCRETE with rebar: (250mm) | | | | | | | | | | | | | | | | |
| 242.6 | | | | | | | | | | | | | | | | | |
| 0.4 | Gravelly SAND, trace silt Very Dense Dark Brown Moist (FILL) | | 1 | SS | 78/ 0.225 | | 242 | | | | | | | | | | 35 62 3 (SI+CL) |
| 241.4 | | | 2 | SS | 50/ 0.100 | | | | | | | | | | | | |
| 1.6 | FILL, mixture of cobbles and boulders, possibly in a sand and gravel matrix Cobbles (75mm to 200mm) | | 1 | RUN | | | 241 | | | | | | | | | | RUN #1 TCR=69% SCR=54% RQD=31% |
| 240.5 | Cobbles | | 2 | RUN | | | | | | | | | | | | | |
| 2.5 | GRANITE slightly to moderately weathered, whitish grey and pink, sub-horizontal to sub-vertical joints, strong to very strong Sub-vertical joint (100mm) at 2.5m and (75mm) at 2.7m Sub-vertical joint (50mm) at 3.1m, 3.7m, 4.1m 100mm at 3.5m, 3.8m and 4.2m Possible sand (50mm) at 3.7m | | 3 | RUN | | | 240 | | | | | | | | | | RUN #2 TCR=100% SCR=58% RQD=42% |
| 238.6 | | | | | | | 239 | | | | | | | | | | |
| 4.4 | END OF BOREHOLE AT 4.4m. BOREHOLE DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | | | | | | | | | | | |

RECORD OF BOREHOLE No 15-P05

1 OF 1

METRIC

W.P. 4011-13-01 LOCATION Perry's Creek N 4 936 599.0 E 407 203.5 ORIGINATED BY ES
 HWY 28 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2015.05.06 - 2015.05.06 CHECKED BY SKP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | | |
|---------------|---|------------|---------|------|--------------|----------------------------|-----------------|--|----|----|----|-----|---|--|--|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | | | | PLASTIC LIMIT w _P NATURAL MOISTURE CONTENT w LIQUID LIMIT w _L | | |
| 243.0 | GROUND SURFACE | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 0.0 | ASPHALT:(150mm) | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 0.2 | Sandy GRAVEL Very Dense Brown Moist (FILL) | | 1 | GS | | | | | | | | | | | | | |
| 241.9 | | | 1 | SS | 75/ 0.200 | | | | | | | | | | | | |
| 1.1 | GRANITE slightly weathered, whitish grey and pink, sub-horizontal to sub-vertical joints, strong to very strong | | 1 | RUN | | | | | | | | | | | | | |
| | Sub-horizontal joint (25mm) at 1.3m | | | | | | | | | | | | | | | | |
| | Vertical joint (25mm to 75mm) at 1.3m, 1.4m, 2.1m 125mm at 1.2m | | | | | | | | | | | | | | | | |
| | Sub-horizontal joint (25mm) at 2.3m and 3.3m | | 2 | RUN | | | | | | | | | | | | | |
| | Sub-vertical joint (150mm) at 3.3m | | | | | | | | | | | | | | | | |
| | Sub-vertical joint (50mm) at 3.7m | | 3 | RUN | | | | | | | | | | | | | |
| 238.7 | | | | | | | | | | | | | | | | | |
| 4.3 | END OF BOREHOLE AT 4.3m. BOREHOLE DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.4m, SAND AND GRAVEL TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | | | | | | | | | | | |

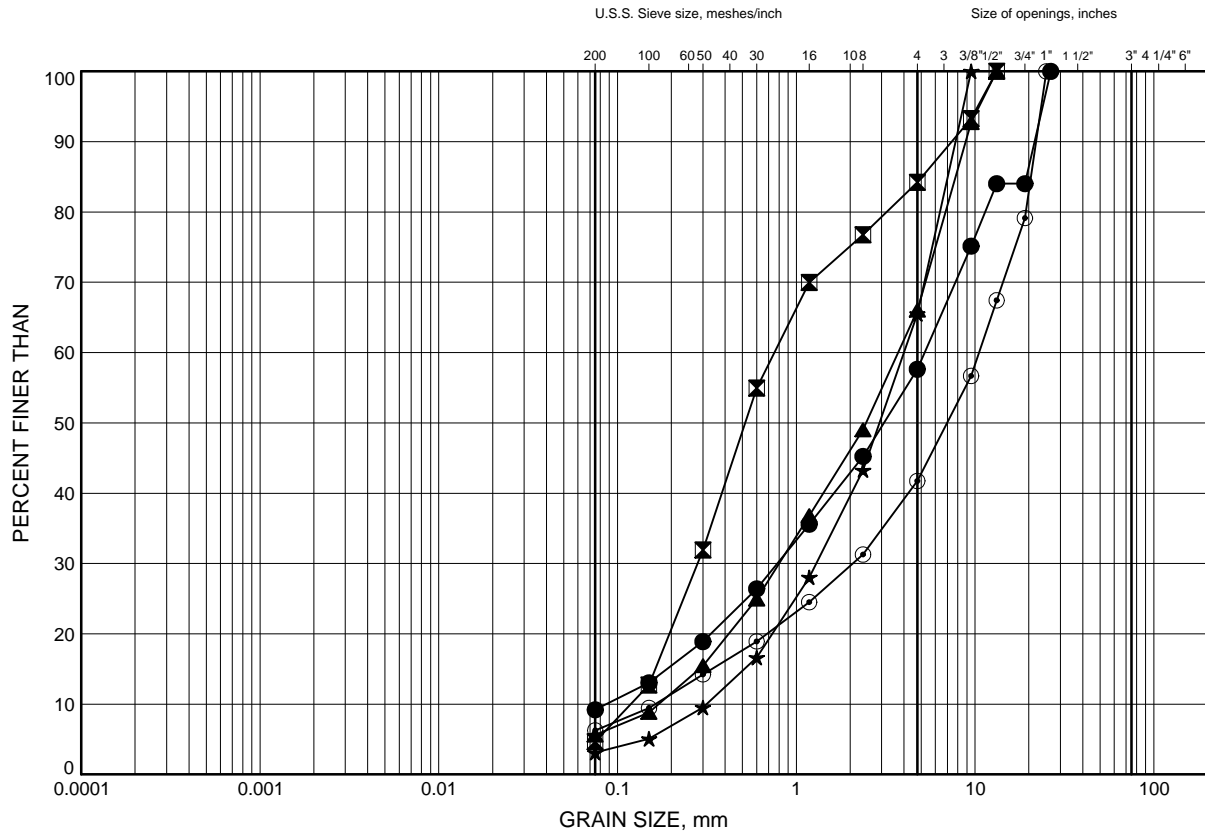
+³, ×³: Numbers refer to
Sensitivity

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15 5
10 (%) STRAIN AT FAILURE

Eastern Rehabilitation 18 Structures
GRAIN SIZE DISTRIBUTION

FIGURE D1

GRAVELLY SAND/SANDY GRAVEL FILL



| | | | | | | |
|---------------|------|--------|--------|--------|--------|-------------|
| SILT and CLAY | FINE | MEDIUM | COARSE | FINE | COARSE | COBBLE SIZE |
| FINE GRAINED | SAND | | | GRAVEL | | |

LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | 15-P01 | 0.93 | 242.07 |
| ⊠ | 15-P02 | 0.97 | 242.03 |
| ▲ | 15-P03 | 0.93 | 242.07 |
| ★ | 15-P04 | 0.95 | 242.05 |
| ⊙ | 15-P05 | 0.93 | 242.07 |

Date June 2015
W.P. 4011-13-01

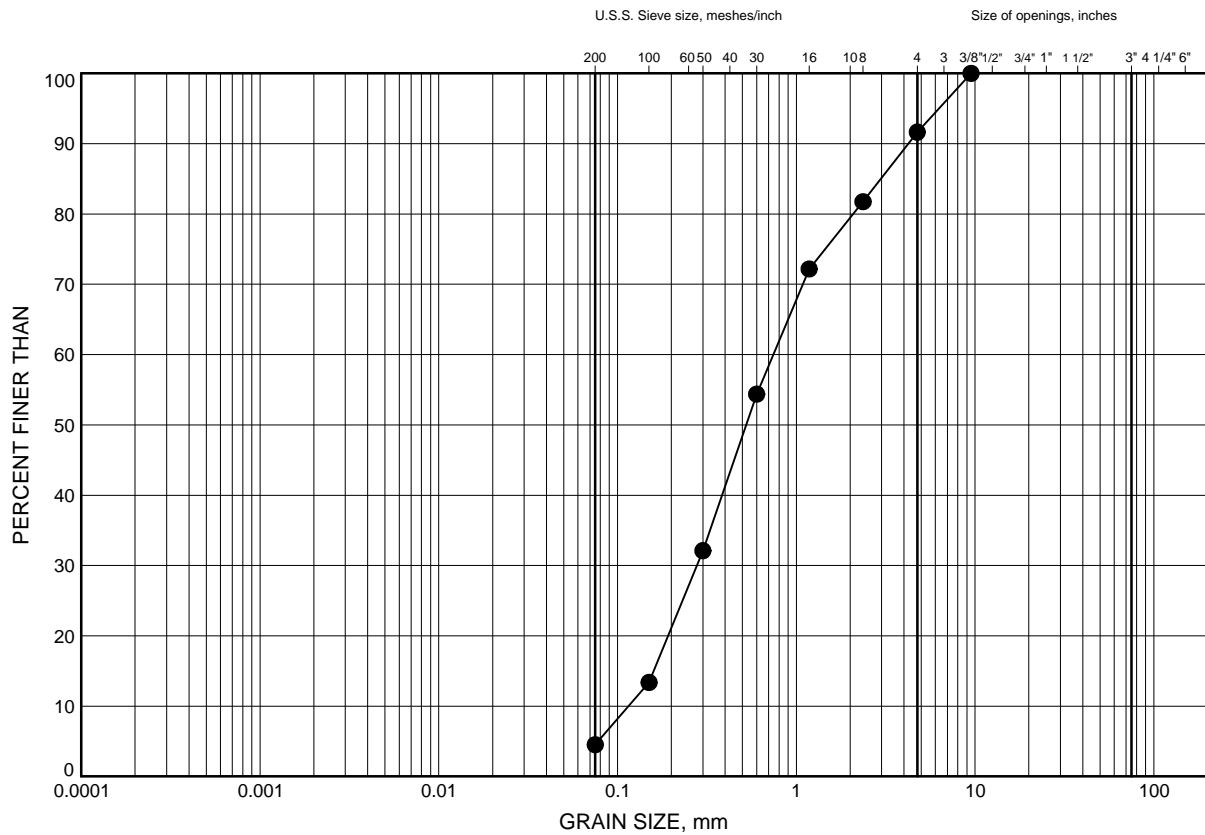


Prep'd AN
Chkd. SKP

Eastern Rehabilitation 18 Structures
GRAIN SIZE DISTRIBUTION

FIGURE D2

SAND (FILL)



| | | | | | | |
|---------------|------|--------|--------|--------|--------|-------------|
| SILT and CLAY | FINE | MEDIUM | COARSE | FINE | COARSE | COBBLE SIZE |
| FINE GRAINED | SAND | | | GRAVEL | | |

LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | 15-P02 | 3.32 | 239.68 |

Date June 2015
W.P. 4011-13-01



Prep'd AN
Chkd. SKP



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-4406-20 Client : AECOM
Date Drilled : 5/7/2015
Project Name : Perry's Creek Bridge Date Tested : 5/11/2015
Core Size : NQ BH No : 15-P01 Tester : ISP

| Test No. | Run No. | Depth (m) | Axial or Diametral | Gauge (MPa) | Diameter (mm) | Length (mm) | UCS (MPa) | Rock Type | Notes |
|----------|---------|-----------|--------------------|-------------|---------------|-------------|-----------|-----------|-------------|
| 4 | 5 | 6.6 | D | 23.6 | 47.3 | 109.5 | 234.0 | Granite | Very Strong |
| 5 | 6 | 8.2 | D | 9.3 | 47.3 | >200 | 92.5 | Granite | Strong |
| 6 | 6 | 8.6 | D | 15.4 | 47.3 | >200 | 152.5 | Granite | Very Strong |
| 7 | | | | | | | | | |
| 8 | | | | | | | | | |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | | | | | | | | |
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| 29 | | | | | | | | | |
| 30 | | | | | | | | | |

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-4406-20 Client : AECOM
Date Drilled : 5/12/2015
Project Name : Perry's Creek Bridge Date Tested : 5/20/2015
Core Size : NQ BH No : 15-P02 Tester : KK / AF

| Test No. | Run No. | Depth (m) | Axial or Diametral | Gauge (MPa) | Diameter (mm) | Length (mm) | UCS (MPa) | Rock Type | Notes |
|----------|---------|-----------|--------------------|-------------|---------------|-------------|-----------|---------------|-------------|
| 6 | 4 | 10.0 | D | 11.8 | 47.3 | 93.4 | 117.1 | Granite | Very Strong |
| 7 | 4 | 10.8 | D | 20.0 | 46.6 | 123.7 | 202.9 | plutonic rock | Very Strong |
| 8 | | | | | | | | | |
| 9 | | | | | | | | | |
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-4406-20 Client : AECOM
Date Drilled : 5/9/2015
Project Name : Perry's Creek Bridge Date Tested : 5/11/2015
Core Size : NQ BH No : 15-P03 Tester : ISP

| Test No. | Run No. | Depth (m) | Axial or Diametral | Gauge (MPa) | Diameter (mm) | Length (mm) | UCS (MPa) | Rock Type | Notes |
|----------|---------|-----------|--------------------|-------------|---------------|-------------|-----------|-----------|-------------|
| 6 | 2 | 2.9 | D | 19.1 | 47.5 | >200 | 188.4 | Granite | Very Strong |
| 7 | 3 | 3.5 | D | 19.3 | 47.4 | >200 | 190.7 | Granite | Very Strong |
| 8 | 3 | 4.0 | D | 20.9 | 47.4 | 151.7 | 206.6 | Granite | Very Strong |
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-4406-20 Client : AECOM
Date Drilled : 5/11/2015
Project Name : Perry's Creek Bridge Date Tested : 5/20/2015
Core Size : NQ BH No : 15-P04 Tester : KK / AF

| Test No. | Run No. | Depth (m) | Axial or Diametral | Gauge (MPa) | Diameter (mm) | Length (mm) | UCS (MPa) | Rock Type | Notes |
|----------|---------|-----------|--------------------|-------------|---------------|-------------|-----------|-----------|--------|
| 9 | 3 | 3.4 | D | 9.4 | 47.2 | 70.8 | 93.3 | Granite | Strong |
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-4406-20 Client : AECOM
Date Drilled : 5/6/2015
Project Name : Perry's Creek Bridge Date Tested : 5/11/2015
Core Size : NQ BH No : 15-P05 Tester : ISP

| Test No. | Run No. | Depth (m) | Axial or Diametral | Gauge (MPa) | Diameter (mm) | Length (mm) | UCS (MPa) | Rock Type | Notes |
|----------|---------|-----------|--------------------|-------------|---------------|-------------|-----------|-----------|-------------|
| 4 | 1 | 1.2 | D | 21.3 | 47.3 | 101.9 | 211.2 | Granite | Very Strong |
| 5 | 1 | 1.6 | D | 19.8 | 47.4 | >200 | 195.9 | Granite | Very Strong |
| 6 | 1 | 2.3 | D | 9.8 | 47.4 | 122.7 | 96.8 | Granite | Strong |
| 7 | 2 | 2.4 | D | 19.6 | 47.4 | 132.6 | 194.4 | Granite | Very Strong |
| 8 | 2 | 2.7 | D | 16.0 | 47.5 | >200 | 158.1 | Granite | Very Strong |
| 9 | 2 | 3.5 | D | 15.0 | 47.4 | >200 | 147.8 | Granite | Very Strong |
| 10 | 3 | 4.0 | D | 24.8 | 47.5 | >200 | 244.7 | Granite | Very Strong |
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



Photograph 1 – Borehole 15-P01, Runs 1 to 6



Photograph 2 – Borehole 15-P02, Runs 1 to 4



Photograph 3 – Borehole 15-P03, Runs 1 to 3

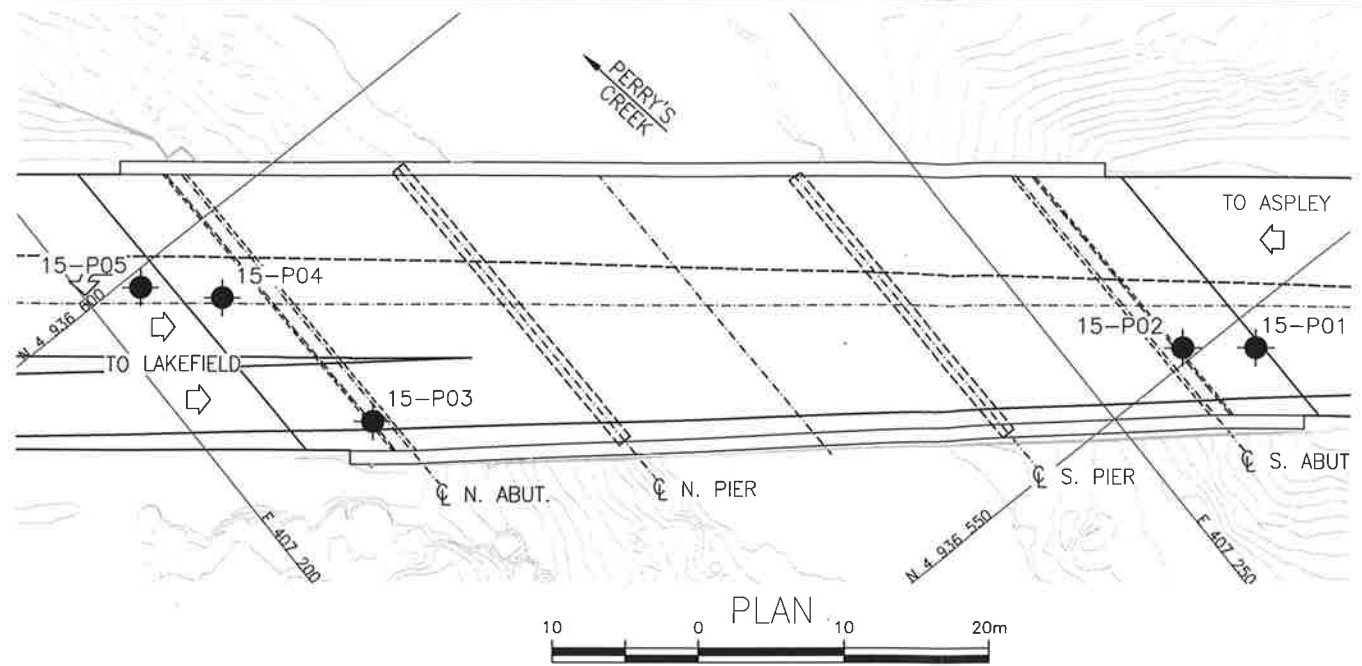


Photograph 4 – Borehole 15-P04, Runs 1 to 3



Photograph 5 – Borehole 15-P05, Runs 1 to 3

MINISTRY OF TRANSPORTATION, ONTARIO



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
WP No 4011-1-01
PERRY'S CREEK BRIDGE
HIGHWAY 28
NEAR BURLEIGH FALLS
BOREHOLE LOCATIONS AND SOIL STRATA



AECOM



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

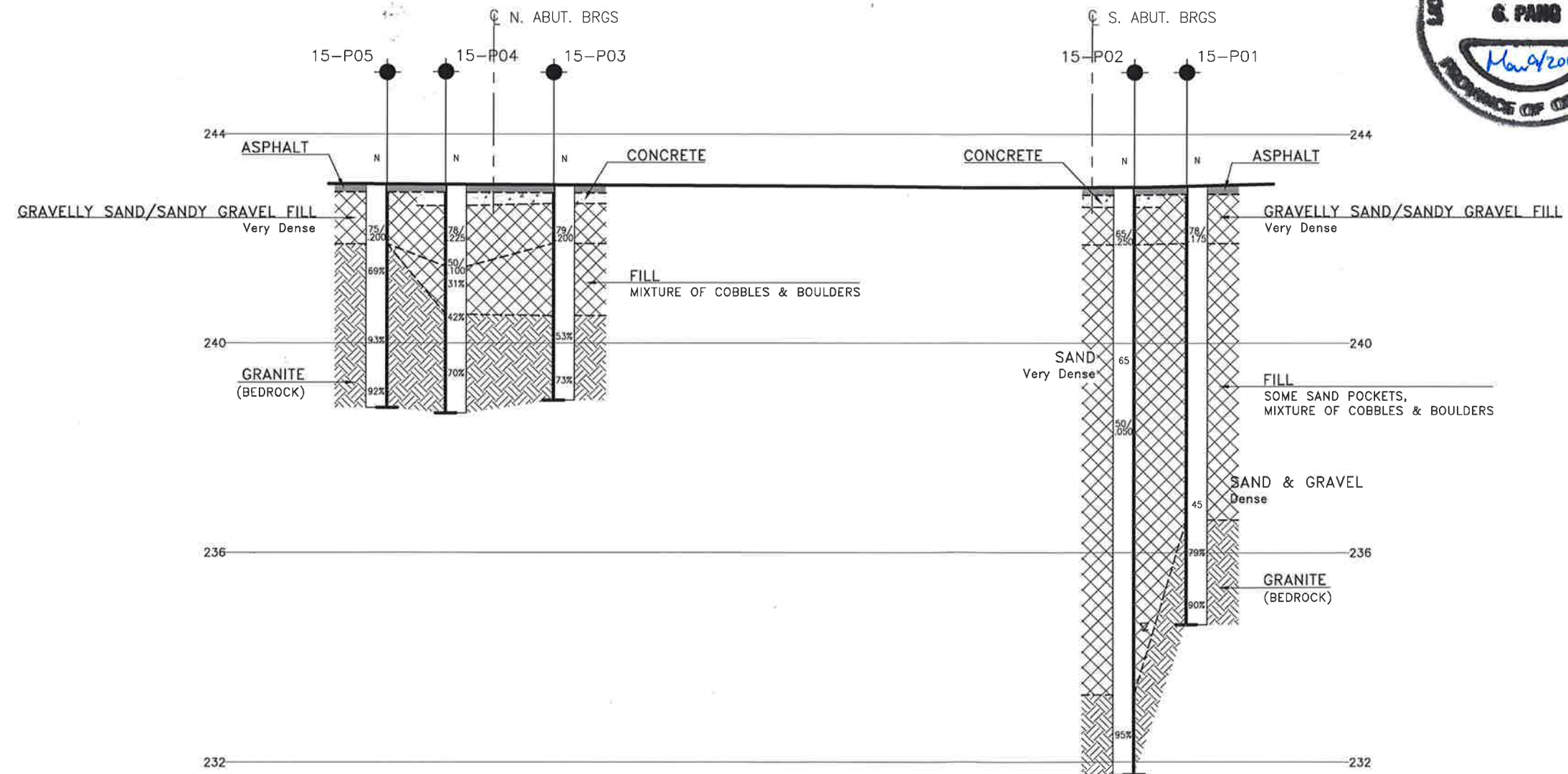
- Borehole
- Borehole and Cone
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60' Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

| NO | ELEVATION* | NORTHING | EASTING |
|--------|------------|-------------|-----------|
| 15-P01 | 243.0 | 4 936 547.8 | 407 260.6 |
| 15-P02 | 243.0 | 4 936 550.9 | 407 256.7 |
| 15-P03 | 243.0 | 4 936 581.8 | 407 210.2 |
| 15-P04 | 243.0 | 4 936 594.9 | 407 207.4 |
| 15-P05 | 243.0 | 4 936 599.0 | 407 203.5 |
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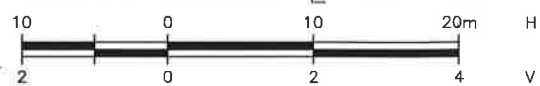
NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31D-654



PROFILE ALONG C HWY 28



* THE BOREHOLES HAVE NOT BEEN SURVEYED AND THE GROUND SURFACE ELEVATION IS APPROXIMATE.

| REVISIONS | DATE | BY | DESCRIPTION |
|-----------|------|----------|-------------|
| DESIGN | SKP | CHK | SKP |
| DRAWN | AN | CHK | SITE |
| LOAD | DATE | MAR 2017 | |
| STRUCT | DWG | 1 | |

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