

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PELICAN RIVER BRIDGE REPLACEMENT
HAUKENESS ROAD, WEST OF HIGHWAY 605
TOWNSHIP OF ETON-RUGBY, ONTARIO
DISTRICT OF KENORA
W.P. 473-00-00, SITE 41S-38**

Geocres Number: 52F-36

Report to

GENIVAR

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed bridge replacement. The existing bridge carries Haukeness Road over the Pelican River, just west of Highway 605 in the Township of Eton-Rugby, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and written descriptions of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to GENIVAR, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0027.

2 SITE DESCRIPTION

The Pelican River bridge is located on Haukeness Road just west of Highway 605 in the Township of Eton-Rugby, Ontario. Haukeness Road intersects Highway 605 approximately 8 km north of the intersection of Highway 17 and Highway 605.

Haukeness Road is an unpaved one-lane road. The existing bridge consists of a three span modular bridge supported on timber pile bents. The length of the bridge is 64.1 m.

At this location, the Pelican River flows from north to south. In general, the Pelican River meanders from its lake of origin, which is located approximately 6.5 km northeast of the bridge site, to the Wabigoon River located to the south of the site.

The lands immediately surrounding the bridge site consist of forested areas. Some lands in the area have been developed for agricultural purposes.

Photographs in Appendix C show the general nature of the surrounding land.

The site is underlain by Precambrian rocks and is covered with Pleistocene and recent deposits. These deposits consist of clays, silts and sands.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out on May 30 and 31, 2011 and consisted of drilling and sampling two boreholes (identified as PRB-01 and PRB-02), through the existing highway embankments at the bridge location. Borehole PRB-01 was drilled near the east abutment and Borehole PRB-02 was drilled near the west abutment. Boreholes were advanced to 12.1 m and 17.7 m depth (Elevations 88.5 and 81.1) where the drill rig encountered refusal on bedrock. Bedrock was proved in both boreholes by NQ size diamond coring. Borehole PRB-01 was advanced 3.1 m into bedrock and terminated at 15.2 m depth (Elevation 85.4). Borehole PRB-02 was advanced 3.6 m into bedrock and terminated at 21.3 m depth (Elevation 77.5). No boreholes were drilled at the proposed pier locations for the modular bridge.

The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing included in Appendix H.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

Drilling was carried out using a rubber-tire drill rig and hollow-stem augers were used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In situ vane shear testing was carried out to assess the undrained shear strength of soft to firm cohesive deposits.

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 and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions were observed in the open boreholes during and upon completion of the drilling operations. One standpipe piezometer consisting of 19 mm diameter PVC pipe with a slotted screen was installed in Borehole PRB-01 and enclosed in filter sand to permit longer term groundwater level monitoring. Borehole PRB-02, where artesian conditions were encountered was backfilled with bentonite in general accordance with O.Reg. 903. This backfilled borehole was periodically inspected after abandonment to confirm that no artesian flow was emerging at the surface of the borehole. The locations and completion details of the boreholes are shown in Table 3.1.

Table 3.1 – Borehole Abandonment Details

Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
PRB-01	11.9 / 88.7	Piezometer with 1.5 m slotted screen installed with sand filter to 10.1 m, bentonite from 10.1 m to 9.9 m, cuttings from 9.9 m to 0.9 m and then bentonite holeplug to surface.
PRB-02	None installed	Backfilled with bentonite holeplug to surface.

The piezometer in Borehole PRB-01 was decommissioned on June 4, 2011 in general accordance with O. Reg. 903.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis and Atterberg Limits testing, where appropriate. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are shown in Table 1 included in Appendix B and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix H. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general terms, the site was found to be underlain by sand, gravel and clay fill over a native silty clay deposit overlying bedrock. At the west abutment, a layer of sand was encountered between the silty clay deposit and the bedrock. More detailed descriptions of the individual strata are presented below.

5.1 Topsoil

A 40-mm thick layer of topsoil was encountered surficially at the location of Borehole PRB-01 drilled at the east abutment.

5.2 Sand and Gravel Fill

Granular fill was encountered below the topsoil in Borehole PRB-01 and at surface in Borehole PRB-02. In Borehole PRB-01 the granular fill consists of dark brown sand with trace gravel and roots and rootlets. In Borehole PRB-02 the granular fill consists of sand and gravel. The thickness of the cohesionless fill was 800 mm and 50 mm in Boreholes PRB-01 and PRB-02, respectively.

A SPT N-value of 5 blows for 0.3 m penetration was recorded in the sand fill in Borehole PRB-01, indicating a loose relative density.

The moisture content of a sample of the sand fill was measured to be 12%.

5.3 Silty Clay Fill

Brown silty clay fill was encountered below the granular fill in both boreholes. The silty clay fill contains trace to some sand and trace gravel. Roots, rootlets, wood pieces and peat were observed near the top of the silty clay fill layer. The thickness of the silty clay fill layer was 3.2 m and 3.1 m in Boreholes PRB-01 and PRB02, respectively.

The depths to the base of the silty clay fill layer were 4.0 m and 3.1 m (Elevations 96.6 and 95.7) in Boreholes PRB-01 and PRB02, respectively.

SPT N-values recorded in the silty clay fill ranged from 4 to 8 blows for 0.3 m of penetration, indicating a soft to firm condition.

The moisture content of samples from the silty clay fill generally varies between 17% and 52%. Higher moisture content values tend to correspond to greater amounts of organics in the fill.

Selected samples of the silty clay fill underwent gradation analysis testing and Atterberg Limits testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A. The grain size distribution curves for these samples are included in Figure B1 of Appendix B.

Soil Particles	Percentage (%)
Gravel	0
Sand	4 to 6
Silt	27 to 58
Clay	38 to 67

Index Property	Percentage (%)
Liquid Limit	40 to 50
Plastic Limit	20 to 22

The results of the Atterberg Limits testing indicate that the silty clay fill is of medium plasticity with a group symbol of CI. These results are illustrated in Figure B3 of Appendix B.

5.4 Silty Clay

Native silty clay was encountered below the silty clay fill in both boreholes. The native silty clay is generally grey in colour.

The thickness of the silty clay varied from 8.1 m at the east abutment to 13.7 m at the west abutment. The depths to the base of the silty clay ranged from 12.1 m to 16.8 m (Elevations 88.5 and 82.0).

SPT N-values recorded in the silty clay ranged from 0 to 3 blows for 0.3 m of penetration, indicating a very soft to soft deposit. Typically, N-values in the native silty clay were 0 blows for 0.3 m penetration. In-situ Shear Vane Tests were also performed where low N-values were recorded. The shear strength of the silty clay ranges from 21 to 38 kPa indicating the clay to be soft to firm.

The moisture content of samples collected from the silty clay layer generally varies between 43% and 72%.

Selected samples of the silty clay underwent gradation analysis testing and Atterberg Limits testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and the grain size distribution curves for these samples are included in Appendix B, Figure B2.

Soil Particles	Percentage (%)
Gravel	0
Sand	0
Silt	23 to 48
Clay	52 to 77

Index Property	Percentage (%)
Liquid Limit	41 to 50
Plastic Limit	20 to 22

The results of the Atterberg Limits testing indicate that the silty clay is of medium plasticity with a group symbol of CI. These results are also illustrated in Figure B4 or Appendix B.

5.5 Sand

A layer of brown sand containing trace of gravel was encountered below the silty clay at 16.8 m depth (elevation 82.0) at the west abutment in Borehole PRB-02. The sand layer is 0.9 m thick.

The depth to the base of the sand layer was 17.7 m (Elevation 81.1).

A SPT N-value of 6 blows for 0.3 m penetration was recorded in the sand layer, indicating a loose relative density.

The moisture content of a sample of the sand was 17%.

5.6 Bedrock

The overburden soils described above are underlain by bluish-grey and white, fresh granite bedrock. Occasional quartz interbeds, occasional mechanical breaks and sub-vertical fractures were noted throughout the bedrock cores.

Bedrock was proved by coring at each borehole. Table 5.1 summarizes depths and elevations to the top of bedrock in the boreholes.

Table 5.1 – Depths and Elevations of Top of Bedrock

Location	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
East abutment	PRB-01	12.1	88.5
West abutment	PRB-02	17.7	81.1

Core recovery in the bedrock was 100% in all cores. The RQD values ranged from 94% to 100%, indicating excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally less than 5.

The estimated unconfined compressive strength of the rock cores ranged from 30 MPa to 163 MPa, indicating a medium strong to very strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Appendix B.

5.7 Water Levels

Water levels were observed in the open boreholes upon completion of drilling operations. A standpipe piezometer was installed in Borehole PRB-01 to monitor water levels after completion of drilling. Artesian conditions were observed in the standpipe piezometer in the days following installation. Artesian conditions were encountered upon reaching the sand layer at 16.8 m depth during drilling operations of Borehole PRB-02. Borehole PRB-02 was sealed off /backfilled with bentonite in general accordance with O.Reg. 903 to control artesian conditions.

The water levels measured in the open boreholes and piezometer are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level (m)		Comment
		Depth	Elevation	
PRB-01	May 31, 2011	1.8	98.8	Open borehole
	June 2, 2011 (am)	0.2*	100.8	Piezometer
	June 3, 2011(am)	0.9*	101.5	Piezometer
	June 3, 2011(pm)	1.0*	101.6	Piezometer
	June 4, 2011 (am)	1.1*	101.7	Piezometer
PRB-02	May 31, 2011	Artesian Flow		Open borehole

* Indicates water level above ground surface, artesian conditions.

The piezometric readings reveal that the groundwater level is 0.2 to 1.1 m above ground surface (elevations 100.8 to 101.7), indicating artesian conditions at this site.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

The Pelican River ice/water level was measured at Elevation 97.2 m on March 9, 2011. GA indicate that the NHWL in the Pelican River at Elevation 97.15.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors retained by Genivar provided plan drawings to obtain the co-ordinates and the ground surface elevations for the boreholes.

Thurber obtained utility clearances for the borehole locations prior to drilling.

TBT Engineering of Thunder Bay, Ontario supplied a reduced-weight rubber-tire drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Ms. Eckie Siu of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program was conducted by Ms. Lindsey Blaine, E.I.T. Interpretation of the data and preparation of this report were carried out by Ms. Lindsey Blaine, E.I.T. and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data provided in the factual report and presents geotechnical recommendations for design of a new bridge to replace the existing bridge that carries Haukeness Road over Pelican River.

The existing bridge is a three span modular bridge with a total length of approximately 64.1 m, consisting of two 18.3 m end spans and a 27.4 m centre span. The existing bridge is supported on two abutment and two piers, each supported on timber pile bents. It is understood the existing pier cap, bracing and wood piles above the channel bottom at the piers and timber cribbing at the abutments will be removed.

The original design for the proposed bridge replacement (as shown on the Preliminary General Arrangement dated April 6, 2011) consisted of a single span modular bridge supported on two abutments. The proposed length of the bridge was 70.1 m with a structure width of 5.7 m. The proposed roadway width was 3.15 m.

The original bridge replacement design was subsequently revised and the revised design consisted of a three span modular bridge supported on two abutments and two piers, as indicated on GA received on February 1, 2012. This design indicates that the abutments and piers will be supported on driven steel H-piles. The span lengths will be 15.2 m, 39.6 m and 15.2 m, resulting in a total bridge length of 70.1 m.

Subsequently, an alternate foundation design shown on a drawing dated February 9, 2012 and provided by Genivar, was proposed for the new modular bridge. This latest alternative consists of supporting the piers on H-piles and the abutment bearings on sleeper slabs built across the roadway. The sleeper slabs will be 6.0 m by 1.5 m in size. The length of the modular bridge will

be 70.1 m. It is understood that this is the desired design alternative for this site. The GA's for the original and alternate designs are included in Appendix I.

It is anticipated that the replacement structure will be constructed on the same alignment as the existing alignment. No temporary detour bridge is required during construction since there is an existing detour route. The existing approach embankments are 3.0 to 4.0 m high and a grade raise of 400 mm to 500 mm is proposed at the west abutment. There is minimal grade raise at the east abutment.

It must be noted that the field investigation at this site was carried out for the original design of single span modular bridge (GA drawing dated April 6, 2011). The design was subsequently changed to a three span modular bridge after the field investigation was completed and no boreholes were therefore drilled at the proposed pier locations.

MTO has indicated that the foundation recommendations for the proposed piers be based on the available borehole information at the abutments. It must be recognized that the stratigraphy, specially the depths to top of bedrock, at the proposed pier locations may vary from the stratigraphy and the depth to bedrock at the abutment locations.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The plans and profiles used for preparation of this report were provided by GENIVAR.

8 STRUCTURE FOUNDATIONS

The stratigraphy encountered in the two boreholes drilled at the east and west abutments revealed two layers of fill consisting of 50 mm to 800 mm of loose sand and gravel over 3.2 m of soft to firm silty clay. A deposit of soft to firm native silty clay was contacted below the silty clay fill. The thickness of the silty clay ranged from 8.1 m to 13.7 m. A layer of sand was contacted below the silty clay in Borehole PRB-02, drilled at the west abutment. Granitic bedrock was encountered directly below the silty clay and the sand layers at 12.1 m and 17.7 m depth.

The piezometric readings reveal that the groundwater level is 0.2 m to 1.1 m above ground surface (elevations 100.8 to 101.7), indicating artesian conditions at this site.

The Pelican River ice/water level was measured at Elevation 97.2 m on March 9, 2011. GA shows that the NHWL in the Pelican River at Elevation 97.15.

In the preparation of the geotechnical design recommendations for the new structure, consideration was given to the following foundation types:

- Spread footings on native soils
- Abutment bearings on precast concrete sleeper slabs
- Augered Caissons (drilled shafts)
- Driven steel H-piles

A comparison of the foundation alternatives based on advantages and disadvantages of each one is included in Appendix D. A foundation scheme preferred from a foundations perspective is recommended.

8.1 Spread Footings on Native Soils

Spread footings founded on native soils are not recommended at this site due to the following reasons:

- Very low geotechnical capacities are available in the overburden silty clay at this site.
- Relatively large settlements under footing loads will occur if footings are placed on the native soils.
- Footing bases will be below the groundwater/creek level and unwatering/groundwater control will be difficult at this site.

8.2 Sleeper Slab Foundation at the Abutment

In light of the fact that modular bridges can accommodate larger settlements and the lightly loaded abutments may be shimmed up periodically to accommodate settlements, the Region and the bridge designers have put forward a cost-effective sleeper slab foundation design at the abutments. The sleeper slab design consists of sitting the two abutment bearings on a transverse precast concrete slab placed across the roadway. The GA attached in Appendix I, illustrates the proposed sleeper slab concept. The bearing slab is proposed to be 6.0 m by 1.5 m in size.

The total ULS and SLS loads at the abutments have been reported by Genivar to be 875 kN and 548 kN, respectively. This implies an applied pressure of 97 kPa (at ULS) and 61 kPa (at SLS).

Analyses have been conducted to assess the settlement at the bridge approaches and stability of the forward slope for the proposed sleeper slab configuration and new imposed loads from the abutments of the modular bridge. The forward slope configuration was based on the latest GA drawing.

The slope stability analysis indicates that the factor of safety of the forward slope including the loads imposed by the abutment is 1.4. This assumes that the natural valley slopes below the river level is 3H:1V or flatter. This factor of safety is considered acceptable. The slope stability analysis is attached in Appendix E.

The placement of new granular fill to raise the existing highway grade and the new loads imposed at the abutments will induce immediate (elastic) settlement in the existing sand fill and silty clay fill as well as time dependent (consolidation) settlement in the underlying native silty clay.

The total immediate and consolidation settlements were assessed using elastic methods and one-dimensional consolidation theory. Based on these analyses, the anticipated immediate and consolidation settlements at the bridge approaches are estimated as follows:

<u>Location</u>	<u>Settlement (mm)</u>
<u>West abutment</u> (settlement induced by new abutment loads and 400 mm to 500 mm of grade raise)	50
<u>East abutment</u> (settlement induced by new abutment loads)	30

The abutments will have to be shimmed periodically to compensate for the above settlements.

The sleeper slab should not be placed directly on the existing silty clay embankment fill at the approaches. It is recommended that the existing embankment fill be subexcavated by 1.0 m and the sleeper slab placed on a minimum 1.0-m thick well compacted granular engineered fill pad.

In order to reduce the contact stress below the base of the slab (zone of influence) and associated soil settlement, due to the imposed loads, consideration should be given to reducing the width of the sleeper slab and increasing the thickness of the engineered fill pad. Design optimization of the sleeper slab dimension and the underlying thickness of the granular pad must be carried by the structural designer based on design and construction considerations.

8.3 Caissons

Augered caisson foundations were also considered for the support of the structure. However, the soft cohesive overburden is not considered suitable for caisson support and the caissons must be founded on the bedrock at depths in the order of 12.1 m to 17.7 m below original ground surface. The base of the caissons would be well below the groundwater level, resulting in difficulties in dewatering, base cleaning and base inspection.

Installation of deep caissons to bedrock is also expected to be a more expensive option than driven piles.

For these reasons, the use of a caisson foundation is not recommended at this site.

8.4 Steel H-Pile Foundations

8.4.1 Abutments

Steel H-piles driven to refusal in bedrock is a suitable and cost-effective option at this site.

The piles are expected to develop the required resistance on the top of bedrock at the elevations given in Table 8.1.

Table 8.1 – Estimated Pile Tip Elevation

Location	Borehole	Anticipated Pile Length below original ground (m)	Anticipated Pile Tip Elevation To Develop Required Resistance
East abutment	PRB-01	12.1	88.5
West abutment	PRB-02	17.7	81.1

The axial, factored geotechnical resistances at Ultimate Limit States (ULS_f) for an H-Pile section 310x110 driven to refusal on bedrock is 2,000 kN.

The SLS condition will not govern for piles founded on the bedrock.

The structural resistance of the pile must be checked by the structural designer.

8.4.2 Pier

Based on interpolation of stratigraphy between the abutment boreholes and assuming that the soil stratigraphy at the piers is similar to the soil stratigraphy encountered at the abutments, supporting the piers on steel H-piles driven to bedrock is a feasible option at this site.

Based on interpolation between the abutment boreholes, the estimated founding elevation from piles driven to bedrock is 81.1 at the west pier and 87.0 at the east pier.

However, since no boreholes were drilled at the proposed piers of the modular bridge, the pile tip depths cannot be established with certainty. The pile tip elevations for the required geotechnical resistance may vary from those predicted in this report and the Contract must allow for this possibility.

Recommendations for factored geotechnical resistance at Ultimate Limit States (ULSf) and geotechnical resistance at Serviceability Limit States (SLS) for H-piles driven to bedrock at the pier, will be similar to the geotechnical resistances provided for the abutments in Section 8.4.1 of this report.

The structural resistance of the pile must be checked by the structural designer.

8.4.3 Pile Tips

If piles driven to bedrock are selected, the tips of all piles should be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

8.4.4 Pile Installation

Pile installation should be in accordance with OPSS 903.

8.4.5 Artesian Conditions

A piezometer installed at the east abutment indicated groundwater level at 0.2 m to 1.1 m (elevations 100.8 to 101.7) above the ground surface. Artesian pressure has the potential to cause flow up the pile shaft, with accompanying loss of fines. However, since there is a considerable thick layer of clay above the bedrock, it is expected that this clay layer will seal around the pile, minimizing the potential for upward flow around a pile shaft.

This is not an issue for the abutments where the abutment bearings will be founded on a sleeper slab.

8.4.6 Downdrag

Downdrag on the piles is not considered to be an issue for the pier piles at this site.

8.5 Lateral Resistance

The overburden soils encountered at this site may be treated as essentially cohesive. Accordingly, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = 67 \cdot S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 \cdot S_u \text{ (kPa) at and below a depth of } 3 \cdot D \text{ (m) reduced to zero at the ground surface}$$

where

$$D = \text{pile width in metres}$$

$$S_u = \text{undrained shear strength (kPa), parameters recommended in Table 8.2.}$$

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

Table 8.2 – Parameters for Lateral Pile Resistance

Location	Elevation	S_u kPa	Unit Weight* (kN/m ³)	Soil Conditions
East Abutment	OGL to 96.6	75	10*	Firm silty clay fill
	96.6 to 88.5 (bedrock)	25 to 30	7*	Soft to very soft silty clay
West Abutment	OGL to 95.7	75	10*	Firm silty clay fill
	95.7 to 82.0	25 to 35	7*	Soft to very soft silty clay

*Buoyant unit weight below the water table.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

The lateral forces on the piers were reported by Genivar to be as follows:

- Unfactored breaking force (longitudinal): 268.3 kN
- Unfactored wind load (transverse): 23.5 kN
- Lateral Load per pile: 61.5 kN

In light of the fact that the pier piles will be installed through soft clay with zero blow count and very with low lateral resistance, at the request of Genivar, a lateral pile analysis

was carried out on two pile sections: HP 310 x 110 and HP 360 x 132. Soil parameters provided in Table 8.2 were used for the analysis.

For the purpose of lateral resistance analyses, the commercially available program LPILE Plus Version 5.0.47 developed by Ensoft Inc. was used.

Two pile head fixity conditions, i.e. fixed head and pinned head, were considered, and two cases of pile head location were analyzed, one at cut-off elevation (EL. 99.446 m) and the other at the base of bracing (EL. 97.446 m). The results of lateral pile analyses are plotted in the attached Figures 1 and 2 in Appendix F. The load deflection curves shown in the figures are for a single pile being loaded in the direction of maximum bending rigidity without accounting for group effect. The lateral load capacity of individual pile in a pile group should be reduced based on pile spacing. The number of piles required to resist the lateral loads in a pier may be determined based on the attached figures in conjunction with the pile spacing. For a lateral load per pile of 61.5 kN, the fixity point for the piles at the west pier is determined to be 5.0 m above the bedrock surface. There is no fixity point for piles at the east pier.

Appropriate cross bracing should be designed for each pile group at each pier so that the piles act as a unit to resist the breaking force.

8.6 Proposed Foundation

Abutments

- The proposed foundation is to found the abutment bearings on a precast concrete sleeper slab placed on a granular compacted engineered fill pad. This design is feasible at this site, provided the abutment slab is shimmed up periodically to compensate for effects of frost heave and settlement at the approaches.
- The abutments could also be founded on steel H-piles driven to refusal on bedrock.

Pier

- Steel H-piles driven to bedrock is the feasible foundation type at the piers. Since no boreholes were drilled at the piers, the pile tip depths have been estimated by interpolation of stratigraphy between the abutment boreholes. The pile tip depths may vary from those predicted in the report and the contract must allow for this possibility.

8.7 Frost Cover

The depth of frost penetration at this site is 2.5 m. The base of pile caps, if employed, must be provided with a minimum of 2.5 m of earth cover as protection against frost action. However, provision of 2.5 m of earth cover for the proposed abutment sleeper slab

foundation is not practical and the sleeper slabs will be subject to heaving due to frost action. The two options for dealing with this issue are:

1. Provision of insulation below the slab.
2. Regular inspection and periodic shimming of the slab, as required, to compensate for the frost heave effects.

9 EXCAVATION

The drawing received on February 9, 2012, which incorporates to the sleeper slabs at the abutments, shows the hatched areas where excavation and flattening of the slopes is proposed at both abutments. The area of excavation is approximately 12.0 m long and 7.3 m to 10.6 m wide. The excavation is above the water level of Pelican River.

Earth excavations at this site will penetrate through sand, gravel and clay fills.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand, gravel and silty clay fill forming the existing embankment may be classified as Type 3 soil above the water table and Type 4 below the water

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. Excavations should be inspected regularly for evidence of instability.

The excavation and backfilling must be carried out in accordance with OPSS 902.

10 UNWATERING

Piezometer indicated groundwater at 0.2 m to 1.1 m above ground surface (elevations 100.8 to 101.7), indicating artesian conditions at this site. However, the groundwater level within the upper fill deposit is expected to be near the water level in the river at the time of construction.

The Pelican River ice/water level was measured at Elevation 97.2 m on March 9, 2011. GA indicate water level in the Pelican River at Elevation 97.15.

Based on the preliminary GA for the bridge structure and the use of pile foundations and/or sleeper slabs, it is not expected that work at the abutments or piers will require excavation below the river/groundwater level.

It is recommended that excavation for removal of existing timber cribbing at the abutments be maintained above the water table.

Pumping from filtered sumps should be implemented to remove any accumulation of water from any excavation base at the abutments prior to placing concrete.

In general, the design of the dewatering system should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility.

11 EROSION PROTECTION

Erosion protection should be provided along any section of forward slope that may be affected by the stream flow.

In general, earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 804.

12 MODULAR BRIDGE BACKFILL

The GA drawing indicates that there is no backfill at the abutments of the modular bridge.

13 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type IV based on a silty clay strata thickness of at least 8.1 m to 13.7 m and bedrock depth less than 60 m. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 2.0 should be used in seismic design.

The foundation soils at the site are assessed as not being prone to liquefaction under current conditions.

14 ROADWAY PROTECTION

Roadway protection will not be required for the modular bridge option at this site.

15 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- The potential variability of pile lengths driven to bedrock, which was contacted at depths ranging from 12.1 m to 17.7 m (elevations 88.5 to 81.1).
- Top of bedrock was not established at the pier locations. Therefore, pile tip elevations will vary at the piers.
- If artesian groundwater flow is observed during pile driving, or any other construction activities, the contractor or QVE must immediately advise the CA. The CA should refer this issue to the design team.
- Excavation should be maintained above the water level in the river.
- Erosion protection should be provided to the header slopes

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Rocío Palomeque Reyna, P.Eng.
Geotechnical Engineer



P. K. Chatterji, P.Eng.
Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$


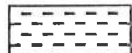



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	(psi)	Field Estimation of Hardness*
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.

TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

METRIC

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	w _P w w _L		
100.6 0.8	TOPSOIL: (40mm) SAND, trace gravel, roots and rootlets Loose Dark Brown Damp (FILL)		1	SS	5							
99.8 0.8	Silly CLAY, trace to some sand, trace gravel, roots and rootlets Firm Brown (FILL)		2	SS	7							
			3	SS	8							
	Mottled Brown and Grey		4	SS	6							0 6 27 67
			5	SS	5							
96.6 4.0	Silly CLAY, occasional peat Soft to Very Soft Grey		6	SS	3							
			1	TW								
			7	SS	0							
			8	SS	0							0 0 47 53

+ 3, X 3: Numbers refer to Sensitivity

METRIC

[illegible]

CONTMT4S 0839 GPJ 6/27/12

+ 3, X 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No PRB-02

1 OF 3

METRIC

W.P. 473-00-00 LOCATION N 8 143.2 E 471.4 Pelican River Bridge ORIGINATED BY ES
 HWY HAUKESS RD. BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Established on site DATE 2011.05.31 - 2011.05.31 CHECKED BY LRB

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						
98.8								20 40 60 80 100						
98.8	SAND and GRAVEL: (FILL)(50mm)							○ UNCONFINED + FIELD VANE						
	Silty CLAY, trace to some sand, trace gravel, occasional wood fibres		1	SS	4		98	● QUICK TRIAXIAL × LAB VANE						
	Soft to Stiff													
	Brown		2	SS	5									
	(FILL)													
			3	SS	8		97						0 4 58 38	
	Occasional wood fibres and peat		4	SS	4		96							
95.7														
3.1	Silty CLAY		5	SS	0		95							
	Very Soft													
	Grey													
			1	TW			94							
			6	SS	0		93						0 0 23 77	
							92							
			7	SS	0		91							
							90							
			8	SS	0									
							89							

Continued Next Page

+³ x³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

ONTMT4S 0839 GPJ 6/27/12

RECORD OF BOREHOLE No PRB-02

3 OF 3

METRIC

W.P. 473-00-00 LOCATION N 8 143.2 E 471.4 Pelican River Bridge ORIGINATED BY ES
HWY HAUKESS RD. BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Established on site DATE 2011.05.31 - 2011.05.31 CHECKED BY LRB

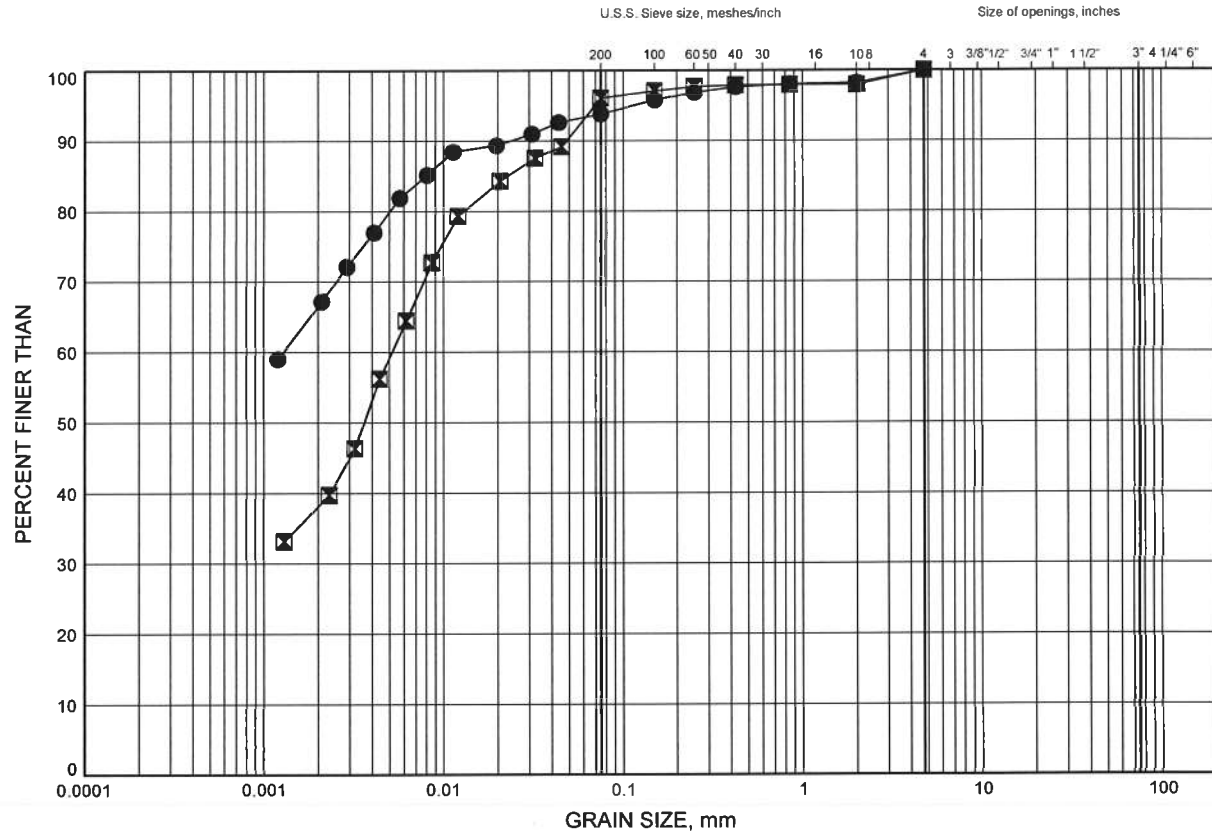
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)				
	Continued From Previous Page							20	40	60	80	100										
	BEDROCK , granite pegmatite, fresh, bluish-grey and white		3	RUN			78															
	Sub-vertical fractures (between 25mm to 100mm) at 20.8m, 21.1m																					
	150mm at 20.1m																					
	150mm at 20.6m																					
	Black mica interbed at 20.8m																					
77.5																						
21.3	END OF BOREHOLE AT 21.3m. ARTESIAN CONDITIONS OBSERVED WITHIN THE SAND LAYER. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.																					

Appendix B
Laboratory Test Results

Pelican River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY FILL



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

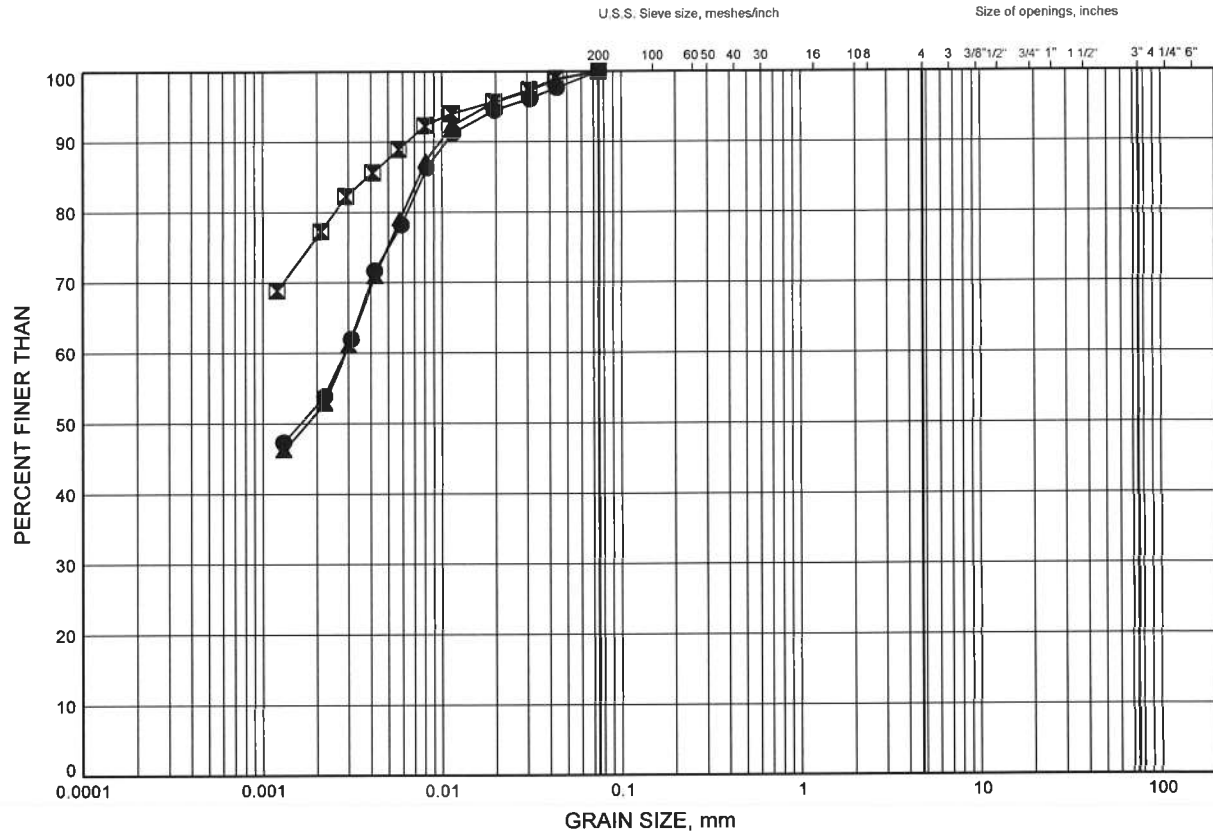
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-01	2.51	98.09
■	PRB-02	1.75	97.05

Pelican River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



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

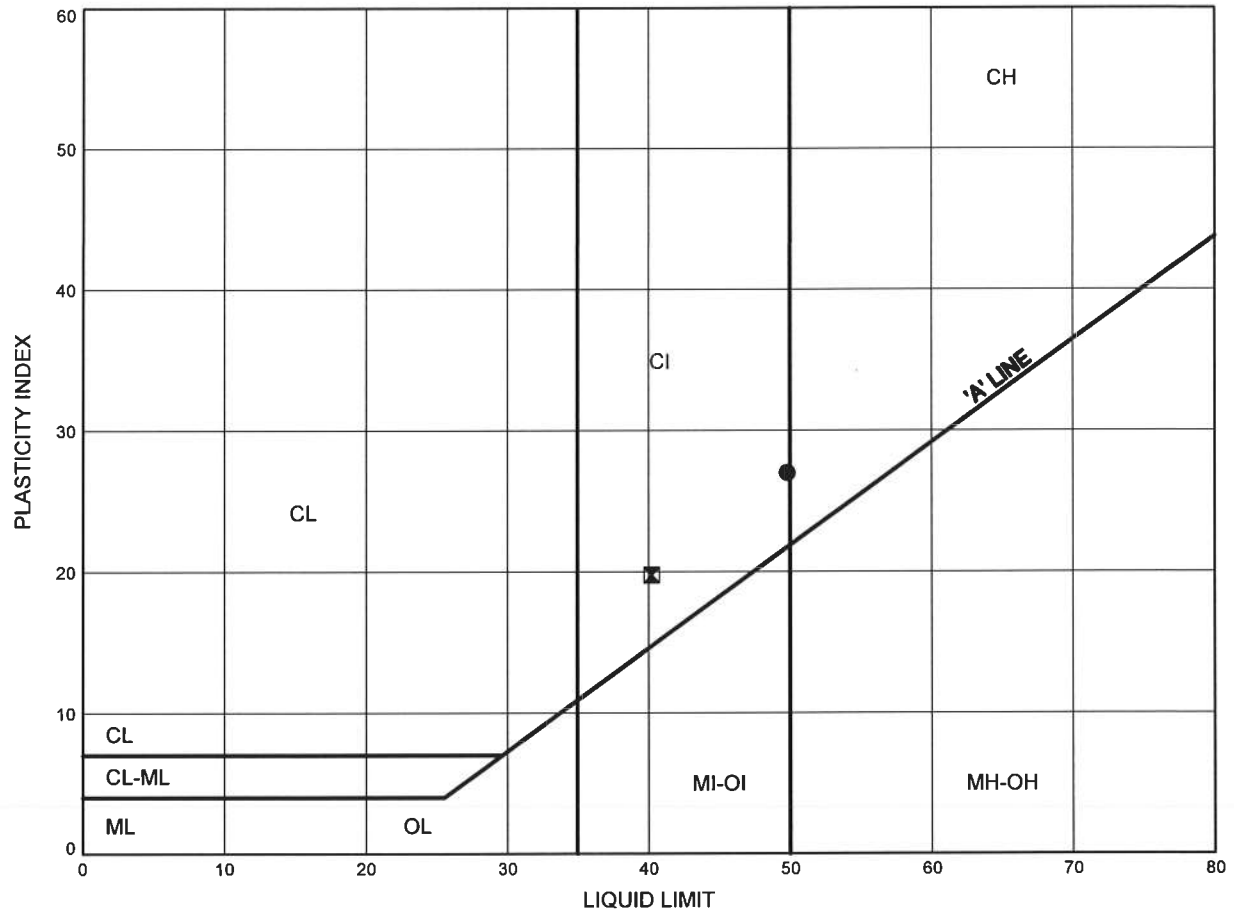
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-01	9.37	91.23
⊠	PRB-02	6.32	92.48
▲	PRB-02	13.94	84.86

Pelican River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B3

SILTY CLAY FILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	PRB-01	2.51	98.09
⊠	PRB-02	1.75	97.05

Date June 2012
 Project 473-00-00

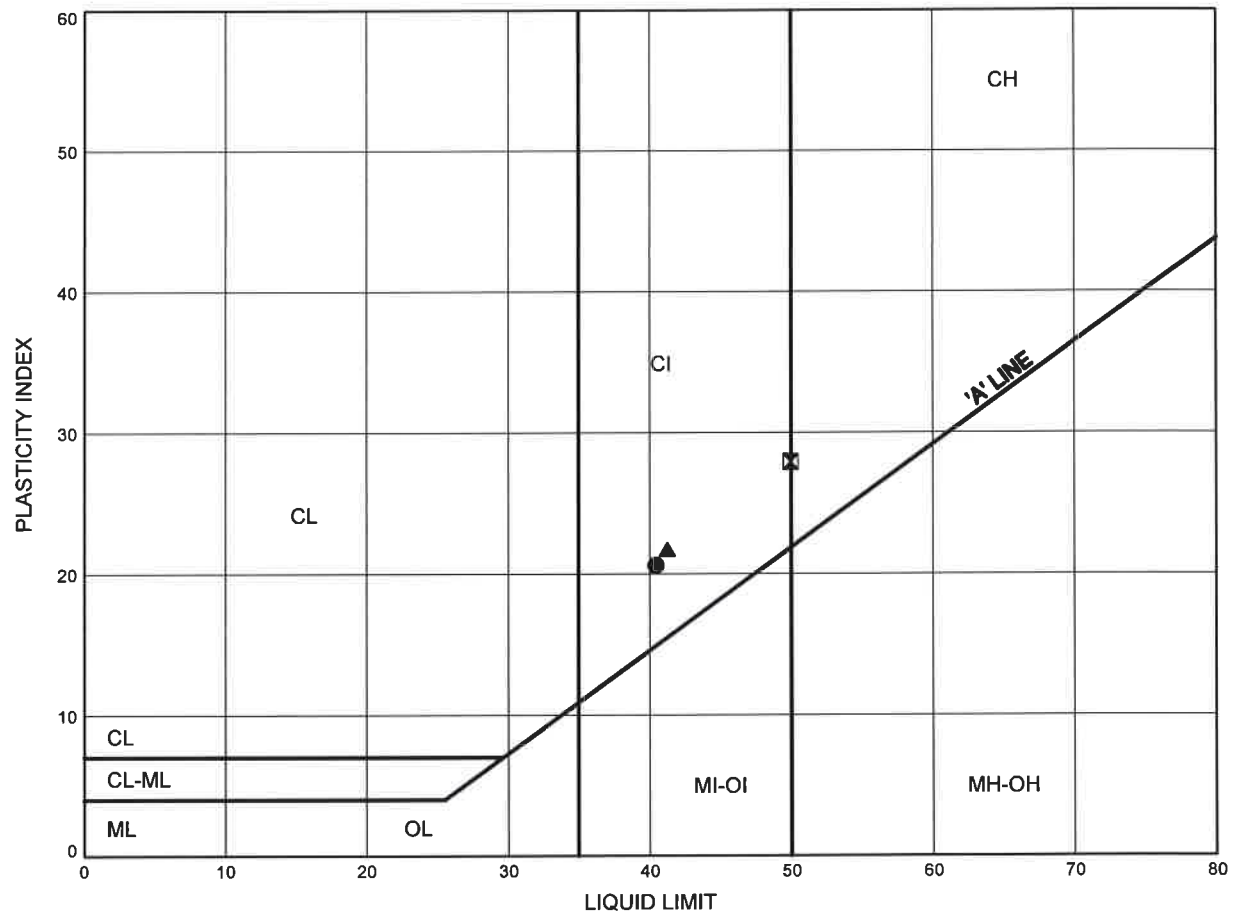


Prep'd AN
 Chkd. RPR

Pelican River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B4

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	PRB-01	9.37	91.23
⊠	PRB-02	6.32	92.48
▲	PRB-02	13.94	84.86

Date June 2012
 Project 473-00-00



Prep'd AN
 Chkd. RPR

TABLE 1 - Point Load Test Results
PELICAN RIVER BRIDGE REPLACEMENT
HAUKENESS ROAD, WEST OF HIGHWAY 607
TOWNSHIP OF RUGBY, ONTARIO
Site 41S-38

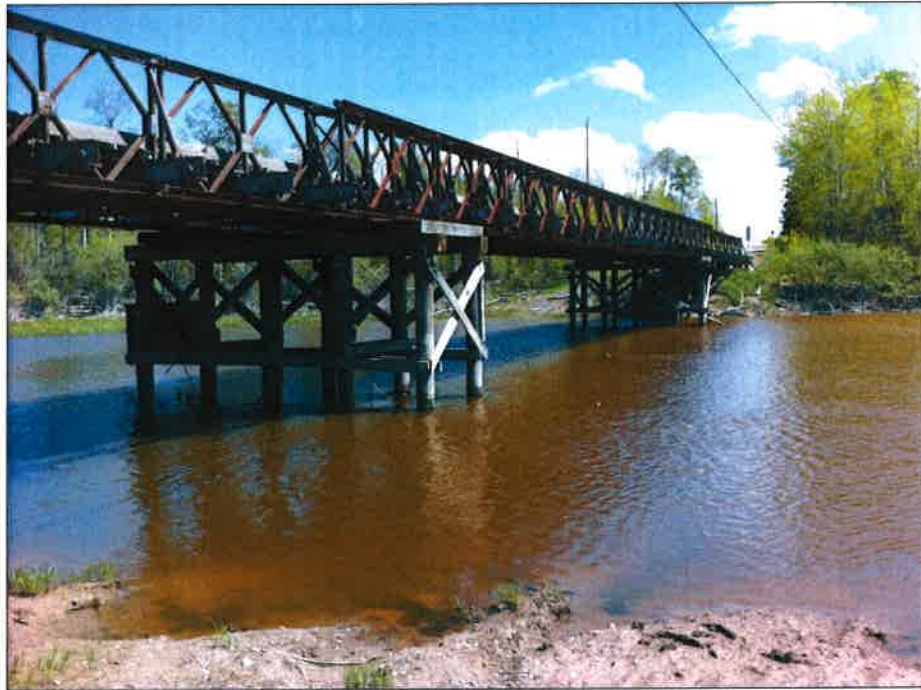
19-5308-39

PRB-01	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	UCS (Mpa)	CONCLUSIONS			
	FT.	IN.	METERS									
RUN #1	39	6	12.04	14.7	D	47.74	82.44	151.20				
	40	7	12.37	6.4	D	48.18	115.00	65.07				
	41	7	12.67	11.8	D	47.17	71.59	124.41				
RUN #2	42	0	12.80	6.9	D	47.78	88.36	71.48				
	43	3	13.18	3.0	D	48.18	74.48	30.91				
	44	7.5	13.60	16.0	D	48.65	62.89	159.86				
	45	10	13.97	6.7	D	48.84	67.35	66.70				
RUN #3	47	0	14.33	9.5	D	48.94	88.61	94.48				
	47	10	14.58	13.4	D	48.60	59.61	134.03				
	48	6	14.78	14.7	D	48.35	74.92	148.25				
RUN #4	49	5	15.06	11.0	D	47.42	68.23	114.64				
									AVERAGE	MAX	MIN	
									RUN #1:	114	151	65
									RUN #2:	82	160	31
									RUN #3:	126	148	94
									RUN #4:	115	115	115

PRB-02	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	UCS (Mpa)	CONCLUSIONS			
	FT.	IN.	METERS									
RUN #1	58	3	17.75	10.3	D	47.71	106.41	106.33				
	60	2	18.34	8.7	D	47.57	66.24	89.81				
RUN #2	61	4	18.69	4.8	D	47.45	79.16	50.39				
	62	8	19.10	10.8	D	47.32	69.20	113.13				
	63	10	19.46	13.2	D	47.11	87.85	138.70				
	65	6	19.96	15.5	D	47.27	65.33	162.54				
RUN #3	66	8	20.32	12.2	D	47.33	92.39	127.31				
	68	6	20.88	14.6	D	47.34	66.66	152.97				
	69	3	21.11	13.4	D	47.73	80.03	138.25				
									AVERAGE	MAX	MIN	
									RUN #1:	98	106	90
									RUN #2:	116	163	50
									RUN #3:	140	153	127

Appendix C
Site Photographs

Pelican River Bridge Replacement, Site 41S-38
Haukeness Road, West of Highway 605



Photograph 1 – Pelican River Bridge, looking northeast



Photograph 2 – Pelican River Bridge, looking east

Appendix D
Foundation Comparison

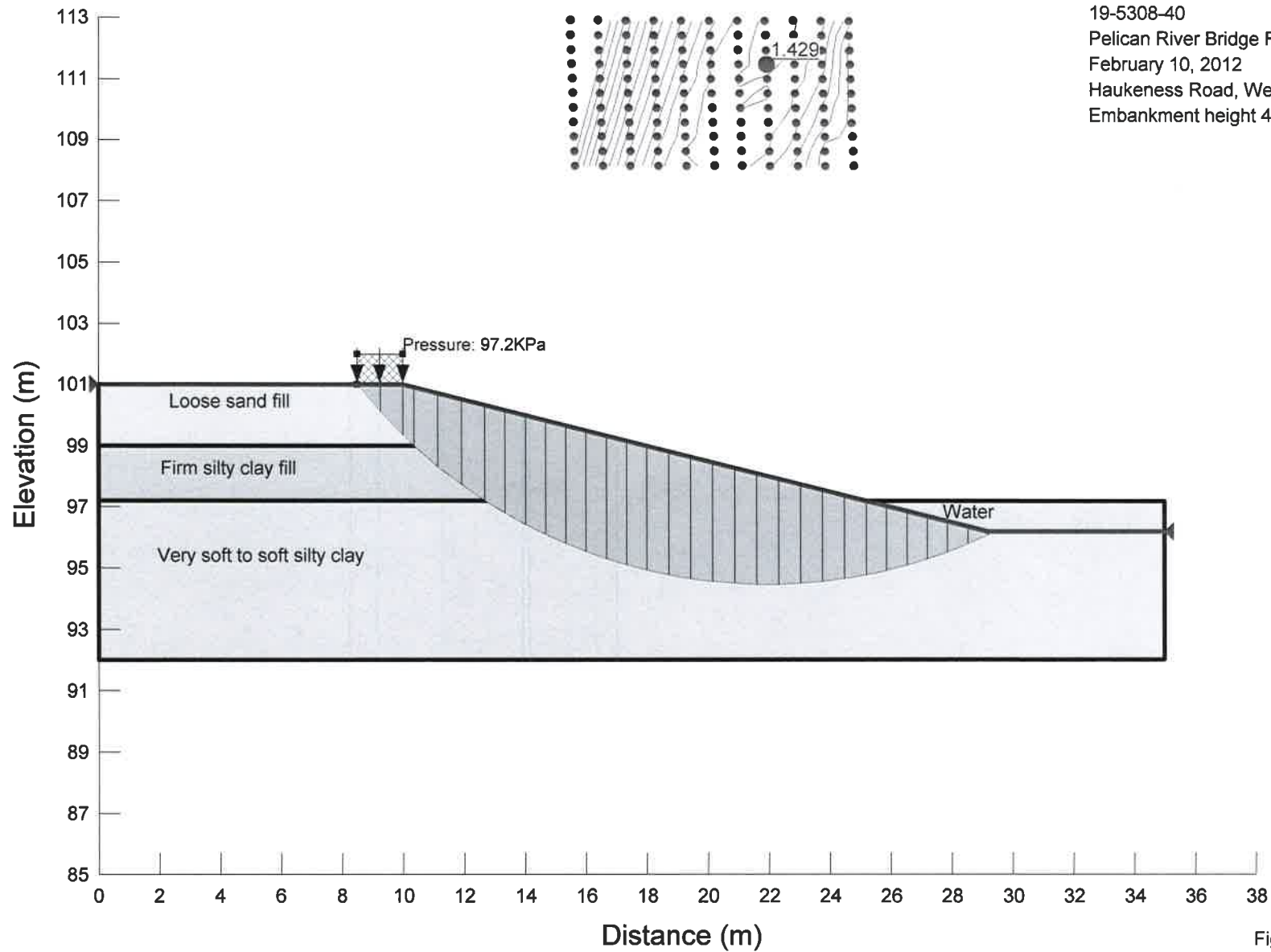
COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Sleeper Slabs at the abutments on Granular Pad	Footings on Native Soil	Caissons	Driven H-Piles to Bedrock
	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction. ii. No deep excavation or removal of soft soils is required. iii. No dewatering necessary. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Potential for settlements. ii. Periodical maintenance to shim up the sleeper slab to compensate for frost action and settlement. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Low available geotechnical resistance in native silty clay deposit. ii. Potential for settlements. iii. High groundwater levels and artesian conditions. iv. Potential disturbance of creek during footing excavation. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for units founded on bedrock. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting bases. iv. Artesian conditions 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to achieve resistance on the bedrock. ii. Installation of piles could continue in freezing weather. iii. Foundation construction may require less volume of excavation than footings. iv. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. No boreholes were drilled at the exact location of the proposed modular bridge piers. iii. Pile lengths required to achieve design resistance may vary.
ABUTMENTS	FEASIBLE	NOT RECOMMENDED	NOT RECOMENDED	FEASIBLE
PIERS	NOT APPICABLE	NOT RECOMMENDED	NOT RECOMENDED	RECOMMENDED

Appendix E

Slope Stability Output

Name: Water Model: (None) Piezometric Line: 1
 Name: Earth Fill Model: Mohr-Coulomb Unit Weight: 21 Cohesion: 0 Phi: 30 Phi-B: 0 Piezometric Line: 1
 Name: Silty Clay Fill Model: Mohr-Coulomb Unit Weight: 18 Cohesion: 0 Phi: 28 Phi-B: 0 Piezometric Line: 1
 Name: Silty Clay Model: Mohr-Coulomb Unit Weight: 17 Cohesion: 0 Phi: 27 Phi-B: 0 Piezometric Line: 1



19-5308-40
 Pelican River Bridge Replacement
 February 10, 2012
 Haukeness Road, West of Hwy 605
 Embankment height 4.0 m, Slope 4.0H:1V

Figure 1

Appendix F
Lateral Pile Analysis

PELICAN RIVER BRIDGE

PIER PILES - LATERAL PILE ANALYSIS

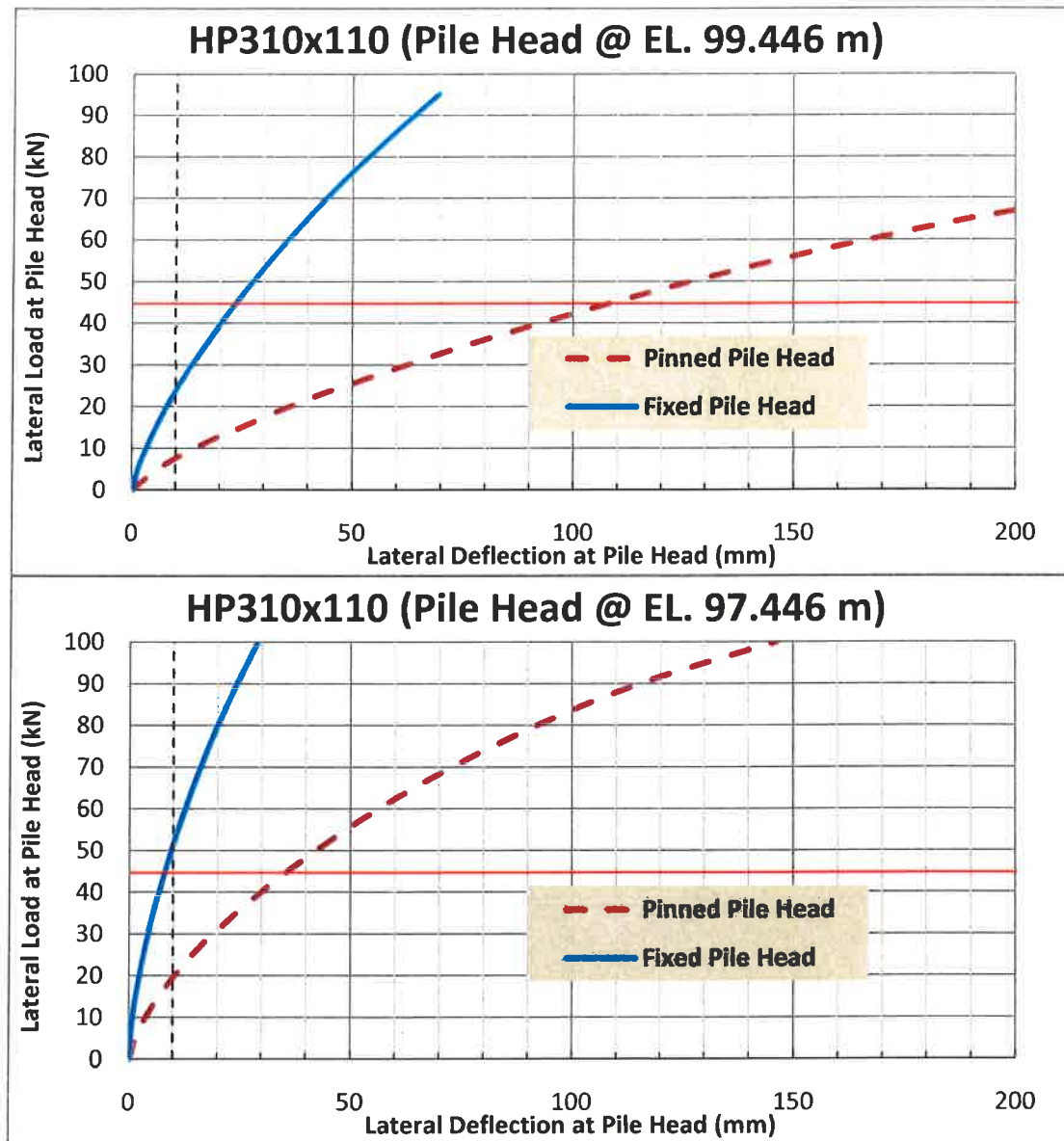


FIGURE 1

PELICAN RIVER BRIDGE

PIER PILES - LATERAL PILE ANALYSIS

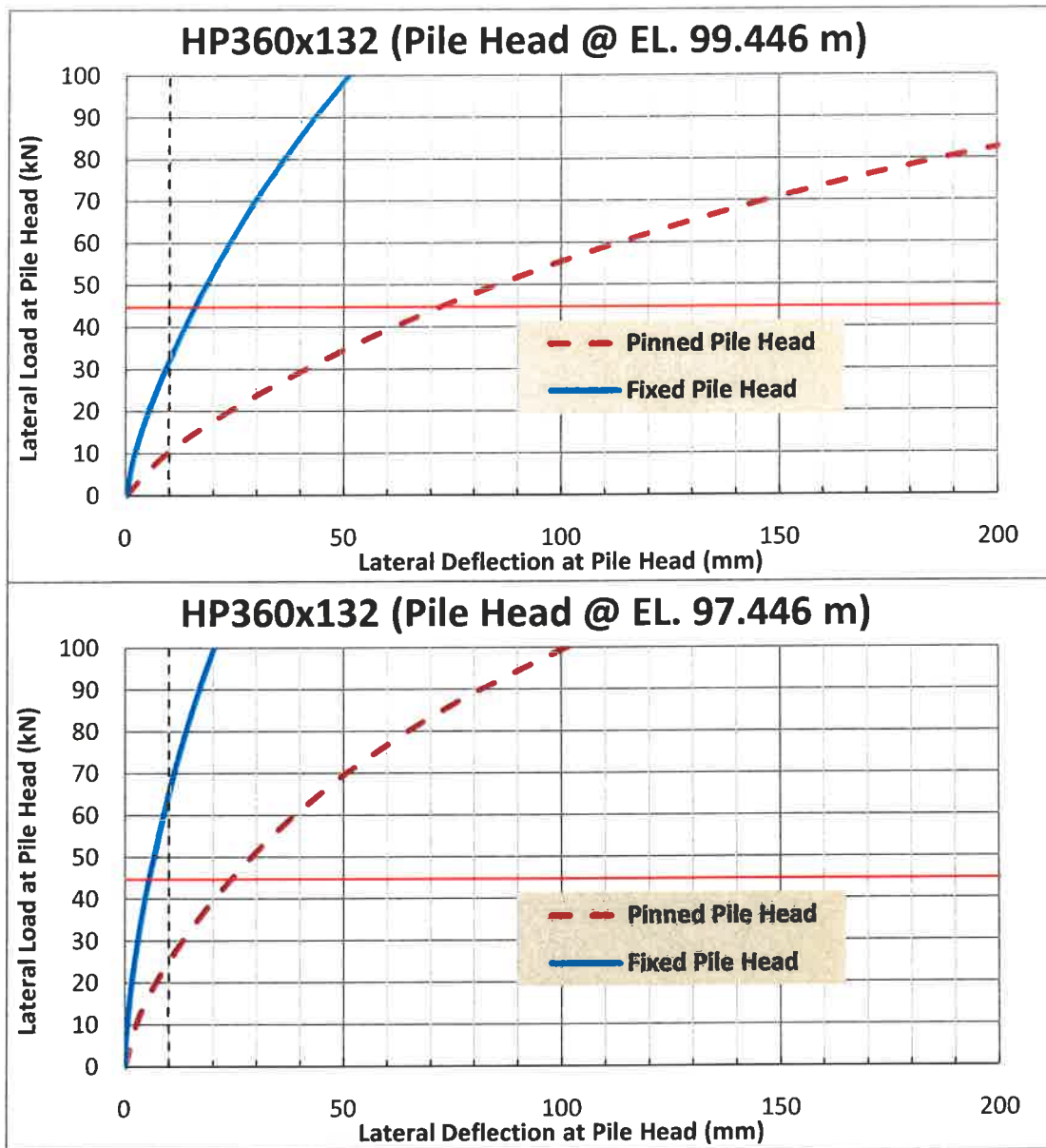


FIGURE 2

Appendix G

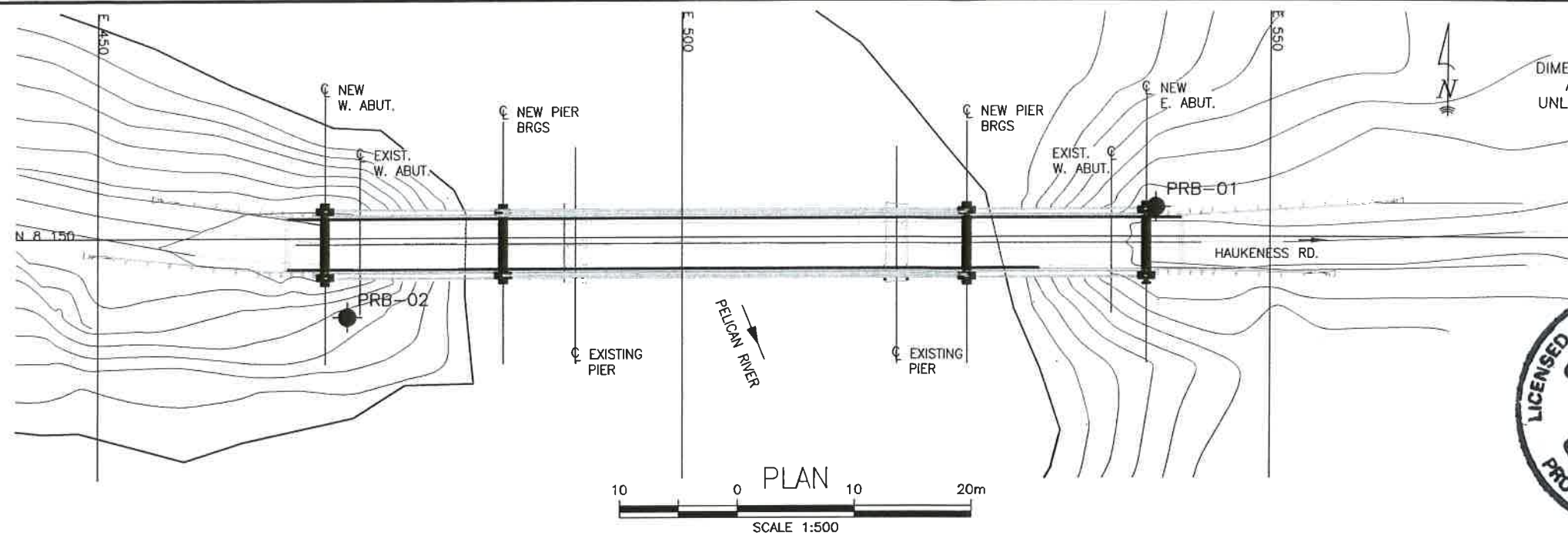
List of SPs and OPSS, and Suggested Text for NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 903, November 2009
- OPSS 804, November 2010
- OPSS 902, November 2010
- OPSD 208.010

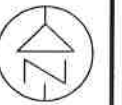
Appendix H

Borehole Locations and Soil Strata Drawings



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



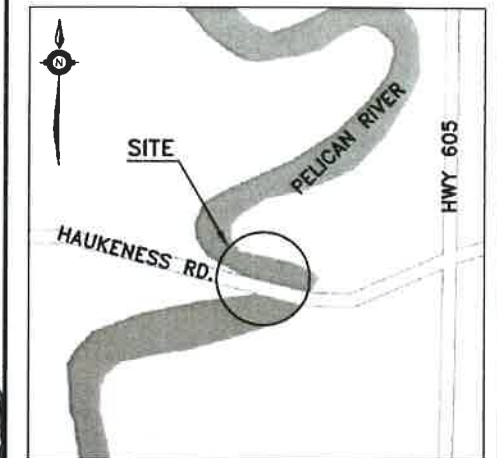
CONT No
WP No 473-00-00

PELICAN RIVER BRIDGE REPLACEMENT BOREHOLE LOCATIONS AND SOIL STRATA






SHEET |

**GENIVAR**

THURBER ENGINEERING LTD.



KEYPLAN
LEGEND

	Borehole
	Dynamic Cone Penetration Test
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
PRB-01	98.8	8 152.6	540.4
PRB-02	100.6	8 143.2	471.4

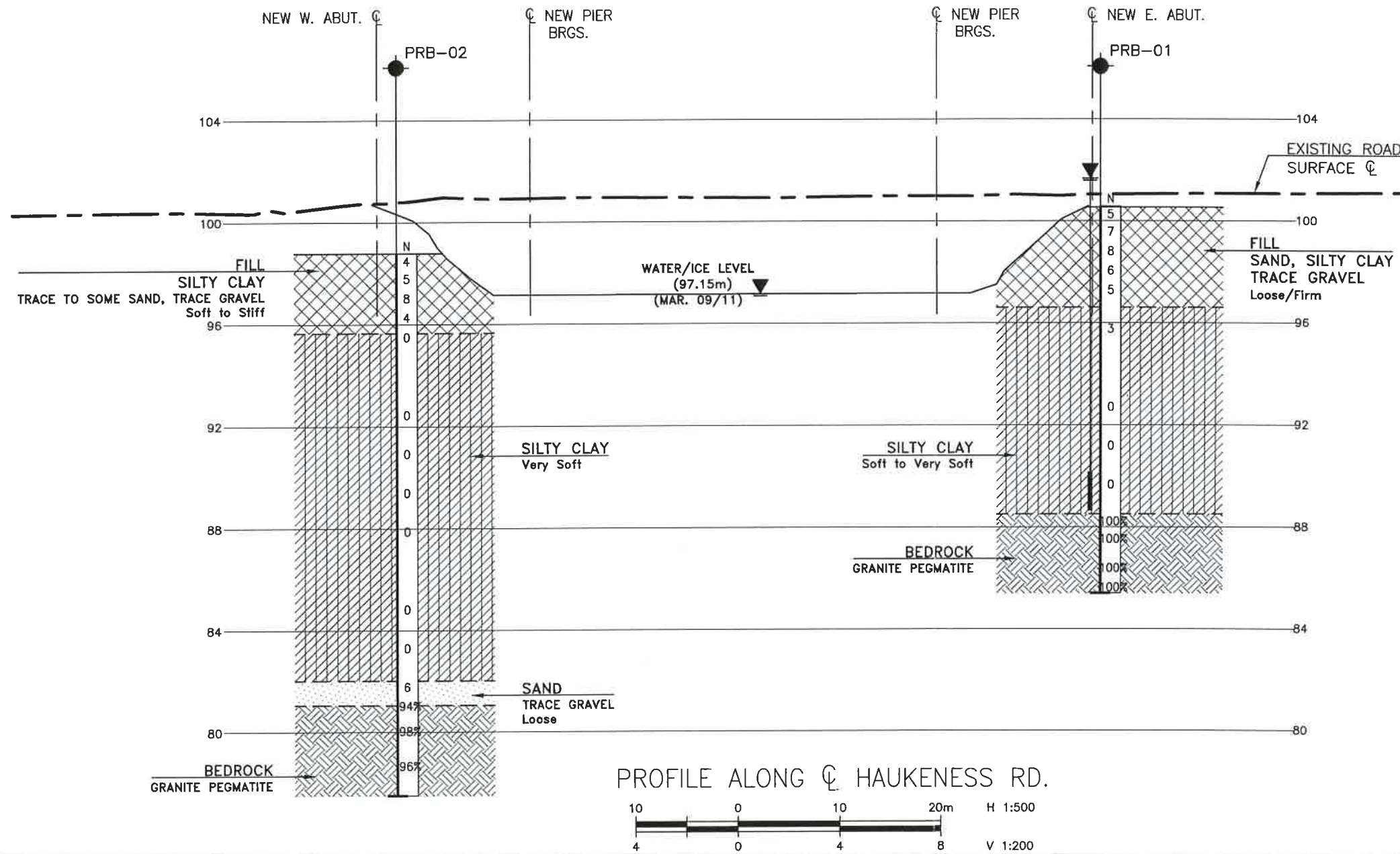
-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- 3) Northing and Easting coordinates obtained from GA drawing provided by MRC.

GEOCRES No. 52F-36

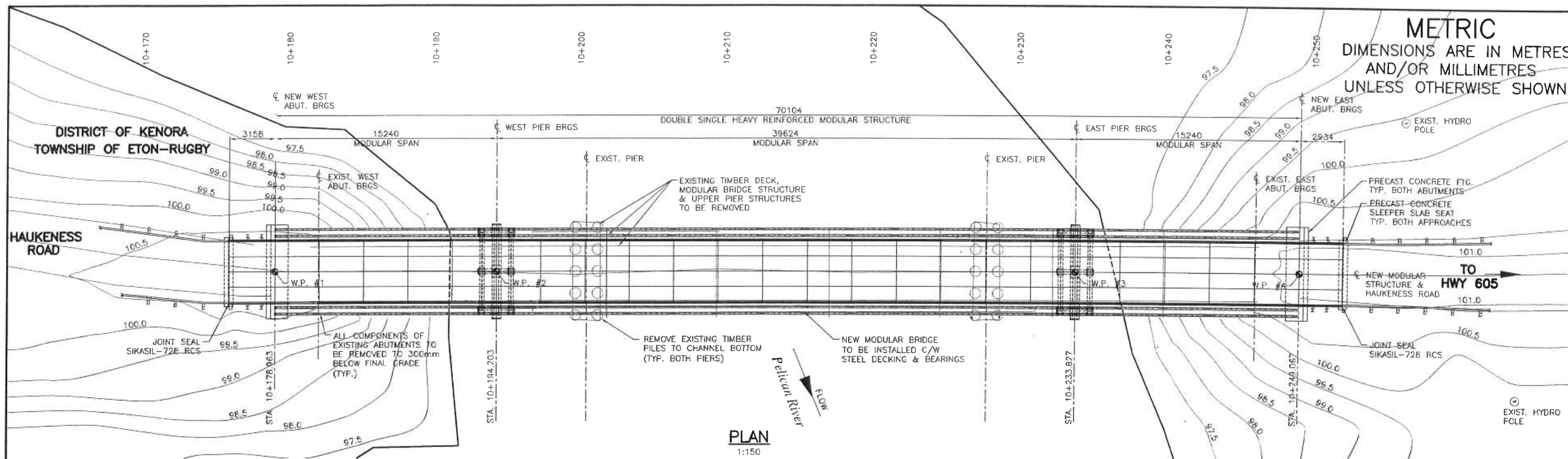
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FILENAME: H:\Draughting\19\5308\38\Ted0839--BoreholePlan.dwg
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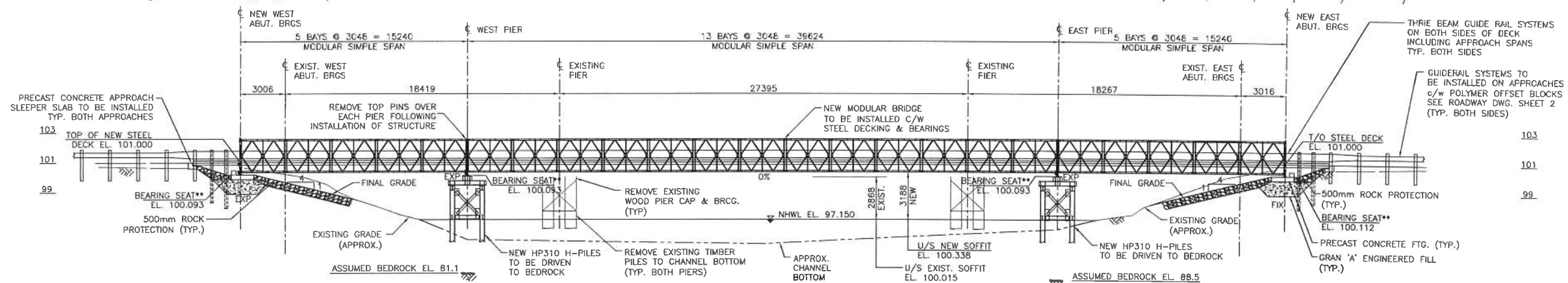


Appendix I

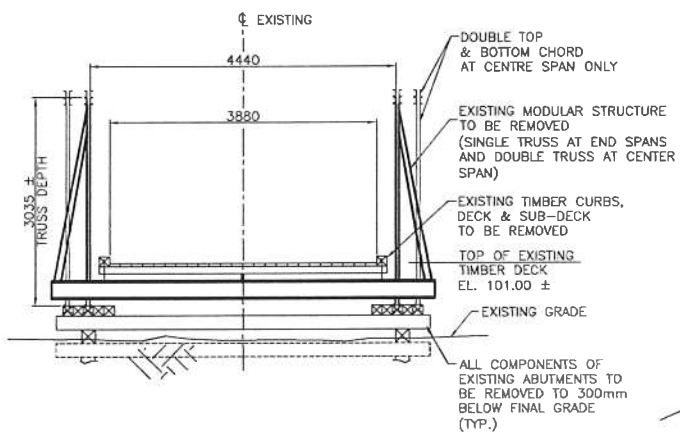
General Arrangement Drawings (GA Drawings)



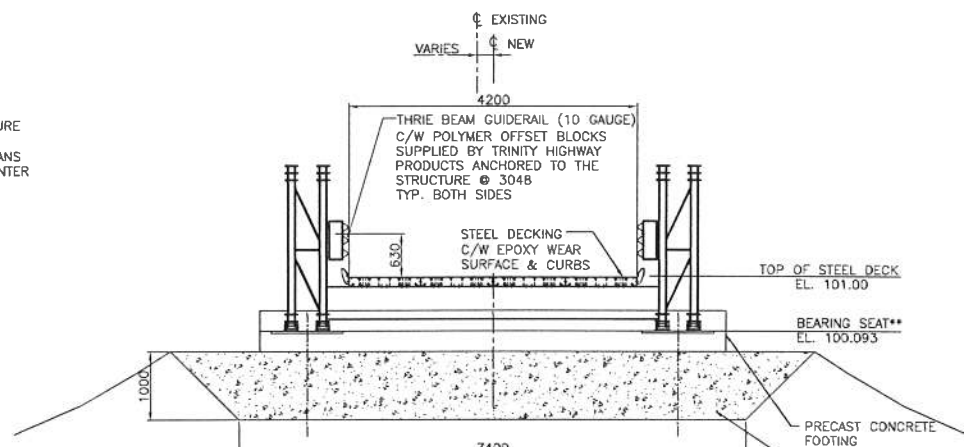
PLAN
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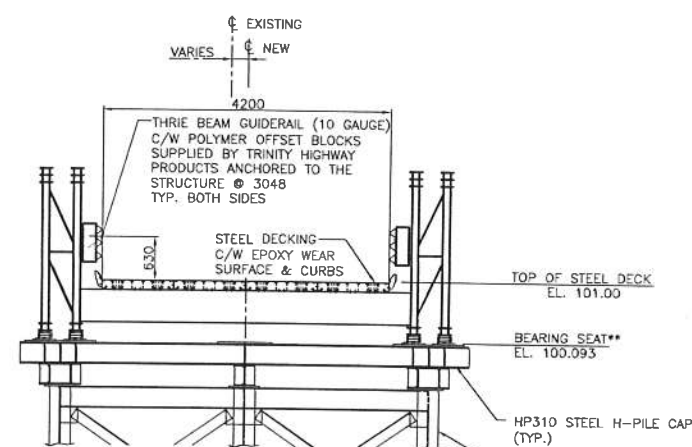
ELEVATION
1:150



TYPICAL SECTION AT
EXISTING ABUTMENTS
1:50



TYPICAL SECTION
@ ABUTMENTS
1:50



TYPICAL SECTION
@ PIERS
1:50

CP-1 BM EL. 100.00
NAIL IN GROUND
25m W. OF W. END OF EXISTING BRIDGE
9.5m S. OF S. EDGE OF ROAD
CP-2 BM EL. 100.94
TOP OF WOOD CURB BOLT
S. W. END OF BRIDGE

** - ELEVATIONS MAY VARY
BASED ON MODULAR SUPPLIER



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

CONT No
WP No

PELICAN RIVER BRIDGE
MODULAR STRUCTURE
GENERAL ARRANGEMENT

GENIVAR
THUNDER BAY, ONTARIO

SHEET
6

- GENERAL NOTES**
- CLASS OF CONCRETE:**
PRECAST SLEEPER SLAB & ABUTMENT FOOTING 40 MPa
- CLEAR COVER TO REINFORCING STEEL:**
CONCRETE - TOP, BOTTOM & SIDES 70±20
- REINFORCING - GENERAL:**
- UNLESS OTHERWISE SHOWN, LAP LENGTHS NOT INDICATED ON THE DRAWINGS SHALL BE CLASS 'B'
 - BAR HOOKS WHERE REQUIRED, SHALL BE MINIMUM LENGTH AND STIRRUPS SHALL HAVE MINIMUM HOOKS UNLESS OTHERWISE SHOWN
 - REINFORCING STEEL SHALL BE GRADE 400W AND BAR MARKS WITH THE PREFIX 'C' DENOTE COATED BARS.

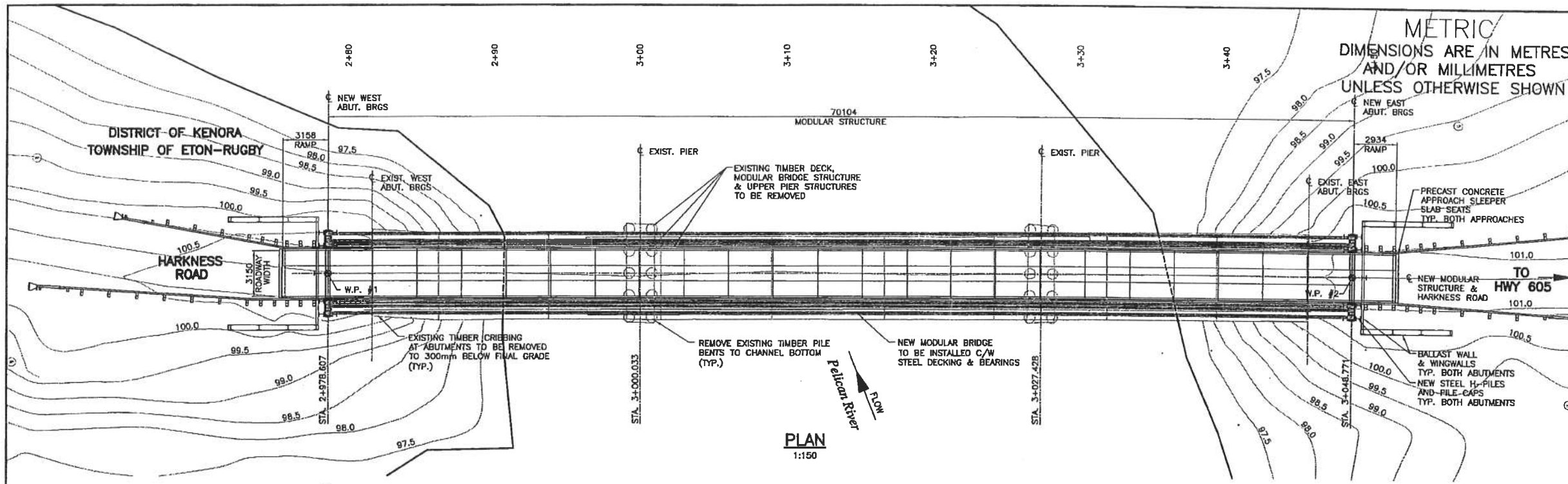
- MODULAR GENERAL NOTES**
- ERECTION AND LAUNCHING OF THE STRUCTURE SHALL BE IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.
 - NEW MODULAR STRUCTURE TO BE CONSTRUCTED ON THE SAME HORIZONTAL ALIGNMENT AS THE EXISTING STRUCTURE.
 - THE CONTRACTOR IS RESPONSIBLE FOR ESTABLISHING ALL LINES AND GRADES CONSISTENT WITH THE DRAWINGS.
 - THE APPROACH RAMP SLEEPER SLAB EKKI TIMBER (50x185) SHALL BE SUPPLIED BY THE MINISTRY AND THE PRECAST CONCRETE SLEEPER SLAB AND ABUTMENT FOOTINGS SHALL BE SUPPLIED AND INSTALLED BY THE CONTRACTOR.
 - THE CONTRACTOR IS RESPONSIBLE FOR THE INSTALLATION OF THE MODULAR BRIDGE STRUCTURE.
 - ALL MODULAR INSTALLATION DIMENSIONS SHALL BE VERIFIED PRIOR TO ANY RELATED WORK COMMENCING.
 - 3 SPAN MODULAR BRIDGE IS SIMPLY SUPPORTED.
 - MODULAR BRIDGE COMPONENTS, INCLUDING ALL HARDWARE AND THREE BEAM GUIDE RAIL COMPONENTS TO BE SUPPLIED BY THE MINISTRY.

- STRUCTURAL STEEL NOTES:**
- ALL STRUCTURAL STEEL SHALL CONFORM TO CSA-G40.20-04 /G40-21-04 (R2009) GRADE 350W, OR ASTM SPECIFICATION A588.
 - THE CONTRACTOR SHALL ENSURE THE STABILITY OF ALL COMPONENTS DURING HANDLING, TRANSPORTATION AND ERECTION UNTIL THE STRUCTURAL STEEL IS IN ITS FINAL LOCATION WITH ALL PERMANENT BRACINGS, CONNECTIONS AND SUPPORTS IN PLACE.
 - UNLESS NOTED OTHERWISE, THE MINIMUM FILLET WELD SHALL BE AS FOLLOWS:
- | MATERIAL THICKNESS OF THICKER PART JOINED (mm) | MINIMUM SIZE OF FILLET WELD (mm) |
|--|----------------------------------|
| TO 12 INCLUSIVE | 5 |
| OVER 12 TO 20 | 6 |
| OVER 20 TO 40 | 8 |
| OVER 40 TO 60 | 10 |
| OVER 60 TO 120 | 12 |

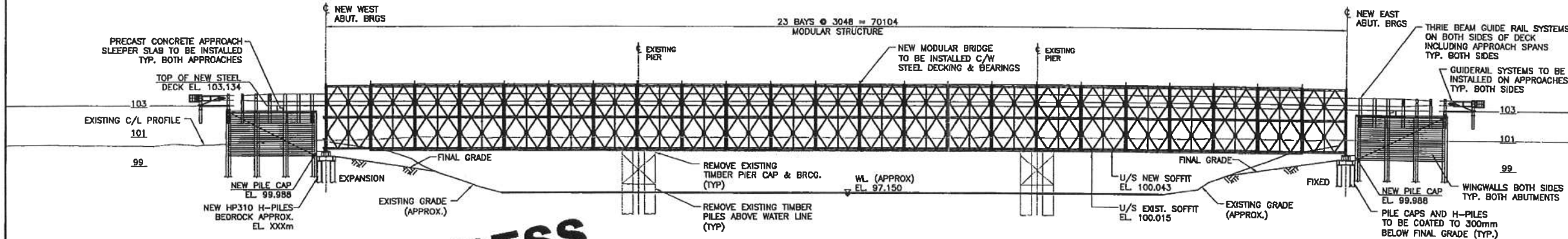
- CONSTRUCTION SEQUENCE**
- INSTALL DETOUR SIGNING.
 - INSTALL ENVIRONMENTAL PROTECTION.
 - REMOVE THE EXISTING MODULAR STRUCTURE AND FOUNDATIONS.
 - DRIVE THE PILES AND CONSTRUCT THE PILE CAPS.
 - INSTALL THE SLEEPER SLABS AND ABUTMENT FOOTINGS.
 - COMPLETE INSTALLATION OF THE NEW MODULAR STRUCTURE.
 - COMPLETE GRADING WORK.

- LIST OF DRAWINGS**
- GENERAL ARRANGEMENT
 - BOREHOLE LOCATIONS & SOIL STRATA
 - ABUTMENT & SLEEPER SLAB DETAILS
 - PILE LAYOUT & DETAILS
 - STRUCTURAL STEEL PIER DETAILS
 - APPROACH RAMPS PLAN & SECTIONS
 - MISCELLANEOUS DETAILS & STANDARDS

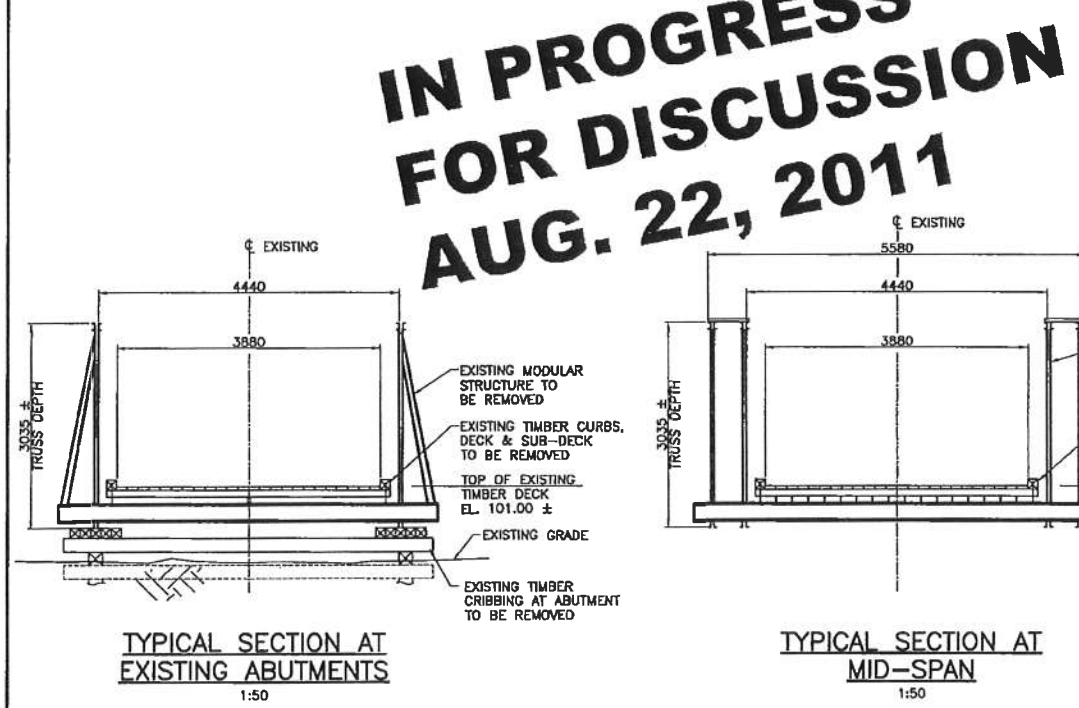
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DRAWN	CSN	CHK	DCR
DATE		DATE	
APRIL 2012		APRIL 2012	
DWG		DWG	
1		1	



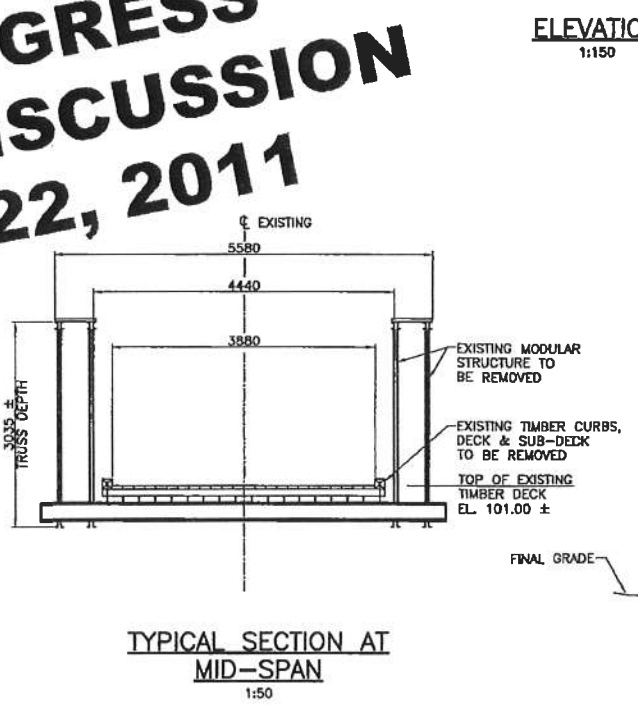
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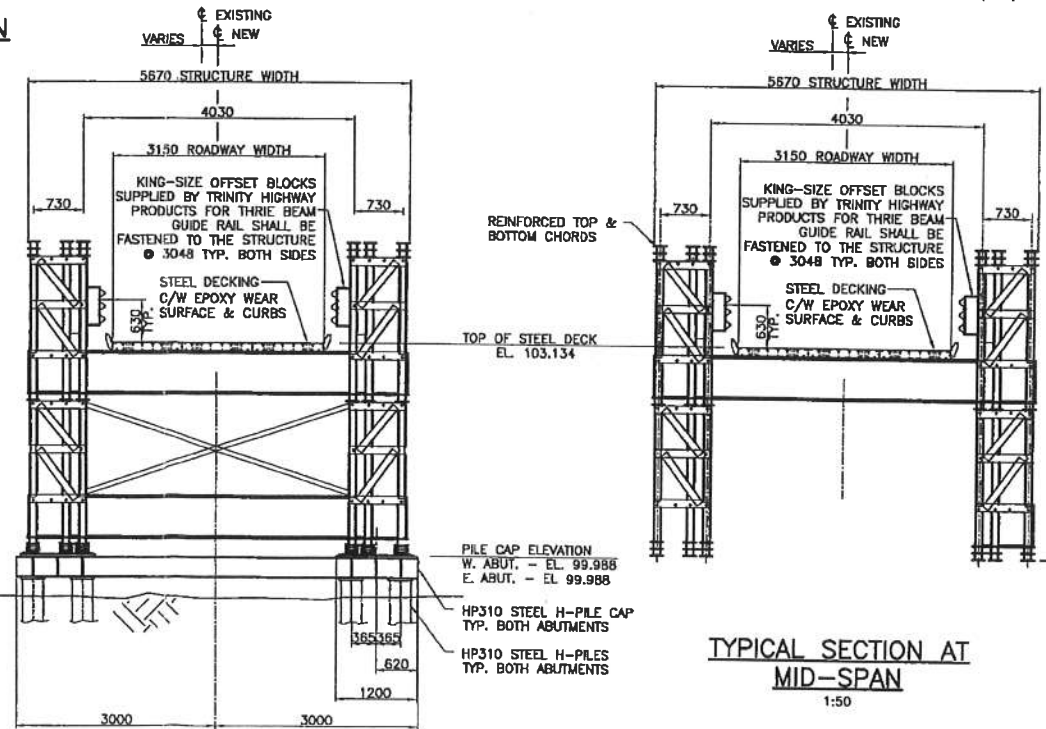
ELEVATION
1:150



TYPICAL SECTION AT EXISTING ABUTMENTS
1:50



TYPICAL SECTION AT MID-SPAN
1:50



TYPICAL SECTION AT NEW ABUTMENTS
1:50

TYPICAL SECTION AT MID-SPAN
1:50

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

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

CONT No
WP No

PELICAN RIVER BRIDGE
MODULAR STRUCTURE
GENERAL ARRANGEMENT



SHEET



GENERAL NOTES

CLASS OF CONCRETE:

PRECAST SLEEPER SLAB 60 MPa

CLEAR COVER TO REINFORCING STEEL:

CONCRETE - TOP, BOTTOM & SIDES 70±20

REINFORCING - GENERAL:

1. UNLESS OTHERWISE SHOWN, LAP LENGTHS NOT INDICATED ON THE DRAWINGS SHALL BE CLASS 'B'.
2. BAR HOOKS WHERE REQUIRED, SHALL BE MINIMUM LENGTH AND STIRRUPS SHALL HAVE MINIMUM HOOKS UNLESS OTHERWISE SHOWN.
3. REINFORCING STEEL SHALL BE GRADE 400W AND BAR MARKS WITH THE PREFIX 'C' DENOTE COATED BARS.

MODULAR GENERAL NOTES

4. ERECTION AND LAUNCHING OF THE STRUCTURE SHALL BE IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.
5. NEW MODULAR STRUCTURE TO BE CONSTRUCTED ON THE SAME HORIZONTAL ALIGNMENT AS THE EXISTING STRUCTURE.
6. THE CONTRACTOR IS RESPONSIBLE TO ESTABLISH ALL LINES AND GRADES CONSISTANT WITH DRAWINGS.
7. THE APPROACH RAMP SLEEPER SLAB EKKI TIMBER (50x185) SHALL BE SUPPLIED BY THE MINISTRY AND THE PRECAST CONCRETE SLEEPER SLAB SHALL BE SUPPLIED AND INSTALLED BY THE CONTRACTOR.
8. THE MODULAR BRIDGE TO BE CONSTRUCTED AT THIS SITE SHALL BE A MABEY COMPACT 200. THE CONTRACTOR IS RESPONSIBLE TO LOAD, SHIP AND UNLOAD THE COMPONENTS AT THE BRIDGE SITE. ALL COMPONENTS ARE LOCATED AT THE MTO JAMES STREET COMPLEX.
9. ALL MODULAR INSTALLATION DIMENSIONS SHALL BE VERIFIED PRIOR TO ANY RELATED WORK COMMENCING.

STRUCTURAL STEEL NOTES:

1. ALL STRUCTURAL STEEL SHALL CONFORM TO THE CURRENT CSA STANDARD, OR CURRENT ASTM SPECIFICATION.
2. THE CONTRACTOR SHALL ENSURE THE STABILITY OF ALL COMPONENTS DURING HANDLING, TRANSPORTATION AND ERECTION UNTIL THE STRUCTURAL STEEL IS IN ITS FINAL LOCATION WITH ALL PERMANENT BRACINGS, CONNECTIONS AND SUPPORTS IN PLACE.
3. UNLESS NOTED OTHERWISE, THE MINIMUM FILLET WELD SHALL BE AS FOLLOWS:

MATERIAL THICKNESS OF THICKER PART JOINED (mm)	MINIMUM SIZE OF FILLET WELD (mm)
TO 12 INCLUSIVE	5
OVER 12 TO 20	6
OVER 20 TO 40	8
OVER 40 TO 60	10
OVER 60 TO 120	12

CONSTRUCTION SEQUENCE

1. INSTALL DETOUR SIGNING.
2. INSTALL ENVIRONMENTAL PROTECTION.
3. REMOVE THE EXISTING MODULAR STRUCTURE INCLUDING PIERS AND EXISTING RIP RAP AND BOULDERS FROM THE CHANNEL.
4. DRIVE THE PILES.
5. COMPLETE INSTALLATION OF THE NEW MODULAR STRUCTURE.
6. COMPLETE GRADING.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE DATA
3. REMOVALS & ENVIRONMENTAL
4. PILING LAYOUT
5. ABUTMENT & SLEEPER SLAB DETAILS
6. MISCELLANEOUS DETAILS & STANDARDS
7. PILE DRIVING CONTROL

REVISIONS	DESCRIPTION
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DRAWN CSN	CHK
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SITE 41S-38	STRUCT SCHEME DWG 1

**IN PROGRESS
FOR DISCUSSION
AUG. 22, 2011**

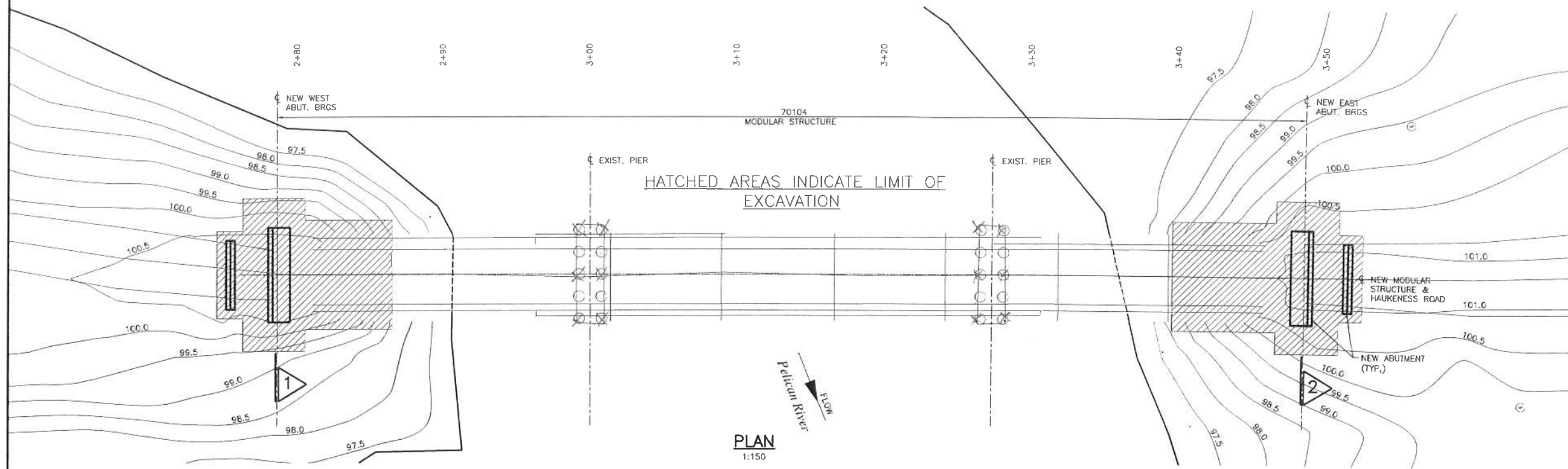
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No

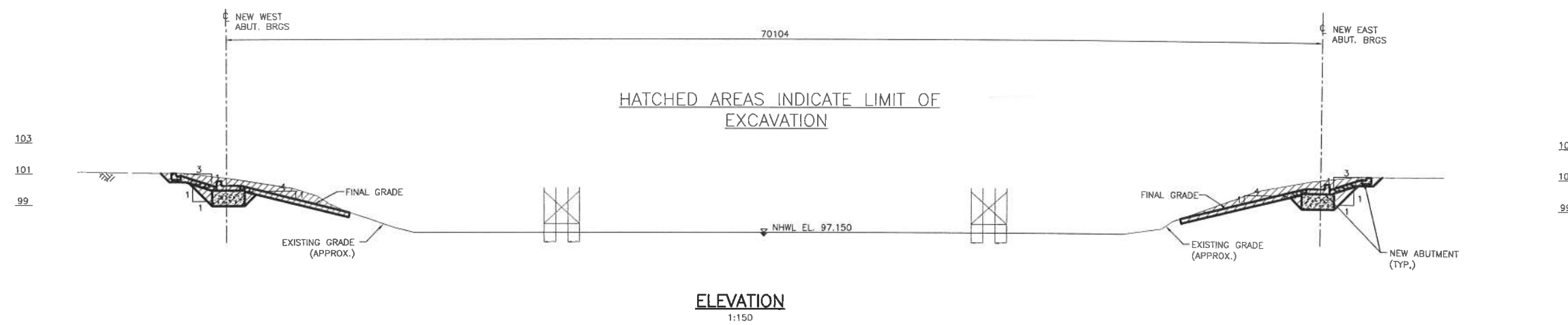
PELICAN RIVER BRIDGE
MODULAR STRUCTURE
ABUTMENT EXCAVATION



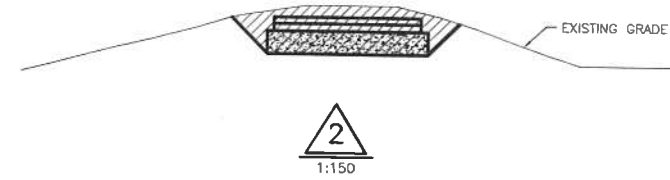
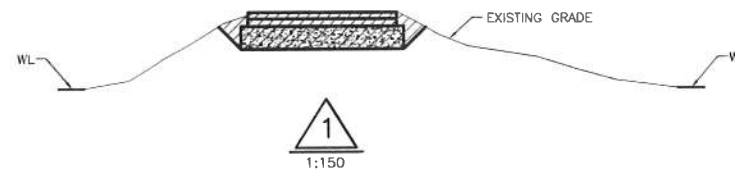
SHEET



PLAN
1:150



ELEVATION
1:150



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

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