BISHOPVILLE ROAD ARCH, NOVA SCOTIA
REDUCTION OF EXCAVATION FOR SOIL FOUNDATION IMPROVEMENT

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ABSTRACT

Applications to reuse materials or reduce “the need” for new materials, and subsequently reduce energy, can take on a multitude of forms. By reducing energy to construct the highway networks, owners can benefit environmentally and economically in the long term. A collective effort, jointly by government and industry, to reduce and reuse highway construction materials can have a positive effect on the environment.

The Nova Scotia Department of Transportation and Public Works (NS TPW) called a tender in July 2007 to replace a corrugated steel tunnel (CST) crossing Highway 101 near Windsor, Nova Scotia. The new concrete arch structure was stipulated as “Design-Build”, allowing the contractor to use innovative construction techniques, acceptable to NS TPW, to provide a cost effective structure replacement. This paper will describe the geotechnical analysis, approval and processes used to reduce the volume of foundation excavation for a new Bishopville Road arch tunnel. It will demonstrate the foundation improvement design did not adversely affect the structural integrity of the new structure. Survey data taken before, during and after final construction of the arch will illustrate the foundation settlement was well within design limits.

The proven results of this case study show that geotechnical analysis of the “in-situ material” along with controlled construction practices can save backfill replacement costs, energy, and emissions to the environment.

INTRODUCTION

In July 2007, Nova Scotia Transportation and Public Works (NSTPW) called a tender namely, “Two Sections of Hants County, Highway 101”, Section 1. From Falmouth Westerly to Kings/Hants County Line; Approximately 5.54 km and Section 2. Bishopville Road Structure (HAN 119) located on Bishopville Road at Highway 101, near Windsor, Nova Scotia.

The existing structure was a galvanized steel elliptical corrugated steel tunnel (CST) which had two lanes of Highway 101 passing over it and Bishopville Road passing through it below (see Figures 1a &1b). The contract required the demolition, removal and replacement of the existing structure with a new “Design/Build” pre-cast concrete arch and all associated foundations, and retaining walls to meet the lines, grades and clearances described in the tender documents and drawings. The contractor had to install the new arch in a staged construction procedure to allow for the continued flow of traffic on Highway 101. Bishopville Road would be closed to local traffic for the duration of this construction sequence.

The contract stipulated that the contractor had the responsibility to Design and Build a new structure meeting NSTPW Bishopville Road clearances, that could support the live and dead loads of Highway 101, and the concrete footings were adequately designed based on geotechnical data provided in the tender documents for supporting foundations. The geotechnical data and spread footing design will be the key element described throughout this paper. It will describe the geotechnical analysis, approval and processes used to reduce the volume of foundation excavation for a new Bishopville Road arch tunnel. It will demonstrate that the foundation improvement design did not adversely affect the structural integrity of the new structure. Survey data taken before, during and after final construction of the arch will illustrate the foundation settlement was well within design limits.

The proven results of this case study show that geotechnical analysis of the “in-situ material” along with controlled construction practices can save backfill replacement costs, energy, and emissions to the environment.

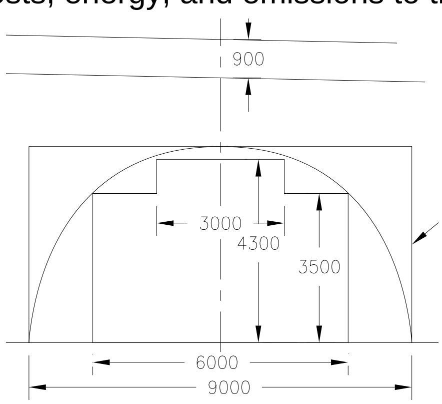

Figure 1a – NSTPW Clearance Envelope

Figure 1b – NSTPW Existing Structure

DESIGN-BUILD STRUCTURE

The NSTPW specified a pre-cast concrete arch with a minimum clear span of 9.0m and a minimum vertical clearance of 4.3m. The overall traffic clearance envelope was pre-determined by NSTPW. The designed structure had to meet the CAN/CSA-S6-06 Canadian Highway Bridge Design Code to withstand the dead load, earth cover above the arch, and the live load designated by CL-625. The concrete spread footings required a design to resist the applied loads through the arch and adequate size to support the new structure on the underlying foundation soils. All concrete components had a minimum compressive strength of 45 MPa at 28 days. The design life of the structure was specified at 75 years (see Figure 2a & 2b).

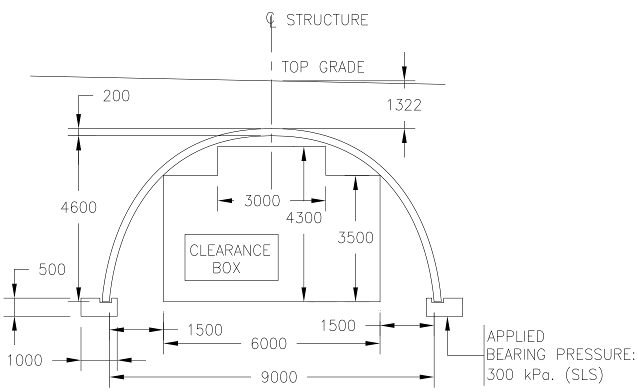

Figure 2a – Design-Build Structure Cross Section

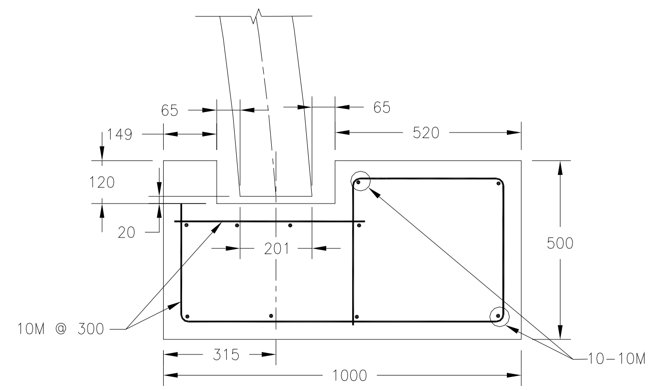

Figure 2b – Spread Footing Design – Section

The minimum specified length of new arch structure was 60.0m excluding any retaining structures used for head walls or wing walls. The horizontal alignment of the new structure closely matched that of the existing multi-plate and had to be within the right-of-way clearances and limits of construction on Highway 101 and Bishopville Road (see Figure 3).

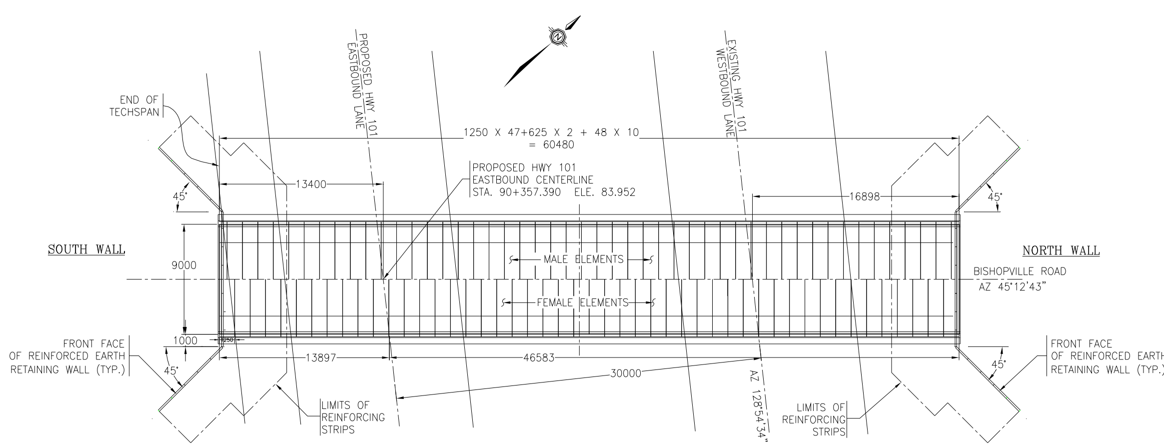

Figure 3 – Design-Build Structure Plan View

The standard NSTPW side slopes of 2:1 horizontal to vertical for this section of highway were to be maintained. The cover over the arch varied from 300 mm minimum in the median to approximately 2400 mm maximum. The new arch structure was a on a vertical longitudinal slope of 0.67% (see Figure 4).

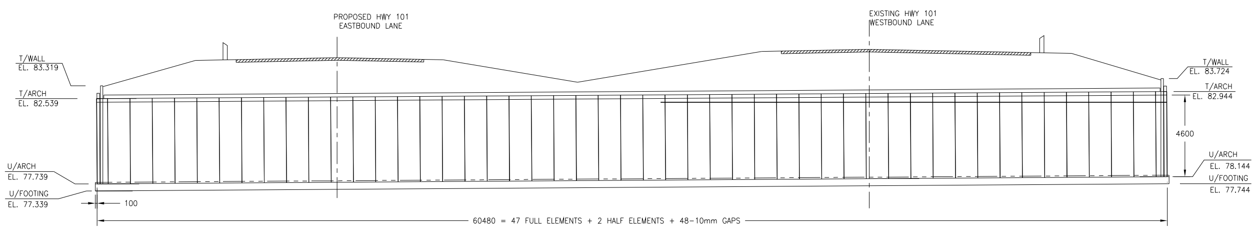

Figure 4 – Design-Build Structure Longitudinal Section

STAGED CONSTRUCTION PROCEDURE

As part of the Design-Build requirement for the structure replacement, the contractor was responsible for traffic control on both Highway 101 and Bishopville Road. The contractor had to propose plan to the NSTPW that would allow traffic to be un-interrupted on Highway 101 by means of staged construction of the new arch structure using detours within the construction limits. In the interest of safety, Bishopville Road was closed to local traffic during the entire demolition, removal and installation of the new arch structure.

The staged construction plan included the design of a temporary Mechanically Stabilized Earth (MSE) wire wall that would be located at the optimum location along Bishopville Road so that it would retain the embankment fill on the eastbound lanes Highway 101 on a temporary basis. This MSE wire wall was temporary in the sense that it was only needed for this portion of the stage construction and for the detouring of the existing two lane traffic of Highway 101. Once the entire new arch was completely installed, backfilled and re-paved on the existing alignment, the temporary MSE walls became redundant are were buried in place (see Figure 5).

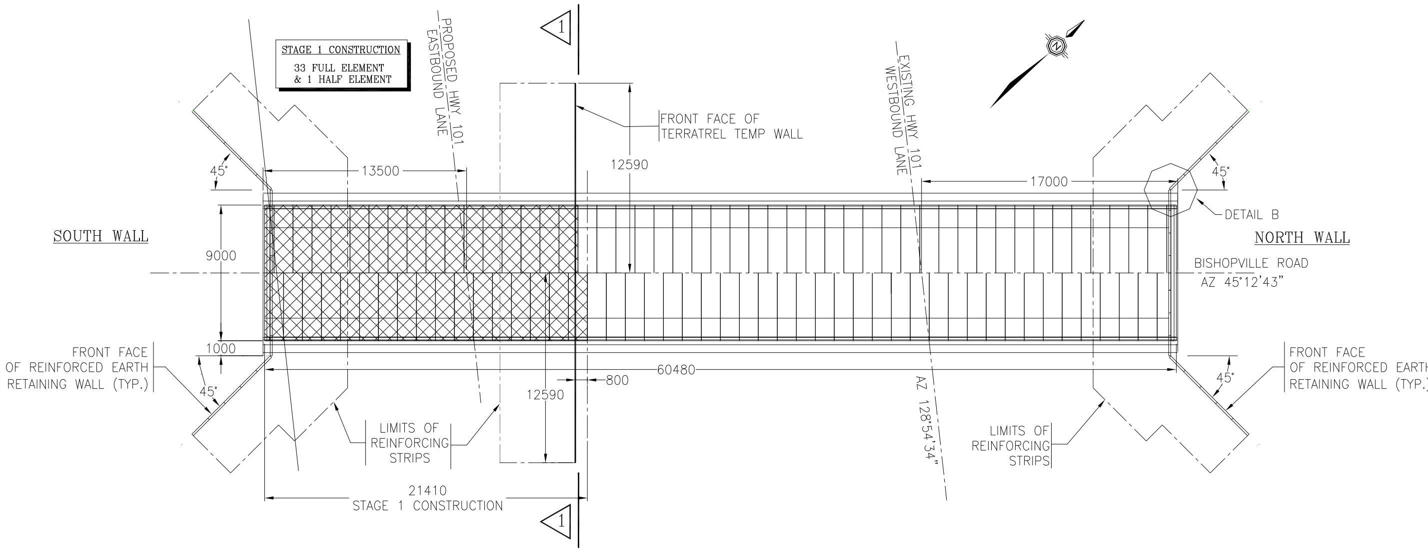

Figure 5 – Staged Construction and Temporary MSE Wall Location

The existing two-lane Highway 101 had 2:1 embankment side slopes. The ends of the CST had cut ends to match these slopes. In order for the contractor to safely maintain traffic flow on Highway 101 and start the staged construction of the new pre-cast concrete arch on the new east bound lanes of highway 101, he had to locate the temporary MSE wire wall 21 m from the south end. This distance would allow for enough driving surface and side slopes for the detour of Highway 101 in stage one of the overall construction (see Figure 6).

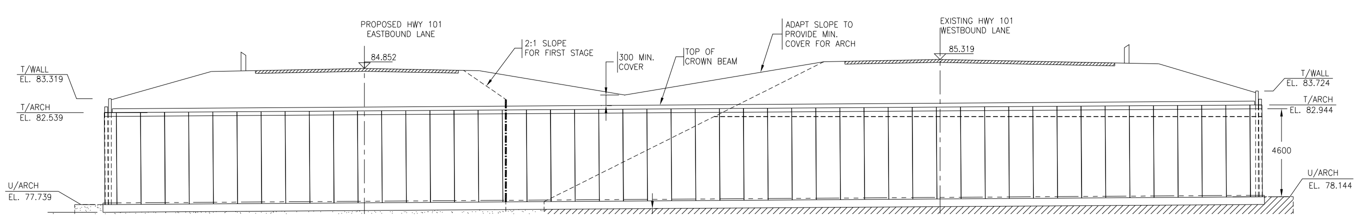

Figure 6 – Staged Construction Temporary MSE Wall Location

The temporary MSE wire wall did not require a specific design life. The only requirement for design was that it had sufficient capacity to maintain the temporary embankment fill and the traffic surcharge from the eastbound lanes of Highway 101 (see Figures 7a & 7b).

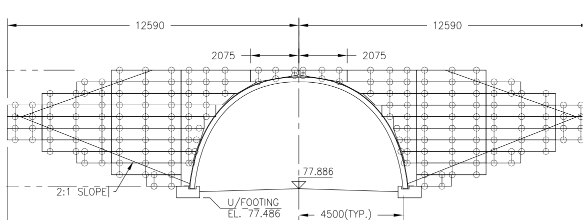

Figure 7a – Temporary MSE Wire Wall

Figure 7b – Temporary MSE Wire Wall

The materials used in the temporary MSE wall were galvanized steel wire basket facing, galvanized steel soil reinforcements with bolt/nut/washer sets, and geotextile used behind the facing to hold the granular backfill in place during the staged construction (see Figures 8a, 8b & 8c).

Figure 8a – Temporary MSE Wall

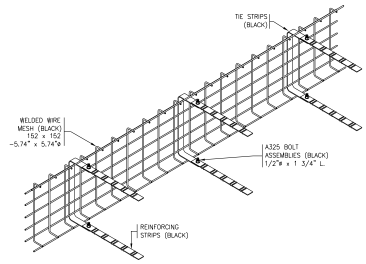

Figure 7b – Temporary MSE Wall

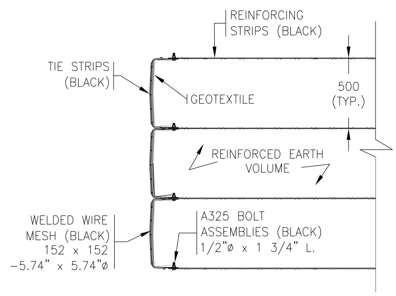

Figure 7b – Temporary MSE Wall

The south end retaining structures were pre-cast concrete faced MSE walls and had to meet the 75 year design life stipulated in the tender documents. They were constructed simultaneously with the temporary walls to have balanced earth pressures on the new arch structure during the backfilling operation (see Figures 9a & 9b)

Figure 9a – Permanent MSE Wall Photo

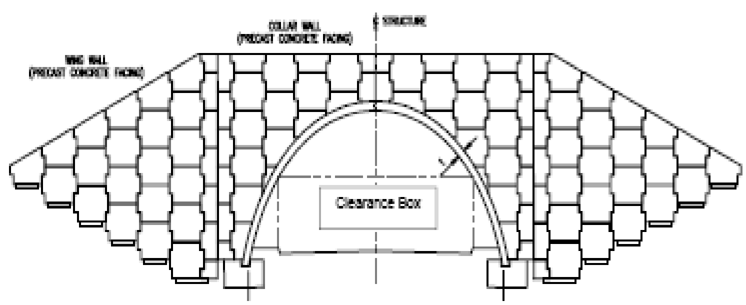

Figure 9b – Permanent MSE Wall Schematic

GEOTECHNICAL ANALYSIS

During the tender period, contractors were to become familiar with job site where the existing Bishopville Road structure was located and the new structure to be placed. A geotechnical investigation was carried out by a geotechnical engineering firm under the direction of NSTPW. This investigation was provided solely for information and the contractors were to draw their own conclusions with respect to the underlying foundation soil condition and its capacity to support the applied pressure of 300 kPa (SLS) from the new concrete arch structure.

As a part of the design-build criteria, the contractor was responsible for foundation design, which included any soil improvement if required. From review of the geotechnical investigation report, two bore holes were drilled at the south end of the new structure. Onsite inspection revealed severely fracture rock was at or near the exposed ground surface at these locations. The north end of the new structure had no boreholes and limited soils information from NSTPW. The contractor had to perform his own foundation testing so he could properly design the new concrete spread footings. Although it was not required by the contractor to locate the rock elevation, it purely provided geotechnical information to be utilized at a later time to reduce the foot print for the soil improvement.

Maritime Testing Ltd. (MTL) was hired to advise the contractor on geotechnical issues. A series of six test pits were excavated. One pit was dug at each of the north and south corners of the new structure location and two at the interface between the existing multi-plate and new concrete arch structure (see Figure 10).

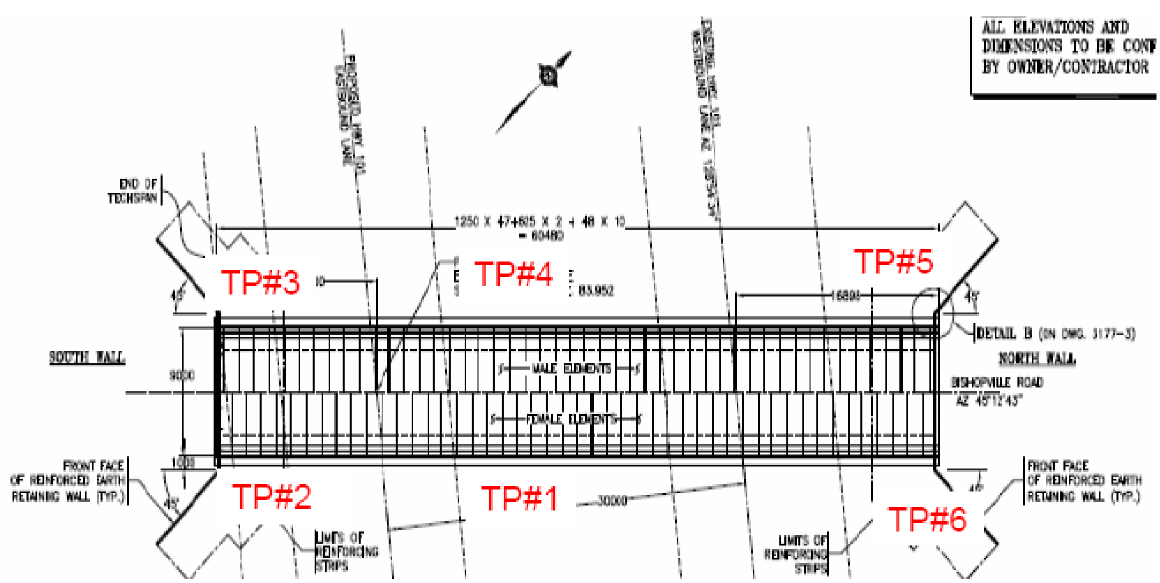

Figure 10 – Test Pit Locations

Bishopville Road Test Pits

Test Pit	Location	Existing Elev (m)	Depth to Rock (m)	Rock Elev (m)	u/s Footing Elev (m)	Fill u/s Ftg (m)
1	Interface West	77.650	2.890	74.760	77.483	2.723
2	SE corner	77.500	1.000	76.500	77.339	0.839
3	SW corner	77.500	0.900	76.600	77.339	0.739
4	Interface East	77.650	2.890	74.760	77.483	2.723
5	NW corner	77.850	3.500	74.350	77.744	3.394
6	NE corner	77.850	5.000	72.850	77.744	4.894

Figure 11 – Test Pit Results

The test pits revealed the rock depths varied from a minimum 0.839 m to a maximum 4.894 m to the underside of footing (see Figure 11). MTL recommended that un-disturbed till could provide 300 kPa (SLS).

General geotechnical practice for undercutting poor foundation soils typically describes an excavated foot print 1:1 horizontal to vertical on either side of the spread footing to an adequate depth of competent soil. For the south end of the structure, the depth of competent soil was determined at 0.5 m to 2.0 m deep. The concrete spread footing was designed at 1.0 m wide. Therefore, the excavation width underneath the footing was calculated to be a minimum 2.0 m wide at the extreme south end of the new structure to a maximum 6.6 m wide at the interface location. MTL recommended a compacted engineered imported rockfill placed at 300-600 mm lifts to within 300 mm of underside of the concrete spread footing. Type 2 gravel was placed and compacted for the remaining lift (see Figures 12a & 12 b).

Figure 12a – Placement of Engineered Fill South End

Figure 12a – Compacted Engineered Fill South End

The underlying foundations soils were originally classified as “unsuitable fill soils”, but with further investigation in stage two of construction, it was re-classified as underlying “sand and gravel layer”. This would have a profound effect on a revised foundation improvement design that would reduce the need for a large excavation in stage two of construction.

REVISED FOUNDATION IMPROVEMENT DESIGN – STAGE TWO

With the current method of mass excavation and replacing the material with imported rockfill and Type 2 gravel on stage one, it was realized by the contractor that the limits of material removal were going to be enormous on stage two. The results of the test pits showed that a maximum depth to competent soil on the north east corner was 5.0 m. This would result in an excavation width of the underlying foundations soils at 18.7 m wide. The northwest corner would have an excavated width of 10.6m. The overall excavation foot print would range from 21.2 m wide at the interface to 29.3 m wide at the north end. The average depth of this excavation would be 4.0 m. The total volume of material to be removed was estimated at 4000 cubic meters. The contractor began exploring alternative methods of improving the bearing capacity of soils while stage one was still in progress.

From the test pit review, it was questioned whether the underlying soils were properly classified as “unsuitable fill soils” or “a native sand and gravel layer”. Upon further analysis the foundation soil classification changed from unsuitable fill with a low bearing capacity to a moderate foundation soil with somewhat increase structural properties. More analysis of the “in-situ” material was needed. Representative samples were taken and it was determined to be native sand and gravel. A revised foundation design could now be proposed to the NSTPW.

It was proposed through GeoTerre Ltd. (GTL) in conjunction with MTL review, that a much reduced excavation foot print could be incorporated with careful control on four major aspects of design and construction of the foundation improvement method for stage two. GTL’s recommendations[1] were:

> 1*. The assumption that the footing load will be distributed through the rockfill to the underlying soil is using simple stress distribution. For this to be applicable, the rockfill needed to be properly compacted in order to develop the necessary lateral resistance, in particular, with the weaker soil (i.e. lower shear strength) along the "outside" edge of the excavation. Compacted backfill placed under the roadway should provide sufficient lateral support for the rockfill on the side between the footings.*
>
> *2. It was proposed to leave in place approximately 1.5 m of existing sand soil above the till/bedrock and then place the rockfill. At the time of construction, an evaluation of the sand was carried out by proof-rolling and weak/soft areas will be sub-excavated and replaced with structural fill. The soil underlying the second stage of the TechSpan had been subjected to loading of the existing roadway for many years. The first stage of TechSpan did not have the benefit of this loading. The distribution of loads was quite different between the existing*

arch and the footings for the TechSpan; however, the underlying native sand layer should have undergone some consolidation during this time.

3. A well-graded rockfill along with Type 2 top layer was required for stage two, similar to that of stage one.

4. If groundwater is encountered in the excavation, it will need to be properly controlled. The excavation should be dry in order to evaluate the existing sand at the base of the excavation prior to placing the rockfill.

Through GTL recommendation and MTL review the revised cross-section of the foundation improvement for stage two was described as a sub-excavation below spread footing grade of 1.5 m in depth and 3.0 m in width (see Figure 13).

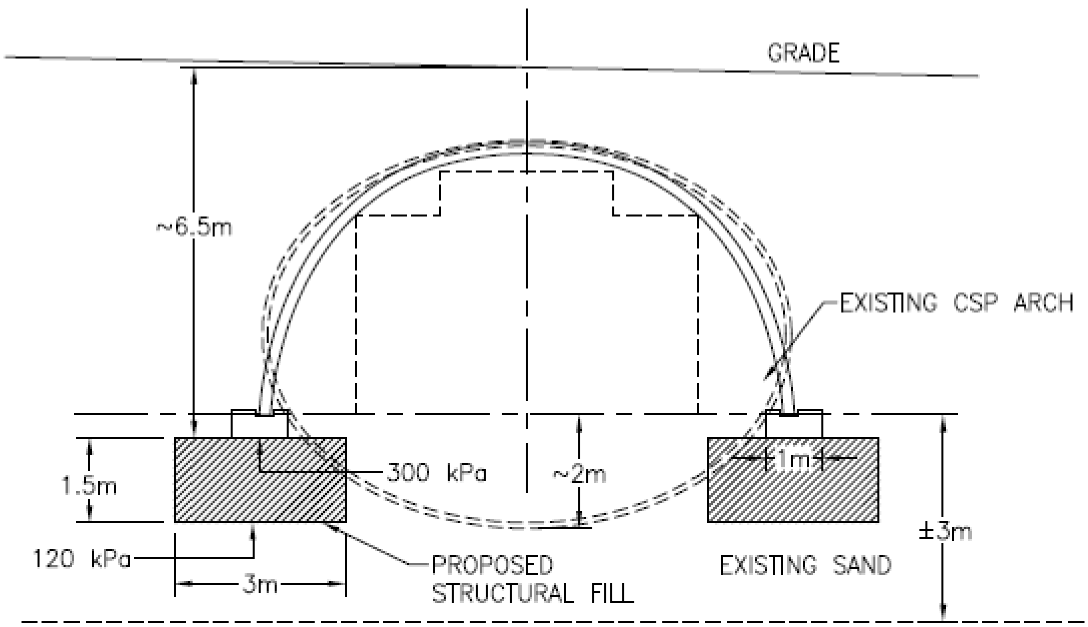

Figure 13 – Revised Foundation Improvement Foot Print – Stage Two

As a part of the revised foundation improvement design, the contractor utilized the services and experience of Maritime Testing Ltd. onsite full time, until stage two foundation improvement was complete. A MTL geotechnical technician inspected the exposed layer of excavation to evaluate soft pockets or areas of concern that required removal. The trench was proof rolled prior to the rockfill and Type 2 placement (see Figures 14a & 14b).

Figure 14a & 14b – Placement and Compaction of Foundation Improvement Material – Stage Two

The contractor requested compaction testing throughout the foundation improvement process, with particular attention to the compaction of the reduced foot print in stage two. The contractor wanted to ensure the backfill material was placed and compacted in strict accordance of the geotechnical design (see Figures 15a & 15b).

Figure 15a & 15b – Compaction Testing Foundation Improvement – Stage Two

To achieve the proper elevation of the underside of the spread footing and to ensure the required compaction value of 100% Standard Proctor Density, a layer of 19 mm minus gravel was placed and compacted (see Figures 16a & 16b).

Test No	Location	Elevation (meters)	Dry Density Kg/m3	% Moisture	% Proctor
1	Southedge of footing	on grade	2203	5.8	99.3
2	Southedge of footing	on grade	2253	7.8	100+
3	Center span of footing	on grade	2248	3.1	100+
4	Retest #1	on grade	2258	5.9	100+
5	Retest #2	on grade	2324	7.6	100+
6	Retest #3	on grade	2329	3.2	100+
7	Northedge of footing	on grade	2230	3.7	100+
8	Northedge of footing	on grade	2218	3.2	100.0

Figure 16a – 19 mm minus Granular Placement

Figure 16b -Compaction Test Results – Stage Two

The transition between stage one and two had to be addressed geotechnically. The maximum excavated depth on stage one was 3.0m versus 1.5 m on stage two. To accommodate for differential settlement between the two stages, a sloped transition engineered fill was placed and compacted (see Figure 17 & 18)

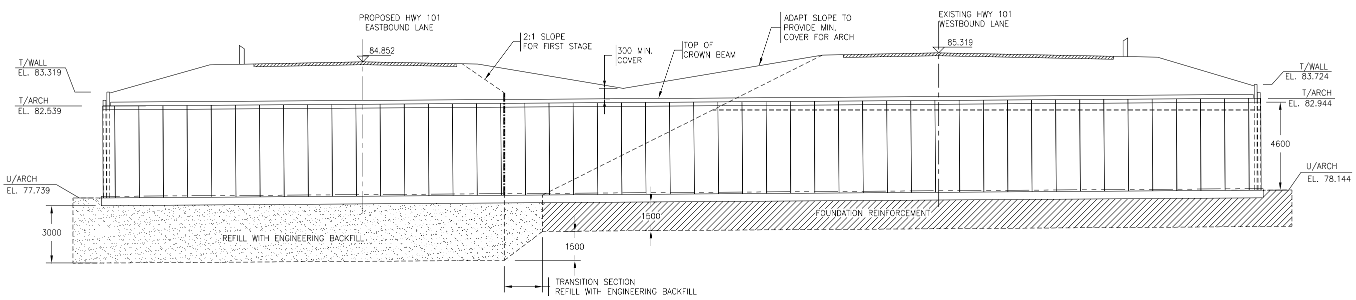

Figure 17 Transition Taper of Engineered Backfill between Stages One & Two

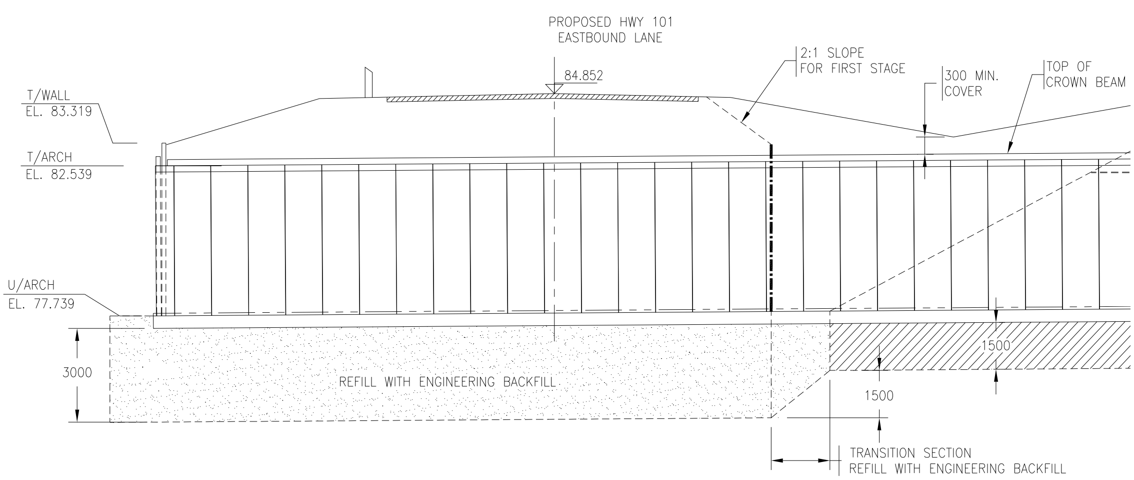

Figure 18 Transition Taper of Engineered Backfill between Stages One & Two

REDUCTION IN EXCAVATION - RECYCLED MATERIALS

By carefully re-analyzing the in-situ foundations soils, test pits results and applying strict control on the construction of the revised improvement design, a reduction in excavated area could be achievable. The contractor applied all necessary quality control on the excavation limits, ground water management, rockfill and Type 2 materials consistency, inspection of the trench(s), placement of the backfill and compaction testing by MTL geotechnical technicians so that the optimum result could be realized.

From the initial assessment and recommendations of the foundation improvement on stage one, it was estimated that a total volume of underlying soil to be removed and replaced in stage two was estimated at 4000 cubic meters at a cost $35,000. To place and compact the engineered rockfill and Type 2 was estimated at $85,000. The total estimated cost was $120,000 for stage two only. The total volume of excavated material and backfill replacement in stage two was 315 cubic meters. A net savings in backfill replacement was estimated at 3685 cubic meters.

SETTLEMENT MONITORING

The contractor wanted to have measured field results for the revised foundations improvement design for stage two to prove the redesign was valid. A plan was developed to set survey targets on the spread footing of stage one and two to measure settlement. Since stage one was entirely installed, backfilled and all anticipated settlement completed by the time stage two revised foundation improvement was approved, it could only be used as a control or base line from which relative settlement between the two stages was measured.

Elevations for stage one were surveyed only once, since this section of arch was completely installed and further movement was not anticipated. The survey data for stage two comprised of:

1. Initial survey shots were taken with concrete arch placed with no backfill.
2. Second survey shots were taken with concrete arch placed with backfill half way up.
3. Third survey shots were taken with the concrete arch placed and backfill complete.

The pre-cast concrete arch can tolerate transverse differential settlement up to 0.5% of the span. The Bishopville Road Arch span measures 9000 mm, therefore the settlement maximum must not exceed 45 mm. The maximum transverse differential settlement recorded at any survey shot location was 4 mm. This proved the revised foundation improvement design for stage two essentially had the same effect as the initial mass excavation concept for stage one.

CONCLUSIONS

The Bishopville Road design-build structure replacement provided challenges for the contractor to evaluate and provide acceptable geotechnical solutions to the NSTPW for foundation improvement. By obtaining experienced geotechnical firms to properly analyze in-situ foundations soils, design for foundation improvement and to have strict quality control on granular material, placement, compaction and testing, it was concluded that minimal differential settlement could be achieved.

The survey field results provided evidence that the spread footings experienced negligible settlement, well within the allowable tolerances for the concrete arch structure. The overall structural integrity of the concrete arch was maintained throughout the construction of stage one and two and will provide the adequate design life necessary in the design-build requirement of this contract.

The revised foundation improvement design in stage two of Bishopville Road arch structure provided a reduction in excavation and replacement of engineered backfill in the amount of 3685 cubic meters and saved one week of machine time.

ACKNOWLEDGEMENTS

The authors acknowledge the participation of the following:

- Nova Scotia Department of Transportation and Works
- Maritime Testing (1985) Ltd.
- GeoTerre Limited
- Alva Construction Ltd.
- Reinforced Earth Company Ltd.

REFERENCES

1. GeoTerre Limited, letter to Reinforced Earth Company Ltd., November 2007