FOUNDATION DESIGN REPORT
HIGHWAY 11
FREDERICKHOUSE RIVER BRIDGE
STRUCTURE REHABILITATION
W.P. 647-90-01

GEOCRES NO. 42H-32

Prepared For:

LEA CONSULTING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1142A
February 1, 2006

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5. DISCUSSION & RECOMMENDATIONS

As shown in Drawing No. 1 and the photographs presented in Appendix C, the Frederickhouse River Bridge is an approximately 135.5 m long, five-span structure. It is approximately 10 m wide and carries the two-lane Highway 11 over the Frederickhouse River near Cochrane, Ontario.

It is our understanding that abutments of the bridge, which was built in late 1940's, have been showing signs of instability. Observations made by LEA indicate that the abutments are overturning. The ballast walls appear to have engaged the bottom chords of the trusses as the bearings are deformed and fixed bearings at the adjacent piers appear to have deformed under the lateral push of the spans.

We understand that some years ago, the instability was more apparent in the west abutment area. The deformation was largely arrested by removing soil from behind the abutment wall, as shown on Drawing No. E-1, Appendix E.

More recently, similar signs of instability were also noted in the east abutment. Calculations made by LEA have shown that overturning of the spread footings which support the abutment is the problem.

Boreholes drilled to the east of the east abutment and to the west of the west abutment show the presence of fill extending to depths ranging between 4 and 13 m below the ground surface. The fill is underlain by a silty clay deposit which extends to a depth of 12 m at the boreholes drilled at the east abutment location and to about 15 m at the west abutment location. The consistency of the silty clay ranges from firm to very stiff, but is generally stiff. The silty clay is in turn underlain by a silt deposit. At the east abutment area, the thickness of the silt is about 4 to 5 m and from the recorded N-values it appears to be in a loose condition. At the west abutment area, the silt is much thicker (e.g. 11.5 m thick at BH 4). The recorded N-values indicate that it is loose to compact near the top, becoming compact to dense below.

Underlying the silt, the boreholes show the presence of coarse-grained granular deposits with frequent cobbles and boulders. The relative densities of these deposits are estimated to be compact to dense, becoming very dense with increasing depth.

In the deep piezometer installed in BH1, the groundwater table at the time of our investigation was recorded at a depth of 12.8 m (El. 244.8 m) or about 4 m above the water level in the River. A shallow piezometer installed in the same borehole indicated the presence of a perched water level in the fill at a depth of 2.4 m below the ground surface. The groundwater table can be expected to be subject to seasonal fluctuations, in response to major weather events as well as fluctuations in the water level in the River.

5.1 FOUNDATIONS

Information available to us shows that the abutments of the bridge are supported on spread footing foundations which are about 4.8 m wide and about 11 m below the existing grades or at about El. 246.7 m (bottom of footing elevation) at the east abutment and El. 247.1 at the west abutment location. The boreholes show that the soil beneath the footing is likely to consist of silty clay for about 1.0 m at the east abutment location, followed by loose silt to about 241 m (i.e. to about 5 to 6 m below the footing bottom) which is in turn underlain by more competent coarse grained granular soils.

Borehole 4 drilled on the west side of the west abutment location shows the presence of silty clay beneath the bottom of the footing for a vertical distance of about 4 m. In Borehole 5, fill was encountered at the footing elevation level but it is unlikely that the footing would have been constructed on fill materials. In this borehole, the fill extends to about El. 245.4 m (i.e. to about 1.7 m below the bottom of the footing), followed by silty clay to about El. 243 m (i.e. to about 4 m below the footing level), similar to Borehole 4. Underlying the silty clay both boreholes contacted an extensive silt deposit. Standard Penetration tests indicate that the silt is generally loose to about El. 237 m and is likely to be compact, becoming compact to dense with depth. More competent coarse granular deposits were contacted below about El. 230 m.

Based on these results, along with field and laboratory test data (including consolidation test data), the following geotechnical resistances would be available at the footing bearing elevation for the 4.8 m wide footing.

Geotechnical Resistance at SLS = 100 kPa
Factored Geotechnical Resistance at ULS = 240 kPa

The ULS value is for vertical loading only and does not include normal reduction for inclined loading which is required as per Canadian Highway Bridge Design Code.

These values are for soil conditions some distance beyond the abutment footings. Immediately below the abutment footings, the soil would have consolidated and settled since the 1940's and depending on the pressures exerted the ULS and especially SLS values will have increased. These values are difficult to quantify but the SLS value is estimated to be of the order of 130 kPa.

At the footing elevation, for the concrete and undisturbed silty clay interface, a friction angle of 24 degrees can be assigned to calculate the unfactored horizontal resistance against sliding.

5.2 BRIDGE ABUTMENT REHABILITATION OPTIONS

We understand that the bridge will be rehabilitated to increase its useful life span by five to ten years.

LEA has considered the following options for this purpose regarding the abutments.

5.2.1 REPLACE THE ABUTMENT WALLS AND FOUNDATIONS

As was mentioned before, the existing abutment foundations (spread footings) are considered to be inadequate with today's standards. If the abutment footings were to be replaced, the use of deep foundations could be considered but this option would not be cost-effective for the proposed rehabilitation. As well, major roadway protection would be required.

To briefly expand on this option from a geotechnical foundations point of view, steel H-piles may be considered. The piles will have to be driven to refusal in the coarse granular soils underlying the silt. They will, however, likely encounter premature refusal on boulders in these deposits at depths ranging between 19 and 25 m at the east abutment location and between 27 m and 30 m at the west abutment or between El. 239 and 228 m. Based on this, the following conservative tentative values of the order of

SLS = 1000 kN/pile
ULS = 1500 kN/pile

can be considered for preliminary design of HP 310 x 110 steel H-piles which are driven to practical refusal in the overburden. Other details such as reinforcing the pile tips can be provided if this approach is considered feasible. The vibrations generated during pile driving may have detrimental effects especially in view of the presence of loose silts and therefore, vibrations will need to be monitored. As such, the use of driven piles carries the risk of settlement of the existing foundations.

Drilled and cast-in-place concrete piles (caissons) generate less vibrations but because of the presence of granular soils below groundwater table as well as the presence of cobbles of boulders they will be difficult (if possible) to install and as such these are not recommended.

5.2.2 EXTEND ABUTMENT FOOTING HEEL

It is our opinion that the resistance values at the existing footing levels are inadequate and any new loadings and widening of the existing footings will likely induce additional settlements. In addition, extensive excavation would be required to reach the bottom of footing elevation which is considered to be impractical. As such, this option is considered to be unreliable as well as being costly.

5.2.3 EXTEND ABUTMENT FOOTING TOE AND BACKFILL WITH LIGHTWEIGHT FILL

As mentioned before in our opinion, the resistance values at the existing footing levels are inadequate. In addition, extensive excavation would be required, which would be costly. Limited access will require complex site access and major roadway protection during construction owing to very deep excavations which would be required. For these reasons, this option is not recommended.

5.2.4 UTILIZE TIEBACKS TO RESIST HORIZONTAL PRESSURES

This approach is considered to be impractical as well as being costly and unreliable, especially since tiebacks will increase the existing vertical loading on the existing abutment footings which are believed to be under designed especially for SLS. It is our opinion that any additional vertical loads will cause increased settlements along with rotation of the existing footing foundations.

In addition, the subsurface conditions are not suitable for the installation of tiebacks for the following reasons:

- The existing silty clay fill cannot be relied upon to provide permanent reliable design resistance. In the underlying silty clay deposit, tiebacks will be subject to relatively large deformations before any resistance can be mobilized and long-term creep is possible, which would not be compatible with the deformation requirements of the abutment.

- The silt deposit underlying the silty clay is wet and water-bearing and as such it will be difficult to install permanent tiebacks, as well as providing low resistance values.

- The coarse-grained granular deposits will provide adequate resistance (i.e. of the order of 60 kPa) but it will be very costly to install the tiebacks within these deposits, as they are below the groundwater table and especially since they contain frequent cobbles and boulders.

5.2.5 CONSTRUCT A NEW 9.5 M APPROACH SPAN WITH A NEW ABUTMENT FOOTING

The existing silty clay fill does not appear to have received systematic compaction and as such can not be relied upon to support the new abutment footing. Because of this and the relatively weak nature of the underlying silty clay, constructing a new spread footing to support the new abutment will be costly as well as being unreliable.

The abutment may be supported on driven piles but in our opinion vibrations created during pile driving will likely be detrimental to the stability of the existing abutment footings. This is because vibration will lead to increased earth pressures on the abutments and may also cause the settlement of the silt deposit where it is loose.

Concrete auger-cast foundations founded in the dense coarse granular soils can be considered but these will unlikely be cost-effective.

5.2.6 CONSTRUCT A NEW APPROACH SPAN WITH RSS

This approach is considered to be unreliable since the silty clay and the underlying loose silt deposits are likely to undergo considerable settlements. The RSS (Reinforced Soil System) wall will require significant excavation, complex site access and extensive roadway protection during its construction.

5.2.7 RELIEVE EXISTING ABUTMENT PRESSURES BY MEANS OF PARTIAL EPS BACKFILL

This approach requires the excavation of the existing fill materials behind the east abutment to a sufficient depth and replacing the fill with EPS (expanded polystyrene blocks) in order to relieve the pressure behind the wall to a sufficient degree to decrease the overturning and vertical pressures on the existing footing to arrest and stabilize the movements of the wall.

We understand that the west abutment was some years ago stabilized by excavating the soil behind it and that this approach appeared to be successful.

In our opinion, a similar approach is likely to be the most practical, albeit short-term solution for the remaining ten year service life of the structure. The excavated area can then be filled with virtually weightless EPS instead of leaving a void space. For design purposes, a bulk unit weight of 0.6 kN/m^3 and a K_o value of 0.1 can be assumed for EPS. Proper drainage should be provided to ensure hydrostatic pressures are relieved; as well the EPS will not be subject to uplift. In general, a pavement cover of not less than 1.2 m is used over the EPS. The 1.2 m cover can consist of reinforced concrete slab underlain by about 0.9 m of granular soil consisting of Granular 'A' or Granular 'B' Type II material.

Drainage can be provided by means of a drainage layer at the bottom of the excavation. This could consist of a 0.2 m thick 20 mm clear crushed stone underlain by a geotextile for separation purposes, such as Terrafix 270R, or approved equivalent. The bottom of the excavation should be graded to avoid undulations where water can accumulate as well it should be sloped towards the abutment wall (say 3%). A perforated drain pipe should be placed near the abutment wall and this should have proper outlet(s) to discharge the collected water. The geotextile should then be extended to wrap the entire system (i.e. top and sides of the blanket) and an 8 cm thick layer of sand should be placed before placing the EPS as a levelling course and to avoid damage to the EPS from the crushed stone.

The design and construction of EPS should be in accordance with MTO Special Provision entitled "Expanded Polystrene Embankment" (a copy of which is given in Appendix F). Sufficient protection is required from the detrimental effects of ultraviolet light (e.g. minimum 0.3 m earth cover) and/or any spillage.

We recommend that the existing fill be sloped at 2H:1V and benched before placing the EPS. The approximate volume of the EPS for preliminary estimating purposes is about 750 cu metres.

We understand that with EPS replacement, the required geotechnical resistance values for the abutment foundations are as follows:

Geotechnical Resistance at SLS = 182 kPa
Factored Geotechnical Resistance at ULS = 245 kPa

These values are substantially lower than the bearing pressures calculated for the existing abutment foundations with the present loadings.

The geotechnical resistances required with EPS replacement (i.e. SLS=182 kPa and ULS=245 kPa) are acceptable based upon the concept that no additional horizontal loads will be applied, but rather there will be a substantial reduction of horizontal overturning loads from the existing conditions, for the proposed 5 to 10-year working life period.

5.2.8 RELIEVE PRESSURE BEHIND THE EAST ABUTMENT WALL BY REMOVING SOIL

Similar to what was done at the west abutment location this may be a cost-effective short-term solution. However, the approach slab will require a foundation support beyond the excavated zone. As was mentioned before, the existing deep fill will not provide sufficient resistance for a spread footing. If some degree of maintenance is acceptable, including periodic raising of the grade under the slab support, the slab can be supported on compacted fill (e.g. 2.0 m thick compacted granular fill). For this purpose, the existing fill at the slab support area (i.e. beyond the void at top) will need to be removed to a depth of about 2.0 m. The exposed fill will be compacted from surface and the soils that were removed (i.e. 2.0 m) will be replaced with compacted Granular 'A' or Granular 'B' Type II soil. As the underlying fill settles, the support of the slab will also settle and thus the slab will need to be raised at the support by such means of lifting and padding and/or grouting. With this approach, the horizontal length of the unsupported portion of the reinforced slab will likely be limited to about 8 m to reduce foundation loadings on the compacted granular soil and the underlying uncompacted fill.

The unsupported face of the existing fill materials should be cut back at slopes no steeper than 2H:1V. It is recommended that at least 0.25 m thick gravel sheeting (Granular 'A' or Granular 'B' Type II material) be provided on the face of the clayey slopes. As well, unsupported height of the slope should not exceed 6.0 m.

The approach slab will however, increase the load on the existing abutment footing and therefore the approach is unlikely to be acceptable.

5.2.9 SUMMARY OF SUGGESTED METHODS

The following is a summary of bridge abutment rehabilitation options discussed in the preceding paragraphs.

Table 5.1
Summary of Short-Term Abutment Foundation Rehabilitation

Foundation Rehabilitation		Comments	Recommendations
5.2.1	Replace Abutment Foundation	Inadequate geotechnical resistance for spread footing. Deep Foundations very costly.	Spread Footings are not recommended based on reliability and cost. Deep Foundations not recommended based on cost.
5.2.2	Extend Abutment Footing Heel	Widening footings will cause additional settlements. Deep excavations required for construction are impractical and costly.	Not recommended based on reliability and cost.
5.2.3	Extend Abutment Footing Toe and Backfill with Light-Weight Fill	Widening footings will cause additional settlements. Deep excavations required for construction are impractical and costly.	Not recommended based on reliability and cost.

Foundation Rehabilitation		Comments	Recommendations
5.2.4	Utilize Tiebacks to Resist Horizontal Pressures	Tiebacks will increase vertical loading and cause additional settlements. As well, subsurface conditions are not well suited for tieback design and installation.	Not recommended based on reliability (i.e. increase in vertical loads) and cost.
5.2.5	Construct a new 9.5 m long approach span with new abutment footing	Existing fill and underlying soils are not suitable for spread footing support. Deep foundation support is costly.	Not recommended based on reliability (spread footings) and cost (deep foundation). May, however, be considered if it reduces cost for temporary shoring required during construction.
5.2.6	Construct a new approach span with RSS	Will likely experience excessive settlements	Not recommended based on reliability and logistics and extensive shoring required during construction.
5.2.7	Relieve existing abutment pressures by means of partial EPS backfill	From previous experience on the west abutment, relieving pressures appear to be suitable.	Recommended as a short-term solution provided structural calculation can verify this approach. Shoring during construction will be required to maintain traffic and this may be costly.
5.2.8	Relieve pressure behind the east abutment wall by removing soil	Duplicate west abutment solution by using a shorter span reinforced concrete approach slab supported on the east end by a 2 m thick compacted granular pad over existing silty clay fill; with a spill through abutment.	Could be considered a short-term, cost-effective solution, provided maintenance can be provided as the soils underlying the slab support settle. However, it is likely to increase the loadings on the existing footings and may not be feasible from reliability point of view.

5.3 CONSTRUCTION

The groundwater table at the site appears to be close to the high water level in the River. However, when carrying out excavations, the presence of the perched water level encountered in the fill deposit should also be taken into consideration.

All excavations, shoring and backfilling should be carried out in conformance with the safety regulations of the province, as well as the following specifications:

SP 539S01 – Protection Schemes
SP 902S01 – Excavation and Backfilling to Structures

Above the water table, the silty clay fill and the natural silty clay deposits can be classified as Type 3 soil, while the granular fills can be classified as Type 3 above water table and Type 4 soil below water table. The silt underlying the silty clay is Type 4 soil.

Assuming that the highway must be kept open and that the traffic will be diverted to a single lane during construction, the construction will be limited to one-half the approach embankment width. Roadway protection should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criteria for the structure performance level. This instance, the Performance Level should be 2.

The possible alternative methods for the rehabilitation of the abutments of the structure require excavation of the fill behind the abutments. To facilitate this excavation, vertical shoring will likely be required. The cost of shoring will increase significantly with the depth of excavation; as such we recommend the chosen rehabilitation method should minimize excavation depths.

The more feasible and cost-effective methods of rehabilitation will probably require excavation of up to about 6 m below the existing road grades. The following are some of the possible methods of constructing the shoring.

5.3.1 CANTILEVER RETAINING WALL

Normally, temporary cantilever retaining walls are about 3 to 4 m in height to limit the deflection. The use of tied back wall is preferred; however, earth anchors have a limited usefulness as was discussed in Section 5.2.4 of this report. In addition, this may be impractical along the mid-way of the high embankment. To construct a soldier pile and lagging type wall will require very heavy steel soldier piles, closely placed, if deflection is to (or can be) controlled. Also, a reinforced continuous caisson wall can be considered. A tied crib-type wall can also be considered.

When considering shoring excavation, an earth pressure coefficient of 0.45 (i.e. close to K_o) should be utilized because of the vehicular traffic. Soil bulk unit weights of 19 kN/m^3 are applicable. The shoring system should be designed by a Professional Engineer experienced in this type of work.

5.3.2 REINFORCED EARTH RETAINING WALL

The vertical excavation can be maintained by incorporating a facing such as shotcrete and soil nails. In this case for the length of soil nails, a preliminary design construction of 0.8H can be assumed where H is the height of the excavation. A typical spacing could be as close as 1 m on centers, both horizontally and vertically. While shotcrete is an acceptable facing, other material such as precast concrete segments or timber can be considered.

The soil nailed wall forms a gravity type retaining wall that should be designed using the gravity wall concept for overturning, sliding, etc.

It is possible that the soil facing of shotcrete or even soldier pile and lagging can be tied back to the existing wingwall. In this case, suitable whaler can distribute the load from the ties and the ties can be located at various elevations. For a soldier pile and lagging base penetration of the soldier piles should be at least 2 m below final excavation level; for the case of the soil nailed facing with shotcrete, the facing will terminate at excavation level. For design purposes, a value of K_a (active) of 0.45 K_p (passive) of 1.0 in the fill and 1.5 in the clay is recommended. These walls as discussed should be considered as gravity retaining walls where sliding and overturning are a consideration. Safe net bearing values (SLS) of 120 kPa can utilized, however, this loading is presently in place.

Once the first one-half of the approach is excavated and as the EPS is placed, ties to support the facing can be attached to the other side wingwall to increase the stability of the EPS section. If required a coefficient of subgrade reaction for the fill and/or clay of 8 MPa/m is recommended.

The earth retained structure may be preferable over the cantilever or soldier pile and lagging wall, as the deflections are easier to control. The practicability and constructability of this method for this site should, however, be discussed with a specialized contractor who should also design it.

5.3.3 POST EXCAVATION OF APPROACH SLAB

A unique construction procedure can consist of the construction of the approach reinforced concrete slab on the existing fill, alternatively on each lane of traffic, followed by the excavation of the soil beneath the slab, if construction equipment can gain access to beneath the slab. The excavation after construction of the approach slab and floating abutment wall may sufficiently relieve the earth pressure on the existing abutment, as discussed in Section 5.2.8 of this report. A final slope beneath the slab is recommended as 2H:1V it may, however, be possible to steepen it to 1½H:1V with surface treatment of the slope, if necessary.

We understand however that the ballast wall will require replacement so the fill will need to be excavated before the slab can be built. Therefore, this may not be a feasible solution. Also the approach slab will increase the load on the existing abutment footing, and this is undesirable.

5.4 FROST PROTECTION

The frost protection for the general area is 2.6 m. Therefore, a permanent soil cover of 2.6 m or its thermal equipment is required where necessary (e.g. foundations). Rock fill can be assumed to provide one-half of frost cover provided by earth fill.

6. CLOSURE

The Limitations of Report, as quoted in Appendix G, are an integral part of this report.

SHAHEEN & PEAKER LIMITED

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Z.S. Ozden, P.Eng.

for K. R. Peaker, Ph.D., P.Eng

LICENSED PROFESSIONAL ENGINEER
K. R. PEAKER
PROVINCE OF ONTARIO

ZO:tr/idrive

Appendix E

General Arrangement Drawing

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

CONT No WP No 647-90-01	
FREDERICKHOUSE RIVER BRIDGE HWY. 11-BRIDGE REHABILITATION GENERAL ARRANGEMENT	SHEET —

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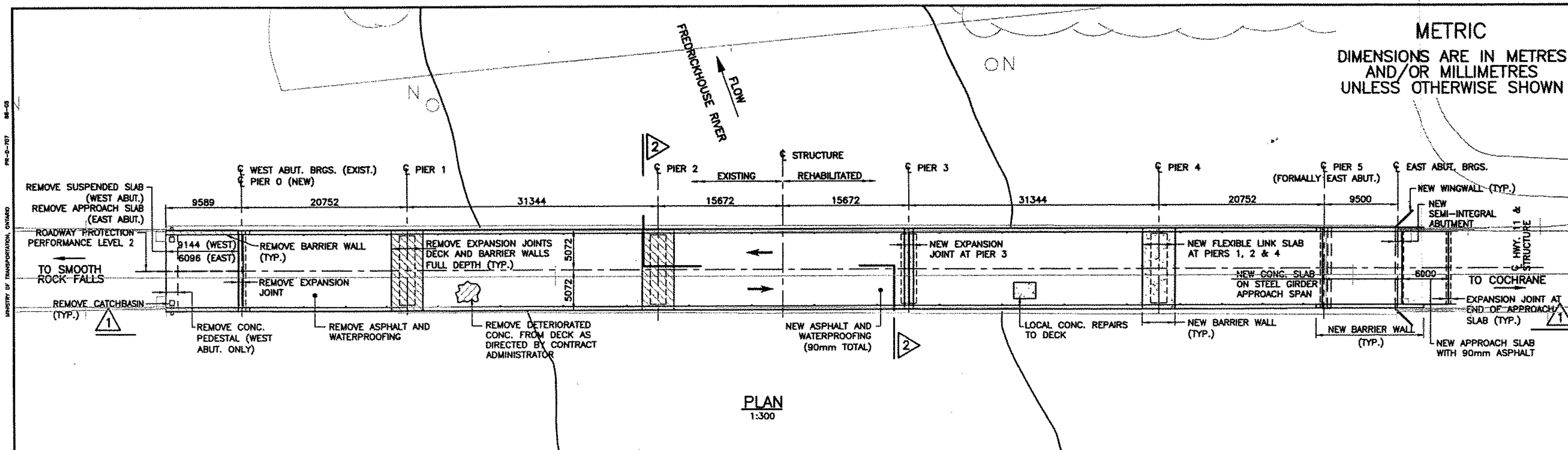
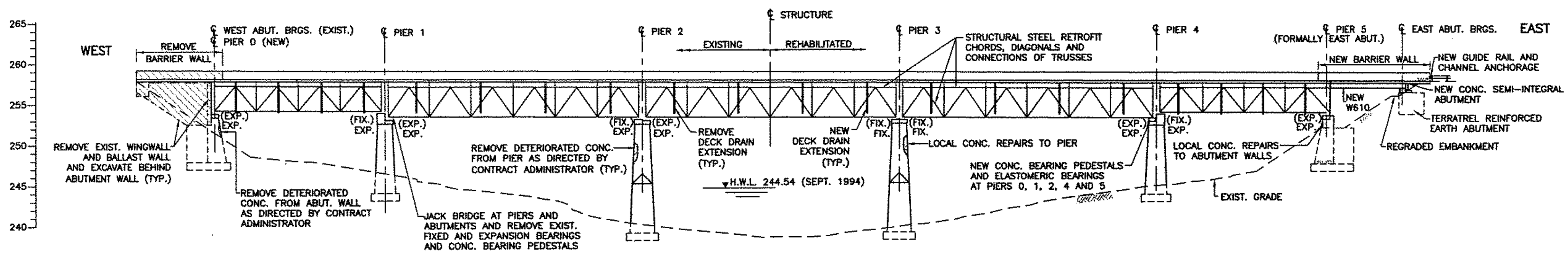
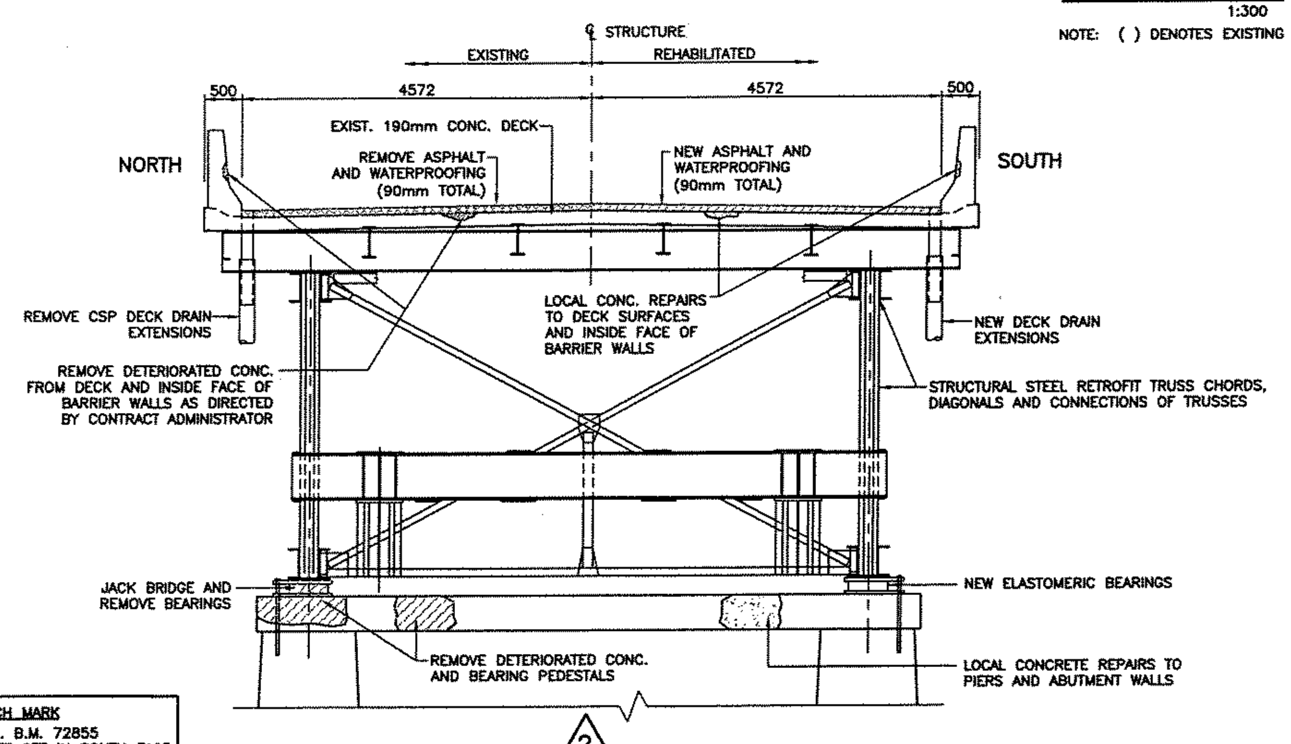
Consulting Engineers and Planners
www.LEA.ca



PLAN
1:300



ELEVATION
1:300
NOTE: ( ) DENOTES EXISTING BEARING



1:50

Drawing No. E-1

General Arrangement Drawing

GENERAL NOTES:

CLASS OF CONCRETE .... 30 MPa

CLEAR COVER TO REINFORCING STEEL

DECK TOP .... 70± 20
BOTTOM .... 40± 10
REMAINDER (UNLESS OTHERWISE NOTED) .... 70± 20

REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.

BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS.

ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1 AND SS12-2 UNLESS INDICATED OTHERWISE.

STRUCTURAL STEEL

ALL STRUCTURAL STEEL SHALL CONFORM TO CSA STANDARDS CAN/CSA-G40.21-M92.

ROLLED SECTIONS SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.21-M92.

ALL BOLTS SHALL BE ASTM A325 M22 TYPE 3.

CONSTRUCTION NOTES

1. CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING STRUCTURE AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH RELATED FIELD WORKS AND/OR SHOP FABRICATION.
2. NO BACKFILL SHALL BE PLACED BEHIND BALLAST WALLS ABOVE BEARINGS UNTIL BALLAST WALL AND DECK CONCRETE HAS REACHED 25 MPa STRENGTH.
3. SAWCUTS IN CONCRETE WHERE DESIGNATED SHALL BE 25mm DEEP OR THE FIRST LEVEL OF REINFORCING STEEL, WHICHEVER IS LESS.
4. ALL CONCRETE TO REMAIN ALONG FULL DEPTH SAWCUTS SHALL BE ROUGHENED BY REMOVING THE TOP 3 mm OF CONCRETE UNLESS OTHERWISE SPECIFIED.

LIST OF DRAWINGS:

1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGING
3. REMOVALS I
4. REMOVALS II
5. JACKING AND BEARING DETAILS
6. STRUCTURAL STEEL REPAIRS – I
7. STRUCTURAL STEEL REPAIRS – II
8. CONSTRUCTION I
9. CONSTRUCTION – II
10. BARRIER WALL – PERFORMANCE LEVEL 3
11. APPROACH SLABS
12. STANDARD DETAILS I
13. QUANTITIES – STRUCTURE
14. QUANTITIES – STRUCTURE

APPLICABLE STANDARD DRAWINGS:

OPSD-912.43	GUIDERAIL SYSTEM, STEEL BEAM STRUCTURE CONNECTION
OPSD-4010.00	GUIDERAIL AND CHANNEL ANCHORAGE
OPSD-3906.02	BRIDGE DECK WATERPROOFING
OPSD-3906.03	BRIDGE DECK WATERPROOFING DETAILS

LIST OF ABBREVIATIONS:

T/A	TOP OF ASPHALT (FINISHED ELEV.)
WP	WORKING POINT
T/F	TOP OF FOOTING
BRGS.	BEARINGS
CONC.	CONCRETE
TYP.	TYPICAL
U/S	UNDERSIDE
EL.	ELEVATION
EXIST.	EXISTING
ABUT.	ABUTMENT

R.T.A. KRUTZLER
P. C. OJALA

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

BENCH MARK
M.T.O. B.M. 72855
TABLET SET IN SOUTH FACE OF WEST CONC. ABUTMENT
ELEV. 257.680m

DESIGN RTK	CHK PCO	CODE CHBDC 00	LOAD ONT CL-625	DATE APRIL 2005	
DRAWN A.A.	CHK K.M.	SITE 39E-045	STRUCT	SCHEME	DWG P1

DRAWING NAME: F:\8000\Clarkson\DRAFTING\FREDERICKHOUSE\GA-FRED-PREL.dwg CREATED: APRIL 2005 MODIFIED: Jun 02, 2005-11:32am

Appendix F

MTO Special Provision -
”Expanded Polystyrene Embankment”

<u>EXPANDED POLYSTYRENE EMBANKMENT FILL</u> - Item No.

Special Provision

1. SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene backfill and associated works as shown on the contract drawings.

2. REFERENCES

This special provision refers to the following standards, specifications or publications.

National Standards of Canada

CAN/CGSB - 51.20 M87

ASTM

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

OPSS - Ontario Provincial Standard Specification

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

OPSS 1010 Aggregates – Granular A,B,M, and Selected Subgrade Material

OPSS 1605 Expanded Extruded Polystyrene Pavement Insulation

OPSS 1860 Geotextiles

3. SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4. DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene

Molded rigid blocks produced by a process of pre-expansion, aging and forming of a petroleum based raw material.

Rigid Extruded Expanded Polystyrene

Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot

The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer

An Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s)of conformance.

5. QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6. SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturers requirement.

6.3 Construction

The contractor shall submit full details of the following.

a. The method of foundation excavation and preparation.

b. Construction of leveling pad.

c. The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis.

d. The method and limits of placement of polyethylene sheeting.

e. The method of placement of 125 mm reinforced concrete base pad (or equivalent).

f. The method of placement of subbase material.

g. The method of placement of side slope cover.

6.4 Quality Verification Engineer

1. The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

2. The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. ***Upon completion of the Expanded Polystyrene Backfill the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.***

7. MATERIALS

7.1 Leveling Pad

The leveling pad shall consist of mortar sand with gradation and physical requirements as specified in OPSS 1004.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.

2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.

3. Certification of compliance of physical and mechanical properties.

4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.

5. The physical and mechanical properties of the rigid expanded polystyrene including:

    1) Geometry
    2) Nominal Density
    3) Compressive Strength
    4) Flexural Strength
    5) Thermal Resistance
    6) Dimensional Stability
    7) Flammability
    8) Water Absorption

6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.

7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.

8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

TABLE 1 – MATERIAL PROPERTIES

Property	**Unit**	**Requirements**	**Test Procedure**
Geometry	mm	1200 x 600 x 300	
- Linear		with tolerances ± 1%	
- Flatness		10 mm in 3 m ± 0.5%	
- Squareness			
- Thickness		-3, +5	
Compressive Strength	kPa (min)	110	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	$m^2 \cdot {}^oC/W$ (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of <u>1200 mm</u> x <u>600 mm</u> x <u>300 mm</u> .

The maximum deviation from the specified linear dimensions shall be ± 1%. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner to corner dimensions (squareness) shall be 0.5%. The thickness shall be within –3 to +5 mm.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 110 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be 0.7 $m^2.^oC/W$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25mm} = \frac{R_{measured}}{\text{thickness (mm)}} \quad x25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less then 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant, limited, or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations and bending as shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material in accordance with OPSS 1010.

9.2 Leveling Pad

Place, level and compact a layer of mortar sand in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

(1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.

(2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90º to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers except at the vertical construction joints.

A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.

(3) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.

(4) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.

(5) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.

(6) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.

(7) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.

(8) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.

(9) The top surface and side surfaces of the expanded polystyrene shall be covered with .6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the blocking. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

10. EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11. QUALITY ASSURANCE

11.1 Sampling and Testing

11.1.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

11.1.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

11.1.3 Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12. MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position based on theoretical dimensions.

13. PAYMENT

13.1 Basis of Payment

The mortar sand leveling bed shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

Appendix G

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.