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**REPORT ON**

**STAGE II  
TEMPORARY ACCESS ROAD OVER THE  
MISSISSIPPI RIVER WETLANDS  
W.P. 451-90-03/04, HIGHWAY 417  
DISTRICT 42, OTTAWA, ONTARIO**

**Submitted to:**

Ministry of Transportation, Ontario  
Planning and Design  
Eastern Region  
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## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO) to carry out a study for various options available for the construction of a temporary access road and launching pads over the wetlands in order to construct the Highway 417 structures over Mississippi River near Arnprior. The Stage I report was issued at the end of July 1999.

As requested, in this report we considered options for the design of the low height access road and launch pads founded above the soft organic wetland subsoil. The ten options considered for the access road and launching pads in the report are given below:

- 1) Full depth excavation of floodplain deposits;
- 2) Displacement of weak materials by embankment overloading;
- 3) Geogrid / geotextile with earth fill (Granular "B");
- 4) Geogrid / geotextile with rockfill;
- 5) Geogrid / geotextile with light-weight fill (slag);
- 6) Geogrid / geotextile with earth fill (Granular "B") and Polystyrene "sandwich";
- 7) Corrugated pipes with granular (Granular "B") cover;
- 8) Sawdust / Barkchips (hog) fill topped with Granular "B" fill;
- 9) Timber mat corduroy construction; and
- 10) Timber Crib construction.

A table was provided in the report which listed the Advantages and Disadvantages of each option. Also given in the table was information on costs including an estimated Cost for each option for the design length of the access roadway and launch pads. The table also contained a section on Risk, which essentially was mainly about the environmental risk. In a conclusion section our preferred options were discussed from three distinct categories.

For the category which had the roadway and pads founded directly on the underlying clay stratum and therefore avoids serious concerns with stability and settlement, Option 2 which involved displacement of the organic layer by embankment overloading was the preferred option.

For options that involved granular / rock / slag material within a geogrid / geotextile envelope (Options 3, 4 and 5), Option 4 involving rockfill was the preferred option.

For options involving light-weight sandwich construction (Options 6 through 10), Option 6 involving polystyrene as the middle of the sandwich and Option 9 which involved timber mat construction were considered to be the preferred options.

## **2.0 TELECONFERENCE OF AUGUST 20, 1999**

A teleconference was held on August 20, 1999 between Louis Tay, Planning and Design Section and David McAvoy, Environmental Section of MTO, Kingston, Tony Sangiuliano, MTO Foundations Section Downsview and Messrs. Murty Devata and Fin Heffernan, Golder Mississauga.

The teleconference began with a brief summary of the ten options by Messrs. Devata and Heffernan.

David McAvoy did not wish to consider Options 1 or 2 because of the impact on the wetlands. He also pointed out that while the water level in the wetlands was low this summer period, especially during the month of July 1999 when no standing water was present, this has been an exceptional dry year and suggested that we should consider that 0.4 m of water would be present through many periods of the year and as much as 1.0 m of water would be present during the spring run-off period. It was stated that construction may take place during the wet fall period and that the roadway could be still in place during the spring run-off. High water conditions during construction period makes all the light-weight "sandwich" options, (Options 6 through 9) to be less practical.

The remaining options (Options 3, 4 and 5) within a geogrid / geotextile envelope could be constructed in high water. David McAvoy vetoed Option 5 as slag fill was not an acceptable material from an environmental standpoint within the wetlands. However previous analysis on the stability and settlement of this option was carried out and is included for completeness. Tony Sangiuliano agreed that we should consider Options 3 and 4. We concluded that Option 4 was preferred for a number of reasons as follows:

- 1) relatively light-weight which leads to increased stability and slightly lesser magnitude of settlements;
- 2) increased permeability due to lack of fines in the rockfill and maintaining water equilibrium on either side of the roadway and access pads; and
- 3) cost.

Fin Heffernan suggested that we give a second look to Option 9, timber corduroy construction because of its limited environmental impacts. The teleconference was concluded that only Option 4, geogrid / geotextile with rockfill is a preferred option and a further review of Option 9 timber mat corduroy construction should be carried out.

Subsequent review by us found that for a water level of about 0.4 m, this "raft" construction (Option 9) would be difficult. Also the cost is somewhat higher than the rockfill option, (Option 4) and the total weight is not much different. This was subsequently discussed with Mr. Tay (MTO, Kingston) and Mr. Dundas (MTO, Downsview) and concluded that Option 4 only will be discussed in Stage II report.

### **3.0 CONSTRUCTION EQUIPMENT**

The design of the temporary access road and launch pads is dependent on the construction equipment that it must support. Early in the study, we were provided with information (weight, size, etc.) regarding a crane that could be used to erect the segments of the bridge superstructure. On August 27, 1999, we met with Mr. Nicholas Theodore from St. Catharines Structural Department of MTO in the office of Mr. David Dundas to discuss this crane and possible alternatives both crawler mounted and wheel mounted.

We also obtained information on pile driving equipment by contacting Mr. A. Fine, P.Eng. former Vice President, Bermingham Construction Ltd., who considered that Link Belt L.S. 108, 35 tonne crane would be capable of installing the foundation piling for the piers in the wetland area. This crane is not suitable for lifting the bridge beams and a stronger crane will be required for that work.

We also investigated concrete trucks by contacting Mr. Dan Brown of Dufferin Custom Concrete and he stated that the largest trucks that they were using at this time weigh 39,000 kg when fully loaded and that the weight was spread fairly evenly over 4 axles. The distance between the rows of axles in the front and back was 1.8 m and between the front and rear axle sets was about 3 m.

#### **4.0 DISCUSSIONS**

This Stage II report provides the details of stability and settlement analysis carried out for the proposed temporary access road and launch pad embankments over the wetlands at the Highway 417 crossing over the Mississippi River. The embankment is required in order to provide access to construct the piers and erect the bridge deck. The potential construction traffic utilizing the access road and launch pads includes cranes, pile driving rigs, concrete trucks, dump trucks, flat bed trucks and dozers. The analysis is presented and summarized, some general guidance on construction traffic is provided and several specific crane configurations are discussed.

##### **4.1 Stratigraphy and Parameters**

The Thurber Environmental Ltd. (Thurber) report entitled "Mississippi EBL & WBL River Bridges, Highway 417, District 42, Ottawa, Geocres No. 31F-117" to MTO and dated September 18, 1995, describes the geotechnical investigation carried out for the bridges. In particular, the geotechnical data is summarized for the central floodplain area on Figure 5 of Thurber's report. Boreholes 95-10, 95-11, 95-12 and 95-13 were put down for the bridge carrying the eastbound lanes and Boreholes 95-3, 95-4, 95-5 and 95-6 were put down for the bridge carrying the westbound lanes.

Golder Associates carried out two shallow boreholes (99-1 and 99-2) on July 20, 1999, in the floodplain area. The boreholes were carried out in order to obtain an in-situ vane shear strength profile and obtain floodplain deposit samples for laboratory testing. Borehole 99-1 was carried out just west of Thurber Borehole 95-12 and Borehole 99-2 just west of Thurber Borehole 95-5.

The Record of Boreholes for 99-1 and 99-2 are attached and describe the encountered stratigraphy and laboratory testing. Two consolidation tests were also carried out on selected samples, see attached summary sheets and Figures 1 and 2. The sample from Borehole 99-1 was selected to represent the organic silt material (0.85 m depth) and the sample from Borehole 99-2 was selected to represent the silty clay / clayey silt material (1.75 m depth).

It should be noted that the stratigraphy of the surficial deposits described in Boreholes 99-1 and 99-2 differs from that described by Thurber in Boreholes 95-12 and 95-5, respectively. For example in Borehole 95-5, Thurber describe the upper 1.5 m of material as an amorphous peat while in Borehole 99-2 only 0.7 m of peat was present underlain by a deposit of organic silt, interlayered with peat. The peat is described as fibrous in Boreholes 99-1 and 99-2 - however it is possible that a surface layer of more amorphous peat was present, because sampling commenced at about 0.3 m below ground surface.

The following simplified stratigraphy and design parameters were utilised in the analyses. The stratigraphy to a depth of 2.2 m was based on Borehole 99-1 and below 2.2 m on Borehole 95-12. The ground surface is at about Elevation 83.1 m and the ground water level was assumed coincident with the ground surface.

Depth (m)	Stratigraphy	Design Parameters		
		Unit Weight (kN/m <sup>3</sup> )	Strength	Compressibility (i)
0 to 0.7	Peat (ii)	12	$\phi' = 30$ degrees (compressed)	$c_c = 2.0$ $e_o = 4.0$
0.7 to 2.0	Organic Silt	13	$S_u = 15$ kPa	In loading range – $c_c = 0.1$ $e_o = 2.1$ from oedometer
2.0 to 2.2	Clayey Silt (iii)	17	$S_u = 20$ kPa	$c_c = 0.25$ $c_r = 0.02$ $e_o = 1.0$ OCR = 15
2.2 to 4.2	Sandy Clay (iii) (clayey silt with sand)	19	$S_u = 25$ kPa	$c_c = 0.2$ $c_r = 0.02$ $e_o = 0.7$ OCR = 7
4.2 to 39	Silty Clay (iv)	17	$S_u$ varies with depth 25 kPa to 80 kPa	$c_c = 0.6$ $c_r = 0.05$ $e_o = 1.2$ OCR varies with depth from 7 to about 1.5
<39	Bedrock			

- NOTE:**
- (i) Compressibility Parameters
    - $c_c$  = compression index
    - $c_r$  = recompression index
    - $e_o$  = initial void ratio
    - OCR = overconsolidation ratio
  - (ii) The parameters for the peat are based on data from The Muskeg Engineering Handbook. An effective stress friction angle ( $\phi'$ ) is used in the stability calculations to represent the compressed state of the fibrous peat.
  - (iii) Compressibility data was based on index property correlation's and OCR based on pre-consolidation stress estimate from undrained strength.
  - (iv) See Thurber report for vane shear strength profiles and compressibility data.

## 4.2 Settlement Analysis

The following options were considered for embankment construction;

- A) Granular "B", unit weight = 21 kN/m<sup>3</sup>,  $\phi' = 35$  degrees
- B) Rockfill, unit weight = 17 kN/m<sup>3</sup>,  $\phi' = 38$  degrees
- C) Light Weight Fill, unit weight = 14 kN/m<sup>3</sup>,  $\phi' = 32$  degrees

The embankment was assumed to be constructed to about 1.5 m above the floodplain, or to about Elevation 84.5 m. The settlements were calculated using the commercially available program UNISETTLE (v2.4) produced by Unisoft Ltd. The maximum calculated settlements were as follows;

- Option A) 570 mm
- Option B) 530 mm
- Option C) 490 mm

About 80 percent of the settlement would occur rapidly due to the compression of the peat deposit. The remaining settlement would develop over a period of months to years due to consolidation of the clay deposits. The difference in settlement due to fill option is minor and should not be a governing factor on fill selection. The settlement values were adjusted to take into account the additional depth of fill required after initial settlement occurred – i.e. an additional 0.5 m of fill is required which in turn causes additional settlement.

The depth of peat makes a significant difference on the likely settlement of the embankment and the volume of material required to construct the embankment. The compression of the peat amounts to about 60 percent of its thickness and if the Thurber report is correct and there is up to about 2.5 m of peat present then the maximum settlement goes up to around 1.5 m to 1.8 m and the embankment requires more material to construct. Estimating the volume required for the embankment will be problematic.

Constructing the embankment will require care. Placement of fill will create a displacement wave of peat. The displacement may create the need for even larger volumes of fill than the settlement estimate indicates. The creation of the mud wave and peat displacement are functions of construction practices, however careful placement and use of a basal geogrid / geotextile will help to reduce the size of the wave but are unlikely to eliminate it.

#### **4.3 Stability Analysis**

Slope stability analyses were carried out on an embankment section using the commercially available program SLOPE/W (version 4.0). The program uses the general equilibrium method of analysis to calculate the factor of safety of numerous potential failure surfaces (both circular and composite) and determine minimum factors of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure.

The embankment was assumed to be 2 m in height, after the underlying peat has compressed, see Figure 3. The embankment is statically stable with a factor of safety greater than 1.5 for any fill option, using 2H to 1V side slopes. However, there is a significant difference in factor of safety

depending on the fill type selected, in general the rockfill option has an 8 percent higher factor of safety than the Granular "B" embankment and the light weight fill has a 6 percent higher factor of safety than the Granular "B" embankment. The factor of safety difference is a function of unit weight and friction angle. Note that the rockfill embankment would require a compacted Granular "B" surface.

The effect of crane loading on the embankment may be divided into two cases; travelling and lifting. For analysis purposes the following dimensions of a tracked crane were assumed, tread width of 1.1 m and an overall width of 3.5 m. The overall width is indicative of a 100 tonne crane but is a conservative assumption as larger cranes typically have similar track widths but overall widths of about 5 m (165 tonne crane).

The access road should be wide enough to safely maneuver a heavy vehicle on without the edge of the embankment failing and to provide a margin of safety against driver error. The crane tracks were modeled as pressure loads and varied between 50 kPa and 100 kPa. These pressures are typical travelling ground pressures for tracked cranes. We recommend that a minimum setback of 1.5 m be adopted to provide maneuvering space. However, the proposed embankment configuration should be checked by an equipment specialist to ensure that there is enough space to safely maneuver the proposed construction vehicles.

The light weight fill embankment was not specifically considered in the following stability analyses although its behaviour may be considered as being similar to the rockfill embankment.

The following table summarizes the results of stability analyses for the travelling case;

<i>Ground Pressure (kPa)</i>	<i>Side Slope</i>	<i>Granular "B" Fill Embankment</i>		<i>Rockfill Embankment</i>	
		<i>Setback</i>	<i>Factor of Safety</i>	<i>Setback</i>	<i>Factor of Safety</i>
50	2H:1V	2	>1.5	1.5	>1.5
75	2H:1V	2	>1.3	1.5	>1.5
100	2H:1V	4	>1.3	2	>1.3
50	4H:1V	1.5	>1.5	1.5	>1.5
75	4H:1V	1.5	>1.5	1.5	>1.5
100	4H:1V	2	>1.3	1.5	>1.3

A factor of safety greater than 1.3 is considered acceptable. The above table therefore indicates a range of acceptable fill and geometry configurations. (Note; the increase in setback requirements and decrease in factor of safety when the load is 100 kPa. The 100 kPa ground pressure level may be assumed as the cut-off for tracked vehicle operations).

The effect of wheeled vehicles using the access road is difficult to assess using a conventional two dimensional slope stability program. The contact pressures for wheeled vehicles are higher than for tracked vehicles (e.g. about 170 kPa for a 150 tonne crane) but there is a significant load spreading effect and using a 2-D slope stability program would be unacceptably conservative. However, bearing capacity theory may be used to assess the performance of the subgrade. The ultimate bearing capacity of the organic silt beneath the embankment is only about 120 kPa. This implies that there will be inadequate factors of safety for many of the wheeled vehicles using the access ramp, even assuming some load spreading before the subgrade is reached. The embankment, therefore, will require geogrid reinforcement to spread the load at the subgrade level and it is also likely that additional geogrid layers will be required.

We understand that the crane has to be capable of lifting 35 tonnes. The 35 tonne lift would likely increase the average ground pressure of a tracked crane by about 10 kPa to 25 kPa depending on the particular crane and lift configuration. The peak ground pressure during a lift would exceed these values and the actual value would be dependent on lift angle and specific crane geometry. However, in order to reduce the pressure loading during the lift and help to even out the pressure distribution the crane can be located on a grillage or some form of pad to spread the load. Assuming the grillage is at least as large as the cranes overall footprint then the following results were obtained;

<i>Maximum Ground Pressure (kPa)</i>	<i>Side Slope</i>	<i>Granular "B" Fill Embankment</i>		<i>Rockfill Embankment</i>	
		<i>Setback</i>	<i>Factor of Safety</i>	<i>Setback</i>	<i>Factor of Safety</i>
50	2H:1V	3	>1.3	2	>1.3
75	2H:1V	6	>1.3	4.5	>1.3
50	4H:1V	2	>1.5	2	>1.5
75	4H:1V	3.5	>1.3	2	>1.3

Setback distance is defined as the distance from the loaded area to the slope crest. The grillage loading should not exceed 75 kPa. The wheeled cranes have outriggers and the bearing pressure during lifting can be controlled by using grillages placed under the outriggers. These outrigger pads / grillages may be sized to restrict the ground pressure to under 75 kPa and then the above table may be used to determine the setback.

In general, based on the analysis results, we recommend that a rockfill embankment is used in preference to Granular "B". A rockfill embankment with 2H to 1V side slopes, a 2 m setback and a lifting restriction on the ground pressure of 50 kPa would be sufficient for most crane configurations.

In addition, when a crane is in position on a grillage (say 6 m x 7 m) some compression of the organic silt will occur. The settlement is likely to occur rapidly and could be up to 25 mm to 50 mm. When lifting the maximum load additional settlement of 10 mm to 25 mm could occur during the lift. The contractor should be made aware of this problem and be prepared to try to use cranes that can lift the required loads as vertically as possible and control the counterweight to try to maintain a uniform ground pressure under the grillage and prevent differential settlement occurring.

Note; we understand that a pile driving rig should have lower or similar ground pressures to the cranes and heavily loaded concrete and dump trucks should have similar or lower ground pressures than the wheeled cranes.

The effect of loading adjacent to the river bank was checked and the above setback values are still applicable.

#### **4.4 Crane Examples**

The above tables allow for combinations of fill type, slope angle and ground pressure. The following specific examples are provided for illustration purposes only, the contractors proposed lifting equipment should be checked by the geotechnical engineer to ensure compliance with the ground pressure restrictions. The contractor may modify crane configurations by removal of optional pieces of equipment or partial disassembly (removal of counterweight).

**165 Tonne American Crawler Crane (Model 9299)**

Dimensions:	Width	(crawlers retracted)	=	4.95 m
	Width	(crawlers extended)	=	5.71 m
	Length		=	7.5 m
Travelling Ground Pressure				= 93 kPa
Ground Pressure on grillage (5.7 m x 7.5 m)				= 33 kPa
Average ground pressure on grillage when lifting				= 41 kPa

A rockfill embankment with 4H to 1V sideslopes and a 2 m setback distance would be acceptable for this crane configuration. The access road would therefore have a width of  $5+2+2 = 9$  m.

**100 Tonne Lorain Crawler Crane – 1.1 m tread width**

Dimensions:	Width (crawlers retracted)	=	3.5 m
	Length	=	6.1 m
Ground Pressure			
		=	46 kPa
Ground Pressure on grillage (3.5 m x 6.1 m)			
		=	35 kPa
Average ground pressure on grillage when lifting			
		=	51 kPa

We understand that this crane may have to maneuver when lifting. It would generally not be acceptable for the crane to move on the grillage pad when lifting because the pressure redistribution would result in some immediate compression of the organic silt and a potential for tipping the crane.

A rockfill embankment with 2H to 1V sideslopes and a 1.5 m setback distance for the access road and 2 m setback for the lifting area (launching pad) would be acceptable for this crane configuration. The access road would therefore have a width of  $3.5+1.5+1.5 = 6.5$  m.

**150 Tonne truck crane - 100 ft boom (P&H 9150)**

Dimensions:	Width	=	3.3 m
	Width with outriggers extended	=	6.6 m
	Length	=	10.9 m

Assume four outriggers on 2.5 m x 2 m grillage pads		
Ground Pressure on grillage (full weight of rear tandem)	=	34 kPa
Average ground pressure on grillage when lifting	=	50 kPa

The length of the crane and the fact it is on wheels will make it difficult to maneuver on the access road and launching (lifting) pads. Additional space may be required over and above the setback criteria.

A rockfill embankment with 2H to 1V sideslopes and a 1.5 m setback distance for the access road and 2 m setback for the lifting area would be acceptable for this crane configuration. The access road would therefore have a width of  $3.3+1.5+1.5 = 6.3$  m.

### Summary of Recommendations / Comments

- A rockfill embankment with 2H to 1V side slopes, a 2 m setback and a lifting restriction on the ground pressure of 50 kPa would be sufficient for most crane configurations.
- Geogrid embankment reinforcement is required to prevent subgrade bearing problems for heavily loaded truck and wheeled cranes. Geogrid reinforcement may also allow sideslopes steeper than 2H:1V.
- A grillage is required during lifting – either the same size as the crane footprint or specially sized for outriggers to keep the ground pressure within allowable levels.
- Settlement of the cranes will occur during lifting. The crane selected should be capable of lifting as vertically as possible in order to prevent differential settlement occurring across the grillage during lifting. (If this movement is unacceptable consideration should be given to constructing a pile supported grillage platform for the crane).
- The contractor should check his proposed equipment to ensure that allowable ground pressures are not exceeded.
- An embankment layout and design (in conjunction with Terrafix) can be made based on this report and assumptions as to contractors equipment, but it is quite likely that the contractor will propose different equipment and the embankment design require modification after contract award.
- The space required for vehicle maneuvering on the embankment should be checked, by an equipment specialist, as a separate issue.
- Calculation of the volume of fill required for the embankment will be problematic given the likely variation in peat thickness and therefore settlement. The embankment should be constructed to some minimum level above the floodplain, 1.5 m as assumed in our calculations but 1 m (a freeboard of 0.6 m) may also be acceptable. The minimum freeboard should be established based on hydraulic considerations.

## 5.0 REVIEW OF TERRAFIX DESIGN

The stratigraphy and soil parameters including Thurber's subsurface information along with our recent borehole data were supplied to Terrafix Ltd. a specialty geogrid / geotextile supplier to develop necessary design details for both the temporary access road and launching pad embankments over the wetlands with the geogrid / geotextile rockfill concept (Option 4). The supplier would develop quantities and specifications for the work. This includes installation and removal of the temporary access road and launch pad embankments. To provide the necessary liaison with the specialty supplier we met with Mr. Phil Perzia, P.Eng. of Terrafix Ltd. initially on August 27, 1999 and again on September 10, 1999 to review their preliminary design details. We also discussed with Mr. John Kerr, P.Eng. of Tensar Earth Technologies Inc. (Tensar) National Sales Manager, Canada Consultant to Terrafix Ltd. by telephone the development of the design requirements for this temporary access road and launching pad geogrid / geotextile rockfill embankment.

Tensar prepared a set of five drawings and these were reviewed by Golder and comments were discussed at a meeting held in our office with Phil Perzia of Terrafix Ltd. on September 17, 1999. Our comments were incorporated on the construction drawings prepared by Tensar which are included in the Appendix of this Stage II report. The details of these drawings are as follows:

<i>Sheet</i>	<i>Description</i>
1	Title Sheet
2	Construction Notes
3	Typical Cross-sections
4	Typical Cross-sections
5	Geogrid Layout

**Sheet 2:** Provides notes for placement of Tensar geogrids / geotextiles and rockfill for an MTO approved "Prism" reinforced system. The rockfill for the construction of access road and pads shall be 300 mm minus. The gradation of a bedding sand fill is shown on the drawings. The notes provide requirements for installation and removal of geogrid / geotextile rockfill embankments for access road and launching (lifting) pads including special provisions.

**Sheet 3:** Illustrates cross-sections of the geogrid / geotextile with rock embankments for both temporary access road and launching pads. The sections provides the types of geogrid and geotextile and their respective locations including the rockfill and sand fill bedding. It should be noted that the biaxial geogrid extends 2.0 m beyond the toe of the fill for any possible spillage of rockfill into the natural ground (wetlands). For a 165 tonne American Crawler Crane the width of the access road is 10 m whereas for the launch pads its top width is 12.0 m for a 1.5 m rockfill embankment. A water level of 0.4 m above the wetland ground surface was assumed.

**Sheet 4:** Provide similar details for the launching pads at the end of the launching (lifting or crane) pads. At these locations additional geogrids are required as shown on the section including sand fill bedding.

**Sheet 5:** Gives the details of the installation of geotextile and geogrid placement step by step (six steps) for the construction of the rockfill embankments in the area of access road and launching pads. The step by step placement details gives the details of the roll direction of the geogrid at various elevations.

The above drawings were prepared based on the access road layout provided by Eastern Region, MTO office, Kingston. If there are any changes pertaining to pier and abutment locations, these drawings will have to be modified to the revised locations of the piers and abutments located in the wetlands.

To provide free flow of the water on either side of the embankment, rockfill was chosen due to its pervious nature. However pipes could be installed to increase the flow. However, since the cover is so small, a very strong pipe is required to withstand the heavy equipment loads. Also it will be difficult to establish the precise invert elevation of the pipe due to anticipated uneven settlements. For these reasons pipes were not considered, however, if it is necessary from an environmental point of view, their use will be reconsidered with a method of placement prior to the installation of upper geogrids in the embankment.

Tensar developed their design for a 1.5 m embankment height over the wetlands for the access road and launching pads to the widths specified on their drawings. The geogrid / geotextile rockfills will settle immediately due to the compression of the peat and further settlements in the organic silt layer due to the imposed loads. The peat and organic silt layer thickness were not fully established except in Boreholes 99-1 and 99-2. Since the subsurface conditions are not uniform, nor fully established, some variations in the magnitude of the settlements can be anticipated. However, their main concern was to establish a free board of 0.6 m for the embankment above the normal high water of 0.4 m which was used in their design. Any uneven settlements can be filled with rockfill and graded with Granular "B" cover to maintain the free board of 0.6 m. During the flooding period this road will be under water for a short period of time and the road surface may be subject to erosion.

The quantities for rockfill may vary since the precise depths of peat and organic silts are not fully established. Based on the Tensar design of 1.5 m height embankment for a geogrid / geotextile rockfill access road and launching pads having a crest width of 10 m and 12 m respectively, a total rockfill volume of about 10,500 c.m will be required. An additional 10% by volume should be a reasonable quantity to compensate for the uneven settlements of the embankment due to uneven depth of peat and organic silt deposits in the upper portion of the wetlands.

We also discussed on September 17, 1999 the concept of removal of the temporary access road and launching pad embankments. These were also added in the construction notes on the drawing. The rockfill and upper layers of geogrid are to be removed in one lift, taking care not to puncture the bottom layers of geogrid. The lower layers of geogrid are peeled up and removed as the main excavation progress. An indentation generally about 0.5 m deep will be left in the

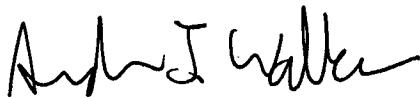
flood plain below the water surface. With time it is expected that this indentation will fill with organic matter.

We also made an estimate of the construction time in consultation with Tensar based on the approximate quantities of geotextile/geogrid placement including the rockfill. Some 4 to 5 weeks are needed for the access road and launching pad construction. The removal could be accomplished in 2 to 3 weeks.

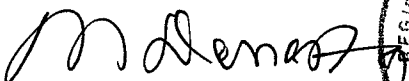
We trust that the details provided in this report are adequate for your needs. Should you require any clarification or further details, please call us.

Yours truly,

**GOLDER ASSOCIATES LTD.**



Andrew J. Walker, P.Eng.  
Associate

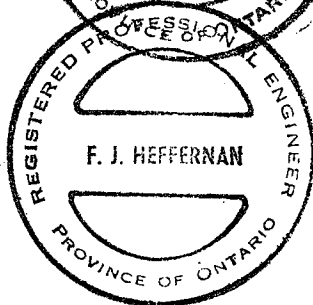
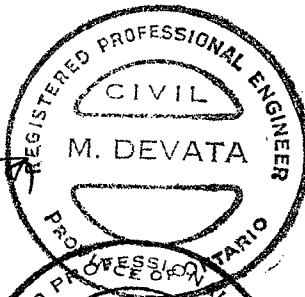


Murty S. Devata, P.Eng.  
Consultant



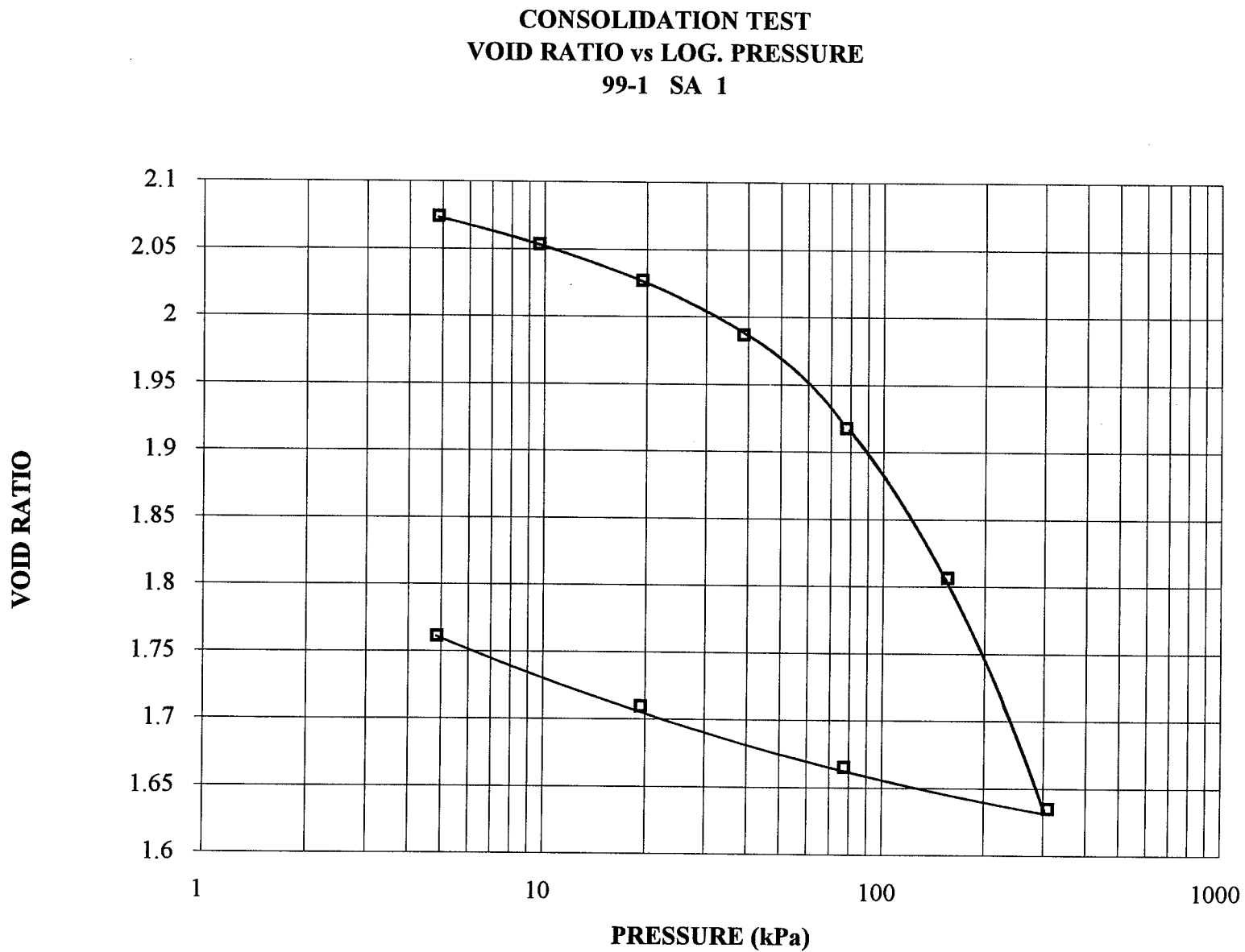
Fintan J. Heffernan, P.Eng.  
Designated MTO Contact

AJW/MSD/FJH/clg  
WORD S/FINALDAT/1100/991-1155/911551R1



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE 1



## OEDOMETER CONSOLIDATION SUMMARY

### SAMPLE IDENTIFICATION

Project Number	991-1155	Sample Number	1
Location	99-1	Sample Depth, m	0.85

### TEST CONDITIONS

Test Type	Quick	Load Duration, hr	(0.15 -0.27)
Oedometer Number	7		
Date Started	99-07-26		
Date Completed	99-07-26		

### SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m <sup>3</sup>	13.51
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	6.83
Area, cm <sup>2</sup>	31.55	Specific Gravity, assumed	2.18
Volume, cm <sup>3</sup>	60.10	Solids Height, cm	0.608
Water Content, %	97.75	Volume of Solids, cm <sup>3</sup>	19.18
Wet Mass, g	82.80	Volume of Voids, cm <sup>3</sup>	40.92
Dry Mass, g	41.87	Degree of Saturation, %	100.0

### TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.905	2.134	1.905				
4.85	1.869	2.074	1.887	5	1.51E-01	3.93E-03	5.81E-05
9.65	1.856	2.053	1.863	26	2.83E-02	1.36E-03	3.77E-06
19.39	1.840	2.026	1.848	15	4.83E-02	8.84E-04	4.18E-06
38.77	1.816	1.986	1.828	13	5.45E-02	6.61E-04	3.53E-06
77.74	1.774	1.917	1.795	17	4.02E-02	5.64E-04	2.22E-06
155.09	1.706	1.806	1.740	25	2.57E-02	4.59E-04	1.16E-06
310.17	1.602	1.635	1.654	17	3.41E-02	3.53E-04	1.18E-06
77.54	1.620	1.665	1.611				
19.39	1.647	1.709	1.634				
4.85	1.678	1.761	1.663				

Notes:

k calculated using cv based on t<sub>90</sub> values.

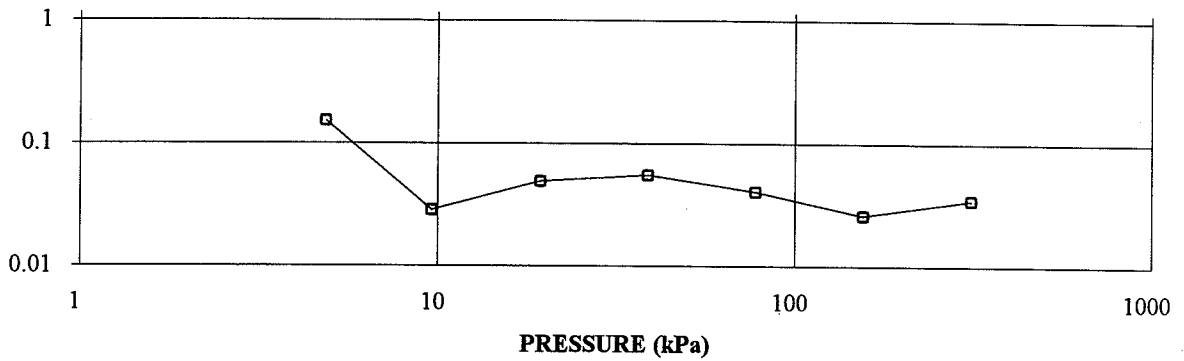
### SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.68	Unit Weight, kN/m <sup>3</sup>	14.74
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	7.76
Area, cm <sup>2</sup>	31.55	Specific Gravity, assumed	2.18
Volume, cm <sup>3</sup>	52.94	Solids Height, cm	0.608
Water Content, %	90.06	Volume of Solids, cm <sup>3</sup>	19.18
Wet Mass, g	79.58	Volume of Voids, cm <sup>3</sup>	33.76
Dry Mass, g	41.87		

# OEDOMETER CONSOLIDATION SUMMARY

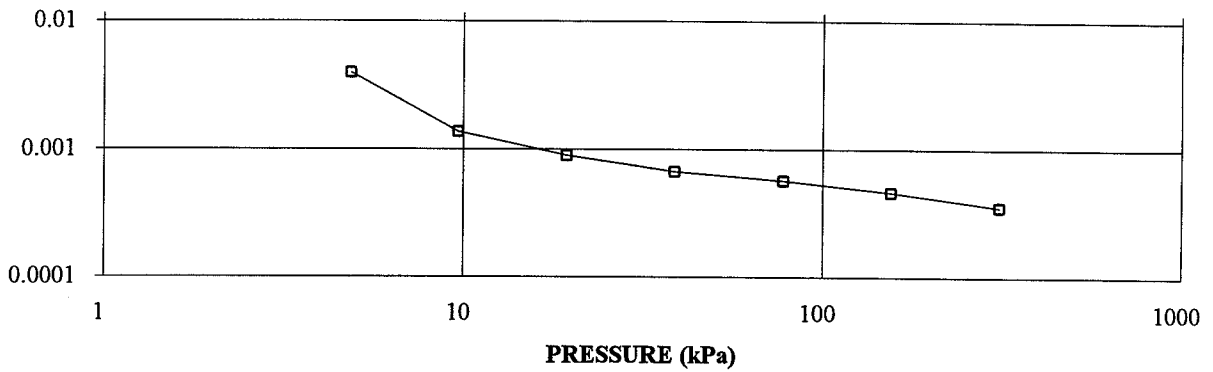
COEFFICIENT OF CONSOLIDATION,  $\text{cm}^2/\text{s}$

CONSOLIDATION TEST  
LOG.  $c_v$   $\text{cm}^2/\text{s}$  vs LOG. PRESSURE (kPa)  
99-1 SA 1



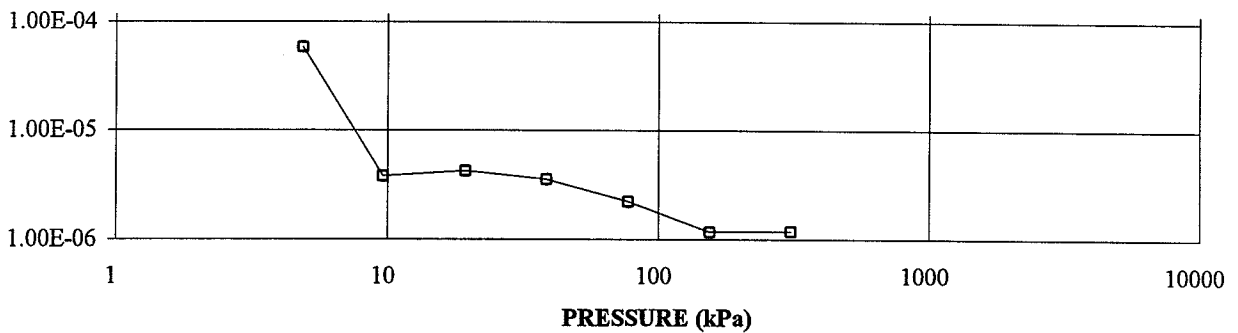
VOLUME  
COMPRESSIBILITY,  
 $\text{m}^2/\text{kN}$

CONSOLIDATION TEST  
LOG.  $m_v$ ,  $\text{m}^2/\text{kN}$  vs LOG. PRESSURE (kPa)  
99-1 SA 1



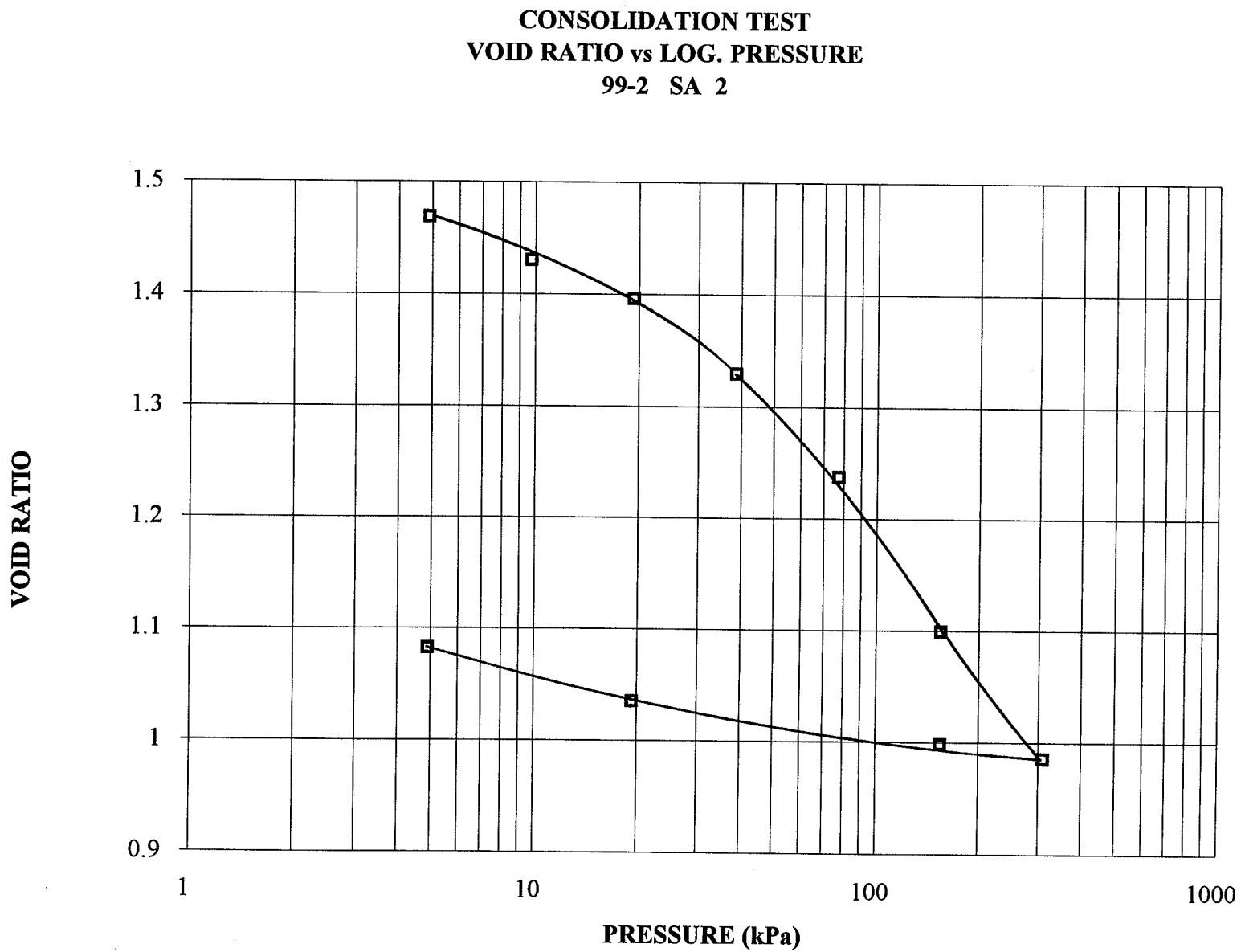
HYDRAULIC  
CONDUCTIVITY,  $\text{cm/s}$

CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs LOG. PRESSURE  
99-1 SA 1



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE 2



## OEDOMETER CONSOLIDATION SUMMARY

### SAMPLE IDENTIFICATION

Project Number	991-1155	Sample Number	2
Location	99-2	Sample Depth, m	1.22-1.83

### TEST CONDITIONS

Test Type	Quick/Standard	Load Duration, hr	(0.15 -18.0)
Oedometer Number	6		
Date Started	99-07-26		
Date Completed	99-07-28		

### SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m <sup>3</sup>	16.21
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	10.31
Area, cm <sup>2</sup>	31.62	Specific Gravity, assumed	2.64
Volume, cm <sup>3</sup>	60.23	Solids Height, cm	0.759
Water Content, %	57.15	Volume of Solids, cm <sup>3</sup>	23.99
Wet Mass, g	99.54	Volume of Voids, cm <sup>3</sup>	36.24
Dry Mass, g	63.34	Degree of Saturation, %	99.9

### TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.905	1.511	1.905				
4.89	1.873	1.469	1.889	13	5.82E-02	3.41E-03	1.95E-05
9.72	1.844	1.430	1.859	1654	4.43E-04	3.19E-03	1.38E-07
19.34	1.818	1.396	1.831	1711	4.15E-04	1.42E-03	5.77E-08
38.69	1.767	1.329	1.793	3474	1.96E-04	1.38E-03	2.64E-08
77.37	1.698	1.237	1.732	3553	1.79E-04	9.46E-04	1.66E-08
154.74	1.593	1.099	1.645	2930	1.96E-04	7.10E-04	1.36E-08
309.49	1.507	0.985	1.550	2473	2.06E-04	2.93E-04	5.91E-09
154.74	1.516	0.998	1.511				
19.34	1.544	1.035	1.530				
4.89	1.580	1.082	1.562				

Notes:

k calculated using cv based on t<sub>90</sub> values.

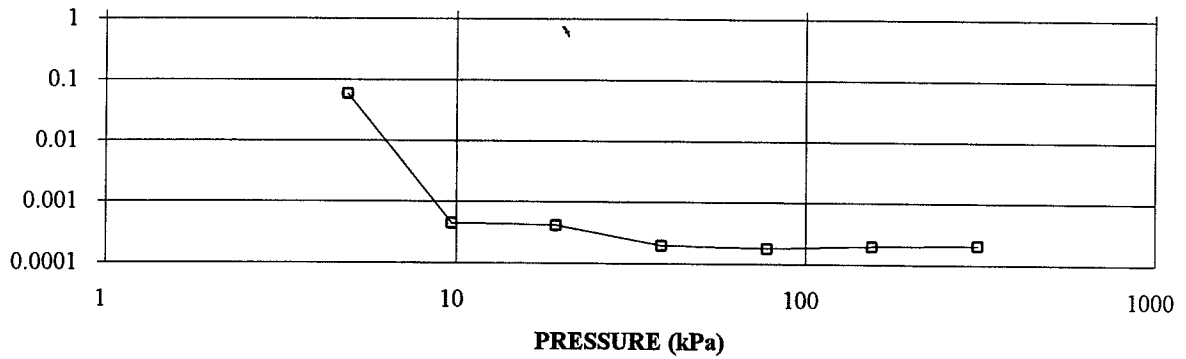
### SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.58	Unit Weight, kN/m <sup>3</sup>	17.91
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	17.78
Area, cm <sup>2</sup>	31.62	Specific Gravity, assumed	2.64
Volume, cm <sup>3</sup>	49.96	Solids Height, cm	0.759
Water Content, %	44.05	Volume of Solids, cm <sup>3</sup>	23.99
Wet Mass, g	91.24	Volume of Voids, cm <sup>3</sup>	25.97
Dry Mass, g	63.34		

# OEDOMETER CONSOLIDATION SUMMARY

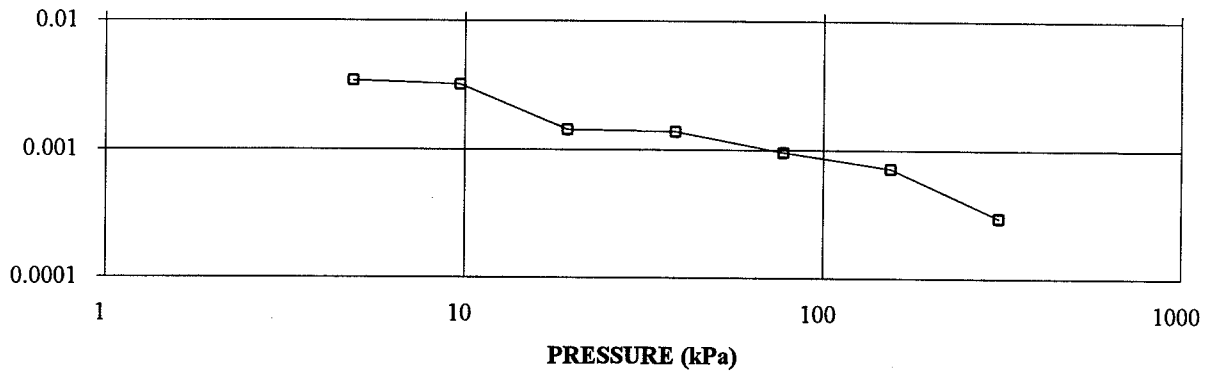
COEFFICIENT OF CONSOLIDATION,  $\text{cm}^2/\text{s}$

CONSOLIDATION TEST  
LOG.  $c_v$   $\text{cm}^2/\text{s}$  vs LOG. PRESSURE (kPa)  
99-2 SA 2



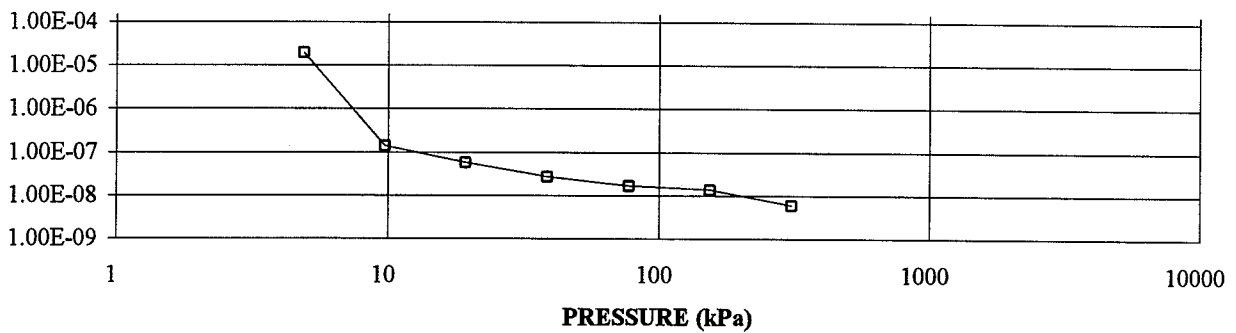
VOLUME  
COMPRESSIBILITY,  
 $\text{m}^2/\text{kN}$

CONSOLIDATION TEST  
LOG.  $m_v$ ,  $\text{m}^2/\text{kN}$  vs LOG. PRESSURE (kPa)  
99-2 SA 2



HYDRAULIC  
CONDUCTIVITY,  $\text{cm/s}$

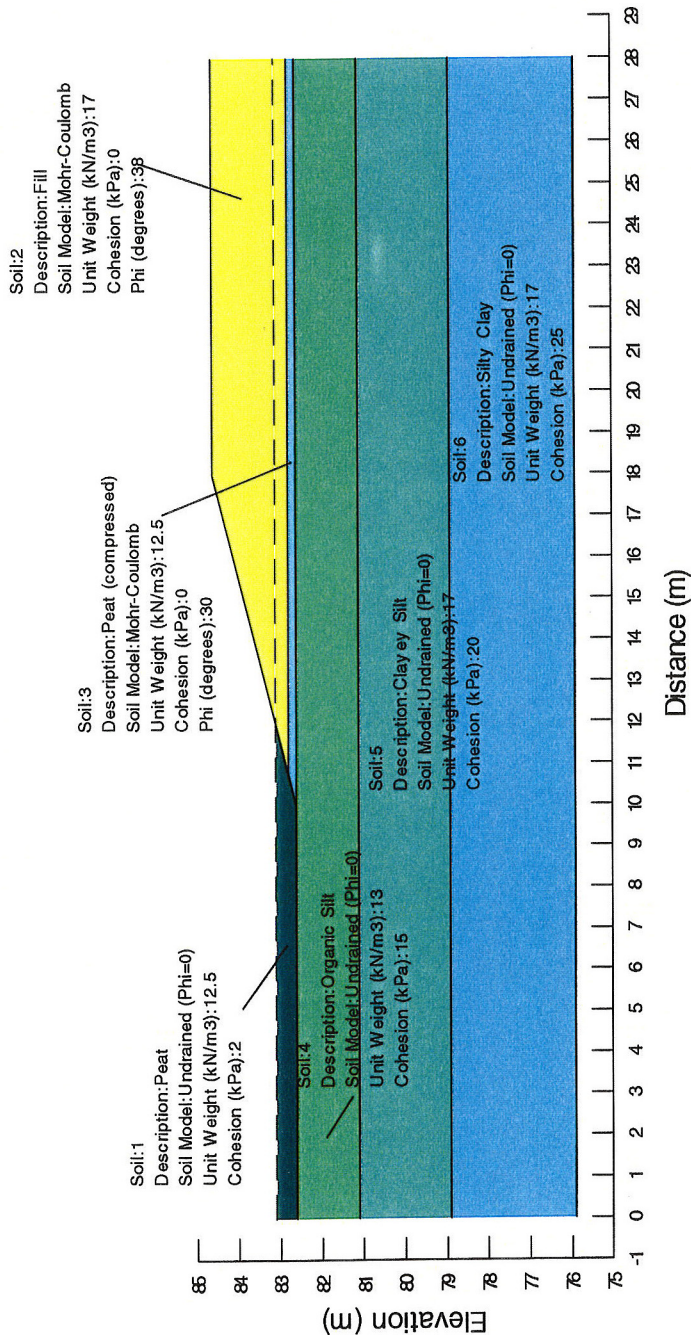
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs LOG. PRESSURE  
99-2 SA 2



# Soil Property Summary 417-Mississippi Flood Plain

FIGURE 3

Description: 991-1155  
Comments: 417 - Construction Embankment  
File Name: missi-6.slp  
Analysis Method: Morgenstern-Price  
Direction of Slip Movement: Right to Left



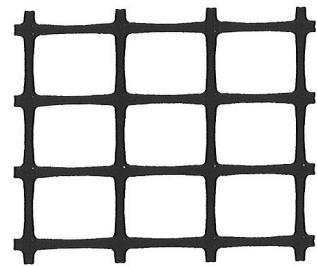
Date ..... 21 September 1999  
Project ..... 991-1155

Golder Associates

Drawn ..... AJW  
Chkd. .... AJW

## **APPENDIX A**

### **TENSAR CONSTRUCTION DRAWINGS**



Tensar  
Earth Technologies, Inc.

CONSTRUCTION DRAWINGS  
Prepared For

# MTO HIGHWAY 417 MISSISSIPPI RIVER TEMPORARY ACCESS ROAD

ARNPRIOR,

ONTARIO

## INDEX

### SHEET

- 1.
- 2.
- 3.
- 4.
- 5.

### DESCRIPTION

Title Sheet  
Construction Notes  
Typical Cross-Sections  
Typical Cross-Sections  
Geogrid Layout

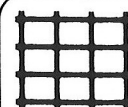


E9950101.DWG

THIS DESIGN IS BASED UPON SPECIFIC PROPERTIES OF TENSAR PRODUCTS (GEOGRIDS, DRAINAGE COMPOSITES AND EROSION MEDIA), WHICH ARE PROPRIETARY TO THE TENSAR CORPORATION 1210 CITIZENS PARKWAY, MORROW GA 30280. ANY SUBSTITUTION OF THE SPECIFIED PRODUCTS WILL INVALIDATE THIS DESIGN. THIS DRAWING IS BEING FURNISHED FOR USE ON THIS SPECIFIC PROJECT ONLY. ANY PARTY ACCEPTING THIS DOCUMENT DOES SO IN CONFIDENCE AND AGREES THAT IT SHALL NOT BE DUPLICATED WHOLE OR IN PART, NOR DISCLOSED TO OTHERS, WITHOUT THE CONSENT OF TENSAR EARTH TECHNOLOGIES, INC.

THIS DRAWING, DESIGN NOTES AND ASSOCIATED CALCULATIONS HAVE BEEN PREPARED BY TENSAR EARTH TECHNOLOGIES, INC. FOR PRELIMINARY DESIGN PURPOSES AND SHALL NOT BE USED FOR FINAL DESIGN OR CONSTRUCTION.

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TENSAR

EARTH TECHNOLOGIES, INC.

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Lakeside Center Suite 450  
Atlanta, Georgia 30328  
(404) 250-1290

**terrafix**  
geosynthetics inc.

425 Attwell Drive  
Rexdale, Ontario  
M9W 5C4  
Tel:(416) 674-0363

**PRISM**<sup>TM</sup>  
FOUNDATION SUPPORT SYSTEM

[ SEAL ]

## REVISIONS \ ISSUE

0	9/3/99	ISSUED FOR REVIEW
1	9/15/99	ISSUED FOR REVIEW

Project Number  
E99501  
Date Drawn  
9/3/99  
Scale  
As Shown  
Designed by  
KL  
Drawn by  
BBC  
Checked by

MTO HIGHWAY 417

ARNPRIOR,

ONTARIO

TITLE  
SHEET

Sheet Number

1 of 5

CONSTRUCTION NOTES FOR PLACEMENT OF TENSAR® GEOGRIDS AND BACKFILL SOILS FOR PRISM® REINFORCED SYSTEM

1.0 MATERIALS

1.1 BACKFILL SOILS

1.1.1 REINFORCED BACKFILL MATERIALS SHALL BE APPROVED BY THE OWNER OR OWNER'S REPRESENTATIVE AND SHALL MEET THE STRENGTH REQUIREMENTS AS DEFINED IN SECTION 6.0. THE ROCK BACKFILL SHALL BE 300 mm MINUS ROCK. THE SAND BACKFILL MATERIAL SHALL MEET THE FOLLOWING GRADATION:

SIEVE SIZE	PERCENT PASSING
101.6 mm	100 - 75 %
No. 4	100 - 20%
No. 40	0 - 60% max.
No. 200	0 - 15% max.

THE PORTION OF THE SAND BACKFILL MATERIAL PASSING THE No. 40 SIEVE SHALL HAVE A LIQUID LIMIT LESS THAN 40 AND A PLASTICITY INDEX LESS THAN 20. SAND BACKFILL MATERIAL SHALL BE CLASSIFIED PER THE UNIFIED SOIL CLASSIFICATION SYSTEM AS LOW PLASTICITY OR NON-PLASTIC SOILS.

1.1.2 FURTHERMORE, REINFORCED BACKFILL MATERIALS SHALL BE FREE OF EXCESS MOISTURE, ROOTS, MUCK, SOD, SNOW, FROZEN LUMPS, ORGANIC MATTER OR OTHER DELETERIOUS MATERIALS. ALL ROCK PARTICLES SHALL BE LESS THAN 300 mm IN THE LONGEST DIMENSION. REINFORCED BACKFILL MATERIALS WHICH DO NOT MEET THIS CRITERIA SHALL BE CONSIDERED UNSUITABLE AND SHALL BE REMOVED.

1.2 GEOGRID REINFORCING SHALL BE TENSAR BIAXIAL AND UNIAXIAL GEOGRIDS MANUFACTURED BY THE TENSAR CORPORATION, MORROW, GEORGIA.

1.3 BODKIN BARS SHALL BE 1.5" X 1/4" HDPE BARS MANUFACTURED BY THE TENSAR CORPORATION, MORROW, GEORGIA.

2.0 TECHNICAL REQUIREMENTS

2.1 THE OWNER OR OWNER'S REPRESENTATIVE SHALL SUBMIT TO TENSAR EARTH TECHNOLOGIES, INC. REINFORCED BACKFILL MATERIAL AND RETAINED SOIL/FILL GRADATIONS FOR APPROVAL PRIOR TO PROCEEDING WITH CONSTRUCTION.

2.2 PRIOR TO CONSTRUCTION OF THE TENSAR REINFORCED EMBANKMENT, THE CONTRACTOR SHALL CLEAR AND GRUB THE REINFORCED BACKFILL ZONE AREA.

2.3 THE OWNER OR OWNER'S REPRESENTATIVE SHALL CONFIRM THAT THE SITE HAS BEEN PROPERLY PREPARED AND THE DESIGN PARAMETERS IN SECTION 6.0 ARE APPROPRIATE PRIOR TO FILL PLACEMENT.

2.4 TERRAFIX NONWOVEN GEOTEXTILE, TENSAR TYPE A BIAXIAL GEOGRID AND TYPE B GEOGRID SHALL BE PLACED ON THE PREPARED GROUND SURFACE AND COVERED BY AN INITIAL LIFT OF 900 mm THICK ROCK FILL. ANOTHER LAYER OF TENSAR TYPE B GEOGRID SHALL BE PLACED ON TOP OF THE INITIAL LIFT AND COVERED BY A LAYER OF 150 mm THICK SAND.

2.5 FILL MATERIALS SHALL BE PLACED FROM THE MIDDLE OF THE REINFORCED ZONE TOWARDS THE ENDS OF THE GEOGRID TO ENSURE FURTHER TENSIONING.

2.6 TESTING METHODS AND FREQUENCY, AND VERIFICATION OF MATERIAL SPECIFICATIONS SHALL BE THE RESPONSIBILITY OF THE OWNER OR OWNER'S REPRESENTATIVE.

2.7 A COMPLETE SET OF CONSTRUCTION DRAWINGS AND CONTRACT SPECIFICATIONS SHALL BE ON-SITE AT ALL TIMES, DURING CONSTRUCTION OF THE PRISM SYSTEM, IF ANY.

3.0 TENSAR GEOGRID PLACEMENT

3.1 TENSAR GEOGRID SHALL BE PLACED AT THE LOCATIONS, ELEVATIONS AND ORIENTATIONS SHOWN ON THE DRAWINGS.

3.2 TENSAR GEOGRID LENGTH SHALL BE AS SHOWN ON THE CONSTRUCTION DRAWINGS.

3.2.1 TENSAR GEOGRID REINFORCEMENT SHALL BE CONTINUOUS THROUGHOUT THEIR EMBEDMENT LENGTH(S). THE BODKIN CONNECTION SHALL NOT BE UTILIZED UNLESS PRE-APPROVED BY THE OWNER OR OWNER'S REPRESENTATIVE PRIOR TO CONSTRUCTION.

3.2.2 IF PRE-APPROVED, TENSAR UNIAXIAL GEOGRIDS MAY BE SPLICED UTILIZING THE BODKIN CONNECTION DETAIL. NO MORE THAN ONE SPlice SHALL BE ALLOWED IN ANY ONE LENGTH OF REINFORCING AND NO SPLICES SHALL BE ALLOWED FOR GEOGRIDS LESS THAN 2.0 m IN LENGTH (EACH).

3.3 PRIOR TO PLACING FILL, THE GEOGRID MATERIALS SHALL BE PLACED TO LAY FLAT AND PULLED TAUT TO REMOVE ANY SLACK IN THE GEOGRIDS.

3.4 TRACKED CONSTRUCTION EQUIPMENT SHALL NOT BE OPERATED DIRECTLY ON THE GEOGRID. A MINIMUM BACKFILL THICKNESS OF 150 mm IS REQUIRED FOR OPERATION OF TRACKED VEHICLES OVER THE GEOGRID. TURNING OF TRACKED VEHICLES SHOULD BE KEPT TO A MINIMUM TO PREVENT TRACKS FROM DISPLACING THE FILL AND/OR THE GEOGRID.

3.5 RUBBER-TIRED VEHICLES MAY PASS OVER THE GEOGRID REINFORCEMENT AT SLOW SPEEDS, LESS THAN 16 KM/HR. SUDDEN BRAKING AND SHARP TURNING SHALL BE AVOIDED.

3.6 LOW GROUND PRESSURE CONSTRUCTION EQUIPMENT SHOULD BE USED; ESPECIALLY DURING THE INITIAL STAGES OF CONSTRUCTION. SMALL WIDE TRACK DOZERS (WITH MAXIMUM 15 kPa GROUND PRESSURE) SHOULD BE USED FOR SPREADING FILL MATERIAL.

3.7 A MINIMUM OF 75 mm OF FILL MATERIAL SHALL BE REQUIRED BETWEEN LAYERS OF BIAXIAL, UNIAXIAL AND FILTER FABRIC, UNLESS OTHERWISE SHOWN.

4.0 CHANGES TO GEOGRID LAYOUT OR PLACEMENT

4.1 NO CHANGES TO THE TENSAR GEOGRID LAYOUT, INCLUDING, BUT NOT LIMITED TO, LENGTH, GEOGRID TYPE, OR ELEVATION, SHALL BE MADE WITHOUT THE EXPRESSED PRIOR WRITTEN CONSENT OF TENSAR EARTH TECHNOLOGIES, INC.

5.0 DRAINAGE

5.1 THE ENGINEERING, DESIGN, ANALYSIS, DETAILING AND MITIGATION OF BOTH SURFACE DRAINAGE AND SEEPAGE OF GROUNDWATER SHALL BE THE RESPONSIBILITY OF THE OWNER OR OWNER'S REPRESENTATIVE.

6.0 DESIGN PARAMETERS

6.1 DESIGN OF THE REINFORCED SOIL STRUCTURE IS BASED ON THE FOLLOWING PARAMETERS:

	FRICTION ANGLE (°)	UNDRAINED PEAK SHEAR STRENGTH (kpa)	MOIST UNIT WEIGHT (kN/m³)
ROCK FILL FOUNDATION SOILS	38	0	17.0
0.0 - 0.7 m	30	0	12.0
0.7 - 2.0 m	0	15	13.0
2.0 - 2.2 m	0	20	17.0
2.2 - 4.2 m	0	25	17.0
4.2 - 8.0 m	0	34	17.0
8.0 - 10.0 m	0	38	17.0
10.0 - 14.0 m	0	45	17.0
14.0 - 22.0 m	0	50	17.0

6.2 FACTORS OF SAFETY:  
MINIMUM FACTOR OF SAFETY FOR GEOGRID PULLOUT  
SOIL-GEOGRID INTERACTION COEFFICIENT

STATIC  
= 1.5  
= 0.9

6.3 GLOBAL STABILITY:  
MINIMUM FACTOR OF SAFETY FOR GLOBAL STABILITY

STATIC  
= 1.3

6.4 LOADINGS:  
CONTACT PRESSURE  
(ASSUME AMERICAN CRAWLER CRANE, MODEL 9299 SUPPORTED  
BY 6.0 m X 7.0 m MAT)

= 70 kPa

6.5 HYDROSTATIC FORCES

NONE

6.6 SEISMIC DESIGN  
SEISMIC ACCELERATION

= NONE

6.7 DESIGN WATER ELEVATION  
NORMAL WATER LEVEL WAS ASSUMED AT 400 mm ABOVE EXISTING GRADE.

6.8 RAPID DRAWDOWN CONDITION

= NONE

7.0 SPECIAL PROVISIONS

7.1 THE DESIGN PRESENTED HEREIN IS BASED ON SOIL PARAMETERS, FOUNDATION CONDITIONS, GROUNDWATER CONDITIONS, AND LOADINGS STATED IN SECTION 6.0. THESE PARAMETERS ARE AS OBTAINED FROM GOLDER ASSOCIATES AND THURBER ENGINEERING LTD., ETOBICOKE, ONATRIO.

7.2 ELEVATION VIEWS, LOCATIONS, AND GEOMETRY OF EXISTING STRUCTURES MUST BE VERIFIED BY THE OWNER OR OWNER'S REPRESENTATIVE PRIOR TO CONSTRUCTION.

7.3 TENSAR EARTH TECHNOLOGIES, INC. AND TERRAFIX GEOSYNTHETICS INC. ASSUME NO LIABILITY FOR INTERPRETATION OR VERIFICATION OF SUBSURFACE CONDITIONS, SUITABILITY OF SOIL DESIGN PARAMETERS AND INTERPRETATION OF SUBSURFACE GROUNDWATER CONDITIONS.

7.4 THE OWNER OR OWNER'S REPRESENTATIVE IS RESPONSIBLE FOR REVIEWING AND VERIFYING THAT THE ACTUAL SITE CONDITIONS ARE AS DESCRIBED IN SECTION 6.0 PRIOR TO AND DURING CONSTRUCTION. THE OWNER OR OWNER'S REPRESENTATIVE SHALL BE ON-SITE TO ASSURE THE PROVISIONS OF THE CONSTRUCTION NOTES ARE FOLLOWED.

7.5 THE SOIL DESIGN PARAMETERS STATED IN SECTION 6.0 SHALL BE VERIFIED BY THE OWNER OR OWNER'S REPRESENTATIVE. WRITTEN VERIFICATION OF DESIGN PARAMETERS SHALL BE SUBMITTED TO TENSAR EARTH TECHNOLOGIES, INC. PRIOR TO COMMENCING WITH CONSTRUCTION.

7.6 PROCEEDING WITH CONSTRUCTION WITHOUT FIRST PROVIDING TENSAR EARTH TECHNOLOGIES, INC. AND TERRAFIX GEOSYNTHETICS INC. A WRITTEN REPORT VERIFYING CONDITIONS DISCUSSED IN SECTION 6.0, SHALL ABSOLVE TENSAR EARTH TECHNOLOGIES, INC. AND TERRAFIX GEOSYNTHETICS INC. FROM ALL LIABILITY FOR THE DESIGN AND CONSTRUCTION OF THIS STRUCTURE AND CONTRACTOR SHALL INDEMNIFY AND HOLD HARMLESS TENSAR EARTH TECHNOLOGIES, INC. AND TERRAFIX GEOSYNTHETICS INC. FROM ALL RESULTING CLAIMS, DAMAGES, LOSSES AND EXPENSES.

7.7 ANY REVISIONS TO DESIGN PARAMETERS STATED IN SECTION 6.0 OR STRUCTURE GEOMETRY SHALL REQUIRE DESIGN MODIFICATIONS PRIOR TO PROCEEDING WITH CONSTRUCTION.

7.8 ONCE THE TEMPORARY ACCESS ROAD IS NO LONGER REQUIRED, ALL OF THE ROCK FILL AND GEOSYNTHETICS SHALL BE REMOVED IN SUCH A MANNER SO AS TO MINIMIZE ANY FURTHER DISTURBANCE TO THE UNDERLYING SOILS.

7.8.1 THE ROCK FILL AND UPPER LAYERS OF GEOGRIDS MAY BE REMOVED IN ONE LIFT. CARE SHALL BE EXERCISED SO AS TO NOT PUNCTURE THROUGH THE BOTTOM LAYERS OF GEOGRID.

7.8.2 THE LOWER LAYERS OF GEOGRID AND THE GEOTEXTILE LAYER SHALL BE PEELED UP AND REMOVED AS THE REMOVAL OF THE ROCK FILL PROGRESSES.

7.9 THE SPACE REQUIRED FOR CONSTRUCTION EQUIPMENT MANEUVERING ON THE EMBANKMENT SHOULD BE CHECKED BY AN EQUIPMENT SPECIALIST.

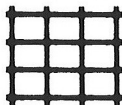


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9/3/99

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As Shown

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KL

Drawn by  
KL

Checked by

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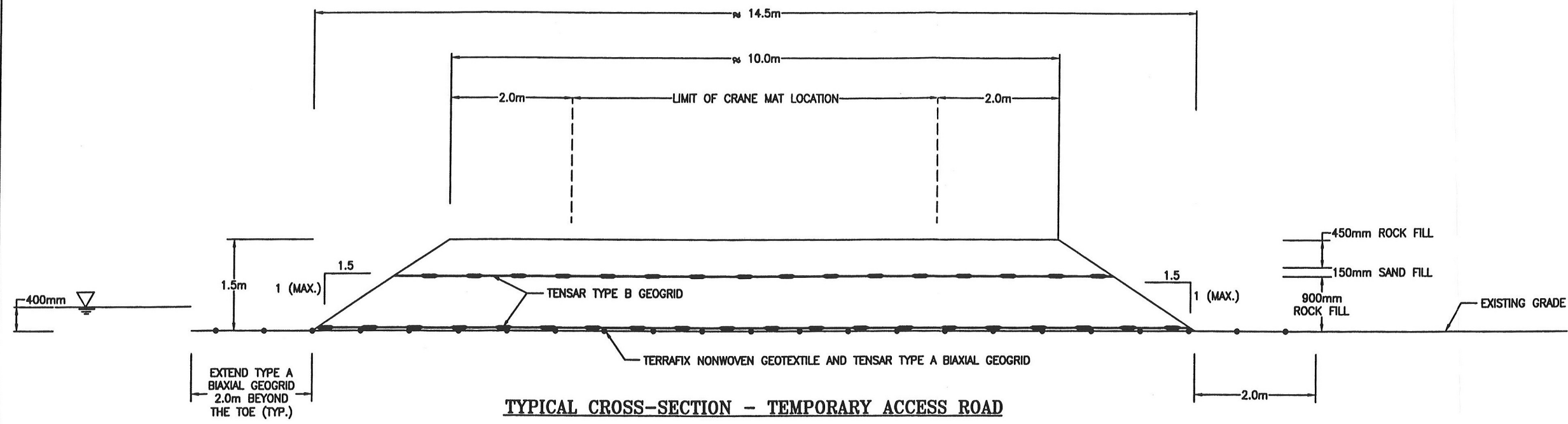
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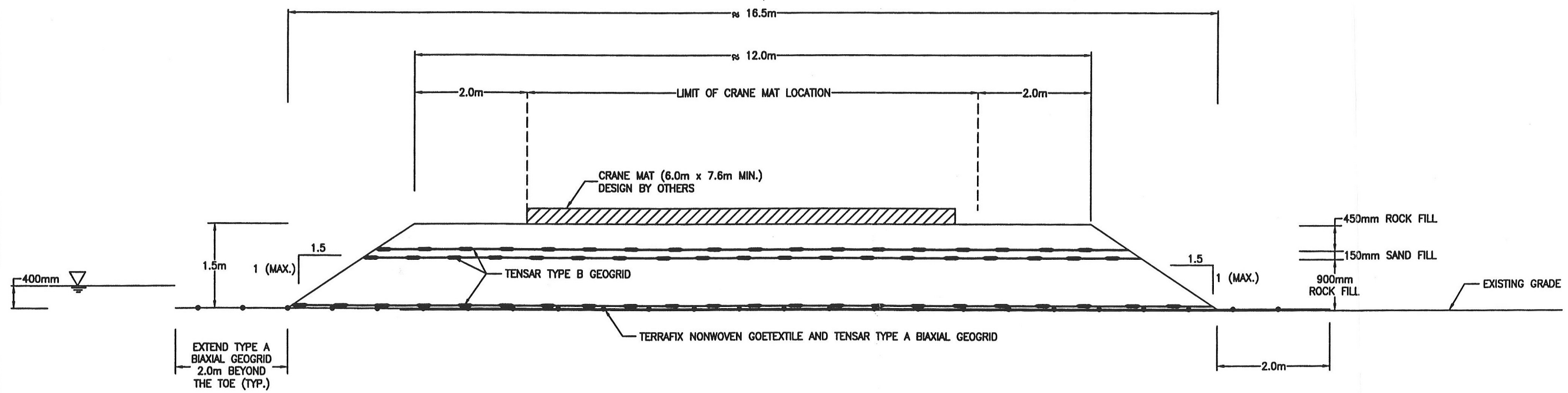
CONSTRUCTION  
NOTES

Sheet Number

2 of 5



**TYPICAL CROSS-SECTION - TEMPORARY ACCESS ROAD**



**TYPICAL CROSS-SECTION - CRANE PAD**

NOTE:  
SEE SHEET 5 FOR GEOGRID LAYOUT.



E9950103.DWG

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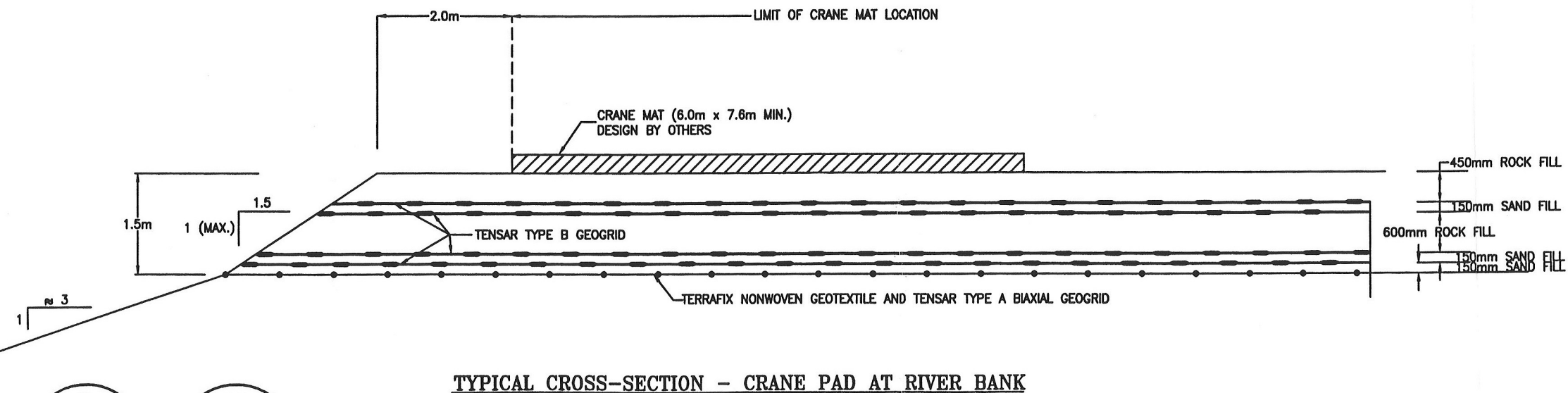
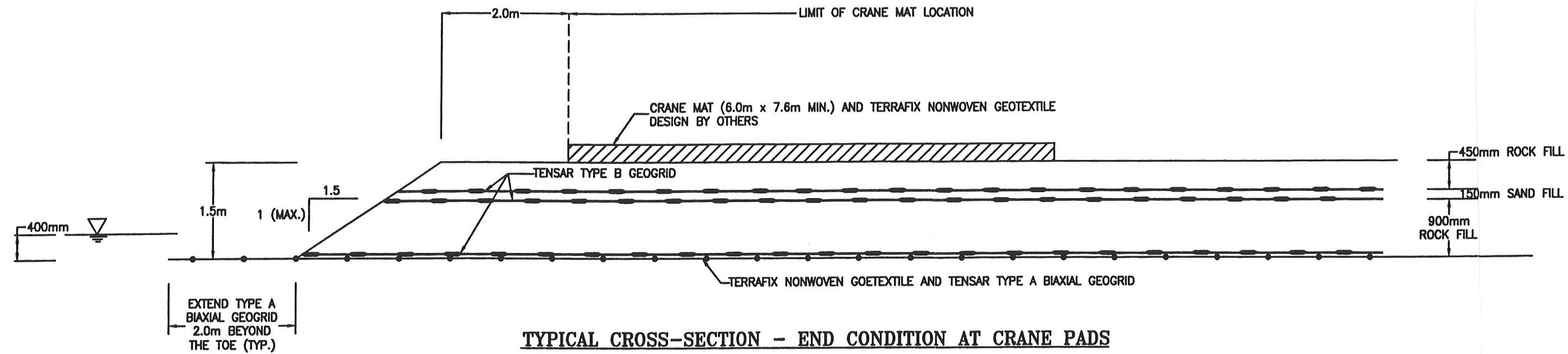
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**TYPICAL CROSS-SECTIONS**

Sheet Number  
**3 of 5**



NOTE:  
SEE SHEET 5 FOR GEOGRID LAYOUT.



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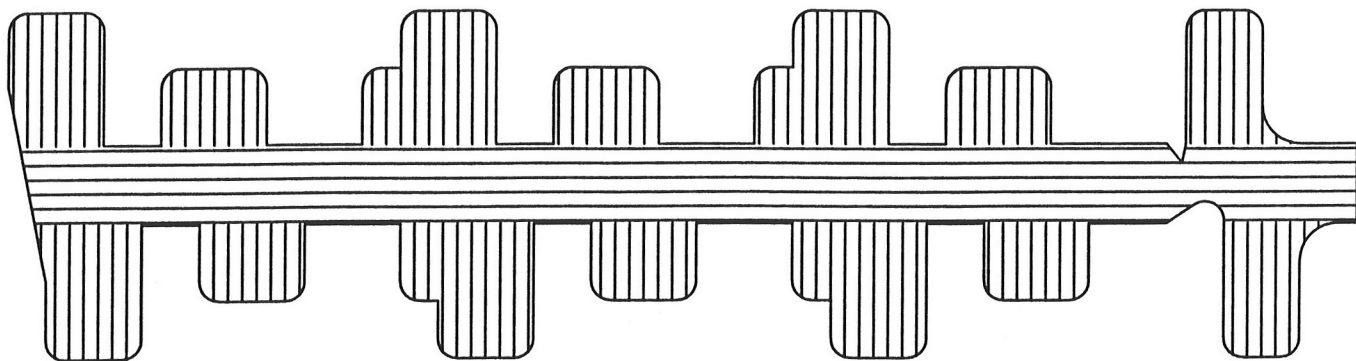
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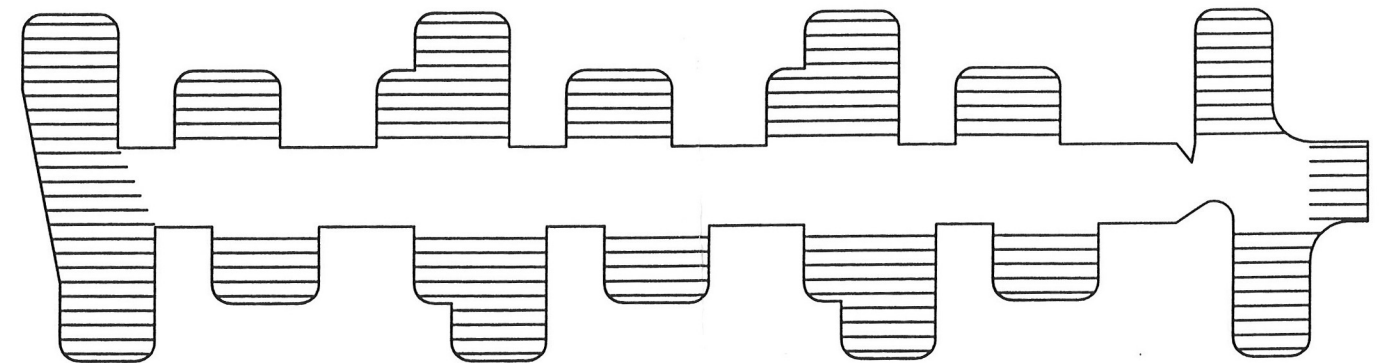
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**TYPICAL  
CROSS-SECTIONS**

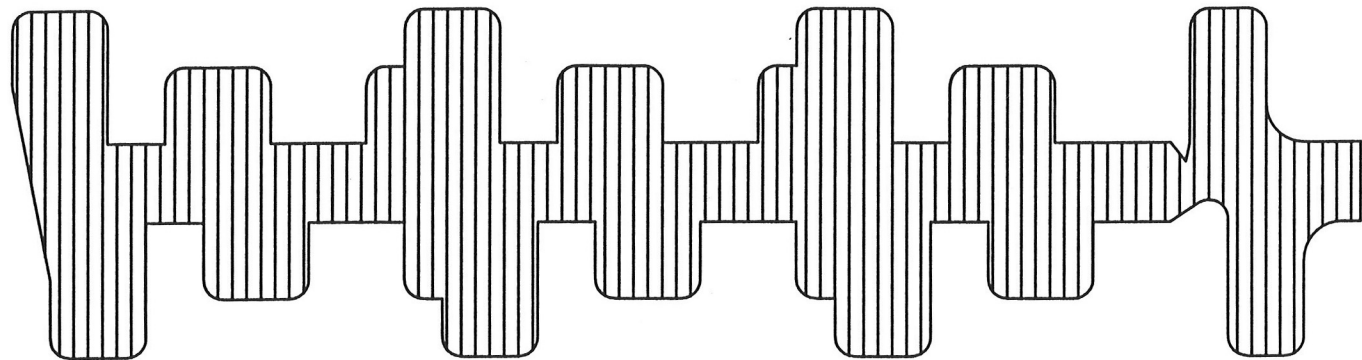
Sheet Number  
**4 of 5**



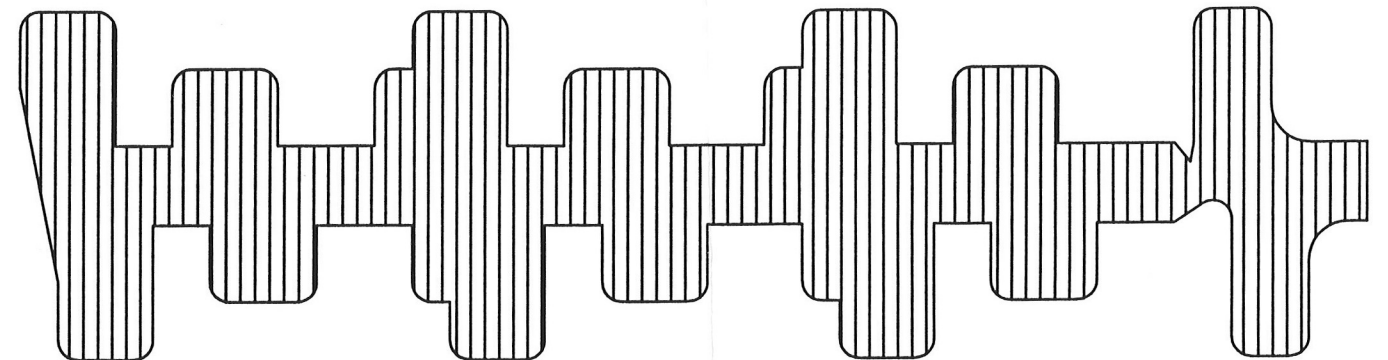
STEP 1: PLACE TERRAFIX NONWOVEN GEOTEXTILE AND TENSAR TYPE A BIAXIAL GEOGRID AT THE BASE OF EMBANKMENT  
TYPE A BIAXIAL GEOGRID SHOULD HAVE A 1.0 m MINIMUM OVERLAP AT THE SIDES AND ENDS OF ADJACENT ROLLS.



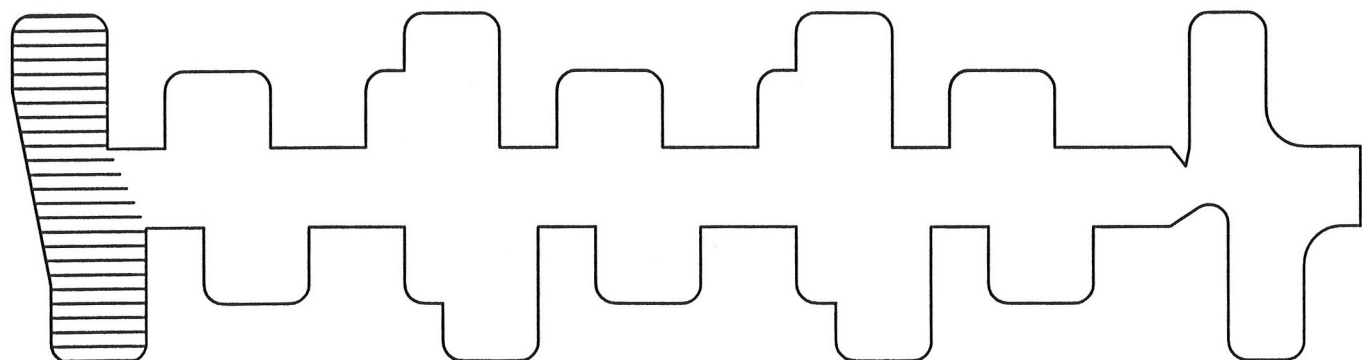
STEP 4: PLACE ONE LAYER TENSAR TYPE B GEOGRID AT 900mm ABOVE THE BASE OF EMBANKMENT



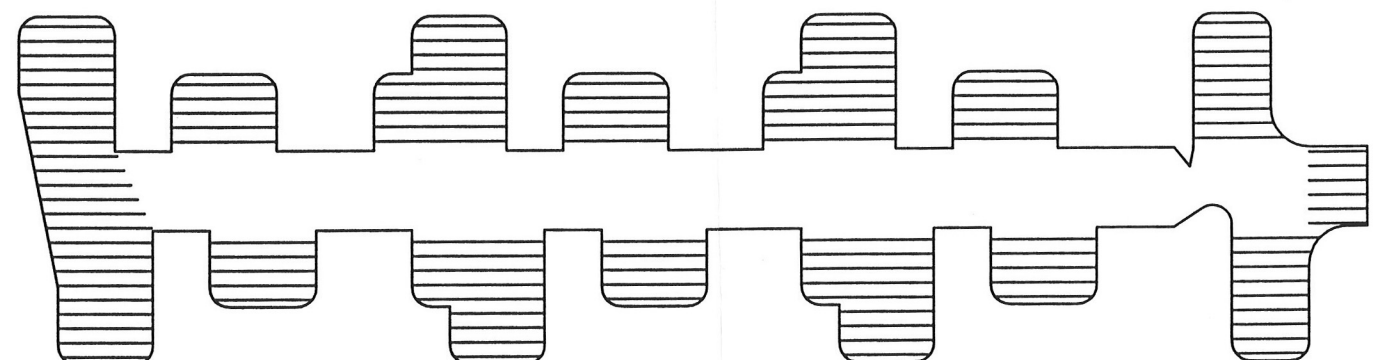
STEP 2: PLACE ONE LAYER TENSAR TYPE B GEOGRID ON TOP OF TYPE A BIAXIAL GEOGRID



STEP 5: PLACE ONE LAYER TENSAR TYPE B GEOGRID ON TOP OF THE TYPE B GEOGRID INSTALLED IN STEP 4



STEP 3: PLACE 2 LAYERS TENSAR TYPE B GEOGRID AT 150mm AND 300mm ABOVE BASE OF EMBANKMENT



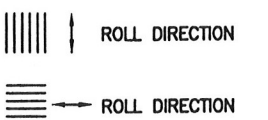
STEP 6: PLACE ONE LAYER TENSAR TYPE B GEOGRID AT 1050mm ABOVE BASE OF EMBANKMENT



E9950105.DWG

NOT TO SCALE

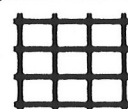
#### LEGEND



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**GEOGRID  
LAYOUT**

Sheet Number

**5 of 5**

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

#### Dynamic Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils

Consistency	$c_u, s_u$ kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
$\gamma$	unit weight

Note:

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I GENERAL

$\pi$	= 3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10} x$ or $\log x$ ,	logarithm of x to base 10
$g$	acceleration due to gravity
$t$	time
$F$	factor of safety
$V$	volume
$W$	weight

### II STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
$u$	porewater pressure
$E$	modulus of deformation
$G$	shear modulus of deformation
$K$	bulk modulus of compressibility

### III SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
$e$	void ratio
$n$	porosity
$S$	degree of saturation
*	Density symbol is $\rho$ . Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

#### (a) Index Properties (con't.)

$w$	water content
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	plasticity Index = $(w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_p) / I_p$
$I_C$	consistency index = $(w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (c) Hydraulic Properties

$h$	hydraulic head or potential
$q$	rate of flow
$v$	velocity of flow
$i$	hydraulic gradient
$k$	hydraulic conductivity (coefficient of permeability)
$j$	seepage force per unit volume

#### (d) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (overconsolidated range)
$C_s$	swelling index
$C_\alpha$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
$U$	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
$p$	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
$q$	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

Notes: 1.  $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

PROJECT 991-1155			RECORD OF BOREHOLE No 99-1			1 OF 1			METRIC															
W.P. 451-90-03/04			LOCATION Stn. 18+067 Eastbound Lanes			ORIGINATED BY FJH																		
DIST 42 HWY 417			BOREHOLE TYPE Hand Augerhole			COMPILED BY JS																		
DATUM Geodetic			DATE 20.7.99			CHECKED BY AJW																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60						80	100	20	40	60	80	100	60	120
83.10	Peat (Fibrous)																							
82.40	Organic Silt with peat layers.		1	SH																				
81.10			2	SH																				
80.90	Clayey Silt, trace sand.																							
80.66	No Sample																							
2.44	END OF BOREHOLE																							
	APPROXIMATE ELEVATION																							

PROJECT 991-1155			RECORD OF BOREHOLE No 99-2				1 OF 1		METRIC						
W.P. 451-90-03/04			LOCATION Stn. 18+027 Westbound Lanes				ORIGINATED BY FJH								
DIST 42 HWY 417			BOREHOLE TYPE Hand Augerhole				COMPILED BY JS								
DATUM Geodetic			DATE 20.7.99				CHECKED BY AJW								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED							
83.00								20 40 60 80 100							
0.00	Peat (Fibrous)		1	SH				20 40 60 80 100							
82.30								20 40 60 80 100							
0.70	Organic Silt with silty clay and occasional peat layers.		2	SH				20 40 60 80 100							
81.40								20 40 60 80 100							
81.20	Silty Clay, trace sand.							20 40 60 80 100							
80.90	No Sample							20 40 60 80 100							
80.90								20 40 60 80 100							
2.10	END OF BOREHOLE APPROXIMATE ELEVATION							20 40 60 80 100							

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