

**Golder Associates Ltd.**

2390 Argentia Road  
Mississauga, Ontario, Canada L5N 5Z7  
Telephone (905) 567-4444  
Fax (905) 567-6561



**Golder  
Associates**

GEOCRES No:  
31C-169

**REPORT ON**

**DESIGN OF TEMPORARY WETLAND ACCESS ROADS  
REHABILITATION OF THE HIGHWAY 401  
SALMON RIVER BRIDGE, SITE NO. 11-207  
G.W.P 82-98-00  
MINISTRY OF TRANSPORTATION, ONTARIO  
DISTRICT 8, KINGSTON**

Submitted to:

Lea Associates Ltd.  
Suite 900, 625 Cochrane Drive  
Markham ON  
L3R 9R9

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## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Lea Associates Ltd. (Lea) on behalf of the Ministry of Transportation, Ontario (MTO) to design the construction access roads and crane pads that will be part of the Salmon River Bridge rehabilitation project. The scope of work was defined in our proposal letter dated January 16, 2004.

Preliminary design guidelines/recommendations for various alternatives for the access roads /crane pads were contained in a letter report and submitted by Golder on December 15, 2003. A meeting was held between the MTO, Lea and Golder on January 14, 2004 to discuss the results of our recommendations and it was decided that a reinforced geogrid embankment was the preferred design alternative and that design should be carried out by the owners representative, rather than by the Contractor.

Design drawings and construction notes will be produced by Tensar Earth Technologies Inc. (Tensar) with input from Forza Solutions (Forza) and review by Golder. A non-standard special provision (NSSP) will be drafted by Golder with input from Forza and Lea for inclusion into the Contract Documents. This design report contains recommendations and assumptions used in design of the geogrid embankments, as well as our review of the final drawings and construction notes by Tensar. The design drawings will be signed and stamped in accordance with MTO guidelines.

Design input and review of draft drawings and draft provisions have been carried out as necessary to facilitate submission of the technical review package in late January 2004.

## **2.0 BACKGROUND**

### **2.1 Background Information**

The existing Salmon River bridge is located on the four-lane stretch of Highway 401 between Belleville and Napanee, Ontario. The bridge is a 31 m wide, 5 span, 126 m long bridge with a 30 degree skew to the highway. The existing bridge was constructed in 1956 and was last rehabilitated in 1983. As part of the current rehabilitation work, widening of the four piers and two abutments will be required, on both sides of the bridge in conjunction with widening of the bridge deck. Additional piles will be required only at the abutment locations. Temporary roads and support pads will be required for access by equipment during construction. These temporary roadways and support pads are to be removed following construction to comply with environmental requirements as described below. Figure 1 shows the location of the access roads/crane pads and the location of the boreholes in plan.

Detailed description of the subsurface conditions are contained in the investigation reports listed below:

- Foundation Investigation and Design Report, Rehabilitation of the Highway 401 Salmon River Bridge, Site No. 11-207, G.W.P. 82-98-00, Geocres No. 31C-168, dated August 2003.
- Foundation Investigation Report, Tyendinaga Bridge No. 5 Crossing the Salmon River, Geocres No. 31C-131, dated May 16, 1955.

### **2.2 Site Description and Environmental Constraints**

In total, eight temporary crane support pads are required to carry out the construction work. In addition, temporary roads will be required to access the crane pads as well as traverse under the bridge in some areas. Crane pads at the north and south sides of Piers A and D are required to support cranes capable of lifting steel girders to the centre span of the bridge and as such, will be relatively large. Crane pads at the north and south sides of Piers B and C will be required for general lifting and will be comparatively smaller.

The floodplain of the Salmon River is considered to be an environmentally sensitive area. In order to comply with environmental constraints, the temporary construction access roads and crane pads are permitted to remain in place for a only a limited duration. It is understood that the large crane pads will be in use for the full duration of construction and that the smaller crane pads

will only be allowed to be in use for one construction season and in addition, the crane pads at Pier B cannot be in use at the same time as the crane pads at Pier C.

It should be noted that based on the current topography and river course, the crane pads at both the north and south side of Pier B will be within the river channel. The remaining access roads and crane pads are not to be within the main channel; however, the river water level fluctuates from season to season and as such, a portion of or all of the roads/pads may be located within the water course at the same time during high water.

### 3.0 SUBSURFACE CONDITIONS

The bridge site is located within the Salmon River valley and the terrain generally consists of open fields, bush areas, swamp areas and limestone bedrock outcrops to the east and west of the site. The ground surface at the borehole locations in the flood plain area generally varies between about Elevation 79.8 m to 80.6 m.

In general, the subsoils at the site consist of a surficial layer of topsoil underlain by alternating thin layers of sand and gravel to silty clay up to 4 m thick containing organics. The surficial soils are underlain by a clayey silt to clay deposit up to 11.5 m in thickness. The clayey silt to clay deposit is underlain by limestone bedrock. A brief outline of the subsoil conditions is provided below. For details of the soil strata, refer to the reports referenced in Section 2.1.

The boreholes advanced within the flood plain area indicate that the surficial deposits range in composition from silty clay to sand to sand and gravel. The Standard Penetration Test (SPT) 'N' values within the surficial silty clay deposits range from 0 blows to 9 blows per 0.3 m of penetration and are typically less than 8 blows suggesting a very soft to firm consistency. The SPT 'N' values within the surficial sand to sand and gravel deposits range from 1 to 12 blows per 0.3 m of penetration and are typically less than about 8 blows indicating a very loose to compact relative density.

The base of the surficial loose/firm deposits is typically between 1.8 m and 3.8 m below the ground surface. Where the boreholes were drilled within the river channel (during the 1955 investigation), the river bed was encountered between 2.1 m and 2.8 m below the water surface and the base of the loose/firm deposits was up to 1.4 m below the river bed.

The thick deposit of clayey silt to clay below the surficial deposits was described as a stiff to very stiff silty clay to clay in the upper portion and a soft to stiff clayey silt within the lower portion. The bedrock surface between the west and east abutments was typically between about 6.4 m and 15.2 m below the ground/water surface, the greater depth towards the middle of the channel.

The water levels measured at the site are consistent with the adjacent river water level; typically less than 1.0 m below the ground surface. The water level in the Salmon River fluctuates seasonally and was surveyed at Elevation 79.1 in August 2003 (by Lea). It should be noted that groundwater levels in the area are subject to seasonal fluctuations. The 1 in 10 year design flood level of the Salmon River is at Elevation 80.4 m (information provided by Lea).

## **4.0 DISCUSSION**

### **4.1 Stratigraphy and Parameters**

The simplified stratigraphy and design parameters given in Tables 1 and 2 attached to this report were utilized in the stability and settlement analyses at each of the crane pad locations.

The strength parameters for the cohesive deposits were based on Standard Penetration Test (SPT) 'N' values for the upper deposits and based on in-situ vane tests in the lower deposits. The strength parameters for the granular deposits were based on SPT 'N' values. A summary of the SPT 'N' data and the undrained shear strength data is given in Figures 2 and 3. A summary of the strength parameters for the various deposits is given in Table 1.

The compressibility parameters of each of the clayey deposits were based on index property correlations and the overconsolidation ratio (OCR) was based on published correlations to undrained shear strength. For the granular deposits, the compressibility was estimated using elastic modulus values estimated based on SPT 'N' values. A summary of the void ratio, compression and recompression indices and the overconsolidation data along with the associated design assumptions are shown on Figures 4 to 7. A summary of the compressibility parameters for the various deposits is given in Table 2.

The static groundwater level assumed for design is at Elevation 79.1 m. The design flood level (1 in 10 year spring flood) is at Elevation 80.4 m.

The design top of pad/road elevation was estimated to be a minimum of 0.5 m above the spring flood level, at Elevation 80.9 m. The proposed thickness of the embankments based on the existing available survey information and depth soundings in the Salmon River are given in Table 3 following the text of this report. The given values represent the average thickness across the crane pad. Variations of at least 0.3 m across the crane pad are expected to occur. The maximum embankment thickness anticipated in the areas of the large crane pads/access roads is 1.7 m. The maximum embankment thickness anticipated in the areas of the smaller crane pads/access roads is 3.0 m. For the access roads that run under the bridge, the embankment heights could be as low as 0.3 m in some areas to allow for sufficient headroom.

### **4.2 Crane Loading**

The crane sizes and loading conditions listed below were provided to us by Lea. A brief description of the cranes to be used at the small and large crane pads is also given below. The following specific examples are provided for illustration purposes only, the contractors proposed lifting equipment should be reviewed to ensure compliance with the ground pressure restrictions.



**400 tonne Mobile Crane (LTM 1400)**

Dimensions:	Width	(retracted)	=	3.0 m
	Width	(outriggers extended)	=	9.5 m
	Length	(between outriggers)	=	10.7 m
	Length	(front to rear wheels)	=	12.0 m
Travelling Ground Pressure				= 185 kPa
Maximum Load per outrigger during lifting				= 1300 kN
Average ground pressure on outrigger pads (4.5 m x 4.5 m) when lifting				= 64 kPa

For manufacturer details of dimensions and loading see Appendix A.

**60 tonne Mobile Crane**

Dimensions:	Width	(retracted)	=	3.0 m (assumed)
	Width	(outriggers extended)	=	6.5 m
	Length	(between outriggers)	=	8.5 m
Travelling Ground Pressure				= 185 kPa (assumed)
Maximum Load per outrigger during lifting				= 520 kN
Average ground pressure on outrigger pads (3.0 m x 3.0 m) when lifting				= 58 kPa

Figures A-1 and A-2 in Appendix A show the crane pad layout for both the 60 tonne crane and the 400 tonne cranes, respectively, assuming a minimum setback distance of 1.5 m from the edge of the outrigger pads. The setback requirements are discussed in Section 4.4.3. The outrigger pads should be designed by the crane supplier and should be capable of distributing the loads such that the maximum stress imposed by the crane during lifting is less than 70 kPa. The outrigger pads could consist of timber mats with a metal (steel) grillage between the outrigger and the timber to assist in load distribution.

**4.3 Settlement Analysis**

Settlement of the subsoils will occur as a result of the embankment loading as well as under the crane loading. Settlement of the granular soils will occur rapidly, during construction as well as during initial lifting. Consolidation of the cohesive soils will be long-term and post-construction settlements will occur. The following options were considered for embankment construction;

1. Rock Fill, unit weight = 19 kN/m<sup>3</sup>,  $\phi'$  = 38 degrees
2. Light Weight Fill, unit weight = 14 kN/m<sup>3</sup>,  $\phi'$  = 35 degrees

The use of granular fills were not considered for use in the geogrid embankment due to environmental constraints. The settlements were estimated using the commercially available program UNISETTLE (v3.0) produced by Unisoft Ltd. The results of the settlement analysis are summarized below:

Location	Assumed Embankment Height (m)	Estimated Settlement (mm)		Maximum Estimated Settlement with Crane Load* (mm)
		Rock Fill	Lightweight Fill	
Large Crane Pad	2.0	170	150	270
Small Crane Pad	3.0	70	50	90

\* Assumes rock fill is used for construction of the embankments. Calculated settlement is under centre of outrigger pad.

It should be noted that these settlements estimated are approximate and local variations of as much as 50 percent of the above values should be considered for project planning. These settlement estimates also do not account for disturbance, rutting, or the localized presence of compressible organic materials. Caution should be exercised when applying these settlement estimates to evaluate construction conditions that are sensitive to settlements.

About half to two thirds of the settlement would occur rapidly due to the compression of the upper granular deposits and the over-consolidated cohesive deposits as a result of the embankment and crane loading. The remaining settlement should develop over a period of months to years due to consolidation of the clay deposits. The difference in settlement due to fill type is minor and should not be a governing factor on fill selection. The contractor should be required to regularly examine settlements under each outrigger pad and adjust outrigger support accordingly to accommodate total and differential settlements.

The effect of the crane loading on the settlement of the subsoils will be dependant on the type of crane utilized and the distribution of loads to the outriggers. In order to lift the steel girders into the centre span of the bridge from the large crane pads adjacent to Piers A and D, the boom will have to reach out a distance of about 48 m. This will create uneven loading conditions for the outrigger pads; however, the difference in settlement between the outrigger pads during lifting is about 25 mm. The contractor should be made aware of this problem and be prepared to control the counterweight to try to maintain a uniform ground pressure under the grillage pads and prevent differential settlement occurring during lifting. At the location of the small crane pads, it is assumed that the required lifting will be mostly vertical and limited differential settlement will occur between the outrigger pads.

Constructing the embankment will require care. Placement of fill may create a displacement wave of mud. The displacement may create the need for larger volumes of fill than the settlement estimate indicates. The creation of the mud wave is a function 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.

#### **4.4 Stability Analysis**

All slope stability analyses were performed using the commercially available program SLOPE/W (Version 5.13), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor 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. A target factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at these sites considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the various embankment heights and geometries.

The embankment is statically stable with a factor of safety greater than 1.3 for the two fill options, using 1.5H to 1V side slopes for the given embankment geometry. The difference in factor of safety between the two options is minimal and it is therefore considered that the use of more expensive lightweight fill will not be warranted.

##### **4.4.1 Effect of Loading**

The effect of crane loading on the embankment may be divided into two cases; travelling and lifting. For analysis purposes, it is assumed that wheeled cranes and construction equipment will be used.

##### **4.4.1.1 Travelling Case**

The following dimensions of a wheeled crane were assumed; tire width of about 0.4 m and an overall width of 3.0 m, based on the dimensions of the 400 tonne crane. The effect of tracked cranes was not analysed and if tracked cranes are being considered by the contractor, the stability analyses should be checked for the tracked case.

The access road should be wide enough to safely maneuver a heavy vehicle without the edge of the embankment failing and to provide a margin of safety against driver error. We recommend that a minimum set back of 1.25 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 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 but there is a significant load spreading effect and using a 2-D slope stability program would be inappropriate. The ultimate bearing capacity of the floodplain deposits beneath the embankment is between about 100 to 125 kPa. Therefore, 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 assist in redistributing the load and reinforcing the embankment materials at the subgrade level and it is also likely that additional geogrid layers within the embankment/road base will also be required.

It should be noted that ground pressure imposed by heavily loaded concrete and dump trucks should be reviewed prior to their use in the temporary access road/crane pad areas.

#### **4.4.1.2 Lifting Case**

We understand that the 400 tonne crane has to be capable of lifting the centre bridge girders into place from the Pier A and D locations. This relates to a lift of 8 tones at a distance of about 48 m. In this case, high loads will be encountered under each of the four outriggers. Somewhat smaller loads are anticipated in the case of the 60 tonne cranes for the small crane pads. In both cases, in order to reduce the pressure loading during the lift and help to evenly distribute the pressure to the subgrade soils, the crane can be located on a grillage or some form of pad to spread the load. Grillage/outrigger mats of 9 m<sup>2</sup> and 20.25 m<sup>2</sup> were assumed for the small and large cranes, respectively. The analyses assumed 1.5H:1V rock fill side slopes. The use of light weight fill as an embankment material was not specifically considered in the following stability analyses as its behaviour may be considered as being similar to the rock fill embankment. The following table summarizes the results of the analyses:

<i>Crane Pad Location</i>	<i>Maximum Embankment Height (m)</i>	<i>Maximum Ground Pressure (kPa)</i>	<i>Minimum Required Setback (m)</i>	<i>Factor of Safety</i>
Small (Piers B and C)	3.0	50	2.0	>1.3
		70	2.5	>1.3
Large (Piers A and D)	1.9	50	1.5	>1.3
		70	2.5	>1.3

Results of the analysis are shown on Figures 8 and 9 for the small and large crane pads, respectively. Setback distance is defined as the distance from the loaded area (i.e. edge of grillage pad) to the slope crest.

Given the environmental constraints at the site, a setback of 1.5 m the maximum setback allowable. A setback of 1.5 m may be acceptable if geogrid reinforcement is utilized in the embankment. The outrigger pads / grillages should be sized to restrict the ground pressure to under 70 kPa. The effect of loading adjacent to the river bank was also checked and the above setback values are still applicable.

## 5.0 SUMMARY OF RECOMMENDATIONS/COMMENTS

- For a rock fill embankment with 1.5H to 1V side slopes, a minimum 2.5 m setback and a lifting restriction on the ground pressure of 70 kPa should 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 side slopes steeper than 1.5H:1V and reduced setback distances to the required 1.5 m.
- Each crane outrigger should be supported on a grillage (built-up heavy timber mat) to assist in spreading the outrigger load where the grillage contacts the embankment materials. A 3.0x3.0 m (minimum) grillage is required for the small crane outriggers and a 4.5x4.5 m (minimum) grillage is required for the large crane outriggers to keep the ground pressure within allowable levels.
- Differential settlement of the cranes will occur during lifting of the steel girder into the centre span. Elsewhere, the crane selected should be capable of lifting as vertically as possible in order to prevent differential settlement occurring across the outrigger pads during lifting. Settlement and level of the cranes should be routinely checked during and after each lifting event.
- The contractor should check his proposed equipment to ensure that the allowable ground pressure of 70 kPa is not exceeded.
- The space required for vehicle maneuvering on the embankment should be evaluated separately by an equipment specialist.
- Calculation of the volume of fill required for the embankment should take into account the proposed settlement. The embankment should be constructed to some minimum level above the floodplain, Elev. 80.9 m as assumed in our calculations. The minimum freeboard should be established based on hydraulic and operational considerations.

## 6.0 REVIEW OF TENSAR DESIGN


The stratigraphy and soil parameters including the subsurface information from our Foundation Investigation and Design Report were supplied to Tensar Earth Technologies Inc. (Tensar), a specialty geogrid embankment designer / manufacturer to develop necessary design details for both the temporary access roads and the crane pads over the floodplain. To provide the necessary liaison with the specialty designer, we retained Mr. Phil Perzia, P.Eng. of Forza Solutions to assist in development of the design requirements for this temporary access road and crane pad geogrid / geotextile rockfill embankment.

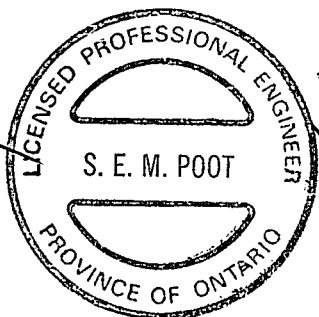
Tensar carried out the design for the reinforced geogrid embankment as well as produced a set of drawings for inclusion into the contract. Discussions were held between Golder and Forza throughout the design stage to ensure that the geometry, soil conditions and design assumptions were carried out in accordance with that which were provided.


The settlement and stability of the embankment is improved by the use of geogrid. The overall stability of the embankment is improved by the addition of layers of biaxial and uniaxial geogrid. The addition of geogrid increases the local stability as well as allows for reduced setback requirements. The factor of safety for global stability increases about 25 percent with the addition of geogrid reinforcement for the 1.5 m setback case. The maximum settlement of the subsoils under the embankments as well as differential settlements will also be reduced with the addition of geogrid layers, although the percentage in reduction of settlement cannot be quantified.

The drawings were prepared based on the access road / crane pad layout and equipment loading provided by Lea Associates. If there are any changes to the layout of the roads/pads or equipment, the drawings will have to be modified by Tensar prior to construction.

### GOLDER ASSOCIATES LTD.

  
Sarah E. M. Poot, P.Eng.,  
Geotechnical Engineer

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



SEP/SJB/MSD/FJH/sep

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**TABLE 1**  
**SUMMARY OF DESIGN PARAMETERS**  
**CRANE PADS - SALMON RIVER BRIDGE**

<i>Location</i>	<i>Elevation (m)</i>	<i>Thickness (m)</i>	<i>Stratigraphy</i>	<i>Assumed Design Parameters</i>	
				<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Strength</i>
Large Crane Pad North Side, Pier A	79.7 – 78.9	0.8	Soft silty clay with organics	18	$s_u = 30$ kPa
	78.9 – 78.2	0.7	Loose sand	20	$\phi' = 30^\circ$
	78.2 – 77.4	0.8	Soft silty clay	18	$s_u = 30$ kPa
	77.4 – 72.1	5.3	Stiff to very stiff silty clay to clay	18	$s_u = 75$ kPa
	72.1 – 66.0	6.1	Firm to stiff clayey silt	18	$s_u = 25$ kPa
Large Crane Pad South Side, Pier A	79.6 – 77.6	2.0	Soft to stiff silty clay with organics	18	$s_u = 30$ kPa
	77.6 – 75.8	1.8	Loose sand and gravel with organics	20	$\phi' = 30^\circ$
	75.8 – 72.0	3.8	Stiff to very stiff silty clay to clay	18	$s_u = 75$ kPa
	72.0 – 65.5	6.5	Firm to stiff clayey silt	18	$s_u = 25$ kPa
Large Crane Pad North Side, Pier D	79.5 – 78.1	1.4	Loose sand and gravel with organics	20	$\phi' = 30^\circ$
	78.1 – 76.7	1.4	Soft to firm silty clay with organics	18	$s_u = 30$ kPa
	76.7 – 71.0	5.7	Stiff silty clay to clay	18	$s_u = 75$ kPa
	71.0 – 68.0	3.0	Soft to firm clayey silt	18	$s_u = 25$ kPa
Large Crane Pad South Side, Pier D	79.7 – 78.2	1.5	Very soft to soft silty clay with organics	18	$s_u = 20$ kPa
	78.2 – 77.4	0.8	Very loose silty sand to sand and gravel	20	$\phi' = 28^\circ$
	77.4 – 75.1	2.3	Stiff to very stiff silty clay to clay	18	$s_u = 75$ kPa
	75.1 – 72.1	3.0	Stiff clayey silt	18	$s_u = 50$ kPa
Small Crane Pad North and South Side Pier B	78.2 – 75.7	2.5	Loose sand and gravel with organics	20	$\phi' = 28^\circ$
	75.7 – 71.9	3.8	Stiff to very stiff silty clay to clay	18	$s_u = 75$ kPa
	71.9 – 65.4	6.5	Soft to firm clayey silt	18	$s_u = 25$ kPa
Small Crane Pad North and South Side Pier C	79.1 – 76.6	2.5	Loose sand and gravel with organics	20	$\phi' = 28^\circ$
	76.6 – 72.6	4.0	Stiff to very stiff silty clay to clay	18	$s_u = 75$ kPa
	72.6 – 66.1	6.5	Soft to firm clayey silt	18	$s_u = 25$ kPa



**TABLE 2**  
**SUMMARY OF COMPRESSIBILITY PARAMETERS**  
**CRANE PADS - SALMON RIVER BRIDGE**

<i>Deposit</i>	<i>e<sub>o</sub></i>	<i>C<sub>c</sub></i>	<i>C<sub>r</sub></i>	<i>OCR</i>	<i>E (MPa)</i>
Soft to firm silty clay with organics	0.90	0.27	0.025	top of layer 1 bottom of layer 4	5
Very loose silty sand to sand and gravel with organics	n/a	n/a	n/a	n/a	5
Stiff to very stiff silty clay to clay	top of layer 0.90 bottom of layer 0.75	top of layer 0.27 bottom of layer 0.12	top of layer 0.050 bottom of layer 0.022	top of layer 6 bottom of layer 2.5	n/a
Soft to firm clayey silt	top of layer 0.75 bottom of layer 0.55	0.12	0.015	top of layer 2 bottom of layer 1.2	n/a

Where

$c_c$  = compression index

$c_r$  = recompression index

$e_o$  = initial void ratio

OCR = overconsolidation ratio

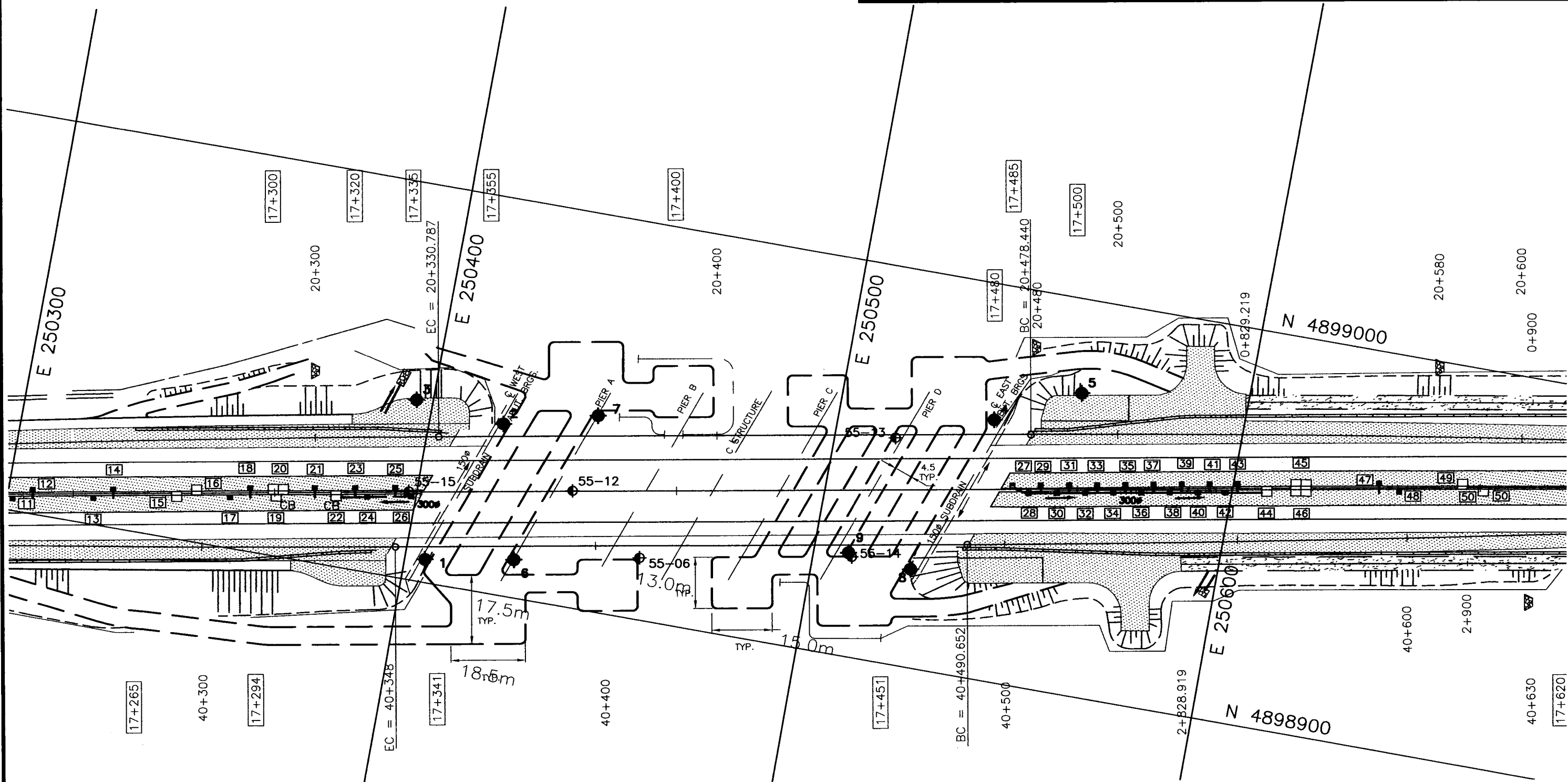
E = elastic modulus

**TABLE 3**  
**SUMMARY OF EMBANKMENT HEIGHTS**  
**CRANE PADS – SALMON RIVER**

<i>Crane Pad Location</i>	<i>Proposed Embankment Average Height (m)</i>	<i>Proposed Embankment Maximum Height (m)</i>
North Side of Pier A	1.2	1.5
South Side of Pier A	1.3	1.6
North Side of Pier B	2.7	3.0
South Side of Pier B	2.5	2.8
North Side of Pier C	1.6	1.9
South Side of Pier C	1.7	2.0
North Side of Pier D	1.4	1.7
South Side of Pier D	1.2	1.5

ACCESS ROADS / CRANE PAD LOCATIONS  
SALMON RIVER BRIDGE

FIGURE 1



LEGEND

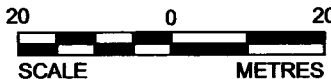
- Borehole - Current Investigation
- ⊕ Borehole - Previous Investigation
- - - Access roads/crane pad

NOTE:

For details at borehole locations and stratigraphic sections, refer to report titled "Foundation Investigation and Design, Rehabilitation of HWY 401, Salmon River Bridge, Site No. 11-207, G.W.P. 82-98-00", Dated August, 2003.

REFERENCE:

Baseplan provided in digital format by Lea Associates Ltd. Drawing titled 2330-C03.dwg, received December 5, 2003.



DATE: MARCH, 2004

PROJECT: 04-1111-006

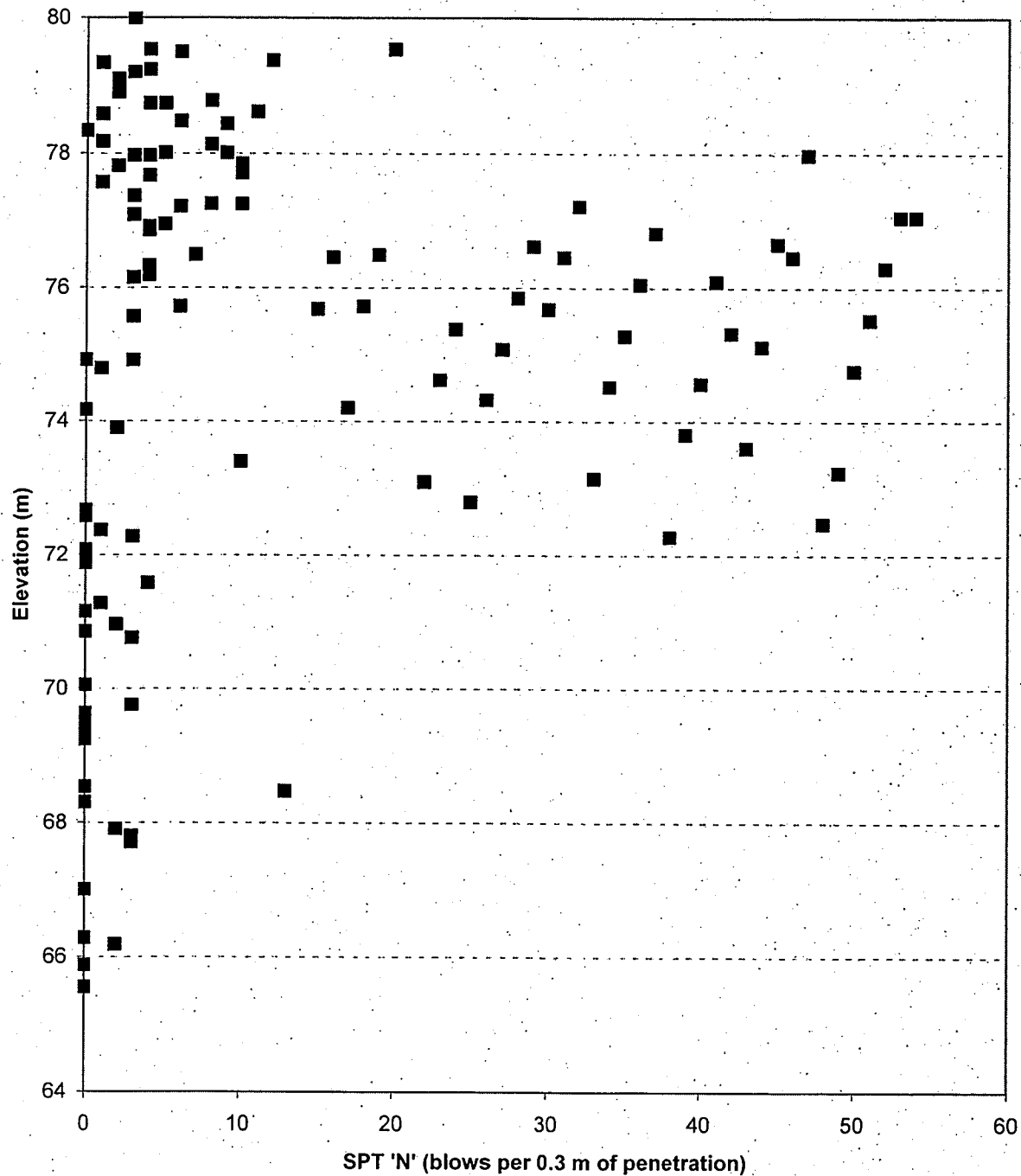


CAD: JDR

CHK: SEP

**STANDARD PENETRATION TEST 'N' VALUES  
SALMON RIVER BRIDGE OVER HWY 401  
TEMPORARY WETLAND ACCESS ROADS**

**FIGURE 2**



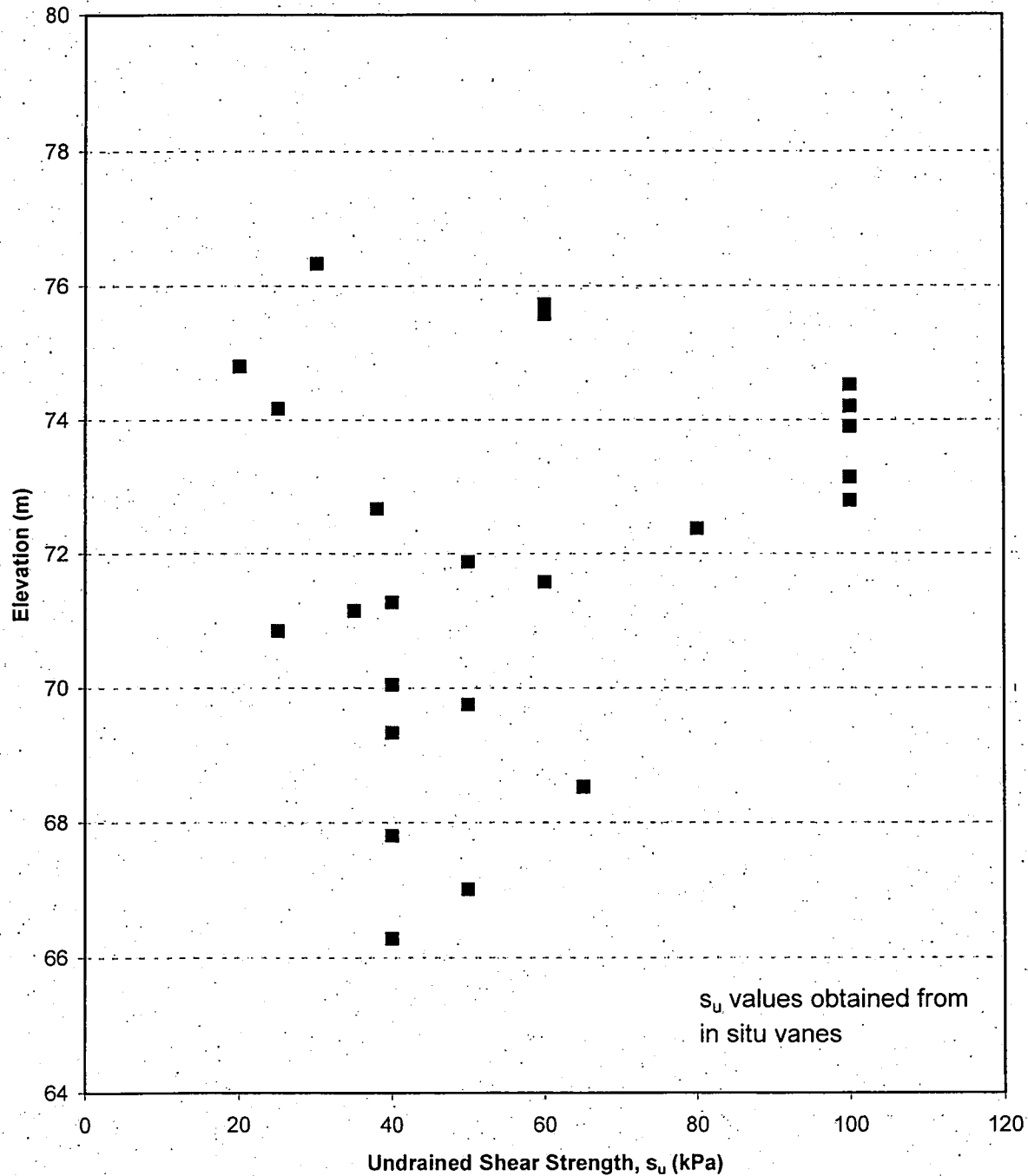
Drawn: SEP  
Checked: FJH

Project No.: 04-1111-006  
Date: March 2004

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**UNDRAINED SHEAR STRENGTH  
SALMON RIVER BRIDGE OVER HWY 401  
TEMPORARY WETLAND ACCESS ROADS**

**FIGURE 3**



Drawn: SEP  
Checked: FJH

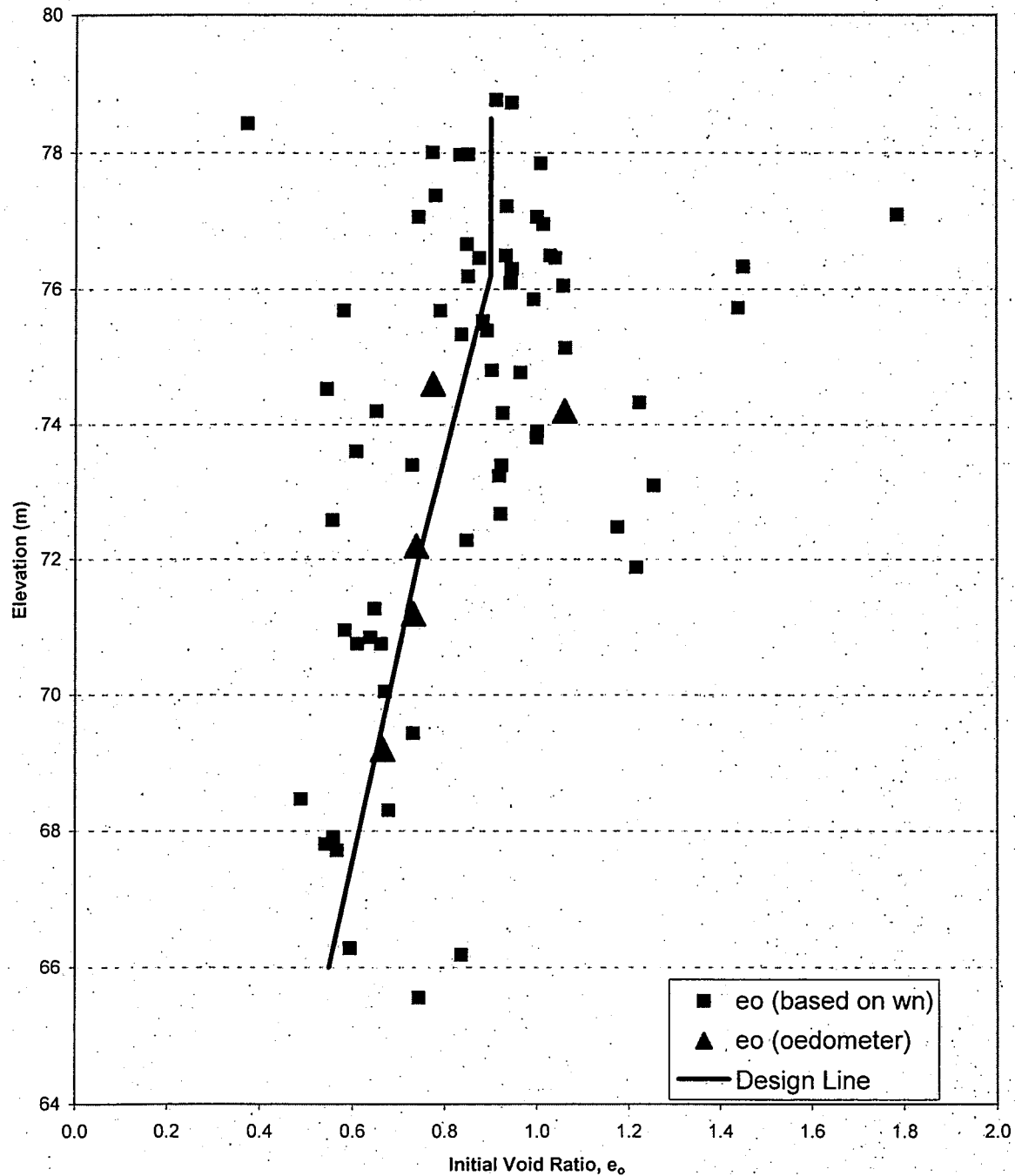
Project No.: 04-1111-006  
Date: March 2004

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# VOID RATIO

SALMON RIVER BRIDGE OVER HWY 401  
TEMPORARY WETLAND ACCESS ROADS

FIGURE 4



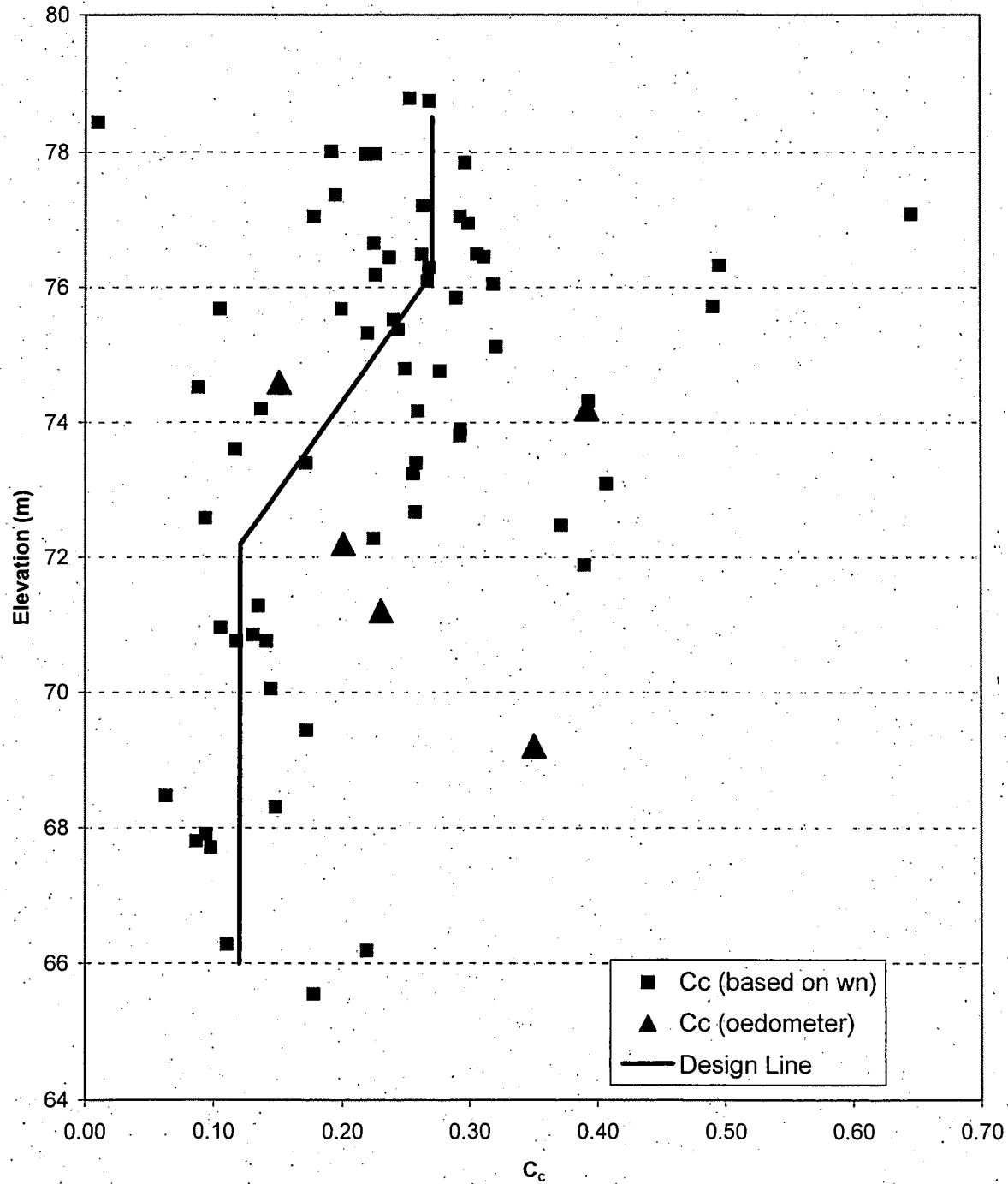
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Checked: FJH

Project No.: 04-1111-006  
Date: March 2004

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**COMPRESSION INDEX**  
**SALMON RIVER BRIDGE OVER HWY 401**  
**TEMPORARY WETLAND ACCESS ROADS**

**FIGURE 5**



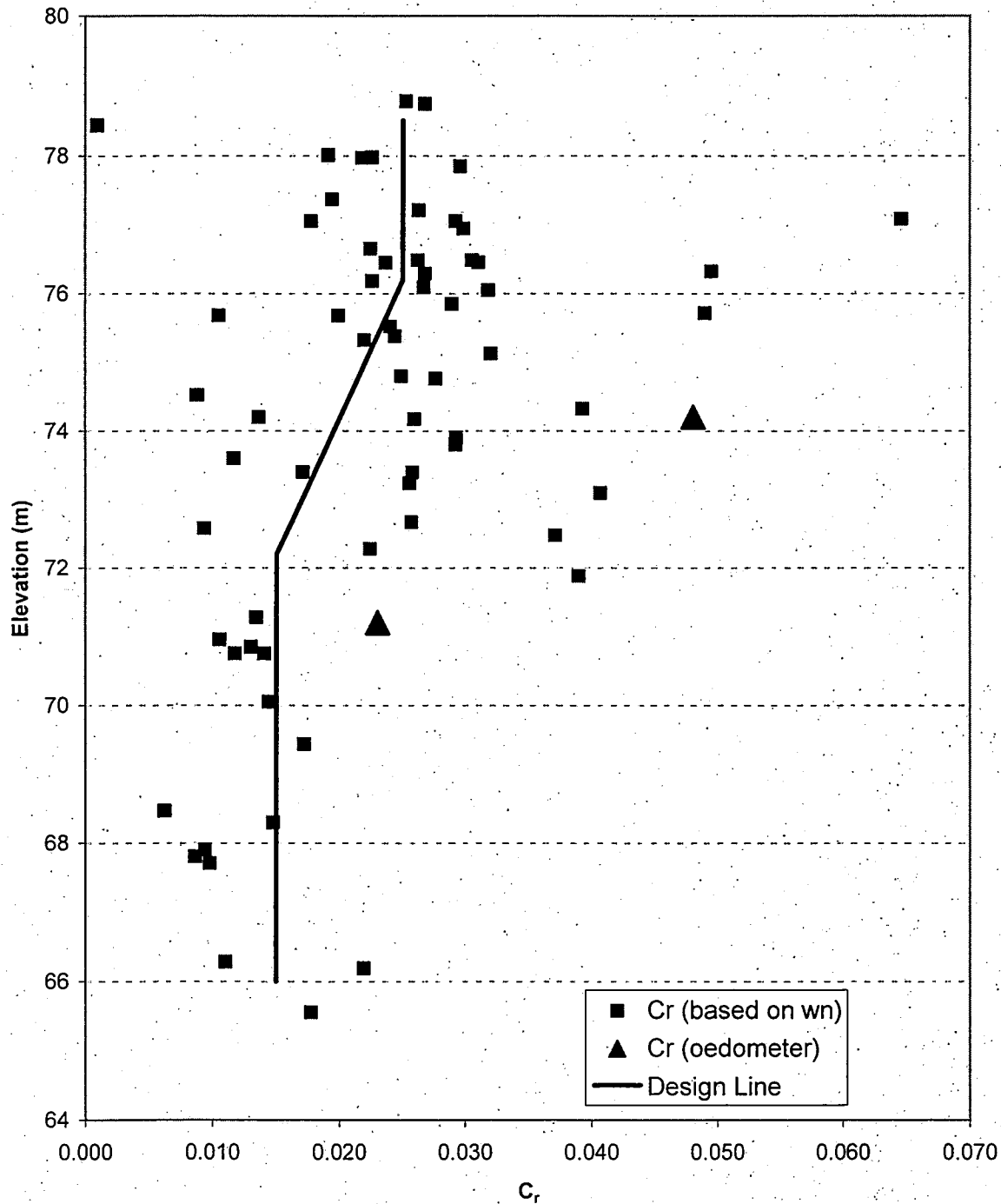
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 Checked: FJH

Project No.: 04-1111-006  
 Date: March 2004

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**RECOMPRESSION INDEX**  
**SALMON RIVER BRIDGE OVER HWY 401**  
**TEMPORARY WETLAND ACCESS ROADS**

**FIGURE 6**



Drawn: SEP  
 Checked: FJH

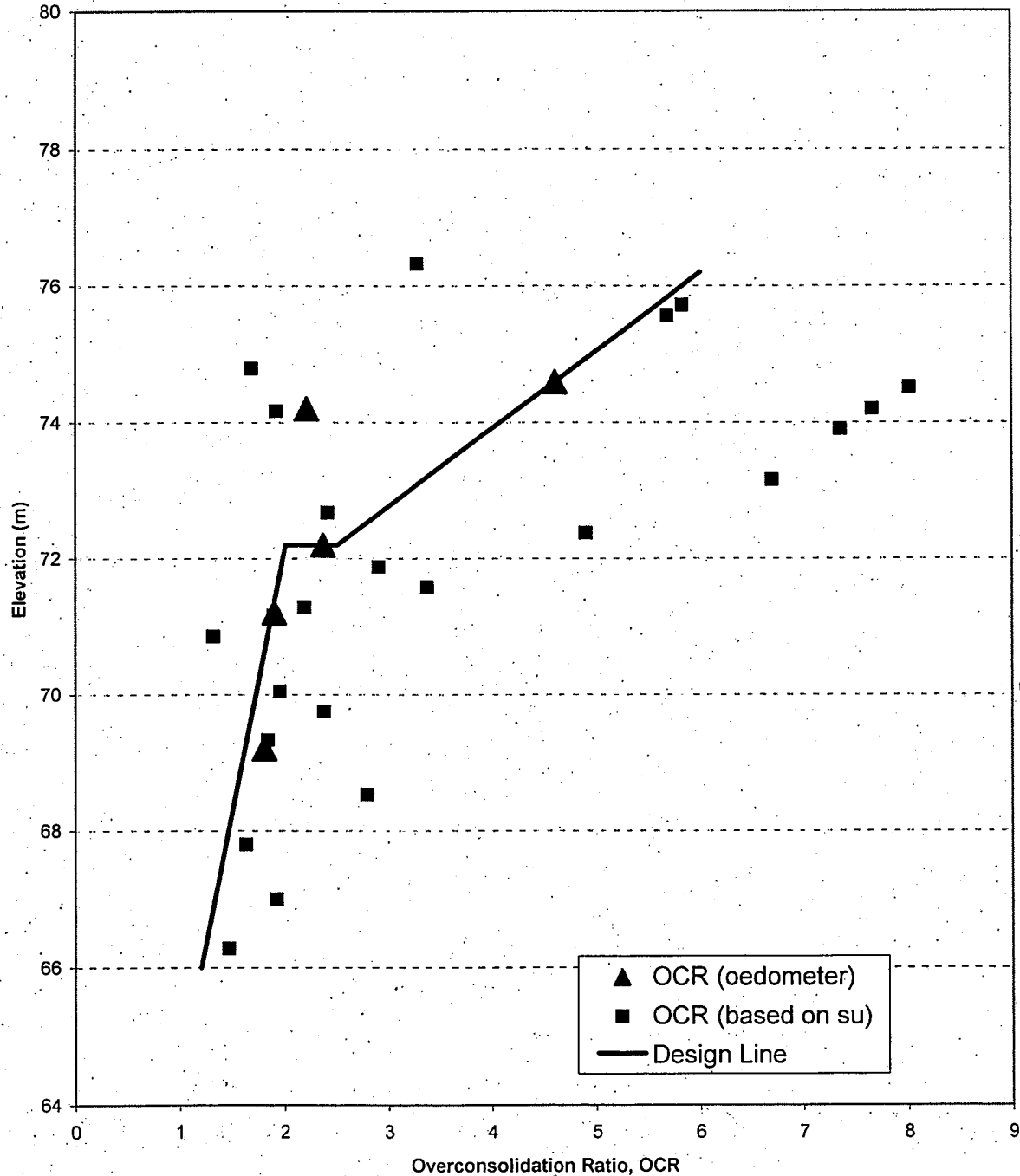
Project No.: 04-1111-006  
 Date: March 2004

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**OVERCONSOLIDATION RATIO  
SALMON RIVER BRIDGE OVER HWY 401  
TEMPORARY WETLAND ACCESS ROADS**

**FIGURE 7**



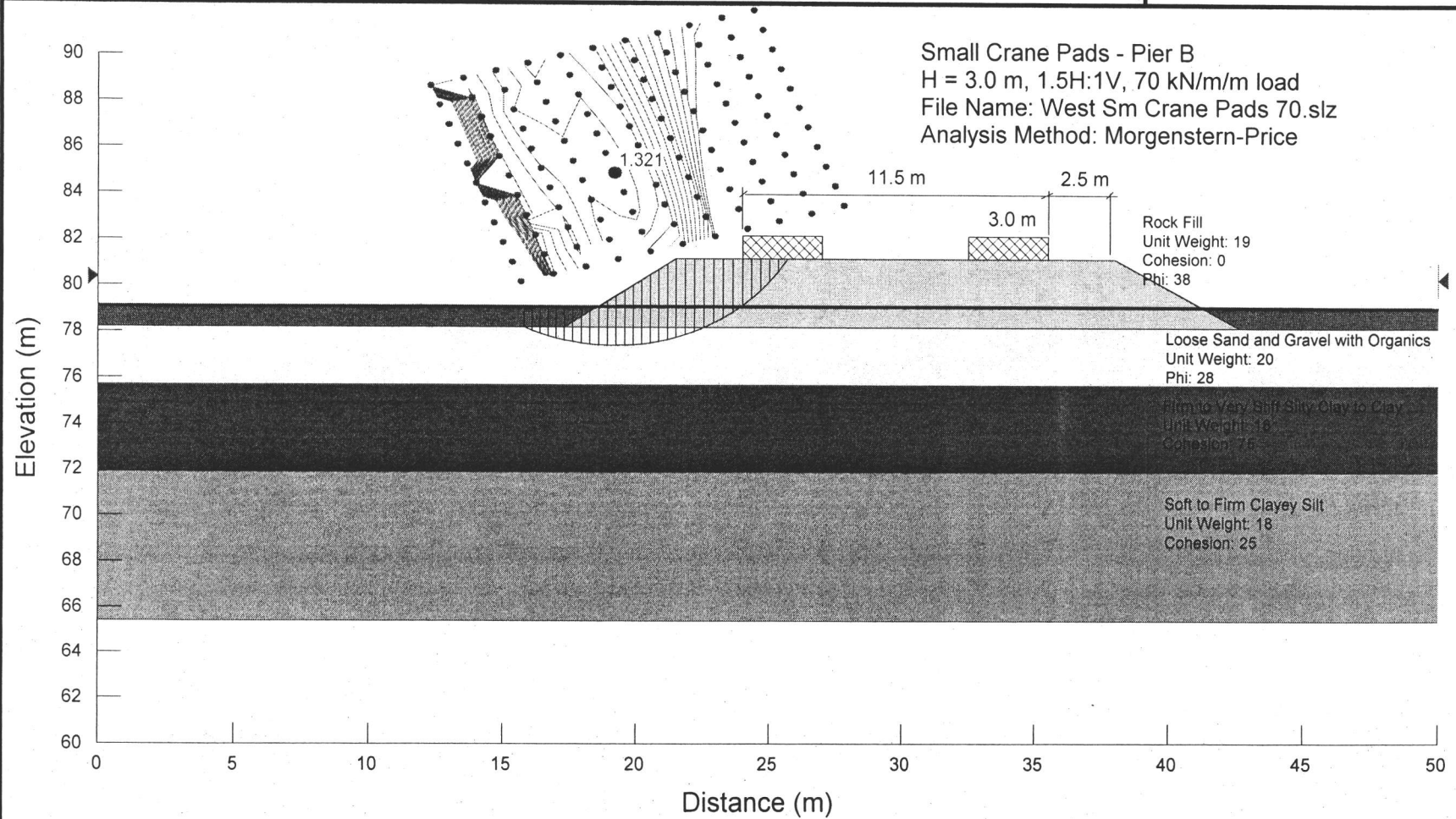
Drawn: SEP  
Checked: FJH

Project No.: 04-1111-006  
Date: March 2004

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**STABILITY SETBACK ANALYSIS - SMALL CRANE PAD**  
**SALMON RIVER BRIDGE OVER HWY 401**  
**TEMPORARY WETLAND ACCESS ROADS**

**FIGURE 8**



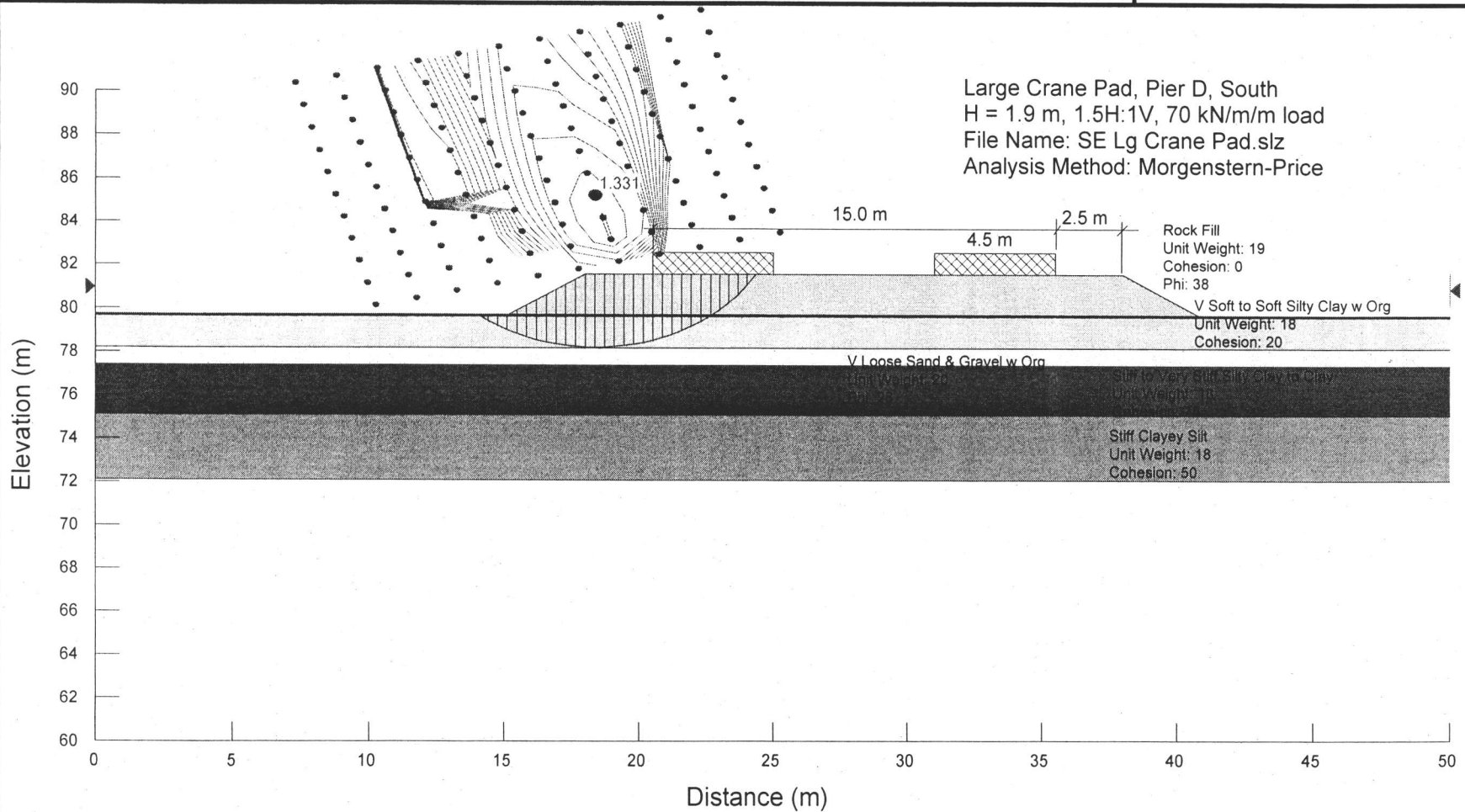
Drawn: SEP  
Checked: FJH

Project No.: 04-1111-006  
Date: March 2004

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**STABILITY SETBACK ANALYSIS - LARGE CRANE PAD  
SALMON RIVER BRIDGE OVER HWY 401  
TEMPORARY WETLAND ACCESS ROADS**

**FIGURE 9**



Drawn: SEP  
Checked: FJH

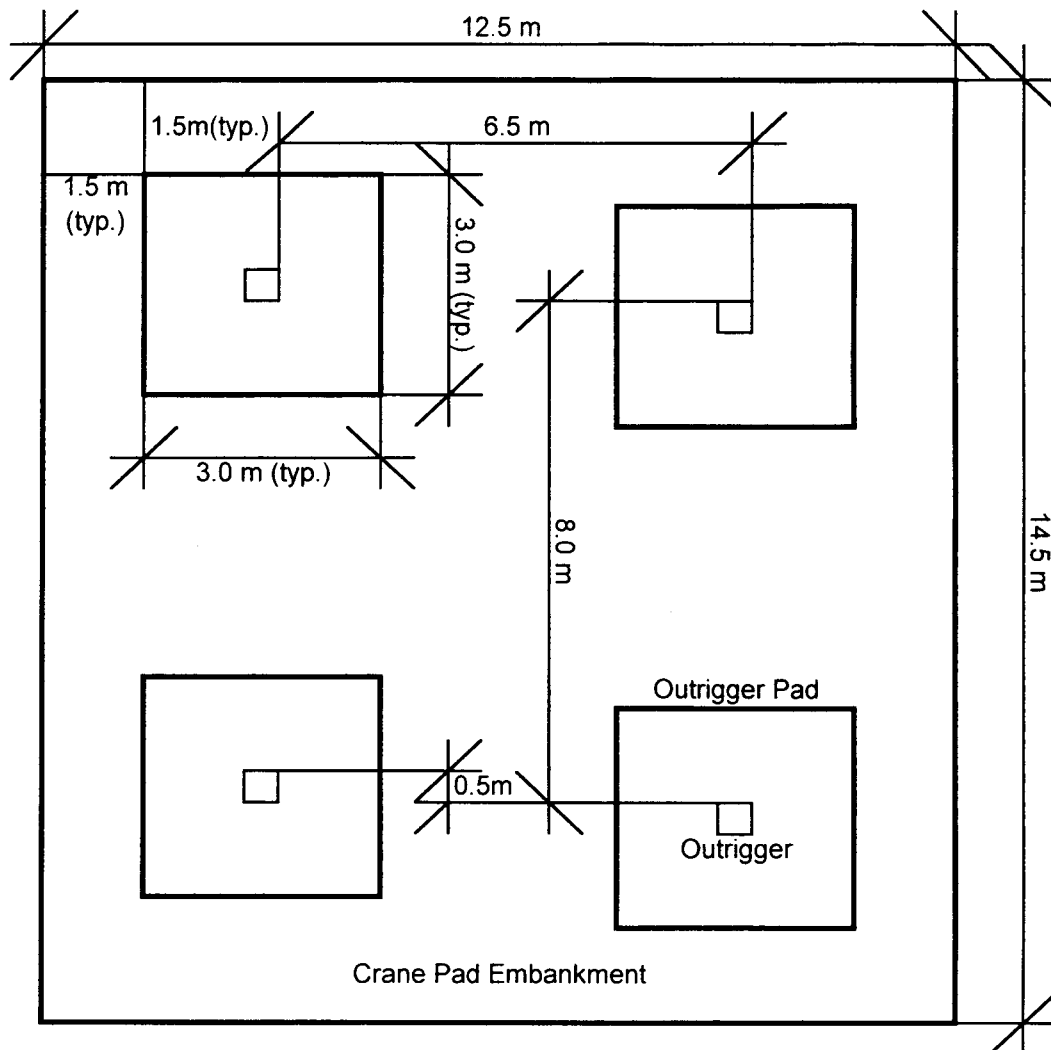
Project No.: 04-1111-006  
Date: March 2004

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**APPENDIX A**  
**CRANE DATA**

**SMALL CRANE PAD LAYOUT  
SALMON RIVER BRIDGE OVER HIGHWAY 401  
TEMPORARY WETLAND ACCESS ROADS**

**FIGURE A-1**



Estimated Load per outrigger pad/grillage (provided by LEA) = 520 kN  
Estimated stress per outrigger pad/grillage = 58 kPa

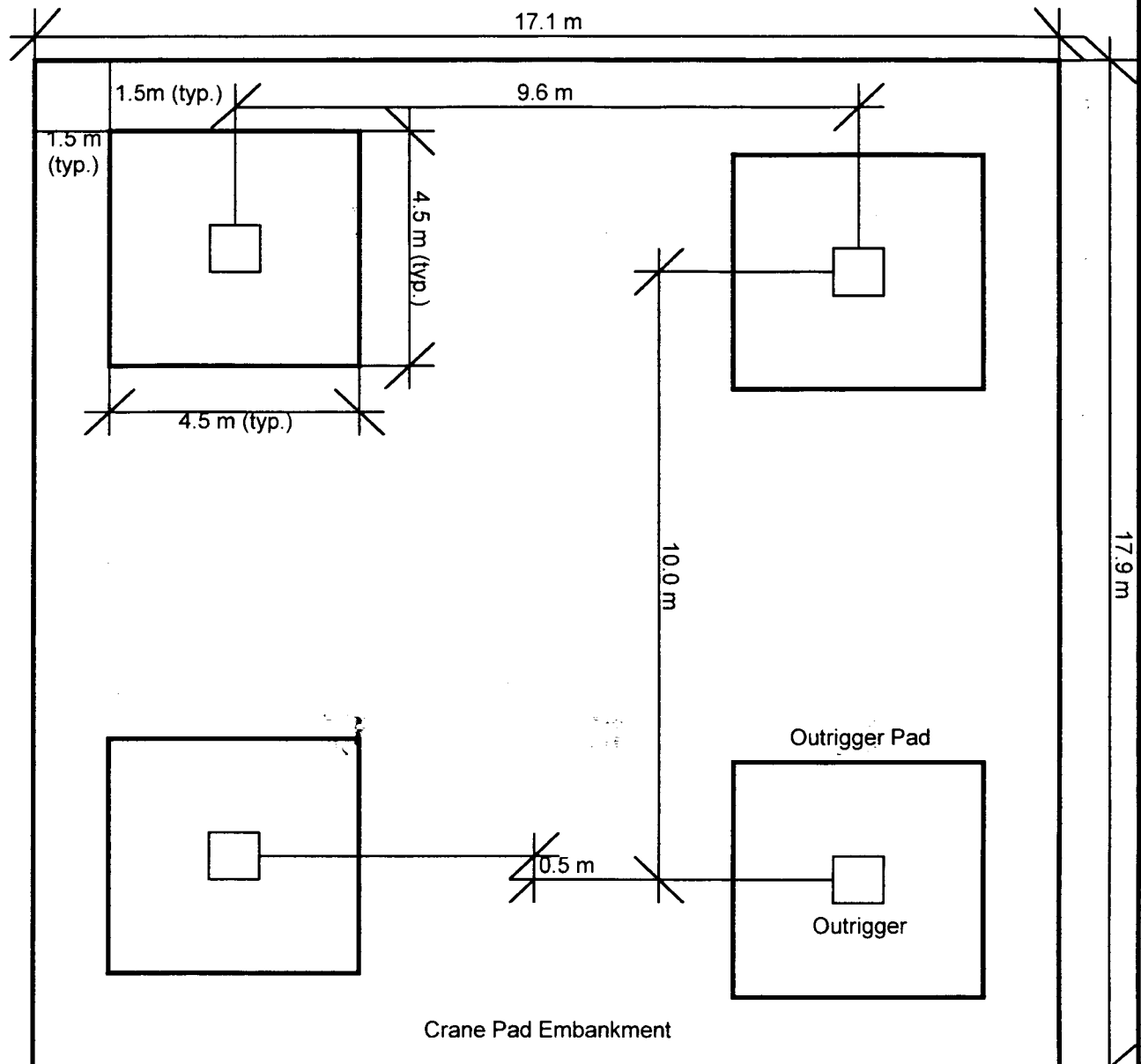
Drawn: SEP  
Checked: FJH

Project No.: 04-1111-006  
Date: March 2004

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**LARGE CRANE PAD LAYOUT**  
**SALMON RIVER BRIDGE OVER HIGHWAY 401**  
**TEMPORARY WETLAND ACCESS ROADS**

**FIGURE A-2**



Estimated Load per outrigger pad/grillage (provided by LEA)  
 Maximum load per outrigger pad/grillage (provided by LEA)

= 1100 kN  
 = 1300 kN

Estimated stress per outrigger pad/grillage  
 Maximum stress per outrigger pad/grillage

= 54 kPa  
 = 64 kPa

Drawn: SEP  
 Checked: FJH

Project No.: 04-1111-006  
 Date: March 2004

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