




## TECHNICAL MEMORANDUM

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**Subject Windsor Essex Parkway Project**  
**Bridge B-15: Preliminary 60% Geotechnical Design – Rev.0**

Revision History					
Revision	Date	Status	Prepared By	Reviewed By	Approved By
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### Drawings (to be inserted in the final report)

- 285380-04-090-SEG0-0018 Location Plan and Profile Sta. 11+500T to Sta. 12+300T (Tecumseh)
- 285380-04-091-SEG1-0128 Location Plan and Sections at Bridge B-15
- 285380-04-091-SEG1-0129 Stratigraphic Sections at Bridge B-15

### Attachments –Existing Borehole Logs (Appendix A)

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## 1.0 INTRODUCTION

This memo provides preliminary 60% geotechnical design for the 2-span Bridge B-15 structure (North Talbot Road underpass). Bridge B-15 will replace the existing North Talbot Road bridge located between North Talbot Road Stations 9+970.5 and 10+029.5 (approximately between Highway 401 Stations 12+100T and 12+300T (Tecumseh), as shown on Figure 1<sup>1</sup>.

The WEMG proposal design for Bridge B-15 incorporated true integral abutments and a centre pier founded on deep end bearing piles as shown in Figure 2. The WEMG proposal design has been accepted as 30% preliminary design. As no geotechnical investigation was carried out to develop the 30% bid designs and no site specific additional investigation has yet been carried out at Bridge B-15 site, the 60% design presented here is based on limited historical investigation information available in the general area away from the structure. In this regard, the soil data interpretations, design assessments and recommendations given hereafter for the Bridge B-15 location are considered preliminary and subject to revision at a later stage when the soil and groundwater data are updated following completion of the proposed additional investigation.

Figure 3 shows the investigation by boreholes and static cone tests (CPT) program being proposed for the design of this structure.

Based on available as-built information, the existing bridge is a one span structure supported on rigid abutments founded on conventional shallow foundation<sup>2</sup>. The approach way embankments are 7.6 m high above the surrounding grades. Based on the available as-built information, it is considered that Bridge B-15 construction will involve the following earthwork, design elements and loading stages:

- Temporary excavations cutting back about 8.5 m below North Talbot Road grade within the north and south embankments;
- Temporary excavations to about 3 m below existing grade for the central bridge piers;
- Installation of piles (HP310x110) for all bridge supports driven to mobilize a ULS factored axial capacity of 2000 kN;
- Installation of 600 mm diameter Corrugated Steel Pipe (CSP) around the abutment pile stickups and filling them with loose dry sand;
- Construction of bridge abutment and deck;
- Construction of Retained Soil System (RSS) wing walls, including provision of reinforced Granular Mat (RGM), if required (subject to condition of the existing fill at the base of the excavation) for the high wing wall segments;
- Completion of drainage works and backfill behind the abutment and wing walls; and

<sup>1</sup> Figures provided at the end of this memorandum text.

<sup>2</sup> Dillon, 1955. Sandwich South Twp Underpass Drawings.

- Completion of the pavements over North Talbot Road and Highway 401.

## 2.0 SIMPLIFIED SUBSURFACE CONDITIONS AND DESIGN PARAMETERS

- No test holes were put down at or within the general vicinity of Bridge B-15 site during the recent 2009 and 2010 investigations carried out by Golder for proposal designs.
- The approximate stratigraphy of the area was based on historic reports<sup>3</sup>, which only shows one borehole numbered BH215-N (log provided in Appendix A) in the vicinity of Bridge B-15. Borehole BH215-N is a shallow hole put down during the investigation for the design of noise barrier walls and is located approximately 50 m west of the north abutment. Additional subsurface information for this site was based on the inferred stratigraphy provided for the North Talbot Road Bridge (referred to as site 6-068), which is also outlined here<sup>4</sup>:

Item		Value
Ground Surface Elevation <sup>5</sup>		191
Groundwater Level Elevation		190.5
Bedrock Surface Elevation		151
Information on Encountered Soil Strata		
Crust	Elevation	190 to 185
	Thickness (m)	5.5
	W <sub>N</sub> (%)	12 to 15
	SPT (N-values)	25
Silty Clay to Clayey Silt (including Transition Layer)	Elevation	185 to 162
	Thickness (m)	23.5
	W <sub>N</sub> , W <sub>L</sub> , W <sub>P</sub> , PI, LI (%)	18, 15, 25, 10, 0.3
	SPT (N-values)	11
Dense Granular Deposit	Elevation	162 to 151
	Thickness (m)	11.2
	W <sub>N</sub> (%)	18
	SPT (N-values)	37

- The successive native soil strata encountered at Bridge B-15 site comprised a 4 to 5 m thick mottled grey brown desiccated clayey silt crust, about 2 to 3 m of a clayey silt transition layer and then an extensive stiff grey silty clay to clayey silt deposit. The grey silty clay stratum was underlain by a stiff clayey silt with sand and gravel traces (referred

<sup>3</sup> Golder Associates, 2009. Foundation Investigation and Design of Noise Barrier Walls, Geocres 40J2-114; Dillon, 1955. Sandwich South Twp Underpass Drawings.

<sup>4</sup> Golder Associates, 2006. Structure Settlement Study – Highway 401 Reconstruction, Geocres 40J2-79

<sup>5</sup> All elevations given in this memo and accompanying drawings and figures are referred to geodetic datum and are in metres.

to as lower clay between elevations 167 m and 162 m) and then granular till deposits to bedrock at about elevation 151 m.

- Nilcon vane and CPT data were not available for Bridge B-15 area. SPT N-values, limited conventional field vane test results and the general trend in the eastern sector of the WEP were used to estimate a preliminary  $S_u$  profile for the site. The design  $S_u$  variation with depth for the grey silty clay stratum was from about 75 to 60 kPa for the crust and transition layers, about 60 kPa for the upper clay, and increasing with depth from 60 to 70 kPa for the lower clay.
- The piezometric level reported within the silty clay stratum was close to the original ground.
- Other relevant soil properties required for various analyses are provided in relevant sections.

### **3.0 DESIGN CRITERIA AND METHODOLOGY**

- The design criteria employed for all geotechnical and foundation designs will respect the criteria set out in Project Agreement (PA) – Schedule 15-2, Part 2, Article 5 and the AMEC's Geotechnical Design Criteria memorandum dated February 25, 2011.
- Foundation designs were based on methods and requirements outlined in Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC) LRFD method for ULS and SLS. Canadian Foundation Engineering Manual (CEFM), Ministry of Transportation (MTO)'s guidelines and Ontario Provincial Standard Specifications (OPSS) were also used as required.
- Slope stability analyses (Limit Equilibrium) were carried out using SLOPE/W Version 2007, adopting the Morgenstern-Price method of analysis and circular failure surfaces.

### **4.0 EXCAVATION AND TEMPORARY CUT SLOPES**

- Excavations are expected to encounter surficial granular soils and compacted clay fill (from the existing approachway embankments), and will be extended 8.5 m below existing grade at elevation 198.5 m to about elevation 190 m into the native stiff clayey silt to silty clay.
- All excavation works should be carried out in accordance with the guidelines outlined in Occupational Health and Safety Act (OHSA) and Ontario Provincial Standard Specification (OPSS) 902. The assumed compacted clay fill may be classified as Type 3 soils. The excavations below the original ground levels may intersect water bearing backfill within trenches of active and/or abandoned utilities. In these cases, Type 4 soil conditions may occur and should be addressed accordingly.
- The complete excavation for Highway 401 does not need to be advanced to the roadway subgrade within the same excavation operation as for the abutment walls. For the present design purposes, the bulk of the general excavation is assumed to be conducted

close to the profile shown on Figure 4. Consideration should also be given to the interactions between abutments and excavation for Highway 401.

- Minor seepage from runoff infiltrations is anticipated which should be controllable by conventional temporary dewatering methods.
- The calculated minimum factor of safety (FS) for 8.5 m deep temporary excavation at 1H:1V slope for the typical abutment section with the slope profile and assumed soil properties shown in Figure 5 was 1.87. This analysis assumed that the excavation for Highway 401 was also completed. However, should the excavations required for construction of Highway 401 sub-grade not commence prior to the completion of the abutment this would result in a higher FS.
- The recommendations provided herein are based on the assumptions that (1) the temporary slopes are properly protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, and (2) the duration of the slope exposure is in general limited to 4 to 5 months.
- To protect the subgrade integrity, the final excavation lift above the design elevation should not be less than 500 mm and should be carried out only when the contractor is ready to prepare and cover the subgrade with the materials specified in the design same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- As the soil below the excavation profile has been loaded previously by the existing bridge embankment, the heave upon completion of the excavation is considered minor.
- Periodic inspection of the condition of the temporary slopes should be carried out by qualified personnel.

## **5.0 DEEP FOUNDATIONS**

- It is understood that HP310x110 steel H piles driven to competent foundation material to mobilize a target ULS capacity of 2000 kN are being considered for the bridge foundations. Based on historic reports available to date at this site, the tips of piles are anticipated to be set at about elevation 151 m.
- The actual pile capacity should be confirmed by static load tests at strategic locations in conjunction with testing using Pile Driving Analyzer (PDA). The static load tests will facilitate proper calibration of the PDA, pile driving equipment performances and determine the appropriate driving criteria (set).

In cases that some of the piles cannot be advanced to the Specified Ultimate Resistance (SUR) due to perceived risk of damaging the piles, consideration should be given to supplement the field testing to prove the actual mobilized resistance and / or alternative options based on the most economical approaches (changes to the driving equipment / method, addition of more piles) may be considered.

- The steel H piles should be installed and monitored in accordance with OPSD 3000.150 and OPSS 903 standards. The piles should be reinforced with Type I shoe flanges as



shown in OPSD 3000.100. Alternatively, approved commercially available shoes (Titus points, or equivalent) may be considered.

- Unless the piles are set on bedrock, provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven.
- While unlikely to occur at this site, considering the general geologic conditions in the region, indications of natural gas venting, water, and fines washout should be monitored during driving. It is recommended that the pile splicing be completed by butt-welding to reduce the risks of creating pathways for potential upward flow of artesian water along the piles to the surface. The type of driving shoes used should be such that the projections of the driving point beyond the pile shaft section are minimal. Provision to mitigate such occurrences (heavy mud, grouting of the cavities, etc.) should be in place.
- Consideration should be given to potential driving difficulties due to the presence of dense lower granular soils and potential presence of cobbles and boulders above the bedrock.
- Vibrations generated by piling should be monitored. It is not expected that the vibrations during piling will have a significant impact on the stability of temporary slopes. Nonetheless, if the vibration intensities at the toe and top of the slopes exceed 10 mm/s, appropriate mitigation measures (slope flattening or vibration dampening by dumping sand around the piles) should be considered.
- The preliminary horizontal subgrade reaction to the pile can be estimated using the following equation:

$$k_x = n_h (z/d) \quad \text{for cohesionless soils, and}$$

$$= 67 (S_u/d) \quad \text{for cohesive soils.}$$

Where:

$k_x$ (MPa/m)	= Soil modulus of horizontal subgrade reaction
$n_h$ (MPa/m)	= Soil coefficient
$S_u$ (MPa)	= Undrained shear strength
$z$ (m)	= Depth below finished grade
$d$ (m)	= Pile diameter/width

- The recommended ranges of soil parameters are tabulated as follows:

Soils Around the Piles	Elevation Range	$n_h$ (MPa/m)	Undrained Shear Strength, $S_u$
Loose Sand within CSP	-	2 to 5	-
Native Silty Clay Crust	190 to 185	-	0.075 MPa
Native firm to Stiff Silty Clay	185 to 182	-	Decreases linearly with depth from 0.075 to 0.06 MPa
	182 to 167	-	0.06 MPa
Native Stiff Clayey Silt	167 to 162	-	Increases linearly with depth from 0.06 to 0.07 MPa



- Batter piles or inclined piles are considered to be more effective in resisting horizontal loads, as a part of lateral load is converted into axial load and consequently the induced bending moments are less. It should be noted that the centerline to centerline of the piles is in excess of six times the pile diameter and the group effect of the piles will be negligible.

## **6.0 ABUTMENTS**

### **6.1 Global Stability**

- Figure 6 and Figure 7 illustrate the stability models for the typical abutment section with an approximate height of 8.5 m. The global stability analyses have been carried out for both short-term and long-term loading conditions.
- The proposed design for the Bridge B-15 structure incorporated 5 m high pile caps. However, to achieve the minimum required FS ( $FS > 1.3$ ) against global instability of abutments, the base of the pile caps should be extended down to approximate elevation 191 m (7.5 m below the proposed top of the pile abutment).

### **6.2 SLS Performance**

- Since the stress increase in the overburden strata at this site will be negligible, the subgrade settlements caused by the backfill restoration to within the general limits of the existing embankments should be nominal. Accordingly the SLS (ground deformations) is not a factor under these assumptions.
- The ground deformations noted above do not include deformations caused by seasonal temperature and moisture variations. Also, they do not include the effects of the long-term compression of the new backfill materials that may occur further to inadequate compaction.

## **7.0 RSS WING WALLS**

- Use of RSS walls was assumed to retain the backfill behind the bridge wing walls. The proposed design comprises three segments of retaining wall panels of approximately 3.6 m length each and 6.0 m, 4.0 m and 2.0 m heights. Analyses have been carried out in the following sub-sections for the highest and middle RSS segments.

### **7.1 RSS Global Stability**

- The global stability analyses have been carried out for both short-term and long-term loading conditions for the RSS walls based on the available geotechnical information (Section 2.0). The actual internal design of the RSS is to be provided by the RSS supplier, and is beyond the scope of this design memo.

- The models used for the stability analyses of the highest wing walls are shown in Figure 8 and Figure 9 for the short-term and long-term conditions. FS were also calculated for the middle height wing walls.
- All calculated factors of safety are in excess of 1.3 against global instability and meet the minimum factor of safety required by the PA.

## 7.2 ULS Bearing Resistance

- In the case of founding the RSS on approved compacted clay fill, the following gross factored geotechnical resistance values ( $q_u$ ) may be used for the design of the wing wall segments:

Wing Wall	Assumed Subgrade Elevation (m)	Condition	$q_u$ (kPa)
Highest (H 6.0 m)	192.5	Short-Term (Undrained)	150 <sup>(1),(2),(3)</sup>
		Long-Term (Drained)	550 <sup>(2),(3)</sup>
Middle (H 4.0 m)	194.5	Short-Term (Undrained)	130 <sup>(1)</sup>
		Long-Term (Drained)	250

(1) Excludes contribution of adhesion between wall and backfill

(2) Below the RGM, if used

(3) RGM may not be required depending on the material and quality of the compacted fill used to restore the foundation grade

- The above resistances are applicable in conjunction with the specific RSS wing wall configurations and sizes described below, and for load inclinations of up to 12%.
- The overall dimensions and makeup of the wing walls at this site have been checked for the following Loading Combinations:
  - SLS Combination (1D+1E+0.9LL)
  - ULS Combination 1a (1.25D+1.25E+1.7LL)
  - ULS Combination 1b (0.8D+1.25E)
  - ULS Combination 9 (1.35D+1.25E)

Where:

D – dead loads (based on an average unit weight of 21 kN/m<sup>3</sup>)

E – Earth pressures

LL – Live Loads (assumed 12 kPa uniformly distributed)

- The following total wing wall dimensions were determined to meet the most severe of the above conditions:

Wing Wall	Total Height, (m) <sup>(1)</sup>	RGM Size, thickness x width (m) <sup>(3)</sup>	RSS Structure Size, width x height (m) <sup>(2)</sup>
Highest (H 6.0 m)	6	1.0 x 6.5	4.5 x 6.0
Middle (H 4.0 m)	4	Not Required	5.5 x 4.0

- (1) Measured from top of finished pavement (198.5 m) to the base of the RSS structure  
 (2) The RSS supplier may require wider structures to meet the internal design requirement.  
 (3) The need for RGM depends on the condition and quality of the supporting fill

## 8.0 RGM FOUNDATION

- A 1.0 m thick, 6.5 m wide, RGM, or equivalent, was considered necessary under the highest wing wall (6.0 m) to meet the ULS bearing capacity requirements for undrained conditions. The RGM may be eliminated if the fill used to restore the foundation grade and prepare the foundation for the wing wall panel consists of 2 m thick Granular B Type 2 compacted to 100%.
- The soil at the RGM base is assumed to be compacted clay fill. The properties used for the compacted clay fill were those defined for the Global Stability analyses:

Unit weight for Clay Fill	21 kN/m <sup>3</sup>
Undrained Strength of Clay Fill	50 kPa
Drained Internal Friction Angle of Clay Fill	30°

- The following loads were estimated to act on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls.

Loading Condition	SLS Stresses (kPa) <sup>(1)</sup>		Max. ULS Stresses (kPa) <sup>(2)</sup>
End of Construction	225	50	215
Long-Term	220	75	230

- (1) SLS load combination (1D+1E+0.9LL) as per CHBDC  
 (2) ULS-1a load combination (1.25D+1.25E+1.7LL) was determined to be the most critical.

## 9.0 BACKFILLING

- Behind the concrete abutment and wing walls, non-frost susceptible and free draining granular fill should be placed in accordance with the CHBDC. Alternatively, a synthetic insulation with drainage blanket and site generated clay fill behind the walls may be considered.
- The backfill should be compacted in maximum 200 mm thick loose lifts in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the backfill. Other aspects of the abutment backfill requirements

with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.

- Sub-drainage should be provided if clay backfill is used between the back of the abutment and the excavation slope face. Alternatively, free draining sand and gravel fill (Granular B Type I, or approved equivalent) may be used for backfill behind the abutment, which will ensure good long-term drainage and keep the phreatic surface low.
- Heavy compaction equipment should not be used immediately adjacent to the walls of the structure. Effects of backfill compaction activities should be simulated as live load over and above the static lateral earth pressure for structural design in accordance with the CHBDC.
- Earth pressures on abutment and wing walls may be calculated on the basis of the following parameters:

Parameter	Group I Soils (*)	Group II Soils(*)	Group III Soils (*)
Fill unit weight (kN/m <sup>3</sup> )	22	21	20.5
Coefficients of static lateral earth pressure:			
‘active’ or unrestrained, Ka	0.27-0.30	0.30-0.35	0.35-0.45
‘at-rest’ or restrained, Ko	0.45-0.50	0.50-0.55	0.60-0.70
‘passive’, Kp	3.3 – 3.7	2.8 – 3.3	2.2 – 2.8

(\*): Compacted to greater than 95% Standard Proctor Maximum Dry Density.

Group I Soils: Coarse grained soils (e.g. Granular A and B Type 2)

Group II Soils: Finer grained than Group I non-cohesive soils (e.g. Granular B Type1, pit run, etc)

Group III Soils: Finer grained soils (e.g. approved site generated silty clay)

## 10.0 DEWATERING

- The design of the dewatering system should comply with the OPSS 517 and 518 provisions.
- Further details of permanent dewatering needs will be determined when additional soil information becomes available for this particular bridge site.

## 11.0 TECHNICAL APPRAISAL FORM (TAF) INSERTS

### 11.1 Design/Assessment Criteria

- The designs were developed as per the Project Agreement – Schedule 15-2, Part 2 – Design and Construction Requirements, Article 5.
- The foundations’ designs are as per the principles of Limit States Design (LS Method) based on Load and Resistance Factors (CHBDC and CFEM).
- Working Stress Design (WS Method) is employed for global stability of the false abutment foundations and/or earthworks.

- Deep foundations are designed to meet or exceed the requirements of MTO's Structural Manual and OPSS 903 of 2009.
- All piles at this project are designed as end-bearing piles generally on bedrock.
- The design pile capacities (axial and lateral loads) will be ensured by suitable driving equipment and procedures.
- The stability of the soil mass containing the retaining wall on piles was checked for all potential surfaces of sliding and to have a minimum factor of safety exceeding 1.3.
- The face batter of the permanent retaining walls will not be steeper than 1H:24V.
- Long-term creep is not a factor since there are no soil stress increases at this structure.

## **11.2 Ground Conditions**

- The soil and groundwater conditions for the current design were based on data from boreholes located far from the bridge site. The soil stratigraphic conditions and soil properties will be interpreted from the results of the 2011 additional investigation to be carried out by AMEC.
- The soil conditions and design parameters will be based on investigation data at the structure location with due consideration for the data in the vicinity.
- As noted in Section 1.0, the geotechnical analyses and design recommendations provided in this memo are preliminary and are subject to change based on interpretation of the updated soil data (combined results of the previous and proposed additional geotechnical investigations).
- Details of geotechnical investigation proposed at Bridge B-15 location to validate basis of design/assessment are listed below:
  - Boreholes B15-1, B15-2 and B15-3;
  - Nilcon Vane Test B15-1; and
  - Core Penetration Tests CPT B15-1 and B15-2.

## **11.3 Description of Foundations**

- Structural foundations are designed on end bearing HP310x110 piles driven to adequate bearing strata using an ULS capacity of 2000 kN. The design capacity and associated driving criteria will be confirmed by load tests and PDA. Driving Refusal (blows/25 mm) and Hiley charts will be developed and calibrated with the static load tests and PDA.
- SLS resistance to vertical loads is not an issue since the bedrock is anticipated to not yield under the ultimate loads. Hence the pile axial deformations should be comparable with the elastic compression of the pile shaft (less than 29 mm for a 40 m long shaft loaded to an estimated SLS = 1400 kN).

- Lateral pile response and axial stress increase due to soil stress increase from approachway fill was considered negligible.

#### **11.4 Groundwater**

- The corrosion potential will be tested and, if required, appropriate mitigation measures will be considered (cathodic protection, sacrificial steel thickness, etc). Elevated content of H<sub>2</sub>S in the groundwater is anticipated.

#### **11.5 Ground Settlement due to Embankment Loading**

- Total post-construction settlement considered negligible due to the fact that the final height of the bridge abutment is equal to the height of the existing bridge.

#### **11.6 Allowable Differential Settlement at Expiry Date**

- 5 mm to 100 mm measured at distances from the back of the abutment stub from 0 m to 100 m at the Expiry Date.

#### **11.7 List of Drawings**

DRAWINGS (to be inserted in the final report):

- 285380-04-090-SEG0-0018 Location Plan and Profile Sta. Tecumseh 12+000 to Sta. 12+400
- 285380-04-091-SEG2-0112 Location Plan and Sections at Bridge B-15
- 285380-04-091-SEG2-0113 Stratigraphic Sections at Bridge B-15

Attachments:

Figures and Existing Borehole Log

SF/dd/nsv

W:\SW8801 - Design Engineering Services\1002 - Geotech Design\Mississauga Group\Bridge B-15\60% Memo\WEP-0000-MEM-rev0-SW8801-2011\_05\_06-Bridge B-15 Preliminary 60% Geotechnical Recommendations\_SF20110504.docx

## FIGURES



Construction North

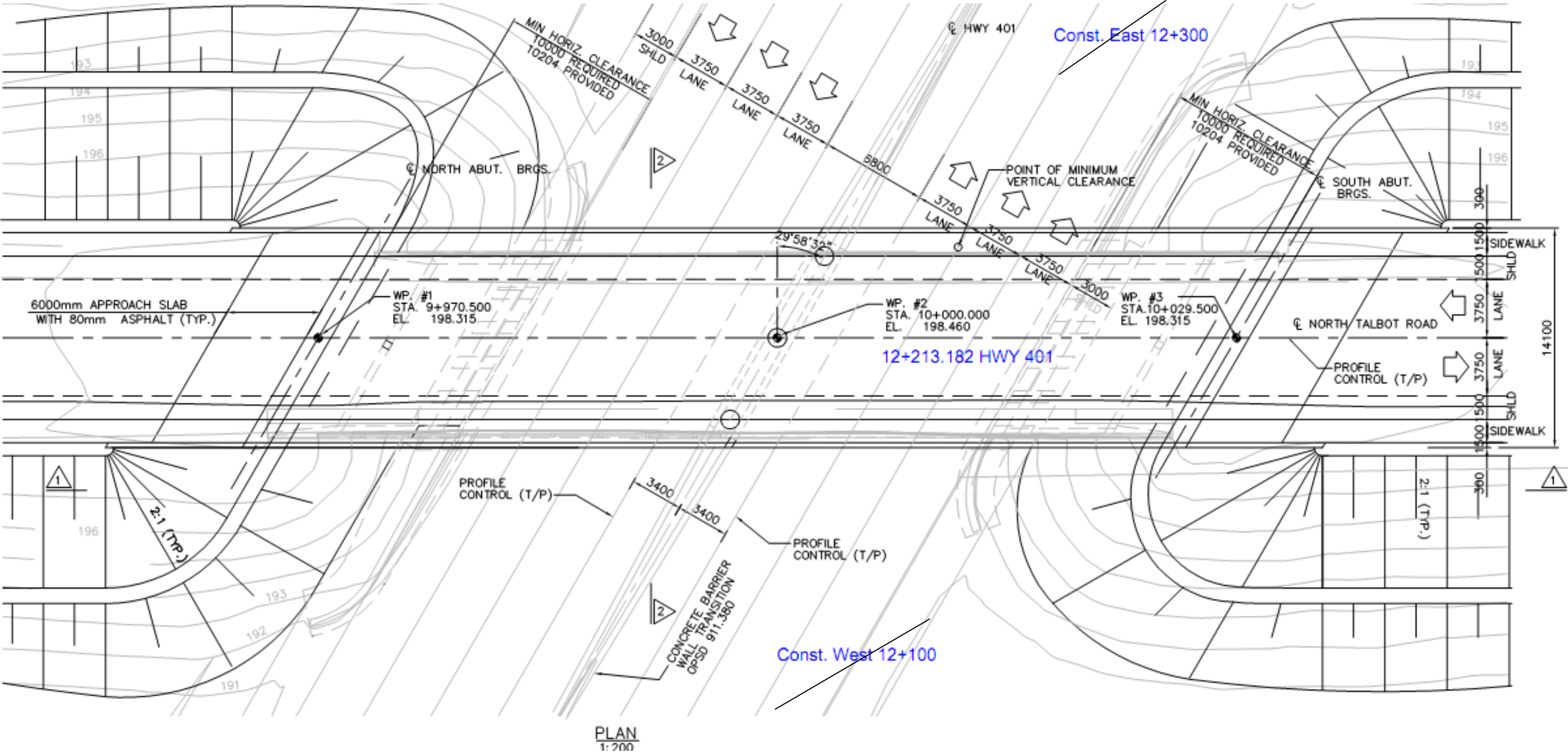


Figure 1: General Plan View at Bridge B-15 Site

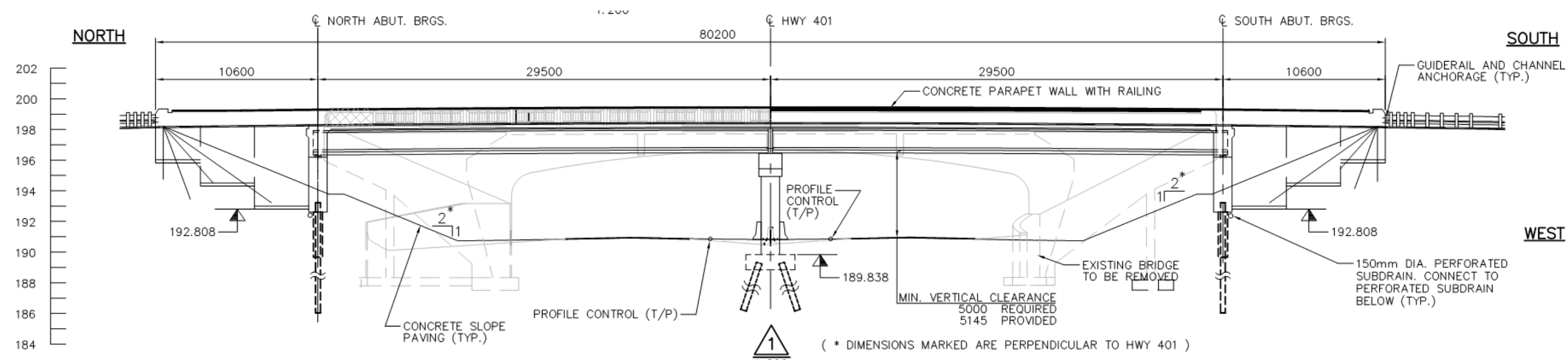


Figure 2: General Profile for Bridge B-15

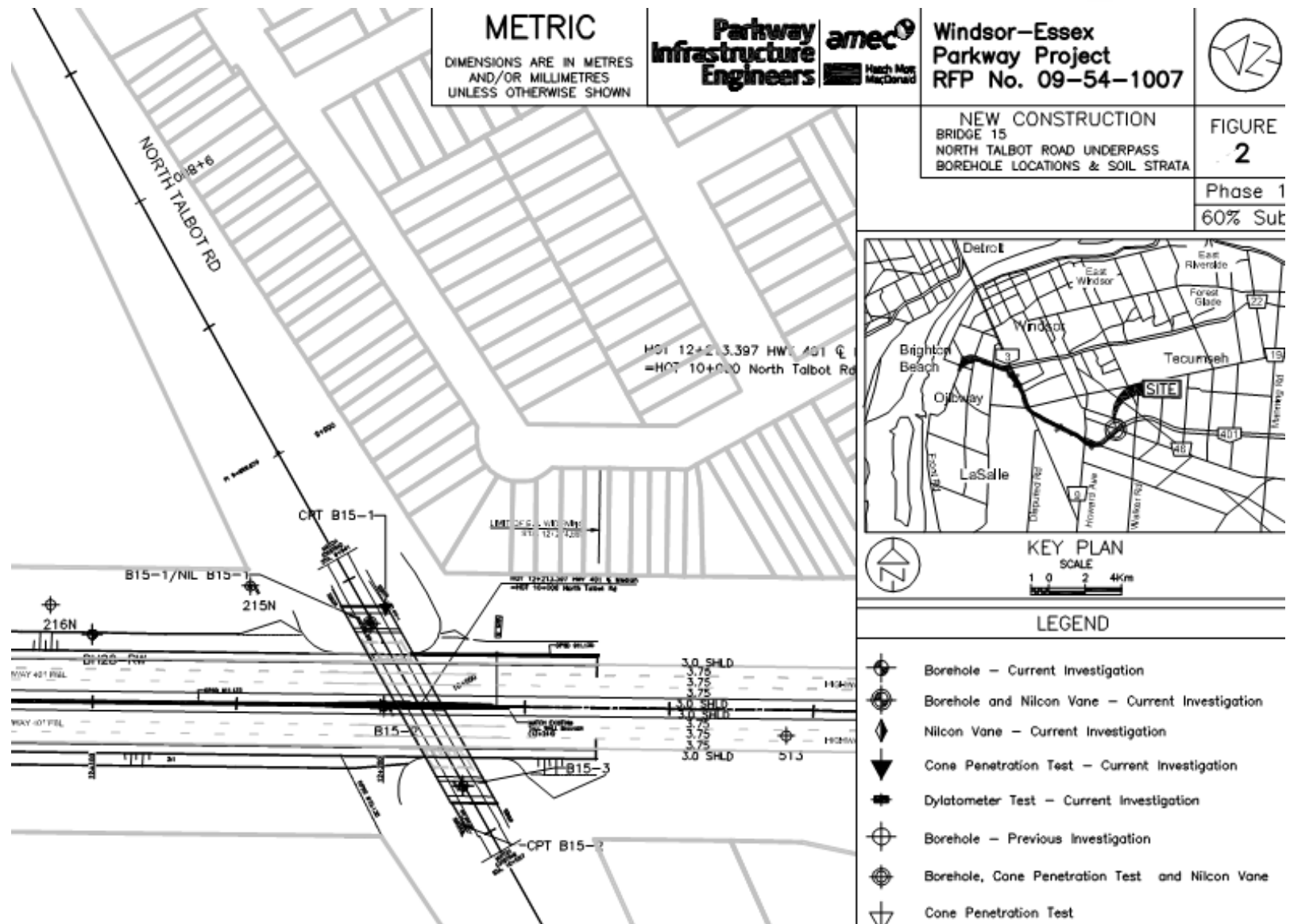


Figure 3: Test Hole Location Plan – Previous and Proposed Investigations

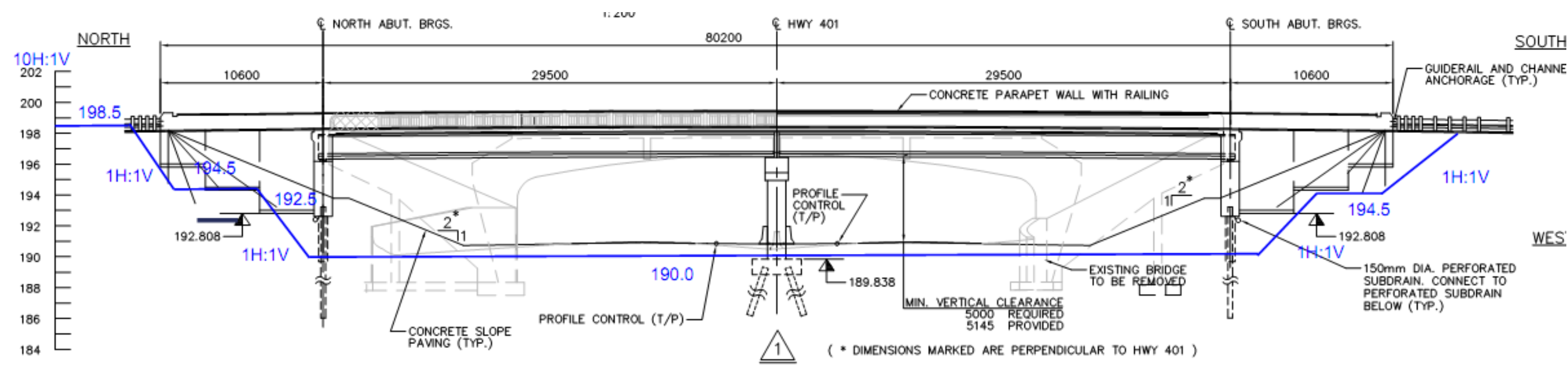


Figure 4: Excavation Profile for Bridge B-15

File Name: B15Abut\_EXC\_20110502.gsz  
 Last Saved: 5/2/2011 - 4:18:42 PM  
 Analysis Method: Morgenstern-Price - E&E

Properties:

Name: Silty Clay Crust Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 75 kPa Phi: 0 °  
 Name: Trans Silty Clay Unit Weight: 22 kN/m<sup>3</sup> C-Datum: 75 kPa C-Rate of Change: -5 kPa/m Limiting C: 60 kPa Elevation: 185 m  
 Name: Lower Grey Silty Clay Unit Weight: 20 kN/m<sup>3</sup> Cohesion: 60 kPa Phi: 0 °  
 Name: Clay Fill (Existing) Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 0 °

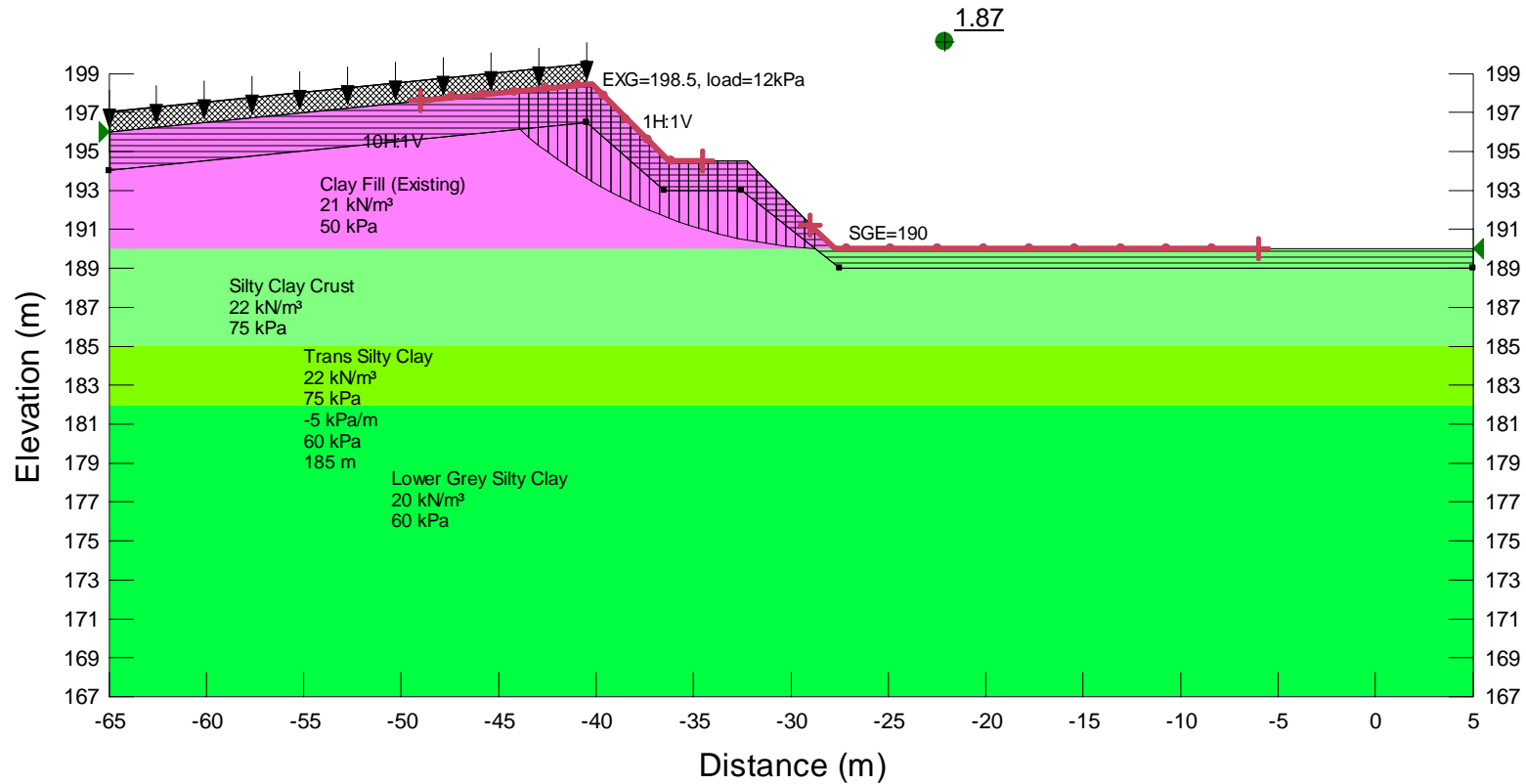


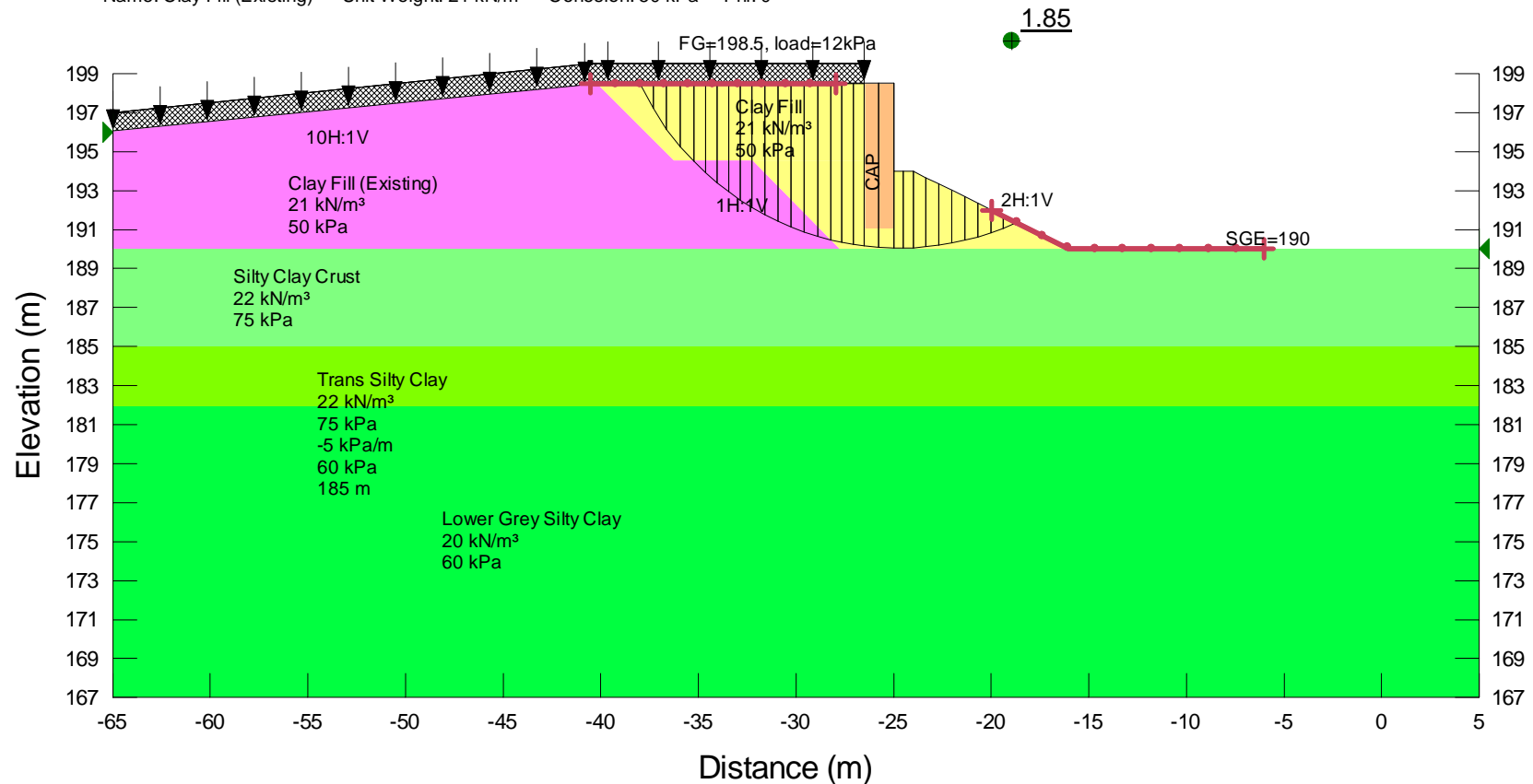
Figure 5: Abutment Global Slope Stability – Temporary Excavation



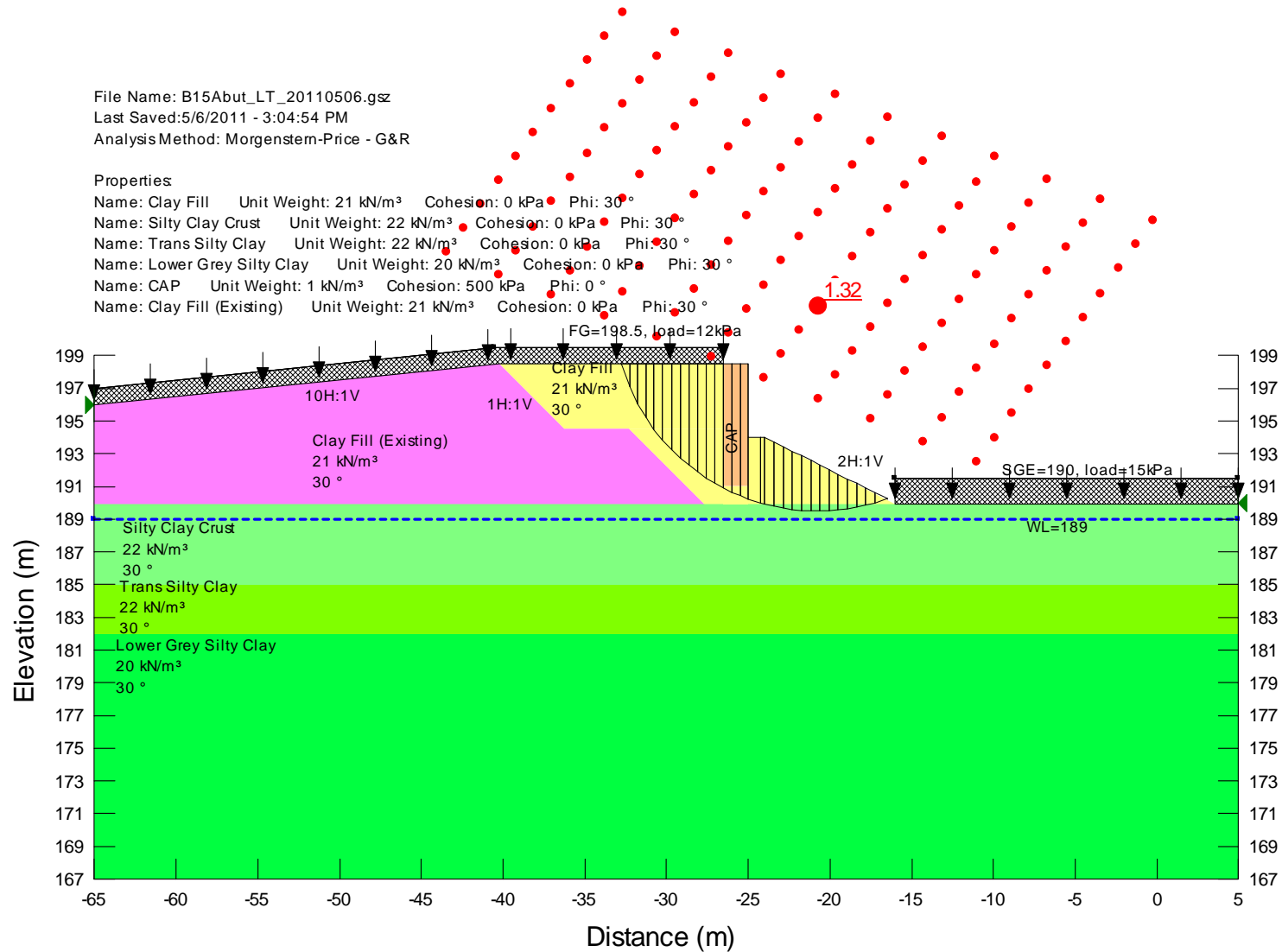
File Name: B15Abut\_ST\_20110506.gsz  
 Last Saved: 5/6/2011 - 3:05:11 PM  
 Analysis Method: Morgenstern-Price - E&E

**Properties:**

Name: Clay Fill Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 0 °  
 Name: Silty Clay Crust Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 75 kPa Phi: 0 °  
 Name: Trans Silty Clay Unit Weight: 22 kN/m<sup>3</sup> C-Datum: 75 kPa C-Rate of Change: -5 kPa/m Limiting C: 60 kPa Elevation: 185 m  
 Name: Lower Grey Silty Clay Unit Weight: 20 kN/m<sup>3</sup> Cohesion: 60 kPa Phi: 0 °  
 Name: CAP Unit Weight: 1 kN/m<sup>3</sup> Cohesion: 500 kPa Phi: 0 °  
 Name: Clay Fill (Existing) Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 0 °

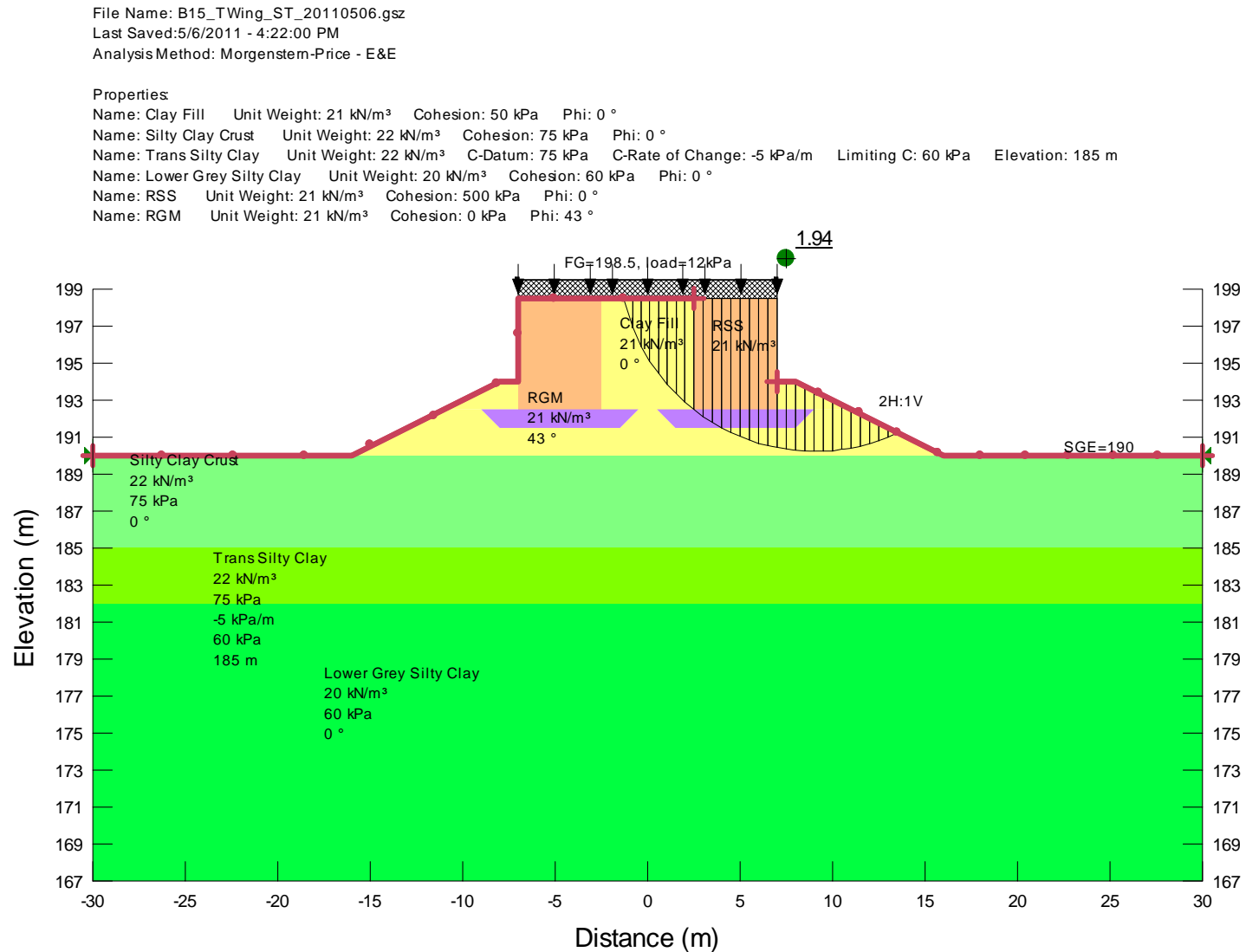


**Figure 6: Abutment Global Slope Stability – End of Construction**

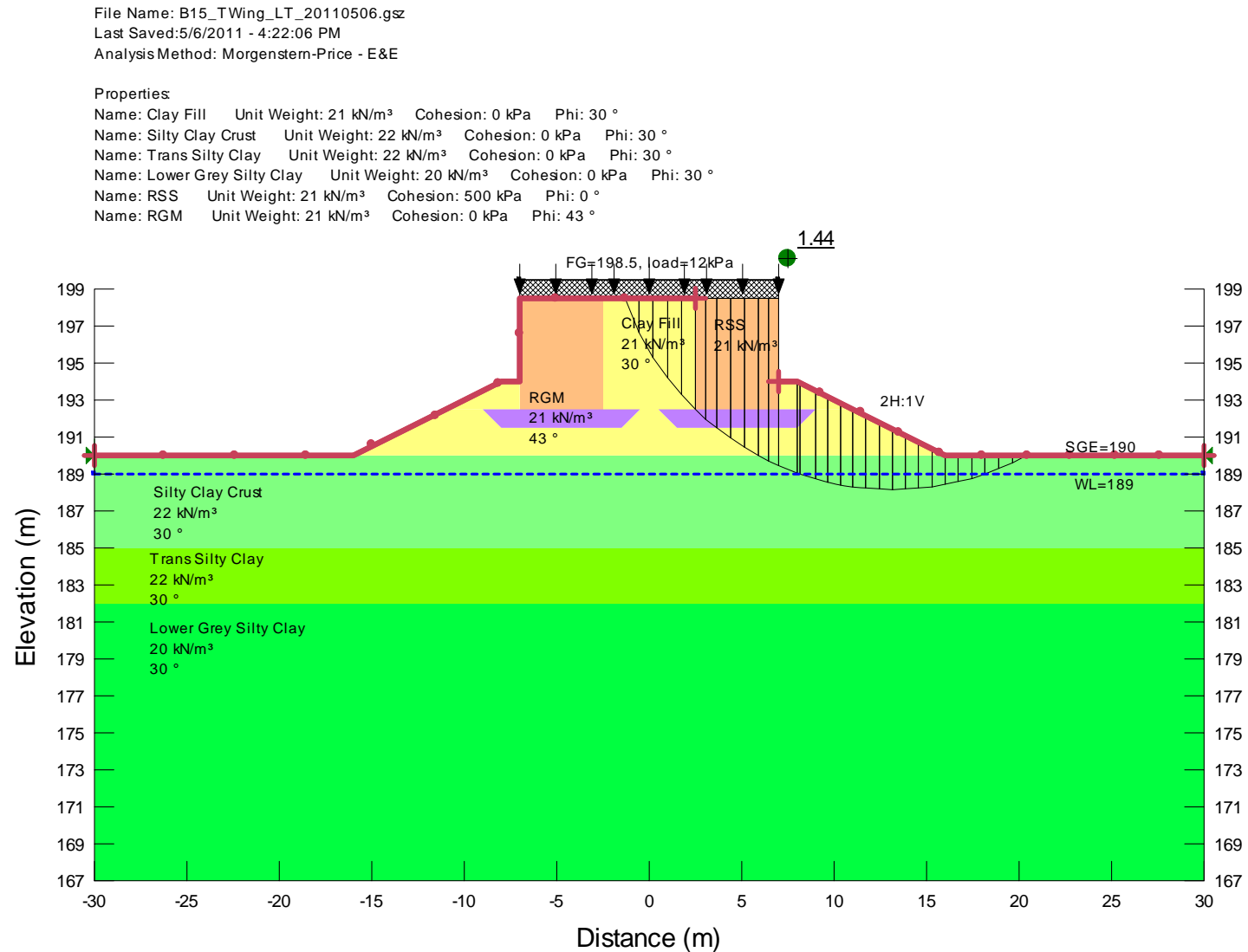


**Figure 7: Abutment Global Slope Stability – Long-term Loading Condition**





**Figure 8: Wing Wall Global Slope Stability – End of Construction**



**Figure 9: Wing Wall Global Slope Stability – Long-term Loading Condition**

## **APPENDIX A – EXISTING BOREHOLE LOGS**

PROJECT 09-1132-0003		LOCATION N 4678473.3 ; E 337006.7		1 OF 1		METRIC						
W.P. 3179-08-01		BOREHOLE TYPE POWER AUGER, HOLLOW STEM		ORIGINATED BY NG		COMPILED BY LMK						
DIST WEST HWY 401		DATE March 12, 2009		CHECKED BY SJB								
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					
189.69	GROUND SURFACE											
0.00	TOPSOIL, clayey Grey											
0.23	CLAYEY SILT, some sand, trace gravel, with oxidized fissures Firm Mottled brown and grey		1	SS	6							
188.32	CLAYEY SILT, some sand, trace gravel, with oxidized fissures Hard Brown		2	SS	37							
1.37			3	SS	69							
186.62	CLAYEY SILT, some sand, trace gravel Very stiff to hard Grey		4	SS	35							
3.07			5	SS	25							
			6	SS	17							
			7	SS	15							
			8	SS	10							
			9	SS	12							
180.09	END OF BOREHOLE											
9.60	Borehole dry during drilling on March 12, 2009.											

LDN\_MTO\_01\_09-1132-0003.GPJ LDN\_MTO.GDT 7/13/09