Golder Associates Ltd.

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

REPORT ON

FOUNDATION DESIGN REPORT
OLD WANUP ROAD UNDERPASS
HIGHWAY 69, SITE 46-498
G.W.P 327-91-00
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 54, SUDBURY

Submitted to:

URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Ltd. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detailed design of the Old Wanup Road Underpass structure along the proposed new Highway 69 alignment.

The terms of reference for the scope of work are outlined in Golder's proposal P31-1084, dated March 2003, that forms part of the Consultant's Agreement (Number P.O.5005-A-000287) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated April 8, 2003. The general arrangement drawings for the various options under consideration for the bridge structure at Old Wanup Road was provided to Golder by URS in June 2003.

The foundation investigation was carried out by Peto MacCallum Ltd. (Peto) and the factual information, including soil stratigraphy and borehole logs is contained in the following report by Peto:

- Foundation Investigation Report for Old Wanup Road Underpass, Site 46-498, GWP 327-91-00, District 54, Highway 69, Sudbury, Ontario, dated May 2003.

The site is situated at the Old Wanup Road crossing of the proposed new alignment of Highway 69. The proposed two-span structure will carry Old Wanup Road traffic over the proposed new Highway 69. Highway 69 runs in the north-south direction in the area of Old Wanup Road.

It should be noted that a contract drawing showing the borehole locations and soil strata with the general arrangement for the bridge will be provided once the General Arrangement drawing has been finalized and sent to us..

2.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the proposed Old Wanup Road Underpass and related earth and rock works. The recommendations are based on interpretation of the factual geotechnical data obtained from the Foundation Investigation Report completed by Peto MacCallum Ltd. in May 2003.

The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

The proposed Highway 69 in this area will be constructed in cut (up to about 6 m). It is understood that this excavation will be carried out prior to construction of the proposed structure. The recommendations given in this report have taken this into account as it pertains to foundation design and construction, excavation, drainage and other considerations.

2.1 Summary of Subsurface Conditions

The subsoils at the site are generally comprised of surficial roadway fill (overlain by asphalt at the east abutment and approach) underlain by discontinuous deposits of native clayey/sandy silty and sand. The relative density of the predominantly non-cohesive deposits were typically loose to very dense and generally compact. Numerous cobbles and boulders were encountered within the sandy soils. The cohesive deposits were typically firm to stiff. The overburden deposits are between 0.6 m and 16.2 m thick and are underlain by bedrock. The groundwater level was typically between 0.3 m and 3.7 m below the existing ground surface. Details of the stratigraphy are given on the borehole logs contained in the Foundation Investigation Report. The location of the boreholes relative to the foundation units are shown on Figure 1.

2.2 General

It is understood that the proposed Old Wanup Road Underpass bridge will be a slab-on-girder structure with abutments located just north and south of the proposed Highway 69 alignment. The spans will be 41 m long each. In order to eliminate the requirement for expansion joints and bearings at the ends of the bridge deck, it is understood that consideration is being given to either integral or semi-integral abutment configurations, as follows :

- Semi-integral abutments supported on shallow spread footings at west abutment and piles at the east abutment; and

- Pile-supported integral abutments with piles placed within a trench excavated into the bedrock at west abutment.

A summary of the advantages, disadvantages, cost and risks/ consequences for the two alternatives is given in Table 1 following the text of this report. The following sections discuss these options and provide foundation recommendations for each alternative.

2.3 Bridge Foundation Options

The existing grade of Old Wanup Road in the vicinity of the proposed underpass structure varies from about Elevation 249.5 m to 250 m at the west and east abutments, respectively. The existing road is built on an embankment which is about 1.5 m to 3.5 m above the adjacent ground surface. Based on the information provided on the Old Wanup Road Underpass General Arrangement Drawing provided by URS in June 2003, the proposed new Old Wanup Road grade varies between about Elevation 253 m and 254 m (which is about 3 m to 4 m above the existing Old Wanup Road surface). Therefore, the total embankment height will be up to about 6.6 m above existing/original ground surface outside the existing road embankment. The proposed Highway 69 grade at the structure location is at about Elevation 246 m for the northbound and southbound lanes which is between 3.6 m and 4.5 m below the existing Old Wanup Road grade and some 7 m to 8 m below the proposed Old Wanup Road grade.

The bedrock surface elevation varies substantially across the site from about 0.6 m below the existing ground surface at the north side of the west abutment to 16.2 m depth at the east abutment. In addition, the bedrock surface at the west abutment and the pier varies by about 4.5 m and 6.8 m, respectively, in the north-south direction, being lower on the south side of the proposed structure. The bedrock surface elevations at the proposed foundation elements, as established at the borehole locations, are given in the table below:

Foundation Element	***Bedrock Surface Elevation (m)***		
	North Side	***Centre***	***South Side***
West Abutment	247.8 (BH ST2-7)	246.7 (BH ST2-1)	243.3 (BH ST2-8)
Central Pier	242.8 (ST2-AP3)* 241.2 (ST2-9)	238.4 (BH ST2-2)	236.0 (BH ST2-3)
East Abutment	235.1 (ST2-AP5)*	233.9 (BH ST2-4)	235.9 (ST2-AP4)*

* Probable bedrock; i.e. inferred based on auger or dynamic cone refusal.

It should be noted that at the east abutment, the bedrock surface elevation has been confirmed only in the borehole at the centre of the foundation element. The ‘south side’ information is based on refusal to dynamic cone penetration (DCPT) refusal. The ‘north side’ information is based on auger refusal. Given the presence of cobbles and boulders within the overburden overlying the bedrock, the DCPT and auger refusal should not be assumed to provide definitive bedrock surface information.

The bedrock, described as gneiss, biotite to granitic gneiss and granite pegmatite, is suitable for support of the bridge on shallow foundations at the west abutment for the semi-integral abutment or on driven steel H-piles.

Since integral abutments are under consideration, steel H-piles can be considered for support of the abutments. Assuming an underside of pile cap at about Elevation 247.4 m and a minimum pile length of 6 m (as required to impart sufficient flexibility of the piles to accommodate bridge deck deflections for an integral abutment structure), the required bedrock excavation at the depth would vary from approximately 1.9 m to 6.4 m across the length of the west abutment to allow installation of the piles.

The pier could be supported on spread footings founded on bedrock or mass concrete. In this case, overburden excavation of up to about 7 m below the proposed ditch level would be required to expose the bedrock surface. Alternatively, the pier could be founded on driven piles; however, in order to achieve a pile length of 5 m at the central pier, bedrock excavation would be required at the north side of the footing. It is understood that consideration is being given to the use of two separate column footings with the north column founded on a spread footing and the south column founded on driven piles or caissons. Without bedrock excavation, the driven pile length could be as little as 3 m based on the borehole in the area of the south column footing. The use of caissons for support of the pier provides flexibility in dealing with the variable but restricted depth to bedrock. However, caissons are not recommended at this site due to the difficulty in socketting the caissons into the extremely strong gneiss bedrock and due to the sloping bedrock surface. Groundwater control for the caisson option would also be difficult.

Recommendations for spread footings founded on bedrock and steel H-pile foundations for the bridge abutments are presented in the following sections.

2.4 Shallow Foundations

It is understood that the majority of the excavation for the west abutment and pier will be encompassed within the excavation for the proposed Highway 69 excavation (cut), which will take place prior to the structure work.

West Abutment

The west abutment may be supported on spread footings placed on or within the biotite to granitic gneiss bedrock or on mass concrete placed on the bedrock. The bedrock surface elevations are given in Section 2.3. Due to the presence of loose to compact sandy soils at the site and potential for differential settlement, shallow spread footings are not recommended for the east abutment.

At the north side and central section of the west abutment footing, following removal of the overburden, bedrock excavation is required given the proposed road grades. Based on a founding level of Elevation 245.5 m (as noted on the preliminary GA drawing), subexcavation of up to about 2.3 m of bedrock would be required at the north side. It is noted that the bedrock is classified as medium to extremely strong (i.e. estimated unconfined compressive strengths in the range of 50 MPa to greater than 250 MPa). This will make excavation potentially difficult particularly in areas where only small depths and narrow zones of removal are needed. Bedrock excavation would likely have to be carried out using line drilling and pre-shearing techniques (as discussed in Section 2.11). This method would provide better control over the configuration of the founding surface, and this procedure would be the preferred approach where deeper excavation into the bedrock is required for footing construction.

Alternatively, consideration could be given to raising the footing elevation to the highest bedrock elevation (about Elev. 247.8 m), with placement of additional mass concrete in order to minimize bedrock excavation.

At the south side of the west abutment foundation element, following the removal of the overburden, the bedrock surface would have to be cleaned and then mass concrete placed to raise the grade as required to the founding level. Provision should be made in the Contract Documents for additional mass concrete placement to accommodate variations in the bedrock surface.

As an alternative to placement of mass concrete, consideration could be given to the use of short caissons for the south side of the west abutment. With the same procedure of overburden removal for the Highway 69 construction extending over to encompass the west abutment, after the bedrock surface is exposed, the caissons could be formed within steel liners placed on the bedrock surface. Some excavation of the bedrock surface in the caisson footprint would have to be carried out to create a level pad on which to place the liner. Difficulty may be encountered in achieving a level surface in the bedrock due to the hardness of the rock. As an alternative, mass concrete may be used to level the caisson area locally before setting the liner. It is assumed that backfilling with rock fill or granular will be used around the liners.

The simplest spread footing option for the west abutment, from a foundation perspective is for a combination of founding on the bedrock surface and mass concrete placed on the bedrock surface which should minimize the bedrock excavation difficulties. The cost effectiveness of each of the foundation alternatives should be considered in the design.

It is noted that footing excavations to expose the bedrock surface over the south portion of the west abutment will, in some places, extend through existing fill embankment as well as water-bearing sands and silt overburden soils. Groundwater control measures (as discussed in Section 2.10) will be required in order to maintain a dry and stable excavation.

Central Pier

The pier foundations may be supported on spread footings placed on or within the granite pegmatite, gneiss to granitic gneiss bedrock or on mass concrete placed on the bedrock. The bedrock surface elevations are given in Section 2.3. Due to the presence of compact to very dense sand at the founding level and potential for differential settlement between the pier column footings and between the abutments (placed on bedrock, mass concrete or driven H-piles), shallow spread footings founded within the overburden are not recommended.

Due to the sloping nature of the bedrock surface across the proposed pier column footings, the excavation through the overburden will vary from about 2 m to 7 m below the proposed Highway 69 median ditch grade to expose the bedrock surface. If spread footings are only considered for the north column, excavation below Highway 69 grade will be reduced. The bedrock surface would have to be cleaned and then mass concrete placed to raise the grade as required to the founding level. Alternatively, the founding level of the footing could be lowered to the bedrock surface to minimize the quantity of mass concrete required; however, mass concrete placement would still be required due to the variability of the bedrock surface across the individual footings. Provision should be made in the Contract Documents for additional mass concrete placement to accommodate variations in the bedrock surface.

It is noted that footing excavations to expose the bedrock surface at the pier will extend through the existing fill embankment as well as water-bearing sands and silts and clayey silt soils. Groundwater control measures (as discussed in Section 2.10) will be required in order to maintain a dry and stable excavation.

2.4.1 Geotechnical Resistance

Spread footings placed on the surface of the properly prepared gneiss bedrock at the west abutment or granite pegmatite to granitic gneiss at the pier, may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 10,000 kPa. For footings placed on a mass concrete pad, the factored geotechnical resistance at Ultimate Limit States (ULS) is as given above for bedrock assuming that the strength of the concrete used to form the pad is at least 25 MPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

Short caissons founded on the surface of the bedrock or a localized concrete leveling pad, may be designed based on end-bearing resistance using the values given above.

All loose, shattered and/or fractured rock within the footprint of the footings and at the footing level should be removed and replaced with concrete. A NSSP should be included in the Contract Documents to address the requirements for field inspection; MTO Special Provision 902S01 – Excavation and Backfilling – should be included in the Contract Documents.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary.*

2.4.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete footings and the bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ, may be taken as 0.70 between the base of the concrete footings and/or mass concrete and the bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the sound bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded.

In the case of the mass concrete placed on the sloping bedrock surface, the design may also take into account the lateral resistance provided by the soil adjacent to the mass concrete. In this case, there is a requirement for some movement of the mass concrete to mobilize the passive resistance and as such, will not strictly be applicable if dowels are used as well at the bedrock/mass concrete interface. In addition, there would be some restrictions needed on the process of excavation and backfilling to ensure that this lateral resistance could be relied upon. A value of K_p of 3.7 for the coefficient of passive lateral earth pressure may be used for the soil in front of the mass concrete below frost depth (2.0 m below final grade).

2.4.3 Frost Protection

For spread footings or mass concrete founded on the properly prepared gneiss bedrock at this site, frost susceptibility is not an issue.

2.5 Steel H-Pile Foundations

Driven piles are suitable for the support of the east abutment and can also be considered at the pier provided sufficient depth below the pile cap to the bedrock is available to provide a 5 m pile length. In the case of the central pier south column, the number and configuration of piles could be adjusted to account for pile lengths of less than 5 m.

As noted in Section 2.3, if steel H-piles are to be considered for integral abutment design, the piles and CSPs at the west abutment would have to be placed in excavations extending about 1.9 m to 6.4 m into the bedrock due to the shallow depth of the overburden at this foundation unit. The gneiss bedrock is medium strong to extremely strong and therefore boring large diameter (i.e. approximately 1 m diameter) sockets, although possible, may not be cost effective. Consideration should be given to trenching/excavating by drilling and blasting into the bedrock to provide a preformed slot into which compacted granular backfill could be placed and the piles could be driven. The width of the trench should be at least 3 m. Bedrock excavation using line drilling and pre-shearing techniques (as discussed in Section 2.11) will be required. In addition, the bedrock excavation for the west abutment will be partially encompassed by the proposed Highway 69 excavation (cut).

It should be noted that groundwater control measures (see Section 2.10) would be required in order to complete such excavation through the water-bearing sands and silts at this site.

2.5.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to refusal on the gneiss bedrock, a factored axial resistance at ULS of 2,000 kN may be assumed for design. In the case of the driven H-piles, this value represents a structural limitation for the pile rather than a geotechnical limitation although it does assume proper seating of the piles into/on the bedrock. In this regard, the piles should be fitted with Titus Ejector rock points or equivalent and appropriate driving procedures must be adopted to ensure adequate/proper seating of the piles without damaging the piles. The driving procedures to enable this seating depend on the type of pile driving rig used and need to be established at the time of construction. Generally, the procedures will involve a reduction in hammer energy once abrupt peaking is met to ease the pile point into the rock.

The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

2.5.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. For vertical piles, the resistance to lateral loading will depend on the soils surrounding the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections. In this case, the lateral resistance of the piles at the west abutment will also be governed by the type of backfill placed in the bedrock trench through which the piles are driven or placed as well as the configuration/width of the trench.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the following equation for granular soils (assuming the bedrock excavation is backfilled with a granular material):

$$k_h = \frac{n_h z}{B}$$

where
n_h is the constant of horizontal subgrade reaction, as given below;
z is the depth (m); and
B is the pile diameter (m).

For cohesive soils, the coefficient of horizontal subgrade reaction is constant with depth and is estimated using the following formula:

$$k_h = \frac{67 s_u}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa), as given below; and
B is the pile diameter (m).

The following range for the value of n_h and s_u may be assumed in the structural analysis:

Soil Unit	*East Abutment*	*West Abutment*	*s_u (kPa)*	*n_h (MPa/m)*
Stiff to very stiff clayey silt	Above Elev. 245 m	n/a	75 - 150	n/a
Backfill around piles and CSPs in bedrock trench excavation (assumed to be compacted granular fill below the groundwater level)	n/a	Above base of trench up to bedrock surface	n/a	5 to 10
Loose to compact sand/sandy silt, trace clay below the groundwater level	Elev. 245 m to 240 m	n/a	n/a	2 to 8
Dense to very dense sand, silty sand, and gravelly sand with cobbles and boulders below the groundwater level	Below Elev. 240 m to bedrock surface	Above bedrock surface	n/a	6 to 16

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	*Reduction Factor*
8d	1.0
6d	0.7
4d	0.4
3d	0.25

2.5.3 Frost Protection

The pile caps should be provided with a minimum of 2.0 m of soil cover for frost protection.

2.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3505.000.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 2.0 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM) or rock fill:

	SSM	Rock Fill
Soil unit weight:	20 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.35	0.27
At rest, K_o	0.50	0.43

- For Case II, the pressures are based on the rock fill as above or on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	**Granular 'B' Type II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for Sudbury is 0.05. Based on experience, for the subsurface conditions at this site, a 10 to 20 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.09$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

- The following seismic active pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	**Case II**	
		Granular A	**Granular B Type II**
Yielding wall	0.32	0.26	0.30
Non-yielding wall	0.37	0.30	0.34

Note : These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta = \phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$K\,\gamma'\,d + (K_{AE} - K)\,\gamma'\,H$$

Where
- K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
- K_{AE} is the seismic active earth pressure coefficient;
- γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 20 kN/m^3 above Elev. 247 m and 10 kN/m^3 below Elev. 247 m for the native materials
- d is the depth below the top of the wall (m); and
- H is the height of the wall above the toe (m).

2.7 Approach Embankment Design

The construction of the Old Wanup Road underpass will require placement of up to about 6.5 m of fill for the approach embankments and will encompass the existing Old Wanup Road fill embankment. The existing embankment soils consist of up to 3.6 m of compact sand and gravel, sandy gravel or gravely sand containing numerous cobbles and boulders. Outside the area of the fill, the subgrade soils will consist of either compact silt, very loose to compact sandy silt or very stiff clayey silt. Within some of areas of the west approach, bedrock was encountered at shallow depth below ground surface. The existing approach embankment fills are considered to be appropriate subbase for the proposed approach embankment; however, all topsoil and softened/ loosened soils should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement. In the following sections, the results of stability and settlement analysis for the new approach embankments are presented.

2.7.1 Stability

Due to the variable stratigraphy and sloping bedrock surface across the site, analyses were performed for the proposed east and west approach embankments of Old Wanup Road to assess stability and liquefaction potential. In addition, stability analysis was carried out for that portion of the slope (adjacent to the abutment) where the fill embankment sits atop the proposed highway cut.

Limit equilibrium 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 was 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 this site 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 proposed embankment heights and geometries.

The subsoils encountered in the area of the west approach embankment are composed primarily of cohesionless soils. In the area of the east embankment, a layer of cohesive clayey silt up to 3 m thick below the ground surface/existing embankment fill and overlying the cohesionless deposits.

For the soils at this site, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT). The parameters used in the analysis are shown on Figures 2 to 4.

The piezometric conditions assumed in the analyses were based on the water levels noted in the boreholes during drilling which were typically at or less than 0.5 m below about the elevation of the natural ground surface (below the existing fill embankment). It should be noted that there were no piezometers installed in the boreholes and as such there is no information available on the stabilized groundwater table.

In the analyses, two different types of fill (i.e. rock fill and earth fill) were considered for the required fill embankments. The fill alternatives provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to native subsoils), construction cost and time, and ease of construction / availability. A brief description of each alternative is described below.

Earth Fill (Sand and Gravel) Embankment

This option assumes that conventional earth fill is used for the construction of the embankment. The main advantage of this option is that post-construction settlements within the fill embankment itself are minimized. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, granular material.

Rock Fill Embankment

This option would employ rock fill rather than earth fill for the embankments. The main advantage of this material is the ability to achieve steeper embankment side slopes, particularly useful in areas with limited right-of-ways. The surplus rock fill available from the highway cuts through the adjacent bedrock outcrops may provide a cost advantage. The disadvantage of using rock fill for the construction of the embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

Stability Analysis Results

At each approach embankment area, due to the variability in thickness of the overburden, the highest (i.e. most critical) approach embankment section on both the north and south side slopes have been analysed. In addition, the most critical section for the abutment side slopes adjacent to the proposed Highway 69 permanent cut have been analysed. The results of the stability analyses for the two options are summarized in the following table presenting the minimum factor of safety for deep-seated, global failure surfaces that would impact the operation of the roadway.

Location	***Embankment Height at Critical Section (m)***	***Earth fill Option***		***Rock fill Option***	
		Recommended Side Slope Profile	***Minimum Factor of Safety***	***Recommended Side Slope Profile***	***Minimum Factor of Safety***
East Approach (see Figure 2)	6.6 (north) 6.0 (south)	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
West Approach (see Figure 3)	4.5 (north) 3.0 (south)				
East Abutment (north side cut) (see Figure 4)	6.6 (approach fill) 4.4 (Hwy 69 cut)	n/a	n/a	1.25H : 1V 2.5H : 1V	≥ 1.3
West Abutment (north side cut)	4.5 (approach fill) 5.2 (Hwy 69 cut)	n/a	n/a	1.25H : 1V 2.5H : 1V	

Figure 4 is for the case of the Old Wanup fill embankments where they encroach on the permanent cut slopes for Highway 69. It should be noted that in the vicinity of the abutment, where stability of the Old Wanup Road embankment could be a concern, the cut slope will actually be backfilled with rock fill prior to construction of the fill embankment for ditch/culvert placement. Proper drainage, as discussed in Section 2.8, should be provided on all permanent cut slopes to prevent surficial sloughing where the cut slopes extend below the water table.

Liquefaction Potential

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, the soils at the site are not considered to be liquefiable under magnitude 7.0 earthquake events. Assuming a ground surface acceleration of 0.06 g, a factor of safety of greater than 1.0 is obtained for magnitude 7.0 earthquake events.

2.7.2 Settlement

Settlement analyses were performed on the critical sections of the proposed approach embankments. For these analyses, the critical sections are assumed to correspond to the greatest new embankment heights.

Over the majority of the area of the east approach embankment (i.e. within about 20 m from the abutment), the subsoils (below the existing embankment fill) consist of a layer of very stiff clayey silt up to about 3 m in thickness overlying loose to very dense cohesionless sands and silt deposits up to 11.5 m thick. Over the majority of the area of the west approach embankment, the subsoils (below the existing embankment fill) consist of very loose to compact sand and silt deposits. Bedrock is exposed at the ground surface at a distance of about 18 m to 20 m west of the west abutment. Surficial topsoil deposits were encountered in some of the boreholes.

Considering that a significant volume of rock fill will be made available during the excavation of the deep cuts through the existing bedrock outcrops in the area, the new approach embankments were analysed assuming a rock fill composition, γ = 19 kN/m^3, 1.25H:1V side slopes and the maximum new height of embankment. The following sections describe the estimated settlement of the foundation soils and the estimated settlements of the rock fill due to the loading imposed by the new approach embankments.

2.7.2.1 Foundation Soils Settlement

The new west approach embankment will be up to about 3 m and 4.5 m in height above the existing/ original ground surface. The immediate compression of the very loose to very dense sand and silt subsoils at the abutments were modeled by estimating an elastic modulus of deformation based on the SPT ‘N’ values. The parameters used in the analysis are shown on Figures 2 and 3. Provided that the topsoil is removed prior to the new embankment fill placement, the settlement of the foundation soils at the west approach is expected to be less than

50 mm as a result of the new embankment construction. These settlements are expected to occur rapidly (i.e. during or shortly after construction).

The new east approach embankment will be up to about 6.6 m in height. The consolidation settlement of the very stiff clayey silt deposit encountered in the boreholes at the east abutment was modeled by estimating the consolidation parameters from correlations with the SPT results and other laboratory test data. The immediate compression of the very loose to very dense sand and silt subsoils at the abutments were modeled by estimating an elastic modulus of deformation based on the SPT 'N' values.

The consolidation settlement of the foundation soils at the east approach is expected to be between 50 mm and 125 mm as a result of the new embankment construction. The majority of these settlements are due to the consolidation settlement of the very stiff clayey silt deposit and about 50 per cent of this settlement will occur within the first six months after loading.

2.7.2.2 Settlement of Rock Fill

Where rock fill is used for the construction of the embankments, in addition to the embankment settlement due to compression of the foundation soils, there will be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in the Special Provision, Amendment to OPSS 206 dated September 1999, the settlement of the newly placed rock fill is expected to be small. In general, it is estimated that for the up to 6.6 m high approach embankments, the settlement of the rock fill will be about 1% of the new effective height of rock fill. Therefore, the settlement of the newly placed rock fill is expected to be approximately 45 mm and 65 mm at the east and west approach embankment fill areas, respectively. It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction.

In some areas, the fill for the new embankments will be required to be placed immediately overtop of the existing Old Wanup Road fill embankments; however, the new embankment will cover a wider footprint and in these areas, there is a potential for some small differential settlement to occur between the existing and new portions of the road. However, since the majority of the settlement is expected to occur rapidly during construction (or within about 1 month following construction), the impact on the long-term performance of the travelled road surface is anticipated to be minor. Consideration should be given to regrading of the existing embankment by spreading the sand and gravel fill over the full proposed embankment area to provide a constant thickness.

2.8 Permanent Cut Slopes

The proposed new Highway 69 will be in cut, approximately 2.5 m to 5 m below existing ground surface in the vicinity of the Old Wanup Road. It is understood that consideration is being given to the use of drainage culverts in the ditches for the proposed Highway 69 in the area of the proposed underpass in order to minimize span length. It is further understood that Granular 'A' will be used to backfill the ditch to a certain height above the culvert (less than 1 m). The excavation for the culvert placement and the abutment construction will in effect result in excavation of the majority of the overburden in front of the abutments. The use of rock fill in front of the abutments above the Granular 'A' backfill will permit the use of permanent rock fill side slopes at 1.5 horizontal to 1 vertical (1.5H:1V). Outside of this area where the permanent cut slopes are formed within the overburden and will be below the existing water table, the side slopes must be no steeper than 2.5H:1V, assuming that proper drainage is achieved.

In order to achieve adequate drainage of the cut slopes and to minimize surficial sloughing, a granular blanket/gravel sheeting should be provided. The drainage blanket should be used on the permanent cut slopes including cut slopes on the proposed Highway 69 beyond the limits of Old Wanup Road, formed within the native soils below the existing ground surface. The granular blanket should be connected to toe drains/interceptor ditches that are adequately drained. The blanket should be a minimum of 600 mm thick.

2.9 Subgrade Preparation and Embankment Construction

Prior to the placement of any fill for the new approach embankment construction, all topsoil should be stripped from the plan limits of the proposed works. Where existing embankments are composed of sand and gravel fill, benching into the existing side slopes should be carried out as per OPSD 208.010.

Where rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision, Amendment to OPSS 206 dated September 1999. The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging shall be minimized by blading, dozing and 'chinking' the rock to form a dense, compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

If earth fill (granular) is to be considered for embankment construction, placement of all granular fill material should be carried out in accordance with OPSS 206.07.07, in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Vegetation cover should be established on all soil slopes to (either earth embankment or slope flattening) protect embankment fill against surficial erosion.

2.10 Design and Construction Considerations

2.10.1 Excavations and Groundwater Control

As noted in Section 2.3, excavations for spread footing or bedrock trench excavation at the west abutment will typically extend through embankment fill as well as water-bearing sand and silt overlying the bedrock. As discussed earlier in the report, it is assumed that the bulk of the excavation (including rock blasting) at the west abutment will be carried out during the excavation for the proposed Highway 69 excavation (cut) and that the ditching and culvert will be in place sufficiently such that some extent of drainage of the overburden soils will occur. In order to allow the excavation to proceed into the bedrock, if required for pile installation, groundwater control measures must be implemented. These measures are required to permit either mass concrete placement or granular backfill compaction (in the bedrock trench) to be carried out in the dry. Similar measures would be required for excavations at the pier. The groundwater level at the site is relatively shallow (ranges from about Elevation 246.8 m to 248.0 m) and groundwater flow into the excavations can be expected where excavations extend below this depth.

At the central pier for the proposed structure, the depth of the excavation required to found the pier on the bedrock surface will extend an additional 2 m and 7 m below the proposed Highway 69 median ditch grade. If two separate column footings are used (one on spread footings and one on driven piles), then the spread footing excavation requirements will be minimized.

By carrying out the general highway cut including the 2.2 m of subexcavation and replacement with rock fill for the pavement design including ditch construction, the dewatering/groundwater control requirements will be minimized. In addition, the bedrock excavation for the ditch/highway cut could be extended to encompass the west abutment area thereby minimizing the additional trenching require for pile installation.

Based on the drainage considerations and sequence of excavation assumed above, groundwater seepage inflow into the excavations through the sand and silt will occur and is expected to be minor, except during periods of sustained precipitation. Pumping from well-filtered sumps located at the base of the excavation should provide adequate groundwater control during foundation excavations for granular or mass concrete placement on the bedrock. Surface water should be directed away from the excavations at all times. The appropriate NSSP should be included in the contract documents.

If space permits, side slopes for temporary excavations (i.e. those that are open only for a relatively short period) through the native materials should be maintained no steeper than about 3H:1V unless prior dewatering is carried out. Temporary excavations through the existing road embankment fill may be made with side slopes no steeper than about 1.5H:1V.

If space and/or staging restrict the use of open cuts, a temporary support system could be constructed to support the excavations and adjacent road embankment in the area of the bridge structure foundations. The temporary excavation support system should be in accordance with MTO Special Provision 539S01. The temporary support system should be designed to Performance Level 2 as defined in SP 539S01. Roadway protection should be as per current MTO Special Provision 539S01.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

2.10.2 Obstructions

The fill soils at the site (i.e. roadway embankment fills) are expected to contain cobbles and boulders as discussed in the investigation report.

Conventional excavation equipment should be suitable for the majority of excavation through the on-site soils; however, the presence of the boulders may interfere with or slow the progress of stripping and excavation. The presence of boulders may also affect the driving of the piles and the piles may “hang up” within the overburden. In addition, the bedrock surface is extremely variable and care will be required during pile installation to ensure the piles are adequately seated without damaging the piles particularly in the case of the very short piles at the pier. The boulders and variable bedrock surface may also impact the installation of soldier piles and soil or rock anchors (tie-backs) if temporary roadway protection measures are required. Ultimately, provision will have to be made in the Contract Drawings to ensure that the Contractor is equipped to handle such obstructions.

2.11 Blasting Recommendations for Rock Excavations

2.11.1 Excavation Considerations

For excavations into the bedrock, the overall slope to the cut face may be formed vertical. The use of controlled blasting techniques are recommended in order to provide a neat excavation line and minimize face instabilities resulting from damage to the rock mass.

2.11.2 Special Provisions

2.11.2.1 Blasting

The use of controlled blasting techniques is recommended for all of the bedrock excavation. It is recommended that a separate Special Provision for the control of all blasting operations be prepared (refer to SP 299F06). The Special Provision should include, but not be limited to, the following:

- Outlining the requirements, procedure and extent of a pre-blast survey. This would include all structures within a radius of about 100 m of the blasting operations, as well as notification to all individuals working or living within 500 m.
- Submission of a blast proposal by the blasting contractor or their blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing procedures, as well as procedures to deal with debris clean-up. This submission would be required prior to the commencement of any blasting operations.
- The requirement for trial blasts for all proposed production and wall control blast procedures.
- The requirements for ground and air vibration monitoring during the blasting operations. This would include details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings.

We recommend limiting ground vibration levels to 50 mm/s for adjacent services and buildings. Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet these limits and how close to the blasting approaches the adjacent structures.

It is recommended that the specification for the blasting require a minimum of 80 percent half barrels (drill hole traces) visible on the cut face after scaling.

GOLDER ASSOCIATES LTD.

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

Anne S. Poschmann, P.Eng.,
Principal

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

SEP/ASP/FJH/sep
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TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES

Options	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
West Abutment and Pier on spread footing on bedrock; excavation to bedrock surface (elevation variable across footing width) East Abutment on H-piles driven to bedrock	Can minimize bedrock excavation depending on design founding level for west abutment.	Sloping bedrock surface at west abutment will require bedrock and soil excavation with mass concrete placement to achieve level footing. Bedrock will have to be blasted. Sloping bedrock surface at pier will require soil excavation with mass concrete placement. Dewatering and temporary shoring required for soil excavation below groundwater table to allow mass concrete placement in the dry. At the west abutment, extent of dewatering expected to be less than for the trenching. At the pier, extent of dewatering could be significant.	Less bedrock excavation required than for H-pile option at west abutment. Dewatering and temporary shoring costs reduced at west abutment.	Full excavation to the bedrock surface required for placement of mass concrete. Careful/controlled blasting techniques must be used to excavate bedrock. Difficulty may be encountered seating piles on sloping bedrock surface. Titus Injector Rock points will be required.
Intregal Abutments: Abutments founded on H-piles driven to bedrock (east) and trenched into bedrock (west). Pier on spread footing on north side and piles to bedrock on south side; excavation to bedrock surface (elevation variable across footing width)		Shallow overburden at west abutment will require up to 5.5 m bedrock excavation at one end of the footing to achieve minimum pile length. Additional bedrock excavation required if concrete socket required for pile fixity. Bedrock will have to be blasted. Dewatering and temporary shoring will be required through the overburden for the excavation below groundwater table. Dewatering is more extensive than the spread footing option to complete the required bedrock excavation for the pile trench. Cobbles/boulders noted throughout non-cohesive overburden may present obstructions to pile driving at pier and east abutment. Titus Injector Rock points will be required.	Significant bedrock trench for H-piles at west abutment will increase costs for blasting and backfilling as compared to costs for bedrock excavation for spread footing option. Dewatering and temporary shoring costs for soil excavation below groundwater table.	Careful/controlled blasting techniques must be used to excavate bedrock trench. Difficulty may be encountered seating piles on sloping bedrock surface. Titus Injector Rock points will be required.

OLD WANUP ROAD UNDERPASS
BOREHOLE LOCATION PLAN

FIGURE 1

LEGEND:

ST2-4	Location of borehole by Peto MacCallum Ltd. (May 2003)
ST2-AP5	Location of auger probehole by Peto MacCallum Ltd. (May 2003)

REFERENCES:

1. MAPPING BASED ON GENERAL ARRANGEMENT CAFLUFFAWHOSIT PROVIDED BY URS IN JUNE 2003

DATE: JUNE 2003

PROJECT: 03-1111-011-1 (5000)

Golder Associates

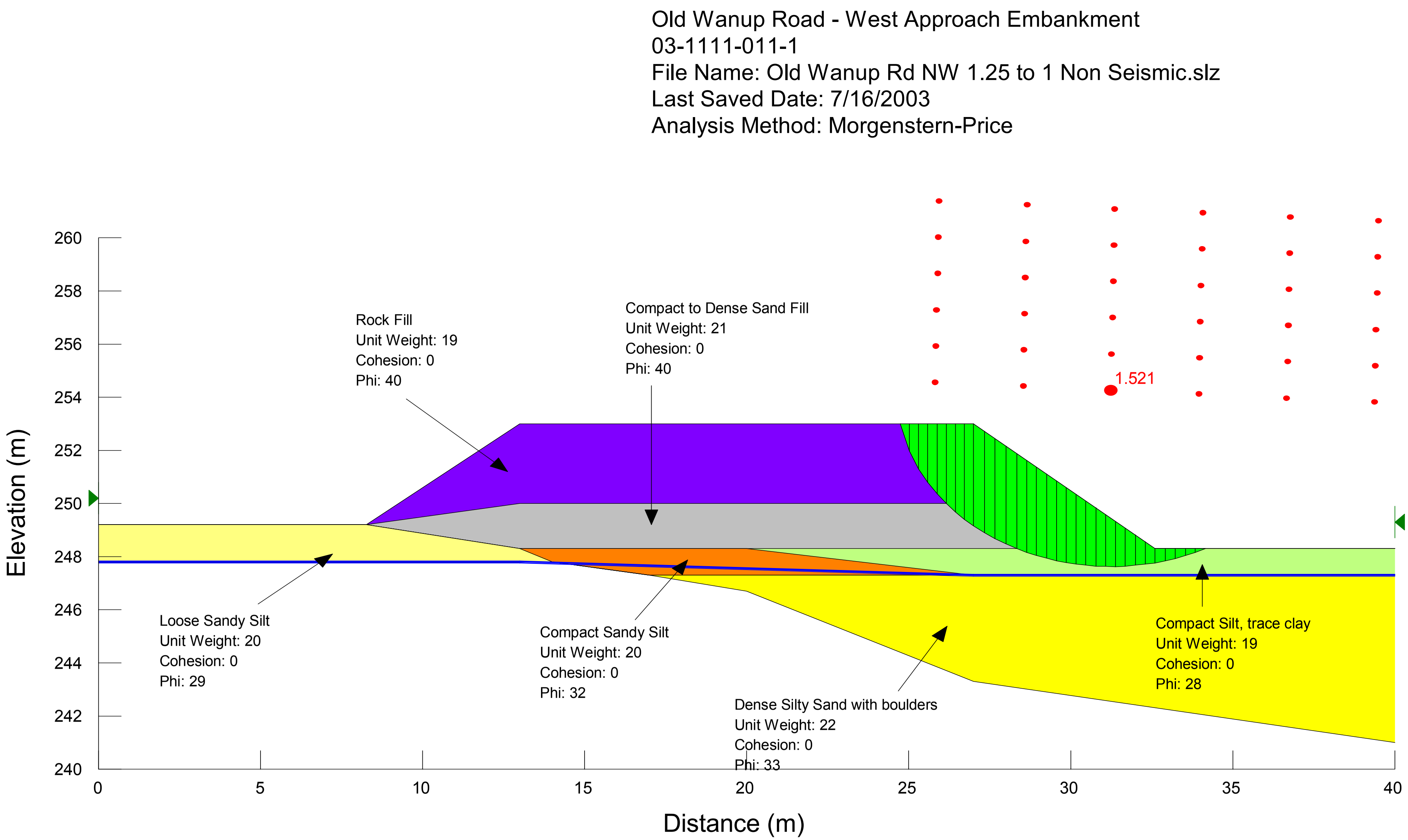
CAD: JDR

CHK: SEP

PLOT DATE: July 25, 2003
FILENAME: T:\Projects\2003\03-1111-011 (URS, Hwy 69)\-AA-\031111011-1-001.dwg

WEST APPROACH EMBANKMENT STABILITY
OLD WANUP ROAD UNDERPASS

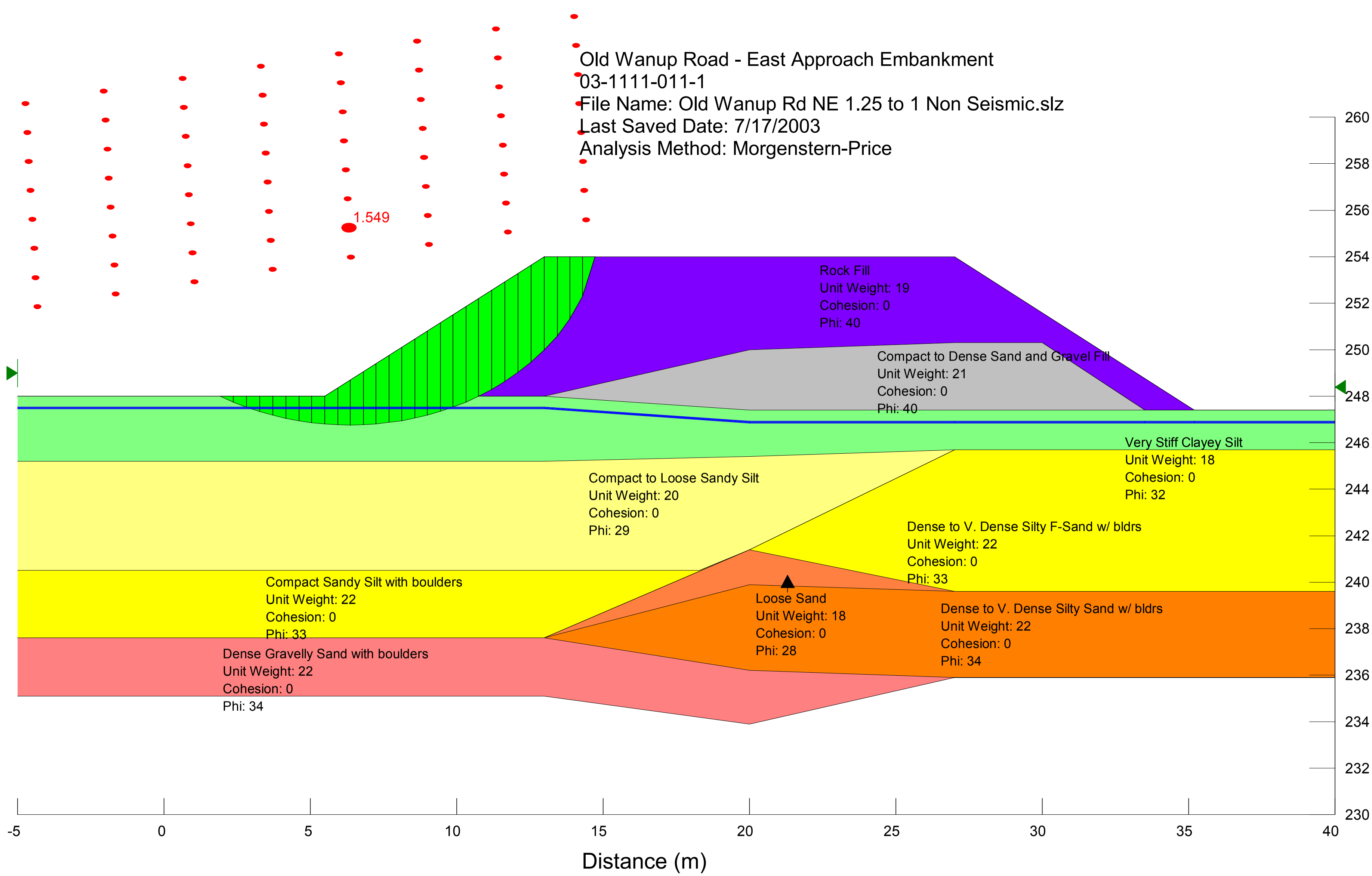
FIGURE 2



JOB NO.	03-1111-011-1	DRAWN	SEP
DATE	JULY 2003	CHECKED	ASP

EAST APPROACH EMBANKMENT STABILITY
OLD WANUP ROAD UNDERPASS

FIGURE 3



JOB NO. 03-1111-011-1

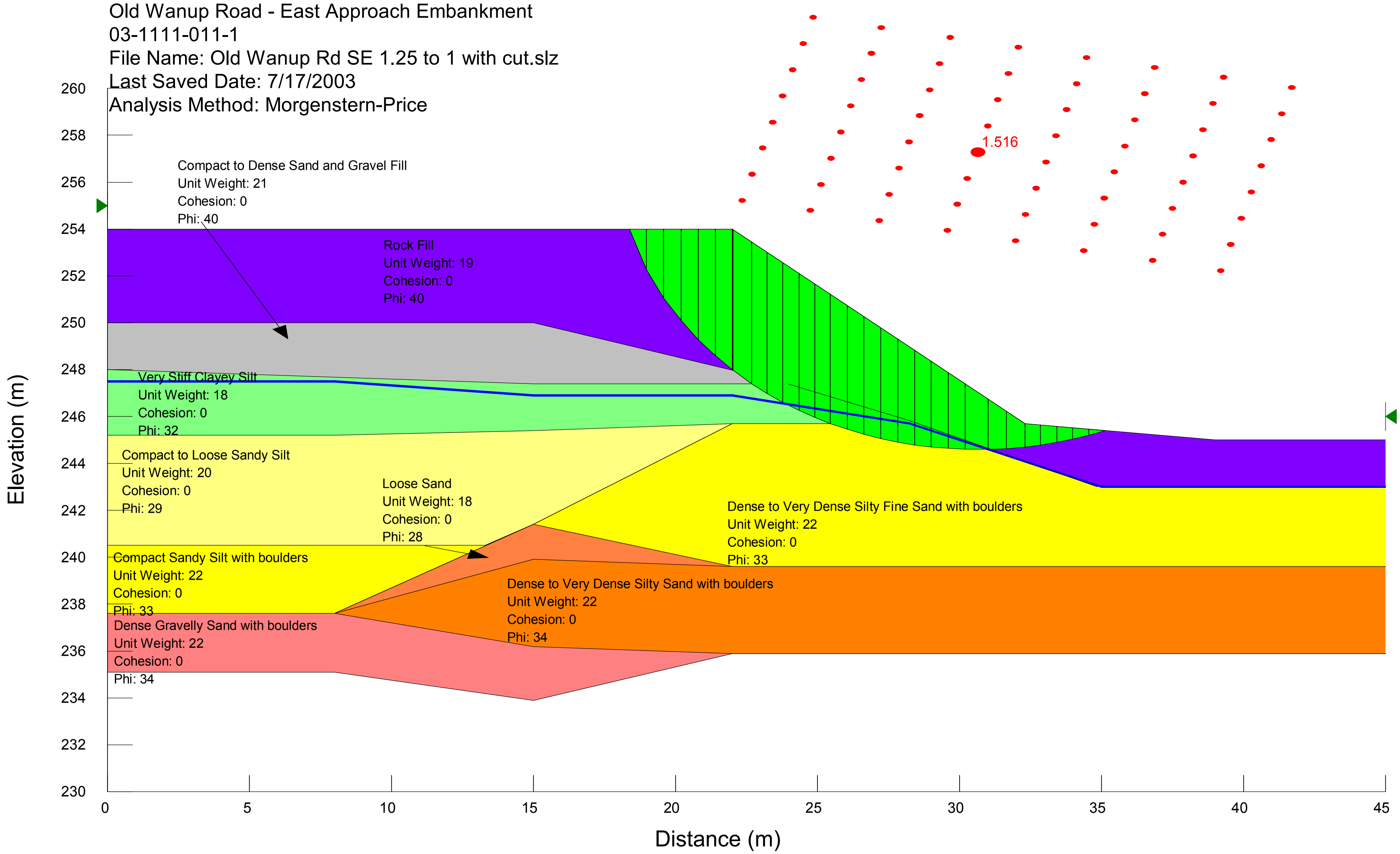
DATE JULY 2003

DRAWN SEP

CHECKED ASP

EAST EMBANKMENT STABILITY OVER CUT SLOPE
OLD WANUP ROAD UNDERPASS

FIGURE 4



JOB NO. 03-1111-011-1

DATE JULY 2003

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CHECKED ASP