Optimizing approach slab design for settlement using soil-structure interaction modelling – A case study

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ABSTRACT
The Ministry of Transportation of Ontario (MTO) is considering alternative approach slab configurations to reduce the frequency and severity of pavement surface distress that is commonly observed at approach slabs of bridges. Excessive embankment settlements will result in approach slab settlements that impact the highway rideability and can lead to serious safety concerns. At the Highway 417 overpass at County Road 3, the settlements have been excessive and resulted in significant degradation of the approach pavement, which required speed reduction warnings. MTO therefore undertook a study to assess potential approach slab rehabilitation options at that location. The work undertaken included geotechnical investigations, analysis to estimate the settlements that had occurred and the potential ongoing settlements. Soil-structure modelling was then carried out to evaluate numerous combinations of approach slab and pavement configurations to arrive at the most effective design for the rehabilitation of the County Road 3 bridge approaches and to provide guidance for approach slab rehabilitation at other bridges.

RÉSUMÉ

Le ministère des Transports de l'Ontario (MTO) envisage d'autres configurations de dalles d'approches afin de réduire la détérioration de la surface de chaussée couramment observée à l'extrémité des dalles d'approches des ponts. Des tassements excessifs de remblais produisent une dégradation des dalles d'approches et peuvent avoir un impact sur la maniabilité et la sécurité routière. Au niveau de l'autoroute 417, à la jonction avec la route 3 du comté de l'est de l'Ontario, les tassements du remblai avaient été excessifs et avaient entraînés une dégradation de la chaussée au niveau de la dalle l'approche, ce qui nécessitait des avertissements de réduction de vitesse. Le MTO a entrepris une évaluation des options de réhabilitation à cet endroit. Les travaux entrepris comprenaient des études géotechniques, analyses pour estimer les tassements historiques et estimer les tassements futurs. Une modélisation sol-structure a été effectuée pour évaluer de nombreuses configurations de dalles d'approches afin d'optimiser une conception pour la réhabilitation des dalles d'approches du pont à cet endroit, pouvant servir comme solutions sur d'autres ponts.

1 INTRODUCTION

Approach slabs are provided at the transition from the approach embankment to the bridge deck. These reinforced concrete cast-in-place slabs are intended to provide a smooth transition from the approach embankments, which can settle, to the relatively rigid bridge deck, which is often placed on pile supported abutments. Ideally, the settlement of the embankment, which is differential to the bridge deck, is less than what can be tolerated (i.e., about 25 mm) and the approach slab can function effectively. However, when embankments are constructed on soft ground, the long terms settlements can be significant in magnitude producing sags at the bridge approach. These sags or dips in the pavements can become a safety hazard to motorists travelling at highway speeds, result in pavement and approach slab damage, and increase maintenance frequency and costs. These hazards and maintenance demands are a significant concern to the Ministry of Transportation of Ontario (MTO).

Reducing ongoing settlement can be a costly endeavour, potentially requiring excavation of the embankment and replacement of the earth embankment fill with lighter weight fills (e.g., extruded polystyrene) or some form of ground improvement. Beyond the capital costs for that work, this can result in significant disruptions to users of the highway. In addition, such investments are not economical for structures that are nearing the end of their lifespan and which may be planned for replacement in the near to medium future (i.e., 10 to 20 years).

The MTO is therefore investigating the potential for alternative approach slab designs that could prove economical by mitigating the embankment settlement entirely or by at least extending the maintenance period, reducing maintenance costs while maintaining the safety of the travelling public. The investigations and modelling work described in this paper at one existing bridge site was intended to aid in the assessment of alternative approach slab designs for bridges and embankments constructed on compressible soils.

2 SITE DESCRIPTION AND HISTORY

The two existing County Road 3 overpasses, each of which carries two lanes of Highway 417 over County Road 3, are located about 4 km west of the village of Casselman, Ontario.

The overpasses constructed in 1973 are about 115.8 m long, 12.4 m wide, and each superstructure is supported on five piers founded on steel piles and semi-integral abutments, also founded on steel piles.

The existing bridge embankments are approximately 3 to 4 m in height above the natural ground level.

The available information and foundation records from the original investigation in 1970 indicate the subsurface conditions at the site generally consist of surficial fill or native sandy and silty soils, overlying a 20 m thick deposit of firm to stiff clay to silty clay underlain by glacial till, over limestone bedrock.

The existing embankment loading over the thick, sensitive and compressible clay deposit has resulted in significant settlement of the embankments since the original construction in 1973.

A major structure rehabilitation was carried out in 2005 and that rehabilitation included removal and replacement of the existing approach slabs, as well as construction of sleeper slabs. Subsequent to the 2005 rehabilitation, additional settlement was identified at the ends of all the approach slabs.

A survey was carried out by a licensed surveyor, J.D. Barnes (JDB), in November 2018 at this site. Based on the design profile grade from the 2005 rehabilitation and the surveying results by JDB, settlements ranging from 150 to 170 mm appears to have occurred at the approaches since the 2005 rehabilitation.

3 GEOTECHNICAL ASSESSMENT

3.1 Subsurface Conditions

3.1.1 Geotechnical Investigation

A geotechnical investigation was carried out by Golder Associates Ltd. (Golder) at the existing approaches between December 2018 and January 2019 to delineate the existing subsurface conditions and establish the current soil engineering parameters at this site.

During the investigation, a total of four boreholes were advanced using 108 mm inside diameter (200 mm outside diameter) continuous-flight hollow-stem augers on a truck mounted drill rig.

Two of the boreholes were located just beyond the sleeper slabs and the other two boreholes were located within the approach slabs. All four boreholes were advanced to depths ranging from about 25 to 30 m when practical refusal to augering was encountered.

Samples of overburden in the boreholes were obtained at intervals of about 0.60 and 0.76 m within the non-cohesive soil layers, using split-spoon samplers. In-situ vane testing was carried out within the clay deposits to measure the undrained shear strength of the cohesive soils. Remoulded shear strengths were also measured at selected intervals.

In addition, a total of 45 relatively undisturbed samples were retrieved throughout the clay deposit using a fixed piston sampler with 73 mm diameter thin-walled Shelby tubes.

Upon completion of drilling, a vibrating wire piezometer (VWP) was installed in each borehole at varying depths. The groundwater levels measured by the VWP's were obtained at various times between January and February in 2019.

A total of four CPT's were also carried out, one adjacent to each of the four boreholes, using a 25-ton truck mounted drill rig. The tests used a 15 cm^2 tip base area probe, with an equal end area friction sleeve, and tip and sleeve capacities of 1,500 bar and 15 bar, respectively.

In each CPT hole, the tip resistance, shaft friction, and pore water pressures were measured at approximately 0.025 m depth intervals. The CPT holes were advanced to refusal at depths ranging from about 26 to 27 m.

A total of four dissipation tests were completed in the CPT holes; i.e., one dissipation test in each completed testhole.

Based on the results of the current investigation, the subsurface conditions at the site consist of pavement structure and embankment fill (3 to 4 m thick), overlying a layer of native sandy silt to sand and silt (about 2 to 3 m thick), underlain by a deposit of soft to stiff compressible clay to silty clay (about 18 to 21 m thick).

3.1.2 Laboratory Testing

A comprehensive laboratory testing program was carried out on selected soil samples, which included: two consolidated direct shear tests on samples from the embankment fill and native granular soils; two consolidated drained (CID) triaxial tests, with unload-reload cycles on samples from the embankment fill and native granular soils; four incremental loaded (IL) consolidation tests; two long-term (LT) consolidation tests; four one-dimensional consolidation testing using constant rate of strain (CRS) on samples from the clay deposit, as well as a number of index and classification tests, consisting of grain size distribution, Atterberg limits, and water content determinations.

It is particularly worth noting the overconsolidation ratio (OCR) derived from the various consolidation tests, as shown in Table 1.

Based on the laboratory testing results on two clay samples from the 1970 original investigation, the OCR was estimated to be about 1.0 and 1.1, which indicated the clay deposit was normally consolidated to slightly overconsolidated, prior to the embankment construction.

As a comparison, based on the laboratory testing results from the 2019 investigation nearly 50 years after construction, the OCR values on eight clay samples range from about 0.8 to 1.4 (except for one sample with an OCR = 0.2, likely due to sample disturbance).

Using the sample quality designation system developed by *Lacasse et al*., (1985), the calculated strain required to reach the existing effective stress is shown to be greater than 4% for these samples, which indicates the samples could have been disturbed. However, the sample quality designation system developed by *Lacasse et al* was based on testing of overconsolidated clays (with OCR ranging from 1.4 to 5.0) and may not be valid for samples where primary consolidation is still ongoing (i.e., that are partially consolidated).

OCR close to but less than 1.0 could potentially indicate that the clay deposits (or portions of) are not fully consolidated under the current embankment loading (i.e., primary consolidation is not yet completed), which could be the case given the ongoing settlement observed at this site. However, given the inherent uncertainties associated with the sampling and testing, the actual degree of ongoing consolidation has been difficult to assess.

Table 1. Derived OCR values of tested clay samples

Borehole ID/ Sample Number	Type of Test	Sample Depth/Elevation (m)	OCR
18-601 / 11	IL	9.7 / 59.1	0.8
18-602 / 14	IL	14.2 / 54.8	1.0
18-603 / 16	IL	17.3 / 51.4	1.2
18-604 / 20	IL	23.5 / 45.3	0.8
18-601 / 15	CRS	15.5 / 53.3	1.0
18-602 / 17	CRS	18.6 / 50.4	1.4
18-603 / 12	CRS	11.0 / 57.7	0.2[2]
18-604 / 14	CRS	14.0 / 54.8	0.9
5 / 10[1]	IL	15.6 / 49.8	1.1
6 / 6[1]	IL	7.9 / 57.3	1.0

[1]laboratory results from 1970 original investigation
[2]sample was likely disturbed

3.2 Settlement Modelling and Results

Based on the original investigation for this site, settlements in the order of 0.8 m, were predicted for the embankments, with about 0.3 m anticipated within the first 1 to 2 years.

In order to estimate the settlements of the clay deposit that had occurred since the original construction in 1970's and to predict the future settlement, analyses were carried out using Settle3D software (Rocscience©).

The analyses incorporated the existing embankment geometry and construction sequence. The parameter inputs to the initial analyses are summarized in Table 2 and described below.

1) Primary Consolidation Parameters: OCR = 1.0, Cc = 1.35, Cr = 0.0675 and e_o = 2.0. These parameters were estimated based on the average values from the results of the 1970 investigation (i.e., prior to the bridge construction), in order to estimate the magnitude of settlement of the clay deposit since the original construction.

2) Secondary Consolidation Parameters: C_α = 0.004 to 0.006 and C_α / C_c = 0.02. These parameters (which were not available from the original investigation) were estimated based on the results of the LT oedometer consolidation testing during the 2019 investigation and are considered reasonable in view of past experiences on similar sites and published values.

3) Time-dependent Consolidation Parameters: C_{vr} = 3 x 10^{-3} cm^2/s and C_v = 6 x 10^{-4} to 6 x 10^{-5} cm^2/s (later refined as described further below). These parameters (which were not available from the original investigation) were estimated based on the IL oedometer consolidation testing from the 2019 investigation. These parameters are difficult to estimate with accuracy while at the same time the settlement analysis results are highly sensitive to the selected values.

4) Groundwater Levels: The groundwater levels were estimated based on the measurements from the 2019 investigation, since the water levels from the 1970 investigation were based on open hole readings (which are likely not representative of stabilized water levels) and there are no reasons to assume that the groundwater conditions have changed significantly since the original construction. The groundwater measurements indicate a downward hydraulic gradient exists within the clay deposit. The clay deposit was therefore modelled as a composite of four soil layers within that stratum, each with its own static water level, in order to approximate that hydraulic gradient. The final effective stress of the clay deposit was computed using the *Boussinesq*'s distribution method (*Boussinesq*, 1883).

5) Construction Sequence: Based on the foundation records, and discussions with a former MTO Chief Foundation Engineer familiar with the original construction, it was assumed that the approach fills were preloaded with 1 m of surcharge for a period of four months before the commencement of the bridge construction.

Table 2. Soil parameters for initial analyses

Soil Parameters	Clay to Silty Clay
e_o	2.0
C_r	0.0675
C_c	1.35
OCR	1.0
C_α	0.004 to 0.006
C_α / C_c	0.02
C_{vr}	3 x 10^{-3} cm^2/s
C_v	6 x 10^{-4} to 6 x 10^{-5} cm^2/s

[1]e_o Initial void ratio
[2]C_c Compression index
[3]C_r Recompression index
[4]OCR Overconsolidation ratio
[5]C_α Secondary compression index
[6]C_{vr} Coefficient of consolidation at recompression
[7]C_v Coefficient of consolidation at compression

As previously noted, the recent survey results indicated that settlements ranging in magnitude from about 150 to 170 mm appear to have occurred since the 2005 approach slab rehabilitation. However, no other settlement records are available for this site to verify the magnitudes of settlement that have actually occurred since the original construction. A sensitivity analysis was therefore carried out using the range of time-dependent consolidation parameters as noted above (C_v = 6 x 10^{-4} to 6 x 10^{-5} cm^2/s) and the results were compared to the field observations.

Based on the results of the sensitivity analysis, the estimated settlement at the centreline of the approach embankments during the period from 2005 to 2019 ranged from about 110 to 150 mm (corresponding to 65 to 70% of consolidation), which are similar in magnitudes to the settlements indicated by the survey. This suggests a reasonable model calibration with the field observations.

However, there are a number of uncertainties associated with the initial analysis that was carried out, which include the degree of consolidation that has occurred to date, the parameter values selected for an unconsolidated deposit, limited survey data and changes at the approaches (e,g., padding) that may have been completed during maintenance to correct sags at the approach slabs.

Given the above uncertainties, the results of the initial settlement analysis may not be representative of the actual ground behaviour and the estimates are possibly conservative. Therefore, consideration was given to the following two scenarios, where 1) the primary consolidation of the clay deposit has been completed and 2) the primary consolidation has nearly been completed (both achieved by adjusting the magnitude of C_v and/or C_{vr} within the model).

The results of the additional analyses are summarized below.

1) Assuming C_v and C_{vr} = 3.6 x 10^{-3} cm^2/s and other parameters remain constant, the primary consolidation would have been completed (i.e., the clay site would have reached 100% of consolidation in 2019). For this scenario, the estimated settlements are in the range of 30 to 60 mm between 2005 and 2019 (which is less than half of the magnitude based on the field observations during the same period).

2) Assuming C_v = 1.35 x 10^{-3} cm^2/s, C_{vr} = 3 x 10^{-3} cm^2/s and other parameters being constant, the primary consolidation would be almost completed at this site (i.e., the clay would be at 90 to 95% of consolidation in 2019) and some degree of secondary compression is occurring. For this scenario, the estimated settlements are in the range of about 90 to 150 mm which is similar in magnitude to the field observations during the same period.

Considering the limitations as previously noted and past experiences with similar project sites, it is believed that the scenario where the primary consolidation is almost completed (i.e., the site currently at 90 to 95% consolidation) would be the most likely, while the other two cases are considered as the worst case (65 to 70% consolidation completed) and best case (100% consolidation completed) scenarios, respectively.

The next step was to estimate the magnitude of the ongoing settlement if pavement rehabilitation is to commence in 2020. Using the same model input parameters (as described above) and assuming no additional load will be applied to the underlying clay deposit, the total settlements for the next 20 years (the typical rehabilitation cycle for bridge approaches) and 30 years (the end of the existing bridge life span) were estimated for each of the scenarios, which are provided in Table 3.

Table 3. Summary of settlement analysis results

Time Period	Best Case Scenario (100% Consolidation Completed)	Most Likely Scenario (90 - 95% Consolidation Completed)	Worst Case Scenario (65 - 70% Consolidation Completed)
20 years	12 – 30	70 – 110	120 – 160
30 years	15 – 35	90 – 150	160 – 240

[1] Assume pavement rehabilitation commences in 2020

These settlements would be entirely differential relative to the overpass structures and would, in general, exceed the usual values accepted by MTO for freeway approaches to bridges.

4. SOIL-STRUCTURE MODELLING

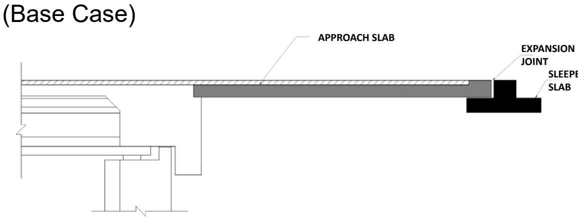
The purpose of the finite element (FE) modelling was to investigate the interaction between the structural components and the geological materials with the aim of understanding the behaviour of the approach slab and the deformations (settlement) of the pavement surface.

Modelling was undertaken for four of the six alternative configurations (see Section 4.1) of approach slabs. The analyses were based on the expected consolidation settlement over the next 20 years, truck loads based on CL-625-ONT, and horizontal expansion/contraction of the bridge deck due to seasonal thermal effects.

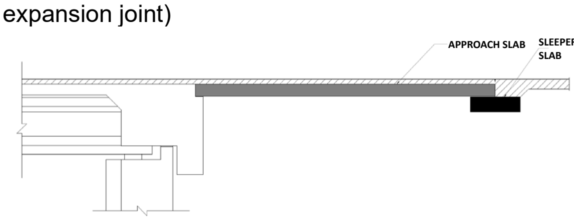
4.1 Approach Slab Alternatives

A literature search resulted in six configurations for consideration:

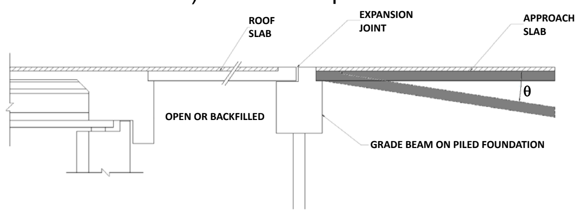
1. Approach slab with sleeper slab and expansion joint (Base Case)



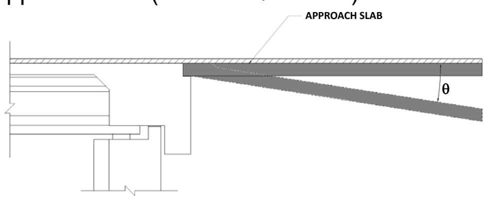
2. Approach slab with sleeper slab (no end dam / expansion joint)



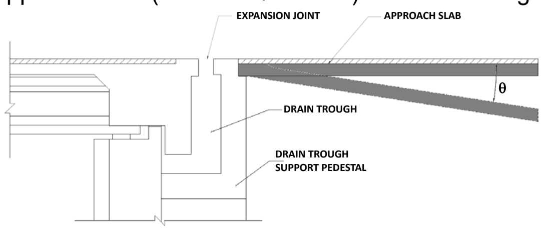
3. Roof slab with grade beam and approach slab (horizontal/inclined) without sleeper slab



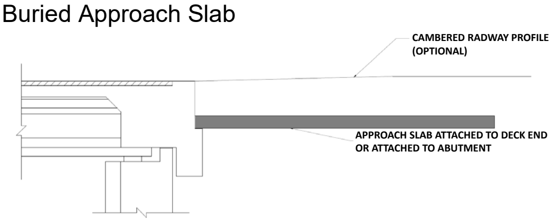
4. Approach slab (horizontal/inclined) without sleeper slab



5. Approach slab (horizontal/inclined) with drain trough



6. Buried Approach Slab



Multiple sub-configurations were analysed for each alternative, which considered 6 m and 12 m long approach slabs, combined with 1 m or 2 m sleeper slabs for the first two alternatives, and flat or inclined slabs for the last four alternatives.

4.2 Finite Element Model

The geometry and stratigraphy for the FE models were developed from the profiles shown in the original design. The stratigraphy was confirmed/updated based on the results of the recent geotechnical investigation carried out by Golder.

The modelling was carried out in two dimensions using the commercially available software package RS2 (Rocscience©), which allows for interaction of structural and geological elements (including contact elements) and a selection of constitutive models. A view of the full model is shown in Figure 1, where the different soil layers and the major components are labelled.

The details of the deck end and abutment were obtained from the drawings prepared for the 2005 rehabilitation. All components of the deck and abutment were modelled, including the girders, bearing seat, EVA foam, and elastomeric bearing strip. The piles underpinning the abutment and pier footings were modelled with springs with stiffness equivalent to the piles. Given that a two-dimensional model considers a section of unit thickness, components that are not continuous in the direction normal to the sections (e.g., girders, piles) were pro-rated to the thickness of the section.

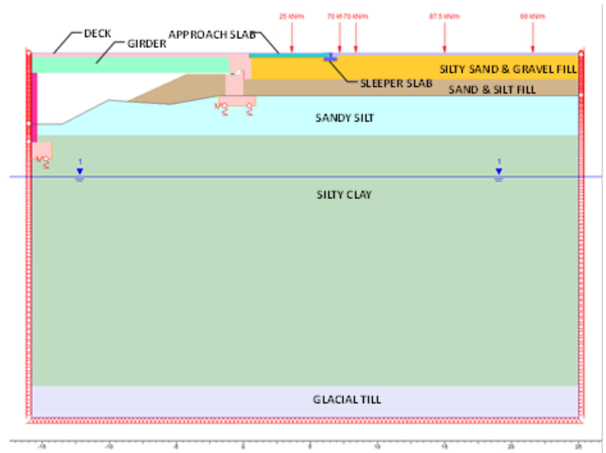


Figure 1. FE model showing stratigraphy and major components

4.3 Material Properties

4.3.1 Geological Materials

Properties for the geological materials were based on laboratory testing conducted as part of a recent geotechnical investigation carried out by Golder. The glacial till properties were based on Golder's experience from previous projects in the Ottawa area. Table 4 shows the material properties for the geological material used in the model.

In addition to the geological materials, the model includes interface between several structural components and the soil. These are represented in the model as contact elements and the adopted properties are as shown in Table 5. Frictional strengths are based on values in Table 1 of Chapter 3 of NAVFAC's DM-7.02 Foundations and Earth Structures.

Table 4: Geological material properties used in FE analyses

Material	Glacial Till	Silty Clay to Clay[1]	Sandy Silt to Sand and Silt	Sand and Silt (Fill)	Silty Sand and Gravel (Fill)
Unit Weight	20 kN/m^3	20 kN/m^3	20 kN/m^3	20 kN/m^3	20 kN/m^3
Poisson's Ratio	0.3	0.3	0.3	0.3	0.3

Material	Glacial Till	Silty Clay to Clay[1]	Sandy Silt to Sand and Silt	Sand and Silt (Fill)	Silty Sand and Gravel (Fill)
Young's Modulus	250 MPa	18 MPa	10 MPa	30 MPa	60 MPa
Failure Criterion	←---------- Mohr-Coulomb ----------→				
Material Type	←---------- Plastic ----------→				
Peak Tensile Strength	50 kPa	5 kPa	5 kPa	5 kPa	5 kPa
Peak Friction Angle	35°	30°	38°	40°	45°
Peak Cohesion	500 kPa	5 kPa	5 kPa	5 kPa	5 kPa
Residual Tensile Strength	50 kPa	5 kPa	5 kPa	5 kPa	5 kPa
Residual Friction Angle	35°	30°	38°	40°	45°
Residual Cohesion	500 kPa	5 kPa	5 kPa	5 kPa	5 kPa
Dilation Angle	5°	0°	0°	0°	5°
Effective stress ratio (in-plane)	0.43	0.5	0.43	0.43	0.43
Effective stress ratio (o-o-plane)	0.43	0.5	0.43	0.43	0.43

[1] Unit weight of silty clay to clay is increased during the model stages to replicate the effect of long-term consolidation settlement

Table 5: Properties used in FE analyses for contact elements

Interface	Concrete on Granular	Concrete on Concrete	Elastomeric strip	Roller	Gap
Kn (normal stiffness)	1e5 kPa/m	1e5 kPa/m	5440 kPa/m	1e5 kPa/m	~0 kPa/m
Ks (shear stiffness)	1e4 kPa/m	1e4 kPa/m	1813 kPa/m	~0 kPa/m	~0 kPa/m
Cohesion	0	0	-	0	0
Friction angle	35°	35°	-	0	0
Tensile strength	0	0	-	0	0

4.3.2 Structural Materials

Structural materials in the model include the concrete deck and deck end, approach and sleeper slabs, the asphaltic concrete layer (both winter and summer properties), the prestressed concrete girders (AASHTO Type 3 I-beam) and the piles.

Most of these elements are continuous in a direction normal to the modelled section; however, the girders and piles are discrete members and are installed at a specified spacing in the direction normal to the section. Therefore, the properties for these two elements in the model need to be pro-rated to the unit thickness represented by the modelled section.

The modulus of the girder material (concrete) was adjusted to satisfy both the flexural stiffness (EI) and the axial stiffness (AE). The unit weight of the girder was also adjusted in order to be pro-rated to that of an equivalent unit thickness. Table 6 shows the material properties for the structural elements represented in the model as continuous in the direction normal to the section.

Table 6: Properties used in finite element analyses for continuum elements

Material	Concrete (deck and slabs)	Girder[1]	EVA Foam	Asphaltic Concrete (summer)	Asphaltic Concrete (winter)
Unit weight	23 kN/m^3	3 kN/m^3	0.1 kN/m^3	20 kN/m^3	20 kN/m^3
Elastic modulus	27,805 MPa	3,475 MPa	0.1 MPa	1,000 MPa	15,000 MPa
Poisson’s ratio	0.3	0.3	0.3	0.4	0.4

[1] Elastic modulus and unit weight of girder were adjusted to reflect spacing

The piles are represented in the model as springs under the foundations of the piers and the abutment. The axial stiffness of the piles was calculated as follows:

$$K = \frac{AE}{L} \quad [1]$$

where A is the cross-sectional area of the pile, E is the elastic modulus of the pile and L is the length of the pile.

The axial stiffness of the piles was pro-rated to the model thickness taking into consideration the spacing of the piles, resulting in stiffnesses of 31,746 kN/m and K_{axial} = 55,294 kN/m for the abutment piles and pier piles, respectively. In the case of the battered piles, the axial stiffness was resolved into a vertical and a horizontal stiffness.

5 MODELLING RESULTS AND DISCUSSION

Modelling of the long-term consolidation settlement was undertaken separately using Settle3D (Rocscience©) as presented in Section 3.2. The FE model was set up to replicate this predicted settlement (in the order of 160 mm) by gradually increasing the unit weight of the silty clay to clay, after the construction and rehabilitation stages in the model were complete.

Figure 2 shows the ground settlement due to primary consolidation.

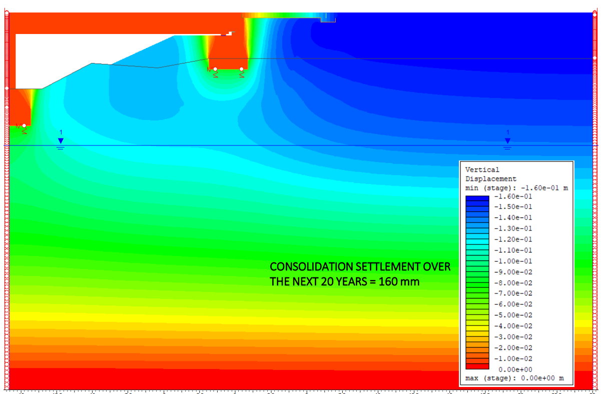


Figure 2. Replication of consolidation settlement expected over the next 20 years

It should be noted that the abutment and piers show negligible settlement as they are supported by piles driven to refusal. The ground near the abutment settles considerably less due to the interaction with the abutment; however, the full settlement can be felt within a few metres from the abutment.

5.1 Results Due to Consolidation Settlement

As a result of the large magnitude of estimated long-term consolidation settlement (on the order of 160 mm over a period of 20 years), all of the alternatives with a 6 m long approach slab resulted in a situation where the approach slab becomes simply supported. Figures 3 and 4 show the typical settlement of the approach slab and the ground underneath the slab for Alternatives 1 and 6.

The only alternatives that maintained partial contact between the approach slab and the ground are the 12 m long slab in an inclined configuration and both 12 m buried slabs (flat and inclined). However, the moments on the 12 m long slabs are considered excessive for the modelled slab thicknesses, even for the partially supported cases.

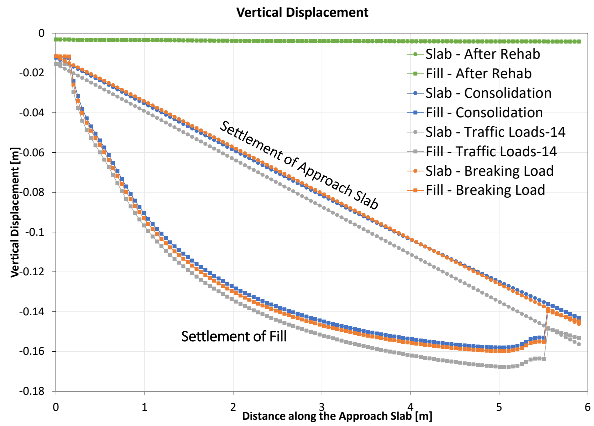


Figure 3. Alternative 1 – Vertical displacement of the 6 m approach slab and the ground underneath it

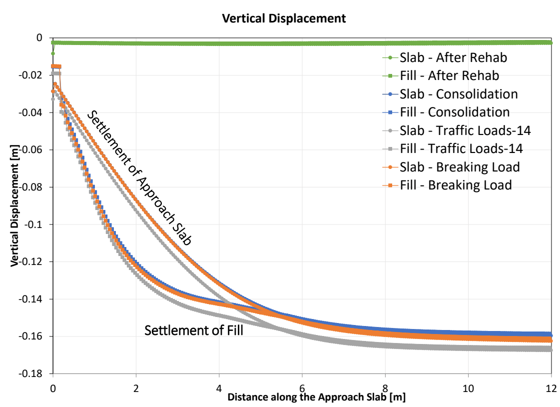


Figure 4. Alternative 6, inclined – Vertical displacement of the 12 m approach slab and the ground underneath it

5.2 Surface Horizontal Strains

The larger surface strains are due to the change in slope ('kink') at the end of the approach slab and due to the large consolidation settlement.

Alternative 1 is the only alternative with a gap (i.e., space for expansion without contact) between the approach slab and the end dam of the sleeper slab. This gap prevents strain to accumulate (see Figure 5). All other alternatives with flat surface approach slabs show surface strains in the order of 0.1 to 0.3%, depending on the slab length. Figures 5 to 7 show a comparison between Alternative 1 (with gap), Alternative 2 and Alternative 4 (inclined slab).

Alternative 2 shows strain concentrations both at the end of the approach slab and at the end of the sleeper slab. Alternative 4 (inclined slab) shows that the strain is distributed over a wider zone, rather than being concentrated in a single location. Figures 6 and 7 also show that, in the case of Alternatives 2 and 4 (both of which have no gap), additional strain will be induced by thermal expansion of the bridge in the summer, and a reduction in strain will result from contraction of the bridge in the winter.

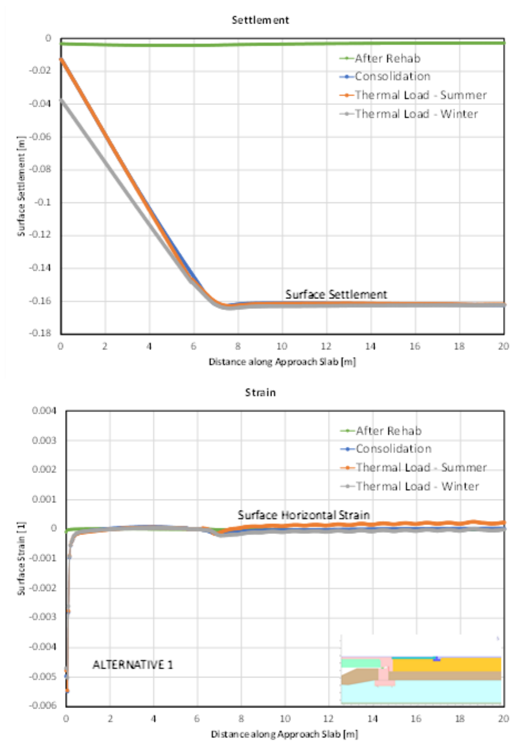


Figure 5. Alternative 1 – Comparison of surface settlement and lateral strains

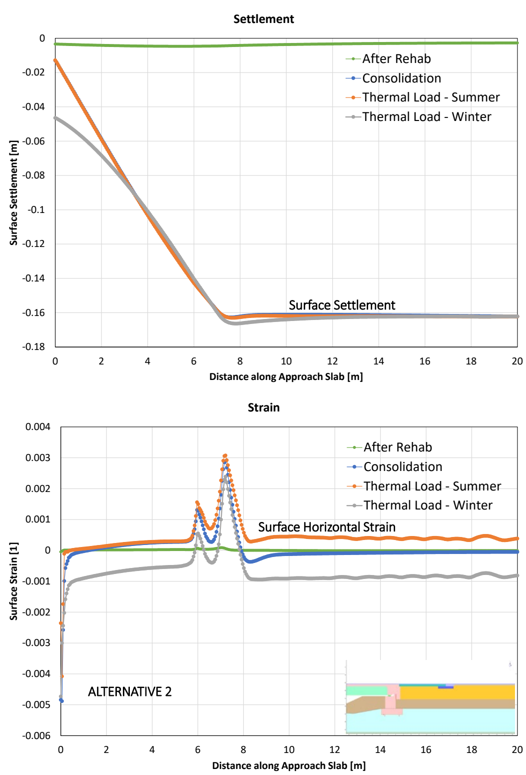


Figure 6. Alternative 2 – Comparison of surface settlement and lateral strains

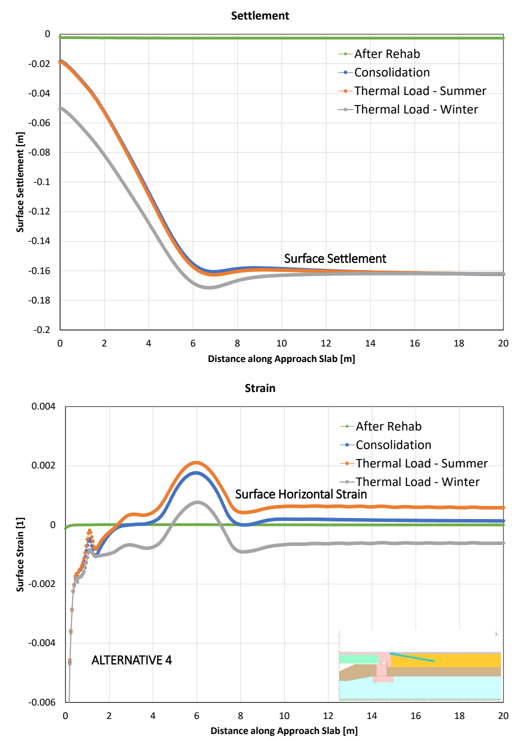


Figure 7. Alternative 4 – Comparison of surface settlement and lateral strains

5.2 Temperature Effects

A more detailed model for Alternative 4 with the 12 m long, inclined approach slab configuration was set up to better understand the effects of (thermal) temperature, separately from the settlement deformations. The thermal movements are based on an effective construction temperature of 15°C. Based on the detailed model, expansion of the bridge in the summer will result in a 10 mm displacement of the approach slab (ΔT = 35.6°C – 15°C = 20.6°C), and a contraction of 22 mm in the winter (ΔT = -32.8°C – 15°C = -39.8°C). Because the approach slab is tied to the deck end, the displacements due to temperature changes will be imparted to the approach slab.

Figures 8 and 9 show the vertical deformations and horizontal strains, respectively. The top graphic of each figure illustrates the behaviour in the summer and the bottom graph shows the behaviour in the winter. The results show that the expansion of the bridge will cause a rise (‘bump’) in the pavement at the end of the buried slab; conversely, contraction of the bridge will create a depression (‘trough’) at the end of the buried slab. The magnitude of the rise and depression is proportional to the lateral movement of the bridge.

5.3 Additional Analyses

Two additional models were analysed to evaluate the impact of reinforcing the subgrade with geogrids and the pavement with fiber modified asphalt. The analyses were conducted on Alternative 4, with the 1:12 inclined approach slab and basically demonstrated that the reinforcement of the subgrade and the pavement do not have an impact on the surface settlement magnitude; however they do show that the contact area between the end of the slab and the ground is increased.

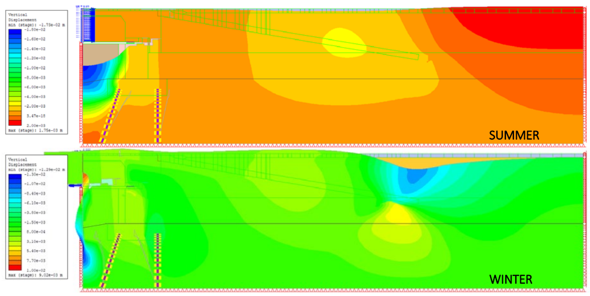


Figure 8. Vertical displacement contours as a result of expansion (summer) and contraction (winter) of the bridge deck end

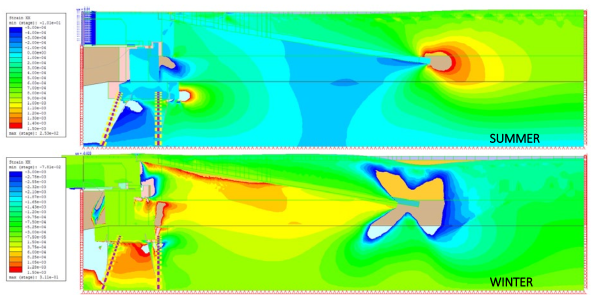


Figure 9. Horizontal strain contours as a result of expansion (summer) and contraction (winter) of the bridge deck end

6 OPTION SELECTED FOR DESIGN

Based on the work carried out above, and considering site constraints and constructability requirements, Alternative 4 with an inclined (1:12) slab was selected for the design of the approach slabs. This work is expected to be tendered in 2020.

The design for the currently planned approach slab rehabilitation includes instrumentation of the approach slab and embankments to assess the amount of settlement after the rehabilitation and the effects of that settlement on the approach slab and soil interaction. The planned instrumentation includes vertical and horizontal Shape Accelerometer Arrays (SAA) and pressure cells (to be installed under the new slabs) at each approach. This instrumentation will allow MTO to assess the effectiveness of the approach slab design.

7 CONCLUSIONS

Results of the analyses show that, for these specific site conditions, the behaviour of the approach slab is governed by the estimated large magnitude of consolidation settlements (on the order 160 mm, corresponding to 20 years), which makes it challenging to identify the 'best' configuration. Nevertheless, Alternatives 3 and 4 were considered to outperform the others in terms of surface horizontal strain and Alternative 4 was chosen for implementation.

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