Cofferdam Failure - Geotechnical Lesson Learned

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ABSTRACT
The existing Kabinakagami River Bridge located on Highway 11, approximately 32 km west of Hearst, Ontario, was replaced with a new bridge structure in 2008-2009. During construction of the West Abutment, the cofferdam failed and the construction was delayed. After the failure, the subsequent geotechnical investigation, commissioned by the Contractor was carried out in December 2008. Based on the results of that investigation, the Contractor contended that the borehole soil data shown in the Contract Documents was significantly different than what was actually encountered in the site during the sheet pile and H-pile installation. It was claimed that the existing soil conditions had contributed to problems encountered during the cofferdam installation and to its failure. Consequently, the third-party verification of the subsurface conditions in the area of interest was requested. To verify the subsurface conditions and clarify discrepancies in data collected during pre-failure and post-failure geotechnical investigations, the additional geotechnical investigation in the vicinity of the failed cofferdam was performed in September 2009, results of which are presented in this paper. The paper also includes the assessment of the original subsurface information available for the cofferdam design and suitability of that design as well as the assessment of installation procedures and sequences applied during the construction. The geotechnical lesson learned is discussed.

RÉSUMÉ
Le pont de la rivière Kabinakagami, situé sur la route 11, à environ 32 km à l'ouest de Hearst, en Ontario, a été remplacé par un nouveau pont en 2008-2009. Lors de la construction de la culée ouest, le batardeau a échoué et la construction a été retardée. Après la défaillance, l'enquête géotechnique suivante, commandée par le contractant, a été réalisée en décembre 2008. Sur la base des résultats de cette enquête, le contractant a affirmé que les données relatives au sol dans les trous de forage figurant dans les documents contractuels étaient très différentes de celles réellement rencontrées dans le site lors de l'installation des palplanches et des pieux en H. Il a été affirmé que les conditions du sol existantes avaient contribué aux problèmes rencontrés lors de l'installation du batardeau et à son échec. En conséquence, la vérification par une tierce partie des conditions du sous-sol dans la zone d'intérêt a été demandée. Afin de vérifier les conditions du sous-sol et de clarifier les divergences dans les données collectées au cours des enquêtes géotechniques effectuées avant et après la défaillance, les recherches géotechniques supplémentaires menées à proximité du batardeau en panne ont été effectuées en septembre 2009 et les résultats en sont présentés dans le présent document. Le document inclut également l'évaluation des informations originales disponibles sur le sous-sol pour la conception du batardeau, ainsi que l'adéquation de cette conception, ainsi que l'évaluation des procédures et séquences d'installation appliquées pendant la construction. La leçon géotechnique apprise est discutée.

1 INTRDUCTION

Kabinakagami River Bridge is located on Highway 11, approximately 32 km west of Hearst, Ontario. The bridge is a five-span structure with steel girders supporting a reinforced concrete deck. It is approximately 107 m long and 10 m wide. The bridge is oriented in an east-west direction and conveys one westbound lane and one eastbound lane of Highway 11 over the Kabinakagami River. It is reported that the structure was constructed in 1942, and that, the northern half of the bridge deck was replaced in 2002. The existing bridge structure is most likely supported on a combination of shallow and deep foundations. The East Abutment and Pier 4 are likely supported by shallow foundations placed on bedrock, while the West Abutment and Piers 1, 2, and 3 are likely supported by deep foundations. Highway 11 at the Kabinakagami River area is generally higher than the surrounding lands. It is built on shallow embankments with wide gravel shoulders. Figure 1 shows the existing Kabinakanagi River Bridge.

In 2007, the Ministry of Transportation (MTO) planned to replace the existing bridge structure at the Kabinakagami River with a new bridge structure. The new structure was planned to be constructed to the south of the existing structure. The new bridge was a three-span bridge constructed with pre-cast pre-stressed concrete girders and a reinforced concrete deck. The new bridge was planned to be supported also by a combination of shallow and deep foundations. The East Abutment and East Pier of the replacement bridge structure was designed to be founded on spread footings resting on the underlying bedrock, while the West Abutment and West Pier would be supported on piles driven to bedrock. The bridge was designed and constructed based on the results of geotechnical investigations at the site performed in 2007 (GI 2007) which was a part of the Contract Documents.

In 2008, the construction of the new bridge was commenced. However, in November 2008, during construction of the West Abutment, the cofferdam failed and the construction was delayed. After the failure, in December 2008, the Contractor commissioned additional borehole investigations (GI 2008) and based on the results, contended that the borehole soil data shown in the Contract Documents was significantly different than what was actually encountered in the site during the sheet pile and H-pile installation. The Contractor claimed that the existing soil conditions had contributed to problems encountered during the cofferdam installation and to its failure.

Figure 1. Kabinakanagi River Bridge (facing east)

To verify the subsurface conditions and clarify discrepancies, MTO engaged the third party to perform additional geotechnical investigations in the vicinity of the failed cofferdam in 2009 (GI 2009). The boreholes were strategically located around the West Abutment cofferdam, close to the previously drilled boreholes to facilitate comparison and to provide good coverage of the area. Figure 2 shows the locations of drilled boreholes around the cofferdam during the GI 2009 as well as during the GI 2007 and GI 2008.

Based on the finding during the GI 2009 the following key issues were discussed in this paper:

- Background and site stratigraphy
- Assessment of the subsurface conditions and any noted inconsistencies in the reported information from three geotechnical investigations.
- Assessment of the cofferdam design carried out by the Contractor, that was based on the conditions reported in the GI 2007.
- Commentary on the impact of subsurface conditions identified by the GI 2009.
- Review and assessments of the installation procedures and sequences applied, and the impact on the reported failure, if any.
- Comments and conclusions pertaining to the above issues.

2 BACKGROUND OF COFFERDAM FAILURE

The shoring and cofferdams for the abutments and piers of the new Kabinakagami River Bridge were designed by the Contractor based on the results of geotechnical investigations at the site performed in 2007 (GI 2007). According to shoring details for construction of the West Abutment, the cofferdam was 15.5 m long by 7.4 m wide as shown on Figure 2. The top of the cofferdam was designed to be at Elevation of approximately 244.6 m. The sheet piles as parts of the cofferdam were design to be embedded into the soil up to the Elevation of approximately 237.0 m. The excavation level inside the cofferdam was planned to be completed at Elevation of 239.5 m. The Elevation of the existing ground was approximately 243.6 m, while the groundwater level was estimated to Elevation 243.0 m. The shoring drawing Revision No. 9 showed the design of the cofferdam with top struts and waler.

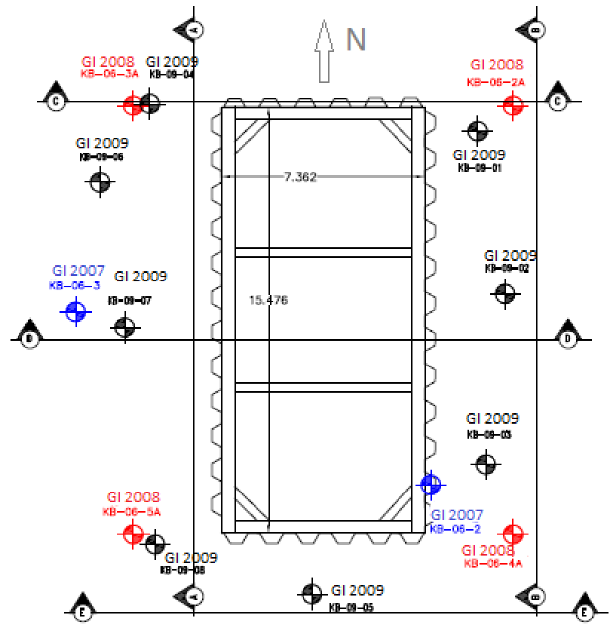


Figure 2. Borehole location plan around the cofferdam for three geotechnical investigations

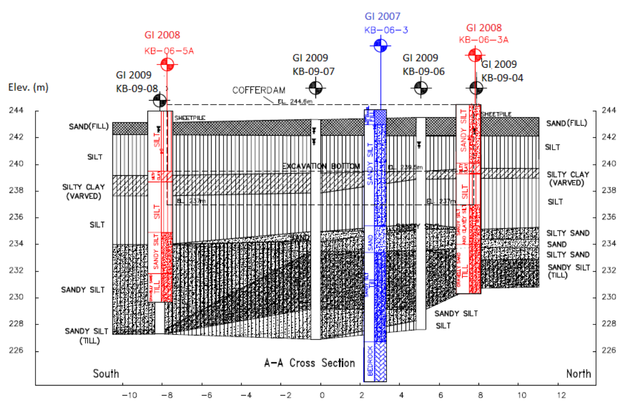
As mentioned above, during construction of the West Abutment the cofferdam failed. A chronological order of cofferdam failure events is reported below:

- October 24, 2008 - Installation of the sheet piling for the West Abutment cofferdam was commenced. During that time, the West Pier cofferdam was being de-watered and excavated to the required elevation for the tremie plug.
- October 29, 2008 – The West Abutment cofferdam was completed and the excavation commenced within the cofferdam. It is noted that only one top waler was installed which was according to the shoring drawing Revision No. 9.
- October 30, 2008 – The excavation of the West Abutment cofferdam was completed. The cross bracing for support of the walls was installed and welded into place.
- October 31, 2008 – It was noticed in the morning that the bottom material in the northwest corner of the West Abutment cofferdam rose slightly. The material visually consisted of greyish saturated clay with a sponge effect when walked on. The H-pile installation within the West Abutment cofferdam commenced that day.
- November 1, 2008 – It was observed that the sheet pile walls of the West Abutment were showing an

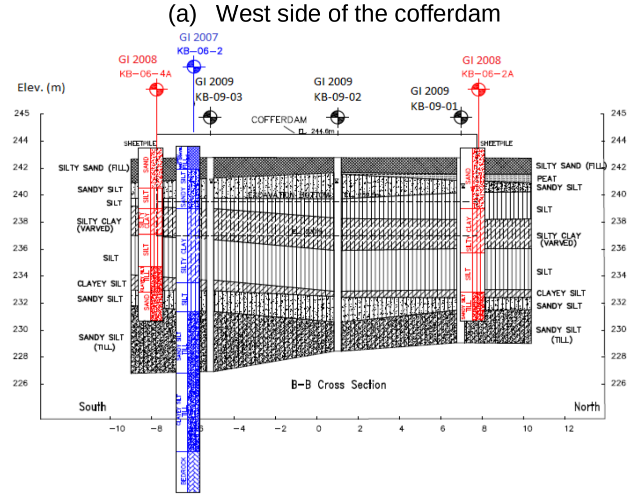
- inward movement, especially near the bottom. The contractor decided to install a second waler closer to the bottom of the cofferdam.
- November 2, 2008 – Distress in the soils (cracking and sinkholes) were observed within adjacent limits surrounding the cofferdam. The site foreman elected to suspend operations at the West Abutment until further notice.
- November 4, 2008 – Ministry of Labour was on-site and issued a stop-work order for entire west side.
- November 25 and 26, 2008 – Remedial work performed to brace the cofferdams. The item work of installing the H-piles at the West Abutment commenced that day.

3 SITE STRATIGRAPHY

The stratigraphy of the area around the failed cofferdam generally consisted of several units of loose to compact sandy and silty soils and soft to firm, low plastic silty clay overlaying a dense glacial till. The bedrock was encountered at Elevations in the range of approximately 220.9 m and 226.7 m. Figure 3 shows the stratigraphy at the site define by three geotechnical investigations.



(a) West side of the cofferdam



(b) East side of the cofferdam

Figure 3. Soil stratigraphy at the site

The following assessments are made following a review of pertinent documentations of subsurface conditions as developed by three investigations:

a. The site is undertaken by complex/variable deposits reflective of the geological setting. Variations in composition occur over relative short distances both horizontally and vertically.
b. The documentation of subsurface stratigraphy illustrated discrepancies among the reports/records presented by three investigations. These discrepancies are:
   - GI 2007 did not identify the silt clay layer in their BHs near the north-west corner of the cofferdam near Elevation 239 m. This layer was identified in GI 2008 and GI 2009, although thickness and levels differ slightly (refer to Figure 3.a).
   - GI 2007 identified a thicker layer of silty clay extending to below Elevation 234 m in the south-east corner. GI 2008 and GI 2009 borings in the vicinity identified the material below Elevation 237 m as silt (refer to Figure 3.b).
   - GI 2008 did not identify a thin layer of sand near Elevation 234 m in their BHs at north-west corner, while both GI 2007 and GI 2009 documented this layer (refer to Figure 3.a).

GI 2009 identified two water bearing strata separated by the low-permeable silty clay layer (an upper and lower water bearing strata) with possible different groundwater regimes. Both GI 2007 and GI 2008 reported only one water bearing stratum.

3 ASSESSMENT OF ORIGINAL COFFERDAM DESIGN

An assessment of the original cofferdam design developed on the conditions reported by GI 2007, was performed by the third-party including stability analyses related to kick-out at the toe of the sheet pile wall, basal stability analyses (piping and heave) and liquefaction potential analyses. According to reviewed documentation the shoring system included sheet piles extending to a depth of 7.6 m (Elevation 237 m) with a strut at Elevation 243.6 m, i.e., 1m below the top of sheet pile wall.

3.1 Stability Analyses

The stability analyses related to kick-out at the toe of the sheet pile wall considered two cases to account for the natural variability of soil strata over the site. Case 1 and Case 2 are chosen to represent two typical soil strata at the north-west corner and at the south-east corner of the cofferdam, according to GI 2007 borehole logs.

a) Case 1 - The deposit soil consists of silt, according to KB-06-3. Within Case 1 two sub-cases, Case 1a and Case 1b were distinguished. Case 1a considers the Elevation of ground surface of 244.1 m reported on KB-06-3, while Case 1b takes into account the average Elevation of ground surface of 243.6 m identified on shoring drawings. In both cases, the ground water level at Elevation 242.6 m was assumed.
b) Case 2 - The deposit soil consists of silt above the excavation bottom, and silty clay between the

excavation bottom and the sheet pile tip, according to KB-06-2. The ground water level at Elevation 242.6 m was also assumed.

Soil parameters utilized in this analysis include: bulk soil unit weight, γ, effective friction angle for silt, ϕ', and undrained shear strength for silty clay, c_u. The unit weights of silt and silty clay were taken as 18 kN/m^3 and 20 kN/m^3, respectively. The effective friction angle for silt was taken as 28 degree as recommended in the GI 2007 report. The note shown on drawings of design shoring systems suggested that the effective friction angle for silt was taken as 32 degree for their design of sheet piles. The design undrained shear strength for silty clay of 50 kPa was recommended by the GI 2007, and that value was considered in the stability analyses. However, the undrained shear strength ranged from 20 kPa to 55 kPa as measured during the GI 2008 and GI 2009 afterwards. Considering these variations in the reported soil parameters, a parameter sensitivity study was performed for this assessment to evaluate the reliability of the stability results with respect to the uncertainty of soil parameters measured from in-situ and/or lab tests. In this study, four (4) different effective friction angles for silt (i.e. 25, 28, 32 and 35 degrees) and two (2) different undrained shear strengths for clay (i.e. 30 and 50 kPa) were selected and used in the assessments.

The stability analyses used the approach recommended in the Canadian Foundation Engineering Manual (CFEM 4th Edition, Chapters 24 and 26). Tables 1.a and 1.b summarizes the results of stability analyses.

Table 1. Results of stability analyses

(a) Case 1

ϕ' of Silt (degree)	Factor of Safety	
	Case 1.a	Case 1.b
25	0.40	0.43
28	0.48	0.52
32	0.62	0.67
35	0.75	0.81

(b) Case 2

ϕ' for Silt (degree)	Cohesion c_u for Silty Clay (kPa)	Factor of Safety Case 2
25	30	1.04
28		1.10
32		1.20
35		1.27
25	50	4.33
28		4.61
32		5.00
35		5.36

As can be seen from the tables, the results of the sensitivity analyses suggest that the original cofferdam design for the excavation in the sandy silt (KB-06-3) was not safe with only one strut. The factor of safety for toe kick-out was less than 1.0 in all cases. On the other hand, the factor of safety against sheet pile toe kick-out in the soil deposit consisting of layers of silt and silty clay (KB-06-2) was above 1.0 (if $c_u \geq 30$ kPa) indicating more favorable soil conditions for sheet pile stability. The results of the analyses show also that the differences in the values of the factor of safety for two different ground surface elevations, Elevation 244.1 m reported by GI 2007 (KB-06-3) and Elevation 243.6 m reported by shoring drawings are small and do not materially affect the conclusions.

4 ASSESSMENT OF POTENTIAL FOR BASAL FAILURE IN THE COFFERDAM

Since excavation for the failed cofferdam was carried out below groundwater levels, instability of the base could potentially result from the differential hydrostatic pressures. Factors which influence these problems are depth of penetration of sheeting, the head of water and the size of the excavation, and these factors are included in most theoretical studies of the flow of water into sheeted excavations. However, these studies were mostly concerned with homogeneous soil conditions, and are, therefore, of limited applicability at this site.

Many case studies (Maryland(1953), Milligan and Lo (1970) and Baur et al. (1980)) show that the most important factors influencing the stability of the excavation base are the groundwater and detailed soil conditions at the site. For example, the presence of coarse sand layers under excess hydrostatic pressure makes flow in fine material more nearly vertical and generally increases seepage gradients in the fine layer compared to the homogeneous soil condition. It is recommended in practice, that when the fine/coarse layer interface is at a depth below the sheeting tips greater than the width of the excavation, then the case of a homogeneous sand stratum applies. However, as the fine/coarse layer interface approaches the sheeting tips more critical conditions will result and the head necessary to cause piping decreases to a very low value. Where the permeability ratio $k_t/k_b{}^2$ is large (more than 10) (where k_t and k_b are hydraulic conductivities of the top and bottom layer, respectively), the critical head sufficient for failure is equivalent to the submerged weight of column of soil between the interface and the bottom of excavation.

As shown on Figure 3, a stratum of soft to firm, varved, silty clay was identified at the site for the West Abutment of the new Kabinakagami River Bridge by all investigators. This stratum was identified by the GI 2008 and GI 2009 in all boreholes drilled around the failed cofferdam. A 5.5 m thick silty clay layer was reported in KB-06-2 (GI 2007) at Elevation from 239.0 m to 233.5 m at the east side of the cofferdam, while no silty clay was identified in KB-06-3 (GI 2007) at the west side of the cofferdam. The distance between these two boreholes is approximately 12 m, as shown in Figure 2. Assuming these records were correct, it is reasonable to assume a transition thickness of silty clay layer between these two boreholes, that varies from about 5.5 m at the east side to zero at the west of the cofferdam. Noting sample intervals, it is possible that a relatively thin layer of silty clay may have been missed during sampling for Borehole KB-06-3.

GI 2008 and GI 2009 identified the stratum of varved, silty clay about 0.6 m (area of the north-west side of the cofferdam) to 3 m (area of the south-east side of the cofferdam) thick underlain by loose silt at the site, which in some cases, contained pervious layers of sand. The thickness of these sandy layers varied from a fraction of a centimeter to 2 m recorded at the north-west side of the cofferdam by the GI 2009. Based on these findings it is reasonable to estimate that the fine/coarse layer interface in the area of the north-west corner of the cofferdam, where the first signs of instability appeared, was at Elevation of approximately 235.4 m. The sheeting tip was at Elevation 237 m. Therefore, a depth below sheeting tips was 1.6 m, which is less than the width of excavation (~7.4 m). This suggests that method of analyses for potential seepage using a homogeneous sand stratum should not be applied in this case.

In the assessment of foundation basal heave stability, the two most critical cases were studied to evaluate the potential for a basal failure:

a. Excavation in a sandy silt layer (k_t~10^{-6} m/s; estimated using grain-size data) overlaying a sand layer (k_b ~ 10^{-4} m/s; estimated using grain-size data) (KB-06-03) – Potential for piping in the sandy silt which is relatively permeable (drained condition).
b. Excavation in a silty clay layer (k_t~10^{-9} m/s; estimated using grain-size data) overlaying a sand layer (k_b ~ 10^{-4} m/s; estimated using grain-size data) – Potential for basal heave in the silty clay which is low permeable (undrained condition). This is considered to be the more likely condition based on a review of all subsurface data available.

4.1 Potential for Piping in Sandy Silt Below the Excavation

Figure 4 shows the stratigraphy considered to assess the potential for piping in the case the excavation for the cofferdam occurred in a sandy silt layer overlaying a sand layer. The soil condition assumed in this case is that there was not the low permeable silty clay layer in the examined area. The groundwater conditions in the area of the excavation just prior to failure are difficult to determine, so it is assumed that the groundwater level was between Elevation 241.7 m and 242.6 m (reported in all geotechnical reports). The bulk unit weight of native sand and silt, γ, is assumed to be equal to 18 kN/m^3.

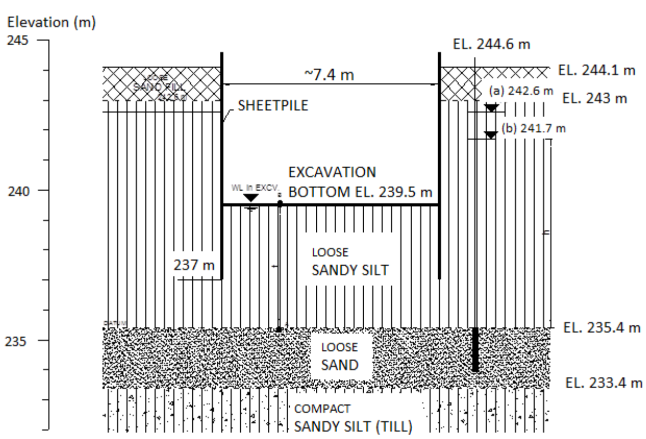


Figure 4. Cofferdam excavation in sandy silt overlaying a sand layer (KB-06-3, GI 2007)

Table 2. Summary of calculation for assessment of the potential for piping in the sandy silt below the excavation

Parameters	Elevation of Groundwater in Piezometers	
	242.6 m	241.7 m
Total Hydraulic Head in Point A h_A (m)	7.2	6.3
Total Hydraulic Head in Point B h_B (m)	4.1	4.1
Thickness of Sandy Silt Layer t (m)	4.1	4.1
Hydraulic Gradient i	0.75	0.54
Critical Hydraulic Gradient i_{cr}	0.84	0.84
Factor of Safety	1.12	1.5

Table 2 presents the summary of the calculation for the potential of piping. As can be seen, the factor of safety against piping for the given hydro-stratigraphic conditions is between 1.12 and 1.5 suggesting the potential for piping failure in the cofferdam. The Canadian Foundation Engineering Manual (CFEM) requires a minimum factor of safety of 1.2 against base instability for basal heave in layered soils which consist of soft to medium stiff cohesive soils and loose sand (CFEM, 4th Edition; Section 26.11.1.3; Pg. 411) or/and 1.43 against base heave of layered soils due to water pressures confined by intervening low permeability soils (CFEM, 4th Edition; Section 22.3.1.1; Pg. 341). The table also illustrates the sensitivity of the actual groundwater level to the factor of safety.

4.2 Potential for Basal Heave of the Excavation in Silty Clay

Figure 5 shows the stratigraphy considered to assess the potential for basal heave of the bottom of the excavation if the cofferdam is excavated in a silty clay layer overlaying loose silt and sand layers. At the site, the silty sand layer was approximately 0.6 to 2.3 m thick. Again, it is assumed that the groundwater level was between Elevation 241.7 m and 242.6 m. The bulk unit weight, γ, is assumed to be equal to 18 kN/m^3 for native sand and silt, and 20 kN/m^3 for silty clay.

Table 3 presents the summary of the calculation for assessment of the potential for basal heave of the excavation in silty clay. The results presented in Table 3 show that the factor of safety for basal heave is below 1.0 indicating that the 0.6 m thick silty clay layer was not thick enough to prevent basal heave of the excavation which is consistent with observations after the failure. It appears that the thickness of the silty clay layer should be more than 1.6 m (to satisfy the CFEM requirement of FS=1.2 against base instability for basal heave in layered soils which consist of soft to medium stiff cohesive soils and loose sand (CFEM) or 2.0 m (to satisfy the CFEM requirement of FS=1.43 against base heave of layered soils due to water pressures confined by intervening low permeability soils (CFEM) to avoid the potential of basal heave. If the silty clay layer below the excavation bottom was 0.6 m thick, the maximum hydraulic pressure head allowable during excavation should not exceed the Elevation of 239.9 m to satisfy the CFEM requirement of FS=1.2 or 239.75 m to satisfy the CFEM requirement of FS=1.43. To allow safe excavation of the cofferdam, the weight of soil/material at the base should be increased, and/or the level of the hydraulic head in the water bearing soil should be lowered.

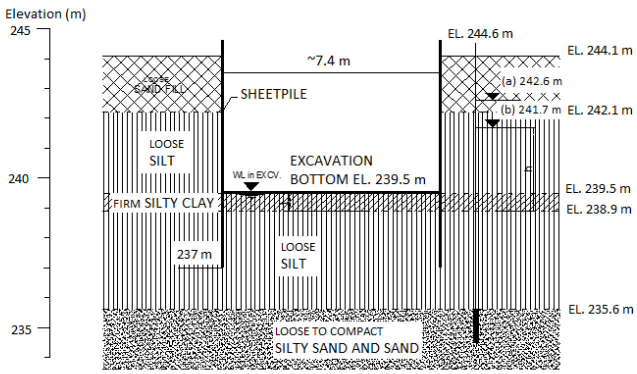


Figure 5. Cofferdam excavation in silty clay with lower more permeable silt and sand

Table 3. Summary of calculation for assessment of the potential for basal heave of the excavation in silty clay

Parameters	Elevation of Groundwater in Piezometers	
	242.6 m	241.7 m
Hydraulic Pressure Head Above Bottom of Silty Clay Layer h (m)	3.7	2.8
Thickness of Silty Clay Layer t (m)	0.6	0.6
Factor of Safety	0.33	0.44
Minimum Thickness of Silty Clay Layer for FS=1.2 or [1.43]* t_{min} (m)	2.2 [2.6]	1.6 [2.0]
Maximum Hydraulic Pressure Head Allowable During Excavation for FS=1.2 or [1.43]* h_{max} $(m)_r$	1.0 [0.85]	1.0 [0.85]

* - FS as required in CFEM

It should be noted that the GI 2007 Foundation Investigation and Design Report recommended the need to account for basal heave due to flow of water beneath the sheet piling in the design of shoring. It is recommended that basal heave and seepage be controlled by extending the shoring below the proposed excavation depth and placing a working mat of concrete, i.e., tremie concrete or a mud mat, at the base of the excavation. Dewatering is recommended to be carried out from within the shoring. If the geometry of the foundations precludes the use of tremie concrete to control groundwater flow, dewatering the excavation with well points was suggested by installing a second set of sheet piles around the first set and installing the well points between the two sets of sheet piles.

4.3 Liquefaction Potential Analysis

Liquefaction in this analysis refers to a sudden loss in strength of soil due to cyclic/vibrational loading effects. The loss arises from a tendency for soil to contract under cyclic/vibrational loading, and such contraction may lead to an increase of excess pore water pressure and decrease of effective stress. As a result, the strength reduces to zero and the soil behaves as a heavy liquid.

As shown in Figure 3, silty clay and lower silt layers dominate in the passive zone of the cofferdam. Consequently, the liquefaction responses of these two layers are critical for the cofferdam stability. Thus, this liquefaction potential assessment was focused on silty clay and lower silt layers.

Silty clay and lower silt are classified as fine-grained soils having a significant portion (over 95%) of fines passing through #200 sieve.

To delineate liquefaction susceptibility of the fine-grained soils, this analysis adopted the empirical criteria that recommended in the Canadian Foundation Engineering Manual (CFEM 4th Edition; Chapter 6; Pg. 111):

a) $w/w_L \geq 0.85$ and $I_P \leq 12$: Susceptible to liquefaction or cyclic mobility

b) $w/w_L \geq 0.80$ and $12 \leq I_P \leq 20$: Moderately susceptible to liquefaction

c) $w/w_L < 0.80$ or $I_P \geq 20$: No liquefaction or cyclic mobility

Based on the above criteria, two evaluation charts presented on Figures 6 and 7 were made to assess the liquefaction potential of silty clay and lower silt, based on the Atterberg limit test results obtained in the GI 2009. Figure 6 shows four (4) data points for silty clay on the liquefaction assessment chart. It can be seen that one data point falls in the liquefaction susceptible zone, one in the non-liquefaction zone, and two in the moderate liquefaction zone. It appears that the silty clay layer is, in general, “moderately susceptible” to liquefaction. Figure 7 shows that lower silt is “susceptible” to liquefaction.

In addition, Figure 8 evaluates liquefaction potential of silty clay based on data from the GI 2007 and GI 2008, suggesting that silty clay is “moderately susceptible” or “susceptible” to liquefaction. This is consistent with the assessment based on the test results from the GI 2009.

As a conclusion, the silty clay and lower silt are susceptible to liquefaction under vibration loads. Thus, the vibration introduced by the pile-driving might lead to a significant loss of strength in silty clay and lower silt layers. Consequently, the factor of safety of the cofferdam stability could decrease due to pile-driving.

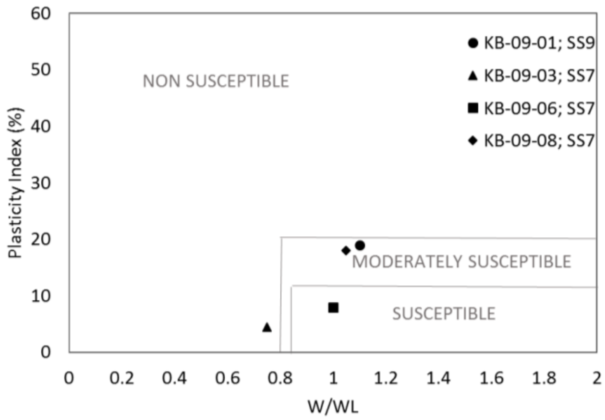


Figure 6. Liquefaction assessment on silty clay (data from GI 2009)

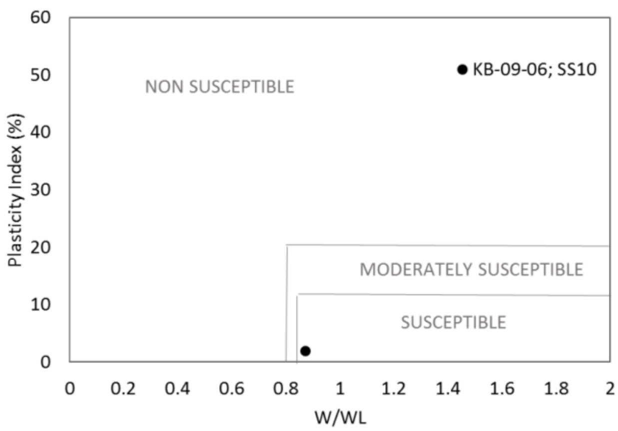


Figure 7. Liquefaction assessment on lower silt (data from GI 2009)

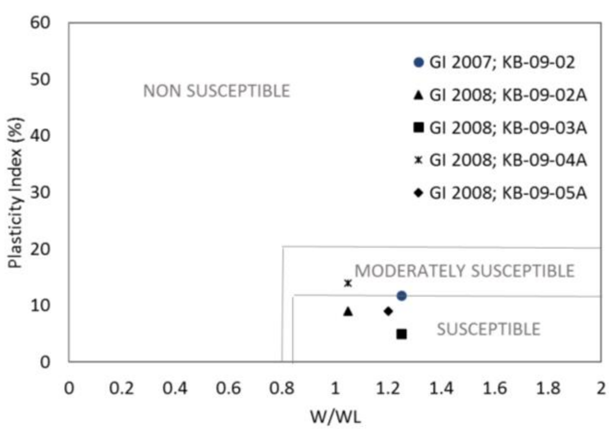


Figure 8. Liquefaction assessment on silty clay (data from GI 2007 and GI 2008)

5 REVIEW AND ASSESSMENT OF INSTALLATION PROCEDURES AND SEQUENCES APPLIED DURING THE CONSTRUCTION

The installation report noted that installation of the sheet piling for the West Abutment cofferdam commenced on October 24, 2008. Excavation within the cofferdam started 5 days after. It is noted that only one top struts and waler was installed in accordance with the shoring drawing. Based on information provided excavation was taken to final grade at approximately 4.5 m depth within two days. The cross bracing for support of the walls was installed and welded into place. It is not clear whether the bracing was installed as per instructions in the shoring drawings or whether this was completed after excavation to full depth. The latter would infer that the sheeting would have been acting as a cantilever for the full height of excavation for a period of time until the bracing was installed. The next day it was noticed that the base of excavation heaved in the area of the north-west corner. However, the H-pile installation within the West Abutment cofferdam proceeded that day. It was not reported that tremie concrete or a mud mat was placed at the base of the excavation to control seepage, as it was recommended in the Geotechnical Foundation and Design Report of GI 2007. The day after, it was observed that the wall of the cofferdam moved in towards the excavation at the level of the base. Consequently, it was decided to install a second waler closer to the bottom of the cofferdam. Distresses in the soils (cracking and sinkholes) were observed within adjacent limits surrounding the cofferdam. No further foundation piles were driven until remedial work was executed to brace the cofferdam.

Considering the installation procedures and sequences applied during construction of the West Abutment cofferdam, the following issues are deemed relevant:

- It is possible that the Contractor excavated to full depth without installation of any strut support, contrary to the sequence directive in the design shoring drawings. This would add to the potential for cofferdam distress.
- It is not readily apparent from the design drawings what specific seepage or dewatering measures or hydrostatic head control were required for this cofferdam. No measures were instituted by the Contractor.
- It is not apparent whether the Contractor requested designer representatives to attend the site for field review for this cofferdam in accordance with the Note ‘K’ of the shoring drawings.
- The third party has not been able to independently verify the actual depth of excavation and length of sheeting installed.

6 DISCUSSION AND CONCLUSIONS

Discussion and conclusions of the assessments performed by the third party for this project are presented below.

(A) Soil Discrepancies

The third-party investigation (GI 2009) and review of the existing data have identified discrepancies in the reported subsurface stratigraphy amongst the submissions from three geotechnical investigations. These discrepancies are:

a. The Contract Documents (GI 2007) did not identify the silty clay layer near Elevation 239 m in the north-west section of the cofferdam (refer to KB-06-3).

b. The Contract Documents (GI 2007) reported a thicker layer of silty clay extending to below elevation 234 m in the south-east corner (KB-06-2).
c. The geotechnical investigation after the cofferdam failure (GI 2008) commissioned by the Contractor did not identify sand near Elevation 234 m in their Borehole KB-06-3A (north-west corner).
d. Groundwater regime at the site identified was less detailed in the GI 2007 and GI 2008 documentations compared to GI 2009 records. Here it is noted that groundwater measurements recorded by the third party (GI 2009) were carried out after the installation of the cofferdam.

In making comparisons, it is important to note that the information obtained during the GI 2007 was carried out as part of a foundation investigation for supporting the bridge design with the associated standard of care expected for such assignments. On the other hand, the GI 2008 and GI 2009 undertaken are, in essence, 'forensic' investigations (i.e. after a failure event) where programs would be expected to be more detailed and geared to specific differentiation.

Further, the stratigraphic sequences at the site are complex, with vertical and spatial variability over relatively short distances that are typical of the fluvial geologic setting. The nature of the materials in these sequences ranges from low-permeable silty clay to sand with much higher level of permeability. The groundwater level is near grade level and the site is quite close to the river. Standard borings may not fully determine requirements for groundwater control in these layered deposits below the groundwater level. Carefully controlled tests using pumped and observation wells would better defined the complex groundwater regime and yield the quantitative data on groundwater volumes, pressure and relationships that are necessary to adequately engineer construction dewatering approaches and systems.

(B) Suitability of the Original Cofferdam Design

Assessments of the original subsurface information provided by the Contract Documents and the cofferdam design proposed for the West Abutment by the Contractor have yielded the general conclusions listed below. It is assumed that the design would have been developed to accommodate the worst case conditions identified in subsurface data provided by the Contract Documents and a reasonable interpretation of the specific data as it might impact the entire structure.

a. Results of stability analysis undertaken by the third party using the cofferdam design and data supplied by the Contract Documents indicate that the cofferdam was not stable with an adequate factor of safety when analyzed in accordance with procedures provided in the Canadian Foundation Engineering Manual (CFEM). The stability analyses suggest that the original cofferdam designed was not stable with only one top strut.
b. There are differences in some of the selected design parameters compared with those recommended by the Contract Documents. Some of these assumptions are less conservative.
c. A cofferdam with two levels of struts would address the stability from toe kick-out provided that the structural requirements in the struts and sheets are met.
d. The base heave/piping elements would need to be addressed by hydraulic pressure relief, control and/or extension of the sheeting below the excavation base.
e. Some susceptibility of the subsoil to liquefaction should have reasonably been anticipated by the Designer and suitable accommodations made in the design and construction.

(C) Impact of Installation Procedures and Sequences

Based on reported construction approach and sequences the following items are considered pertinent:

a. Based on the diary records, it appears that the excavation was undertaken to full depth prior to installation of the bracing. This would add to the potential for cofferdam distress.
b. The Contractor did not employ any measures for hydraulic pressure relief and seepage control. It is not clear what specific measures, if any, were required by the shoring designer.

In summary, the results of the third-party investigation (GI 2009) have identified some material differences in subsurface conditions compared to those reported in Contract Documents and used for initial design of the West Abutment cofferdam. Further, it was noted that some recommendations and comments provided in the FIDR appear to have been missed or not accommodated in the design. Based on the assessments and analyses escribed above, it was the third-party opinion that the failure of the West Abutment cofferdam at the site occurred due to basal heave primarily in its north-west corner followed by toe kick-out. Pile driving would have aggravated the situation due to increase in excess porewater pressure during driving. This was consistent with the site observations.

7 REFERENCES

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