Highway Embankment Failure on Weak Ground – Case History

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ABSTRACT
This paper describes the failure of a highway embankment near Sault Ste. Marie, Ontario, Canada. The failure occurred in 2005 during the construction of the Highway 17 (Trans-Canada Highway) Sault Ste. Marie Bypass, in a wet area over a layer of very loose surficial silty sand, underlain by very soft to soft silty clay to clayey silt. The grade raise for the proposed roadway was only 1.4 m, along with a 1.0 m surcharge, for a total embankment height of 2.4 m. In view of the weak nature of the subsoil, the height of the embankment was carefully chosen, with a sufficient factor of safety against instability. As such, the failure came as a surprise to the geotechnical designers.

This paper describes the project background, subsurface conditions, causes of the unexpected instability and the corrective measures taken to rectify the situation.

RESUME
Cet article décrit la défaillance d'un remblai routier près de Sault Ste. Marie, Ontario, Canada. La panne est survenue en 2005 lors de la construction de l'autoroute 17 (route Transcanadienne) Sault Ste. Marie Contourner dans une zone humide sur une couche de sable limoneux de surface très meuble, sur laquelle repose une argile silteuse très douce à molle à une limon argileux. La pente de la route proposée n'était que de 1,4 m, avec une surcharge de 1,0 m, pour une hauteur totale de remblai de 2,4 m. Compte tenu de la faiblesse du sous-sol, la hauteur du remblai a été soigneusement choisie, avec un facteur de sécurité suffisant contre l'instabilité. En tant que tel, l'échec est venu comme une surprise pour les concepteurs géotechniques.

Cet article décrit le contexte du projet, les conditions du sous-sol, les causes de l'instabilité inattendue et les mesures correctives prises pour corriger la situation.

1 INTRODUCTION

The Trans-Canada Highway (Highway 17) was constructed through the Town (now City) of Sault Ste. Marie in the mid-1900's (Figures 1 and 2). With the subsequent urban sprawl around the highway across the town, the Province of Ontario proposed the construction of a 32 km long City of Sault Ste. Marie By-Pass (Figure 2). The construction of the middle approximately 22 km section of the By-Pass started in about the year 2000 as a design-build contract. An 8 km section in the east end of the By-Pass (Figure 3) was undertaken as a conventional total project management (TPM) design and construction tendering process, i.e., design-bid-build. This eastern section was constructed between 2004 and 2008. During the construction, instability of the new embankment was encountered between Stations 13+080 and 13+160 (Figure 3). The height of the embankment at the time of instability in this 80 m long section was 2.4 m, including a 1.0 m surcharge.

This paper describes the failure of this section of the highway embankment under construction, causes of the failure and remedial measures implemented to rectify the failure condition.

Figure 1 – Ontario Map – City of Sault Ste. Marie

Figure 2 – Sault Ste. Marie – Highway 17 (red dash line)

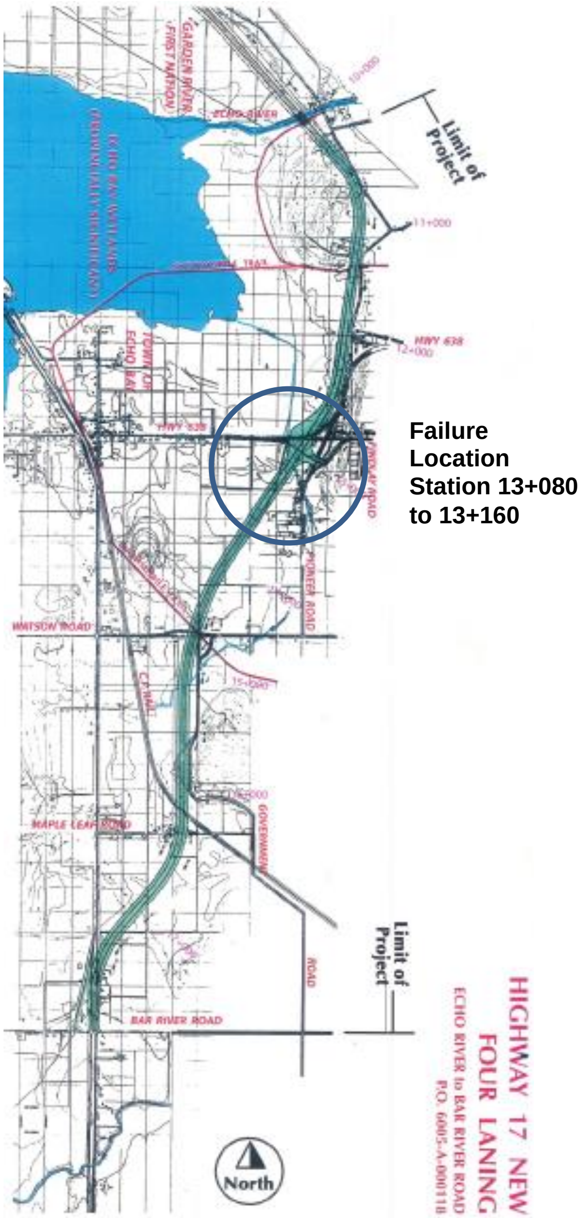

Figure 3 – Project Limits – Echo River to Bar River Road

2 SITE DESCRIPTION, GEOLOGY AND SUBSURFACE CONDITIONS

Typically in the general area, the low-lying areas are characterized by surficial peat and topsoil, overlying glaciolacustrine deposits. The glaciolacustrine deposits typically consist of clay and silt, with minor sand deposited in basin and quiet water environments. The depth of the clay in these areas can exceed 40 m. In the higher lying areas, bedrock of undifferentiated igneous and metamorphic classifications (Southern Province) is exposed at surface forming shallow hills. These rocks are generally Pre-Cambrian formations while some Cambrian unconformities are also noted. The bedrock at the site consists of Cambrian sandstone of Jacobsville Formation at the interface with Pre-Cambrian Lorrain Formation which consists of quartzite, siltstone, greywacke and conglomerate. Geology of the City of Sault Ste. Marie is described by Cowan et al (1998).

The site of the failure lies immediately west of secondary Highway 638 (near Station 13+000) which connects the existing Highway 17 to the Town of Echo Bay, east of the City of Sault Ste. Marie. At the west end of the 8 km long project section, near Station 10+000, Echo River flows into the Echo Bay Wetlands, considered a significant wetland by the Province of Ontario. To the west of Echo River is Garden River First Nation territory. The section of the highway alignment between Stations 11+600 and 13+200 is essentially a continuation of the wetlands and was referred to as the 'swamp' section of the new by-pass. In this section, access by vehicle and foot traffic was only possible upon freezing of the ground surface in the winter months.

A subsurface investigation was conducted at the project site in 2002 and 2003 which showed that this low lying area is characterized by a thin veneer of organic soils (typically 0.4 to 0.6 m thick), underlain by weak clays to depths in excess of 40 m. The measured undrained shear strength of the clay in the upper 3 m generally ranged between only 4 and 15 kPa. The presence of artesian conditions was noted during the soil investigation at intermittent depths below the ground surface (Ozden et al, 2017). The secondary Highway 638 runs north-south, at about Station 13+100 towards the eastern perimeter of this swamp section, on an embankment about 1 m high. Failures of the embankment for Highway 638 have been reported when grade raises were attempted in the past. Starting at about Station 13+020, immediately east of Highway 638, a surficial silty fine sand layer was encountered, which increased in thickness towards the eastern fringes of the 'swamp' section.

The failure section between Stations 13+080 and 13+160 was characterized by a veneer of peat (0.2 to 0.4 m thick), underlain by very loose fine sand with silt and silty fine sand zones. These extended to 0.9 to 2.1 m below the ground surface and were underlain by a clay to clayey silt deposit with measured in situ undrained shear strengths generally ranging from 6 to 25 kPa (Figures 4 and 5). The groundwater table was generally at the ground surface but during the summer months dropped to about 0.3 m below the ground surface.

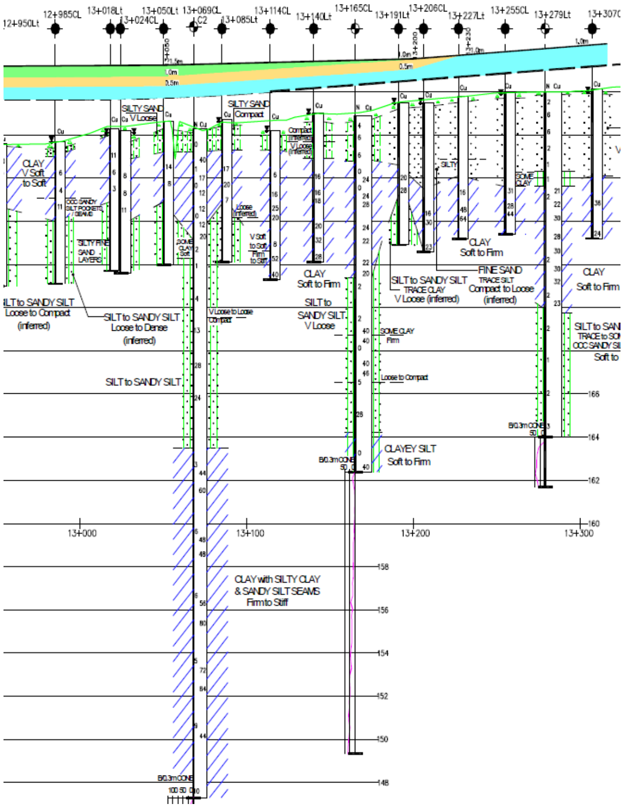

Figure 4 – Subsurface Conditions – 12+950 to 13+300

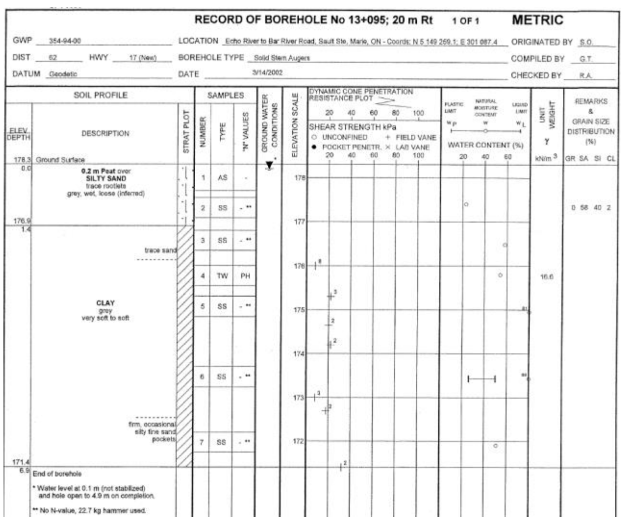

Figure 5 – Record of Borehole at Station 13+095 20 m Rt

3 THE PROJECT

The embankment that failed during its construction was part of a highway construction project which extended 8 km from Station 10+000 at the west end near the Echo River to Bar River Road at Station 18+000 at the east end, connecting to the existing Highway 17.

In the general area between Station 11+600 and 13+200, referred to as the 'swamp' section, the original grade (o.g.) sloped from about El. 183 m at Station 11+750 near the fringes of the swamp down to El. 178 m at about Station 11+900. Further eastbound, the o.g. remained at a relatively low level at about El. 177 m to about Station 13+000, near Highway 638. Beyond this point, the grade rose very gradually to about El. 180 m at about Station 13+250.

Initially, flooding concerns dictated a higher vertical grade design in the 'swamp' section. However, due to unfavourable subsoil conditions, the lowering of the embankment height was recommended. With some improvements in the drainage conditions (e.g. larger and more frequent culverts), this was deemed possible by the highway designers. In this 'swamp' section, very soft to soft clay soils extended to great depths and undrained shear strengths as low as 4 kPa were measured. Because of geotechnical concerns, in addition to lowering the proposed vertical grade, a two to three stage surcharging design was proposed for implementation along with field instrumentation to measure the porewater pressures and settlements. Figures 4 and 5 depict the subsurface conditions (borehole logs) and surcharging requirements along the eastern part of the swamp section.

The construction of the eastbound lanes east of Station 13+050 (near Highway 638) commenced in the spring of 2005. The placement of the fill to the final road grade of 1.4 m above o.g. took place rather rapidly. As well, a surcharge of 1.0 m was applied as per the geotechnical design to effect the majority of the anticipated settlements prior to paving the highway, bringing the total height of the fill to 2.4 m above o.g. According to the contractor, during this period the eastbound lanes, while being constructed and surcharged, were used as the only haul route for the construction traffic, including heavy granular material haul and rock fill trucks. The rate of the truck traffic on some days was about one loaded truck per minute. This was because the local municipality would not allow construction truck traffic on their streets.

It should be pointed out that immediately adjacent to the failure site (between the failure location and Highway 638), a set of twin concrete culverts (each 5.7 m wide and 2.2 m high) was being constructed. Due to the anticipated settlements of the subsoil, site preparation for the culverts was started at an earlier date. The finished grade (i.e. top of pavement elevation) of the highway at the culvert locations was higher than the adjacent failure site. As well, the design surcharge height was 1.6 to 1.8 m (versus 1.0 m) to be placed in stages subject to field instrumentation readings. The first stage of this surcharge (0.6 m) had already been placed by the end of March 2005. The culvert embankment is shown in Figures 6 and 7 in the background of the photographs.

It should also be noted that the geotechnical firm (represented by the primary author) that carried out the geotechnical design was not involved with the construction until the failure occurred, in spite of various attempts to obtain and review the field instrumentation readings. For this reason, the news of the failure came as a surprise. The readings were being taken and interpreted by another geotechnical firm. It is however believed that they were not involved in the day-to-day construction activities.

Figure 6 – Highway 17 construction (2005) – culvert embankment in background

Figure 7 – Highway 17 construction (2005)

4 THE FAILURE

In early May 2005, an approximately 80 m long embankment section of the highway under construction failed between Stations 13+080 and 13+160 along the east bound lanes (Figures 8 to 10). The failure occurred shortly after the embankment reached 2.4 m above o.g. level (full height of embankment, including surcharge fill). Upon failure, some ground heave was noted towards the south side, but not within the median area to the north of the east bound lanes.

Fortunately, the failure did not adversely affect the on-going construction between Stations 13+050 and 13+080, immediately adjacent to the failed 80 m long section. This section remained intact despite the fact that the height of the embankment was 3.0 to 3.2 m (i.e. 0.6 to 0.8 m higher than the failed section to the east) and the subsoil conditions were similar or slightly less favourable.

Figure 8 – May 10, 2005 – Embankment Failure

Figure 9 – May 10, 2005 – Embankment Failure

Figure 10 – May 10, 2005 – Embankment Failure

A field investigation was conducted in the failed area on May 12, 13 and 14, 2005 and this consisted of drilling six boreholes as shown in Figure 11. Five of the boreholes were drilled along the centre of the failure and the sixth borehole (BH 23) was placed outside the failure zone. The boreholes were drilled to depths of about 10 to 11.5 m below existing ground surface. Extensive field vane testing and continuous sampling by means of Standard Penetration Test method were performed in these boreholes. The findings of the boreholes are presented on the soil profile in Figure 12. Comparison of the undrained shear strengths measured in the boreholes in this investigation with the undrained shear strengths in the original investigation boreholes indicated that, in general, the newly measured undrained shear strengths were the same or higher than those used in the original design.

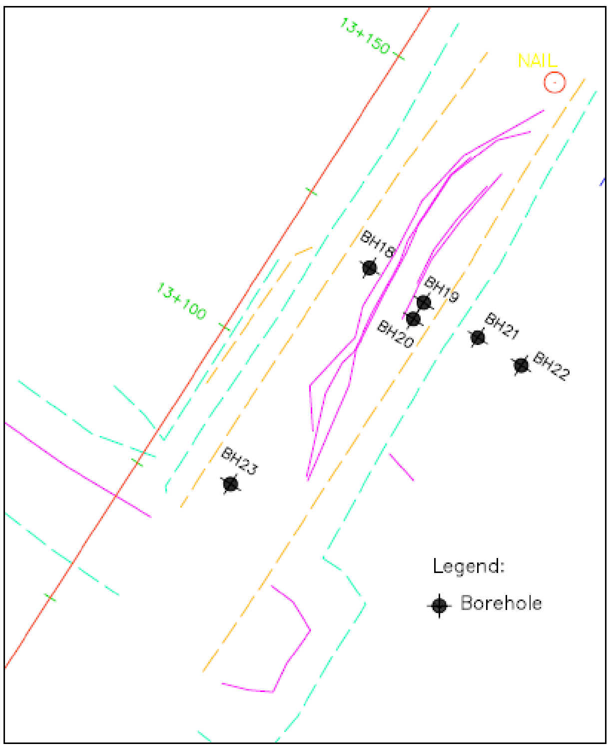

Figure 11 – Borehole Location Plan (NTS)

5 CAUSES OF FAILURE

The design height of the embankment was carefully chosen, with an adequate factor of safety of 1.4 for stability analysis, provided that pore pressures were given a chance to dissipate. This involved a sufficiently slow rate of construction and this aspect was specified in the geotechnical report. The embankment was constructed in the spring months, when the groundwater table was at the ground level and, according to the daily construction records, the construction took place at a rapid pace. More importantly, during its construction, the embankment was utilized to carry intense construction traffic as the only access road to the tune of up to one loaded truck per minute. Because the construction of the embankment and the construction traffic were on-going simultaneously, there was no chance for pore pressures to dissipate. As well, the granular fills used for the construction of the embankment were very silty, thus precluding the rapid dissipation of the pore pressures.

The presence of the higher embankment built for the construction of the twin concrete culverts, as shown in the background in the photographs in Figures 6 and 7, also contributed to high pore pressures. When the loaded trucks, especially the unarticulated rock haul trucks, moved down from the top of the culvert embankment to the failure area at a significant speed and frequency, they caused considerable impact on the ground and tended to generate excessive pore pressures in the underlying soils, thus causing a failure condition. According to the contractor, they were under intense pressure to keep the opening schedule for the highway and thus the rate of the construction truck traffic was about one loaded truck per minute.

In summary, the main causes of the embankment failure were rapid construction and heavy truck traffic, particularly the unarticulated rock haul trucks.

6 REMEDIAL MEASURES

The failed section of the highway under construction was part of the only construction and haul access to continue with a speedy construction schedule, as the local municipalities did not allow the existing municipal roads in the vicinity to be used for this purpose. The unexpected failure deprived the contractor from utilizing the only avenue available for east-west access. The tight construction schedule meant that a very quick decision had to be made to come up with remedial measures and to implement them to keep the heavy construction traffic moving as soon as possible. Unfortunately, in this stretch of the new highway alignment, the right-of-way was narrower than most of the rest of the alignment. Therefore, the design carried a relatively narrow median compared to a 30 m wide median along the rest of the swamp section to the west. As well, the available land along the failed east bound lanes (i.e. towards the south side) was only 4 to 5 m wide. This restricted the ability of the geotechnical engineer to recommend an effectively wide stabilizing berm on the outside of the embankment for remediation.

The sketches of the recommended reconstruction are given in Figures 13 to 15. As shown, the failed section of the east bound lanes was reconstructed using two layers of biaxial geogrid with some granular fill in between. These extended across the 4 m wide stabilizing berm, i.e. all the available property, which was recommended as part of the remediation measures. It was also recommended that several vibrating wire piezometers and two settlement monitoring plates with rods be installed between Stations 13+120 and 13+126, with tip elevations of the piezometers ranging from El. 178.2 to 171.4 m.

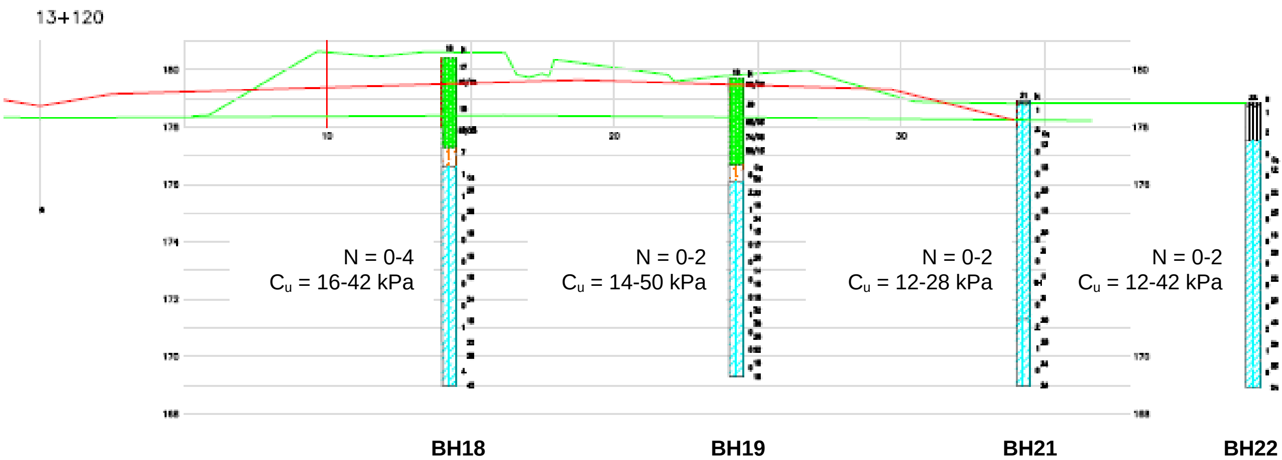

Figure 12 – Soil Profile at Station 13+120 EBL (blue = clay to silty clay, red dots = silty sand, green = fill)

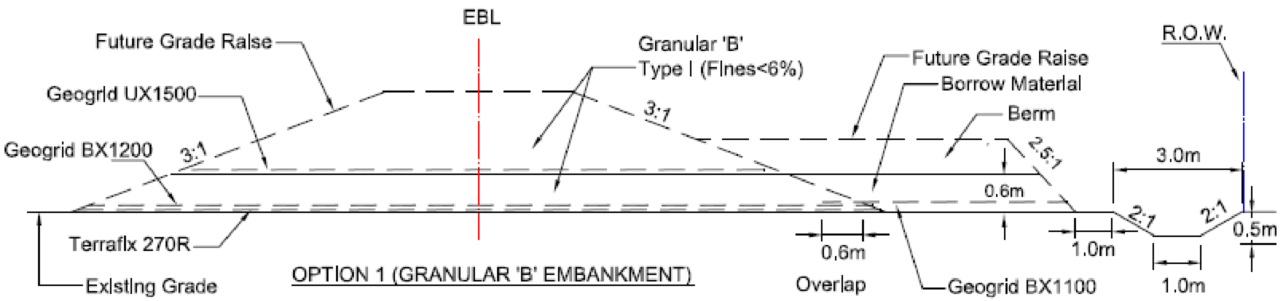

Figure 13 – Cross-Section at Station 13+100 EBL Failure

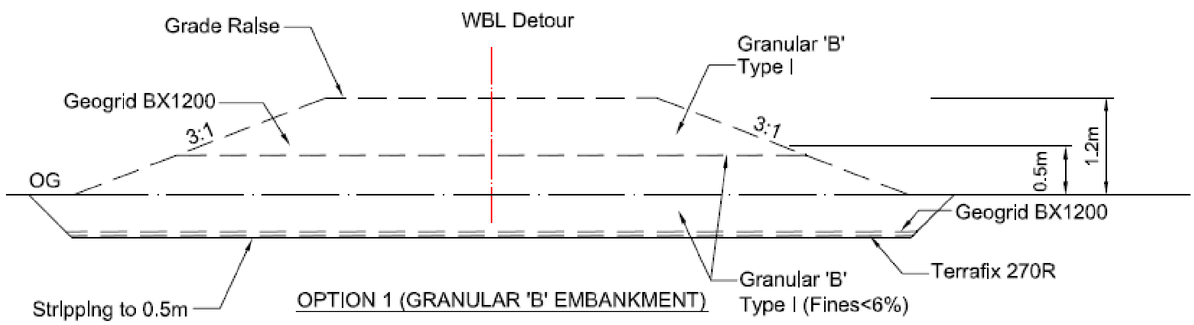

Figure 14 – Cross-Section at WBL Detour

In addition, it was recommended that when the height of the embankment reached the finished road surface (i.e. top of asphalt elevation), the placement of the fill be stopped for a period of several months (subject to field instrumentation readings) to allow for the pore pressures to dissipate prior to placement of the surcharge fill. The frequency of the loaded truck traffic was reduced from the pre-failure rate of one truck per minute to one truck every 4 minutes. In view of the anticipated excessive settlements, the height of the surcharge was increased by 0.6 m from 1.0 m to 1.6 m, to be placed in two stages, with three weeks of heavy truck traffic restriction in between.

Figure 16 shows the predicted and the measured settlements under the weight of the embankment built after the failure. In spite of the fact that some of the settlements had already occurred under the weight of the embankment prior to failure (in addition to lateral yield), another 0.6 m of settlement materialized in addition to the originally predicted 0.7 m for a total of 1.3 m, some of which can be attributed to the rebound when the failed embankment weight was removed for reconstruction and some to further lateral yield.

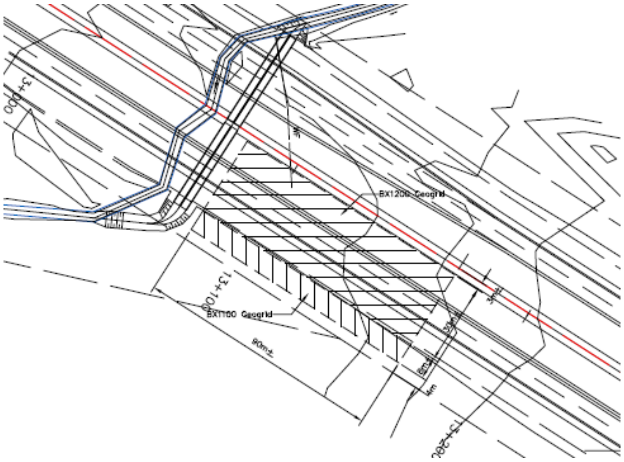

Figure 15 – Geogrid Placement at Station 13+100 EBL

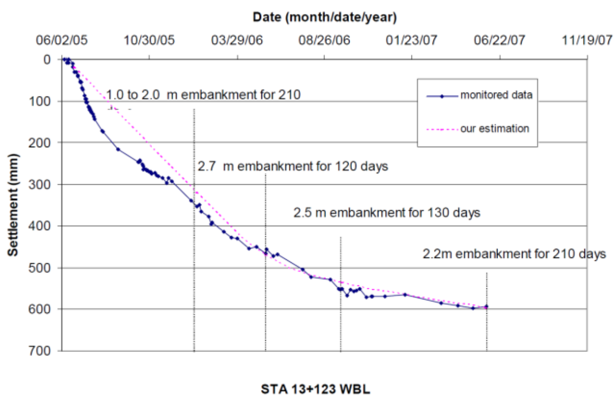

Figure 16 – Predicted and measured settlements under the weight of the embankment built after the failure

Immediately after the failure of the east bound lanes (EBL), the contractor started to build the west bound lanes (WBL) in an effort to carry on with material haul from west to east. Similar to the EBL, the construction of the WBL was being conducted while it was used as a haul road but at a somewhat reduced frequency of loaded truck traffic. At this time, the primary author decided to check the site for proper implementation of the recommended corrective measures described above. Figures 17 to 19 show the second (upper) layer of the reinforcing geogrid along the EBL to rectify the failure and construction of the WBL in progress at the same time.

At the background of the photograph in Figure 7 is the surcharged embankment for the twin concrete culverts and the heavy vibratory compactor in operation in front of it. At one point during this site visit, the primary author was trying to convince the contractor to stop applying vibration for compaction of the embankment fill or to at least reduce it to a minimum so as to minimize generating excess pore pressures in the silty granular fill being used to construct the embankment and in the underlying wet native silty fine sand foundation soils with the groundwater table being at the o.g. level. By coincidence, the ground surface in the median area started to visibly rise towards the left side of the photograph. This phenomenon convinced the contractor to heed the advice from the geotechnical engineer (primary author) and to not only cease applying vibration during the compaction process but also to immediately provide a culvert pipe along the median for drainage and then immediately backfill the median area to form a quick mid-height stabilizing berm. They were also advised to place a mid-height stabilizing berm on the north side. The width of the berm was limited to 5 to 6 m, as the property beyond this point had not been acquired by the government. The mid-height stabilizing berms were placed uncompacted, except for several passes of a dozer. These measures were taken on the same day, immediately after noticing the rising ground surface and there were no further visible movements along the WBL.

Figure 17 – June 14, 2005 – Embankment Reconstruction

Figure 18 – June 14, 2005 – Embankment Reconstruction

Figure 19 – June 14, 2005 – Biaxial (BX) Geogrid

7 CONCLUSIONS AND RECOMMENDATIONS

Failures during construction on weak soils are not uncommon. This is primarily because every so often unforeseen situations arise which were not accounted for by the geotechnical designers. The construction stage design factor of safety against stability failure should therefore be 1.5, rather than the conventional 1.3 (common MTO practice) during construction on weak clays (Ozden and Staseff, 2016), unless the designers can be sure that the construction will proceed according to all assumptions made during the design. As well, small differences in measuring the undrained shear strength of the weak clays can lead to large errors (Ozden et al, 2017).

Construction access and traffic conditions during embankment construction on weak soils, including haul truck weights and frequency, are some of the issues that are frequently not well visualized and anticipated during the design stage. In as much as possible, these should be properly identified and specified so that if there are unforeseen changes, they can be addressed during the construction. If not, they can lead to excess pore pressures causing failures, as it happened in this case.

Site instrumentation and monitoring is very useful and can act as an insurance policy to warn against the danger of impending failures, as well as facilitating possible construction schedule changes. However, their use is diminished if they are not properly utilized. If the original geotechnical designers are not commissioned for taking and interpreting the field instrument readings then some liaison between those who carry out this task and the geotechnical design engineer would be very useful.

Figure 20 shows the project site in 2017. The repaired section is in the lower left quadrant of the plan view.


Figure 20 – Highway 17 in 2017 (Google Map)

8 REFERENCES

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