# Interpretation and analysis of a test embankment on soft clay

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ABSTRACT
This paper presents the foundation performance of a test embankment constructed on instrumented thick deposits of soft clays in Northern Ontario, with and without prefabricated vertical drain (wick drains) ground improvement. The test site consists of low to high plastic, slickensided, lightly overconsolidated clay deposits. Wick drains were installed in three areas of the test embankment foundation with 2 m, 3 m and 4 m triangular spacings. A fourth area of the test embankment foundation did not have wick drains installed. The embankment was constructed in stages and loaded to achieve normally consolidated condition in the clay foundation. Aspects such as strength gain of the clay deposits due to loading and ground improvement, the effect of wick drain spacing, and consolidation parameters are discussed.

RÉSUMÉ
Cet article présente la performance de la fondation d'une berme d'essai construite sur d'épais dépôts instrumentés d'argiles molles dans le nord de l'Ontario, avec et sans amélioration de sol par drain vertical préfabriqué (drains à mèche). Le site d'essai est constitué de dépôts d'argile striée légèrement surconsolidée, faiblement à fortement plastique. Des drains à mèches ont été installés dans trois zones de la fondation de la berme d'essai avec des espacements triangulaires de 2 m, 3 m et 4 m. Aucun drain à mèche n'a été installé dans un quatrième secteur de la fondation de la berme d'essai. La berme a été construite par étapes et chargée de façon à obtenir un état normalement consolidé dans les fondations d'argile. Les aspects tels que le gain de résistance des dépôts d'argile dû à la charge et à l'amélioration du sol, l'effet de l'espacement des drains à mèche et les paramètres de consolidation sont abordés.

## 1 INTRODUCTION

Large embankments are proposed to be constructed in Northern Ontario. The site consists of up to 40 m of low to high plastic, slickensided lightly overconsolidated clay deposits requiring ground improvement to support more than 50 m high embankments, scheduled to be constructed in a relatively short timeframe. A test fill was constructed to asses the performance of ground improvement using prefabricated vertical drains (wick drains). The test fill was constructed to a final height of approximately 20 m, a crest width of 50 m, a crest length of 110 m and with side slopes of 9:1 (horizontal: vertical).

This paper provides a summary of test fill construction, instrumentation installed, and a summary of the observations.

## 2 FOUNDATION CONDITIONS

The soil stratigraphy generally consists of (in order from ground surface to bedrock):

- A thin organic surficial layer;
- upper glaciolacustrine;
- clay till;
- lower glaciolacustrine;
- granular till; and
- bedrock.

Four boreholes and eight CPTs were completed to characterize the foundation condition of the test fill. The upper glaciolacustrine unit has a range of thickness of 2.1 m to 4.1 m and generally consists of an upper weathered silty clay and varved silt and clay. The clay till unit has a range of thickness of 11.0 m to 13.0 m and generally consists of silty clay to clay. The lower glaciolacustrine unit has a range of thickness of 0.6 m to 1.5 m and generally consists of varved silt and clay. The underlying granular till encountered consists of sand and gravel to gravelly sand.

Disturbed and undisturbed samples were collected for laboratory testing and Nilcon field vanes were carried out to measure the undrained strength of the foundation clays prior to the test fill construction.

The three cohesive soil units (upper and lower glaciolacustrine and clay till) are of primary interest in this study, as they ultimately control foundation stability of the embankment.

## 3 CONSTRUCTION

The construction comprised the installation of the wick drain foundation improvement, followed by placement of the embankment. Wick drain foundation improvement included construction of a drainage blanket to provide drainage from the wick drains at ground surface.

### 3.1 Foundation Improvement

The test fill was constructed with four foundation areas, consisting of 2 m, 3 m and 4 m triangular wick drain spacing, and one area where no wick drains were installed. Table 1 presents dimensions of each area.

Table 1. Wick drain installation summary

| Area | Triangular Wick Drain Spacing (m) | Installation Area Length (m) | Installation Area Width (m) |
|---|---|---|---|
| TF-1 | 3 | 20.8 | 15 |
| TF-2 | 2 | 20.8 | 12 |
| TF-3 | 4 | 20.8 | 20 |
| TF-4 | 0 | 0 | 0 |

A granular drainage blanket underlain by a geogrid and geotextile composite was installed to provide drainage at ground surface in the wick drain foundation areas. The drainage blanket doubled as a construction platform to allow installation of the wick drains over the relatively weak foundation soils. The geogrid and geotextile composite provided separation of the drainage blanket from the underlying soil and provided ground reinforcement support to allow for a reduced granular thickness necessary to support construction equipment.

The granular drainage blanket material consists of 3-inch minus crushed aggregate. The constructed thickness varied from 0.3 m to 0.5 m. The geogrid and geotextile composite used was Titan TE-BXC30 Bi-Axial Geogrid Composite. Titan TE-BXC30 is made of bi-axial polypropylene geogrid heat bonded to non-woven polyester geotextile. The geotextile provides separation between the granular drainage blanket material, and underlying soil, while the geogrid provides subgrade reinforcement to allow for a reduced granular thickness.

The wick drains installed are Mebra-Drain MD-7407/3401. The wick drains had a width of 100 mm and a thickness of 3.6 mm and were anchored at their base with a steel plate. The wick drains were installed using a hydraulic wick installation rig mounted to a Komatsu PC-650 tracked excavator. The mandrel used was a 60 mm x 120 mm rectangular steel tube. Wick drains were pushed to refusal in the granular till unit underlying the cohesive units to provide a double drainage condition within the foundation clay. The Ontario Provincial Standard Specification (OPSS 220, 2014) was followed for the wick drains installation.

### 3.2 Embankment Construction

Test fill construction consisted of hauling rockfill with 230 Ton haul trucks and spreading by dozer. A total of six lifts were placed with an average thickness of 3.3 m (ranging from 1.8 m to 5.8 m thick) to a final height of 20 m. Each lift was placed in three stages, as follows:

- Haul trucks free dumped rockfill material over the lift area.
- Free dumped piles were infilled, and top surface dozed flat by a bulldozer.
- A buffer was maintained during rockfill placement around vertical conduit pipes placed inside the embankment (to allow subsequent in-situ testing of the foundation clay), which were subsequently backfilled with clay overburden carefully placed by excavator in order to protect the conduits from damage.

Heavy construction machinery was not allowed to traffic within 3 m of the conduit pipes and overburden material was mounded around the pipes prior to the placement of each rock fill lift. Care was taken so that the conduit pipes would remain vertical and not be damaged during fill placement and conduit pipe raising.

## 4 INSTRUMENTATION AND MONITORING

Instrumentation consists of vibrating wire piezometers (VWP) and inclinometers. Vibrating wire piezometers and inclinometers were installed in boreholes and grouted in place using a Portland Cement-Water-Bentonite grout backfill selected to be appropriate for the surrounding soil stiffness. Inclinometers were anchored a minimum 3 m in bedrock to provide a stable anchor point.

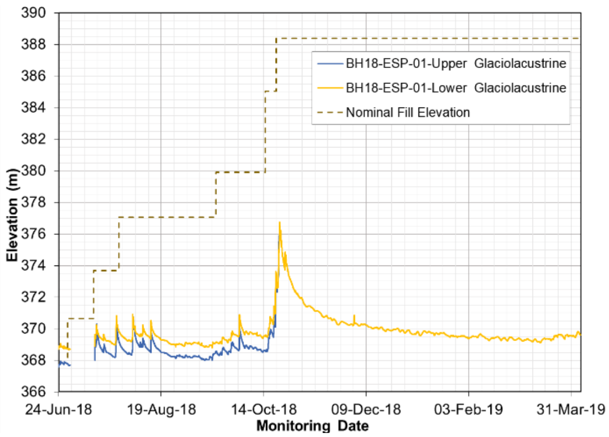
Three nested vibrating wire piezometers were installed at each borehole location (12 VWPs in total) at specified depths to target each of the clay units. Following the wick drain installation, the VWP cables were placed in a trench within the wick drain platform and brought to the toe of the test fill, where they were connected to dataloggers. The VWP cables were snaked along the base of the trench allowing for any ground settlement which may occur during stockpiling. Two inclinometers were installed at the embankment toe.

In addition to VWPs and inclinometers, six conduit pipes (two at each of the 3 wick drain area) were installed within the test fill. The conduit pipes consist of 75 mm steel pipes anchored to a 1,000 mm by 1,000 mm square, 10 mm thick, steel base plate. The conduit pipes were installed overtop of a 300 mm by 300 mm layer of geotextile placed above a minimum of 0.5 m of sand. The sand replaced the crushed granular material of the drainage blanket to allow for penetration of the CPT through the drainage blanket layer following embankment construction. The primary purpose of these conduit pipes is to provide a vertical conduit through the embankment for pushing of CPTs into the underlying subgrade following construction of the test fill. Completion of CPTs following stockpile construction adjacent to locations where CPTs were pushed prior to construction allowed for a direct measurement of undrained shear strength gain and dissipation of excess pore pressures within the foundation soils along continuous vertical profiles. At each area, one CPT was pushed shortly after the end of the test fill construction and one CPT was pushed about one week later to measure strength gain due to the wick drain installation. Dissipation tests were carried out for every pushed CPT. The target of the tests was to achieve 50% of consolidation time ($t_{50}$). In addition to acting as a conduit, the top of the conduit pipes was surveyed periodically throughout fill placement to provide a measurement of settlement due to the added fill weight.

All conduit pipes and VWPs were installed at midpoints between the installed wick drains to provide the most critical condition.

## 5 TEST FILL MONITORING RESULTS

Construction induced piezometric pressures and subsequent dissipation were monitored throughout embankment construction. The resulting piezometric water elevation due to embankment loading elevations are displayed in Figure 1. A few of the VWPs stopped working during test fill construction, likely due to excessive settlement and subsequent tearing of the cables.

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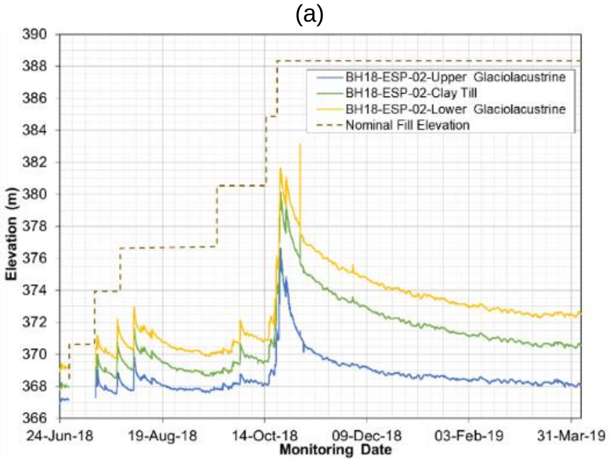


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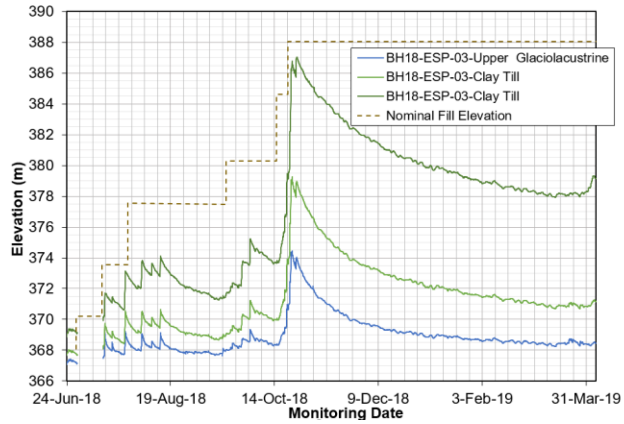


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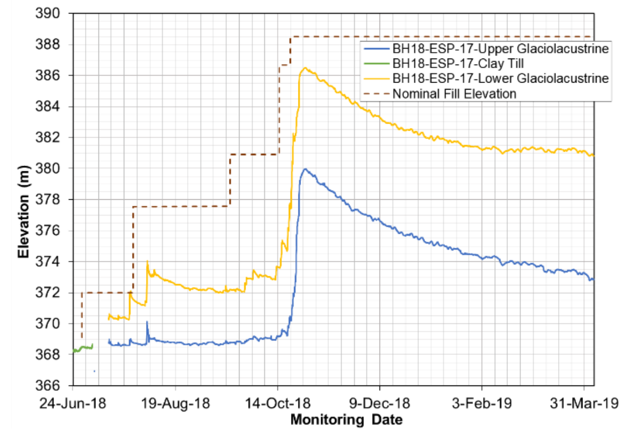



(d)

Figure 1. Piezometric porewater pressure response (a) 2 m spacing wick drain area, (b) 3 m spacing wick drain area, (c) 4 m spacing wick drain area and (d) no wick drain area.

Incremental B-Bar responses (i.e. B-Bar response to individual lift placement) were calculated based on the piezometric response measured in each vibrating wire piezometer during each fill placement. B-Bar responses were calculated as the sum of all porewater pressure increases during lift placement, divided by the change in vertical total stress due to the lift placement. The results of these calculated B-Bar responses are shown in Figure 2.

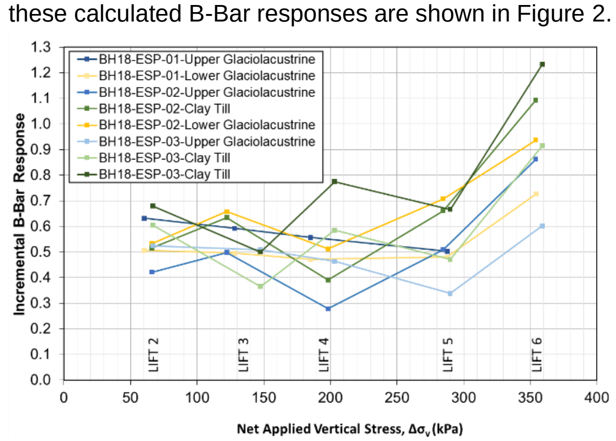



Figure 2. B-bar calculated at WVP locations

Observed B-Bar responses varied from 0.3 to 0.8 under applied vertical stresses less than 300 kPa and increased to a range from 0.6 to 1.2 under the higher applied vertical stresses of 350 to 400 kPa during final lift placement. The increase of B-Bar responses at higher applied vertical stress is a result of transitioning from the over consolidated stress range to the normally consolidated stress range, where incremental B-bar of approximately one represents the normally consolidation condition, as described by Leroueil et al. (1978). Figure 3 presents the comparison of preconsolidation pressure estimated using VWPs, CPTs, Odeometer and Constant Rate Strain (CRS) laboratory tests. A good agreement between the methods has generally been observed. However, Casagrande (1936) method estimates lower value of preconsolidation pressure in comparison to the other methods. Kulhawy and Mayne (1990) method was used to calculate the overconsolidation ratio of the CPT data.

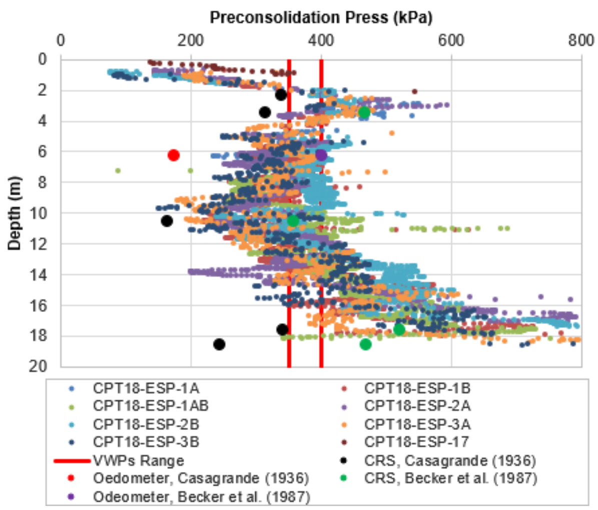



Figure 3. Foundation Clay Preconsolidation Pressure

According to Figure 1, the first few test fill raises show a much faster porewater pressure dissipation than the last raise over the same period. Based on Figure 2, the foundation clay is still within the overconsolidated stage for the first few raises; whereas, the last raise generally represents the normally consolidation condition. This observation is important for projects where the applied load will result in existing overconsolidated clay becoming normally consolidated. The rationale behind this is that the coefficient of consolidation ($c_v$) decreases significantly at the normally consolidated stage as shown by Ladd and DeGroot (2004).

Construction induced piezometric pressure generation and subsequent dissipation, are shown for each of the three primary clay soil units on Figures 4 and 5, respectively.

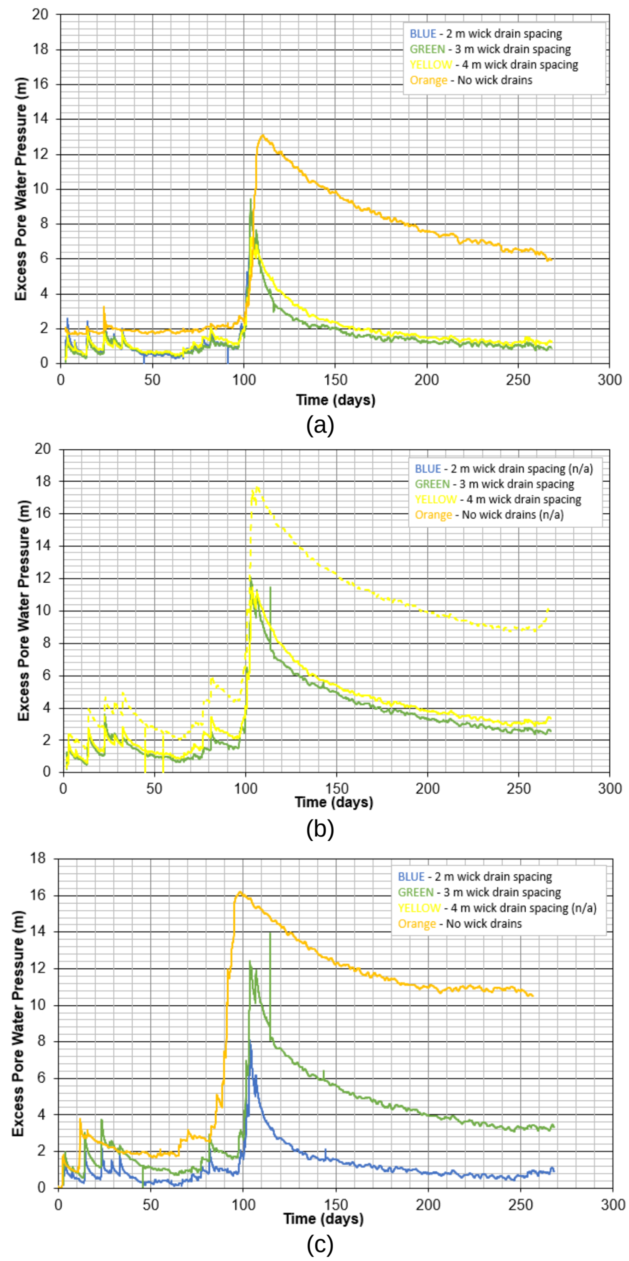
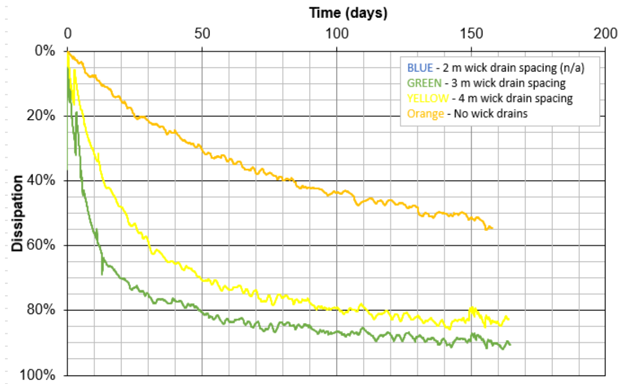
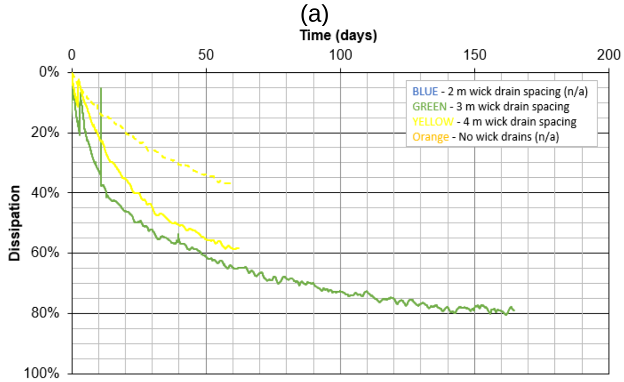



Figure 4. Excess porewater pressure generation due to loading (a) upper glaciolacustrine (b) clay till (c) lower glaciolacustrine




(a)

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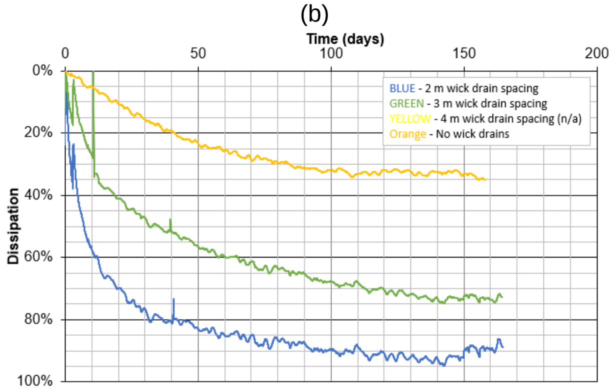


(c)

Figure 5. Porewater pressure dissipation following the end of construction (a) upper glaciolacustrine (b) clay till (c) lower glaciolacustrine

Excess porewater pressure generation with time shown on Figure 4 indicates that porewater pressure build up with loading occurred during construction. It is evident from the data shown that where no wick drains were installed very little dissipation of excess porewater pressure occurred, compared with the locations where wick drains had been installed. Further, the closer the wick drain spacing, the smaller the build up of excess porewater pressures.

The excess porewater dissipation with time plots shown on Figure 5 present the dissipation after final loading. For comparative purposes, a summary of calculated dissipation achieved at 50 days for the different wick spacings and soil units is provided in **Error! Reference source not found.**2. As shown, for all soil units, a significant increase in the rate of dissipation can be seen with the installation of wick drains. The rate of dissipation increases further with closer wick spacing. Although two out of the three 2 m spacing wick drain piezometers stopped working at the end of the test fill construction, it is anticipated that most of the dissipation had already occurred under this condition after 50 days of construction, even when the clay is at normally consolidated stage.

Table 2. Summary of excess porewater dissipation at 50 days

| Foundation Condition | Dissipation at 50 Days – Upper Glacio-lacustrine | Dissipation at 50 Days – Clay Till | Dissipation at 50 Days – Lower Glacio-lacustrine |
|---|---|---|---|
| No Wick Drains | 30% | N/A | 23% |
| 4 m spacing | 71% | 36% – 56% | N/A |
| 3 m spacing | 81% | 61% | 57% |
| 2 m spacing | N/A | N/A | 84% |

Readings of the two inclinometers installed at the toe of the test fill indicate no significant ground movement during fill placement, implying that the test fill was not approaching a failure condition. With the flat sideslopes, the embankment remained stable despite the porewater pressures that were generated in the foundation.

Settlement monitoring was completed during test fill loading through survey of the top of conduit pipes installed prior to and throughout fill placement. Due to a relatively wide scatter in the data attributed to movement of the conduit pipes during fill placement, as well as potential error introduced during raising of the conduit pipes, incremental rates of settlement cannot be reliably determined from the data available and is not presented herein. However, a general trend of increase in settlement due to closer wick drain spacing and construction of the test fill was observed.

Two series of CPTs were pushed following the completion of the test fill in a about one week of timespan to check strength gain due to loading. Undrained strength profiles estimated from CPTs indicated increase in undrained shear strength where the wick drains were installed. Minimal strength gains were observed for the no wick drain area. Figure 6 shows the foundation clay strength gain due to the wick drains construction. The blue colour and olive brown colours represent the CPTs pushed preconstruction and postconstruction of the test fill, respectively. As shown strength gain has been achieved.

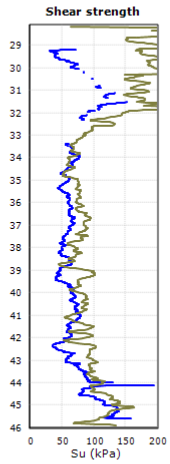

Figure 6. 3 m triangular spacing wick drains CPT data

## 6 CONCLUSION

The monitoring results of the test fill construction indicate that installation of wick drains greatly increased the rate of dissipation of construction induced excess porewater pressures, and the associated consolidation and undrained shear strength gain. The greatest ground improvement was observed where wick drains were installed at 2 m triangular spacing, with a decreasing impact observed with increasing spacing of wick drains (3 m and 4 m areas). The available data on 2 m triangular spacing wick drains indicate considerable improvement in comparison to 4 m triangular spacing wick drains. Significantly less consolidation and strength gain were achieved were no ground improvement was used. Review of the data indicates much lower excess porewater pressure dissipation when the vertical effective stress in the foundation clay passed the preconsolidation pressure due to the test fill construction because the coefficient of consolidation decreases significantly at the normally consolidated stage. This fact could have significant effect on the strength gain required for the planned staged construction of the ultimate embankments to be constructed and should be considered during the design.

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