May 6th, 12:00 AM

Dykes on Clay Foundation at Bersimis 2

S.Z. Akber

Follow this and additional works at: http://scholarsmine.mst.edu/icchge

Recommended Citation

This Article - Conference proceedings is brought to you for free and open access by the Geosciences and Geological and Petroleum Engineering at Scholars' Mine. It has been accepted for inclusion in International Conference on Case Histories in Geotechnical Engineering by an authorized administrator of Scholars' Mine. For more information, please contact weaverjr@mst.edu.
SYNOPSIS
This paper discusses the behaviour (settlement, high foundation pore pressure, horizontal displacement and seepage) of two dykes founded on a thick clay deposit, since their completion in December, 1958 to date. In the first case, coarse and fine sand layers successively cover an over sixty meters thick clay deposit; in the second case, an eight to thirty meters thick clay layer is underlain by a heterogeneous granular deposit. Foundation treatments including problems encountered during the foundation works are discussed. Results of static and pseudo-static analyses carried out using the measured high pore pressures are compared with the factors of safety obtained during design. General conditions of the dykes together with the remedial works carried out to repair the upstream slope protection of dyke No. 1 are reviewed. The factors leading to the disintegration of the downstream rip-rap of dyke No. 2 are also discussed.

INTRODUCTION
The Bersimis 2 Project, completed in 1959 and located some 300 miles north-east of Montreal, comprises a 665 MW open air power house, an intake channel, a power tunnel, penstocks, a gravity dam and two auxiliary earthfill dykes founded on a very thick clay deposit (fig. 1).

DESCRIPTION OF THE DYKES
Dyke No. 1 is situated 2440 m (8000 ft.) north-west of the main dam, while dyke No. 2 lies 915 m (3000 ft.) south-east of the main dam (fig. 1).

The two dykes are composed of a 4 m wide central clay core flanked by 90 cm (3 ft.) wide silty sand filters (transitions), which in turn are supported by sand and gravel shells (figs 2 and 3).

Dyke No. 1 is 1038 m (3400 ft.) long and 15 m (50 ft.) high on the average, except for a restricted area at the north abutment where it reaches a maximum height of 30 m (90 ft.). In this area the dyke was founded on top of the clay and as such the clay core was widened to 9 m (30 ft.) at the base to accommodate foundation settlements and to increase the contact area with the steep rock abutment. Other safety precautions which
were made at the north abutment were the inclusion of berms and a 3 m (10 ft.) thick layer of rockfill over a 60 m (200 ft.) length above the central core to protect it from loose rock falling down the very steep and high rock face (fig. 2).

Dyke No. 2 is slightly longer, 1190 m (3900 ft.), but its height is less, averaging about 11 m (35 ft.).

towards the south abutment. Transversely, the thickness of the clay increases gradually downstream from the dyke, except at the south abutment where it's absent and a 10 to 30 m (30 - 100 ft.) thick layer of till covers the granitic bedrock. The clay layer is badly weathered and fissured on the surface (1 m - 3 ft.); the next 3 to 4 m (10 to 15 ft.) of the clay is stiff and desiccated. Below this depth, the clay becomes soft to firm and is grey in colour.

FOUNDATION OF THE DYKES

Dyke No. 1

The surface layer about 0.5 m (1.5 ft.) thick consists of a natural blanket of wind blown sand and humus followed by a 6 to 9 m thick layer of coarse sand and gravel. The thickness of the coarse sand layer increases gradually southeastwards to about 9 m (30 ft.) at the dam site. The deposit is strongly current bedded and of varied grain size, containing seams from fine sand to "open-work" gravels. Below the coarse sand, a deposit of fine laminated sand about 5 m thick generally overlies a clay deposit, except in the northern zone where it is eroded and the coarse sand lies directly on the clay. Between the ch. 4+00 (north abutment) and ch. 19+00, a laminated silt layer about 5 to 13 m thick (15 to 40 ft.) is sandwiched between the fine sand layer and the marine clay deposit. The marine clays over 60 m (200 ft.) in thickness are stratified silty clays with interbedded seams and lenses of silt and fine sand. At the north abutment, the small gully below the steep rock cliff has exposed the clay deposit underlying the sand mantle. Across the northern half of the dam (ch. 5+00 to ch. 15+00), a thin clay seam with perched water above it exists between the coarse and fine sands (fig. 2). The fine sand deposit ends at ch. 25+00, and from there to the south abutment the coarse sands directly overlay the clays.

Dyke No. 2

Dyke No. 2 is founded directly on a firm clay layer which reaches a maximum thickness of 30 m (100 ft.) near the north abutment and thins out to 8 to 9 m (25 ft.)

Heterogeneous granular deposits of unknown thickness and composed of fine sand with pockets or lenses of coarse-grained materials exist below the clays. These outcrop at the surface in the terraces upstream from the dam, but pass below the clays at the dam site and probably thin out further downstream.

SEEPAGE CONTROL MEASURES

Dyke No. 1

To reduce foundation seepage the following measures were adopted:

- The surface layer (366 m long - 1200 ft.) composed of fine organic silty sand and humus was conserved intact as an impervious blanket and tied into the impervious core.
- The core trench was extended down to the fine sand or clay layer along the entire length of the dam. By thus confining the seepage to the fine sand and silt strata, the seepage quantity could be reduced to an acceptable amount and the hydraulic gradient reduced to a level that would not endanger the structure. To prevent disruption of the natural blanket by the core trench excavation, the cut-off core was designed to slope downstream (fig. 2).
- Where it was not possible to excavate a core trench to the desired depth, due to the presence of the water table, (ch. 19+20, to ch. 21+80), (ch. 23+20 to ch. 27+30), (ch. 28+70 to ch. 31+40), a sheet piling was driven through the permeable soils into the clay layer.
At the north abutment, the steel sheet piling was supplemented by a 120 m (400 ft.) long blanket of compacted clay extending from the central core. This measure was taken to compensate for the absence of the natural fine sand blanket. The piles were driven a minimum of 1.5 m (5 ft.) into the clay and protruded 1.5 m (5 ft.) into a 3 m (10 ft.) thick blanket of clay placed in the core trench.

Dyke No. 2

Because of the weathered surface of the clay, a core trench 1 m deep (3 ft.) was excavated in the clay layer along the core. From ch. 29+50 to 31+50, the core trench was deepened due to the presence of thin layers of fine sand in the clay deposit. At the south abutment, the core was also deepened because the till layer on which the core is founded contained seams of gravel.

During the excavation of the core trench, a fissure 2.5 cm (1 inch) in width was discovered with rising watertable, near the north abutment (around ch. 21+00) - fig. 3. The excavation of the core trench was extended to 7 m depth due to the presence of this fissure. The excavation down to 7 m depths revealed the presence of other fissures, even though the quantity of water flowing into the fissures had diminished. To take into account the presence of these fissures, sheet piles were driven into the clay layer from this depth and the core trench backfilled with impervious material up to the original ground surface.

INSTRUMENTATION

Figs. 2 and 3 show the location of the instruments installed to observe the performance of the dykes and are summarized below:

- Settlement points (11 for dyke 1 and 8 for dyke 2) were established along the crest of the dykes to record movements of the dykes and their foundations after construction;
- Telescoping settlement gauges were installed in the downstream shell at 1.5 m (5 ft.) intervals down to the natural ground, to measure the settlement of the dykes foundation and the settlement profile within the fill of the dykes;
- Piezometers (32 for dyke 1 and 11 for dyke 2) were installed downstream from the dykes. The piezometers consist of a stand pipe with either a well point (dyke 1) or a porous tip (dyke No. 2) at its bottom.

DESIGN PARAMETERS

The design parameters used are described below:

Foundation

Coarse sands:
- Water content above ground water: 4%
- Dry density: 15.7 - 18.9 kN/m³ (100 - 120 lbs/ft³)
- Coefficient of permeability: $1 \times 10^{-2}$ feet/min (Determined from laboratory tests).
- Silty sand (lab.) = $3 \times 10^{-6}$ ft./min
- The shear parameters for the coarse sand layer are similar to those of the sand and gravel used for the dykes supporting zones i.e.: $\phi' = 38$ to 44°

Fine Sand:
- Effective friction, $\phi' = 40°$

Clays:
- Clay samples for dyke No. 1 were obtained from the northern part of the dyke.
- Total weight: 18.9 kN/m³ (120 pcf)
- Water content: 31 - 46%
- Plasticity Index: 10 - 25%
- Undrained shear strength:
  - for dyke 1: 114 to 134 kPa (1.2 to 1.4 T/sq.ft.)
  - for dyke 2: 76 to 114 kPa, av. 81 kPa (0.8 to 1.3 T/sq.ft.)
**HORIZONTAL MOVEMENTS**

**Dyke No. 1**

Fig. 2 shows the location of the settlement points installed at the center of the dyke's crest. The readings of the settlement points (fig. 4) show that in general the crest of the dyke moves downstream, except at the settlement points 31, 41 and 61 where the movements are towards the upstream. These upstream movements are considered negligible because of their small magnitude (0.3 to 1.6 cm).

The maximum displacement of 6.9 cm towards the downstream is measured at settlement point No. 80. Elsewhere, displacements in the downstream direction vary from 1.0 to 2.7 cm. It is also to be noted that these movements have practically ceased since 1969.

**Dyke No. 2**

The readings (fig. 5) taken since May 15, 1980, show that the crest of the dyke in general moves upstream; in the southern part, the displacements are between 1.5 and 4.5 cm, while in the northern section, they vary between 0.5 to 1.2 cm.

It is difficult to visualize why the dyke has moved upstream instead of downstream. The only possible explanation may be that at the south abutment the thickness of the foundation clay increases upstream from the dam; at the north abutment, even though the clay thickness increases downstream from the dam, it's possible that these upstream movements were influenced by the presence of the intake channel excavated in clay. The distance between the upstream toe of the dyke and the eastern end of the intake channel varies between 50 m (±) (170 ft.) to 120 m (±) (400 ft.) (fig. 1). The deepest intake channel excavation in clay was 16.5 m (55 ft.).

All horizontal movements have stabilized since 1965 - 1966.
Dyke No. 1

Fig. 2 shows the location of the settlement points while Fig. 4 illustrates the evolution of settlement with time. The dykes were completed in December, 1959. The reservoir filling started on the 24th of August and finished by the end of September, 1959. The last reading taken before the reservoir impounding show that the settlement at the northern end of the dyke (Ch. 3+50), where the dyke was founded directly on the clay deposit and where it attains a maximum height of 30 m, was about 12 cm. Part of this settlement can be attributed to the compression of the fine sand and silt seams present in the clay deposit and the rest being due to consolidation of the clay core and the foundation clay. However, the reservoir filling reduced the effective stresses on the clay. As a result, a general slowdown in the rate of settlement and in certain cases a rebound is observed.

A maximum settlement of 45 cm was predicted at the north abutment at the time of design, assuming a thickness of compressible clay equal to 60 m (200 ft.). The maximum settlement measured to date has been about 26.5 cm at the north abutment, well within the predicted maximum (Fig. 4). Since the base of the settlement points was established on top of the clay core, the measured settlement comprises the settlement of the clay core plus that of the clay foundation. The consolidation of the clay core is unknown due to lack of instrumentation. However, it is considered to be rather small for two reasons. First, the clay core was compacted, in general, on the dry side of the optimum water content. Secondly, the arching effect reduces the load acting on the core thereby limiting the consolidation of the core. Therefore, the majority of the settlement has been due to compression of the clay deposit. The graphical plot proposed by Asoaka giving a total settlement of 60 m, for the northern part of the dyke, i.e. a further settlement in the order of 3 cm is to be expected. Fig. 4 also reveals that the settlements since 1967 have been minimal. Since no piezometers were installed in the clay deposit, it is difficult to say with certainty when the primary consolidation ended. But a study of the time vs settlement curve leads to the conclusion that most probably the primary settlement has terminated and presently secondary consolidation is taken place. Calculations made using the secondary compression index, for Quebec clays, give the secondary settlement up to 1982 in the order of 5 cm, and a further settlement of about 5 cm in the next 25 years (C_c: primary compression index, C_s: secondary compression index).

Dyke No. 2

Maximum settlements of 4 to 6 cm (Fig. 5) (settlement points at chainages 0+15, 0+40 and 0+80) have occurred in the northern part of the dyke, where the dyke attains its maximum height (15 m) and the maximum thickness (30 m) of the foundation clay. The settlements diminish towards the south abutment in relation to the dyke’s height. Settlements measured in the southern part of the dyke are small because the applied stresses due to the weight of the dam are much less than the preconsolidation pressure of the foundation clay. With the filling of the reservoir and the rise of the main water table, the effective loads on the clay were reduced to less than those existing immediately after construction and hence a rebound was noted, especially in the southern part of the dyke, which continued at a gradually decreasing rate until equilibrium conditions were achieved around 1965. Settlements since 1965 have been minimal.

Foundation Pore Pressures

Dyke No. 1

Thirty-one piezometers were installed (Fig. 2) in the fine sand layer overlying the clay deposit between elev. 97 and 110 m (320 to 390 ft.) and only one in the silty clay layer at elev. 91,15 m (399), the depth of the piezometers varies between 7 and 16 m. The piezometers readings started in January, 1959, after the end of construction in December, 1958. The reservoir filling started in August, 1959 and terminated at the end of September of the same year.

Figs. 6 and 7 show the development of pore pressures in the sand layer since the reservoir filling to date. Soon after the filling of the reservoir, high pore pressures immediately downstream from the northern part of the dyke were observed, indicating seepage through the dyke’s foundation. They have remained more or less the same ever since. The pore pressures are artesian at the northern end of the dyke, but they decrease further downstream from the dyke and do not exceed the ground level; the head difference being about 5 – 10 m between the first (nos.1 to 14) and second rows (nos.15 to 22) of piezometers, and only 2 to 6 m between the 2nd and 3rd (nos. 23 to 29) rows of piezometers.

The impervious core was taken down to the upper clay layer, which exists in the foundations across the northern half of the dam, to allow seepage to pass below this layer through the silty sands. Across the southern half of the dam, the channels filled with sand and gravel were cut off by steel sheet piling, and elsewhere the backfilling was carried for several months. As a result of this design, it was anticipated that seepage would occur under the clay layer at the northern end of the dyke causing locally high uplift pressures downstream from the dyke which would dissipate with time. It was also realized that it was possible for some water to pass through and around the sheet piling.

Seven other piezometers installed at shallow depths (1 - 3 m) in the coarse sand layer between piezometers 1 to 8, in the northern part of the dyke, confirm that seepage takes place only through the fine sand layer. The decreasing pore pressures downstream from the dyke indicate an underground flow which is confirmed by the foundation seepage measured at the two extremities of the dyke. The horizontal hydraulic gradient at piezometer No. 5 attains a value of 0.2 to 0.25 but decreases to 0.002 near piezometer No. 30. The seepage at the northern end of the dyke emerges through the coarse sand, about 80 m (250 ft.) downstream from the piezometer No. 15 (Fig. 2). The exit gradient calculated vary between 0.1 and 0.2.

The piezometers Nos 6 and 5, adjacent to No. 4 show a piezometric difference of more than 15 m, giving a hydraulic gradient in the order of 0.2 to 0.25 between these piezometers.

Immediately downstream, the pore pressure starts to diminish towards the south abutment; the piezometers 10 to 13 show the water level to be at the bottom of the hole (Fig. 6).

The water escaping the sheet piling nearest to the piezometer No. 12 creates underground flow downstream from the dyke, towards the south abutment. However, the gradient between piezometer 21 to 22 is rather small, i.e. 0.025.
Weir No. 3 located much further downstream shows a lower real seepage which is only 1/3 to 1/2 that of the northern section (fig. 10).

**Stability**

**Dyke No. 1**

Stability analysis of this dyke was carried out in 1982 using the computer program "STABIL"; this program is based on the simplified Bishop's method. The section used for the stability check corresponds with the location of piezometer No. 4, where the highest pore pressure (el. 124.5 m against the reservoir el. of 132.4 m) was recorded in the fine sand layer overlying the clay deposit (fig. 7). An effective friction angle of 35 degrees was used for the clay core. The factor of safety obtained using measured pore pressure is 1.56 compared to long-term F.S. of 1.8 obtained during design, for a section near the north abutment where the dam attains its maximum height of 30 m; ground water downstream from the cut-offs was assumed to be at elev. 112.8 m (370 ft.) at the time of design. Effective stress parameters of the impervious soils and pore pressures used at the time of design were obtained from consolidated undrained triaxial tests and flow nets.

No dynamic stability analysis was carried out at the time of the design since the project location was considered to be in an area of low seismicity. According to the present Hydro-Quebec's standard, the two dykes fall in seismic zone 2, which corresponds to coefficients of seismic acceleration of 0.12 g horizontal and 0.06 g vertical. The F.S. obtained for pseudo-static stability, using the previous section (piez. No. 4), falls to 1.18.

**Dyke No. 2**

A cross-section near the south abutment using piezometer No. 2 was used in the stability analysis. The highest pore pressure measured in this piezometer is 132.5 m against the reservoir head of 132.92 m. This piezometer is installed at el. 109.27 m in the sand layer below the clay deposit. The F.S. obtained is 1.72 against 1.85 calculated for long-term steady seepage at the time of design.

The pseudo-static stability of the dyke using the same coefficients as for dyke No. 1 is 1.66.

However, the liquefaction potential of the fine sand foundation remains to be investigated.

**General Conditions of the Dykes**

**Dyke No. 1**

In 1964, only four years after the reservoir filling, changes in the upstream slope geometry, above elevation 132 m (433 ft.), were observed. Repair of the rip-rap was carried out during October and December, 1964.
During October, 1972 inspection, slumps were observed on the upstream rip-rap. On July 13, 1973, during a routine survey, cracks were observed on the upstream side of the dyke's crest. Because of the gravelly nature of the dyke's crest, it was difficult to evaluate the depth and importance of the cracks. The following measures were taken to study the effect of the cracks on the stability of the dyke:

- Three test pits were dug by hand on the crest;
- A review of the instrumentation results was carried out;
- Seventy-four new movement gauges, on both sides of the crest, were installed.

The test pits revealed that the cracks did not extend more than 30 cm (1 ft.) into the fill and that the cracks were filled with sand; the clay core was intact and in very good condition.

The review of the instrumentation results did not show any anomalies.

Since no significant movements were registered with the new gauges, between 16 and 30 of July 1974, the readings of the new gauges were discontinued.

It was thus concluded that these surface cracks did not in anyway affect either the stability of the dyke or imperviousness of the core.

The 1982 inspection revealed that the upstream rip-rap had again slumped in several places and that the rip-rap had been exposed at several locations. Remedial works were recommended and are planned for 1984.

The rip-rap problem has been caused due to inadequate rockfill size. The inadequate rockfill size (smaller than recommended) was produced due to the presence of fractures in the rock quarry.

The rip-rap for the 1984 repair work will be obtained from a quarry with sound rock to assure that the recommended dimensions of the rockfill are respected.

Fine sand conforming to AASHO-W-147-granulometry-D - will be added to the crest during the 1984 rip-rap repair work to allow for the observation of any future cracks.

**Dyke No. 2**

**Upstream Rip-Rap**

Minor slumps observed on the upstream rip-rap were repaired in 1965. No fissures were observed on the dyke's crest. Recent inspections show that the rip-rap is in excellent condition.

**Drainage**

About 122 m (400 ft.) north of piezometer no. 2, a trickle of water coming out of the dyke foundation, most probably through the toe drain, was noticed during the 1978 inspection. The toe drain was provided to drain away any water collecting on the downstream side of the dam. This drain was provided with a graded filter, and offtake ditches to connect it to the natural drainage system of the terrace. The seepage together with runoff accumulates in the form of a pond, slightly further downstream. This pond communicates with a lake situated further east of the pond. This accumulation of water is the result of the original drainage ditches being filled with all sorts of debris over a period of time.

Excavation of the old drainage ditches and installation of a weir is planned as part of normal maintenance.

**Downstream Rip-Rap**

During the 1978 inspection, the downstream rip-rap composed of gneiss containing biotite and pyroxene, was found to have desintegrated into small fragments (coarse sand and gravel). However no adverse effects were noted on the downstream slope of the dyke, probably due to the additional protection provided by the underlying one-foot thick layer of gravel.

A petrographic analysis carried out on a thin layer of representative rock sample shows that gneiss is friable and contains an appreciable amount of scattered magnetite. It is therefore the oxidation of mineral containing iron (specially magnetite and the accompanying biotite and pyroxene) which has caused the disintegration of the rock under the effect of inclement weather. The burning of the floating wood, collected from the upstream, on the downstream slope has also contributed to the desintegration process.

No replacement of the downstream protection is envisaged at this time.

**CONCLUSION**

The two dykes completed in December, 1958 have performed more or less as predicted. They are stable under the steady state seepage even though the stability analysis carried out using measured pore pressure show some reduction in the factor of safety. As far as the stability during earthquake is concerned, they are stable for the coefficients of acceleration (0.12 g horizontal and 0.06 vertical) of zone 2 of seismic zoning (pseudo-static condition); the liquefaction potential of the foundation sand remains to be investigated.

The high pore pressures, immediately downstream from the dykes, and seepage have remained almost constant since the reservoir filling. The fact that seepage has remained constant and has shown a slight tendency to reduce in recent years confirms the integrity of the foundation. The settlement of both dykes are well within the predicted values.

In general the dykes have performed very well except for the upstream rip-rap problem of dyke No. 1 and the desintegration of the downstream rip-rap of dyke No. 2.

**REFERENCES**


The author wishes to thank Hydro-Québec for permission to publish this article and to Messrs. O. Dascal, R. Piché, H. Parent & Mrs. S. LeBel for helping me with the preparation of this article.