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STABILIZATION OF THE DIVIDING WALL AT A DRINKING WATER RESERVOIR

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ABSTRACT

The Reservoir is a component of the water supply system of a major city in the northeastern United States. The reservoir receives water conveyed by aqueducts and pressure tunnels from several watershed reservoirs. It is a balancing reservoir that regulates water flow and maintains the elevation head needed for the water tunnel distribution system to the south of the reservoir, which services the city inhabitants. The reservoir water surface is approximately 90 acres, and is contained by a man-made rim earth embankment, 40 feet in average height. The reservoir, which has an oval shape, is lined with a paneled concrete slab and is divided into an east and west basin of about equal size by a concrete dividing wall that traverses the reservoir along its longer axis. The wall, completed in 1916, stands 45 feet tall and has a 34.7 feet wide base. A by-pass aqueduct was built within the wall. After about eighty years of uninterrupted operation, the basins were cleaned of accumulated sediments. The paper describes the stability issues of the dividing wall during cleaning the basins. The original wall design did not account for dewatering one basin while the other remains in service with the uplift pressures developed in the foundation since start of first operation in 1917. The stability of the wall had to be improved in order to make possible the complete draining of one basin while the other remains fully operational. Although, initially, post-tensioned anchors into bedrock were considered to increase the wall stability against unbalanced water thrusts, additional assessments of the foundation material lead, instead, to the use of a temporary well-point dewatering system installed along the full length of the wall 20 feet into the underlying Glacial Till. The wellpoints were used to reduce uplift pressures under the wall base. The paper describes the stabilizing and monitoring procedures adopted during dewatering. It describes the analyses and monitoring of safe drawdown rates of the reservoir basins to prevent failure of the rim embankment slopes. It also describes in detail the designs, new factors of safety and test results, and construction of permanent stabilizing mass concrete buttresses that provide the necessary passive resistance for future dewatering of either basin for maintenance, cleaning and/or additional future construction.

INTRODUCTION AND SITE DESCRIPTION

The reservoir is located near a major northeastern metropolitan center and was built between the years 1908 and 1915. The reservoir is used to balance variations in the supply of water from other reservoirs. It is an essential component of that city's water supply system. The reservoir water surface is approximately 90 acres, and is contained by a man-made earth embankment. The approximately 40-foot high embankment dam encloses the reservoir and was constructed of local material removed during the excavation to form the reservoir basins. The reservoir is lined with a paneled concrete slab and is divided by a concrete dividing wall into an east and west basin of about equal size. The reservoir depth when full is 36.5 ft.

After 80 years of continuous operation it was determined that as much as 4 to 6 inches of soft sediments had accumulated in the reservoir bottom. To maintain water quality it was decided to clean both basins one at a time. The dividing wall had to be permanently raised before the emptying of any one basin could proceed.

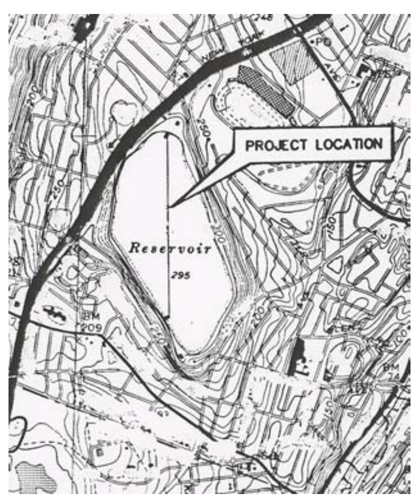

Fig. 1. Reservoir Layout

Prior to 1998, the basins of the reservoir had never been fully emptied since the initial filling of the reservoir around 1917. It was recognized that the rate of drawdown of the reservoir could create potential sliding failure of the earth rim embankment, and unequal levels of water in the reservoir basins could lead to instability of the dividing wall.

Before the emptying of the basins in 1998 there was also concern regarding uplift of the reservoir's bottom slabs. However, after careful monitoring during the emptying of the basins for the reservoir cleaning, it was concluded that slab uplift was not a concern at the adopted reservoir drawdown rates. No slab uplift was observed from daily high resolution side-scan monitoring during drawdown performed from a small floating vessel during drawdown.

The stability of the rim embankment was achieved by limiting the drawdown rate of the basins whereas the stability of the dividing wall was controlled by an active foundation dewatering system installed along the wall. Stability of the embankment and dividing wall was monitored around the clock during basin dewatering and empty conditions by instrumentation installed for that purpose.

SITE GEOLOGY

The reservoir site is located within the Manhattan Prong of the New England Upland region of the Appalachian Highland Physiographic Province. Before the construction of the reservoir, basal glacial soil deposits at the site consisted of substantial thickness of relatively unsorted sand, gravel and clayey-silt with frequent cobbles and boulders randomly distributed in the material. The glacial till material was deposited primarily during the most recent glacial advance/retreat designated as the Wisconsin glacial stage. Underlying the till is igneous, intrusive bedrock, known as the Yonkers Granite. The rock is of widely variable composition and commonly shows strong gneissic (light and gray banding of minerals) characteristics. The average thickness of the dense basal till under the bottom of the reservoir varies from 60 to 80 feet. The original natural ground water level at the reservoir was low. However, 80 years of full reservoir operation has created a ground water dome fully saturating the materials underlying the reservoir and its vicinity. Most of the natural drainage of the reservoir subsurface is toward the river valley to the south-east about 250 feet below.

EXPLORATION OF DIVIDING WALL SUBSURFACE

The mass concrete dividing wall is 2,750 feet in length and runs from the uptake building on the north side to the downtake building on the south side of the reservoir. The wall, symmetrical along its vertical axis, was constructed out of concrete in monoliths, typically about 30 feet in length. Vertical joints between monoliths are keyed and water-stopped with metal strips. A circular by-pass aqueduct, twelve feet in diameter was cast within the wall near its base. The wall has a base width of 34.7 ft and its original height was 36.5 ft. Before proceeding with the emptying of the basins, it was necessary to investigate the nature of the foundation materials, depth to bedrock and the uplift conditions under the wall base in order to establish the variables controlling its stability. For this purpose five sections along the wall were selected. Drillholes were advanced through the sides of the wall, approximately 10 ft from its centerline, on opposite sides several feet into the underlying Glacial Till foundation, Fig.2.

The drillholes were 6 inches in diameter and were drilled from a drilling rig mounted on a floating barge. All equipment, including the sectional floats, drill rig, casings, supplies and tools, were steam cleaned and sprayed with chlorine solution prior to being placed in the reservoir or transported to the barge. Other measures required for the protection of water quality included the use of propane fuel and non-petroleum based lubricants. Drilling water was recirculated and suspended concrete and soil allowed to settle out. The drill spoils were brought to shore for disposal. An absorbent boom was placed at the front of the barge around the drill casing.

Three of the drillholes, one at the north side, one at the center and one at the south side of the wall, were taken down to bedrock, see Fig. 2. The need to know the bedrock depth and its quality stemmed from the initial thought of using post-tensioned anchors to increase the wall stability. Because of the gravel and cobbles present in the Glacial Till, only disturbed samples were recovered for index properties and grain size distribution analyses. The sieve analyses showed,

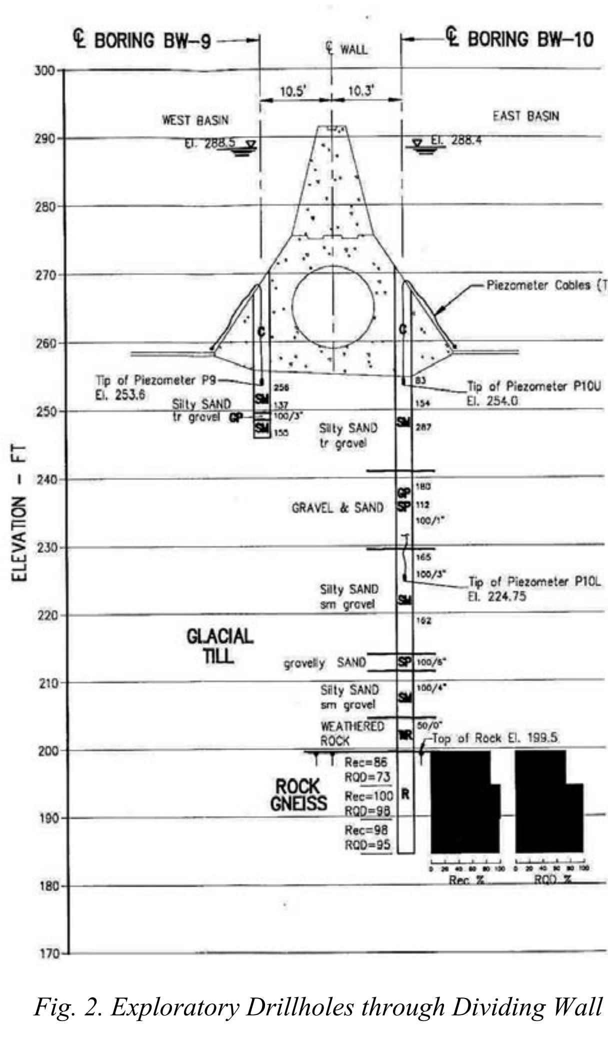

Fig. 2. Exploratory Drillholes through Dividing Wall

typically, that the Glacial Till under the wall base had fines content, soil particles smaller than 0.074 mm (No. 200 mesh) between 20 and 40%, and of low plastic index. The Glacial Till encountered directly under the wall was very dense comprised of boulders, gravel, sand and silt with some clay and was similar to materials encountered in prior explorations and representative of the Glacial Till confirming that the wall was founded directly on it.

Piezometers were installed in each boring at the contact between the wall base and the foundation. In the deep drillholes a second piezometer was installed at depth. The piezometers were of electric vibrating-wire type. The selection of piezometer type and their purpose was to measure uplift pressures and its changes under the wall base with a quick response device. The deep piezometers were installed to establish piezometric heads at depth and hydraulic gradients in the Glacial Till. Before draining the basins and wall stabilization implementation, drawdown tests of the basins were performed to El. 278 ft, approximately 12 feet below the average operating level of the reservoir, to obtain hydraulic head response in the foundation material and magnitude of change of the uplift pressures under the wall. The drawdown tests were also used to calibrate the finite element seepage model that encompassed the reservoir, rim embankment and foundation.

It was established that the piezometric response was relatively rapid to changes of reservoir level and that in general the hydrostatic heads in the Glacial Till under the base of the wall are 30% to 40% lower than the hydrostatic head of the free standing water in the reservoir. Subsequent investigations of additional observation wells along the length of the wall, and installation of piezometers under the bottom slab of the reservoir confirmed the hydrostatic head difference found.

DIVIDING WALL EXTENSION

The removal of sediments and the cleaning of the reservoir were undertaken in compliance with a stipulation agreement with the Department of Health. However, before performing the cleaning of the basins, the dividing wall had to be raised. The raising from its original top elevation 291.5 ft was necessary to be able to operate the reservoir at its maximum water level of El. 295 ft, above the original wall top elevation, during the period of time when one basin is out of service for cleaning. The wall was permanently extended in height to El. 300 ft with a reinforced concrete extension 8 feet wide by 8.5 feet high (Fig. 3). This work was completed in 1997 after which the basins were cleaned in the dry, one at a time, in the Winter/Spring 1997/98 and fall of 1998, respectively. Subsequently both basins were again fully drained, one at a time, to construct the dividing wall stabilizing buttresses. The construction of permanent stabilizing buttresses along both sides of the dividing wall toes was necessary to increase the wall stability for future dewatering of basins without the use of the active foundation dewatering system, described in detail in the next Section, used during cleaning of the basins in 1998. The buttresses act as a passive system by providing additional weight and lateral resistance against sliding due to unbalanced water levels in the reservoir basins. Detailed description and purpose of the buttresses is described in subsequent Sections. Construction of the west basin buttress was done in the autumn of 1999 and the east basin buttress was completed in the spring of 2000.

STABILIZATION OF THE WALL DURING CLEAN-UP

The original dividing wall, constructed around 1912, was built on pre-existing very dense Basal Glacial Till, which is about 60 ft in thickness overlying Gneissitic Bedrock. The formation of the reservoir caused the buildup of the groundwater level in the area. This buildup created uplift pressures under the wall that lowered the original factors of safety against sliding under unbalanced water conditions between the two basins to less than one.

Under unequal basin water levels, the dividing wall is subject to unbalanced lateral water thrusts that need to be resisted by frictional resistance along the wall base. The net excess horizontal water thrust increases as the unbalanced water levels between the two basins gets larger. Because the frictional resistance derived from effective contact stresses due to gravitational forces acting at the wall base were not sufficient to meet the necessary factors of safety against sliding, and also to maintain the resultant of all forces close to or within the middle-third of the wall’s base, it became

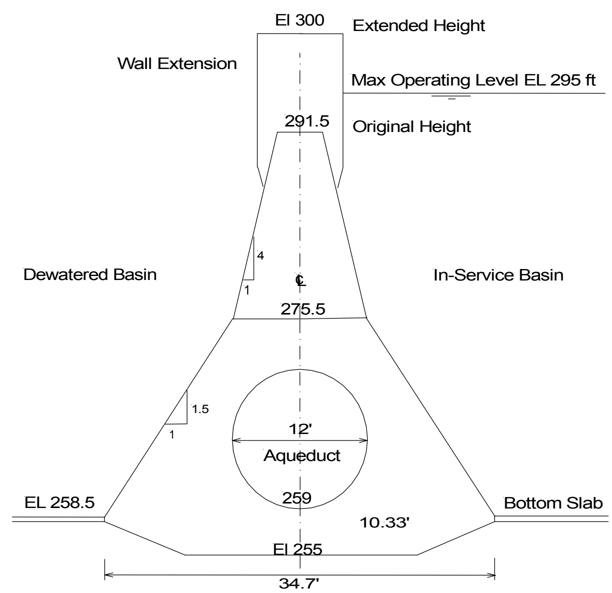

Fig. 3. Dividing Wall and New Extension

necessary to investigate methods that would increase the normal effective stress at the base of the wall. Based on the index properties of soil laboratory tests of samples recovered from under the wall base it was decided to treat the foundation formation as a purely frictional material with no cohesion. The adopted friction angle for base-shear failure was 35 degrees.

The increase in normal force at the wall base can be achieved by (1) adding an external force to the wall, such as vertical post-tensioned anchors or (2) the reduction of hydrostatic uplift pressures.

Initially, it was considered to increase the stability to the wall by installing post-tensioned high capacity tendons anchored into the underlying bedrock on average about 70 feet below the base. Calculations indicated that three 675-kip anchors at ten feet on center per wall monolith would be required to obtain an acceptable factor of safety against sliding. The length of the anchors was to be approximately 140 feet. The total amount of post-tensioned anchors required would have been 300. However, based on grain size distribution analysis of samples obtained from the wall foundation investigations, together with the calculation of exit hydraulic gradients from underseepage flow net analyses, and uncertainties about the behavior of the bottom slab regarding their uplift near the wall on the dewatered basin, it was concluded that the foundation material was susceptible to piping of its fine fraction. The piping potential would have created foundation undermining leading to wall tilting and greatly decreasing the usefulness of the anchors if installed. Although the post-tensioned anchor solution was regarded as a permanent long-term stabilization measure, the risk of foundation undermining was regarded too high. Therefore, to attain the necessary wall stability, in the short term, it was resolved to reduce the hydrostatic uplift forces instead. The investigation of these uplift pressures and their effect, on the dividing wall stability was extensively studied starting in the fall of 1995. The wall has at present 12 permanent monitoring sections, about 220 feet apart, where uplift conditions under the wall were measured. The observation wells consist of open standpipes and electric-type piezometers. The open standpipes were furnished with pressure transducers lowered to the bottom of the pipes for automatic remote reading. The transducers and electric-type piezometers were read at remote readout units connected to data loggers installed in the uptake and downtake buildings at each end of the wall.

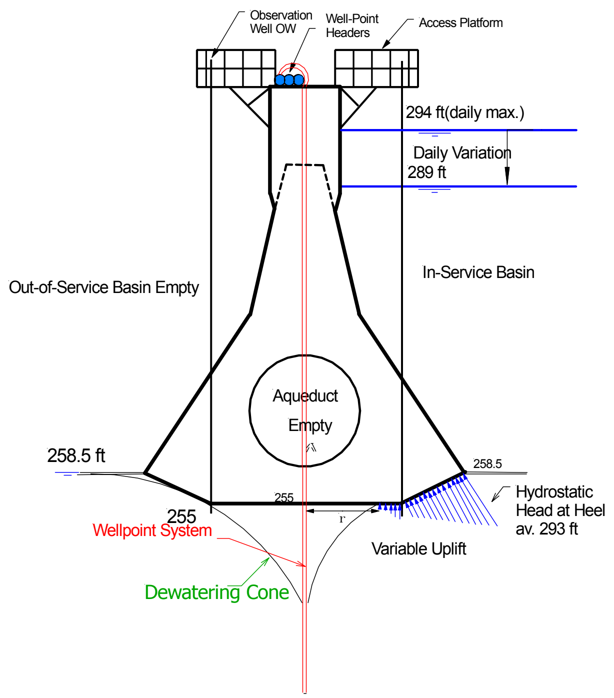

Fig. 4. Foundation Dewatering and Observation Wells. Dewatering Cones Reduced Uplift under the Wall

Based on these measurements, the results of two limited drawdown tests and the reduction of uplift obtained at three test sections along the wall using wellpoints, an ejector-vacuum pumped wellpoint system was arrived at and constructed. The system consisted of 457 ejector wells, at 6 feet on center, 20 feet deep into foundation ground under the centerline of the wall. The wellpoints were installed in 6-inch diameter holes drilled from the top of the wall. This arrangement became the active uplift pressure reduction system used to maintain wall stability during the time the basins were cleaned, and subsequently for the construction of the permanent stabilizing scheme (passive buttresses) when both basins had to be fully emptied again.

The wellpoints were continuously pumped through ejector heads installed on each well. To diminish the safety risks of pump breakdown or power failure, the wells were hooked up to three separately pump-operated heather pipes. This mechanical system reduced uplift by creating overlapping groundwater depression cones under the wall (Fig. 4), reducing uplift pressures to meet the desired factor of safety of 1.75 against wall sliding.

Although successful as a temporary stabilizing method, it was decided that future draining of the basins could be better achieved by a passive and simpler system to increase the factors of safety. Such a passive system would replace the active dewatering used during the first cleaning of the reservoir basins. The active system used to perform the first cleaning and subsequently to build the permanent stabilizing scheme, entailed considerable monitoring and round-the-clock maintenance to perform the necessary work within acceptable risk levels. The use of this active system for future dewatering and cleaning was deemed too costly and complex for permanent operations, and its replacement by a permanent, risk-free, passive system was highly desirable. The passive system entailed concrete buttresses acting as wall toe support. They were designed to aid the dividing wall to withstand the maximum operating level plus two feet of surcharge in one basin during the time the other basin is fully drained and empty.

DIVIDING WALL BUTTRESSES

The toe supports provided a permanent passive system that consists of mass-concrete buttresses built along both sides of the wall to add to the frictional resistance of the wall. The buttresses also aid the wall against potential overturning. Uplift under the wall and buttress is controlled by longitudinal gravel-filled underdrains built into the new toe supports. These underdrains run along the toes of the original wall, and act as seepage collectors. The gravel underdrains, which are 6 feet in width, are wrapped in geotextile fabric. The fabric acts as a filter to prevent migration of fines from under the wall foundation from seepage flow. A 12-inch diameter perforated PVC collector pipe runs along the full length of the drain and discharges at two valved outlets. When the basin is empty, the valves are opened for drain seepage water to discharge into the empty basin through a sump pit. The valve operating wheels are accessible by a ladder from the top of the dividing wall to the top of the buttress at El. 270 ft, and an aluminum platform walkway to the valve wrench location.

The buttresses were built in 60-foot long monoliths, of non-reinforced concrete poured in two lifts of approximately 6 and 5.5 feet in height each. The horizontal lifts were provided with two longitudinal keys. Vertical expansion joints between monoliths were provided with two vertical keys. Buttress monoliths were constructed in a checkerboard pattern (Fig. 5). The buttresses were not structurally

Fig.5. Buttress Construction in Checkerboard Fashion

connected to the dividing wall and are 16 feet wide at their base and 11.5 ft high, and are founded on the existing 6-inch thick bottom concrete slab. Each buttress took approximately two and one-half months to complete and a placed total mass concrete volume of 25,000 cyds per buttress. The final modified wall section is shown in Fig. 6 below.

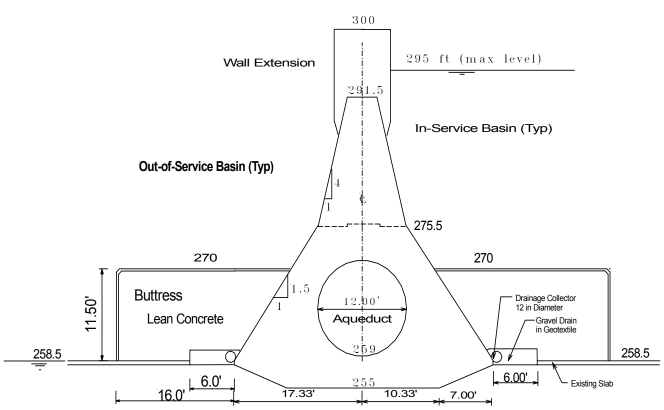

Fig.6. Dividing Wall & Permanent Concrete Buttresses

Factors of Safety of the Buttressed Wall

The factors of safety considered for the stability of the dividing wall were (1) stability against sliding and (2) stability against overturning. The first is obtained by the sum of the net frictional resistance along the base of the wall and buttress over the driving forces. The stability against overturning is assured by keeping the resultant of all forces close or within the middle third of the wall base. The buttresses were designed and dimensioned to satisfy these two stability considerations. It was found, however, that the stability against sliding is the controlling mode of failure. Given the possibility of leakage from the basins through buttress expansion joints and other seepage paths into the buttress underdrain, it was assumed that hydrostatic pressures in the soil beneath the wall are related to the water level in each basin. Thus, when the water level in one basin is lower than the other basin and the wall is functioning as a

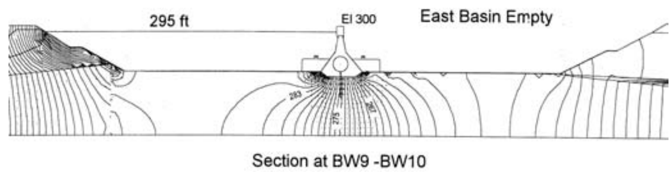

Fig. 7. Global Seepage Analysis

dam, the hydrostatic pressure in the soil at the heel of the wall (the side of the wall with higher water level) along the buttress underdrain, was assumed equal to the height of water above the heel multiplied by the unit weight of water. Furthermore, the hydrostatic pressure at the toe of the wall (the side of the wall with the lower water level) was assumed equal to the height of water above the toe underdrain multiplied by the unit weight of water. Therefore, for the stability analyses the uplift distribution was considered to have a trapezoidal distribution when one basin is partially dewatered and a triangular distribution when one basin is completely empty. Flow net analyses results confirmed that the assumed distributions were conservative (Fig.7).

In view of the risk and potential consequences of failure of the wall would have in the water supply system, the minimum factor of safety against sliding for one basin empty and the other in operation with a water level at El. 295 ft was set at 1.75 with the by-pass aqueduct within the wall full of water. The minimum factor of safety without water in the aqueduct is 1.63. Both factors of safety were considered adequate for future dewatering of the basins.

A stability test was performed on the east basin buttress after its construction completion and before filling the basin. The test was carried out over a 5-day period to assess the foundation response to uplift pressures created by the west basin, which was in service. The results are summarized in a graphical display (Fig. 8, next page) of typical uplift pressure distributions, with their associated factors of safety, which can be expected in the future. Although the west basin buttress was not tested, a similar uplift distribution pattern is expected to occur in the future when the west side of the reservoir is out of service and the east basin is operational.

The test results show that for an operating level of the in-service basin equal to 293 ft, the average factor of safety of the wall against sliding is 2.07, with the lowest value of 1.90 at an instrumented section at the south side of the wall. The calculated factors of safety were for the empty aqueduct condition, which was the case during the test. The uplift response at the section with highest readings (section 1R/P2u, at the south side of the wall) was extrapolated for an in-service basin at its maximum operating level of El. 295 ft.

For that condition, the anticipated factor of safety was calculated to be 1.81 for the aqueduct full of water condition, 1.69 for the empty aqueduct. Both values are above the minimum design values.

The graphical presentation below shows the measured distribution of uplift under the wall during the test period compared to the maximum theoretical distributions. Factors of safety are shown for three typical sections. Piezometric head readings at other sections fall generally between the limits shown.

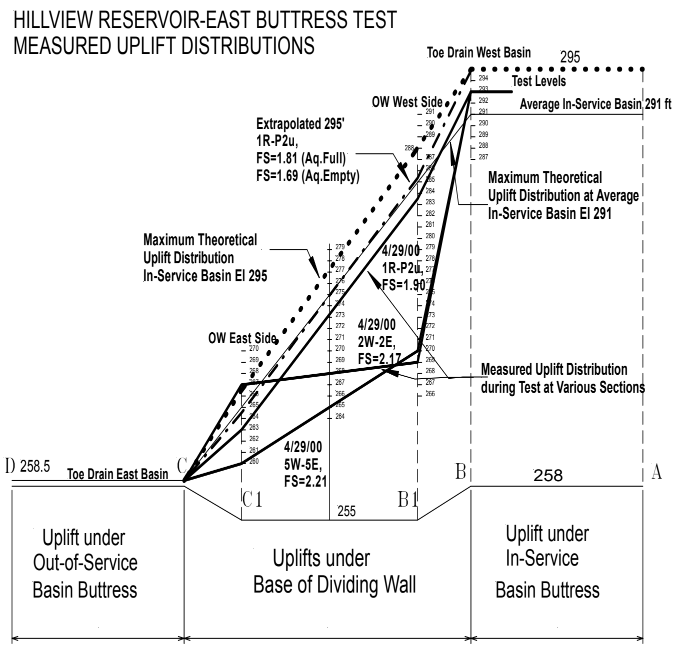

Fig. 8. Stability Test of East Side Buttressed Wall

<u>Discussion of Factors of Safety</u>

The active dewatering system used to lower the uplift pressures under the dividing wall during cleaning and construction of the buttresses was essential to maintain adequate factors of safety against sliding of the wall when one basin is completely empty. Without it, the wall could have failed. The computed factor of safety of the wall, using a 2D analysis, without the uplift control is less than one. By installing the foundation dewatering system described earlier and limiting the maximum operating level to between two and three feet below the maximum El. 295 ft, it was possible to increase the average factor of safety of the wall to a value of 1.7±. Thus, the stability techniques used during these operations reduced the risk of failure to an acceptable level. Although the temporary system provided stability as long as the system was working, any serious breakdown of the system, such as freezing during cold weather, loss of power from the primary grid, or failure of the backup power generator, could have seriously jeopardized safe conditions of the work in the empty basin. Therefore, there was a desire to find a permanent solution regarding the stability of the wall during all future basin emptying that would not impose limitations of the maximum water level in the in-service basin and at the same time provide an acceptable factor of safety via a more passive and simplified system.

This was a particularly pressing consideration for the possible construction of a concrete cover over the reservoir, which would entail keeping each basin empty for considerable periods of time. The inherit risk of an active system together with restrictions of reservoir level in the active basin to ensure the dividing wall stability was unacceptable. This goal was accomplished by replacing the active dewatering system with passive supports, i.e., stabilizing buttresses, on each side of the wall. The benefits obtained in stability with the addition of the concrete buttresses is clear, and is summarized below in terms of new permanent factors of safety of the wall. The risk of failure has been virtually eliminated by these new structures. The quoted factors of safety in Table 1 are for the case when one basin is empty and the other is operating at a maximum water level equal to 295 ft. Values have been calculated both for the case with the by-pass aqueduct in operation and full with water and the empty case.

Table 1. New Computed Factors of Safety

	New Factors of Safety of the Dividing Wall		
	Uplift Conditions		
Aqueduct	Normal (*)	Extreme (**)	"Worst Case" (***)
Full	1.75	1.44	1.33
Empty	1.65	1.33	1.21

(*) Triangular distribution. The full hydrostatic head on in-service basin side of the wall dissipates linearly from the heel to the toe of the wall base.
(**) Full hydrostatic head of the in-service basin extends to the wall base centerline and dissipates thereafter linearly to the toe.
(***) Full hydrostatic head of the in-service basin extends to the toe-end of horizontal portion of the wall base and dissipates thereafter linearly to the wall toe.

Uplift distributions during testing of the east basin buttress all fell below the normal triangular distribution. Thus, higher calculated factors of safety were actually obtained during the test. The west buttress has not been tested, but is expected to behave satisfactorily and similar to the east buttress based on the piezometric response gained during the full dewatering of that basin.

EMBANKMENT STABILITY

The embankment is approximately 8,500 feet long following an oval shape. The inner slope is 2H to 1V extending from El. 258.5 ft above mean sea level, which is the bottom of the reservoir, to the embankment crest at El. 300 ft. The slope is interrupted at about mid-height, El. 280 ft, by a 10-foot wide bench. The lower portion of the slope is concrete lined. Above El. 280, the side slopes of the embankment rise generally at about 2.5 H to 1V; however, there are some portions of the embankment with steeper slopes. Above the bench the slope is covered by hand-placed stones up to about two feet in size, followed by dumped riprap with stones typically between four and fifteen inches in size to El. 297 ft. Grass covers the remaining top portion of the slope. The concrete liner from El. 280 to the toe of slope is 8 inches

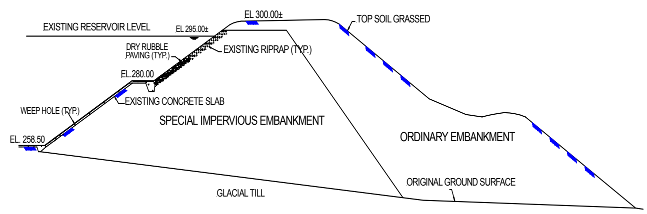

Fig. 9. Typical Embankment Section with Fill Materials

thick and is provided with pattern weepholes (eight weepholes per joint and joints are typically 8 ft apart). The upper section of the slope above the bench is free draining.

The embankment was constructed with two fill zones. The inner portion of the embankment was built of special impervious fill, which consists of screened Glacial Till obtained directly from the excavation of the reservoir. It was placed in 4-inch lifts and compacted with 10-ton sheepfoot rollers. The outer slope or stabilizing shoulder, the ordinary embankment zone, was built of unsorted Glacial Till and less compacted then the impervious zone; see Fig. 9.

The stability of the inner embankment slope during drawdown of the basins was also a concern. During drawdown of the reservoir, the water contained within the select fill will slowly drain from the embankment into the Glacial Till beneath and reservoir. The rate of drainage and pore-pressure dissipation in the select fill of the inner slope is a function of the reservoir drawdown rate, properties of the materials and efficiency of the weepholes in the concrete lined lower slope. The rate of drawdown was studied theoretically by means of flow nets and integrated stability analyses.

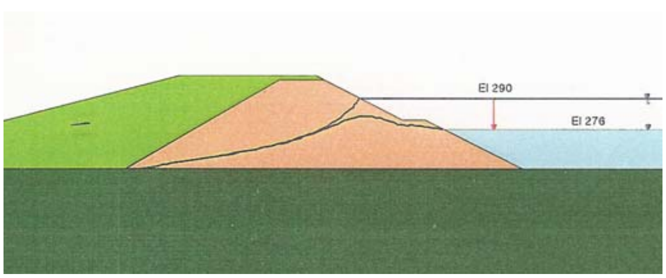

Fig. 10. Embankment Phreatic Change under Drawdown

The east basin was the first to be dewatered. For this basin an initial rate of 6 inches per 24 hours was adopted. It was found that for that rate the lowest acceptable factor of safety of the slope of 1.25 occurred when the pool had been drawdown 26 feet to about El. 265 ft, i.e., and a remaining pool depth of about 7 feet. To verify the analyses and confirm that the adopted rates of drawdown were acceptable, five instrumented sections were installed on each the east and west rim embankments. The instrumented sections were about 700 ft apart and entailed the installation of three pneumatic piezometers 5 to 10 feet deep into the reservoir side slope. The need of deeper piezometer tips was not necessary since all stability analyses indicated rather shallow failures entailing the lower portions of the slope below the 10-foot bench. Therefore, one was installed in the El. 280 ft mid-height 10-foot bench, the second on the 2H:1V slope, typically at about half the slope height, and a third one a few feet from the toe of the embankment slope in the Glacial Till under the bottom slab. These piezometers were read on a daily basis and the data was used as input to check drawdown analyses and compared with residual pore pressures obtained from the drawdown model. The piezometric heads were used to run fresh stability analyses on a daily basis to ascertain the slope safety and the adjustment of the adopted drawdown rates, if any, to ensure adequate pore-pressure dissipation to maintain slope safety.

The shear strength parameters used in the embankment slope stability were obtained from triaxial testing of undisturbed samples taken during the site investigations executed for the purpose of strength investigations of the embankment materials. A friction angle of 39 degrees was obtained from the tests. Friction angles adopted in the stability analyses were between 36 and 39 degrees. Triaxial test results also served to confirm and validate the friction angle used in the foundation material under the dividing wall. Shear resistance values for the outer slope material and intact Glacial Till were derived from SPT blow counts.

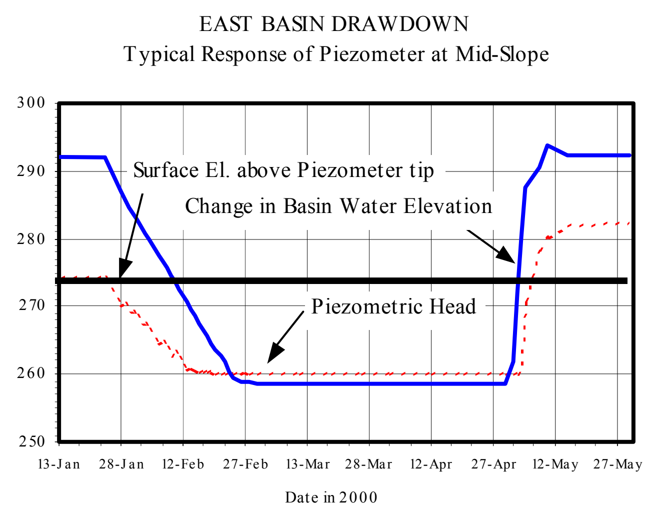

Fig. 11. Second East Basin Drawdown & Piezometric Response

Based on daily checks on pore-pressure dissipation and stability runs it was decided to increase the drawdown rate of the east basin to 9 inches per day. This would shorten the time needed to empty the basin and increase the time available to clean the basin before the coming summer season starting around the beginning of June of each year, when both basins need to be fully operational to meet the warm/hot water demands of the city. Calculated factors of safety for the most critical pool level of around El. 268 ft were 1.20 at one of the instrumented sections without considering three-dimensional restrains.

From the experience learned at the east basin and piezometric measurements taken during the initial favorable response of drawdown at the west basin, together with the knowledge that negative pore-pressures that develop due to shear strain in the slope will increase the shear resistance, it was decided to drawndown the west basin at a rate of 12-inch per day. Periodic pore pressure measurements at five instrumented slope sections confirmed that the adopted drawdown rate was safe. The rate increase from 9 to 12 inches in 24 hours created a saving of 10 days.

Figure 11 shows the adopted drawdown rate at the east basin during second emptying for the construction of the east wall buttress. The figure also shows the response of the piezometer installed at mid height of the lower slope. The drawdown rate adopted this time around was 12 inches in 24 hours. The behavior and calculated factors of safety were satisfactory and no distress was observed on the slope or at the 10-foot bench. From the experience gained during emptying each basin twice it was concluded that a maximum safe drawdown rate for both basins should not exceed 12 inches in a 24-hour period.

MONITORING

Extensive monitoring was performed during the emptying of the basins. From the start of basin drawdown to putting the basin back in-service took approximately 4.5 months. During that period of time and in agreement with the client's request, surveillance and comprehensive monitoring of the structures involved were performed around-the-clock seven days a week. The dividing wall monitoring entailed biweekly topographical survey of the top of the wall at 182 fixed survey points. Daily measurements of clinometers installed on each wall monolith and daily readings of crackmeters to an accuracy of 1 mm installed across the monolith expansion joints formed part of the monitoring effort. Twice-daily manual readings of the observation wells installed along the wall at 12 sections were performed to cross-check the automated readings stored in a data acquisition system. However, as mentioned above, pressure transducers installed near the bottom of the wells read pressures continuously and the automatic readout unit was hooked up to an alarm system with speakers at both intake and downtake chambers. The trigger levels were set based on water levels measured in the observation wells that could pose risks to the wall stability. In the event of such occurrences, the in-service basin level would be lowered through available overflow facilities in the downtake building to lower uplift pressures and maintain adequate safety factors.

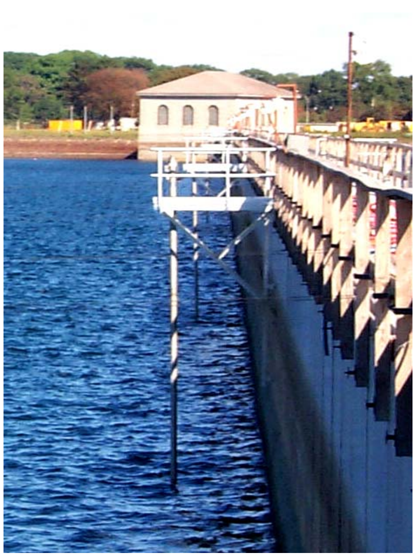

Fig. 12. Dividing Wall Monitoring Station for Wall Uplift

The 6-inch reservoir bottom slab was monitored for potential displacement using geophysical survey techniques that included echo-sounding and high-resolution side scan sonar. The bottom slab performance to uplift pressures was carried out daily by profiling the basin floor along established track lines, parallel to the dividing wall, at a spacing of approximately 50 feet on center. The physical collection of data from a small vessel was usually performed in the mornings and the analysis, evaluation and graphical presentation of data was done in the afternoon and discussed with the engineers.

The embankment slopes were checked by measuring piezometric levels at the various installed piezometers. Readings were performed twice a day and stability of the slopes was updated daily using measured pore water pressures.

In addition, the dividing wall was visually inspected every two hours and the inner reservoir slope at 4-hour intervals by walking along the exposed mid-slope bench.

There were weekly briefing meetings at the site with the various client's divisions, namely design, construction and operations as well as the consulting engineering team involved and invited guests to review the wall stability and its performance as well as the rim embankment behavior and change in the course of action if required. In great part the success achieved can be attributed to very open channels of communications among the parties involved, and the overall appreciation by all of the interest and technical challenge needed to accomplish a successful operation within tolerable risks understood by all.

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