15 Apr 2004, 1:00pm - 2:45pm

Drake Lake Dam — A Performance Case History

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ABSTRACT

Drake Lake Dam was constructed during the summer of 2000 as part of the development of the Jim Edgar/Panther Creek State Fish and Wildlife Area in Cass County, Illinois. The earthen dam on a natural soil foundation has a length of 450 feet and a height of 45 feet with 3H:1V upstream and downstream slopes. The dam has a principal spillway structure consisting of a cast-in-place concrete drop inlet tower, a 36-inch reinforced concrete pipe (RCP) outlet pipe, and an impact-basin-type energy dissipater/outlet structure. Construction of the earthen embankment began in July 2000 and ended in September 2000. During the latter stages of construction, observed and measurable distress of the dam was detected in the inlet tower and RCP outlet pipe. The movement measured in the principal spillway intake tower and the 36-inch RCP was the result of lateral spreading and vertical cracks in the embankment dam, both in the downstream and upstream direction, due to rapid embankment filling over the soft, silty clay foundation soils. Redesign of the dam structure was required to account for these unexpected site conditions. Results of monitoring the dam during and after construction are presented.

PROJECT BACKGOUND

Drake Lake Dam was constructed as part of the development of the Jim Edgar/Panther Creek State Fish and Wildlife Area within Cass County, Illinois. The dam was constructed across a tributary stream to Cox Creek. At normal pool, Drake Lake will have a surface area of 34 acres. The 70,000-cubic-yard earthen dam has a height of 45 feet and sideslopes of 3H:1V for both the downstream and upstream sides of the dam. The crest elevation of the dam is +591.75 feet. The normal pool elevation is +585.0 feet, and the high reservoir level is elevation +589.5 feet. A site plan of the Drake Lake Dam is shown in Figure 1.

The dam has a primary spillway structure consisting of a cast-in-place concrete inlet tower, 36-inch-diameter RCP, and an impact basin type, energy dissipater/outlet structure. The inlet structure is supported on a driven timber pile foundation and the energy dissipater/outlet structure is supported on a mat foundation. The RCP was constructed using 20-foot pre-cast segments. After the RCP was put in place, but before placement of any fill around or over the pipe, it was leak tested using 45 feet of head pressure and no leaks were observed.

This earthen dam was constructed from July to September 2000 and experienced observable and measurable distress during the latter periods of the construction. Upstream tilting of the principal inlet structure tower was observed. The upstream tilt was measured in the range of 9 inches from the current top of the spillway structure to the top of the pile cap. In addition, the segmental, 36-inch-diameter RCP beneath the embankment elongated horizontally and settled a maximum of 10.2 inches at the joint, 20 feet downstream of the spillway intake tower, with lesser amounts of settlement at pipe joints near the toe. The pipe joints had total joint separation in the range of 13 inches (8 inches downstream of the intake structure and 5 inches upstream of intake structure at the base of the spillway intake tower) measured from the original intake to energy dissipater. The RCP had radial cracks internally near the separated joints at some locations. While drilling a

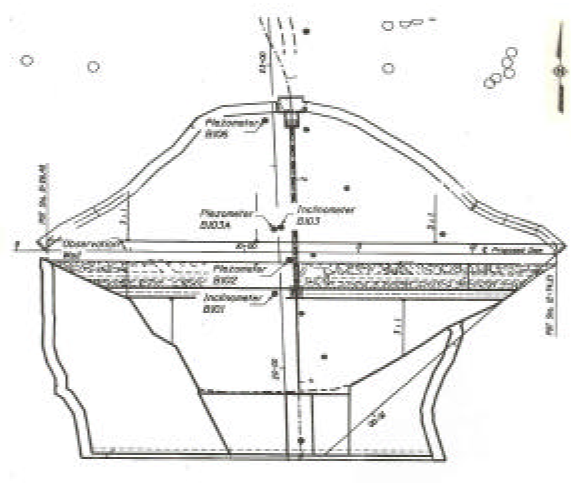

Fig. 1. Drake lake Dam – Plan View

borehole 10 feet from the RCP, drilling fluid was observed entering the pipe through the joints.

SITE EXPLORATION

In October 2000, a subsurface exploration program (consisting of six mud rotary test borings ranging in depth from 22 to 80 feet) was performed to evaluate the dam embankment and foundation soils. Representative soil samples were obtained using split-barrel and Osterberg and Shelby tube sampling techniques. Vane shear tests were conducted within selected borings to estimate the in-situ undrained shear strength. After completion of selected borings, inclinometer casings or vibrating wire piezometers were installed. All other borings were backfilled by tremying cement-bentonite into the borehole. The locations of the soil borings are shown on Figure 1.

Laboratory Testing Program

Routine and index testing was performed on the collected embankment and foundation soil samples. To determine the dispersive characteristics of the embankment soil, Double Hydrometer testing was conducted. Pinhole dispersion tests were also performed to determine the embankment soil dispersive or piping potential. Hydraulic conductivity tests were conducted using the rising tailwater method in a triaxial permeameter. A series of undrained and drained triaxial and direct shear tests were performed to determine the shear strength of the dam and foundation soils. One dimensional consolidation tests were conducted on representative soil samples to determine the compressibility of the embankment and foundation soils.

EXPLORATION RESULTS

A cross-section depicting the subsurface conditions with the spillway structure super-imposed on the section as shown on Figure 2.

Embankment Earth

Drake Lake Dam is a compacted earthen embankment. Fill for the embankment was obtained from two borrow areas, which are located along the left abutment, west and southwest of the embankment site. Borrow areas contained three distinct soil types immediately below surficial topsoil. The upper zone was plastic clay overlying a silty clay and silt materials. The design of the embankment required that the plastic silty clay fill material be placed within the lower 15 to 20 feet of the embankment to form an impermeable core. A lesser plastic silty clay material was to be placed above this material to the crest of the dam. In general, the US Department of Agriculture (USDA) – Natural Resources Conservation Service (NRCS) soil survey of the borrow area sites identifies the site soils as loess.

Upper Embankment Fill

Test boring results indicated that the upper dam zone, extending down to approximate elevations of +558.6 feet to +563.0 feet, consisted of silty clay (CL) and clayey silt [(ML) and (CL-ML)] material, with little to some amounts of fine sand. Unconfined compressive strengths, ranged from 1.5 to 4.5 tons per square foot (tsf), indicated that the consistency of the upper embankment fill ranged from stiff to hard.

Atterberg limits tests within this zone indicated the liquid limit ranged from 25% to 30% with a plastic limit of 21% to 23%, resulting in plastic indexes ranging from 3% to 9%. Grain size distributions indicated that the majority (99.6%) of this material passed the No. 200 sieve, and 11% to 29% passed the 0.002-millimeter clay fraction grain size. This index testing indicated that the upper embankment fill material has a small fraction of clay particle sizes and has a USCS classification of (ML), (CL-ML), or (CL).

Results of the triaxial test indicated that the effective stress angle of internal friction of the upper fill zone ranged from 36.3 degrees to 41.7 degrees, with a value of 36.0 degrees used in the stability analysis.

Lower Embankment Fill

The borings encountered a plastic clay fill material in the lower portions of the dam. This material extended down to elevations +543.5 and +546.6 feet. This material was primarily obtained from the upper portion of the borrow areas. Estimates of the unconfined compressive strength were obtained using calibrated penetrometers and ranged from 1.75 to 4.25 tsf, indicating a stiff to hard consistency.

Atterberg limits tests indicated that the liquid limit was 59% and the plastic limit was 23%, resulting in plastic index (PI) of 36%. Based on these results, the USCS classification for this layer is (CH). Hydrometer test results indicated that approximately 99% of the material passed the No. 200 sieve, and 36% passed the 0.002-millimeter clay grain size criteria.

Dam Foundation

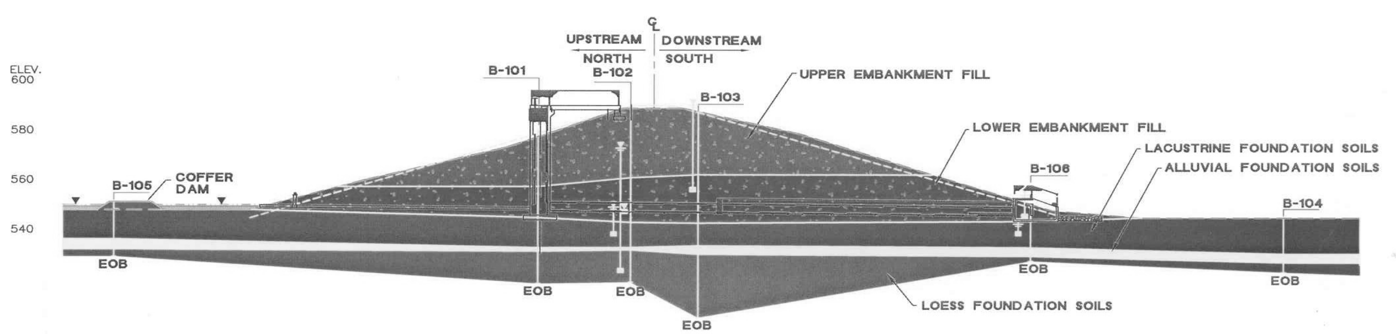

Fig. 2. Drake Lake - Cross Section

According to the NRCS soil survey of Cass County, Illinois, the foundation soil found along the base of the dam is classified as a friable silt loam. The soil is typically found in flood plains. During construction the surface topsoil was stripped from beneath the base of the foundation and a 5- by 5-foot key trench was excavated along the centerline of the dam. The contractor reportedly had difficulty maintaining stable cut slopes for this key trench excavation in this silty material.

Lacustrine and Alluvial Foundation Soils. Immediately beneath the embankment fill soils and at the surface within upstream and downstream free field borings, lacustrine soils, consisting of a dark gray silty clay (CL) with trace amounts of organics and trace to a little amount of gravel and fine sand, were encountered. Estimates of the unconfined compressive strength, using the calibrated penetrometer, ranged from 0.9 to 3.1 tsf, indicating a medium to very stiff consistency. Generally, the consistency was greater in samples obtained from beneath the centerline of the dam, and softer consistency soils were encountered in samples from the free field borings away from the dam. Measured water contents within the lacustrine soils below the embankment ranged from 23.0 to 28.8%, while water contents within the free field borings and at the downstream toe boring ranged from 15.0 to 45.0%. The difference in water contents indicates that a considerable amount of consolidation occurred beneath the dam. The organic content from selective samples obtained from this stratum ranged from 1.3 to 1.6%.

In-situ vane shear test results indicated that the undrained shear strength immediately below the embankment had increased to a range of 1,980 to 4,300 pounds per square foot (psf). A plot of the vane shear results is presented in Figure 3.

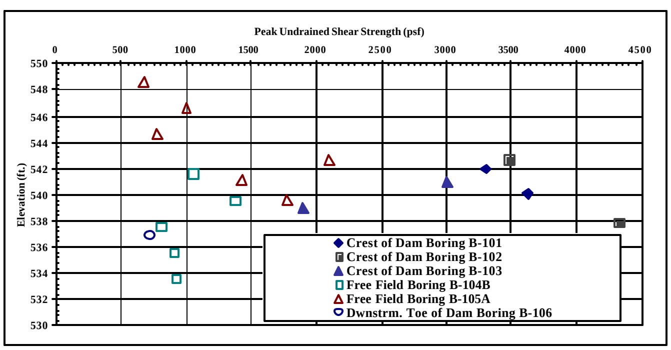

Fig. 3. Vane Shear Results

Undrained shear strengths from vane shear tests performed in the free field borings were in the range of 675 to 1,775 psf. Unconsolidated undrained (UU) triaxial tests indicated that the undrained shear strength ranged from 800 to 1,800 psf for samples from beneath the embankment and 400 to 600 psf in the free field borings. Direct shear tests indicated that the drained angle of internal friction of the lacustrine soil ranged from 33.6 to 37.2 degrees.

A layer of alluvial silty sand (SM) and silty gravel (GM) was found immediately beneath the lacustrine clay. This layer ranged in thickness from 1 to 2 feet. The SPT blow counts ranged from 4 to 11 bpf, indicating the relative density ranged from loose to medium dense.

Loess Foundation Soils. Underlying the lacustrine and alluvial deposits were consolidated loess soils, consisting of a light brown to dark gray non-plastic silt (ML) with trace amounts of fine sand. SPT blow counts ranged from 22 bpf to 115 blows per 10 inches, indicating a medium to extremely dense relative density.

Instrumentation

Inclinometer casings were installed within two borings to monitor lateral movements of the dam and foundation soils. The readings at the inclinometer casing installed on the upstream slope of the embankment) indicated some steady movement of up to 0.8 inch in the south (upstream) direction and 0.5 inch in the east direction. The inclinometer casing installed on the dam crest and downstream of the dam centerline had movement of 0.25 to 0.5 inch to the north (downstream) between the reading taken on October 26, 2000, and February 7, 2001. However, between December 27, 2000, and February 7, 2001, there appears to be about 0.5-inch movement. Results of the inclinometer readings are included in Figure 4.

In general, the readings were somewhat erratic. The readings appear to indicate some movement had occurred within and just beneath the dam embankment. If the dam embankment and foundation had moved the magnitude indicated by the inclinometer readings, there would be some concern that the embankment had not yet stabilized. However, the other instrumentation, such as the vibrating wire transducers and measurements of the outlet pipe joints, did not support these inclinometer observations. The apparent movement was attributed to instrumentation or system error since two different sets of inclinometer reading equipment was utilized.

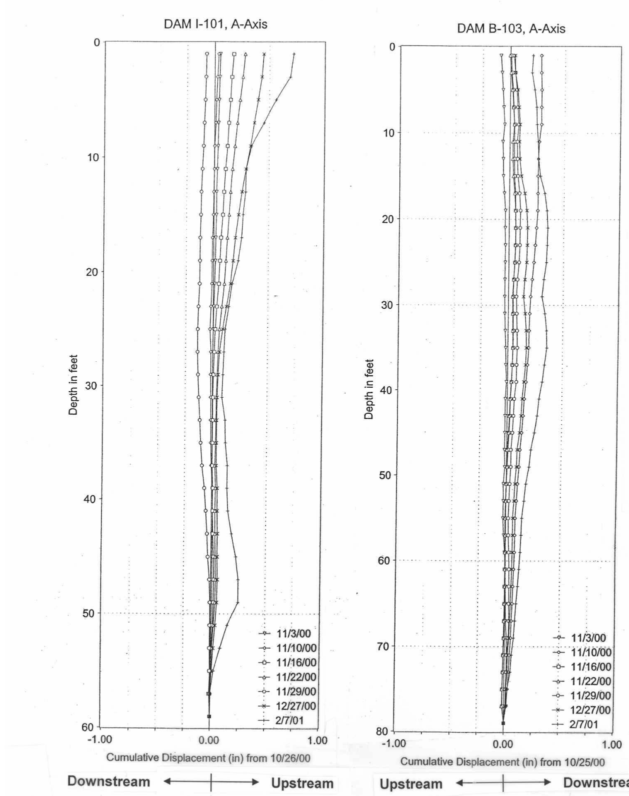

Fig. 4. Inclinometer Readings

A total of six vibrating wire pressure transducers were installed within three borings to measure changes in the pore water pressure within the dam and foundation soils. The locations of the transducers are shown on Figure 2. With the exception of several readings affected by trapped drill water, since the beginning of November 2000 readings have shown only slight fluctuations in the range of 0.2 to 0.8 foot.

The steady pore water pressure readings indicate that the construction-induced increase in pore water pressure had dissipated and the primary settlement of the lacustrine clay foundation soils is complete. Results of the vibrating wire piezometer monitoring program are shown on Figure 5.

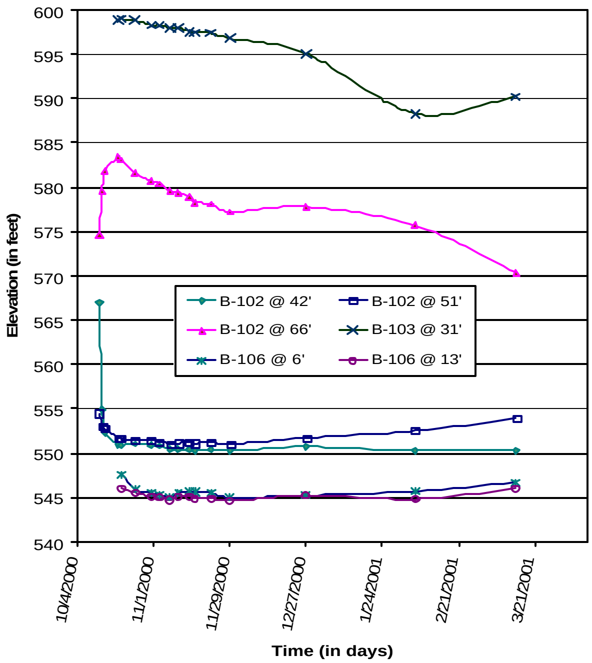

Fig. 5. Piezometer Readings

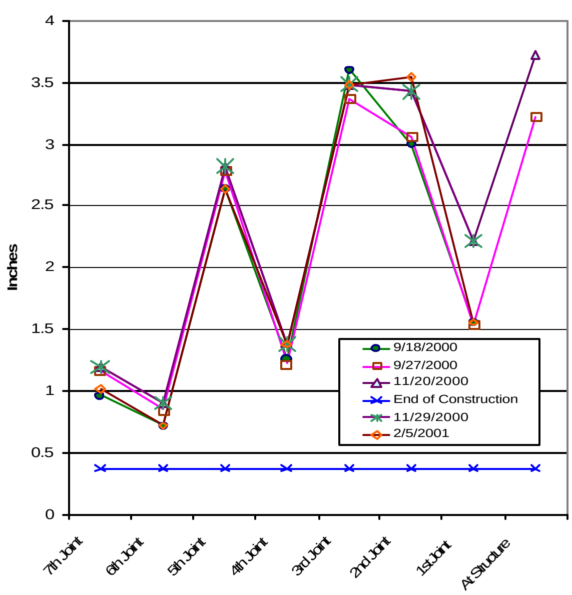

Fig. 6A. Average Joint Separation

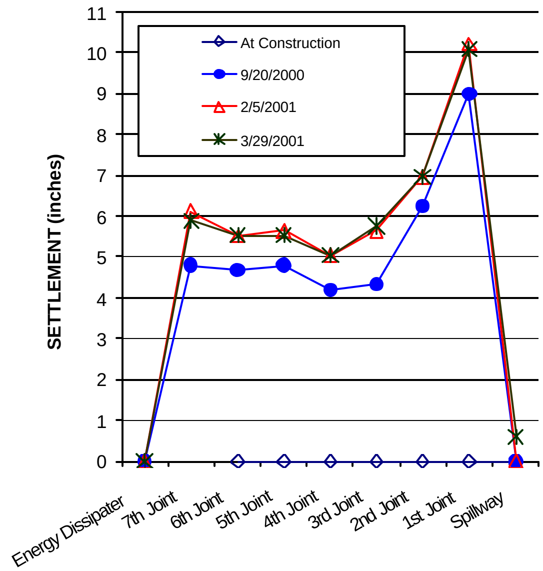

Fig. 6B. Settlement at the Joints

Measurements of the movement of the seven joints between the 20-foot RCP sections were also taken. The pipe joints were originally placed with minimal separation between the joints. The soils movement resulted in the separation of the joints. The joint separation readings indicated that the joint separation ranged from 0.72 to 3.54 inches. The greatest amount of movement occurred within Joint No. 2, located 40 feet downstream of the inlet structure.

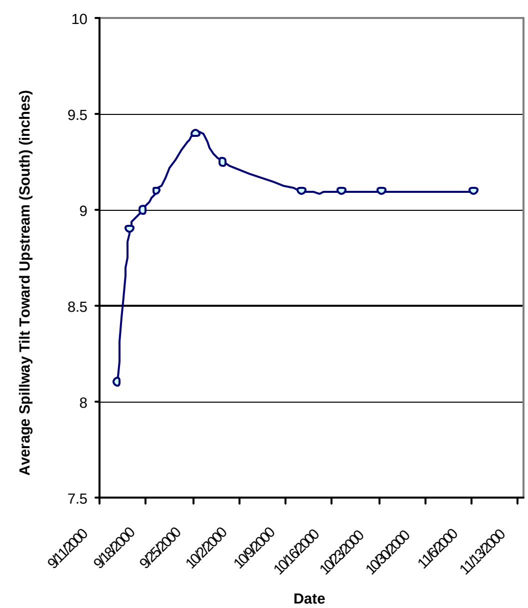

Fig. 6C. Average Spillway Tilt Toward Upstream (South)

Readings were taken of the joint separation along the outlet pipe, settlement measured at joints along the outlet pipe, and the amount of tilting that has occurred to the inlet structure. Results of these measurements are included on Figure 6A, 6B, and 6C. Readings were also obtained to record the amount of tilting of the inlet structure in the south (upstream) direction. Assuming that the inlet structure was constructed level, the measurements indicate that the spillway tilted approximately 9.06 to 9.12 inches toward the upstream (south).

ANALYSES

Seepage Analyses

Input parameters for the seepage analyses using a finite element model, were estimated based on results of gradation data, laboratory permeability testing, and engineering judgment. In estimating the permeability of the compacted (CH) embankment soils that had undergone cracking, the horizontal coefficient of permeability (K_h) was assumed to be 6.0×10^{-6} feet per minute (ft/min) and the assumed vertical coefficient of permeability (K_v) was 6.0×10^{-4} ft/min.

Four seepage conditions (cases) were analyzed to predict the steady-state phreatic surface. Note that the analyses were based on a steady-state seepage condition. The four cases assume various combinations of isotropic and anisotropic permeability conditions to evaluate the effect of cracking in the compacted (CH) embankment soils. An upstream impervious clay blanket was also modeled in our analyses as a method to control future seepage through the cracked embankment.

Based on results of the four seepage analyses, it was concluded that utilizing a clay blanket on the upstream embankment slope would be effective in lowering the seepage emergence point across the downstream slope. A downstream filter blanket drain was also recommended to safely intercept the phreatic surface and stabilize the downstream toe area. The predicted phreatic surfaces were subsequently utilized for the embankment dam stability analyses.

Stability Analyses

The stability of the dam was evaluated to determine if there had been stability failure within the foundation during construction. Stability analyses were also performed for the post-construction (existing condition), normal pool (elevation +585.0 feet), and high reservoir level (elevation +589.5 feet). The normal pool and high reservoir analyses utilized the predicted phreatic surface, considering the effect of the upstream clay blanket and downstream filter blanket drain. The “During Construction” analysis modeled a short-term condition with no reservoir pool assuming that the natural lacustrine clay (CL) strata located immediately beneath the dam embankment had not completely consolidated. Therefore, soil strengths from the free field borings were used to model the foundation soil. Undrained conditions were assumed for cohesive embankment and foundation soils.

Field evidence indicated that the (CH) embankment fill material had developed tension cracks and therefore, anisotropic shear strength parameters were assigned to this deposit. The anisotropic properties include a minimal strength 20 psf aligned in the near vertical direction (parallel to the cracks) and a horizontal undrained shear strength of 500 psf (perpendicular to or across the cracks).

Both short-term and long-term analyses were performed for the “After Construction” condition with no reservoir pool. The short-term analysis assumed that the cohesive soils were in an undrained state. Since tension cracks formed in the (CH) fill, we adapted an anisotropic strength of 20 psf in the near-vertical direction and 1,500 psf in the horizontal direction. We also assumed that the natural lacustrine (CL) soils below the dam had also undergone significant consolidation, which increased the undrained shear strength of the foundation soils near the center of the dam. The shear vane testing indicated that the strength of the foundation soils increased from 250 psf near the toe to 3,000 psf under the center of the dam.

The long-term analysis was performed assuming effective stress strength parameters (angle of internal friction with zero cohesion). Anisotropic friction angle of 10 degrees in the near vertical direction and 34 degrees, friction angle in the horizontal direction were utilized for the cracked (CH) fill.

Stability analysis for “Normal Pool” and “High Reservoir” conditions were performed using the same parameters utilized for the “After Construction” condition. These analyses adopted the phreatic surface determined by the seepage analyses including the upstream clay blanket and the downstream filter blanket and toe drain.

The stability analysis indicated that “During Construction” loading condition, a circular sliding surface had a factor of safety of 1.01, and a sliding block surface through the natural (CL) foundation materials had a factor of safety of 1.06, indicating that a foundation stability failure had likely occurred. As a minimum, these low factors of safety indicated high shear stress levels within the dam foundation, which would have led to excessive embankment and pipeline deformations. The analysis shown also indicated acceptable factors of safety, greater than 1.5, for all other cases, assuming the upstream clay blanket and downstream filter were constructed.

Embankment Foundation Consolidation Analyses

Consolidation analyses were performed to determine the amount of settlement, which might occur from constructing the dam embankment over the compressible natural lacustrine silty clay strata and to determine the time required for completing the settlement. The combination of fill induced shear stresses and consolidation of the saturated silty clay layer resulted in lateral spreading, vertical settlement, and cracking of the outlet structure, the outlet pipe, and the embankment.

Based upon consolidation test results, the over-consolidation ratios (OCR) of the foundation soils ranged from 0.51 under the center of the dam to 4.8 in the free field. An OCR of greater than 1 is an indication that the soil is over-consolidated, meaning that the soil has experienced a maximum stress sometime in the past, which is greater than the current overburden stress. The OCR of the samples from the toe of the dam and the free field boring indicated that the original state of the natural lacustrine silty clay layer was over-consolidated. This may have been the result of depressed groundwater table or desiccation stresses, which could have consolidated the clay material resulting in an estimated vertical pre-consolidation pressure ranging from 1.1 to 1.5 tsf. At the time that the sample was obtained from beneath the embankment, 46 feet of fill had been placed over the lacustrine silty clay layer. The OCR of less than 1.0 under the center of the dam indicates that the additional overburden stress surpassed the past maximum stress applied to this layer, resulting in the clay becoming normally consolidated.

Settlement of the dam was evaluated in two components. The first component was the settlement due to the primary consolidation of the natural silty clay layer. Based upon the geometry of the dam and the estimated thickness of the natural silty clay layer, the estimated primary settlement was 4.3 inches. The second component was the settlement due to the volumetric change from lateral squeezing of the soft natural silty clay layer caused by fill induced shear strains. Based upon readings obtained from the elongation of the pipe, lateral spreading was in the magnitude of 13 inches. Assuming a horizontal to vertical strain ratio of 0.12 (Report on Investigation of Deformations in Foundation of Earth Embankments Containing Concrete Pressure Pipe Conduits, by Moran, Proctor, Mueser, and Rutledge), referenced in "Computation of Joint Extensibility Requirements" USDA (1969), we estimated a deflection due to volumetric change of 4.7 inches. Therefore, the total estimated settlement or deflection from both components was 9 inches. Based upon survey readings in the outlet pipe, at least 10.2 inches (0.85 foot) of settlement had actually occurred. This relatively close agreement between the theoretical predicted settlement and the measured settlement, indicated that foundation settlement was essentially complete.

According to published sources (Ladd, 1991), the ratio of settlement to horizontal movement (dh/ds) should not exceed 0.4 to avoid cracking and lateral spreading of fills on compressible foundations. Based upon the observed and calculated settlement of 10.2 inches and a horizontal movement of 13 inches, the actual ds/dh ratio is approximately 0.78. Thus, cracking in the embankment would be anticipated.

Fill placement for the embankment occurred over a 60-day period, starting on July 19, 2000. Based on the results of the consolidation tests, the time to complete 50% of the primary settlement would be 45 days, and 90% of the primary settlement would be complete within 149 days or by December 2001. The exploration began 82 days after fill placement and hence, a major portion of the settlement of the embankment would have already taken place.

Dispersion Analyses

Index testing and visual examination indicate the embankment fill was potentially dispersive. The embankment soil was evaluated for the potential of being a dispersive soil. A dispersive soil erodes by the individual colloidal clay/silt particles going into suspension in practically still water. Dispersive soils may result in failure of a dam by either: 1) internal erosion or piping through the dam; or 2) severe erosion on the embankment slope resulting in deep gullies, which could weaken or breach the embankment.

To confirm that dispersive soils were used in the dam construction, both Pinhole and Double Hydrometer tests were performed. Three pinhole tests were conducted on samples from the upper layer of the dam embankment, the lower layer of the embankment, and from a borrow source sample, which will be used to construct the upstream seepage blanket. The results indicate the upper embankment layer had a dispersion classification of D-1, which is the "most dispersive" classification. A Double Hydrometer test was also conducted on a sample from the upper layer of the dam embankment. The percent dispersion for the sample was 68%, also indicating a dispersive soil. The sample from the lower embankment layer and the borrow sample had dispersion classifications of ND1-nondispersive. Based on comparison of index properties, the upper embankment fill was classified as dispersive. Since pinhole testing and soil type confirm that dispersive soils were present, we proceeded with designing an upstream clay blanket to reduce dam seepage and a downstream filter blanket to intercept seepage.

CAUSES OF DAM DISTRESS

Findings indicate the dam has been constructed of an upper layer of low plasticity clayey silt (ML), silty clay (CL), and a lower layer of high plasticity clay (CH) material. Both fill soils were likely of loess origin. Natural foundation materials consisted of a compressible, low shear strength, saturated silty clay of lacustrine origin. The embankment fill experienced several problems during construction. The fill was reportedly difficult to compact due to fill moisture contents above Standard Proctor optimum. Water contents in the fill ranged from 20.2 to 24.1%, which was 4 to 8% wet of the standard Proctor optimum moisture content of 15 to 16%. This resulted in reported "waving, spreading, rolling or shoving of the fill under the weight of construction traffic." This was evidenced during construction by shifting grade stakes. The downstream face of the dam had a noticeable downstream bulge or rolling surface that we believe was caused by these excess soil fill moisture content conditions.

Horizontal movement measured in the principal spillway intake tower and the 36-inch RCP was apparently caused by lateral spreading of the embankment dam in both a downstream and upstream direction due to compressible and lower strength lacustrine foundation soils. We believe that rapid filling over the compressible lacustrine soil layer created a short time period of low shear strength in the relatively soft foundation soil due to excess positive pore water pressure. As a result, the soft foundation soil was overloaded, developed high positive pore pressures and essentially experienced plastic yielding or horizontal shear failure. Since the low strength foundation soils were restrained above by the embankment and below by the dense underlying till, the low strength silt crept laterally due to sustained driving shear stresses greater than the resisting stresses. Since the embankment dam was resting on a compressible silt layer, the dam and embedded structures moved horizontally and settled vertically in response to foundation soil movement. We believe that the principal spillway and horizontal pipe movements were caused by lateral spreading and vertical consolidation of the compressible lacustrine silt. The shear strength of the softer foundation soils material was considerably lower than anticipated, and the soft foundation soils could not consolidate fast enough under fill loading rates to limit lateral movement. Loss of drilling fluid in our mud rotary cased borings indicates that there were vertical cracks parallel to the dam alignment across the center portions of the recently completed dam with no pool. The ratio of settlement to horizontal movement that was measured in this dam (e.g., dh/ds equaled 0.7) should have caused cracking in the embankment. The maximum recommended dh/ds ratio for stable fills on compressible foundations should not exceed 0.4 per Ladd (1991).

Field vane shear tests indicate that the foundation soils below the dam gained substantial strength (e.g., S_u equals 1,980 to 4,300 psf), due to consolidation under the weight of the dam. Peak vane strengths beyond the toe of the dam were considerably less (S_u equals from 675 to 1,775 psf). These measurements corroborate foundation soil strength gain due to consolidation had occurred under the dam and confirmed that primary consolidation was essentially complete.

REMEDIAL MEASURES

Site investigation indicated defects in the dam consisting of vertical cracks in the embankment, separated joints in the spillway pipe structure, and an out-of-plumb and cracked drop inlet structure. Various remedial measures were implemented to repair the structure.

Constructing Impervious Upstream Seepage Blanket

To reduce seepage through and around the dam, a compacted, 3-foot-thick impervious clay blanket was placed over the upstream dam slope and over the left and right abutments. The blanket created an additional seepage barrier across the upstream face of the dam and reduced the risk of excess seepage through the dam and upstream dam/abutment contact. In addition, the blanket effectively increased the seepage path through the dam and abutments, thereby decreasing exit gradients and reducing the potential for piping. This was a critical issue since there are known cracks along the centerline of the dam, which could move the phreatic surface downstream through dispersive fill soils. The clay blanket is illustrated on Figure 7.

Placing Downstream Filter Blanket

A downstream granular filter blanket was constructed over the toe of the dam and abutment contacts to intercept and control uncontrolled dam and foundation seepage. This was addressed due to concerns with observed embankment cracking and the piping potential of the embankment dam and abutment soils. Also, there were indications of groundwater springs along the west abutment. The drainage blanket extended onto the east and left (west) downstream abutments to intercept any "short-circuiting" of seepage water through the known saturated silt and sand seams in the abutments during reservoir filling. Situating the filter blanket over the downstream toe of the dam also increased the slope stability of the embankment. The filter blanket consisted of 12 inches of clean IDOT FA-3 type sand placed against the dam fill overlain by 18 inches of washed IDOT CA-2 type gravel. A sketch of the downstream filter blanket is also contained in Figure 7.

Lining the RCP Outlet Pipe

The 36-inch RCP had joint separations up to 3.54 inches wide. Also, some sections had cracks along the pipe crown. The pipe was considered unsuitable to convey water through the embankment. The open joints and cracks could allow water to enter the dam embankment, which could result in a piping failure of the dam. Also, if the pipe was subjected to high flows, the openings in the pipe may act as a siphon, which would suction embankment material into the outlet pipe, resulting in voids at the openings. Due to these factors, the pipe was lined. A synthetic felt material, impregnated with heat-activated resin and HDPE liner systems, were evaluated for this project. The 30-inch diameter HDPE liner was selected since it is more ductile and able to accommodate future lateral and horizontal strains anticipated during the life of the structure. The HDPE liner pipe selected was a 1,000 Series PE 3408, Driscopipe with a 50 psi pressure rating.

After installation, the annulus between the existing pipe and the HDPE was filled with a cementatious grout. The HDPE has a relatively high coefficient of thermal expansion, and therefore, it was important not to restrain the pipe at both ends. The HDPE liner was fixed at the upstream end with a flanged seal, while the downstream end was free to expand and contract.

Prior to installation of the liner system, the existing RCP joint cracks were grouted as an additional preventive measure against water seepage from the pipe into the surrounding subgrade. This grouting was performed from within the pipe at each joint using an expansive urethane foam grout. Figure 8 shows the HDPE liner system and the joint grouting detail.

Grouted Anti-Seepage Collars and Spillway Tower

Due to the outlet pipe movements, the original filter fabric and stone drain placed around the outlet pipe during construction was not considered reliable. There were concerns that large voids or channels may have formed along the pipe, which could lead to serious internal erosion. Therefore, grout collars, were constructed around the RCP pipe, to limit the seepage flow from passing along the pipe directly to the filter fabric covered stone drain. Also, any known voids along the pipe exterior were filled with grout.

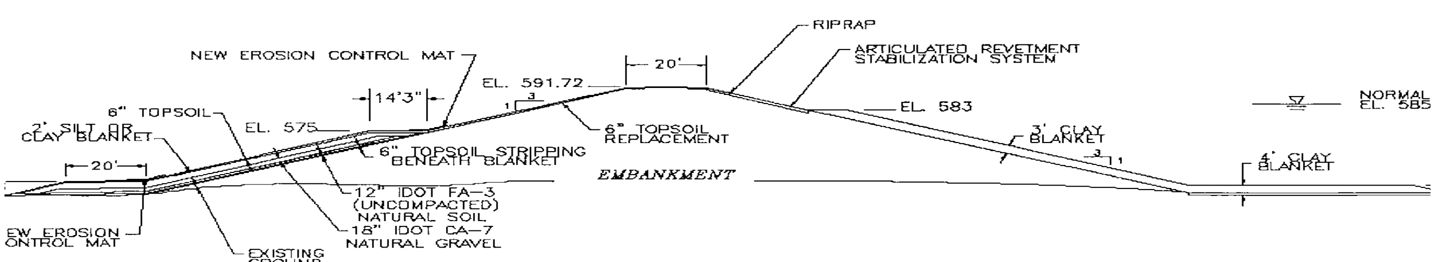

Fig. 7. Embankment Repairs

Two cement grouted "dough-nuts" or anti-seepage collars were installed along the pipe between Joints 1 and 2 and between Joints 2 and 3. These collars were created with a pressure-grouted zone extending 1 to 2 feet outside the pipe. Figure 8 shows the grout collar concept.

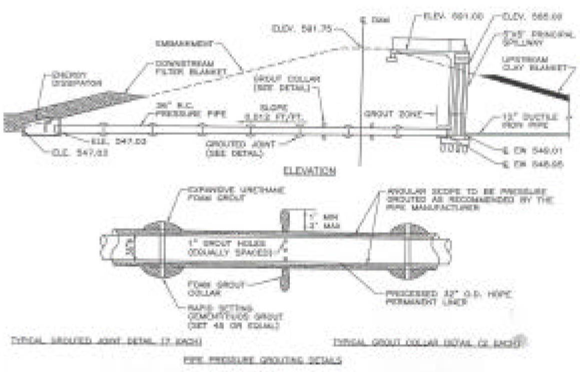

Fig. 8. Outlet Pipe Repairs NTS

During construction, horizontal joint cracks within the reinforced spillway structure were observed. The cracks formed when the base of the structure moved. The soils in the upper zone of the dam were determined to be dispersive and have a high potential for forming piping channels, which could result in failure of the dam. Therefore, cracks in the outlet structure were sealed using a non-shrinkable epoxy grout.

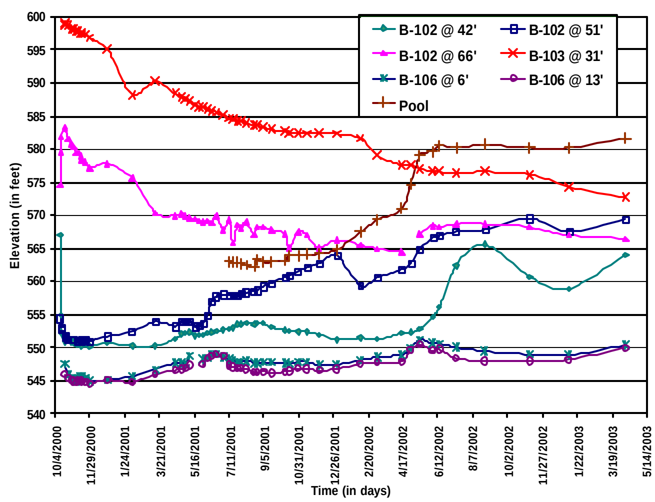

Fig. 9. Piezometer Results as of April 2003

RESULTS OF LONG-TERM MONITORING

As of May 2002, the lake level had risen to within 6 feet of the normal pool level. Instrumentation monitoring has continued to evaluate the dam performance as the reservoir fills. As indicated by the inclinometer and piezometer readings, the structure appears to be performing satisfactorily. Pore water pressure within the lower fill continues to dissipate as shown on Figure 9. There has been some continued settlement of the dam and its foundation as noted by inclinometer casing movement towards the centerline of the dam, shown on Figure 10. It is anticipated that some settlement of the crest of the dam will continue as the cracks within the embankment continue to heal by filling in with softened embankment soils. To date, the dam has performed as anticipated in the redesign of the structure. However, monitoring of the dam will need to continue.

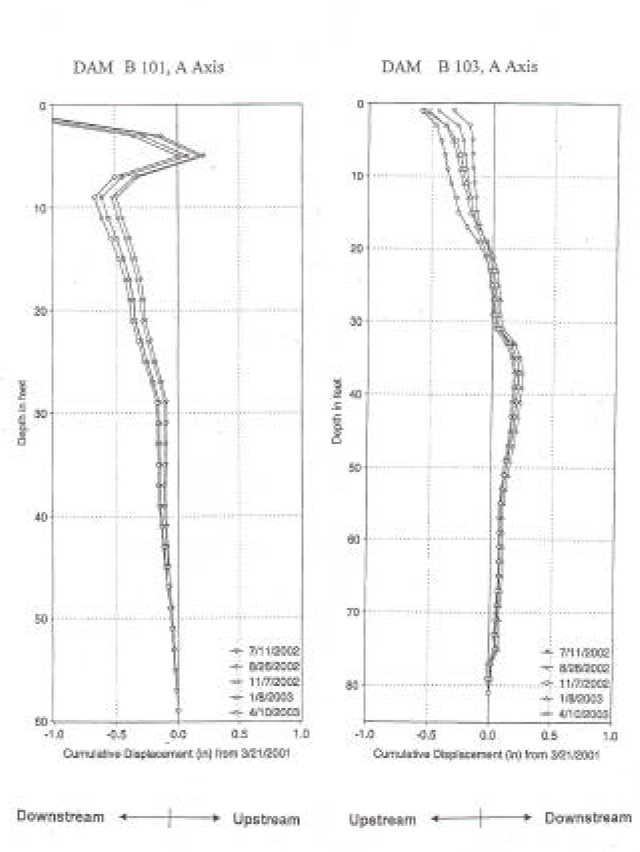

Fig. 10. Inclinometer Results as of April 2003

ACKNOWLEDGEMENTS

The authors acknowledge the State of Illinois for allowing us to prepare this paper. We would also like to thank Mr. Alan Grelck, the Resident Engineer for the Illinois Department of Natural Resources, for his help in the field during and after the explorations. His knowledge of dam construction events and details was invaluable. We would also like to thank Mr. Jerry Reince, Ms. Carol Moore, and Ms. Barb Vanderheiden for their help on this paper.

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