DAM SAFETY MONITORING REPORT
for
Hope Mills Dam

Town of Hope Mills
North Carolina

Prepared for:

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September 30, 2009

1.0 BACKGROUND INFORMATION

The Town of Hope Mills was granted conditional approval to impound by the North Carolina Dam Safety Section (Dam Safety) via regulatory correspondence dated July 15, 2009. The conditional approval was contingent upon implementation of the approved monitoring plan dated July 10, 2009 (prepared by others) to address Dam Safety's regulatory requirements for monitoring outlined in their May 29, 2009 correspondence. The monitoring plan is to be implemented through June 30, 2013 to maintain compliance with the conditional approval to impound.

Based on conversations with Dam Safety on September 18, 2009 regarding implementation of the approved monitoring plan, Dam Safety indicated that vibration monitoring would be required through June 2013 and that the alternative approach outlined in the approved monitoring plan (by others) would not be sufficient to meet the requirements for vibration monitoring. However, according to Dam Safety personnel, continuous vibration monitoring will not be required. Initial vibration monitoring will be conducted on a weekly basis during periods in which greater than three (3) inches of flow occurs over the labyrinths. Subsequent vibration and major storm event monitoring frequencies will be determined based on the results of the initial monitoring data.

Due to the recognized time constraints associated with meeting the September 30, 2009 reporting deadline, Dam Safety approved the following revised scope of work to be included in this initial report:

- Observe the labyrinth spillway, earthen embankment, and overall structure to evaluate existing conditions and performance of the dam and spillway;
- Collect and interpret data from existing instrumentation (i.e. pressure gauges, and piezometers);
- Provide "X,Y,Z" survey data from fifteen (15) locations on the spillway structure;
- Provide an implementation schedule for the required vibration monitoring program.

Mosher Engineering was verbally authorized to initiate implementation of the monitoring program on September 23, 2009 by the Town of Hope Mills. The required submittal date for this initial report is September 30, 2009.

2.0 FIELD INVESTIGATION

On September 24, 2009, Mr. Timothy L. LaBounty, P.E., with Mosher Engineering visited the site to observe existing conditions and to collect data pertinent to the implemented monitoring program required by Dam Safety. Monitoring data incorporated into this report prior to September 24, 2009 was collected by others and has been provided to supplement recently collected data for longer-term analysis and potential establishment of initial data trend(s). This report addresses the revised scope of monitoring authorized by Dam Safety for the initial report submittal due September 30, 2009. We plan to address the complete monitoring scope outlined by Dam Safety, including vibration monitoring, in subsequent reports. "Left" and "right" used in this report are referenced to the downstream direction consistent with typical water resources standard convention. Tables, drawings, photographs and appendices referenced in this report are included in the appropriate tabbed section.

Published antecedent precipitation for the 72-hour period prior to our site visit revealed total cumulative precipitation depths of 0.35 inches and 0.62 inches for the Fayetteville Airport and Fort Bragg Climate Retrieval and Observations Network of the Southeast (CRONOS) database reporting stations of the State Climate Office of North Carolina, respectively. Hope Mills Dam watershed is similar in terms of size and hydrologic condition to the watershed monitored by USGS Gage 02104220 located on Rockfish Creek at Raeford, NC. Based on review of the most recent 60 days of flow data reported, the maximum flow rate was about 191 cubic feet per second (cfs) recorded on September 26, 2009. The data also revealed low base flow conditions for extended periods during the 60-day reporting period. Meteorological and stream gage data is included in Appendix A.

2.1 Earthen Embankment

The existing earthen embankment is approximately 650 feet in length with a maximum height of about 34 feet. Lakeview Drive is located along the crest of the dam that connects Highway 59 (Main Street) to Legion Road and has been in service since 2005. The primary reconstructed portion of the earthen embankment was located in the vicinity of the previously breached section and was completed during the Lakeview Drive Bridge Repair project. Compacted select fill was placed in a controlled manner to fill the breach section. A toe and blanket drain system was installed in the downstream slope between the hydro sluice and the left bridge abutment retaining wall during the Lakeview Drive repairs. The toe drain system daylights on the downstream, riprap lined, slope adjacent to the left (east) spillway bridge abutment retaining/wing wall. The existing hydro sluice was plugged during the recent Hope Mills Dam Repair project. Grout was pressure-injected into the subsurface on both sides of the hydro sluice on the upstream side of Lakeview Drive during the Lakeview Drive Bridge Repair project. Pertinent portions of the upstream slopes were armored with NCDOT Class 2 Riprap and are secured with fencing to reduce the potential for unauthorized access.

A walkover was conducted along the upstream and downstream embankments. Evidence of significant movement or other potential embankment concerns including tension cracking, scarps, slides, depressions, erosion, sloughs, rutting, etc. were not observed on the downstream embankments. Evidence of uncontrolled seepage was not observed on the downstream earthen embankments. Excluding two approximately five (5) feet by five (5) feet localized eroded areas (Photos 1 and 2) and an undermined area beneath the left upstream bridge abutment slope concrete apron (Photos 3 and 4), evidence of significant movement or other potential embankment concerns including tension cracking, scarps, slides, depressions, sloughs, erosion and rutting, etc. were not observed on the upstream embankments. Grass cover located on portions of the dam was sparse in a few locations including the upstream right (west) abutment (Photo 5) and crest area located between the dedication monument and the spillway (Photo 6) and central portions of the downstream embankment (Photo 7). The riprap lining placed on the upstream slope and the downstream stream banks was in satisfactory condition. Evidence of movement or other potential embankment concerns associated with Lakeview Drive (i.e. the crest of the dam) were not observed and the pavement and guardrails were observed in satisfactory condition.

Nuisance vegetation was observed on the right (west) downstream bridge abutment that included small (less than 6 inch diameter) trees, weeds and bushes (Photo 8). Small bushes and weeds were observed on the left (east) downstream bridge abutment (Photo 9). Small trees were also observed on the downstream slope between the left dam abutment and the hydro sluice channel (Photo 10). Active flow at the hydro sluice inlet or in the hydro sluice channel was not observed but some minimal moisture and standing water (approximately (1) one inch in depth) was observed at various locations in the abandoned channel.

Nine (9) piezometers are located within the earthen embankment section as shown on Drawing 1. The piezometers were observed in satisfactory condition and excluding PZ-9, were readily accessible. PZ-9 is located within the riprap lined portion of the upstream right (west) spillway abutment slope. Access to this area is prevented by the existing chain link fence which does not include provisions for access (Photo 11). Depth to water level measurements were collected from each piezometer (Table 1) to evaluate phreatic conditions within the embankment. Based on review of the data collected, phreatic levels decreased from the upstream to downstream locations in central piezometer set PZ-3, PZ-2 and PZ-1 and west piezometer set PZ-9, PZ-8 and PZ-7. The east piezometer set PZ-4, PZ-5 and PZ-6 revealed an increasing trend in the phreatic surface in the downstream direction during the current monitoring period which was consistent with previous monitoring data collected on April 17, 2009 under similar lake operating conditions (i.e. lake level at approximately 103.5 feet msl). The monitoring data collected on August 18, 2009 reflected lower lake water surface levels at about elevation 97.5 feet msl, but also revealed similar trends for the data reported.

2.2 Spillway Structure

The spillway structure consists of a 4-cycle reinforced concrete labyrinth spillway and fish ladder that discharge to interior portions of the spillway and concrete discharge channel, respectively, prior to discharging to Little Rockfish Creek. The spillway structure is equipped with four (4) bottom drain gate valves that can be operated independently. Two nominal 36-inch diameter and two nominal 30-inch diameter gate valves with externally mounted riser stems are located on the east and west sides of the structure, respectively. The spillway also includes a sheet-pile cutoff wall (combination of vinyl and steel) beneath the labyrinth and fishway weir wall footings and a drainage system beneath the interior footprint of the structure to control uplift pressures on the structure. A safety/marker buoy system is also located in the lake immediately upstream of the spillway structure that was observed in satisfactory condition (Photo 12). The spillway is secured with controlled access fencing to reduce the potential for unauthorized access. Portions the concrete spillway crests, accessible to authorized personnel, are equipped with safety railing.

The interior walls and floor of the spillway structure revealed evidence of sealed cracks in the spillway. Based on preliminary conversations and information provided by others involved during construction, cracks developed and were subsequently sealed with a hydrophilic, liquid polyurethane pressure injected resin. Random crack width measurements collected from the walls (Photo 13 and 14) and floor (Photo 15) typically revealed widths of 0.1 to 0.2 mm and 0.2 to 0.4 mm in thickness, respectively. More frequent and extensive cracking was observed on the labyrinth floors which were topped with an unreinforced concrete/grout section (i.e. topping slab) on top of the underlying structural section for flow purposes (Photos 16, 17, 18 and 19). The majority of the sealed cracks throughout the spillway structure appeared to be performing satisfactorily. A few locations as described in the following paragraphs did reveal evidence of minor seepage. Some localized staining was also observed at the sealed crack locations.

Seven (7) locations on the eastern wall of Labyrinth 1 revealed evidence of minor seepage which included damp concrete and accumulated moisture (Photos 20 and 21). Two (2) wall locations and four (4) wall/floor joint locations on or adjacent to, respectively, the eastern wall of Labyrinth 2 revealed similar evidence of minor seepage (Photos 22 and 23). Minor seepage was also observed on the floor near the eastern wall of Labyrinth 2 in close proximity to the seepage locations on the eastern wall of Labyrinth 2. Two very minor seepage locations were observed on the western wall of Labyrinth 2 (Photo 24). One (1) minor seepage location was observed near the wall/floor intersection of the eastern spillway abutment wall (Photo 25). Discharge was not observed from the fish ladder internal drainage discharge pipe (Photo 26). Clear-water discharge was observed from the partially submerged slab under-drain outlets located in the eastern portion of the spillway down gradient of Labyrinths 1 and 2 (Photo 27).

One minor seepage location was observed on the spillway wall located downstream of the western bottom drain gates (Photo 28). Four (4) locations on the western wall of Labyrinth 4 revealed evidence of very minor seepage which included small localized areas of damp concrete (Photos 29 and 30). Two (2) wall locations on the eastern wall of Labyrinth 4 revealed evidence of minor seepage (Photo 31). Minor seepage was observed on the floor near the mouth of Labyrinth 4 (Photos 31 and 32). Two minor seepage locations were observed on the eastern wall of Labyrinth 3 (Photo 33). Three (3) minor localized seepage areas were observed in the floor of the western spillway area (Photos 34, 35, and 36). Discharge was not observed from the fish ladder internal drainage discharge pipe (Photo 37). Clear-water discharge was observed from the partially submerged slab under-drain outlets located in the western portion of the spillway down gradient of Labyrinths 3 and 4 (Photo 38). The largest flow was observed discharging from the drain outlet located closest to the exterior spillway abutment wall.

A few (i.e. 3 to 4) inches of flow was observed discharging over the entrance weir to the fish ladder during our site visit. The discharge was contained within the fish ladder structure with no discharges observed over other portions of the spillway structure (Photo 39). The exterior wall of the fish ladder did not reveal indications of seepage and the internal drain outlet pipes located in the fish ladder sidewalls were not discharging. Crack sealing performed on the exterior walls appeared to be in satisfactory condition.

The bottom drains were closed but were observed discharging flow. The bottom drains were reportedly closed some time during the week beginning September 14, 2009 following completion of recent tree removal activities from within the lake by others. Flow rates from each bottom drain gate valve were approximated by volume metering. The southern and northern 36-inch diameter gate valves located on the east side of the spillway were metered at approximately 30 gallons per minute (gpm) and 75-90 gpm, respectively. The southern and northern 30-inch diameter gate valves located on the west side of the spillway were metered at approximately 50 to 75 gpm and 10 gpm, respectively. Closer observation of each gate valve revealed the presence of organic debris (i.e. sticks, roots, wood, etc.) lodged between the gate and seal which prevented proper sealing of each gate valve (Photo 40). The operating mechanisms were observed in satisfactory condition and had been recently operated to close the gate valves.

Eight (8) pressure gages were installed during construction in the basal portions of the spillway to monitor uplift pressures beneath the slab (Drawing 1). The pressure gages were in satisfactory condition and were readily accessible. On September 23, 2009, pressure gage readings were collected from each gage. The data are summarized in Table 1.

Fifteen (15) "X,Y,Z" survey points were installed on the structure for monitoring purposes in accordance with Dam Safety requirements. In addition, we attempted to collect survey data that corresponded to the approximate locations of previous data collected during preparation of the as-built drawings in April 2009. Coordinate data points were obtained at various points on the spillway structure as indicated on the as-built survey dated April 29, 2009 prepared by McKim & Creed. The data was obtained using a Topcon total station instrument and prism pole held at the locations shown on Drawing 1.

On September 22, 2009, field survey crews obtained approximate elevation data because the as-built points were not previously marked on the structure when the as-built data was obtained in April 2009. The current locations used to collect elevation data represent approximate locations utilized for collection of the as-built elevation data. We estimate that the points surveyed on September 22, 2009 are within a few inches of the original as-built point data.

The recently surveyed approximate as-built location data is shown in bold text with the original data shown in screened text on Drawing 1. The recent data was obtained using a surveyor's level and rod, not the total station as was previously used in April 2009 for the as-built drawings. Due to the different methods used and the likely error in obtaining data at the same location, some error is possible.

To avoid similar issues in future monitoring events, field survey crews have placed nails in the wall at fifteen (15) locations as shown on Drawing 1. These points were placed using a drill bit to prepare a hole into which a nail was placed and then set using an epoxy grout. Most of these points have been set about six (6) inches below the top of the wall where there is less likelihood of damage and where they are visible from one of two control points located on the earthen dam. This will allow survey crews to obtain "X,Y, Z" coordinates of the points using a prism-less total station without having to physically access each point. Physically accessing each point presents personnel hazards due to the potential for dangerous (i.e. wet, icy) conditions and having to place a ladder to reach each point that is about fifteen (15) feet above the floor of the spillway structure. The coordinates of each "X,Y,Z" point surveyed on September 22, 2009 are included in Table 2 which will be updated to incorporate monitoring data collected during each reporting period.

2.3 Vibration Monitoring

Vibration monitoring will be conducted as required by Dam Safety targeting active labyrinth discharge events associated with major storm events (i.e. initially greater than three (3) inches of flow depth over the labyrinth crest) to evaluate the potential for possible vibration-induced effects resulting from activation of the spillway. An automated continuous flow depth device is planned to augment the vibration monitoring instrumentation. The flow depth device is intended to provide the necessary flow depth information to trigger preparation of required major storm event reports and collection of vibration monitoring data. Based on the interpretation of results of the initial monitoring reports, subject to regulatory approval, the trigger threshold (i.e. greater than three (3) inches of flow depth over the labyrinths) may be adjusted which could include increasing or decreasing the threshold.

We are currently in the process of expediting implementation of the vibration monitoring program. More specifically, we are investigating previous vibration monitoring efforts including instrumentation, interviewing previous personnel, obtaining and reviewing data as well as identifying and evaluating alternative systems. While we endeavor to expedite implementation of the vibration monitoring program and provide vibration monitoring data in the next report cycle consistent with regulatory requirements, we must emphasize that there are conditions and/or uncertainties which may impact the implementation schedule which include upcoming Holidays, weather conditions, reaction time and/or availability of other consultants/personnel, vendors, suppliers, equipment installation, regulatory review and concurrence and coordination with the Town of Hope Mills, etc. which are beyond our control. The following schedule represents our current anticipated timeframe for implementation which may be subject to adjustment based on actual progress obtaining required information, necessary approvals and coordinating with others.

October 31, 2009: Submit summary report of proposed vibration monitoring system to Dam Safety for regulatory review.

December 31, 2009: Installation of vibration monitoring instrumentation. Potential for initial baseline data collection.

2.4 Ancillary Structures

In addition to the spillway and dam, we also observed the general condition of ancillary structures including Lakeview Drive Bridge that included a twelve (12) inch diameter suspended ductile iron pipe (DIP) water line, pedestrian bridge, and 42-inch diameter sewer line located within or proximal to the dam and spillway structure. Our evaluation consisted of surficial observations which were limited in nature and were not intended to replace or represent routine inspections and/or evaluations by others including regulatory authorities, municipalities, utilities, etc.

Lakeview Drive Bridge is located approximately 70 feet downstream of the labyrinth spillway and fish ladder terminus and spans approximately 120 feet across the reinforced concrete discharge channel of the spillway (Photo 41 and 42). The bridge support structure consists of structural steel girders spanning between reinforced concrete end bents supported on steel H-piles. The bridge pavement surface is concrete and generally slopes toward the east (i.e. from NC 59 toward the left (east) dam abutment). Concrete parapet walls and bridge drainage outlets (approximately 6-inch diameter PVC) are also located on the bridge. Two (2) storm drain inlets are located in each bridge approach section on each side of Lakeview drive (total of four (4) storm drains) which are connected to fifteen (15)-inch diameter high density polyethylene (HDPE) stormwater pipes that are day-lighted to riprap lined portions of the downstream bridge abutment slopes (Photo 43). The bridge abutments include reinforced concrete retaining walls that retain soil backfill that slopes beneath the bridge structure and around the bridge end bents. The sloping soil backfill is protected with either NCDOT Class 2 riprap or concrete. An approximate five (5) feet long and thirty (30) inch deep undermined area (previously mentioned in the earthen embankment section) which extended about one (1) foot beneath the concrete apron was observed on the upstream east bridge abutment slope (previous Photos 3 and 4). Sedimentation was observed immediately downstream within riprap voids and on the concrete stormwater conveyance channel beneath the bridge which was similar to soils that remained in the undermined area. Adjacent to and extending approximately 100 feet downstream of the brink, the receiving stream and banks are lined with an approximate 3.0 feet thick layer of NCDOT Class 2 riprap which was observed in satisfactory condition. The bridge structure also supports a twelve (12)-inch diameter DIP water line suspended by pipe hangers on the upstream side of the bridge which was observed in satisfactory condition (Photo 44). The water line penetrates both reinforced concrete end bents of Lakeview Drive bridge to provide water supply across the spillway structure. Excluding the undermined concrete apron area, based on our limited surficial observations, Lakeview Drive Bridge appeared to be in satisfactory condition.

The newly installed prefabricated steel pedestrian bridge is located just upstream of Lakeview Drive bridge approximately 30 feet downstream of the labyrinth spillway and fish ladder terminus (Photo 45). The pedestrian bridge is supported by the reinforced concrete spillway abutment walls and is attached to the top of the walls with steel bolted connections anchored in the concrete (Photo 46). The walkway platform/surface on the bridge is timber (Photo 47). Access to the bridge structure is by concrete sidewalk/approach sections located on each abutment. Based on our limited surficial observations, the pedestrian bridge appeared to be in satisfactory condition.

The forty-two (42)-inch diameter sewer line is located approximately 100 feet downstream of the spillway brink (Photo 48). The pipe is oriented curvilinear in plan view and supported on steel H-pile and reinforced concrete pier supports above Little Rockfish Creek (Photo 49). The sewer line was in service prior to and remained in service during the Lakeview Drive Bridge Repair and Hope Mills Dam Repair projects. Reinforcement of the support structure(s) were implemented by Public Works Commission (PWC) of Fayetteville prior to the completion of the Hope Mills Dam Repair project. Riprap lining was observed in and adjacent to the stream channel around the pile and concrete foundations. Based on our limited surficial observations, the 42-inch sewer line appeared to be in satisfactory condition.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our field activities and data collected from Hope Mills Dam we have concluded the following:

- Excluding the following minor localized areas in need of maintenance, the embankment was observed to be in satisfactory condition. The following localized areas will need to be addressed:
    - Two, approximately 5 feet by 5 feet, localized depressions/eroded areas located on the upstream slope (Drawing 2) adjacent to the security fence will need to be reworked consistent with the requirements of the approved plans and specifications.
    - The localized undermined area beneath the Lakeview Drive Bridge left (east), upstream abutment, concrete apron will need to be repaired. We recommend

that the undermined area beneath the apron be properly formed and filled with flowable fill (minimum compressive strength of 1000 psi) and that the adjacent five (5) feet wide strip of riprap voids be grouted. In addition, we recommend that the street curbing be elevated to match the elevation of the adjacent bridge curbing and extended behind the inlet to enhance stormwater flow into the existing drain.

- o The denuded/sparse ground cover areas of the earthen embankment should be reseeded with a cool season grass cover (i.e. rye) as outlined in the operation and maintenance plan.

- Six-inch diameter bust height (DBH) and smaller trees and nuisance vegetation (i.e. undergrowth weeds, bushes, brush, etc.) shown in the areas indicated on Drawing 2 shall be flush cut and removed from the embankment and at least 15 feet beyond the downstream toe. We recommend that the Town of Hope Mills schedule these activities concurrent with future vegetation/clearing activities to expedite removal and potentially reduce costs. Based on the current availability of Town resources, a phased approach over the next two years could be implemented due to the current maturation level of the trees and nuisance vegetation. Please note that the more time the trees and vegetation are allowed to mature the more effort will be required to complete the scope of work. Also, later Fall and Winter seasons are typically more preferable times to conduct these activities when compared to the Spring/Summer due to decreased amounts of foliage as well as the reduced presence of insects, pests and public accessing Hope Mills Lake.

- Based on our observations, the spillway structure was observed in satisfactory condition. The crack sealing operations appear to have been successful in stopping the majority of the leaks previously observed in the spillway structure. However, a few minor seepage areas were observed in previously sealed areas that will need to be addressed. We recommend that the areas documented in this report as well as areas that may develop following issuance of this report be coordinated with the contractor that performed the sealing operations relative to satisfying performance requirements of the initial crack sealing work. Subsequent monitoring events will include continued observation of the sealed crack locations that will be used to establish longer-term trends relative to the performance of the sealed cracks. Subsequent monitoring reports will also address, as is applicable, activities by the contractor regarding necessary future crack sealing activities. Excluding the aesthetic nuisance condition created by the minor seepage observed in a few locations, we have no evidence to support structural-related concerns with the spillway at this time. Excluding the minor seepage areas and sealing issues with the bottom drain gates, the spillway structure including the fish ladder and discharge channel was observed to be in satisfactory condition.

- The piezometer monitoring data collected from the central (PZ-3, PZ-2 and PZ-1) and west (PZ-9, PZ-8 and PZ-7) abutment piezometer sets, revealed phreatic (line of saturation) conditions consistent with typical seepage conditions through an earthen embankment. Specifically, the phreatic surface decreased in elevation from upstream to downstream (Drawings 2 and 3). Localized mounding or adverse phreatic surface slope conditions were not reflected in the data collected at these locations. Based on our interpretation of the data, saturation conditions within the earthen embankment at these locations appear consistent with expected trends associated with typical seepage conditions through an earthen embankment. However, current phreatic levels may not represent stabilized seepage conditions through the embankment. Subsequent monitoring events will include supplemental piezometric data that will be used to establish longer-term trends of the phreatic surface through the embankment.

- The piezometer monitoring data collected from the eastern piezometer set, PZ-4, PZ-5 and PZ-6 revealed slightly adverse phreatic surface slope conditions through the embankment in the downstream direction (Drawings 2 and 3). Comparison of the current phreatic levels from piezometers PZ-3/PZ-4 and PZ-5/PZ-2 revealed a 9.4 foot and 12.8 foot difference, respectively, in upstream elevations of the phreatic surface between monitoring locations separated by a horizontal distance of only about 80 feet. East piezometer monitoring set PZ-4, PZ-5 and PZ-6 is located proximal to a portion of the embankment that was not reconstructed with the select fill as was used to fill the breach section. Based on review of boring log information prepared by others, subsurface soils near PZ-4, PZ-5 and PZ-6 consist of very firm to firm silty to slightly silty fine sand fill to approximately elevation 97 feet msl at which depth residual materials were reported which consisted of hard to very stiff silty plastic clays. In addition, subsurface soils in close proximity to PZ-4 and PZ-5 were pressure-injected with micro-fine cement grout to about elevation 94 feet msl during the Lakeview Drive Bridge Repair project.

As mentioned previously, the majority of select fill placed in the embankment was located in the breached section that was repaired during the Lakeview Drive Bridge Repair project. Select fill placed in the breach section extended to approximately elevations 76 to 79 feet msl. A toe and blanket drain system was also installed in the downstream embankment during the Lakeview Drive Bridge Repair project which extended from approximately the downstream bridge abutment wall to the hydro sluice channel and was located above elevation 97 feet msl in the portion of the embankment near the east piezometer set PZ-4, PZ-5 and PZ-6. While both the pressure injected grout and drainage systems could affect the location of the phreatic surface, the water surface elevations (91.6 to 93.9 feet msl) reported in the east piezometer set PZ-4, PZ-5 and PZ-6 were below both features.

One potential explanation for the lower phreatic conditions observed between the eastern and central piezometer sets is comparison of the select fill and the clayey residual soil permeabilities. Clayey soils would be expected to have lower permeabilities than the select fill material used during Lakeview Drive. The water levels in the eastern piezometer set PZ-4, PZ-5 and PZ-6 were located in residual (clayey) materials based on the soil profiles provided in the boring logs. Because the select fill extends to deeper depths (elevation 76 to 79 feet msl) as you approach the spillway structure from the direction of the hydro-sluice it is possible that this area behaves similar to a drainage system relative to the residual soils.

Because the current piezometer data may not represent stabilized seepage conditions, we will continue to monitor conditions in accordance with the requirements of the monitoring program and may alter our conclusions and/or monitoring strategy to reflect potential concerns which may develop based on future data. Based on our observations and evaluation of the piezometric data to date, we have no evidence to indicate that there is an immediate concern at this time with respect to the data observed at this location.

- Access to PZ-9 will be required. Our recommendation is to replace the end section of the existing black chain link fence with a secured gate.

- Eight (8) pressure gages are located in the spillway basin area that were installed to monitor uplift pressures on the spillway. Four (4) pressure gages are located in the eastern half of the spillway and the remaining four (4) pressure gages are located in the western half of the spillway (Drawing 1). Because some pressure gage data was not reported in previous monitoring events, the current pressure gage data reported essentially represents a baseline data set for future trending of uplift pressures applied to the base of the structure. The reported pressure readings summarized in Table 1 ranged from 0.0 psi in PG-7 to 0.8 psi in PG-4, both located in eastern portions of the spillway. The maximum reported pressure of 0.8 psi in PG-4, located near the mouth of Labyrinth 1, corresponds to approximately 1.85 feet of hydraulic head which is below estimated uplift pressures utilized in the design that ranged from 5 to 7 feet of hydraulic head in the area of the gage locations. The available hydraulic heads upstream of the labyrinth structures and at the counter-forted spillway abutment walls during typical base flow operating conditions (i.e. no labyrinth flow) are approximately 20 and 28 feet, respectively, at the bearing elevation of the footings. Based on the pressure data we collected, the seepage cutoff and drainage measures installed to control seepage (uplift) pressures appear to be performing satisfactorily. Future pressure gage data will be compared to the current baseline data set for appropriate trending and subsequent evaluation of the seepage control measures.

- Fifteen (15) survey points were established and corresponding "X,Y,Z" data were obtained at each location to establish a baseline data set for future trending of elevation and location data to monitor the potential for movement of the spillway structure (Drawing 1). Because the newly installed points were located in areas that were not previously surveyed, data analysis is not feasible in the current report. However, to provide an approximate evaluation of potential movement since April 2009 in the current reporting period, survey data (elevation only) from twelve (12) approximate previously surveyed as-built locations were obtained. This approximate method will be replaced by the 15-point "X,Y,Z" data points in future reports. The maximum variance reported from the twelve (12) approximate data points was a decrease of 0.04 feet (0.48 inches) located on the top of the intersection of the western spillway and abutment walls. The maximum variance reported for the labyrinths was a uniform decrease of 0.03 feet (about 0.36 inches) at three locations along Labyrinth 1. Elevation decreases on the eastern labyrinths (Labyrinths 1 and 2) trended consistently from a maximum decrease in Labyrinth 1 of 0.03 feet (0.36 inches) to 0.02 feet (0.24 inches) decrease along Labyrinth 2 and the fish ladder parapet wall located on the eastern portion of the spillway to virtually no reported change in Labyrinths 3 and 4 on the western portion of the spillway. A 0.01 feet (0.12 inch) increase was reported at the apex of Labyrinth 3 but is considered to be within the margin of error of the approximate method used to obtain data from the twelve (12) locations. Based on comparison of the approximate survey data (elevation only), conditions consistent with minor settlement may be indicated in the walls of Labyrinths 1 and 2, the eastern portion of the fish ladder entrance, and at the intersection of the western spillway and abutment walls. We expect that some of the variance noted in this comparison (i.e. 0.01 feet or 0.12 inches) could be attributed to the approximate methods used and minor structural variances. However, this would not likely account for the total amount of decrease reported. We did observe that the majority of the minor wall seepage locations were located in Labyrinth 1 which revealed the most relative movement. Relatively fewer and generally smaller seepage locations were observed in the western labyrinths which revealed negligible to no movement. While we do not consider the amount of movement a significant concern at this time, we will continue to monitor this situation and may revise the survey monitoring program based on future data.

- The bottom drain gate valves were observed discharging flow greater than the typical allowable rate of 0.1 gpm per foot of gate valve perimeter (About 8 gpm and 10 gpm for 30- and 36-inch diameter gates, respectively) due to lodged debris preventing proper closure of the valve. Debris migration toward the gates would be expected to occur during prolonged operation of the gate valves. Tree removal activities and previously existing debris in the lake during the initial filling phase of the reservoir and naturally occurring debris deposition from the watershed that was trapped in the lake during periods of lower lake operating levels have also likely contributed to elevated concentrations of debris in the lake. We recommend, to the extent it is feasible to do so without damaging the gates and/or structure, that attempt(s) be made to carefully dislodge the debris from the inside of the spillway structure. We also recommend that gate operation be limited to once per year during the 4-year conditional operation period consistent with the approved operation and maintenance plan to provide time for natural abatement of accumulated debris within the lake. We anticipate that debris levels in the lake will attenuate over time and allow for more normal operation of the gate valves with respect to proper sealing. Should excessive debris issues continue to present similar operational conditions long term, alternative measures for reducing and/or controlling debris accumulation at the gates may be considered which could include structural measures to reduce debris impacts to the gates.

- The vibration monitoring program is currently being developed to satisfy regulatory requirements outlined in the conditional approval to impound issued by Dam Safety. We plan to have our proposed vibration monitoring strategy submitted to Dam Safety by October 31, 2009. Following approval of the plan by Dam Safety and the Town of Hope Mills, we plan to have the vibration monitoring implemented prior to December 31, 2009.

- Based on our limited surficial observations, and excluding the undermined concrete apron and Lakeview Drive stormwater drainage issue previously addressed in other sections of this report, the ancillary structures including Lakeview Drive Bridge and the suspended 12-inch water line, the 42-inch diameter sewer line and pedestrian bridge were observed to be in satisfactory condition.

- We recommend that this report be submitted to Dam Safety on or before 5:00 pm, September 30, 2009.