PHASE 2 FINAL REPORT

REVISION 1

for
Kingdom Dam
on Lincoln Pond in the
Town of Elizabethtown, NY

VOLUME 1

Submitted to

**NEW YORK STATE
OFFICE of
GENERAL SERVICES**

Mr. Howard Hasenbein

Submitted by

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CORRESPONDENCE
Project # 15470

Mr. Howard Hasenbein
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State of New York
Office of General Services
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Re: Kingdom Dam Phase 2 Final Report
Located on Lincoln Pond, Wadhams, New York
OGS Project No.: S2153
CHA Project No.: 15470

Dear Mr. Hasenbein,

Clough Harbour & Associates LLP (CHA) is pleased to present this report documenting the current condition of the Kingdom Dam structure located on Lincoln Pond. This report also includes recommendations for correcting deficiencies in the dam structure.

BACKGROUND

Kingdom Dam is located on Lincoln Pond in Wadhams, NY. On June 18, 2004 representatives of NYS Department of Environmental Conservation, NYS Office of General Services, and CHA visited the site of the Kingdom Dam for the purposes of performing a reconnaissance of the general condition of the dam.

CHA was asked by OGS and DEC to provide a Phase 1 report of the general condition of the dam structure including structural, geotechnical and hydrological evaluations. CHA's Phase 1 report dated September 27, 2004 included the general condition of the structure and preliminary cost estimates for the rehabilitation and upgrades to the dam.

Based on the Phase 1 report recommendations and the condition of the structure, OGS and DEC requested a more in depth Phase 2 analysis and report. This report includes the findings from the site investigation, our analysis of the structure and recommendations for providing a dam structure that will be adequate structurally and meet the hydraulic requirements.

PURPOSE

This engineering report was commissioned to determine the condition of the existing dam structure and recommend repair options. The condition assessment of the structure evaluates the structural stability and spillway capacity of the dam. Downstream hazards in the event of a dam breech are also addressed.

DESCRIPTION OF STRUCTURE

The Kingdom Dam on Lincoln Pond is a buttressed concrete gravity dam built in 1911. The dam is approximately 255 feet long and 24 feet high. The main section of the dam is approximately 178 feet long. The remainder of the structure consists of buried end dykes. The dam has received patching and overlay type repairs to the exposed concrete surfaces at unknown times in the past.

There is a 45 foot long fixed crest spillway which works in conjunction with an outlet structure for control of the pond level. The outlet structure consists of a relief valve 36 inch diameter outlet pipe and log stop. The interior dimension of the outlet chamber is 6 feet wide by 26.5 feet long. The top of the outlet structure is covered with open steel grating. The outlet structure is accessed by a steel ladder fixed to the concrete wall of the chamber. Beneath the relief valve there is an abandoned 2 foot diameter penstock. The upstream end of the penstock appears to be blocked by a metal gate valve.

A two span steel foot bridge spans the top of the spillway for access. Each span of the foot bridge is approximately 23 feet long. There is a concrete pier supporting the bridge which is centered on the spillway. At each end of the dam there is a gate to restrict access to the top of the dam and spillway. The gate consits of a steel frame in-filled with timber planks.

SCOPE OF INVESTIGATION

Hydraulic Dam Hazard Assessment Scope

Refer to volume 2 of this report for the hydraulic and hydrologic data.

1. Hydrologic and Hydraulic Dam Evaluation

   a. CHA developed a hydrologic & hydraulic model and evaluated various storm events including the 2-year, 10-year, 50-year, 100-year and 150% of the 100-year (spillway design flood based on the current hazard coding). The purpose of this task was to determine the discharge capacity of the existing spillway. CHA utilized Hastead Method PondPack Version 10.0 software.
   b. CHA prepared a summary of the performance of the dam spillway (service and emergency) for each identified storm event.
   c. CHA recommended replacement/rehabilitation options (if required) for the service and emergency spillways, in order to achieve current DEC design standards.
   d. CHA identified areas of potential impact along the Black River Corridor downstream of Kingdom Dam. In particular, our engineers identified and documented any residential structures which may fall within the potential inundation limits and which could lead to a reclassification of the dam.
   e. Once a structure has been identified as at risk, our engineer worked with the survey crew to determine its elevation relative to the streambed.
   f. CHA developed an estimate for the peak flow associated with a sunny-day dam failure. This estimate was then used in conjunction with the survey data taken at the critical areas described above to determine an approximate water surface elevation.

g. Based on the results of the preliminary determination, CHA provided recommendations to whether the current hazard classification is adequate, or that a more detailed analysis is warranted.

Structural Scope

1. Structural Dam Evaluation
   a. Field measurements of the dam structure were obtained during our site visit.
   b. CHA performed a visual inspection, mapping areas in need of rehabilitation on all above water portions of the dam.
   c. CHA performed a diving inspection of all underwater portions of the dam including the upstream face and relief valve inlet and inlet chamber. Divers concentrated on the identification of substantial areas of deteriorated concrete.
   d. CHA performed a dam stability analysis utilizing the results of the hydraulic analyses and the proposed dam configuration.
   e. Eight concrete core samples were taken to determine the compressive strength, depth of reinforcing and condition of the existing concrete.
   f. CHA developed preliminary retrofits for anchoring the dam against overturning and sliding.
   g. CHA detailed the repairs required and the associated costs. Work was itemized into an estimate of the life expectancy of the repairs.

Geotechnical Scope

1. Geotechnical Evaluation of the Dam Structure

   a. CHA determined the bedrock quality beneath the dam.
   b. A review was preformed to determine the seepage conditions across the dam.
   c. CHA evaluated the seepage through the dam abutments.
   d. Sediment depth and characteristics on the upstream side of the dam were obtained.
   e. The earthen embankment and concrete dike construction impounding Lincoln Pond on the northeast side were reviewed.

FINDINGS

Site Visit

Between the weeks of April 10 and June 12, 2006 CHA engineers, surveyors, geotechnical drillers and structural drivers performed site visits. During these site visits they obtained soil and bedrock data, completed a topographic survey, inspected the dam structure and the structures located downstream of the dam which would be affected by a breech.

Field Observations

Structural Observations

Above Water Inspection

a. <u>Top of Dam and Downstream Face (Refer to Appendix C, Photos C-1 through C-10)</u>

On April 21, 2006 representatives of CHA performed an above water inspection of the Kingdom Dam for the purposes of assessing the general condition of the exposed portions of the dam and the outlet structure.

At the time of the inspection, there was no flow over the spillway, which allowed concrete sounding and measurements to be taken. This condition also allowed a visual identification of areas of leakage. The current conditions of the dam elements are as follows:

Downstream Face
The downstream face of the dam, including the buttresses and training walls, showed evidence of past rehabilitation with a thin parge coating system that was applied to approximately 70% of the surface. Much of the parge coating is hollow sounding and exhibits cracking, efflorescence and moderate to severe spalling. The cracks are up to 1/8th inch wide and the spalling is up to 4 inches deep in isolated areas. The concrete that remains beneath the spalled areas is soft and one inch of its surface is easily removed.

Top of Dam
Along the top of the dam there are several areas of spalled concrete and light cracking. Additionally, there are isolated areas of hollow sounding concrete. Most of the hollow sounding concrete is located on top of the spillway. The parge coating system at this location appears to be in good condition but the hollowness indicates that it did not bond to the underlying concrete.

Foot Bridge
One of the eight anchor bolts on each of the foot bridge spans are ineffective due to spalled concrete. The foot bridge exhibits widespread paint failure but has no significant section loss. In addition, a few broken welds on the railings were noted during our investigation.

Access Gates
The gates which restrict access to the top of the dam are comprised of steel frames and timber planks. The steel is in good condition, though it exhibits paint failure and light rust. The timbers are aged with isolated areas of deterioration.

Buttresses
Buttresses 1 and 2 exhibit cracking and efflorescence on their surfaces and will require concrete patching. Buttress 3 is in worse condition. There are spalls up to 8 inches deep on both sides of the buttress near its base. The concrete at the base of the spalls is soft. The remaining concrete surface appears to be an overlay that is heavily cracked. This buttress will need to be entirely replaced.

There is active leakage at the base of Buttress 1 and at the junction of Buttress 3 and the downstream face of the dam. Additionally, there was a damp spot above Buttress 3 indicating a minor leak.

Expansion Joints
All joints appear to be water tight with the exception of the joint located at Sta. 0+31 which exhibits a small amount of seepage. (See Appendix D for Plan of Dam)

b. Outlet Structure and Gate Valve (Refer to Appendix C, Photos C-11 through C-13)

The concrete on the outside of the outlet structure has areas of cracking with efflorescence but sounds solid when struck with a hammer. Both the north and south interior wall surfaces have an 1/8 inch wide vertical crack with horizontal extension. The area below the crack along the north wall is damp indicating leakage. The walls also have areas of hollowness with light spalling and scaling in the thin concrete overlay.

The bottom eight feet of the access ladder and log stop support angles is severely corroded. The open steel grating exhibits paint failure and section loss. The grating deflects noticeably when walked on.

The exposed portion of the trash rack appeared to be in good condition but extensive deterioration below the water surface was discovered during the diving inspection. The gate valve operating mechanism also appeared to be in good condition and was functioning at the time of the inspection. However, when the valve was fully closed, a small amount of leakage was evident. Some of the leakage is attributed to debris lodged behind the valve seal.

Diving Inspection

On May 5, 2006 representatives of CHA and Seaway Diving & Salvage Co. Inc. performed an underwater inspection of the Kingdom Dam for the purposes of assessing the general condition of the upstream face of the dam and the outlet gate valve.

The condition of the entire structure below the water line was observed by the diver and communicated via a continuous audio and video link between the engineer and the diver. The diver performed a thorough hands-on inspection of the dam. However due to murky water and debris it is difficult to discern detail in the video record of the inspection. Refer to the Elevation of the Front Face sketch in Appendix D for a record of the conditions found.

a. Upstream Face of the Dam

Widespread hairline cracking and several locations of light to moderate spalling were revealed in the concrete during the diving inspection. Most of the spalling occurs at a horizontal cold joint which are vary in height between 2 feet and 10 feet below the top of the dam. This condition occurs at several locations along the dam and with several spalls having exposed rebar. At one location, large cobble sized aggregate was exposed.

Also there are several random cracks in the dam face that are generally 1/8" wide. At various locations, spalling up to four inches wide has developed along these cracks. The entire concrete surface of the dam face exhibits minor scaling, which is evidenced by exposed aggregate.

The vertical joint seals, located adjacent to each end of the spillway, were inspected and appear to be intact and in good condition.

b. Outlet Structure and Gate Valve

There are four individual steel grates that compose the trash rack. The outlet structure inspection revealed that they are in poor condition. The third rack from the top is missing and another is loose and at risk of becoming completely detached. The trash racks that are still in place exhibit areas with severe deterioration of the steel with up to 100% section loss.

The gate valve has two loose bearings on the valve shaft. The lower bearing is completely separated from the guide. This condition causes the valve shaft to deflect heavily under load. The shaft exhibited surface rust but it appears to be minor deterioration. One of the valves seals is missing and another is partially displaced. A small amount of water leaks through the top seal, which is still intact.

Geotechnical Observations

Subsurface Investigation

A subsurface investigation consisting of nine borings was conducted at the Kingdom Dam site between May 16 and June 1, 2006. The borings, identified as P-1 through P-5, and auger probes, identified as AP-1 through AP-3, were advanced to depths ranging from 13.5 to 42 feet. The location and ground surface elevations of the borings and auger probes were established by measuring from existing site features and estimated from available site topographic mapping. The surface elevation at boring P-5 and the auger probes was taken to be the approximate top of spillway elevation plus the height of the barge deck above the water surface (i.e., Elevation 1034 feet). Borings P-1 through P-3 and P-5 were located to provide a cross section through the dam adjacent to the present outlet structure. Auger probes AP-1 through AP-3 and boring P-5 were located to provide a sediment and overburden depth profile along the centerline of the dam, and boring P-4 was drilled to determine bedrock elevation at the downstream side of the dam. The approximate locations of the borings are shown on the Boring Location Plan, Figure 1, included in Appendix B. The boring locations and elevations should be considered accurate to the degree implied by the method used to determine them.

CHA retained New England Boring Contractors of Glastonbury, Connecticut to advance the borings and auger probes. The field investigation was observed by CHA geotechnical engineers, who ensured that proper drilling and sampling methods were utilized, inspected and classified samples, and prepared field logs documenting the subsurface conditions.

Borings P-1 through P-4 were advanced with an ATV mounted drill rig. Boring P-1 and auger probes AP-1 through AP-3 were advanced with a skid mounted drill rig from a barge within the lake using 3-inch flush joint casing. Split spoon samples were obtained continuously in general accordance with American Society for Testing and Materials guidelines (ASTM D 1586). The split spoon samples were advanced by a 140 (±) pound hammer free falling 30 (±) inches. "Blow counts" are recorded on the boring logs, and indicate the penetration resistance for a six-inch advancement of the split spoon. Initially, the spoon is driven six inches to seat the sampler in undisturbed material. The number of blows required to drive the sampler the next 12 inches is taken as the standard penetration resistance or "N" value. This value is indicative of the soil's in-place compactness or consistency. The final six-inch increment that the spoon is driven is not included in the determination of "N". Sampler refusal is defined as a resistance of greater than 50 blows per six inches of penetration.

A NQ size core barrel (approximately 1.875 inch core diameter) was used to obtain bedrock core samples in borings P-1, P-2, P-4, and P-5. A larger diameter HQ core barrel (approximately 2.500 inch core diameter) was used to sample the bedrock in boring P-3 This core is available for direct shear testing where detailed design requires this site specific testing. The Rock Quality Designation (RQD) value was then determined for the core samples. RQD is defined as the sum of the length of core pieces 4 inches and longer, divided by the length of the core run, expressed as a percentage. The RQD values provide an indication of the relative degree of jointing or fracturing.

Methods and Techniques

Geotechnical Methods

Regional Geology

Based upon review of the *Surficial Geologic Map of New York – Adirondack Sheet* (Cadwell, D.H. & Pair, D.L. (1991)), the soil overburden is glacial till comprising sand and gravel/cobbles with varying silt content.

The *Geologic Map of New York – Adirondack Sheet* (Fisher, D.W. and Isachsen, Y.W. (1970)) maps the local bedrock as metagabbro and olivine metagabbro to leucogranitic gneiss containing sodium plagioclase and local biotite, hornblende, pyroxene, and garnet. This map also indicates an unknown bedrock deposit from the Quaternary age, in addition to local fault downstream of the project site.

Subsurface Stratigraphy

Subsurface conditions encountered in the individual borings are detailed and described on the subsurface logs included in Appendix B. These conditions can generally be described as follows, in order of increasing depth:

<u>Topsoil</u> – A topsoil layer approximately 0.2 to 0.3 inches thick was encountered at the ground surface in borings P-1 and P-4. Topsoil comprising wet, dark brown to black silt, some fine sand, little organics and decayed wood was also encountered in boring P-3 to approximately 2.5 feet below the surface. A Standard Penetration Test (N-value) of 1 bpf, reflective of a very soft material consistency, was recorded in this soil. At all other drilling locations the borings and auger probes were advanced on top of old sand and gravel/rip-rap or off a drilling barge and no topsoil was encountered at these locations.

<u>Fill</u> – Fill comprising brown to black sand, little silt, and gravel to cobbles and boulders and coarse gravel with varying amounts of silt and fine to coarse sand was encountered to depths ranging from 4.0 to 10.0 feet below the surface. N-values varied widely from 8 to greater than 100 bpf in this material. The fill was generally in a moist to wet or saturated condition when sampled.

<u>Sand</u> – Brown to light brown and grayish brown fine to coarse sand with little to some fine to coarse gravel was encountered in borings P-1, P-4, and P-5, and auger probes AP-1, AP-2, and AP-3. Little silt was observed in the sand sampled in boring AP-1, and some organic debris was noted in sample S-1 of the material encountered in boring AP-3. N-values in this soil at the auger probe locations and boring P-5 generally ranged from 1 to 6 bpf, indicating very loose to loose conditions where the material is submerged beneath approximately 5.0 to 14.5 feet of water. Exceptions were noted in samples taken immediately above the apparent bedrock interface, where N-values of 30 and 75 bpf were recorded at auger probes AP-2 and AP-1, respectively, denoting compact to very compact conditions. Material encountered downstream of the dam at P-1 and P-4 was subjected to approximately 4.0 to 7.0 feet of fill surcharge, and N-values ranged from 46 to 77 bpf, reflecting compact to very compact soil. The sand was generally in a moist to saturated condition when sampled.

Gravel and Sand / Gravel and Cobble Fragments – Brown fine to coarse gravel and fine to coarse sand with trace to little silt, to brown and black gravel and cobble fragments was encountered in borings P-2, P-3 and P-4 below depths ranging from 8.0 to 10.0 feet and exposed to depth ranging from 11 to 17.5 feet below the surface. N-values in these materials were greater than 70 bpf, with refusal noted where encountered adjacent to the bedrock interface in borings P-3 and P-2. This material was visually classified as saturated.

Cobbles/Boulders – Cobbles and boulders with traces of gravel and sand was encountered in borings P-2 and P-5 at depths ranging from approximately 6.5 to 17.0 feet below the surface, and was disclosed to about 10.0 feet and 22.0 feet deep, respectively. An attempt to core the material recovered about 1.5 feet of a 3.5-foot sample interval in boring P-2 and refusal was recorded in split-spoon samples driven in the material in boring P-5. A saturated condition was observed in the cobble and boulder material.

Weathered Bedrock – Weathered bedrock was encountered in boring P-3 and auger probes AP-1 and AP-3 at depths ranging from approximately 11.0 to 13.0 feet below the surface. Contact with this material resulted in spoon refusal at the auger probe locations, while roller bit drilling tools were employed to advance the boring approximately 2.0 feet through the material and set the rock core barrel at 13.0 feet deep in boring P-3.

Granite Bedrock – Moderately hard to hard, moderately weathered, bluish gray and white/pinkish white to gray granite bedrock was encountered in boring P-3 below approximately 12.0 to 13.0 feet deep and cored to a depth of 23.0 feet below the surface. The measured RQD (Rock Quality Designation) values ranged from 20 percent in the 13 to 18-foot sample to 55 percent in the 18 to 23-foot sample, indicating very poor to fair rock quality. At boring P-2 light gray and white, very hard, slightly weathered granite was evident from approximately 11.5 feet to 19.0 feet deep, and an RQD of 65 percent was measured, indicating fair rock quality. Fracture spacing in the recovered samples was close to very close.

Gneiss Bedrock – Massive, dark gray and gray with trace purple gneiss bedrock was encountered in borings P-1, P-2, P-4 and P-5 at depths ranging from approximately 12.0 to 22.0 feet below the surface. This bedrock was observed to be in a hard to very hard, unaltered to slightly weathered condition. Exceptions were noted at boring P-4, where the bedrock was in a medium or moderately hard condition, and in boring P-1, where the rock was soft and highly weathered below 34.0 feet. The measured RQD varied from as low as 27 percent to 100 percent, but was generally greater than 60 percent, indicating fair to excellent rock quality. Fracture spacing was generally close to medium or wide in the recovered core samples, with exceptions noted in the rock encountered at boring P-1 where very close fracture spacing was observed in the core recovered below 34.0 feet deep. Cross section A-A', and Section 3.4 of this report illustrate and discuss in greater detail the observation made of the bedrock factures and quality.

Groundwater

Piezometers were installed at boring locations P-1, P-2, P-3, and P-5. Piezometers in P-2, P-3, and P-5 were installed along cross sectional alignment A-A' while the piezometer at P-1 was installed adjacent to the south abutment. The piezometers have enabled CHA to obtain piezometric head levels at these locations in the underlying bedrock ground water flow regime. Water levels measured in these piezometers are summarized in Table 1 below. Seasonal factors, which include temperature and precipitation, affect groundwater levels. Therefore subsequent water levels may differ from those presented in this report.

Table 1
Piezometer Measurements

Boring	Date of Measurement	Estimated Ground Surface Elevation (ft)	Water Depth (Note #1) (ft)	Water Elevation (ft)
P-1	5/19/06	1025.0	2.0	1023.0
P-1	6/01/06	1025.0	2.5	1022.5
P-2	5/19/06	1014.0	1.5	1012.5
P-2	6/01/06	1014.0	1.5	1012.5
P-3	5/19/06	1015.0	1.5	1013.5
P-3	6/01/06	1015.0	1.2	1013.8
P-5	5/19/06	1032.5*	3.0	1029.5

*Note: Elevation at P-5 is approximate lake level.

Seepage Conditions

Using the bedrock quality data gathered and the piezometric level data recorded CHA has evaluated the seepage across Kingdom Dam and drawn a generalized section that illustrates the existing bedrock and seepage conditions. Cross-section A-A', adjacent to the outlet structure and lake drain, shows the generalized bedrock quality with depth across the site. The borings along this cross-section, as well as boring P-1, have been plotted with rock quality designation (RQD) percentages as obtained from rock cores. Presenting the RQD data in this manner permitted the interpretation of zones of bedrock stratigraphy as excellent, good, fair, poor and very poor bedrock quality as designated under standard geotechnical engineering classification. Using this approach CHA was able to evaluate the bedrock and make the following generalized conclusions:

a. The foundation bedrock, gneiss of gabbro origin, is fairly level beneath the cross-section of the dam, apparent from approximately Elevation 1012 to Elevation 1013 between boring P-5 and boring P-1. At some point beyond the downstream toe, this bedrock formation begins to drop in elevation, appearing at approximately Elevation 991 in boring P-2. Rock quality in the foundation material is generally good to excellent, particularly at boring P-5.
b. Moderately weathered, close fracture spacing was observed in the rock core taken below Elevation 991 in boring P-1 where the core indicated a fair rock quality. At elevations closer to the base of the dam foundation elevation, the recovered cores indicated slight weathering and a good to excellent rock quality.
c. Piezometers installed in borings P-2 and P-3 indicate a head loss of 19 to 20 feet along what is taken to be a critical cross-section of the dam. This head loss can be attributed to the relatively high RQD, limited weathering, and tight or closed discontinuities in the foundation bedrock. Projecting P-1 head measurements to the critical section, however, suggests that uplift pressures equivalent to 14 feet of hydraulic head may be present at the toe of the dam.

Structural Methods

A stability analysis was performed on the dam in accordance with the New York State Department of Environmental Conservation's Guidelines for Design of Dams, revised January 1989 and the United States Department of the Interior, Bureau of Reclamation's Design of Small Dams, revised reprint 1977.

The load cases used for this analysis included:

a) Load Case 1: water at the normal reservoir level.
b) Load Case 2: water at the normal reservoir level with an ice line loading applied.
c) Load Case 3: water surface at the spillway design flood level, which is a 150% of the 100-year storm for existing dams.
d) Load Case 4: a seismic loading condition.

The dam section was analyzed for three stability criteria: sliding, overturning and cracking. When the resultant of the overturning forces falls outside of the middle third of the dam base tension exists on the upstream side of the structure, which can result in cracking of the concrete. The "cracking" condition is acceptable in existing dams except for load case 1, per the NYSDEC's Guidelines for Design of Dams, revised January 1989.

The analysis performed has assumed:

- That the dam is not keyed into bedrock because no plans or documentation are available indicating the construction of a shear key.
- The spillway design flood level produces the maximum hydrostatic loading condition as there is a negligible tail water pool below the dam.
- The spillway section of the dam is the critical section
- The interface between the base of the dam and bedrock foundation was the assumed failure surface. A failure surface wholly within the bedrock was not considered given the bedrock is metamorphic lithology that has good to excellent bedrock quality (i.e. RQD values greater than 75%)
- Uplift forces at the toe of the dam are based on measured values at piezometer P-1 which is offset from the critical section 50 feet to the south.

Based on assumptions made above the results of the existing condition stability analysis yield factors of safety below 1.0 indicating that the dam is unstable under sliding forces for the loading conditions analyzed. It is stable for overturning except under ice loading (load case 2). The results of the existing condition stability analysis are as follows:

a.) <u>Load Case 1: Normal Reservoir Level</u>
The factor of safety for sliding is 0.99, which is below the required 2.0 factor of safety. The resultant of the overturning forces is within the middle third of the base width, as required. This also meets the requirement for cracking.

b.) <u>Load Case 2: Normal Reservoir Level with Ice Load</u>
The factor of safety for sliding is 0.73, which is below the required 2.0 factor of safety. The resultant of the overturning forces is just outside of the middle three quarters of the base when it is required to be in the middle half of the base width. This does not meet the requirement for cracking.

c.) Load Case 3: Spillway Design Flood Level
The factor of safety for sliding is 0.79, which is below the required 1.5 factor of safety. The resultant of the overturning forces is within the middle half of the base width, as suggested. This also meets the requirement for cracking.

d.) Load Case 4: Seismic
The factor of safety for sliding is 0.70, which is below the required 1.25 factor of safety. The resultant of the overturning forces is within the middle half of the base which meets the suggestion that it be within the base width. This also meets the requirement for cracking.

Remedial action on this dam is recommended based on these calculations due to the deficient sliding factors of safety. However, the stability calculations are based on available information obtained for the scope of the investigation performed. Corrective measures may be scaled back based by applying the geotechnical recommendations discussed in subsequent sections.

RECOMMENDATIONS

Structural Recommendations

Two alternatives to correct the deficiencies of the Kingdom Dam were explored. These alternatives were:

Alternative #1: Rehabilitate the Dam and Provide Stabilization with Rock Bolts

Concrete Repairs Needed:
The majority of the original concrete on the downstream face of the dam appears to have been rehabilitated with a thin concrete overlay. The buttresses, training walls and spillway exhibit various degrees of deterioration and need repair. The downstream face of the dam exhibits spalling, cracks, efflorescence, and hollow sounding concrete over most of its area.

These conditions should be repaired by completely removing the deteriorated concrete, and applying concrete patching that will be doweled to the remaining sound material with reinforcing. Any of the existing concrete patching that sounds hollow or delaminated will be completely replaced. Exposed rebar that is corroded will be sandblasted. If the exposed rebar has significant section loss, it will be replaced by doweling new rebar into sound concrete. The remaining areas, where the existing concrete is sound but has light cracking, should be cleaned and have the cracks sealed. Where similar but less severe conditions exist on the outlet structure, this repair can be used as well. On the upstream face of the dam, the spalled areas should be repaired using concrete patches.

Dam Stability:
The dam can be modified to meet the stability requirements with the addition of rock bolts. The bolts can be drilled through the dam at an angle normal to the downstream face and embedded in the bedrock underneath the structure. Installing the anchors at an angle will take greater advantage of their ability to resist sliding.

Miscellaneous Repairs Needed:
There is evidence of one expansion joint leaking toward the north end of the structure. This joint should be sealed on the upstream face. It would be prudent to re-seal the remainder of the joints as well. The gate valve should be repaired and cleaned of debris to restore proper operation.

The trash racks and the ladder accessing the outlet structure chamber should be replaced. The bottom eight feet of the log stop angles, located in the outlet chamber, should be replaced if there is a need for them.

The pedestrian bridge requires cleaning and repainting. The hand railing posts also require welding repairs. The steel grating over the outlet structure should be replaced, with welded galvanized grating, as its structural integrity is questionable. Rotted timber in the access gates requires replacement as well.

The repairs listed above address the structural concerns of the dam. It is estimated that these repairs will last at least 50 years before the dam will need additional significant rehabilitation.

Alternative 2: Rehabilitate the Dam, Apply a Three Foot Thick Concrete Overlay to the Entire Upstream Face of the Dam and Stabilize with Rock Bolts

This alternative is similar to Alternative 1 except that the upstream face of the dam will be overlayed with an approximately 3 foot thick layer of concrete. This action has two main objectives. One objective is to ensure that the upstream face of the dam is dense and non-permeable. The second objective is to reduce the size of the rock bolts. Application of concrete to the upstream face will add to the dam's mass to help resist sliding and overturning forces.

The repairs listed above address the structural concerns of the dam. It is estimated that these repairs will last at least 50 years before the dam will need additional significant rehabilitation.

Geotechnical Recommendations

Based on the data obtained during the geotechnical drilling investigation and the subsequent evaluation of this data CHA has prepared the following conclusions and recommendations.

a. The gneiss bedrock comprising the foundation material has been categorized according to RQD measurements and is generally rated as very good to excellent within about 10 to 15 feet of the foundation-concrete interface. Exceptions were detected below about Elevation 1000 in boring P-1, where fair rock quality was measured in 2 of 3 core samples. This is just below the screened interval in this boring, and this zone of fair rock quality may be the conduit through which the elevated hydrostatic pressures can possibly affect the downstream toe of the dam. The dam was analyzed in its present condition by projecting a 14 foot pressure head measured at this location to the critical section of the dam, which resulted in a condition where this uplift theoretically reduces the factor of safety in uplift to 1.0 or lower. Since the dam has been standing in its location for a long period of time, with the analysis using the elevated uplift pressure indicating an unstable condition, one can conclude that the present information concerning the hydrostatic conditions below the dam may benefit from further investigation. In this case, site clearing and dozer work can be done to access the downstream toe of the dam, where one or two additional piezometers can be installed immediately adjacent to the critical section of the dam.

b. The potential for a failure in sliding at the base of the dam and foundation-bedrock interface has been analyzed. A friction angle (phi) equal to 35 degrees, typical of mass concrete cast against competent bedrock, has been recommended for this purpose. Since the dam is presently stable, a theoretical sliding failure will most likely indicate that the uplift pressures reducing the effective sliding resistance are not completely understood as mentioned previously, or that the critical structural section is actually keyed into the foundation bedrock. If a foundation rock key is present, even a minimal one of a 6 to 12 inches in depth, it would develop of fairly significant passive resistance to lateral movement. This passive resistance would more than likely bring the factors of safety above 1.0 but may not be deep enough to meet the factor of safety set forth in the NYSDEC's Guidelines for Design of Dams. Unless construction plans of the dam depicting a rock key are available, additional investigation would be necessary to determine if a rock key is present. One way to accomplish this is to drill an inclined boring through the face of the dam above the downstream toe. This boring would extend into the bedrock to determine the rock key depth by comparing the results with previous elevation data. The boring would have to be inclined steeply to avoid penetrating the dam, so its effectiveness may be limited if the rock key is located at the upstream heel of the dam. A similar approach may be attempted from the upstream side of the dam in such cases.

c. The gneiss bedrock formation comprising the foundation provides an adequate support condition for the gravity dam. A bearing capacity equal to 100 tons per square foot (tsf) and a point load index equal to 7500 pounds per square inch (psi) have been conservatively estimated for analysis. Given the apparent intact strength and rock quality of this formation, theoretical bearing capacity limitations under typical loading conditions are not anticipated.

d. CHA recommends that additional investigation be completed as mentioned previously to obtain more information concerning the hydrostatic pressure head at the downstream toe and the possible existence of a rock key. In the absence of such additional investigation, or if the additional investigation does not alter the present analysis model, CHA recommends tie down anchors or downstream concrete facing with rock bolts to counteract the uplift and provide an accurate Factor of Safety for the various failure mechanisms.

e. Post-tensioned tie-down anchors placed along the longitudinal axis of the dam can help stabilize the dam against overturning behavior and sliding behavior. Vertical anchors, extending into the bedrock formation from the top of dam, would be most efficient for resisting overturning. While inclined anchors placed above the downstream toe, would be most efficient to resist lateral movement. A grout-rock bond force equal to 100 psi has been estimated for temporary design purposes.

f. An upstream concrete facing can be placed on the dam to increase the weight of the structure and resist uplift. The additional weight and resulting wider base will also increase the overturning resistance. After initially scaling the exposed concrete surface to remove deteriorated material, this concrete facing could be dowelled in to the exposed structure to establish structural continuity between the new concrete and the existing structure. Further stability is possible by extending the concrete facing into a rock key excavated at the downstream toe. An allowable cohesion equal to 3000 psi has been estimated for design of such a rock key.

g. The upstream concrete facing noted previously may be installed with rows of inclined rock bolts placed above the heel in lieu of excavating a rock key. These bolts would extend into the foundation bedrock and may act as passive anchors or be post-tensioned and function as active tie down anchors. The need for post-tensioning will depend on the level of increased stability required.

h. Excavation of the gneiss as observed in the borings will likely require controlled blasting with pre-splitting drilling if a rock key is constructed. Excavations should be performed in accordance with the Occupational Safety and Health Administration (OSHA) standards and applicable state and local codes. The design of the temporary excavation system shall be designed by a Registered Professional Engineer in the State of New York.

COST ESTIMATE

Considering the two alternatives discussed and based on the results of the phase 2 investigation, the approximate costs of the different options are as follows:

Alternative # 1: Rehabilitate the dam and provide stabilization with rock bolts = $925,000
Alternative #2: Rehabilitate the dam, apply a 3 foot thick concrete overlay to the entire upstream face of the dam and stabilize with rock bolts = $1,200,000

The opinions and recommendations contained in this report are based on visual field investigations performed as part of this project. This report does not address any other portions of the structure other than those areas mentioned, nor does it provide any warranty, either expressed or implied, for any portion of the existing structure.

If you have any questions regarding these issues, please feel free to call me at (518) 453-3945.

Very truly yours,

CLOUGH HARBOUR & ASSOCIATES LLP

Thomas L. O'Brien, P.E.
Partner

TOB/rja
Enc.
CC: C.Dooley

APPENDIX A

SURVEY DATA

APPENDIX B

GEOTECHNICAL DATA

APPENDIX C

STRUCTURAL PHOTOS

APPENDIX D

STRUCTURAL DATA

APPENDIX E

STABILITY CALCULATIONS