Report of Engineering Investigation of the Hadlock Pond Dam Failure

Fort Ann, New York

Prepared for:

New York State Department of Environmental Conservation

Division of Water

Bureau of Program Resources & Flood Protection

Submitted by:

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CHA Project No. 14559.2000.1502

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EXECUTIVE SUMMARY

This report summarizes the field investigations and evaluations performed by Clough Harbour & Associates LLP (CHA) as part of the Engineering Investigation of the Hadlock Pond Dam failure. CHA was retained by the New York State Department of Environmental Conservation (NYS DEC), through a term contract with the New York State Office of General Services, to investigate the failure of the Hadlock Pond Dam which occurred on July 2, 2005. The Hadlock Pond Dam is located in the Town of Fort Ann, New York. The primary objective of the engineering investigation was to gather data and review available documentation to render an opinion, if possible, as to the potential cause or causes of the dam failure.

Six types of incidents were evaluated that could cause an accident or failure of an earthen dam such as the Hadlock Pond Dam:

- overtopping of the dam by the impounded waters;
- settlement of the dam embankment;
- slope instability of the dam embankment;
- occurrence of a seismic event which could lead to a weakening of the dam or the foundation soils;
- vandalism which could cause degradation of the dam, and;
- piping or internal erosion.

Based on the information gathered during the investigation and an evaluation of available documentation at the time this report was prepared, overtopping, settlement, slope instability, seismic activity and vandalism are not supported as types of incidents leading to the failure of the Hadlock Pond Dam.

Based on the information gathered during the investigation and an evaluation of available documentation at the time this report was prepared, it is our opinion that the Hadlock Pond Dam failed due to internal erosion through or beneath the dam. The definitive preferential pathway cannot be identified at the breach location, but based on the observations and data obtained from the

field investigations and review of available information, it is reasonable to conclude that internal erosion occurred along one or more of the following potential preferential pathways:

- the old to new embankment contact zones where existing oversized rock-fill had not been properly removed from the surface and/or no soil transition material had been placed;
- the contact between loosely compacted new embankment backfill and the vertical west face of the Roller Compacted Concrete (RCC) base or abutment wall;
- a disturbed soil zone within the new embankment that formed a preferential pathway through loose soils as a result of saturated or frozen material placement or burial of frozen material on the surface of an embankment lift;
- a disturbed soil zone within the new embankment that formed a preferential pathway through coarse granular materials as a result of access road base aggregates left in place above subgrade soils.

A number of studies have been conducted in the past to summarize types of incidents leading to failures or accidents for all types of dams. This summary indicated that of earthen dams that were reported to have failed, excluding overtopping as a failure mode, at least 25% of dam failures occurred during first filling. Given the statistic cited above, regular observations during first filling may have allowed for identification of symptoms of potential latent defects within the dam at a critical time in its service. Review of the available documentation did not indicate that regular observations occurred during the first filling of Hadlock Pond.

The findings of this engineering investigation into the failure of the Hadlock Pond Dam are based on the information gathered during the investigation and an evaluation of available documentation at the time this report was prepared. It is possible that additional information, not available during the investigation, could exist regarding the design and construction that could more definitively identify the specific mechanism leading to failure.

This executive summary briefly presents the findings of the engineering investigation; to fully understand the investigatory findings and evaluations, the report should be read in its entirety.

This report has been prepared and reviewed by the following qualified engineers employed by Clough Harbour & Associates LLP.

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1.0 INTRODUCTION

This report summarizes the field investigations and evaluations performed by Clough Harbour & Associates LLP (CHA) as part of the Engineering Investigation of the Hadlock Pond Dam failure. CHA was retained by the New York State Department of Environmental Conservation (NYS DEC), through a term contract with the New York State Office of General Services, to investigate the failure of the Hadlock Pond Dam which occurred on July 2, 2005. The Hadlock Pond Dam is located in the Town of Fort Ann, New York and is shown on the Site Location Map, Figure 1, included in Appendix A.

The primary objective of the engineering investigation was to gather data and review available documentation to render an opinion, if possible, as to the potential cause or causes of the dam failure. The following information was gathered during the field investigation and is presented in Sections 2.0 through 9.0:

- Topographic and Three Dimensional Survey;
- Soil Test Borings;
- Test Trenches;
- Subsidence Test Trenches;
- Geophysical Survey;
- Dam Breach Sections;
- Interviews;
- Detailed Visual Inspection.

Review of available documentation for the Hadlock Pond Dam rehabilitation produced by HTE Northeast, Inc. (HTE); Kubricky Construction Corp.; Atlantic Testing Laboratories, Limited; Copeland Environmental and NYS DEC at the time this report was prepared included:

- HTE design drawings and documents;
- HTE contract drawings, specifications and documents;
- correspondence between Kubricky Construction Corp. and HTE;

- HTE Project Status Reports;
- Kubricky Construction Corp. foreman reports;
- correspondence and observation reports from Copeland Environmental regarding wetland remediation;
- Atlantic Testing Laboratories material testing results and reports;
- historic information of Hadlock Pond Dam from NYS DEC files;
- correspondence between involved parties and the NYS DEC;
- photographs showing the dam rehabilitation work produced by HTE, Kubricky Construction Corp., and Atlantic Testing Laboratories;
- photographs of the Hadlock Pond Dam failure taken by the Lake Hadlock Association, Inc. and NYS DEC;
- videotapes of the Hadlock Pond Dam failure, taken by local residents, provided by NYS DEC.

The original Hadlock Pond Dam was noted in various sources to have been built in 1896 by the Kanes Falls Electric Company. A Glens Falls newspaper article dated September 12, 1917 reported that the dam had been "blown up" the previous evening. In 1924, an inspection report indicated the dam length was about 850 feet long, and was constructed of rock filled timber crib with sand, gravel and boulder fill on the upstream side, and stone and boulder fill on the downstream side. The dam was raised in 1933 with the addition of soil and boulder fill to the upstream and downstream sides as well as above the rock filled timber crib structure. In 1977, a portion of the dam was reconstructed due to an accident at the dam that occurred in 1975 as a result of deterioration of two drain pipes beneath the dam. Reconstruction in 1977 included a new service spillway with a drop inlet structure and a 48-inch diameter reinforced concrete outlet pipe. A portion of the timber cribbing and embankment placed in 1933 was removed during reconstruction and replaced with compacted sand and gravel. After 1977 and prior to the most recent dam remedial work, modifications to the dam included placement of the 30-inch diameter winter drain pipe and reconstruction of the drop inlet structure above the pipe.

The most recent dam remedial work began in September 2004. The remedial work was designed by HTE and constructed by Kubricky Construction Corp.; in addition, Atlantic Testing Laboratories provided material testing services. The remedial work included construction of a multi-stage auxiliary spillway system which included an ogee spillway, fuse plug spillway sections and reconstruction of portions of the existing embankment. A photographic timeline of the remedial work is provided in Appendix G. This timeline was developed from photographs produced by HTE, Atlantic Testing Laboratories, and Kubricky Construction Corp. According to item #2 of HTE Project Status Report No. 33, dated May 2, 2005, "all work on the dam structure, including the spillway, fuse plugs, concrete barrier, backfilling and loam and seeding…" had been completed as of that date. On July 2, 2005 failure of the dam occurred, resulting in a breach west of the new ogee spillway.

2.0 TOPOGRAPHIC AND THREE DIMENSIONAL SURVEY

A topographic survey of the existing breach, existing dam, and downstream rubble area, which began on July 7, 2005 and was completed on August 4, 2005, was performed by CHA and is shown on the Topographic Survey, Figure 2, included in Appendix A. Latitude, longitude and elevations were obtained using geodetic grade GPS referenced to HDF1:Hudson Falls 1 Cors, PID-AI8827 located in Hudson Falls, NY. The latitude and longitude are referenced to UTM zone 18, NAD 83, and the elevations and contours are referenced to NAVD88. The elevations shown on the topographic survey differ from the elevations shown on the HTE construction drawings for the dam rehabilitation; the elevations on the HTE drawings appear to be based on an arbitrary reference. The survey documented existing features, including but not limited to:

- surface elevations;
- water level elevations;
- water mark on concrete ogee spillway;
- timber cribbing;
- soil stratification;
- partial collapsed fuse plug area;
- steel sheet piling;
- RCC surface elevations;
- elevation profile of precast concrete wall unit;
- ground subsidence on the dam embankment east side of the ogee spillway;
- locations of sediment discharge on the downstream side of the dam;
- service spillway;
- location of soil test borings, test trenches and geophysical investigation points.

A three dimensional survey of the dam failure area was performed by Badey & Watson, Surveying and Engineering of Cold Spring, New York. A High Density Laser Scan was performed to provide an accurate model of the dam failure area for precise measurement and detail. Topographic information encompassing the failure area is shown on Figure 2.

3.0 SOIL TEST BORINGS

The investigation included 11 soil test borings identified as B-1 through B-10 and B-6A. Soil test boring B-2 was advanced through the embankment just west of the failure area. Soil test borings B-5, B-7, B-9 and B-10 were advanced through the dam embankment east of the ogee spillway. Soil test borings B-6 and B-6A were advanced on the upstream side of the dam embankment in the area of the east fuse plug. Soil test boring B-8 was advanced on the upstream side of the dam embankment adjacent to the north end of the subsidence. Soil test borings B-3 and B-4 were advanced on the downstream side of the dam embankment; B-3 was located southwest of the failure area and B-4 was located in the area of the east fuse plug. The soil test borings were advanced to depths ranging from 10.5 feet to 77.4 feet. Subsurface conditions encountered in individual soil test borings are described in the subsurface logs included in Appendix B. In addition, photographs of selected soil test borings are attached following the subsurface logs.

Soil boring locations were surveyed at the completion of each boring. The locations and elevations are shown on the Investigation Location Plan, Figure 3, included in Appendix A.

New England Boring Contractors of Glastonbury, Connecticut was retained by CHA to advance the soil test borings. The boring program began on July 20, 2005 and was completed on July 28, 2005. The soil test borings were performed under the full-time observation of CHA geotechnical engineers. Proper drilling and sampling methods were observed to have been utilized. CHA geotechnical engineers also inspected and classified samples and prepared field logs documenting subsurface conditions. The soil test borings were also observed by representatives from GEI Consultants, Inc.; Haley & Aldrich, Inc.; O'Brien & Gere Engineers, Inc.; Robson Forensic, Inc.; and NYS DEC.

A Mobil Drill B-53 ATV mounted drill rig was used to advance soil test borings B-1, B-3, B-4, B-6, B-6A and B-8; a Mobil Drill B-53 truck mounted drill rig was used to advance soil test borings B-2,

B-5, B-7, B-9 and B-10. The soil test borings were typically advanced with four inch flush joint casing with exceptions noted on individual subsurface logs.

Split spoon samples were typically obtained continuously until boring completion in general accordance with American Society for Testing and Materials (ASTM) guidelines D 1586-99. The split spoon samples were advanced with 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". Refusal is defined as a resistance of greater than 50 blows per six inches of penetration.

Field permeability testing was also performed during advancement of soil test borings B-2, B-5 and B-6A at select depths. The data collected was analyzed in accordance with the procedures outlined in *Analysis of Permeability by Variable Head Tests, Case F-1* (NAVFAC, DM-7.1, 1986). Results from the analysis are included following individual subsurface logs in Appendix B.

Thin-walled Shelby tube sampling was used to obtain relatively undisturbed soil samples in soil test borings B-2, B-3 and B-7 in general accordance with ASTM D 1587-00. The samples were obtained by pressing a thin-walled metal tube with a three inch outside diameter into the in-situ soil at the bottom of the boring, removing the soil-filled tube, and applying seals to the soil surfaces inside the tube to prevent soil movement and moisture gain or loss.

3.1 Laboratory Testing

GeoTesting Express, Inc. of Boxborough, Massachusetts was retained by CHA to perform soil laboratory testing. Laboratory testing was performed on selected samples collected during the advancement of the soil test borings. Laboratory testing included Atterberg Limits (ASTM D 4318), Grain Size (ASTM D 422), Moisture Content (ASTM D 2216), Permeability (ASTM D 5084) and Triaxial Shear (ASTM D 4767). Laboratory results are included following individual subsurface logs in Appendix B.

4.0 TEST TRENCHES

The investigation included nine test trenches identified as TT-1 through TT-8 and TT-12. Test trenches TT-4, TT-7 and TT-8 were excavated downstream of the dam embankment with the remaining test trenches advanced upstream of the dam embankment. Subsurface conditions encountered in individual test trenches are described in the subsurface logs included in Appendix C. In addition, photographs documenting conditions encountered are included following individual subsurface logs.

Test trench locations were surveyed at the completion of the excavation. The locations and elevations are shown on the Investigation Location Plan, Figure 3, included in Appendix A. Elevations shown on the subsurface logs were based on the approximate center of the test trench.

Cutting Edge Group LLC of Lake George, New York excavated the test trenches using a Komatsu PC58UU mini-excavator and a Komatsu PC308LC excavator. The test trench excavations were performed on July 21 and 22, 2005 with the exception of test trench TT-12 which was excavated on August 4, 2005. The test trenches were performed under the full-time observation of CHA geotechnical engineers who directed the excavation, inspected and classified soil samples and prepared field logs documenting subsurface conditions. The test trenches were also observed by representatives from GEI Consultants, Inc.; Haley & Aldrich, Inc.; O'Brien & Gere Engineers, Inc.; Robson Forensic, Inc.; and NYS DEC.

4.1 Laboratory Testing

GeoTesting Express, Inc. of Boxborough, Massachusetts was retained by CHA to perform soil laboratory testing. Laboratory testing was performed on selected samples collected during the excavation of the test trenches. Laboratory testing included Atterberg Limits (ASTM D 4318), Grain Size (ASTM D 422) and Moisture Content (ASTM D 2216). Laboratory results are included following individual subsurface logs in Appendix C.

5.0 SUBSIDENCE TEST TRENCHES

The subsurface investigation of the subsidence area, which was located approximately 60 feet east of the ogee spillway, began on August 3, 2005 and was completed on August 5, 2005. The subsidence investigation included three test trenches, identified as TT-9 through TT-11. Test trench TT-9 was excavated on the downstream side of the crest wall in the area of the subsidence, TT-10 was initiated about 16 feet north (upstream) of the crest wall and was continued to about 44 feet south (downstream) of the crest wall, and TT-11 was initiated about 60 feet downstream of the crest wall and was continued to about 14 feet upstream of the crest wall. Subsurface conditions encountered in the test trenches are briefly described in the following text and illustrated in the photographs and on the Subsidence Test Trench Plan, Figure 4, included in Appendix E. Significant points in the test trenches, as well as the test trench boundaries, were surveyed throughout the excavation and are shown on Figure 4.

Cutting Edge Group of Lake George, New York excavated the test trenches using a Komatsu PC58UU mini-excavator and a Komatsu PC308LC excavator. The test trenches were observed by representatives from GEI Consultants, Inc.; Haley & Aldrich, Inc.; O'Brien & Gere Engineers, Inc.; Robson Forensic, Inc.; and NYS DEC, and were directed by a geotechnical engineer from CHA with agreement from all parties present. In addition, CHA inspected the trenches, collected representative soil samples for laboratory testing, took photographs and prepared field notes documenting the subsurface conditions. A NYS DEC Bureau of Public Outreach Photographer also took photographs and videotaped the test trenches. The photographs shown in Appendix E were provided by CHA and NYS DEC.

5.1 Proposed Test Trench Work Plan

The goal of advancing the test trenches in the area of the subsidence was to assess the cause of the subsidence. The initial work plan, as outlined below, was developed to minimize disturbance to the remaining dam embankment:

- On the downstream side of the crest wall in the area of the subsidence, excavate with a small backhoe to the base of the concrete crest wall from the approximate top-of-dam elevation of 457 feet (based on CHA Draft Topographic Survey 7/15/05). It was estimated that the width of the excavation would be about four feet.
- On the upstream side of the crest wall in the area of the subsidence, excavate with a small backhoe to the base of the concrete crest wall from the approximate top of dam elevation of 457 feet (based on CHA Draft Topographic Survey 7/15/05). It was estimated that the width of the excavation would be about four feet.
- Remove concrete crest wall panels as necessary to continue the excavation. It was estimated that two panels would be required to be removed to facilitate the excavation.
- Continue to excavate north to south in about one foot vertical lifts to approximately elevation 446 feet, or two feet below the lowest visible subsidence elevation of about 448 feet. The excavation would proceed in a benched fashion, requiring a four foot horizontal bench for every four feet of vertical excavation. A small backhoe would be utilized until its reach was overextended, at which time a larger backhoe would be used.
- Once the excavation reached about elevation 446 feet, the need for further excavation would be collectively evaluated. If further evaluation below elevation 446 feet was determined necessary, additional excavation would proceed as indicated above with a four foot horizontal bench for every four feet of vertical excavation to a maximum depth below the top-of-dam of 20 feet, or about elevation 437 feet.

5.2 Actual Test Trench Progression

The subsidence test trenches, identified as TT-9 through TT-11, were advanced in a manner similar to the initial work plan as outlined below and are illustrated in the photographs and on the Subsidence Test Trench Plan, Figure 4, included in Appendix E:

- On the downstream side of the crest wall in the area of the subsidence, hand digging was initially performed to remove topsoil at the ground surface. A nuclear density test ND-1 was performed in the subsidence area. Grab sample Grab-1A was obtained after the nuclear density test to determine the maximum laboratory dry density in accordance with the Modified Proctor test (ASTM D 1557). TT-9 was then excavated to about elevation 454 feet (see photo 1). At this elevation, soft and saturated gray silty soils were encountered. Three nuclear density tests ND-2 through ND-4 were performed, and grab samples Grab-1 and Grab-4 through Grab-6 were obtained for laboratory testing. The excavation of TT-9 continued to approximately elevation 448 feet; observation of color change in the soil strata was noted at about elevation 453.3 feet (see photos 2 & 3).
- On the upstream side of the crest wall, the rip-rap and geotextile over the subsidence was removed (see photos 4 & 5). Survey points were taken to document the contour of the ground surface at the subsidence. This was followed by probing at select locations with a piece of rebar to locate soft spots within the subsidence, hand digging and obtaining grab samples Grab-2 and Grab-3 for laboratory testing.
- About 16 feet upstream of the crest wall in the area of the subsidence, the first cut of TT-10 was initially excavated to about elevation 445 feet. One nuclear density test ND-5 was performed, and grab samples Grab-7 through Grab-10 were obtained for laboratory testing. Soil color changes were noted and surveyed (see photos 6 & 7). Three more successive cuts (identified as second cut through fourth cut) were made in TT-10 in the downstream direction (see photos 8 through 14), and grab samples Grab-11 through Grab-15 were obtained. After the third cut was made, one nuclear density test ND-6 was performed. The third cut also revealed a soft spot in the gray sandy silt soil which was identified by probing with a piece of rebar (see photo 11). During each cut,

the bottom of TT-10 was maintained at about elevation 445 feet. After the fourth cut was made, the excavation had progressed to about four feet north of the crest wall.

- Three concrete crest wall panels were removed to facilitate the continuation of TT-10 in the downstream direction (see photo 15). The concrete panels remained intact upon removal and sustained only slight damage.
- A fifth cut was made in TT-10 to about the location of the crest wall after removal of the crest wall panels. A nest of cobbles and boulders with significant voids was observed at the east side of the cut (aligned approximately below the subsidence) at about elevation 448.5 feet (see photos 16 & 17). Four more successive cuts (identified as sixth cut through ninth cut) were made in TT-10 to about 29 feet downstream of the crest wall, maintaining the bottom of TT-10 at about elevation 445 feet. Cobbles and boulders were continually encountered along the east side and sand and silt with little cobbles were encountered along the west side of TT-10 as the trench was progressed. Color changes in the soil, cobbles and boulders, timber and grab samples were noted and surveyed (see photos 18 through 24). Grab samples Grab-16 through Grab-19 were taken for laboratory testing.
- After the ninth cut, the remaining 15 feet of the dam embankment was removed to elevation 445 feet (see photo 25). TT-11 was then initiated about 60 feet downstream of the crest wall with the bottom elevation at about 438 feet, and was excavated to approximately 14 feet upstream of the crest wall with the bottom elevation at about 441 feet. In general, cobbles and boulders were continually encountered along the east side of TT-11 downstream of the crest wall as the trench was progressed; sand and silt with little cobbles were encountered along the west side (see photos 26 through 30). A piece of timber was noted at about three feet upstream of the crest wall at about elevation 443.6 feet. Grab sample Grab-21 was taken for laboratory testing.

After the investigation was complete, the test trenches were backfilled with the material removed using the Komatsu PC308LC excavator. The material was placed in approximately three foot lifts, and tamped with the bucket of the excavator. The backfill was shaped to match the existing

embankment slope as near as possible, and the rip-rap on both the upstream and downstream side was replaced. The three concrete crest wall panels were not reinstalled, and were left lying flat on the upstream slope just west of the subsidence test trenches.

5.3 Nuclear Density Testing

The nuclear density tests were conducted by a qualified CHA representative in general accordance with ASTM D 5195. The results of the nuclear density tests are shown in Table 1 below:

Table 1: Nuclear Density Test Results

Test Number	Wet Density (pcf)	Dry Density (pcf)	Moisture Content (%)	Associated Grab Sample Number
ND-1	117.9	106.9	10.2	Grab-1A
ND-2	131.8	115.3	14.3	Grab-4
ND-3	137.5	124.3	10.6	Grab-5
ND-4	132.8	115.0	15.5	Grab-6
ND-5	124.5	115.2	8.1	Grab-9
ND-6	136.4	125.4	8.8	Grab-14

5.4 Laboratory Testing

GeoTesting Express, Inc. of Boxborough, Massachusetts was retained by CHA to perform soil laboratory testing. Laboratory testing was performed on selected samples collected during the excavation of the subsidence test trenches, and the results are included in Appendix E. Laboratory testing included Atterberg Limits (ASTM D 4318), Grain Size (ASTM D 422) and Moisture Content (ASTM D 2216). CME Associates, Inc. of Albany, New York was retained by CHA to perform the Modified Proctor test (ASTM D 1557) and Grain Size (ASTM D 422) on selected samples. In addition, grab sample Grab 20 was sent to Advance Testing of Campbell Hall, New York for

Petrographic Analysis (ASTM C 295) to determine if the sample contained cement. This sample was taken from the downstream slope of the dam embankment about 10 feet east of TT-11, and its location is shown on Figure 3.

6.0 GEOPHYSICAL SURVEY

A geophysical investigation was performed by NDT Corporation of Worcester, Massachusetts on July 21 and 22, 2005. The investigation included seismic refraction, electrical resistivity and ground penetrating radar. Data was collected along the dam embankment and along the upstream and downstream toe of the dam embankment to assess the condition of the embankment and investigate the geology in the area of the dam. The Geophysical Investigation report is included in Appendix D.

7.0 DAM BREACH SECTIONS

This section will discuss the visual observations made by CHA geotechnical engineers of the area of the dam that was breached on July 2, 2005. CHA inspected the dam breach faces, collected representative soil samples for laboratory testing, took photographs and prepared field notes documenting the dam embankment and subsurface conditions. The dam breach faces were also observed by representatives from GEI Consultants, Inc.; Haley & Aldrich, Inc.; O'Brien & Gere Engineers, Inc.; Robson Forensic, Inc.; and NYS DEC.

The breach of the dam caused the removal of the entire section of the dam over an approximate width of 110 feet to 125 feet (at the crest), including a portion of the western fuse plug (fuse plug No. 2). The western limits of the breach (at the crest) were located approximately 200 feet west of the western training wall of the ogee spillway. The eastern limits of the breach (at the crest) were located approximately 75 feet west of the western training wall of the ogee spillway.

The breach area included a scour hole extending to about elevation 425 feet, approximately 14 feet into the foundation soils. The scoured area extended north (upstream) from the centerline of the dam embankment crest approximately 110 feet with widths between 40 feet and 85 feet, and extended south (downstream) from the centerline of the dam embankment crest approximately 120 feet with widths between 60 feet and 90 feet. A debris field containing soil, cobbles, boulders, concrete crest panels, small pieces of sheet piling and roller compacted concrete (RCC) pieces extended approximately 250 feet downstream of the scoured area, with widths between 50 feet and 70 feet.

Exposed steel sheet piles were observed just west of the eastern edge of the breach and extended approximately 45 feet to the west. The western 20 feet of steel sheet piles was bent in the upstream direction and had a lower top-of-sheet pile elevation compared to the eastern 25 feet, which was aligned nearly vertical. Immediately downstream of the sheet piles was an RCC slab, approximately 20 feet wide by 40 feet long, with a mass of soil resting on top of the RCC slab approximately 12 feet wide by 15 feet long and 3.5 feet to 7 feet high.

The exposed western section of the dam breach, as shown on Dam Breach West Face Section, Figure 5, included in Appendix F, revealed newly placed silty sand fill above approximately elevation 449 feet to the top of the dam crest. Below the newly placed fill and downstream of about station 1+05, materials consisted of the old cobble and boulder fill, and upstream of about station 1+05 material consisted mainly of the old silty sand fill. Below the old fill materials, at approximately elevation 441 feet and downstream of about station 1+05, natural sandy silt foundation soils were observed. In addition, two pieces of timber were observed extending horizontally out of the west face of the breach at approximately elevation 439.5 feet at stations 1+15 and 1+21. Grab samples Grab-1 through Grab-12 were taken for laboratory testing. Grab samples Grab-4 through Grab-6 were composed of RCC material and were not sent for laboratory testing.

The exposed eastern section of the dam breach, as shown on Dam Breach East Face Section, Figure 6, included in Appendix F, revealed the section of the dam at fuse plug No. 2. Three grab soil samples were obtained and labeled by continuing the numbering used on the west face. Grab-13 was a sample of the clay material remaining within the exposed RCC fuse plug spillway. Grab samples Grab-14 (Till Above RCC) and Grab-15 (Erodible Soil Plug) were of two distinct soil types within the erodible soil zone of the fuse plug. The east face also exposed a PVC pipe at about station 1+05, located just above the RCC fuse plug spillway surface at about elevation 447.3 feet. Grab samples Grab-14 and Grab-15 were sent for laboratory testing.

During the investigation of the dam breach, additional grab samples were taken east of the ogee spillway on the downstream side of the dam embankment. The grab samples were taken approximately 24 feet east of the ogee spillway training wall (identified as 24 ft. E of Spillway) and near the 48 inch RCP outlet (identified as Sediment Discharge), and their locations are shown on Figure 3. The grab samples were sent for laboratory testing, and the results are included in Appendix F.

7.1 Laboratory Testing

Laboratory testing was performed on selected samples to determine the soil classification. Testing included Atterberg Limits (ASTM D 4318), Grain Size (ASTM D 422) and Moisture Content (ASTM D 2216). GeoTesting Express, Inc. of Boxborough, Massachusetts was retained by CHA to perform the laboratory analysis; the results are included in Appendix F.

8.0 INTERVIEWS

As part of the geotechnical investigation, in coordination with the NYS DEC, CHA conducted six interviews with six different parties who observed conditions at the dam prior to or during the failure. NYS DEC Investigator Fred Stannard and Washington County Sheriff's Department Investigators, Anthony LeClaire and Bruce Hamilton, took statements shortly after the failure from a number of individuals who had witnessed events leading up to and after the failure. Six parties were selected by Investigator Stannard to be interviewed with consultation by CHA. Michael Quinn, P.E., an Associate with CHA, conducted the interviews and asked the witnesses to describe specifically when, where and what they observed with respect to the dam. The interview notes are arranged in chronological order from the time that observations were made along the crest wall in mid-June to the failure observed on July 2, 2005.

8.1 Interview with Eyewitness #1 – July 25, 2005

At the time of the interview of Eyewitness #1, Fred Stannard was present. Eyewitness #1 stated that he walked along the dam crest wall in mid-June.

- Eyewitness #1 noted small subsidence depressions at joints in precast wall segments.
- The subsidence observed during CHA's detailed visual inspection (Section 9.0), approximately 50 feet east from ogee spillway, was not observed during the visit by Eyewitness #1.
- Eyewitness #1 did not see any subsidence at the crest wall east of the newly constructed service ogee spillway. Eyewitness #1 also did not notice any seepage, but was not down at the toe of the embankment.

8.2 Interview with Eyewitness #2 – July 27, 2005

At the time of the interview of Eyewitness #2, Fred Stannard was present. Eyewitness #2 first visited the dam on Wednesday or Thursday, June 22nd or 23rd, 1.5 weeks prior to the dam failure.

- Eyewitness #2 stated that he went to the dam to go fishing with a friend (Eyewitness #3).
- Eyewitness #2 walked out on the dam crest to a path leading down the embankment to the RCP outfall pipe that discharges to the north end of the beaver pond. Eyewitness #2 stood and fished in the beaver pond from the top of the pipe.
- Eyewitness #2 explained that while he was fishing, he noticed water discoloration from sediment entering the pond from a discharge point located approximately five feet above and five feet to the west side of the outlet pipe. The discolored water was discharging over the ground surface and into the beaver pond.
- Eyewitness #2 then said he walked back up the path leading to the crest of the embankment to look at Hadlock Pond. From the top of the embankment Eyewitness #2 noticed the subsidence located east of the ogee spillway adjacent to the crest wall. Eyewitness #2 stated that the subsidence, at that time, was not as large as the present subsidence, with the approximate dimensions being three feet in diameter by two feet deep.

The second visit by Eyewitness #2 to the dam was on July 2, 2005.

- Between 4 p.m. and 5 p.m., Eyewitness #2 returned to the dam. Eyewitness #2 noticed that the seepage observed on his previous visit was not flowing and he thought it had dried up.
- Eyewitness #2 then walked to the ogee spillway fence at the top of the embankment and saw seepage with sediment flowing into the beaver pond from an area at the toe of the embankment west of the ogee spillway. When asked specifically where seepage was coming from, Eyewitness #2 said the seepage was coming from the base of the portion of the dam that had failed.

- Eyewitness #2 did not walk beyond the east fence of the ogee spillway. After observing the seepage, Eyewitness #2 left the site.

8.3 Interview with Eyewitness #3 – August 5, 2005

At the time of the interview of Eyewitness #3, Fred Stannard was present. Eyewitness #3 made a single visit to the dam prior to the date of failure.

- Eyewitness #3 took us to the RCP outfall following the same path as Eyewitness #2 and described what he saw with Eyewitness #2 approximately 1.5 weeks prior to the failure. Eyewitness #3 said that he had observed seepage coming from the same point Eyewitness #2 had observed, but he also said that seepage was coming from multiple points along the base of the dam (east of the newly constructed ogee spillway).
- The flow and sediment was clouding the west shore of the beaver pond. After seeing this, Eyewitness #3 went back to swim at the beach adjacent to the east dam abutment. He said Eyewitness #2 went up to the dam and walked over to the ogee spillway. Since Eyewitness #3 did not go to the ogee spillway along the crest wall, he did not observe the subsidence that Eyewitness #2 had observed.

8.4 Interview with Eyewitness #4 – July 28, 2005

At the time of the interview of Eyewitness #4, Fred Stannard was present. Eyewitness #4 indicated that his observations were made downstream of the dam, approximately 200 feet from the west abutment on July 2, 2005 between 5 p.m. and 6 p.m.

- Eyewitness #4 described initially hearing loud clunking sounds of boulders hitting each other and water flow.
- Eyewitness #4 went outside to look at the dam and saw a large spout of water shooting up from the toe of the downstream side of the embankment at the west end of the dam. Rocks, both cobble and boulder size, were being thrown from the spout. The flow

continued for approximately 15 minutes when the spouting appeared to move backwards toward the dam and upward toward the crest walls in an inverted V-shape.

- Once reaching the crest wall, the wall segments toppled into the flow and the breach widened laterally releasing the lake downstream. Eyewitness #4 described the flow out through the breach as an enormous tunnel of water which was very frightening.

8.5 Interview with Eyewitness #5 – August 30, 2005

At the time of the interview of Eyewitness #5, Fred Stannard and John Aspland, Esq. (Town Attorney) were present.

- Eyewitness #5 was asked what his role was regarding maintaining the pond level during construction. Eyewitness #5 explained that he checked the water level typically once a day and sometimes twice a day. Based on his monitoring and the needs of Kubricky Construction Corp. representatives, Eyewitness #5 notified the Town Highway Superintendent to have the valve opened or closed to achieve the desired water level in the pond.
- The water level was held at an elevation that kept the construction site dry. When asked if the construction site was ever swamped, Eyewitness #5 answered no. Typically the pond level was held two feet above the low level intake pipe to prevent freezing of the valve.
- When asked if he ever saw frozen material buried, Eyewitness #5 said no; although, he said he was not specifically looking at the backfilling procedures during each visit.
- On the day of the failure at about 4 p.m., Eyewitness #5 said he was down at the south end of the lake in his boat and noted the water going over the ogee spillway crest at a height of approximately two to three inches. Eyewitness #5 said that he had not observed the water level at the ogee spillway any higher than this since the filling of the pond began.

- Eyewitness #5 returned to his home and received a call between 5 p.m. and 6 p.m.; the phone call was from Eyewitness #6 informing him that the pond was losing water and that a whirlpool had developed north of the dam embankment. Eyewitness #5 returned immediately back to the south end of the lake. When he got to the island, several hundred yards north of the dam, he saw the crest and crest wall of the dam crumble into the dam breach. Eyewitness #5 immediately turned the boat around and headed back to his home telling all boaters he saw to get off the lake and that the dam was failing. When Eyewitness #5 got back to his dock he could see that the lake level had dropped more than eight inches in the time he was gone.

8.6 Interview with Eyewitness #6 – August 30, 2005

At the time of the interview of Eyewitness #6, Fred Stannard was present. Eyewitness #6 indicated that his observations were made while standing at the west upstream end of the dam crest.

- On July 2, 2005 between 5 p.m. and 6 p.m., Eyewitness #6 said he initially noticed a whirlpool approximately 50 to 60 feet north of the dam embankment and approximately 100 feet east of the west shore of the pond. Eyewitness #6 said he went down to the dam crest and that the whirlpool was about three feet in diameter and had moved south towards the dam embankment.
- Eyewitness #6 did not notice any flow on the downstream side, but he could not say he looked over the crest in that direction. When asked if he saw Eyewitness #4 from his vantage point, Eyewitness #6 said that he did not notice if Eyewitness #4 was outside.
- Upon seeing the whirlpool and the lake dropping, Eyewitness #6 attended to his boat, and then went to help his neighbor. When he returned the lake was in full flow through the breach.

9.0 DETAILED VISUAL INSPECTION

The Hadlock Pond Dam may be characterized as an earthen dam formed by a composite of the older sections of earth dam embankment and rock fill structure (i.e., the remaining east and west abutment sections) and the recently completed multi-stage auxiliary spillway system. The multi-stage auxiliary spillway system includes an ogee spillway and two fuse plug spillways that were tied into the original dam structure by constructing new embankment sections. The dam is located adjacent to Hadlock Pond Road and is approximately ¼ mile upstream of New York State Route 149 in the Town of Fort Ann. Figure 1, Site Location Map illustrates the dam site and vicinity.

Multiple inspections of the various components of the failed dam were conducted between July 6 and July 24, 2005. On July 24 and 25, 2005 and August 3, 2005 a formal detailed visual inspection of the dam was conducted. The weather during the detailed visual inspection was sunny and warm, and temperatures ranged in the upper eighties to the low nineties. The precipitation during the spring and summer seasons had been close to average amounts of seasonal rainfall. The current reservoir level at the time of inspection was effectively empty with only a shallow channel flowing across the bottom of the lake bed to the 30-inch RCP intake pipe (invert elevation at about 432.8 feet) at the service spillway. Prior to the failure, the reservoir level typically overtopped the service spillway drop inlet structure at an elevation of 451.95 feet.

During the inspection, the recently constructed ogee spillway (part of the multi-stage auxiliary spillway system), with a crest elevation of 452.55 feet, was observed to have a three inch high water mark above the ogee crest on the training walls. The concrete ogee spillway is located approximately at the center of the dam. The fuse plug spillways and breach area are located west of the ogee spillway near the west abutment as shown on Figure 2, Topographic Survey.

9.1 Embankments, Abutments, Foundation and Downstream Channel

The inspection began by walking east to west on the crest of the dam along the older section of the

dam earth embankment (see photo 1); no signs of distress were evident until approximately the area where the old embankment section adjoins the east side of the newly constructed embankment section. At a distance of approximately 60 feet from the east training wall of the new ogee spillway, there was a subsidence that extended from the crest wall down the upstream face of the dam, causing a channel depression across the shoreline riprap (see photos 2 and 3). This subsidence has been extensively investigated and the results of this investigation are presented in Section 5.0. The presence of this subsidence prompted a detailed inspection of the downstream embankment slope for evidence of soil loss by seepage discharge. From the east training wall of the ogee spillway eastward along the toe of the embankment slope to the RCP outlet into the beaver pond, as well as up on the downstream slope of the embankment (see photos 4 and 5), several points of apparent seepage discharge were observed, consisting of gray fine sand, silt and clay.

West of the ogee spillway, fuse plug No. 1 is intact. At fuse plug No. 2, the east limit of the dam breach is defined with the collapse of the roller compacted concrete (RCC) at approximately mid-span of the west fuse plug (see photo 6), as well as exposure and bending of the sheet pile cutoff wall beneath the RCC. West beyond the limits of the collapsed fuse plug No. 2 is the scour hole (see Figure 2 and photo 7), which extends 40 to 50 feet from the collapsed RCC and sheeting to the west face of the dam breach. The west and the east faces of the breach area are discussed in Section 7.0.

Beyond the west limit of the breach, the remaining dam embankment consists mainly of the older section of the dam which shows no signs of distress (see photo 8). The contact zone between the old and new sections of the embankment was destroyed during the dam breach.

In general, the undisturbed segments of dam embankment consist of an upstream slope (3 horizontal to 1 vertical) with rip-rap armoring along the length of the dam, approximately 15 feet wide. Grasses cover the upstream embankment above the rip-rap line. The downstream slope (2 horizontal to 1 vertical) consists of grass and brush with a predominance of cobbles and boulders (see photo 9) at the ground surface about halfway down the slope to the toe of the embankment. The cobbles and boulders along the old embankment section are part of the older rock filled structure. Brush and

small tree growth was observed for about 100 feet east of the new construction, and the remaining downstream side of the east embankment included older growth with large trees. Remnants of the downstream slope at the west abutment had significant overgrowth with trees. Seepage through the embankment was not observed, but given that the water level in the lake was lower than the upstream embankment toe, no seepage was expected. Probing the embankment for soft areas was not performed given the rocky nature of the embankment fill and absence of seepage across the dam. No piezometers were found and no water levels were taken within the embankment.

The crest of the dam is typically 15 to 20 feet wide and vegetated with grasses. The crest from the east provides a dirt access road to the service spillway and the new ogee spillway, and the crest from the west provides access to the dam breach. Running west of the west breach face and running east of the ogee spillway is a crest wall which was installed as part of the recent dam construction. The crest wall is composed of pre-cast concrete sections. At the joint between each pre-cast section, there are small depressions in the ground surface, which would indicate that fill soil is being lost into the joint below grade by localized erosion. On August 3, 2005, excavation along the two joints 50 feet from the ogee spillway revealed that joint filler does not extend below grade and that the joint had filled with soil.

Foundation soil, as observed from the lake bed, consists of silty soils with varying amounts of fine sand (see photo 10). The soils appear to be glacial lake deposits and not glacial till. A series of soil test borings and test trenches were advanced to investigate and characterize the foundation soils. This data is presented in Section 3.0 and Section 4.0 of this report.

The downstream conditions consist of a beaver pond that functions as a stilling basin leading to a stream channel. Significant quantities of embankment and dam foundation soils have been deposited immediately downstream of the dam, west of the beaver pond. A debris pile consisting of boulder and cobbles, timber, RCC and pre-cast concrete has been deposited nearest to the dam (see photo 11). Beyond the debris piles are deposits of sands and silt that reach into the wetlands south of the beaver pond (see photo 12).

9.2 Multi-stage Auxiliary Spillway System

The Hadlock Pond Dam multi-stage auxiliary spillway system consisted of three segments: an ogee spillway at elevation 452.55 feet, and two fuse plug spillways constructed of erodible soil above a RCC base (see photo 13). Fuse plug No. 1 and No. 2 have crest elevations of 456.75 and 457.75 feet, respectively. The downstream face of the ogee spillway has an ogee shape and stilling basin which promotes efficient flow over the dam. This surface is in a relatively unused condition and there are no significant areas of weathering, spalling or cracking of the concrete. The observed high water mark within the ogee spillway was about elevation 452.7 feet (see photo 14).

9.3 Service Spillway

The service spillway consists of a 30-inch diameter RCP intake (see photo 15) with an invert elevation of 432.8 feet, which connects to the drop inlet structure (see photo 16) that has a crest elevation of 451.95 feet. In addition to the intake pipe, there is a winter reservoir drain consisting of a 24-inch diameter ductile iron pipe (see photo 17) that connects to the drop inlet at elevation 446.4 feet. Flow out of the drop inlet is conveyed through the dam embankment by a 48-inch diameter RCP to the outlet on the north shore of the beaver pond at elevation 430.3 feet.

The outlet works observed above grade, which consist of a manually operated gate valve, are in good condition. A Town official (Eyewitness #5) reported during an on-site conversation that the valve is in good working condition. The catwalk and platform above the inlet structure are in good operational condition and have been secured with gating and locks.

10.0 DISCUSSION AND EVALUATION OF TYPES OF INCIDENTS LEADING TO EARTHEN DAM FAILURES

This discussion, regarding the failure of Hadlock Pond Dam, will describe potential types of incidents that could lead to accidents or failures. Dam failures are defined as a breach or uncontrolled releases of the impoundments; whereas, dam accidents do not result in a total failure of the structure, but do require that timely repairs be made. Under the above definitions, the Hadlock Pond Dam incident is categorized as a dam failure. Earthen dams can experience a number of incidents that could lead to noticeable distress on or around the embankment or that could cause an uncontrolled release of the impounded water.

This section will discuss and evaluate the following six types of incidents that could cause an accident or failure of an earthen dam:

- overtopping of the dam by the impounded waters;
- settlement of the dam embankment;
- slope instability of the dam embankment;
- occurrence of a seismic event which could lead to a weakening of the dam or the foundation soils;
- vandalism which could cause degradation of the dam, and;
- piping or internal erosion.

The sub-sections below will briefly describe the above types of incidents and the associated factors, as well as evaluating whether or not the type of incident was a potential cause of the failure of Hadlock Pond Dam.

A number of studies have been conducted in the past to summarize types of incidents leading to failures or accidents for all types of dams. The publication by the National Academy of Sciences (1983), provides a summary of two separate surveys conducted by the International Commission on Large Dams (ICOLD) and the United States Committee on Large Dams (USCOLD) for incidents

occurring on large dams (greater then 50 feet in height).

This summary indicated that of earthen dams that were reported to have failed, excluding overtopping as a failure mode, at least 25% of dam failures occurred during first filling. Although Hadlock Pond Dam does not meet the definition of a large dam used during the ICOLD and USCOLD surveys, it is an earthen dam susceptible to the failure modes discussed in the surveys. Given the statistic cited above, regular observations during first filling may have allowed for identification of symptoms of potential latent defects within the dam at a critical time in its service. Review of the available documentation did not indicate that regular observations occurred during the first filling of Hadlock Pond.

10.1 Overtopping

Overtopping of an earthen dam will occur if the impounded water reaches a level that is higher than the dam crest elevation. Once water begins to flow over the crest of the dam, it will begin to erode the embankment material which could develop a breach in the dam. This type of incident could lead to an accident, or it could lead to a failure if the water level was not reduced by some other means.

The following points are highlighted from documentation produced by HTE and information learned during the field investigation:

1) The construction drawing for Hadlock Pond Dam, *Existing and Proposed Conditions Plan* by HTE dated July 1, 2003, indicates that the lowest elevation along the crest of the dam embankment (fuse plug No. 1 pilot channel) was designed at elevation 456.8 feet (based on NAVD88) or elevation 100 feet (referenced to an assumed datum on the design drawings); the crest of the ogee spillway was designed at elevation 452.6 feet (95.8 feet on the drawing); and the top of the embankment crest at the location of the breach was designed at an approximate elevation of 458.8 feet (102 feet on the drawing).

2) The survey conducted by CHA, as shown on Figure 2, indicates that the fuse plug No. 1 pilot channel elevation was at 457 feet, the ogee spillway crest elevation was at 452.55 feet and the embankment crest at the west edge of the breach was at elevation 458.5 feet.
3) During the detailed visual inspection of Hadlock Pond Dam, the highest water mark observed on the ogee spillway training wall was about elevation 452.7 feet (see photo 14 in Appendix G).
4) Eyewitness #5 indicated in an interview (see Section 8.5) that on the day of the failure, at about 4 p.m., he noted the water going over the ogee spillway crest at a height of approximately two to three inches above the crest. Eyewitness #5 also indicated that he had not observed the water level at the ogee spillway any higher than this since the filling of the pond began.

Based on the four points listed above, on July 2, 2005 the elevation of the water impounded behind Hadlock Pond Dam did not attain an elevation higher than elevation 452.7 feet, which was 4.3 feet and 5.8 feet below fuse plug No. 1 pilot channel elevation and the top of embankment crest respectively. Since the impounded water did not reach an elevation that would allow it to overtop the embankment crest at the location of the breach, overtopping is not supported as an incident leading to failure of the dam.

10.2 Settlement of the Dam Embankment

Settlement of the dam embankment could occur by two methods: bearing capacity failure or consolidation of underlying soils. Dam embankment settlement could result in sloughing of the embankment slopes as well as differential settlement along the crest. This movement could develop cracks within the soil mass that could lead to internal erosion.

10.2.1 Bearing Capacity Failure

A bearing capacity failure of the foundation soils would occur if the bearing pressure at the bottom of the earth embankment exceeded the ultimate bearing capacity. A bearing capacity failure would

cause horizontal and vertical movement of the earth embankment, which could result in cracks within the dam and a lowering of the crest. This type of distress would be readily observable by looking along the crest of the dam.

Based on the information produced by HTE, a review of the general history of the dam, and geotechnical experience of CHA engineers, the following points are highlighted:

1) The *Topographic Map of Existing Conditions* by W.J. Rourke Associates dated January 26, 2000; and the *As-Built Plan Showing the Post-Existing Conditions of the Hadlock Pond Dam* by W.J. Rourke Associates dated May 9, 2005 show minimal change between the pre-construction and post-construction embankment grades along the dam, and specifically at the breach location.
2) The *As-Built Plan Showing the Post-Existing Conditions of the Hadlock Pond Dam* by W.J. Rourke Associates dated May 9, 2005 indicates an average embankment crest elevation west of the RCC abutment of 458.8 feet (102 feet with respect to HTE drawings). HTE Project Status Report No. 25 dated January 11, 2005 also indicates that the bottom of the RCC base extended to about elevation 437.8 feet (81 feet with respect to HTE drawings). In addition, a review of construction photos (see Appendix H) indicates that excavation west of the RCC abutment did not extend below the bottom of the RCC base. The embankment height west of the west RCC abutment is therefore estimated to be 21 feet.
3) Based on field density testing (see Section 5.3) and laboratory analysis of the Modified Proctor densities (see CME Associates, Inc. reports in Appendix E) of the material observed in the old and new embankment fill zones along the east contact zone of the dam during excavation of the subsidence test trench on August 3 and 4, 2005, and review of the Modified Proctor test results and field density reports produced by Atlantic Testing Laboratories during construction, the average unit weight of each of the old and the new embankment fill material is conservatively estimated to be 140 pounds per cubic foot (pcf).

4) The foundation soils generally consist of silt with trace to some fine to medium sand. These soils have a compactness that ranges from loose to medium compact based on the N values obtained during the subsurface investigation, which indicates allowable bearing capacity of the foundation soils in the range of 1.5 tons per square foot (TSF) to 2.0 TSF (NAVFAC, DM-7.2, 1986).
5) The construction drawing, *Fuse Plug Sections and Details* by HTE dated revision March 29, 2005, indicates that the RCC base extends from elevation 446.8 feet (90 feet with respect to HTE drawings) to 437.8 feet (81 feet with respect to HTE drawings); these elevations were confirmed through review of the Project Status Reports produced by HTE. Above the RCC base, the fuse plug fill material extends up to an average elevation of 457.8 feet (101 feet with respect to HTE drawings). Based on this information, the RCC base is estimated at nine feet thick, and the fuse plug fill material is estimated at 11 feet thick.
6) Based on review of the Modified Proctor test results as well as field density reports produced by Atlantic Testing Laboratories during construction of the RCC, the unit weight of the RCC is estimated to be 150 pcf.
7) Above the RCC base, the fuse plug fill material consisted of Glens Falls Cement Company overburden pit sand and gravel, Peckham clay, and Shimmerhorn Pit sand and gravel. Based on review of the Modified Proctor test results as well as the field density reports produced by Atlantic Testing Laboratories during construction for each of these materials, the aggregate unit weight for all three materials is conservatively estimated to be 140 pcf.

Points 1) through 4) indicate that the bearing pressure from the pre-construction and post-construction dam embankment west of the west RCC abutment did not exceed the allowable bearing capacity of the foundation soils. Assuming a pre-construction and post-construction embankment height of 21 feet and similar embankment geometry, and a conservatively estimated unit weight for each of the old and new embankment fill materials of 140 pcf, the approximate bearing pressure on the foundation soils in the area of the breach was 1.5 TSF, which is equal to the lowest allowable bearing capacity (1.5 TSF) of the foundation soils.

At the fuse plug location, the bearing pressure might be anticipated to be higher than at the embankment due to the presence of RCC; however, points 5) through 7) indicate that the approximate post-construction bearing pressure on the foundation soils at the bottom of the RCC base was also 1.5 TSF. This is based on a nine foot thick RCC base with a unit weight of 150 pcf and fuse plug embankment material thickness of 11 feet with an aggregate unit weight of 140 pcf.

Since the foundation soils have been subjected to minimal change in pre-construction and post-construction embankment bearing pressure, the foundation soils have adequate bearing capacity to support the earth embankment, and no observations were made prior to the dam failure of any sloughing or settlement along the crest of the dam, bearing capacity failure is not supported as an incident leading to failure of the dam.

10.2.2 Consolidation of the Underlying Foundation Soils

Consolidation of the underlying foundation soils would occur if there were a material increase in the net pressure of the post-construction embankment relative to the pre-construction embankment (i.e., a difference between the weight of the post-construction embankment and the pre-construction embankment). An increase in the net pressure at the surface of the foundation soils would cause an increase in pressure at a given depth within the underlying soils. The increased pressure would result in consolidation of the foundation soils and a subsequent settlement of the embankment. The consolidation would occur rapidly during construction (within a few weeks) if the foundation soils have a high permeability (e.g., loose sands or gravels), or the consolidation would occur over a longer period after construction (three months to several years) if the foundation soils have a low permeability (e.g., silts or clays). Settlement of the embankment would cause mostly downward vertical movement, which could result in cracks within the dam and a lowering of the crest. This type of distress would be readily observable by looking along the crest of the dam.

Based on information produced by HTE, a review of the general history of the dam, and geotechnical experience of CHA engineers, the following points are highlighted:

1) The foundation soils generally consist of silt with trace to some fine to medium sand. Undisturbed samples were taken within the foundation soils near the location of the failure and were measured for permeability; the results are included behind the individual soil boring logs in Appendix B. In soil boring B-3 sample ST-1 the permeability was measured at 3.1×10^{-7} centimeters per second (cm/sec), and in soil boring B-2 sample ST-1 the permeability was measured at 4.2×10^{-6} cm/sec. These values would indicate that this material would take no less than three months and more likely six to 12 months to consolidate if the net load upon the foundation soils was increased.

2) The *Topographic Map of Existing Conditions* by W.J. Rourke Associates dated January 26, 2000, and the *As-Built Plan Showing the Post-Existing Conditions of the Hadlock Pond Dam* by W.J. Rourke Associates dated May 9, 2005 show minimal change between the pre-construction and post-construction embankment grades along the dam, and specifically at the breach location.

3) As stated in Section 10.2.1 of this report, the approximate bearing pressure on the foundation soils from each of the pre-construction and post-construction embankment fill materials was 1.5 TSF.

4) According to *Plan and Sections, Timber Dam at Hadlock Pond* dated July 29, 1924, the dam configuration near the breach location consisted of a 20 foot high and 15 foot wide rock-filled timber crib with a 10 foot high "stone backing" on the downstream side of the timber crib and a 23 foot high "earth fill" embankment on the upstream side of the timber crib. The dam was raised in 1933 with the addition of soil and rock fill to the upstream and downstream sides, as well as above the rock filled timber crib structure. The configuration or geometry of the dam embankment near the breach location did not change significantly between 1933 and the pre-construction time period; therefore, this indicates that the foundation soils below Hadlock Pond Dam had experienced pre-construction loads for many years, which would have allowed for full consolidation to have occurred based on that loading.

5) Section 10.2.1 illustrates that the bearing pressure at the bottom of the RCC base and the post-construction embankment soils west of the west RCC abutment were both approximately 1.5 TSF, which indicates an equivalent loading of the foundation soils near the breach location.

Point 1) indicates that the time required for the silt foundation soils to consolidate significantly enough to generate cracking in the embankment from the post-construction embankment load would be at least three months and more likely six to 12 months. However, based on points 2) and 3) it appears that consolidation of the foundation soils would not have occurred, since the pre-construction and post-construction loads between the ogee spillway and the west earth abutment would have been similar. In addition, point 4) indicates that the foundation soils were fully consolidated due to the many years of pre-construction embankment loads prior to construction in September, 2004.

Point 5) indicates that differential settlement between the fuse plug spillway and the earth embankment west of the west RCC abutment would not have occurred, since the post-construction loads imparted to the foundation soils were equivalent.

Another event that may have induced settlement of the foundation soils below the RCC base was the driving of fuse plug cut-off sheet piles. Due to the low permeability of the silt foundation soils (see point 1 above) and their susceptibility to liquefaction (see Section 10.4), driving of sheet piles could induce liquefaction in the silt soils. Liquefaction of the silt foundation soils during the driving operations would only occur within about three feet on either side of the sheet pile (NHI, 1998). An adverse result of liquefaction would be consolidation of the soil matrix exposed to the liquefaction. Assuming that the silt foundation soils experienced liquefaction and consolidated, this condition would only have occurred along an approximately six foot wide area around the fuse plug cut-off sheet piles. Due to the limited area of potential settlement of the foundation soils during pile driving operations below the RCC base, settlement of the RCC base is not likely to have occurred.

Based on the information above and that no observations were made prior to the dam failure of any sloughing or settlement along the crest of the dam, consolidation of the silt foundation soils is not

supported as an incident leading to failure of the dam.

10.3 Slope Instability of the Dam Embankment

Slopes maintain stability when there is a balance between the resisting and driving forces within the slope, or when the resisting forces are greater than the driving forces. An earthen dam embankment consists of an upstream and a downstream slope. Either of these slopes could become unstable (i.e., unbalanced) based on one or a combination of the following reasons:

- removal of material from the toe of the embankment or addition of a surcharge load at the top of the embankment;
- a slope was constructed too steeply;
- embankment material was not placed with enough compaction to develop the necessary internal strength to resist the driving forces; or
- a saturated soil condition developed that would reduce the internal strength of the embankment material, resulting in a reduction of the resisting force.

Slope instability would cause sloughing down the face of the slope, lowering of the crest, or tension cracks in the embankment. This type of distress would be readily observable by looking at the downstream slope of the embankment, where this type of distress is most likely to occur, or along the crest of the dam.

Based on information produced by HTE, Engineering Ventures, Inc., and Copeland Environmental, and stability evaluations performed by CHA, the following points are highlighted:

1) Based on *Mitigation Plan for Wetland Disturbance* by Engineering Ventures, Inc. dated July 1, 2002 and review of the daily reports produced by Copeland Environmental, the wetland mitigation consisted of excavations, which extended no deeper than three feet, to create pocket wetlands at the base of the fuse plug multi-stage auxiliary spillway.

2) Based on review of the available information, no other excavations at the base of the downstream embankment slope were known to have occurred.

3) After review of the available documentation, there is no indication that a significant surcharge load (i.e., a large construction service load or soil stock pile) was placed at the top of the embankment crest between the apparent end of the construction (as noted in HTE Project Status Report No. 33, dated May 2, 2005) and the day of the failure (July 2, 2005).

4) The construction drawing, *Existing and Proposed Conditions Plan* by HTE dated July 1, 2003 indicates that the downstream embankment slope was designed at 2 horizontal to 1 vertical (2H:1V). The upstream embankment slope was designed at 1.5H:1V from embankment crest elevation of 458.8 feet (102 feet on the drawings) to elevation 450.8 feet (94 feet on the drawings), changing to a 3H:1V at elevation 450.8 feet (94 feet on the drawings) to elevation 441.8 feet (85 feet on the drawings), and then grading relatively flat into the pond (see Figure 10 in Appendix A). This was confirmed after review of the *As-Built Plan Showing the Post-Existing Conditions of the Hadlock Pond Dam* by W.J. Rourke Associates dated May 9, 2005. This information identifies the geometry of the embankment slope at the location of the breach.

5) Based on review of the Modified Proctor test results as well as the field density reports produced by Atlantic Testing Laboratories during construction, both Shimmerhorn Pit sand and gravel and Glens Falls Cement Company overburden pit sand and gravel were placed in the embankment west of the west RCC abutment, and were compacted to 95% of the maximum dry density as determined by the Modified Proctor test (ASTM D 1557). This information aids in identifying the internal strength of the embankment material.

The removal of material at the downstream toe of the embankment during the wetlands creation, as indicated in point 1), was limited to less than three feet in depth; in addition, this work was of limited extent and not at a location which would create instability in the slope. Furthermore, point 2) indicates that removal of material from the toe of the embankment west of the RCC abutment did not appear to have occurred. Therefore, instability in the embankment slope west of the RCC abutment would not have occurred by means of removal of material from the toe.

Point 3) indicates that a surcharge load was not placed at the top of the dam embankment. Therefore, instability in the slope would not have occurred by means of excessive surcharge loading.

Based on points 4) and 5), which define the geometry of the embankment slopes and the compaction effort applied to the embankment material to achieve internal strength, our stability analysis of the upstream and downstream embankment slopes under a saturated dam embankment condition indicates the slopes met applicable stability criteria.

Another event that could lead to slope instability is rapid drawdown of the impoundment. This event was not considered as a likely cause, since there were no reported incidents of rapid drawdown between when filling of the pond began and the date of failure, and since the release of the impoundment, which would create a rapid drawdown condition, was a result of the failure.

Since the factors that could lead to slope instability were not evident at the crest or toe of the post-construction embankment, the stability analysis indicated that the post-construction embankment slopes met applicable stability criteria, and sloughing along the downstream slope of the embankment or tension cracks along the crest of the dam were not observed prior to the failure, slope instability failure is not supported as an incident leading to failure of the dam.

10.4 Seismic Event

A seismic event could cause damage to a dam embankment through two mechanisms: (1) liquefaction of the foundation soils, resulting in consolidation of the foundation soils and subsequent settlement of the embankment; or (2) shifting along a fault line, potentially resulting in a transverse crack across the embankment. McCook (2004), pp. 46, states that, “most cases of embankment failure caused by seismicity are related to liquefaction of saturated, cohesionless material in the foundation or embankment. Major deformation of the dam from fault movement is an exception to this general rule.” This type of incident could result in either an accident or failure, depending on the magnitude of the seismic event.

The field investigation revealed that the foundation soils generally consist of wet to saturated non-plastic silt with trace to some fine to medium sand. Based on the N values obtained during the subsurface investigation, the foundation soil's compactness ranges from loose to medium compact. These soils are susceptible to liquefaction based on Polito and Martin (2001). During the field investigation no evidence of liquefaction of the silt foundation soils was observed.

Seismic data available from the Lamont – Doherty Cooperative Seismographic Network (LCSN), a group which monitors earthquakes that occur primarily in the eastern United States of America, was reviewed to determine if any seismic activity was present around the time of the dam failure. The nearest recording station monitored by the network is located in Glens Falls, New York and operated by Adirondack Community College. The most recent local earthquake prior to the failure, recorded by LCSN, was a 2.2 (M_c) magnitude earthquake 15 miles southwest of Plattsburgh, New York at 7:06 AM (EST) on July 1, 2005. Based on the magnitude and distance of this earthquake from the Hadlock Pond Dam, the accelerations generated by the earthquake would have dissipated to very low levels prior to reaching the dam. This is substantiated by the fact that there were no known reports of earth movement by residents in the area of the dam. Furthermore, a review of the paper by Youd (1998) indicates that the potential for liquefaction of soft soils (i.e., the silt foundation soils) for magnitudes less than 5.2 is negligible. It is noted that seismic activity recorded by the LCSN on July 2, 2005 (i.e., day of the failure) was attributed to a seismic event off the coast of Nicaragua.

Movement along a fault line was also investigated. Review of Plate 1, from "Bedrock Geology of Glens Falls – Whitehall Region, New York" (Fisher, 1985) included as Figure 7 in Appendix A indicates the presence of two faults in the area of Hadlock Pond Dam. The Hadlock Pond fault is oriented southwest to northeast along the southeast side of Hadlock Pond. The Copeland Pond fault is oriented southeast to northwest, roughly 16 miles east of the east side of the dam and bisects the Hadlock Pond fault east of the north end of the pond. The Hadlock Pond fault is considered "inactive" according to Robert Fickies of New York State Museum Geological Research. In addition, no evidence of movement along the Hadlock Pond fault was observed during the visual

inspections by CHA during July and August, 2005, or during a subsequent field investigation along the fault alignment by a CHA geologist on September 19, 2005.

Based on information obtained from the field investigation, examination of geological and seismic data and a review of literature, a seismic event is not supported as an incident leading to failure of the dam.

10.5 Vandalism

Vandalism was considered as a potential incident that could lead to dam failure. Understanding that the Hadlock Pond dam is an earthen embankment dam and the area of the breach was composed of rock fill, roller compacted concrete, dense compacted sand and silty sand backfill soils, explosives and/or heavy construction equipment would have been necessary to place a rift in the embankment large enough to cause a failure similar to that observed at Hadlock Pond Dam. NYS DEC Investigator Fred Stannard considered the potential for vandalism as a cause of the dam failure and, based on his 31 years of experience as a police officer, concluded that there is a very low probability that any vandalism occurred that would have caused the breach in the embankment. Investigator Stannard found that the fencing restricting access on the beach side of the dam was intact and secure, and that there were no complaints or reports of vandalism or explosions. It is also Investigator Stannard's opinion that given the time of day, late afternoon and early evening on a Saturday, and the considerable activity on and around Hadlock Pond, that the possibility of vandalism going unnoticed is very remote. Based on this information, vandalism is not supported as an incident leading to failure of the dam.

10.6 Piping and Internal Erosion

Dam failures or accidents caused by uncontrolled flow of water through or under an earth embankment may be categorized as either piping or internal erosion incidents. In our evaluation, the reference to piping or internal erosion follows the definitions presented in a paper by McCook

(2004). Piping is defined as flow developing solely within and propagating through a single erodible soil unit, whereas internal erosion is defined as flow seeking and eroding a preferential flow path within or below the dam. Both of these modes of failure have the common process of soil removal from within the embankment, and, if not corrected, will accelerate over time and ultimately lead to the collapse of the earth embankment structure. These two mechanisms are discussed in detail in the following subsections as they pertain to the specific site conditions at Hadlock Pond Dam.

10.6.1 Piping

Piping initiates the soil removal process through inter-granular flow within the soil mass as the discharge hydraulic gradient across the dam exceeds the critical hydraulic gradient of the soil (i.e., individual soil particles become buoyant) and the soil particles nearest the point of discharge move out of the soil structure. This process progresses from the downstream point of discharge toward the reservoir side of the embankment, creating a pipe-shaped tunnel. Cohesionless very fine sand and silt soils offer the least resistance to piping; non-dispersive cohesive clay soils offer the most resistance. Other site conditions necessary for the development of piping include:

- a hydraulic gradient and a soil pore structure that permits inter-granular flow;
- an unprotected exit such as excessively coarse soil or an open point of discharge for soil to escape;
- erodible materials that make up the earth embankment; and
- embankment material, which piping is occurring through, capable of arching and forming a roof or pipe structure.

Upon review of various piping scenarios as presented by McCook (2004) and given our understanding of the Hadlock Pond Dam earth embankment cross section and the underlying stratigraphy and the location of the failure, two scenarios of piping are considered plausible modes of dam failure.

The first scenario is piping of newly placed embankment soils into an overly coarse existing rock-filled embankment. Piping would begin where fine grained non-plastic backfill is in contact with the

rock-filled portion of the old embankment. At this interface, seepage forces could develop as the phreatic surface develops across the dam. The exit gradient from the backfill would, without the presence of an engineered transition material, dislodge fine grain particles which would then flow with seepage into the rock-fill and exit the dam. This process could progress and develop a tunnel back to Hadlock Pond.

Factors supporting piping of newly placed embankment soils into an overly coarse existing rock-filled embankment include:

- the presence of unprotected rock-fill within the old embankment in contact with new backfill soils. Based on observations made by CHA during excavation of the subsidence test trenches (TT-9, TT-10 and TT-11) at the east contact zone, no transition materials between the old and new embankment backfill materials were present.
- the absence of properly graded material between the old and the new embankment materials that satisfies filter criteria. CHA has evaluated the grain size distribution curves for both the Glen Falls Cement Company overburden pit sand and gravel and the Shimmerhorn Pit sand and gravel soils and used this data to compute the criteria for an effective filter as defined in Cedergren (1989). This evaluation found that the D_{50} and D_{15} particle sizes necessary to protect these materials from piping are not characteristic of the rock-fill materials found in the old embankment (see filter criteria illustrated on Figures 11 and 12 in Appendix A).
- the presence of erodible materials in the new embankment backfill as observed during excavation of the subsidence test trenches (TT-9, TT-10 and TT-11) at the east contact zone.
- Eyewitnesses #2 and #3 during interviews on July 27 and August 5, 2005 indicated the presence of turbid seepage flow from both the east and west sides of the spillway prior to the failure date; and
- the presence of fine grain soil deposition along the downstream toe of the dam in the vicinity of the east limits of construction.

Factors not supporting piping of newly placed embankment soils into an overly coarse existing rock-filled embankment include:

- the estimated time period of six months to develop a phreatic surface across the new embankment materials and establish a hydraulic gradient necessary to drive flow through the new embankment soil pore structure. The hydraulic gradient condition for piping failure is not satisfied based on review of the field density reports produced by Atlantic Testing Laboratories and an evaluation by CHA of phreatic surface progression through these new embankment materials. The field density reports indicate compact silty sand and gravel soils, which CHA estimates to have a permeability of 1×10^{-4} cm/sec to 5×10^{-5} cm/sec. The evaluation of phreatic surface progression assumes that the filling of Hadlock Pond began on or about May 2, 2005, when HTE reported (Project Status Report No. 33) that the rehabilitation work was complete, and the estimated range of permeability for the new backfill soils which governs seepage progression. The results of the evaluation indicate that the development of the phreatic surface and hydraulic gradient necessary to drive flow through the new embankment soil pore structure would take at least six months after the filling began.
- the absence of backfill soils capable of a full soil arch necessary to create a tunnel for flow. The earth embankment subsidence observed east of the ogee spillway is evidence of soil collapse into a void (see photos 4 and 5 in Appendix E).

Based on the factors listed above, which do not support piping of newly placed embankment soils into an overly coarse existing rock-filled embankment, this scenario of piping is not supported as an incident leading to failure of the dam.

A second scenario for piping failure would be piping through foundation soils beneath the dam. This process would involve excessive discharge gradients at the toe of the slope resulting in the dislodging of subgrade soils at the ground surface or into the rock-fill portion of the embankment. As the subgrade soils are removed, the piping would progress back under the earth embankment to Hadlock Pond. As the piping beneath the dam develops back towards the pond, the exit gradients

would increase and accelerate the rate of soil loss.

Factors supporting piping through foundation soils beneath the dam include:

- the presence of unprotected exits of seepage flow into rock-filled portions of the earth embankment or the ground surface.
- the presence of subgrade soils that are susceptible to piping. Borings and laboratory testing show the subgrade soils are primarily non-plastic to very low plasticity silts with trace to some amounts of sand, which are susceptible to piping.
- the presence of soil stratification that may support the development of tunnel structure. Observations of the limits of the scour hole and lake bed show that the soil can form cavities.

Factors not supporting piping through foundation soils beneath the dam include:

- the absence of an exit gradient across the dam sufficient to develop flow that would mobilize soil particle movement within the inter-granular pore space of the undisturbed subgrade soils. The long history of the dam empirically demonstrates that piping within the undisturbed subgrade soils has not been an issue. The dissipation of hydraulic head within the subgrade soils can be attributed to the low to very low permeability, as evaluated by testing undisturbed samples taken during the geotechnical investigation (see Appendix B for the laboratory testing results and point 1 in Section 10.2.2). Within the failure area, review of design and construction records indicates that only steel sheeting penetrated the subgrade soils. This penetration appears to have been limited to several feet and would not alter flow conditions in the subgrade soils in a manner that would make the subgrade soils more susceptible to piping.

Based on the factors listed above, which do not support piping through foundation soils beneath the dam, this scenario of piping is not supported as an incident leading to failure of the dam.

10.6.2 Internal Erosion

The second mode of soil loss is internal erosion. This process removes soil particles by seepage flow through a preferential path within the soil mass comprising the embankment. Internal erosion develops when a defect, discontinuity or crack extends though the embankment to a point of discharge. Internal erosion occurs when the velocity of flow is sufficient to erode the sides of the defect and transport the soil particles, causing the defect to enlarge. This is in contrast to piping where flow occurs through pore space of intact soil mass which has no internal discontinuities. Upon filling of the reservoir, seepage flow develops within the preferential pathway and soil loss begins along the walls of the preferential flow path, enlarging the defect. The expanding defect progresses back toward the reservoir (i.e., Hadlock Pond), which is the source of flow. Internal erosion incidents are much more common than piping incidents and are commonly misidentified as piping since the erosion channel through the embankment has a tunnel or pipe-shaped appearance.

Site conditions necessary for the development of internal erosion include:

- a hydraulic gradient or head generating a flow. This gradient or head does not need to be large, since seepage flow is achieved when a defect is in hydraulic connection with the water source;
- an unprotected exit such as excessively coarse soil or an open point of discharge for soil to escape;
- erodible materials that make up the earth embankment; and
- internal cracks, defects or discontinuities within the embankment which are exposed to water flow from the reservoir.

The first three site conditions were present at Hadlock Pond as discussed in Section 10.6.1. The role of the fourth site condition, which includes internal cracks, defects or discontinuities and are collectively referred as preferential pathways, is discussed further in the following paragraphs. Preferential pathways can occur in earth embankments for a variety of reasons. Some of the more common conditions leading to preferential pathways include:

- hydraulic fracturing, differential settlement and drying cracks that occur during the interruption of fill placement;
- different compaction conditions for soils placed around conveyance pipes or above excessively coarse fills;
- poor bonding at the contact between embankment materials and a structure that extends into or through the embankment;
- soft wet backfill or subgrade soils resulting from inclusion of frozen materials in backfill or burying of frozen subgrade; and
- defects or irregularities in bedrock surfaces (bedrock was not encountered during subsurface investigations at Hadlock Pond Dam).

Upon review of various internal erosion scenarios presented in the paper by McCook (2004), and given our understanding of the Hadlock Pond Dam earth embankment cross section, the underlying stratigraphy and the location of the failure, two scenarios of internal erosion are considered plausible modes of dam failure.

The first is internal preferential pathways within the earth embankment extending completely through the dam. Such a mode of failure was apparently underway at the new to old embankment contact zone east of the ogee spillway construction, since there was an area of subsidence and soft soil observed at the inclined contact between the old embankment and the newly constructed embankment. At this location it appears that compaction conditions were not satisfactory, resulting in poor bonding between the old and new embankment materials. There was also the lack of transition materials to protect erodible soils from migrating into the rock fill portion of the old dam. Based on construction photographs and observations made during the excavation of the subsidence area at the east contact zone, oversized materials were present at this interface, which inhibited proper compaction of backfill lifts at the contact with old embankment fills (photo 10 in Appendix E shows the inclined contact surface consisting of oversized materials). Further excavation along the contact zone and into the old embankment uncovered nested cobbles (i.e., an erosion channel) as shown in Photos 16 and 19 of Appendix E. The poorly compacted new embankment soils directly

above the nested cobbles appear to have provided a preferential flow path for internal erosion of embankment soils. Seepage through this zone appears to have discharged into the rock-filled portion of the dam embankment, resulting in gray fine sand and silt deposition along the downstream toe of the dam.

The conditions leading to this failure scenario on the east side (i.e., the old to new contact has oversized material, an absence of transition materials, and the presence of erodible backfill soils) can be presumed to have been present on the west limit of construction, with the difference being that the progression to failure was more rapid on the west side. This rapid progression could have resulted in a breach of the dam that would wash away evidence of the failure progression that is evident at the east interface. Although this mode of failure is plausible, it is hypothesized that this process would result in a top-down breach of the dam, given the observations on the east side. A top-down failure is inconsistent with eyewitness accounts of the west side embankment collapse, which indicate that the breach began at or near the base of the embankment and progressed backwards and up to the crest wall in an inverted V-shape. A bottom-up failure progression could, however, have developed under this scenario if the loose interface soils which develop the internal erosion channel were located near the base of the new to old embankment contact surface. The potential for this bottom-up dam embankment failure condition is possible, given the presence of loose, disturbed, wet and muddy embankment material described in HTE Project Status Reports No. 28 through 33, documenting construction from March 28, 2005 through May 2, 2005.

The second internal erosion scenario meets the basic criteria outlined above for this soil removal mechanism, with the difference being that the preferential path develops along a structure that extends through the earth embankment. Under this scenario the seepage flow would develop along the contact between the new embankment backfill soils and the west RCC abutment wall or vertical section of the RCC base. Based on past experience with dam and other earthwork projects, the soil in contact with the RCC abutment wall or base could be loose due to the difficulty in compacting against this observed vertical irregular surface. This vertical irregular surface appears to have existed along the west RCC base, upstream to downstream, between elevations 441.8 feet (85 feet on

the drawings) and 437.8 feet (81 feet on the drawings), with no protrusion of the RCC cut-off wall steel sheeting beyond the vertical irregular surface. This information is based on the construction drawing, *Fuse Plug Sections and Details* by HTE revision dated March 29, 2005, and HTE Project Status Reports No. 22 through 27, documenting construction from January 5, 2005 through January 12, 2005. In addition, separation could develop between the RCC and the soil as these two materials undergo different strain responses under the hydraulic load imparted during the filling of Hadlock Pond.

Review of construction documentation also identified the potential for preferential flow path development along the steel sheeting and beneath the construction access road, presuming that this road was built by placing a granular road base. Soil boring B-6A shows that a loose to medium compact sand with little gravel and silt was present north of a segment of the steel sheeting (the RCC cut-off wall) immediately west of the ogee spillway. This boring indicates that the sheeting at the eastern end of the RCC cut-off wall extended approximately three feet below the pervious fill into the silt foundation soils. Considering the varying types of backfill soils, steel and concrete dam components, and the different methods of construction, the actual preferential flow path could have been a complex circuitous path involving multiple interfaces. Observations during the visual inspections of sediment deposition at the toe of the embankment on the east side of the ogee spillway and eyewitness accounts support that the pathway, however complex, ultimately links seepage flow to the rock-fill portion of the old embankment that was left in place (see Generalized Dam Profile and Geologic Cross Section, Figure 8; Generalized Dam Cross Section at Subsidence Location, Figure 9; and Generalized Dam Cross Section at Breach Location, Figure 10 included in Appendix A).

Based on the discussion presented above, the conditions necessary to develop internal erosion were present within the dam prior to the date of failure; therefore, internal erosion is supported as an incident that lead to the failure of the dam.

10.7 Conclusion

Six modes of failure that were considered plausible have been evaluated in this report. Given our review of both the old and new dam designs, construction photographs, construction practices used in the new multi-stage spillway and new embankment construction, interviews with eyewitnesses, and the evidence observed at the east contact zone between the old and new embankment materials, it is our opinion that the Hadlock Pond Dam failed due to internal erosion through or beneath the dam.

The definitive preferential pathway cannot be identified at the breach location, but based on the observations and data obtained from the field investigations and review of available information, it is reasonable to conclude that internal erosion occurred along one or more of the following potential preferential pathways:

- the old to new embankment contact zones where existing oversized rock-fill had not been properly removed from the surface and/or no soil transition material had been placed;
- the contact between loosely compacted new embankment backfill and the vertical west face of the RCC base or abutment wall;
- a disturbed soil zone within the new embankment that formed a preferential pathway through loose soils as a result of saturated or frozen material placement or burial of frozen material on the surface of an embankment lift;
- a disturbed soil zone within the new embankment that formed a preferential pathway through coarse granular materials as a result of access road base aggregates left in place above subgrade soils.

11.0 CLOSURE

The findings of this engineering investigation into the failure of the Hadlock Pond Dam are based on the information gathered during the investigation and an evaluation of available documentation at the time this report was prepared. It is possible that additional information, not available during the investigation, could exist regarding the design and construction that could more definitively identify the specific mechanism leading to failure.

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