02-0603

GE
159 Plastics Avenue
Pittsfield, MA 01201
USA

May 9, 2006

Ms. Dale C. Young
Lead Administrative Trustee
The Commonwealth of Massachusetts -
Executive Office of Environmental Affairs
100 Cambridge Street
Boston, MA 02114-2524

Re: Biennial Structural Integrity Assessment (April 2006)
Quarterly Inspection Report (April 2006)
Woods Pond Dam, Housatonic River, Lee/Lenox, MA

Dear Ms. Young:

On November 29, 2005, GE's consultant MWH Americas, Inc. (MWH) conducted a structural integrity assessment of Woods Pond Dam. While this assessment was not schedule until November 2007, it was performed one year in advance as a precaution due to the high Housatonic River flows in October 2005. The results of this inspection are presented in the enclosed report "Woods Pond Dam: 2005 Structural Integrity Assessment", which was prepared by MWH.

This biennial inspection of Woods Pond Dam is part of GE's overall operation and maintenance program for the dam. This program includes monthly, quarterly and biennial inspections. GE conducts the monthly inspections and the quarterly inspections, which are more detailed than the monthly inspections. The biennial inspections are conducted by a registered professional engineer and assess the structural integrity of the dam. The next biennial inspection is scheduled for November 2009.

Also enclosed is the April 2006 Quarterly Inspection Report.

If you have any questions associated with the Biennial Structural Integrity Assessment or the Quarterly Inspection Report, please contact me at (413) 448-5910.

Very Truly Yours,

Kevin Mooney

Kevin G. Mooney
GE Project Manager

Cc: Kenneth Finkelstein, NOAA/CPRD
Ken Munney, USFWS
Susan Peterson, CTDEP
Susan Svirsky, USEPA
Susan Steenstrup, MADEP
Roderic McLaren, GE*
Andrew T. Silfer, GE*
Michael T. Carroll, GE*
James Bieke, Goodwin Procter*
Laurence S. Kirsch, Goodwin Procter
Sam Gutter, Sidley Austin Brown & Wood*
Mario Finis, MWH

* Without copies

1.0 GENERAL COMMENTS AND SUMMARY OF INSPECTION

Date of Inspection: 4-27-06

Inspection By:
1. SEAN COYLE
2. MARK WASNEWSKY
3. J. LEVESQUE
4. KEVIN MOONEY

Weather: 55 °F SUNNY

Comments:

2.0 CONCRETE OVERFLOW SPILLWAY

Is there any evidence of:

		YES	NO
A.	Discontinuity of smooth spillway overflow?		✓
B.	Accumulation of large debris upstream?		✓
C.	Seepage from face of abutment walls?		✓
D.	Settlement or movement of walls or slabs?		✓
E.	Unusual conditions?		✓
F.	Vandalism?		✓

Comments:

Status of any detrimental conditions, if any, observed during last inspection:

3.0 WEST (RIGHT) ABUTMENT

Is there any evidence of:

		YES	NO
A.	Erosion of material upstream or downstream?		✓
B.	Settlement or cracking of concrete?		✓
C.	Dislocated or missing riprap?		✓
D.	Seepage around the end of the abutment?		✓
E.	Excessive flow downstream of the abutment?		✓
F.	Significant change in wetland area?		✓
G.	Unusual conditions?		✓
H.	Vandalism?		✓

Comments:

Status of any detrimental conditions, if any, observed during last inspection:

4.0 EAST (LEFT) ABUTMENT

Is there any evidence of:

		YES	NO
A.	Erosion of material upstream or downstream?		✓
B.	Settlement or cracking of concrete?		✓
C.	Dislocated or missing riprap?		✓
D.	Seepage around the end of the abutment?		✓
E.	Excessive flow downstream of the abutment?		✓
F.	Unusual conditions?		✓
G.	Vandalism?		✓

Comments:

Status of any detrimental conditions, if any, observed during last inspection:

5.0 RACEWAY CLOSURE STRUCTURE

Is there any evidence of:

		YES	NO
A.	Damage to chain lock or handrails?		✓
B.	Damage to hoisting mechanism?		✓
C.	Missing or damaged concrete stoplogs?		✓
D.	Settlement or cracking of concrete deck?		✓
E.	Sheetpile bowing or interlock distress?		✓
F.	Loss of interior fill?		✓
G.	Seepage?		✓
H.	Unusual conditions?		✓
I.	Vandalism?		✓

Comments:

Status of any detrimental conditions, if any, observed during last inspection:

6.0 RACEWAY EMBANKMENT

Is there any evidence of:

		YES	NO
A.	Local subsidence, sinkholes, animal burrows, or depressions?		✓
B.	Erosion at the water line (raceway or river side)?		✓
C.	Seepage on downstream face of embankment?		✓
D.	Large trees or heavy vegetation impeding inspection?		✓
E.	Accumulation of debris in raceway channel?		✓
F.	Settlement of crest?		✓
G.	Sloughing or slides?		✓
H.	Unusual conditions?		✓
I.	Vandalism?		✓

Comments:

Status of any detrimental conditions, if any, observed during last inspection:

7.0 RACEWAY STOPLOG SLUICE STRUCTURE

Is there any evidence of:

		YES	NO
A.	Missing or damaged stoplogs?		✓
B.	Substantial leakage through stoplogs?		✓
C.	Cracking or movement of concrete walls?		✓
D.	Leakage from crack(s) in concrete walls?		✓
E.	Seepage around the walls or under the apron?		✓
F.	Accumulation of debris on stoplogs or apron?		✓
G.	Settlement of fill?		✓
H.	Deterioration of concrete?		✓
I.	Unusual conditions?		✓
J.	Vandalism?		✓
K.	Deterioration or damage to upstream masonry walls?		✓

Comments:

Status of any detrimental conditions, if any, observed during last inspection:

8.0 PIEZOMETERS

Is there any evidence of:

		YES	NO
A.	Damage to casing?		✓
B.	Is the cap locked and in place?		✓
C.	Is there debris or other obstruction inside the casing?		✓
D.	Is there ice inside the casing?		✓
E.	Is there settlement around the piezometers?		✓
H.	Unusual conditions?		✓
I.	Vandalism?		✓

PIEZOMETER READINGS

Piezometer	Elevation at Top of Pipe (ft) (a)	Depth to Water (ft) (b)	Water Elevation (ft) (c) = (a) – (b)
BH-1	952.82	9.70	943.02
BH-2	953.79	11.24	942.55
BH-3	954.03	6.10	947.93

Note: Elevation at top of pipe resurveyed on 28 November 2005.

Comments:

Status of any detrimental conditions, if any, observed during last inspection:

9.0 SURFACE WATER READINGS

Location	Elevation at Benchmark (ft)[1] (a)	Depth to Water (ft) (b)	Water Elevation (ft) (c) = (a) – (b)
Reservoir	954.14	4.85	949.29
Raceway Channel	954.10	7.10	946.00
River (downstream)	944.26	2.90	941.36

[1] Locations of Benchmarks:

Reservoir: Chiseled square on the northwest corner of east abutment of raceway closure structure.

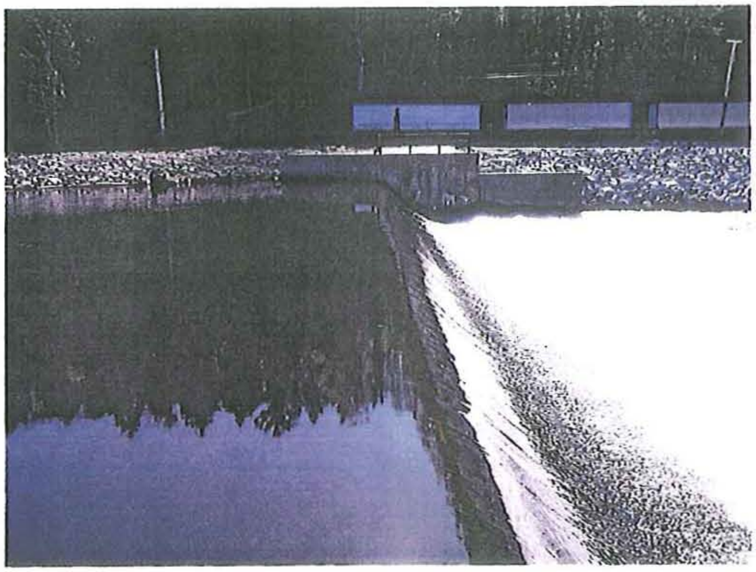
Raceway Channel: Chiseled square on the southwest corner of east abutment of raceway closure structure.

River: Chiseled square on the southwest corner of north wingwall of raceway stoplog structure.

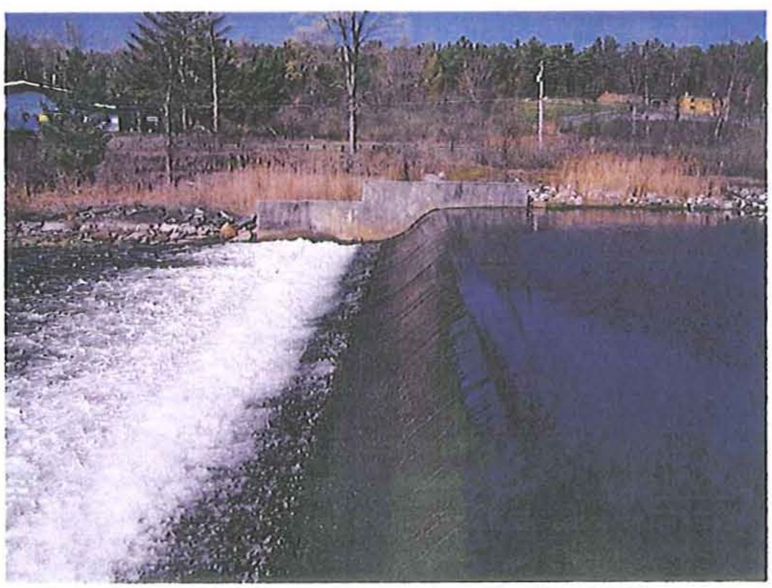
10.0 PHOTOGRAPHS

Typical photographs of project features to be inspected as well as conditions encountered in the past are presented in Section F. Locations of each photograph are shown on the Photo Location Map. Suggested photographs to be taken at the Quarterly Inspection are highlighted in green on the Photo Location Map.

#	Photograph	Taken?
2	Railroad area at west (right) abutment	JPG #99
3	Crest of west (right) abutment	~~101~~ 113
4	View of reservoir from upstream of west (right) abutment	101
5	Wetlands area from downstream right bank looking upstream	114
6	Overflow spillway and left training wall from right training wall	102
7	Riprapped riverside of raceway embankment taken from wetlands area downstream of toe	103
8	End of base of right spillway training wall	104
9	Concrete cap of raceway closure structure	105
10	Left upstream masonry wall upstream of raceway closure structure	106
11	View of raceway channel from raceway closure structure	107
12	End and base of left spillway training wall and depression area at spillway toe	108
13	Right spillway training wall	—
14	View of right spillway wall from raceway closure structure	109
15	Crest of raceway embankment	110
18	Upstream face of raceway stoplog sluice structure	112
20	Downstream of raceway stoplog sluice structure	111
21	View of concrete overflow spillway from raceway embankment	—



Woods Pond Dam Quarterly Inspection – April 27, 2006
Crest of Spillway Looking West to East



Woods Pond Dam Quarterly Inspection – April 27, 2006
Crest of Spillway Looking East to West

Picture was not recorded

Woods Pond Dam Quarterly Inspection – April 27, 2006
View of Dam Looking Upstream

Woods Pond Dam Quarterly Inspection – April 27, 2006
Raceway Channel Looking Downstream

WOODS POND DAM

2005 STRUCTURAL INTEGRITY ASSESSMENT

PREPARED FOR

GENERAL ELECTRIC COMPANY

MWH

APRIL 2006

Woods Pond Dam
2005 Structural Integrity Assessment

Prepared for

General Electric Company

By

MWH

April 2006

PREFACE

At the request of General Electric Company, an independent inspection of the Woods Pond Dam was performed to assess the structural integrity of the dam, including conditions and circumstances that could lead to catastrophic failure of the dam and/or substantial release of the sediments contained in the impoundment behind the dam. This is the fourth assessment conducted pursuant to Paragraph 123.a of the Consent Decree (CD) executed by General Electric and various federal and state agencies, which was effective upon approval by the court on October 27, 2000. At GE's request, this assessment was performed one year in advance of the scheduled biannual assessment as a precaution due to high flows in October 2005. The inspection was performed on 29 November 2005 by Mario Finis, P.E. and Manoshree Sundaram, P.E., both of MWH Americas, Inc. (MWH).

The reported condition of the dam is based upon onsite observations and data available to the inspection team at the time of the inspection.

COMMONWEALTH OF MASSACHUSETTS
MARIO
FINIS
STRUCTURAL
NO. 41193
REGISTERED
PROFESSIONAL ENGINEER

Signed: ______________________
Mario Finis, P.E.
MWH

TABLE OF CONTENTS

LIST OF EXHIBITS

APPENDICES

1.0 PROJECT DESCRIPTION AND BACKGROUND

1.1 Background

Woods Pond Dam is located on the Housatonic River near the towns of Lee and Lenox, Massachusetts, as shown on Exhibit 1. The original dam was reportedly constructed in about 1864. The original structure consisted of a 9-foot high rock-filled timber crib overflow dam with a 14-foot-high earthfill embankment parallel to the river channel as its east (left) abutment. The original structure was replaced in 1989 with a new concrete overflow spillway and new abutments with the centerline of the new dam located downstream from the crest of the old dam. An earthfill (raceway) embankment forms a 30- to 50-foot wide raceway channel east of the main river channel. The raceway channel conveys flow to a small mill pond, which at one time was a forebay for a water-powered mill that has been retired. The dam impounds the Woods Pond Reservoir, which is part of the Housatonic River Valley Wildlife Management Area. No design or construction drawings for the original project structures are known to be available.

Prior to the 1989 rehabilitation and dam replacement, the original Woods Pond Dam was in a deteriorated condition and could not pass a flood greater than about the 10-year flood event without overtopping the raceway embankment at the east (left) abutment. Overtopping of the embankment at the east (left) abutment could have caused a breach of the embankment, resulting in the uncontrolled release of the reservoir, including silt, which had settled to the bottom of the reservoir.

Beginning in 1979, studies and investigations were performed to assess the condition of the dam and to develop recommendations for repair and rehabilitation of the dam. In 1983, the timber planking and heavy gage sheet metal on the main overflow dam were replaced with an 18-inch-thick reinforced concrete cap. After placement of the cap, a leak developed at the east abutment masonry wall, presumably due to the cut-off of natural seepage through the timber crib dam. The joints in the masonry wall were filled with mortar shortly after the 1983 construction, temporarily stopping the leakage, which therein contributed to pressure buildup behind the wall resulting in lateral movement of the wall.

In 1988, a plan was developed to rehabilitate the dam to provide a safer, more reliable project structure. Based on a geotechnical investigation, a detailed design report for the dam rehabilitation was prepared in 1989 [Reference 1]. The rehabilitation was carried out in two stages. The first stage consisted of the construction of a raceway closure structure with concrete stoplogs to divert flow away from the raceway and mill pond. The purpose of the closure structure is to protect the raceway embankment from overtopping and potential failure during high flows. During later (second stage) construction, the closure structure was used to divert water through the raceway channel while work was performed on the new spillway and abutments. The second stage consisted of the construction of a replacement spillway and non-overflow gravity sections a short distance downstream of the original spillway. A rockfill berm was also constructed on the west (right) bank of the river between the original spillway and new spillway, and the upper 2.5 feet of the original timber crib spillway were demolished. A general plan of the rehabilitated project is shown on Exhibit 2. Exhibit 3 presents typical cross sections of the structures. The original stoplog sluice gate structure at the downstream end of the raceway embankment was rehabilitated in 1991. This structure controls the water level in the Mill Pond and raceway channel. The drawings for the dam and raceway closure structure were provided in an Appendix to the 2000 Structural Integrity Report [Reference 6].

1.2 Project Features and Project Classification

The dam consists of a 140-foot-long concrete overflow spillway, a concrete non-overflow gravity section at the west (right) abutment, and a concrete and steel sheetpile raceway closure structure at the east (left) abutment, all constructed since 1989. The dam has a maximum height of approximately 14 feet. The ogee spillway has a crest elevation of 948.3 ft National Geodetic Vertical Datum (NGVD). The raceway closure structure forms the east (left) abutment of the dam and has a top elevation of 954.0 ft NGVD. The west (right) abutment is a non-overflow gravity structure with a sloped downstream face and a top elevation of 954.0 ft NGVD. The new structure is completely independent of the original structure and is located about 200 feet downstream of the original structure. The new structure does not rely on any of the original structures for stability.

The structures are founded on shallow "marbleized" bedrock, which is vertically bedded and is generally fine grained, hard with variable medium to close joint spacing. This was determined during the 1988 geotechnical investigation program, which also included soil borings and water pressure testing of the rock. Details of the subsurface field investigation can be found in the General Design Report for Woods Pond Dam Rehabilitation [Reference 1].

In accordance with the Massachusetts Department of Conservation and Recreation, Division of State Parks and Recreation (DCR) Regulations (302 CMR 10.00, November 2005), Woods Pond Dam is designated as a "large" size dam with a "significant" hazard (Class II) potential. This size classification is based on the storage capacity that occurs during the spillway design flood, while the hazard potential classification is based on the presumption that a failure of the dam may result in the loss of life and significant property damage immediately downstream of the dam. A failure of the dam could also result in the uncontrolled release of sediments from the reservoir upstream of the dam.

1.3 Previous Inspections and Reports

Harza Engineering Company, Inc. (Harza), a predecessor to MWH Americas, Inc. (MWH), inspected the dam in 1991, shortly following completion of the project rehabilitation [Reference 2]. Harza inspected the dam in 1998, with a report prepared in March 1999 [Reference 3]. Harza also prepared a Downstream Raceway Embankment Slope Stability Analysis in March 2000 [Reference 4]. In addition, the Massachusetts Department of Environmental Management (DEM, now DCR) performed an inspection of the dam in 1998 [Reference 5]. Harza then performed the first Structural Integrity Assessment of Woods Pond Dam in December 2000 with a report submitted in January 2001 [Reference 6]. This was performed in accordance with the Consent Decree (CD) executed by General Electric Company (GE) and various agencies. As a result of the 2000 assessment, various modifications to the project were completed to improve the structural integrity of the project. According to GE, at the invitation of the Lead Administrative Trustee (LAT) under the CD, a DEM staff member reviewed the modifications to the dam during a site visit in January 2002. MWH performed the second Structural Integrity Assessment of the Woods Pond Dam in October 2002. As part of that work, MWH reviewed the modifications performed as a result of recommendations made in the 2000 report and prepared record drawings to reflect the work performed in 2001. These drawings were provided in an Appendix to the 2002 Structural Integrity Report [Reference 10]. MWH performed the third Structural Integrity Assessment of the project in November 2004 and, as part of that report, made recommendations for minor repair work [Reference 11]. The work was completed in 2005.

1.4 Recent Project Activities

As part of the Structural Integrity Assessment performed in 2004 by MWH [Reference 11], several recommendations were made to improve the structural integrity of the project. These were implemented in 2005 and included the following:

- Stabilization of the remaining section of river-side raceway embankment slopes immediately downstream of the spillway by placing riprap to a minimum of three feet above the normal water line;

- Repair of voids and deterioration at the waterline on the upstream face of the left and right masonry training walls at the stoplog sluice structure (at the downstream end of the raceway channel).

Observations on these modifications are discussed in Section 2.0.

2.0 FIELD INSPECTION

2.1 General

The field inspection was performed on Tuesday, 29 November 2005. The weather was cloudy with drizzle, with a temperature of approximately 53°F at the time of the inspection. Weather records for Pittsfield, Massachusetts indicate a high temperature of 60°F and low temperature of 50°F for that day with average precipitation of 0.10 inches. Temperatures were higher than typical for this time of year. A very thin ice cover was observed over most of the water surface in the raceway channel as snow and cooler weather was experienced in the preceding week. However, flow over the spillway was fairly strong and no ice formation was observed in the reservoir, Mill pond, or main river channel.

The reservoir water level (upstream of the overflow spillway) was measured at approximately El. 949.4 ft NGVD, 13 inches above the spillway crest, which is at El. 948.3 ft NGVD. The tailwater was at approximately El. 946.7 ft NGVD.

The inspection was made by Mr. Mario Finis, P.E. and Ms. Manoshree Sundaram, P.E., both of MWH. Prior to visiting the project site, the inspection team met with Mr. Kevin Mooney, Mr. John Levesque, and Mr. John Novotny of General Electric Company as well as Mr. John Powers, Mr. Mark Wasnewsky, Mr. Jim Roff, and Mr. Sean Coyle of O'Brien & Gere, Inc. to conduct refresher training sessions both in the office and in the field for personnel who are responsible for conducting and overseeing the monthly and quarterly inspections of Woods Pond Dam.

The inspection performed by MWH involved observations of the portions of the structures visible at the time of the inspection. No subsurface or underwater inspections were included as part of this field inspection. Photographs taken during the inspection are included with this report.

2.2 Reservoir and Overflow Spillway

The reservoir rim and water surface in the immediate vicinity of the project structures were inspected. No sloughing, slides, or other indications of instability or unusual conditions that could affect the integrity of the project structures were observed along the reservoir rim near the project structures.

The surface of the overflow spillway was not clearly visible during the inspection due to nearly thirteen inches of water flowing over the crest. The flow over the spillway crest was generally smooth with no observations of irregular flow patterns or evidence of differential movement of the monoliths (Photos 1 and 2). Observations made with this flow over the spillway indicate that the concrete on the crest and downstream face of the spillway is in good condition (refer to Appendix A for explanation of project feature condition scale), with no observed deterioration since the last inspection. Algae accumulation observed on the downstream face of the spillway does not appear to impede flow over the spillway or cause other potential problems. A horizontal line across the spillway located several feet upstream of the toe appears to cause a slight disruption of the flow. However, this appears to be the location of the end of the forms used to construct the spillway ogee, and marks the transition between the formed and unformed surfaces. There is no evidence of erosion at this location based on observations, but this area should be monitored as part of the quarterly inspections and should be observed during a period of low flow, if possible. Flow at the toe of the spillway was relatively level, with no visual indication of scour or undermining. No areas of visible spalling, cracking, erosion, or signs of concrete deterioration across the spillway were observed. The concrete at the spillway-abutment contacts is also in good condition.

An investigation consisting of probing and sounding of the riverbed downstream of the toe of the concrete overflow spillway was performed in 2002 and revealed a small depression near the left training wall approximately three to four feet lower than the average riverbed elevation. At this time, it is unclear if this depression existed at the time of construction of the new overflow spillway, or if this depression has developed since construction was completed. Results of the 2002 investigation as well as exhibits and photographs showing the top of rock as observed and recorded during construction are included in the 2002 Structural Integrity Assessment Report [Reference 10].

Visual inspection of this small depression area during this inspection did not reveal cloudy or muddy seepage or signs of erosion in the general area of the depression; however, it is recommended to observe the area in a time of low flow to allow closer inspection of the low spot. Also, it is recommended to perform a survey within the next year to compare with the results of the 2002 baseline survey to determine if the depression area has enlarged or experienced additional deterioration.

To facilitate estimates of the water levels in the reservoir, and reduce risks to personnel taking measurements, particularly during high flows, we suggest a staff gage be installed or elevations be painted on the spillway training walls. Elevations could be painted on the left training wall to facilitate view from Crystal Street or from the railroad tracks/west abutment, and on the right training wall to facilitate reading from the crest of the raceway embankment or from near the closure structure. Figure 1 below illustrates a suggested location for a staff gage or to paint reference elevations to facilitate reservoir level readings.

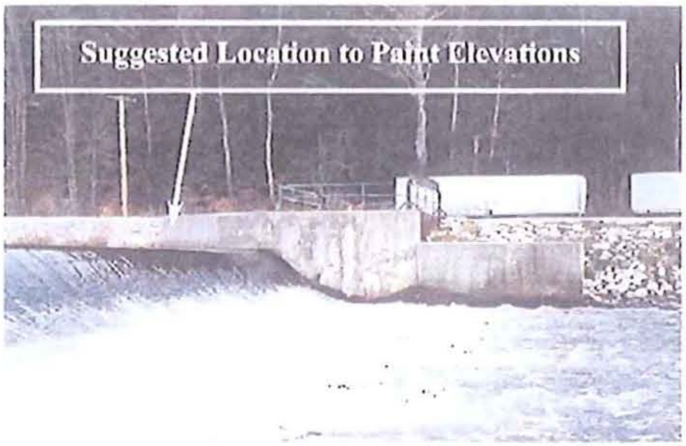


Figure 1. Left Abutment - Suggested location for staff gage or to paint reference elevations (right abutment to be similar)

2.3 West (Right) Abutment Non-overflow Gravity Section

The west (right) abutment consists of a training wall adjacent to the spillway, which extends upstream and downstream of the spillway crest, and a mass concrete gravity section that ends at the railroad tracks at a sheetpile wall. The abutment was observed to be in good condition. No signs of differential movement of the concrete gravity monoliths, areas of visible spalling, significant cracking, erosion, exposed reinforcement, or other signs of deterioration on the visible portions of the upstream face, crest, downstream face, or riverside face were observed. The joints above ground were observed to be in good condition. A few small cracks that have been observed in the past three inspections in the top of the right training wall at the right abutment do not appear to have changed. The riprap along the west embankment, both upstream and downstream of the spillway, appeared to be in good condition with no scour or undermining of the riprap observed (Photo 3).

The training wall at the end of the overflow spillway is in good condition with no large cracks or significant deterioration or signs of movement or undermining observed (Photo 4). Some deterioration and rust-colored stains have been noted near the base of the training wall downstream of the spillway crest. The concrete in this area should continue to be monitored for signs of deterioration or rusting reinforcing steel.

Upstream of the spillway crest, the slight bulge at just above the waterline along the vertical joint in the training wall, which was observed in the 2004 inspection, does not appear to have changed or moved.

The concrete abutment ends at the east side of the Housatonic (formerly Boston and Maine) Railroad tracks with riprap placed against the sheetpile (Photo 5). Approximate measurements of the top of railroad bed were made. The sheetpile extends to approximately 4 inches above the top of the concrete abutment. The distance from the top of the sheetpile to the top of the rail was measured at 7½ inches while the distance from the top of the rail to an approximate top of the bedding was measured at 16 inches. This area should be routinely monitored for any seepage through or around the concrete non-overflow section or for signs of muddy or cloudy flow. No adverse conditions, such as settlement, depressions, or sinkholes in the surface of the railroad bedding were noted during the inspection.

The wetland area noted in previous inspections downstream of the west abutment along the right bank was wetter than noted in the previous visit due to recent precipitation. The water observed was clear, with no high velocities. No erosion of the backfill behind the abutment wall was observed. The area where the 12-inch diameter drainage pipe was identified beneath the railroad tracks should continue to be monitored for changes in the rate of flow or for signs that the water is muddy or cloudy. Flow from the drainage ditch along the western edge of the railroad feeds the wetland area via this drainage pipe; however, if signs of muddy or cloudy water are noted, these could indicate potential seepage from the reservoir. The vegetation in the area is reasonable and does not currently obstruct observation. However, it is recommended that the woody vegetation in this area be kept to a minimum to facilitate inspection for potential problems.

2.4 East (Left) Abutment and Raceway Closure Structure

The east (left) abutment consists of a steel sheet pile cell-type structure with concrete cap and provisions for stoplogs controlling the flow into the raceway (raceway closure structure). The right-side face of the sheetpile structure is covered with concrete and forms the left spillway training wall. A small concrete retaining wall on the downstream side of the structure acts as an extension of the spillway training wall and retains fill for the raceway embankment. Masonry and sheetpile walls upstream of the stoplogs act as training walls. Overall the structure is in good condition.

The minor cracking in the concrete facing on the left spillway training wall, just downstream of the crest of the spillway, has not changed or worsened from previous inspections (Photo 6). This concrete is a facing on the steel sheet pile and is not critical to the integrity of the water retaining structures, but helps with the approach hydraulics and smooths flow over the spillway at the abutment interface. The cracks have no structural integrity significance. The concrete cap between the sheetpiles is in good condition (Photo 7). The joints have been cleaned and refilled, and are holding up well. The steel sheetpile is in good condition with no indication of bowing or interlock distress in any of the piles or of loss of fill from within the sheetpile.

The repairs made in 2001 to the left masonry wall upstream of the entrance to the raceway stoplog closure structure are in fair condition and have not changed since the last inspection. The chipped corners of the patch appear stable and have not appreciably changed since past inspections. The purpose of this patch was to minimize the potential for water to seep through the bank and around the end of the abutment under Valley Road. The area around the left abutment and the surface of Valley Road in this area should continue to be monitored for signs of seepage or piping. The trailers noted during the last inspection remain stored in this area. Inspection of the ground surface and slope at Valley Road showed no sinkholes, depressions, erosion, or other signs of seepage around the abutment or movement of the abutment.

The raceway closure (stoplog) structure is in good condition. Several of the ladder rungs located on the downstream side of the stoplogs remain bent. Also, there was some leakage through the stoplogs, providing a small amount of flow into the raceway channel. These conditions do not affect the integrity or operation of the dam.

We recommend the current stoplog operating procedure be revised. The current procedure involves placing additional stoplogs during rising reservoir water levels, then removing them to lower levels for normal conditions. In terms of practicality, especially in times of flood, we suggest placing all the stoplogs under normal conditions and leaving them in-place all the time except for maintenance. If the need to lower the reservoir arises, the stoplogs can be removed. The practice of maintaining flow in the raceway channel by having water spill over the top of the stoplogs is no longer necessary. Flow in the raceway is now maintained by leakage through the stoplogs. If necessary, spacers can be placed between two stoplogs to maintain a minimal flow in the raceway channel. As little as a one-inch space between two stoplogs should suffice to maintain a minimum flow in the raceway channel. The small amount of flow going through the one-inch opening during a flood would not be an issue in terms of dam safety. Leaving the stoplogs in-place all the time will avoid the need to install stoplogs as water levels are rising, alleviating a potential personnel safety concern, and potential dam safety concerns if the stoplogs were not able to be placed for some reason while water was rising.

2.5 Raceway Embankment

The raceway embankment upstream of the new dam consists of a masonry wall around the embankment and grouted and ungrouted riprap over the surface of the embankment. This feature is no longer a water-retaining structure and serves no dam safety function. The left wall of the upstream raceway embankment (to the right of the raceway closure structure) continues to move and separate from the raceway embankment. The wall is leaning out at the top nearly six inches. If the movement continues, the wall could collapse into the reservoir. This wall upstream of the raceway closure structure serves no structural purpose so further deterioration or collapse of the wall will not affect the overall integrity of the water retaining structures.

Downstream of the dam and raceway closure structure, the raceway embankment consists of earthfill and riprap. This raceway embankment, which is about 12 to 14 feet high, is located between the river channel and the raceway channel. Under normal conditions, the water surface in the raceway channel is about one to two-feet lower than the reservoir water level, and about 4 to 5 feet higher than the water level in the river channel.

All of the vegetation along the raceway channel side of the raceway embankment has been removed since the previous inspection. The entire length of the raceway channel side of the raceway embankment was regraded and fully riprapped, and is in good condition (Photo 8). The landside (left side) embankment along the raceway channel was also riprapped in 2001 for a distance of approximately 130 feet downstream of the raceway closure structure to remediate erosion and locally steep slopes near the project structures. This area is also in good condition. Some erosion along the remainder of the left embankment was observed. However, the left slope of the raceway channel downstream of the riprapped section serves no function concerning the structural integrity of the dam. The erosion should be monitored for possible sloughing of material into the raceway channel or for trees falling into and blocking the channel, which could then back up water in the channel so that it might overtop the embankment. The masonry wall on the left side of the culvert at the end of the raceway channel was observed to be leaning outward (Photo 9) more than during the last inspection. This masonry wall does not serve any dam safety or water retaining function, so its condition is not relevant to the structural integrity of the dam.

All significant vegetation on the river-side of the raceway embankment has also been removed and the slopes have been protected with riprap from the toe to the crest. The short length of embankment just downstream of the spillway previously without riprap was now covered with riprap like the rest of the embankment and is in good condition (Photos 10a and 10b). The slopes were measured to vary from approximately 2H:1V to 2.5H:1V and are in good condition with minimal vegetation. The riprap is of good quality and is predominantly well graded. A few areas were noted where large pieces of riprap have loosened or where a higher percentage of finer materials exist (Photo 11). A few stones at the base of the slope may have moved or rolled into the riverbed, possibly as a result of the recent high flows. These stones should be restored to their proper positions as part of routine maintenance each spring. The riprap near the water lines should be inspected each spring as part of the quarterly inspections for signs of damage due to freeze/thaw action and/or high flows.

The three piezometers installed along the crest are marked with orange traffic cones. The locking caps for the piezometers were damaged during the recent riprap placement activity and therefore new locking caps were placed with additional visible identification (Photo 12). The newly repaired piezometers have been resurveyed to provide for accurate piezometer readings. Readings from these instruments are currently taken quarterly and are discussed further in Section 4.0. Based on discussions between GE and MWH, the frequency of the readings will be increased to a monthly basis.

Some ruts, likely due to earthmoving equipment used in the riprap placement activity, were observed along the crest, especially along the length where new riprap was placed earlier in 2005. Other minor depressions were also noted. The crest of the embankment should be periodically surveyed, at least once every 10 years, to verify the crest elevation is at El. 952 ft or above, and all areas below El. 952 feet should be raised (Photo 13). In the event that the ruts and depressions extend to a significantly greater depth than currently is the case, the potential for overtopping of the raceway embankment would increase.

Beaver activity observed during this visit appeared to be at a minimum however, some activity was observed and should continue to be monitored so that blockage of flow in the raceway channel is avoided.

2.6 Raceway Stoplog Sluice Structure

The raceway stoplog sluice structure is a concrete and masonry structure located at the downstream end of the raceway channel, just upstream of the Mill Pond. The purpose of the structure is to maintain the water level in the Mill Pond and raceway channel and to allow lowering of the water in the reservoir or raceway channel and Mill Pond for inspection and/or maintenance.

As noted in past inspections, the upstream masonry wing walls of the structure are deteriorated, particularly at the waterline, and are in marginal condition. As first noted in the 1999 inspection report [Reference 3], the walls appear to be leaning outward slightly. The voids identified in the 2004 inspection on the left and right masonry walls at the waterline and within the walls themselves have been repaired and are in good condition. Some deterioration was noted on the right wall at the waterline, as well as a vertical crack approximately six feet to the right of the edge of the stoplog structure. These should be repaired within one year to minimize possible seepage or piping which may lead to washout around the structures and/or movement of the walls (Photo 14).

Several branches, possibly due to beaver activity, and other debris were located on and behind the stoplogs. The debris over the stoplogs should be kept cleared so that it does not cause excessive backup of water in the raceway channel, which might overtop the embankment.

The wingwalls downstream of the stoplog sluice structure are in fair condition. The vertical cracks noted on the north and south abutment walls during the previous inspection have been repaired as have the small holes previously observed in the south abutment wall. No evidence of settlement of the structure or surrounding backfill, or seepage around the structure, was observed. However, flow of about 1 GPM was observed coming from the base of the vertical crack on the left wall (Photo 15). The accumulation of sand in this area was thought to be a result of the high flows from a recent high water event, rather than from the flow through the crack. The sand and debris in this area should be cleared and the flow rate and quality of the flow (clear, turbid, presence of deposits, etc.) should be monitored visually as part of the monthly inspection. In addition, the flow rate should be measured quarterly. The source of the leakage should be investigated, perhaps by fluorescent dye test or other means. If the point at which the water is getting though the upstream wingwall can be identified, it should be sealed from the upstream side. The crack and leakage should not be sealed or plugged from the downstream side, as this could result in a buildup of pressure behind the wall or in the water finding a new exit path which could cause erosion, piping or other problems

3.0 SPILLWAY ADEQUACY

The new spillway was designed in 1988 and constructed in 1989. The design criteria for the spillway required that:

- There would be no change in normal Woods Pond reservoir levels; and
- There would be no change in the 100-year flood reservoir level.

The design and construction of the dam was approved in 1989 by the DEM, under Chapter 253 Dam Safety Permit (Waterways Application No. 89W-077, License No. 2028, August 2, 1989). The DEM/DCR regulations were subsequently revised in December 1996, May 2004, and November 2005. Adequacy of the spillway under the revised regulations was discussed in the March 1999 inspection report [Reference 3]. Section 10.14 (6) of the regulations, Spillway Design, has not changed since the March 1999 inspection report. The discussion presented in that report concluded that the 500-year flood flow can be passed without failure of the dam.

The area to the right of the west (right) abutment at the railroad bed has been previously identified as a low spot and would be the first area to be overtopped during a major flood. The potential for erosion and subsequent failure of this area was explored and is summarized in the 2002 Structural Integrity Assessment Report [Reference 10]. While this area at the railroad bed is not part of the dam structure reconstructed in 1989, it does form part of the water retaining features of the project. The plan view of the area, shown on Figure 2 below shows the location of the railroad and its proximity to the project. A schematic illustrating the profile along the crest of the dam (viewed looking downstream) is shown in Figure 3.

During the 100-year flood, the reservoir level is estimated at El. 954.6 ft., while the tailwater level is estimated at El. 951.5 ft. [Reference 3]. The reservoir level during the 500-year flood is estimated at El. 955.8 ft. with associated tailwater estimated at El. 952.8 ft. [Reference 3]. These flows will overtop the east and west concrete abutments by 1.8 feet, which can be safely withstood by the structures. As illustrated in Figure 3, the lowest area along the project profile other than the spillway is the railroad bed. During the 100-year and 500-year floods, water will flow through this low area. Evaluation of flow velocities and durations for these floods concluded that flows will overtop the railroad bed area, but that the size of the bedding stone and configuration of the project features should not result in the failure of the project structures or uncontrolled release of the reservoir for flows up to and including the 500-year flood (see Appendix C and Appendix D).

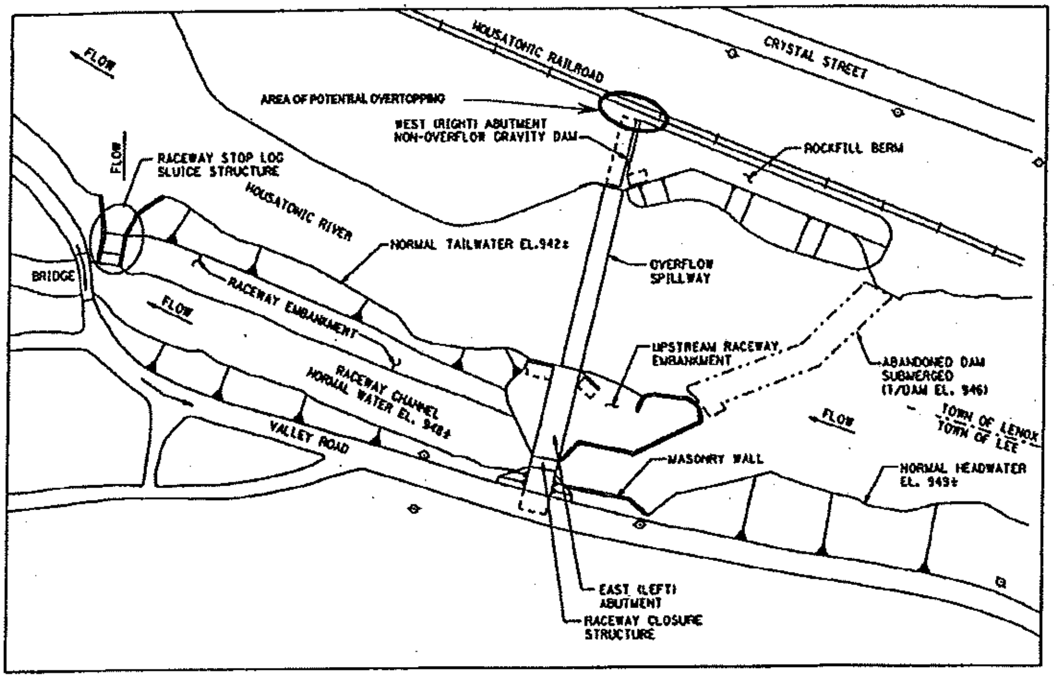


Figure 2. Plan View of Project.

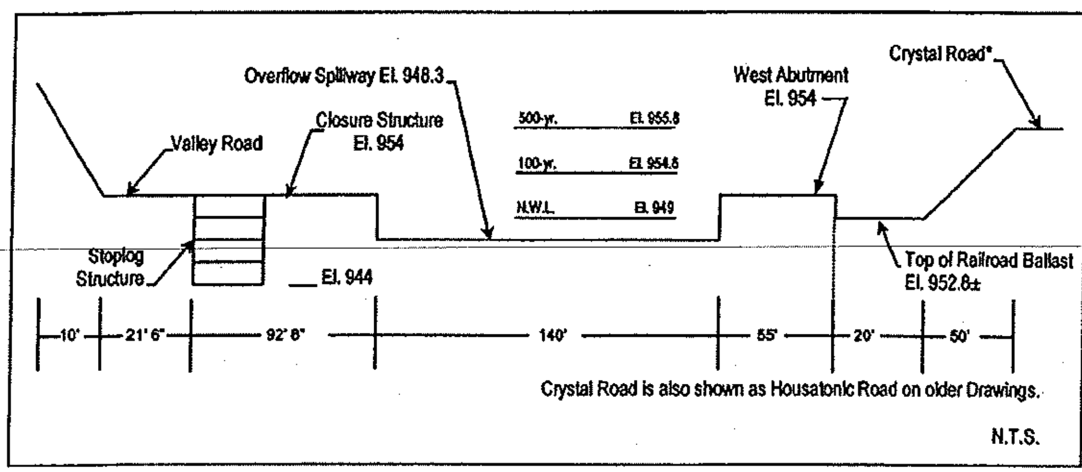


Figure 3. Schematic Profile Along Crest of Dam Looking Downstream.

3.1 October 2005 Flood

The high flow event of 9 October 2005 had a reported peak flow at the U.S. Geological Survey stream gaging station in Coltsville, upstream of Woods Pond Dam, of 6,510 cfs, which was reported to be between the 50-year and 100-year recurrence interval flood. This is reported to be the highest flow on record at the gage which has records going back 70 years, to 1936, exceeding the previous peak of 6,400 cfs in 1938 (Note: provisional USGS data, provided in Appendix B, indicates this flow to be between the 5-year and 10-year recurrence interval flood, while also noting that this is the flood of record for the gage which has records going back to 1936. The footnotes also state that the flow had a 50 year recurrence interval excluding October 2005 peaks. It is MWH's opinion that the 5-10 yr designation may be in error, and that the 50-yr return interval designation is correct). The Coltsville gage drainage area is about 1/3 that of the Woods Pond dam.

For the same event, the peak flow recorded at the USGS stream gaging station in Great Barrington was 8,080 cfs, which is about the 20-year recurrence interval flood.

MWH believes that the Great Barrington gage is a better representation of the historical flows at Woods Pond Dam than the Coltsville gage because of the small size of the Coltsville gage drainage area relative to the drainage area of the Woods Pond dam.

Based on the flow at the Great Barrington gage, the peak flow at Woods Pond Dam is estimated to have been about 4,900 cfs, with an associated water level of about El. 952.8 ft. The actual water level appeared to be a bit lower than this, and did not result in flow going over the low point at the railroad bed. Photos 16 and 17 show water levels at the project as observed by GE staff immediately following the high flow event.

The high flows at Woods Pond dam in October 2005 do not appear to have caused any significant damage to the project structures or to have threatened the integrity of the facility. The only observed damage that might possibly be attributed to the high flows is the movement of some of the riprap stones at the base of the raceway embankment along the left side of the river channel. These few stones can be restored or replaced as part of normal maintenance measures. Overall, the project performed very well during the high flow event.

4.0 INSTRUMENTATION

Beginning in 2003, as part of its quarterly inspection procedures, GE began taking readings at each of the three piezometers located on the crest (BH-1, BH-2, and BH-3) to monitor the phreatic surface (i.e., the depth to groundwater) within the raceway embankment. Reference elevations of the water levels in the reservoir, river channel, and raceway channel are also taken at each reading. The instrumentation data collected to date were reviewed prior to this inspection and a time history plot of the data was developed to evaluate the data and help to identify possible trends and/or anomalies in the data. Exhibit 4 presents the time history plot of the readings from the piezometers since April 2003. For evaluation purposes, reference elevations of the water levels in the reservoir, raceway channel, and river are also plotted. As can be seen on Exhibit 4 and in Tables 1 and 2 below, the piezometer readings generally remain between El. 942 ft. and El. 945 ft. and remain between the water levels in the raceway channel and the river channel and below the reservoir elevation. The readings taken in October 2005 are slightly higher and reflect elevated water levels associated with the high rainfall and high flows in the river. It is also noted that readings taken for piezometers BH-2 and BH-3 may have been inadvertently switched in April 2004 and then back again in January 2005. The slightly higher readings reported by piezometer BH-2 reflect the location of this piezometer near the raceway edge of the embankment crest, whereas piezometers BH-1 and BH-3 are located near the center of the embankment crest. Exhibits 5 and 6 present cross-sections through the raceway embankment showing the October 2005 and July 2005 readings, respectively for each of the piezometers. This cross-section graphically presents the level of saturation of the raceway embankment between the water levels in the raceway channel and the river channel. The higher values recorded on 28 October 2005 reflect the recent storm event, and may be due to surface water infiltrating the embankment, higher water levels in river channel, or both.

Table 1 – Water Level Elevations

	Reservoir			Raceway Channel			River		
Date of Reading	Benchmark Elev. (ft)	Depth (ft)	Elevation (ft)	Benchmark Elev. (ft)	Depth (ft)	Elevation (ft)	Benchmark Elev. (ft)	Depth (ft)	Elevation (ft)
28-Apr-03									
29-Jul-03			948.67			946.27			940.38
28-Oct-03	954.14	3.96	950.18	954.10	7.45	946.65	944.26	1.25	943.01
28-Jan-04		4.40	949.74		7.40	946.70		2.60	941.66
22-Apr-04		4.65	949.49		7.80	946.30		2.80	941.46
27-Jul-04									
10-Nov-04		5.03	949.11		8.29	945.81		3.28	940.98
20-Jan-05		4.10	950.04		7.00	947.10		2.30	941.96
22-Apr-05		5.13	949.01		8.18	945.92		3.40	940.86
27-Jul-05		7.45	946.69		8.10	946.00		3.91	940.35
28-Oct-05		3.99	950.15		8.10	946.00		1.35	942.91

Table 2 – Piezometer Readings

Note: ** Well casings noted damaged during Oct. 2005 inspection; well casings to be repaired and resurveyed.

	BH-1			BH-2			BH-3		
Date of Reading	Top of Casing El (ft.)	Depth to Water (ft.)	Piezometric El. (ft.)	Top of Casing El (ft.)	Depth to Water (ft.)	Piezometric El. (ft.)	Top of Casing El (ft.)	Depth to Water (ft.)	Piezometric El. (ft.)
28-Apr-03	952.29	9.28	943.01				953.17	8.48	944.69
29-Jul-03		9.87	942.42					8.82	944.35
28-Oct-03		8.93	943.36					7.9	945.27
28-Jan-04		7.96	944.33		frozen			frozen	
22-Apr-04	953.13	8.88	944.25	954.07	9.31	944.76	954.14	11.53	942.61
27-Jul-04		10.62	942.51		9.71	944.36		12.37	941.77
10-Nov-04		10.80	942.33		10.08	943.99		12.2	941.94
20-Jan-05		10.09	943.04		11.57	942.50		10.18	943.96
22-Apr-05		9.35	943.78		11.78	942.29		8.99	945.15
27-Jul-05		11.35	941.78		12.65	941.42		10.16	943.98
28-Oct-05	**	8.98	944.15	**	10.32	943.75	**	8.44	945.70

Based on discussions between GE and MWH, the piezometers will continue to be read at monthly intervals coinciding with the monthly inspections. Readings should be plotted to help identify irregular readings, readings that do not correlate with those taken in the past, changes in the trends that have been observed in the past, or readings that exceed the water elevations in the reservoir, river channel, or raceway channel. These types of observations may indicate an erroneous reading or can indicate a potential seepage or piping problem. In the event of an irregular reading, a set of verification readings should be taken to confirm the irregular reading. The plot included in Exhibit 4 provides guidance regarding the general acceptable range of readings at the Project.

5.0 STRUCTURAL STABILITY

Structural stability and embankment slope stability are discussed in the 2001 Structural Integrity Assessment Report [Reference 6]. The dam is a concrete gravity structure founded on rock. The stability analyses performed for the new structures in 1988 are in accordance with the analyses required by the DEM/DCR regulations in 302 CMR 10.00 and the required factors of safety were met. Detailed design information and calculations are contained in the General Design Report for Woods Pond Dam Rehabilitation [Reference 1]. The horizontal force coefficient used for pseudo-static earthquake design loading was 0.1. This design value is considered equivalent to a peak seismic acceleration of about 0.15g to 0.2g. Full uplift was assumed for all loading cases. The value used in the 1988 design for cohesive strength at the interface of a concrete dam founded on bedrock was 100 psi for the non-overflow west abutment, with a coefficient of friction of 0.75. In the 1999 report [Reference 3], the factors of safety for the stability analyses were recalculated to evaluate the sensitivity of the factors of safety relative to values of cohesive strength. Revised factors of safety were calculated using values of 10 psi for cohesive strength with a 0.75 coefficient of friction. The bedrock encountered during construction was angular rock with some irregularities; therefore, the values for cohesive strength and coefficient of friction are conservative for this analysis. As shown in the 2000 Structural Integrity Assessment Report [Reference 6], all factors of safety, even with this much lower value of cohesion, are acceptable and the dam is in compliance with the DEM/DCR regulations in 302 CMR 10.00.

Slope stability analyses were performed for the raceway embankment, and are included in the Downstream Raceway Embankment Slope Stability Analysis [Reference 4], prepared by Harza in March 2000. The embankment acts as a dike between the raceway and the main channel with a hydraulic head differential of approximately 5 feet under normal conditions. The most critical section of the raceway embankment existing at that time, *i.e.*, the narrow section previously located approximately 100 feet downstream of the new dam, was selected for analysis. Soil parameters were established using field classification of the subsurface materials, standard penetration test N-values, grain size distribution, and water content from a subsurface exploration program conducted in 1999 (see Ref. 4 for results of the subsurface investigation). No cohesion was used in the analysis. A number of variations of the phreatic surface through the embankment were estimated for the different analysis cases. These phreatic surfaces correlate with the actual piezometer readings observed. The results of this analysis indicated that the overall stability of the embankment satisfied the recommended factors of safety for stability.

As observed during this inspection, the addition of riprap along the slopes of the raceway embankment provides additional slope protection and stability to the embankment. The previous narrow spot in the embankment has been filled and no longer exists. The oversteepened slopes have been flattened, and the erosion at the toe of the embankment has been repaired. The placement of riprap along the short riverside section of the raceway embankment just downstream of the spillway also improves the stability of the raceway embankment. The modifications made over the past several years have improved the stability of the embankment. All minimum factors of safety for embankment stability have been met or are exceeded.

6.0 SUMMARY AND RECOMMENDATIONS

Overall, the Woods Pond Dam is in good condition and has been well maintained. The 1989 structures were designed in accordance with the DEM (now DCR) Dam Safety Rules and Regulations (302 CMR 10.00) applicable at that time, and were approved by DEM. The dam safety rules and regulations have been revised since the design of the structures, in 1996, in May 2004, and, most recently in November 2005. The new dam, raceway closure structure, and riprap constructed in 1989 are in good condition and the modifications completed between 2001 and 2005 are also in good condition. The spillway and tailrace were under flow at the time of the inspection, but there was no indication of deterioration or distress. The overall condition and integrity of the water retaining structures have been significantly improved as a result of the modifications made since 1998, most recently with the riprap added to the raceway embankment in 2005. The structures safely withstood the high flows in October 2005, estimated at about the 25-year return interval flood, with no noticeable deterioration or degradation, except for the possible movement of some of the riprap at the base of the raceway embankment. With implementation of the modifications recommended herein, the structural integrity of the project water retaining structures should remain intact for flows up to and including the 500-year flood.

The original dam structures are in a deteriorated state; however, these structures, including the upstream raceway embankment, serve no water-retaining function and are not relevant to dam safety. The cracking and movement observed in the walls of the upstream raceway embankment are not critical to the integrity of the water retaining structures or dam safety.

On the basis of our 2005 visual inspection, implementation of the following physical modifications is recommended.

1. Install a staff gage or paint reference elevations on the spillway abutment walls for ease in estimating the reservoir water levels, especially in times of high flow *(Section 2.2)*.

2. The deterioration along the waterline in the upstream face of the right masonry training wall at the raceway stoplog sluice structure should be repaired. Also, the vertical crack in the upstream face of the right masonry wall located approximately six feet from the right wall edge should be repaired *(Section 2.6)*.

3. The seepage at the base of the left downstream training wall at the raceway stoplog sluice structure should be investigated to determine its source, and monitored regularly. A dye test or other test is recommended to determine if the water is coming from the raceway channel. If the raceway channel is identified as the source of the leakage and point of leakage through the upstream walls can be identified, repairs to the upstream walls should be implemented to stop the leakage. The water level in the Mill pond and raceway channel could also be lowered to look for possible deterioration at and/or below the water line on both the left and right upstream wingwall sections *(Section 2.6)*.

4. Riprap similar to that placed along the raceway embankment should be placed along the area just downstream of the right (west) abutment for a distance of approximately 20 to 40 feet *(Appendix C and D)*. Given the close proximity to the railroad tracks, placement of the riprap will likely require access permission from the Housatonic Railroad.

In addition, GE and its contractors should continue the quarterly and monthly inspections to inspect and monitor the project site in order to identify changes in site conditions that may indicate a problem with the dam. Areas of the project site that should be closely monitored during these inspections were covered during the training session and are listed below.

1. The overflow spillway should be evaluated during a period of no/very low flow to determine the condition of the concrete and joints, and determine if there has been any movement of the monoliths and if there is any damage to the overflow structure itself *(Section 2.2)*.

2. The riverbed downstream of the spillway toe should be monitored during a period of low flow to allow close investigation of the depression identified as the result of a survey performed in 2002. This area of depression should be monitored for any cloudy or muddy seepage or any signs of erosion or enlargement that might undermine the dam or abutment *(Section 2.2)*.

3. The cracks in the right (west) abutment non-overflow wall should be monitored for changes *(Section 2.3)*.

4. The wet areas at the west abutment (the downstream side) should be routinely monitored for any increase in seepage through or around the concrete non-overflow section or for signs of muddy or cloudy flow *(Section 2.3)*.

5. The rust colored stains at base of right (west) concrete training wall should continue to be monitored for signs of deterioration/rebar corrosion *(Section 2.3)*.

6. Revise the stoplog operations procedures for the stoplog closure structure by installing all stoplogs under normal conditions, with a one-inch gap left between two stoplogs to provide a base flow in the raceway channel *(Section 2.4)*.

7. The raceway embankment downstream of the new dam should continue to be monitored for signs of erosion, piping, seepage and instability *(Section 2.5)*.

8. An inspection of riprap along the river channel side and raceway channel side of the raceway embankment should be performed each spring as part of the quarterly inspections to identify areas where high flows or freeze/thaw action may have caused damage *(Section 2.5)*.

9. The raceway channel, areas upstream and downstream of the raceway stoplog sluice structure, and areas upstream and downstream of the raceway stoplog sluice structure should be monitored for beaver activity to prevent blockage of the raceway channel *(Sections 2.4, 2.5, and 2.6)*.

10. The crest of the raceway embankment should be surveyed periodically (at least every 10 years) and should be raised as necessary to the minimum design elevation *(Section 2.5)*.

11. The east abutment of the raceway closure structure, left upstream masonry wall at the entrance to the raceway closure structure, and upstream masonry wing walls at the raceway stoplog sluice structure should be monitored for any indications of piping or seepage. Indications of piping may include (but are not limited to) washout around the structures and settlement of fill materials behind the structures. Movement may be indicated by relative movement of the structure away from adjacent fill materials, heaving of adjacent fill materials, or differential movement of the structure (indicated by cracking) *(Sections 2.4 and 2.6)*.

12. Any changes in site conditions identified during the inspections should be reviewed and evaluated by a registered professional engineer with experience in dam inspection and rehabilitation.

Additionally, the Operation and Maintenance Manual and Emergency Action Plan for the project will be updated to reflect the current condition of the project, specifically the raceway embankment. Procedures for maintenance of the riprapped slopes on the raceway embankment should be included. Similarly, the record drawings for the project will be revised to reflect the current condition of the project and should be included in the Operation and Maintenance Manual as well as the Emergency Action Plan, for reference.

REFERENCES

1. General Design Report for Woods Pond Dam Rehabilitation, Prepared for General Electric Company, Pittsfield, MA by Harza Engineering Company, Chicago, IL, April 1989.

2. First Annual Inspection Report of Woods Pond Dam, Prepared for General Electric Company, Pittsfield, MA by Harza Engineering Company, Chicago, IL, March 1991.

3. Woods Pond Dam Inspection Report (1998), Prepared for General Electric Company, Pittsfield, MA by Harza Engineering Company, Chicago, IL, March 1999.

4. Downstream Raceway Embankment Slope Stability Analysis, Prepared for General Electric Company, Pittsfield, MA by Harza Engineering Company, Chicago, IL, March 2000.

5. Inspection/Evaluation Report for Woods Pond Dam, Prepared for Massachusetts Department of Environmental Management, Office of Dam Safety, by Root Engineering, based on inspection conducted on May 27, 1998 (report undated).

6. Woods Pond Dam Structural Integrity Assessment Report (2000), Prepared for General Electric Company, Pittsfield, MA by Harza Engineering Company, Chicago, IL, January 2001.

7. Letter to Andrew Silfer, General Electric Project Coordinator from Dale C. Young, Lead Administrative Trustee of The Trustees of The Commonwealth of Massachusetts Executive Office of Environmental Affairs, July 9, 2001.

8. Phase II Investigation Report at Woods Pond Dam, Lee, Massachusetts, Prepared for General Electric Company, Pittsfield, MA by Harza Engineering Company, Chicago, IL, June 1988.

9. Hydraulic Design Criteria, Sheet 712-1, Stone Stability – Velocity vs. Stone Diameter, by U. S. Army Corps of Engineers, revised 9-70.

10. Woods Pond Dam Structural Integrity Assessment Report (2002), Prepared for General Electric Company, Pittsfield, MA by MWH, Chicago, IL, May 2003.

11. Woods Pond Dam Structural Integrity Assessment Report (2004), Prepared for General Electric Company, Pittsfield, MA by MWH, Chicago, IL February 2005.

12. Railroad Design and Rehabilitation (2000), Technical Instructions TI 850-02 by U. S. Army Corps of Engineers, March 2000.

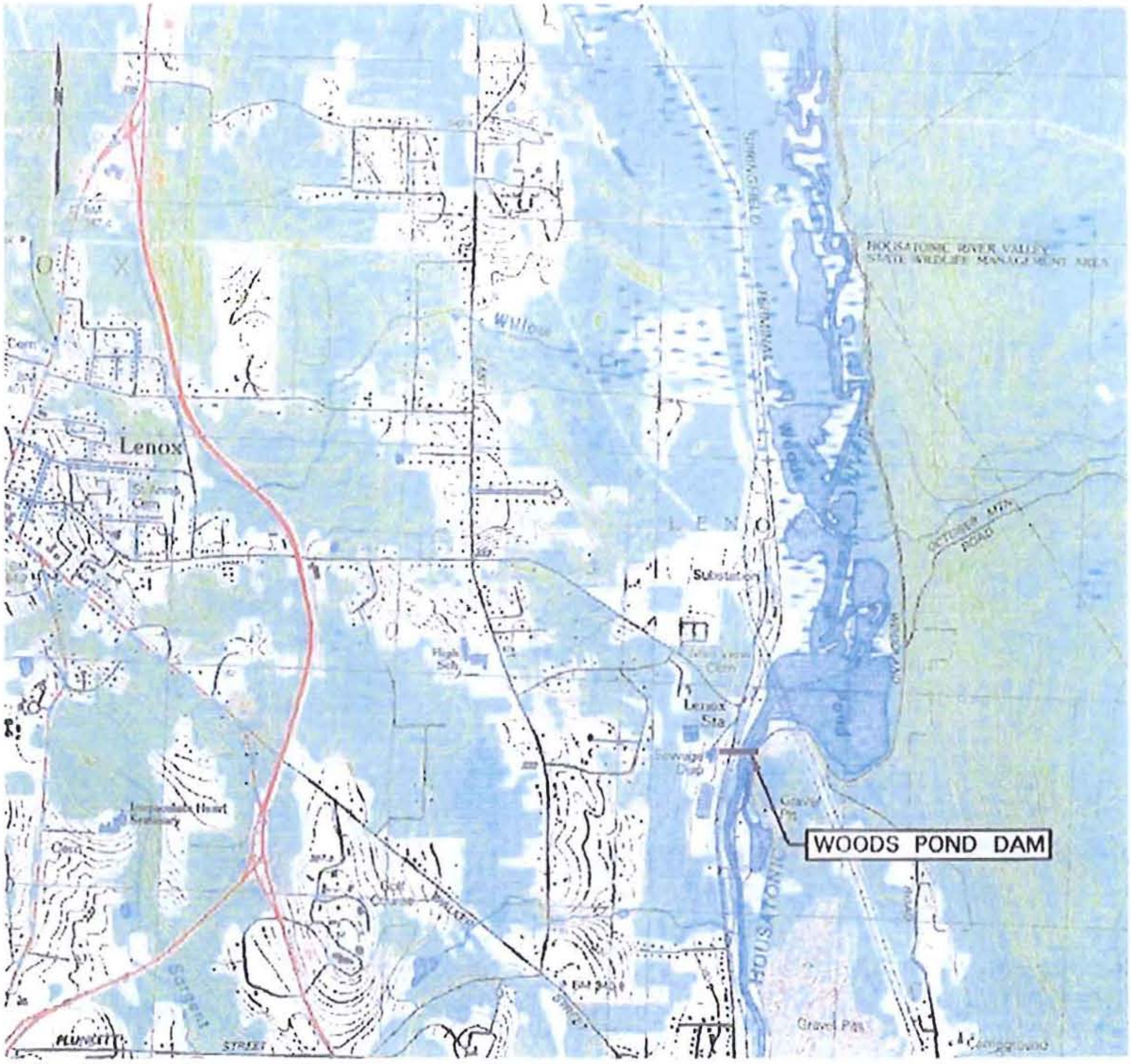
DISCLAIMER

This report and the recommendations contained within are based solely on the information made available. This report is intended for discussion and advisory purposes only. Conclusions and recommendations may materially vary due to events and circumstances that are not reasonably foreseeable, are beyond the scope, or not part of this report, or due to inaccurate or incomplete data provided. This report is provided strictly for the benefit of GE. MWH is not responsible for use, dissemination, or disclosure of the information contained herein by GE to any third party. MWH is also not responsible for updating this report to reflect events or circumstances occurring after the date of submission.

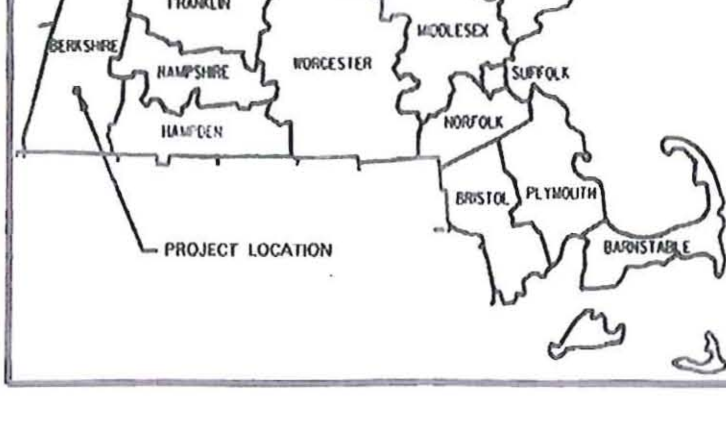
EXHIBITS

LIST OF EXHIBITS

EXHIBIT 1

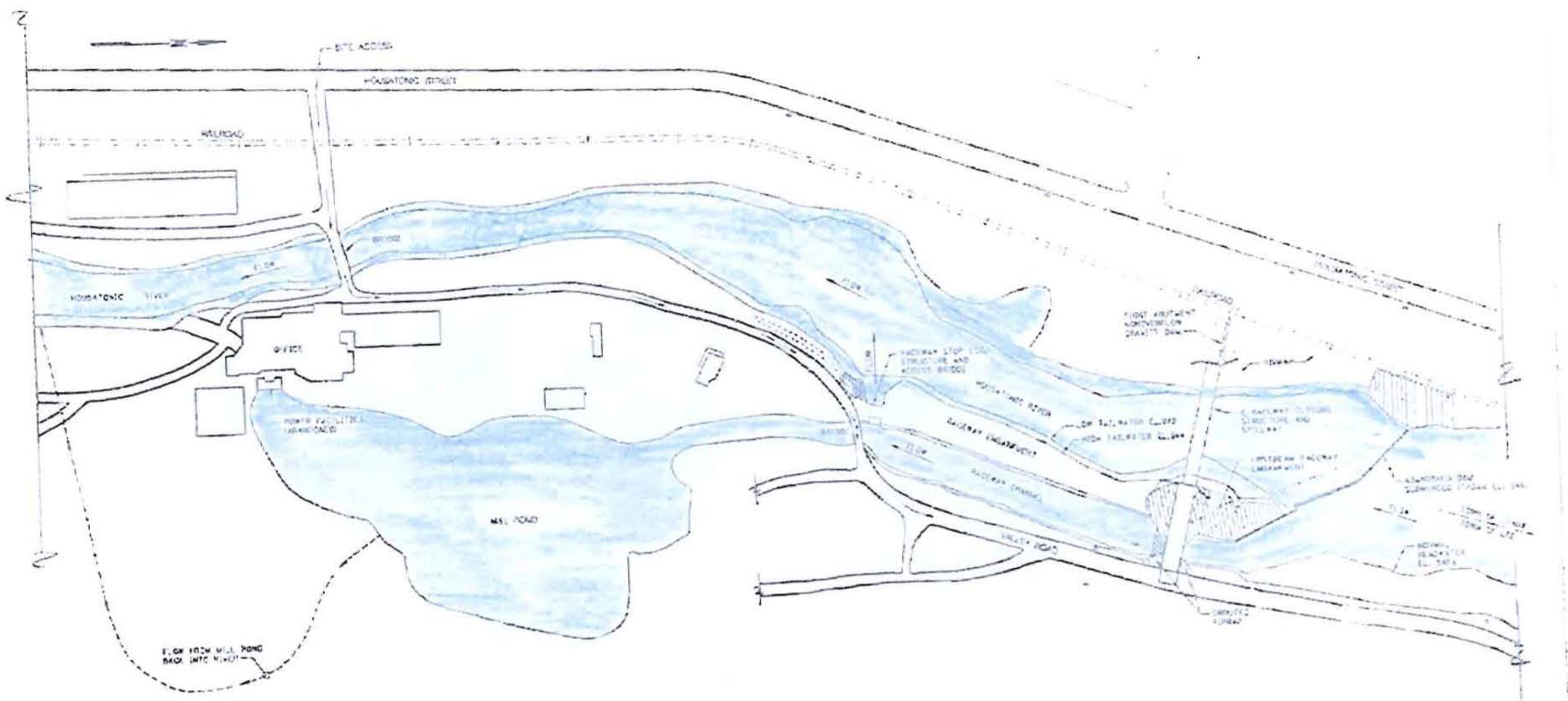


SOURCE: USGS QUADRANGLE MAPS



GENERAL ELECTRIC COMPANY PITTSFIELD, MASSACHUSETTS		
WOODS POND DAM	MODIFICATIONS	
PROJECT PLAN		
PREPARED BY: MWH		
CHICAGO, ILLINOIS	DATE	DWG.NO.

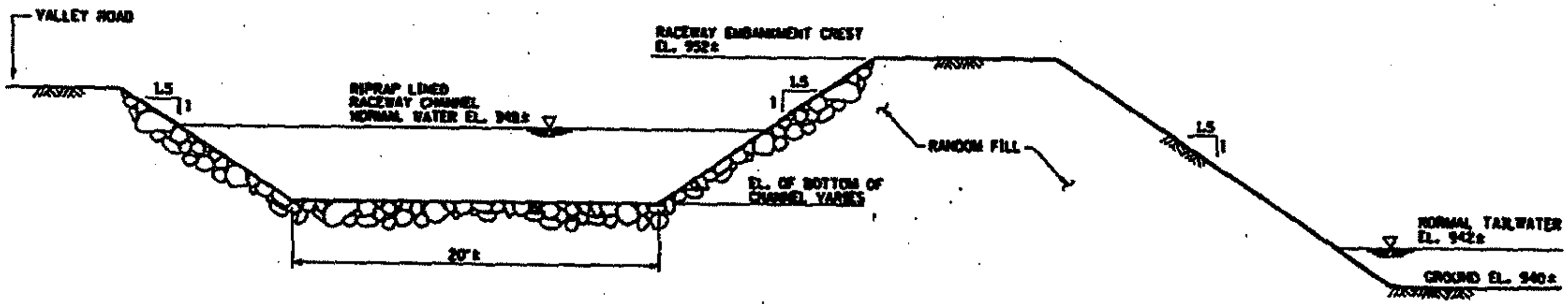
EXHIBIT 2



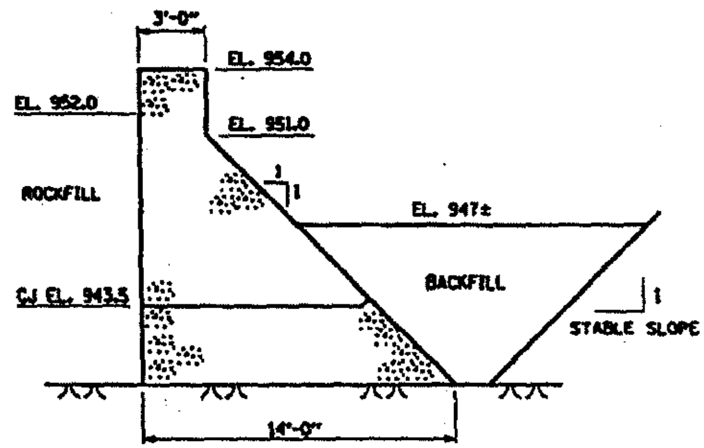
GENERAL SITE PLAN

No Scale

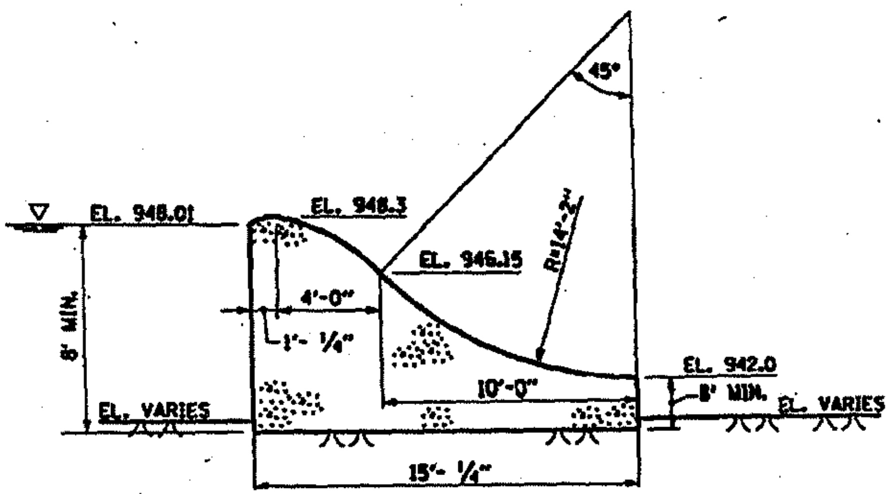
GENERAL ELECTRIC COMPANY WOODS POND DAM
GENERAL PLAN
HARZA Engineering Company
Date Dwg. No.



TYPICAL SECTION THROUGH RACEWAY CHANNEL AND RACEWAY EMBANKMENT



TYPICAL SECTION NON-OVERFLOW GRAVITY SECTION



TYPICAL SECTION OVERFLOW SPILLWAY

GENERAL ELECTRIC COMPANY
WOODS POND DAM

TYPICAL CROSS SECTIONS

MWH
MONTGOMERY WATSON HARZA

DATE	November 2002	DWG. NO.	1349K

EXHIBIT 3

Woods Pond Dam Piezometer Readings

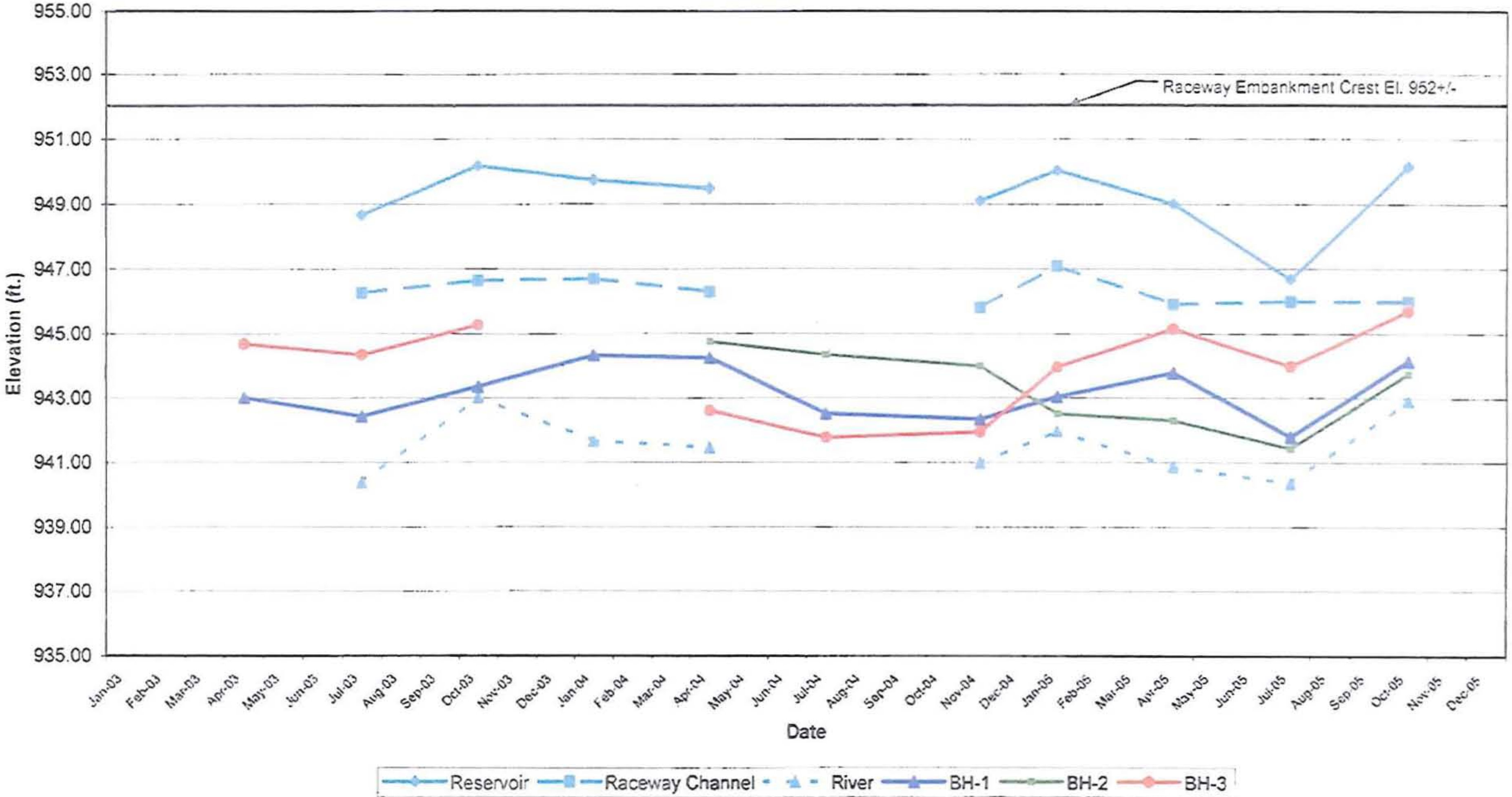
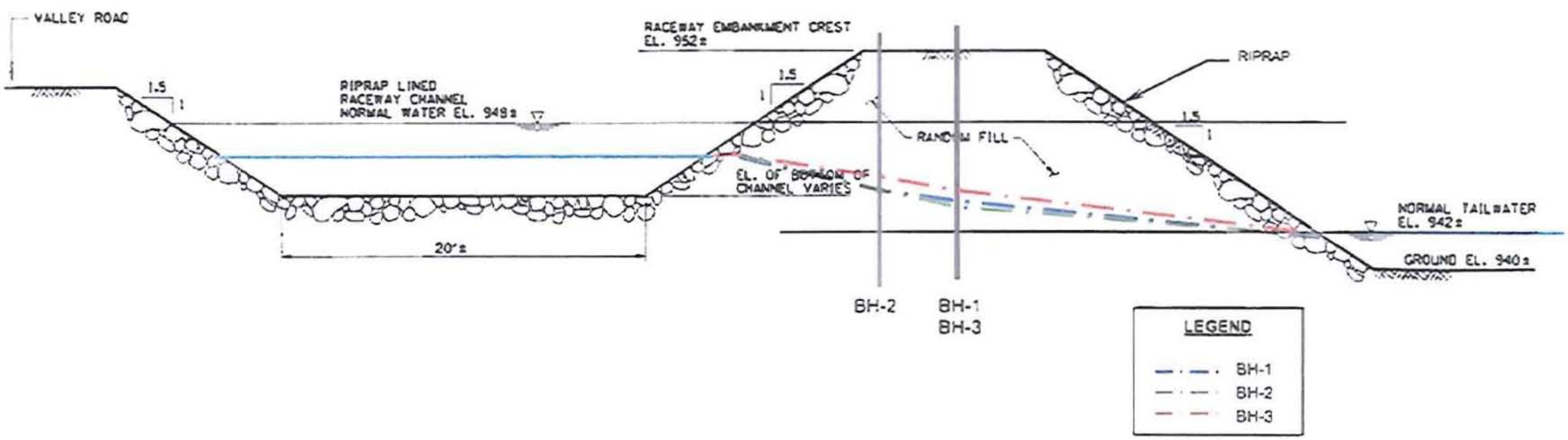
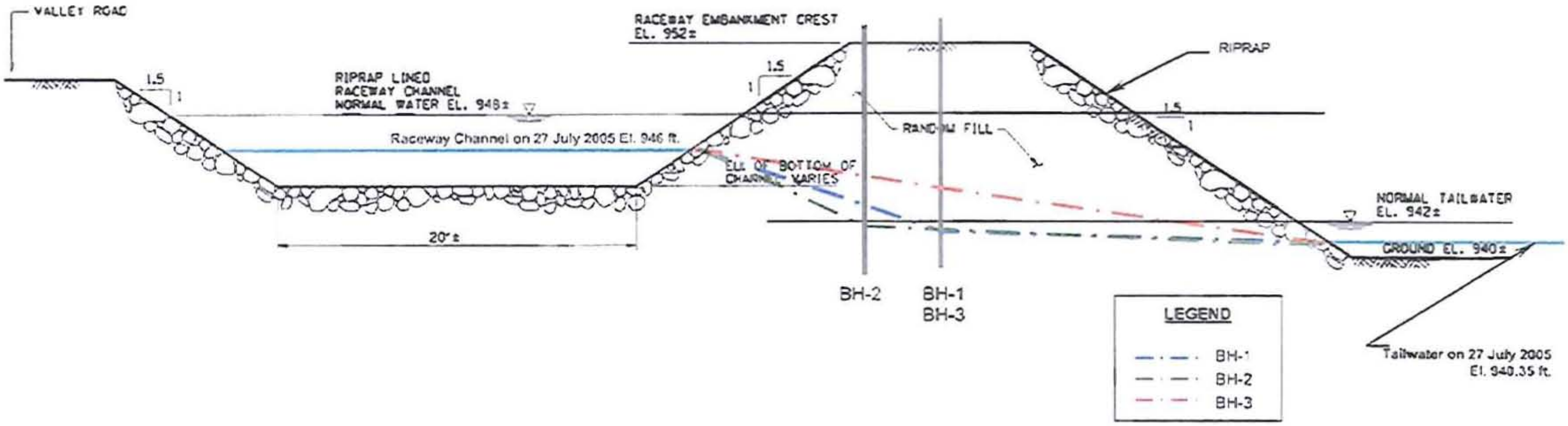


EXHIBIT 4



TYPICAL SECTION THROUGH RACEWAY CHANNEL AND RACEWAY EMBANKMENT
SHOWING PIEZOMETERS BH-1, BH-2, AND BH-3
28 OCTOBER 2005 READINGS

EXHIBIT 5



TYPICAL SECTION THROUGH RACEWAY CHANNEL AND RACEWAY EMBANKMENT
SHOWING PIEZOMETERS BH-1, BH-2, AND BH-3
27 JULY 2005 READINGS

EXHIBIT 6

PHOTOGRAPHS

LIST OF PHOTOGRAPHS

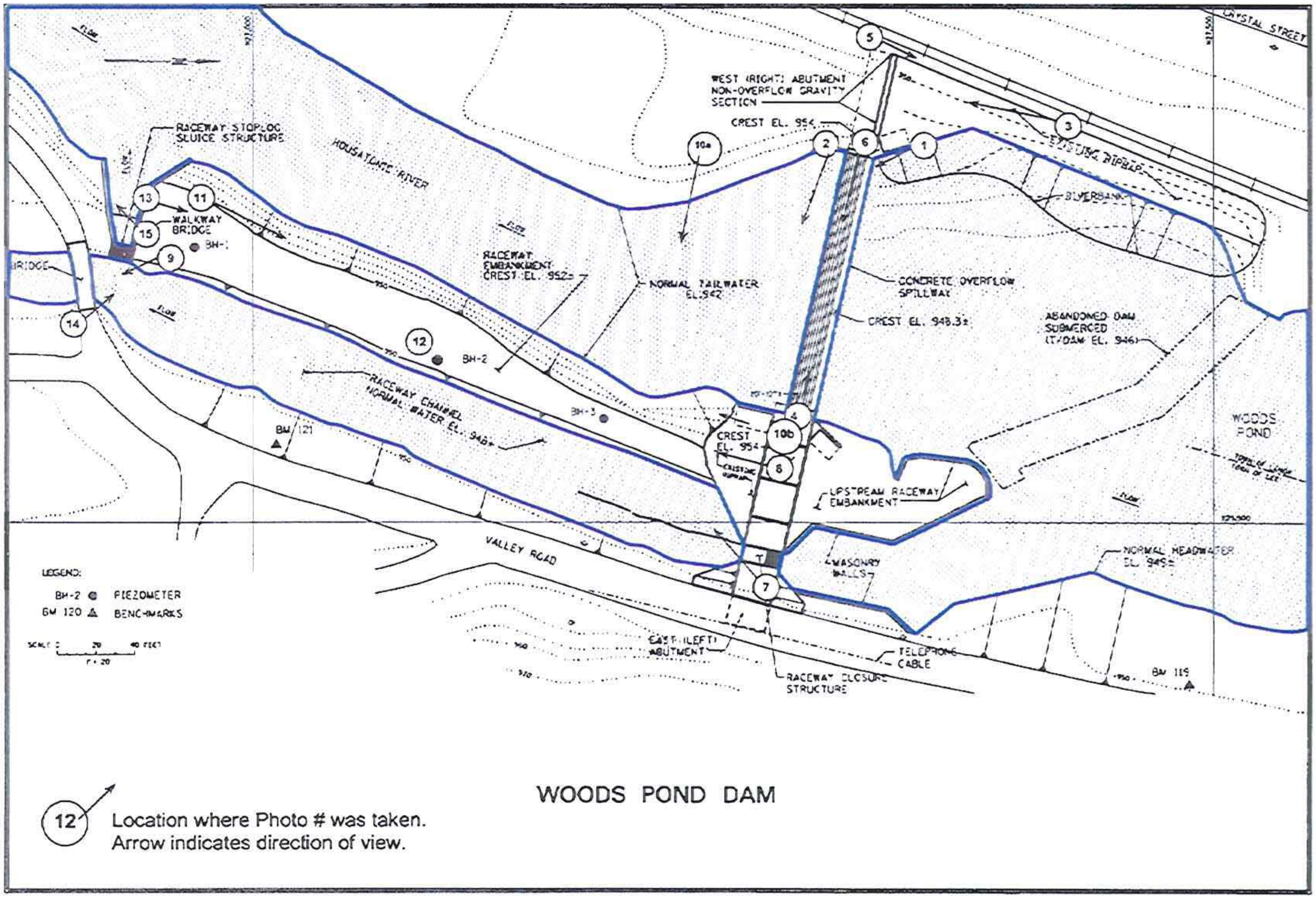


Photo Location Map

Photo 1 View of reservoir, overflow spillway and left.walls and abutment from upstream of right abutment training wall.

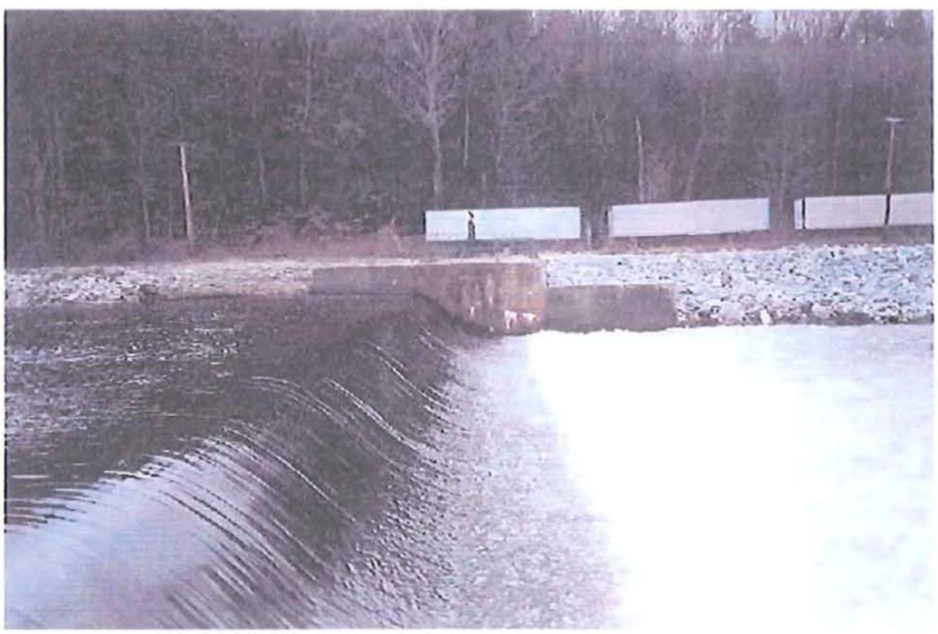


Photo 2 View of overflow spillway and left training wall from right training wall.

Photo 3 **Riprap protection along right bank upstream of overflow spillway; railroad at right abutment. Viewed from the right bank upstream of the overflow spillway.**

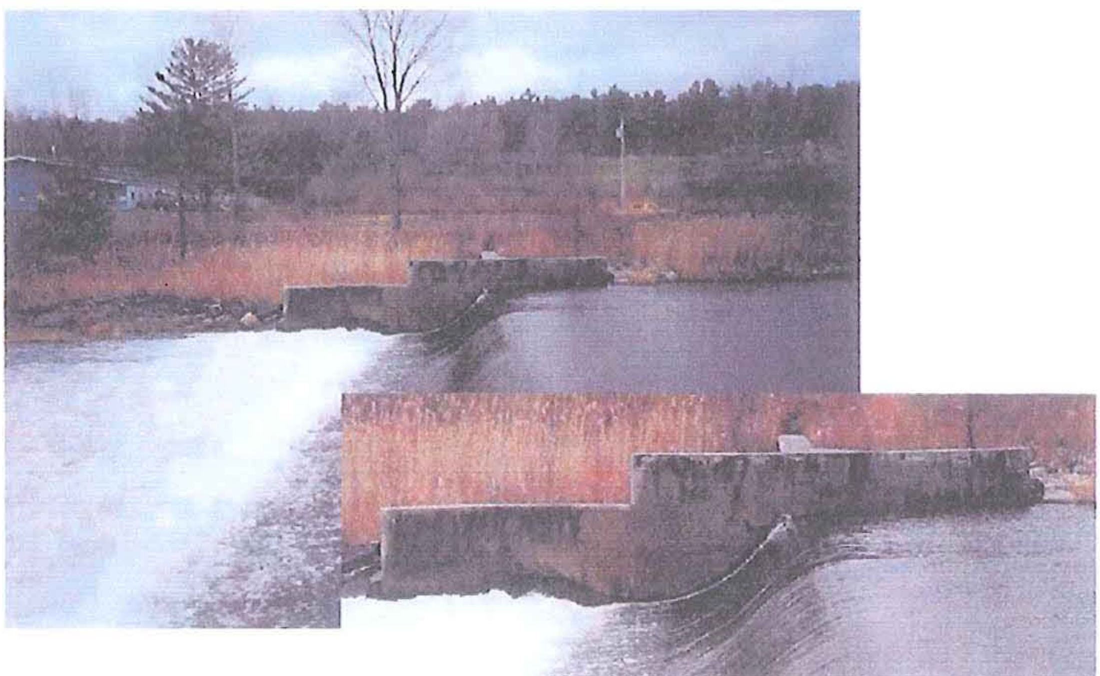


Photo 4 **View of right training wall from raceway closure structure**

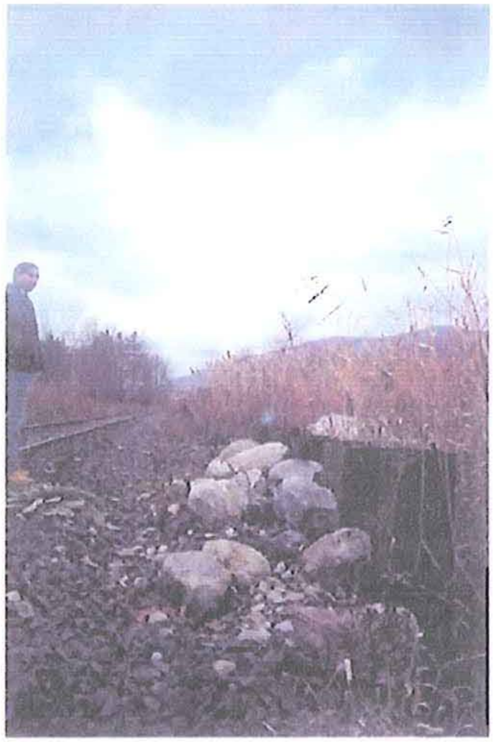


Photo 5 View of riprap in railroad area on right (west) abutment looking upstream (north).

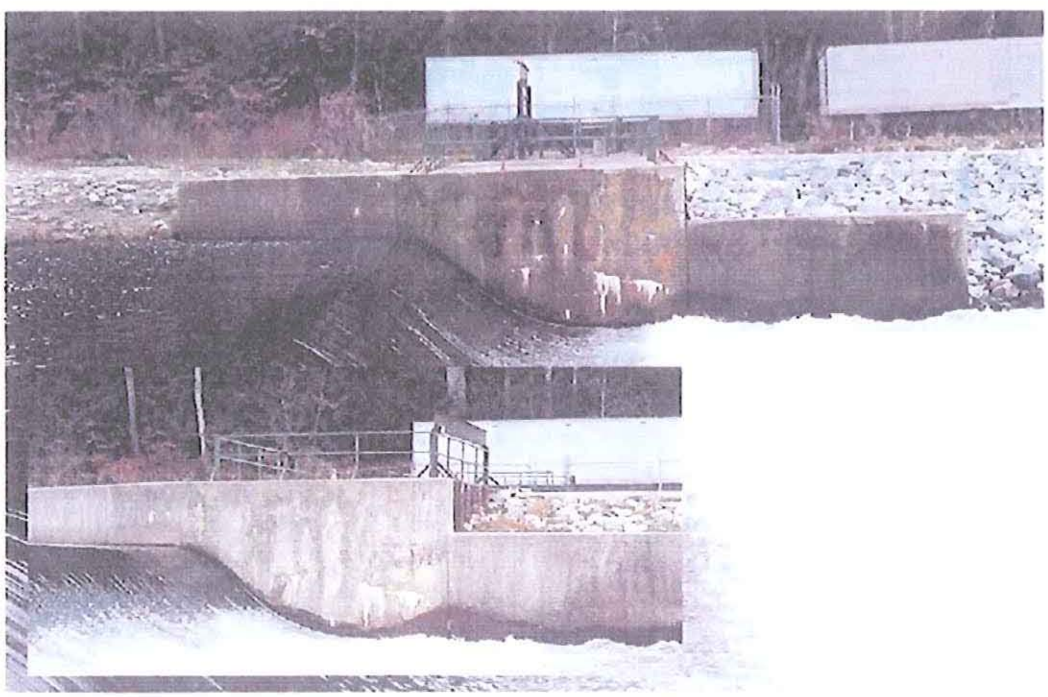


Photo 6 View of overflow spillway and left concrete training wall from right training wall. Inset is photo taken in 2004 for comparison.

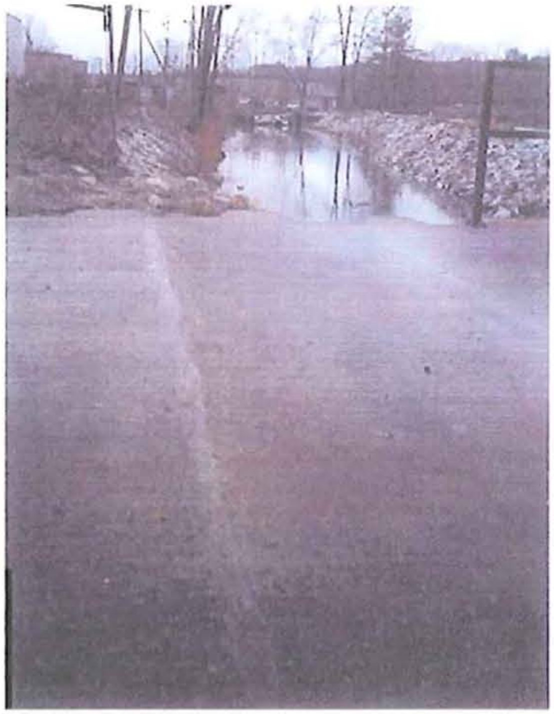


Photo 7 View of concrete cap on raceway closure structure from upstream (north) edge looking downstream (south).

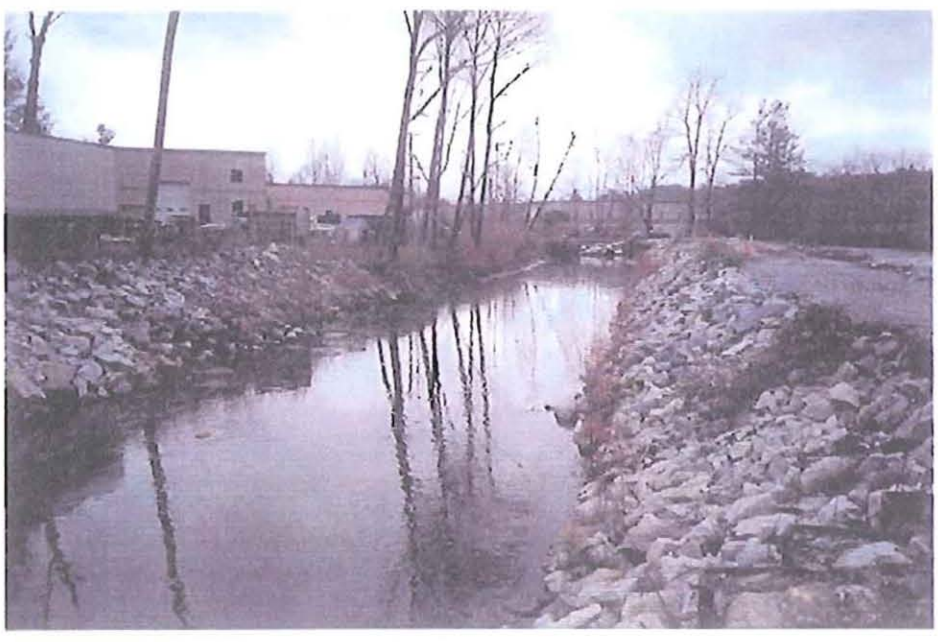


Photo 8 View of raceway channel from raceway closure structure looking downstream; thin layer of ice on water.

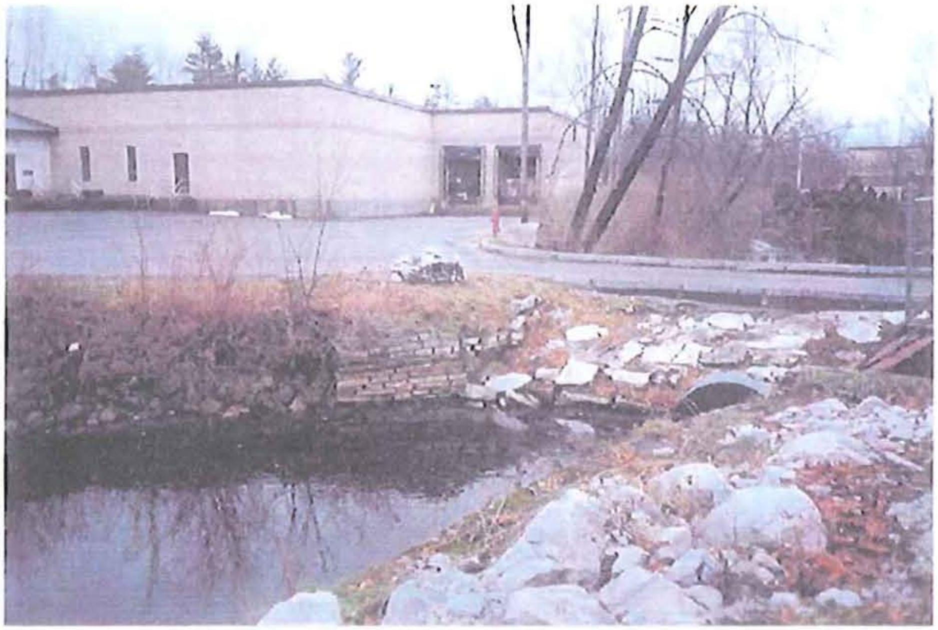
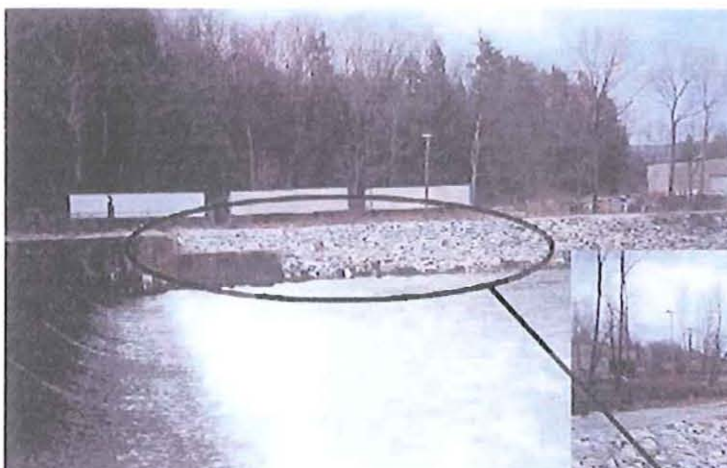


Photo 9 **View of downstream end of raceway channel from crest of raceway embankment.**



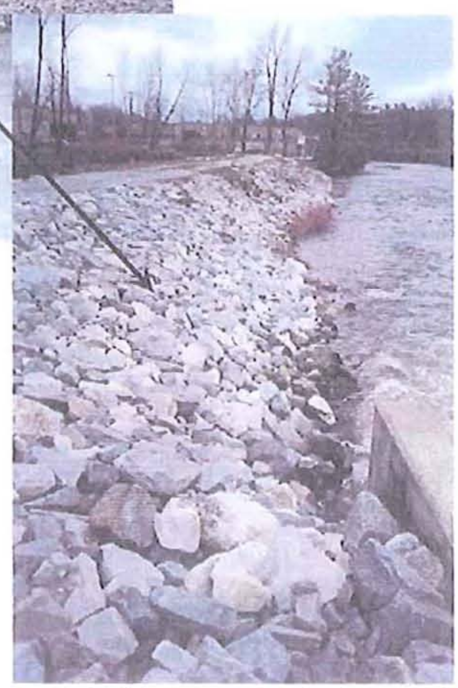


Photo 10a **View of riverside of raceway embankment from downstream toe of right (west) training wall. Note newly riprapped area.**

Photo 10b **View from closure structure along riprapped riverside slope of raceway embankment looking downstream (south).**

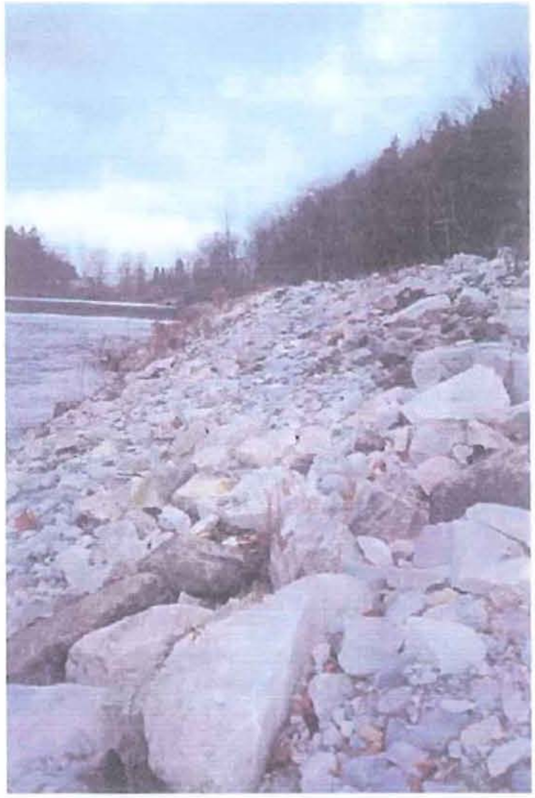


Photo 11 **View of riprapped riverside of raceway embankment from the downstream (south) end looking upstream (north).**

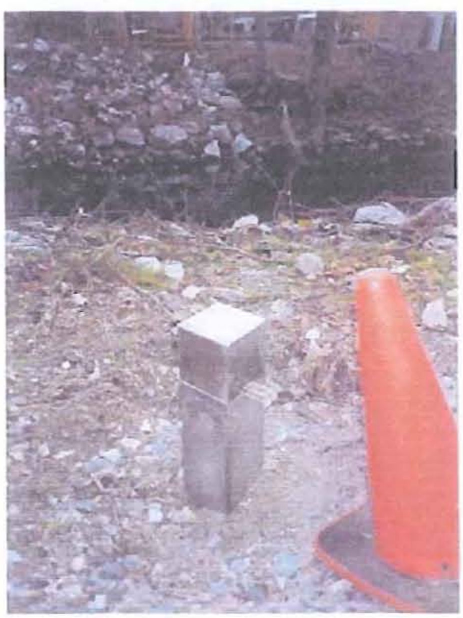


Photo 12 **Orange cone identifying piezometer locations on crest.**

Photo 13 View of crest of raceway embankment from downstream (south) end near raceway sluice structure looking upstream (north). Orange cone shows indicates location of crest piezometer.

Photo 14 Upstream face of raceway stoplog sluice structure. Note damage at waterline and vertical crack (with vegetation) on right wall.

Photo 15 **Flow observed at base of vertical crack in left downstream training wall downstream of raceway stoplog sluice structure.**

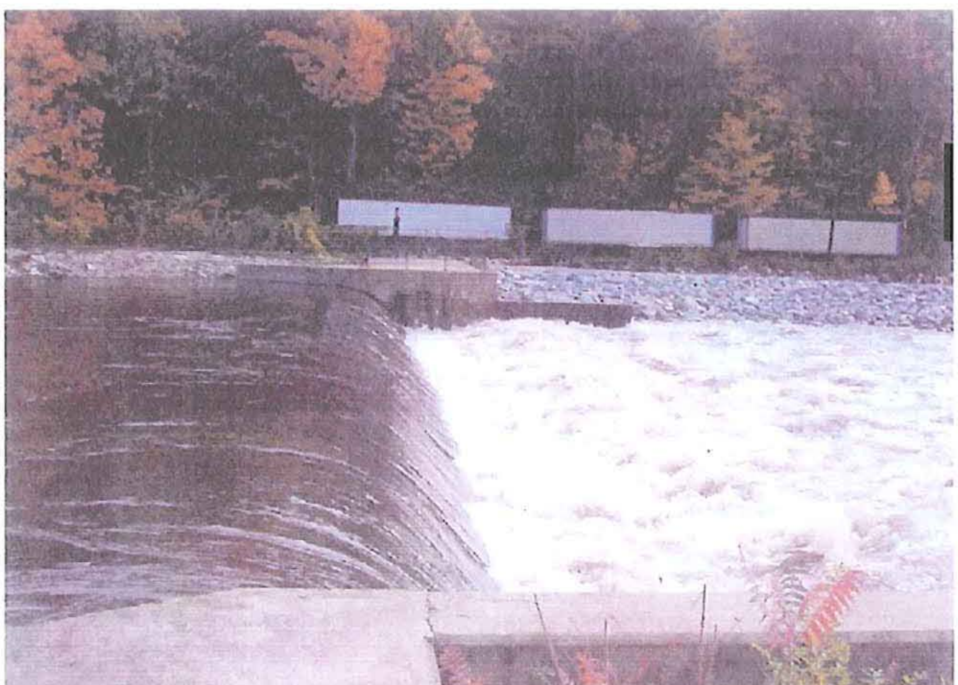


Photo 16 **View from right (west) abutment during October 2005 high flow event.**

Photo 17 **View of downstream river channel from right (west) abutment during October 2005 high flow event.**

APPENDICES

APPENDIX A

PROJECT FEATURE CONDITION INDEX SCALE

PROJECT FEATURE CONDITION SCALE

CONDITION	DESCRIPTION
Excellent	No noticeable defects. Some aging or wear may be visible.
Good	Only minor deterioration or defects are evident.
Fair	Some deterioration or defects are evident, but function is not significantly affected.
Marginal	Moderate deterioration. Function is still adequate.
Poor	Serious deterioration in at least some portions of the structure. Function is inadequate.
Very Poor	Extensive deterioration. Barely functional.
Failed	No longer functions. General failure or complete failure of a major structural component.

Note: Condition Scale taken from REMR Condition Index Scale, "The REMR Condition Index: Condition Assessment for Maintenance Management of Civil Works Facilities", REMR TN OM-CI-1.2.

APPENDIX B

U.S. GEOLOGICAL SURVEY STREAMFLOW DATA

U. S. Geological Survey Massachusetts - Rhode Island Water Science Center

PEAK RIVER STAGES, DISCHARGES, AND APPROXIMATE RECURRENCE INTERVALS DATA FOR FLOODS OF OCTOBER 8 - 19, 2005 (updated 10/24/2005)

USGS Gaging Stations Where Flood Recurrence Intervals Equaled or Exceeded 2 Years

(Data are provisional and are subject to revisions)

Note: A 5-year flood has a one in five chance of occuring in any one year; a 100-year flood has a one in one hundred chance of occuring in any one year

Station Number	Station name	Date of Peak	Time of Peak (hours)	Peak Gage Height (feet)	Peak Discharge (cubic feet per second)	Flood Recurrence Interval (years)	Start Year Of Gaging Station
MERRIMACK RIVER BASIN							
01094400	NORTH NASHUA RIVER AT FITCHBURG, MA	15-Oct	0800	7.11	2,360	5	1935
01096000	SQUANNACOOK RIVER NEAR WEST GROTON, MA	15-Oct	2130	6.73	2,170	5	1949
01097000	ASSABET RIVER AT MAYNARD, MA	16-Oct	0730	5.48	1,460	3	1941
01099500	CONCORD R BELOW R MEADOW BROOK, AT LOWELL, MA	19-Oct	0400	7.47	2,550	2	1936
01100000	MERRIMACK RIVER BL CONCORD RIVER AT LOWELL, MA	16-Oct	2015	53.51	55,500	5	1923
01100600	SHAWSHEEN RIVER NEAR WILMINGTON, MA	16-Oct	1100	7.04	630	3	1963
MYSTIC RIVER BASIN							
01102500	ABERJONA RIVER AT WINCHESTER, MA	15-Oct	2100	12.99	467	3	1939
CHARLES RIVER BASIN							
01103500	CHARLES RIVER AT DOVER, MA	19-Oct	0215	6.09	1,840	5	1937
01104500	CHARLES RIVER AT WALTHAM, MA	15-Oct	1715	4.1	1,310	2	1931
NEPONSET RIVER BASIN							
01105000	NEPONSET RIVER AT NORWOOD, MA	15-Oct	1500	9.96	860	10	1939
01105500	EAST BRANCH NEPONSET RIVER AT CANTON, MA	15-Oct	1430	5.88	1,100	10	1952
WEYMOUTH BACK RIVER BASIN							
01105600	OLD SWAMP RIVER NEAR SOUTH WEYMOUTH, MA	15-Oct	1545	5.92	458	10	1966
SOUTH COASTAL RIVER BASIN							
01105870	JONES RIVER AT KINGSTON, MA	15-Oct	2245	4.99	297	5	1966
TAUNTON RIVER BASIN							
01108000	TAUNTON RIVER NEAR BRIDGEWATER, MA	17-Oct	1200	12.38	3,860	25	1929-76, 1985-88, 1996 -
01109000	WADING RIVER NEAR NORTON, MA	16-Oct	0300	10.79	1,060	25	1925
BLACKSTONE RIVER BASIN							
01109730	BLACKSTONE RIVER, W. MAIN ST., AT MILLBURY, MA	15-Oct	0930	11.76	5,960	unknown	2002
01110000	QUINSIGAMOND RIVER AT NORTH GRAFTON, MA	15-Oct	1600	4.61	657	50	1939
01110500	BLACKSTONE RIVER AT NORTHBRIDGE, MA	15-Oct	1600	13.65	7,410	25	1939-77, 1995 -
01112500	BLACKSTONE RIVER AT WOONSOCKET, RI	15-Oct	2400	15.34	16,400	25	1929
MOSHASSUCK RIVER BASIN							
01114000	MOSHASSUCK RIVER AT PROVIDENCE, RI	15-Oct	1000	8.26	2,020	25	1963
WOONASQUATUCKET RIVER BASIN							
01114500	WOONASQUATUCKET RIVER AT CENTERDALE, RI	16-Oct	0400	4.86	1,670	50	1941
PAWTUXET RIVER BASIN							
01116000	SOUTH BRANCH PAWTUXET RIVER AT WASHINGTON, RI	16-Oct	0400	4.86	1,670	20	1940
01116500	PAWTUXET RIVER AT CRANSTON, RI	15-Oct	1430	13.68	4,260	40	1939
PAWCATUCK RIVER BASIN							
01117420	USQUEPAUG RIVER NEAR USQUEPAUG, RI	16-Oct	0730	8.09	932	10	1959, 1976 -
01117468	BEAVER RIVER NEAR USQUEPAUG, RI	15-Oct	1345	3.97	321	10	1974
01117500	PAWCATUCK RIVER AT WOOD RIVER JUNCTION, RI	16-Oct	1615	5.98	1,110	10	1940

MILLERS RIVER BASIN							
01162500	PRIEST BROOK NEAR WINCHENDON, MA	16-Oct	0330	5.68	589	5	1916
01166500	MILLERS RIVER AT ERVING, MA	15-Oct	1345	7.09	5,930	5	1915
DEERFIELD RIVER BASIN							
01168500	DEERFIELD RIVER AT CHARLEMONT, MA	9-Oct	0415	13.53	25,700	10	1913
01169000	NORTH RIVER AT SHATTUCKVILLE, MA	9-Oct	0515	12.32	18,800	100	1939
01170000	DEERFIELD RIVER NEAR WEST DEERFIELD, MA	9-Oct	0730	13.35	37,500	15	1940
01170100	GREEN RIVER NEAR COLRAIN, MA	9-Oct	0500	9.14	6,030	150	1967
CONNECTICUT RIVER BASIN							
01170500	CONNECTICUT RIVER AT MONTAGUE CITY, MA	9-Oct	1045	35.04	121,000	5	1904
01171500	MILL RIVER AT NORTHAMPTON, MA	9-Oct	0415	13.95 [a]	8,860 [a]	200 *	1938
					New peak discharge		
			Previous peak discharge		**6,300 cfs, 8/19/55**		
CHICOPEE RIVER BASIN							
01173500	WARE RIVER AT GIBBS CROSSING, MA	15-Oct	1815	8.54	5,310	10	1912
01174500	EAST BRANCH SWIFT RIVER NEAR HARDWICK, MA	15-Oct	1930	21.38	1,210	5	1937
01175670	SEVENMILE RIVER NEAR SPENCER, MA	15-Oct	0545	12.95	400	25	1960
01176000	QUABOAG RIVER AT WEST BRIMFIELD, MA	15-Oct	0600	9.32	3,500	25	1912
01177000	CHICOPEE RIVER AT INDIAN ORCHARD, MA	15-Oct	1430	12.92	12,000	10	1928
WESTFIELD RIVER BASIN							
01181000	WEST BRANCH WESTFIELD RIVER AT HUNTINGTON, MA	9-Oct	0300	14.35	27,900	5-10 *	1935
					New peak discharge		
			Previous peak discharge		**26,100 cfs, 8/19/55**		
01183500	WESTFIELD RIVER NEAR WESTFIELD, MA	9-Oct	0900	16.59	18,400	5	1914
FARMINGTON RIVER BASIN							
01185500	WEST BRANCH FARMINGTON RIVER NEAR NEW BOSTON,	9-Oct	0300	8.81	6,460	5	1914
HOUSATONIC RIVER BASIN							
01197000	EAST BRANCH HOUSATONIC RIVER AT COLTSVILLE, MA	9-Oct	0530	8.14 [b]	6,510 [b]	5-10 *	1936
					New peak discharge		
			Previous peak discharge		**6,400 cfs, 9/21/38**		
01197500	HOUSATONIC RIVER NEAR GREAT BARRINGTON, MA	9-Oct	1530	10.34	8,080	20	1913
HUDSON RIVER BASIN							
01332500	HOOSIC RIVER NEAR WILLIAMSTOWN, MA	8-Oct	2200	12.36	9,840	25	1940
		Prior recurrence interval excluding October 2005 peaks					
01171500	MILL RIVER AT NORTHAMPTON, MA					>500	
01181000	WEST BRANCH WESTFIELD RIVER AT HUNTINGTON, MA					100	
01197000	EAST BRANCH HOUSATONIC RIVER AT COLTSVILLE, MA					50	
* Return periods for these stations were recalculated following Bulletin 17-B guidelines from the annual series of peak flows including the most recent October 2005 peaks. This assumes that the most recent peaks are the maximum flow for the 2006 water year, and thus are subject to change.							
[a]	Based on peak-stage indicator						
[b]	Values may be higher pending results of survey of high-water marks						

APPENDIX C

REVIEW BY OTHERS OF PREVIOUS REPORTS

REVIEW BY OTHERS OF PREVIOUS REPORTS

The Lead Administrative Trustee (LAT) requested Woodlot Alternatives, Inc. (Woodlot) to review the Woods Pond Dam Structural Integrity Assessments performed in 2000, 2002, and 2004. Based on their review, several items were identified for which clarification was requested. These are discussed below.

C1.1 Apparent Elevation Discrepancies

Woodlot noted apparent discrepancies between water surface elevations presented in Section 3.0 of the 2002 Structural Integrity Assessment Report (2002 Report) and those presented in the hydraulic analyses in Appendix C of the 2002 Report. The apparent discrepancies resulted from conservative assumptions used in hydraulic analyses in Appendix C of the 2002 Report. The elevations presented in Section 3.0 of the 2002 Report are correct based on available information. The primary purpose of the hydraulic analyses performed and presented in Appendix C of the 2002 Report were to obtain the duration of overtopping of the railroad bed west of the end of the right (west) abutment. The inflow hydrograph was conservatively developed using rainfall in the watershed and through calibration of the hydrologic model to match recorded peak flows at the downstream USGS gaging station at Great Barrington. The elevations and calculations have been revised to eliminate the discrepancy and any potential confusion and are presented in Appendix D of this report. The revised elevations do not affect or alter the previous results or conclusions.

C1.2 Railroad Ballast Stability

A question was raised regarding the average versus maximum particle size of the railroad ballast and its stability under overtopping flow. A missing subscript in the 2002 Report inadvertently led to a comparison of the required *median* stone size to the maximum size of the ballast. The calculations for the required median stone size did not change, and remains 1.3 inches. As shown on Table 1 in this section, a review of typical railroad ballast gradations recommended by the American Railway Engineering and Maintenance of Way Association *(AREMA)* shows the closest match for the materials observed at Woods Pond Dam is the gradation for material 4A with a maximum particle size of 2½ inches. The median size for this 4A material is 1.5 inches, larger than the 1.3 inches required. This matches MWH's field observations which indicate the material to have a maximum particle size of about 3 inches and a median size of about 1.5 inches (see Photo C1). The revised calculation sheets correcting the typographical error are included in

Appendix D of this report.

Table 1 Ballast Gradations per AREMA

Table 6-8 Recommended Ballast Gradations

Size No	Nominal Size Square Opening (in.)	Amounts Finer Than Each Sieve (Square Opening) Percent by Weight							
		2-1/2 in.	2 in.	1 1/2 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4
4A	2 to 3/4	100	90-100	60-90	10-35	0-10		0-3	
4	1-1/2 to 3/4		100	90 100	20 55	0 15		0-5	
5	1 to 3/8			100	90-100	40-75	15-35	0-15	0-5

(2) For smaller projects, where less than 200 tons of ballast is needed, and where the nearest suppliers do not stock AREMA gradations, the following AASHTO (highway) gradations may be substituted: CA5 for AREMA 4 or 4A, and CA7 for AREMA 5

Source: Reference 12.

Photo C1 - Typical Railroad Ballast

C1.3 Overtopping of Right Abutment/Railroad Bed

Woodlot noted that the railroad bed at the right abutment will be overtopped by less than a 50-year event and questioned whether the existing slope protection downstream of the west (right) abutment was sufficient. Stability analyses for the west abutment show that the west abutment structure meets the required factors of safety under flood loading conditions without any of the downstream earthfill being in place. The design of the west abutment appears to have considered the possibility that the backfill downstream of the west abutment structure might erode during a flood, and designed the structure to meet required factors of safety under that loading condition. The potential erosion of the railroad embankment was reviewed as well and is discussed in the following section.

C1.4 Potential for Railroad Embankment Erosion

A question was raised regarding the possibility for scouring of the fill downstream of the west (right) abutment, compromising the integrity of the railroad grade, and ultimately mobilizing sediments in the upstream impoundment. The situation was reviewed and evaluated as a potential failure mode for floods up to and including the 500-year flood.

Because of the rapid rise in tailwater levels, the available energy head at the 500-year flood flow is small at 3.1 feet and it discharges into a very high tailwater pool at El. 951.5 ft. Scour would stop once equilibrium was reached, which would not extend far below El. 951.5 ft. Erosion and scour also would be resisted by the railroad ballast (with a ballast size up to 3") and by vegetation. Vegetation is used in some spillways for erosion protection for flow velocities of 3 to 5 feet per second. The steel sheetpile at the end of the dam would also limit undermining and erosion of the railroad embankment adjacent to the dam. There could be sloughing of the railroad embankment laterally into a scour hole, downstream of the sheetpile, with erosion then proceeding in the upstream direction (headcutting). To connect to the reservoir, there is a very large volume of material in the railroad embankment that would have to be eroded and moved, which is unlikely with short duration and low energy flow associated with floods up to the 500-year flood.

Moreover, the entire right bank of the river upstream of the abutment between the old dam and the new abutment was built-up with a rockfill berm on top of the railroad embankment, with a minimum crest width of 10 feet. The crest of the rockfill berm immediately upstream of the west abutment is much wider and extends the full 54-foot width of the concrete abutment. This is large riprap that would not be subject to erosive forces, and was placed in large quantities.

Nevertheless, some small potential for erosion and scour exists. To be conservative, GE could consider minimizing the potential for scour and erosion downstream of the west (right) abutment by placing riprap in this area for a distance of approximately 20 to 40 feet (Appendix D).

APPENDIX D

UPDATED HYDRAULIC ANALYSIS OF RAILROAD BED OVERTOPPING

Notes for Updated Hydraulic Analysis

To conduct the analysis of the railroad bed overtopping during a flood event, it was necessary to create a simulated inflow hydrograph that could be routed through the reservoir in order to obtain the depth and duration of overtopping at the railroad embankment. NOAA's TP-40 Rainfall Frequency Atlas data were used as the starting point for this analysis. Runoff hydrographs were adjusted based on site specific conditions to result in peak flows that would match previously determined peak flows for various recurrence interval storms. A comparison of the output from this simulation model to data contained in this and previous reports shows the following:

Flow (cfs)

Event (return int-duration)	Report	Model	Difference	% Difference
500 yr-24 hr	12100	12113	13	0.11
100 yr-24 hr	8600	8626	26	0.30
50 yr-24 hr	-	7441	-	-

Stage (ft)

Event (return int-duration)	Report	Model	Difference	
500 yr-24 hr	955.8	955.68	-0.12	
100 yr-24 hr	954.6	954.50	-0.10	
50 yr-24 hr	-	953.98	-	

Based on this comparison, the simulated hydrograph and routing model were considered sufficiently accurate for our purposes. The slight differences in peak reservoir elevation between the model and the report are acknowledged here but are considered insignificant, and well within the accuracy of the modeling.

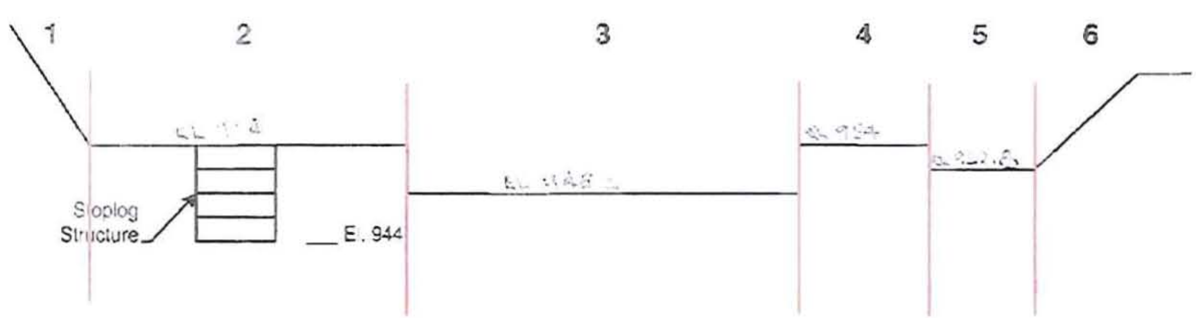
Revised Woods Pond Dam Stage-Storage-Discharge Curve

	Overflow ID	1	2	3	4	5	6	
	Overflow Elevation (ft)	957.00	954.00	948.30	954.00	952.80	954.40	
	Weir Length (ft)	10.0	114.0	140.0	55.0	20.0	50.0	
Elevation (ft)	Storage (acre-ft)	Valley Road (cfs)	Closure and Stoplog Structure (cfs)	Spillway Outflow (cfs)	West Abutment Overflow (cfs)	RR Overflow (cfs)	Crystal Road (cfs)	Combined Outflow (cfs)
948.30	0.00	0	0	0	0	0	0	0
948.50	0.03	0	0	40	0	0	0	40
949.00	0.10	0	0	273	0	0	0	273
949.50	0.18	0	0	634	0	0	0	634
950.00	0.25	0	0	1095	0	0	0	1096
950.50	0.32	0	0	1649	0	0	0	1648
951.00	0.39	0	0	2283	0	0	0	2283
951.50	0.47	0	0	2989	0	0	0	2989
952.00	0.54	0	0	3764	0	0	0	3764
952.50	0.61	0	0	4606	0	0	0	4606
953.00	0.69	0	0	5514	0	5	0	5519
953.50	0.76	0	0	6479	0	31	0	6510
954.00	0.83	0	0	7402	0	70	0	7472
954.50	0.91	0	108	8345	52	119	4	8624
955.00	0.98	0	304	9357	147	174	52	9982
955.50	1.05	0	559	10445	270	237	154	11511
956.00	1.12	0	861	11586	415	305	270	13168
957.00	1.27	0	1582	[illegible]	763	460	580	15969
958.00	1.42	27	2435	[illegible]	1175	633	912	20834
959.00	1.55	78	3403	[illegible]	1542	824	1317	25251
960.00	1.71	139	4473	[illegible]	2158	1032	1750	29877

Notes:

1. Spillway values taken from Appendix G of MWH March 1999 Inspection Report
2. All overflow sections, with except for the spillway section, assume a discharge coefficient of 2.67
2. Overflow lengths and elevations taken from Record Drawings.

D-9

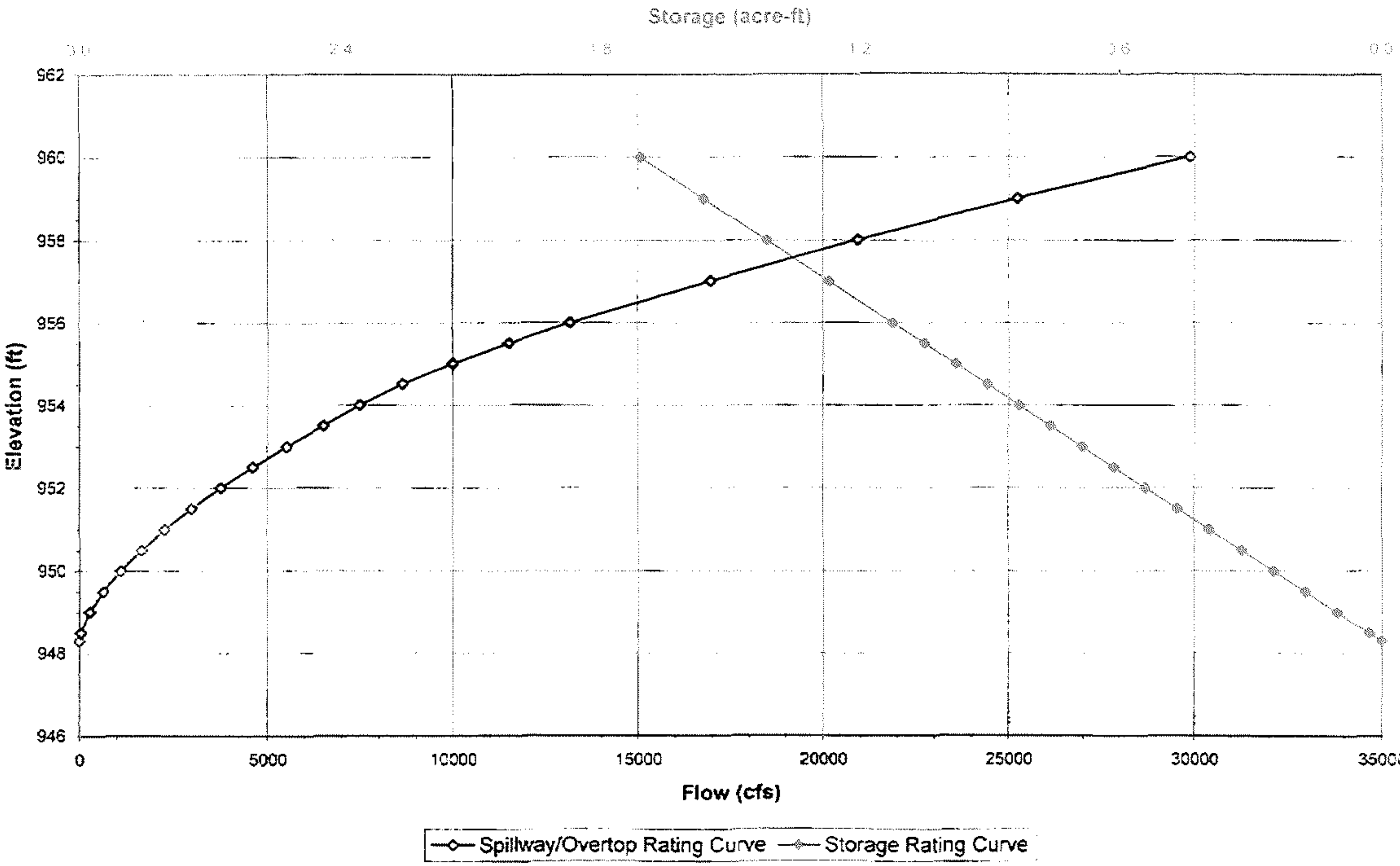


Crystal Road is also shown as Housatonic Road on older Drawings.

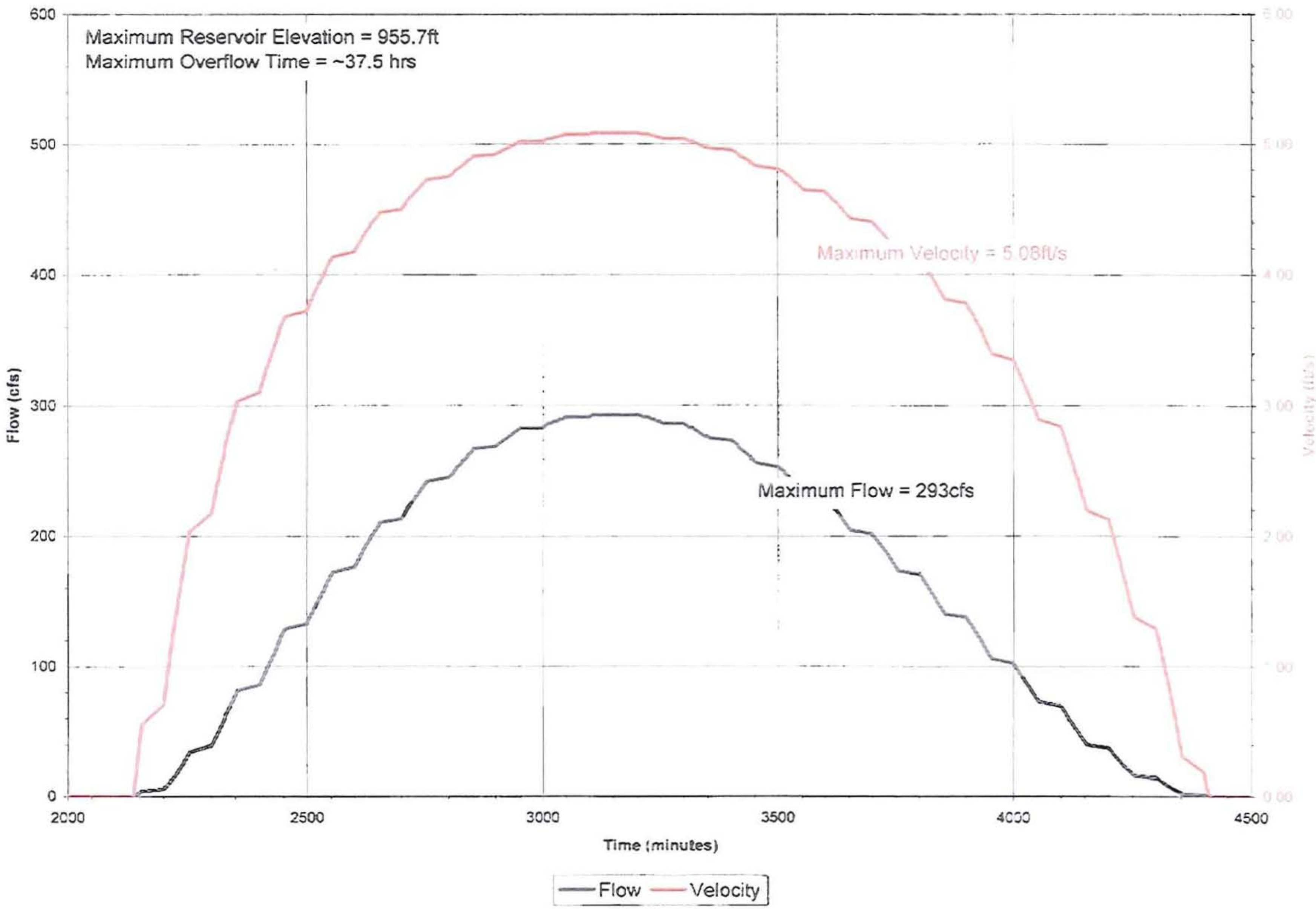
N.T.S.

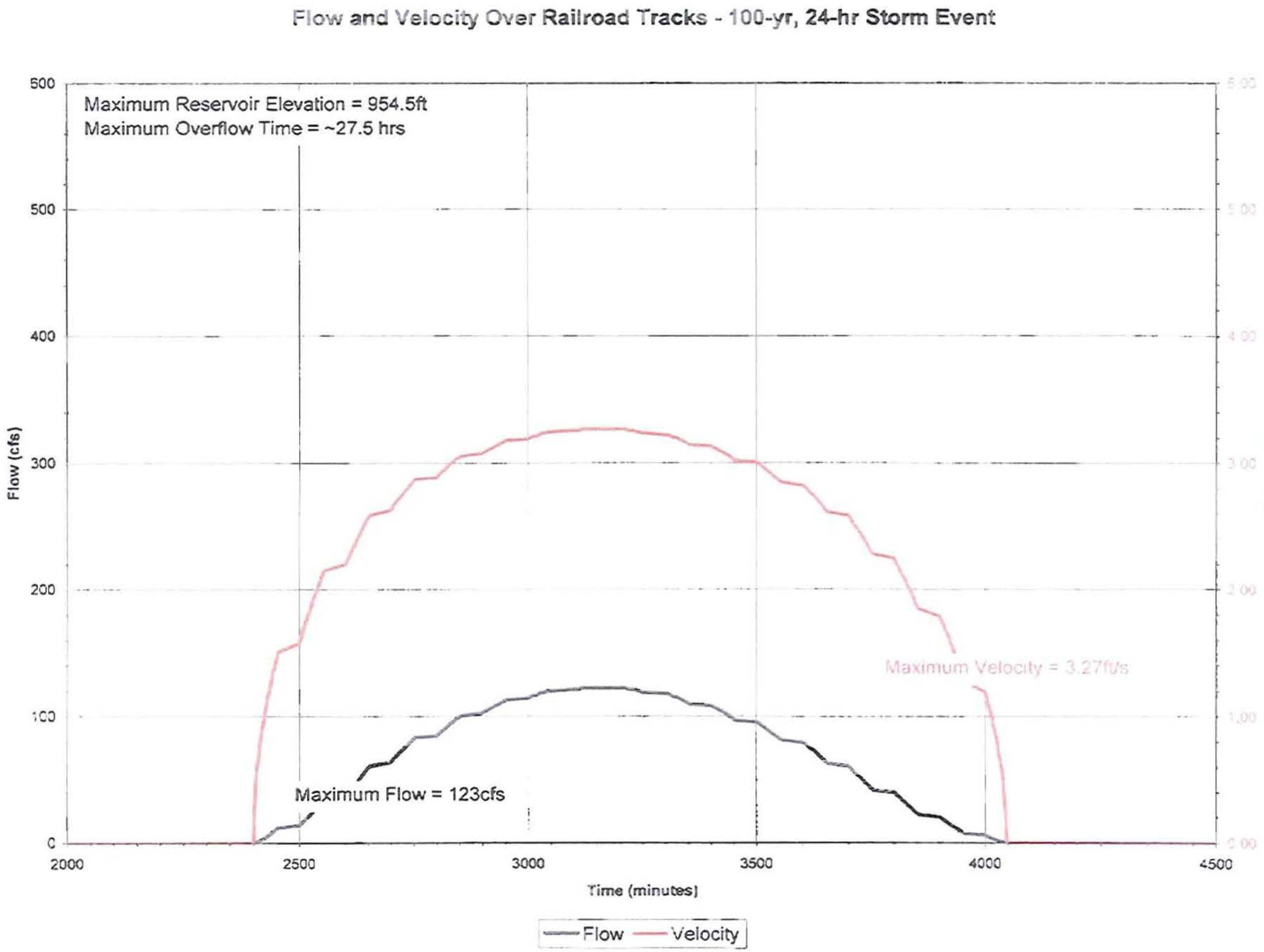
Hydraulic Analysis

Woods Pond Dam - Revised Rating Curves



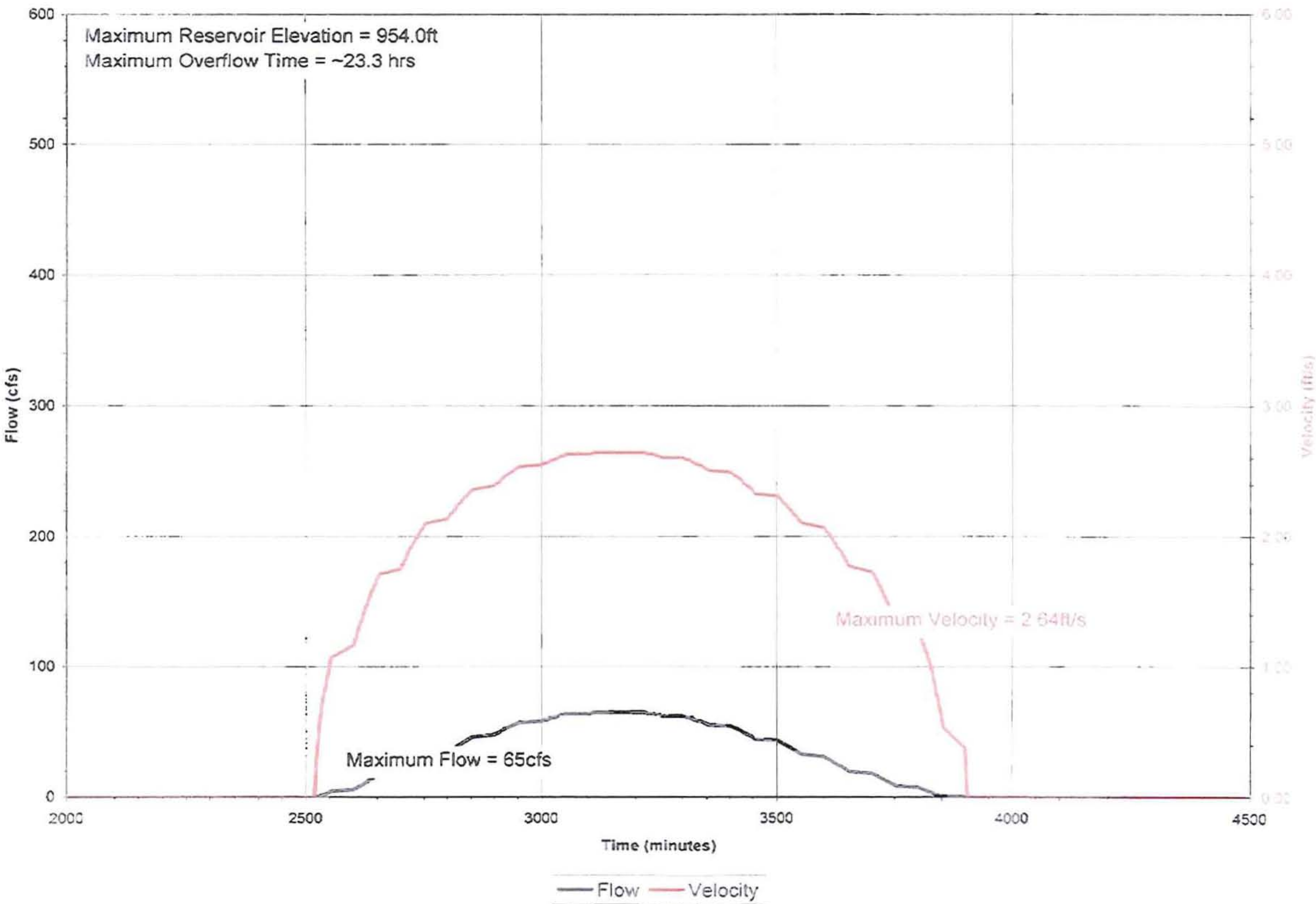
Flow and Velocity Over Railroad Tracks - 500-yr, 24-hr Storm Event





Flow and Velocity Over Railroad Tracks - 100-yr, 24-hr Storm Event
Maximum Reservoir Elevation = 954.5ft
Maximum Overflow Time = ~27.5 hrs
Maximum Velocity = 3.27ft/s
Maximum Flow = 123cfs
Flow (cfs)
Velocity (ft/s)
Time (minutes)
0
100
200
300
400
500
600
0.00
1.00
2.00
3.00
4.00
5.00
6.00
2000
2500
3000
3500
4000
4500
Flow
Velocity

Flow and Velocity Over Railroad Tracks - 50-yr, 24-hr Storm Event



MWH

MWH AMERICAS – GLOBAL ENERGY & INDUSTRY

COMPUTATION CHECKOUT SHEET

Project Name: Woods Pond Dam

Job Number: 1003423/1003559

Project Feature: Spillway

Purpose of Computation: Evaluation of Erosion Potential due to 500-Year Flood

Mario Finis/ Wade Moore	Justin Bartels/Manoshree Sundaram	Manoshree Sundaram/ Justin Bartels	Justin Bartels/Manoshree Sundaram
Supervisor (1)	Originator (1)	Checker (1)	Backchecker (1)

COMPUTATION CHECKOUT AND APPROVAL RECORD

Sequence Number	Responsible Individual	Procedure	Date Completed	Initials (2)
1	Supervisor	Check Originator's familiarity with DG-001. Discuss criteria, references, and planned procedures.	4/2/03	[signature]
2	Originator	Review material and study scope of computation. Prepare computation outline; list criteria, references, and planned procedures.	4/7/03-4/9/03 18 April 2003	JDB ms
3	Supervisor	Review and approve computation outline, criteria, and procedures.	4/18/03 4/9/03	[signature] WAM
4	Originator	Prepare computations, including sketches.	4/9/03 18 April 2003	JDB ms
5	Checker	Check computations.	4/30/03	JDB
6	Backchecker	Backcheck computations.	5/12/03 1 May 2003	JDB ms
7	Supervisor	Review, approve, and release checked computations and sketches for preparation of drawings, reports, or other purposes.	5/28/03 5/12/03	[signature] WAM

(1) Full name.
(2) In own handwriting (not printed).

Revised February 2006 [signature]

MWH
MONTGOMERY WATSON HARZA

By M. SUNDARAM Date 18 APRIL 2003 Client OF - PITTSFIELD Sheet 1 of 3

Chkd. By JDB 4/20/03 Description CHANNEL VELOCITY - WOODS POND Job No. 1003557.020101

OBJECTIVE : Determine if the anticipated velocities at the railroad embankment / low spot in the Woods Pond Dam profile are high enough to erode and/or transport the existing railroad ballast material.

ASSUMPTIONS : • Based on flow-velocity calculations completed in ~~April 2003~~ : January 2006
Velocity at RR for 100-year flood = ~~4 ft/s~~ 3.2 ft/sec
Velocity at RR for 500 year flood = ~~5 ft/s~~ 5.1 ft/sec
• Railroad Ballast is minus 3" material
(D_{50} estimated at 1.5" based on field observations)

REFERENCES : ① USACE, Hydraulic Design Criteria Sheet 712-1 "Stone Stability - Velocity vs. Stone Diameter".

② Handbook of Hydraulics, 7th ed.

CONCLUSIONS :

Using references ① & ②, the velocities anticipated at the railroad / low section at the Woods Pon Dam are not high enough to erode the railroad ballast material. In addition, head cutting is not assumed to be a problem since the downstream area is sufficiently protected by concrete slabs & riprap. Therefore no additional treatment or protection is required.

MWH
MONTGOMERY WATSON HARZA

By M. SUNDARAM Date 18 APRIL 2003 Client GE · PITTSFIELD Sheet 2 of 3

Chkd. By ______ Description CHANNEL VELOCITIES - WOODS POND Job No. 1003559

① $V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D_{50})^{1/2}$ ✓ MY

V = velocity, ft/s
C = coefficient = 1.20 for low turbulence ✓
g = gravity = 32.2 ft/s² ✓
γ_s = unit weight of stone = 160 lb/ft³ ✓
γ_w = unit weight of water = 62.4 lb/ft³ ✓
D_{50} = stone diameter in feet.

$$D_{50} = \left(\frac{V}{C\left[2g\left(\frac{\gamma_s-\gamma_w}{\gamma_w}\right)\right]^{1/2}}\right)^2 \quad ✓$$

For 100-year flood :

$$D_{50} = \left(\frac{4\ ft/s}{1.2\left[2(32.2\ ft/s^2)\left(\frac{160\ lb/ft^3 - 62.4\ lb/ft^3}{62.4\ lb/ft^3}\right)\right]^{1/2}}\right)^2$$

$$= \left(\frac{4\ ft/s}{1.2\ (100.7)^{1/2}}\right)^2$$

D_{50} = 0.11 ft ✓ ⇒ 1.3 in ✓ LESS THAN ~~3"~~ · OK ✓

FOR V_{100} = 3.3 fps, D_{50} REQ'D = 0.075 FT ⇒ 0.90 inches REQD

FOR V_{500} = 5.1 fps D_{50} REQD = 0.18 FT ⇒ 2.1 INCHES REQD MY

NOTE · V_{500} MAY BE OVERESTIMATED, SEE ATTACHMENT 3, PG 2, WHERE V_{500} = 4.27 fps, D_{50} REQD = 1.5"

Based on this review, the flow for the 100-yr flood can be easily withstood, while the flow from the 500-yr flood may cause some incipient motion of the bedding material. Because of this, some additional investigation and sensitivity analysis has been conducted.

The Isbash equation yields armor that is essentially 'non-moving' under the action of the design forces considered. The existing stone bedding, assumed D50 of 1.5 inches, would be considered non-moving for velocities up to 4.26 feet-per-second, which is greater than the 100-year flood but less than that for the 500-yr flood. There are several conservative inputs and assumptions in this analysis and in using the Isbash equation that should be considered. The required D50 is determined based on an average flow velocity, a recognized conservative assumption (see paragraph 4 and 5 of the attached Sheet 712-1, Attachment 1). The actual velocity at the boundary conditions, where the flow is actually in contact with the stones, is typically less than the average flow velocity, and sometimes as little as ½ the average velocity. Secondly, the Isbash equation itself may be conservative in this application. The Isbash equation is appropriate for specifying material in dis-equilibrium with existing channel conditions, and sizes stone for essentially no movement. This is typical when considering riprap and other local scour protection around piers or at the ends of stilling basins. For more channel-like conditions which may exist at the railroad bed due to the long horizontal approach distance and long, continuous horizontal stone bedding layer itself, stability methods for channel linings may be more appropriate.

There are various methods that can be used for stability of channel linings. Table 2-5 of EM-1110-2-1601 (Attachment 2) provides suggested maximum permissible mean channel velocities for various materials, and indicates that fine gravel (<3/4 inch in size) can withstand velocities up to 6 fps. Additional analysis was done using equations for stability of stone blankets in a current field from other references (Attachment 3). This analysis shows that stone with D50 of 1.5 inches would resist the expected velocities for the 100 and 500-yr floods.

Given these factors, the existing stone bedding at the railroad bed is considered acceptable. Some movement of the smaller stones could be expected, but significant erosion is not anticipated.

END OF SECTION

HYDRAULIC DESIGN CRITERIA

SHEET 712-1

STONE STABILITY

VELOCITY VS STONE DIAMETER

1. Purpose. Hydraulic Design Chart 712-1 can be used as a guide for the selection of rock sizes for riprap for channel bottom and side slopes downstream from stilling basins and for rock sizes for river closures. Recommended stone gradation for stilling basin riprap is given in paragraph 6.

2. Background. In 1885 Wilfred Airy[1] showed that the capacity of a stream to move material along its bed by sliding is a function of the sixth power of the velocity of the water.[1] Henry Law applied this concept to the overturning of a cube,[2] and in 1896 Hooker[2] illustrated its application to spheres. In 1932 and 1936 Isbash published coefficients for the stability of rounded stones dropped in flowing water.[3,4] The design curves given in Chart 712-1 have been computed using Airy's law and the experimental coefficients for rounded stones published by Isbash.

3. Theory. According to Isbash the basic equation for the movement of stone in flowing water can be written as:

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D)^{1/2} \qquad (1)$$

should be D_{50} ?

where

V = velocity, fps

C = a coefficient

g = acceleration of gravity, ft/sec^2

γ_s = specific weight of stone, lb/ft^3

γ_w = specific weight of water, lb/ft^3

D = stone diameter, ft

The diameter of a spherical stone in terms of its weight W is

$$D = \left(\frac{6W}{\pi\gamma_s}\right)^{1/3} \qquad (2)$$

Substituting for D in equation 1 results in

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}\left(\frac{6W}{\pi\gamma_s}\right)^{1/6} \tag{3}$$

which describes Airy's law stated in paragraph 2.

4. Experimental Results. Experimental data on stone movement in flowing water from the early (1786) work of DuBuat[5] to the more recent Bonneville Hydraulic Laboratory tests[6] have been shown to confirm Airy's law and Isbash's stability coefficients.[7] The published experimental data are generally defined in terms of bottom velocities. However, some are in terms of average flow velocities and some are not specified. The Isbash coefficients are from tests with essentially no boundary layer development and the average flow velocities are representative of the velocity against stone. When the stone movement resulted by sliding, a coefficient of 0.86 was obtained. When movement was effected by rolling or overturning, a coefficient of 1.20 resulted. Extensive U. S. Army Engineer Waterways Experiment Station laboratory testing for the design of riprap below stilling basins indicates that the coefficient of 0.86 should be used with the average flow velocity over the end sill for sizing stilling basin riprap because of the excessively high turbulence level in the flow. For impact-type stilling basins, the Bureau of Reclamation[8] has adopted a riprap design curve based on field and laboratory experience and on a study by Mavis and Laushey.[9] The Bureau curve specifies rock weighing 165 lb/ft^3 and is very close to the Isbash curve for similar rock using a stability coefficient of 0.86.

5. Application. The curves given in Chart 712-1 are applicable to specific stone weights of 135 to 205 lb/ft^3. The use of the average flow velocity is desirable for conservative design. The solid-line curves are recommended for stilling basin riprap design and other high-level turbulence conditions. The dashed line curves are recommended for river closures and similar low-level turbulence conditions. Riprap bank and bed protection in natural and artificial flood-control channels should be designed in accordance with reference 10.

6. Stilling Basin Riprap.

a. Size. The W_{50} stone weight and the D_{50} stone diameter for establishing riprap size for stilling basins can be obtained using Chart 712-1 in the manner indicated by the heavy arrows thereon. The effect of specific weight of the rock on the required size is indicated by the vertical spread of the solid line curves.

b. Gradation. The following size criteria should serve as guidelines for stilling basin riprap gradation.

(1) The lower limit of W_{50} stone should not be less than the weight of stone determined using the appropriate "Stilling Basins" curve in Chart 712-1.

(2) The upper limit of W_{50} stone should not exceed the weight that can be obtained economically from the quarry or the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(3) The lower limit of W_{100} stone should not be less than two times the lower limit of W_{50} stone.

(4) The upper limit of W_{100} stone should not be more than five times the lower limit of W_{50} stone, nor exceed the size that can be obtained economically from the quarry, nor exceed the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(5) The lower limit of W_{15} stone should not be less than one-sixteenth the upper limit of W_{100} stone.

(6) The upper limit of W_{15} stone should be less than the upper limit of W_{50} stone as required to satisfy criteria for graded stone filters specified in EM 1110-2-1901.

(7) The bulk volume of stone lighter than the W_{15} stone should not exceed the volume of voids in the revetment without this lighter stone.

(8) W_0 to W_{25} stone may be used instead of W_{15} stone in criteria (5), (6), and (7) if desirable to better utilize available stone sizes.

c. Thickness. The thickness of the riprap protection should be $2D_{50\ max}$ or $1.5D_{100\ max}$, whichever results in the greater thickness.

d. Extent. Riprap protection should extend downstream to where nonerosive channel velocities are established and should be placed sufficiently high on the adjacent bank to provide protection from wave wash during maximum discharge. The required riprap thickness is determined by substituting values for these relations in equation 2.

7. References.

(1) Shelford, W., "On rivers flowing into tideless seas, illustrated by the river Tiber." Proceedings, Institute of Civil Engineers, vol 82 (1885).

(2) Hooker, E. H., "The suspension of solids in flowing water." Transactions, American Society of Civil Engineers, vol 36 (1896), pp 239-340.

(3) Isbash, S. V., Construction of Dams by Dumping Stones in Flowing

Water, Leningrad, 1932. Translated by A. Dorijikov, U. S. Army Engineer District, Eastport, CE, Maine, 1935.

(4) ________, "Construction of dams by depositing rock in running water." Transactions, Second Congress on Large Dams, vol 5 (1936), pp 123-136.

(5) DuBuat, P. L. G., Traite d'Hydraulique. Paris, France, 1786.

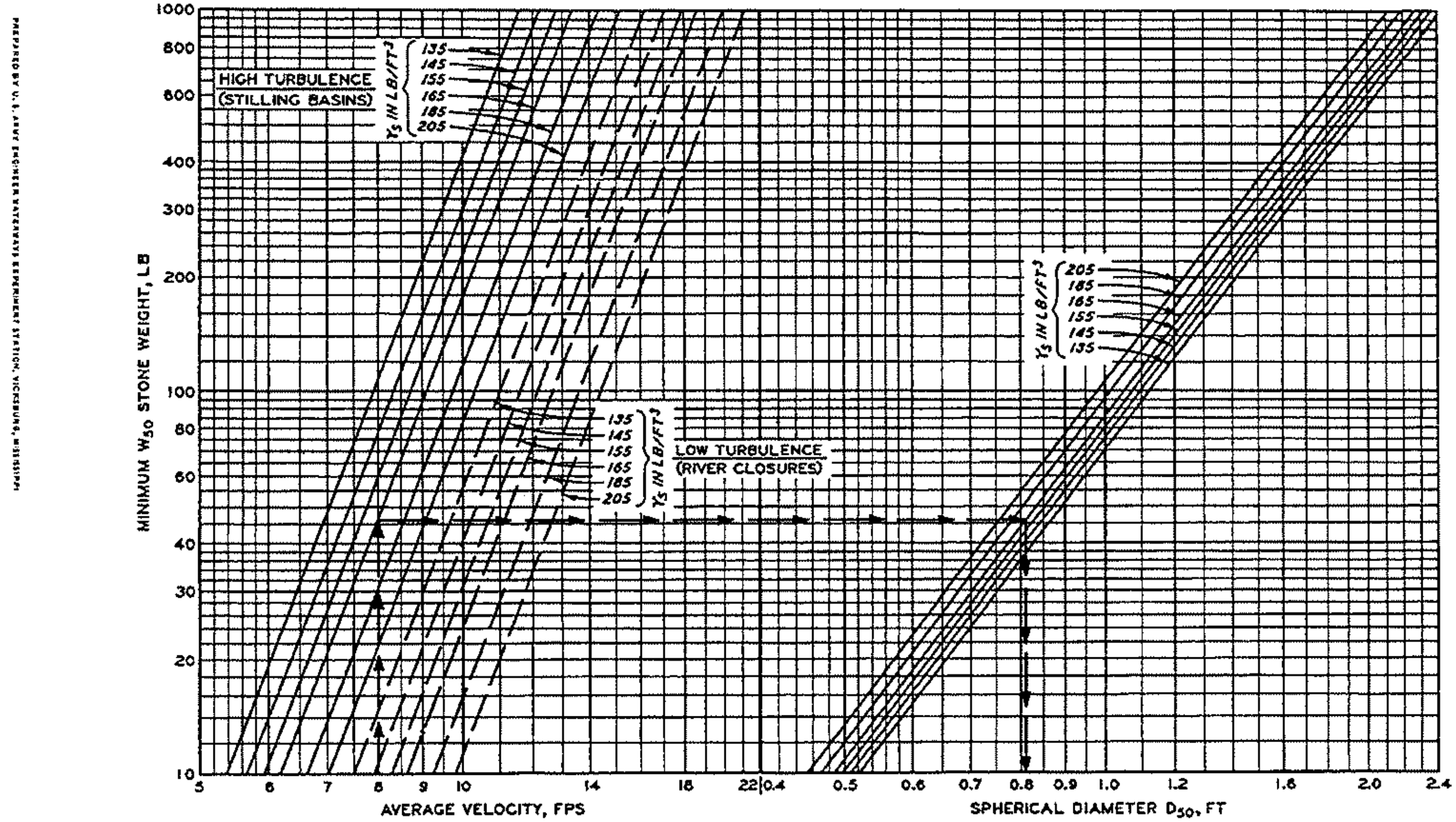
(6) U. S. Army Engineer District, Portland, CE, McNary Dam - Second Step Cofferdam Closure. Bonneville Hydraulic Laboratory Report No. 51-1, 1956.

(7) U. S. Army Engineer Waterways Experiment Station, CE, Velocity Forces on Submerged Rocks. Miscellaneous Paper No. 2-265, Vicksburg, Miss., April 1958.

(8) U. S. Bureau of Reclamation, Stilling Basin Performance; An Aid in Determining Riprap Sizes, by A. J. Peterka. Hydraulic Laboratory Report No. HYD-409, Denver, Colo., 1956.

(9) Mavis, F. T. and Laushey, L. M., "A reappraisal of the beginning of bed movement - competent velocity." Second Meeting, International Association for Hydraulic Structure Research, Stockholm, Sweden, 1948. See also Civil Engineering, vol 19 (January 1949), pp 38, 39, and 72.

(10) U. S. Army, Office, Chief of Engineers, Engineering and Design; Hydraulic Design of Flood Control Channels. EM 1110-2-1601, Washington, D. C., 1 July 1970.



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_S-\gamma_W}{\gamma_W}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_S}\right)^{1/3}$$

WHERE: V = VELOCITY, FPS

γ_S = SPECIFIC STONE WEIGHT, LB/FT^3

γ_W = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3

W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.

D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}

C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)

g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

5/5

Woods Pond Railroad Track Erosion Potential Assessment

Stone blanket stability design equation (for stable stone blankets in current fields)

$$\frac{d_{30}}{h} = S_f\ C_s \left[\left(\frac{\gamma_w}{\gamma_a - \gamma_w} \right)^{1/2} \left(\frac{\bar{u}}{\sqrt{K_1 g\, h}} \right) \right]^{5/2} ; \qquad K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \varphi}}$$

The critical depth-integrated velocity ($\bar{u}_c$) (obtained by re-arranging the above equation)

$$\bar{u}_c = \left(\frac{1}{S_f\ C_s} \right)^{2/5} \left(\frac{h}{d_{30}} \right)^{1/10} \left[g\, K_1 \left(\frac{\gamma_a - \gamma_w}{\gamma_w} \right) d_{30} \right]^{1/2}$$

where

$\bar{u}$ = depth-integrated mean flow velocity

$\bar{u}_c$ = critical depth-integrated mean flow velocity

h = flow depth associated with $\bar{u}$

d_{30} = stone or riprap size of which 30% in finer by weight

γ_w = the specific weight of water

γ_a = the specific weight of armor stone

S_f = safety factor (minimum = 1.1) to allow for debris impacts or other unknowns

C_s = stability coefficient for incipient motion
= 0.30 for angular stone (0.38 for rounded stone)

K_1 = a side slope correction factor to account for blankets placed on sloping channel side walls

θ = channel sidewall slope

φ = angle of repose of blanket armor (40° approx. for riprap)

References:

1. Coastal Engineering Manual (Draft), Engineer Manual EM 1110-2-1100 (Part VI), Fundamentals of Design, US Army Corps of Engineers, 2003
2. Hydraulic Design of Flood Control Channels, Engineer Manual EM 1110-2-1601, Engineering and Design, US Army Corps of Engineers, 1991.
3. Boundary Shear Stress Distributions in Open Channel Flow, D.W. Knight, K.W.H. Yuen, and A.A.I.Al-Hamid, Mixing and Transport in the Environment, John Wiley & Sons Ltd., 1994.

critical scour velocities is given by the Task Committee on Preparation of Sedimentation Manual (1966). Table 2-5 gives a set of permissible velocities that can be used as a guide to design nonscouring flood control channels. Lane (1955) presents curves showing permissible channel shear stress to be used for design, and the Soil Conservation Service (1954) presents information on grass-lined channels. Departures from suggested permissible velocity or shear values should be based on reliable field experience or laboratory tests. Channels whose velocities and/or shear exceed permissible values will require paving or bank revetment. The permissible values of velocity and/or shear should be determined so that damage exceeding normal maintenance will not result from any flood that could be reasonably expected to occur during the service life of the channel.

Table 2-5
Suggested Maximum Permissible Mean Channel Velocities

Channel Material	Mean Channel Velocity, fps
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel[1]	6.0
Earth	
Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass-lined Earth (slopes less than 5%)[2]	
Bermuda Grass	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
Sandy Silt	5.0
Silt Clay	7.0
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	20.0

Notes:
1. For particles larger than fine gravel (about 20 millimetres (mm) = 3/4 in.), see Plates 29 and 30.
2. Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

sediment transport in alluvial channels and design of canals has been ably presented by Leliavsky (1955). Fundamental information on bed-load equations and their background with examples of use in channel design is given in Rouse (1950) (see pp 769-857). An excellent review with an extensive bibliography is available (Chien 1956). This review includes the generally accepted Einstein approach to sediment transport. A comparative treatment of the many bed-load equations (Vanoni, Brooks, and Kennedy 1961) with field data indicates that no one formula is conclusively better than any other and that the accuracy of prediction is about ± 100 percent. A recent paper by Colby (1964b) proposes a simple, direct method of empirically correlating bed- load discharge with mean channel velocity at various flow depths and median grain size diameters. This procedure is adopted herein for rough estimates of bed-load movement in flood control channels.

c. Design curves. Plate 27 gives curves of bed-load discharge versus channel velocity for three depths of flow and four sediment sizes. The basic ranges of depths and velocities have been extrapolated and interpolated from the curves presented in Colby (1964a) for use in flood control channel design. Corrections for water temperature and concentration of fine sediment (Colby 1964a) are not included because of their small influence. The curves in Plate 27 should be applicable for estimating bed-load discharge in channels having geologic and hydraulic characteristics similar to those in the channels from which the basic data were obtained. The curves in this plate can also be used to estimate the relative effects of a change in channel characteristics on bed-load movement. For example, the effect of a series of check dams or drop structures that are provided to decrease channel slope would be reflected in the hydraulic characteristics by decreasing the channel velocity. The curves could then be used to estimate the decrease in sediment load. The curves can also be used to approximate the equilibrium sediment discharge. If the supply of sediment from upstream sources is less than the sediment discharge computed by the rating curves, the approximate amount of streambed scour can be estimated from the curves. Similarly, deposition will occur if the sediment supply is greater than the sediment discharge indicated by the rating curves. An example of this is a large sediment load from a small side channel that causes deposition in a major flood channel. If the location of sediment deposition is to be controlled, the estimated size of a sediment detention facility can be approximated using the curves. An example of the use of a sediment discharge equation in channel design is given in USAED, Los Angeles (1963).

2-7. Stable Channels

a. General.

(1) The design of stable channels requires that the channel be in material or lined with material capable of resisting the scouring forces of the flow. Channel armoring is required if these forces are greater than those that the bed and bank material can resist. The basic principles of stable channel design have been presented by Lane (1955) and expanded and modified by Terrell and Borland (1958) and Carlson and Miller (1956). An outline of the method of channel design to resist scouring forces has been given in Simons (1957). The most common type of channel instability encountered in flood control design is scouring of the bed and banks. This results from relatively large discharges, steep channel slopes, and normally limited channel right-of-way widths. These factors frequently require the use of protective revetment to prevent scouring.

(2) While clay and silt are fairly resistant to scour, especially if covered with vegetation, it is necessary to provide channel revetment when tractive forces are sufficiently high to cause erosion of channels in fine material. Little is known about the resistance of clay and silt to erosion as particles in this size range are influenced to a large extent by cohesive forces. A summary of some of the effects is given by the Task Committee on Preparation of Sedimentation Manual (1966). Suggested maximum limiting average channel velocities for noncohesive materials are listed in c below and plotted in Plate 28.

b. Prevention of scour. Scour and deposition occur most commonly when particle sizes range from fine sand to gravel, i.e., from about 0.1 mm through 50 mm (Plate 28). Erosion of sands in the lower range of sizes is especially critical as the sand particle weight is small, there is no cohesion between grains, and there is usually little vegetation along the channel. This particle size range comprises the majority of the bed and suspended load in many streams. Paragraph 2-6 above discusses sediment movement and presents a sediment rating curve as a guide to predicting channel stability.

c. Permissible velocity and shear. The permissible velocity and shear for a nonerodible channel should be somewhat less than the critical velocity or shear that will erode the channel. The adoption of maximum permissible velocities that are used in the design of channels has been widely accepted since publication of a table of values by Fortier and Scobey (1926). The latest information on

Project information and pertinent data

Railroad track invert = El. 952.8 ft
Effective channel width = 20 ft
K_1 = 1.0 (no side slope correction assumed)

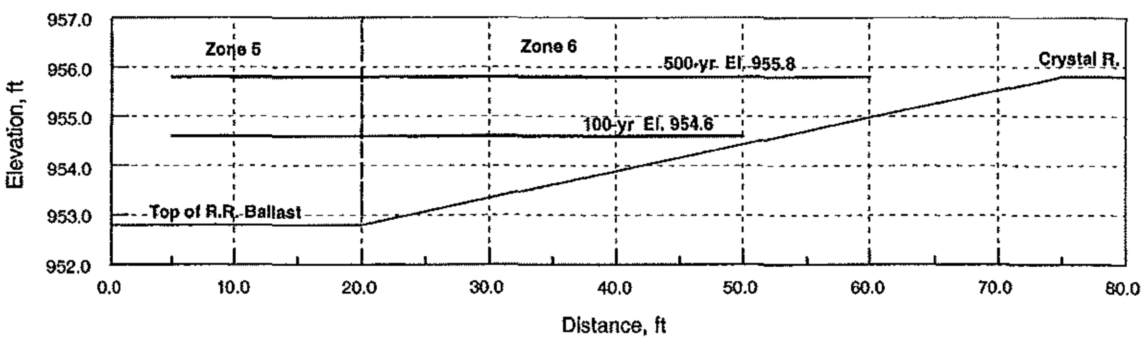
Hydraulic conditions (from separate analysis)

	100-yr flood	500-yr flood
W.S. elevation	954.60	955.80
Flow depth, h	1.80 ft	3.00 ft

Approximation of d_{50}

$\gamma_{50} = 1.7\,\gamma_{30}$ (approx.); $d_{50} = 1.7\,d_{30}$ (approx.) (Reference 2)

Section mean-velocity, V_{ave} and depth-integrated velocity, $\bar{u}$



Flood year	Flow depth, ft	Q, cfs			Area, ft^2			V_{ave}, ft/s	$\bar{u}$
		Zone 5	Zone 6	total	Zone 5	Zone 6	total	Q_{total}/A_{total}	ft/s
100	1.8	118.0	4.0	122.0	36.0	27.0	63.0	1.94	2.13
500	3.0	306.0	270.0	576.0	60.0	75.0	135.0	4.27	4.48

Note:
$\bar{u} = 1.05\,V_{ave}$ (Reference 3 – Figure 1)
Basic hydraulic data obtained from a separate study.
- Froude number, $F_r = 0.44$

Critical scour velocity, $\bar{u}_c$

d_{30} inches	d_{50} inches	γ_a lb/ft^3	S_f	flood years	depth, h ft	$\bar{u}$ ft/s	$\bar{u}_c$ ft/s
0.88	1.50	160	1.1	100	1.80	2.13	4.14
				500	3.00	4.48	4.36
0.88	1.50	160	1.0	100	1.80	2.13	4.30
				500	3.00	4.48	4.53
1.00	1.70	160	1.1	100	1.80	2.13	4.33
				500	3.00	4.48	4.56
1.00	1.70	165	1.1	100	1.80	2.13	4.43
				500	3.00	4.48	4.68
1.00	1.70	165	1.0	100	1.80	2.13	4.62
				500	3.00	4.48	4.86

Summary

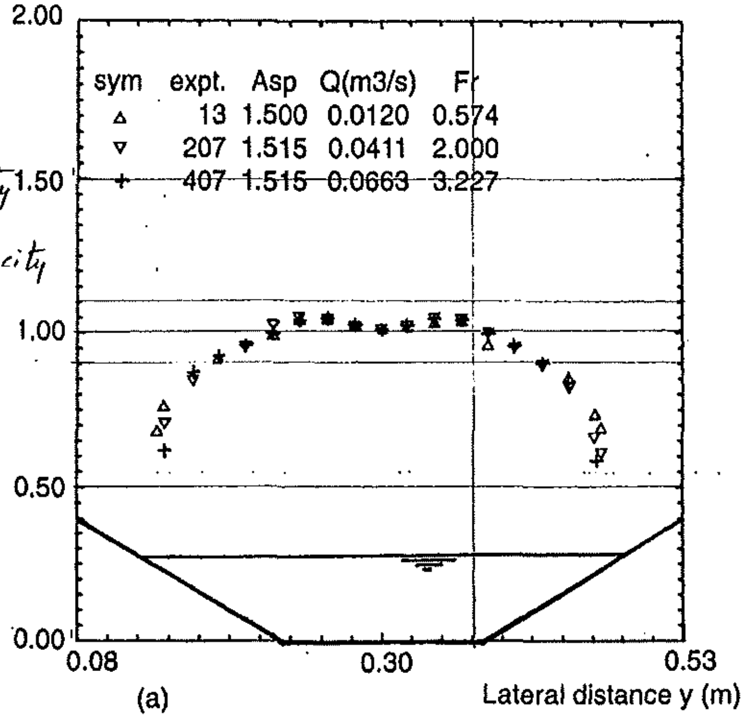
For all cases considered above, the critical depth-averaged velocities ($\bar{u}_c$) computed based on d_{30} are greater than the depth-averaged velocities ($\bar{u}$) computed based the flow area including the area between R.R. and Crystal Rd. Significant movements of ballast stones are not anticipated under both 100- and 500-yr flood conditions due to interlocking nature of the ballast stones.

2/21/06

Figure 1 4/5

U_o = section-mean velocity

U_d = depth-averaged velocity



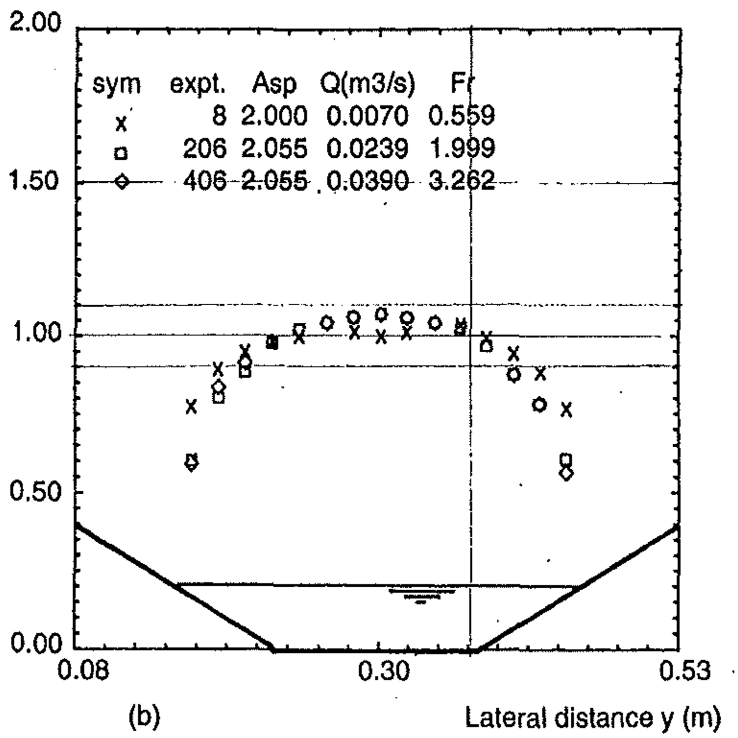


Figure 4.4 Variation of depth-averaged velocity across trapezoidal channels for constant aspect ratios and various Froude numbers

Revised Woods Pond Dam Stage-Storage-Discharge Curve

	Overflow ID	1	2	3	4	5	6	
	Overflow Elevation (ft)	957.00	954.00	948.30	954.00	952.80	954.40	
	Weir Length (ft)	10.0	114.0	140.0	55.0	20.0	50.0	
Elevation (ft)	Storage (acre-ft)	Valley Road (cfs)	Closure and Stoplog Structure (cfs)	Spillway Outflow (cfs)	West Abutment Overflow (cfs)	RR Overflow (cfs)	Crystal Road (cfs)	Combined Outflow (cfs)
948.30	0.00	0	0	0	0	0	0	0
948.50	0.03	0	0	40	0	0	0	40
949.00	0.10	0	0	273	0	0	0	273
949.50	0.18	0	0	634	0	0	0	634
950.00	0.25	0	0	1096	0	0	0	1096
950.50	0.32	0	0	1648	0	0	0	1648
951.00	0.39	0	0	2283	0	0	0	2283
951.50	0.47	0	0	2989	0	0	0	2989
952.00	0.54	0	0	3764	0	0	0	3764
952.50	0.61	0	0	4606	0	0	0	4606
953.00	0.69	0	0	5514	0	5	0	5519
953.50	0.78	0	0	6479	0	31	0	6510
954.00	0.83	0	0	7402	0	70	0	7472
954.50	0.91	0	108	8346	52	118	4	8624
955.00	0.98	0	334	9357	147	174	62	9982
955.50	1.05	0	558	10445	270	237	154	11511
956.00	1.12	0	851	11586	415	306	270	13168
957.00	1.27	0	1582	14155	763	460	560	16959
958.00	1.42	27	2435	16864	1175	633	912	20934
959.00	1.55	76	3403	19306	1642	824	1317	25251
960.00	1.71	130	4473	22075	2158	1332	1769	29877

Notes
1. Spillway values taken from Appendix G of MWH March 1999 Inspection Report
2. All overflow sections with except for the spillway section, assume a discharge coefficient of 2.67
2. Overflow lengths and elevations taken from Record Drawings