Dam Assessment Report

New Rochelle Reservoir No. 1 Dam
DEC Dam ID # 215-0207

August 10, 2020

City of New Rochelle	Town Houses at Lake Isle Association	Block 138 Corporation c/o United Corporation Services	Mott MacDonald NY, Inc John K. Ruschke, PE NY PE Lic. No. 071267-1

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Issue and revision record

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Contents

1 Introduction

This Engineering Assessment of New Rochelle Reservoir No. 1 Dam was prepared for the City of New Rochelle, (other Owners, Town Houses at Lake Isle Association and Block 138 Corporation). According to New York Codes, Rules and Regulations, Title 6, Part 673.13, the owners of Class C dams shall have an Engineering Assessment Report performed on their dam(s) every 10 years, which must be submitted to the New York State Department of Environmental Conservation (NYSDEC). This document is the first Engineering Assessment to be performed on the New Rochelle Reservoir No. 1 Dam (aka Lake Innisfree) after New York State passed new dam safety regulations in August 2009.

In order to prepare this report and develop the material contained in this report, the following tasks were completed:

- Obtain and review the records and relevant reports kept by the City of New Rochelle (dam owner);
- Obtain and review the records and relevant reports kept by the New York State Department of Environmental Conservation;
- A hazard classification reconnaissance;
- A dam safety inspection; and
- Engineering review and/or calculations on hydrology, hydraulics and stability.

The New Rochelle Reservoir No. l Dam is located on the Hutchinson River in the Town of Eastchester and the City of New Rochelle, Westchester County, New York. According to NYSDEC records, the dam is presently owned by:

City of New Rochelle:

Left embankment section and left masonry section from the center of the spillway to left abutment

Block 138 Corporation:

Right masonry section from the center of the spillway to the right abutment

Town Houses @ Lake Isle:

Right embankment section.

1.1 Description

The New Rochelle Reservoir No. 1 Dam was originally constructed in or around 1894 by the New Rochelle Water Company and was used for water supply. The reservoir is no longer used by the water company and is presently used primarily for recreational purposes. Construction and As-built drawings of the dam are not available. The dam geometry considered in this report is based on data presented to NYS DEC in the 1979 Phase 1 Inspection Report for the New Rochelle Reservoir No. 1 Dam and updated by recent survey data performed by Mott MacDonald in January 2020. All elevations referenced herein refer to NAVD 1988 datum and left and right directions are from a looking downstream perspective.

The New Rochelle Reservoir No.1 Dam is a relatively long dam on the southern end of the impoundment. The dam consists of a central masonry section composed of stone blocks and mortar with earthen embankment dikes on both ends. The masonry section is approximately 670 feet long, with a maximum section 30 feet high near the center down to 6 feet at the northeastern end. The crest width is about 6 feet with a crest elevation of 186.4±.

The dam includes a service spillway within the masonry section of the dam. The spillway is situated approximately 274 feet from the northeastern end (left end) of the masonry section. It has a crest length of 30 feet with a crest elevation of 182.28, which is about 4 feet lower than the masonry section. The spillway has a crest width of 6 feet and then slopes off at approximately a 2 vertical on 1 horizontal slope. The bottom and sides of the spillway channel are lined with stone block and convey the water approximately 70 feet downstream which is well beyond the toe of the dam.

There is a stone building/gate house structure on the masonry dam located approximately 145 feet from the southwestern end (right end) of the masonry section. Old inspection reports indicate that there were several pipes which passed through the gate house. However, the spot where these pipes exited has been sealed and are no longer operational. Therefore, the structure currently has no low-level outlet to drain the reservoir.

The left and right earthen dikes extend beyond the ends of the masonry sections approximately 95 feet and 650 feet, respectively, with a maximum section about 10 feet high, crest widths of 6 feet to 8 feet and a crest elevation ranging from 186.48 to 188.4 at the left dike and 188.11 to 189.2 at the right dike.

The impoundment is known as Reservoir No. 1 (also known as Lake Isles) and stores 582± acre-feet of water at the spillway crest. The New York State Department of Environmental Conservation (NYSDEC) has classified the structure as a Hazard Class “C”, high hazard dam, due to the presence of the Hutchinson River Parkway and highly developed areas downstream of the dam, where a catastrophic event would have a significant impact and may endanger human life and property.

Information relative to the dam and reservoir are included in the following table.

Table 1: New Rochelle Reservoir No. 1 Dam

Geographical Location	New Rochelle/Eastchester, Westchester County, NY
NYSDEC ID#	215-0207
NATDAM ID #	NY00020
Date of completion	1894 (no available design or construction records)
Latitude	(N) 40° 57’ 24”
Longitude	(W) 73° 47’ 56”
Purpose	Conservation/Recreation
NYSDEC Hazard Classification	Class “C”, High Hazard Potential
Drainage Area	2.2 square miles (1,292 acres)
Dam Type	Stone Masonry Dam w/Earthen Dikes at each end
Maximum Dam Height	30 ft.
Dam Length	NE Earthen Dike approximately 95 ft. Masonry Dam 670 ft. SW Earthen Dike approximately 650 ft.
Masonry Dam	U/S face: vertical D/S face: 3V:2H Crest Elevation: 186.6± ft. Crest Width: 6 ft.
Earth Dikes	U/S slope: about 1V:2H D/S slope: varies from 1V:1.5H to 1V:3H Earth Dikes Crest Elevation: Varies from 186.48 ft to 189.2 ft. Crest Width: 6 to 8 ft.
Spillway	un-gated overflow channel Spillway length: 30 ft. Spillway Crest Elevation: 182.28 ft. Crest Width: 6 ft. Drop: 12 ft w/2V:1H d/s slope

Geographical Location	New Rochelle/Eastchester, Westchester County, NY
Normal Pool Elevation	182.3 ft.
Reservoir Area	62 acres w/water level at top of service spillway 81 acres w/water level at top of masonry dam 92 acres w/water level at top of earth dikes
Normal Storage Capacity	582 acre-ft. w/water level at top of service spillway 869 acre-ft. w/water level at top of masonry dam 1,043 acre-ft. w/water level at top of earth dikes
Spillway Capacity	744 cfs. (w/water level @ top of masonry dam)
Low Level Outlet	Gate House located 145 ft. from SW end of masonry section. Several pipes pass thru gate house. The pipes have been sealed and the low-level outlet is currently not operational.

1.2 Records Review

As part of the Engineer's Assessment report, it is necessary to perform a review of existing dam information. The engineer must analyze the dam's design, construction, and operational records, if available, to become fully acquainted with the present physical features as well as construction and performance history of the dam.

As noted previously, this is the first Engineering Assessment report for the New Rochelle Reservoir No. 1 Dam. While not termed an Engineering Assessment, several reports or studies have been performed for the dam. A review of the records held by the City of New Rochelle and the NYSDEC yielded the following significant documents.

- New Rochelle Reservoir No. 1 Dam Phase I Inspection Report
  New York District Corps of Engineers
  February 1979

- Emergency Action Plan New Rochelle Reservoir No. 1
  City of New Rochelle, Office of Emergency Management
  Latest Revision Date November 2016

- Inspection and Maintenance Plan for New Rochelle Reservoir No. 1 Dam
  Paul C. Rizzo Engineering – New York, PLLC., Engineers & Consultants
  Latest Revision Date June 12, 2013

- Hydraulic Analysis of The Hutchinson River and Potential Improvements
  Leonard Jackson Associates, Consulting Engineers
  Latest Revision Date March 25, 2008

- Drawing Titled "Reservoir No.1 – Soundings, Top of Rock, North Side of Dam Wall", City of New Rochelle, Dept. of Public Works – Bur. Engineers
  September 21, 1979

2 Dam Safety Inspection

The Engineer should perform at least two specific on-site activities; a thorough dam safety inspection and a hazard classification reconnaissance. Additional on-site investigations; i.e., soil borings, test pits, topographic and boundary surveys, etc., may be necessary dependent on what additional information is required to complete the Engineering Assessment. Interviews with maintenance personnel and other people familiar with the dam are also recommended.

2.1 Document Review

Prior to visually inspecting the dam, existing documentation was reviewed which included the following:

- New Rochelle Reservoir No. 1 Dam, Phase I Inspection Report, National Dam Safety Program,
- New York District Corps of Engineers dated February 1979.
- Dam Safety Inspection Report, prepared by Paul C. Rizzo Engineering – New York, PLLC, dated April 2, 2013.
- NYSDEC certified letter to listed Dam Owners, dated October 12, 2018, RE: Notice of Condition Rating "Unsound, Deficiency Recognized"

According to the latest NYSDEC visual safety inspection conducted on August 3, 2018, and the certified letter to dam owners dated October 12, 2018, this dam has been assigned a condition of **Unsound, Deficiency Recognized**, in accordance with 6NYCRR Part 673.16. Appendix C contains a copy of the Dam Safety Inspection Report, including inspection photographs, evaluations of observations, and recommendations.

2.2 Description

The dam safety inspection is a comprehensive examination of the visible physical features of the dam and its appurtenant structures. The dam safety inspection requirements are presented in 6NYCRR Part 673.12(d). The visual inspection should comment on previously observed deficiencies, including the deficiencies' progression or advancement and identification of continuing or new corrective actions, as necessary, with recommendations. Corrective actions and recommendations may be deferred until after the technical analysis.

All observations and unusual features should be recorded, regardless of how insignificant they may seem. Photographs are a permanent record of the condition of the dam and provide a means to compare dam conditions at different points in time. Photographs are a required element of the Dam Safety Inspection Report.

A good source of information for conducting a dam safety inspection can be found in the NYSDEC "An Owners Guidance Manual for the Inspection and Maintenance of Dams in New York State" which can be found at http://www.dec.ny.gov/lands/4991.html. Other references can be found on the Internet.

Underwater inspections may be required as the condition and design of underwater features is not well known.

2.3 Physical Inspection

The physical condition of the New Rochelle Reservoir No. 1 Dam was inspected on December 10, 2019, by Mott MacDonald. Weather conditions on the day of the inspection were cool and sunny. At the time of inspection, the reservoir level was at the crest of the service spillway with flow going over the spillway. The following are observations made during the inspection of the key components of the dam. Appendix A contains a complete copy of the Dam Safety Inspection Report, including a visual inspection checklist, inspection photographs, evaluations of observations, and recommendations.

2.3.1 Masonry Dam

The majority of the masonry dam is in fair to good condition. The portion of the masonry dam to the right of the gate house had significant vegetative growth over the structure which limited the inspection of same. The vertical and horizontal alignments are in good condition with no signs of movement. No signs of structural instability were observed. Masonry blocks are in overall good condition. Mortar joints are in fair condition with some areas of loose or missing mortar were observed. Mortar repairs should be made as needed to ensure long term performance.

Minor seepage and wet spots were observed on the downstream face of the gate house structure, approximately 30 feet to the right of the spillway, and along the joints (left and right side) between the masonry dam and the spillway wing walls. In addition, a soft/wet spot was observed in the earth near the toe of the masonry dam approximately 20 feet to the left of the primary spillway. Visual seepage through the masonry Dam should be eliminated to ensure long term performance of the dam.

Inspection of the upstream masonry face was limited to the exposed portion above the water elevation. The masonry block appears to be in good condition. The mortar joints in several locations are also in need of repair.

Concrete blocks have been set on top of the crest adjacent to the gate house which are not part of the original dam construction. The blocks appear to have been placed to restrict access around the gate house.

Vines and ivy cover a large portion of the downstream face of the masonry dam along the right side of the structure. Root intrusion into the masonry dam was noted in several locations. All mortar joints impacted by root intrusion should be cleaned to remove the existing vegetation and repaired.

Dense brush and large trees were also observed near the toe of the masonry dam. The presence of woody vegetation and trees represents a potential risk that could impact the foundation of the dam if uprooted from storms or overtopping of the Dam. The prolongation of roots in close proximity of the dam can also cause conduits for uncontrolled excessive seepage. They also hinder close visual inspection of the structure. All woody vegetation and trees in close proximity of the dam should be removed.

The base of the masonry dam is not protected from high erodible forces if overtopping of the masonry dam occurs.

A shallow steel pipe was noted left of the gate house structure running perpendicular with the dam. A small discharge of water was noted from the subject pipe. The origin and configuration of the piping in relation to the dam is unknown and should be further investigated.

Security fencing in and around the dam has been removed or cut in several locations. Overall, Security fencing is in poor condition.

Despite the Dam's reasonably good structural appearance, there are no records available that demonstrate acceptable factors of safety for structural stability for all potential loading conditions. A detailed engineering assessment is being performed and will be outlined in a separate report.

2.3.2 Concrete Spillway

The service spillway is an uncontrolled overflow channel located approximately 265 feet from the northeastern end (left end) of the masonry dam. It is approximately 30 feet wide and has a crest elevation approximately 4 feet lower than the crest of the rest of the masonry sections. The spillway has a crest width of approximately 6 feet and then slopes off at approximately a 2 vertical on 1 horizontal slope. The bottom and sides of the channel are lined with stone block to carry the water well beyond the downstream toe of the dam.

A wrought iron fence has been placed along the spillway crest. Debris from high flows is likely to become hung up on this fence. This will significantly reduce spillway discharge capacity and will also increase stress on the structure. The fence should be removed, and alternative safety measures considered.

A footbridge has been installed over the lower portion of the spillway discharge channel. The bottom of the bridge is above the sidewall of the discharge channel. The bridge could be impacted if the capacity of the channel is exceeded or if heavy debris is passed through the channel. The presence of the bridge will be evaluated under the engineering assessment being performed.

Concrete blocks have been set on top of the stone spillway training walls which are not part of the original dam construction. The concrete blocks appear to have been placed to restrict access onto the dam and spillway.

Some undermining of the spillway wing walls was observed along the spillway chute/channel. Also, the downstream end of the spillway chute is cracking and collapsing into the plunge pool. Some erosion of the downstream channel observed at the plunge pool likely due to the collapsing spillway chute. The subject damage appears to have been present since the Army Corp inspection that was for performed in 1979.

2.3.3 Gate House

The gate house was reported to contain several pipes which passed through dam, presumably for low-level control of the reservoir water level and a water supply intake by the New Rochelle Water Company. The intake structure for the piping within the reservoir is submerged and not readily accessible. At the base of the gate house is two chambers housing piping and valves. The steel doors to these chambers are in poor condition and do not prohibit access into the structure. A large boulder was placed in from of one of the doors. The floor of both chambers was flooded restricting inspection of the piping and valves. The condition and age of the valves are unknown. If part of the original construction, the piping and valves would be over 100 years old. The pipes within the structure were reportedly sealed. Further investigation is necessary to verify if the pipes have been properly sealed.

Dam safety regulations require that, assuming no inflow, the service spillway, or low-level outlet, should have sufficient capacity to evacuate 75 percent of the storage between the auxiliary spillway crest and the service spillway crest within 7 days. Since the gates and valves in the gate house are arguably inoperable, the reservoir presently has no emergency maintenance drain capability.

Wet spots were observed in the masonry at the downstream face of the gate house. These areas should be monitored for further consideration and possible correction action if necessary.

2.3.4 Left Earthen Dike

The left dike is approximately 75 feet long and 6 feet high. The crest elevation is approximately 2 feet higher than the crest elevation of the adjacent masonry dam. The left dike has a steep side slope on the upstream side which is protected with riprap stone from wave action and ice. The crest of the embankment is not well defined and varies from approximately 4 to 6 feet in width. There is a chain link fence along the crest of the embankment. The downstream side slope is moderately sloped with a height that varies with the transition into the surrounding topography. The entire dike has been cleared of the shrubs and trees. The stumps of same were visible from the inspection. The vertical and horizontal alignments are in good condition with no signs of notable movement or settlement.

There were no signs of seepage noted along the dike.

2.3.5 Right Earthen Dike

The right dike is approximately 700 feet long and approximately 10 feet high. The crest elevation is about 2 feet higher than the crest elevation of the adjacent masonry dam. The upstream slope has riprap stone protection from wave action and ice and includes access points to the lake for recreation. There is a stone pathway along a portion of the crest of the dike that serves for recreational purposes and provides access to the lake for the adjacent townhouse community. The downstream slope is approximately 2H:1V and is maintained lawn areas. The townhouse community is situated just downstream of the toe of the dike.

The dike is covered with numerous large trees and brush on the upstream side and a few large trees on the crest and downstream slope. Minor erosion/depressions were observed along the upstream slope. Trees and brush should be removed from the dike.

There is a wooden fence that runs across the dike at the transition from the right earth dike and the right masonry dam. Some minor erosion was observed in the dike along the wooden fence.

There were no signs of seepage noted along the dike.

3 Hazard Classification

The dam's hazard classification influences the regulatory and design standards for the dam and is therefore a critical component to the assessment. A dam's hazard classification will typically dictate spillway capacity criteria, which in turn can affect much of the remainder of the assessment, as well as the EAP requirements.

The first step in determining a dam's hazard classification is to perform a review of the downstream area of the dam. Before beginning the field reconnaissance, the engineer should become familiar with the dam's features and setting through a desktop review of the downstream area using readily accessible data such as USGS 7 ½ minute quadrangle maps, recent orthoimagery, and similar tools to determine potential downstream features that may be impacted by the dam's failure flood wave. The engineer can then perform the downstream field investigation to verify the buildings, roads, railroad, environmentally sensitive areas, etc. that will be within the flood wave's path. In most cases, field work will also be necessary to assess potential impacts and recommend a hazard classification.

A previous dam breach analysis was developed by New Rochelle as part of an Emergency Action Plan for Reservoir No.1 with a latest revision date of November 2016. The report presented results from the analysis which appeared to use a HEC-RAS steady state backwater analysis for a "sunny day" breach and the "flood event" breach which is the 50% Probable Maximum Flood (PMP). However, the report did not present any supporting backup data, such as breach parameters and modeling output reports, to verify the validity of the analysis and the methodology. Therefore, a revised analysis was developed to verify the dam's hazard classification.

To assess the dam's hazard classification in accordance with the guidelines set forth in the DEC's Department of Water (DOW) Technical & Operational Guidance Series (TOGS) 3.1.5, "*Guidance for Dam Hazard Classification*", failure of the dam was simulated to determine potential impacts to roadways and structures downstream of the dam. A comprehensive 2D model of the downstream conditions along the Hutchinson River was prepared using HEC-RAS computer program developed by the US Army Corp of Engineers. The model and terrain data were developed using topographic and geospatial data provided by the City of New Rochelle and with Westchester County's geospatial data downloaded from their website. The data was supplemented by recent topographic survey data developed by Mott MacDonald, including a bathymetric survey of New Rochelle Reservoir No. 1. The model of the Hutchinson River and its overbank areas was extended from the upstream end of the impoundment area of the dam to the crossing with Pelham Parkway, a distance of approximately 5.3 miles downstream of the dam. Two dam failure scenarios, the sunny day and spillway design flood conditions, were simulated as described below.

Inflow hydrographs used for the HEC-RAS 2D model were developed using HEC-HMS computer program developed by the US Army Corp of Engineers.

3.1 Failure Simulation and Parameters

In order to estimate the impact of flows on the downstream roadways and structures, two different dam failure modes were evaluated in the HEC-RAS model. One failure scenario is of the dam under normal flow conditions (termed "sunny day" failure) and the other being failure during the spillway design flood (50% PMP event).

In addition to the failure scenarios noted above, a third scenario representing the spillway design flood without failure scenario was modeled. From the hydrologic modeling discussed in Section 4, the peak inflow at the reservoir is 5,110 cfs and the peak outflow over the dam is 4,804 cfs during the spillway design flood (SDF).

The sunny day failure was performed to model the peak outflow from the dam in the event of its failure under normal pool conditions. Under such conditions, there is no precipitation and upstream pool elevations are at normal levels. The sunny day breach could be caused by an earthquake, sabotage, detected or undetected structural problems, or other causes. This type of breach is normally not predictable and is generally considered to have the most potential for loss of life and property damage due to unsuspecting residents and creek users downstream of the dam. This event was modeled with a normal water surface elevation of 182.3, which is equivalent to the spillway crest elevation. The mode of failure considered was a breach that would occur at the primary spillway using similar dimensions.

The sunny day failure was assumed to begin at elevation 182.3 feet, progressing in size until a full breach has occurred to elevation 161.3 feet. It was assumed that the total breach height would be 21 feet and would occur at the primary spillway. Since the dam's construction is mostly a combination of a stone masonry and earthen dam structure, it has been assumed that the breach parameters will fall within the Federal Energy Regulatory Commission (FERC) Guidelines.

At the primary spillway, the dam is a stone masonry structure with a stone spillway chute extending downstream of the dam. At the rest of the dam, the stone masonry section will control the overall breach parameters as it is on the upstream side of the earthen embankment. The stone block construction will begin to fail resulting in a reasonably narrow breach with nearly vertical side slopes. The average breach with for the sunny day failure was assumed to be 30-feet wide (width of the spillway). Given that the FERC guidelines recommend a range of failure time for a masonry dam from 0.1 to 0.3 hours, a failure time of 0.3 hours was assumed as the dam retains only a relatively small height of water. During this sunny day failure scenario, the peak outflow from the New Rochelle Reservoir No. 1 under the assumed breach was determined to be 6,153 cfs.

The hydrologic and hydraulic analysis determined that the during the SDF, the masonry portion of the dam would be overtopped by approximately 1.7± feet with the SDF water surface elevation at 188.1± feet. Since the dam would be overtopped during the SDF, the SDF failure was assumed to occur where the dam is at its maximum height. Failure of the dam would begin at 188.1 feet and working its way down through the stone masonry section and embankment to elevation 162 feet. It was assumed that the breach height would be 24.4 feet. The average breach width was assumed to be 60 feet. Given that the FERC guidelines recommend a range of failure time for a masonry dam from 0.1 to 0.3 hours, a failure time of 0.1 hours was assumed as the dam retains only a relatively small height of water. The FERC breach parameters have been supplied in Appendix B. In the HEC-RAS model, the dam's failure was set to coincide with the peak outflow and elevation from the dam, representing a worst-case scenario. During the spillway design flood scenario, the peak outflow from the New Rochelle Reservoir No. 1 Dam under the assumed breach was determined to be 19,061 cfs. A copy of the modeling reports and supporting data are provided in Appendix B.

The following table represent the breach parameters used for the two (2) failure scenarios:

Table 2: Breach Parameters

Breach Scenario	Breach Width (ft)	Breach Slope	Failure Time (hr.)	Trigger Elev. (ft)
SDF Event	60	0	0.3	188.1
Sunny Day Event	30	0	0.3	182.3

The results of the HEC-RAS model for the two (2) failure scenarios are shown on Figures 2, 3, 4, & 5 and discussed below.

3.2 Downstream Impacts

Consistent with DEC's DOW TOGS 3.1.5, "*Guidance for Dam Hazard Classification*", the sunny day dam failure, SDF failure, and the SDF without failure scenarios were considered when determining the hazard impact. Specific to TOGS, the potential for loss of human life (Section D.1), impact to emergency services (Section D.2), damage to homes (Section D.3) and impact to main highways versus roads (Section D.4) were considered. Two of the major determinations between the assignment of a hazard classification of "B" or "C" to a dam lie in the potential for loss of human life and damage to main versus minor roadways. Section D.3 of the TOGS contains specific information related to flow depth and damage to homes, as well as how many homes can be impacted and the resulting hazard classification. Table 3 is provided directly from Section D.3 of the TOGS. Commercial buildings within the inundation limits are not considered in the hazard classification assessment since these buildings are assumed not occupied at night and may be readily evacuated during their normal business hours in the event of a dam failure.

Table 3: Section D.3 of TOGS, Damage to Homes

Flood Depth	1 to 10 homes	11 to 99 homes	100 to more homes
Up to 1 foot above lowest occupied floor	**A**	**B**	**C**
Greater than 1 foot above lowest occupied floor	**B**	**B**	**C**
Above the Low Danger Zone	**C**	**C**	**C**

Section D.4 contains a table showing the New York State Department of Transportation (NYSDOT) functional class of the roadways and the subsequent hazard classification if the roadway is damaged. Table 4 is provided directly from Section D.4 of TOGS.

Table 4: Section D.4 of TOGS, Road Classification

NYSDOT Functional Class (#)	Hazard Classification (if impacted)
Urban- Principal Arterial Interstate (11)	C
Rural- Principal Arterial Interstate (1)	C
Urban- Principal Arterial Expressway (12)	C
Urban- Principal Arterial- Other (14)	C
Urban- Minor Arterial (16)	B
Rural-Principal Arterial- Other (4)	B
Rural- Minor Arterial (6)	B
Urban- Collector (17)	B
Rural Major Collector (7)	B
Rural Minor Collector	A
Rural Local	A
Urban Local	A

A roadway is considered impacted if the dam failure model shows overtopping of the road embankment at any depth, since even 6 inches of water flowing at a high velocity can move a vehicle.

3.2.1 Impacts on Roadways

Roads are considered to be impacted by a dam's failure if there is overtopping of the road embankment. Roadways that could be impacted by failure of New Rochelle Reservoir No. 1 are listed in Table 5 along with their NYSDOT Functional Classification and the hazard classification which would be assigned to each under the guidelines set forth in Section D.4 of DEC's DOW TOGS 3.1.5.

Table 5: Downstream Road Impacts

Roadway	NYSDOT Functional Class	Dam Hazard Class
Hutchinson River Pkwy DS of Dam 1	Urban Principal Arterial Expressway	C
Mill Road	Urban Principal Arterial Other	C
Hutchinson River Pkwy at Dam 3	Urban Principal Arterial Expressway	C
Bon Air Avenue	Urban Local	A
Norman Road	Urban Local	A
Hutchinson River Pkwy DS of Dam 2	Urban Principal Arterial Expressway	C
Robins Road	Urban Local	A
New Rochelle Road	Urban Minor Arterial	B
Hutchinson Boulevard	Urban Local	A
Hutchinson River Pkwy at Lincoln Ave.	Urban Principal Arterial Expressway	C
Lincoln Avenue	Urban Principal Arterial Other	C
Hutchinson River Pkwy at Boulevard West	Urban Principal Arterial Expressway	C
Colonial Avenue	Urban Minor Arterial	B
Pelham Parkway	Urban Minor Arterial	B

Upon review of Table 5, failure of New Rochelle Reservoir No. 1 Dam could impact fourteen (14) roadways, with the highest NYSDOT Functional Classification of a road being “Urban Principal Arterial Expressway”.

To assess the impact on roads downstream of the dam, deck roadway elevations for each roadway were obtained from a combination of available downstream topographic mapping and limited survey data and compared with the results of the HEC-RAS model. Table 6 below summarizes the results, listing key roadways and roadway structures that are located within the inundation zone.

Table 6: HEC-RAS Summary – Downstream Roadway Impacts

Roadway Crossing Location	Distance DS of Reservoir No. 1 (mi)	Roadway Crossing Elevation	Peak Water Surface Elevation (Sunny Day)	Peak Water Surface Elevation (SDF)	Peak Water Surface Elevation (Overtopping)
Hutchinson River Pkwy DS of Dam 1	0.1	137.9'±	148.3'±	155.8'±	147.3'±
Mill Road	0.1	155.0'±	148.3'±	155.8'±	147.3'±
Hutchinson River Pkwy at Dam 3	0.9±	118.8'±	111.6'±	120.3'±	114.3'±
Bon Air Avenue	1.4±	93.8'±	93.9'±	97.0'±	95.1'±
Norman Road	1.6±	74.8'±	75.2'±	77.9'±	75.7'±
Hutchinson River Pkwy DS of Dam 2	2.0±	60.1'±	66.3'±	71.8'±	69.6'±
Robins Road	2.5±	68.0'±	66.1'±	71.9'±	69.6'±
New Rochelle Road	2.5±	64.0'±	65.9'±	70.8'±	68.8'±
Hutchinson Boulevard	3±	42.2'±	46.0'±	52.2'±	50.1'±
Hutchinson River Pkwy at Lincoln Ave.	3.4±	26.0'±	31.6'±	38.6'±	35.9'±
Lincoln Avenue	3.4±	32.0'±	31.4'±	38.4'±	35.8'±
Hutchinson River Pkwy at Boulevard West	4.2±	8.7'±	13.2'±	21.5'±	19.2'±
Colonial Avenue	4.5±	8.4'±	11.7'±	18.8'±	17.2'±
Pelham Parkway	5.3±	10.0'±	4.2'±	12.7'±	11.1'±

Table 6 shows the roadways that are overtopped or flooded due to the SDF event, sunny day failure and overtopping event of the New Rochelle Reservoir No. 1 Dam.

3.2.2 Impacts to Residents

Section D.3 of the TOGS 3.1.5 discusses the impact of dam failure in homes and the depth of flooding and flow velocity that would constitute different upstream dam hazard classifications. In that publication the following guidance on hazard classification is given:

"If flooding is up to 1 foot above the lowest occupied floor, but 10 homes or less are likely to be damaged, then the dam may generally receive a hazard classification of Class A. If flooding is up to 1 foot above the lowest occupied floor and more than 10 homes are damaged; or if flooding more than 1 foot above the lowest occupied floor in a single home, then the dam may generally receive a hazard classification of Class B. If flooding is above the "Low Danger Zone" in Figure 2 or 3 of ACER-11, this would indicate Class C. Damage to 100 homes or more, implies Class C, even if flooding is less than 1 foot above the lowest occupied floor."

To assess the impact on structures downstream of the dam during the sunny day failure and SDF failure, various houses that are within the inundation zones were identified and compared within the results of the HEC-RAS model. Tables 7, Table 8, and Table 9 summarize the results, showing the number of houses within a residential street/area that is flooded during the sunny day failure, SDF failure, and Overtopping event, respectively.

Table 7: Downstream Structure Impacts (Sunny Day Failure)

Street	No. of Resident Houses Impacted	Depth of Water (ft)	Average Flow Velocity (fps)
Norman Road	4	0.4'±	1.8± fps
Bon Air Ave	0	0.1'±	0.1± fps
Seacord Road	0	N/A	N/A
Rosehill Ave	7	0.4'±	2.1± fps
Interlake Ave	14	1.9'±	1.2± fps
Robbins Rd. Area	7	<1.0'	0.5± fps
New Rochelle Rd. Area	12	1.9'±	1.4± fps
Hutchinson Blvd	4	3.8'±	3.0± fps
River Ave Area	3	1.9'±	2.8± fps
First Ave	8	4.0'±	2.6± fps
Second Ave	0	N/A	N/A
Brookside Ave E	1	1.9'±	1.0± fps
Farrell Ave	4	1.7'±	1.1± fps
Sparks Ave	3	1.4'±	2.4± fps
Manning Cr.	0	N/A	N/A
Hillside Ave	1	0.4'±	0.3± fps
Brookside Ave W	6	1.7'±	1.6± fps
Stellar Ave	0	N/A	N/A
Wolfs Ln	0	N/A	N/A
Carol Pl.	7	2.7'±	2.4± fps
Iden Ave	0	N/A	N/A
Total Houses Impacted	**81**		

Note: The flood depths and velocities were estimated from the HEC-RAS 2D results using the maximum values within the vicinity of the inundation zone for each area.

Table 7 indicates that approximately 81± houses would be impacted with flooding during the sunny day failure. Since the first-floor elevations (FFE) are unknown, it can't be determined if any of the houses are flooded 1 foot above the FFE or above the Low Danger Zone with regards to flood depth and velocity. Therefore, based on the results of the sunny day failure alone, we are unable to classify the dam since the number of impacted houses is less than 100. Figures 2 through 5 (New Rochelle Reservoir No. 1 Inundation Maps) show the houses within the inundation limits.

Table 8: Downstream Structure Impacts (Spillway Design Flood Failure)

Street	No. of Resident Houses Impacted	Depth of Water (ft)	Average Flow Velocity (fps)
Norman Road	16	3.6'±	5.6± fps
Bon Air Ave	8	3.2'±	5.2± fps
Seacord Road	8	2.5'±	6.9± fps
Rosehill Ave	14	2.7'±	6.6± fps
Interlake Ave	17	7.5'±	1.8± fps
Robbins Rd. Area	60	3.9'±	1.4± fps
New Rochelle Rd. Area	30	7.8'±	6.5± fps
Hutchinson Blvd	15	10.0'±	8.0± fps
River Ave Area	20	9.0'±	6.2± fps
First Ave	12	10.9'±	3.2± fps
Second Ave	10	3.0'±	1.1± fps
Brookside Ave E	4	8.7'±	2.9± fps
Farrell Ave	15	4.3'±	4.9± fps
Sparks Ave	10	4.9'±	4.4± fps
Manning Cr.	7	4.0'±	3.4± fps
Hillside Ave	13	5.0'±	2.1± fps
Brookside Ave W	13	9.0'±	3.9± fps
Stellar Ave	9	4.6'±	0.3± fps
Wolfs Ln	27	8.6'±	5.2± fps
Carol Rd.	25	8.6'±	3.0± fps
Iden Ave	10	8.1'±	3.8± fps
Total Houses Impacted	**343**		

Note: The flood depths and velocities were estimated from the HEC-RAS 2D results using the maximum values within the vicinity of the inundation zone for each area.

Table 8 indicates that approximately 343± houses would be impacted with flooding during the SDF failure. According to Section D.3 of the TOGS 3.1.5, Damage to 100 homes or more, implies Class C, even if flooding is less than 1 foot above the lowest occupied floor. Therefore, based on the results of the SDF failure, the dam would be classified as Class C. Figures 2 through 5 (New Rochelle Reservoir No. 1 Inundation Maps) show the houses within the inundation limits.

Table 9: Downstream Structure Impacts (Overtopping)

Street	No. of Resident Houses Impacted	Depth of Water (ft)	Average Flow Velocity (fps)
Norman Road	7	0.9'±	3.3± fps
Bon Air Ave	7	1.3'±	2.0± fps
Seacord Road	6	0.9'±	1.7± fps
Rosehill Ave	10	1.2'±	1.6± fps
Interlake Ave	15	5.1'±	1.8± fps
Robbins Rd. Area	18	1.6'±	0.6± fps
New Rochelle Rd. Area	16	5.2'±	3.7± fps
Hutchinson Blvd	11	7.9'±	5.8± fps
River Ave Area	10	6.2'±	4.9± fps
First Ave	9	8.2'±	2.8± fps
Second Ave	5	1.2'±	0.4± fps
Brookside Ave E.	3	6.6'±	2.2± fps
Farrell Ave	9	2.1'±	3.1± fps
Sparks Ave	8	2.9'±	3.6± fps
Manning Cr.	6	1.7'±	1.7± fps
Hillside Ave	7	2.7'±	1.3± fps
Brookside Ave W	11	6.7'±	3.3± fps
Stellar Ave	8	3.0'±	0.3± fps
Wolfs Ln	24	7.0'±	4.5± fps
Carol Pl.	24	7.1'±	2.6± fps
Iden Ave	9	6.6'±	3.9± fps
Total Houses Impacted	**223**		

Note: The flood depths and velocities were estimated from the HEC-RAS 2D results using the maximum values within the vicinity of the inundation zone for each area.

Table 9 indicates that approximately 223± houses would be impacted with flooding during an overtopping scenario without failure of the dam during the SDF.

3.3 Hazard Classification Assessment

The dam failure scenarios summarized in the previous section have identified the SDF failure as the most severe. Under this event, overtopping of an Urban Principal Arterial Expressway and Urban Principal Arterial Roadway at multiple locations as well as several other Urban Local and Minor Arterial roadways. In addition, over 100 residential houses lie within the inundation area and would be impacted from flooding.

In accordance with Section D.3 and D.4 of DEC's DOW TOGS 3.1.5, the impact of the dam's failure to over 100 residential houses downstream and on an Urban Principal Arterial Expressway would result in the dam being assigned a hazard classification of "C", or "High Hazard". Given that over 100 residential homes would be subject to potential damage and an Urban Principal Expressway (Hutchinson River Parkway) would be overtopped, occupants of the residences and vehicular traffic on the roadways would be in serious danger with the potential for loss of life due to the flood water. Accordingly, the dam's current hazard classification of "C", as assigned by the Dam Safety Section of DEC, is considered appropriate and fits the definition set forth in in DEC's DOW TOGS 3.1.5, *"Guidance for Dam Hazard Classification"*, where failure of a Class "C" or "High Hazard" structure would:

> *"result in widespread or serious damage to home(s); damage to main highways, industrial or commercial buildings, railroads, and/or important utilities, including water supply, sewage treatment, fuel, power,*

cable, or telephone infrastructure; or substantial environmental damage; such that the loss of human life or widespread substantial economic loss is likely."

4 Hydrologic & Hydraulic Assessment

4.1 Document Review

A previous hydrologic and hydraulic study was conducted by the New York District Corps of Engineer's entitled New Rochelle Reservoir No. 1, Phase 1 Inspection Report, National Dam Safety Program, February 1979. Based on the Phase 1 study, it was determined that the spillway does not have the capacity to safely convey the full or the ½ probable maximum flood (PMF). The spillway capacity was estimated to be 744 cfs and the full PMF and ½ PMF were estimated to be 4,036 cfs and 2,018 cfs respectively.

The previous hydrologic study prepared by the New York Corps of Engineers utilized Hydrometeorological Report No. 33 (HMR-33) to determine the PMF and incorporated the "Snyder Synthetic Unit Hydrograph method" and the "Modified Plus" flood routing procedure to determine the associated flows. In accordance with the NYSDEC DOW 3.1.4 – Guidance for Dam Engineering Assessment Reports, HMR-33 has been replaced by Hydrometeorological Report No. 51 and 52 (HMR-51/52). In addition, the hydrology model should use the appropriate geographic and hydrologic setting of the dam being studied.

4.2 Design Rainfall Analysis

According to the guidelines set forth in the DEC's DOW "*Guidelines for Design of Dams*", existing dams that are being rehabilitated should have adequate spillway capacity to pass the following floods without overtopping:

Hazard Classification	Spillway Design Flood (SDF)
A	100 year
B	150% of 100 year
C	50% of PMF

In addition, the service spillway design flood (SSDF) for existing dams is the same as shown for the new dams on Table 10 below:

Table 10: New and Existing Dams Hydrologic Design Criteria

Hazard Classification	Size Dam	Spillway Design Flood (SDF)	Service Spillway Design Flood (SSDF)	Minimum Freeboard (ft)
A	*Small	100 year	5 year	1
A	*Large	150% of 100 year	10 year	2
B	Small	225% of 100 year	25 year	1
B	Large	40% of PMF	50 year	2
C	Small	50% of PMF	25 year	1
C	Large	PMF	100 year	2

*SMALL - Height of dam less than 40 feet. Storage at normal water surface less than 1000-acre feet.
*LARGE - Height at dam equal to or greater than 40 feet. Storage at normal water surface equal to or greater than 1000-acre feet.

With a High Hazard classification (Class "C"), the spillway design flood for New Rochelle Reservoir No. 1 Dam is that developed under 50% of the probable maximum flood (PMF) with the service spillway design flood the 25-year storm event since size falls under the "small" criteria. Rainfall during the PMF estimated from the HMR 51/52 from the National Oceanic and Atmospheric Association (NOAA) equates to 34 inches of rainfall in 24-hours using a Type III distribution. Rainfall during the 25-year storm for the watershed were obtained from the point precipitation frequency estimates for Eastchester, NY area generated by the NOAA National Weather Service web site which was 6.54 inches. Refer to Appendix B for rainfall data.

4.3 Watershed Analysis

Runoff from the watershed was evaluated for the PMF storm event and the 25-year storm event using HEC-HMS developed by the US Army Corp of Engineer's (ACOE). The basic runoff computation methodology used the SCS method with a Type III 24-hour rainfall distribution.

The hydrologic analysis was based on a hydrologic and hydraulic study of the Hutchinson River developed by Leonard Jackson Associates in 2006 for the Federal Emergency Management Agency Flood Insurance Study (FEMA Study). Key parameters used in the FEMA study, such as runoff coefficient and times of concentrations, were reviewed and verified for accuracy and incorporated into the update HEC-HMS model. Delineation of the watershed tributary to New Rochelle Reservoir No. 1 Dam was determined using USGS topographic mapping and the USGS StreamStats website. The total drainage area to the dam is approximately 1.86 square miles which was subdivided into two sub areas, Sub-F and Sub-IE, for the hydrologic model. To be consistent with the FEMA Study modeling, the discharge generated from Sub-F is lagged before it is combined with the discharge generated from Sub-IE and then routed through the reservoir. The following table below shows the summary of the hydrologic parameters used in the HEC-HMS model.

Table 11: Hydrologic for Sub Areas to New Rochelle Reservoir No. 1 Dam

Sub Area	Drainage Area (Sq. Mi)	Runoff Coefficient (CN)	Tc Lag Time (min)
Sub-F	0.68	86	53.8
Sub-IE	1.18	81	66.7

The soils data for the site consisted of mostly Urban Land with other complexes which were classified as hydrologic soil groups B and C. The primary land use for the site consists of residential districts ranging from 1/8 acre to ½ acre lots. Other less significant land uses consisted of wood, open space, commercial and business districts and impervious cover.

The watershed delineations are shown on Figure 6 – Drainage Area Map and the Results of the HEC-HMS modeling is included in Appendix B.

4.4 Existing Spillway Capacity Analysis

With its high hazard classification, the spillway design flood for New Rochelle Reservoir Dam is 50% of the PMF. A center-weighted 24-hour storm distribution based on precipitation values obtained from the NOAA HMR-51 to determine the probable maximum precipitation (PMP). The PMP 24-hour precipitation is 34-inches for the watershed. Based in the NYSDEC Guidelines for Design of Dams, to correctly determine the peak flow, the rainfall values will be used for the PMF and the appropriate peak discharge will be computed. The peak discharge value, not the precipitation, is then multiplied by the appropriate percentage which in the case of the New Rochelle Reservoir No.1 Dam is 50%. For the analysis, the PMF was estimated to be 10,220 cfs which established 50% of the PMF to be 5,110 cfs. To generate the spillway design flood in the HEC-HMS modeling, the full PMF hydrographs for each contributing sub basin were generated and then exported into Excel where the resulting PMF hydrographs (34"/24hr) were reduced by 50% to represent the appropriate SDF hydrographs. The SDF hydrographs were then imported back into HEC-HMS at the associated sub basin using the Source Creation tool in HEC-HMS. The resulting source input hydrographs now represent 50% of the PMF in accordance with the Guidelines for Design of Dams.

As a basis for determining the dam's spillway capacity and weather the dam would be overtopped during the spillway design flood, a detailed survey of the New Rochelle Reservoir No. 1 Dam was performed, including a bathymetric survey of the impoundment area to develop the flood routings. From the survey, it was determined that the primary spillway crest has an elevation of 182.3 feet, and that the spillway has a width of 30.7-feet. In addition, the other sections of the dam, which include the masonry sections and earthen dikes, would be modeled as five (5) different sections. The left earthen dike was determined to have a crest elevation that varied from 186.48 feet to 188.4 feet with a width of 93 feet. The left masonry dam section was determined to have an average crest elevation of 186.41 feet with a width of 265.6 feet. The right masonry dam section adjacent to the spillway was determined to have an average crest elevation of 186.38 feet with a width of 210.2 feet. The right masonry dam section adjacent to the right earthen dike was determined to have an average crest elevation of 186.94 feet with a width of 144 feet. The right earthen dike was determined to have an average crest elevation of 188.61 feet with a width of 665 feet.

The SDF storm event (50% of the PMF) and the 25-year storm event were simulated over the watershed and routed through the reservoir. Below are the HEC-HMS results of the analysis.

Table 12: New Rochelle Reservoir No. 1 Dam – Existing Conditions

	Storm Event	
	25-Year	SDF (1/2 PMF)
Peak Flow to Dam (cfs)	1,487	5,110
Peak Elev. (ft)	185.36	188.14
Depth over Spillway (ft) Top of Spillway 182.3 (ft)	3.06	5.84
Dam Overtopping Depth (ft) Top of Masonry Dam 186.38± (ft)	N/A	1.76
Freeboard with Dam (ft)	1.02	0
Peak Flow Leaving Dam (cfs)	507	4,804

The peak water surface elevation as a result of the SDF is approximately 1.76 feet above the crest elevation of the masonry dam. During the storm event, flow is overtopping all portions of the masonry dam and a small portion of the earthen dike at the left end of the dam. During the 25-year storm event, all the flow is conveyed through the spillway.

According to the requirements of Section 5.3 of DEC's publication "Guidelines for Design of Dams", the dam's spillway does not have the capacity to safely pass the SDF and the dam is overtopped. Therefore, the dam is not in compliance with DEC's standards and the dam will have to be modified.

4.4.1 Alternative 1 – Spillway Modifications

To address the requirements of Section 5.3 of DEC's publication "Guidelines for Design of Dams", the dam's spillway needs to safely convey the SDF without overtopping. Since the current spillway does not have sufficient capacity, a viable alternative to bring the dam into compliance would be to modify the spillway.

A revised HEC-HMS model was developed that simulated a modified spillway that would provide sufficient capacity to convey the SDF. The modified spillway included a revised primary spillway with a lowered crest elevation of 179.5 feet and a revised width of 25.0 feet. The modifications also included an auxiliary spillway. The auxiliary spillway would replace the left earthen dike and left masonry dam section with a spillway crest elevation of 184.0 feet and a width of 345 feet. The remainder of the dam would remain at its existing conditions with the right masonry dam section adjacent to the spillway with an average crest elevation of 186.38 feet and a width of 210.2 feet. The right masonry dam section adjacent to the right earthen dike with an average crest elevation of 186.94 feet and a width of 144 feet. The right earthen dike with an average crest elevation of 188.61 feet and a width of 665 feet.

The SDF storm event and the 25-year storm event were simulated over the watershed and routed through the reservoir with the modified spillway. Below are the HEC-HMS results of the analysis.

Table 13: New Rochelle Reservoir No. 1 Dam – Proposed Conditions Alternative 1

	Storm Event	
	25-Year	SDF (1/2 PMF)
Peak Flow to Dam (cfs)	1,487	5,110
Peak Elev. (ft)	182.95	186.34
Depth over Spillway (ft) Top of Spillway 179.5 (ft)	3.45	6.84
Dam Overtopping Depth (ft) Top of Masonry Dam 186.38± (ft)	N/A	N/A
Freeboard with Dam (ft)	3.43	0.04
Peak Flow Leaving Dam (cfs)	494	4,580

Based on the results of the HEC-HMS analysis, the modified spillway under proposed Alternative 1 will provide the necessary capacity to convey the SDF while maintaining compliance with the DEC's requirements for dams. In addition, the spillway will have minimal impacts to the downstream communities with regards to increased discharges or water surface elevations when compared to existing conditions for the range of storms analyzed. Various configurations of additional spillway modifications were studied but were determined not to be practical since they typically resulted in increased flows downstream for some of the storms analyzed.

Although the modified spillway will have minimal to no adverse impact to the downstream communities for the storm events analyzed, the normal pool elevation in the impoundment area would be lowered 2.8± feet. This will have a major impact to the upstream communities surrounding the reservoir and the recreational aspect of the reservoir itself. This proposed alternative would have significant obstacles to overcome to get the necessary approval and is not a preferred option.

4.4.2 Alternative 2 – Overtopping Protection

For this alternative, the dam would remain in its existing conditions hydraulically but would be armored to allow the dam to be overtopped so it can safely convey the SDF without the potential for a breach. Under this scenario, the dam would be overtopped by approximately 1.76 feet during the SDF, similar to the existing condition described above. The dam and downstream slopes would need to be modified or armored to ensure the integrity and stability of the dam during an overtopping event. Overtopping protection can include a variety of materials or measures such as concrete, gabion basket, articulated concrete blocks, or other measures and can be further analyzed during the design phase.

Structural measures may be required to the dam to ensure the stability of the structure during overtopping. This may require, but not be limited to, increasing and enhancing the downstream slopes at some locations to address the existing height of the dam. Appropriate structural measures can be further analyzed during the design phase.

This alternative would be preferred since it would maintain the existing conditions both downstream with regards to discharges from the dam and upstream with regards to maintaining the existing normal pool elevation. Therefore, no changes would occur that would impact downstream and upstream communities with regards to flooding.

4.4.3 Alternative 3 – Dam Removal

Another alternative to address the conditions at the dam would be to provide a permanent breach of the dam. This would essentially remove the dam and impoundment area and eliminate the recreational aspect of the structure. A detailed hydrologic and hydraulic analysis was not investigated for this alternative at this time.

4.5 Low-Level Outlet

The existing gate house at the dam was reported to contain several pipes which passed through dam, presumably for low-level control of the reservoir water level and a water supply intake by the New Rochelle Water Company.

However, the pipes within the structure were reportedly sealed and no longer operable. Since the gates and valves in the gate house are arguably inoperable, the reservoir presently has no emergency maintenance drain capability. Further investigation is necessary to verify the conditions of the pipes and if they have been properly sealed.

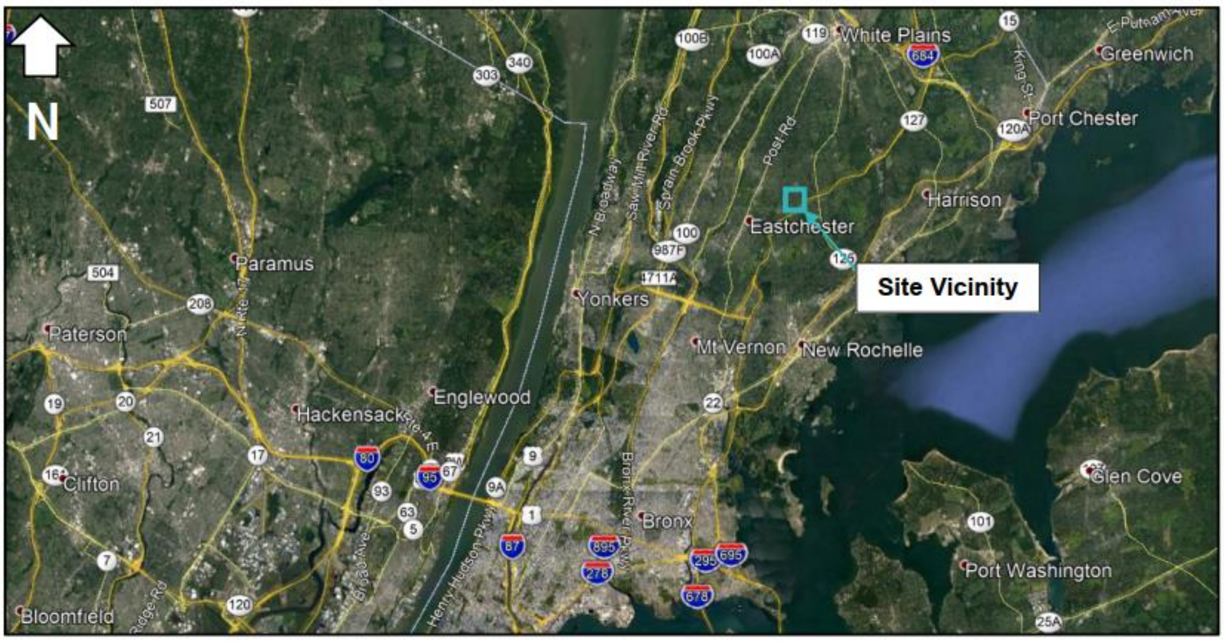
According to the DEC's regulations, a low-level outlet conduit or drain is required for emptying or lowering the water in case of emergency; for inspection and maintenance of the dam, reservoir, and appurtenances; and for releasing waters to meet downstream water requirements. The outlet conduit may be an independent pipe, or it may be connected to the service spillway conduit. The low-level drain is required to have sufficient capacity to discharge 90% of the storage below the lowest spillway crest within 14 days, assuming no inflow into the reservoir.

5 Subsurface Investigation

5.1 Introduction

Mott MacDonald assessed the current subsurface conditions at the New Rochelle Reservoir No. 1 Dam along the Hutchinson River. A subsurface investigation was performed consisting of four test borings, B-MM-1 to B-MM-4, advanced to split-spoon refusal, and eight rock probes, RP-1 to RP-8, advanced to roller-bit refusal. In borings B-MM-1 and B-MM-3, rock cores were advanced between ten and fifteen feet below the split-spoon refusal depth. Temporary groundwater wells were installed in each of the four borings, and groundwater measurements of the wells were taken throughout the duration of the site investigation. The purpose of the investigation was to obtain and report factual information regarding the soil overburden, depth to bedrock and groundwater conditions at selected locations downstream of the dam crest. Figure 7 indicates the approximate site location, and Figure 8 shows a close-up view of the project site. An as-drilled Boring Location Plan has been included in Appendix C.

Figure 7: Site Vicinity Map



Source: Google Earth, accessed 3/9/2020

Figure 8: Site Location Map

Source: Google Earth, accessed 3/9/2020

5.2 Geologic Desktop Study

Mott MacDonald conducted a desktop study of local geology within the project area using publicly available references such as published maps and online geologic databases. Findings from our desktop study are summarized within this Section and included in Attachment B.

5.2.1 Surficial Geology

Review of the University of the State of New York's "Surficial Geologic Map of New York – Lower Hudson Sheet," shows that the general facility of the project area is underlain by till. This till is described as clay, silt-clay, or boulder clay in valleys and the uplands, and gives way to a sandy till when underlain by gneiss or sandstone.

Mott MacDonald also reviewed the mapping from the Natural Resource Conservation Service's (NRCS) Web Soil Survey application. NRCS was initially created for agricultural purposes, however it also provides preliminary information related to soil chemistry within five feet of grade. NRCS identifies the project area to pose a high risk of corrosion to concrete, and a medium risk of corrosion to steel.

Table 14: NRCS Soil Properties

Soil Unit	Drainage Class	Available Water Storage	Depth to Water Table
Charlton Chatfield Complex	Well Drained	Moderate (~8.7 inches)	More than 80 inches
Urban Land-Paxton Complex	Well drained	Low (~4.1 inches)	About 18 to 37 inches

5.2.2 Bedrock Geology

Based on review of the mapping from the University of New York's "Geologic Map of New York – Lower Hudson Sheet," the project site lies within the Manhattan Formation, which consists of mica, schist, and calcite marble.

Bedrock outcrops are common in this area. Small amounts of overburden soils are expected throughout the project area, giving way to shallow bedrock in most areas.

5.3 Subsurface Investigation

Mott MacDonald retained Aquifer Drilling and Testing (ADT) Inc. of Mineola, New York to advance four geotechnical test borings (B-MM-1 through B-MM-4) and eight rock probes (RP-1 through RP-8) at select locations near the southern and eastern portions of the dam, downstream of the dam crest. Upon achieve refusal soil drilling in borings B-MM-1 and B-MM-3, a minimum ten-foot rock core was performed to identify bedrock composition.

5.3.1 Methodology

Each soil boring was sampled continuously to split-spoon refusal. Samples were taken using a CME-45C track mounted drill rig. Soil samples were collected using the Standard Penetration Test (SPT) Method in accordance with ASTM Standard D1586. The boreholes were advanced mud rotary drilling methods. Rock core samples were collected using an NQ core barrel and a diamond core bit utilizing wireline sampling technique. Upon boring completion, boreholes were grouted with cement-bentonite mix and restored to grade. Boring locations were determined using a handheld GPS unit while the elevations were interpolated from a site topographic map.

5.3.2 Generalized Subsurface Profile

A generalized profile of the subsurface conditions encountered is provided below. Typed soil boring logs are provided in Attachment C and should be consulted for a more detailed understanding of the site.

SAND (SM/SP): was encountered at ground surface in borings B-MM-1 and B-MM-2 and extended to split spoon refusal at 7.6 feet below ground surface (BGS) and 4.0 feet BGS, respectively. This stratum was generally described as medium dense to dense, brown to light gray coarse to fine sand, with varying amounts of silt and gravel.

SILT (ML): was encountered at ground surface in borings B-MM-3 and B-MM-4 and extends to two feet BGS. This stratum was generally described as medium stiff to stiff brown silt, with varying amounts of coarse to fine sand and trace coarse to fine gravel.

SAND (SM): was encountered underlying the silt stratum in borings B-MM-3 and B-MM-4 and extended to split spoon refusal at 2.8 feet BGS and 3.4 feet BGS, respectively. This stratum was generally described as medium dense to very dense, light brown coarse to fine sand with varying amount of coarse to fine gravel and silt.

BEDROCK (BR): Mica Schist was encountered under the sand layers in all borings and extended to 18.5 feet BGS. The Mica Schist stratum was generally described as light gray, coarse grained, slightly weathered to fresh, strong rock with moderate discontinuity spacing. Recovery was recorded to be between 93% and 100% while the Rock Quality Designation (RQD) was recorded to be between 70% to 98%.

5.3.3 Bedrock Depth

ADT utilized a Bobcat MT55 with an open-end rod sampler to advance eight rock probes along the existing dam. The table below lists the depths to top of bedrock in each rock probe and boring.

Table 15: Depth to Refusal

Rock Probe ID	Depth to Top of Rock (ft)
RP-1	3.0
RP-2	2.0
RP-3	10.0
RP-4	4.0
RP-5	4.0
RP-6	5.0
RP-7	4.0
RP-8	4.0

Boring ID	Depth to Top of Rock (ft)
B-MM-1	7.6
B-MM-2	3.9
B-MM-3	1.9
B-MM-4	4.4

5.3.4 Groundwater

During our investigation, temporary groundwater monitoring standpipes were installed in each of the four borings. Readings were taken immediately after the well was pumped dry by an electric pump. Readings were taken an additional two times over the duration of the investigation. Table 3 depicts the well construction information of each boring, while Table 4 lists the groundwater readings obtained at each location. The temporary standpipes were removed upon completion of the field investigation.

Table 16: Well Construction Information

Boring ID	Depth to Bottom of Boring (ft)	Depth to Top of Rock (ft)	Depth to Bottom of Slotted Screen (ft)	Depth to Top of Slotted Screen (ft)	Depth to Top of Well Sand in Annulus (ft)	Depth to Top of Bentonite Seal in Annulus (ft)
B-MM-1	18.5	7.6	18.5	8.5	7.5	6.5
B-MM-2	7.0	3.9	7.0	2.0	1.0	grade
B-MM-3	18.5	1.9	18.5	1.5	0.5	grade
B-MM-4	7.0	4.4	7.0	2.0	1.0	grade

Table 17: Groundwater Information

Boring ID	Initial Reading* (ft)	Elapsed time to second reading (hr:min)	Second Reading* (ft)	Elapsed time to final reading (hr:min)	Final Reading (ft)
B-MM-1	16.4	19:46	8.6	22:46	8.4
B-MM-2	6.8	18:15	6.3	21:15	6.2
B-MM-3	16.0	27:00	1.9	44:50	1.9
B-MM-4	5.0	25:23	4.1	42:53	4.4

*Depth below adjacent grade

5.4 Laboratory Testing

Representative rock core samples were submitted to Atlantic Testing Laboratories (ATL) of Canton, New York, an accredited geotechnical laboratory for compressive strength testing on rock cores, in accordance with ASTM D7012, Method C. Laboratory results have not yet been received, but will be summarized in the table below, and attached once received in Appendix C.

Table 18: Rock Testing Results

Boring ID	Sample	Depth (ft)	Unconfined Compressive Strength (psi)
B-MM-1	R-1	10.2 - 11.2	2,620
B-MM-3	R-2	11.3 - 12.5	2,680

5.5 Limitations

The information presented within this Section is based on our limited geotechnical investigation and laboratory testing performed in March 2020 and reflects geotechnical conditions we observed to be present at the time of our work. If there are changes to the proposed scope of work or if conditions change, Mott MacDonald should be given the opportunity to review and adjust the geotechnical information presented.

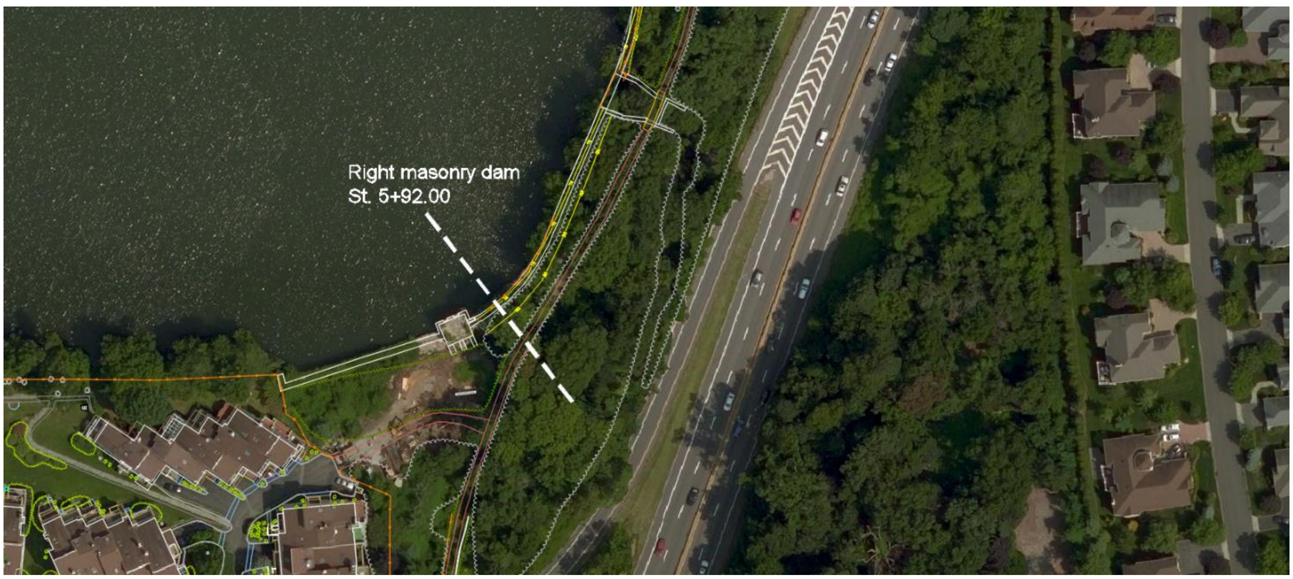
6 Stability Analysis

6.1 Introduction

Mott MacDonald assessed the current stability of the New Rochelle Reservoir No. 1 Dam. The dam itself consists of two earthen dikes and a central masonry section. The dam was constructed in 1894 with the reservoir now used primarily for recreational purposes. A previous study undertaken by the New York District Corps of Engineers (USACE 1979[1]), noted that several deficiencies exist and classified the structure as “high hazard”.

Mott MacDonald proceeded to undertaken ground investigations at the site and undertake further stability assessments of the main masonry dam and the earthen embankments. The analysis includes the selection of rock strength parameters based on the ground investigation data and the structural stability analysis undertaken for the masonry section of the New Rochelle Reservoir No. 1 Dam.

Figure 9: Masonry dam of New Rochelle Reservoir No. 1

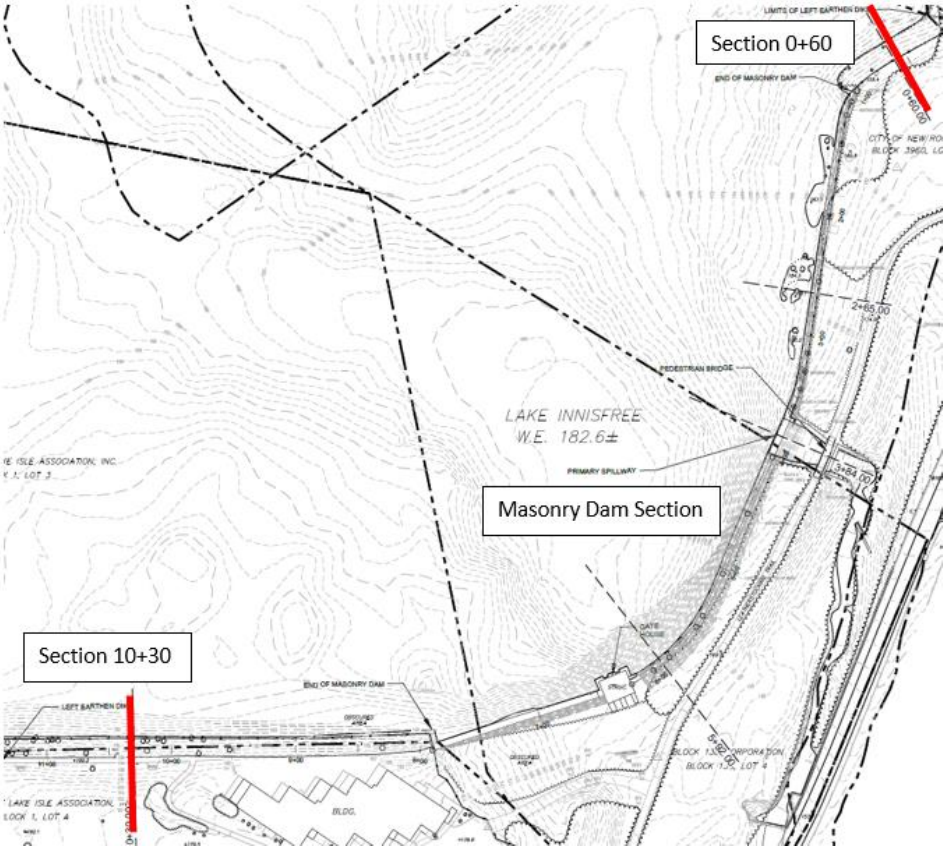


Source: Mott MacDonald, DigitalGlobe CNES (2020)

Two sections of the embankment (either side of the dam) have also been analyzed. The analysis is carried out using the software Geostudio SlopeW and SeepW. Figure 10 shows the locations of the sections (highlighted by the red line). The left dyke section, taken at chainage 0+60, is shown in Figure 11. The right dyke, taken at chainage 10+30 is shown in Figure 12.

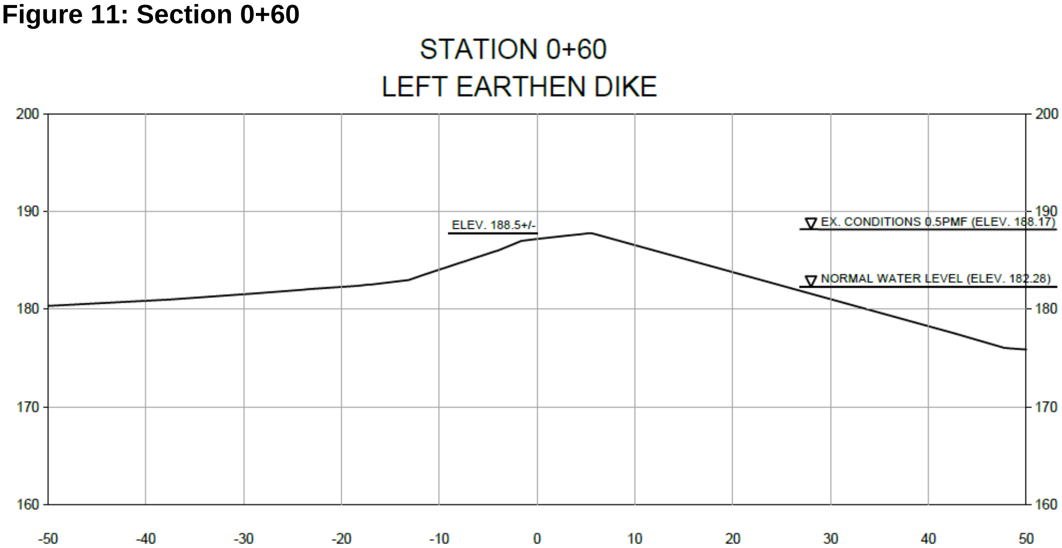
[1] USACE 1979. New Rochelle Reservoir No. 1. National Dam Safety Program. New York District Corps of Engineers.

Figure 10: Dam Layout and Embankment Section Location



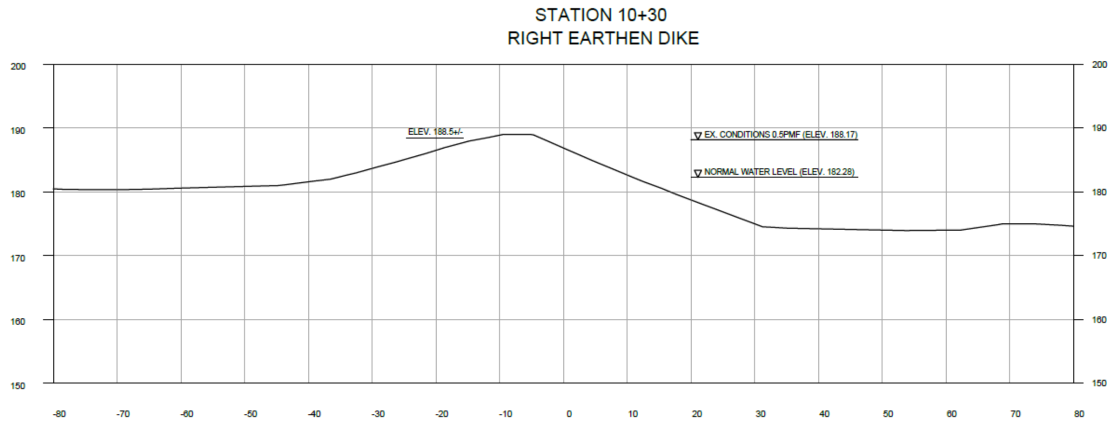
Source: Mott MacDonald

Figure 11: Section 0+60



Source: Mott MacDonald

Figure 12: Section 10+30



Source: Mott MacDonald

6.2 Masonry Dam

Source of Data

Basic geotechnical information is presented in Section 5 Subsurface Investigation. Based on the data collected as part of the ground investigations it was necessary to interpret appropriate ground strength parameters in order to inform the dam stability analysis.

Failure Modes and Strength Models

Due to practicality of undertaking surveys no intrusive investigations were made within the dam footprint. As such the depth of embedment of the dam into the rock has not been proven, and as such has to be taken as minimal for purposes of this assessment.

An initial review of the rock discontinuities and nature of the rock material found that the failure mode involving multiple wedges, as described in EM 1110-2-2200 (USACE 1995[2]), is not considered applicable due to there not being evidence of multiple sets of discontinuities in unfavorable orientations. Examination of the rock core logs indicates that the natural fabric of the rock (schistocity) is steeply inclined and the majority of the discontinuities at relatively shallow depth beneath the dam are also inclined at steep angles (40 - 65°). Sliding is therefore assumed to take place along the base of the dam, or close to it, were a plane of weakness in an unfavorable orientation was assumed to exist.

For such a failure, in order for sliding to take place, failure would have to occur along one of the following interfaces:

1. Interface 1: Along a pre-existing plane of weakness, such as a joint or along inherent weaknesses in the rock, for example along planes formed by the schistose fabric of the rock;
2. Interface 2: Along a failure surface through the rock mass (which would include failure partly along existing planes of weakness and partly through in-tact rock);
3. Interface 3: Along a new failure surface formed through solid rock;
4. Interface 4: Along the actual interface between the dam concrete and the rock surface.

Were a failure to occur, this would form along the interface that provides the least resistance to the imposed horizontal actions. Based on this assumption empirical models of rock strength exist for failure modes 1. to 3. above, however no such model exists for failure mode 4.

[2] USACE 1995. Gravity Dam Design. EM 1110-2-2200.

The rock strength models that have been used to derive strength parameters to assess resistance to sliding failure were therefore taken as the following:

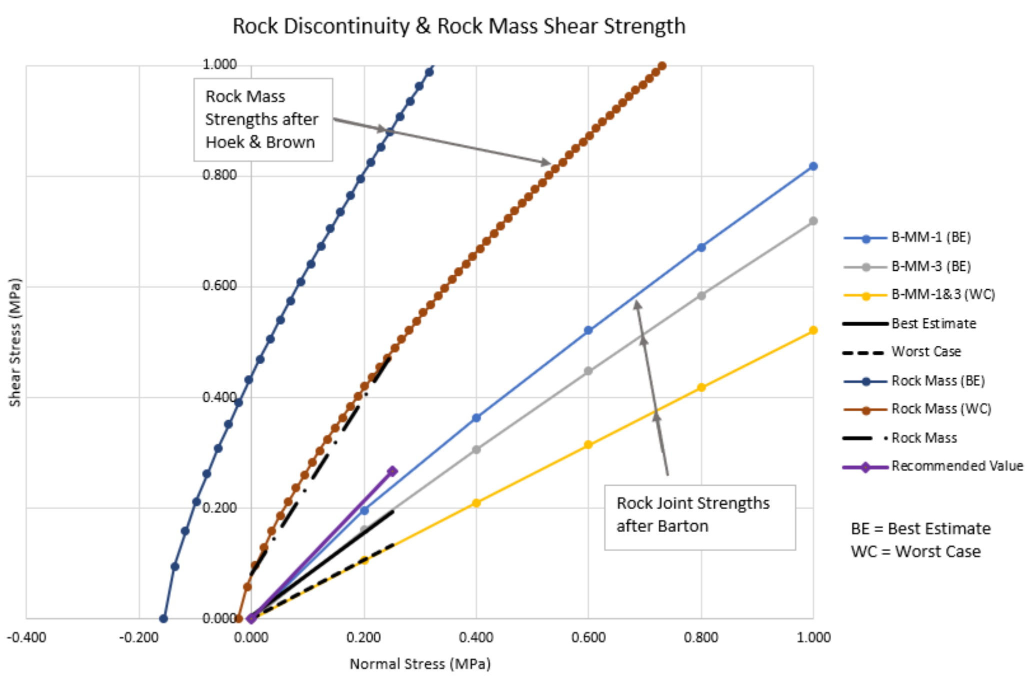
1. Interface 1: Pre-existing planes of weakness (joints, schistocity) analyzed using the 'Barton' or 'Barton and Chouby' method (Barton and Choubey 1977[3]);
2. Interface 2: Failure through the rock mass is analyzed using the 'Hoek-Brown' strength model (Hoek et al. 2002[4]);
3. Interface 3: Failure through in-tact rock would have a very high strength and will, by inspection, not be the worst case, so is not considered further;
4. Interface 4: For this case, it is necessary to assess which of the above methods are most analogous to this situation and thus most applicable.

Discussions

Based on the analysis methods described above non-linear shear strength envelopes were derived. However, for simplicity of input for the dam stability analysis, linear Mohr-Coulomb approximations were then derived to model these curves over the appropriate stress range.

For the assessment of pre-existing planes of weakness (Interface 1), the data for each borehole from boreholes B-MM-1 and B-MM-3 were evaluated to consider both a 'best estimate' and a 'worse case' scenario (blue, grey and yellow lines Figure). Following this, interface 2 - rock mass assessments were undertaken using all data from the boreholes to also provide a 'best estimate' and 'worse case' scenario (dark blue and brown lines Figure13). The results of the analyses are shown on Figure13.

Figure 13: Shear strength envelopes derived for rock joints and rock mass at the dam



Source: Mott MacDonald

[3] Barton, N.R., Choubey, V. 1977. The shear strength of rock joints in theory and practice. Rock Mechanics V 10(1-2), pp.1-54.

[4] Hoek, E., Carranza-Torres, C., Corkum, B., 2002. Hoek-Brown failure criterion-2002 edition. Proceedings of NARMS-Tac, 1(1), pp.267-273.

Table **19: Results of Assessment of Rock Strength Parameters**

Rock strength model	Angle of Friction, ϕ (deg.)	Cohesion (MPa)	Cohesion (psi)	Cohesion (kipf/ft²)
Schistose foliation (worst case)	28	0.000	0.0	0.00
Rock joint (best estimate)	37	0.005	0.7	0.10
Rock mass	58	0.080	11.6	1.67
Proposed value for stability check	47	0.000	0.0	0.00

Source: Mott MacDonald

The assessment of the strength along a plane of foliation within the schist bedrock was undertaken to assess the worst case in terms of strength along a pre-existing discontinuity. In this case the failure would occur along a joint plane characterized by a large proportion of weak, flaky mica crystals. There is no data for the base angle of friction of schist from the current investigation, but data from Le Cor et al. (2015[5]) suggests a basic friction angle of 27°. Taking a low estimate of rock strength and the least favorable rock joint conditions as recorded on the borehole logs, this results in a worst-case overall friction angle of the schistose foliation planes of 28°. However, there is no evidence that the schistose fabric of the rock at new Rochelle Dam is consistently aligned parallel with base of the dam. Indeed, the core photos suggest that it is steeply dipping. Therefore, it is considered that this mode of failure is not applicable.

The shear strength of 'normal' rock joints has also been assessed and the strength parameters $\phi = 37°$, $c = 0.1 \text{kipf/ft}^2$ have been derived. However, as with the foliation orientation, the dip of joints was not consistently horizontal, but varied significantly from 3° to 65° with an average of 25°. The only near-horizontal natural joints (with dips < 10°) are at over 10ft depth below rockhead in both cored boreholes. With the dam constructed in a sloping valley, it is conceivable that any near-horizontal joints will 'daylight' at rock head level somewhere along the dam footprint, and thus a joint could exist quite close to the underside of the dam in certain places. However, such locations would be only very localized and thus a failure along a single rock joint is very unlikely and accordingly the rock joint strength criteria should not apply to the general stability case.

This leaves failures through the rock mass as the most appropriate strength model. The rock mass strength criteria are much higher and include a significant cohesion. The parameters $\phi = 58°$, $c = 1.67 \text{kipf/ft}^2$ have been derived. The use of these criteria would require the assumption that the dam concrete is keyed sufficiently into the rock to force the failure surface through the rock mass. Given the likely method of excavation of the rock - probably by blasting then cleaning with hand tools - an undulating rock surface is likely to have been formed and this assumption is therefore likely to hold true. However, a surface between the dam concrete and the rock is not exactly the same as a surface within a rock mass, so some caution is required. Due to placement issues, shrinkage and so on a crack may exist at the dam/rock interface, and therefore it is considered a prudent measure to ignore the cohesion derived in the above calculation. In addition, it is common practice (for example see BS EN 1997-1 (2004) Clause 9.5.1) to reduce the friction angle of geo-materials when considering their friction angle against concrete structures. By reducing $\tan\phi$ by a factor of 2/3, an angle of friction of 47° is derived.

Therefore, the parameters recommended to be used for the base sliding stability check for New Rochelle Dam are $c' = 0 \text{kipf/ft}^2$, $\phi' = 47°$. The equivalent shear strength envelope is shown on Figure 2.1, and it can be seen to be a cautious estimate of the mobilized strength when compared to the rock mass, and comparable to that of natural rock joints in reasonable condition.

Various conceptual models have been considered to determine the strength of the interface between the base of the dam and the underlying schist foundation rock. Failure directly along the natural schistose fabric of this rock is considered to be too conservative an assumption. Similarly, a rock joint model is considered not to be applicable.

[5] Le Cor, T., Rangeard, D., Merrien-Soukatchoff, V. Simon, J., 2015. Mechanical characterization of weathered schists. Engineering Geology for Society and Territory Vol. 6.

Failure through the rock mass is considered more appropriate, but this should be applied with some caution. Cohesion has been ignored and a reduced angle of friction taken on this interface.

Therefore, the parameters recommended to be used for the base sliding stability check for New Rochelle Dam are $c' = 0 kipf/ft^2$, $\phi' = 47°$.

6.2.1 Dam Stability Analysis

Previous Study

The original calculations for the analysis of the structural stability of the masonry dam of New Rochelle Reservoir No. 1 were performed in 1979 by the New York District Corps of Engineers (USACE 1979).

The assessment reported that the available data concerning the masonry dam were extremely limited and that the stability of the structure was analyzed based on information from the 1913 and 1915 New York State Conservation Commission inspection reports and measurements made during the inspection in 1979. Therefore, the geometry (crest 28ft high and base 20ft long) , reservoir levels (water 24ft deep in normal conditions and 28.6ft deep at 50% of PMF) and mechanical properties of the dam and foundation (angle of friction 31°) as considered in this report are different from the ones used in the present report due to more up to date information available.

However, safety factors calculated for the dam masonry by USACE (1979) are presented in Table 20 for the purposes of comparison. It is noted that cracked section analysis and seismic analysis of the stability of the dam were not carried out.

Table 20: Dam stability analysis results from 1979 USACE inspection report

Loading case	Sliding safety factor	Overturning safety factor	Position of resultant force
Normal conditions, water at service spillway crest, no ice load	1.345	1.824	59.03% of base from U/S
Ice load of 5000psf, water at service spillway crest	1.052	1.403	73.93% of base from U/S
½ PMF, flow over non-overflow section, no ice load	0.939	1.519	68.70% of base from U/S
PMF, flow over non-overflow section, no ice load	0.886	1.450	71.26% of base from U/S

Source: USACE (1979)

The authors of the report concluded that the factors of safety against both overturning and sliding were less that those recommended by the USACE, although the actual safety factors of the dam may be higher than those calculated because conservative assumptions concerning the foundation conditions and depth of the embedment of the downstream toe made due to the lack of information.

6.2.2 Stability Assessment

Methodology

The static and seismic analysis of the stability of the masonry dam of New Rochelle Reservoir No. 1 was carried out in accordance with the guidelines from the New York State Department of Environmental Conservation (DEC 1989[6]). Table 21 presents the four loading conditions that were analyzed.

Table 21: Loading conditions and minimum safety requirements for an existing dam

Loading case	Description	Minimum safety requirements
Case 1	Normal loading condition; water surface at normal reservoir level.	• Overturning: resultant force in the middle third of the base; • Cracking: not permitted;

[6] DEC 1989. Guidelines for Design of Dams. New York State Department of Environmental Conservation.

Loading case	Description	Minimum safety requirements
		• Sliding: minimum safety factor is 1.5.
Case 2	Normal loading condition; water surface at normal reservoir level plus an ice load of 5,000 pounds per linear foot.	• Overturning: resultant force within the middle half of the base; • Cracking: permitted; • Sliding: minimum safety factor is 1.25.
Case 3	Design loading condition; water surface at spillway design flood level.	• Overturning: resultant force within the middle half of the base; • Cracking: permitted; • Sliding: minimum safety factor is 1.25.
Case 4	Seismic loading condition; water surface at normal reservoir level plus a seismic coefficient applicable to the location.	• Overturning: resultant force within the limits of the base; • Cracking: permitted; • Sliding: minimum safety factor is 1.00.

Source: DEC (1989)

The evaluation of the structural stability of a single 2D monolith of the gravity dam-foundation reservoir system was performed using the computer program CADAM2D 2.0.4. based on the gravity method (rigid body equilibrium and beam theory) and only considered failure along the base of the dam.

The seismic safety analysis was performed using the pseudo-static method. Case 4 was analyzed under Maximum Design Earthquake (MDE) as defined in publication ER 1110-2-1806 from USACE (2016[7]). Peak ground and spectral accelerations were estimated for the dam location (40.957°N, -73.799°E) using the USGS Unified Hazard Tool for the seismic event with a 5% probability of exceedance in 50 years (average return period of 975 years). Inertia forces induced by the earthquake were computed by means of a seismic response coefficient determined in accordance with ASCE/SEI 7-10 (ASCE 2013[8]).

Model Assumptions

The stability analysis of the masonry dam was performed assuming that:

- The dam consists of a full stone masonry structure.
- The depth of embedment of the dam into the rock was believed to be minimal. The dam was therefore assumed to be founded directly on rock and the foundation was assumed not to be pinned or anchored into the rock. Sliding was therefore assumed to take place along the base of the dam.
- The material strength properties at the dam-rock foundation interface were estimated based on available data from ground investigation carried out in the vicinity of the dam. Parameters used for the base sliding stability were determined assuming a “rock mass” analogy (failure through the rock mass). This assumption was applied with some caution by ignoring cohesion, reducing the angle of friction on the interface and assuming tensile strength equal to zero.
- Elevation of the base of submerged sediment material accumulated along the upstream face of the dam was estimated from current survey at the level where the slope of the bottom of the reservoir intercepts the dam heel. Because no information was available, properties of silt were determined as per literature.
- No passive earth pressure acting on the downstream face of the dam was included in the stability analysis.

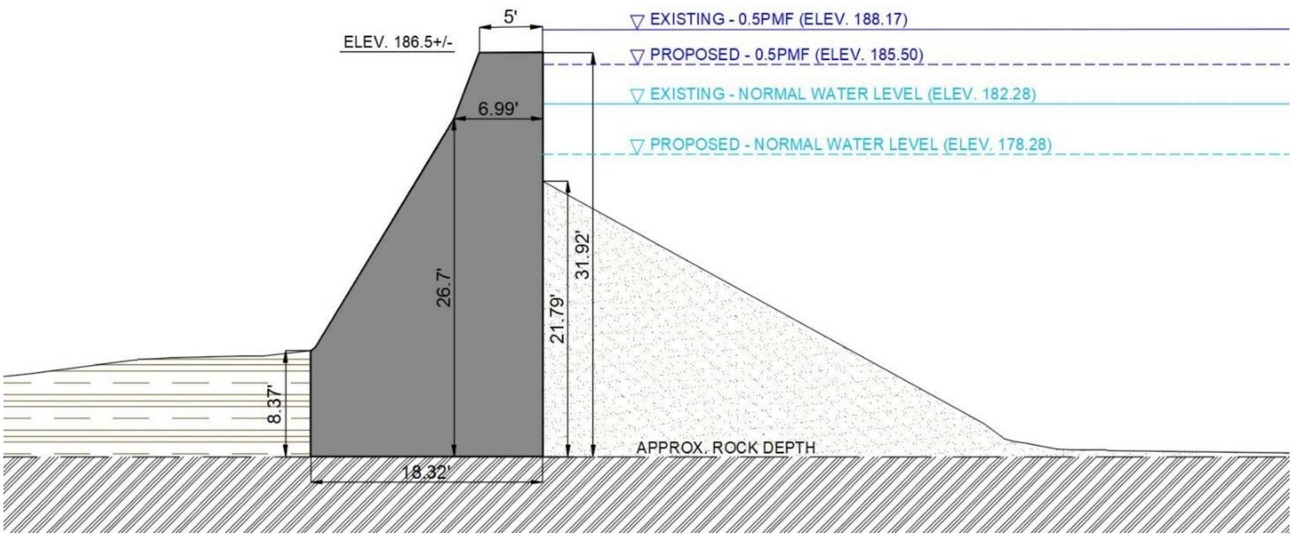
Crack initiation and propagation and effect of cracking on uplift pressures were evaluated during the analysis based on USBR (1976[9]).

[7] USACE 2016. Earthquake design and evaluation of civil works projects. ER 1110-2-1806. U.S. Army Corps of Engineers.
[8] ASCE 2013. Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-10. American Society of Civil Engineers.
[9] USBR 1976. Design of Gravity Dams.

Model Scenarios

The stability analysis of the masonry dam was conducted along a section through the dam located at station 5+92, where the height and inclination of the downstream slope appears to be the greatest. Overall dimensions of the section geometry as per current survey data are specified in Figure14.

Figure 14: Right masonry dam section (station 5+92)



Source: Mott MacDonald

Two scenarios were considered in the stability analysis of the dam:

- **Existing scenario:** considers the current geometry of the dam. Normal water level of the reservoir is elev. 182.28 ft (depth 27.74 ft). Spillway design flood level of the reservoir is elev. 188.17 ft (depth 33.63 ft), corresponding to the 50% of PMF.
- **Proposed scenario:** considers the construction of a new spillway by lowering the entire left side of the existing masonry dam. Normal water elevation of the reservoir is lowered by 4 feet to 178.28 ft (depth 23.74 ft). Maximum elevation for the 50% of PMF would be 185.50 ft, 1 foot below the remaining portion of the masonry dam (depth 30.92 ft). This scenario assumes 1 foot of freeboard. **Final stability calculations will be required for the final design of the modifications to spillway and further investigation of the composition of the dam.**

In order to assess the effect of crack initiation and propagation on the stability of the dam, two scenarios were evaluated considering the following cases:

- Cracking is allowed to take place during the analysis at the base of the dam;
- No cracking is possible at the base of the dam and the analysis is performed assuming linear elastic properties.

The adoption of such an approach assumes that in the first case no reliance on the tensile strength both between the foundation and rock mass and the masonry structure itself is made. However, where tensile strengths were found at the heel of the dam this assumption leads to the propagation of a crack through the foundation which can set up a feedback effect where small increases in loading cause significant reductions in factors of safety. As such, for comparison, a second case assumed that, where tension is present at the heel of the dam, this does not exceed the tensile strength of either the foundation rock mass interface; nor that of the masonry structure itself. As such, for this case, minimum allowable tensile strength values required to prevent crack initiation at the heel of the dam for each load combinations are also presented.

6.2.3 Results and Discussions

Existing Scenario

Results of the loads acting on the masonry dam of New Rochelle Reservoir No. 1 are presented for each loading combination from Table 22 to Table 25. It should be noted that uplift forces both prior to cracking and post cracking are presented demonstrating the increases in loading at foundation level resulting from assuming no tensile strength within the dam foundation.

Table 22: Case 1: Normal loading condition

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	23.965	9.246
Uplift (no cracking)	15.827	6.106	–	–
Uplift (cracking)	21.207	6.633	–	–
Silt	–	–	2.797	7.265

Source: Mott MacDonald

Table 23: Case 2: Normal loading condition plus ice load

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	23.965	9.246
Uplift (no cracking)	15.827	6.106	–	–
Uplift (cracking)	31.654	9.160	–	–
Ice	–	–	5.000	27.238
Silt	–	–	2.797	7.265

Source: Mott MacDonald

Table 24: Case 3: Design flood loading condition

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	35.132	11.154
Crest overtopping	-0.432	2.308	–	–
Uplift (no cracking)	19.188	6.106	–	–
Uplift (cracking)	38.376	9.160	–	–
Silt	–	–	2.797	7.265

Source: Mott MacDonald

Table 25: Case 4: Seismic loading condition

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	23.965	9.246
Uplift (no cracking)	15.827	6.106	–	–
Uplift (cracking)	21.207	7.328	–	–
Silt	–	–	2.797	7.265
Inertia dam	–	–	4.682	12.734
Inertia reservoir	–	–	2.089	11.095
Inertia silt	–	–	0.414	8.717

Source: Mott MacDonald

Safety Factors (cracking allowed)

Global factors of safety and location of resultant force for the masonry dam of New Rochelle Reservoir No. 1 are presented in Table 26.

Table 26: Dam stability analysis results assuming cracking is allowed

Loading case	Sliding safety factor		Position of Resultant force		Comments
	Calculated	Required	Calculated	Required	
Case 1	1.487	1.500	77.99% of base from U/S	In middle third of base	Calculated sliding safety factor is unsatisfactory. Due to unacceptable resultant location, tensile stresses are present at the heel of the dam leading to crack propagation at the base of the dam.
Case 2	0.900	1.250	105.88% of base from U/S	In middle half of base	Calculated sliding safety factor is unsatisfactory. Resultant falls outside of the base of the dam. Tensile stresses are present at the heel of the dam leading to cracking through the base.
Case 3	0.576	1.250	131.47% of base from U/S	In middle half of base	Calculated sliding safety factor is unsatisfactory. Resultant falls outside of the base of the dam. Tensile stresses are present at the heel of the dam leading to cracking through the base.
Case 4	1.172	1.000	90.71% of base from U/S	Within the base	Calculated sliding safety factor is in excess of the minimum required value. Resultant location is acceptable, although tension at the dam heel still results in cracking at the base of the dam.

Source: Mott MacDonald

Safety Factors (cracking not allowed)

Results of the stability analysis of the masonry dam of New Rochelle Reservoir No. 1 are presented in Table 27. Minimum allowable tensile strength values required to prevent crack initiation are presented in Table 28.

Table 27: Dam stability analysis results assuming cracking not allowed

Loading case	Sliding safety factor		Position of resultant force		Comments
	Calculated	Required	Calculated	Required	
Case 1	1.703	1.500	73.78% of base from U/S	In middle third of base	Calculated sliding safety factor is satisfactory. Resultant location is unacceptable, leading to tensile stresses at the heel of the dam.
Case 2	1.435	1.250	91.27% of base from U/S	In middle half of base	Calculated sliding safety factor is satisfactory. Resultant location is unacceptable, leading to tensile stresses at the heel of the dam.
Case 3	1.119	1.250	100.04% of base from U/S	In middle half of base	Calculated sliding safety factor is unsatisfactory. Resultant location is unacceptable, as it falls outside of the base of the dam.
Case 4	1.342	1.000	84.88% of base from U/S	Within the base	Calculated sliding safety factor is satisfactory. Resultant location is acceptable.

Source: Mott MacDonald

Table 28: Minimum tensile strength at the dam-rock foundation interface to prevent cracking

Loading case	Normal upstream stress at the heel of the dam base (kipf/ft^2)	Tensile strength initiation coefficient from USBR (1976)	Minimum tensile strength of the dam base to prevent cracking (kipf/ft^2)
Case 1	0.990	3	2.969
Case 2	3.425	3	10.274
Case 3	4.325	2	8.649
Case 4	2.534	1	2.534

Source: Mott MacDonald

Stability analysis for the masonry section of the dam indicates that in the existing scenario the factors of safety are unsatisfactory for several loading conditions. These factors of safety indicate a critical deficiency in the stability of this structure even if cracking is assumed not to take place during the analysis. However, the results shown above should be viewed with respect to the following limitations of the modelling

1. No allowance for passive resistance at the downstream toe of the dam was included;
2. It has been assumed that the foundation is at rock level with no significant embedment;
3. No allowance for 3D distribution of loads has been made;
4. The results present both the case which assumes no tensile strength and the tensile strength is not exceeded. It is likely that the real case is between the two;
5. The foundation strength parameters may underestimate the foundation strength properties;

In particular we note that despite some of the very low factors of safety the dam has stood for a significant amount of time, which would suggest that at no point during its lifetime has the factor of safety been lower than 1.

Proposed Scenario

Results of the loads acting on the masonry dam of New Rochelle Reservoir No. 1 following lowering of the spillway are presented for each loading combination from Table 29 to Table 32.

Table 29: Case 1: normal loading condition

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	17.552	7.913
Uplift (no cracking)	13.545	6.106	–	–
Uplift (cracking)	13.545	6.106	–	–
Silt	–	–	2.797	7.265

Source: Mott MacDonald

Table 30: Case 2: normal loading condition plus ice load

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	17.552	7.913
Uplift (no cracking)	13.545	6.106	–	–
Uplift (cracking)	18.865	6.783	–	–
Ice	–	–	5.000	23.238
Silt	–	–	2.797	7.265

Source: Mott MacDonald

Table 31: Case 3: design flood loading condition

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	29.778	10.307
Uplift (no cracking)	17.643	6.106	–	–
Uplift (cracking)	35.285	9.160	–	–
Silt	–	–	2.797	7.265

Source: Mott MacDonald

Table 32: Case 4: seismic loading condition

Load type	Vertical load (kipf/ft)	Position from upstream (ft)	Horizontal load (kipf/ft)	Elevation (ft)
Dam weight	-58.319	7.357	–	–
Hydrostatic reservoir	–	–	17.552	7.913
Uplift (no cracking)	13.545	6.106	–	–
Uplift (cracking)	13.545	6.106	–	–
Silt	–	–	2.797	7.265
Inertia dam	–	–	4.682	12.734
Inertia reservoir	–	–	1.530	9.495
Inertia silt	–	–	0.414	8.717

Source: Mott MacDonald

Safety Factors (cracking allowed)

Results of the stability analysis of the masonry dam of New Rochelle Reservoir No. 1 are presented in Table 33.

Table 33: Dam stability analysis results assuming cracking is allowed

Loading case	Sliding safety factor		Position of resultant force		Comments
	Calculated	Required	Calculated	Required	
Case 1	2.360	1.500	61.63% of base from U/S	In middle third of base	Calculated sliding safety factor is satisfactory. Resultant location is acceptable. No cracking is expected.
Case 2	1.669	1.250	79.76% of base from U/S	In middle half of base	Calculated sliding safety factor is satisfactory. Resultant falls outside the middle half of the base but resulting tensile stresses and cracking at the base of the dam do not compromise the stability of the dam.
Case 3	0.758	1.250	102.63% of base from U/S	In middle half of base	Calculated sliding safety factor is unsatisfactory. Resultant falls outside of the base of the dam. Tensile stresses are present at the heel of the dam leading to cracking through the base.
Case 4	1.780	1.000	71.11% of base from U/S	Within the base	Calculated sliding safety factor is in excess of the minimum required value. Resultant location is acceptable.

Source: Mott MacDonald

Safety Factors (cracking not allowed)

Results of the stability analysis of the masonry dam of New Rochelle Reservoir No. 1 are presented in Table 34. Minimum allowable tensile strength values required to prevent crack initiation are presented in Table 35.

Table 34: Dam stability analysis results assuming cracking not allowed

Loading case	Sliding safety factor		Position of resultant force		Comments
	Calculated	Required	Calculated	Required	
Case 1	2.360	1.500	61.63% of base from U/S	In middle third of base	Calculated sliding safety factor is satisfactory. Resultant location is acceptable.
Case 2	1.894	1.250	75.80% of base from U/S	In middle half of base	Calculated sliding safety factor is satisfactory. Resultant falls outside the middle half of the base but resulting tensile stresses at the base of the dam do not compromise the stability of the dam.
Case 3	1.339	1.250	87.03% of base from U/S	In middle half of base	Calculated sliding safety factor is satisfactory. Resultant falls outside the middle half of the base but resulting tensile stresses at the base of the dam do not compromise the stability of the dam.
Case 4	1.780	1.000	71.11% of base from U/S	Within the base	Calculated sliding safety factor is satisfactory. Resultant location is acceptable.
Case 4	1.780	1.000	71.11% of base from U/S	Within the base	Calculated sliding safety factor is in excess of the minimum required value. Resultant location is acceptable.

Source: Mott MacDonald

Table 35: Minimum tensile strength at the dam-rock foundation interface to prevent cracking

Loading case	Normal upstream stress at the heel of the dam base (kipf/ft²)	Tensile strength initiation coefficient from USBR (1976)	Minimum tensile strength of the dam base to prevent cracking (kipf/ft²)
Case 1	-0.738	3	0.000
Case 2	1.339	3	4.017
Case 3	2.713	2	5.426
Case 4	0.652	1	0.652

Source: Mott MacDonald

The proposed rehabilitation measures appear to improve the structural stability of the masonry dam by reducing the water level in the reservoir. In the proposed scenario the likelihood of failure along the base of the dam is significantly reduced for most of the considered loading conditions. However, it is noted that the tension that develops at the heel of the dam for design loading condition (Case 3) is significant enough to lead to cracking through the whole base of the dam resulting in the possible failure of the structure.

6.2.4 Conclusion

Stability analysis for the masonry section of the dam of New Rochelle Reservoir No. 1 indicates that, for the existing scenario, the factors of safety are less than those recommended by the New York State Department of Environmental Conservation (DEC 1989). Although this result is consistent with the conclusions from the 1979 New York District Corps of Engineers report (USACE 1979), it is noted that, for the purpose of the analysis, certain conservative assumptions were made, and the actual safety factors may be higher than those calculated. It is therefore possible that additional investigations such as those to determine the composition of the dam core or dam foundation level would allow a better assessment of the structural stability of the masonry dam.

Modelling of the proposed lowering of water levels within the reservoir are expected to improve the structural stability of the dam by reducing the horizontal hydrostatic pressure acting on the upstream dam face during both normal and flood conditions. However, it is noted that, even assuming that no cracking can take place at the base of the dam, the minimum safety requirements recommended by the New York State Department of Environmental Conservation (DEC 1989) are not still met for all the considered loading conditions, although this does not necessarily suggest ultimate failure. As noted the proposed scenario assumed one (1) foot of freeboard. If the new spillway was raised to eliminate all freeboard the hydrostatic loading conditions would increase thereby reducing the factors of safety. **Final stability calculations will be required for the final design of the modifications to spillway and further investigation of the composition of the dam.**

Finally, it is noted that, regarding the modelling of the structure behavior under tension, the tensile strengths for concrete is between 41 kipf/ft² and 104 kipf/ft² and for masonry around 2kipf/ ft². From the results presented within Table 28 and Table 35, assuming that the structure were constructed entirely from masonry then the tensile strength of the structure is likely to be exceeded in most cases with results tending towards the lower end of the estimates, however were the core to be constructed from concrete then results are likely to tend towards the upper estimates presented here under the conditions presented above.

6.3 Earthen Dikes

This analysis also follows the requirements set out in the New York State Department of Environmental Conservation (DEC 1989) and USACE EM 1112-2-1902: Slope Stability. These guidelines outline five cases that should be analyzed for existing dams (Table 36). USACE EM 1112-2-1902: Slope Stability outlines the load cases and target factors of safety for new earth dams (Table 3-1) and advises that these should be taken into consideration for existing dams. The target factors of safety have been adopted from this guidance, however as it is an existing dam, the guidance suggests that the factors can be less than those for the design of new dams, considering the benefits of being able to observe the actual performance of the embankment over a period of time.

As Case 2 and 4 target factors of safety were not explicitly advised upon in the guidance, the factors are adopted from experience with similar projects and are considered reasonable.

Table 36: Loading conditions and minimum safety requirements for an existing dam

Loading case	Description (DEC 1989)	Description (USACE EM 1112-2-1902)	Target Factor of Safety
Case 1 (DEC 1989)	Normal loading condition; water surface at normal reservoir level.	Long-term (Steady seepage or maximum storage pool)	1.5 (USACE EM 1112-2-1902)
Case 2 (DEC 1989)	Normal loading condition; water surface at normal reservoir level plus an ice load of 5,000 pounds per linear foot.	Long-term (Steady seepage or maximum storage pool)	1.3
Case 3 (DEC 1989)	Design loading condition; water surface at spillway design flood level.	Maximum Surcharge Pool	1.3 (USACE EM 1112-2-1902)
Case 4 (DEC 1989)	Seismic loading condition; water surface at normal reservoir level plus a seismic coefficient applicable to the location.	ER 1110-2-1806 for guidance	1.1
Case 5	Rapid drawdown loading condition; water surface at normal reservoir level and then is drawn down to reservoir empty level.	Rapid Drawdown from maximum storage pool	1.3 (USACE EM 1112-2-1902)
Case 6	Rapid drawdown loading condition; water surface at full reservoir level and then is drawn down to reservoir empty level.	Rapid Drawdown from maximum surcharge pool	1.1 (USACE EM 1112-2-1902)

6.3.1 Geotechnical Inputs

The geotechnical inputs were taken from the subsurface investigations reference in Section 5 of this report. The ground water level information was inconsistent, so therefore to be conservative the downstream ground water level has been assumed to be at ground level. Table 37 shows the material parameters used for the seep and slope analysis.

Both the left (0+60) and right (10+30) dykes were analyzed as they have different material layers. The right dyke section was chosen as it has the most critical geometry and the left dyke section was taken as that prepared from the CAD model and assumed to be the most critical section. The geological models for the right and left dyke are shown in Figure 15 and Figure 16, respectively.

The assumptions are as follows:

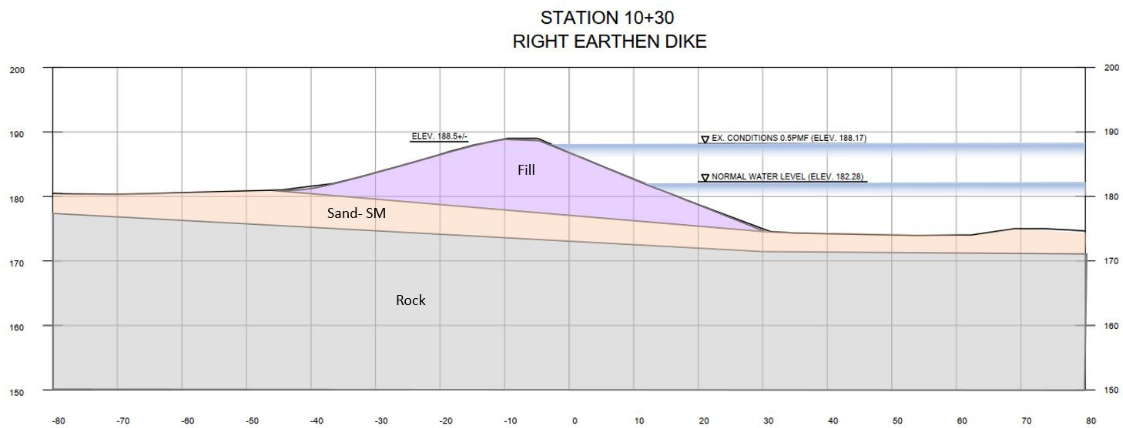
- Geological sections assumed from the slope geometry, but also with recognition of soil depths in areas outside the dam footprint.
- Soil densities assumed from ranges given in BS 8002, 2015 and experience.
- Soil strengths assumed from relationships in BS 8002 2015, and experience.
- Permeabilities assumed from experience.
- Asterix after values in table below signify that it was determined from investigation data.

Table 37: Material Parameters

Material	Moisture Condition	Unit Weight (pcf)	Cohesion (psf)	Phi (deg)	Permeability (ft/s) *	Compressibility (1/psf) *	Saturated Water Content *
Fill	NMC	121	0	28	3.3x10-7	0.014	0.25
Silt	NMC	114.6	0	29	3.3x10-7	0.001	0.16
Silty Sand	NMC	114.6	0	32	3.3x10-6	0.001	0.2
Rock	Saturated	159.2	208.85*	47*	3.3x10-8	0.0002*	0.04

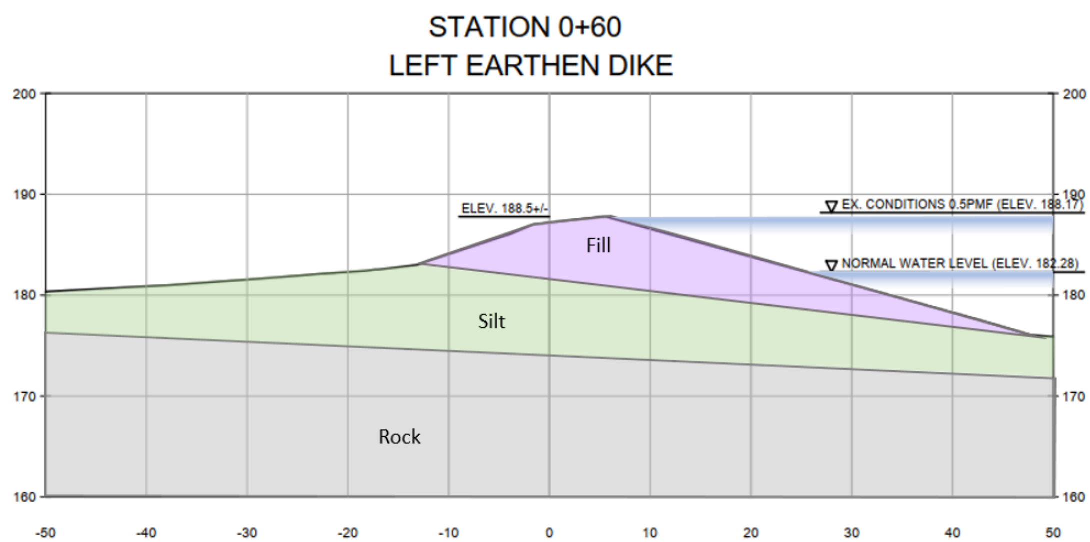
Source: Mott MacDonald [NMC = Natural Moisture Content; all parameters determined from published references and experience, except '*' which were determined from investigation logging, sampling & testing.]

Figure 15: Section 10+30 Geological Model



Source: Mott MacDonald

Figure 16: Section 0+50 Geological Model



Source: Mott MacDonald

6.3.2 Seep Analysis

A seep analysis was carried out to establish the piezometric line through the embankment. In the cases where the normal (operational) level in the reservoir was considered using a steady-state analysis assuming long term maintenance of the design water level.

In the case of the flood event the flood hydrograph was converted to a water level vs time relationship and inputted into the model in order to undertake a transient condition assessment and establish design embankment pore water pressures. However, given uncertainties around material properties and likely variability actual inflow conditions a second steady-state analysis was also carried out assuming that the full reservoir level has been maintained for a long time to give a lower bound series of results.

In the case of the rapid drawdown case a drawdown rate of 3ft/day was assumed (from normal level to reservoir empty level (maximum storage pool) and from full level to reservoir empty level (maximum surcharge pool)) for the purposes of this assessment.

The flood event and rapid drawdown graphs of water level vs time are presented in the Appendix.

6.3.3 Slope Stability Analysis

The slope stability was carried out on both the upstream and the downstream banks of the sections. The assessment was completed using global factors of safety assuming drained conditions. It has been assumed that there is no loading on the crest for any of the cases. The following summarizes the assumptions for each case.

Inputs for Case 2: The depth of ice was assumed to be 1ft. A horizontal force of 5000 lbf was applied 0.67ft below the normal (operational) water level to represent ice flows that could occur during winter times. This value was recommended by the New York State Department of Environmental Conservation (DEC 1989).

Inputs for Case 3: In order to conservatively assess the flood case, 3 separate scenarios were considered: (1) steady-state piezometric line, (2) transient piezometric line and (3) addition of pore water pressure (Ru=0.2) in the top 1.5ft of the embankment fill to represent heavy rainfall during a flood event.

Inputs for Case 4: The horizontal seismic coefficient is 0.08 and the vertical seismic coefficient is 0.04 as established by seismic calculations carried out in the New Rochelle Reservoir No. 1 Dam Stability Analysis (Mott MacDonald, 2020).

6.3.4 Discussion

The results from the slope stability analysis are shown in Table 38. It can be seen that the following scenarios have not met the target factors of safety:

- Left Bank (D/S) – Case 3 – Full (Steady State)
- Left Bank (U/S) – Case 5 – Rapid Drawdown from maximum storage pool (normal level)
- Left Bank (U/S) – Case 6 – Rapid Drawdown from maximum surcharge pool (full level)
- Right Bank (U/S) – Case 1 – Normal
- Right Bank (U/S) – Case 4 – Normal with seismic
- Right Bank (U/S) – Case 5 – Rapid Drawdown from maximum storage pool (normal level)
- Right Bank (U/S) – Case 6 – Rapid Drawdown from maximum surcharge pool (full level)

The left bank full steady state is considered to be highly conservative as it assumes that the flood remains at that level for an infinite amount of time. Whilst unrealistic it was principally considered due to the assumptions made regarding the material parameters and likely inflow hydrograph conditions. Therefore, due to the high factor of safety achieved with the transient analysis and the marginal failure of the steady state, this case can be deemed stable.

The left and right bank were found to both fail in the rapid drawdown cases due to the high pore water pressures which still remain in the embankment whilst the water level is low in the reservoir. The drawdown rate of 3ft/day has been assumed.

The right bank normal case was found to not meet the required criteria principally due to the slope of the upstream bank being 1:2.5. This is typically steeper than most soil embankments. As such it was also found that the normal with seismic case did not meet the required criteria.

The average depth of slip circles which would cause a breach in the embankment (crest edge to toe slip circles) is 5ft. Smaller, less critical, slip circles have an average depth of 1ft. Figures of the model runs which have failed are presented in the appendix.

Table 38: Stability Analysis Results

Section	Case	Bank	Target Factor of Safety	Achieved Factor of Safety
Left Embankment (0+60)	Case 1 - Normal	U/S	1.5	1.70
	Case 1 - Normal	D/S	1.5	1.69
	Case 2 - Normal with Ice	U/S	1.3	1.77
	Case 2 - Normal with Ice	D/S	1.3	1.69
	Case 3a - Full (Steady-State)	U/S	1.3	2.01
	Case 3a - Full (Steady-State)	D/S	1.3	1.28
	Case 3b - Full (Transient)	U/S	1.3	4.69
	Case 3b - Full (Transient)	D/S	1.3	1.62
	Case 3c - Full with Ru	U/S	1.3	N/A
	Case 3c - Full with Ru	D/S	1.3	1.99
	Case 4 - Normal with Seismic	U/S	1.1	1.18
	Case 4 - Normal with Seismic	D/S	1.1	1.34
	Case 5 - Rapid Drawdown from maximum storage pool	U/S	1.3	0.36
	Case 5 - Rapid Drawdown from maximum storage pool	D/S	1.3	2.00
	Case 6 - Rapid Drawdown from maximum surcharge pool	U/S	1.1	0.32
	Case 6 - Rapid Drawdown from maximum surcharge pool	D/S	1.1	1.14
Right Embankment (10+30)	Case 1 - Normal	U/S	1.5	1.25
	Case 1 - Normal	D/S	1.5	1.92
	Case 2 - Normal with Ice	U/S	1.3	1.34
	Case 2 - Normal with Ice	D/S	1.3	1.91
	Case 3a - Full (Steady-State)	U/S	1.3	1.55
	Case 3a - Full (Steady-State)	D/S	1.3	1.40
	Case 3b - Full (Transient)	U/S	1.3	3.66
	Case 3b - Full (Transient	D/S	1.3	2.01
	Case 3c - Full with Ru	U/S	1.3	N/A
	Case 3c - Full with Ru	D/S	1.3	1.62
	Case 4 - Normal with Seismic	U/S	1.1	0.97
	Case 4 - Normal with Seismic	D/S	1.1	1.48

	Case 5 - Rapid Drawdown from maximum storage pool	U/S	1.3	0.21
	Case 5 - Rapid Drawdown from maximum storage pool	D/S	1.3	2.06
	Case 6 - Rapid Drawdown from maximum surcharge pool	U/S	1.1	0.11
	Case 6 - Rapid Drawdown from maximum surcharge pool	D/S	1.1	1.44

6.3.5 Conclusion

Based on the above the following recommendations are therefore made

1. The geotechnical investigation should be expanded to include soil borings within the earthen dikes. The assumptions made in the analysis should be validated.
2. The earthen dikes must be monitored during water drawdowns. Consider reducing the upstream slopes on both the right and left upstream embankments in order to establish suitable factors of safety for drawdowns.

7 Conclusion & Recommendations

Documentation reviewed for the dam included historic documents, past inspections reports, the dam's inspection and maintenance plan (IMP) prepared in January 2020, the dam's EAP report prepared in January 2020, and a Safety Inspection Report dated January 2020. Based upon our review of this information, the Safety Inspection, the spillway capacity and stability calculations performed for this Engineering Assessment, we agree with the current rating of "Unsound, Deficiency Recognized" and recognize that the dam does not meet all of DEC's dam safety criteria. The following sections summarize the conditions requiring correction or further investigation.

7.1 Maintenance Deficiencies

7.1.1 Masonry Dam and Earthen Dikes

Significant trees and woody vegetation were observed on the downstream slope of the dam embankment at the masonry section, specifically along the downstream toe to the right of the primary spillway. In addition, significant trees and woody vegetation were observed along the upstream slope of the right earthen dike. A development plan should be implemented for the removal of the trees and woody vegetation along the dam. The plan should include removal of large stumps and large roots, clearing the vines and root intrusion on the masonry dam, restoring/replenishing of riprap slope protection as needed, filling in eroded areas and depressions and reestablishing grass vegetation.

Several of the mortar joints along the masonry dam need repair due to erosion and root intrusion. In addition, some minor seepage or wet spots were observed along the stone masonry dam. The damaged or missing joints should be repaired to help reduce the potential for seepage. Any observed seepage or wet spots should continue to be monitored for changes in the current conditions.

There is security fencing along the crest of the masonry section of the dam. Consideration should be given to removing or relocating the fencing from the crest of the dam. The fencing along the crest could be a safety concern to the structural integrity of the dam during an overtopping event.

7.1.2 Spillway and Discharge Channel

Some undermining of the spillway wing walls was observed along the spillway chute/discharge channel. Also, the downstream end of the spillway discharge channel is cracking and collapsing into the plunge pool. Some erosion of the downstream channel was observed at the plunge pool likely due to the collapsing discharge channel. The subject damage appears to have been present since the Army Corp inspection that was for performed in 1979.

A wrought iron fence has been placed along the spillway crest. Debris from high flows is likely to become hung up on this fence. This will significantly reduce spillway discharge capacity and will also increase stress on the structure. The fence should be removed, and alternative safety measures considered.

7.2 Stability Assessment

7.2.1 Masonry Dam

The stability analysis for the masonry section of the dam of New Rochelle Reservoir No. 1 indicates that, for the existing scenario, the factors of safety are less than those recommended by the New York State Department of Environmental Conservation (DEC 1989). The stability analysis for the proposed lowering of water levels within the reservoir are expected to significantly improve the structural stability of the dam by reducing the horizontal hydrostatic pressure acting on the upstream dam face during both normal and flood conditions. However, when assuming cracking can take place at the base of the dam, the minimum safety requirements recommended by the

New York State Department of Environmental Conservation (DEC 1989) are not still met for Case No. 3 loading conditions.

With regards to the behavior of the structure under tension, if the structure was constructed entirely from masonry then the tensile strength of the structure is likely to be exceeded in most cases with results tending towards the lower end of the estimates, however were the core to be constructed from concrete then results are likely to tend towards the upper estimates presented here under the conditions presented in Section 6 of this report. Additional investigations such as those to determine the composition of the dam core or dam foundation level would allow a better assessment of the structural stability of the masonry dam. Under existing conditions, it is anticipated that structural modification to the dam will likely be required to achieve acceptable factors of safety.

It is possible that further investigation may reveal that under proposed conditions the masonry portion of the dam may be stable. However, in light of the results of the stability assessment as described in this report, a number of rehabilitation methods were considered that could be implemented to improve the overall structural stability of the masonry dam under the proposed conditions assuming cracking is allowed. Following preliminary discussions, the following alternative options were considered:

1. Remove silt from upstream face;
2. Provide key / extend structure on upstream face;
3. Anchor bolts from the crest;
4. Place selected backfill on downstream face;
5. Construct earthen berm / placement of concrete above dam downstream face and provide anchor bolts from the berm;
6. Provide drainage to the dam foundation to relieve uplift pressures.

Each option was evaluated from a constructability point of view, by inspection or by undertaking modelling in order to determine its feasibility.

Option 1

The first option considered the removal of sediment from the upstream face of the dam allowing a reduction of the horizontal static thrust of the submerged silt deposited along the upstream face.

Results of the stability analysis for the design loading condition (Case 3) are presented in Table .

Table 39: Dam stability analysis results for the proposed scenario with Option 1 (cracking allowed)

Loading	Sliding safety factor		Position of resultant force		Comments
case	calculated	Required	Calculated	Required	
Case 3	0.871	1.250	97.82% of base from U/S	In middle half of base	Calculated sliding safety factor is unsatisfactory. Resultant falls outside of the base of the dam. Tensile stresses are present at the heel of the dam leading to cracking through the base.

Source: Mott MacDonald

Although desilting is feasible from a practical point of view, this option alone cannot ensure that the masonry dam can meet the stability criteria from DEC (1989).

Option 2

The second considered alternative consisted in placing material on the upstream side of the dam to increase the self-weigh of the dam. This option was discounted because it is not practical from a constructability point of view requiring emptying of the reservoir as well as significant construction works.

Option 3

The third option consisted of drilling anchor bolts from the crest. Vertically installed post-tensioned anchors can add normal force, increasing the sliding frictional resistance and preventing the development of tension at the heel of the dam.

The minimum post-tension anchor force that must be applied from the center of the crest to ensure that the masonry dam can meet the stability criteria from DEC (1989) was found to be 10.5kipf/ft. Results of the stability analysis for the design loading condition (Case 3) are presented in Table .

Table 40: Dam stability analysis results for the proposed scenario with Option 3 (anchor force 10.5kipf/ft, cracking allowed)

Loading case	Sliding safety factor		Position of resultant force		Comments
	Calculated	Required	Calculated	Required	
Case 3	1.544	1.250	74.77% of base from U/S	In middle half of base	Calculated sliding safety factor is in excess of the minimum required value. Resultant location is acceptable, although tension at the dam heel still results in cracking at the base of the dam.

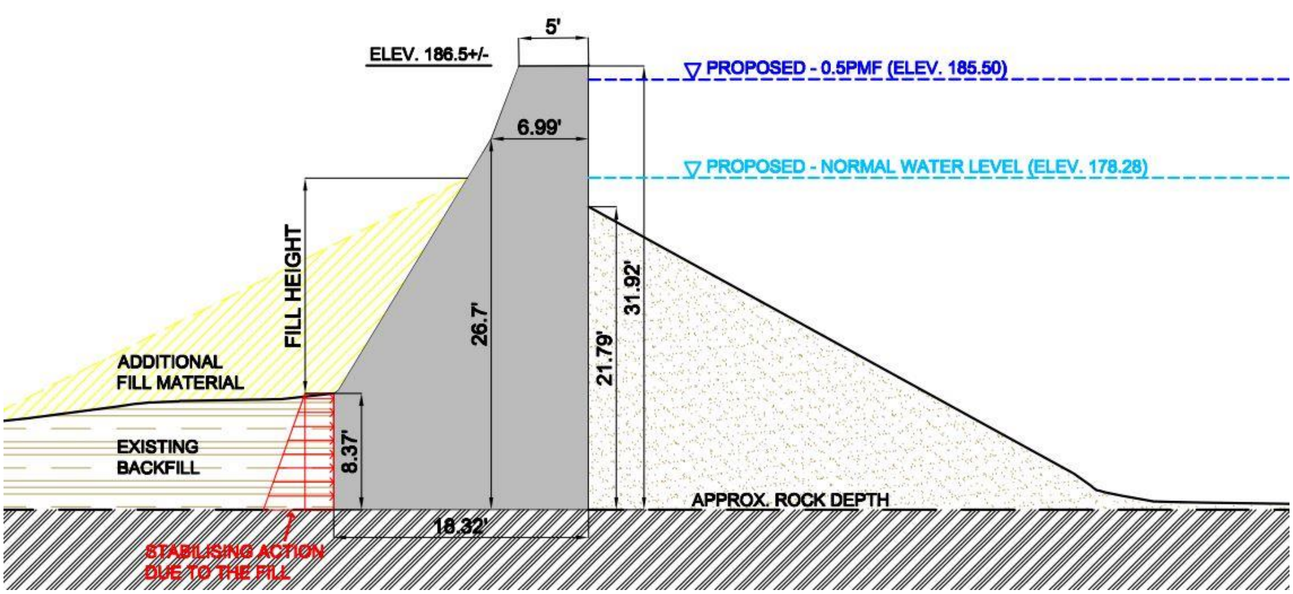
Source: Mott MacDonald

Access to the crest of the dam is extremely difficult. The preferred approach to implementation this option would be to have the reservoir lowered. Consideration of mobilizing equipment on barges could be further evaluated. Implementation of this approach may require special equipment which will greatly increase the overall cost.

Option 4

The fourth option considered the placement of earth material on the downstream face of the dam in order to add mass to the downstream portion of the structure to resist dam movement (Figure).

Figure 17: Dam masonry section with additional earth material on the downstream face (Option 4)



Source: Mott MacDonald

This option was evaluated considering the following assumptions:

- Additional earth material placed on a 2:1 slope;
- Additional earth material had the density of on-site soils;
- Height of the additional earth material was varied from ground level up to crest level (Elev. 186.5ft).

In analyzing the potential resistance of the soil wedge of the additional earth material, three mechanisms were identified that could provide increased resistance to the movement of the dam:

1. Action of the additional surcharge on the downstream dam face providing additional resistance to overturning and sliding;
2. Action of additional surcharge on the earth material located at the toe of the dam increasing passive resistance of the material located at the toe of the dam;
3. Passive resistance of the wedge itself.

Of the three mechanism above, only the resistance from Item 2 was considered in the analysis. Item 3 was excluded because the movement required to realize the passive resistance would have been too great and, furthermore, due to the sloping nature of the backfill, the overall resistant wedge would have been unlikely to provide for significant resistance. Item 1 was excluded because it was unlikely to provide significant improvements over and above Item 3.

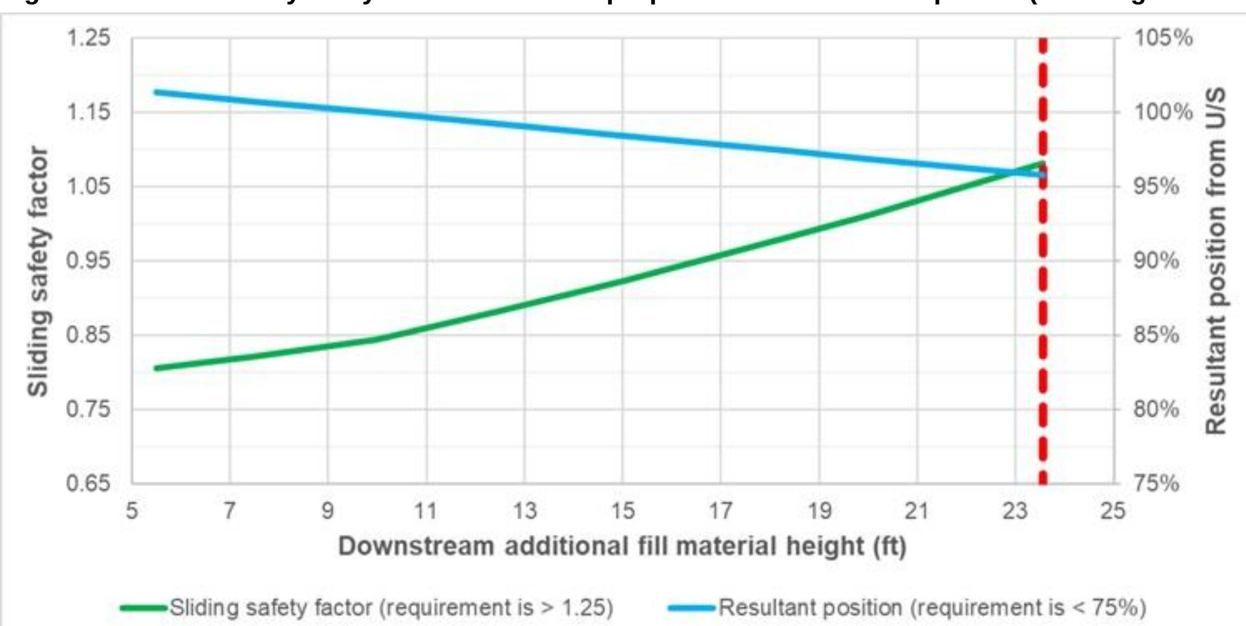
The assumed stress block resulting at the toe of the dam as a result of the action of the additional overburden is shown in Figure .

Figure shows the value of the sliding safety factor (SSF) and the position of the resultant varying the height of the additional earth material.

The graph suggests that insufficient resistance would be generated to meet the safety requirements (SSF > 1.25 and resultant in the middle half of the base).

Furthermore, the factors of safety were such that the resistance increase resulting from the self-weight of the soil wedge acting on the downstream face of the dam (Item 1) was unlikely to significantly alter the results.

Figure 18: Dam stability analysis results for the proposed scenario with Option 4 (cracking allowed)

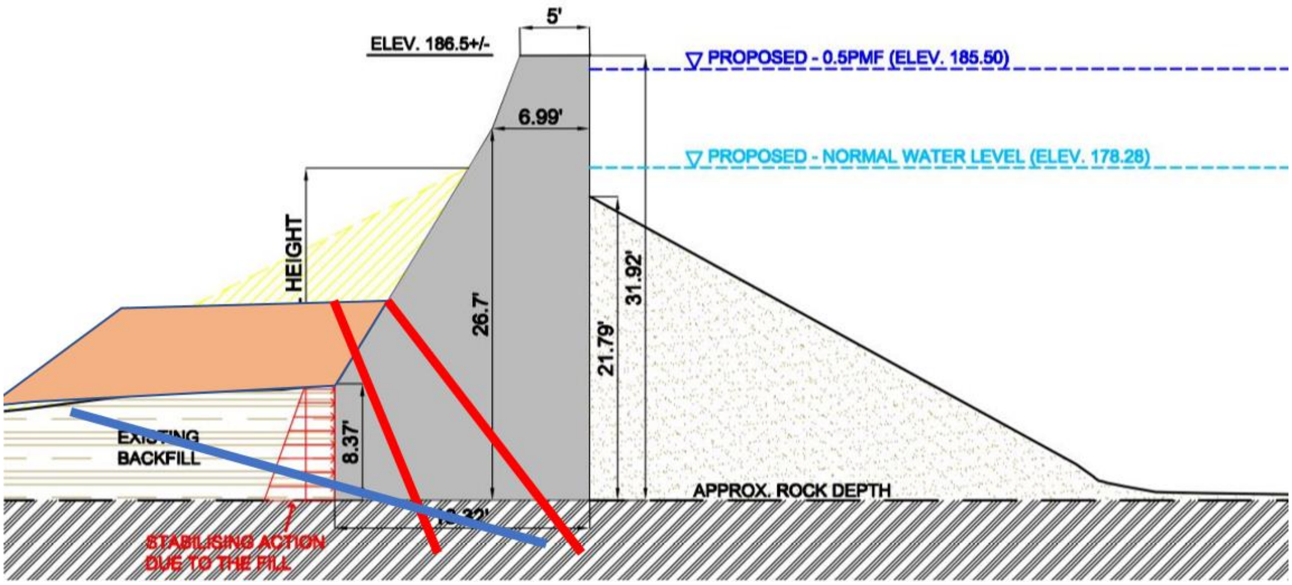


Source: Mott MacDonald

Option 5

The fifth option considered the construction of an earthen berm or concrete slab above the dam downstream face with anchor bolts then driven from this platform into the dam foundation (Figure).

Figure 19: Dam masonry section with earthen berm / concrete on the downstream face (Option 4)



Source: Mott MacDonald

The berm can add mass to the downstream portion of the structure to resist sliding, while the anchors installed at an angle can provide additional sliding resistance by directly offsetting applied horizontal forces.

Although at this stage modelling was not required, it is believed that this solution, if properly designed, would generate sufficient resistance to meet the safety requirements from DEC (1989), although additional investigations are required to evaluate the practicality of the proposed design option.

Option 6

The masonry dam of New Rochelle Reservoir No. 1 is believed to not include any foundation drainage systems. As such the sixth option considered providing drainage to the dam foundation in order to reduce uplift pressures.

The preliminary assessment of this option was carried out considering the following assumptions according to USBR (1976, 1987[10]):

- Drain position located underneath the crest;
- Drain effectiveness was 2/3;
- No drain effectiveness under any cracking condition;
- Uplift pressures considered as an external load acting on the surface of the joint.

Results of the stability analysis for the design loading condition (Case 3) are presented in Table 7.1.

[10] USBR 1987. Design of Small Dams.

Table 7.1: Dam stability analysis results for the proposed scenario considering Option 6 (cracking allowed)

Loading	Sliding safety factor		Position of resultant force		Comments
Case	Calculated	Required	Calculated	Required	
Case 3	0.758	1.250	102.63% of base from U/S	In middle half of base	Calculated sliding safety factor is unsatisfactory. Resultant falls outside of the base of the dam. Tensile stresses are present at the heel of the dam leading to cracking through the base disrupting drain functionality.

Source: Mott MacDonald

Although the installation of a foundation drainage systems appears to be feasible from a practical point of view, this option alone cannot ensure that the masonry dam can meet the stability criteria from DEC (1989). This is the case as cracking still occurs through the whole base of the dam resulting in the disruption of the foundation condition, which could compromise the effectiveness of the drain system. However, although not effective in its own right, this option may be considered along with the interventions as discussed in Option 5 to lead to a more effective overall solution.

Conclusion

Advantages and disadvantages of each considered potential rehabilitation method are summarized in

Table 7.2. From a constructability point of view, this preliminary analysis suggests that the most promising solution appears to be Option 5, eventually coupled with reservoir desilting (Option 1) and drain installation (Option 6). Option 3 would achieve an acceptable factor of safety; however, specialized equipment would be needed.

Table 7.2: Preliminary summary of potential rehabilitation methods of the masonry dam

Option	Description	Advantages	Disadvantages
1	Remove silt from upstream face	This option is feasible from a constructability point of view.	This option alone cannot ensure that the dam can meet the stability criteria.
2	Provide key / extend structure on upstream face	Likely to provide the required improvement in stability	Significant constructability challenges and need to both lower reservoir water levels and undertake desilting.
3	Anchor bolts from the crest	This option can ensure that the dam meets stability criteria.	High construction cost to implement due to accessibility.
4	Place selected backfill on downstream face	This option is feasible from a constructability point of view.	This option alone cannot ensure that the dam can meet the stability criteria.
5	Construct earthen berm / placement of concrete above dam downstream face and provide anchor bolts from the berm	This option may be feasible from a constructability point of view and may ensure that the dam meets stability criteria.	Additional exploratory investigations are required to assess the practical feasibility of this option and to evaluate the stability of the dam.
6	Provide drainage to the dam foundation to relieve uplift pressures	This option is feasible from a constructability point of view.	This option alone cannot ensure that the dam can meet the stability criteria.

Source: Mott MacDonald

In order to evaluate and model the feasibility of each option, it is recommended that additional exploratory investigations be undertaken to evaluate the masonry/rock interface and the internal dam construction. This work may require dismantling a portion of the dam to confirm existing conditions. **Final stability calculations will be required for the final design of the modifications to spillway and further investigation of the composition of the dam.**

7.2.2 Earthen Dikes

Based on assumed parameters, the earthen dikes are believed to have an adequate factor of safety for Case No. 1 through 4 as outlined by the New York State Department of Environmental Conservation (DEC 1989). However, the geotechnical investigation should be expanded to include soil borings within the earthen dikes to validate the assumptions made. The factor of safety of the dikes during rapid drawdowns was calculated to be low. Consideration of reducing the upstream slopes on both the right and left upstream embankments should be evaluated in the future. The earthen dikes should be monitored when the reservoir is lowered to identify any observed instability.

7.3 Spillway Capacity

As part of the hydrologic and hydraulic analysis, it was concluded that the existing dam does not have the spillway capacity to safely convey the SDF (50% of the PMF storm event). Multiple alternatives were discussed in Section 4 of this report that would bring the dam into compliance with the requirements of the DEC dam safety. The alternatives consisted of the following:

1. Alternative 1 – Spillway Modifications – Under this alternative, the dam would need to be modified to increase the spillway capacity. A scenario to modify the dam and increase the spillway capacity was analyzed which included lowering the primary spillway by approximately 4 feet and converting the left masonry dam into a secondary or auxiliary spillway. The results demonstrated that the modifications would the necessary spillway capacity and have minimal to no impacts downstream of the dam. However, this alternative would have a significant impact to the impoundment area and the upstream communities by lowering the normal pool elevation by more than 4 feet. The project cost for this alternative is in the range of $4 Million dollars. This estimate assumes no structural improvements are required on the remaining portion of the masonry dam.
2. Alternative 2 – Overtopping Protection - Under this alternative, the dam would be overtopped during the SDF maintaining its existing hydraulic condition. However, the dam would require modifications to the downstream slopes to ensure the integrity and stability of the dam during an overtopping event. This would typically require providing structural measures to stabilize the dam and provide armoring or overtopping protection of the downstream slopes. Overtopping protection can include a variety of materials or measures such as concrete, gabion basket, or articulated concrete blocks and will depend on the design requirements. This alternative would maintain the existing conditions both downstream with regards to discharges from the dam and upstream with regards to maintaining the existing normal pool elevation. Improvements would also be required to raise portions of the earthen dikes to provide adequate freeboard over the entire length. The project cost for this alternative is in the range of $8 Million dollars.
3. Alternative 3 – Dam Removal – For this alternative, the dam would be permanently breached which would eliminate the dam and the reservoir. A detailed hydraulic evaluation of this alternative was not conducted at this time. However, our analysis of other alternatives shows that the reservoir attenuates downstream stream flows during significant rainfall events. Removing the dam will have potential downstream impacts. As such, it is anticipated that a structure will be needed within the proposed stream bed to control stream flow to order to maintain existing flow conditions. Removing that dam would also require establishing a stabilized channel within the reservoir. The estimated cost for this alternative is currently projected to be in the range of $3 Million dollars.

7.4 Low-Level Outlet

As part of the DEC's requirements for dam safety, a low-level outlet is required for dams for emptying or lowering the water in case of emergency; for inspection and maintenance of the dam, reservoir, and appurtenances; and for releasing waters to meet downstream water requirements. The outlet conduit may be an independent pipe, or it may be connected to the service spillway conduit and required to have sufficient capacity to discharge 90% of the storage below the lowest spillway crest within 14 days, assuming no inflow into the reservoir.

Since the gate house and pipes that pass through the New Rochelle Reservoir No. 1 Dam are reportedly sealed and inoperable, the structure currently has no low-level outlet capable of lowering the dam for emergencies or inspections. As part of the rehabilitation efforts to bring the dam into compliance, further investigation is necessary to verify the conditions of the pipes. The detailed investigation should verify the conditions of the pipes and valves and if they can be repaired or restored. If the existing pipe and valves are determined to be permanently inoperable, it will be necessary to ensure that they are properly sealed, and measures will be necessary to provide a design for a new low-level outlet as part of the modifications to rehabilitate the dam. The newly designed low-level outlet shall be capable of dewatering the reservoir per the DEC's dam safety requirements.

8 Closure

This Engineering Assessment of the New Rochelle Reservoir No. 1 Dam has been based upon the results of a Safety Inspection, dam stability analyses, and hydrologic and hydraulic analysis of the pond and its watershed. The inspections and studies have identified deficiencies which prevent it from being in compliance with DEC regulations. In addition, we concur that the dam possesses a rating of "**Unsound, Deficiencies Recognized**".

Very truly yours,

Mott MacDonald

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9 Related References

Dams shall be compared with safety criteria contained in:
“Guidelines for Design of Dams,” NYS DEC, 1989.
http://www.dec.ny.gov/lands/4991.html

New York State Laws:
http://public.leginfo.state.ny.us/menugetf.cgi?COMMONQUERY=LAWS

Environmental Conservation Law 15-0507:
Structures impounding waters; structures in waters; responsibility of owner; inspection

Environmental Conservation Law 15-0503:
Protection of water bodies; permit

Executive Law Article 2-B:
State and Local Natural and Man-Made Disaster Preparedness

New York State Regulations:
http://www.dec.ny.gov/regulations/regulations.html

Title 6 of the Official Compilation of Codes, Rules and Regulations of the State of New York (6 NYCRR):
Part 608 – Protection of Waters
Part 621 – Uniform Procedures (includes Emergency Authorization)
Part 673 – Dam Safety

Other Guidance Documents:

Guidance on Inspection and Maintenance of Dams is contained in the publication “Owners Guidance for the Inspection and Maintenance of Dams in New York” NYS DEC, June 1987, http://www.dec.ny.gov/lands/4991.html.

Guidance on Emergency Action Plans is contained in the publication “DOW 3.1.3 – Emergency Action Plans for Dams,” NYS DEC, June 2010, http://www.dec.ny.gov/lands/4991.html.

New York State Environmental Conservation Law "Article 15-0503".

New York Code of Rules and Regulations (6NYCRR) "Part 621 - Uniform Procedures".

New York Code of Rules and Regulations (6NYCRR) "Part 673 - Dam Safety Regulations"
New York Code of Rules and Regulations (6NYCRR) "Part 500 - Flood Plain Development Permits"

New York Code of Rules and Regulations (6NYCRR) "Part 373 - Hazardous Waste Management"

An Owners Guidance Manual for the Inspection and Maintenance of Dams in New York State.

New York State Education Law "Article 55".

Soil Conservation Service; U. S. Department of Agriculture SCS National
Engineering Handbook; August 1972 "Section 4 -Hydrology
Corps of Engineers; U. S. Army

Hydrologic Engineering Center

"HEC-1 Flood Hydrograph Package"; 1981

ETL 1110-2-256; June 1981
"Sliding Stability for Concrete Structures".

EM 1110-2-1902; April 1970
"Stability of Earth and Rock-Fill Dams"
Bureau of Reclamation; U. S. Department of the Interior

"Design of Small Dams", 1977 Revised Reprint

"Design of Gravity Dams", 1976

National Oceanic and Atmospheric Administration National Weather
Service; U.S. Department of Commerce

Hydrometeorological Report 33; April 1956 "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from
10 to 1000 Square Miles and Durations of 6, 12, 24 and 48 Hours"

Hydrometeorological Report 51; June 1978 "Probable Maximum Precipitation
Estimates, United States East of the 105th Meridian"

Technical Paper 40; May 1961 "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from
1 to 100 Years"

Technical Paper 49; 1964 "Two-to-Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States"

Figures

Appendix A – Dam Safety Inspection

Appendix B – Hydrologic and Hydraulic Data

Appendix C – Subsurface Investigation Data

Appendix D - Masonry Dam Stability Calculation Sheets

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