A Case study and Lessons Learned Support of a Retaining Wall-Maid of the Mist Winter Storage D/B Project

Ground Improvements
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Agenda

- Project Overview
- Site History
- Present Site Conditions
- Schoellkopf Plant Wall Evaluation
- Design & Construction Considerations
- Proposed Construction
- Other Project Elements

Project Overview

Project Overview

- MOTM Canadian contract expired end of 2013
- Winter Ice Conditions and Whirlpool Rapids require removal of boats for winter
- No means to remove the boats without dismantling
- New site on US side must be established to at least provide for boat removal by October 2013

Site History

Site History

- **Schoellkopf Plants 3A, 3B, & 3C in place**

Site History

- **Schoellkopf Plants 3A, 3B, & 3C before collapse**

Site History

- **Schoellkopf after collapse before water stopped**

SCHOELLKOPF POWER STATION DISASTER
Thursday June 7th 1956
The Most Destructive Rock Fall In History

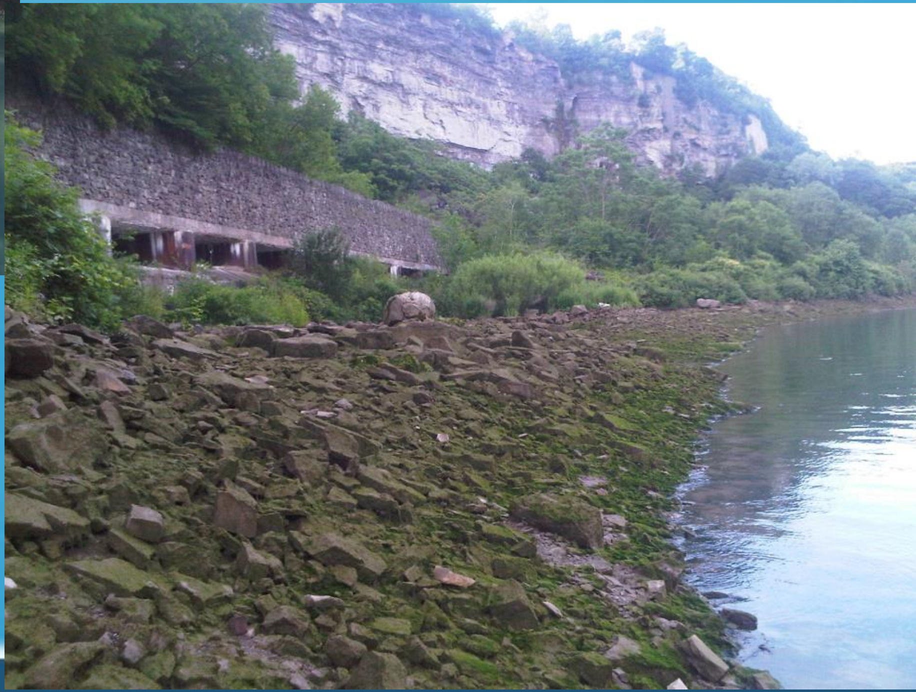
Blame Erosion And Seepage For Mud Slide

A team of geologists examined the scene after the collapse and virtually certain that erosion and water seepage combined to cause the devastating rock slide that crushed 2/3 of the Schoellkopf Power Station. They estimated that 120,000 tons of rock or an estimated 1,000,000 cubic feet of rock plunged into the gorge crushing the power station. A section of rock measuring 400 feet long - 200 feet high and 20 feet thick had broken loose from the top of the cliff.

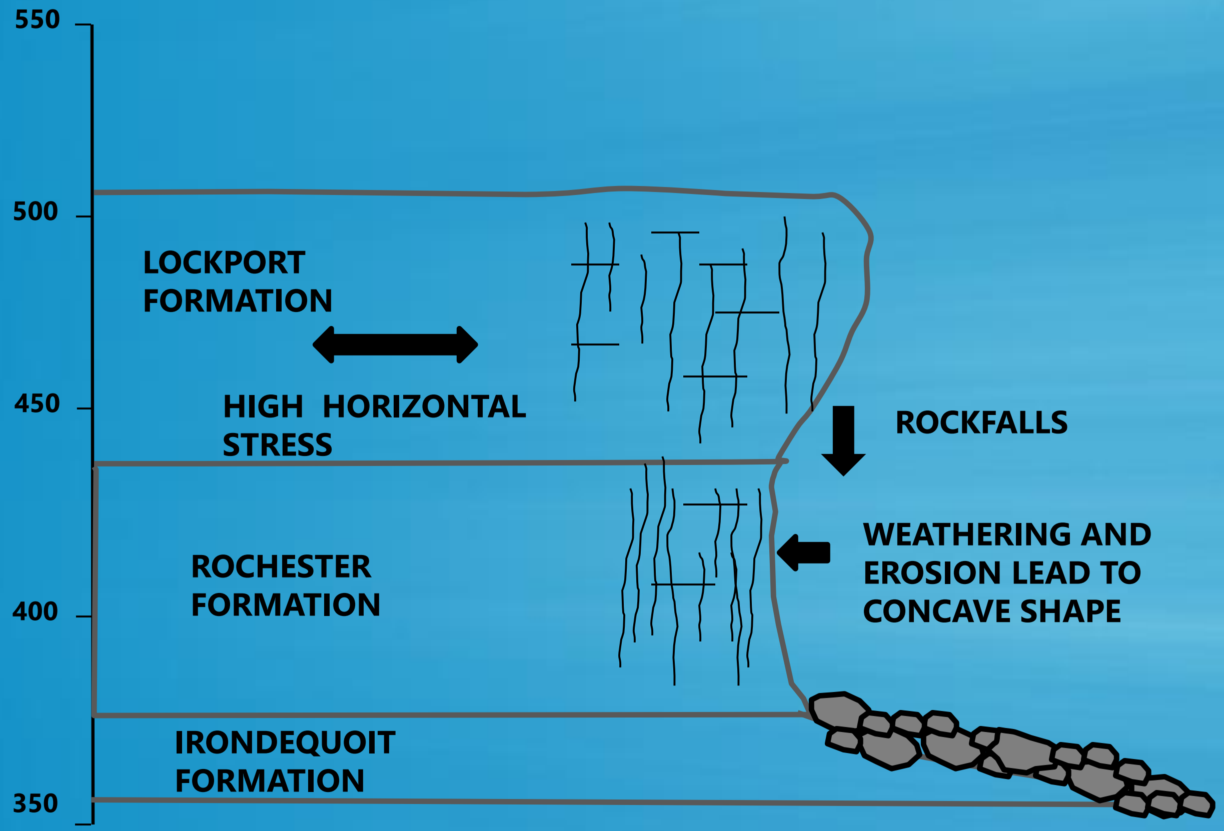
https://www.youtube.com/watch?v=ftUQlvYCpkc

Present Site Conditions

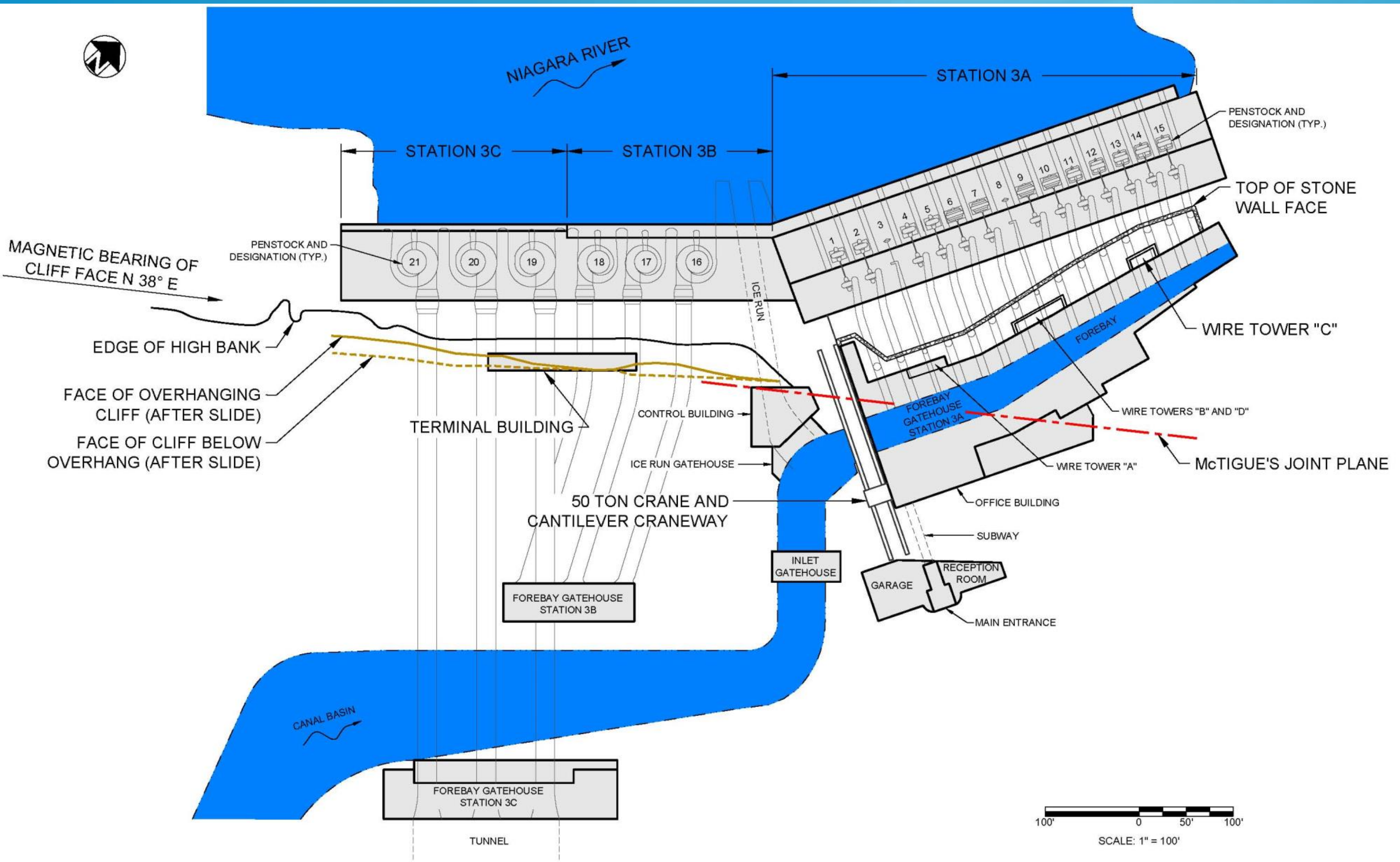
Present Site Conditions



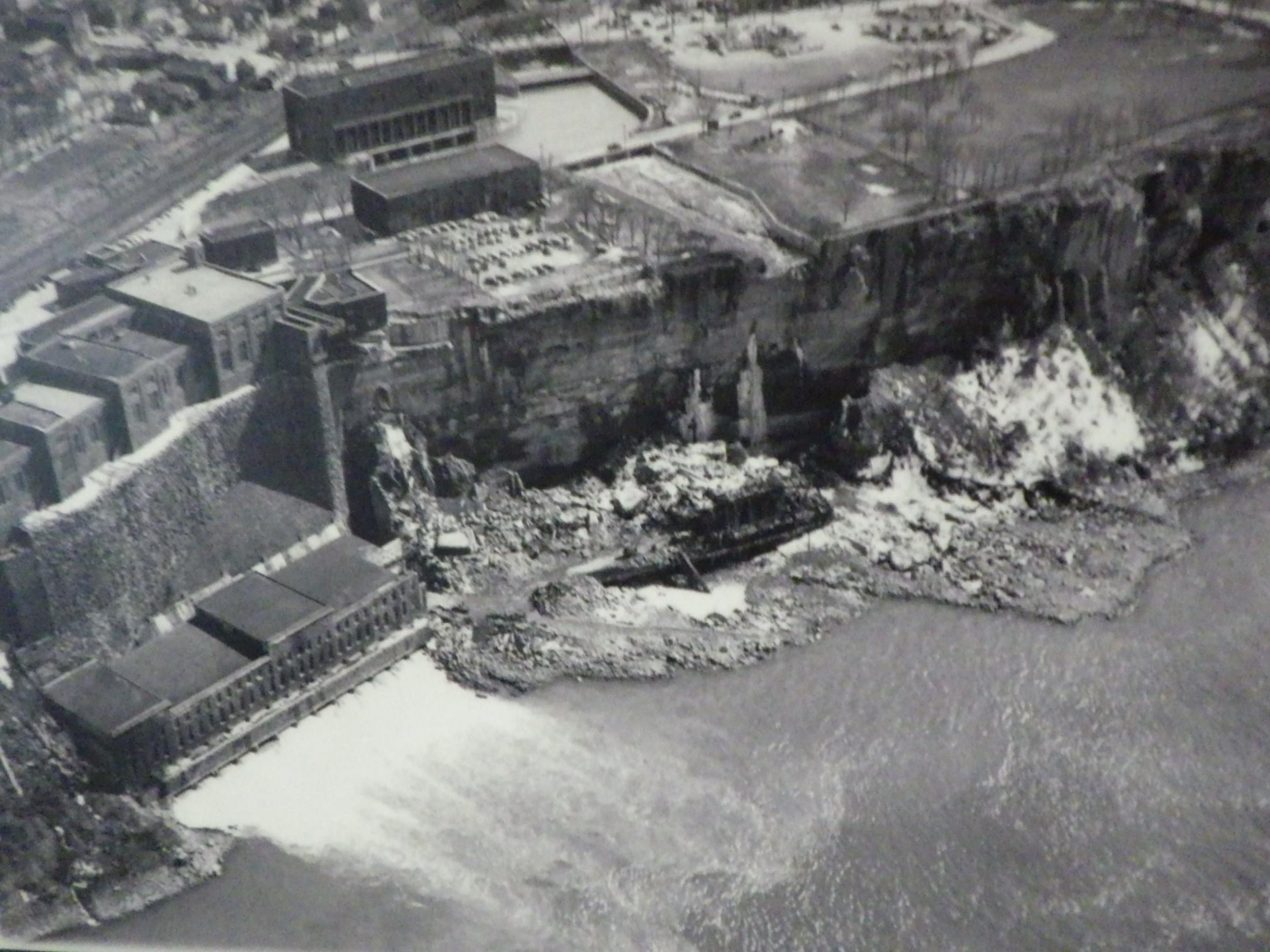
Evaluation of Stability of the Schoellkopf Power Generation Plant



Reference: "Preservation and Enhancement of the American Falls at Niagara," US Army Corps of Engineers, June 1974 PLT C-24.



NIAGARA RIVER
STATION 3A
STATION 3C
STATION 3B
PENSTOCK AND DESIGNATION (TYP.)
TOP OF STONE WALL FACE
MAGNETIC BEARING OF CLIFF FACE N 38° E
PENSTOCK AND DESIGNATION (TYP.)
21
20
19
18
17
16
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
ICE RUN
EDGE OF HIGH BANK
WIRE TOWER "C"
FOREBAY
FACE OF OVERHANGING CLIFF (AFTER SLIDE)
FACE OF CLIFF BELOW OVERHANG (AFTER SLIDE)
TERMINAL BUILDING
CONTROL BUILDING
FOREBAY GATEHOUSE STATION 3A
WIRE TOWERS "B" AND "D"
McTIGUE'S JOINT PLANE
ICE RUN GATEHOUSE
WIRE TOWER "A"
50 TON CRANE AND CANTILEVER CRANEWAY
OFFICE BUILDING
SUBWAY
INLET GATEHOUSE
GARAGE
RECEPTION ROOM
MAIN ENTRANCE
FOREBAY GATEHOUSE STATION 3B
CANAL BASIN
FOREBAY GATEHOUSE STATION 3C
TUNNEL
100'
0
50'
100'
SCALE: 1" = 100'




Rock Anchor

Design and Construction Considerations

Ice

- Ice accumulates in the Gorge as a result of ice flowing over the Falls
- The ice bonds together, piles up, and jams

Ice

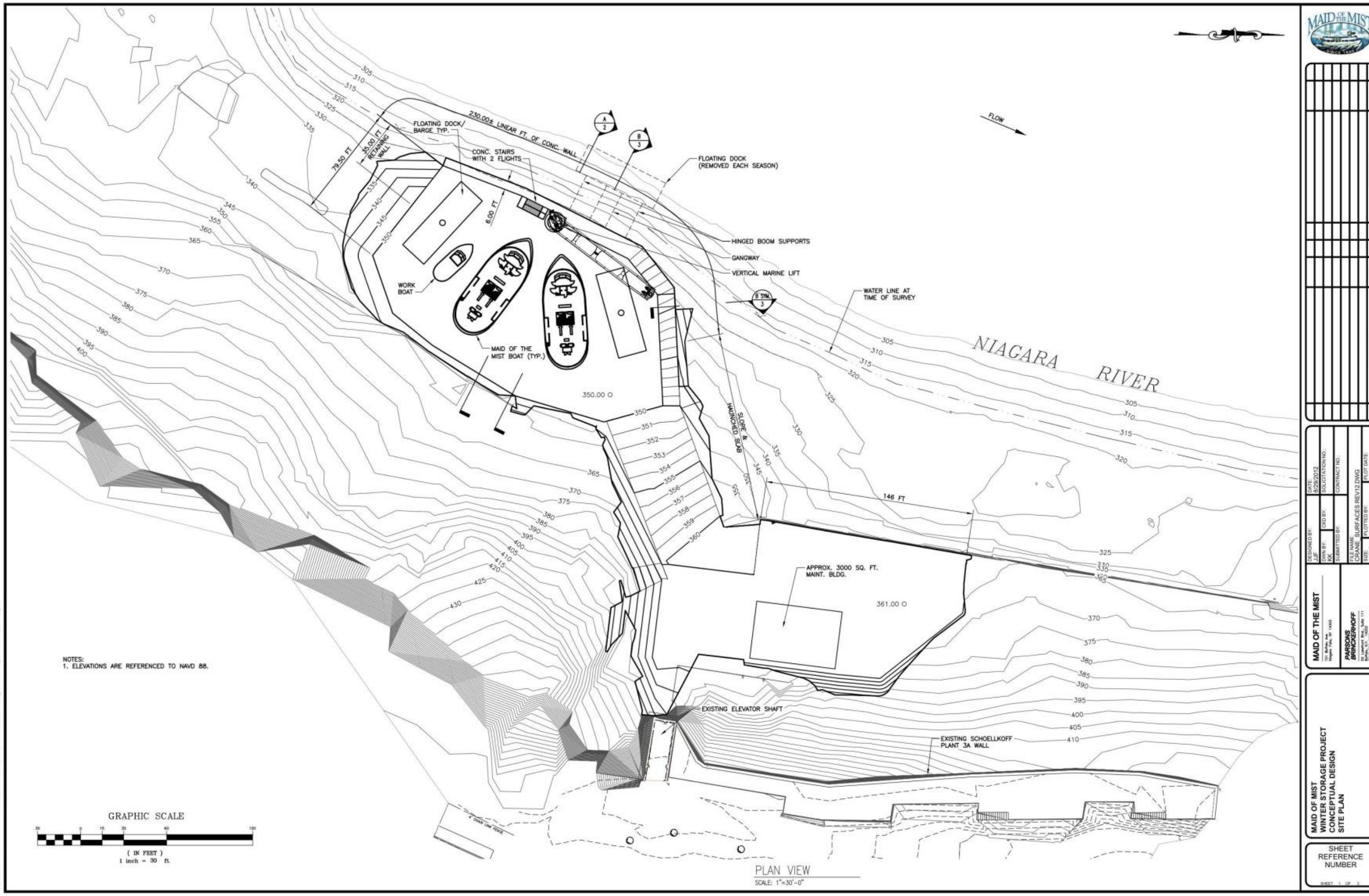
- Water is forced to flow under and through the ice
- At times the ice creates a temporary dam and causes the river level to rise

Ice

Debris

New Storage Facility Design & Construction

Vertical Marine Lift



Vertical Marine Lift - Winter

- **Advantages**
  - Limited or no work in water
  - Smaller footprint and limit of disturbance
  - No infrastructure in the water where it could be impacted by ice
  - Less susceptibility to ice accumulation and having to remove ice in the spring
  - Can allow boat removal during low water conditions
  - Rock removal and deep excavation avoided

Wall Innovative Design

In early 2012, Maid of the Mist Corporation lost a lease to store the tour boats on the Canadian bank of the river and hence had to arrange with New York State to build a new facility to accommodate the ferry boats at the site of the former Schoellkopf Power Station in the Niagara River Gorge on the U.S. bank of the river, which is about 488 meters (1,600 feet) north of the Rainbow Bridge. The new facility had to be built on the U.S. side of the river and be operational before ice began to accumulate in late 2013 – necessitating an October operation.

The project was set up similar in a design-build delivery Method. While the design started in late 2012, the later packages met with unknown conditions that were encountered due to limited prior access (in a gorge) and some significant design modifications had to be made during construction, which began in May 2013.

The project experienced the following significant design and construction challenges during the project's duration;

- The Maid's lease in Canada expired at the end of 2013 so the site had to be ready to remove the boats from the water by then and before the ice established itself in the river since there was no feasible way to remove the boats in whole from the gorge as there are downstream rapids in the river, and there is extremely significant ice accumulation below Niagara Falls.
- The winter storage facility is located on the Schoellkopf site, which is on the Federal Register of Historic Places, and is home to endangered plant species.
- The site is in a 200 feet deep gorge that has no vehicular access so all equipment and materials had to be dropped in or brought down on a temporary elevator.
- Only limited geotechnical information could be obtained before construction began and equipment mobilized into the gorge.

- During construction, 150 to 200 bags of cement had to be pumped to construct a test micropile in talus deposits south of the project site, and it took one to two days to complete the 60 to 80 feet deep test micropiles.
- To mitigate grout loss, low mobility grouting (LMG) or pregrouting/redrilling of the hole was implemented, but with little success. Given the expensive and lengthy process of the LMG or pregrouting/redrilling, foundation design of the wall had to be modified to suit the in situ field subsurface conditions.
- Despite the significant operational hurdles, a fast-track design was delivered for an unusually complex project site that presented numerous logistical, geotechnical, historical and environmental challenges.

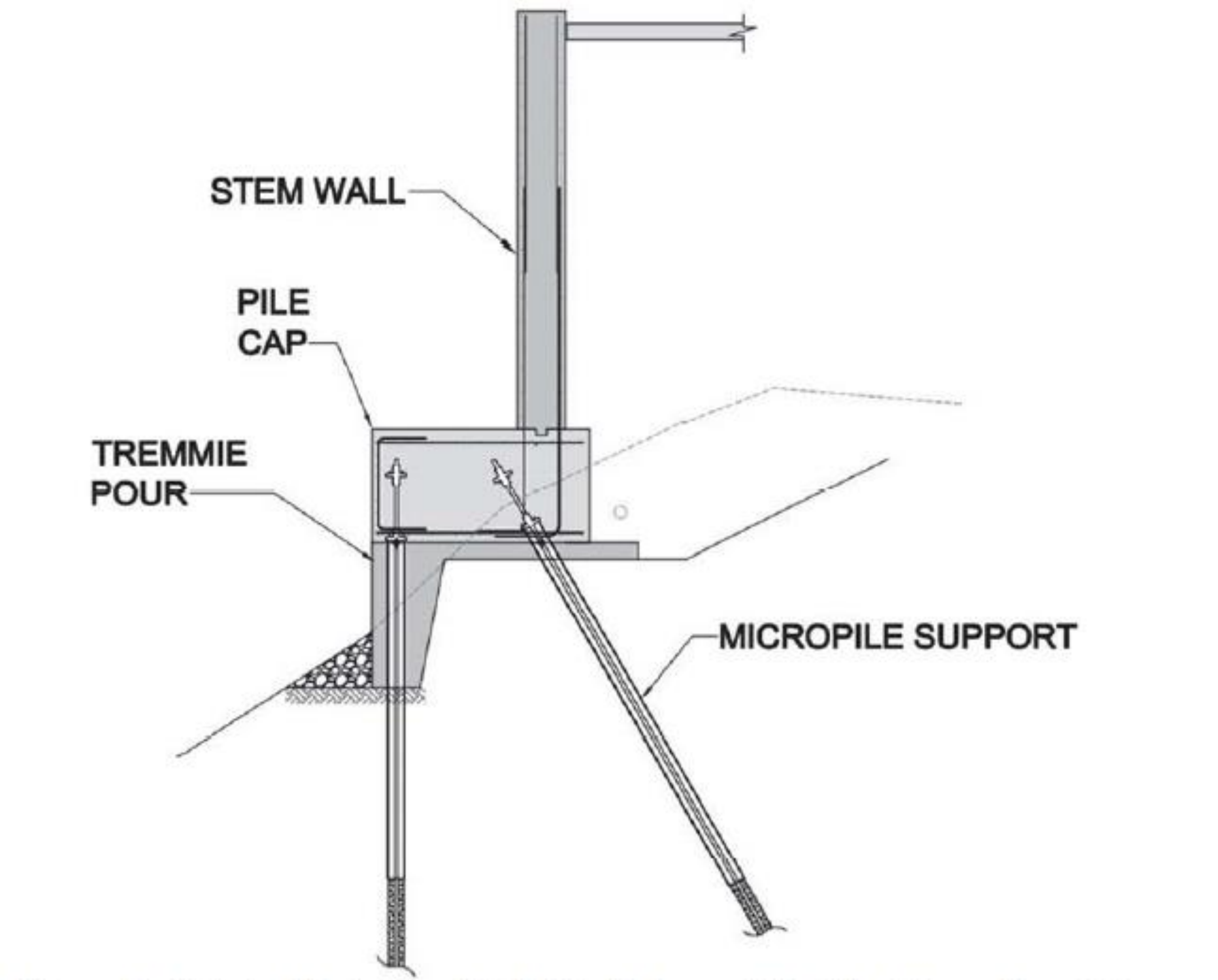


Figure 2: Original Retaining Wall Details Layout Used for Crane Foundation

Construction Problems

Due to the construction problems related to substantial grout loss during installations of test micropiles in the talus deposits (composed of rock fragments of different size and shape) in the south side of the project site, and the concerns over the lengthy construction delays required for installing production micropiles within the talus deposits, the designer was requested by the contractor to re-evaluate the design of the originally proposed micropile-supported retaining wall with an objective to either reducing the number of or removing all of the micropiles for support of the retaining walls.

(Alternative Design)

1. The retaining wall heel slab extends to about 27 feet behind the wall stem, while the toe of the wall extends to about 25 feet in front of the wall stem;
2. The wall footing thickness is generally three feet, but is increased to 4.9 feet at the toe; and
3. The backfill of the on-site soils above the heel is reinforced with layers of Tensar geogrid fabrics. The purpose of Bi-Axial BX 1200 Geogrid was to:

a. Improve the long term performance of the retained fill,
b. Mitigate wall and ground deformations, and
c. Uniformly spread out fill load for more evenly distributed settlements.

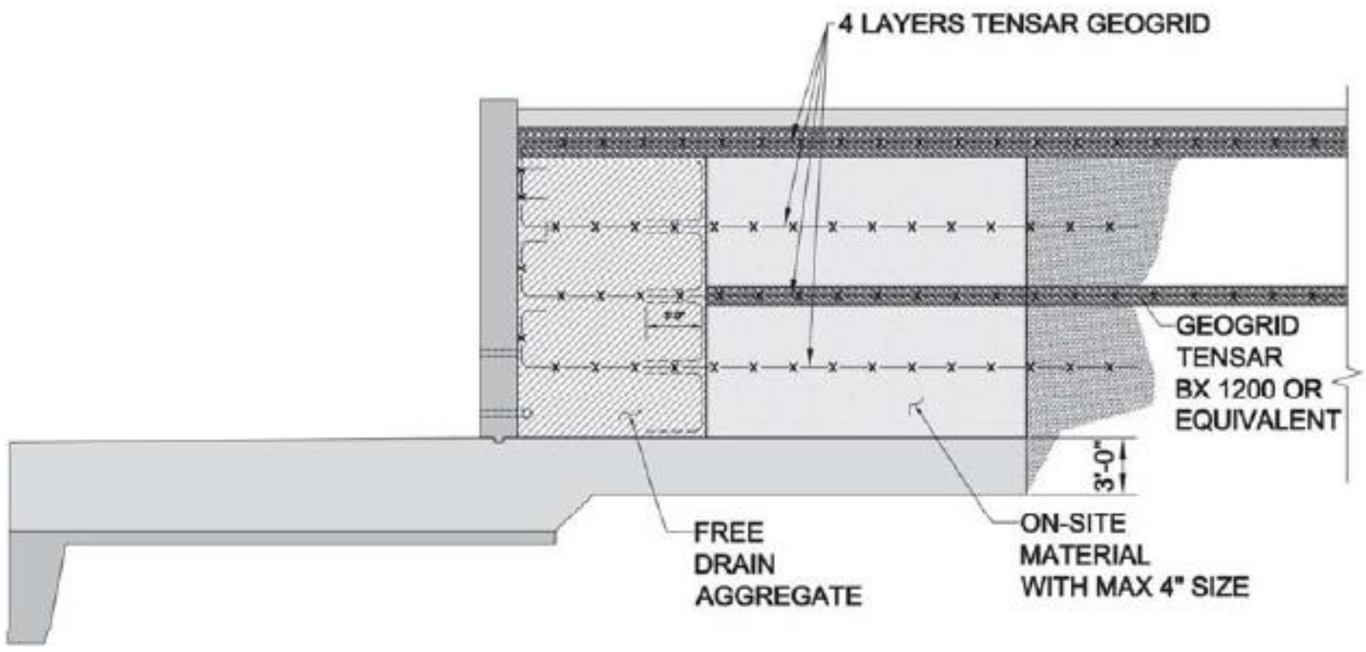


Figure 3: Proposed Alternative Retaining Wall Layout Details

STABILITY ANALYSES AND SETTLEMENT CALCULATIONS

A detailed global stability analysis was performed and evaluated. Slope stability analyses were conducted using GSTABL Version 7.0 computer software developed by Gregory Geotechnical. Analyses were also performed to assess the immediate settlement of the Talus material due to the loads induced by the 18-foot fill. The settlement was evaluated using Settle3D software developed by Rocscience Inc. Based on our analysis, the immediate settlement was estimated to be in the order of approximately one-half to one inch. External stability analyses against sliding and overturning along with bearing capacity and uplift analyses for the proposed wall configurations were performed.

Design Challenges included;

1. Variable rock elevations;
2. Daily water level elevations changes at the adjacent Niagara River ranging from elevations of 314 to 344 feet, with a typical daily range of change from 10 to 12 feet; and
3. A rock layer was encountered at the north side of the proposed crane at El.280 but at the south side of the crane location, no rock layers were encountered during drilling, which progressed to a depth of 95 feet.

Table 1: Summary of Factor of Safety Values for Global Stability

Water Level Elevation (ft)	FS
314 (NO ROCK) Unusual Condition with 1,000 psf ice loading	1.50
314 (NO ROCK) with 100 psf ice loading	1.51
344 (NO ROCK) with 100 psf ice loading	1.60
Rapid Drawdown (344 to 314) (NO ROCK)	1.50

1. 30 feet long Tensar geogrid Type BX 1200 of four layers.
2. Heel extended to about 27 feet behind the CIP wall. Toe is about 25 feet in front of the wall.
3. Internal friction angle of Talus of 38 degree.
4. Surcharge of 250 psf;

The results of the external stability analyses met the minimum FS for Bearing Capacity (2.0), Sliding (1.5), Overturning (2.0) and Uplift (1.5).

TALUS DEPOSITS AND PROPERTIES

The project is located at the Talus formation. Based on many literatures and previous experience with similar site conditions coupled with field observations of the talus slope for other projects, a friction angle (ø') of 38 degrees was adopted for the Talus to be considered for wall design. Justifications for selecting ø' = 38 degrees are as follows:

1. The subsurface conditions underlying the project site are composed of fill and talus underlain by bedrock consisting of the interlayering of angular sandstone, limestone and shale with soils.
2. Groundwater varies from 314 to 344 foot elevations.
3. The existing slope is composed mainly of fill and talus with a surface inclination varying from 25 to 40 degrees. This slope has been subject to fluctuations of river water and ice loading and has performed very well without any sign of distress.
4. Based on the field observation and historical data of the project site, the talus is composed of predominantly large, competent sandstone/limestone rock fragments and broken smaller shale rock which has degraded into finer soils to fill in the open-graded talus deposits. In addition, finer soils are added into the talus matrix by water transport and fluctuations of the groundwater table to form the existing composite matrix. This composite matrix has been further compressed by big piles of rockfall and construction/rockslide debris for many decades (per historical pictures from the 1956 Schoellkopf Slide) and densified by hydrocompaction through fluctuation of the water levels over the last hundreds of years.

The majority of the talus is composed of sandstone and limestone (approximagely 50 to 70 percent) with some shale rock fragments (approximately 10 to 30 percent) and soils (0 to 40 percent). The angle of repose of the talus is estimated to range from 32 to 40 degrees according to the existing topography. The talus deposits are composed of angular and slab shape rock fragments. In light of the factors mentioned in item 3 above, the internal friction angle of the talus would range between 36 and 45 degrees.

1. The design value for the internal friction angle (ɸ') of 38 degrees is substantiated by:
   a. Using the empirical equation proposed by Barton and Kjaernsli (1981) according to the Slope Stability Reference Guide for National Forests in the United States, EM-7170-13;
   b. Recommended values provided by FHWA design manuals such as EM-1110-1-1905;
   c. Navy Design Manual DM-7; and
   d. Review of consensus of geo-professionals on the subject of crushed stone where internal friction angle of 35 (i.e., relative density of 0 percent dumped) to 45 degrees (relative density of 100 percent) can be used.
2. Allowable bearing capacity of the talus deposits of 10,000 psf was selected for design of the retaining wall. This value was selected based on the historical data where substantial rockslide and construction debris at the project site previously supported large cranes with heavy loads during construction of the old buildings at the Schoellkopt site, a few decades ago.

CONSTRUCTION MONITORING

Crane testing was performed on October 10, 2013. The pedestal crane was loaded to 110 percent of its rated capacity, and rotated 360 degrees. Expected pedestal deflections were modeled, calculated and compared with on-site measurements. The calculated lateral deflections of approximately three quarters of an inch matched those observed during testing. The crane pedestal is structurally independent from the retaining wall, and the deflections from crane testing loads returned to zero after the load was removed.

Figure 5: Maid of the Mist Crane Load Test Picture.

Boat live loads were applied in October of 2013. No significant ice loading was experienced on the upper boat storage deck that season. A re-survey of the points was conducted in December of 2013. This data indicated wall movements with no trending direction, and below the accuracy of the method of measurement. Due to the stored boats and docks, these points were taken with a prism and rod, limiting the precision of the data. A visual inspection of the wall shows no abnormal cracking or visible wall movement.

CONCLUSION AND LESSONS LEARN

- The alternative design includes conventional cantilever retaining wall supported on footing over the talus deposits with geogrid layers behind the wall.
- Micropiles were required at the proposed crane location to resist the heavy crane loads.
- Engineering properties of talus are extremely difficult to determine in the lab or in-situ given the intrinsic heterogeneity of the talus formation.
- No drilling methods or geophysical testing method can reliably assess the engineering properties of the talus.
- A useful and practical method for determining shear strength properties in talus is to analyze an existing slope failure, if any and/or observation of the angle of repose and performance of the talus at the site coupled with engineering judgment and common sense.
- Construction in talus is usually difficult because of the typical heterogeneity of the deposits and corresponding unfavorable characteristics such as particle size and strength variations and large void spaces. In addition, there is the possibility of long term creep movement. Large settlements are also possible in talus. Foundations for structures in talus should extend through the deposit and bear on more competent material.

CONCLUSION AND LESSONS LEARN

- A reinforced soil mass behind the wall, coupled with widened and thickened wall footing, was used to accommodate future differential settlements, uplift and external stability requirements.
- Through partnering, the engineer and the contractor built good will and trust, encouraged open communication, which helped to eliminate surprises and adversarial relationships. In addition, it enabled all parties to participate and resolve problems and avoid or minimize disputes through informal conflict management procedures.
- It was a win-win for all parties involved. This project is a good example of how partnering can help advance a project to meet construction schedule and to minimize construction claims.
- The retaining wall to support the ferry boats for winter storage was completed in late 2013, ahead of the schedule and within budget.

https://www.youtube.com/watch?v=x1brJQodRnw&feature=youtu.be