

Safety Inspection of Brown Bridge Dam

**The City of Traverse
City Engineering Office
400 Boardman Avenue, 2nd Floor
Traverse City, Michigan 49684**

**STS Project No. 200802039
September 17, 2008**

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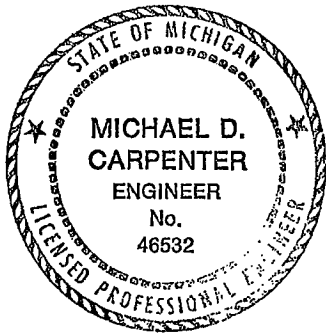
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1.0 Title Sheet

Name of the Dam: Brown Bridge Dam
Inventory Identification Number: 0544
County: Grand Traverse
Watercourse: Boardman River
Owner/Operator: City of Traverse
Public Services Department
625 Woodmere Avenue
Traverse City, Michigan 49686
Contact Person: Mr. Robert E. Cole, P.E.
Director of Public Services
(231) 922-4910
Hazard Potential Classification: High
Name of Inspectors: STS
Michael D. Carpenter, P.E.
Richard J. Anderson, P.E.
Date of Inspection: June 18, 2008
Engineer in Charge of Inspection: Michael D. Carpenter, P.E.
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9/18/08
Date

2.0 Introduction

STS was contracted by the City of Traverse (City) to perform a physical inspection of Brown Bridge Dam. The purposes of this study were to:

- evaluate the current physical condition of the dam,
- identify any deficiencies that, if left uncorrected, could lead to failure of the dam,
- develop recommendations for repair,
- identify required on-going monitoring,
- identify additional field investigations or studies required to bring the dam into compliance with State of Michigan dam safety requirements, and
- provide recommendations regarding modifications to current spillway and decommissioned hydroelectric plant so that the project can safely operate as a run-of-river facility and pass the design flood as a passive structure.

The Brown Bridge Dam (ID No. 0511) is a high hazard dam located on the Boardman River approximately 14 miles upstream of Traverse City, Michigan. Beginning in November 2006, the dam has been regulated under Part 315, Dam Safety, of the Natural Resources and Environmental Protection Act, 1994 PA 451, as amended (Act 451), Sections 31501 through 31529. Prior to then, the dam, along with the downstream Boardman and Sabin Hydroelectric Projects, was owned and operated by Traverse City Light and Power Company (TCL&P) for the purpose of generating hydroelectric energy and were regulated as run-of-river plants under Federal Energy Regulatory Commission (FERC) Project License No. 2978. All three hydros were decommissioned by TCL&P in 2006. The generating equipment at Brown Bridge is still present but the generators have been disconnected and the turbine wicket gates closed. If needed, the City has the ability to open the wicket gates manually to pass some portion of flow or, by closing the head gates, drain the turbine bays.

All river flows currently passes through the upper tainter gates and the log sluice. The log sluice and upper left tainter gate are operated manually, while the upper right gate can be remotely operated to maintain a desired pool elevation. The lower radial tainter gates are not normally used and were not operated during this inspection. Currently the TCL&P monitors reservoir elevation; however, the City is expected to take over all operations of the Brown Bridge development including remote monitoring duties.

The scope of services for this project consisted of performing a visual inspection of the dam, a diving inspection of the submerged portion of the upstream and downstream structures, a bathymetric survey of the tailrace and river just downstream of the plant, a horizontal and vertical control point survey of the embankment and concrete structures, preparing a report that summarizes our findings including recommendations for repair and modification of the spillway to function as a passive structure, and preparing opinions of probable cost for engineering and construction. All references to "right" and "left" within this report are with respect to looking in the downstream direction.

3.0 Conclusions and Recommendations

Overall, the dam is well-maintained and in generally fair condition. Based on the results of this inspection and review of previous hydrologic, hydraulic, and stability analyses, we conclude the following:

1. The spillway capacity is likely inadequate for the current high-hazard rating. The height of the dam, as defined by the State of Michigan (Part 315) for the purpose of establishing required spillway capacity, is the difference in elevation between the 200-year design flood and the lowest elevation downstream of the streambed. The normal operating reservoir elevation is 796.6¹ +/- 1.3 feet. Presuming both upper tainter gates and the sluice gate are fully open, the reservoir elevation computed for the 200-year flood event is elevation 797.8 feet (Ref. 9). According to the project drawings, the top of the spillway apron at the spillway/draft tube outlet is at elevation 756.5 feet. Therefore, the height of the dam per Part 315 is 41.3 feet. High hazard dams 40 feet in height or greater must be capable of safely passing half the probable maximum flood (50% PMF). PMF studies performed by Mead & Hunt (2001) established the PMF at 8,100 cfs, therefore 50% PMF is equal to 4,050 cfs. The project supporting technical information document (STID) reports a spillway capacity of 3,650 cfs at elevation 797.5 feet (normal headwater) and 5,670 cfs at elevation 802.2 feet (top of dam). However, these numbers are based on flow through all four tainter gates, which is operationally impossible. No information is currently available that provides the spillway capacity through the upper tainter gates plus sluice gate only. It should be noted that the two lower gates were refurbished by TCL&P in 2002; however, these gates are only intended for low-level drawdown of the pool and will not operate under head (pool level must be below elevation 791 feet (Section 1, Ref. 1). They therefore cannot be used to pass flood flows and should not be included in the computation of spillway capacity.
2. Active seeps and wet spongy conditions were observed over most of the downstream toe of the right embankment. A blanket of washed stone and drainage pipes was placed over several areas by TCL&P during the 1990's. Most all of the drainage pipes were observed flowing ranging from a low of 1 to 2 gpm to as high as 4 to 5 gpm. Localized ground settlement upstream of the drainage pipes and movement of sand (piping) from the embankment was observed. Probing with a 5-foot-long piece of 1/2-inch diameter rebar revealed very loose, saturated conditions along much of the lower third of the slope and along the downstream toe of the dam. Many of the seeps showed evidence of movement of fines out of the embankment. While we understand that seepage has been present since original construction, the development of localized settlement and movement of fines appears to be gradually increasing over time. Furthermore, previous slope stability analyses indicated factors of safety under normal pool conditions which are less than typically required for a high hazard embankment dam. Surveys performed during this 2008 inspection showed that the downstream slopes are actually steeper than that assumed during the previous

¹ All elevations in this report are in feet and are referenced to National Geodetic Vertical Datum (NGVD 29) unless otherwise indicated.

analyses, which would yield an even lower safety factor. Using the latest survey and well data (Piezometers 3 and 4), STS performed revised stability analyses of the embankment for normal, seismic, and rapid drawdown conditions. Based on these results, the embankment does not have an adequate factor of safety against failure and is meta-stable under normal pool, steady state seepage conditions.

Based on these conclusions and other observations made during our inspection, we recommend the following actions be taken within the timeframes listed:

3.1 Immediate (by end of 2008)

1. We recommend that pool be drawn down to elevation 792.6 feet (0.1 foot above the upper gate sill elevation) by locking the two upper flow tainter gates in the fully open position. Since flows from the sluice gate create a strong eddy within the tailrace area and flows impinge toward the end of the spillway apron, we recommend closing the log sluice. While the dive inspection and bathymetric soundings did not detect active scour or undermining in this area, the current operation has only been in effect for approximately 1-1/2 years. Continuous spills from the log sluice have the potential to erode the tailrace over the long-term, particularly during high flow periods before tailwater rises. Some erosion of the left bank was observed downstream of the spillway training wall. The log sluice should be opened only if needed for additional spillway capacity during high flow conditions or to pass debris.

Lowering the pool level to elevation 792.6 feet will also help reduce the phreatic levels and seepage gradients within the embankments and foundations, thereby reducing the amount of seepage and potential for piping, and improve the stability of the downstream slope. During the drawdown, measurements should be obtained daily until stable measurements are documented within the embankment wells.

2. We recommend a formal operation, maintenance, and monitoring (O&M) plan be developed and implemented by the City.

3.2 Short-Term (within 2 years)

1. Perform an evaluation of spillway capacity and flood routing under the design flood inflow hydrograph (50% PMF) under various lowered normal permanent pool scenarios, taking into account environmental and recreational considerations. The analysis should take into account loss of head due to the trash racks (including some amount of clogging) and updated weir flow and orifice coefficients. The analysis should also consider the option of modifying (larger bar spacing) or completely removing the trash racks, because of their susceptibility to zebra mussel clogging. Provide recommendations for modification to the dam spillway and project operational procedures, as needed, to safely pass the design flood with adequate freeboard.

3.3 Long-Term (within 5 years)

1. Implement the recommended modifications to the dam spillway and reservoir operation procedures, as needed, to safely pass the design flood.
2. Stabilize the right and left embankments, taking into account the affect of a lowered normal permanent pool on the phreatic levels within the embankment. Stability measures could include flattening the downstream slope and/or installing a drainage system, which would control seepage gradients, lower the phreatic surface, and reduce the potential for sliding or piping. Specific design requirements should be based on re-evaluation of the phreatic regime within the embankment after the pool has been drawn down to the new, lower pool elevation for a period of 1 to 3 years.
3. Perform concrete repairs to the powerhouse/spillway spillway structure. Remove the existing severely deteriorated concrete support for the log sluice gate hoist and design, fabricate, and install a new steel hoist support frame. Repair the deteriorated concrete around the upper spillway gates. Repair the localized severely deteriorated concrete on the training walls and log sluice spillway.
4. Update the project Operation and Maintenance Plan.
5. Update the project Emergency Action Plan to account for the status of the project spillway capacity.

4.0 Project Information

4.1 Available References

The following references were provided by the City and were used as a reference for this Project Information section and this report:

1. Gannett Fleming (2005) Supporting Technical Information Document, Brown Bridge Hydroelectric Project, FERC Project No. 2978, prepared for City of Traverse, December.
2. Grand Traverse County, Boardman River HEC-RAS GIS H&H Report (on CD), dated April 2008.
3. Mead & Hunt (1999) Consultant's Safety Inspection Report, Brown Bridge Hydroelectric Project, FERC Project No. 2978, prepared for City of Traverse, October.
4. Mead & Hunt (1994) Consultant's Safety Inspection Report, Brown Bridge Hydroelectric Project, FERC Project No. 2978, prepared for City of Traverse, November.
5. Material Testing Consultants (1995), Report of Field and Laboratory Geotechnical Testing, Brown Bridge Dam, October.
6. University of Wisconsin Department of Civil, Environmental and Geotechnical Engineering (1988) Evaluation of the Liquefaction Potential and Slope Stability of Brown Bridge Hydroelectric Dam Embankments and Foundation, prepared for the City of Traverse, August.
7. U.S. Army Corps of Engineers Detroit District and Engineering Consulting and Technology, Inc. (2008) Boardman River Feasibility Study – Preliminary Evaluation of Existing Structures – DRAFT, January.
8. Mead & Hunt, (2001) Probable Maximum Flood Study, Brown Bridge and Boardman Hydroelectric Projects, FERC Project Nos. 2978 and 2979, December.
9. Grand Traverse County, Boardman River HEC-RAS GIS H&H Report (on CD), dated April 2008.
10. Grand Traverse Conservation District, Letter of Agreement, unknown date

The above references were reviewed prior to inspection for comparison between current and past conditions. These references were also used to help generate the following project description and historical information.

4.2 Project History and Purpose

Brown Bridge Dam was originally constructed in 1921 for TCL&P, and it generated electricity continuously up to its decommissioning in November 2006 (Ref. 1). One of the turbines was replaced in 1941, the other is original. Both generators are original equipment, but were taken off line and rendered incapable of generating electricity in 2006. In 1984, TCL&P installed new control equipment in the powerhouse. To date, none of the generating or control equipment has been removed from the powerhouse.

4.3 Project Description

From left to right, the Brown Bridge Dam consists of an approximately 400-foot-long left embankment, a combined powerhouse/spillway structure, and an approximately 1,150-foot long right embankment. A log chute with slide gate is located adjacent to the right wall of the powerhouse. An abandoned fish ladder is located on the right embankment just right of the log chute. Selected project drawings are included in Appendix B.

Embankments

The lower portion of the embankments consists of hydraulic fill, and the upper portion of the embankments consists of compacted fill. There is a concrete core wall along the entire upstream length of the both earth embankments with a nominal top elevation of 798.4 feet. The project drawings show the core wall extends vertically to a depth of eight feet except at the powerhouse/spillway spillway structure where it functions as a cutoff wall and extends vertically below the upstream wall of the powerhouse/spillway spillway and is keyed two feet into the clayey till. The wall extends laterally at this depth left and right of the upstream approach walls for a distance of 20 feet beyond the wall footings. The minimum crest elevation of the embankments identified during the 2008 centerline survey was 802.0 feet. Based on the original design drawings, the design embankment crest elevation was 802.4 feet. The embankment crest width varies from 12 to 15 feet. The downstream slopes are reported to vary from 2H:1V to 2.5H:1V (Ref. 3), however the 1994 stability analysis assumed downstream slopes as steep as 1.8H:1V. Cross sections surveyed during our 2008 inspection showed downstream slopes on the right embankment as steep as 1.5H:1V. The left embankment adjacent to the left powerhouse/spillway spillway wall appeared to be steeper than 1.5H:1V.

Spillway

As part of the combined powerhouse/spillway spillway structure, the Brown Bridge spillway contains two upper 12-foot wide by 5.5-foot high tainter gates. The upper spillway sill is at elevation 792.5 feet. The two lower 12-foot wide by 5.5-foot high tainter gates function as a turbine bypass and cannot be opened if the water level is above elevation 791 feet (Ref. 1). The lower spillway sill is at elevation 786.7 feet. In addition, there is a log chute with a slide gate measuring 6-foot wide by 6-foot high adjacent to the powerhouse. The log chute sill is at elevation 792.5 feet. The log chute is intended for additional discharge capacity but has been used to pass base river flows since November 2006.

Powerhouse

The powerhouse is a brick structure supported on a reinforced concrete substructure. The powerhouse contains two vertical shaft Francis turbines with an installed capacity of 830 kW. The turbines consist of one Leffel Type Z rated at 690 hp and one Leffel Type F rated at 375 hp (Ref. 3). The powerhouse was constructed in 1921 and is an integral part of the original dam project and was in continuous operation until November 2006 when TCL&P surrendered its operating license and decommissioned the plant. All of the turbine-generating and control equipment are still in the powerhouse.

Intake and Outlet Structures

The intake structure is integral to the powerhouse. Left and right concrete approach (wing) walls flank either side of the intake bays. Inclined trashracks are located on the upstream side of the intake. In the current mode of operation with the upper tainter gates open and the wicket gates closed, water passes through the inclined trashracks and flows over the upper tainter gate concrete sill at elevation 792.5 feet. With the wicket gates open, water passes through a set of horizontal trashracks inside the structure at elevation 792.5 feet, through the turbines, and drops into a short tailrace under the powerhouse. The tailrace discharges to the spillway apron at invert elevation 756.5 feet.

Reservoir

The Brown Bridge Dam is operated as a run-of-river facility. The normal headwater elevation of the Brown Bridge Reservoir is 796.7 feet. At normal pool elevation, the surface area of the pond is 191 acres, and the storage volume is approximately 1,900 acre-feet. The drainage area of the Boardman River at the dam is 151 square miles (Ref. 3).

5.0 Field Inspection

Messrs. Michael D. Carpenter, P.E. and Richard J. Anderson, P.E. inspected the Brown Bridge Dam on June 18, 2008. At the time of inspection, the weather was clear with temperatures in the low 70's. Visible portions of the project were inspected, including the embankments, powerhouse/spillway spillway structures, the upper tainter gates, the sluice gate and hoist. Both powerhouse turbine bays were completely dewatered and an internal inspection of the powerhouse structure was also performed. In addition, a horizontal and vertical control point survey was performed for the embankment dam and concrete structures using the same points monitored during previous FERC 5-year Part 12D Inspections. Bathymetric soundings of the spillway tailrace area were also obtained. A dive inspection was also performed by Seaview Diving Contractors, Inc. of Seymour, Wisconsin under subcontract to STS. The diver inspected the below-water structures along the upstream and downstream face of the dam, and along the downstream end of the spillway apron for signs of signs of scour or undermining. Messrs. Timothy Lodge, Ken Gregory, and dam tender Ms. Robin Johnson of the City of Traverse were present during the inspection. Mr. Joe Kaltenbach of TCL&P as well as TCL&P operations staff were present on-site during the inspection to answer questions, provide additional information, and assist with the operation of the spillway gates and dewatering of the powerhouse. The reservoir and tailwater elevations at the time of our inspection were observed to be at 797.0 and 767.2 feet NGVD, respectively. A visual inspection checklist was completed during the inspection and is included in Appendix E. Photographs documenting the condition of the above-water structures are included in Appendix C. The result of the control point survey and bathymetric soundings are included in Appendix F. A copy of the dive inspection report and DVD are included in Appendix G.

Right Embankment

Overall, the right embankment appears to be in fair condition (Photo 6). The crest elevation undulates slightly but shows no visible signs of lateral movement or cracking. A centerline survey performed during the inspection showed elevations varied from a high of 804.25 feet near the right abutment, to a low of 802.60 feet near the powerhouse. The grass cover is fair but is sparse in several localized areas. The upstream slope between the crest and the concrete core wall is fairly uniform and exhibits no signs of settlement, sloughing, or sliding. The slope upstream of the core wall, is submerged but appears to be fairly uniform and flat. There is no rip rap or other forms of erosion protection other than the core wall. The top of the core wall is in good condition with no obvious signs of cracking or displacement and only minor, localized deterioration of the concrete surface (Photo 12). Control point surveys performed during the inspection indicated the core wall crest varied from elevation 798.35 to 798.63 feet. The upstream slope grass cover is fair but is sparse in several localized areas. The upper portion of the downstream slope is very steep, but exhibits no signs of settlement, sloughing, or sliding (Photos 7 and 8). Some localized settlement has occurred on the lower portion between the toe and slope break at the third point above the toe, particularly right of the bend in the core wall (Photo 8). Most of the settlement appears to be due to loss of ground above perforated drain pipes placed within gravel blankets constructed during the 1990's to address seepage (Photo 10).

Most of the areas just above and below the downstream toe are wet and spongy, with numerous seeps visible (Photos 9 and 10). Gravel blankets with internal drain pipes were installed at multiple times by TCP&L, with the latest being installed in 1996. This area is located near the right end of the embankment and is approximately 250 feet long. TCL&P reportedly installed a 12 to 18 inch layer of washed stone with perforated drain pipes and 10 PVC outlet pipes which discharge flows to the toe. All of these discharge pipes were observed to be flowing from a low of 1 to 2 gpm to a high of 4 to 5 gpm. Many of the drain pipes and seeps showed evidence of movement of sand out from the embankment (Photo 10). It is uncertain if filter fabric was used in this repair, or if the drainage materials were properly selected in accordance with published guidance documents to prevent piping of materials.

A 5-foot-long piece of 1/2 inch diameter rebar was used to probe areas along the downstream toe (Photo 9). Downstream of the toe near the right abutment, the entire length of rebar was easily pushed the soil. Most areas along the downstream toe, particularly right of the core wall bend, were penetrated 18 inches to 30 inches with little difficulty. Several areas on the downstream slope above past gravel blanket repairs were easily probed to a depth of 3 to 4 feet and were noted as saturated within a foot of ground surface. The soundings indicate a high phreatic surface exiting above the toe of the dam and loose to very loose soil conditions near the ground surface.

The entire area downstream of the toe is wet and marshy. Water is collected in two intercepting ditches, one located 10 to 20 feet downstream of the toe (Photo 11), and one located approximately 100 feet downstream of the toe. Flow volumes are monitored in the ditch furthest downstream with a pair of v-notch weirs.

Spillway / Powerhouse Structure

Overall, the spillway powerhouse superstructure is in good condition (Photo 13). The interior of the powerhouse is fairly clean and is well maintained. At some time in the past, TCL&P coated much of the concrete comprising the exterior of the powerhouse/spillway spillway above waterline an approximately 3/16 inch to 1/4 inch thick layer of latex-based sand grout (Thoroseal). In many locations, the Thoroseal coating is beginning to delaminate from the base concrete and is showing signs of pattern cracking. The exterior and interior portions of the structure were observed during this inspection. The following paragraphs summarize the observed conditions.

Interior Dewatered Inspection – Both intakes were dewatered by installing the head gates and draining through the turbine wicket gates. The intakes and turbine pits were then visually inspected. In generally, the concrete substructure and integrated spillway were observed to be in fair to good condition with exception to a few deteriorated areas. The concrete support beams under the upstream and downstream walkways and powerhouse walls are highly deteriorated and exhibit significant cracking, efflorescence, and deterioration (Photos 22, 23, 24, and 25). Some moderate surface spalling and delamination was observed at the head gate slots and at the water line within the intake (Photos 23 and 24). The turbine pits were found to have 1 to 2 foot thick layer of zebra mussels on the floor and a thick coating on the

walls and appurtenances (Photos 14, 15, and 16). In general, the right unit bay walls, floor, and trashracks were more heavily coated than those on the left due to flows being regularly passed through the right intake. The lower tainter gates were heavily coated with zebra mussels and the condition of the members could not readily be observed (Photo 16). According to Mr. Joe Kaltenbach of TCP&L, the lower tainter gates were refurbished to working condition in 2002. Due to the heavy coating of mussels, the risk of not being able to re-close the gates, and the need to manually lower the gates with come-alongs, the lower tainter gates were not tested during the inspection.

Both the left and right upper tainter gates were raised to a height of approximately 18 inches (Photo 21). The gates operated smoothly with no binding, sticking, or racking. TCL&P personnel did not have on hand the needed chains to lift the gates higher than 18 inches. The skin plate and gate members are in good condition with no bending or significant loss of section (Photos 17 and 19). The paint coating is in good condition. The rubber gate seal at the sides and bottom of the right gate are completely missing. This gate is the automatic gate and is operated much more frequently than the left gate. The gate seals at the left gate are in good condition. The lifting chains for both gates are in good condition. The left and right interior concrete walls of both spillways are generally deteriorated near the gate seals (Photos 18 and 19). There is steel mesh exposed under delaminated concrete near the lower left corner at the gate where a previous repair was made and has failed. The concrete around the left and right trunnion pins is in good condition. The rollway concrete has some loss of material and surface delamination. Much of the Thoroseal coating over the concrete pier walls is showing signs of pattern cracking and delamination. Chipped out concrete was observed along the gate alignment for both spillways (Photos 17 and 20). The concrete was likely removed to prevent binding but was not removed to the steel stops, which indicates the gates may not be able to fully open.

Exterior Inspection – On the exterior of the structure, some moderate concrete deterioration was observed on the top of the right and left training walls, on the log chute, and on the log chute gate hoist support (Photos 26 and 27). The upstream left and right approach (wing) walls to the powerhouse/spillway spillway are in good condition and exhibit no signs of significant cracking, deterioration, or movement. The diving inspection showed an approximately 1-inch thick buildup of zebra mussels on concrete surfaces below water line. The upper 3 to 4 feet of upstream, inclined trashracks have minor buildup of zebra mussels. Below a depth of approximately 4 feet, zebra mussels fully clog the trash racks. Similarly, the horizontal trashracks inside the powerhouse at elevation 792.5 feet are 10% to 60% clogged with zebra mussels – the right (automatic gate) bay being more heavily coated than the left bay.

During the diving inspection, the diver entered the draft tube area below the turbines and found the concrete apron and walls to be in good condition. The tailrace apron was generally covered with riprap,

except for a small area near the right, downstream side where concrete was exposed. The left and right training walls were in good condition below waterline. There was no evidence of undermining or significant erosion was found at the downstream end of the concrete apron.

The manual hoist, steel gate supports, rack and pinion, and gate of the log chute was observed to be in good working condition and operates with no binding. However, the paint coating is peeling and the concrete support for the hoist is significantly deteriorated and should be replaced (Photo 27). The concrete comprising the head walls, side walls, and floor of the log chute are in generally good condition and exhibit only minor surface deterioration. Much of the Thoroseal coating is deteriorating and delaminating. The tailrace area shows no signs of scour or undermining, however full flow in the log chute, combined with flow from the right, upper tainter gate, creates a strong eddy in the tailrace area and is beginning to erode the left bank.

Left Embankment

Overall, the left embankment appears to be in good condition. The crest elevation undulates slightly but shows no visible signs of lateral movement or cracking. A centerline survey performed during the inspection showed elevations varied from a high of 802.90 feet near the left abutment, to a low of 802.02 feet at a distance of 200 feet left of the powerhouse. The grass cover is fair but is sparse in several localized areas. The upstream slope between the crest and the concrete core wall is fairly uniform and exhibits no signs of settlement, sloughing, or sliding. The slope upstream of the core wall is submerged but appears to be fairly uniform and flat. There is no rip rap or other forms of erosion protection other than the core wall. The top of the core wall is in good condition with no obvious signs of cracking or displacement. Control point surveys performed during the inspection indicated the core wall crest varied from elevation 798.40 to 798.47 feet. The upstream slope grass cover is fair but is sparse in several localized areas. The downstream slope closest to the powerhouse/spillway is very steep but exhibits no signs of settlement, sloughing, or sliding (Photo 2). Seepage was observed exiting the toe of slope at the left end of the downstream training wall. Seepage was estimated at 8 to 10 gpm. A 5-foot-long piece of 1/2 inch diameter rebar was used to probe along the downstream slope where the left bank meets the left embankment downstream slope. The rebar was pushed 4-1/2 feet with little effort and a high phreatic line was encountered up to about the 1/3 height of the slope (Photos 2, 3, and 4).

The end of the left training wall is deteriorated and shows signs of erosion and undermining (Photo 5). Much of the riprap along the left bank downstream of the left training wall is missing and the bank soils are beginning to be undercut (Photo 3).

6.0 Structural Stability

6.1 Embankments

Stability of the earth embankments was last analyzed by Mead & Hunt as part of the June 1987 Report on Inspection (see Ref. 1) for the following loading conditions:

Table 6.1 - Embankment Stability Loading Conditions – Brown Bridge Dam

Loading Case	Slope Analyzed
Normal maximum pool with steady-state seepage	Upstream and Downstream
Normal maximum pool with earthquake loading	Upstream and Downstream
Steady-state seepage with surcharge (flood) pool	Downstream
Rapid drawdown form normal maximum pool	Upstream

The analysis was performed using the STABL2 computer program, which uses a limit equilibrium approach as applied to the method of slices to determine slope stability. Factors of safety were computed using Bishop's method. Factors of safety were calculated for the upstream slope for the rapid drawdown condition as part of the 1994 Report on Inspection (Ref. 4) using the STABL6 computer program.

The subsurface conditions were modeled based upon soil parameters performed in the embankments in 1985 (see Ref. 1). An additional six soil borings were performed in 1992 as part of the 1994 analysis of rapid drawdown stability (Ref. 4). An additional boring was performed in 1995 (Ref. 1). Based on the soil borings, the embankment and foundation soils were concluded to be non-cohesive. Unit weight and friction angles were estimated from published correlations with SPT N-values and are present in the Table 6.2.

Table 6.2 - Embankment Soil Properties – 1987 Brown Bridge Dam (Mead & Hunt)

Layer	Moist Unit Weight (pcf)	Saturated unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
A (dumped embankment fill)	110	120	0	30
B (upper hydraulic fill)	110	120	0	33
C (lower hydraulic fill)	100	110	0	27
D (foundation – sand)	120	130	0	33

Note: For the 1994 rapid drawdown analysis, a different set of unit weight and strength parameters were used in the stability model. The explanation for this was not provided in either Reference 1, 3 or 4.

Presumably, the 1994 analysis took into account the new soil boring data collected in 1992 and 1995.

However, the other loading cases were not re-analyzed using the new soil boring data.

Phreatic conditions within the embankment and foundation were based upon data collected from the five right embankment piezometers (P-5, 6 and 7 right of the powerhouse and P-3 and 4 right of the core wall bend). A

downstream slope of 1.8H :1V was assumed. The factors of safety for each of the loading cases analyzed in 1987 and 2004 are summarized in Table 6.3.

Table 6.3 - Summary of 1987 and 2004 Stability Analysis Results (Mead & Hunt)

Case	Slope	FERC-Recommended Safety Factors	Calculated F.S.	Comments
Normal condition Steady seepage*	D/S	1.5	1.2	Below minimum required
Seismic condition**	D/S	1.0	1.1	O.K.
Surcharge pool***	D/S	1.4	1.2	Below minimum required
Rapid drawdown****	U/S	1.0	1.9	O.K.

- * Normal headwater elevation 797.6 feet assumed
- ** Pseudostatic seismic coefficient = 0.05
- *** Surcharge pool elevation 804.2 feet (top of right embankment)
- **** Rapid drawdown from elevation 797.6 to 788.5 feet assumed

Because the computed factors of safety did not meet the FERC minimum criteria for all loading cases, Mead & Hunt recommended re-analysis of sliding stability as part of the 2004 Part 12 Inspection. This re-analysis was never performed while the project was under FERC jurisdiction.

As part this Inspection report, STS re-analyzed the stability of the right embankment using the following:

- piezometer data available through 2008 (see Appendix F),
- updated soil density and shear strength parameters using the 1985, 1992, and 1995 soil boring data, and
- updated cross section geometry using the survey data obtained during the 2008 inspection (see Appendix F).

The right embankment was reanalyzed at Station 8+60, which was found to have the steep upper slope and available piezometer and boring data. This was the same section analyzed by Mead & Hunt in 1987 and 1994. Based upon our review of the available data, and published correlations between unit weight, friction angle, and SPT blow counts, the following revised soil properties were used in our analyses:

Table 6.4 - Embankment Soil Properties – 2008 Brown Bridge Dam (STS)

Layer	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
A (upper embankment fill – medium dense)	115	120	0	30
B (lower embankment hydraulic fill – loose to very loose)	110	115	0	28
C (foundation sand / till)	125	130	0	33

Factors of safety computed from the STS re-analysis are presented in Table 6.5. Critical surfaces and assumed phreatic conditions are presented in Appendix H.

Table 6.5 - Summary of 2008 Stability Analysis Results (STS)

Case	Slope	1997 Reported F.S. (Mead & Hunt)	2008 Calculated F.S. (STS)	Comments
Normal condition Steady seepage	D/S	1.2	1.0	Meta-stable
Seismic condition	D/S	1.1	0.9	Unstable
Surcharge pool	D/S	1.2	0.8	Unstable
Rapid drawdown	U/S	1.9	1.5	O.K.

Our analysis shows that in its current condition, the right embankment is meta-stable. Under seismic and flood pool conditions, the embankment is considered unstable.

We recommend that the City immediately draw down the impoundment to elevation 792.6 feet to reduce the phreatic levels and improve downstream slope stability. The City should also undertake studies to flatten the downstream slope and provide internal drainage to control seepage gradients.

6.2 Concrete Structures – Powerhouse / Spillway

The stability of the powerhouse structure was analyzed by Mead & Hunt for the 1987 Inspection Report, then revised in the 1994 Addendum No. 2 to the 1987 Inspection Report, and again revised in the 1995 Composite Addendum No. 1 to the 1994 Inspection Report (Ref. 1). The 1994 addendum was prepared to account for the revised PMF loading conditions. The 1995 addendum was prepared to take into account new piezometers (P-9 and P-10) installed to verify uplift conditions. In the 1999 Inspection Report (Ref. 3), the powerhouse stability was again updated to take into account uplift data from the piezometer installed through the intake deck in 1997.

The following loading conditions were evaluated for the powerhouse/spillway structure (Ref. 3):

Case 1 – Normal reservoir and tailwater levels, plus silt and full uplift

Case 1A – Case 1 with one unit dewatered

Case 4 – PMF reservoir and tailwater water elevations, plus silt and full uplift.

Foundation material properties for the powerhouse were determined based upon direct shear testing of samples taken during the installation of the piezometer under the intake pier in 1995. The soil samples were described as a brown sandy, lean clay (CL). Direct shear testing yielded a cohesion value of 2.5 ksf with an internal angle of friction of 5.0 degrees.

Lane's weighted creep method was used to calculate uplift pressures for the powerhouse. The method was determined to be approximate due to the concrete core wall which the as-built drawings show extends two feet into the clay till under the upstream edge of the powerhouse. Uplift was calculated as full headwater pressure at

the upstream heel of the powerhouse structure, to the uplift levels recorded in the powerhouse through the intake deck, to full tailwater at the downstream toe.

The results of the stability analysis results for the powerhouse/spillway spillway structure is summarized as follows in Table (Ref. 1).

Table 6.6 - Summary of Stability Analysis Results of Concrete Structures (Mead & Hunt)

Structure	Case 1	Case 1A	Case 4	Comments
Headwater level (feet)	796.6	796.6	802.7	
Tailwater level (feet)	767.6	767.6	782.2	
Powerhouse				
Maximum net base pressure (ksf)	3.35	2.82	4.16	O.K.
Shear friction factor (SFF)	2.45	2.43	2.10	O.K.
Minimum FERC-req'd SFF	3.0	2.0	2.0	O.K.

The combined powerhouse/spillway spillway structure is considered stable with respect to sliding stability.

7.0 Hydrology and Hydraulics

As a project previously licensed under the Federal Energy Regulatory Commission, the Brown Bridge project was evaluated for the probable maximum flood (PMF). In their report dated November 2001, Mead and Hunt (Ref. 8) identified the PMF to have an inflow of 8,100 cfs. The inflow design flood (IDF) for the project was equal to the PMF (Section 6, Ref. 1). The summary of the hydrology and hydraulics included in the project STID is adequate and is based on the 2001 PMF report and previous Mead & Hunt studies, with exception of a spillway rating table.

7.1 Hydrology

The PMF for this project used the Electric Power Research Institute (EPRI), Probable Maximum Precipitation Study for Wisconsin and Michigan (published July, 1993). Runoff from the water shed was simulated using the HEC-1 computer model and the Michigan State Soil Geographic Database (STATSGO). Four rain events were used to calibrate the HEC-1 model and develop the PMF inflow hydrograph. A peak PMF inflow of 8,100 cfs resulted from the 2001 PMF analysis.

In the 1995 Addendum No. 1 PMF study, Mead & Hunt performed a statistical estimation of flood return frequency for 25, 100, and 200 year events (Ref. 1). In addition, the Michigan DEQ estimated various discharge frequencies for the Brown Bridge Dam. Table 7.1 summarizes the maximum flows estimated by Mead & Hunt and the MDEQ. A copy of the Request Record from the MDEQ containing estimated design flood flows is included in Appendix D.

Table 7.1 - Summary of Discharge Frequencies

Discharge Frequency	M&H, 1995 Flow (cfs)	2008 MDEQ Flow (cfs)
10 year return		750
25 year return	815 - 907	
50 year return		950
100 year return	1,107 – 1,259	1,000
200 year return	1,204 – 1,446	1,100
500 year return		1,300

The height of the dam, as defined by the State of Michigan (Part 315) for the purpose of establishing required spillway capacity, is the difference in elevation between the 200-year design flood and the lowest elevation downstream of the streambed. According to the project drawings, the top of the spillway apron at the toe of the dam is at elevation 756.5 feet. The reservoir elevation computed for the 200-year flood event is elevation 797.8 feet assuming a steady-state condition and both upper tainter gates and the log sluice gate are fully open (Ref. 9). Therefore, the height of the dam per Part 315 is 41.3 feet. High hazard dams 40 feet in height or greater must be capable of safely passing half the probable maximum flood (50% PMF).

7.2 Hydraulics

Discharge rating tables were developed by Mead & Hunt in their 1995 Addendum No. 2 and 2001 PMF reports. The 1995 rating table was not available for review or inclusion into this report. Table 7.2 summarizes the current spillway rating table for the Brown Bridge project.

Table 7.2 - Spillway Rating Table

Reservoir Elevation (ft, NGVD)	Spillway Flow * (cfs)
797.5 (normal pool)	3,650
802.2 (top of embankment)	5,670

* - Flow assumes all four gates are open.

The spillway discharge flows reportedly include a head loss resulting from the trash racks. As summarized above, the two lower gates reportedly will not operate under flood conditions (Section 1, Ref. 1). Therefore the lower gates cannot be used to pass flood flows and should not be included in the computation of spillway capacity rating table or curve. Based on our review of the available information, a rating table could not be found that quantifies the contribution of the lower gates. Because the spillway capacity during flood conditions is currently unknown, the reservoir pool elevation cannot be estimated using steady state or flood routing methods.

Using the available information and assuming an approximate 50% reduction in spillway capacity to remove the contribution of the lower gates, it is possible that the reservoir could overtop and breach the embankments with a 50% PMF inflow of 4,050 cfs. Therefore, additional flood routing studies should be conducted to estimate the reservoir elevation during the design flood event under possible alternative reservoir operational scenarios. As part of this study, a spillway rating table that identifies the individual contribution of the five gates should be generated. Possible future operational scenarios are discussed further in Section 8.0.

8.0 Operation and Maintenance

Currently there is a letter of agreement in place between the Grand Traverse Conservation District (GTCD), Traverse City, and TCL&P outlining requirements and responsibilities for the operation and maintenance of the Brown Bridge development. The GTCD maintains staff at the project site who visits the dam daily to monitor the reservoir elevation and confirm the pool elevation remotely monitored by the TCL&P. In the future, we understand the City will be accepting full responsibility for monitoring of the project. Because of this transfer, we recommend a formal operation, maintenance, and monitoring (O&M) plan be developed by the City prior to accepting this responsibility. The above referenced agreement can be used as a basis for the O&M plan; however, we recommend adding the following flood-condition items:

- Identify the staffing, operation, and maintenance requirements needed during a flood event
- Define what constitutes a flood event
- Provide procedures to keep the trash racks clean during a flood event

In order to balance the desire to maximize long-term normal reservoir pool, versus the need to safely pass the design flood, the following alternative operational scenarios were considered:

1. Maintain the reservoir at the historic level (elevation 796.6 feet), and provide additional spillway capacity
2. Use the turbine wicket gates to provide additional flow capacity to maintain the reservoir at historic or lower elevations
3. Maintain the reservoir at elevation 792.6 feet, as recommended in Section 3.1, which assumes the upper tainter gates are locked full open
4. Maintain the reservoir at elevation 786.8 feet, which assumes both the upper and lower tainter gates are locked full open
5. Maintain the reservoir at an elevation intermediate to those in Scenarios 3 and 4 (between elevation 786.8 feet and 792.6 feet) by locking both the upper and lower tainter gates full open, and cutting down the top of the headgates to control the normal reservoir elevation.

A representation of these scenarios is depicted on the sketches included in Appendix I. Scenario 1 is considered unrealistic due to the instability of the project embankments and the inadequate spillway capacity. This scenario would require significant capital investment in additional spillway capacity and embankment remediation and is considered to be the least cost-effective option. Scenario 2 is not considered viable because the wicket gates would need to be automated (not a passive system) to maintain a run-of-river condition. The controls and remote monitoring would require active monitoring and maintenance of the equipment. Furthermore, both turbine wicket gates combined provide only an additional 300+/- cfs of flow capacity (Ref. 1), which is insufficient to pass anything other than minimum base flows.

Scenarios 3, 4, and 5 may be potentially viable options depending on the results of the spillway capacity evaluation recommended in Sections 3.2 and 7.2, and should be investigated further. Scenario 3 is described in Section 3.1 and would result in passive spillway system with the log chute available to pass debris. Scenario 4 would require permanent opening of the lower gates and would result in the lowest permanent reservoir elevation of the scenarios considered. Because the log chute would be above the level of the permanent pool, a new procedure for passing debris downstream would need to be developed. For Scenario 5, the head gate would be cut down to maintain a pool elevation between Scenario 3 and 4. In this case, the lower gates would also be kept completely open but the top of the head gate would control the normal reservoir pool elevation.

Although a plan of the possible limits of the reservoir is provided in Appendix I, it should be considered approximate as the reservoir bathymetric contours are only accurate to within 10 feet. A more accurate bathymetric survey would need to be conducted to estimate the location of the future shoreline.

9.0 Emergency Action Plan

STS reviewed the City's Emergency Action Plan (EAP) as part of the inspection. The EAP was found to be complete and appropriate for the Brown Bridge Dam; however, it should be updated based on the long-term operational scenario chosen.

10.0 Cost Estimates

Concept-level opinions of probable cost for the recommendations provided in Section 3.0 are summarized in the table below.

Table 10.1 - Concept-level Opinions of Probable Cost

Recommendation	Estimated Cost	Comments
Immediate (by end of 2008)		
Lower reservoir to elevation 792.6 feet	No cost	To be completed by City staff
Short Term (within 2 years)		
Evaluate hydraulic capacity of project and perform flood routing under various operational scenarios	\$15,000 to \$25,000	Engineering study. Cost depends upon number of scenarios under consideration.
Long Term (within 5 years)		
Modify the spillway structures to pass the design flood under the new permanent reservoir pool and operational scenario (see Section 8.0 above)	\$0 to \$2,500,000	Range of costs for design, construction engineering, and construction of spillway modifications for Operation Scenarios 3, 4 or 5, as described in Section 8.0 above. Lower bound number assumes Scenario 3 is sufficient to pass the design flood. Upper bound number assumes a new overtopping spillway is required to pass the design flood. Costs to remove the turbines, generators, and auxiliary equipment are not included.
Repairs to spillway / powerhouse structure concrete and install new log sluice hoist and support frame	\$150,000 - \$250,000	Including design, construction engineering, and construction
O&M plan and EAP updates	No cost	To be completed by City staff
Stabilize right and left embankments	\$200,000 to \$1,000,000	Extent of remedial measures depends upon how much permanent pool is lowered and reduction of phreatic surface. Could vary from relatively minor slope flattening or berms to major regrading and installation of internal drainage system

These opinions are based upon 2008 unit costs for similar projects and engineering judgment based on experience with similar projects. They are not based on any actual designs, engineering analysis, quantity calculations, or contractor bids. Actual costs will vary based on design, permit conditions, contractor's perceived risk, site access, weather, and market conditions. No warranty concerning the accuracy of these costs are expressed or implied.

The staffing needs required to actively control flows through the dam should not be as great as currently required, assuming a passive long-term approach is selected. Therefore, the cost for maintenance, monitoring, and operation could possibly decrease from current operational expenditures. In addition, no long-term maintenance of the turbine shaft, bearings, and wicket gates should be required, provided the plant is intended to be

decommissioned indefinitely. Zebra mussel growth will continue to be a maintenance problem for this project by clogging the trash racks and reducing the ability to safely pass flows. The mussels can be mechanically removed from the trash racks using the existing or a slightly modified trash rake that is present at the project site.

Appendices

Appendix A – Project Location Map

Appendix B – Project Drawings:

- Topography of Dam Site (1921)
- Brown Bridge Plan (1921)
- Fish Chute Plan (1921)
- Tainter Gate Plan (1921)
- Steel Details (1921)
- Powerhouse Details (1921)
- Log Chute Details (1921)
- Penstock & Spillway Details (1921)
- Penstock & Spillway Details (1921)
- Downstream Apron Embankment Walls (1921)

Appendix C – Photographs (29)

Appendix D – Hydrological Data

MDEQ Flood Discharge Request Record

Appendix E – Dam Inspection Checklist

Appendix F – Control Point Survey, Bathymetric Soundings, Piezometer, and Weir Data

Appendix G – Dive Inspection Report and DVD

Appendix H – Embankment Stability Analyses

Appendix I – Operational Scenario Sketches

Appendix A

Project Location Map



STS | AECOM

**SITE LOCATION MAP
BROWN BRIDGE DAM
GRAND TRAVERSE COUNTY, MI**

Drawn:	SJE	8/13/2008
Approved:	MDC	8/13/2008
Scale:	AS SHOWN	
PROJECT NUMBER	200802039	
FIGURE NUMBER	1	

847.279.2500
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Appendix B

Project Drawings:

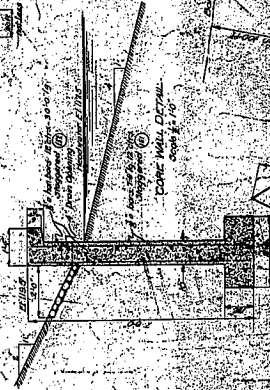
- Topography of Dam Site (1921)
- Brown Bridge Plan (1921)
- Fish Chute Plan (1921)
- Tainter Gate Plan (1921)
- Steel Details (1921)
- Powerhouse Details (1921)
- Log Chute Details (1921)
- Penstock & Spillway Details (1921)
- Penstock & Spillway Details (1921)
- Downstream Apron Embankment Walls (1921)

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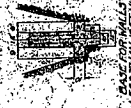
BAR LIST - CORE WALL

No.	Location	Wd.	Thk.	Bar	Length
1	at E. West Vert.	1500	3'	7-8"	
2	" "	575	3'	5-9"	
3	at West Vert.	1425	3'	5-9"	
4	" "	1000	3'	5-9"	

CONTRACTION JOINT
FOR LOW WALL
to be spaced not
more than 30 ft apart



TYPICAL SECTION THROUGH CONCRETE ENABLING WALL

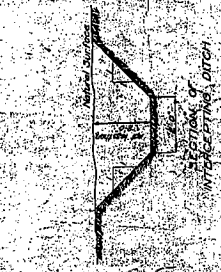


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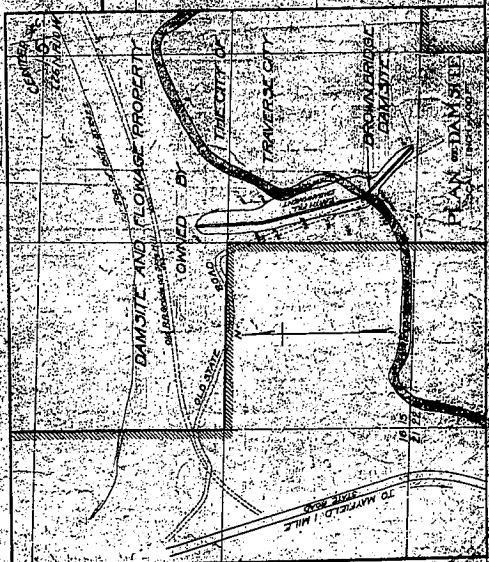
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PLAN SECTION THROUGH CONCRETE ENABLING WALL AT INTERSECTION



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INTERSECTION AT RICH



DAM SITE AND FLOWAGE PROPERTY

OWNED BY
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TRAVERS CITY

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LOCATION PLAN

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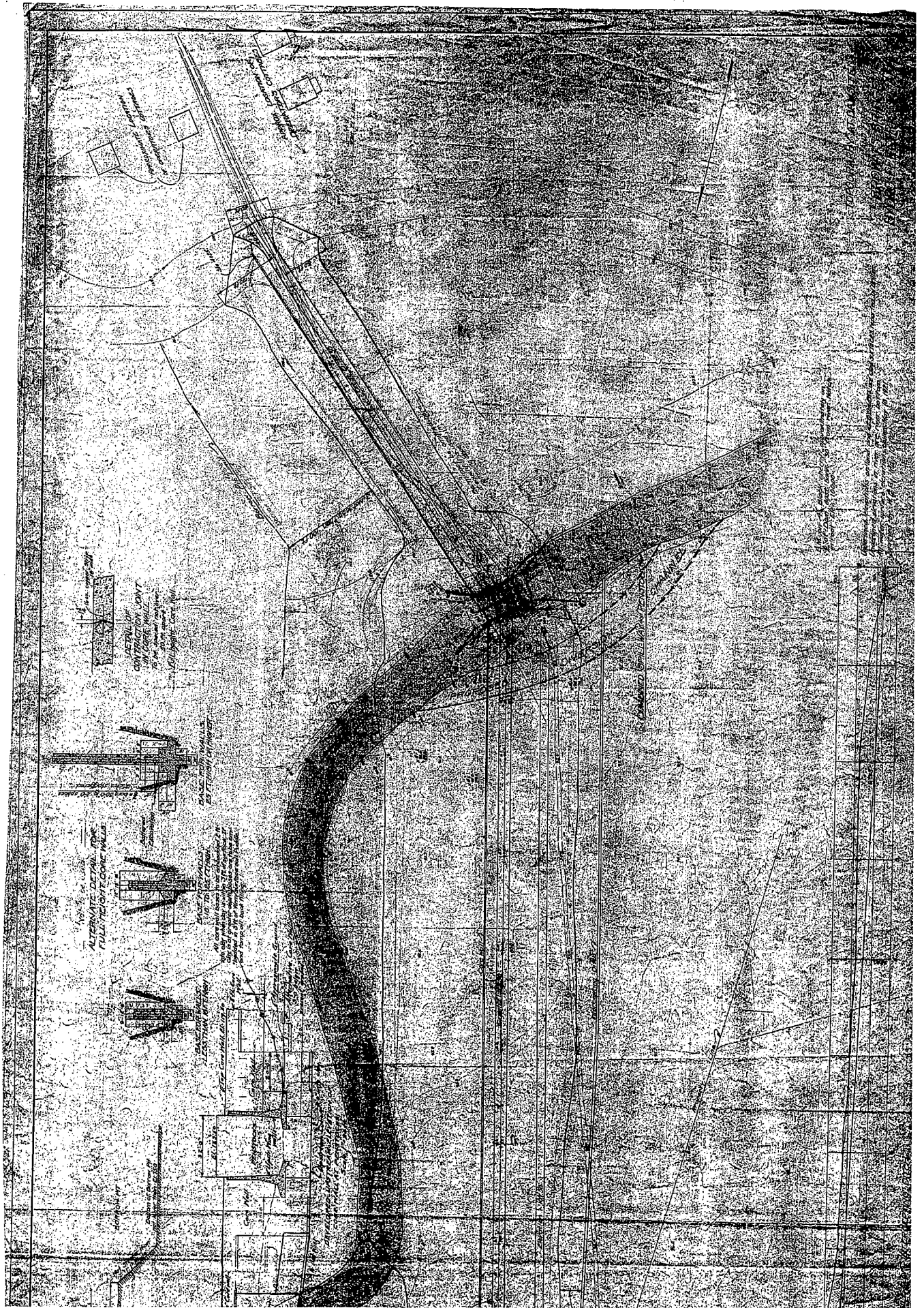
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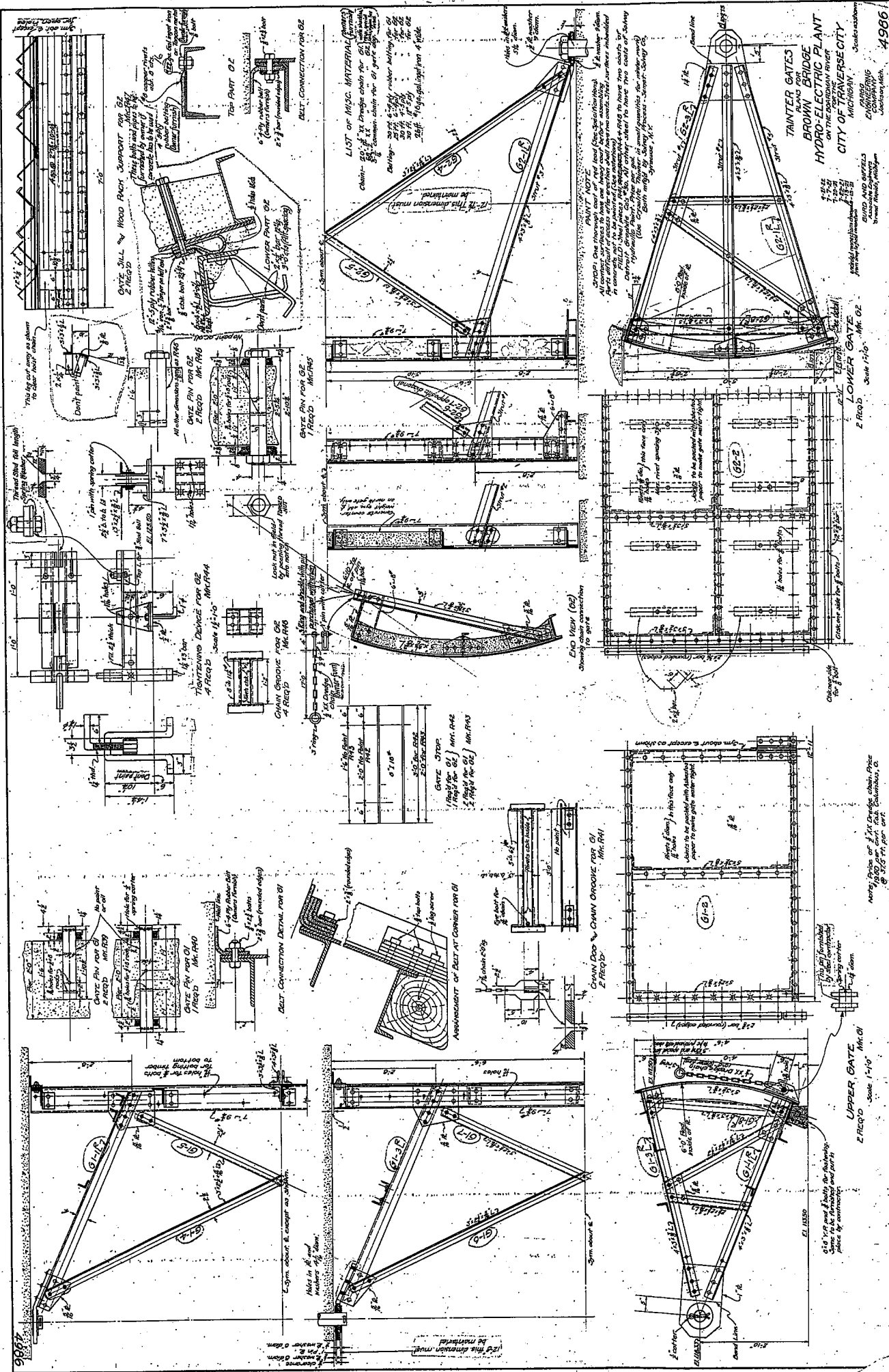
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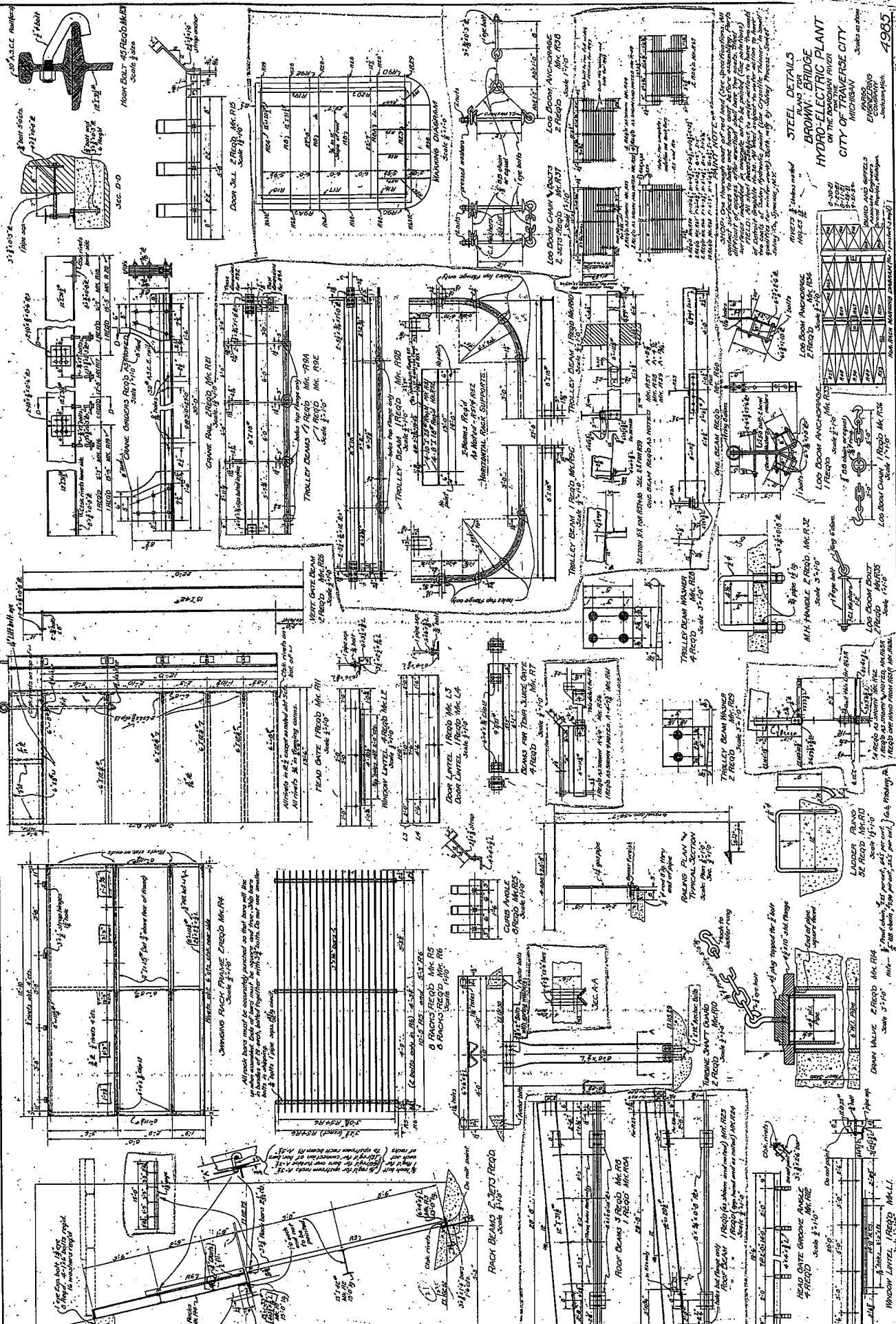
11014

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11



STEEL DETAILS
BROWN BRIDGE
HYDRO-ELECTRIC PLANT
 ON THE BAYBURN RIVER
 CITY OF TRAFALGARE CITY
 MICHIGAN

Scale as shown
 ENGINEERING
 JACKSON, MICH.
 4985

SECTION 81 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1
 SECTION 82 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 83 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 84 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 85 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 86 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 87 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 88 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 89 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 90 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 91 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 92 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 93 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 94 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 95 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 96 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 97 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 98 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 99 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

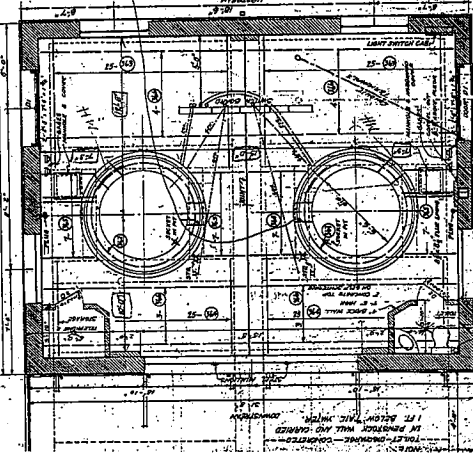
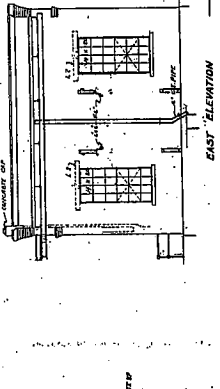
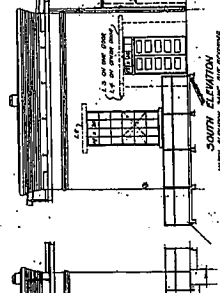
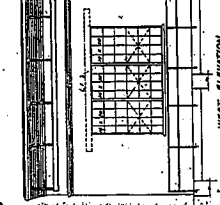
SECTION 100 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 101 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

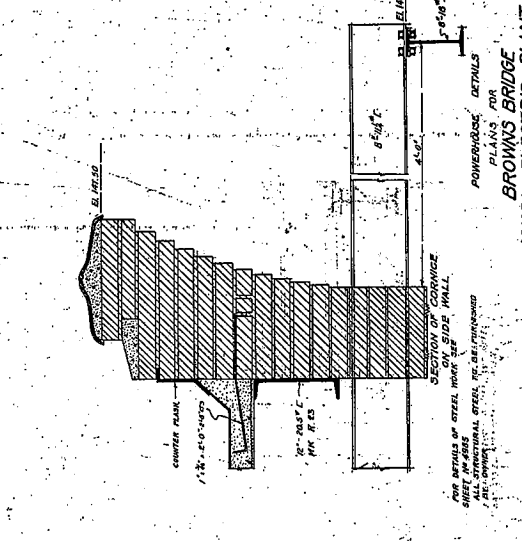
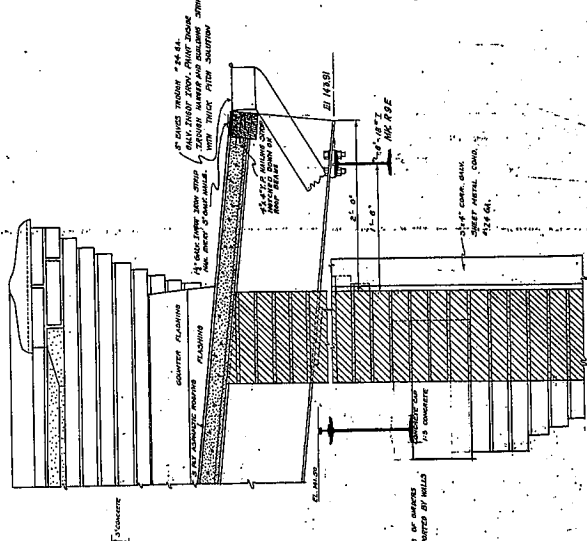
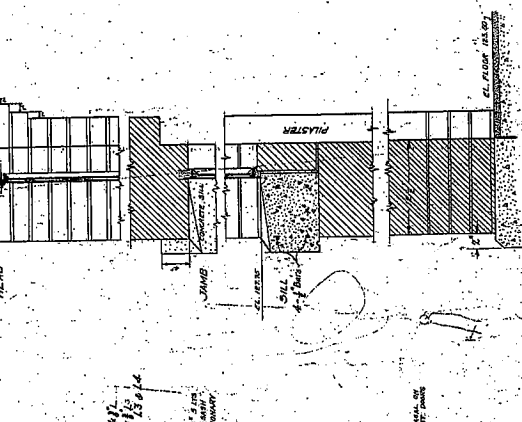
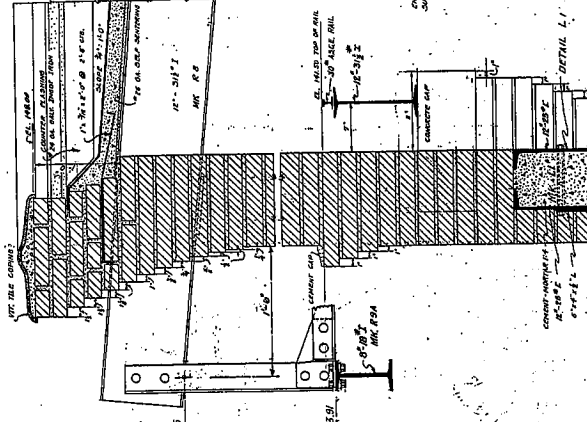
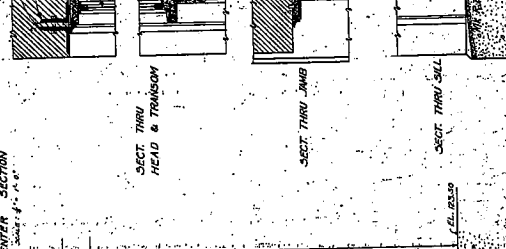
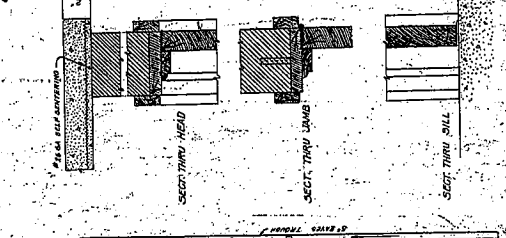
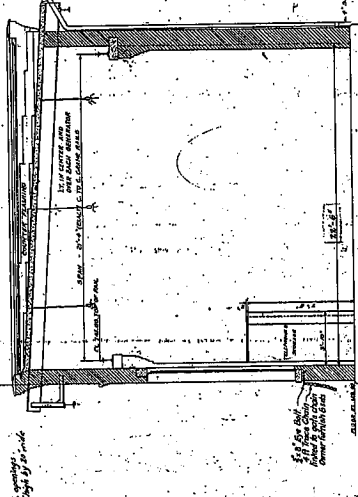
SECTION 102 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

SECTION 103 FOR BEAMS 32, 31, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1

Notes: 1. All steel beams must be connected to concrete as shown on this drawing. 2. All steel beams must be connected to concrete as shown on this drawing. 3. All steel beams must be connected to concrete as shown on this drawing. 4. All steel beams must be connected to concrete as shown on this drawing. 5. All steel beams must be connected to concrete as shown on this drawing. 6. All steel beams must be connected to concrete as shown on this drawing. 7. All steel beams must be connected to concrete as shown on this drawing. 8. All steel beams must be connected to concrete as shown on this drawing. 9. All steel beams must be connected to concrete as shown on this drawing. 10. All steel beams must be connected to concrete as shown on this drawing.



NOTE: SEE OTHER SHEETS FOR DETAILS OF ROOF, FLOOR, AND WALLS. ALL DIMENSIONS SHOWN UNLESS OTHERWISE NOTED. MATERIALS TO BE USED AS SHOWN ON THE DRAWINGS UNLESS OTHERWISE NOTED.

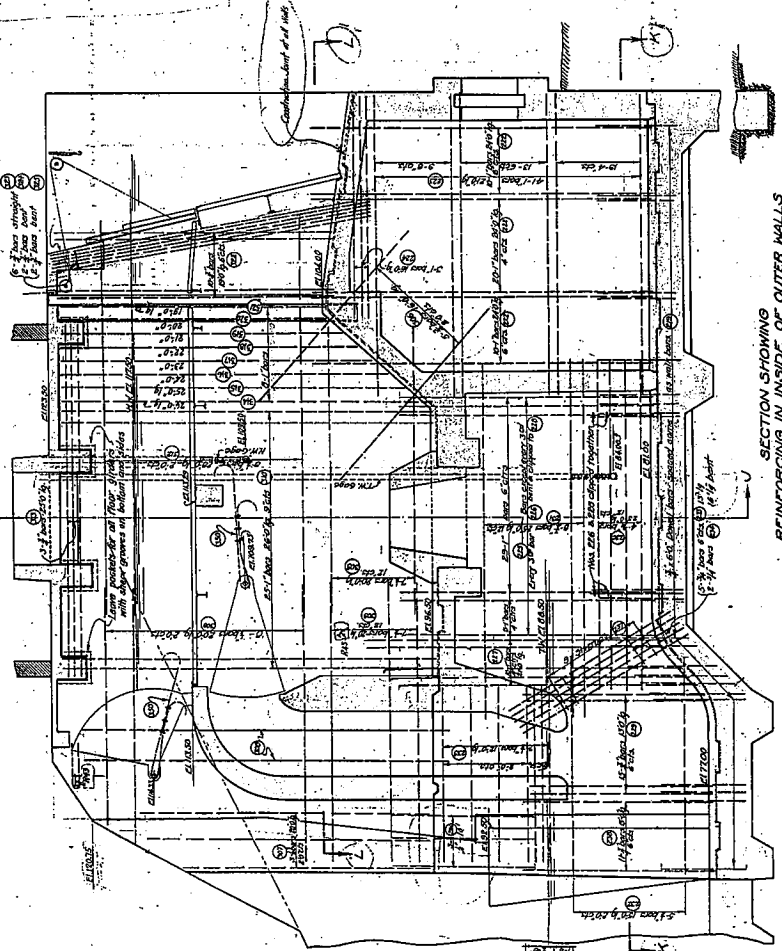
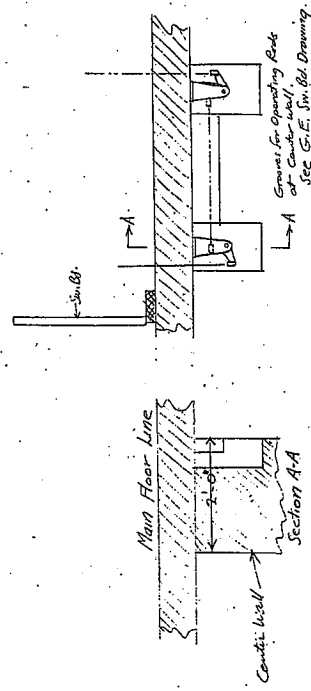


PLANS FOR
BROWNS BRIDGE
HYDRO-ELECTRIC PLANT
ON THE
BROWNS RIVER
CITY OF TRAVERSE CITY
MICHIGAN

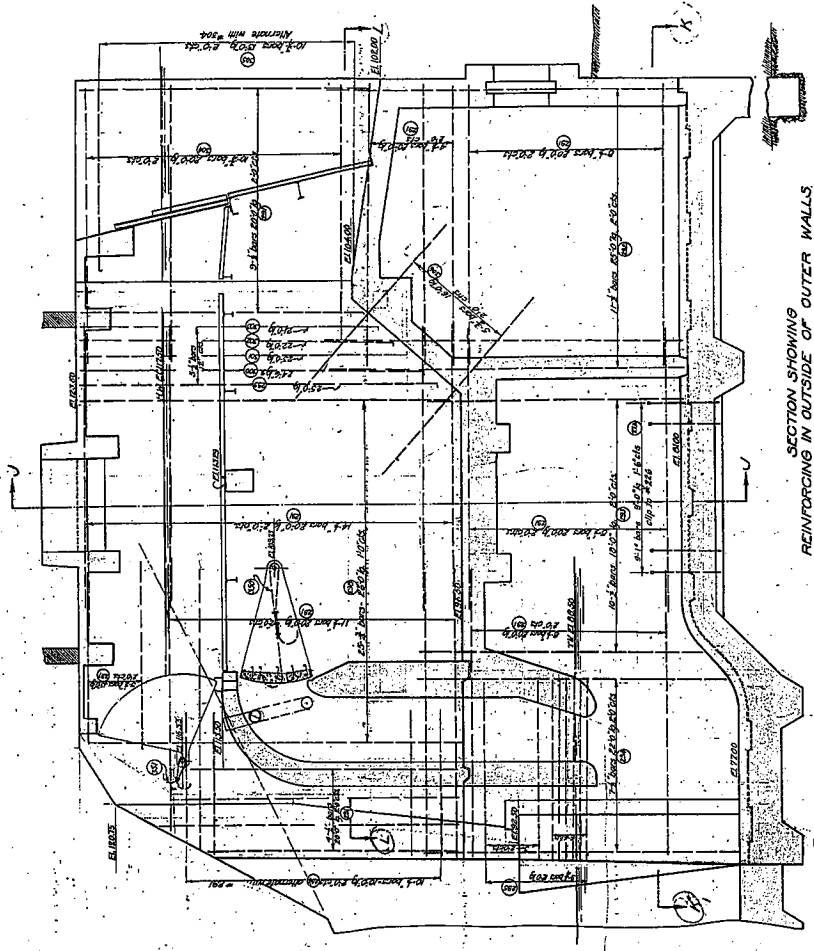
POWERHOUSE DETAILS
SHEET NO. 4984
SCALE: 1/4" = 1'-0"

BAR LIST

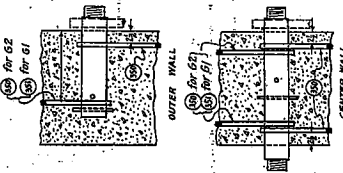
BAR NO	SIZE	LOCATION
101	1/2"	Columns
102	3/4"	Columns
103	1"	Columns
104	1 1/4"	Columns
105	1 1/2"	Columns
106	2"	Columns
107	2 1/2"	Columns
108	3"	Columns
109	3 1/2"	Columns
110	4"	Columns
111	4 1/2"	Columns
112	5"	Columns
113	5 1/2"	Columns
114	6"	Columns
115	6 1/2"	Columns
116	7"	Columns
117	7 1/2"	Columns
118	8"	Columns
119	8 1/2"	Columns
120	9"	Columns
121	9 1/2"	Columns
122	10"	Columns
123	10 1/2"	Columns
124	11"	Columns
125	11 1/2"	Columns
126	12"	Columns
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523	210 1/2"	Columns
524	211"	Columns
525	211 1/2"	Columns
526	212"	Columns
527	212 1/2"	Columns
528	213"	Columns
529	213 1/2"	Columns
530	214"	Columns
531	214 1/2"	Columns
532	215"	



SECTION SHOWING REINFORCING IN INSIDE OF OUTER WALLS



SECTION SHOWING REINFORCING IN OUTSIDE OF OUTER WALLS



REINFORCING DETAIL OF REINFORCING AROUND GATE PILING Scale 1/4\"/>

PENSTOCK AND SPILLWAY DETAILS
 SHOWING REINFORCING IN OUTER WALLS
 PLANS FOR
BROWNS BRIDGE
HYDRO ELECTRIC PLANT
 ON THE GRAND TRAVERSE RIVER
 CITY OF TRAVERSE CITY
 MICHIGAN

And a copy
 prepared
 Grand Rapids, Mich

August 9, 1921
 Drawing
 October 2, 1921
 Location, Mich

Scale 1/4\"/>

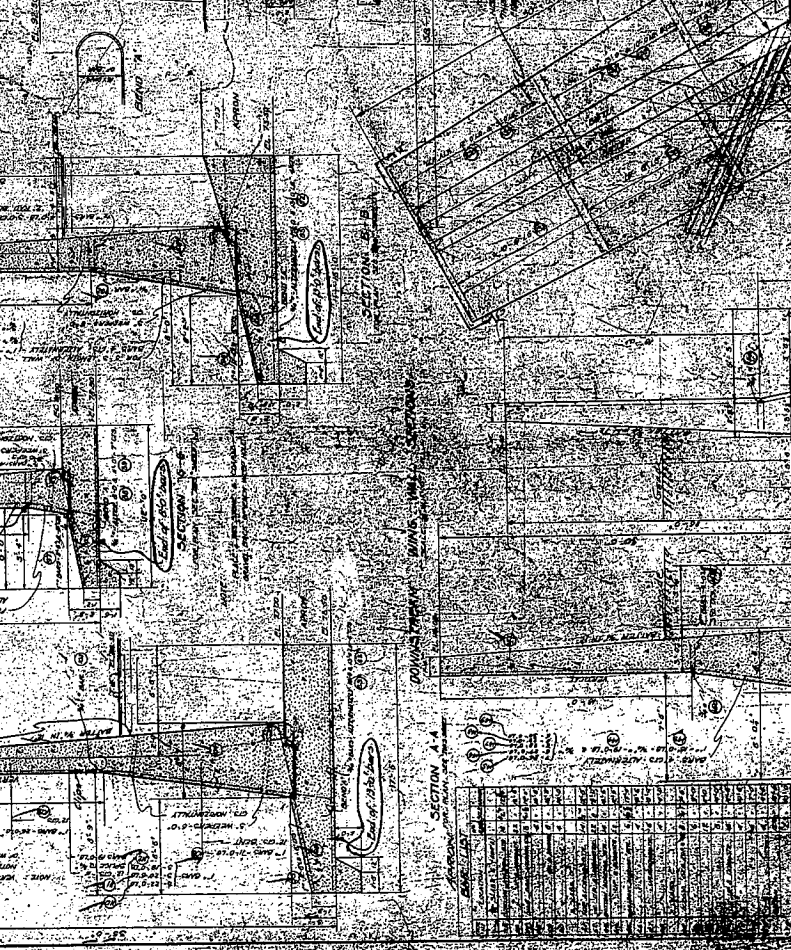
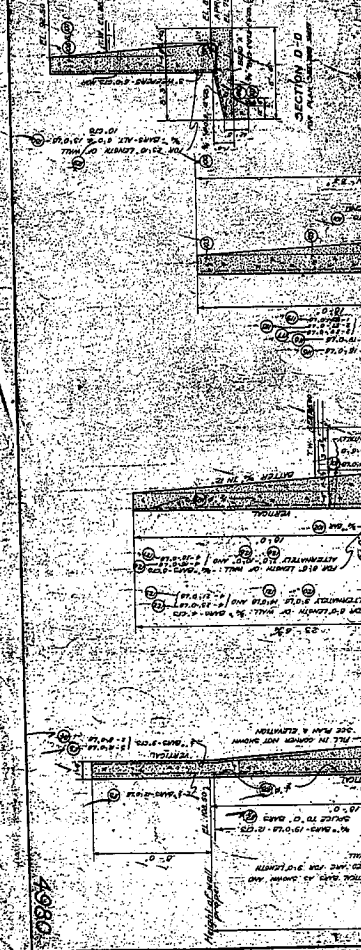
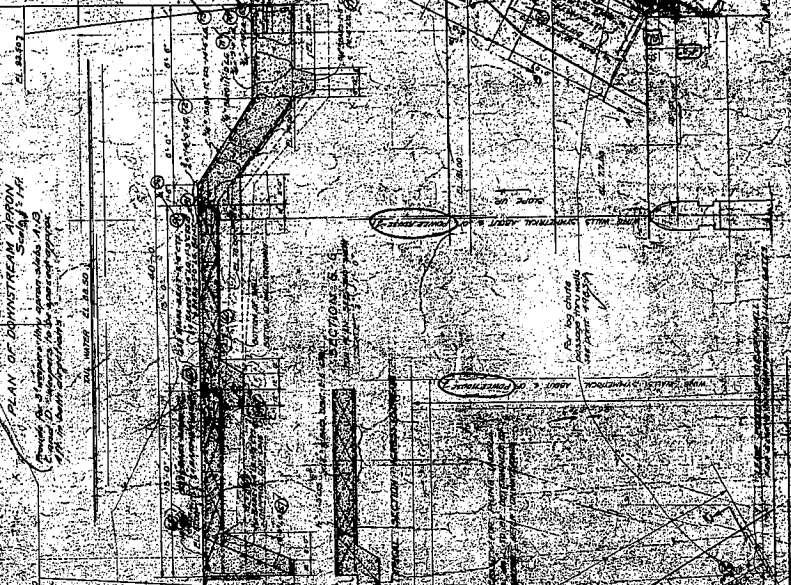
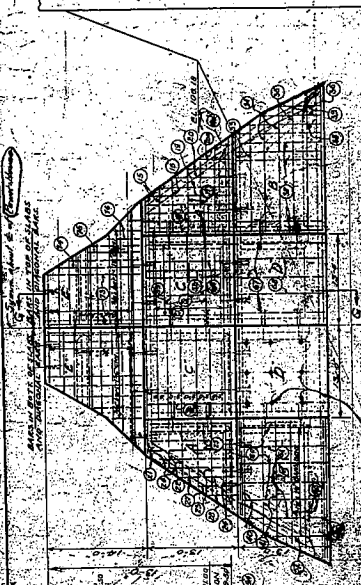
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BAR LIST - HONG MILLS

NO.	DESCRIPTION	QUANTITY	UNIT	REMARKS
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