Submitted to:
Milwaukee, County Department of Public Works
RCM Site Development Section
Milwaukee, WI

Submitted by:
AECOM
Vernon Hills, IL
Project No. 60159452
September 28,2010

Structural Engineering Evaluation of the Estabrook Park Dam

Milwaukee River, Milwaukee, WI

Estabrook Dam Aerial View, with Features

AECOM

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September 28, 2010

Mr. Karl D. Stave, P.E.
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Milwaukee, County Department of Public Works
City Campus – Room 216
2711 West Wells Street
Milwaukee, WI 53208

RE: Structural Engineering Evaluation of the Estabrook Park Dam Located on the Milwaukee River, Milwaukee, Wisconsin – AECOM Project No. 60159452

Dear Mr. Stave:

We have completed our structural evaluation of the current physical condition of Estabrook Dam on the Milwaukee River. The attached report details the results of the condition evaluation, both gated spillway and overflow spillway stability analyses, and provides repair recommendations identified as a result of our evaluation. We also discuss structural maintenance of the facility and provide our opinion of probable costs of recommendations to bring the facility back to good structural condition. Once rehabilitation is completed, we expect the facility to function for 20 more years, with annual operations and maintenance.

Our scope of work consisted of performing a visual inspection, non-destructive testing and material testing of the gated spillway structure, as well as a visual inspection of the serpentine overflow section and the tooth-like icebreakers upstream of the gated spillway. Our main directive was to update STS Consultant's 2006 structural evaluation report (STS project No. 5-87996) dated September 8, 2006. Of primary concern is the structural stability of the dam components with an estimate of costs to bring the dam structures back to structurally stable condition.

Per the Wisconsin DNR Order to Repair or Abandon the Dam, you may also consider demolishing the dam structures and restoring the river to open flow conditions. We have not evaluated that option in this scope of services, but could assist you, should you desire to investigate that option.

If you have questions regarding the attached report, please call Steve Elver at 847-279-2476.

Respectfully,

Charles D. Dean, P.E.
Project Engineer

Steven A. Elver, S.E., P.E.
Principal Engineer

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cc: Don Pirrung – AECOM

Certification Page

I, Steven A. Elver, hereby certify that I am a registered professional engineer in the State of Wisconsin, registered in accordance with the requirements of ch. A-E4, Wis. Adm. Code; that this document has been prepared in accordance with the Rules of Professional Conduct in ch. A-E8, Wis. Adm. Code; and that, to the best of my knowledge, all information contained in this document is correct and the document was prepared in compliance with all applicable requirements in Chapters 31 and NR333 Wis. Adm. Code.

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State of Wisconsin
Department of Regulation and Licensing

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1.0 Executive Summary

Estabrook Park Dam is located on the Milwaukee River in the northeast corner of the City of Milwaukee, Milwaukee County, Wisconsin. The dam features were built in the late 1930's, with recent repairs performed in 1989. The facility consists of three main structural features:

- A gated spillway, approximately 220 feet long with eleven vertical slide gates,
- A serpentine overflow spillway, approximately 562 feet long, with wood stoplog section, and
- A line of 24 triangular shaped concrete ice guards or icebreakers, upstream of the gated spillway.

With spillway gates closed, Estabrook Dam raises the impoundment water level by approximately five feet. The gates have typically been closed from May to November each year, raising the impoundment across the summer. The gates have been opened across the winter months, draining the reservoir. This keeps ice from overloading the gated spillway structure. Currently, the gates are open and the reservoir is drained, per a WDNR directive for dam safety. If repairs are made to the dam, including reinforcing the dam for ice loading, then the reservoir could be filled year-round.

AECOM Technical Services, Inc. (AECOM) was contracted by Milwaukee County Department of Transportation and Public Works to evaluate the current structural condition of Estabrook Park Dam features. This evaluation is an update to a prior structural evaluation performed by STS Consultants, Ltd. (STS) approximately four years ago. That report is numbered 5-87996 and dated September 8, 2006. Note that STS joined AECOM in 2007, so while the company name has changed, it is essentially the same entity contracted for both studies.

This 2010 report makes numerous references to the 2006 STS report. The 2010 Report repeats the main features of the 2006 Report for ease of understanding, but does not repeat all details. Both the 2006 report and this 2010 update should be read to understand the structural evaluation findings behind the repair recommendations given within.

This 2010 report discusses the current structural condition of the dam components. It then discusses requirements for repairing deteriorated components and adding structural reinforcing to the gated spillway for ice loading conditions. It also recommends continued Operations and Maintenance to keep the structures in good condition for the next 20 years. Alternately, the County may decide to abandon the dam. Results of this structural evaluation will be an important part of the decision process by Milwaukee County on which option, repair or abandonment, to pursue.

<u>The dam features remain in structurally fair to good condition</u> (with the exception of two failed icebreakers). No imminent failure of the dam is anticipated unless further concrete deterioration weakens the structure. A dive inspection was performed in 2006 to evaluate possible scouring and undercutting of the dam structure. The dam structures are founded on shallow bedrock. No scouring or undercutting was found.

Overall, the concrete and masonry structures are in weathered condition, typical for 70 year old concrete in a Wisconsin marine environment. The 2006 STS Report outlined several repair recommendations to keep the structure in good, well maintained condition. Those repair recommendations still apply in 2010,

with 4 more years of deterioration adding to the quantity of restoration required. We also added a requirement to anchor the upstream side of the gated spillway to bedrock with grouted ties. That allows for keeping the impoundment full across the winter without overloading the gated structure from lateral ice loading. A summary of the recommended repairs is provided in the following table.

Recommended Repairs

Estabrook Dam Feature	**Repair Type**
Gated Spillway	Remove and replace deteriorated concrete below water level at piers, +/-
Gated Spillway	Remove and replace deteriorated concrete above water level at piers, bridge deck and walls
Gated Spillway	Repair the expansion joints at bridge deck and walls
Gated Spillway	Seal top of bridge deck to reduce penetration of water into concrete
Gated Spillway	Properly prepare and paint vertical steel gates and grease/lube operating components
Gated Spillway	Add grouted tie down anchors (steel) into bedrock, upstream side of all piers, for winter ice loading if impoundment is not drained in the winter.
Shoreline	Add geofabric and riprap along eroding shoreline near gated spillway
Overflow Spillway	Repair/replace broken stoplogs and steel posts
Overflow Spillway	Patch deteriorated concrete at crest and tuckpoint joint between concrete and stone masonry facing
Icebreakers	Repair deteriorated concrete on tops and at water line. Replace failed icebreaker and replace one leg of second icebreaker.
Whole facility	Develop and implement a log and debris cleanup program for all three dam features. (Alternate - Consider new icebreaker design/layout to divert logs to north bank of river for ease of maintenance.)

2.0 Condition Evaluation

The project consists of three distinct sections – the gated spillway, the serpentine overflow spillway and the triangular icebreakers upstream of the gated spillway. These structures were given a preliminary inspection by Steven A. Elver, P.E., S.E., on June 25, 2010. Follow up inspections were made by Steve Elver on August 12 and September 1, 2010, after a large flood occurred on July 22, 2010.

2.1 Gated Spillway

The gated dam section was more thoroughly inspected by Charles D. Dean, P.E. (structural engineer) on Tuesday, July 6, 2010 and Thursday, July 8, 2010. In addition, on Thursday, July 8, 2010, a two-man non-destructive testing crew was on site to perform ultrasonic pulse velocity testing on selected piers. The weather conditions on both days were sunny with temperatures around 90 degrees Fahrenheit. At the time of our field inspections, the gates were in the up position and the water level flowing through the gated section was approximately 1.5 to 2.0 feet deep (Approximate Elevation +30.75). The orientation of the structure is described as left to right, as viewed from the upstream side of the dam in the direction of water flow.

Existing conditions of the structure were documented on copies of the 2006 STS Report Existing Condition Elevation Views. In this manner, we were able to determine if the current deterioration was the same as, or worse than, the conditions observed four years ago. These elevation views are included in Appendix A to this report.

2.1.1 Left Abutment

The left abutment walls were in overall fair condition. The walls at the left end near the pedestrian gate exhibited cracking, efflorescence and spalling. Efflorescence is a whitish deposit left on the surface from water flowing through cracks in the concrete and evaporating on the surface. Spalling is the cracking and falling off of the concrete surface from the concrete substrate behind. The spalling was more frequent near the ground line and at the top of the walls. Both abutment walls running left to right have cracks and spalls at the top of the walls and above stair treads on the inside face of the walls. Concrete disintegration, due to freeze thaw damage, was present at all of the stairs. The right side of the abutment pier exhibited widespread cracking and efflorescence, in addition to delaminations on the upstream side of the pier above Elevation 37.75 feet and spalling on the downstream side of the pier below Elevation 34.0 feet. Delaminations are defined as spalls that have not detached from the substrate yet.

2.1.2 Right Abutment

The right abutment walls were in overall fair condition with several areas of observed deterioration. The walls at the right end of the abutment near the pedestrian gate exhibited cracking, efflorescence and isolated spalling. Both abutment walls, running left to right, were cracked and spalled at the top of the walls. Concrete disintegration, due to freeze-thaw damage, was present at the top six stair nosings. The downstream right side of the abutment pier exhibited cracking with efflorescence above elevation 34.00. The left side of the abutment pier has exposed reinforcing steel below concrete patch spalls on the downstream side of the pier. Both exposed walls on the left side of the pier were cracked, delaminated and had efflorescence on the concrete surface. Map cracking and delaminations were observed at the top of the stairs.

2.1.3 Expansion Joints

Expansion joints were installed in the operating bridge over pier Nos. 1, 4, 8 and 11 (piers are numbered sequentially from left to right as shown on Figure 1 in Appendix A). They are called Joint Numbers 1, 2, 3 and 4, accordingly. Asphalt board type filler was placed in the expansion joints. The filler material was squeezed out of the joints at most locations. The board type joint filler was also in place in the bridge deck portion of the joint, but at two bridge deck expansion joints, it was missing.

2.1.4 Operating Bridge Walls

In general, the operating bridge walls were in fair condition. The operating bridge walls were rotated from the vertical position. The left wall section at Pier P1 and the right wall section at P11, extending from Elevation 37.75 to 49.00 feet, had approximately 1-inch outward displacement at the top relative to the adjacent wall. Similar conditions were observed at Piers P4 and P8, except the outward displacement ranged from 3/4 to 1-inch at the top of the adjacent walls.

2.1.4.1 Upstream Operating Bridge Wall

The upstream operating bridge wall was generally in fair condition with isolated areas of deterioration. Cracking was typically observed at the top and bottom of the walls. Vertical cracking, full height of the wall, was observed near Pier P2. Isolated delaminations, with or without exposed reinforcing, were observed on the inside face of the wall. The concrete was disintegrating at the top of the wall at Pier P1. Cracking, efflorescence and isolated areas of disintegration of concrete were observed on the downstream face of the wall at the right abutment.

2.1.4.2 Downstream Operating Bridge Wall

The downstream operating bridge wall was generally in fair condition with isolated areas of deterioration. Cracking was typically observed at the top and bottom of the walls. Vertical cracking, full height of the wall, was observed along the downstream face of the wall. Isolated delaminations, with or without exposed reinforcing, were observed on the inside face of the wall. The concrete was disintegrating at the top of the wall at Pier P1. Cracking, efflorescence and isolated areas of concrete disintegration were observed on the upstream face of the wall.

2.1.5 Operating Bridge Deck

The operating bridge deck appeared in overall fair to good condition, with isolated areas of topside and underside deterioration. The topside of the deck was surveyed by dragging a chain and tapping with a geologist's hammer. These "sounding methods" help identify areas of delaminated concrete, which sound duller than adjacent, sound areas. Isolated areas of delaminated or disintegrating concrete were identified on the topside of the bridge deck. The areas of distress were noted as follows:

- Left of expansion joint No. 4, Pier 11 (same as in 2006),
- Right of the expansion joint 1 at Pier 1 (includes spalling) and 3 feet to the right of the joint,
- At previous locations of gate hoist bases for gates 5 to 7, and
- Deck above Pier P5.

Cracking was observed on the top side of the deck near joint Nos. 2 and 4 and at isolated locations on the underside of the deck (viewed from below). The underside surface of the deck was wet at locations of through deck penetrations. Isolated spalling, with exposed reinforcing steel, was observed along the

downstream side from below. Refer to Field Notes in Appendix A for locations of distress observed on the underside of the deck.

2.1.6 Gate Hoist Blocks

The gate hoist blocks on the topside of the deck appeared in overall fair to good condition, with isolated blocks experiencing deterioration. The left block for Gate 1 was disintegrating with exposed reinforcing steel. The block at Gate 2 was spalled with exposed reinforcing steel. Cracking was observed in the blocks for Gates 5, 6 and 10.

2.1.7 Piers

The bridge deck and steel gates are supported by vertical concrete piers, numbered 1 to 11, left to right. The piers appeared in fair to good condition above elevation 37.75 and generally in poor condition below elevation 37.75. Full pool water level is at Elevation 36.00 Feet. Refer to Photographs in Appendix B and Figures 1 through 24 in Appendix A. The sides of the piers are labeled left and right as viewed from upstream. All reachable areas were acoustically sounded with a geologist's hammer. Widespread surface repairs below elevation 37.75 have separated from the original concrete substrate and were delaminated, spalled or cracked. More recently applied surface repair material (gunite) was soft and could be scratched easily with light pressure on a screw driver. Isolated concrete delamination was identified on certain vertical faces, approximately 2 to 3 feet above elevation 37.75. Vertical cracking and map cracking, with efflorescence, observed on the vertical sections of the piers. Map cracking is defined as random, interconnecting cracks as if roads on a map.

The visual and sounding survey was supplemented by limited destructive testing (concrete core sampling) and non-destructive testing (ultrasonic pulse velocity) to help determine the quality of concrete in the piers.

2.2 Limited Non-Destructive Testing of the Gated Spillway

2.2.1 Evaluation Procedure

The Ultrasonic Pulse Velocity (UPV) method was used to evaluate the quality of pier concrete, per ASTM Standard C-597. The velocity of an ultrasonic pulse through concrete is a function of the modulus and density of the material, and is, therefore, a guide to concrete quality. Where multiple measurements are performed at different locations on a concrete element, comparison of the data provides an assessment of concrete uniformity.

The UPV test equipment consists of two piezoelectric transducers, a transmitter and a receiver, connected to an electronic pulse generation and timing system. The two transducers are applied to the surface of the concrete. A control system generates an electrical impulse that causes the transmitter to emit a pulse of ultrasound that propagates through the concrete to the receiver. The control system measures the time taken for the pulse to travel through the concrete. If the distance between the two transducers is known, then the velocity of the pulse can be calculated.

There are three possible geometries for UPV measurements:

- Indirect transmission, where both transducers are on the same face of the element being tested, spaced some distance apart.

- Semi-direct transmission, where the two transducers are on adjoining faces of the element being tested, such as across a corner.
- Direct transmission, where the two transducers are on opposite, parallel faces of the element being tested.

We used the direct transmission method, with transmitter on one side of the pier and receiver on the other. This is a more accurate method than the indirect or semi-direct methods.

2.2.2 Test Locations

Six concrete piers were selected to be tested by UPV (P3, P6, P7, P8, P9 and P10). These locations were selected randomly among the piers without debris blocking access for our crew. A 1-foot by 1-foot grid was marked in a 4-foot by 4-foot area on the left and right face of each pier. Refer to Figure Nos. 2 and 3 for locations and grid pattern. We were only able to take UPV measurements at selected points within some grid patterns because of soft, delaminated surface repair material interfering with good contact of our probes.

Measurements were performed between the left and right face of the selected piers, using direct transmission through the piers.

2.2.3 UPV Test Results

The calculated UPV values are listed in Table D1, Appendix C. For ease of evaluation, the calculated UPV values have been summarized graphically.

A velocity of 10,000 ft/sec is approximately equivalent to 3,000 psi normal weight concrete. Original concrete compressive design strength for the piers is 3,000 pounds per square inch (psi), per the original construction documents circa 1939. Values obtained on site varied above and below this amount.

UPV field test results were inconsistent from point to point, mainly due to the irregular shotcrete surface and interior voids at many locations. Table 2, Appendix C, listed the locations and values where results could be obtained. Where results were obtained, the concrete varied from fair to good in quality. Mixing these readings with the locations where readings were not obtainable, the average pier varied from poor to good, depending on exact spot of inquiry. Results were inconsistent, suggesting a varying degree of deterioration within each grid tested.

2.3 Material Testing and Results

We extracted twelve (12), 3.5-inch nominal diameter, concrete cores from six gated spillway piers. These core samples were taken to our Vernon Hills, Illinois Material Testing Laboratory for concrete compressive strength testing and correlation to UPV field results. The core locations were chosen on the basis of variety from poor to good appearing concrete in accessible locations (no log debris). A set of two cores (one vertically and one horizontally) were extracted from six concrete piers (P1, P4, P7, P9 and P11) per American Society for Testing and Materials (ASTM) C-42. Refer to Figures 1 and 2 for location of cores. The concrete cores were photographed and properly prepared for compressive strength testing.

All concrete core samples came out less than whole, with inherent cracks and/or concrete deterioration present. Refer to Photographs in Appendix B. The concrete in core Nos. C-3 (V) and C-3A (H) mainly consisted of gunite concrete repair material. The remainder of the concrete in these two cores appeared

to be original concrete and came out as rubble. Full-depth and partial depth cracking was observed in Cores C-2A, C-5, C-6 and C-6A. This type of cracking is possibly caused by alkali-silica reactivity (ASR). ASR is the reaction between high-alkali cement, aggregate and water. The introduction and uptake of water by the alkali-silica reaction produces a gel like material around the aggregate which causes the concrete to swell. This generates expansive, tensile stresses that lead to the formation of fine cracks in the concrete. Another possible cause of the observed cracking is exposure to many cycles of freezing and thawing for concrete that most likely does not include air-entrainment.

We were able to perform concrete compressive strength testing (per ASTM C42) on 8 of the 12 cores extracted. The remaining four cores could not be tested because of short piece length due to cracking or deterioration. The following is a summary of the compressive strength results.

Table 1: Summary of Compressive Strength Testing Results

Core ID	Location (Pier No.)	Compressive Strength (psi)
C-1A(H)	P1	(rubble)
C-1(V)	P1	12,140
C-2A(H)	P3	5,440
C2(V)	P3	(cracked)
C3A(H)	P4	(rubble)
C-3(V)	P4	6,770
C-4(V)	P7	6,500
C-4A(H)	P7	7,830
C-5(V)	P9	4,190
C-5A(H)	P9	4,720
C6A(H)	P11	(cracked)
C-6(V)	P11	3,080

Notes:
(V) – Cores extracted vertically from topside of Pier
(H) – Cores extracted horizontally from vertical face of Pier

2.4 Gates

Gate Numbers 1 through 10, left to right, are equipped with electrically operated steel slide gates. These gates and pier gate slots were rehabilitated in 1989. The gates were in the up (open) position during our survey. The gates have a steel face plate connected to horizontal structural steel beams. The gates had widespread uniform surface corrosion. The gate guides and side seals appeared in fair to good condition, with surface corrosion on the bolts and channels of the guides. The gates were not operated as part of this structural evaluation.

2.5 Upstream and Downstream River Banks

There is minor erosion and undercutting along the riverbanks downstream of the gated spillway. There is some minor undercutting along the banks upstream of the gated spillway structure, to the icebreakers along the left and right banks.

2.6 Scour Evaluation

The dam features are founded on shallow bedrock. An underwater dive survey was performed on these structures as part of the 2006 Condition Survey. No undercutting of the structure or scour of the bedrock was found during the 2006 Survey. We did not perform an updated underwater dive survey as part of this 2010 Evaluation. We saw no evidence of undercutting or scour from above the water during our recent survey and do not expect that the bedrock has deteriorated noticeably since the 2006 survey.

2.7 Overflow Weir Spillway Section

The overflow spillway section is a serpentine shape approximately 562 feet long. There are approximately 87 feet of timber stoplogs in the overflow spillway, supported by built-up steel posts socketed into concrete. On the upstream side of the stoplog section only, loose masonry is placed rubble style on a 1 to 1 slope, with a 4 inch thick, reinforced concrete topping.

The spillway is a vertical wall concrete dam socketed and doweled into bedrock. The top two feet of the wall, upstream side, is tapered at a 45 degree angle. This angle aids in releasing any ice forces as expanding ice would slide up the wall instead. On the downstream side, the masonry is laid up with grout and creates an aesthetic facing for water flowing over the spillway. Per original construction drawings, there are 5 levels of reinforcing steel rods, spaced at 2 feet on center horizontally, to tie the masonry and concrete sections together.

There was a massive accumulation of woody debris and miscellaneous trash upstream of the overflow spillway. Most of this logjam washed downstream during a large rainstorm on July 22, 2010. Currently, there are no regular plans for removing woody debris and other objects from this overflow spillway area, which is hard to access from the north shore of the river. The south shore, adjacent to the overflow spillway, is owned by a private entity. We recommend the County study and implement measures to periodically remove this debris.

The recent flood of July 22, 2010 exposed the upstream face and crest of the overflow spillway, which aided in our visual evaluation. Areas of the overflow spillway needing rehabilitation are limited in area and do not extend into the sediment upstream of the crest.

The overflow spillway section appeared in generally good condition, with localized areas of deterioration. There has been some concrete crest deterioration and loss of masonry on the downstream face. The top row of horizontal reinforcing steel in the concrete crest is exposed in several locations. The horizontal joint between the top of the masonry and top of concrete has weathered, leaving an open joint. Weathering of masonry stone is particularly evident at the flashboard section, where flows are concentrated. However, the loss of masonry material is not considered sufficient to negatively impact the overall section stability.

The overflow spillway concrete section is socketed into bedrock a minimum of 20 inches, with doweled-in reinforcing steel into bedrock. The dowels are 1 ¼ inch square bars, spaced at 14 inches on center and 5 foot six inches long (below the 20 inch trench socketing the concrete into the bedrock). The overflow section is broad with low hydraulic head. Stability of the section was not considered an issue in our 2006 Report, but has been analyzed in 2010 per WDNR request. See Section 3.0 of this report.

The flashboards were damaged during the July 2010 food, as seen in the photographs, Appendix B. Recent plans include removing the remaining flashboards for the winter and reconstructing them when the dam is rehabilitated. The concrete foundation at stoplog steel supports was observed in good condition, but may need spot rehabilitation within 10-20 years.

The foundation bedrock was visible along the entire length of the overflow spillway downstream toe. No scour holes or undermining of the overflow spillway was evident.

The right and left masonry abutments were in good condition. There is no evidence of visible seepage or erosion.

3.0 Stability Analysis of Spillways

3.1 Introduction

A structural stability analysis was performed for the gated spillway structure as part of the 2006 structural evaluation. As a result of that report, STS recommended use of grouted tie-down anchors in the upstream pier extensions. This was due to inadequate Factor of Safety under full impoundment with full ice loading, a condition that would exist in only the harshest winters with the pond full. The overflow spillway was not analyzed as part of the 2006 report, since it was deemed stable by engineering judgement. Per the WDNR Order, we have analyzed the overflow spillway at this time and include the results below.

3.2 Methodology

The stability of both the gated spillway and the overflow spillway has been evaluated using the standard shear friction factor of safety method. We followed the same procedures used in the 2006 Evaluation, namely the U. S. Army Corps of Engineers Guideline EM 1110-2-2200, Gravity Dam Design, as well as the Federal Energy Regulatory Commission (FERC) Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 3 – Gravity Dams. A conservative, two-dimensional analysis was performed for both structures.

3.3 Material Properties

The same material properties were used for this evaluation as was used in the 2006 report for the Gated Spillway Evaluation. For the overflow spillway evaluation, we also used masonry density at 120 pcf.

Table 2-1
Summary of Material Properties for Stability Analysis of the Gated Spillway

Analysis Properties	Details
Concrete Unit Weight	150 pcf
Concrete Compressive Strength	3,000 psi
Concrete Tensile Strength	150 psi
Concrete Shear Strength	109 psi
Allowable Bedrock Contact Pressure	420 psi
Uplift Pressure	Linear between full headwater/tail water pressure

3.4 Key Lateral Dimensions

Key dimension for the dam section were taken from project drawings.

3.5 Summary Table Computed Factors of Safety and Minimum Required Values

Computations for each loading case are included in Appendix D. The results are summarized as follows:

Table 2-2 Summary of Structure Stability Analyses – 2010 Update

Section/Case Description	Sliding Factor of Safety		Overturning Resultant w/in Middle Third or Middle Half?
	Required*	Computed	
GATED SPILLWAY WITH 80 KIP TIE DOWN ANCHORS IN PIERS			
1. Normal Pool	2.0	6.02	Yes – O K.
2a. Normal Pool with Ice**	1.7	1.7	Yes – O K
OVERFLOW SPILLWAY			
1. Normal Pool	2.0	***	***
2a. Normal Pool with Ice	1.7	>8.3	Yes

* Minimum recommended factor safety, U.S. Army Corps of Engineers (EM 1110-2-2200, 1995)

** 5000 lb/lf ice loading assumed per FERC Guidelines for Gravity Dams (2002)

*** By Inspection, Load Case 2a controls overflow spillway analysis. Load Case 1 stable.

3.6 Conclusions

The existing gated spillway section is structurally adequate under normal pool conditions, unless ice loading is added. Currently, the pool is drained and flows freely through the winter months, so it remains stable with that operating requirement. To keep the pool full during the winter months, structural tie down reinforcing will be required to keep acceptable Factors of Safety for that higher loading condition. One structural tie down will be placed in each pier, upstream extension, approximately 2 feet from the face of the vertical dam wall. The tie downs will be anchored into bedrock deep enough to provide 80,000 pounds (80 kips) of hold down force. Exact design to be determined based on rock testing and material specifications.

The overflow spillway is structurally stable under all loading conditions. Its serpentine shape aids in its structural stability beyond the two dimensional (straight line) analysis performed for this evaluation.

4.0 Repair Recommendations

In general, the structural elements of Estabrook Dam are in fair condition and, with preventive maintenance, can likely remain in-service for 20 years. In our opinion, there were no deficiencies identified during our study which were of immediate concern to dam safety. Furthermore, the dam is under a drawdown order, so it is not loaded at this time. If the County decides to repair the dam, we expect it would be done in stages, with the gated spillway and icebreakers done at one time, and the overflow spillway done before or after that. We expect that the river would be cofferdammed upstream and downstream of this construction to allow work in the dry.

Should the County decide to remove the dam instead of repairing it, we envision similar cofferdamming during demolition activities. The extent of structure removal would need to be discussed with WDNR. An evaluation of dam abandonment was not part of this scope of services, but is an option open to the County.

If the County decides to rehabilitate the dam, money should also be set aside for annual Operations and Maintenance of the facility. To achieve a 20 year life for the repaired structure will require regular maintenance, including debris removal and shoreline maintenance.

A number of repair items are recommended to ensure safe operations of the dam. Concrete repairs, which incorporate the majority of our repair recommendations, should be performed by experienced concrete repair contractors using industry standard repair techniques. Such repairs would follow guidelines of the International Concrete Repair Institute (ICRI) and the American Concrete Institute (ACI). A full set of repair documents, consisting of plans and specifications, will need to be prepared for this work.

Our repair recommendations are summarized by structure as follows.

4.1 Gated Spillway

4.1.1 Abutments

Most elements of both dam abutments are showing signs of their age, including walls, stairs and piers. Final repair recommendations will include removal of deteriorated concrete to sound substrate, then filling back with structural concrete patch material. Reinforcing dowels will be recommended to connect patch material to substrate for thicker patches.

4.1.2 Expansion Joints and Operating Bridge Walls

The rotation of the operating bridge walls appear to be driven by ingress of moisture and progressive freeze-thaw cycles between overlapping sections of the bridge wall. The magnitude of the wall rotation appears symmetrical about the left/right center of the dam, with greater rotation occurring on the wall that overlaps the expansion joint. The freeze-thaw phenomenon also explains why the asphalt board joint material was displaced out of the joint, toward the center of the operating bridge –path of least resistance is the side of the joint without concrete.

The expansion joints appear to be moving in excess of the joint's intended design (joint widths close 75% from design joint width in winter months). We expect, and as evidenced by the concrete disintegration around the joint, that the joint closes further in warmer months. Deterioration around the joint will continue as fractured concrete and debris that has fallen into the joint in the cooler months serves as a mechanism to fracture "sound" concrete around the joint.

AECOM recommends that the County replace the joint material in all expansion joints, implement measures to prevent water ingress into the joints by sealing the joints and monitor rotation of the walls at expansion joints. Expansion Joint Number 1 should be repaired and possibly retrofitted with a joint system capable of accommodating all the expansion and contraction from the leftmost 46.25 feet of the operation bridge.

We also recommend that the County address locations of the bridge wall cracking, disintegration and spalling by implementing routing and sealing of cracks and partial depth repair of spalls on the spillway bridge walls to prevent further deterioration.

4.1.3 Operating Bridge Deck

The operating bridge deck between the right abutment and pier P1 is severely deteriorated, experiencing cracking, delaminations, and disintegration. The topside deterioration does not appear to have advanced to the bottom of the deck. However, if left unattended, the deterioration will likely manifest through the full thickness of the deck.

AECOM recommends that the County repair the concrete deterioration by performing partial depth repairs of the operation deck between the right abutment and Pier 1 and at isolated deteriorated areas across the remainder of the deck.

4.1.4 Piers

With the exception of Pier P1 and Pier 11, the portion of the piers above elevation 37.75 appear to be in overall fair condition and may not need extensive patching, based on this condition assessment. Below elevation 37.75, the majority of the piers appear to have a combination of failed surface repairs, cracking, spalling and other decay.

The cracking of an 8-inch wide portion of the pier between elevations 36.75 and 38.25 feet at Piers P1 through P11 does not likely impact the strength and stability of the dam. It is unlikely that the cracks are caused by hydrostatic or ice loading on the dam, since the 8-inch wide element is above the normal pool level. The structure above the normal pool level serves to support the gate frames and operating deck. These cracks should be properly routed and sealed to prevent water ingress and deterioration.

4.1.5 Gates and Hoists

The gates appeared in overall fair condition for their age. We recommend the County install a pair of adequately sized drain holes in the web of the steel horizontal beams at each gate. While the gate skin plates and guide channels have a layer of surface rust, we did not observe significant loss of cross sectional areas.

AECOM recommends that the gates and guide channels be abrasively grit blasted and re-painted with water-based urethane paint suitable for this application. No existing paint is visible on the gate surface so lead paint control should not be an issue. Abrasive grit blasting and painting can probably be performed most economically on the field as opposed to removal to an off-site shop.

4.1.6 Icebreakers

The top of the tripod shaped concrete structures, as well as the legs at the low, winter water line, have deteriorated due to freeze-thaw action. All deteriorated locations should be properly chipped to sound substrate and properly filled back to original dimensions with properly specified concrete patching material. The collapsed tripod and the collapsed leg of the second tripod should be rebuilt, giving special care to check the condition of the reinforcing bars doweled into the bedrock for likely reuse. Loose steel angles on the edges of the tripod legs should be repaired or replaced during this project.

4.2 Overflow Spillway

4.2.1 Flashboard Section

The stop logs and vertical steel support posts have been recently been removed by County Park Department employees for the winter. We recommend these stoplogs be kept out of the spillway until the rehabilitation project is complete. The vertical steel supports for the stoplogs should be checked for deterioration and primed/painted as deemed necessary.

4.2.2 Regular Overflow Section

The top of the concrete wall in the overflow spillway shows signs of localized spalling, with exposed reinforcing steel showing. These areas should be properly prepared and patched with concrete to approximately original dimensions, aiming to provide proper concrete cover thickness over the reinforcing steel.

The overflow dam masonry wall is generally in good condition, but needs spot tuckpointing in areas. There is mortar missing from the joint between the masonry and the concrete wall – probably due to ingress of water and progressive freezing and thawing over time. This joint should be tuckpointed to reduce ingress of water.

We also recommend the regular removal of the accumulated woody debris upstream of the overflow section. This will be part of the recommendations for sediment removal, a simultaneous project to the recommended dam repair. After initially removing the remaining debris at this time, a program and budget should be implemented to remove new debris on a periodic basis.

4.3 Slope Protection

There is minor erosion and undercutting along the right river bank downstream of the gated spillway. There is also some minor undercutting along the banks just upstream of the gated spillway along the left and right banks. These areas should be properly prepared, covered with a suitable geotextile, then filled in with suitable sized riprap to prevent erosion.

Trees and shrubs should be removed from the island between the dam sections and along the shore near the dam, to prevent erosion when they fall over and uproot. Proper riprap should be placed at the normal pool water line.

Consideration should be given for access above and below the dam for a canoe portage as well as for debris cleanup. Proper materials should be specified for these paths / launches.

4.4 Dam Abandonment Option

Although not addressed directly in this report, the WDNR Order for Drawdown allows for an option to abandon the dam. The amount of existing structure to remove for dam abandonment and the amount of shoreline restoration will need to be agreed upon by the dam owner and regulatory agencies (WDNR). The cost of dam abandonment could be evaluated once the extent of work is better known.

5.0 Opinion of Probable Construction Cost

AECOM Cost Estimate - Estabrook Park Dam
Structural Repair Option - $1,500,000 Budget

9/28/2010 Note: This estimate does not include sediment removal, which is covered in a separate, environmental cost estimate.

Opinion of Probable Structural Project Costs

Client: Milwaukee County DPW
Address: 2711 West Wells St, Milwaukee, WI

	Description	Units	Unit Cost	Quantity	Total Cost	Comments
1.00	General					
1.1	Mobilization / Demobilization	ls	$35,000.00	1	$35,000	segmental barge, small crane, incidental equipment
1.2	Erosion Control	ls	$4,000.00	1	$4,000	silt fence and turbidity barriers
1.3	Site Restoration	ls	$15,000.00	1	$15,000	repair pavements, grading, seeding and mulching
1.4	Diversion of water	ls	$25,000.00	1	$25,000	Porta-dam installation during pier repairs, with stoplogs removed on overflow spillway
2.00	Gated Spillway - Concrete Repairs					
2.01	Concrete surface repairs - abutments and stairs	sf	$75.00	700	$52,500	reconstruct stairs, complete abutment surface reconstruction
2.02	Concrete surface repairs - bridge deck and walls	sf	$75.00	500	$37,500	partial depth repair of deck, partial to full depth repair of walls, sawcut closed expansion joints and replace joint material
2.03	Pier reconstruction - below el. 37.75	sf	$90.00	2800	$252,000	complete surface reconstruction below el. 37.75', all 11 piers
2.04	Concrete surface repairs - pier above el. 37.75	sf	$75.00	800	$60,000	partial depth repair above el. 37.75 as needed, all 11 piers
2.05	Install grouted tie down anchors in upstream piers, for stability with ice loading at full pool	ls	$300,000.00	1	$300,000	Includes 11 tie down anchors into bedrock, with mobilization
3.00	Gated Spillway - Gate Repairs					
3.01	Prepare and paint slide gates	ea	$3,500.00	10	$35,000	sandblast, prime and paint all 10 slide gates in place
3.02	Misc. repairs to gates, guides and seals	ls	$10,000.00	1	$10,000	as needed based upon inspection after cleaning
4.00	Ice Breakers - Concrete Repairs					
4.01	Concrete surface repairs	ea	$3,500.00	24	$84,000	chip, install anchors, polymer modified concrete, incl. replacing 1 and 1/3 ice breakers
5.00	Overflow Spillway					
5.01	New flashboards	ls	$1,000.00	1	$1,000	4"x8"x7'4" timber, recently replaced with new ones - budget for stockpile of new ones
5.02	Repair / replace bent supports	ls	$2,000.00	1	$2,000	assume 4 to be repaired
5.03	Concrete surface repairs	sf	$75.00	500	$37,500	chipping, anchors, and polymer modified concrete repair of crest
6.00	Slope Protection					
6.01	Riprap	cy	$70.00	600	$42,000	24" layer of riprap, left and right banks u/s and d/s of gated spillway
6.02	Geotextile	sy	$3.00	1,600	$4,800	
7.00	Debris Removal	ls	$25,000.00	1	$25,000	upstream of gated spillway and ice breakers, not incl. environmental cleanup area
8.00	Miscellaneous					
8.01	Repair / replace handrails, fences, gates, etc.	ls	$5,000.00	1	$5,000	
8.02	Misc. Site Electrical Work	ls	$15,000.00	1	$15,000	Per Milwaukee County recommendation

Construction Sub-Total:		$1,042,300
Construction Contingency:	20%	$208,460
Construction Estimate:		$1,250,760

9.00 Engineering		
9.01 Engineering design		$115,000
9.02 Resident engineering and contract administration		$80,000
9.03 Prepare EAP and IOM plans		$15,000
Engineering Subtotal:		$210,000
Engineering Contingency:	15%	$31,500
Engineering Estimate:		$241,500

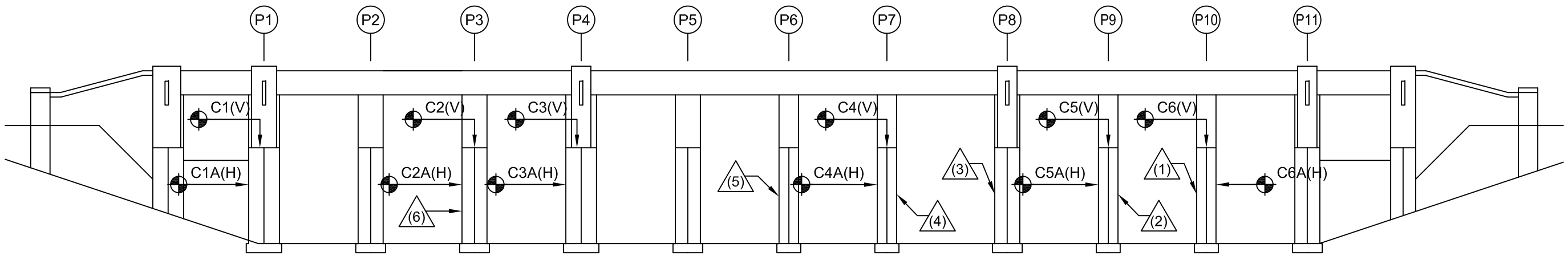
Total Project Cost Estimate: $1,492,260

Information presented on this sheet represents our opinion of probable costs in 2010 dollars, based upon previous unit rates and quantities updated from the 2006 STS Cost Estimate. Unit and lump-sum prices are based on costs for similar projects, our engineering judgment, and/or published cost data. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied.

Note: This rehabilitation estimate is to extend the life of the dam structures for 20 years, with some regular maintenance. This estimate is for repairs of existing structural components only, not including the nearby transformer building. Costs for replacement or remodeling of structures is not included.

Appendix A

Test Locations and Updated 2006 Condition Survey Figures



1 UPSTREAM DAM ELEVATION - TEST LOCATIONS

LEGEND:

APPROXIMATE CORE LOCATION AND NUMBER
H = HORIZONTAL CORE
V = VERTICAL CORE

UPV TEST LOCATION AND TEST NUMBER

P# PIER NUMBER

NOTE:

1. LOCATIONS WITH SHOTCRETE REPAIRS COULD NOT BE TESTED WITH UPV TEST (SOFT MATERIALS OR DEBONDED REPAIR MATERIAL)

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UPSTREAM DAM ELEVATION
TEST LOCATIONS
ESTABROOK PARK DAM
MILWAUKEE COUNTY, WISCONSIN

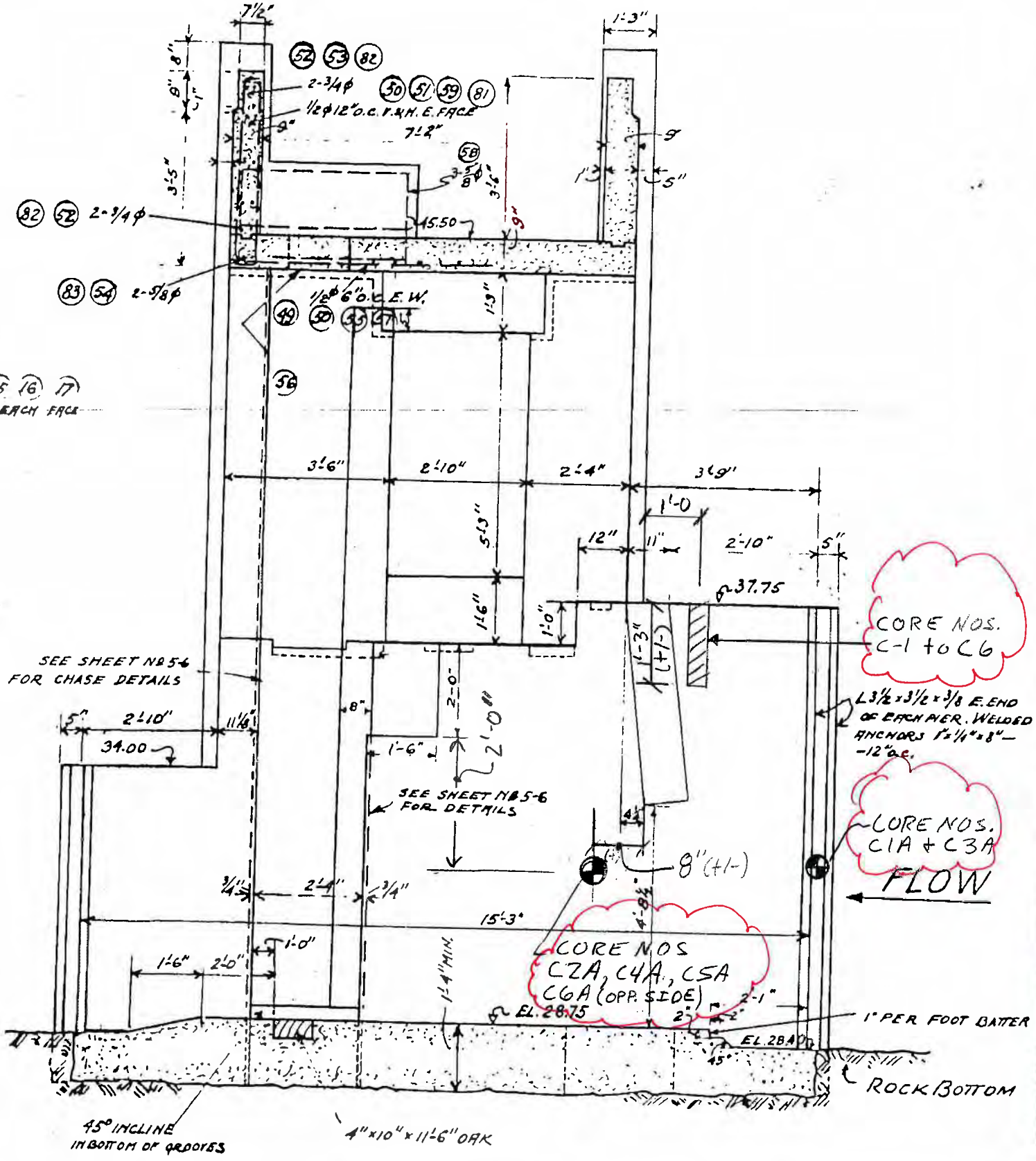
Drawn :	DJD	08/30/2010
Checked:	CDD	08/30/2010
Approved:	SAE	08/30/2010

PROJECT NUMBER 60159452

FIGURE NUMBER FIG-1

X:\PROJECTS\60159452\000_CAD\001_DRAWINGS\SHEETS\60159452_FIG-1.dwg; 8/31/2010 7:35:40 AM; DEARMAN, DANIEL; STS.stb

'ON.



CORE LOCATION DIAGRAM

NTS

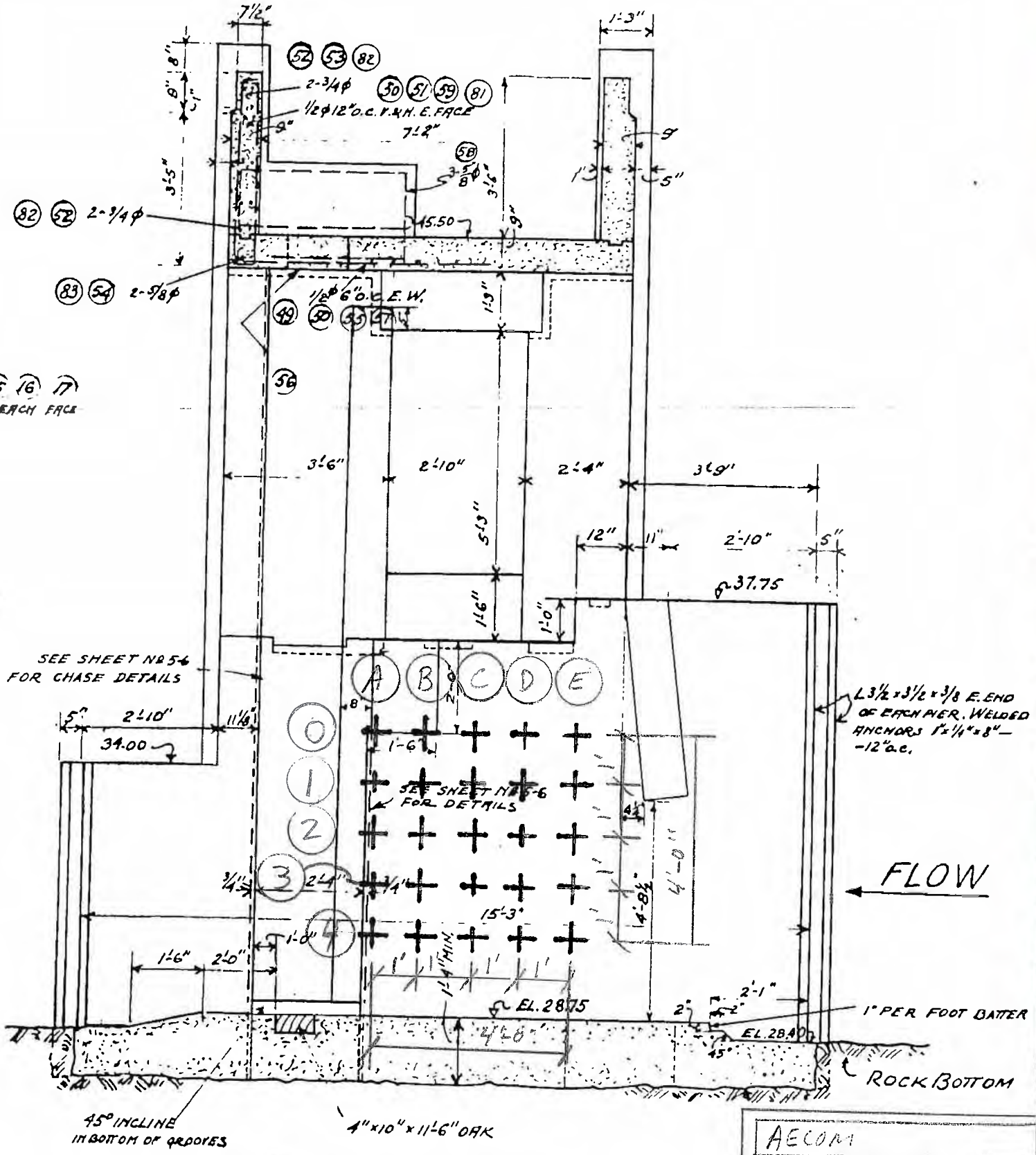
SECTION B-B

SCALE 3/8"=1'-0"

AECOM
PROJ. NO. 60159452

DRAWN BY: CDD 7/9/16
CHECKED BY: SAE

FIG. 2



UPV TESTING GRID

NTS

SECTION B-B

SCALE 3/8"=1'-0"

AECOM	
PROJECT NO. 60159452	
DRAWN BY:	CDD 7/9/2015
CHECK BY: SAE	FIG. 3

FIGURE 1

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: UPSTREAM CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDD	DATE: FEB. 3, 2006	CHECKED BY: SKE	DATE: Sept '10	CALC. NO.	REV. NO

REVISED July 2010

Reference: Structure Drawing No. 9006-4-8, (right half reverse image)

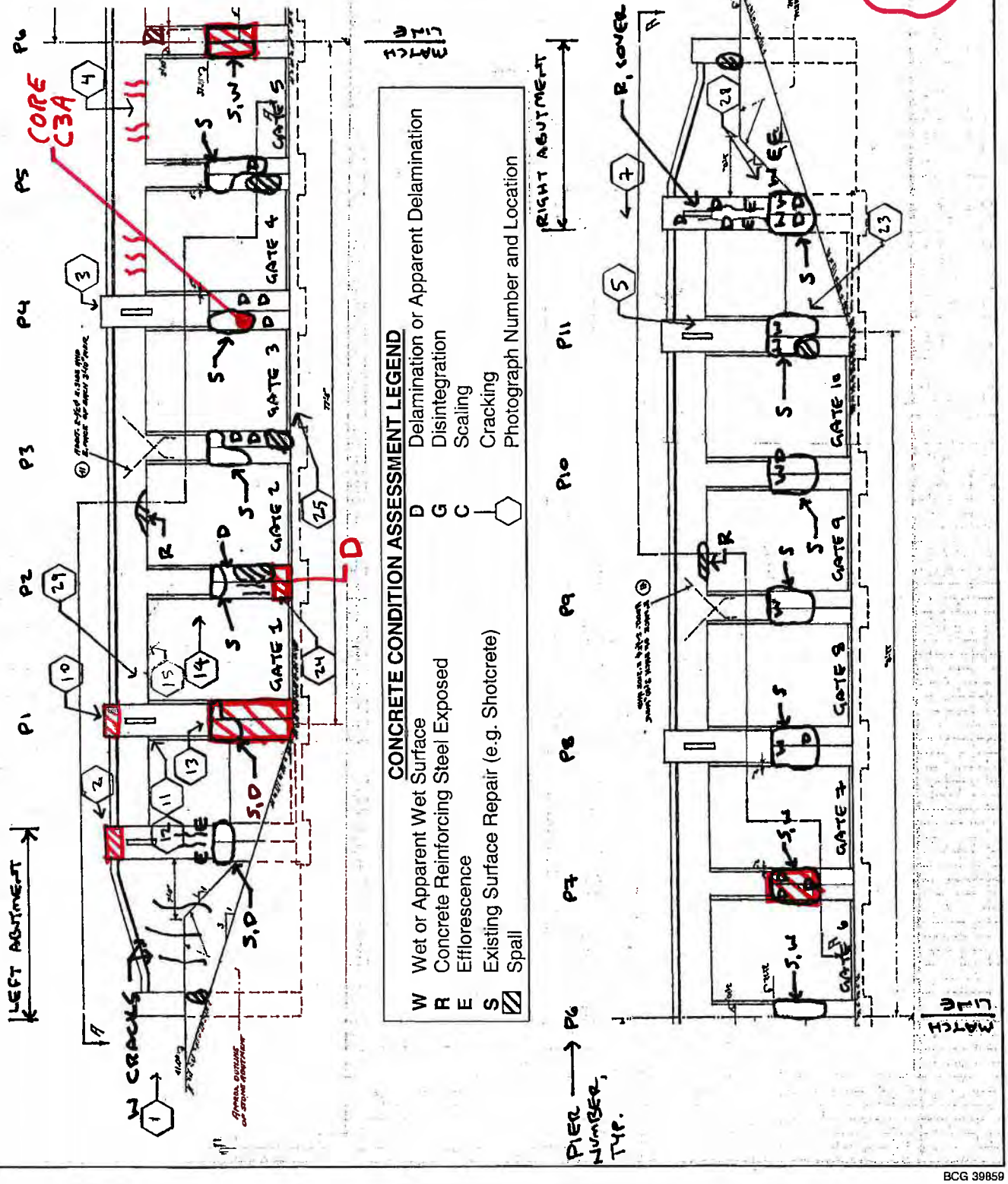


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FIGURE 2

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: DOWNSTREAM CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / GDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. 10	CALC. NO.	REV. NO

REVISED JULY 2010

Reference: Structure Drawing No. 9006-4-8, (left half reverse image)

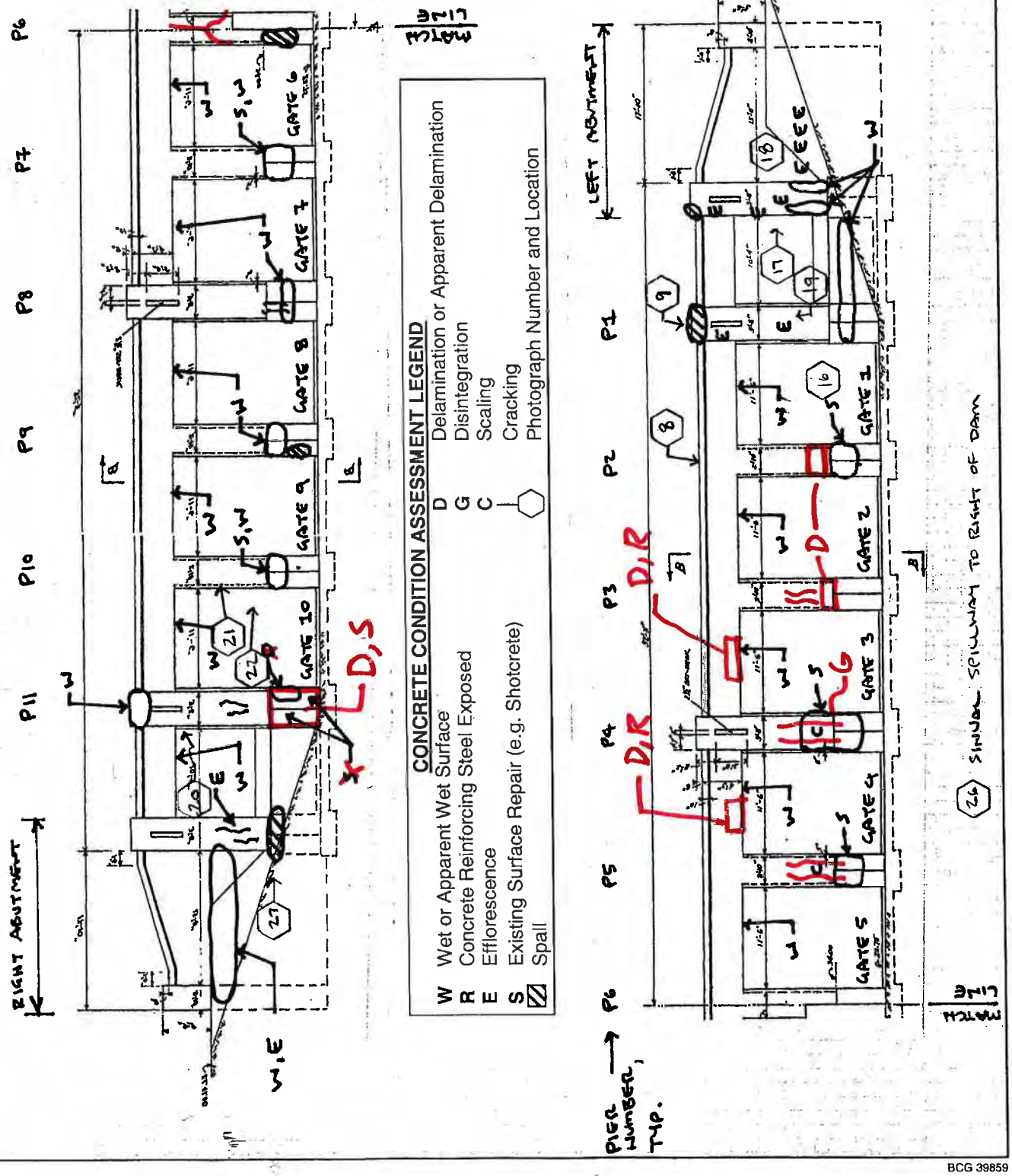

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FIGURE 3

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 1 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C

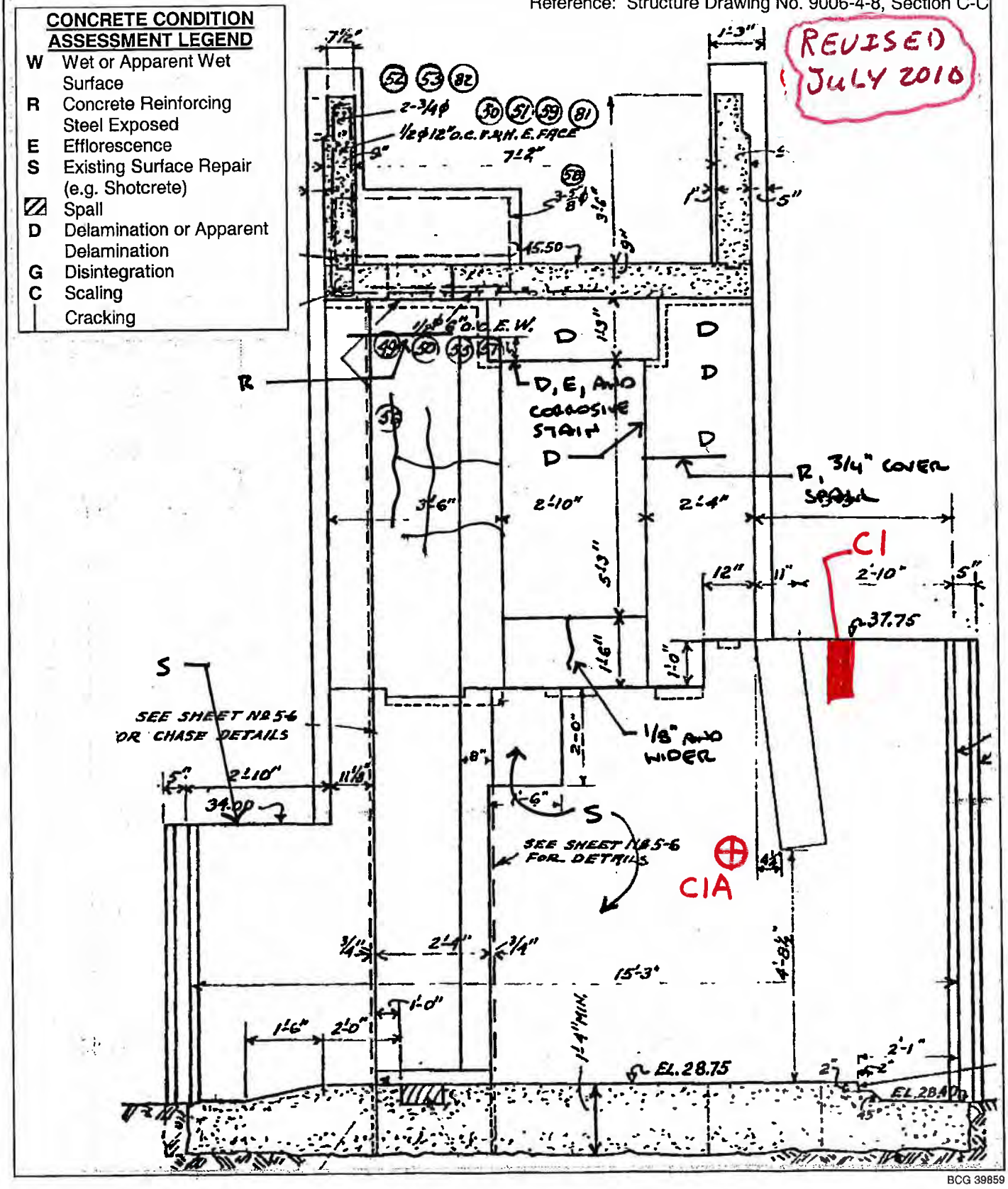

FIGURE 4

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 2 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / GDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. 10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C

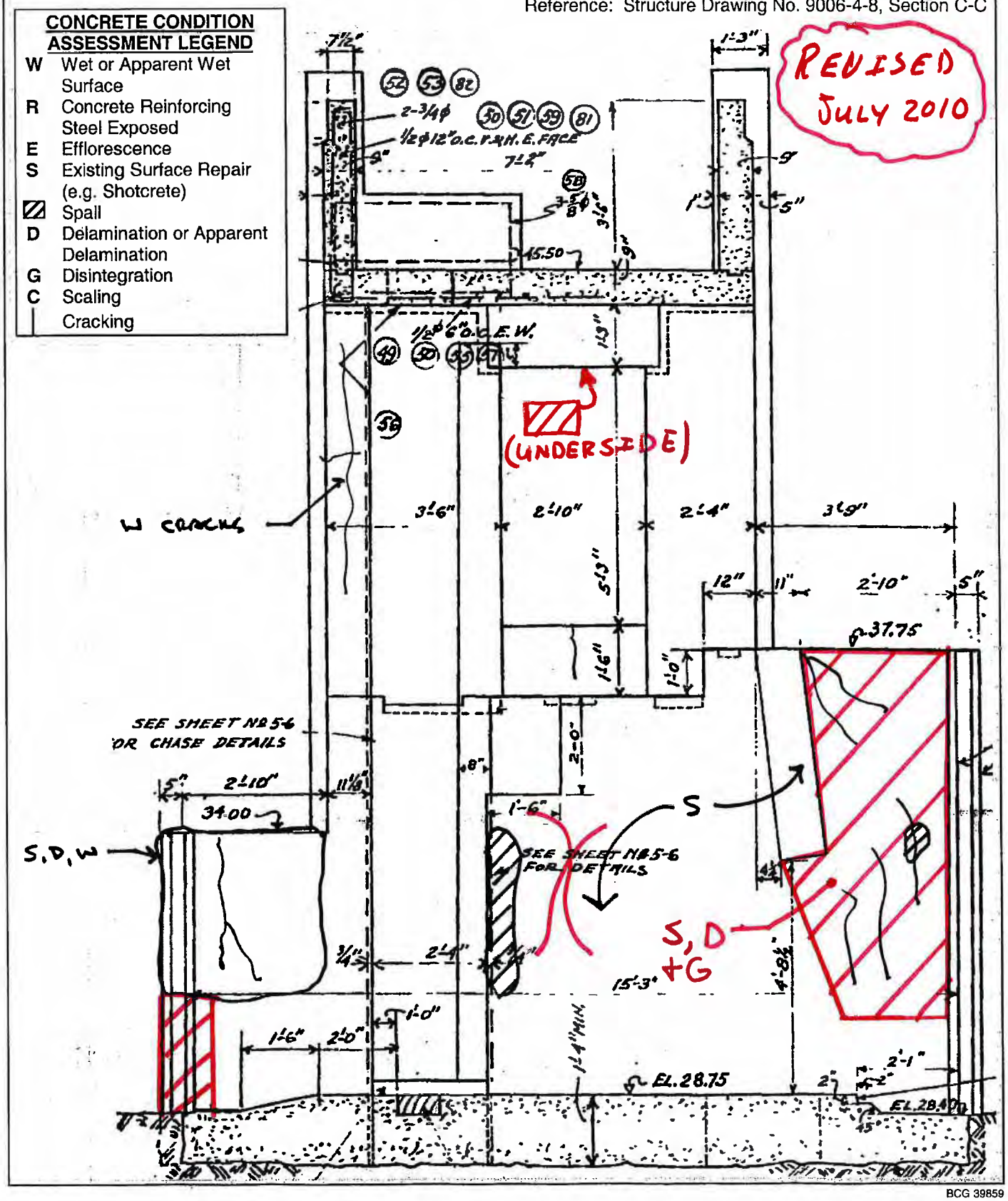

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FIGURE 5

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 3 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDP	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C

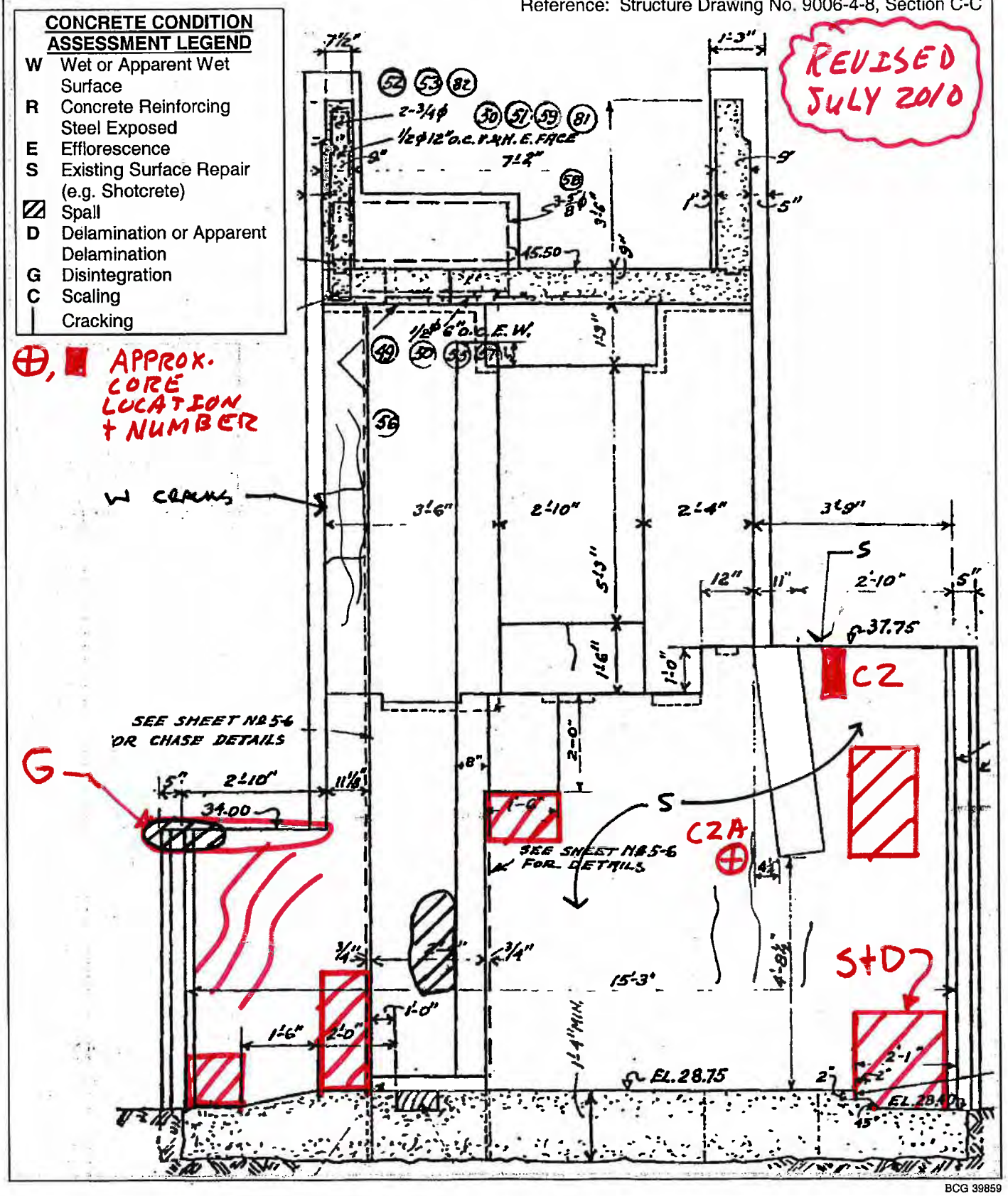

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FIGURE 6

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 4 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C

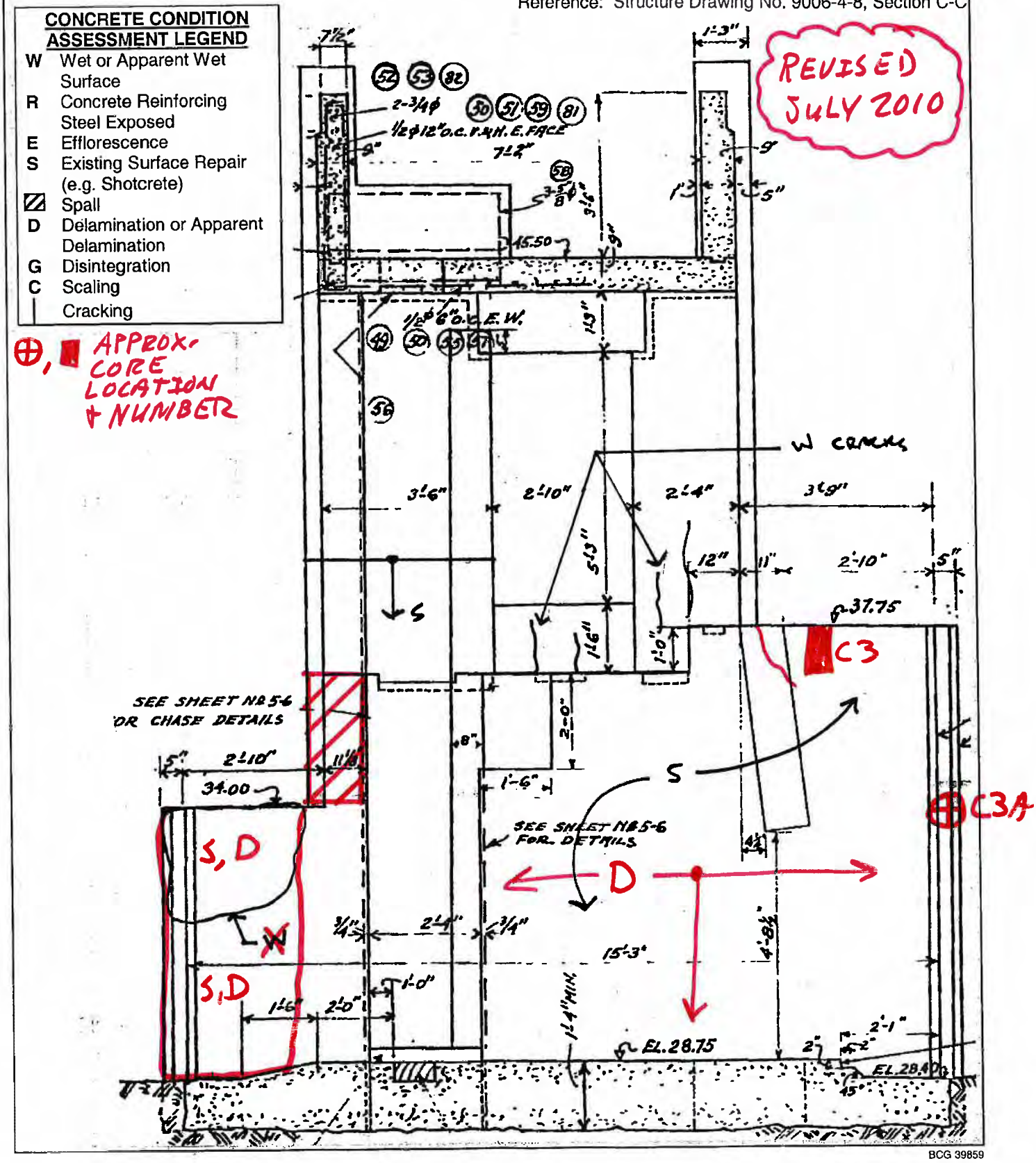

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FIGURE 7

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 5 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C

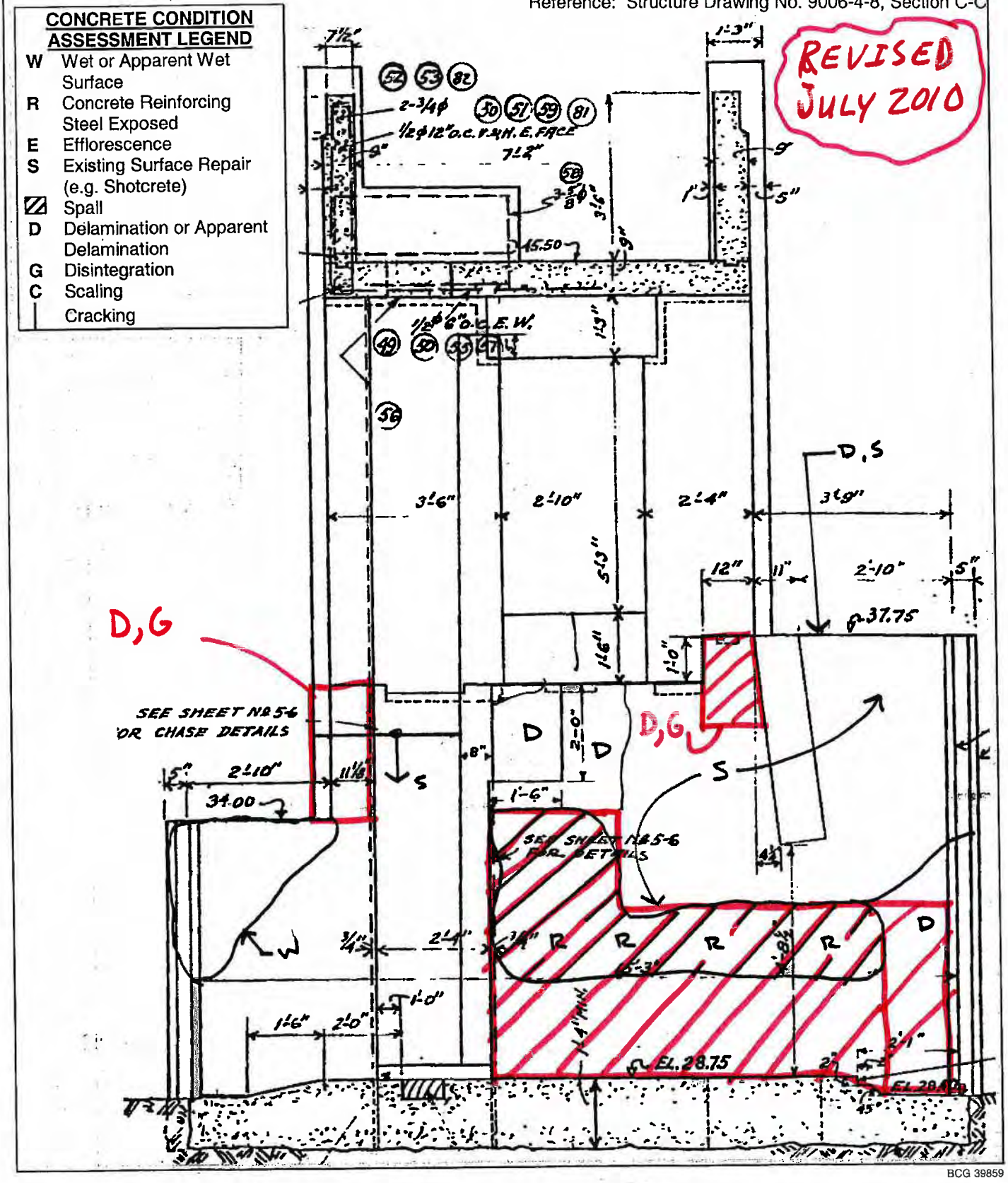

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FIGURE 8

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN			STS JOB NO. X587996	
	SUBJECT: PIER 6 – LEFT SIDE CONDITION OBSERVATIONS			SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. '10	CALC. NO. / REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C

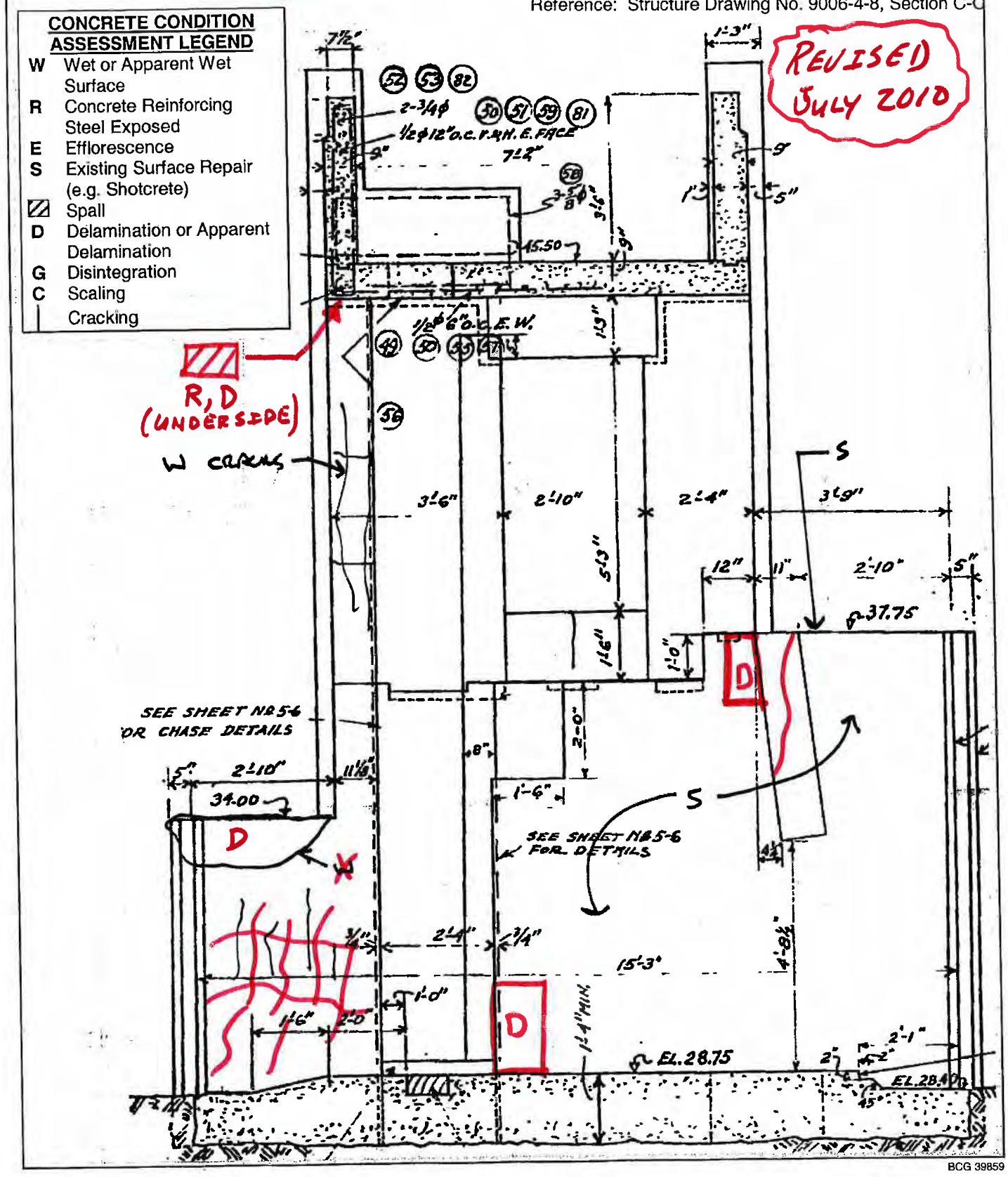

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FIGURE 9

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 7 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDP	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. '10	CALC. NO.	REV. NO

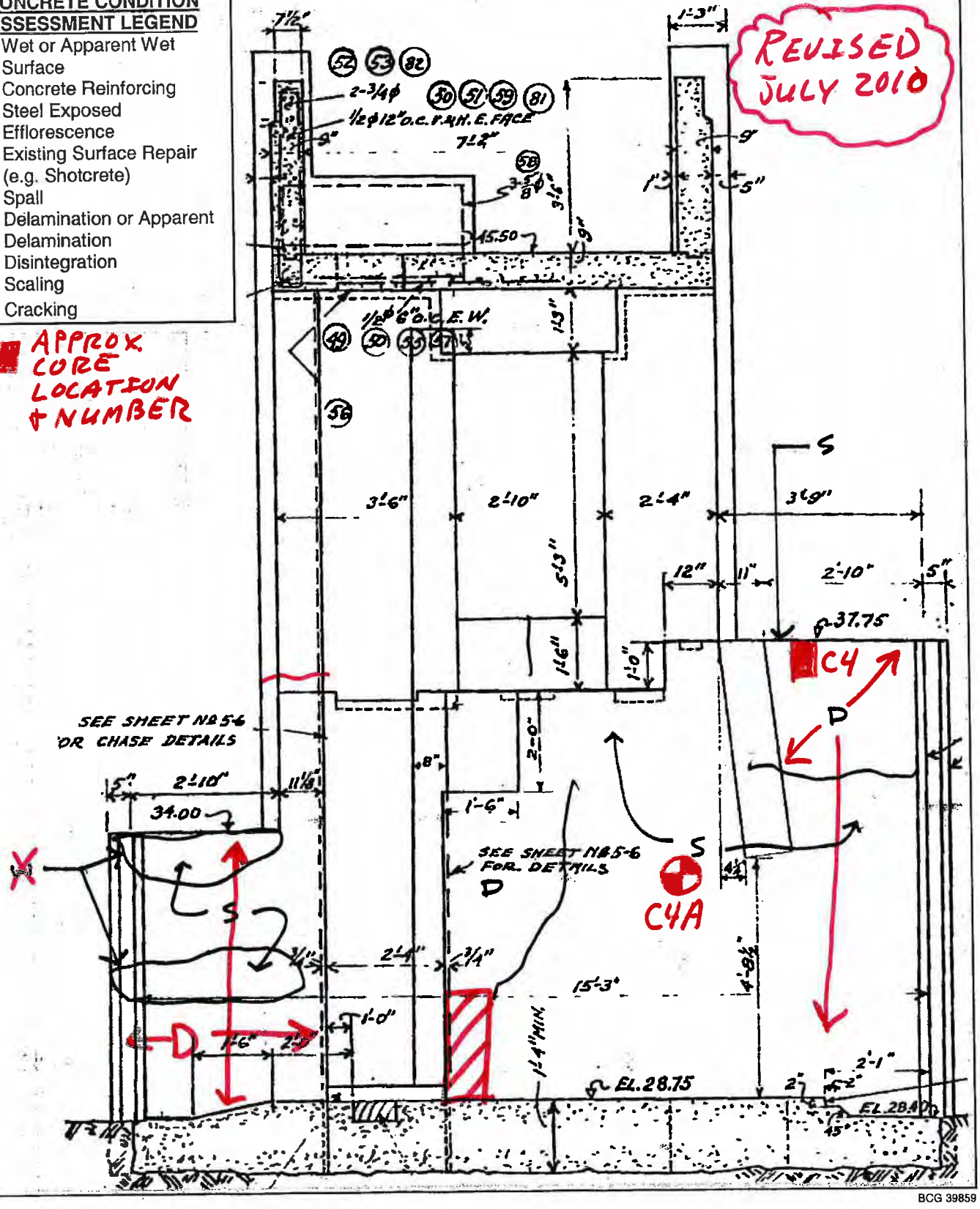
Reference: Structure Drawing No. 9006-4-8, Section C-C

CONCRETE CONDITION ASSESSMENT LEGEND

- W Wet or Apparent Wet Surface
- R Concrete Reinforcing Steel Exposed
- E Efflorescence
- S Existing Surface Repair (e.g. Shotcrete)
- Spall
- D Delamination or Apparent Delamination
- G Disintegration
- C Scaling
- Cracking

REVISED JULY 2010

APPROX. CORE LOCATION + NUMBER

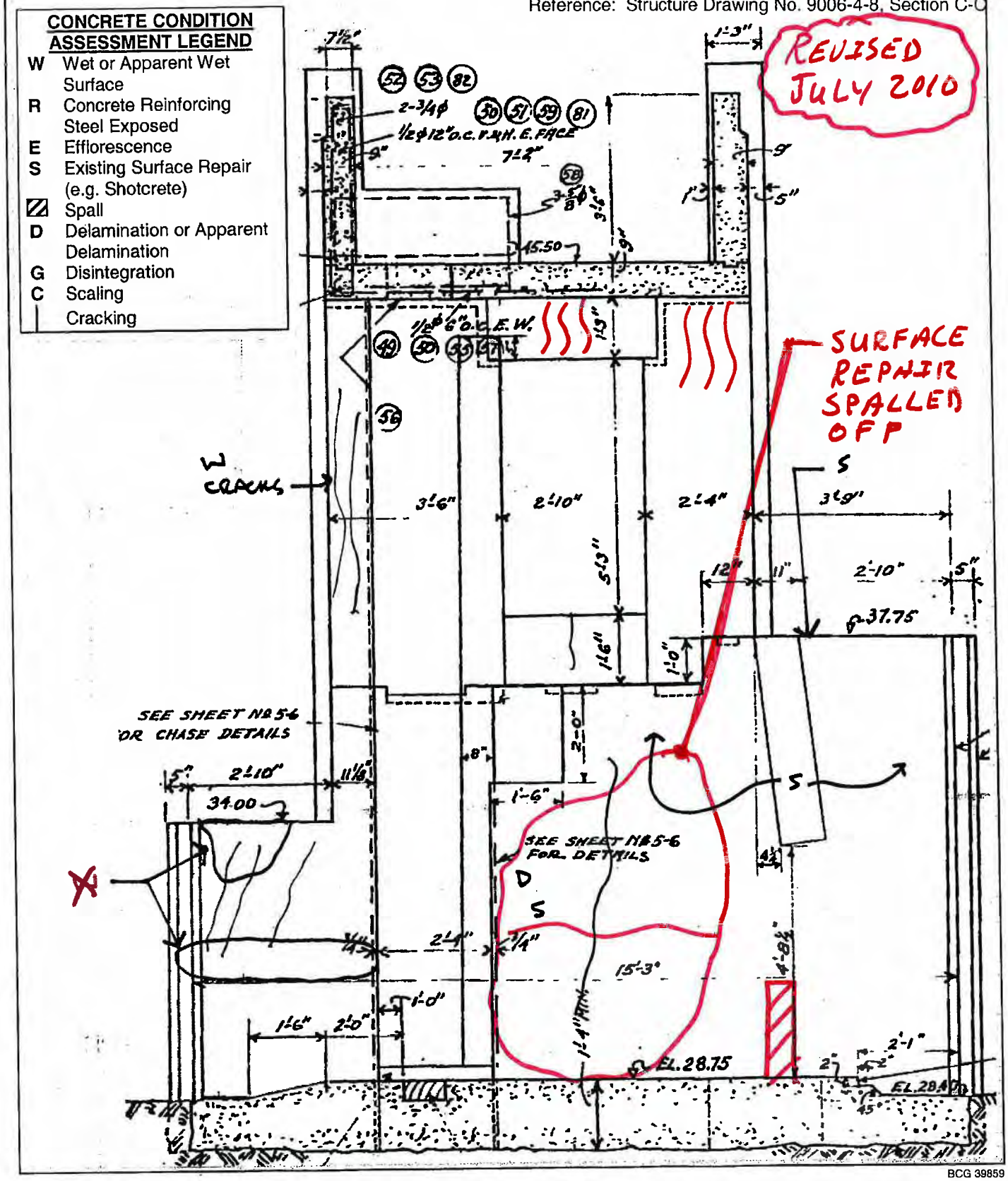


BCG 39859

FIGURE 10

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 8 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C



BCG 39859

FIGURE 11

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 9 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept'10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C

CONCRETE CONDITION ASSESSMENT LEGEND

- W Wet or Apparent Wet Surface
- R Concrete Reinforcing Steel Exposed
- E Efflorescence
- S Existing Surface Repair (e.g. Shotcrete)
- ▨ Spall
- D Delamination or Apparent Delamination
- G Disintegration
- C Scaling
- | Cracking

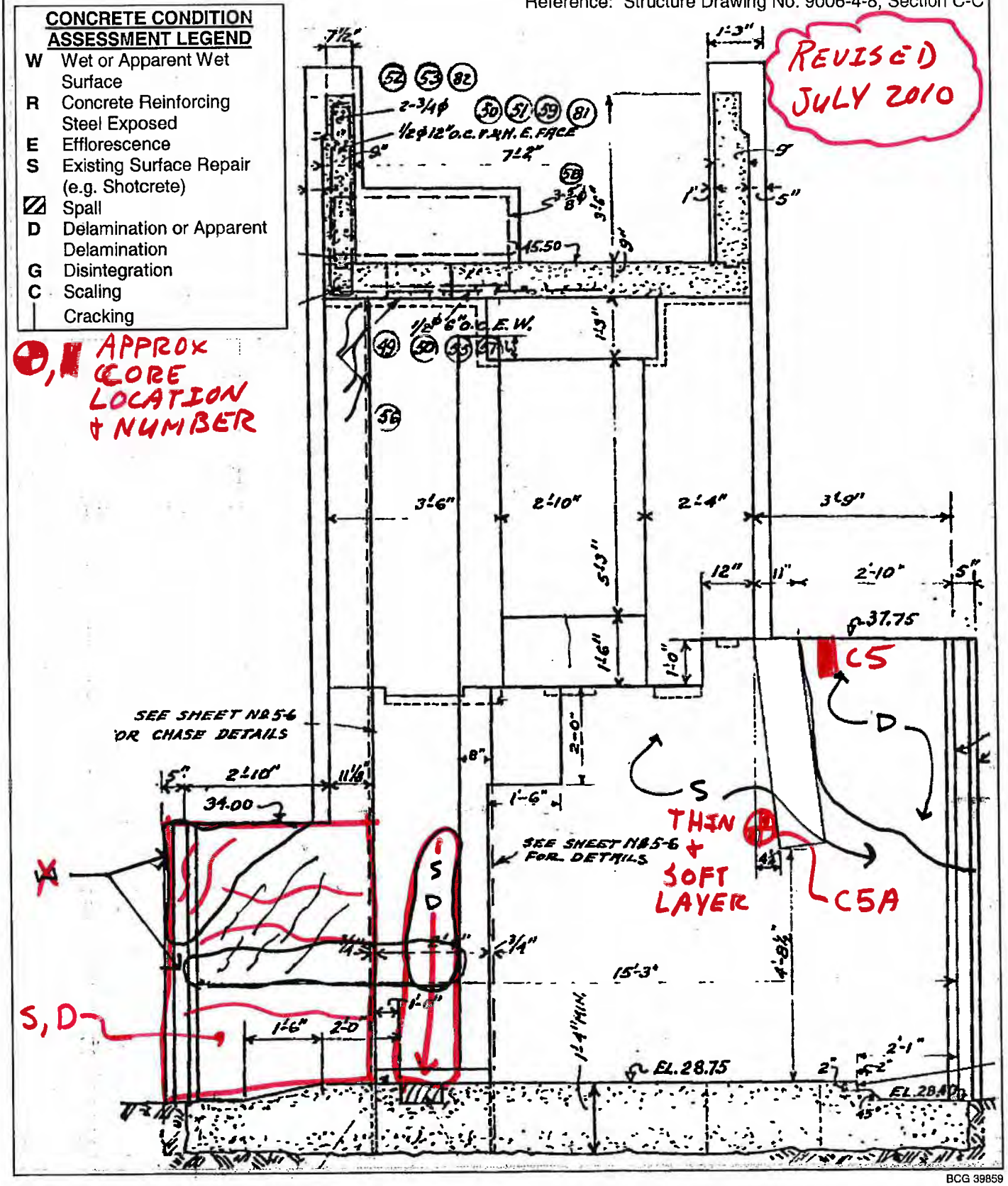


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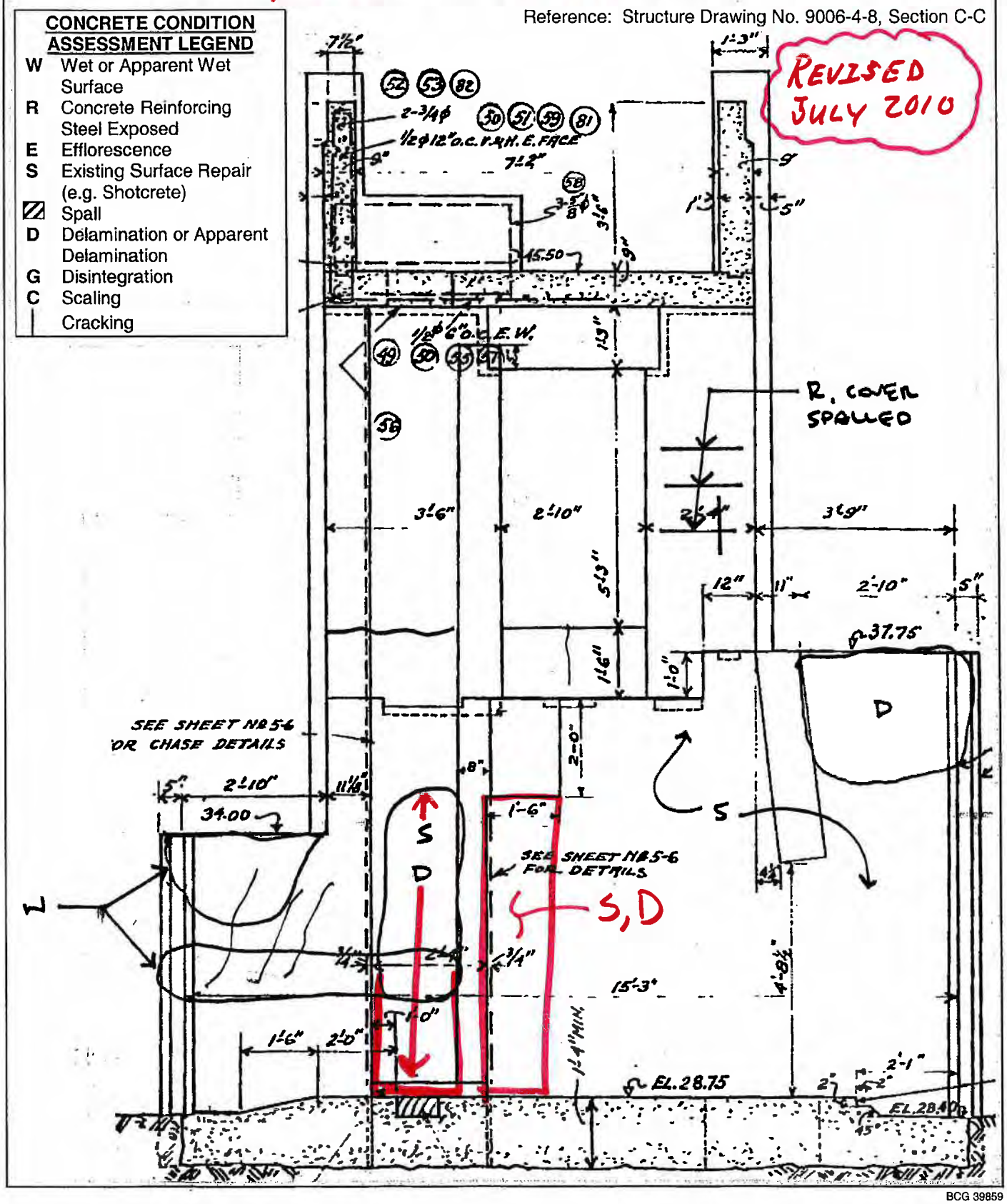
FIGURE 12

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 10 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept '10	CALC. NO.	REV. NO



BCG 39859

FIGURE 13

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT: PIER 11 – LEFT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY: JMK / CDD	DATE: FEB. 3, 2006	CHECKED BY: SAE	DATE: Sept. '10	CALC. NO.	REV. NO

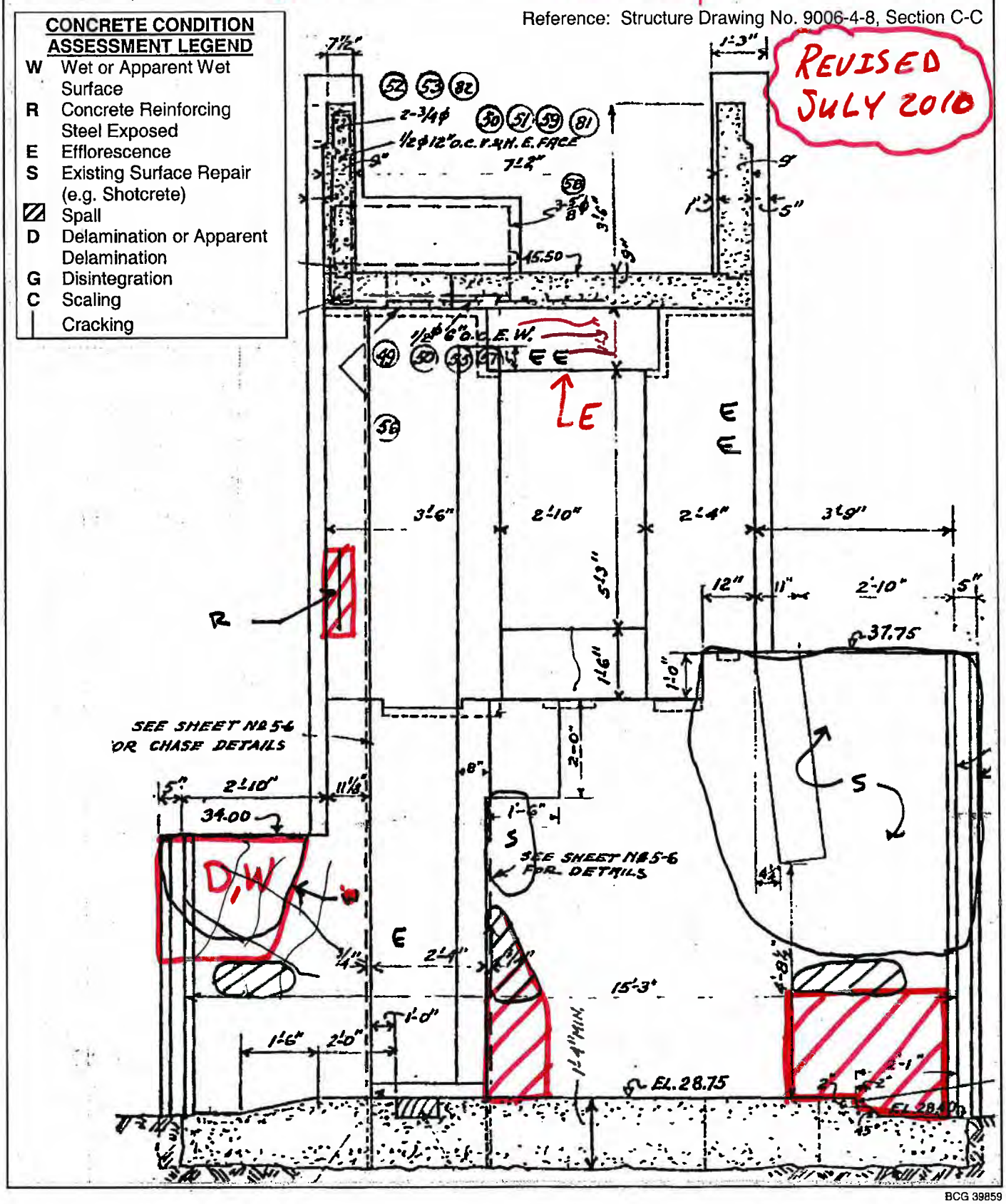
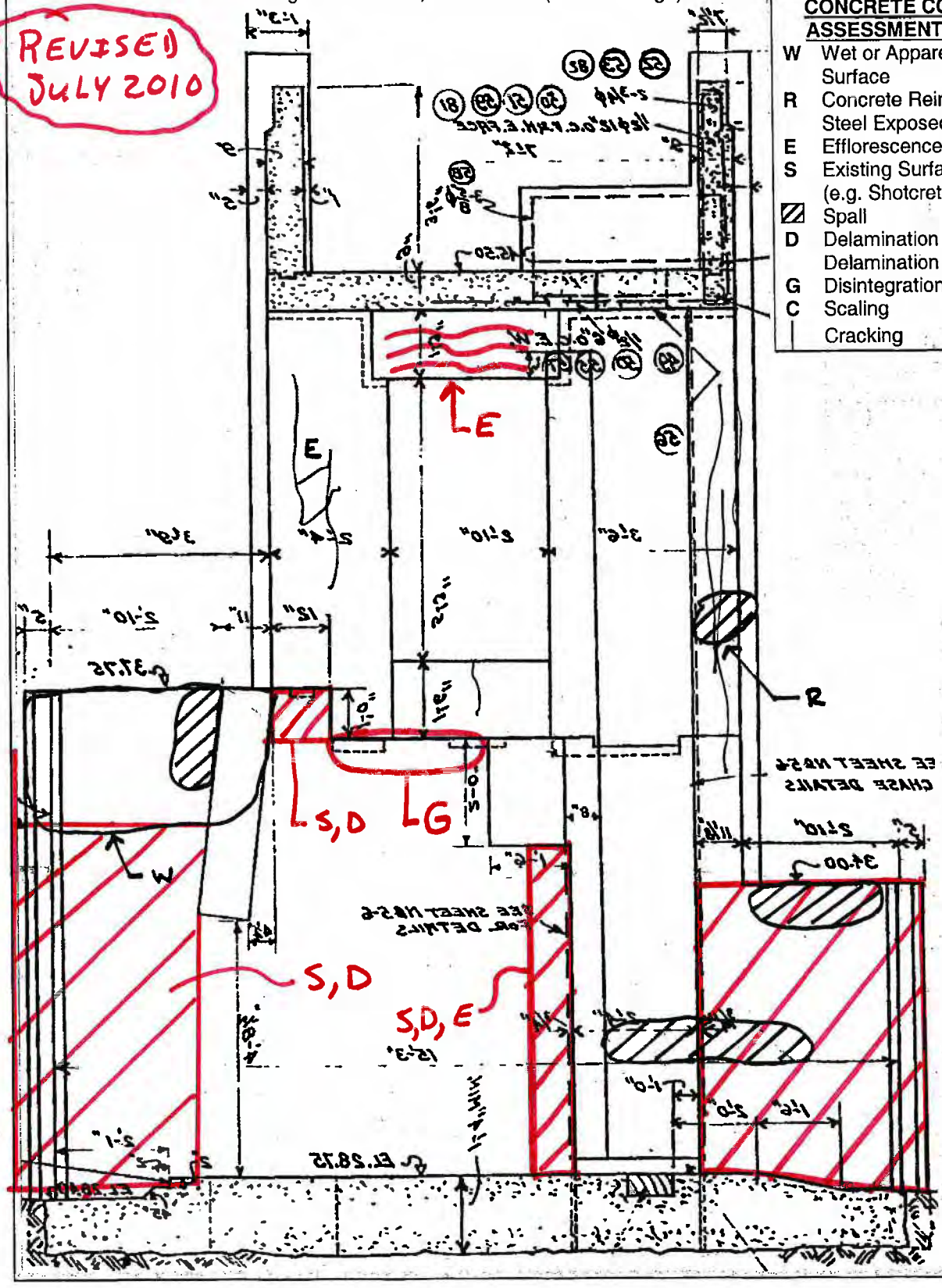


FIGURE 14

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 1 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



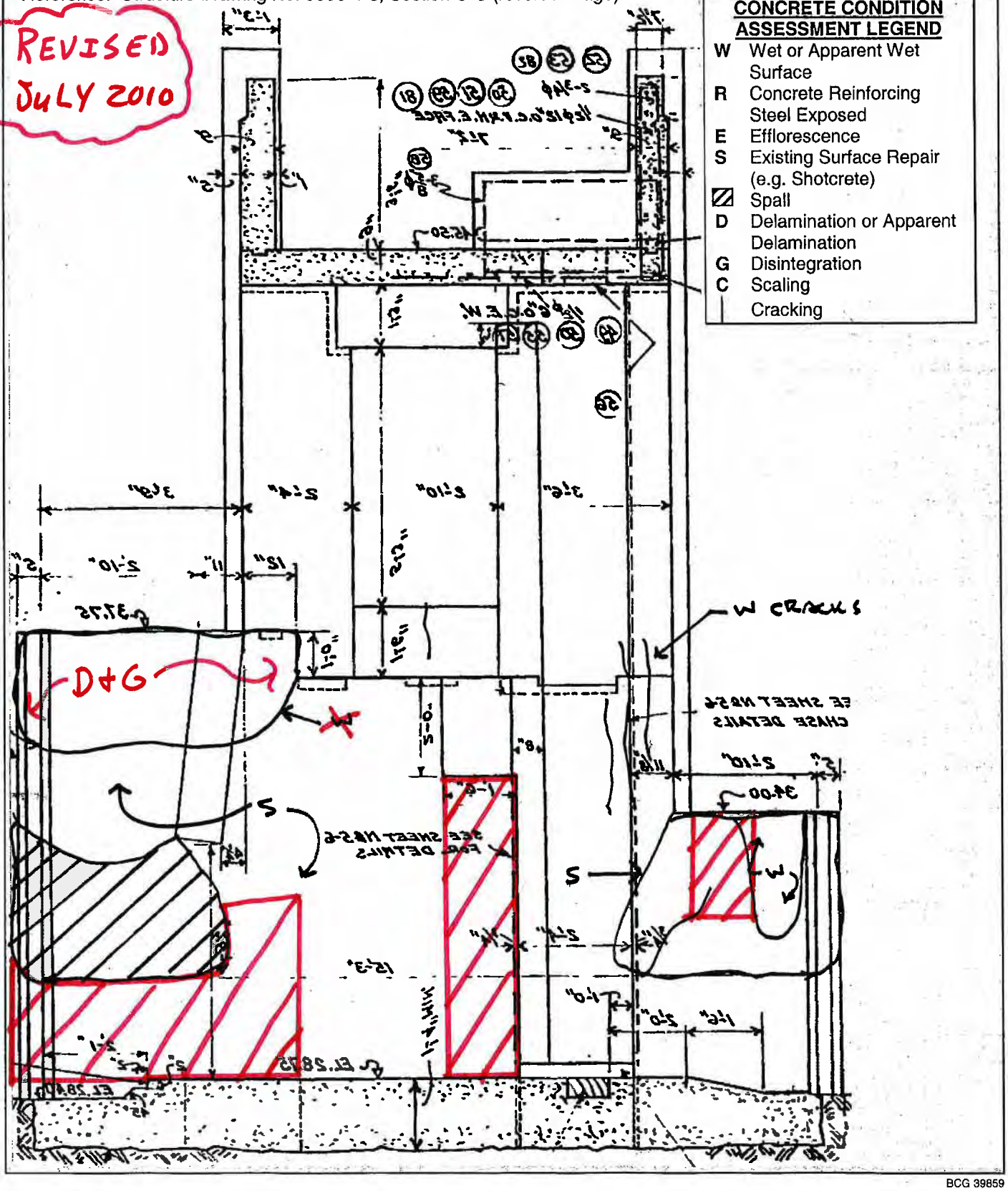
BCG 39859

FIGURE 15

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 2 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



BCG 39859

FIGURE 16

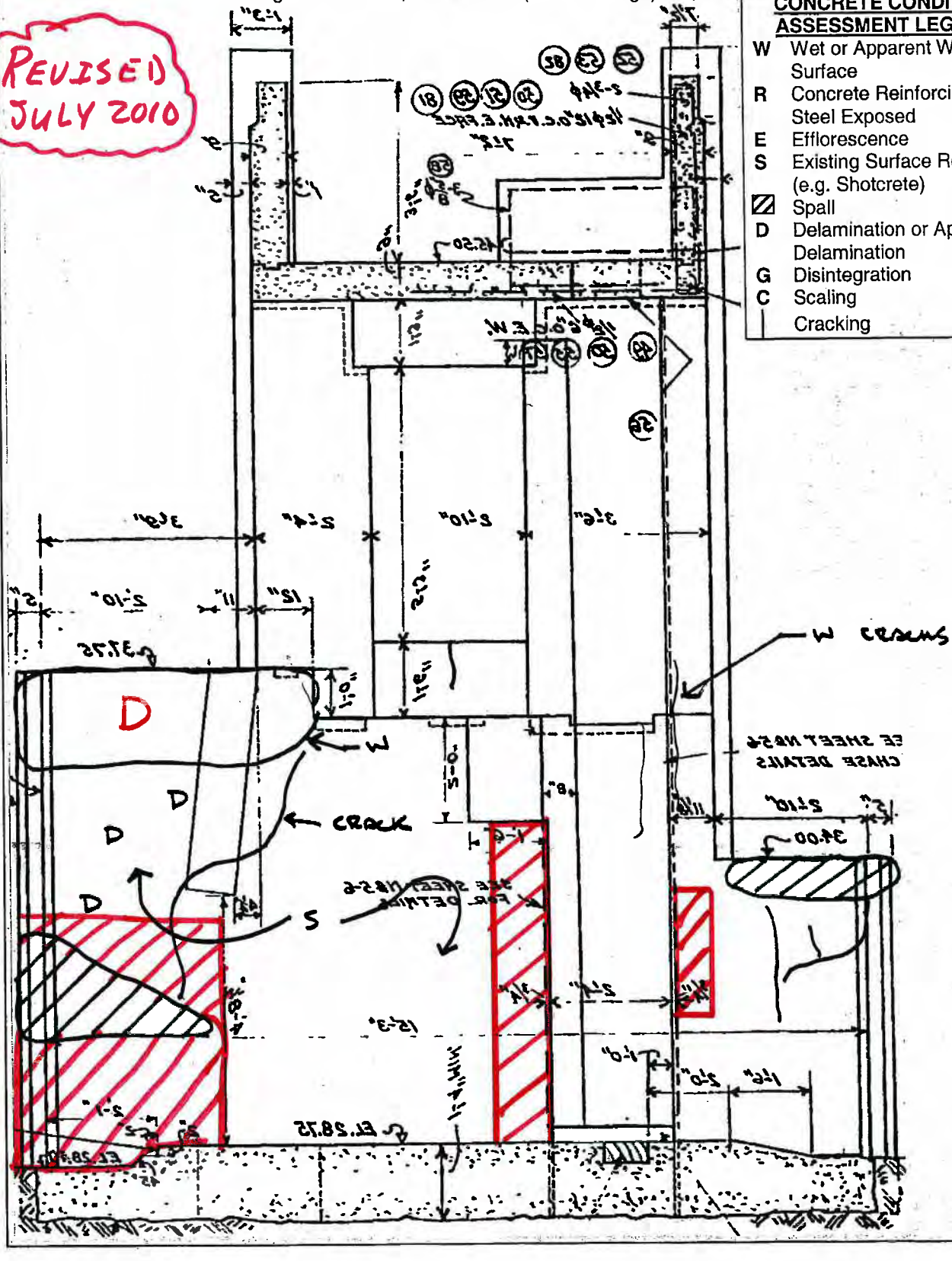
STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 3 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010

CONCRETE CONDITION ASSESSMENT LEGEND

- W Wet or Apparent Wet Surface
- R Concrete Reinforcing Steel Exposed
- E Efflorescence
- S Existing Surface Repair (e.g. Shotcrete)
- ▨ Spall
- D Delamination or Apparent Delamination
- G Disintegration
- C Scaling
- | Cracking



BCG 39859

FIGURE 17

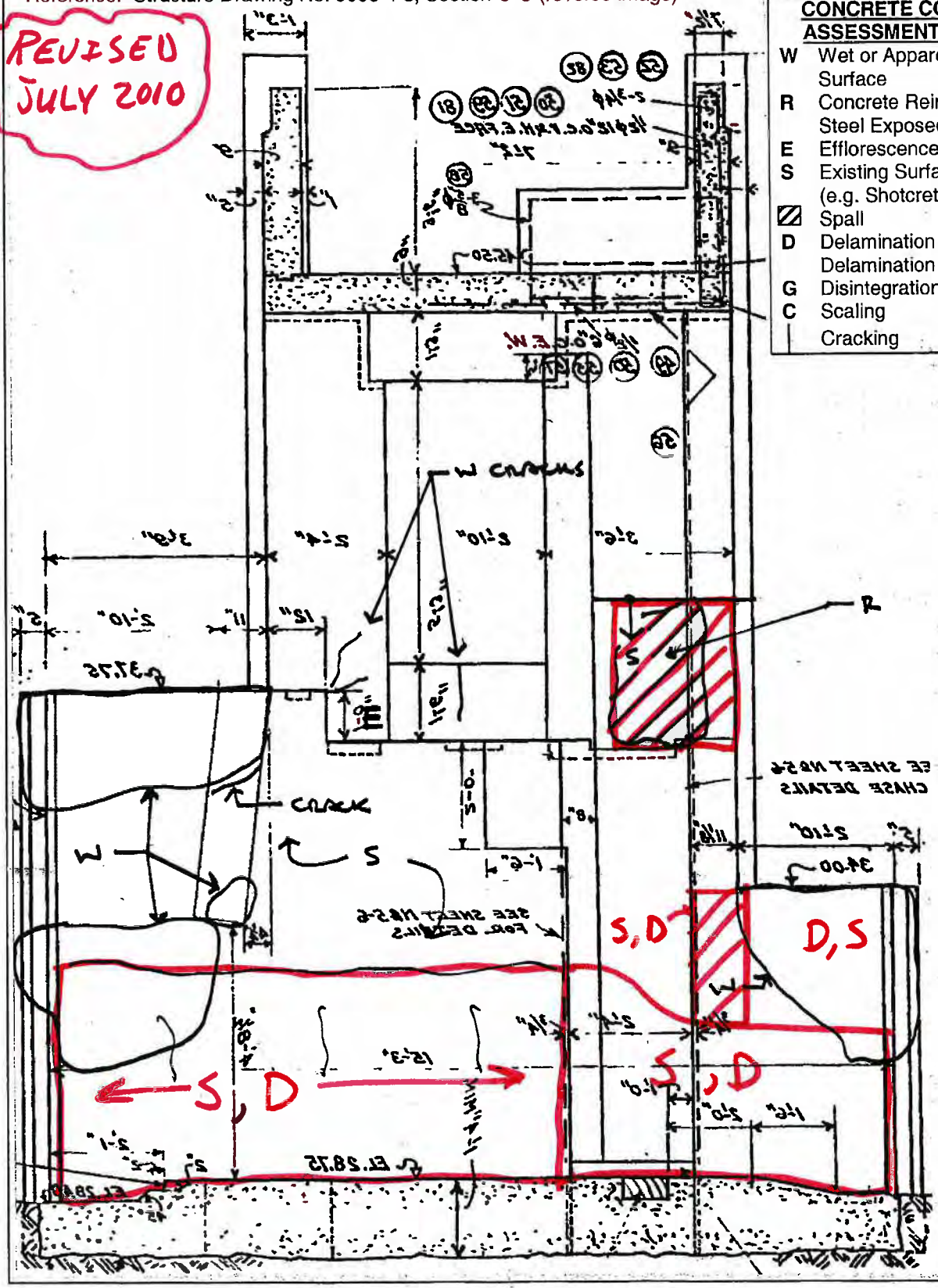
STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 4 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010

CONCRETE CONDITION ASSESSMENT LEGEND

- W Wet or Apparent Wet Surface
- R Concrete Reinforcing Steel Exposed
- E Efflorescence
- S Existing Surface Repair (e.g. Shotcrete)
- ▨ Spall
- D Delamination or Apparent Delamination
- G Disintegration
- C Scaling
- | Cracking



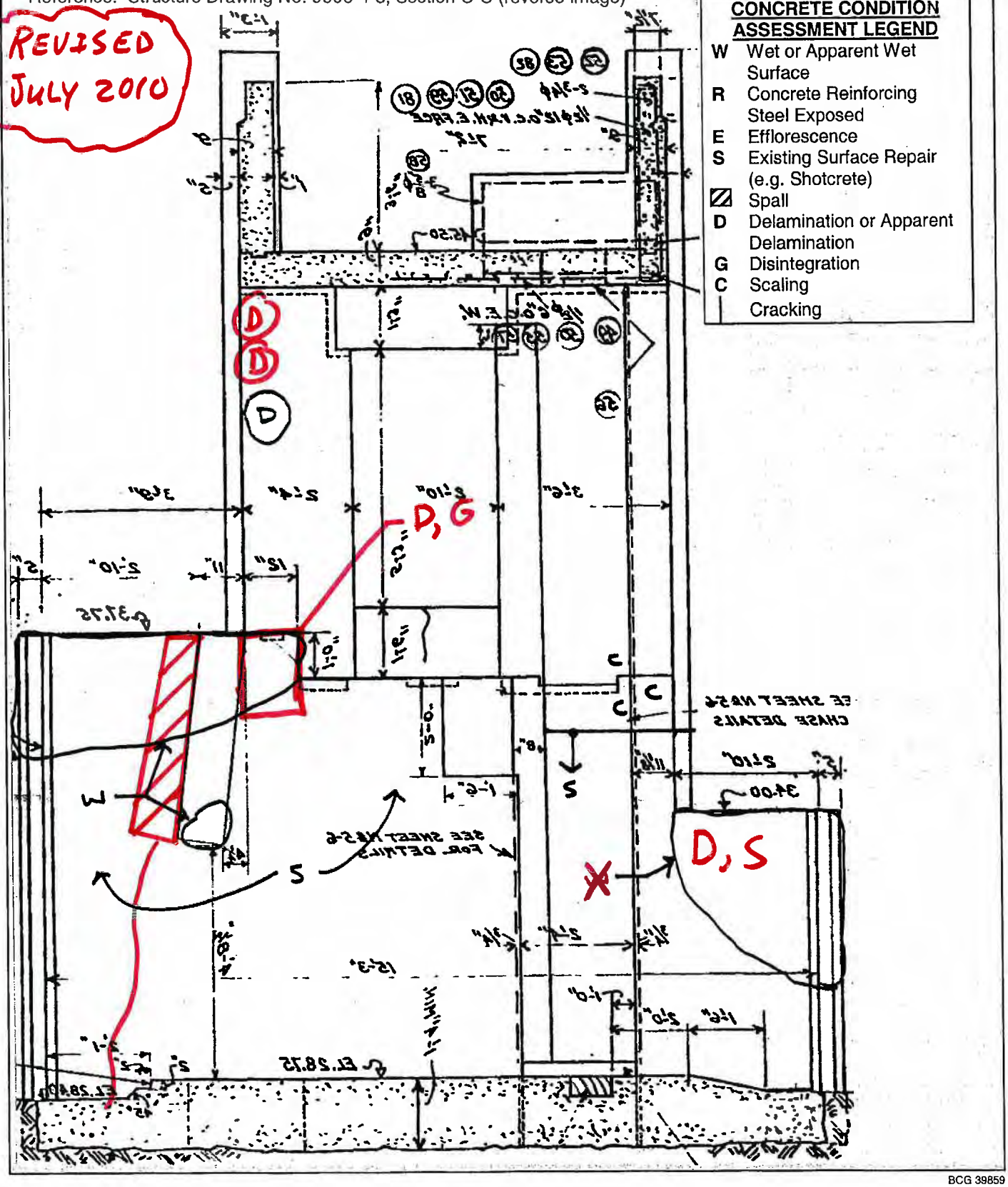
BCG 39859

FIGURE 18

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 5 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



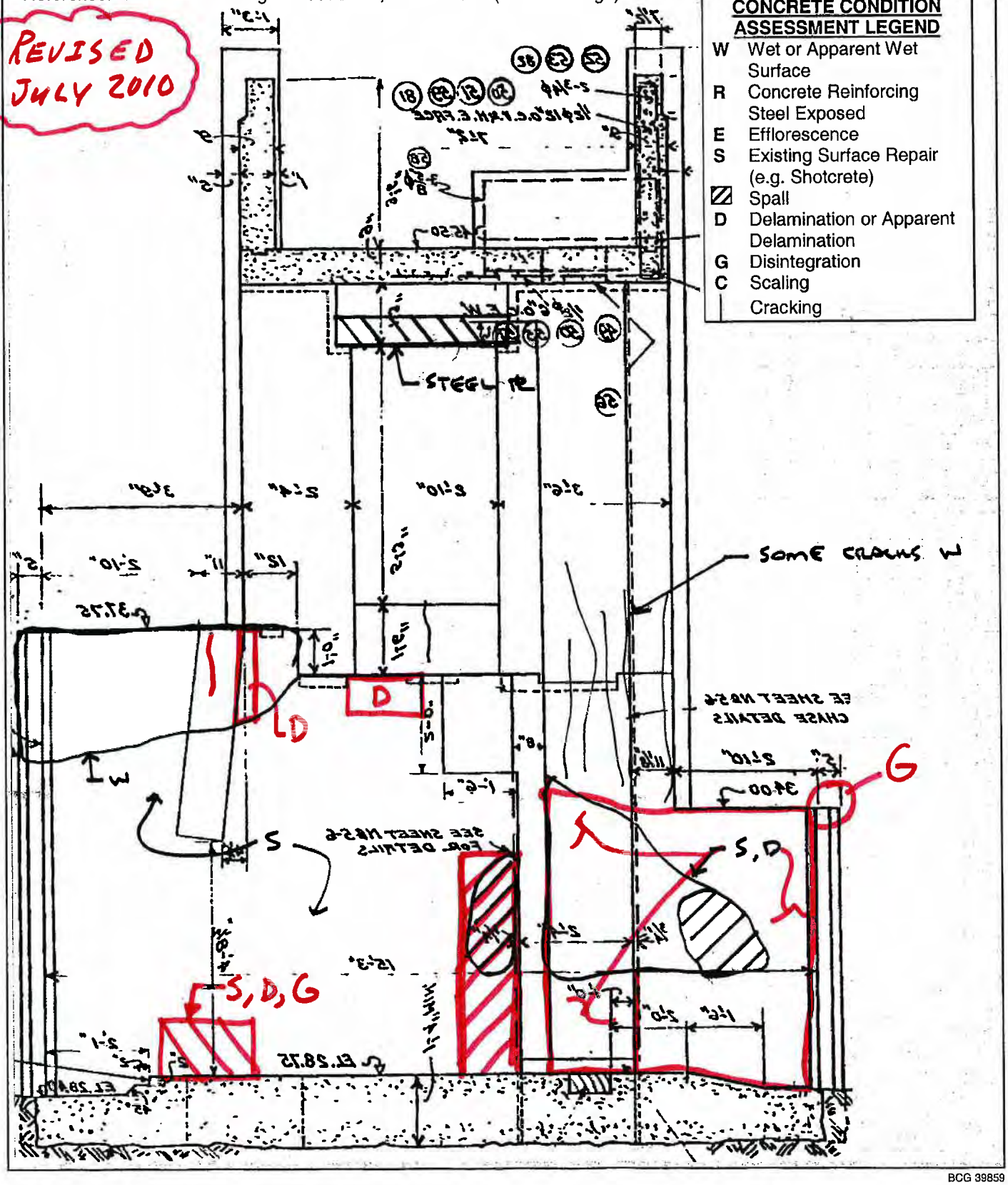
BCG 39859

FIGURE 19

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 6 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



BCG 39859

FIGURE 20

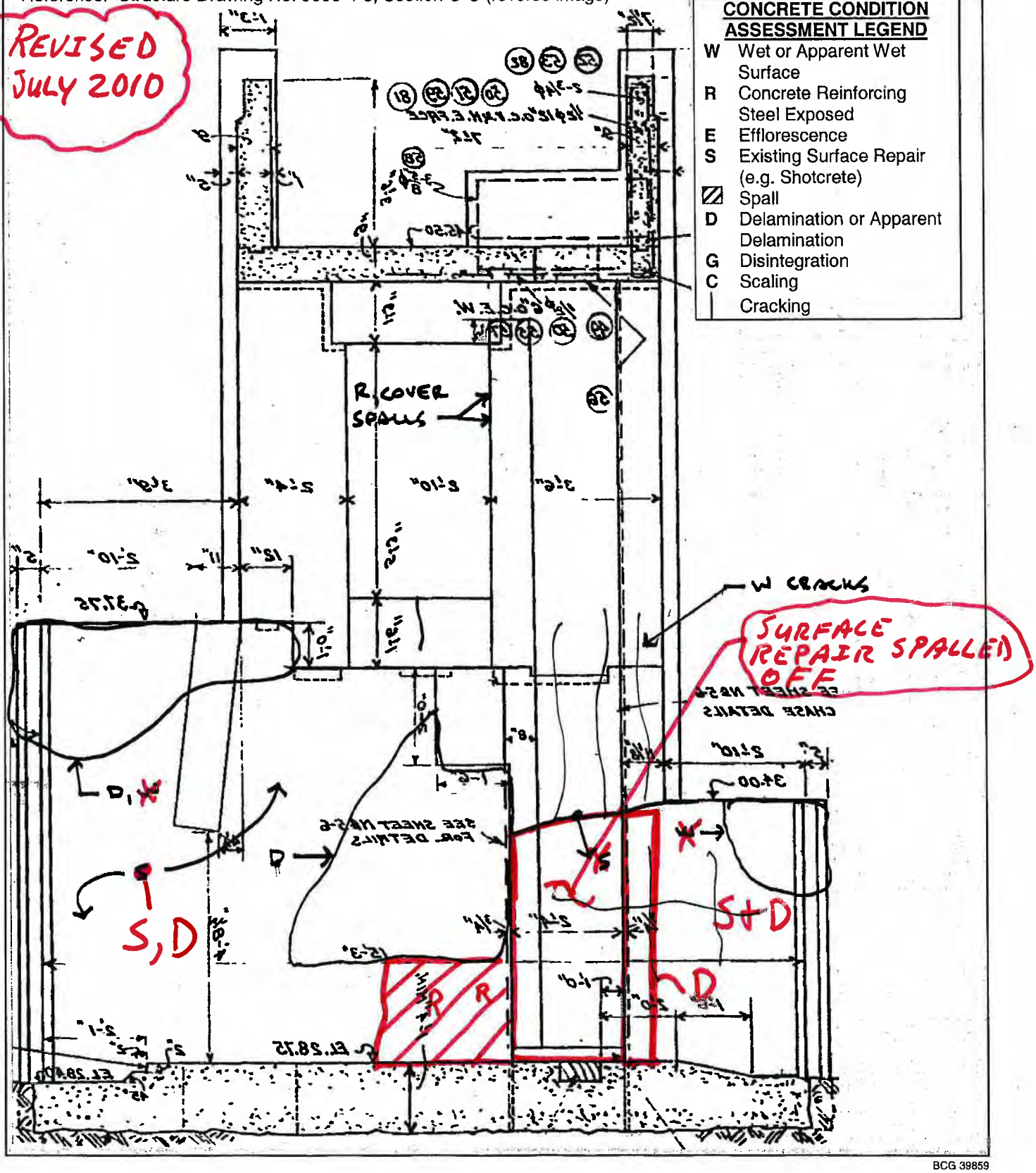
STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 7 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010

CONCRETE CONDITION ASSESSMENT LEGEND

- W Wet or Apparent Wet Surface
- R Concrete Reinforcing Steel Exposed
- E Efflorescence
- S Existing Surface Repair (e.g. Shotcrete)
- ▨ Spall
- D Delamination or Apparent Delamination
- G Disintegration
- C Scaling
- | Cracking



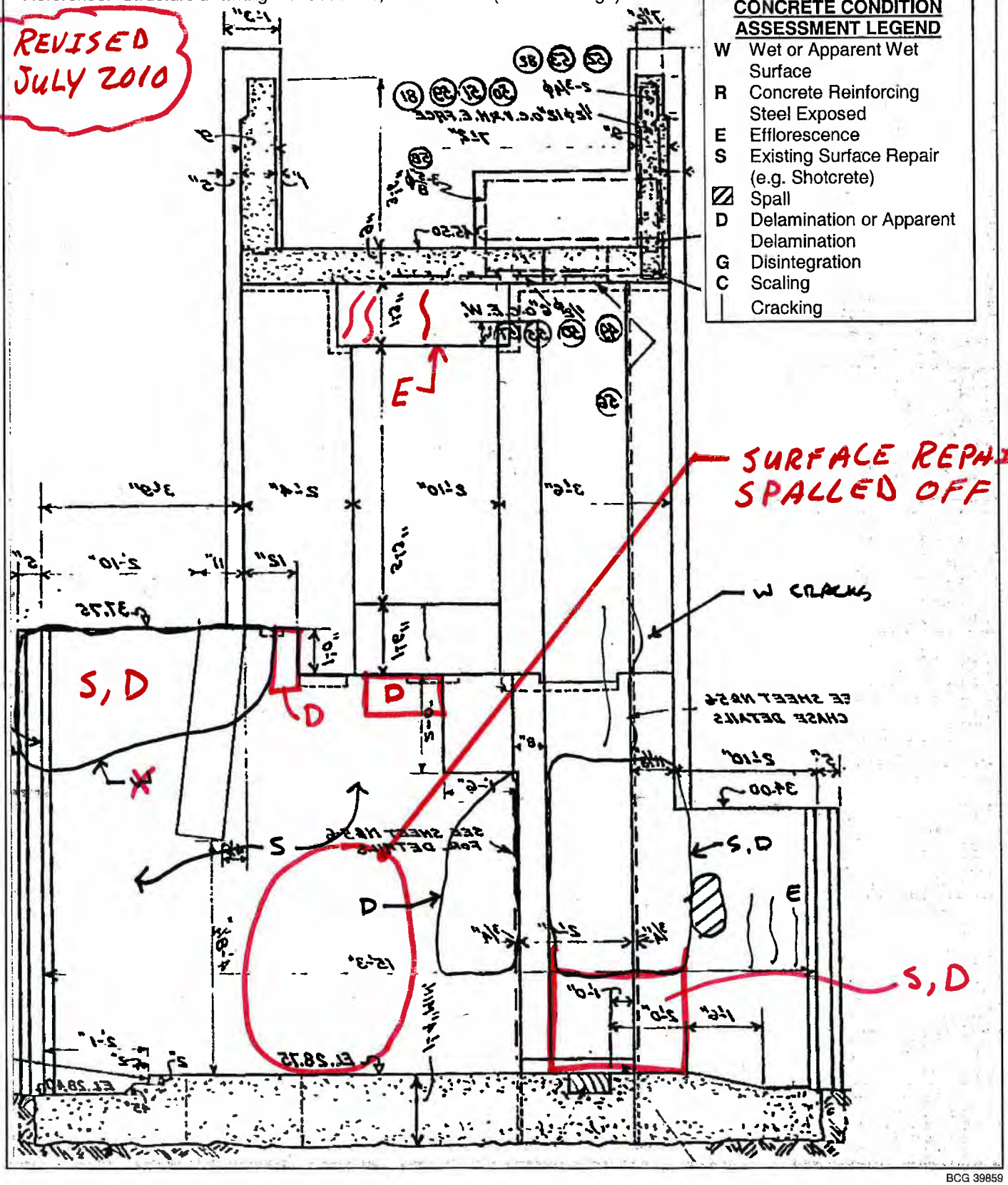
BCG 39859

FIGURE 21

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 8 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



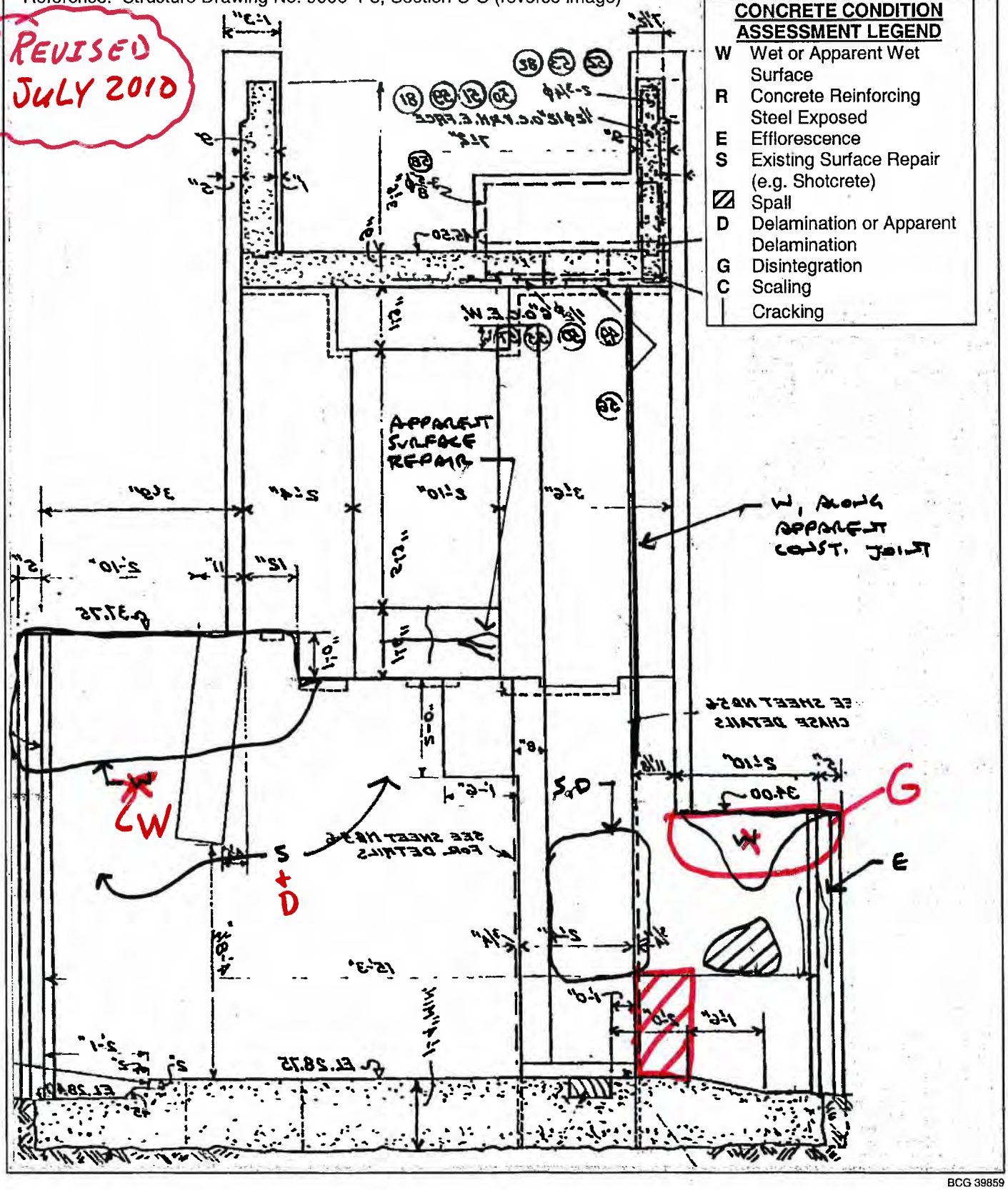
BCG 39859

FIGURE 22

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 9 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD SAE	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



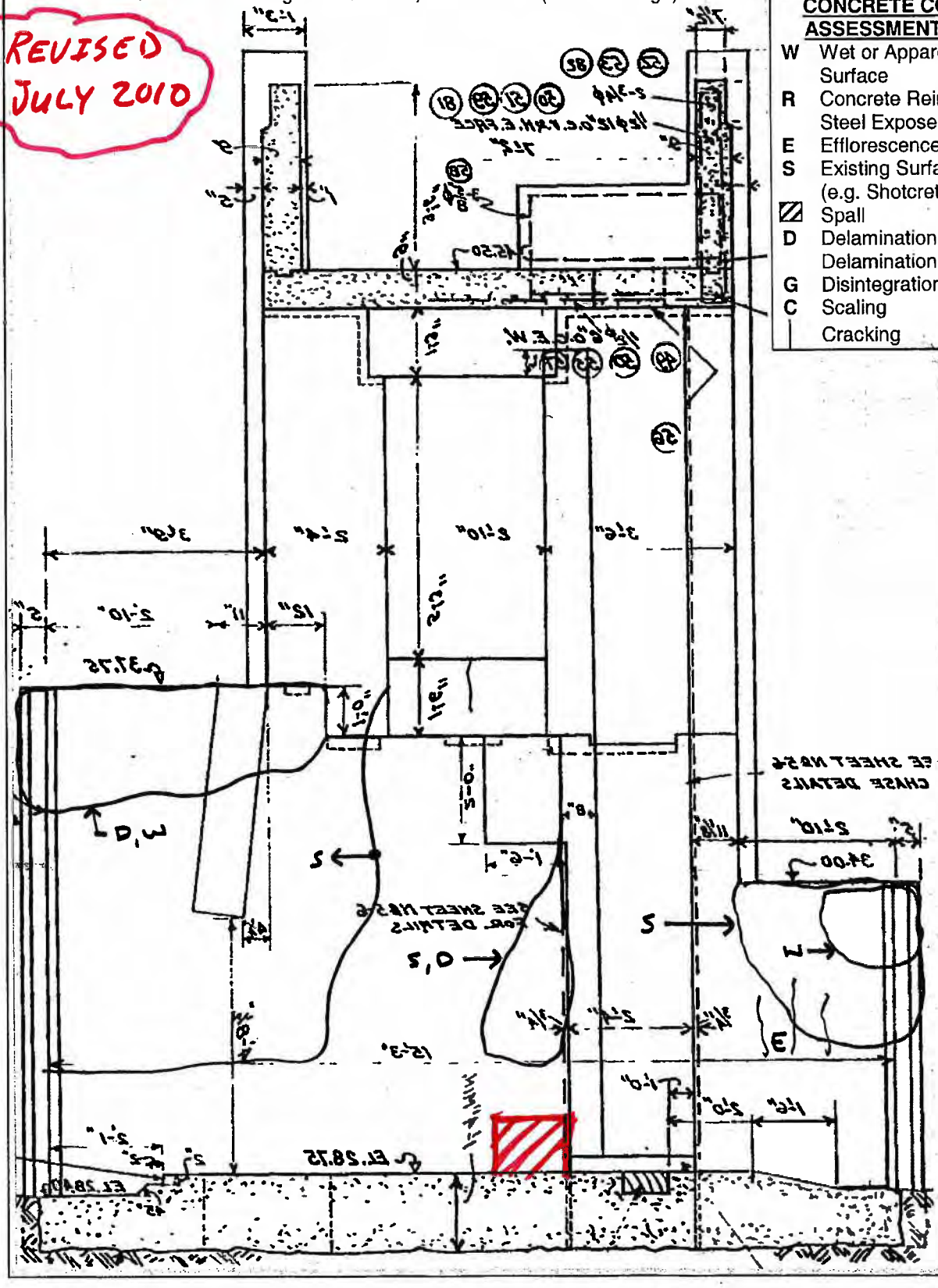
BCG 39859

FIGURE 23

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 10 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



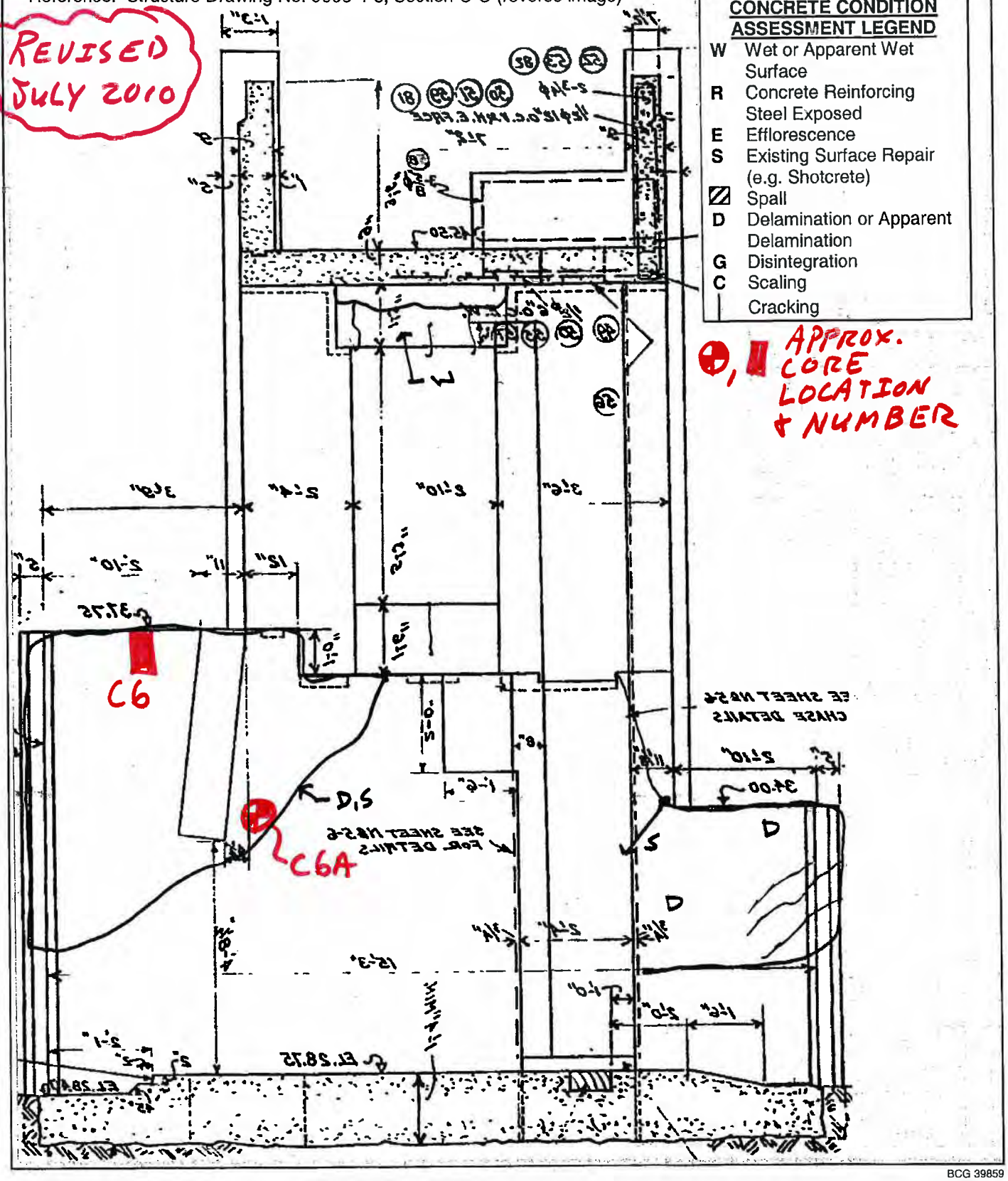
BCG 39859

FIGURE 24

STS Consultants Ltd. CALCULATION SHEET	PROJECT ESTABROOK PARK DAM, MILWAUKEE, WISCONSIN				STS JOB NO. X587996	
	SUBJECT PIER 11 – RIGHT SIDE CONDITION OBSERVATIONS				SHEET NO. 1 OF 1	
	ORIGINATED BY JMK / CDD	DATE FEB. 3, 2006	CHECKED BY SAE	DATE Sept. '10	CALC. NO.	REV. NO

Reference: Structure Drawing No. 9006-4-8, Section C-C (reverse image)

REVISED JULY 2010



BCG 39859

Appendix B

Project Photographs

July 6 and 8, 2010 Photographs

Photo 1: Upstream view of gated spillway structure of Estabrook Dam, gates open.

Photo 2: Downstream side of the gated spillway structure of Estabrook Dam, gates open.

July 6 and 8, 2010 Photographs

Photo 3: View of the right side of the gated spillway structure. Note arrow pointing to the right abutment.

Photo 4: View of deterioration on the right abutment steps.

July 6 and 8, 2010 Photographs

Photo 5: View of the downstream side wall at the right abutment. Note map cracking (arrow) and spalling of the wall.

Photo 6: View of the operating deck from the right abutment, looking north.

July 6 and 8, 2010 Photographs

Photo 7: View of the expansion joint in the operating deck at Pier 1. Note joint material extending outward from joint at wall (right) and spalling at the joint (arrow).

Photo 8: Typical spall of the operating bridge wall (arrow) and expansion joint filler material squeezed outward from the joint.

July 6 and 8, 2010 Photographs

Photo 9: View of concrete deterioration on pedestal and disintegration of concrete of the operating deck and gate hoist block at Pier 1.

Photo 10: View of the underside of operating bridge deck (arrows) with water staining and isolated spalls with exposed reinforcing steel.

July 6 and 8, 2010 Photographs

Photo 11: View of the separation of wall from Pier 1 facing (see photo 12) and white efflorescence at cracks of the operation bridge walls.

Photo 12: Concrete disintegration and Pier 1 separation at the operating bridge wall, upstream side (see photo 11).

July 6 and 8, 2010 Photographs

Photo 13: View of efflorescence and exposed reinforcing steel (arrow) of the left abutment.

Photo 14: View of the left abutment. Note disintegration of the top five stair nosings (arrow).

July 6 and 8, 2010 Photographs

Photo 15: Typical view of the upstream side of piers (Piers P4, P5 and P6).

Photo 16: Typical view of the downstream side of piers (Pier 9, 10 and 11, from right to left).

July 6 and 8, 2010 Photographs

Photo 17: Upstream side of Piers P1 and P2 (left to right). Note previous surface repair is failing on Pier P2.

Photo 18: Upstream side of Piers P3 to P5 (left to right). Note previous surface repair is failing on Pier P3.

July 6 and 8, 2010 Photographs

Photo 19: Upstream side of Piers P6 to P8 (left to right). Note previous surface repair is failing on Pier P7 and wet surface on all three piers.

Photo 20: Upstream side of Piers P9 to P11 (left to right). Note previous surface repair is failing on Piers P9 and P10. Wet surface observed on all three piers.

July 6 and 8, 2010 Photographs

Photo 21: Typical view of failing surface repair at the upstream tip of pier (Pier P2).

Photo 22: Typical view of the right side of Pier 7 with surface repair. Note blue chalk marks are UPV grid lay-out for testing and concrete core location (arrow).

July 6 and 8, 2010 Photographs

Photo 23: Typical view of the left side of piers, looking downstream, with minimal distress (Pier P6).

Photo 24: Typical view of the right side of piers, looking downstream, with disintegrating concrete and delaminated surface repairs (arrows) (Pier P6).

July 6 and 8, 2010 Photographs

Photo 25: Severe erosion of pier with exposed reinforcing steel on Pier P5.

Photo 26: Typical view of cracking and efflorescence on the piers above Elevation 37.75'.

July 6 and 8, 2010 Photographs

Photo 27: Map cracking at the downstream tip of Pier P6.

July 6 and 8, 2010 Photographs

Photo 28: View of disintegrated concrete on top and delaminated concrete on side, downstream tip of Pier P3.

July 6 and 8, 2010 Photographs

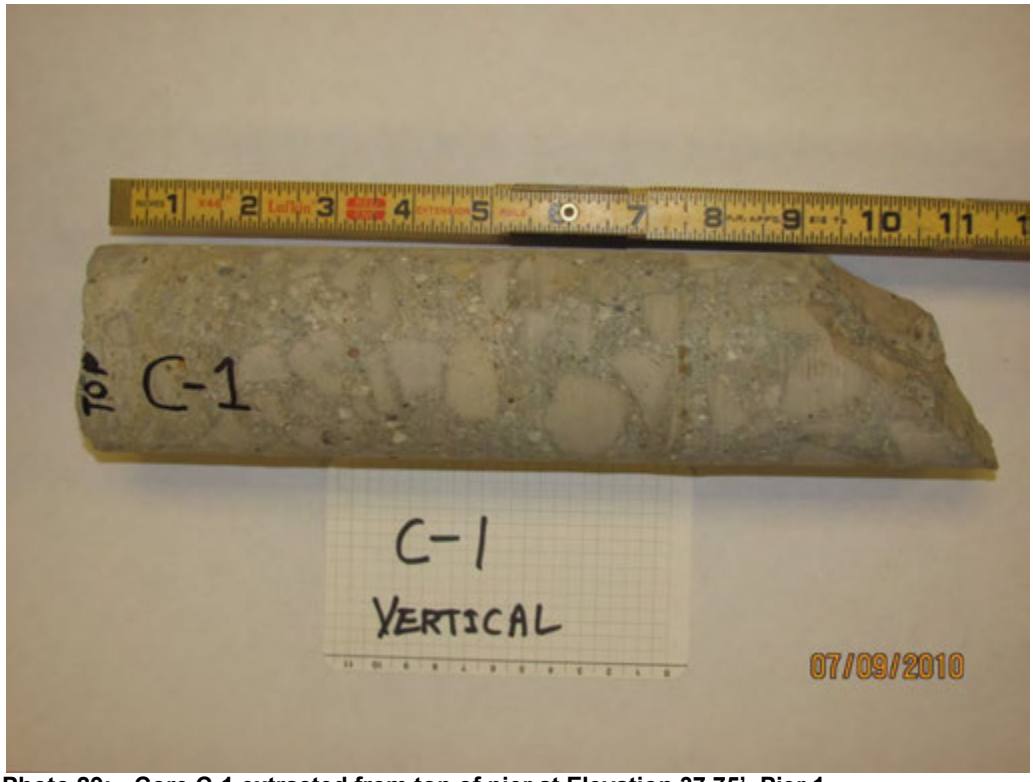


Photo 29: Core C-1 extracted from top of pier at Elevation 37.75', Pier 1.

Photo 30: View of Core C-1A, Pier 1. The core broke apart during coring operations.

July 6 and 8, 2010 Photographs

Photo 31: View of Core C-2A (arrows), Pier 3. The core broke apart during coring operations.

Photo 32: View of Core C-2 extracted from top of Pier 3 at Elevation 37.75'. Core broke apart during coring operations.

July 6 and 8, 2010 Photographs

Photo 33: View of Core C-3. Note concrete above 9-inches is previous repair material.

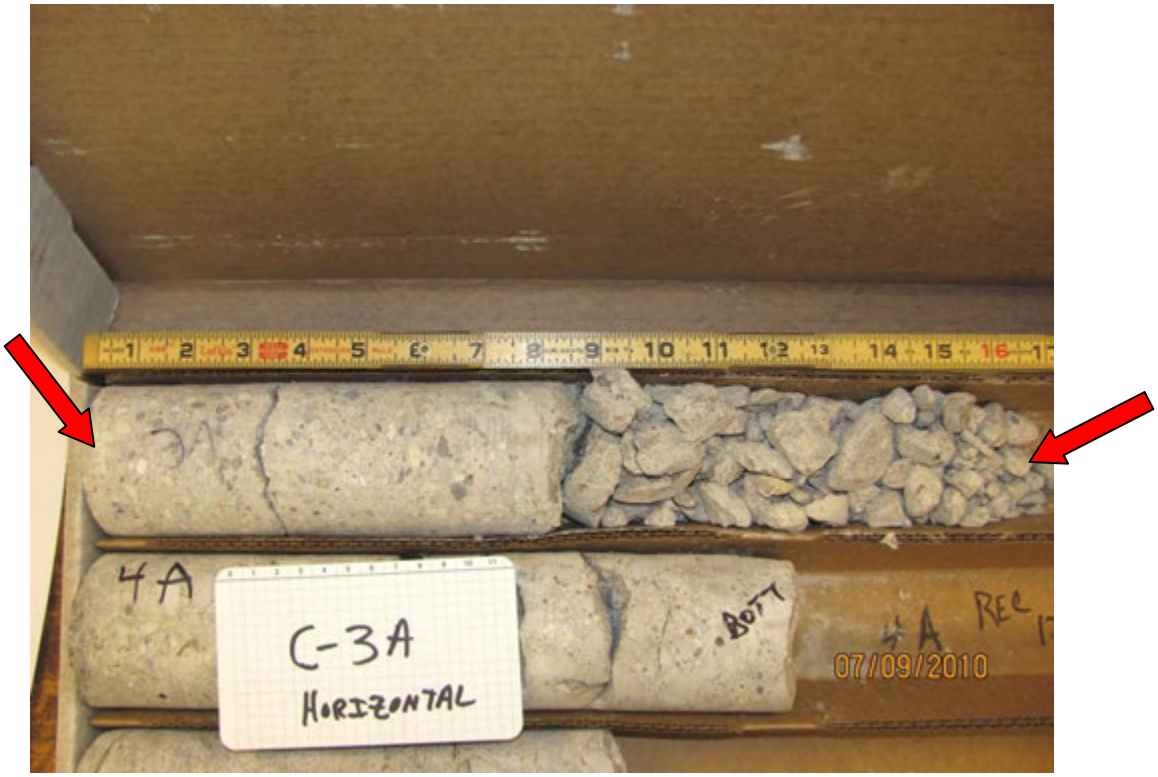


Photo 34: View of Core C-3A (between arrows). Note concrete for first 9-inches is previous repair material. The remainder is original deteriorated concrete.

July 6 and 8, 2010 Photographs

Photo 35: Core C-4 extracted from top down of Pier 7 at Elevation 37.75'.

Photo 36: View of Core C-4A; note on photo should read C-4A, Horizontal.

July 6 and 8, 2010 Photographs

Photo 37: Core C-5 extracted from top down of Pier 9 at Elevation 37.75'.

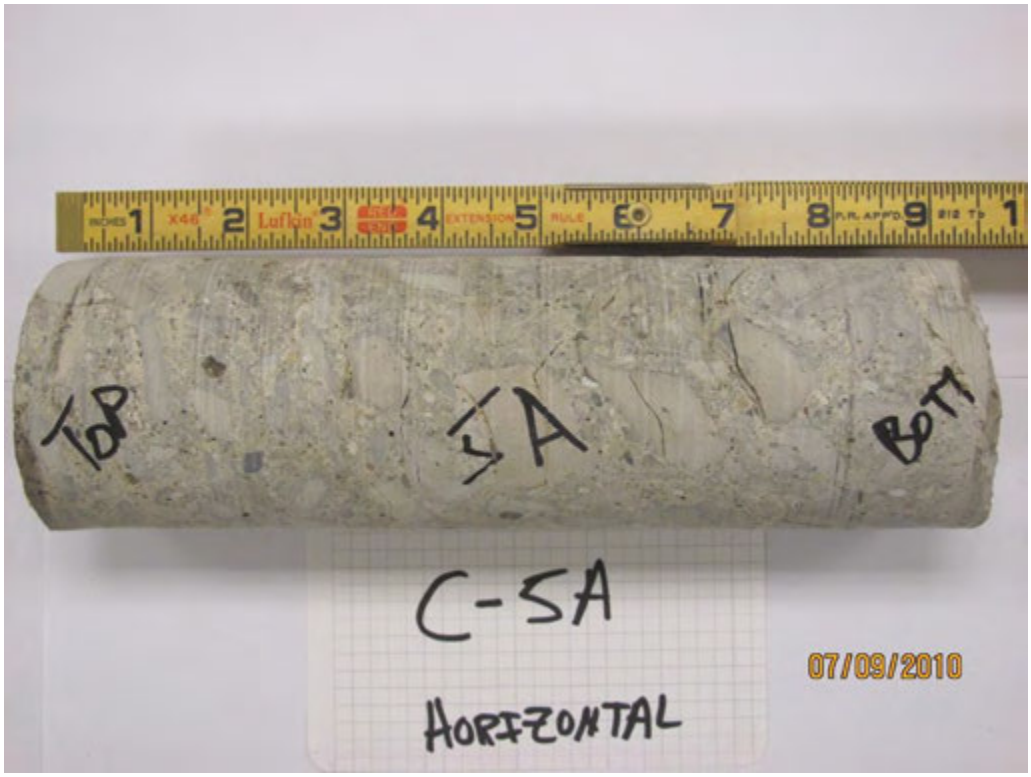


Photo 38: View of Core C-5A. Note cracking in the aggregate and cement paste.

July 6 and 8, 2010 Photographs

Photo 39: View of Core C-6A extracted from top of Pier 11 at Elevation 37.75'. Note cracking in the aggregate and cement paste.

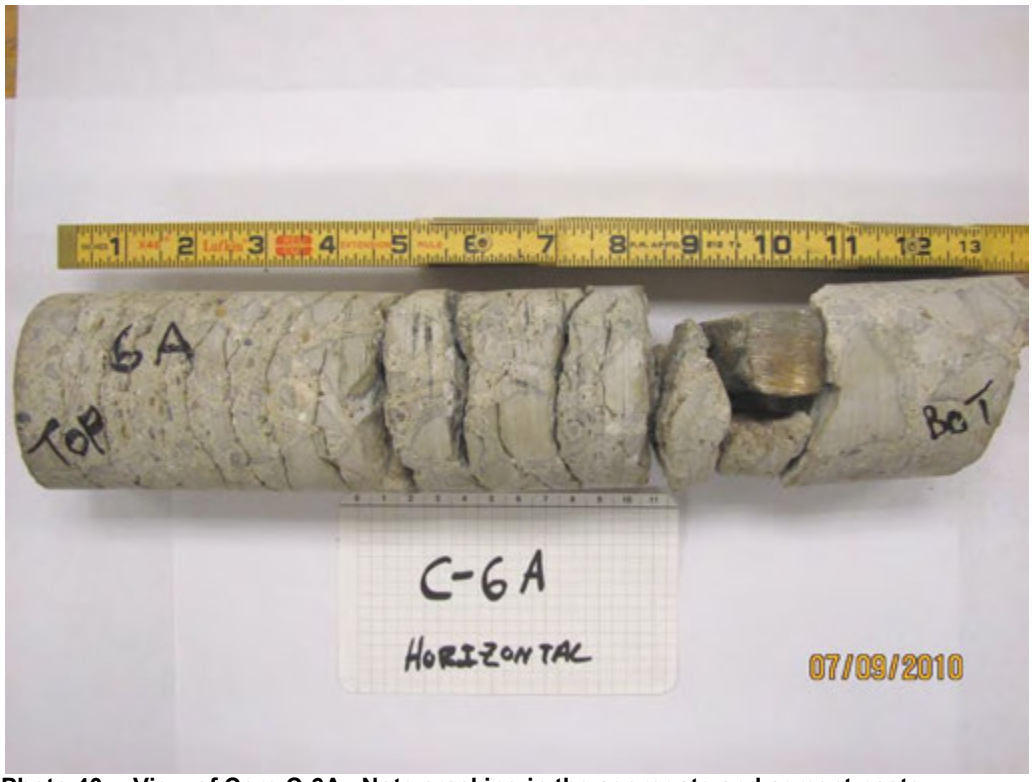


Photo 40: View of Core C-6A. Note cracking in the aggregate and cement paste.

August 12, 2010 Photographs

Photo 41: Typical condition of concrete ice breakers.

Photo 42: Deteriorated top of ice breaker, typical.

August 12, 2010 Photographs

Photo 43: Ice breakers act as "trash racks". Note missing ice breaker at arrow.

Photo 44: View of ice breakers with spillway beyond.

June 25 and August 12, 2010 Photographs

Photo 45: View of spillway at stoplog section, June 2010.

Photo 46: View of spillway at stoplog section, after July 2010 flood. Note bent wood stoplog post.

August 12, 2010 Photographs

Photo 47: Close-up of bent stoplog post, see photo 44.

Photo 48: Masonry facing on spillway near south shoreline.

August 12, 2010 Photographs

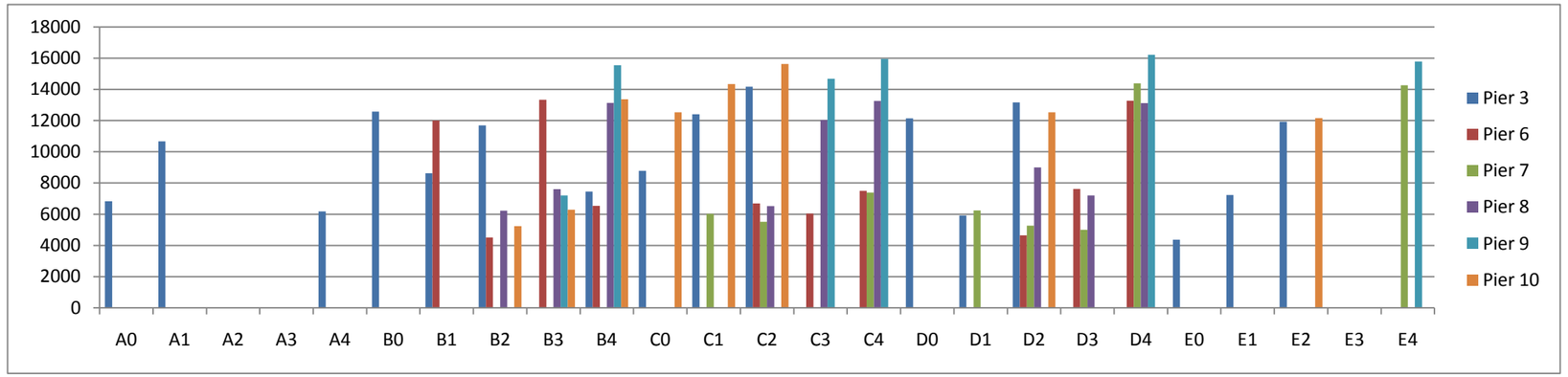
Photo 49: Typical spillway cap condition. Note weathering of long joint with masonry facing and spalling of concrete at arrow.

Photo 50: Spalled concrete at spillway cap with exposed reinforcing steel. Also note weathered mortar joint between concrete and masonry facing.

Appendix C

Laboratory Data UPV, Compressive Strength

Table C - 1: UPV and Laboratory Test Results				
Core Number	Travel Path (inches)	Time (u sec)	Velocity (ft/sec)	Strength (psi)
C1	5.250	25.6	17090	12,140
C2A	4.250	37.5	9444	5440
C3	5.250	28.7	15244	6770
C4	4.500	25.8	14535	6500
C4A	5.000	28.6	14569	7830
C5	4.875	27.3	14881	4190
C5A	4.625	33.6	11471	4720
C6	5.000	41.6	10016	3080



Summary Key -

Vertical Scale - Velocity, feet per second
Horizontal Scale - Grid locations on 6 piers, combined
See Figures 1 and 3, Appendix A

Note - spread of velocity data suggesting variability in piers at 12 inch grids horizontal and vertical

Note - UPV Data not obtained at all grid locations, depending on surface roughness involved

Table C - 2: Field UPV Test Results													
		Pier 3		Pier 6		Pier 7		Pier 8		Pier 9		Pier 10	
Test Location	Travel Path (inches)	Time (u sec)	Velocity (ft/sec)	Time (u sec)	Velocity (ft/sec)	Time (u sec)	Velocity (ft/sec)	Time (u sec)	Velocity (ft/sec)	Time (u sec)	Velocity (ft/sec)	Time (u sec)	Velocity (ft/sec)
A0	44	-	-	-	-	-	-	-	-	-	-	-	-
A1	44	-	-	-	-	-	-	-	-	-	-	-	-
A2	44	536.9	6829	-	-	-	-	-	-	-	-	-	-
A3	44	343.6	10671	-	-	-	-	-	-	-	-	-	-
A4	44	-	-	-	-	-	-	-	-	-	-	-	-
B0	44	-	-	-	-	-	-	-	-	-	-	-	-
B1	44	593.2	6181	305.6	11998		-	-	-	-	-	-	-
B2	44	291.7	12570	813.6	4507	-	-	589.3	6222	-	-	701.0	5231
B3	44	425.2	8623	274.9	13338	-	-	482.6	7598	509.6	7195	583.4	6285
B4	44	313.7	11688	561.2	6534	-	-	279.3	13128	235.8	15550	274.4	13362
C0	44	-	-	-	-	-	-	-	-	-	-	292.6	12531
C1	44	491.9	7454	-	-	608.2	6029	-	-	-	-	255.7	14340
C2	44	417.8	8776	548.3	6687	664.3	5520	562.4	6520			234.6	15629
C3	44	295.6	12404	607.4	6037	0.0		304.8	12030	249.8	14678	-	-
C4	44	258.7	14173	489.3	7494	496.5	7385	276.6	13256	229.9	15949	-	-
D0	44	-	-	-	-	-	-	-	-	-	-	-	-
D1	44	-	-	-	-	586.9	6248	-	-	-	-	-	-
D2	44	301.9	12145	789.2	4646	695.2	5274	407.4	9000	-	-	292.6	12531
D3	44	620.3	5911	481.6	7614	732.4	5006	508.6	7209	-	-	-	-
D4	44	278.4	13170	276.3	13271	254.9	14385	279.4	13123	226.1	16217	-	-
E0	44	-	-	-	-	-	-	-	-	-	-	-	-
E1	44	-	-	-	-	-	-	-	-	-	-	-	-
E2	44	838.6	4372	-	-	-	-	-	-	-	-	301.6	12157
E3	44	507.4	7226	-	-	-	-	-	-	-	-	-	-
E4	44	307.4	11928	-	-	257.2	14256	-	-	232.4	15777	-	-

AECOM

Compressive Strength of Concrete Cores
ASTM C-42

Laboratory Services Group *750 Corporate Woods Parkway Vernon Hills, Il 60061* *Phone: (847) 279-2500 Fax: (847) 279-2550*

Project No.: **60159452** **7/16/10**
Project Name: **Estabrook Dam Rehab**

Date Sampled: **7/9/10** Test Condition: Cut and capped
Date Received: **7/12/10**
Date Tested: **7/16/10**

Summary of Test Results

Core No.	Ht. (in.)	Cap Ht. (in.)	Dia. (in.)	Area $(in)^2$	H/D Ratio	Corr. Factor	Load (lb)	Strength (psi)	Weight (gm.)	Unit Wt. (pcf) *	Location
C-1	5.26	5.40	2.69	5.69	2.01	1.0000	69090	12,140	1210.2	154.1	Pier 1 (Vertical) orig
C-2A	4.29	4.55	2.75	5.94	1.66	0.9728	33200	5,440	975.6	145.8	Pier 2 (Horizontal) orig
C-3	5.25	5.44	2.69	5.70	2.02	1.0000	38570	6,770	1155.1	147.1	Pier 4 (Vertical) repair
C-4	4.59	4.83	2.69	5.70	1.79	0.9832	37680	6,500	1036.1	150.9	Pier 7 (Vertical) orig
C-4A	5.04	5.29	2.74	5.90	1.93	0.9944	46480	7,830	1169.1	149.8	Pier 7 (Horizontal) orig
C-5	4.89	5.14	2.70	5.72	1.91	0.9928	24160	4,190	1102.6	150.0	Pier 9 (Vertical) orig
C-5A	4.68	4.91	2.74	5.89	1.79	0.9832	28260	4,720	1088.5	150.4	Pier 9 (Horizontal) orig
C-6	5.12	5.32	2.70	5.71	1.98	0.9984	17630	3,080	1142.9	149.0	Pier 11 (Vertical) orig

Note: * Unit Weight Calculated Using Weight/Volume Relationship at Time of Test

Appendix D

Updated Structural Stability Analysis – Gated Spillway and Overflow Spillway

GATED SPILLWAY - SHEETS 1 THRU 10

Estabrook Dam - Gravity Stability Analysis

with 80 kip tie down anchor considered in case 2a

Loading Conditions Analyzed: Sep-10

Case 1: Normal Water Levels
Case 2: Design Flood *(Note: Dam is low hazard and submerges under 100 yr flood - no stability analysis needed)*
Case 2a: Normal Water Levels Plus Ice

	Case 1 (Usual)	Case 2a (Unusual)
Headwater Level	36.0	36.0
Tailwater Level	30.75	30.75
Ice Level	N/A	35
USACE Minimum FS	2.0	1.7
Resultant location at base	middle third	middle half

Assumptions:

Single bay analysis - conservatively assume no transfer of loads between piers
Conservatively ignore dead weight of gate, stem, hoist and pillow blocks
Conservatively ignore weight of concrete spillway apron between piers for sliding computations
Conservatively ignore passive resistance of bedrock in front of concrete piers
Conservatively ignore cohesion at concrete / bedrock interface (need cores to verify if cohesion is assumed)

Pier Length =	16.08	ft	
Pier Width =	3.67	ft	
Section Length (mid gate to mid gate) =	15.17	ft	
Rock/Concrete interface elevation =	27.42	ft	
phi (concrete / rock interface) =	45	degrees	(assumed)
Unit Weight (water) =	0.0624	kcf	
Unit Weight (concrete) =	0.15	kcf	
Cohesion =	15.12	ksf	
Unit pressure of Ice =	5.00	k/Lf	

Dead Loads:

Concrete Pier and Walkway

Section	Volume (cf)	Unit Weight (kcf)	Forces (kips) V	Forces (kips) H	Arm (ft)	Moment (ft kips) V	Moment (ft kips) H
1	79.24	0.15	11.89		4.58	54.44	
2	49.92	0.15	7.49		10.33	77.32	
3	10.01	0.15	1.50		7.75	11.63	
4	22.10	0.15	3.31		1.22	4.04	
5	24.15	0.15	3.62		2.33	8.44	
6	277.01	0.15	41.55		6.88	285.67	
7	126.24	0.15	18.94		12.59	238.32	
8	34.69	0.15	5.20		14.86	77.32	
9	38.91	0.15	5.84		3.21	18.71	
10	38.91	0.15	5.84		11.13	64.93	
11	98.64	0.15	14.80		7.17	106.02	
Total	**799.83**		**119.97**			**946.83**	

Live Loads:

	Unit Weight (kcf)	Forces (kips) V	Forces (kips) H	Arm (ft)	Moment (ft kips) V	Moment (ft kips) H
Headwater						
All Cases: H = 0.5 * (36 - 27.42)^2 * 15.17 * 0.0642	0.0624		-34.84	3.75		-130.55
Tailwater						
All Cases: H = 0.5 * (30.75 - 27.42)^2 * 15.17 * 0.0624	0.0624		5.25	2.00		10.48
Ice						
Case 2a: H = 5 * 15.17			-75.85	7.58		-574.94

Uplift:

	Unit Weight (kcf)	Forces (kips) V	Forces (kips) H	Arm (ft)	Moment (ft kips) V	Moment (ft kips) H
All Cases:						
U = (30.75 - 27.42) * 16.08 * 3.67 * 0.0624	0.0624	-12.26		8.04	-98.59	
U = 1/2 * (36.0 - 30.75) * 16.08 * 3.67 * 0.0624	0.0624	-9.67		10.72	-103.62	

Anchor: 80 13.92 1113.6

Load Case 1 - Normal Water Levels

with 80 kip tie down anchor

Load Summary: Sep-10

Forces (kips)		Moments (ft kips)		
V =	199.97	Mv =	2060.43	includes 80 kip tie down
H =	-29.59	Mh =	-120.06	
U =	-21.93	Mu =	-202.22	

Eccentricity (Overturning Analysis):

mu = (Mv + Mh +Mu) / (V + U)
e = (B/2) - mu
% Base = mu / B

Mv + Mh + Mu (ft kips) =	1738.15
V + U (kips) =	178.04
mu (ft) =	9.76
e (ft) =	-1.72
% Base =	**61%**

Resultant falls within the middle third of the base, therefore o.k.

Sliding Stability Analysis:

Factor of Safety:

Where:	tan(phi) =	1.00
	C (ksf) =	0
	Shear Area (ft^2) (base of pier) =	59.01

Assume only the base area of pier is available to resist sliding

Factor of Safety (FS) = {(V+U) * tan(phi) + (Shear Area * C)}/H

FS w/o cohesion = **6.02**

No cohesion required to meet USACE recommended FS = 2.0, therefore o.k.

Note, for phi (concrete / rock) = 31 degrees, FS = 2.0

Load Case 2a - Normal Water Levels Plus Ice

with 80 kip tie down anchor

Load Summary:

Sep-10

Forces (kips)		Moments (ft kips)		
V =	199.97	Mv =	2060.43	includes 80 kip tie down
H =	-105.44	Mh =	-695.01	
U =	-21.93	Mu =	-202.22	

Eccentricity (Overturning Analysis):

mu = (Mv + Mh + Mu) / (V + U)
e = (B/2) - mu
% Base = mu / B

Mv + Mh + Mu (ft kips) =	1163.21	
V + U (kips) =	178.04	
mu (ft) =	6.53	
e (ft) =	1.51	
% Base =	41%	**w/in middle half of base**

Sliding Stability Analysis:

Factor of Safety:

Where:

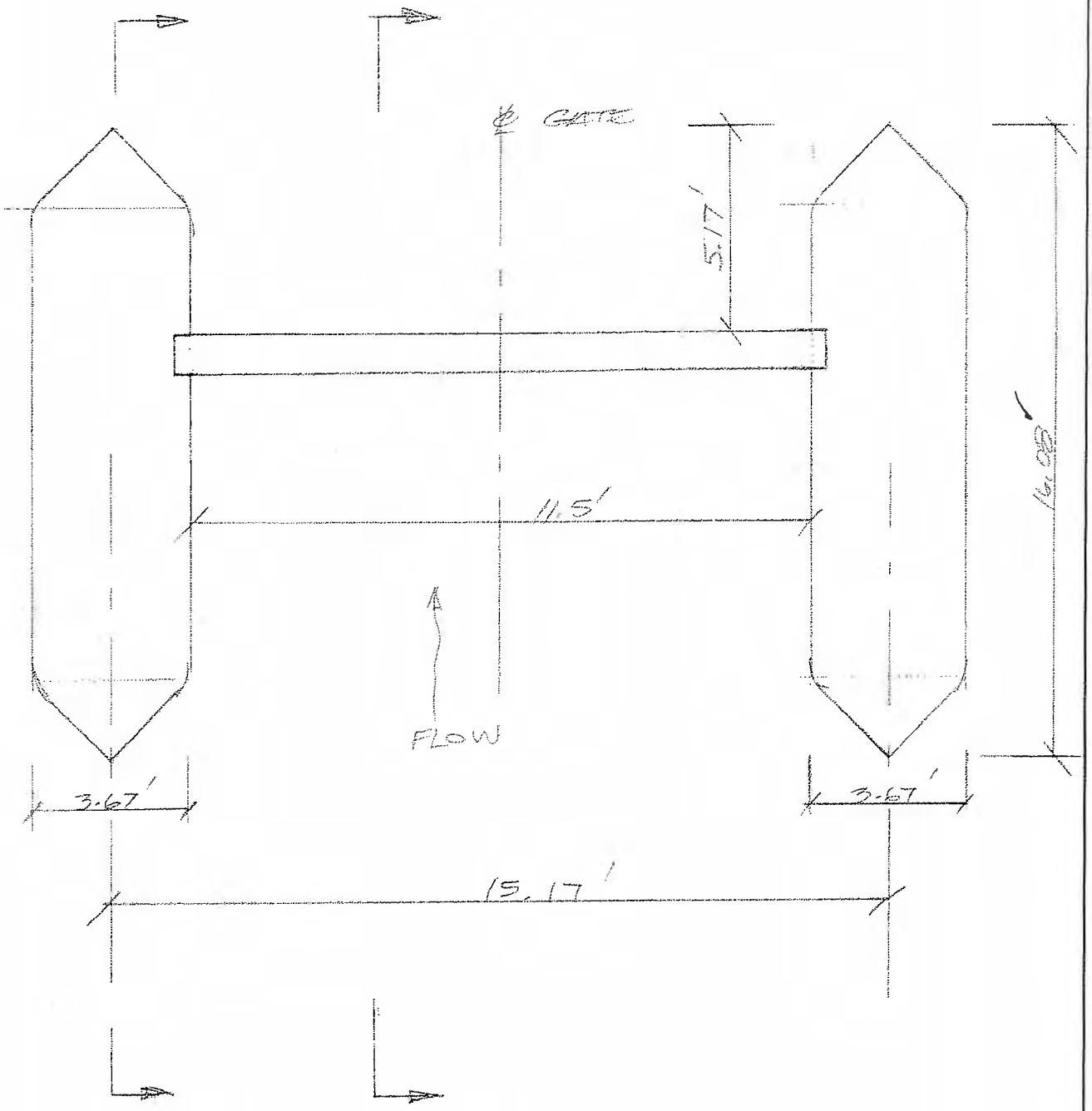
tan(phi) =	1.00	
C (ksf) =	0	
Shear Area (ft^2) (base of pier) =	59.01	Assume only the base area of pier is available to resist sliding.

Factor of Safety (FS) = {(V+U) * tan(phi) + (Shear Area * C)}/H

FS w/o cohesion =	**1.69**	**minimum of 1.7 req'd**

Note that this does not include passive resistance of concrete base against rock at socketed connection, downstream side. Adding the passive resistance will raise the Factor of Safety against sliding.

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK DAM				STS JOB NO.: 587996	
	SUBJECT: GATED SPILLWAY STABILITY				SHEET NO.: 4 OF 16	
	ORIGINATED BY: [illegible]	DATE: 7/14/06	CHECKED BY:	DATE:	CALC. NO.:	REV. NO.:



STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK DAM				STS JOB NO. 587996	
	SUBJECT: PIER CONTROL				SHEET NO. 5 OF 16	
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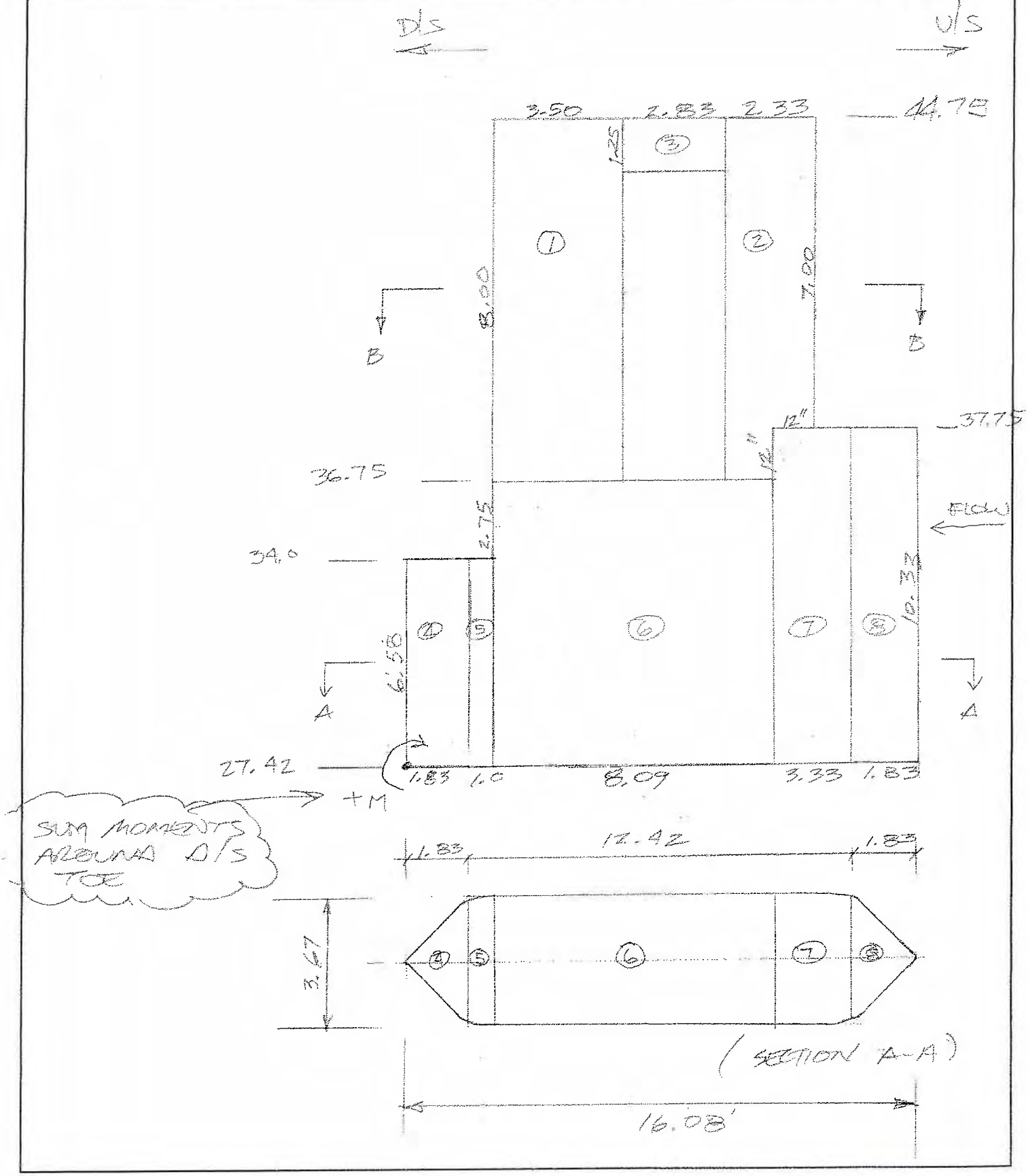

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK DAM				STS JOB NO. 87996	
	SUBJECT: GATE SECTION				SHEET NO. 6 OF 16	
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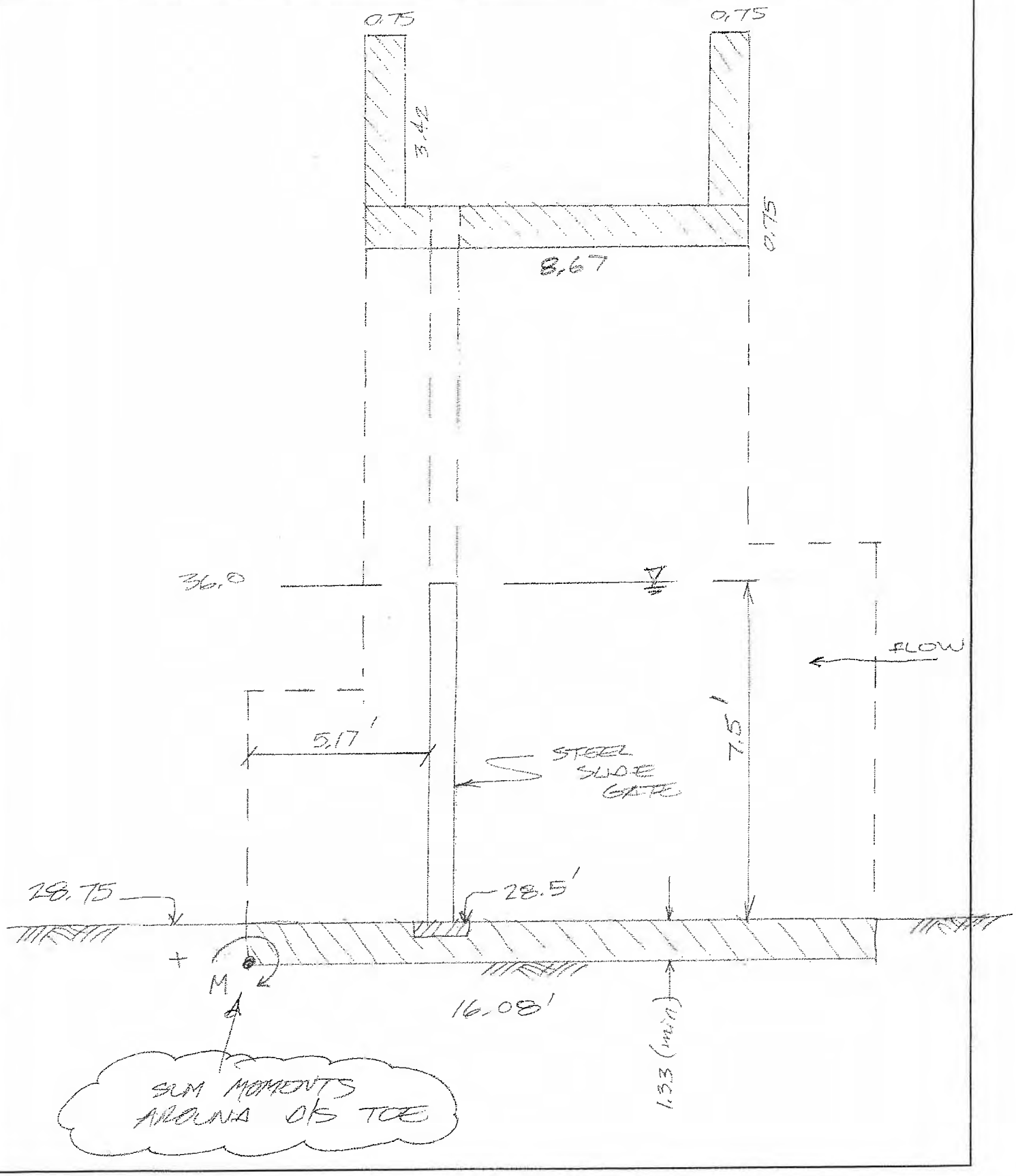
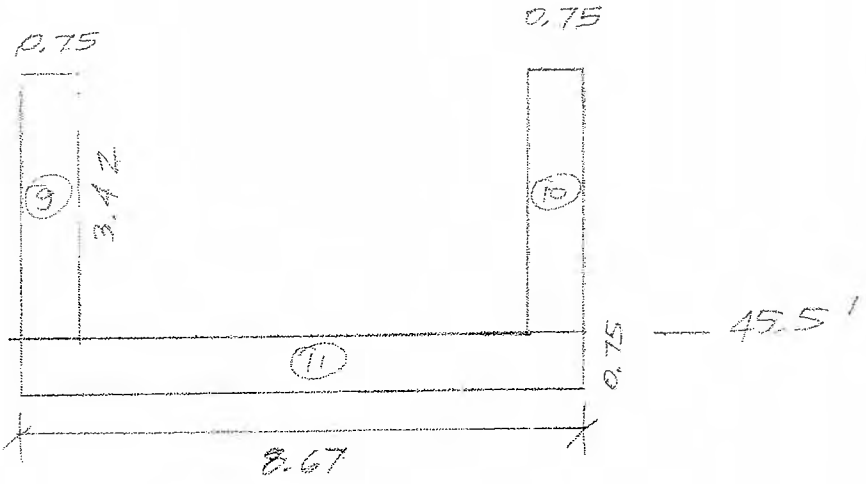
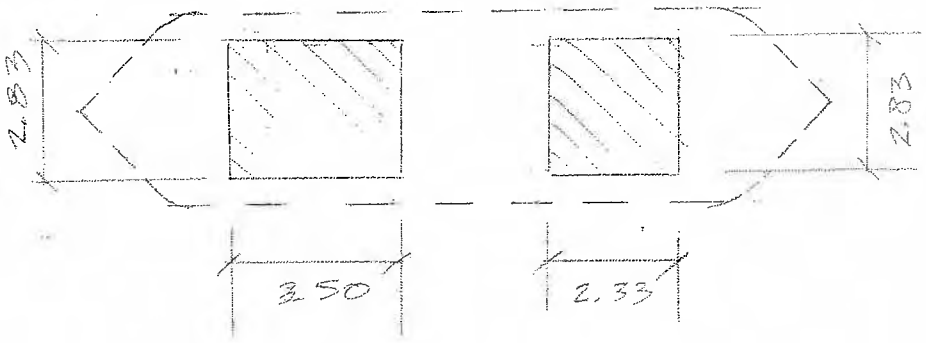


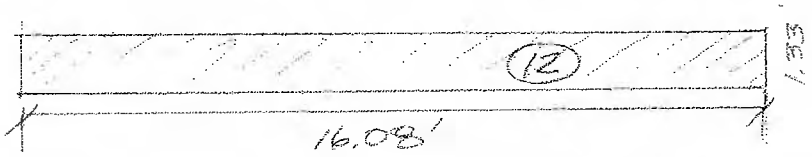
STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK DAM				STS JOB NO.: 587996	
	SUBJECT: WALKWAY & PIER SECTIONS				SHEET NO.: 7 OF 16	
	ORIGINATED BY: RJA	DATE: 7/14/06	CHECKED BY:	DATE:	CALC. NO.:	REV. NO.:



(WALKWAY)



(SECTION B-B)



(SPILLWAY APRON BETWEEN PIERS)

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTABROOK DAM				STS JOB NO.: 587996	
	SUBJECT: GATED SPILLWAY STABILITY				SHEET NO.: 8 OF 16	
	ORIGINATED BY: RJA	DATE: 7/14/06	CHECKED BY:	DATE:	CALC. NO.:	REV. NO.:

CONCRETE WEIGHT

① 0.15(8.0)3.5(2.83) = 11.886 K

② 0.15[(7.0)2.33(2.83) + (1.0)1.33(2.83)] = 7.488 K

③ 0.15(2.83)1.25(2.83) = 1.502 K

④ 0.15(1/2) 3.67(1.83) 6.58 = 3.314 K

⑤ 0.15(1.0) 3.67(6.58) = 3.622 K

⑥ 0.15(8.09) 9.33(3.67) = 41.552 K

⑦ 0.15(10.33) 3.33(3.67) = 18.937 K

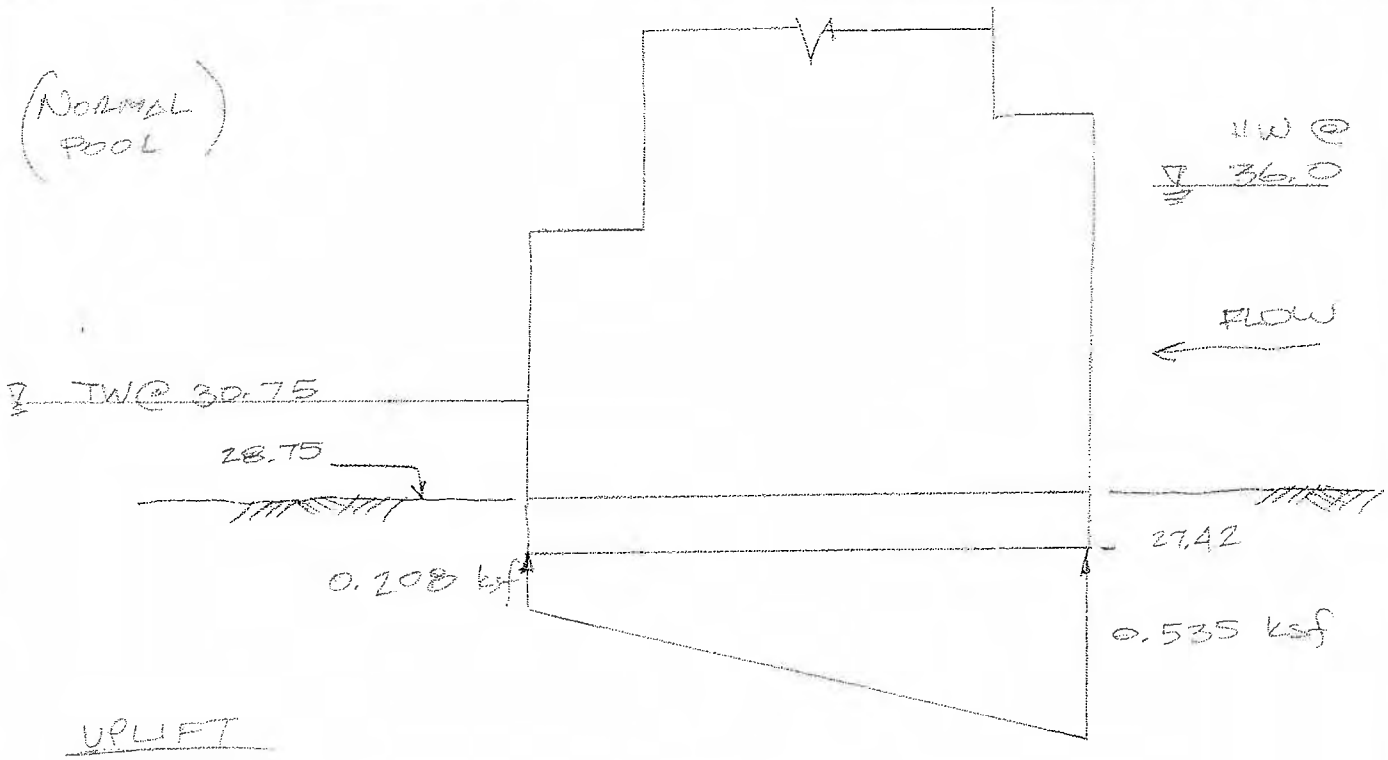
⑧ 0.15(1/2) 3.67(1.83) 10.33 = 5.203 K

⑨ 0.15(0.75) 3.42(15.17) = 5.837 K

⑩ 0.15(0.75) 3.42(15.17) = 5.837 K

⑪ 0.15(0.75) 8.67(15.17) = 14.796 K

STS Consultants Ltd. CALCULATION SHEET	PROJECT: ESTERBROOK DAM				STS JOB NO.: 587996	
	SUBJECT: UPLIFT & HYDROSTATIC LOADS				SHEET NO.: 9 OF 16	
	ORIGINATED BY: RJA	DATE: 7/14/06	CHECKED BY:	DATE:	CALC. NO.:	REV. NO.:

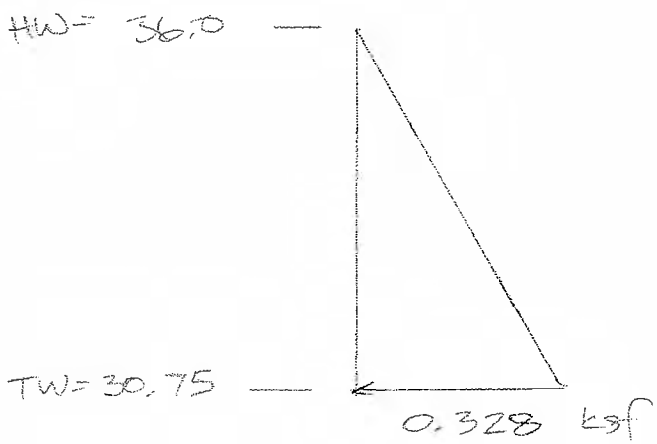


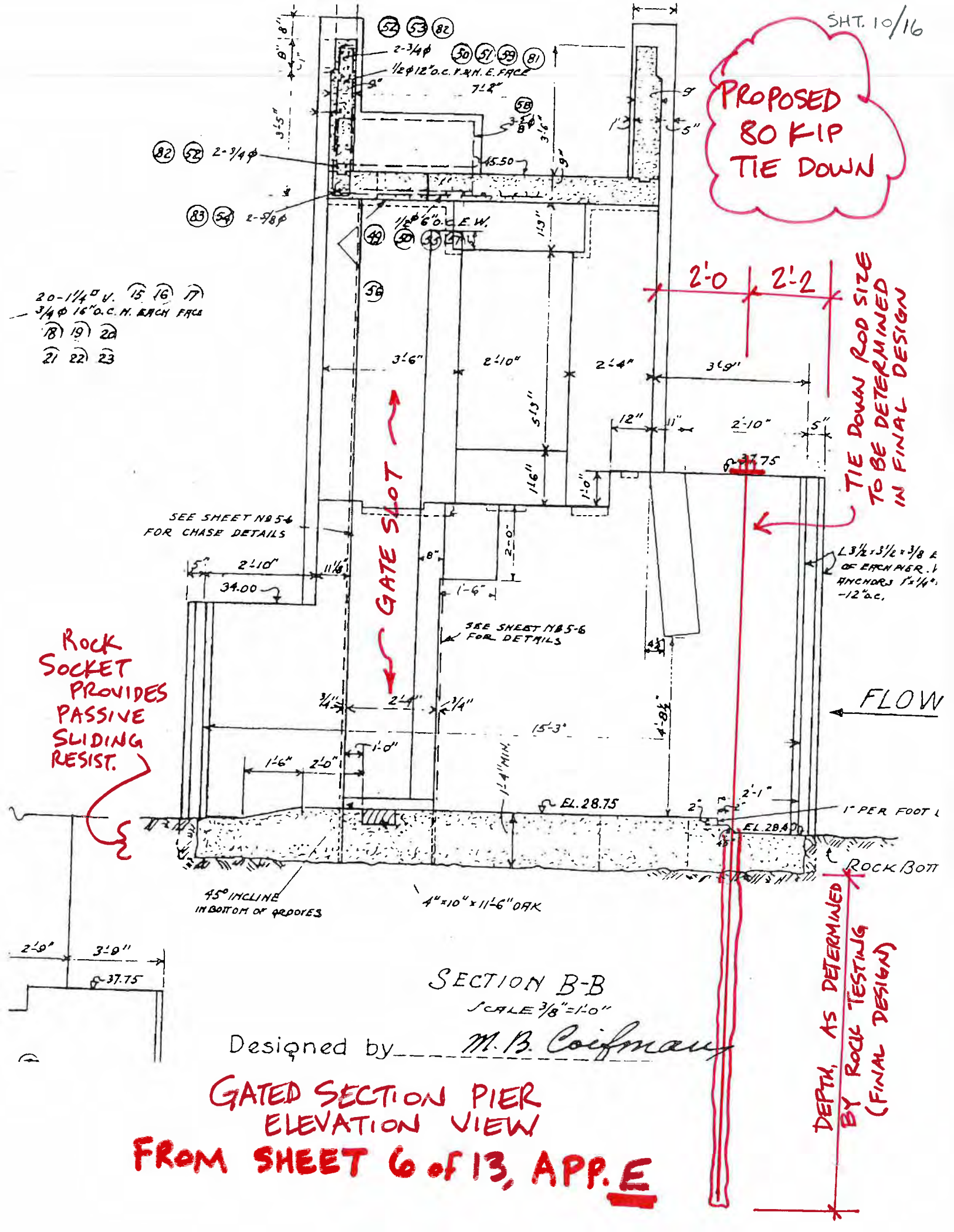
UPLIFT

$$u_{u/s} = 0.0624(36.0 - 27.42) = 0.535 \text{ ksf}$$

$$u_{d/s} = 0.0624(30.75 - 27.42) = 0.208 \text{ ksf}$$

HYDROSTATIC LOADS





SECTION B-B

SCALE 3/8"=1'-0"

Designed by M. B. Coifman

GATED SECTION PIER ELEVATION VIEW

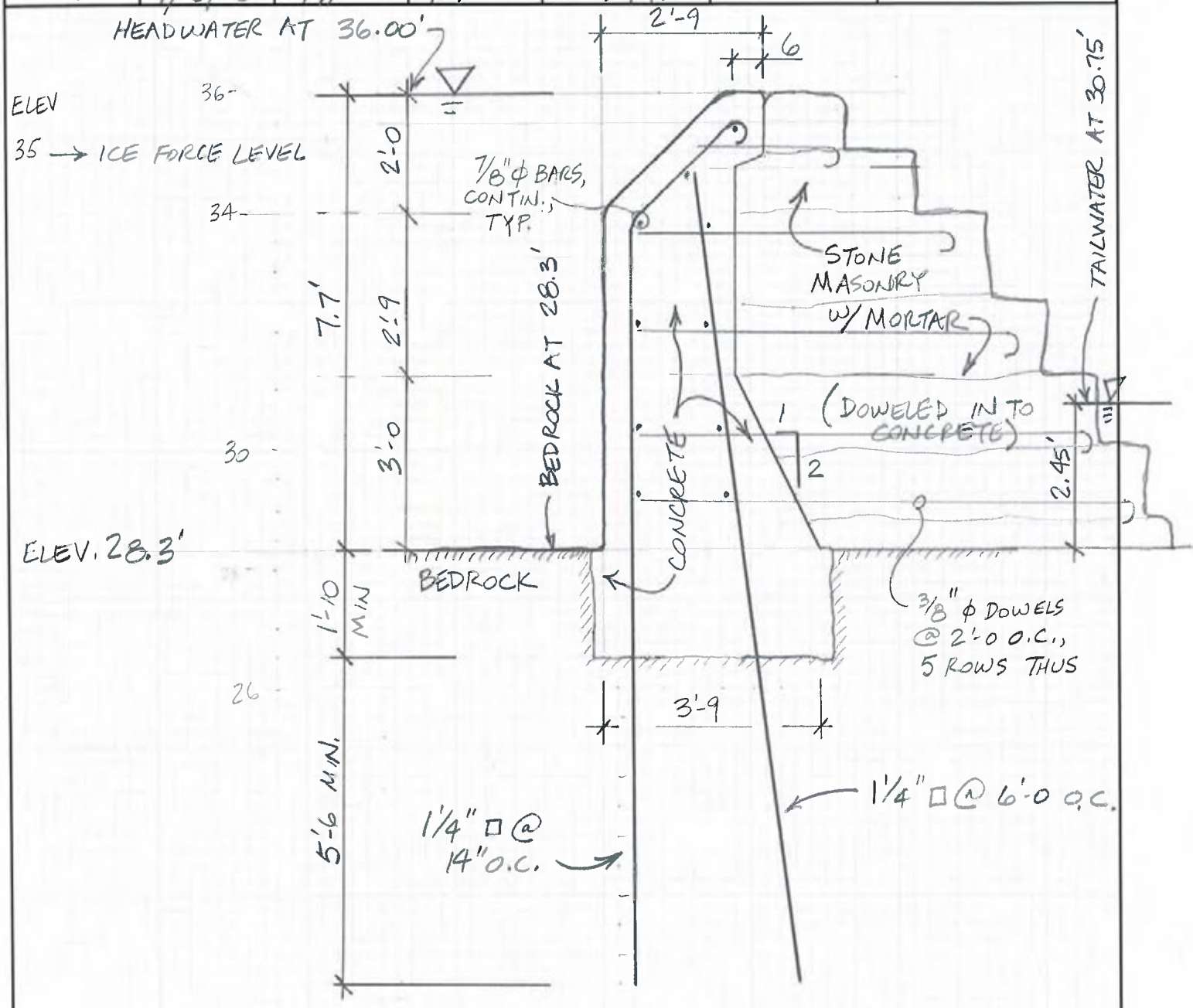
FROM SHEET 6 of 13, APP. E

AECOM

Calculation Sheet

Project	Subject
ESTABROOK DAM	OVERFLOW SPILLWAY ANALYSIS

Originated By	Date	Checked By	Date	~~STS~~ Job No.	Scale	Sheet No.
SAE	9/6/10	TH	9-7-10	60159452		11 of 16



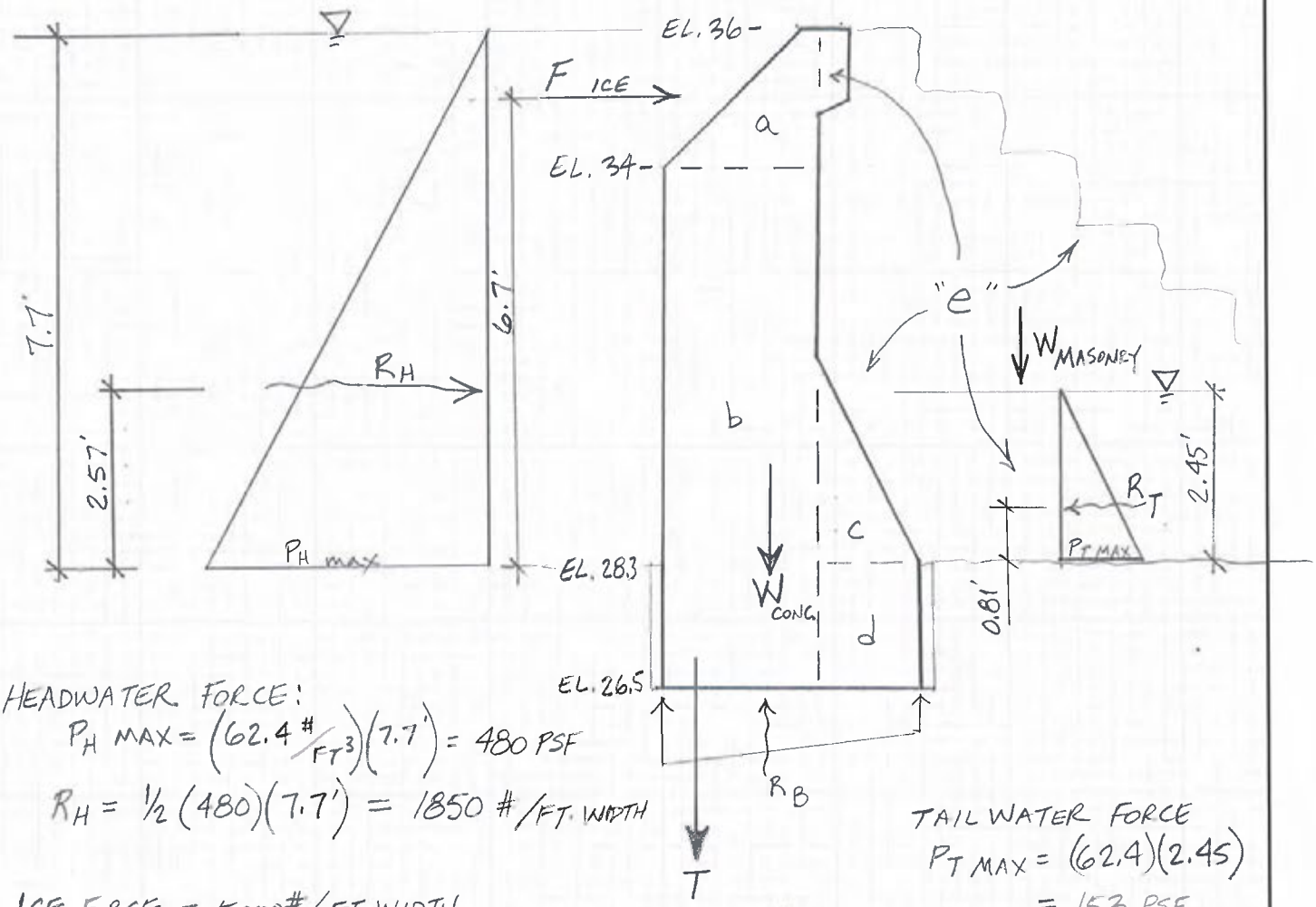
OVERFLOW SPILLWAY CROSS SECTION

DETAILS FROM ORIGINAL 1937 DRAWINGS

NOTE: SPILLWAY IS CURVED SHARPLY THROUGH OUT ITS 562 FOOT LENGTH. WITH CONTINUOUS HORIZONTAL REINFORCING, THIS GEOMETRY ADDS GREAT STABILITY TO THIS 2-D ANALYSIS.

Calculation Sheet

Project	Subject					
ESTABROOK DAM	OVERFLOW SPILLWAY ANALYSIS					
Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No. 12 Of 16
		TH	9-7-10			



HEADWATER FORCE:

$P_{H\,MAX} = (62.4\,\#/FT^3)(7.7') = 480\ PSF$

$R_H = \frac{1}{2}(480)(7.7') = 1850\ \#/FT.\ WIDTH$

ICE FORCE = 5000#/FT. WIDTH
F_{ICE}

TAIL WATER FORCE

$P_{T\,MAX} = (62.4)(2.45)$
$= 153\ PSF$

$R_T = \frac{1}{2}(153)(2.45)$
$= 187\ \#/FT.\ WIDTH$

LOADING DIAGRAM

PER FOOT WIDTH

AREA OF CONCRETE

$a = \frac{1}{2}(2.0' \times 2.3') = 2.3\ FT^2$

$b = (34.0 - 26.5)(2.3') = 17.25\ FT^2$

$c = \frac{1}{2}(3' \times 1.5) = 2.25\ FT^2$

$d = 1.8' \times 1.5' = 2.7\ FT^2$

$\Sigma = 24.5\ FT^2$

$W_{CONCRETE} = 24.5\ FT^2 \times 150\ \#/FT^3 = 3675\ \#/FT.\ WIDTH$

THE MASONRY IS MORTARED AND DOWELED TO THE CONCRETE.

CONSERVATIVELY, AREA OF MASONRY EQUALS 1¼ TIMES AREA OF CONC.

$W_{MASONRY} \simeq 1.25(24.5)(120\ PCF) = 3675\ \#$

Calculation Sheet

Project	Subject					
ESTABROOK DAM	OVERFLOW SPILLWAY ANALYSIS					
Originated By: SAE	Date: 9/6/10	Checked By: TH	Date: 9-7-10	STS Job No.	Scale	Sheet No. 13 of 16

BOUYANCY UPLIFT FORCES -

AT UPSTREAM SIDE $B_{UP} = \left(\frac{62.4\#}{FT^2}\right)(36.0' - 26.5') = 593\#/FT^2$

AT DOWNSTREAM SIDE OF CONC. SECTION $B_{DN} = (62.4)(30.75' - 26.5') = 265\#/FT^2$

UNDER MASONRY $B_{MASY} = (62.4)(30.75 - 28.3) = 153\#/FT^2$ (TAIL WATER)

$B_2 = (153\#/FT^2)(6' \text{ LONG}) = 918\#/\text{FT WIDTH}$

AVG. "B1": $\frac{593+265}{2} = 429\#/FT^2$

$B_1 = 429 \times 3.75' = 1608\#/\text{FT WIDTH}$

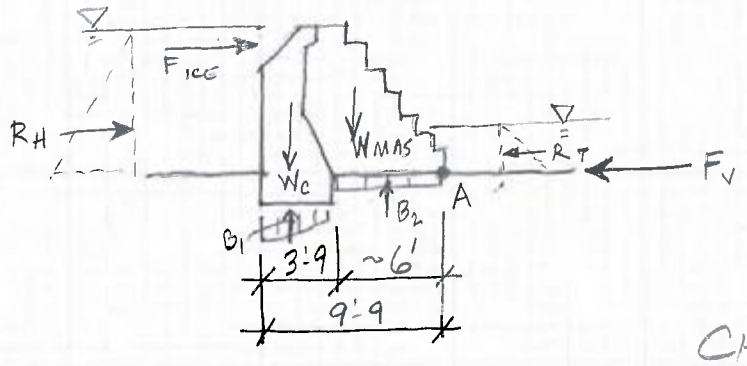
SHEAR FORCE THROUGH CONCRETE KEY

(CONSERVATIVELY IGNORE FRICTION UNDER MASONRY)

$f'_c = 3000\,psi$

$F_V = 2\sqrt{f'_c} = 110\,psi$

$110\,psi\,(43'')\left(\frac{12''}{FT}\right) = 56.5^K/\text{FT WIDTH}$



CHECK SLIDING FACTOR OF SAFETY UNDER FULL POOL WITH ICE

FORCES PUSHING $= F_{ICE} + R_H = 5000\# + 1850\#$ PER FT WIDE

$= 6850\#$

FORCES RESISTING $= R_T + F_V = 187\# + 56{,}500\#$

$= 56{,}687\#$ (per foot width)

CONSERVATIVE: CONCRETE ONLY $\rightarrow F_S$ SLIDING $= \frac{56687}{6850} = 8.3 > 1.7$ ✓

AECOM

Calculation Sheet

Project	Subject					
ESTABROOK DAM	OVERFLOW SPILLWAY ANALYSIS					
Originated By SAE	Date 9/6/10	Checked By TH	Date 9-7-10	STS Job No.	Scale	Sheet No. 14 of 16

CHECK OVERTURNING FACTOR OF SAFETY w/o CONSIDERING ANCHOR TIE DOWN ASSISTANCE.....

ABOUT POINT "A", PREV. PAGE

$$\Sigma M_{OVERTURNING} = F_{ICE} \times (35.0 - 28.3) \quad \leftarrow 33.5^{K\cdot FT}$$

(F_{ICE} = 5 K/LF)

$$+ R_H \times 2.57' \quad \leftarrow 4.75\ K\cdot FT$$

$$+ B_1 \times 9' \quad \leftarrow 14.5\ K\cdot FT$$

(B_1 = 1.6 KB; CONSERV. ESTIMATE)

$$+ B_2 \times 3' \quad \leftarrow 2.75\ K\cdot FT$$

(B_2 = 0.9 KB)

$$M_o = 55.5\ K\cdot FT$$

$$\Sigma M_{RIGHTING} = W_C \times 8' \quad \leftarrow 3.675^K \times 8' = 29.4\ K\cdot FT$$

$$+ W_{MASY} \times 4' \quad \leftarrow 3.675 \times 4' = 14.7\ K\cdot FT$$

$$+ R_T \times 0.81' \quad \leftarrow 0.187^K \times 0.81' = 0.15\ K\cdot FT$$

$$M_R = 44.25\ K\cdot FT$$

FACTOR OF SAFETY, OVERTURNING w/o TIE DOWN OR 3-D CURVED ANALYSIS $= \frac{44.25}{55.5} = 0.80 < 1.7$ NO GOOD

DETERMINE TIE DOWN FORCE REQ'D FOR F.S. = 1.7, NOT CONSIDERING 3-D CURVED CONSERVATISM....

$$FS = 1.7 = \frac{M_R}{M_o} \Rightarrow M_R = 1.7\ M_o \quad M_{IN} = 94.4\ K\cdot FT$$

$$M_{TIE\ DOWN} = 94.4 - \underbrace{44.25}_{\text{REST OF } M_R} = 50\ K\cdot FT$$

MOMENT ARM OF UPSTREAM TIE DOWNS $\doteq 9.4'$

FORCE (per foot) FOR TIE DOWNS $= \frac{50\ K\cdot FT}{9.4\ FT} = 5.3^K/FT$ WIDTH

AECOM

Calculation Sheet

Project	Subject					
ESTABROOK DAM	OVERFLOW SPILLWAY ANALYSIS					
Originated By SAE	Date 9/6/10	Checked By TH	Date 9-7-10	STS Job No.	Scale	Sheet No. 15 of 16

TIE DOWN, CONT'D --

DESIGN DRAWINGS CALL OUT 1¼" □ BARS AT 14" O.C. (1.17')

LOAD PER BAR (NOT INCL. DOWNSTREAM BARS @ 6'-0 O.C.)

$$= \frac{5.3^{K} \quad 1.17' \text{ APART}}{\text{FT WIDTH}} = 6.24^{K}/\text{BAR}$$

$$\text{STEEL STRESS} = \frac{6.24\,K}{1.25^2\ IN^2} = 4.0\ KSI$$

ALLOWABLE STEEL STRESS ≐ 20 KSI

OK FOR STEEL STRESS

CHECK GROUTING LOAD.....

LENGTH OF EMBEDMENT = 5.5'

$$\text{LOAD PER FOOT OF GROUTED ROD} = \frac{6.24^{K}}{5.5'} = 1.13^{K}/FT$$

OK BY INSP.
(PLUS ADD'L DOWELS @ 6' O.C.)

CONCLUSION -

THE OVERFLOW SPILLWAY, MADE OF CONCRETE TIED TO BEDROCK AND INTEGRAL MASONRY ADDED DOWNSTREAM, WAS ANALYZED AS A SIMPLE 2-DIMENSIONAL (STRAIGHT LINE) STRUCTURE. IT WAS FOUND TO BE STRUCTURALLY ADEQUATE FOR SLIDING AND OVERTURNING. THE SERPENTINE SHAPE OF THE SPILLWAY ADDS TO ITS STRUCTURAL STABILITY

OK.

ALSO NOTE THAT THE 45° ANGLE SLOPE AT THE TOP OF THE WALL CAUSES THE ICE SHEET TO SLIDE UP AND OVER THE WALL, REDUCING THE ICE LOADING ON TOP OF THE STRUCTURE.

AECOM

Calculation Sheet

Project	Subject					
OVERFLOW SPILLWAY ANALYSIS	LOAD CASE 2a - NORMAL POOL + ICE					
Originated By: SAE	Date: 9/6/10	Checked By: TH	Date: 9/28/10	Job No.: 60159452	Scale:	Sheet No. 16 of 16

FIND LOCATION OF RESULTANT (WITHIN MIDDLE 1/2 ?)

SEE SKETCH, SHEET 3, OVERFLOW SPILLWAY ANALYSIS....

LOAD SUMMARY - (PER FOOT OF SPILLWAY LENGTH)

COMPONENT	FORCES	MOMENT ARM (ABOUT A)	MOMENT
R_H	1850#	2.57'	+ 4.75 K·FT
F_{ICE}	5000#	6.7'	+ 33.5 K·FT
$W_{C.}$	3675#	8'	- 29.4 K·FT
B_1	1608#	9'	+ 14.5 K·FT
$W_{MAS.}$	3675#	4'	- 14.7 K·FT
B_2	918#	3'	+ 2.75 K·FT
R_T	187#	0.81'	- 0.15 K·FT
TIE DOWN	5.3K	9'	- 47.7 K·FT
		$\Sigma M =$	- 36.5 K·FT

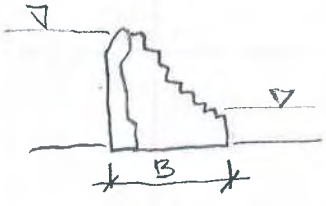
$$\Sigma F_V = \underset{W_C}{3675} - \underset{B_1}{1608} + \underset{W_{MAS.}}{3675} - \underset{B_2}{918} + \underset{\text{TIE DOWN}}{5300}$$

$$\Sigma F_V = 10{,}124\#$$

LOC'N OF RESULTANT $\rightarrow \ell_R = \dfrac{\Sigma M}{\Sigma F_V} = \dfrac{36500\ \#FT}{10124\#} = 3.61\ FT$

B = Base width of 9.9

$$e = \frac{B}{2} - \ell_R = \frac{9.75'}{2} - 3.61' = 1.27'$$



$$\% \text{ of base} = \left(\frac{3.61'}{9.75'}\right) 100 = 37\%$$

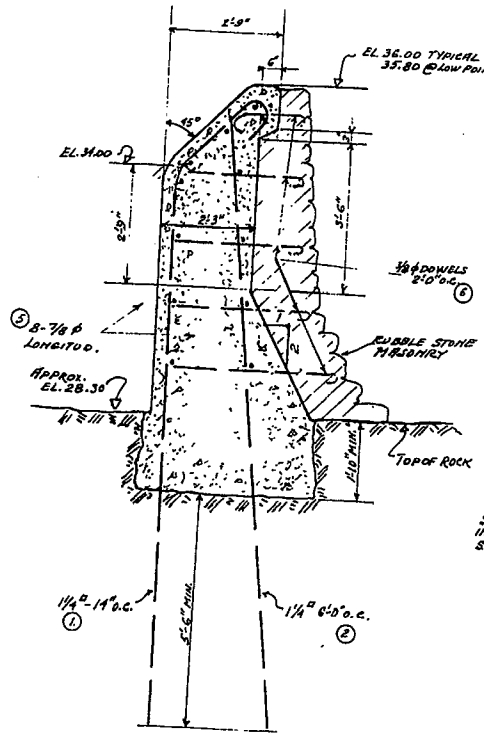
$75\% > \underline{37\%} > 25\%$ OK

CONCLUSION - WITH MINIMUM TIE DOWN FORCE OF 5.3K ASSUMED (FOR OVERTURNING), RESULTANT FORCE IS WITHIN MIDDLE 1/2 OF BASE. ✓OK

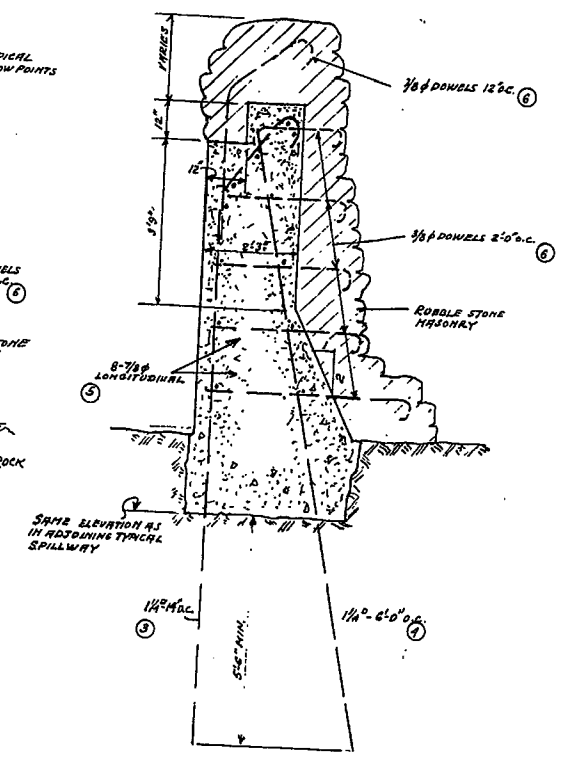
AGAIN, THE 45° ANGLE AT TOP 2 FEET OF WALL SHOULD CAUSE THE 5000 PLF ICE FORCE TO SLIDE UP, RELIEVING THE DAM AND ADDING TO THE FACTOR OF SAFETY OF THE OVERFLOW SPILLWAY SECTION

Appendix E

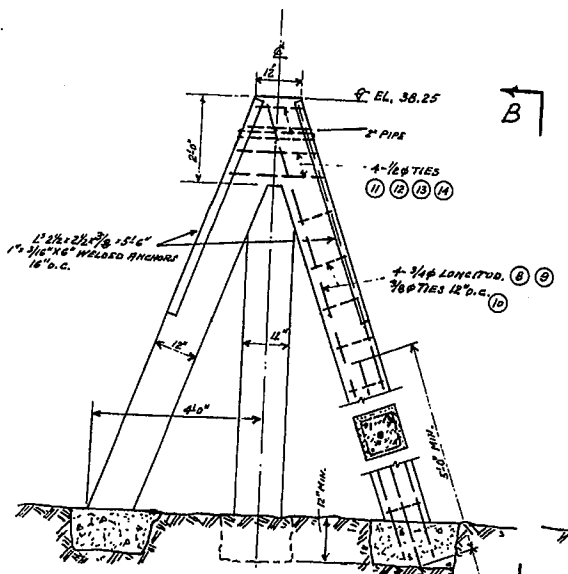
Original 1937 Construction and 1988 Repair Drawings



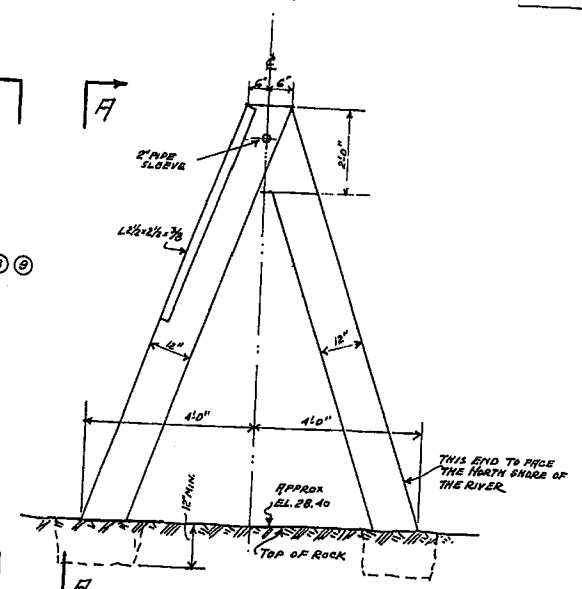
TYPICAL SPILLWAY SECTION
SCALE 1/2" = 1'-0"



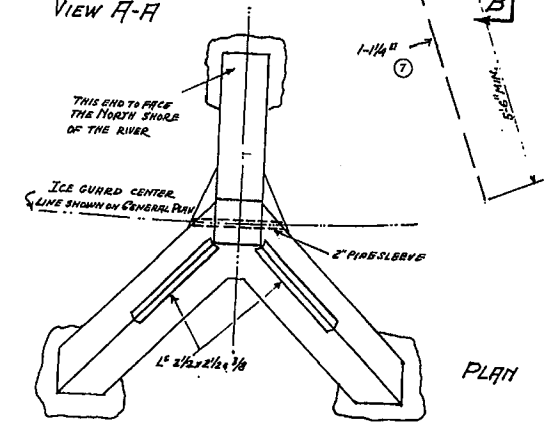
SPILLWAY ABUTMENT SECTION
SCALE 1/2" = 1'-0"



VIEW A-A



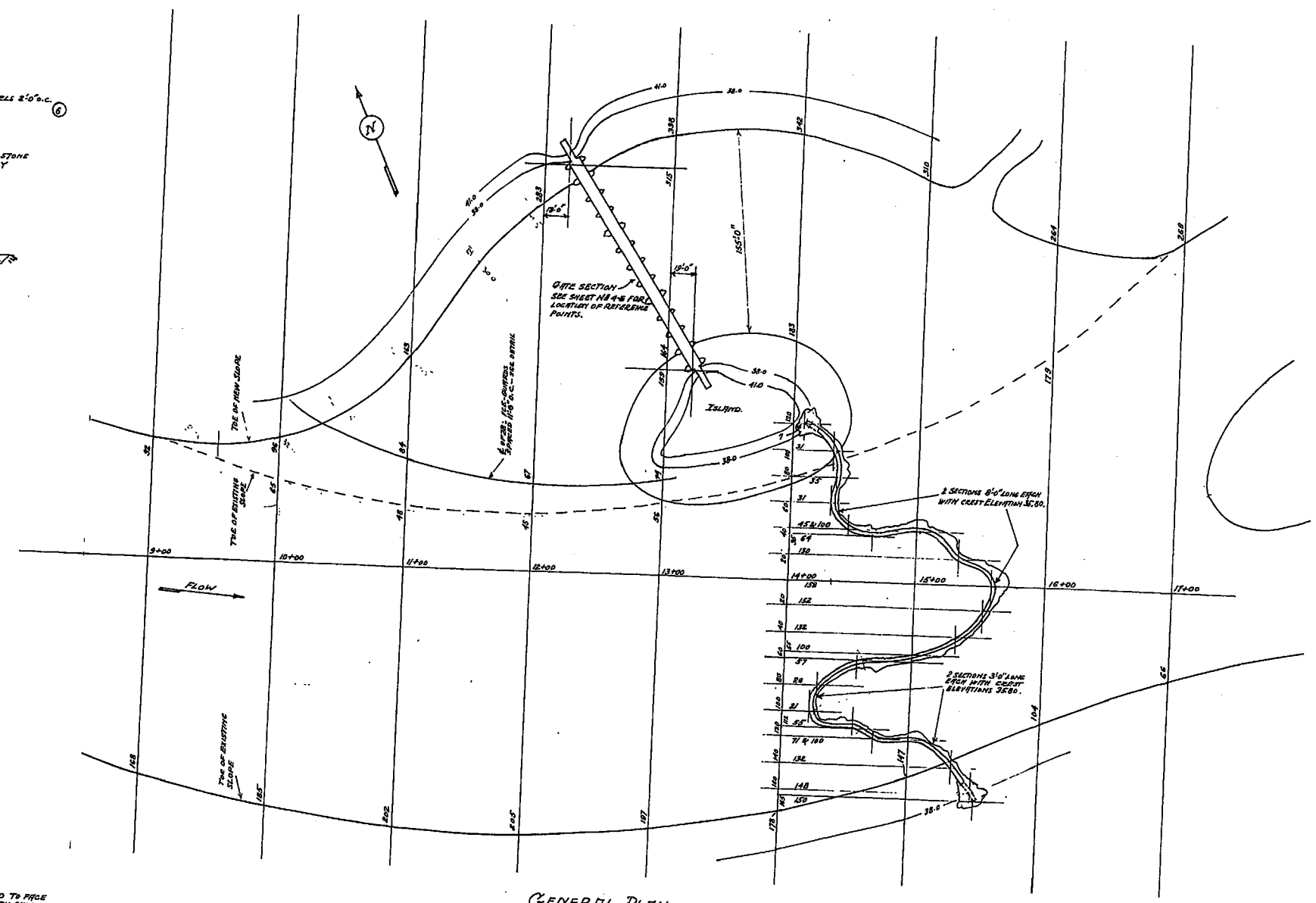
VIEW B-B



PLAN

ICE-GUARDS
(28 REQUIRED)
SCALE 1/2" = 1'-0"

24 INSTALLED
SAE 9/28/10



GENERAL PLAN
SCALE 1" = 50'-0"

NOTE:

DISTANCES FROM STATION 14+00 TO CURVED SPILLWAY REFER TO THE UPSTREAM FACE OF THE SPILLWAY

TOTAL LENGTH OF CURVED SPILLWAY 562 FEET OF WHICH 513 F. WITH CREST ELEVATION 36.00 AND 22 F. WITH CREST ELEVATION 35.80.

RUBBLE STONE MASONRY SHALL EXTEND UPSTREAM AND DOWNSTREAM OF ABUTMENTS NOT LESS THAN 12 F. AND SHALL BE BUILT UP TO EL. 40.00 MINIMUM, ON EACH END OF THE DAM SECTIONS.

ON THE EAST SHORE OF THE ISLAND, NEAR CURVED SPILLWAY ABUTMENT, A WINDING FISH LADDER CHANNEL OF RUBBLE STONE MASONRY SHALL BE PROVIDED.

CONSTRUCTION JOINTS SHALL HAVE 2"×4" KEYS SPACED 8' O.C.

REINFORCING BARS WAS SHOWN MOSTLY. IN TYPICAL LOCATIONS, BUT SHALL BE PROVIDED IN ALL SIMILAR LOCATIONS.

ALL 1 1/4" BARS ANCHORED IN THE ROCK SHALL BE PLACED IN CLEAN 2 1/2" DIAM. HOLES, MIN. 5'-6" DEEP AND FILLED IN ADVANCE WITH GROUT THROUGH 4" PIPE EXTENDING TO THE BOTTOM OF THE HOLE.

ALL REINFORCING BELOW EL. 38.00 SHALL BE PLACED 2 1/2" CLEAR FROM THE ADJOINING CONCRETE FACE.

ABOVE EL. 38.00 CONCRETE COVERING SHALL BE 1" IN SLABS AND PARAPET WALLS AND 1 1/2" ELSEWHERE. IN ICE-GUARDS THE CONCRETE COVERING SHALL BE 1 3/4".

THREE PIECE OF BAR HOOKS SHALL BE 9 1/2 CIRCLE OF A DIAMETER EQUAL TO 5 BAR SIZES.

SPLICES IN REINFORCING BARS SHALL BE MIN. 45 DIAMS LONG AND IN GROUP OF PARALLEL BARS THEY SHALL BE STAGGERED.

MASONRY DOWELS WILL BE CUT IN FIELD FROM 30'-0" BARS.

STEEL 1" ANCHORS IN PIERS AND ICE-GUARDS SHALL BE SANDBLASTED AND HOT-DIP GALVANIZED AFTER FABRICATION.

QUANTITIES.

CONCRETE — 1190 CU. YDS.
REINFORCING STEEL — 148,200 LBS.

24X

MICROFILMED

Designed by M. B. Coifman
Consulting Engineer

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

ESTABROOK PARK DAM

PLOT PLAN; ICE GUARD & SPILLWAY DETAILS

LANDSCAPE DESIGN SECTION	ENGINEERING DESIGN SECTION
Alfred L. Bourum, Landscape Architect	Roy Behrens, Senior Ass't. Civil Engineer

Survey
Plan
Tracing
Checked

Approved: E. A. Normand, Supervising Engineer — Date 4/24/37
Approved: W. [illegible], Highway Commissioner & Planning Engineer — Date 5/20/37

DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
AND
MILWAUKEE COUNTY PARK COMMISSION
CO-OPERATING

BRANCH OF PLANNING — STATE PARK DIVISION

Prepared by Jerome A. [illegible] Date
Secretary Milwaukee County Park Commission

Recommended Theo. P. Whitnall Date 7-21
Inspector

Approved Date
Regional Officer

S.P.-WIS.
9006-3-8
SCALES AS SHOWN

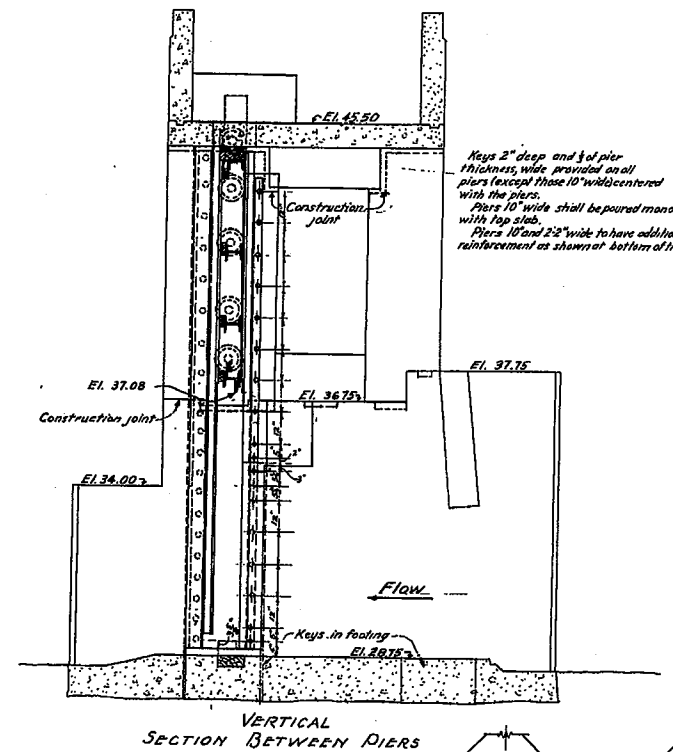
SYMBOL
NUMBER 130
DATE

SHEET 3 OF 13 SHEETS

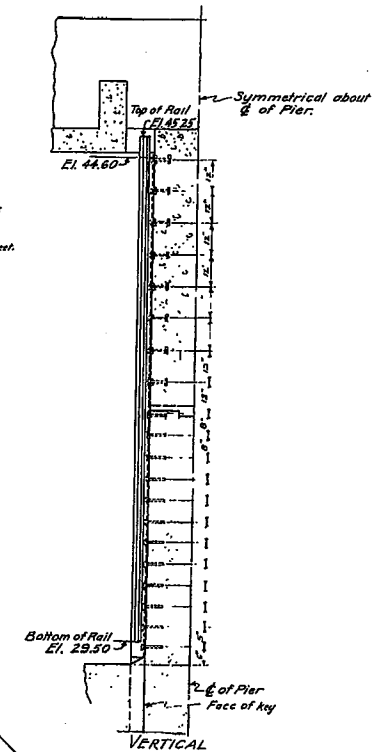
49-D-30

BILL OF REINFORCING BARS

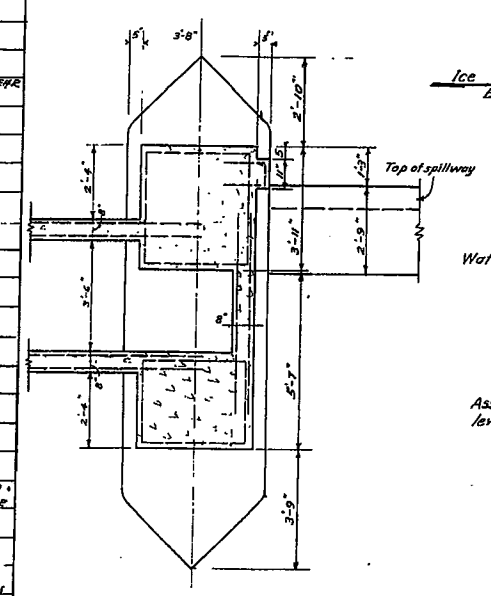
MARK	NO.	SIZE	LENGTH	SPACING	LOCATION	REMARKS
1	459	1¼□	16-3	14"	CURVED SPILLWAY	
2	90	1¼□	14-3	6'-0"	"	
3	24	1¼□	16-0	14"	"	ABUTMENTS
4	4	1¼□	14-6	6'-0"	"	"
5	170	7/8 φ	30-0		"	LONGITODINAL
6	164	3/8	30-0	2'-0"	"	MASONRY DOWELS CUT IN FIELD
7	84	1¼□	10-9		ICE - GUARDS	
8	224	3/4 φ	11-9		"	
9	112	3/4 φ	11-0		"	
10	644	3/8 φ	3-6	12"	"	TIES
11	28	1/2 φ	7-6		"	"
12	28	1/2 φ	6-6		"	"
13	28	1/2 φ	5-3		"	"
14	28	1/2 φ	4-3		"	"
15	91	1¼□	12-0		PIERS	DOWNSTREAM END
16	78	1¼□	14-9		"	CENTER
17	91	1¼□	15-9		"	UPSTREAM END
18	56	3/4 φ	16-9	16"	3'-8" PIERS	LOWER 4 ROWS
19	28	3/4 φ	9-6	16"	"	INTERMID. 2 ROWS
20	14	3/4 φ	5-6	16"	"	TOP ROW, UPSTREAM END
21	48	3/4 φ	17-9	16"	4'-6" PIERS	LOWER 4 ROWS
22	24	3/4 φ	10-0	16"	"	INTERMID. 2 ROWS
23	12	3/4 φ	6-0	16"	"	TOP ROW, UPSTREAM END.
24	143	1□	15-6		PIERS ON DOWNSTREAM END	
25	21	1□	19-0		"	BELOW RAILING IN 2'-10" PIERS.
26	24	1□	19-6		"	BELOW RAILING IN 3'-8" PIERS.
27	109	1□	12-9		PIERS ON UPSTREAM END	
28	21	1□	16-3		"	BELOW RAILING IN 2'-10" PIERS
29	24	1□	16-9		"	BELOW RAILING IN 3'-8" PIERS
30	112	1/2 φ	12-3	12"	PIERS ON DOWNSTREAM SIDE	2'-10" PIERS
31	32	1/2 φ	14-9	12"	"	3'-8" PIERS
32	64	1/2 φ	11-9	12"	"	2'-6" PIERS
33	77	3/8 φ	8-3	10"	"	10" PIERS
34	77	3/8 φ	3-9	10"	"	"
35	16	1/2 φ	19-6		"	3" THICK BITUMINOUS FIBRE EXPANSION JOINTS
36	124	1/2 φ	6-3	12"	PIERS	"
37	16	1/2 φ	16-9		PIERS ON UPSTREAM END	"
38	91	1/2 φ	10-0	12"	"	2'-10" PIERS
39	26	1/2 φ	12-6	12"	"	3'-8" PIERS
40	52	1/2 φ	9-6	12"	"	2'-6" PIERS
41	56	1/2 φ	7-6		RAILING OVER 2'-10" PIERS	
42	80	1/2 φ	6-6		8" WALLS BETWEEN PIERS	
43	12	1/2 φ	10-0		8" WALLS IN ABUTMENT PIERS	
44	116	3/4 φ	8-3		DOWELS INTOP OF PIERS	
45	110	1/2 φ	6-0		RAILING OVER PIERS	
46	16	1/2 φ	2-9		RAILING RETURN FOR ABUTMENT PIERS	
47	102	3/4 φ	30-0	12" o.c.	BOTTOM SLAB	
48	84	3/4 φ	14-9	16" o.c.	"	
49	188	1/2 φ	9-6	12"	TOP SLAB BETWEEN PIERS	(ALTERNATE 49 AND 50 BARS THAT WOULD CROSS THE CHAIN DRAINING SPALLS WILL BE PLACED NEXT TO IT.
50	138	1/2 φ	17-6	12"	"	
51	280	1/2 φ	8-0	12"	RAILING BETWEEN PIERS	
52	40	3/4 φ	17-0		BOTTOM OF RAILING	(AT SPLICES PLACE BARS IN 2 LAYERS
53	40	3/4 φ	18-0		TOP OF RAILING	
54	40	5/8 φ	17-0		SLAB BELOW RAILING	
55	180	1/2 φ	17-0	6"	SLAB	LONGITODINAL
56	10	1/2 φ	10-0		"	INLINE WITH CHAIN OPENINGS
57	30	1/2 φ	16-3	6"	"	END PANELS LONGIT.
58	60	1/2 φ	13-0		GRIDS FOR SHAFT BEARINGS	
59	120	1/2 φ	17-0	12"	RAILING	LONGITODINAL
60	68	3/8 φ	8-0		UNDER PILLOW BLOCKS	HORIZ. ENDS AND BENDUP IN RAILING.
61	34	1¼□	13-0	20" & 36"	ABUTMENTS	
62	136	1/2 φ	19-6	12"	"	HORIZ. INSIDE WALLS
63	88	1/2 φ	18-0	12"	"	VERT. IN SIDE WALLS. LENGTHS TO BE FITTED IN FIELD.
64	48	1/2 φ	19-6	12"	"	
65	128	1/2 φ	6-6	6"	"	SLAB AND STRIPS
66	12	1/2 φ	20-0	12"	"	"
67	12	5/8 φ	8-6	12"	"	STIFFENER WALL
68	32	5/8 φ	7-0	12"	"	BENT LIKE 65
69	12	1/2 φ	13-0	12"	"	END WALL VERTIC.
70	24	1/2 φ	7-6	12"	"	" - BENT LIKE 65
71	24	1/2 φ	4-6	12"	"	"
72	12	1 φ	14-0		SPILLWAY AT THE END OF GATE SECTIONS	
73	24	3/4 φ	13-6		"	
74	14	3/4 φ	13-6	16"	"	
75	14	1/2 φ	10-0	16"	"	
76	30	1/2 φ	7-0	10"	TOP OF SLAB UNDER HOISTS	SEE NOTE ON SHEET NO 2
77	98	1/2 φ	9-0	6"	TOP OF SLAB	OVER 2'-10" PIERS
78	40	3/8 φ	4-6	8"	"	AT EXPANSION JOINTS
79	62	3/8 φ	6-0	10"	10" PIERS DOWNSTREAM END	
80	62	3/8 φ	1-9	10"	"	
81	24	1/2 φ	16-6	12"	RAILING AT END PANELS	LONGIT.
82	16	3/4 φ	16-6		"	TOP & BOTTOM
83	8	5/8 φ	16-6		"	SLAB BELOW
84	35	½ φ	6-6	2.4"	PIERS ON DOWNSTREAM SIDE	2'-10" PIERS
85	20	½ φ	2-0	"	"	3-8" PIERS
86	20	½ φ	2-3	"	"	3-8" PIERS



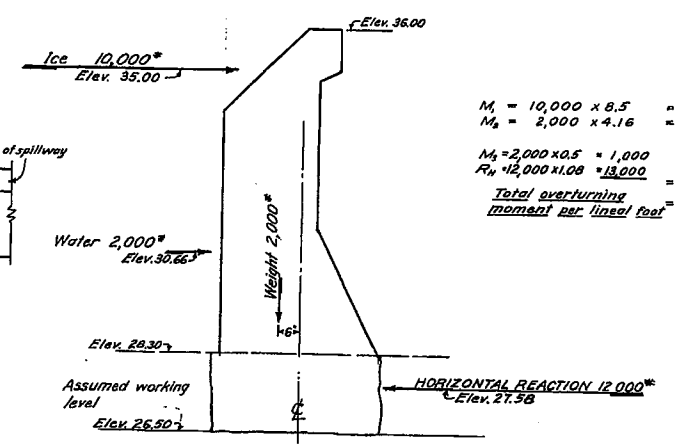
VERTICAL SECTION BETWEEN PIERS WITH GATE RAISED TO MAX. ELEV.
SCALE ⅜" = 1'-0"



VERTICAL SECTION THRU GATE PIER AT GATE GUIDE
SCALE ⅜" = 1'-0"

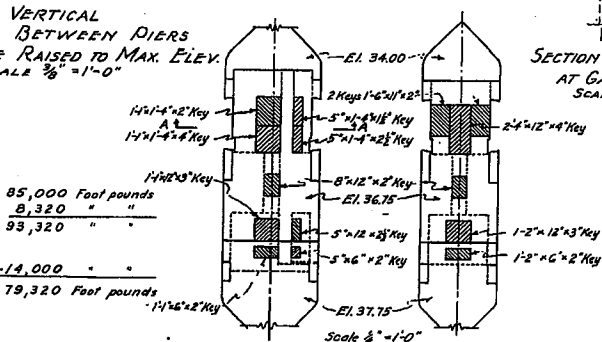


END PIER - SECTIONAL PLAN AT ELEVATION 40.00
Reinforcement as called for in 3'-8" piers

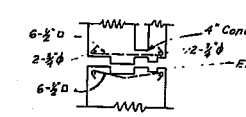


M_1 = 10,000 x 8.5 = 85,000 Foot pounds
M_2 = 2,000 x 4.16 = 8,320 " "
93,320 " "
M_3 = 2,000 x 0.5 = 1,000
R_H = 12,000 x 1.08 = 13,000
= -14,000 " "
Total overturning moment per lineal foot = 79,320 Foot pounds

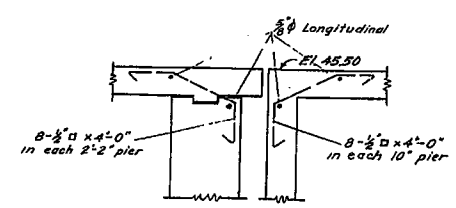
DIAGRAM OF FORCES APPLIED TO SPILLWAY
SCALE ½" = 1'-0"



ARRANGEMENT OF KEYS AT CONSTRUCTION JOINT IN PIERS



SECTION AA
SHOWING INTERLOCKING OF KEYS



ADDITIONAL STEEL IN 2'-2" AND 10" PIERS

Designed by M. B. Coifman
Consulting Engineer

Revised	May 18, 1937.	
Revised	May 19, 1937.	E.J.E
Revised	Sept. 29, 1937.	E.J.E
	Nov. 2, 1937.	E.J.E

20X

MICROFILMED

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

ESTABROOK PARK DAM

REINFORCING STEEL LIST — TYPICAL SECTIONS

LANDSCAPE DESIGN SECTION — Landscape Architect

ENGINEERING DESIGN SECTION — Senior Ass't. Civil Engineer

Survey ..... Plan ..... Tracing ..... Checked .....

Approved: Supervising Engineer — Date 4/24/37

Approved: Highway Commissioner & Planning Engineer — Date 5/20/37

DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
AND
MILWAUKEE COUNTY PARK COMMISSION
CO-OPERATING

BRANCH OF PLANNING — STATE PARK DIV.

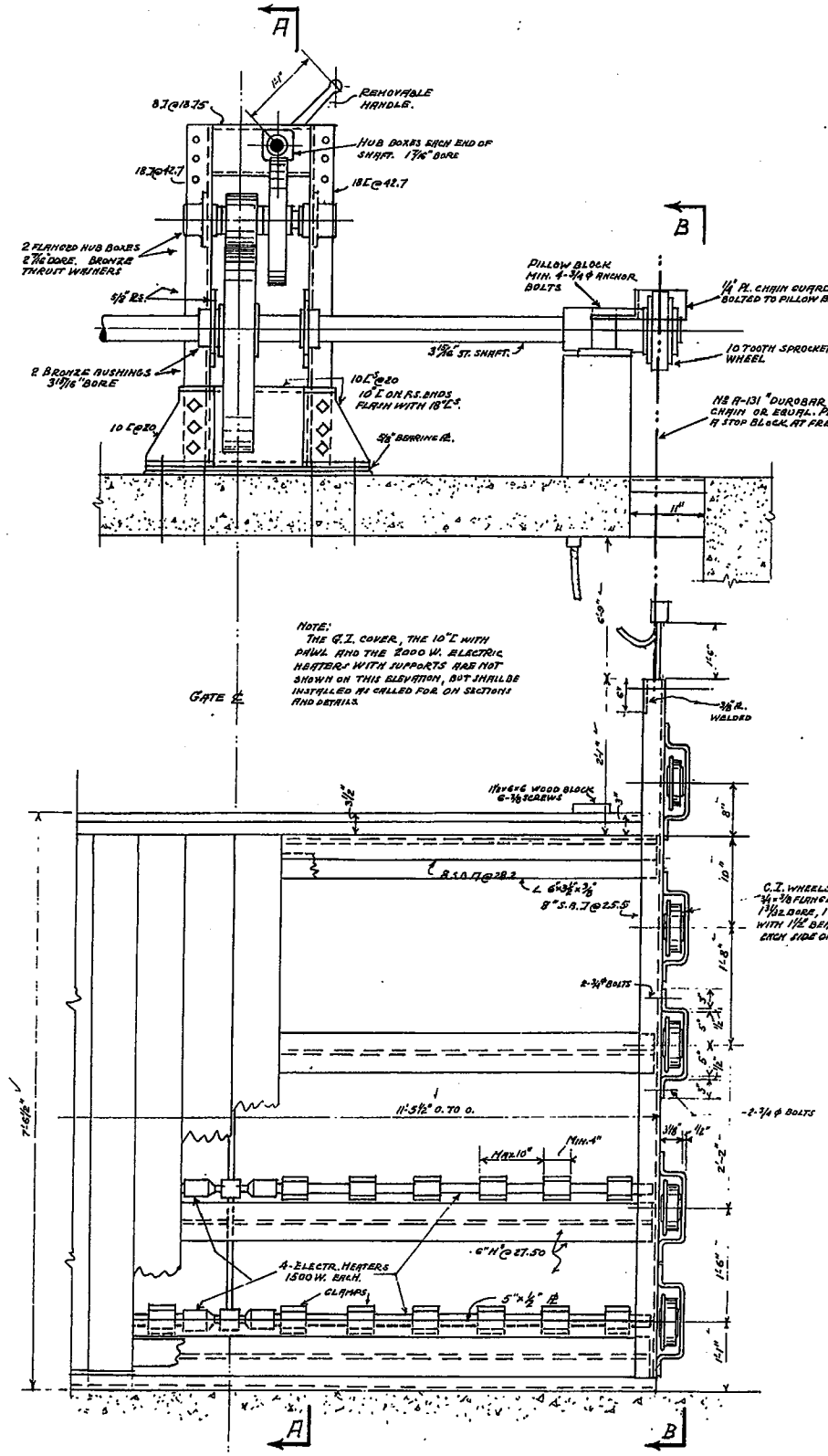
Prepared by ..... Date ..... Secretary Milwaukee County Park Commission

Recommended ..... Date 5/21 Inspector

Approved ..... Date ..... Regional Officer

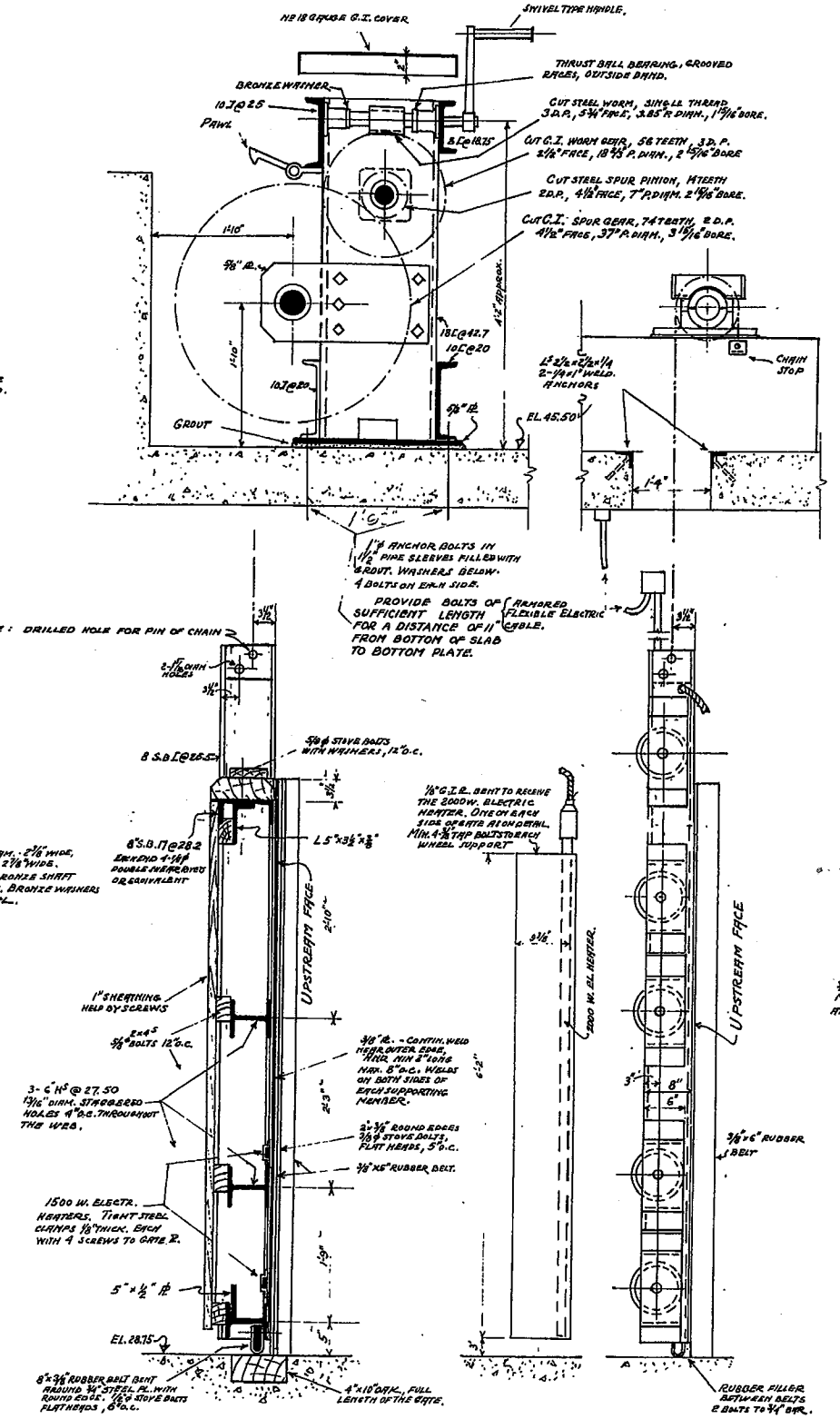
S.P.-WIS. 9006-G-8

SHEET 4 OF 18 SHEETS



DOWN STREAM ELEVATION

SCALE 1"=1'-0"

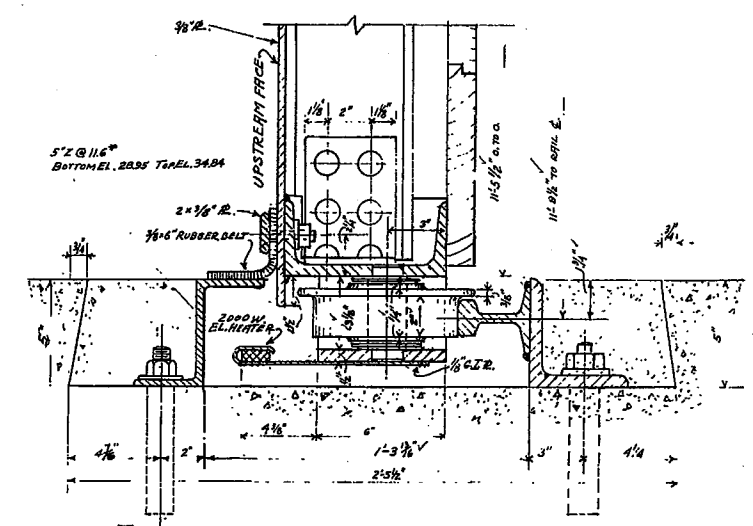


SECTION A-A

SCALE 1"=1'-0"

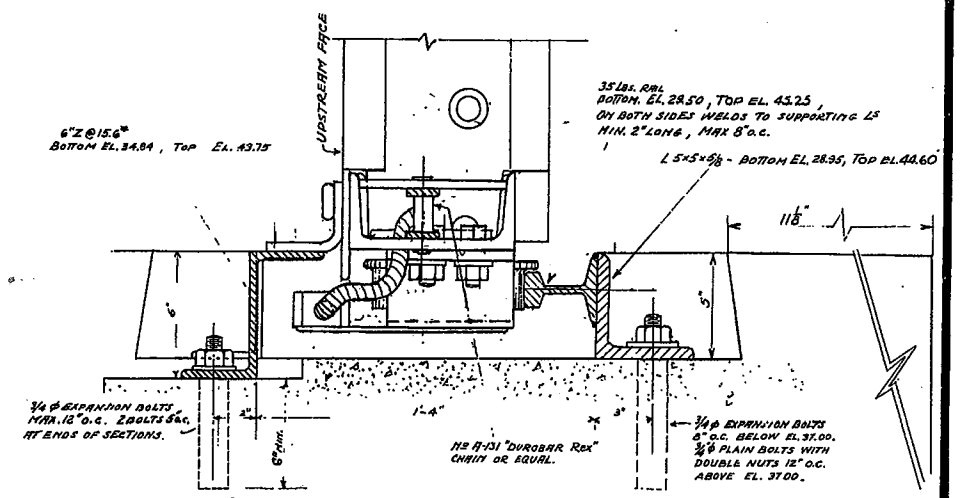
SECTION B-B

SCALE 1"=1'-0"



TYPICAL GATE BEARING BELOW EL. 34.75

SCALE 3"=1'-0"



TYPICAL GATE BEARING ABOVE EL. 36.75

SCALE 3"=1'-0"

NOTE:

GATES, HEATERS AND ACCESSORIES ARE SYMMETRICAL ABOUT GATE ℄, EXCEPT THAT ELECTRIC CONNECTION FOR HEATERS IS AT NORTH END OF EACH GATE

TEN GATES WITH ALL ACCESSORIES AS SHOWN SHALL BE INSTALLED ON THE DAM.

ALL BOLTS, NUTS, SCREWS AND WASHERS SHALL BE CADMIUM PLATED AFTER FABRICATION OF EACH, EXCEPT THOSE IN GATE GUIDES

WELDED CONNECTIONS OF EQUIVALENT STRENGTH CAN BE USED IN LIEU OF RIVETS

ALL ELECTRIC WIRING ON THE GATES AND CONNECTIONS SHALL BE ENTIRELY WATERPROOF.

ALL WOOD USED IN GATE CONSTRUCTION SHALL BE CREOSOTED.

ALL BOLTS SHALL HAVE WASHERS AND HEXAGON NUTS.

BOLTS, NUTS & WASHERS HOLDING THE GATE GUIDES SHALL BE HOT DIP GALVANIZED AND THE NUTS SHALL HAVE OVERSIZE THREADS TO ALLOW EASY OPERATION AND THE STRENGTH OF STANDARD NUTS.

IN THE HOIST ALL HOLES SHALL BE REAMED AND BOLTS CONNECTING STRUCTURAL SHAPES MIN. 7/8" DIAM.

ALL SHAFT BEARINGS OF THE HOIST SHALL BE BABBITT OR BRONZE AND SHALL BE PROVIDED WITH "ALEMITE" GREASE FITTINGS.

ALL GEARS SHALL BE STANDARD MAKE OF A SINGLE MANUFACTURER AND SHALL BE EQUAL TO THOSE MADE BY A. O. JAMES CO., W. R. JONES CO. OR FOOTE BROS.

CHAINS AND SPROCKET WHEELS SHALL BE STANDARD MAKE OF A SINGLE MANUFACTURER AND EQUAL TO THOSE MADE BY CHAIN BELT CO., LINK BELT CO. OR JEFFREY MFG. CO.

PROVIDE SUITABLE KEYS AND SCREWS TO HOLD IN PROPER POSITION ALL GEARS, WHEELS AND SHAFTS UNDER THE MAXIM. STRAIN THAT THESE MEMBERS CAN EXERT.

GUIDES FOR THE GATES SHALL BE SAND-BLASTED AND HOT-DIP GALVANIZED AFTER FABRICATION.

IN GATES AND HOIST ALL EXPOSED EDGES CUT SHARP SHALL BE ROUNDED OUT.

Designed by M. B. Coifman Consulting Engineer

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

ESTABROOK PARK DAM

GATES AND ACCESSORIES

LANDSCAPE DESIGN SECTION	ENGINEERING DESIGN SECTION
Alfred L. Brenner, Landscape Architect	Roy E. Behrens, Senior Ass't. Civil Engineer

Survey ..... Plan ..... Tracing ..... Checked .....

Approved: E. A. Stonecrop, Supervising Engineer ... Date 4/24/37

Approved: M. B. Coifman, Highway Commissioner & Planning Engineer ... Date 5/20/37

Revised May 18, 1937 R.E.B.
Revised May 19, 1937 R.E.B.
Revised Sept. 29, 1937
Revised Oct. 18, 1937
" Nov. 8, 1937 E.L.E.
" Nov. 30, 1937 R.E.B.

DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
AND
MILWAUKEE COUNTY PARK COMMISSION
CO-OPERATING

BRANCH OF PLANNING — STATE PARK DIVISION

Prepared by ..... Secretary Milwaukee County Park Commission ... Date .....

Recommended ..... Inspector ... Date 5-7-21

Approved ..... Regional Officer ... Date .....

S.P.-WIS
9006-5-8

SCALES AS SHOWN

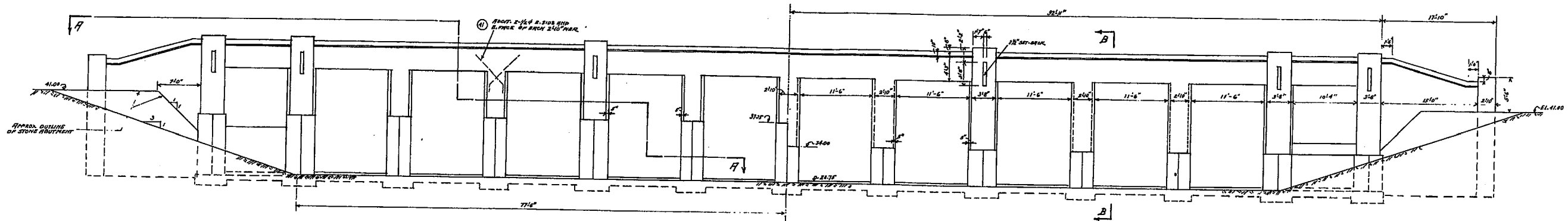
SYMBOL — NUMBER 130 — DATE

24X

MICROFILMED

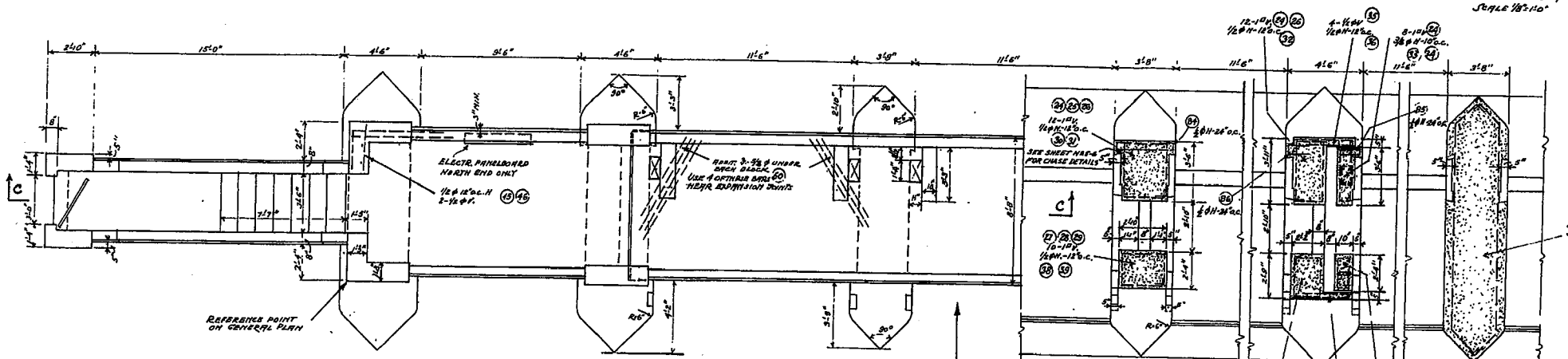
SHEET 5 OF 13 SHEETS

49-D-30

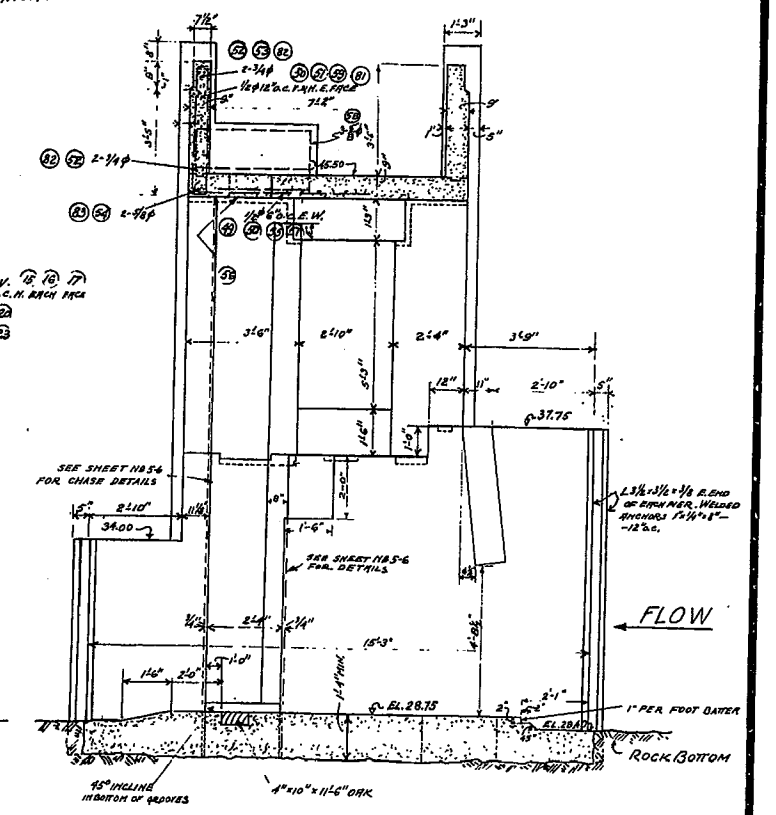


HALF UPSTREAM ELEVATION
SCALE 1/8"=1'-0"

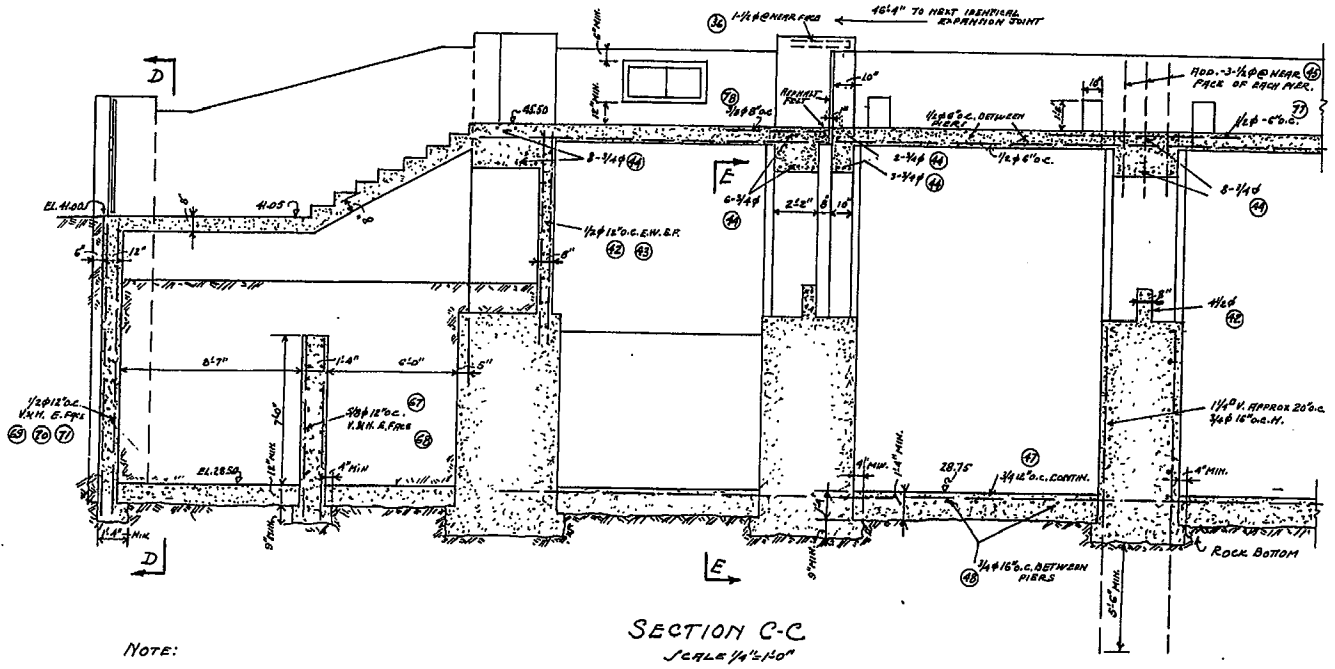
HALF DOWNSTREAM ELEVATION.
SCALE 1/8"=1'-0"



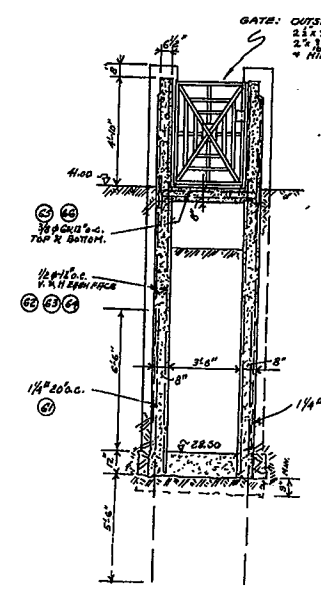
SECTION A-A
SCALE 1/4"=1'-0"



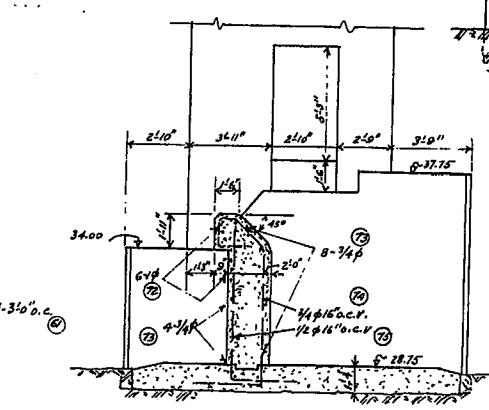
SECTION B-B
SCALE 3/8"=1'-0"



SECTION C-C
SCALE 1/4"=1'-0"



SECTION D-D
SCALE 1/4"=1'-0"



SECTION E-E
SCALE 1/4"=1'-0"

NOTE:

THE GATE SECTION OF THE DAM IS SYMMETRICAL ABOUT THE CENTER LINE, EXCEPT THE ELECTRIC PANELBOARD WHICH IS ON NORTH END ONLY.

THE TEN GATES WITH HOISTS, GUIDES, ANCHOR BOLTS, SLEEVES ETC. ARE NOT SHOWN ON THIS DRAWING, BUT SHALL BE PROVIDED AS CALLED FOR ON SHEET NO. 5-6

PROVIDE A 3/4 CHAMFER ON ALL SHARP CONCRETE EDGES EXCEPT STAIR NOSING WHERE A 3/8" CHAMFER SHALL BE PROVIDED.

EXPANSION JOINTS AT RIGHT ANGLE TO THE LONGITUDINAL DIRECTION OF THE DAM SHALL BE FILLED WITH 1" ASPHALT FELT. SLIDING SURFACES AT THE EXPANSION JOINTS SHALL BE SEPARATED BY MEANS OF BUILDING PAPER.

IN ABUTMENTS ALL VERTICAL WALLS SHALL BE POURED CONTINUOUS WITHOUT JOINTS UP TO EL. 38.00.

REINFORCING BARS ARE SHOWN MOSTLY IN TYPICAL LOCATIONS, BUT SHALL BE PROVIDED IN ALL SIMILAR LOCATIONS.

VERTICAL BARS IN PIERS SHALL EXTEND TO 1 1/2" FROM THE TOP OF THE SLAB, EXCEPT THE BARS BELOW RAILINGS WHICH SHALL EXTEND UP TO 2" FROM THE TOP OF THE RAILING.

PROVIDE 3-1/2φ x 7'-0" - 10" O.C. EACH WAY UNDER EACH HOIST IN TOP OF THE SLAB. (76)

Designed by M. B. Coifman
Consulting Engineer

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

ESTABROOK DARK DAM

CONCRETE DETAILS - GATE SECTION

LANDSCAPE DESIGN SECTION	ENGINEERING DESIGN SECTION
Alfred L. Bowman, Landscape Architect	Ray E. Behrens, Senior Ass't. Civil Engineer

Survey ..... Plan ..... Tracing ..... Checked .....

Approved: E. R. Howard, Supervising Engineer — Date 4/24/37

Approved: W. [illegible], Highway Commissioner & Planning Engineer — Date 5/20/37

Revised May 18, 1937.
Revised May 19, 1937. [illegible]
Revised Sept. 29, 1937. [illegible]
Revised Nov. 30, 1937 [illegible]

DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
AND
MILWAUKEE COUNTY PARK COMMISSION
CO-OPERATING

BRANCH OF PLANNING — STATE PARK DIVISION

Prepared by Jerome [illegible] Date ..... Secretary Milwaukee County Park Commission

Recommended Hobe P. Hoffman Date 5/21 Inspector

Approved ..... Date ..... Regional Officer

S.P.-WIS.
9006-4-8
SCALES AS SHOWN

SYMBOL
NUMBER 130
DATE

24x

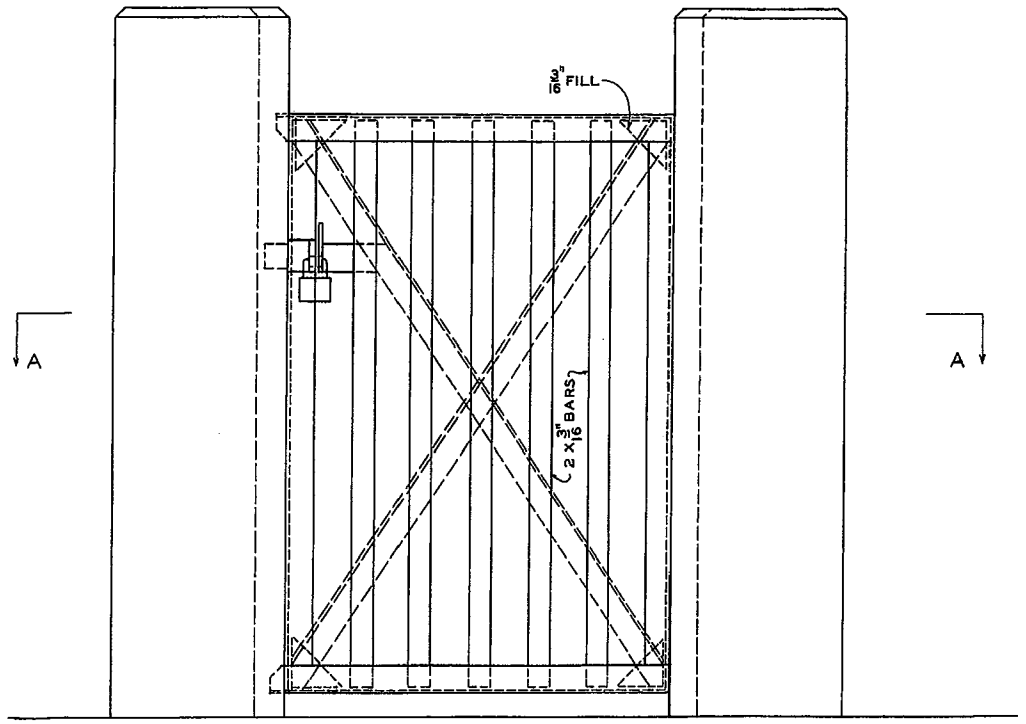
MICROFILMED

SHEET 6 OF 13 SHEETS

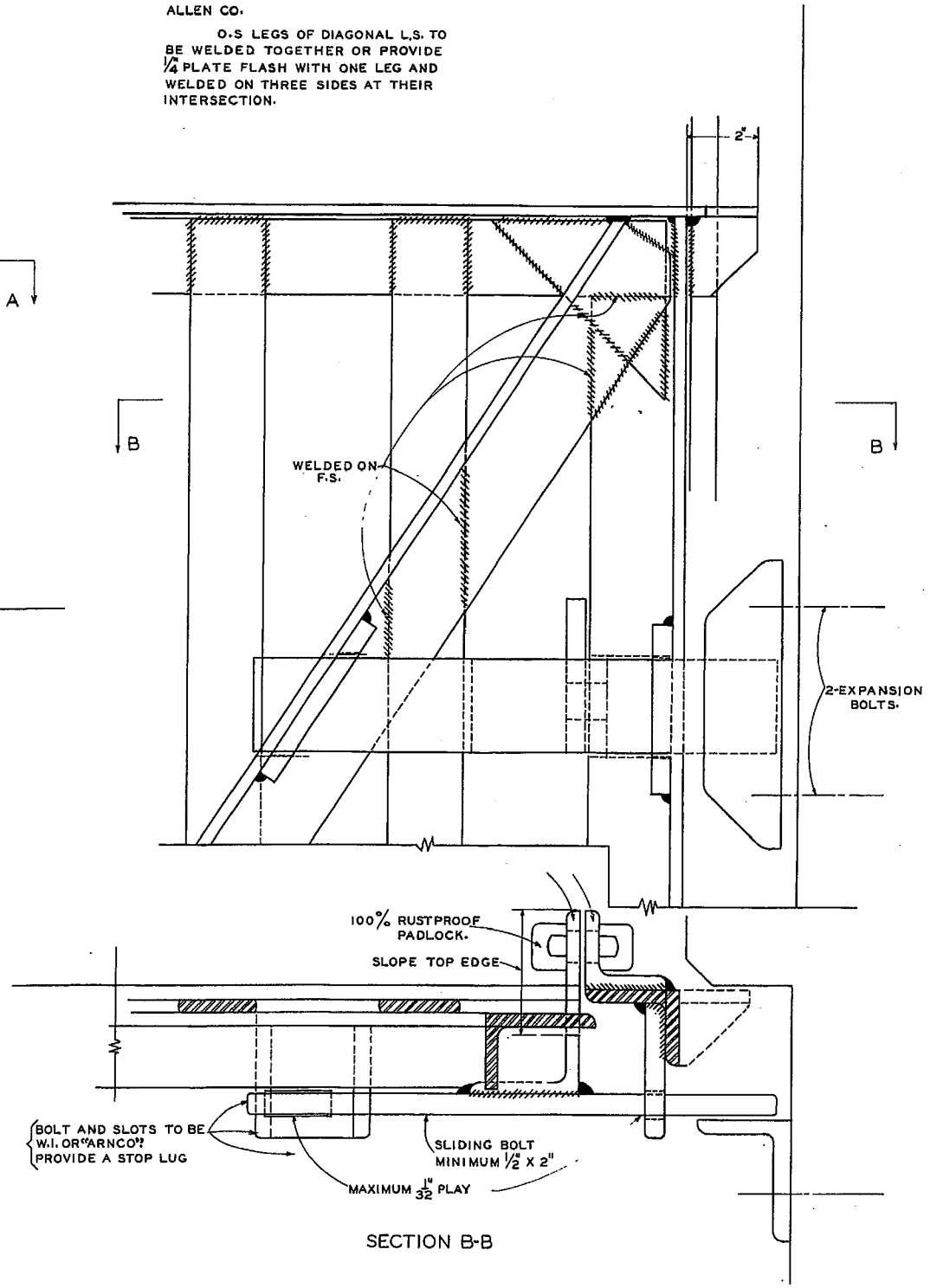
49-B-30

NOTE: – EXCEPT AS SHOWN ON THIS DRAWING FOLLOW SHEET Nº 3., CONTRACT 10-781 OF WORDEN-ALLEN CO.

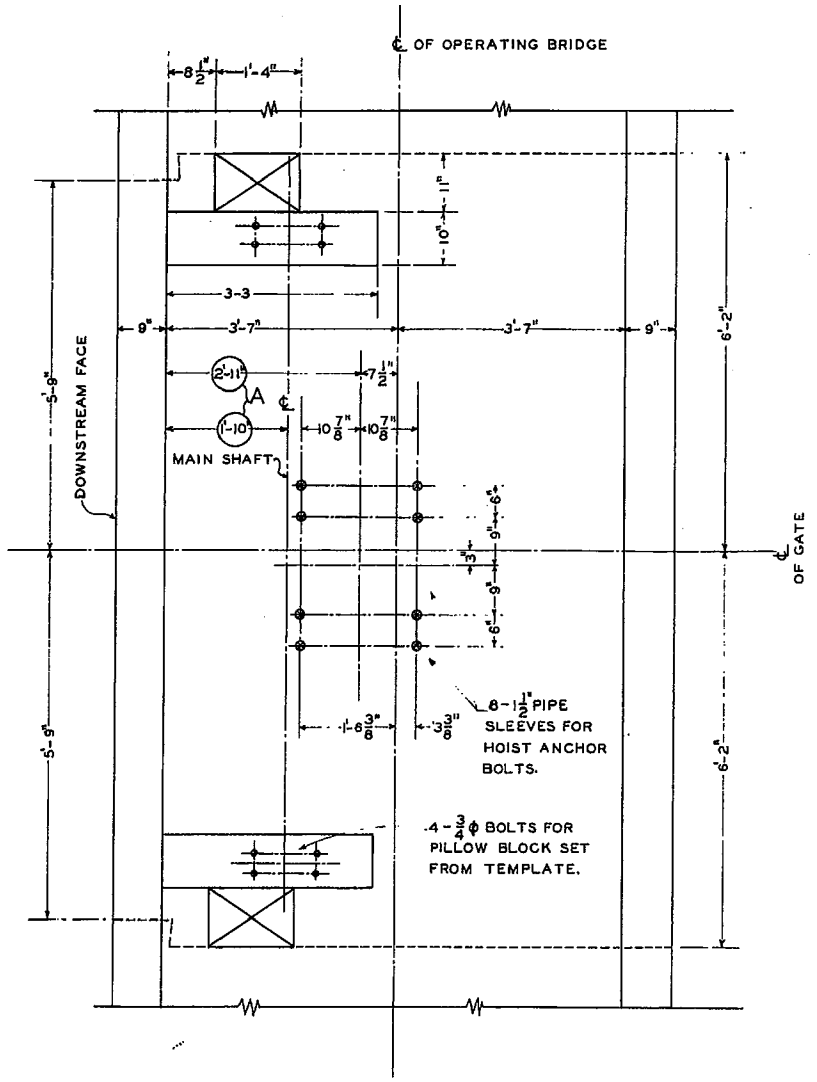
O.S LEGS OF DIAGONAL L.S. TO BE WELDED TOGETHER OR PROVIDE 1/4" PLATE FLASH WITH ONE LEG AND WELDED ON THREE SIDES AT THEIR INTERSECTION.



SECTION A-A



SECTION B-B



LOCATION OF ANCHOR BOLTS FOR HOISTS

SCALE- 3/4" = 1'-0"

(A) REVISED FEB. 11-1938. (REDUCED BY 1/2")

24x

MICROFILMED

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

ESTABROOK PARK DAM
REVISED ENTRANCE GATES
AND LOCATION OF ANCHOR BOLTS FOR HOISTS

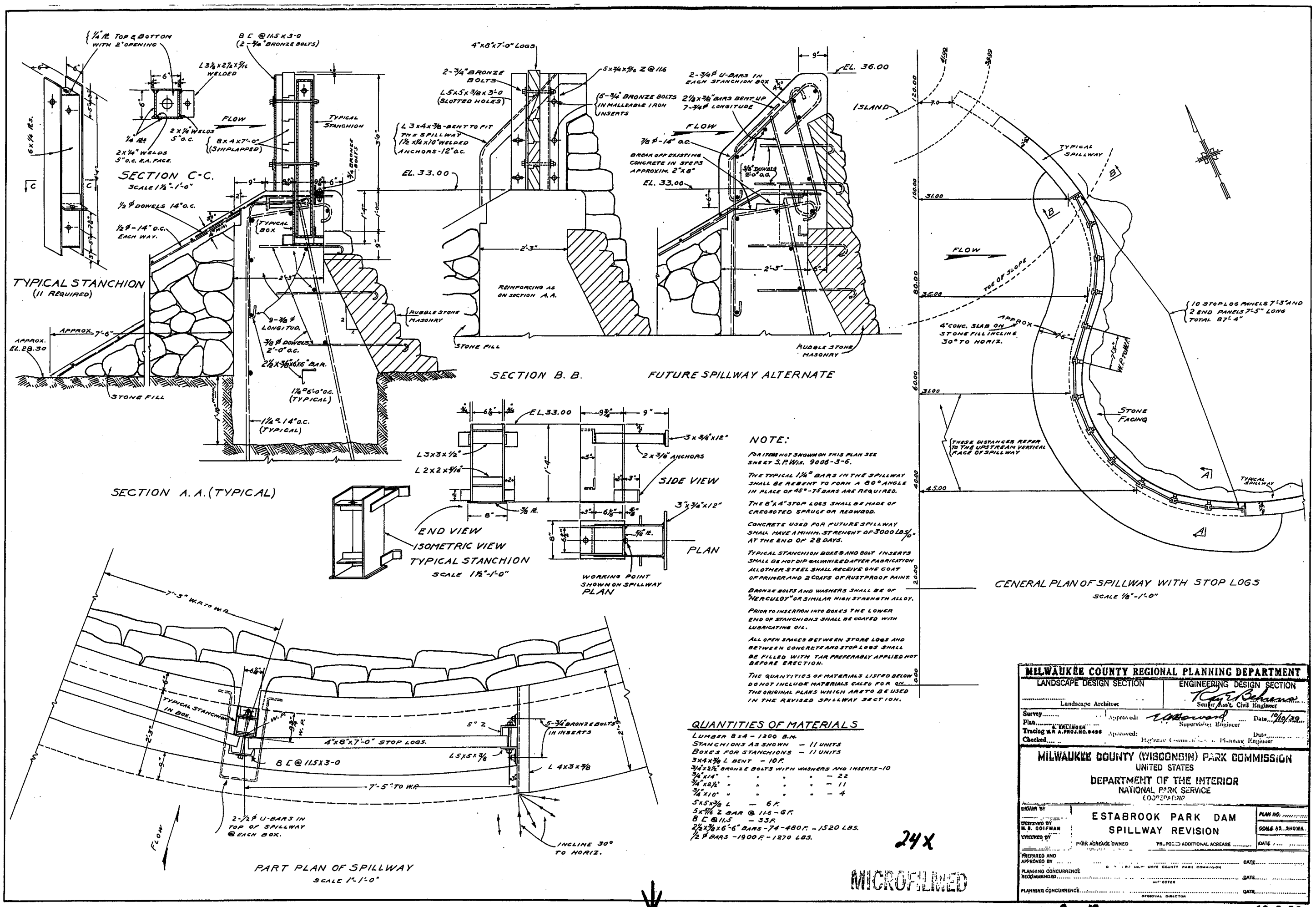
SCALE: AS SHOWN — FEBRUARY. 11, 1938

LANDSCAPE DESIGN SECTION	ENGINEERING DESIGN SECTION
LANDSCAPE ARCHITECT	SENIOR ASS'T. CIVIL ENGINEER

SURVEY ____ NOTES ____
PLAN M. B. COIFMAN
TRACING A. VAN BUREN W. P. A. PROJ NO. 9495
CHECKED ____

APPROVED: ____ SUPERVISING ENGINEER — DATE ____
APPROVED: ____ HIGHWAY COMMISSIONER AND PLANNING ENGINEER — DATE ____

SHEET 7 OF 13 SHEETS — FILE — TRACING 49-D-30 — PRINT ____



NOTE:

FOR ITEMS NOT SHOWN ON THIS PLAN SEE SHEET S.P. WIS. 9006-3-6.

THE TYPICAL 1¼" BARS IN THE SPILLWAY SHALL BE REBENT TO FORM A 80° ANGLE IN PLACE OF 45°-75 BARS ARE REQUIRED.

THE 8"x4" STOP LOGS SHALL BE MADE OF CREOSOTED SPRUCE OR REDWOOD.

CONCRETE USED FOR FUTURE SPILLWAY SHALL HAVE A MINIM. STRENGHT OF 3000 LBS/□" AT THE END OF 28 DAYS.

TYPICAL STANCHION BOXES AND BOLT INSERTS SHALL BE HOT DIP GALVANIZED AFTER FABRICATION ALL OTHER STEEL SHALL RECEIVE ONE COAT OF PRIMER AND 2 COATS OF RUSTPROOF PAINT.

BRONZE BOLTS AND WASHERS SHALL BE OF "HERCULOY" OR SIMILAR HIGH STRENGTH ALLOY.

PRIOR TO INSERTION INTO BOXES THE LOWER END OF STANCHIONS SHALL BE COATED WITH LUBRICATING OIL.

ALL OPEN SPACES BETWEEN STORE LOGS AND BETWEEN CONCRETE AND STOP LOGS SHALL BE FILLED WITH TAR PREFERABLY APPLIED HOT BEFORE ERECTION.

THE QUANTITIES OF MATERIALS LISTED BELOW DO NOT INCLUDE MATERIALS CALED FOR ON THE ORIGINAL PLANS WHICH ARE TO BE USED IN THE REVISED SPILLWAY SECTION.

QUANTITIES OF MATERIALS

LUMBER 8x4 - 1200 B.M.
STANCHIONS AS SHOWN - 11 UNITS
BOXES FOR STANCHIONS - 11 UNITS
3x4x⅜ L BENT - 10 F.
¾"x2½" BRONZE BOLTS WITH WASHERS AND INSERTS - 10
¾"x14" " " " " - 22
¾"x2½" " " " " - 11
¾"x10" " " " " - 4
5x5x⅜ L - 6 F.
5x5/16 Z BAR @ 11.6 - 6 F.
8 [@ 11.5 - 33 F.
2½x⅜x6'-6" BARS - 74 - 480 F. - 1520 LBS.
½ ϕ BARS - 1900 F. - 1270 LBS.

24X

MICROFILMED

CENERAL PLAN OF SPILLWAY WITH STOP LOGS
SCALE ⅛"-1'-0"

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

LANDSCAPE DESIGN SECTION — Landscape Architect
ENGINEERING DESIGN SECTION — Senior Ass't. Civil Engineer

Survey
Plan — C. ERLINGER
Tracing W.R. A. PROJ. NO. 8496
Checked
Approved: Supervising Engineer — Date 10/10/39
Approved: Highway Commissioner & Planning Engineer

MILWAUKEE COUNTY (WISCONSIN) PARK COMMISSION
UNITED STATES
DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
COOPERATING

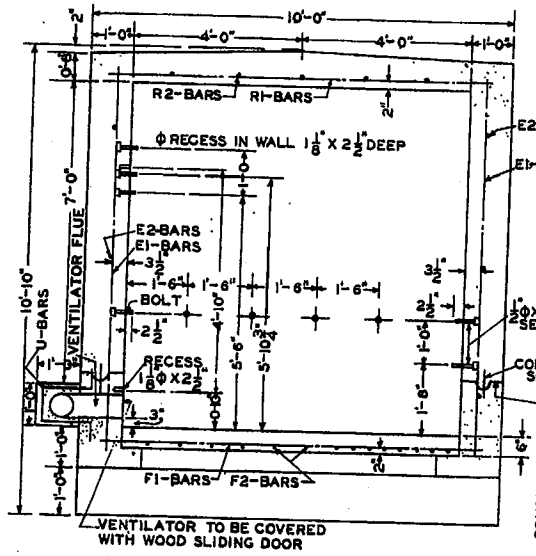
DRAWN BY
DESIGNED BY M. B. GOIFMAN
CHECKED BY

ESTABROOK PARK DAM
SPILLWAY REVISION

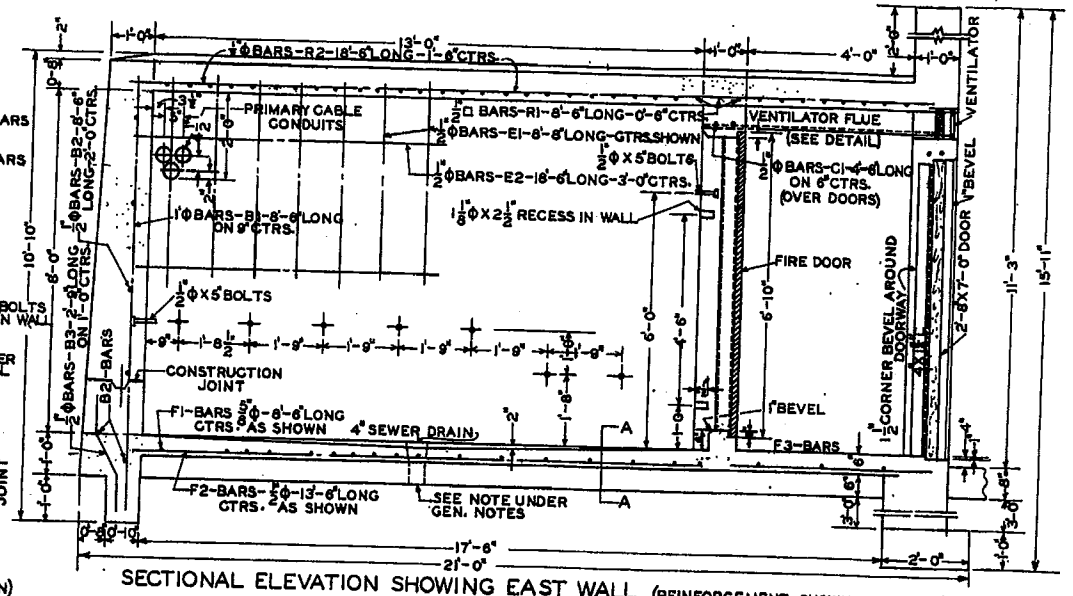
PARK ACREAGE OWNED — PROPOSED ADDITIONAL ACREAGE
PLAN NO.
SCALE AS SHOWN
DATE
PREPARED AND APPROVED BY — DATE
PLANNING CONCURRENCE RECOMMENDED — DATE
PLANNING CONCURRENCE — DATE

SHEET 8 OF 13 SHEETS

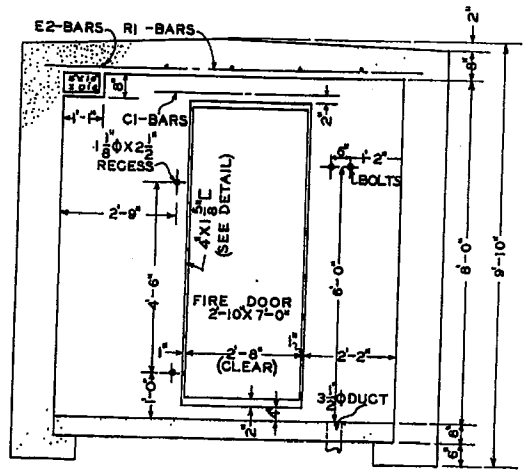
49-D-30



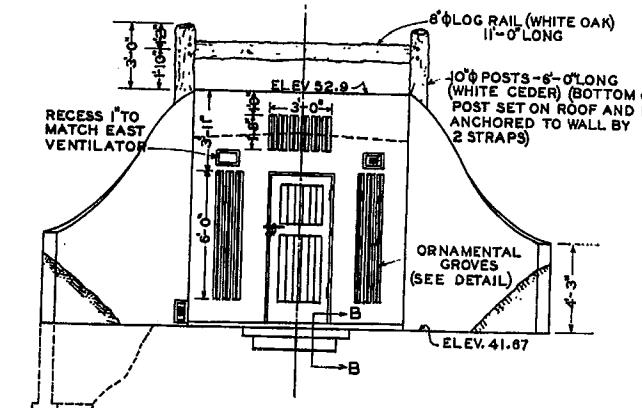
ELEVATION REAR WALL (NO REAR WALL REINFORCEMENT SHOWN) (SCALE ½" = 1'-0")



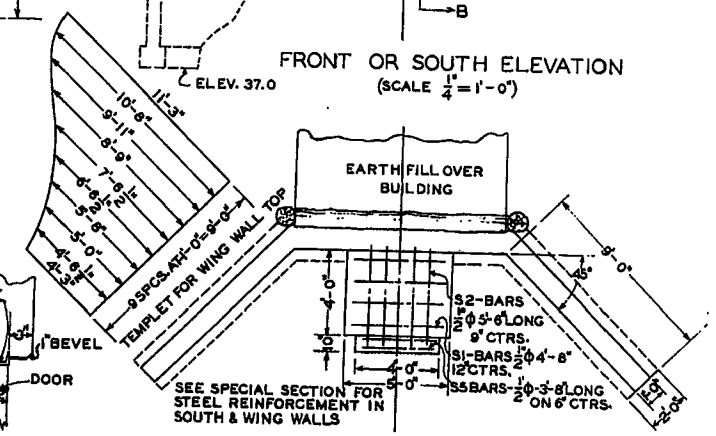
SECTIONAL ELEVATION SHOWING EAST WALL (REINFORCEMENT SHOWN PARTIALLY)



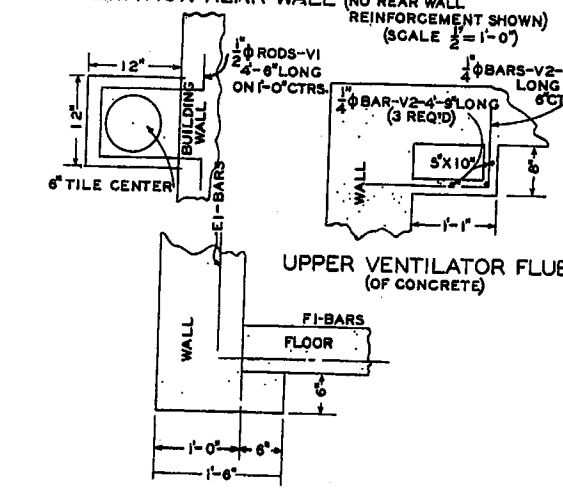
ELEVATION OF SOUTH WALL OF TRANSFORMER ROOM (LOOKING SOUTH) (STEEL REINFORCEMENT SHOWN IN ROOF & OVER DOOR) (SCALE ½" = 1'-0")



FRONT OR SOUTH ELEVATION (SCALE ¼" = 1'-0")

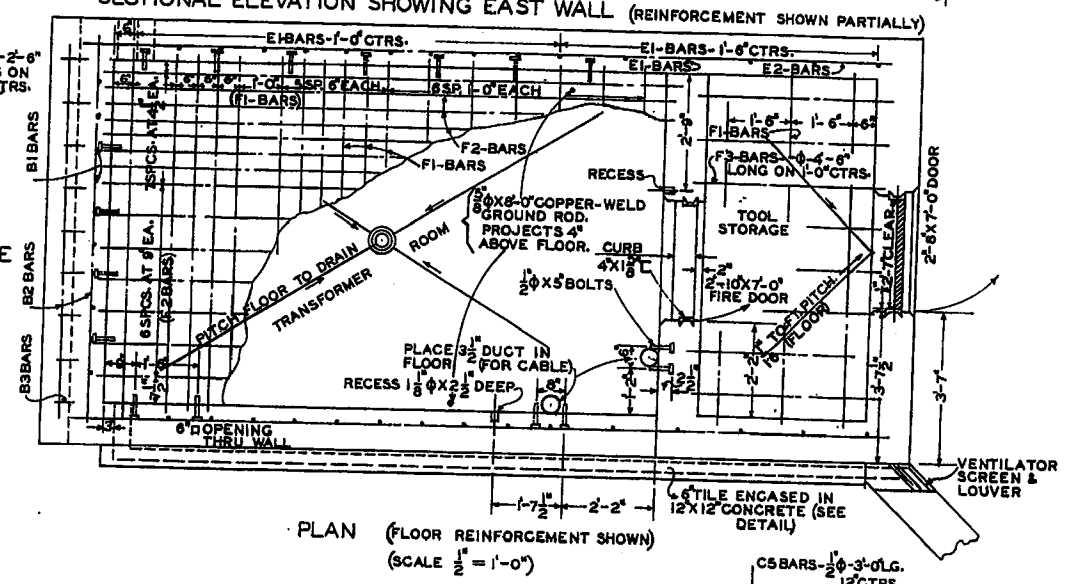


PLAN - ENTRANCE STEP (REINFORCEMENT FOR STEPS SHOWN) (SCALE ¼" = 1'-0")

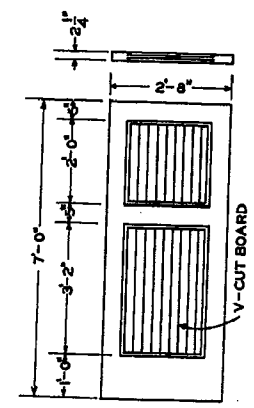


SECTION A-A-FOUNDATION FOR SIDE WALLS

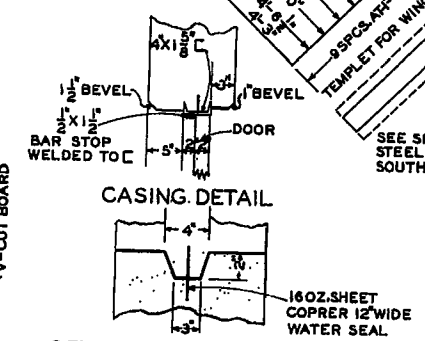
UPPER VENTILATOR FLUE (OF CONCRETE)



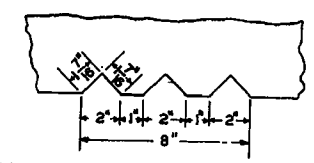
PLAN (FLOOR REINFORCEMENT SHOWN) (SCALE ½" = 1'-0")



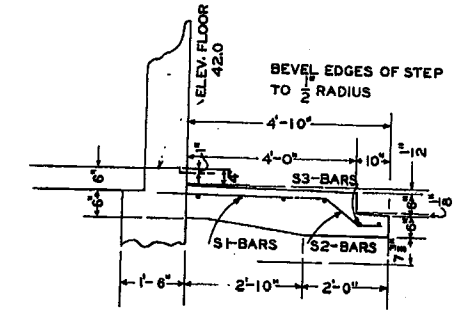
OUTSIDE DOOR



DETAIL CONSTRUCTION JOINT

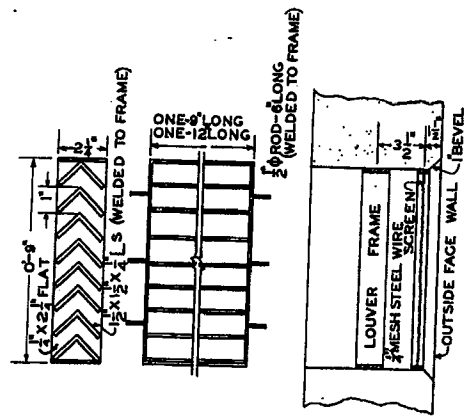


SECTION SHOWING GROVES



SECTION B-B (FRONT STEP)

24X

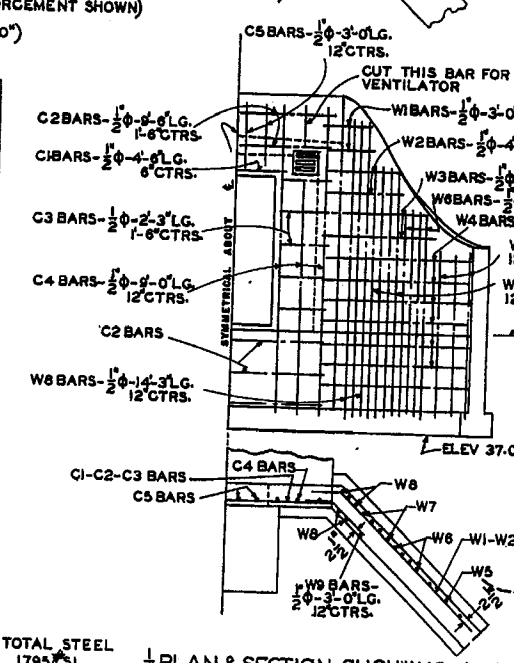


LOUVER DETAIL (2 REQUIRED)

WALL SETTING OF LOUVER & SCREEN

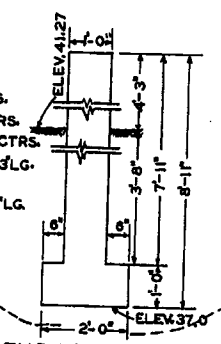
MARK	NO REQ'D	SIZE	LENGTH	SPACING	LOCATION & DETAIL
B1	10	1" Φ	8'-6"	9"	NORTH WALL
B2	6	½" Φ	8'-6"	2'-0"	
B3	8	"	2'-9"	12"	B3 12" 45° BEND
C1	4	½" Φ	4'-6"	6"	SOUTH WALL
C2	5	"	9'-6"	1'-6"	
C3	10	"	2'-3"	1'-6"	
C4	6	"	12'-0"	12"	
C5	4	"	3'-0"	12"	
E1	32	½" Φ	8'-6"	SHOWN	SIDE WALL
E2	6	"	16'-6"	3'-0"	
F1	20	5/8" Φ	8'-6"	SHOWN	FLOOR
F2	14	½" Φ	13'-6"	"	
F3	8	"	4'-6"	12"	
R1	36	½" Φ	8'-6"	6"	ROOF
R2	5	"	16'-6"	1'-6"	
S1	4	½" Φ	4'-8"	12"	STOOP 4'-0" S2 6"
S2	5	"	5'-8"	9"	
S3	2	"	3'-8"	6"	45° BEND
W1	4	½" Φ	3'-0"	12"	WING WALL W1-W2-W3-W4 45° BEND
W2	4	"	4'-9"	"	
W3	4	"	6'-9"	"	
W4	16	"	8'-9"	"	
W5	6	"	8'-3"	"	
W6	8	"	10'-0"	"	
W7	8	"	12'-0"	"	
W8	10	"	14'-3"	"	
W9	26	"	3'-0"	"	
V1	20	½" Φ	4'-6"	12"	VENTILATOR V1 14" 5" SYM. ABOUT ℄
V2	9	¼" Φ	2'-6"	6"	V2 1'-6" 2"

DOORS. Use 3 hinges on each door. Padlock latches required for both doors.

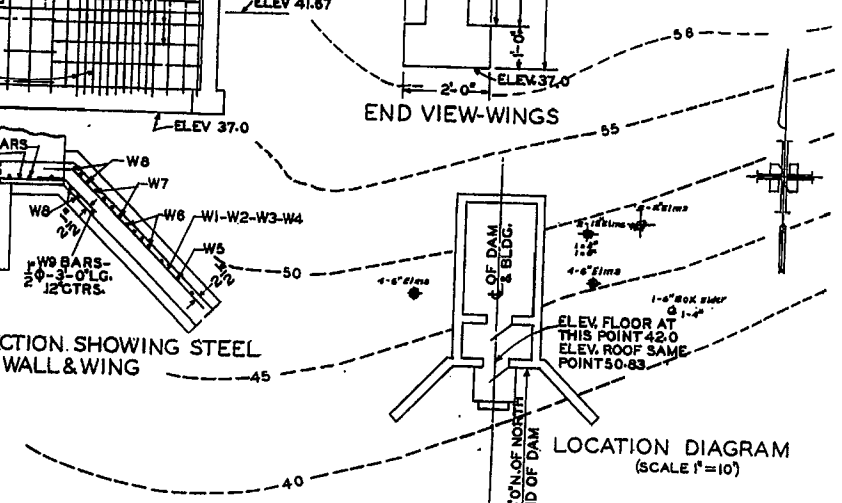


TOTAL STEEL 1795# SI.

PLAN & SECTION SHOWING STEEL IN SOUTH WALL & WING



END VIEW-WINGS



LOCATION DIAGRAM (SCALE 1" = 10')

GENERAL NOTES

ROOF shall be covered with a primer coat, 3 ply membrane fabric and 4 moppings of asphalt. Use 140#S asphalt per 100 square feet.

SIDE & BACK WALL. Place fabrice over construction joint and coat with 1 coat primer and 2 coats hot asphalt for waterproofing

Inside opening of VENTILATOR to be covered with sliding wood door. Use no metal.

FLOOR is to be poured integral with walls to insure water tight seal,

VENTILATOR screens to be made of ¼ mesh steel wire set in metal frame and cast into concrete.

PLANTING & LANDSCAPE work shown on another plan.

BOLTS to be set in walls are to be threaded SAE for entire projecting length of 2½ inches

FLOOR DRAINS. Outlet pipe to be 4" sewer tile and run to water in river. Distance approximately 15FT. Outlet must be screen protected.

GROUND RODS are to be 5/8" Φ x 8'-0" long and projecting 4 inches above floor. Use copper weld rods.

MICROFILMED

LANDSCAPE DESIGN SECTION	ENGINEERING DESIGN SECTION
Landscape Architect	Senior Ass't. Civil Engineer
Survey	Approved: Date
Plan G. F. DARGETT	Supervising Engineer
Tracing K. R. A. VAN BUREN	Approved: Date
Checked	Highway Commissioner & Planning Engineer

UNITED STATES
DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
COOPERATING WITH
MILWAUKEE COUNTY PARK COMMISSION

DRAWN BY
DESIGNED BY
CHECKED BY

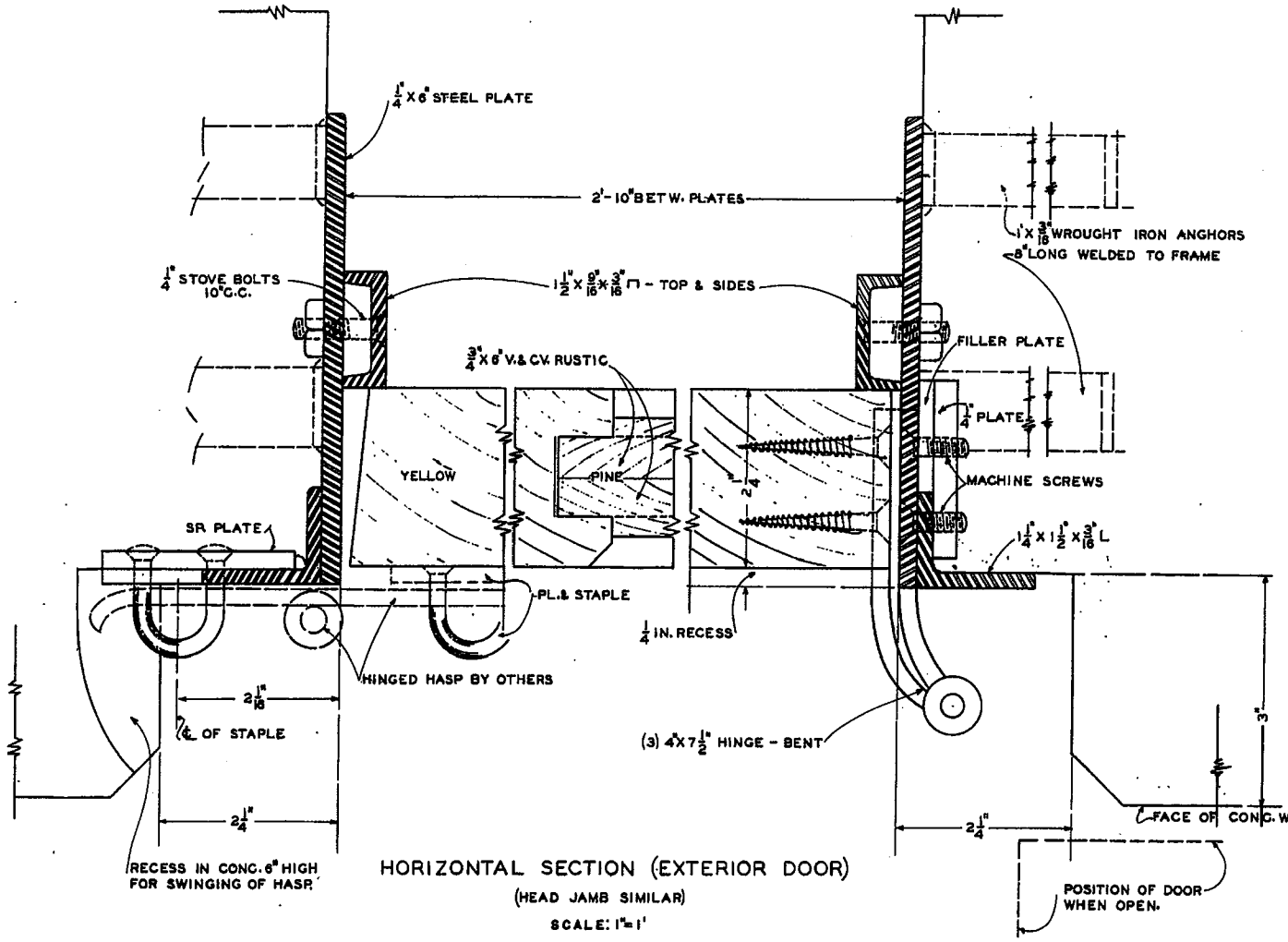
ESTABROOK PARK DAM
TRANSFORMER BUILDING

PLAN NO.
SCALE AS SHOWN
S.P.
DATE NOV. 1939.

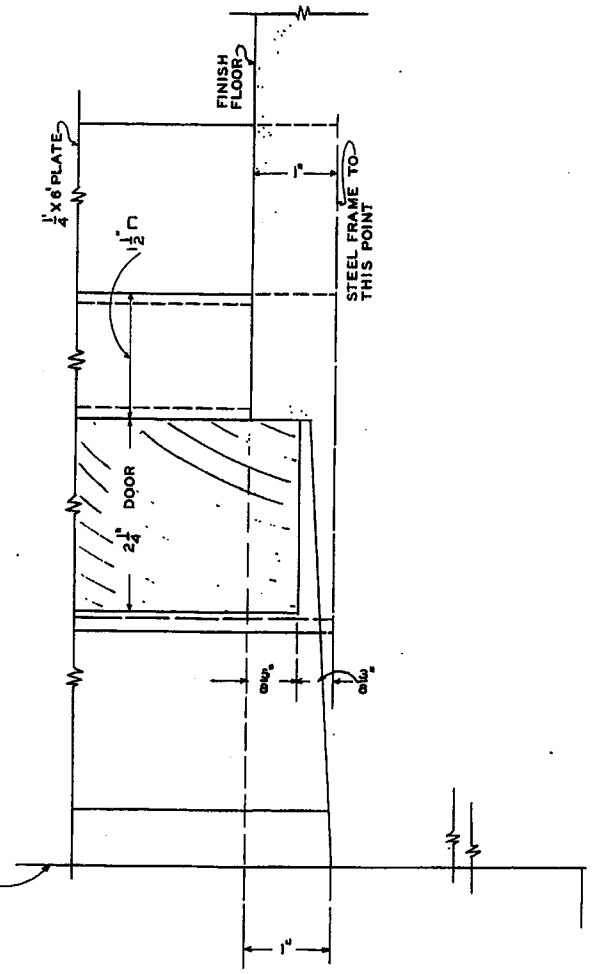
PREPARED BY — SECRETARY MILWAUKEE COUNTY PARK COMMISSION — DATE
RECOMMENDED — INSPECTOR — DATE
RECOMMENDED — DIVISIONAL OFFICER — DATE
APPROVED — ASSISTANT DIRECTOR — DATE

SHEET 9 OF 13 SHEETS

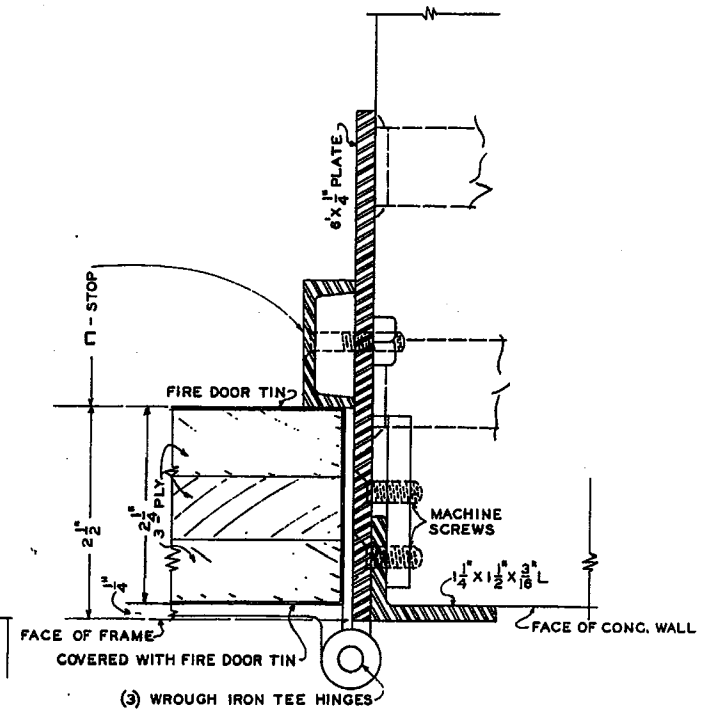
49-D-30



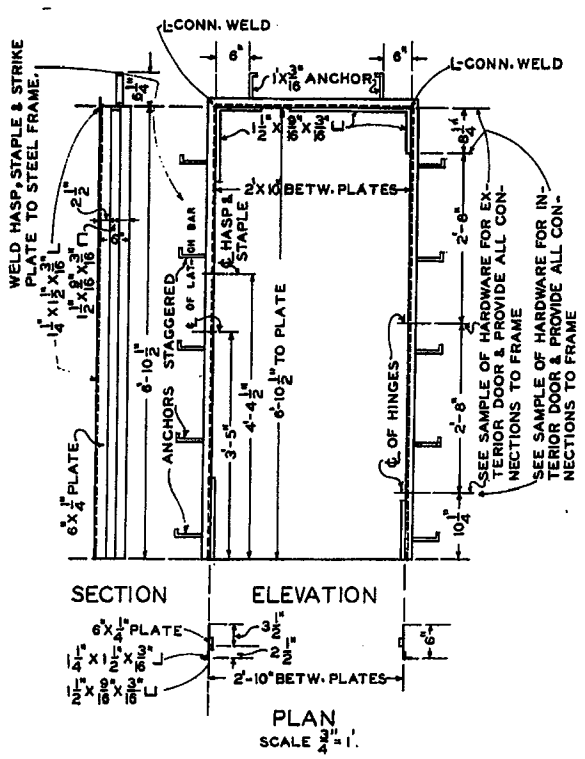
HORIZONTAL SECTION (EXTERIOR DOOR)
(HEAD JAMB SIMILAR)
SCALE: 1" = 1'



SECTION THROUGH DOOR SILL
(EXTERIOR DOOR)
SCALE: 1" = 1'

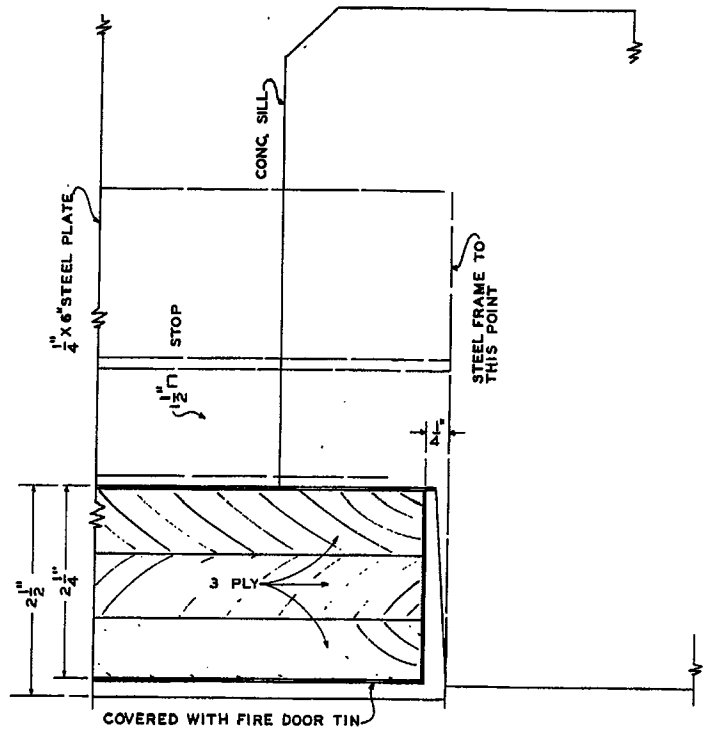


HORIZONTAL SECTION
(INTERIOR DOOR)
SCALE: 1" = 1'



MAKE ONE (1) STEEL DOOR FRAME AS NOTED (EXTERIOR)

MAKE ONE (1) STEEL DOOR FRAME AS NOTED (INTERIOR)



SECTION - DOOR SILL
(INTERIOR DOOR)
SCALE: 1" = 1'

HARDWARE
EXTERIOR DOOR
HINGES - (3) NO. 2410 - LAWRENCE CATALOG OR EQUAL. 4" X 7 1/2" - BENT AS PER DETAIL DRAWING. . . . NO HOLES. - 1/2 WOOD - 1/2" MACHINE SCREWS.
LATCH - (1) NO. 1264 - STANLEY CATALOG, OR EQUAL. EXTRA HEAVY WITH STRIKE BOLT & BAR-LATCH 8" LONG.
HINGE-HASP-STAPLES (2) EXTRA HEAVY AS PER DETAILS WITH PLATE.
INTERIOR DOOR
HINGES - (3) NO. SC 844 - REVERSIBLE TEE HINGE - PRITZLAFF CATALOG OR EQUAL - 10" LONG - 1/2" MACHINE SCREWS.
LATCH - (1) NO. 1264 - STANLEY CATALOG, OR EQUAL. EXTRA HEAVY WITH STRIKE BOLT & BAR - LATCH - 12" LONG.
HINGE - HASP - STAPLES (2) EXTRA HEAVY AS PER DETAILS WITH PLATE.
FINISH - DULL BLACK.
NOTE - HASPS & PADLOCKS BY OTHERS.

NOTE:-
STEEL WORK TO RECEIVE ONE SHOP COAT - RED LEAD & OIL.
ALL JOINTS NOT EXPOSED TO VIEW WELDED., OTHERS BOLTED & SCREWED.

24 X
MICROFILMED

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

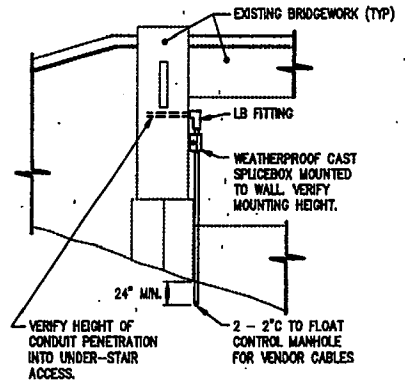
ESTABROOK PARK DAM
TRANSFORMER BUILDING
STEEL DOOR FRAMES

SCALE: AS SHOWN — DECEMBER. 26, 1938

LANDSCAPE DESIGN SECTION	ENGINEERING DESIGN SECTION
LANDSCAPE ARCHITECT	SENIOR ASS'T. CIVIL ENGINEER

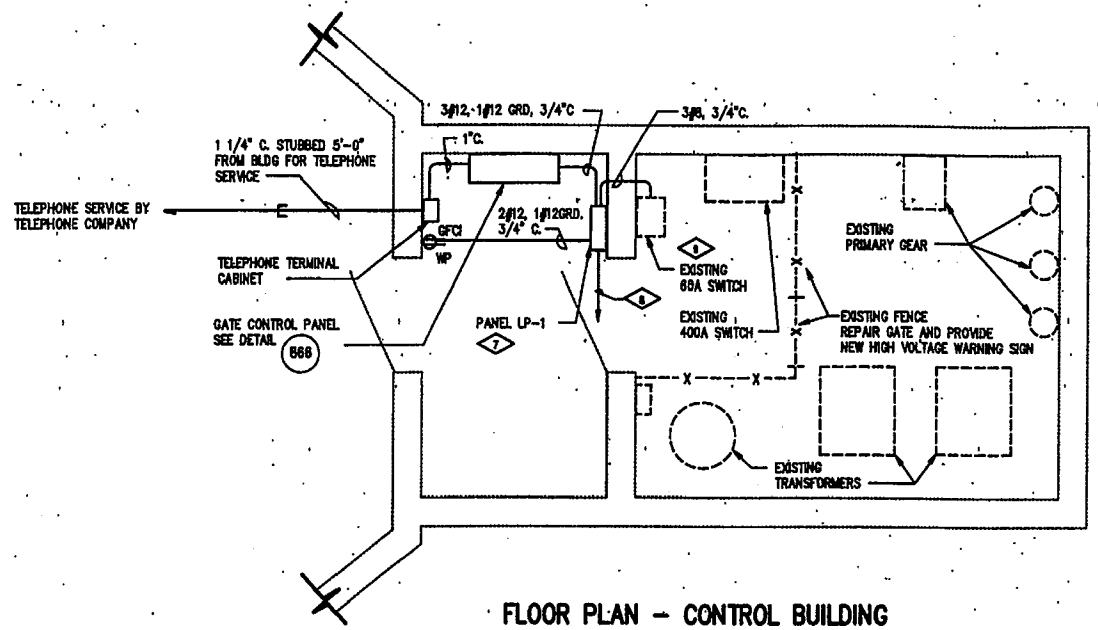
SURVEY ____ NOTES ____
PLAN ____ A. VAN BUREN
TRACING ____ W. P. A. 2999
CHECKED ____
APPROVED: ____ SUPERVISING ENGINEER ____ DATE ____
APPROVED: ____ HIGHWAY COMMISSIONER AND PLANNING ENGINEER ____ DATE ____

SHEET 13 OF 13 SHEETS — FILE — TRACING 49-D-30 — PRINT ____



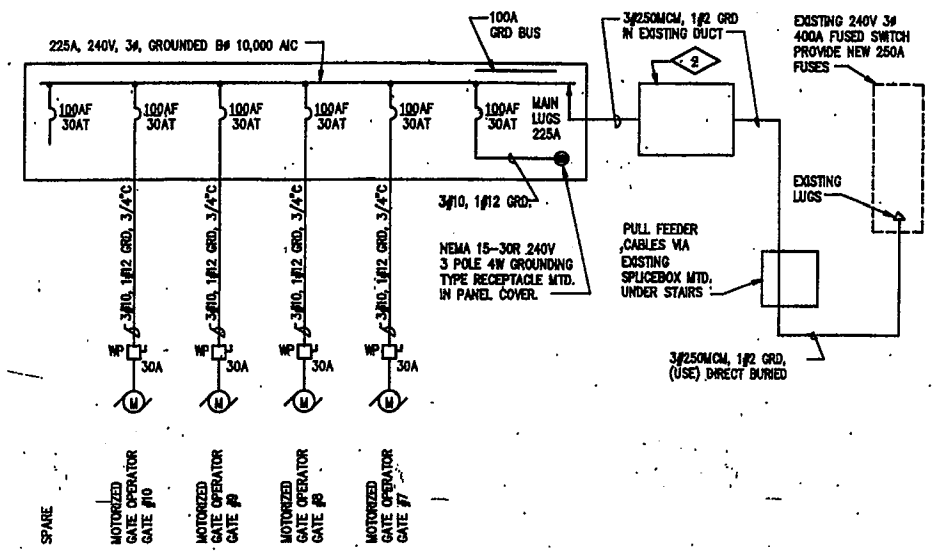
SPLICEBOX DETAIL (940)

NTS



FLOOR PLAN – CONTROL BUILDING

SCALE: 1/2"=1'-0"

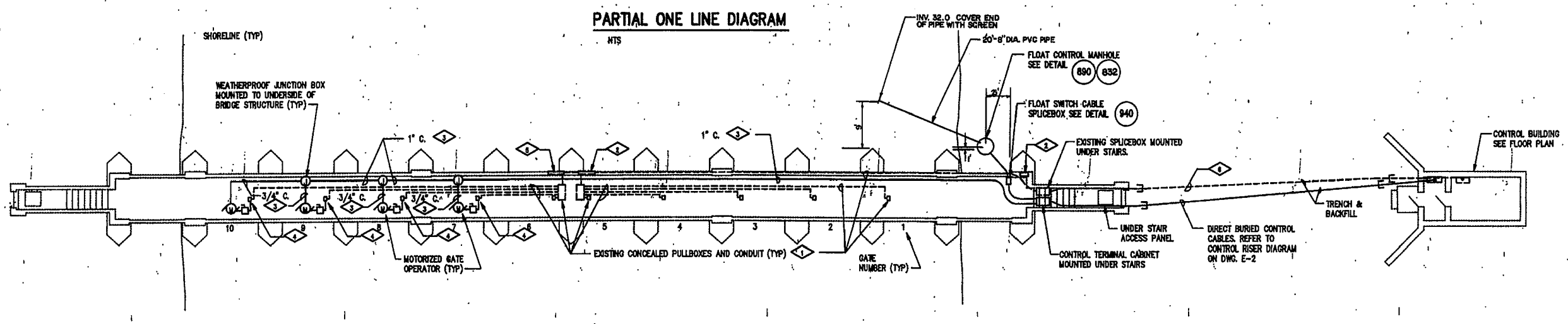


PARTIAL ONE LINE DIAGRAM

NTS

PLAN NOTES

1. DISCONNECT AND REMOVE EXISTING WIRE, CABLE, AND TERMINATIONS BACK TO SOURCE.
2. DISCONNECT AND REMOVE EXISTING ELECTRICAL EQUIPMENT AND ASSOCIATED WIRING IN RECESSED PANEL. PROVIDE NEW WEATHERPROOF COVER AND DRIP SHIELD.
3. ROUTE CONDUIT ON UNDERSIDE OF EXISTING BRIDGEWORK. DO NOT ROUTE ON VERTICAL BRIDGE FACES. REFER TO CONTROL RISER DIAGRAM ON DRAWING E-2.
4. PROVIDE WEATHERPROOF CONNECTION TO EXISTING PULLBOX UNDER BRIDGE: ROUTE CABLES IN EXISTING CONCEALED RACEWAY TO PANEL.
5. DISCONNECT AND REMOVE EXISTING ELECTRICAL EQUIPMENT AND ASSOCIATED WIRING IN RECESSED PANEL. PROVIDE NEW 225A, 240V, 3ø, 3W PANEL IN EXISTING BOX. REFER TO ONE LINE DIAGRAM THIS SHEET. PROVIDE NEW WEATHERPROOF COVER AND DRIP SHIELD.
6. REMOVE AND REPLACE EXISTING DIRECT BURIED POWER CABLE. PULL CABLES VIA EXISTING SPLICEBOX MOUNTED IN UNDER STAIR ACCESS.
7. 120/240V, 1ø, 3W SQUARE D TYPE QO CIRCUIT BREAKER LOAD CENTER WITH 100A MAIN LUGS AND MINIMUM OF 12 – 1 POLE CIRCUITS. PROVIDE WITH 10 – 20A 1 POLE CIRCUIT BREAKERS.
8. PROVIDE CONDUIT, WIRE, AND CIRCUIT BREAKERS TO RECONNECT EXISTING 120V CIRCUIT(S) IN CONTROL BUILDING.
9. DISCONNECT AND ABANDON EXISTING LOAD SIDE TERMINATIONS. PROVIDE NEW 50A FUSES AND CONNECT TO NEW PANEL LP-1.



ELECTRICAL SITE PLAN

NTS

No.	Revisions	By	Date

Scale NOTED
Date 5/9/88
Designer DRT
Drafter RSR
Checker JGM
Approver DJS

Donohue
Engineers & Architects
COMPUTER AIDED DESIGN/DRAFTING

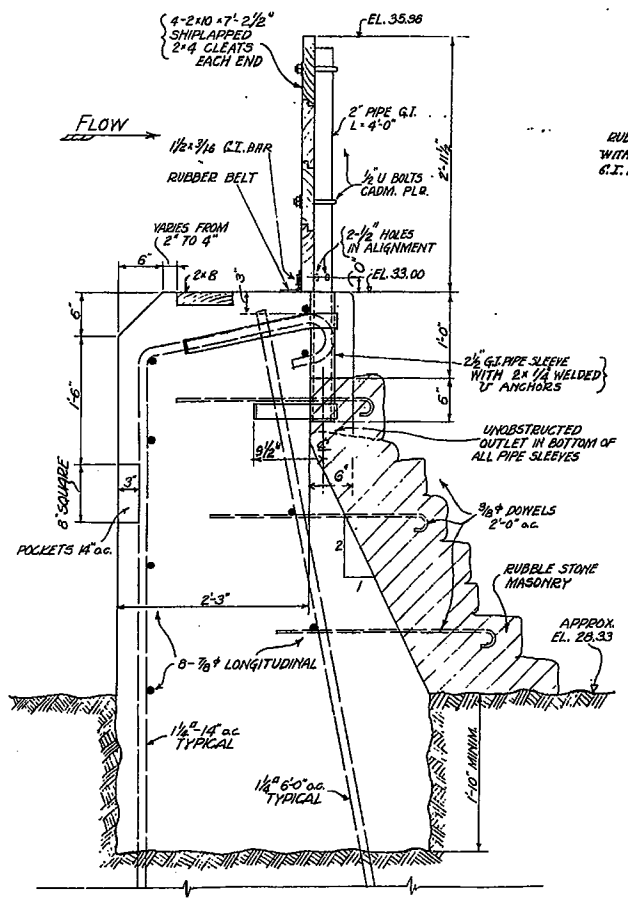
ELECTRICAL PLAN AND DETAILS

MILWAUKEE COUNTY
ESTABROOK PARK DAM
RESTORATION
PROJECT No. 88-06-4684

Sheet No. 2 OF 5
File No. WA-1515
Project No. 16070
Drawing No. E-1

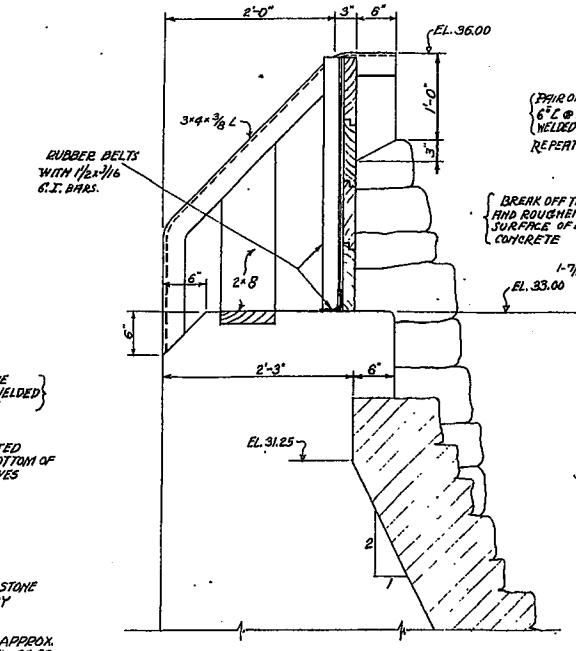
PLOT SCALE
PLOT NAME
(FULL SIZE)-FULL

05487 DESIGN FILE 163-8-78



SECTION A-A (TYPICAL)

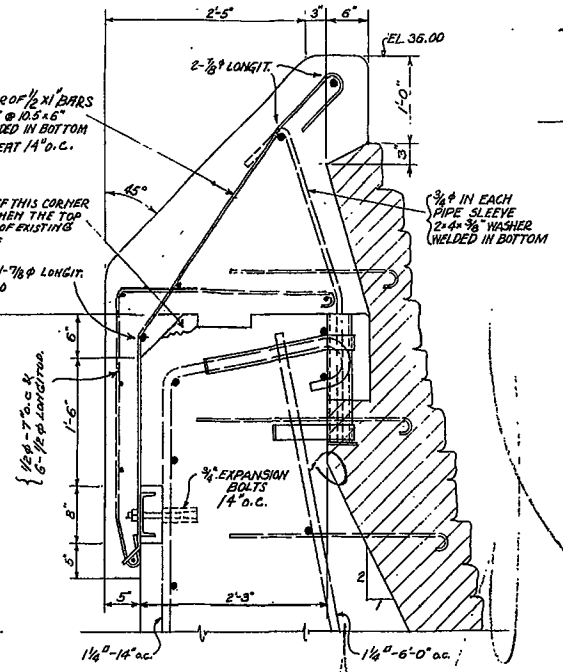
SCALE 1"=1'-0"



SECTION B-B

SCALE 1"=1'-0"

REINFORCING AS IN SECTION A-A



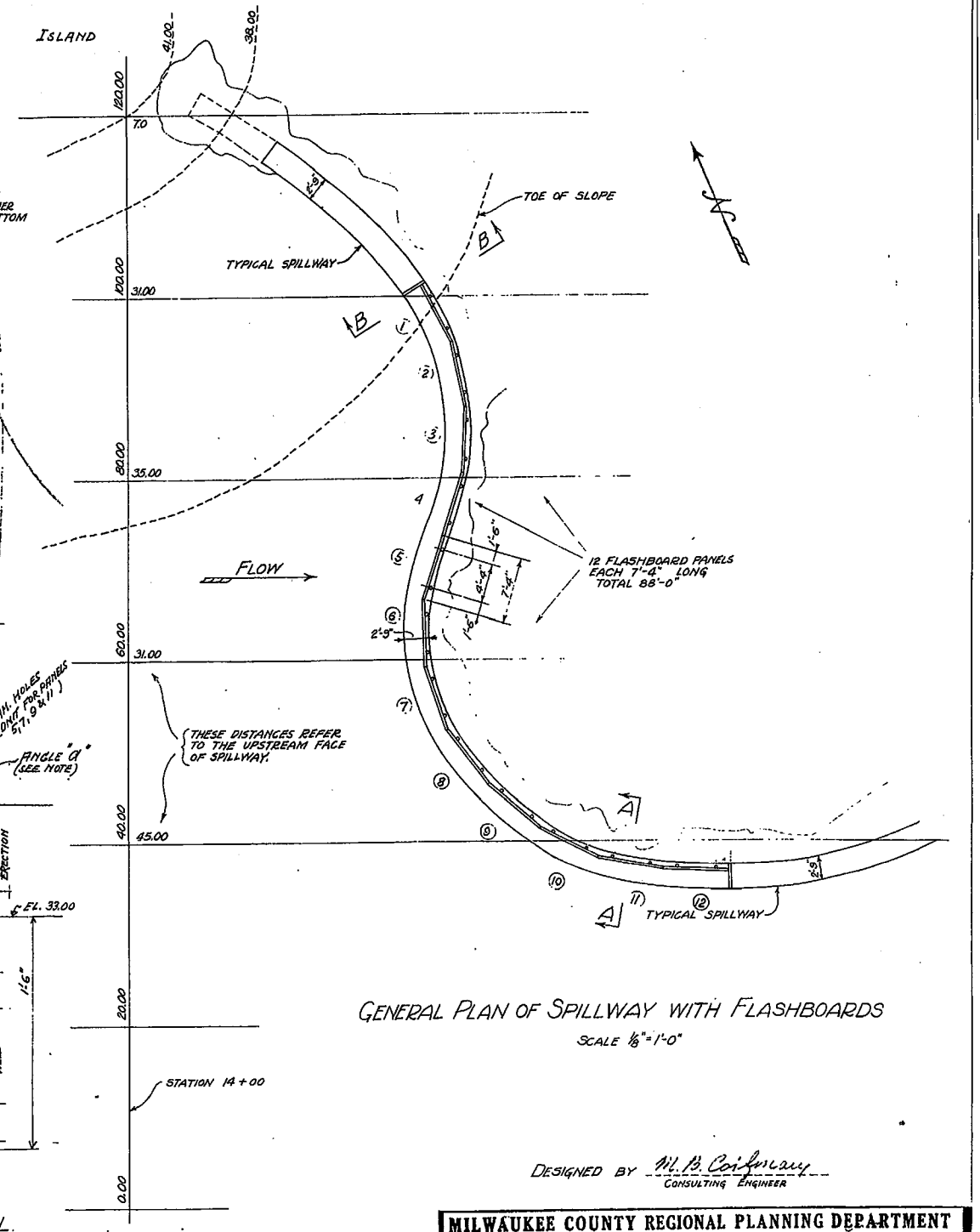
SCALE 1"=1'-0"

FUTURE SPILLWAY ALTERNATE

SECTION A-A

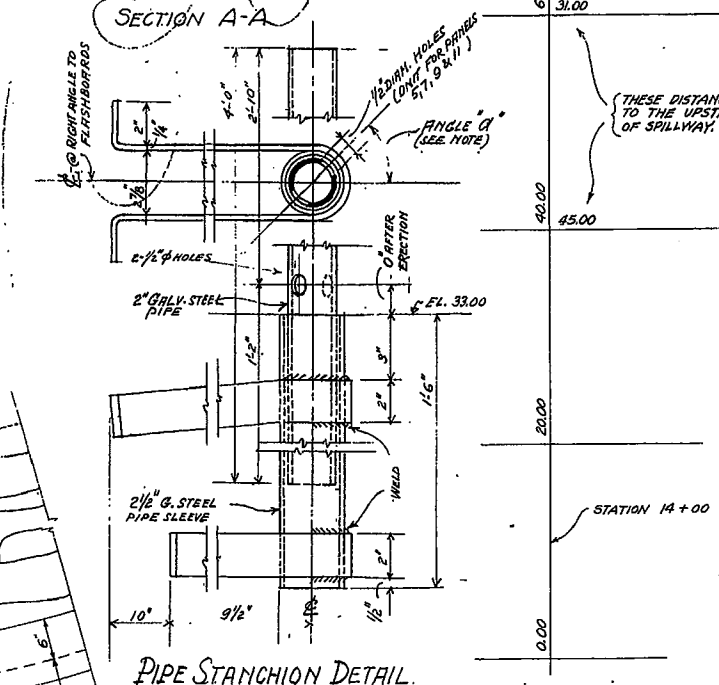
QUANTITIES OF MATERIALS.

LUMBER - 600 B.M.
GALV. PIPE - 2" — 100 F.
" 2½" — 40 F.
BARS 2×1/4 — 140 F.
BOLTS ½" U WITH WASHERS — 48 BOLTS
RUBBER BELT - 6×3/16 — 160 F.
GALV. WOOD SCREWS - 1½" SIZE — 3 GROSS. WITH WASHERS.
3×4×3/8 L - BENT - 10 F.
1½×3/16 G.I. BARS - 160 F.



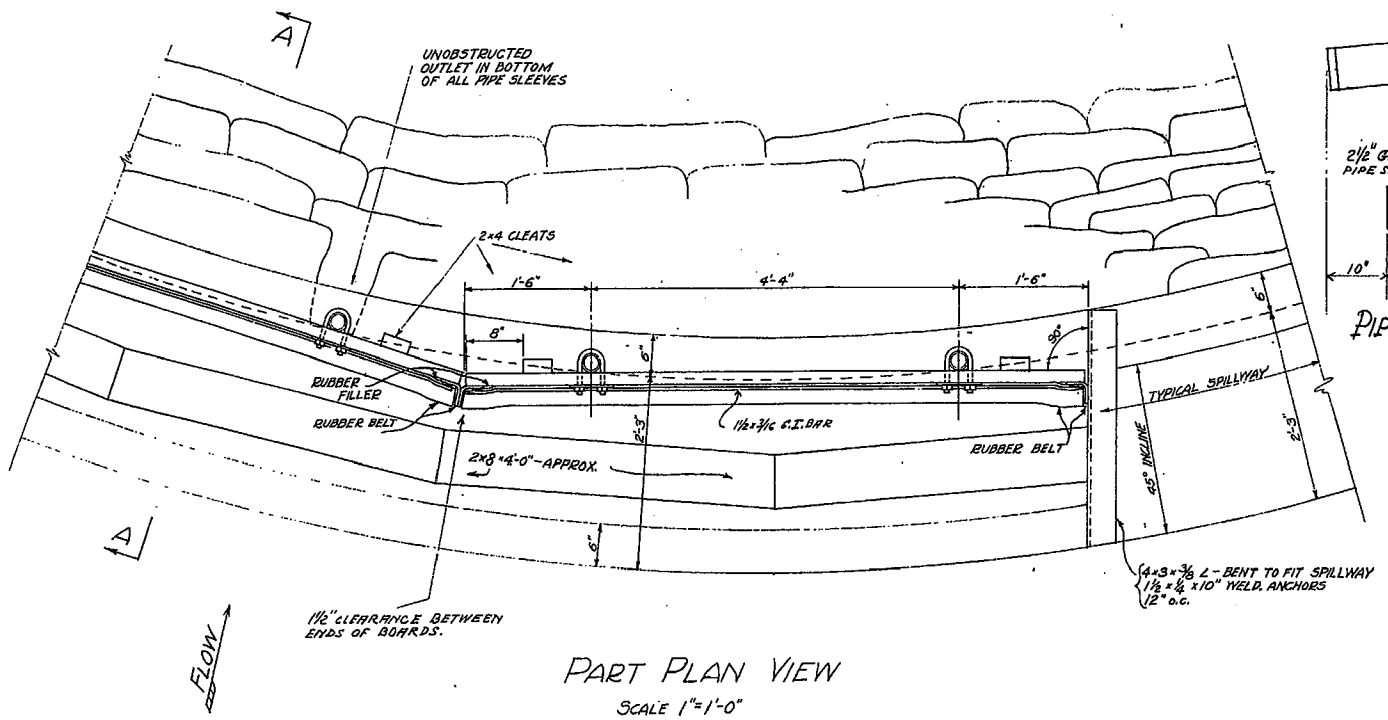
GENERAL PLAN OF SPILLWAY WITH FLASHBOARDS

SCALE 1/8"=1'-0"



PIPE STANCHION DETAIL.

SCALE 1/4"=1"



PART PLAN VIEW

SCALE 1"=1'-0"

NOTE:

FOR ITEMS NOT SHOWN ON THIS PLAN SEE SHEET S.P. WIS.-9006-3-6

FLASHBOARD STANCHIONS SHALL BE MADE OF A 2" PIPE HAVING AN ULTIMATE STRENGTH OF 70,000 LBS/□".

THE ANGLE "Q" SHOWN ON THE STANCHION DETAIL SHALL BE 0° (ZERO) FOR PANELS 1, 2, 3 & 4 & 45° FOR PANELS 6, 8, 10 & 12. OMIT THE HOLES PANELS 3, 5, 9 & 11. — IF THE STRENGTH OF PIPE MATERIAL VARIES FROM 70,000 LBS./□" THE ANGLE "Q" SHALL BE CHANGED SO AS TO MAINTAIN THE SAME RESISTING MOMENT AT THE BASE OF 2" PIPE.

THE 1¼ TYPICAL BENT BARS IN THE SPILLWAY WITH FLASHBOARDS SHALL BE REBENT IN TOP TO FORM AN 80° ANGLE IN PLACE OF 45°. 75 OF THOSE BARS ARE REQUIRED.

ALL RUBBER BELTS SHALL BE 6" × 3/16" WITH DUCK BACKING, FASTENED TO FLASHBOARDS BY G.I. SCREWS MAXIM. 5" o.c. & 1½×3/16 G.I. BARS. RUBBER FILLERS SHALL FILL THE VOIDS BETWEEN BELTS TO FORM A WATERTIGHT JOINTS.

ALL FLASHBOARDS SHALL BE MADE OF SPRUCE OR REDWOOD. THEY SHALL RECEIVE 1 COAT OF PRIMER AND 2 COATS OF PAINT. NUMBERS SHOWN ON PLAN SHALL BE PAINTED ON EACH PANEL IN FIGURES 6" HIGH.

THE TOP PORTION OF SPILLWAY AS SHOWN ON FUTURE SPILLWAY ALTERNATE SHALL HAVE A 28 DAY STRENGTH OF NOT LESS THAN 3000 LBS/□".

THE QUANTITIES OF MATERIALS DO NOT INCLUDE MATERIALS CALLED FOR ON THE ORIGINAL PLANS THAT CAN BE USED IN THE REVISED SPILLWAY SECTION.

PRIOR TO INSERTING THE PIPE STANCHIONS INTO THE SLEEVES THEY SHALL BE BOTH COATED WITH HEAVY LUBRICATING OIL.

DESIGNED BY W. B. Conway
CONSULTING ENGINEER

MILWAUKEE COUNTY REGIONAL PLANNING DEPARTMENT

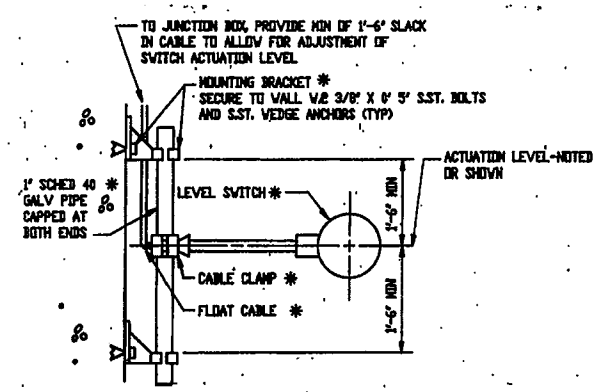
LANDSCAPE DESIGN SECTION	ENGINEERING DESIGN SECTION
Landscape Architect	Roy E. Behrens, Senior Ast. Civil Engineer
Survey / Plan / Tracing / Checked	Approved: Supervising Engineer — Date 5/3/39
	Approved: Highway Commissioner & Planning Engineer — Date 7/4/39

UNITED STATES
DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
COOPERATING WITH
MILWAUKEE COUNTY PARK COMMISSION

DRAWN BY		PLAN NO.
DESIGNED BY		SCALE S.P.
CHECKED BY		DATE

PREPARED BY — SECRETARY MILWAUKEE COUNTY PARK COMMISSION — DATE
RECOMMENDED — INSPECTOR — DATE
RECOMMENDED — REGIONAL OFFICER — DATE
APPROVED BY — ASSISTANT DIRECTOR — DATE

49-5

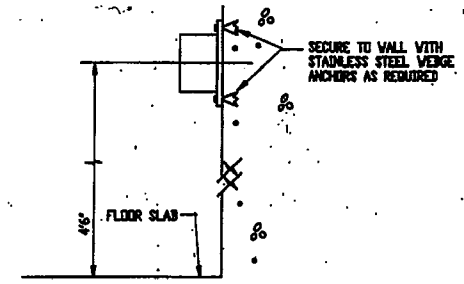


NOTES

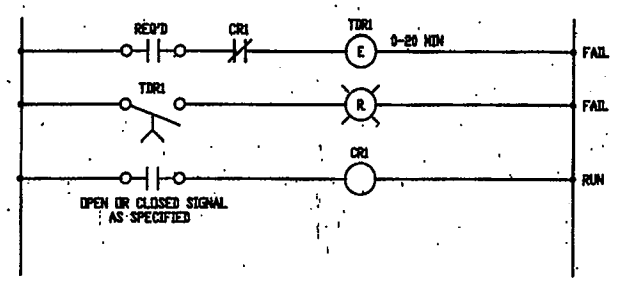
1. * DENOTES ITEMS FURNISHED BY SWITCH MFR

2. INSTALL ALL FLOAT SWITCHES ON SEPARATE 1" SCHED 40 GALVANIZED PIPE. THE FIVE PIPES SHALL BE MOUNTED WITH EVEN SPACING ON CIRCUMFERANCE OF FLOAT CONTROL MANHOLE WALLS. FLOAT ACTIVATION LEVELS ARE 36.1', 36.3', 36.5', 36.7', AND 36.9' ABOVE MILWAUKEE DATUM. PIPE SHALL EXTEND 1'-6" MIN ABOVE AND BELOW SWITCH ACTIVATION LEVEL. PROVIDE SUFFICIENT SLACK IN EACH SWITCH CABLE TO LOWER CLAMP 1'-6" BELOW ACTIVATION LEVEL. ALL SWITCHES SHALL BE INSTALLED SO AS NOT TO ALLOW INTERFERANCE WITH OPERTATION OF ANY OTHER SWITCH.

FLOAT TYPE LEVEL SWITCH (832)
NTS

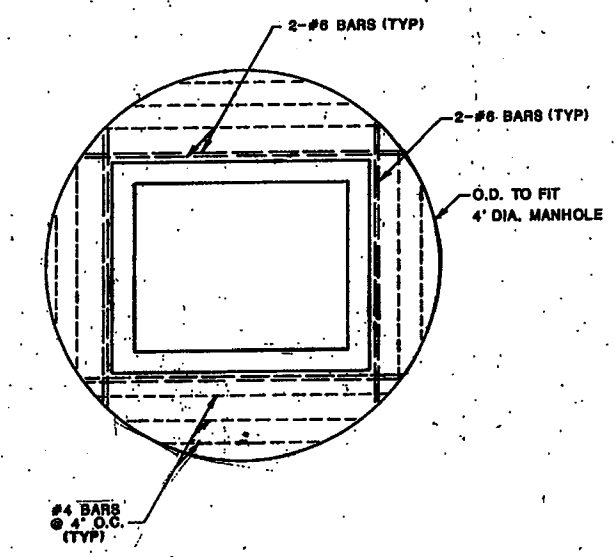


WALL MOUNTED INSTRUMENT INSTALLATION (866)
NTS

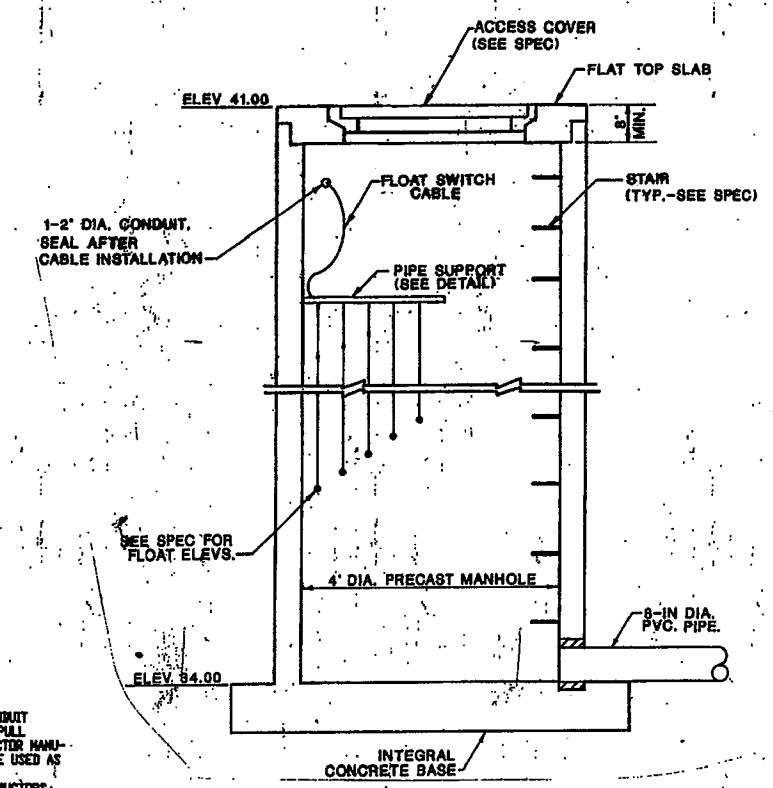


TYPICAL RUN/FAIL CIRCUIT (870)
NTS

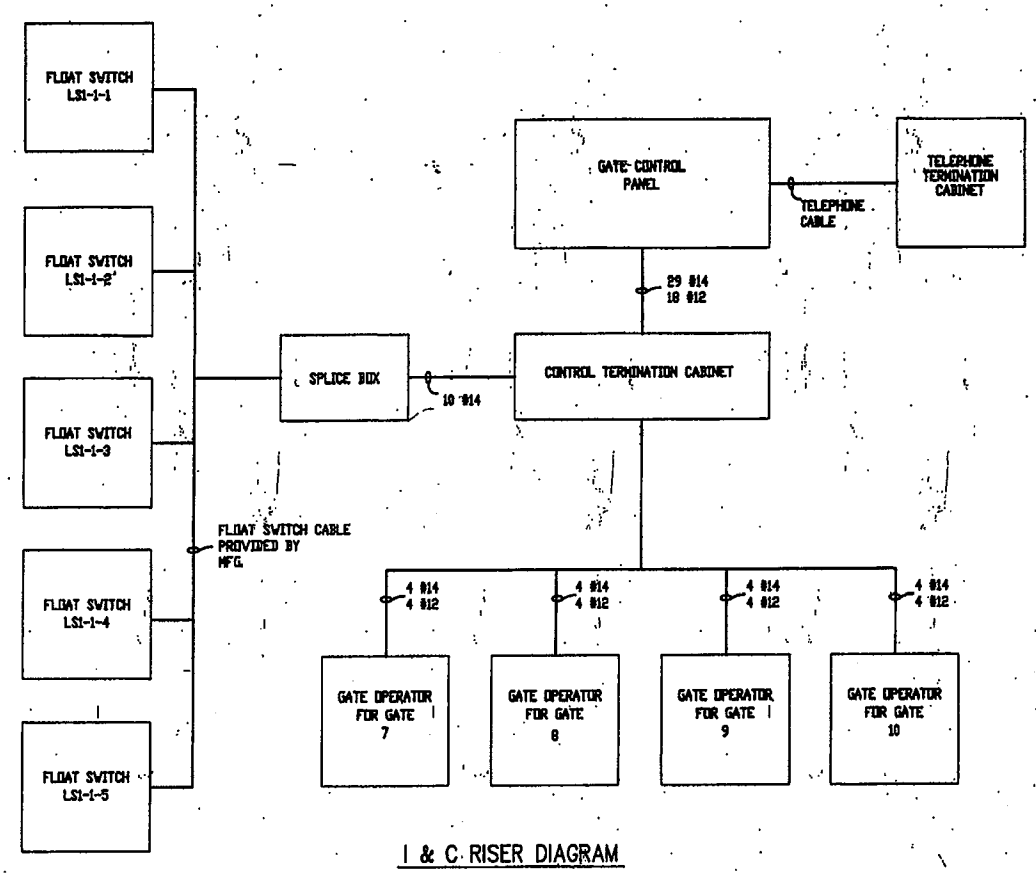
NOTE: SIZE & LOCATE OPENING TO ACCOMODATE FRAME OF COVER SPECIFIED



FLAT TOP SLAB
NTS



FLOAT CONTROL MANHOLE (890)
NTS



I & C RISER DIAGRAM

LEGEND

( ) #12	(QUANTITY) #12 THWN CONDUCTORS
( ) #14	(QUANTITY) #14 THWN CONDUCTORS
( ) SH. PR.	(QUANTITY) SHIELDED PAIR

GENERAL NOTES

1. THIS DRAWING SHOWS PROCESS INSTRUMENTATION AND CONTROL WIRING REQUIREMENTS. ASSOCIATED AC POWER AND GROUNDING CONDUCTORS ARE NOT SHOWN. CONTRACTOR SHALL BE RESPONSIBLE FOR ALL WIRING WHETHER SHOWN OR NOT, NECESSARY FOR A COMPLETE AND OPERABLE SYSTEM.
2. THIS DRAWING SHOWS RELATIVE LOCATIONS OF DEVICES AND PANELS AND IS NOT DRAWN TO SCALE.
3. ALL SHIELDED AND UNSHIELDED CONDUCTORS SHALL BE RUN IN CONDUIT. SHIELDED CONDUCTORS SHALL NOT BE COMBINED WITH UNSHIELDED CONDUCTORS IN ANY CONDUIT. NEITHER SHIELDED OR UNSHIELDED CONDUCTORS SHALL BE INCLUDED IN THE SAME CONDUIT AS THREE PHASE POWER.
4. THIS DRAWING DOES NOT SHOW CONDUIT SYSTEMS. PROVIDE, AS A MINIMUM, PULL BOXES AS RECOMMENDED BY CONDUCTOR MANUFACTURER. CONDULET SHALL NOT BE USED AS PULL BOX.
5. CONDUIT SHALL BE SIZED FOR CONDUCTORS SHOWN PLUS REQUIRED SPARES.
6. SHIELDED AND UNSHIELDED CONDUCTORS SHALL HAVE A MINIMUM OF 6" SEPARATION BETWEEN CONDUIT ON PARALLEL RUNS.
7. SHIELDED CONDUCTORS SHALL BE SEPARATED FROM UNSHIELDED CONDUCTORS BY STEEL BARRIERS IN ALL TERMINAL AND PULL BOXES.
8. FOR EACH CONDUIT CONTAINING MORE THAN TWO CONDUCTORS, PROVIDE A MINIMUM OF TWO CONDUCTORS OR 10% OF TOTAL CONDUCTORS IN CONDUIT, WHICHEVER IS GREATER AS SPARES. TAG BOTH ENDS OF EACH SPARE. TERMINATE EACH END OF SPARE CONDUCTOR AT TERMINALS WHENEVER POSSIBLE.
9. CONDUCTORS SHALL NOT BE SPLICED EXCEPT AT TERMINALS.

Scale	NONE				
Date	5/07/98				
Designer	RJN				
Drafter	RJN				
Checker	AJP				
Approver	AJP	No.	Revisions	By	Date

Donohue Engineers & Architects
COMPUTER AIDED DESIGN/DRAFTING

CONTROL PLAN AND DETAILS

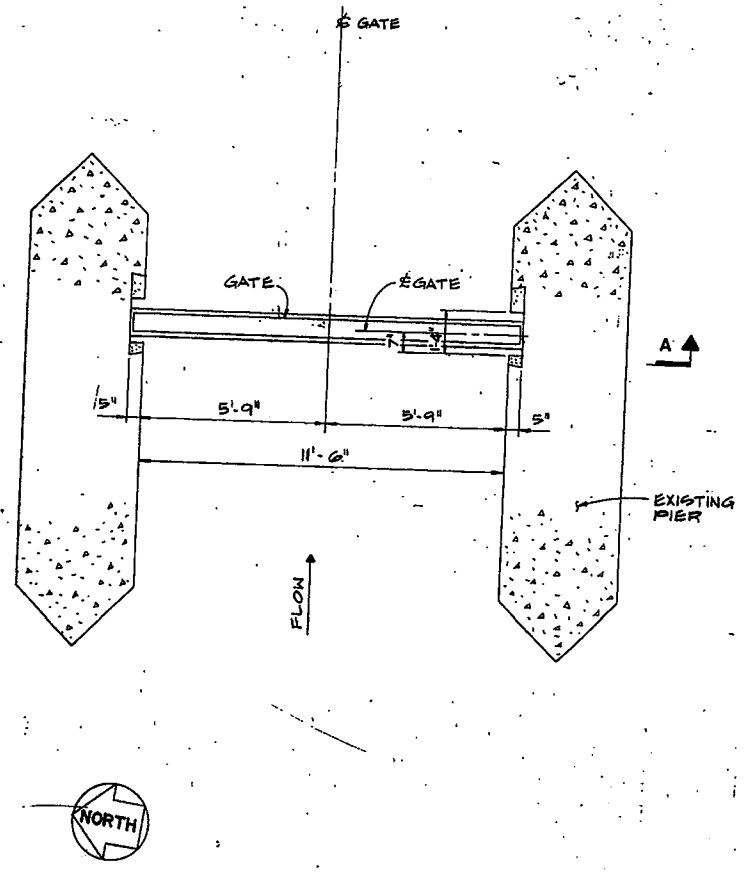
MILWAUKEE COUNTY
ESTABROOK PARK DAM
RESTORATION
PROJECT No. 88-06-4684

Sheet No. 3 OF 3
File No. WA-1516
Project No. 16070
Drawing No. E-2

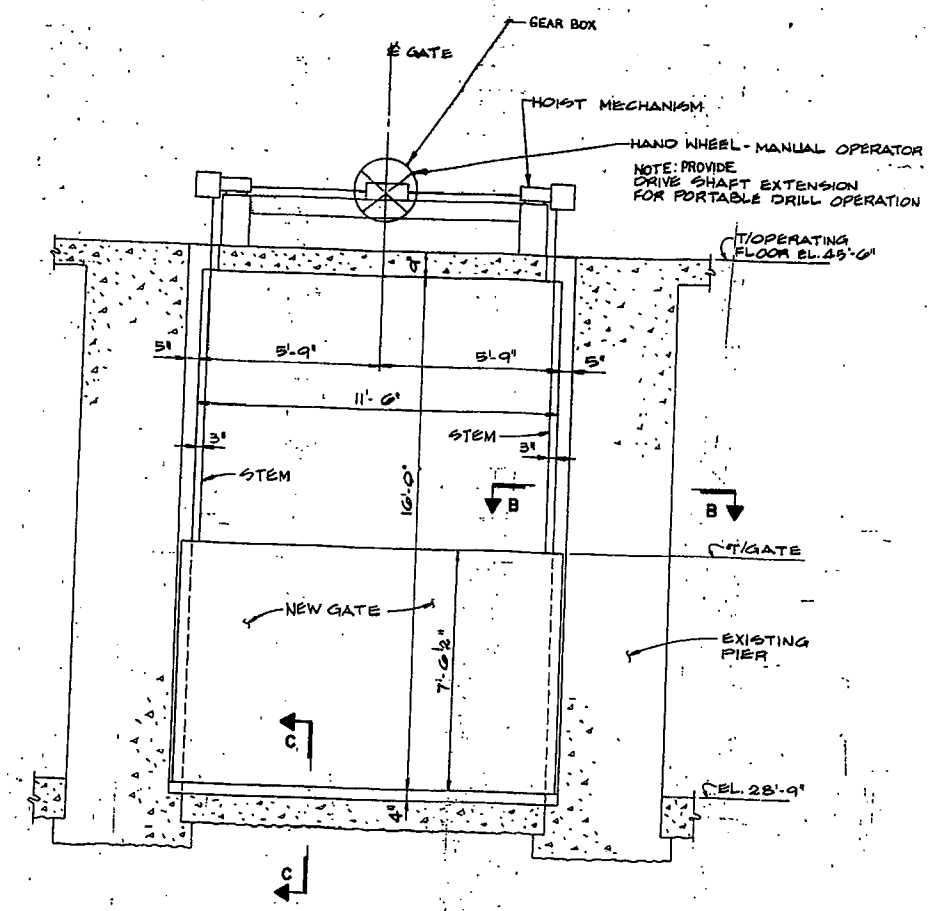
REFERENCE FILES: R1=QSX/CS3001D0R0CBLACK R2= R3=

V0- PLOT SCALE- PLOT NAME- GHALF SIZE

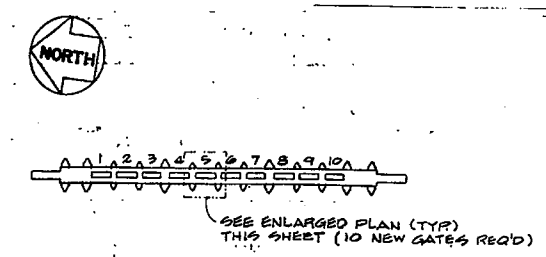
V0- PLOT SCALE- PLOT NAME- DETAIL1 GFULL SIZE-FULL



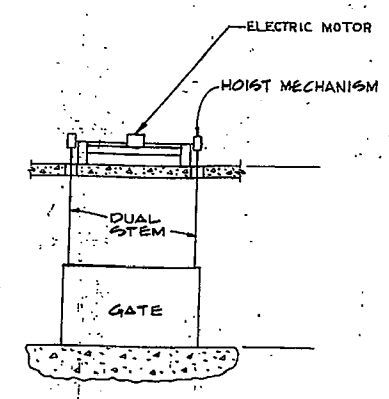
PLAN



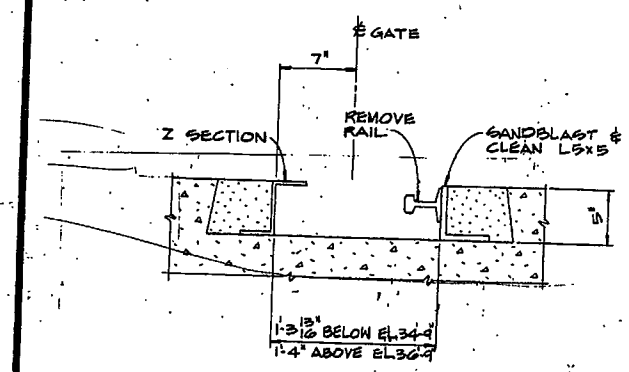
ELEVATION A



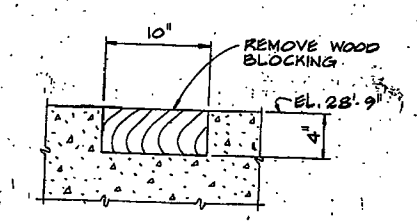
KEY PLAN



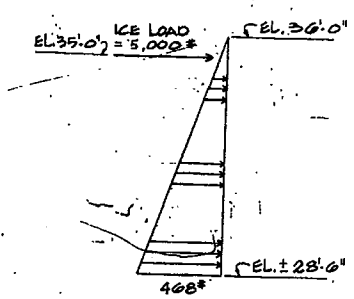
GATE WITH ELECTRIC OPERATOR



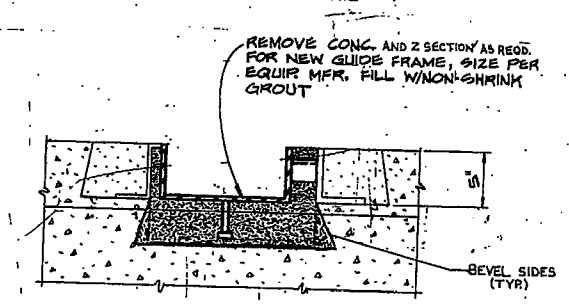
SECTION B
EXISTING GUIDE DETAIL



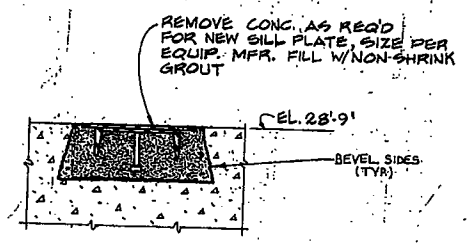
SECTION C
EXISTING SILL DETAIL



GATE PRESSURE DIAGRAM



SECTION B
PROPOSED GUIDE DETAIL



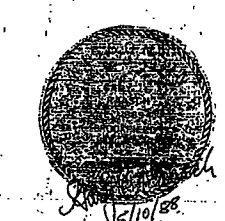
SECTION C
PROPOSED SILL DETAIL

NOTES

1. ALL EXISTING INFORMATION FOR THE DAM IS OBTAINED FROM SHEETS 3, 4, 5, 6, 7 OF 13 PROVIDED BY MILWAUKEE COUNTY.
2. CONTRACTOR SHALL VERIFY ALL FIELD CONDITIONS AND DIMENSIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH WORK.
3. ALL WORKS ON THIS PROJECT SHALL CONFORM TO THE FOLLOWING LATEST SPECIFICATIONS:
   A. AMERICAN CONCRETE INSTITUTE
   B. AMERICAN INSTITUTE OF STEEL CONSTRUCTION
   C. AMERICAN WELDING SOCIETY
   D. AWWA SLUICE GATE SPECIFICATIONS

REFERENCE DRAWINGS

THE CONTRACTOR SHALL REFER EXISTING DRAWINGS FOR THE DAM, AVAILABLE AT THE OFFICE OF MILWAUKEE COUNTY, DEPARTMENT OF PUBLIC WORKS



Donohue
Engineers & Architects

Scale	1"=10'-0"
Date	5-9-88
Designer	
Drafter	
Checker	
Approved	

No.	Revisions	By	Date

GATE PLAN AND DETAILS

MILWAUKEE COUNTY
ESTABROOK PARK DAM
RESTORATION
PROJECT No. 88-06-4684

Sheet No. 1 OF 3
File No.
Project No.
Drawing No. M-1

Appendix F

Wisconsin DNR Order to Repair or Abandon the Estabrook Dam

State of Wisconsin \ DEPARTMENT OF NATURAL RESOURCES

Jim Doyle, Governor
Matthew J. Frank, Secretary
Gloria L. McCutcheon, Regional Director

Southeast Region Headquarters
2300 N. Dr. Martin Luther King, Jr. Drive
Milwaukee, Wisconsin 53212-3128
FAX 414-263-8606
Telephone 414-263-8500
TTY Access via relay - 711

Certified Mail
Return Receipt Requested

FILE REF: 8600

July 28, 2009

Ms. Sue Black, Director
Milwaukee County Parks Department
9480 Watertown Plank Road
Wauwatosa, WI 53226

Dear Ms. Black:

Public health and safety concerns require the Department to ensure that the Estabrook Dam meets acceptable design standards. An Order to Repair or Abandon the Estabrook Dam is attached to this letter. This Order establishes:

- A deadline for completing a drawdown of the Estabrook Dam
- A requirement that a professional engineer registered in the State of Wisconsin complete and certify plans and specifications for repairs identified in the October 16, 2007, revised work schedule
- A requirement that a competent contractor complete repairs in accordance with approved plans and specifications
- A requirement that a professional engineer registered in the State of Wisconsin certify that the repairs are completed and conform to the approved engineering plans
- A deadline for performing detailed stability analyses for the entire structure under all loading conditions

The impoundment shall not be refilled until all repairs identified in the October 16, 2007, revised work schedule have been completed. The authorization to refill the impoundment cannot be granted until the detailed stability analyses are reviewed and approved by the Department. If the analyses conclude that structural improvements to the dam are needed to meet stability requirements, it may be necessary to complete these modifications prior to refilling the impoundment. After receiving the stability analyses, the Department will identify and outline any additional requirements and deadlines in writing. The impoundment shall remain drawn down until the Department provides written authorization that requirements of the Order are complete.

The Order allows Milwaukee County the option to consider dam abandonment. State law and Wisconsin Supreme Court decisions make it clear that the dam owner decides whether to repair or abandon their dam.

While deadlines for completing the project stages are included in the Order, we encourage the County to proceed in an expeditious manner. For example, the County may begin by concentrating efforts on removing debris and sediment behind the fixed crest spillway and completing the stability analyses under all loading conditions. Past experience has shown that stability analyses inform dam owners about full repairs needed at the dam. Owners can then refine cost estimates and make fully informed long-term financial decisions regarding the dam.

Please note that prior to performing any work on the dam, you must receive Department approval of the plans and specifications.

If you have any questions regarding this Order, please contact Tanya Meyer at (414) 263-8641. Thank you for your cooperation.

Sincerely,

Tanya L. Meyer
Water Management Engineer
Milwaukee Service Center

William D. Sturtevant, P.E.
State Dam Safety Engineer
Bureau of Watershed Management

cc: Milwaukee County Parks, Energy and Environment Committee
Milwaukee County Supervisor Willie Johnson, Jr.
Mr. Jim Keegan, Chief of Planning, Policy and Development – Milwaukee County Parks Department
Mr. Greg High, Director – Milwaukee County Architectural, Engineering, & Environmental Services
Ms Julie Esch, Senior Research Analyst – Milwaukee County Board of Supervisors
Ms. Gloria McCutcheon, Regional Director – DNR Southeast Region Headquarters
Ms. Meg Galloway, Dam Safety & Floodplain Section Chief – DNR GEF II, WT/3

BEFORE THE
DEPARTMENT OF NATURAL RESOURCES

IN THE MATTER of the Unsafe Condition of the Estabrook Dam, Located on the Milwaukee River, Milwaukee County.

ORDER FOR DAM REPAIR OR ABANDONMENT

FINDINGS OF FACT

The Department finds that:

1. The Estabrook Dam is located on the Milwaukee River in the Northwest Quarter of the Northeast Quarter of Section 5, Township 7 North, Range 22 East, in the Cities of Glendale and Milwaukee, Milwaukee County. The Estabrook Dam impounds the Milwaukee River.

2. The Estabrook Dam has an estimated hazard rating of low. The dam consists of a gated spillway section, an overflow weir spillway section, and concrete tripods. The gated spillway section is approximately 220 feet in length with 10 vertical steel slide gates. The sinuous overflow weir spillway section is approximately 580 feet in length. It is concrete with rubble stone masonry and has stoplog sections near the left end, which are approximately 88 feet in length. A row of 28 concrete tripods, also known as "ice breakers", is located upstream of the gated spillway section.

3. The Estabrook Dam is currently owned by Milwaukee County. The Milwaukee County Parks Department, 9480 Watertown Plank Road, Wauwatosa, WI 53226, oversees the inspection, operation, and maintenance of the Estabrook Dam.

4. An inspection of the dam was performed by Department of Natural Resources (Department) Dam Safety Staff on July 13, 1994. The following deficiencies were noted and deadlines were established:
   a. Remove debris – fixed crest spillway (May 1, 1995)
   b. Repair concrete – all piers (October 1, 1997)
   c. Remove trees and brush – all abutments (June 1, 1995)
   d. Install dam warning and portage signs (June 1, 1995)
   e. Develop Inspection, Operation and Maintenance Plan (October 1, 1996)
   f. Establish benchmarks (June 1, 1996)
   g. Monitor seepage at gated spillway abutment (Ongoing)

   A detailed explanation of the inspection findings is provided in the January 23, 1995, Dam Safety Inspection Report. Many of the deficiencies require the dam owner to employ the services of a professional engineer registered in the State of Wisconsin to design the repairs.

5. An inspection of the dam was performed by Department Dam Safety Staff on October 27, 2004 and the following deficiencies were noted and deadlines were established:
   a. Remove debris – fixed crest spillway (December 31, 2005)
   b. Repair concrete – all piers (December 31, 2006)
   c. Remove trees and shrubs – all abutments (June 1, 2005)
   d. Install dam warning and portage signs (March 1, 2005)
   e. Develop Inspection, Operation and Maintenance Plan (December 31, 2005)
   f. Complete structural analysis of the dam (December 31, 2005)
   g. Complete scour/undermining analysis (December 31, 2005)
   h. Replace stoplogs and supports – fixed crest spillway (December 31, 2005)

5. (Continued)
   i. Repair or remove ice breakers. If removal is proposed, complete structural analysis of the dam including ice and debris loading on the gated spillway prior to removal. (December 31, 2006)
   j. Stabilize erosion at banks – abutments of gated spillway (June 1, 2005)
   k. Sandblast and clean gates, clean or replace gate seals, repair Gates #6 and #7, and install manual override for all gates (December 31, 2006)
   l. Rebuild concrete access stairs (December 31, 2005)
   m. Work with the Department to review the operational water level order for the dam (No deadline established)

   A detailed explanation of the inspection findings is provided in the November 22, 2004, Dam Safety Inspection Report. Many of the deficiencies require the dam owner to employ the services of a professional engineer registered in the State of Wisconsin to design the repairs.

6. Milwaukee County completed the following work by the dates noted:
   a. Installed dam warning and portage signs (Approximately June 2005)
   b. Completed structural analysis (September 8, 2006)
      - Findings of Fact, Sections 9 and 10 provide additional information regarding the stability analysis
   c. Completed scour/undermining analysis (September 8, 2006)
   d. Repaired and upgraded gates (2001-2002)
   e. Repaired stoplogs and supports – fixed crest spillway (July 10, 2009)
   f. Removed accumulated woody debris upstream of the ice breakers (Occurs annually, typically in May)

7. Deadlines for outstanding repairs have passed. A time extension was granted to Milwaukee County for completing the required repairs to accommodate the Milwaukee County budget process. The time extensions are provided in an October 16, 2007, Revised Work Schedule. A number of deadlines on the Revised Work Schedule have also passed.

8. To date, Milwaukee County has not budgeted the money necessary to complete all repairs required in the Dam Safety Inspection Reports.

9. In addition to Department dam safety inspections, Milwaukee County contracted with STS Consultants, Ltd., to perform an engineering evaluation and scour/undermining analysis for the Estabrook Dam. The September 8, 2006, report detailed results of the condition evaluation and stability analysis, provided recommended repairs, and detailed an opinion of probable costs for recommended repairs. Findings of Fact, Section 10 provides additional information regarding the stability analysis. The following repairs were recommended in the executive summary:
   a. Remove and replace deteriorated concrete on abutments, operating bridge deck, and piers of the gated spillway section.
   b. Repair expansion joints on spillway operating deck and replace missing expansion joint material.
   c. Sandblast and paint spillway gates. Drill drain holes in horizontal cross members to promote drainage.
   d. Place riprap along left bank downstream of gated spillway and left and right banks upstream of gated spillway to prevent further undercutting.
   e. Replace overflow spillway flashboards and repair bent supports. Repair surface deterioration of concrete on overflow weir crest.
   f. Repair deterioration of tops of ice breakers.
   g. Remove accumulation of woody debris upstream of overflow spillway section.

9. (Continued)
Repairs recommended by STS Consultants, Ltd., that were not previously identified in the Department 1995 and 2004 Dam Safety Inspection Reports were added to the October 16, 2007, Revised Work Schedule.

10. STS Consultants, Ltd., completed a stability analysis of Estabrook Dam and summarized the findings in the September 8, 2006, engineering evaluation. The report states that the gated spillway section was found to be stable for overturning and sliding under normal pool conditions (e.g., normal water levels with gates closed). Under full pool conditions with ice loading, the section was found to be unstable. Based upon these preliminary findings, the Department requires a detailed stability analyses for the Estabrook Dam under all loading conditions. The requirement for detailed stability analyses is identified in the October 16, 2007, Revised Work Schedule and the established deadline is October 1, 2010.

11. The accumulation of woody debris behind the fixed crest spillway has reduced the hydraulic capacity of the dam. The woody debris could cause extreme loading conditions of the fixed crest spillway. The woody debris is co-mingled with contaminated sediment and man-made debris. The debris has not been removed since at least the 1994 Department Dam Safety Inspection, resulting in 15 years without annual maintenance for debris removal behind the fixed crest spillway.

12. On September 25, 2008, Department personnel evaluated Estabrook Dam to assess possible damage to a section of the fixed crest spillway. The dam was determined to have stoplogs missing from one section of the fixed crest spillway. The partial failure of a portion of the dam caused an uncontrolled release of water from the impoundment. The Department issued a Repair Order on September 26, 2208. Repairs were completed on July 10, 2009.

13. Another partial failure of a portion of the dam occurred during the 2008-2009 winter season. One ice breaker is no longer intact.

14. The dam in its present condition is not sufficiently strong and is unsafe and dangerous to life, health and property.

CONCLUSIONS OF LAW

The Department concludes that:

1. The Department of Natural Resources has authority pursuant to sections 31.02 and 31.19, Wisconsin Statutes, to inspect or cause an inspection to be made of any dam or reservoir.

2. The Department of Natural Resources has authority pursuant to sections 31.02 and 31.19, Wisconsin Statutes, to order alterations and repairs to any dam that is not sufficiently strong or is unsafe and is dangerous to life, health and property.

3. The Department of Natural Resources has authority pursuant to sections 31.02 and 31.19, Wisconsin Statutes, to order the drawdown of the impoundment above a dam that is not sufficiently strong or is unsafe and dangerous to life, health and property.

ORDER

It is therefore ordered that:

1. The owner of the Estabrook Dam shall maintain the current drawn down condition of the impoundment by keeping all gates fully open and secured in place. With this Order in effect, the dam shall remain drawn down until all conditions within the Order are in compliance. The owner shall inspect the dam on a daily basis to ensure that no additional materials to obstruct flow are placed in the structure, and shall promptly remove any such obstructions found. Obstructions could include debris carried in the channel or unauthorized activities to re-impound water.

2. As required in the October 16, 2007, Revised Work Schedule, the owner of the Estabrook Dam shall engage the services of a professional engineer registered in the State of Wisconsin to perform detailed stability analyses for the entire structure under all loading conditions in compliance with chapter NR 333, Wisconsin Administrative Code. The stability analyses must be submitted for Department review and approval by **October 1, 2010.**

3. The owner of Estabrook Dam shall engage the services of a professional engineer registered in the State of Wisconsin to inspect the dam and to develop plans and specifications in compliance with chapter NR 333, Wisconsin Administrative Code, for repair of the dam. Repairs shall include items identified on the October 16, 2007, Revised Work Schedule. The owner must submit those plans and specifications to the Department of Natural Resources for review and approval by **July 29, 2011.**

4. Upon review and approval of the plans and specifications by the Department, the owner shall select a competent contractor to repair the structure in accordance with the approved plans and specifications by **July 27, 2012.** All permits necessary for construction and sediment removal shall be obtained from the Southeast Region Office, Milwaukee, prior to initiation of construction, including proper testing, removal, transport, and disposal of contaminated sediment.

5. The owner shall provide the Department with documentation and photographs demonstrating that the work has been completed in accordance with the approved plans and specifications. A professional engineer registered in the State of Wisconsin shall certify that the repairs are completed as designed.

6. The impoundment shall not be refilled until all repairs identified in the October 16, 2007, revised work schedule have been completed. The authorization to refill the impoundment cannot be granted until the detailed stability analyses are reviewed and approved by the Department. After receiving the stability analyses, the Department will identify and outline any additional requirements and deadlines in writing. The impoundment shall remain drawn down until the Department provides written authorization that requirements of this Order are complete.

7. Not more than 10 days after the completion of the repairs of the Estabrook Dam, the owner of the Estabrook Dam shall submit an Inspection, Operation and Maintenance Plan and an Emergency Action Plan to the Department for review and approval.

8. In lieu of provisions 2 through 5 and provision 7 of this Order, the owner of the Estabrook Dam shall submit an application for a permit to abandon the dam pursuant to section 31.185, Wisconsin Statutes, by **January 28, 2011.** If an application is submitted, the owner of the Estabrook Dam shall remain responsible for the dam until a permit to abandon the dam is issued and all the conditions of that permit have been met.

9. The owner of the Estabrook Dam shall provide the Department written notification of its intent to repair or abandon the dam, by **January 28, 2011.**

NOTICE OF APPEAL RIGHTS

If you believe that you have the right to challenge this decision, you should know that Wisconsin statutes and administrative rules establish time periods within which requests to review Department decisions must be filed.

To request a contested case hearing pursuant to section 227.42, Wisconsin Statutes, you have 30 days after the decision is mailed or otherwise served by the Department of Natural Resources. The filing of a request for a contested case hearing is not a prerequisite for judicial review and does not extend the 30-day period for filing a petition for judicial review.

This decision was mailed on July 28, 2009.

STATE OF WISCONSIN DEPARTMENT OF NATURAL RESOURCES
For the Secretary

By Tanya L Meyer July 28, 2009

Tanya L. Meyer
Water Management Engineer
Milwaukee Service Center

Date

By William D Sturtevant 7/28/09

William D. Sturtevant, P.E.
State Dam Safety Engineer
Bureau of Watershed Management

Date

EROSION CONTROL PLAN CHECKLIST

The following checklist serves as guidance in the completion of the Erosion Control Plan necessary to meet the requirements of the Chapter 30 and NR 216 Permits. The Erosion Control Plan shall contain at minimum all of the following components:

- ☐ **Project Description**
  Briefly describe the project applicable to the erosion control plan.

- ☐ **Erosion Control Methods and Materials**
  - ☐ The types of erosion control methods that will be used during project construction to protect disturbed areas. Include:
    - ☐ Where applicable
    - ☐ Soil and slope stabilization
    - ☐ Seeding and mulching
    - ☐ Matting, tracking pads, inlet filters, silt fences, drainage swales, sedimentation basins, stockpile protection
    - ☐ Dewatering-related erosion control
    - ☐ Channel protection
    - ☐ Outfall protection
    - ☐ Any other appropriate erosion control measures
  - ☐ Details and typical section drawings of all the erosion control methods utilized.

- ☐ **Erosion Control Measure Site Plan**
  Include a site plan view and typical drawings illustrating:
  - ☐ Construction site boundary
  - ☐ The location of all erosion control measures
  - ☐ Location of stockpiled soil
  - ☐ Vehicle and equipment access sites
  - ☐ Areas of disturbance
  - ☐ The drainage area configuration
  - ☐ Surface water diversion measures
  - ☐ Pre- and post-construction drainage patterns
  - ☐ Methods of collecting, transporting, and dispersing storm water flow
  - ☐ Topography
  - ☐ Existing floodplains and wetlands
  - ☐ Location of trees and unique vegetation

- ☐ **Sequence of Erosion Control Measures**
  List and give a detailed description of the sequence of erosion control measures that will occur (i.e. placed, relocated, and replaced) during all phases of construction including:
  - ☐ Clearing and Grubbing
  - ☐ Concrete removal
  - ☐ Material installation
  - ☐ Channel construction

- Revegetation processes
- Seeding and mulching/matting
- Maintenance

Note the erosion control sequence to be followed for each event of the project.

- **Off-site Diversion Methods**
  - Identify off-site contributions of water effecting project site
  - Methods of controlling off-site water contributions
  - Site plan indicating:
    - Where the off-site water is originating from
    - Locations of diversion measures on-site

- **Computations**

  The following computations should be submitted if applicable to the project:
  - Designed erosion control measures
    - Sediment tap efficiency
    - Runoff volumes
  - Contributing watershed area to each erosion control measure
    - On-site watershed area
    - Off-site watershed area
  - Peak discharge and velocities
  - Direction and destination of flows
  - Ditch and Drainage easement sizing

- **Provisions for Inspection and Maintenance**

  Document the provisions for:
  - The regular inspection of all erosion control efforts
    - Who will perform the inspections
    - When will the inspections occur
    - Any special circumstances initiating an inspection
  - The regular maintenance of all erosion control efforts
    - Who is responsible for the maintenance
    - How often will the site be maintained
    - Corrective action is site is not maintained according to provisions

MATERIALS MANAGEMENT PLAN CHECKLIST

The following checklist serves as guidance in the completion of the Materials Management Plan necessary to meet the requirements of the Chapter 30 and NR 216 Permits. The Materials Management Plan shall contain at minimum, specific details on all of the following components:

- ☐ **Access Point Locations**
  - ☐ List the locations that will be used to gain access to the work site
  - ☐ Include a plan view of all access points

- ☐ **Haul Routes**
  Indicate how and where hauled materials will be routed, including:
    - ☐ Inbound materials
    - ☐ Outbound materials
    - ☐ Clean fill materials
    - ☐ Contaminated materials
    - ☐ Others
  - ☐ Alternate locations if necessary
  - ☐ Include a haul route diagram indicating haul route locations

- ☐ **Stockpile Areas**
  List and describe:
    - ☐ Material to be stockpiled
    - ☐ Where will material be stockpiled on-site
    - ☐ Measures to protect stockpiled areas if applicable
  - ☐ Provide a plan view diagram indicating stockpile area locations

- ☐ **Equipment**
  - ☐ Staging Areas:
    - ☐ Where equipment will be stored on-site
    - ☐ Include a plan view of equipment storage areas on-site
  - ☐ On-site use and Placement
    - ☐ Spill control and kits on-site

- ☐ **Field Screening Protocol for Contaminant Testing**
  If contaminated materials (i.e. soil) are encountered on-site, indicate:
  - ☐ How will the materials be screened
  - ☐ Where will the materials be tested
  - ☐ What protocols will be followed
  - ☐ How work will be impacted

  Notify WDNR and Solid Waste of contaminated material encountered on-site.

- **Contaminated Materials Management Protocol**
  (Will obtain from DNR)

- **Estimated Types, Concentrations and Volumes of Contaminated Materials**
  If contaminated materials are know to exist on-site, list and describe:
  - The type of contaminant
  - Where the contaminant is located on-site
  - Media in which the contaminant is located within (i.e. soil, water, etc.)
  - The estimated concentration of the contaminant
  - The estimated volumes of the contaminant

- **Excavation Methods**
  List and describe:
  - What materials will be excavated
  - Where the excavated materials are located
  - How the materials will be excavated and removed
  - How will excavated materials be exported from site
  - Where will excavated materials be exported to

- **Methods for Dewatering of Excavated Materials**
  If free water is found present in excavated materials, list and describe:
  - What methods will be used to correct the situation (i.e. how will water be removed)
  - Where these methods will take place on-site

- **Estimated Volumes of In-channel and Upland Excavated Materials**
  - Volume of Dredged Materials (cubic yards)
    - Excavation from bed and bank of waterway
    - Excavation from wetland
  - Volume of Upland Materials (cubic yards)
    - Excavation from areas outside of waterway and wetlands
  - Total Volume of Materials (cubic yards)

Estimated Volumes and Location of Re-used In-Channel and Upland Excavated Materials
- Reuse of Dredged Materials
  - Total Volume of Reused Dredged Materials (cubic yards)
  - Location
    - Indicate on project plans OR
    - Provide off-site address, property owner, site map drawn to scale
  - Purpose of Dredged Materials (i.e. grading, trench backfill, etc.)
- Reuse of Upland Materials
  - Total Volume of Reused Upland Materials (cubic yards)
  - Location
    - Indicate on project plans OR
    - Provide off-site address, property owner, site map drawn to scale

- Purpose of Upland Material Usage

- **Off-site Disposal Plans for Contaminated Materials and Non-contaminated Materials**
  - Disposal of Dredged Materials
    - Total Volume of Disposed Materials (cubic yards)
    - Disposal Site Location
      - Type of Disposal Site (i.e. confined disposal facility, landfill, etc.)
      - Disposal Site Information
        - Site name
        - Site address
  - Disposal of Upland Materials
    - Total Volume of Disposed Upland Materials (cubic yards)
    - Disposal Site Locations
      - Type of Disposal Site (i.e. confined disposal facility, landfill, etc.)
      - Disposal Site Information
        - Site Name
        - Site address
  - Solid Waste Program Approvals

DEWATERING PLAN CHECKLIST

The following checklist serves as guidance in the completion of the Dewatering Plan necessary to meet the requirements of the Chapter 30 and NR 216 Permits. The Dewatering Plan shall contain at minimum all of the following components:

- ☐ **Dewatering/Diversion of Flow**
  Provide detailed plans for the dewatering / diversion of flow/ standing water removal
  - ☐ Typical dewatering / diversion measure plans

  Provide specifications for the dewatering / diversion of flow/ standing water removal
  - ☐ Methods employed to dewater / divert flow/ treat water (if applicable)
  - ☐ How will methods be employed
  - ☐ Where will methods be employed
  - ☐ Capacities and capabilities

- ☐ **Downstream Impact Minimization**
  List and describe:
  - ☐ Methods of minimizing downstream impacts during high flow conditions

- ☐ **Analysis of Possible System Overload Scenarios**
  Provide the following information if the stream is overloaded:
  - ☐ Estimated volume of system overload (i.e. what rainfall overloads the system)
  - ☐ Estimated frequency of system overload (i.e. how often will the system be overloaded)
  - ☐ Actions taken if stream is to be overloaded

- ☐ **Impacts of System Overload on Construction Activities and Water Quality**
  List and describe:
  - ☐ Anticipated number of lost work days
  - ☐ Possible water quality impacts
  - ☐ Methods of deterring adverse changes in water quality

- ☐ **Discharge Locations**
  - ☐ Indicate the following regarding the discharge of water:
    - ☐ Where the water be discharged to
    - ☐ How the water will be discharged
  - ☐ Provide a site map indicating discharge locations

- ☐ **Details of a Back-up System**
  If a back-up system becomes necessary, indicate:
  - ☐ What type of back-up system will be used (include backup and standby equipment/power supply)
  - ☐ Conditions when the system will be needed
  - ☐ How the back-up system operated
  - ☐ Where the system will be located

- **High Flow Plan**
  When flooding is likely to occur, list and describe the following:
  - How the water be removed from the site
    - Methods of water removal (i.e. pumping)
    - Methods of minimizing water contamination (i.e. treatment methods)
  - Protocol for evacuating materials from the flood conveyance channel including:
    - List of materials that would require evacuation during high flow periods
    - How will the materials be evacuated from the flood conveyance channel
    - Where will the materials be temporarily placed on-site
    - How will the materials be transported
    - Methods of protecting the materials
    - Include a site map indicating the location of temporary placement
  - Protocol for evacuating machinery from the flood conveyance channel including:
    - Type of machinery that would require evacuation during high flow periods
    - How will the machinery be evacuated from the flood conveyance channel
    - Where will the machinery be temporarily placed on-site
    - Include site map indicating possible locations of temporary machinery placement

- **Contaminated Water**
  List and describe what measures will be taken if contaminated water is found on-site including:
  - Methods of isolating the contaminated water
  - Methods of analyzing the contaminated water
  - Where the water will be tested
  - Methods of removing contaminated water from site
  - How the water will be treated and/or disposed of

SEQUENCE OF WORK PLAN CHECKLIST

The following checklist serves as guidance in the completion of the Sequence of Work Plan necessary to meet the requirements of the Chapter 30 and NR 216 Permits. The Sequence of Work Plan shall contain at minimum all of the following components, all which should be closely related to the Dewatering, Erosion Control, and Materials Management Plans:

- ☐ **Sequencing of Construction**
  - ☐ Include a detailed description of the planned step-by-step events that will take place during construction including:
    - ☐ Construction Stage Activities
      - ☐ Demobilization
      - ☐ Clearing and Grubbing
      - ☐ Erosion control measures
      - ☐ Concrete removal
      - ☐ Material installation
      - ☐ Channel construction
      - ☐ Revegetation processes
      - ☐ Seeding and mulching/matting
      - ☐ Maintenance

- ☐ **Project schedule indicating description of work and anticipated start and finish date for each activity, including erosion controls, dewatering, and materials management**

VEOLIA ENVIRONMENTAL SERVICES

Emerald Park Landfill
Protocol II Acceptance Limits

General Parameters	
pH	$2.0 \leq pH \leq 12.5$
Free Liquids	No Free liquids
Acidity in %	Analyze if pH < 4
Alkalinity in %	Analyze if pH > 10
Flash Point	>140^0 F
Phenol	<2000 mg/l
Reactive Cyanide	<250 mg/l
Reactive Sulfide	<500 mg/l
% Chlorine	<1%
PCBs	<50 ppm

Metals	TCLP Limit
Arsenic	<5.0 mg/l
Barium	<100.0 mg/l
Cadmium	<1.0 mg/l
Chromium	<5.0 mg/l
Copper	<200.0 mg/l
Lead	<5.0 mg/l
Mercury	<0.2 mg/l
Nickel	<35.0 mg/l
Selenium	<1.0 mg/l
Silver	<5.0 mg/l
Zinc	<500.0 mg/l

Pesticides and Herbicides *	
Chlordane	<0.03 mg/l
2,4 D	<10.0 mg/l
Endrin	<0.02 mg/l
Heptachlor	<0.02 mg/l
Lindane	<0.04 mg/l
Methoxychlor	<10.0 mg/l
Toxaphene	<0.05 mg/l
2,4,5-TP (Silvex)	<1.0 mg/l

* = Generator may certify without testing, unless the waste is suspected to contain pesticides or herbicides

Questions? Call John Budzinski at 414-217-6131

TCLP List Organic Compounds (volatiles and semi-volatiles)	
Parameter	TCLP Limit
Benzene	<0.5 mg/l
Carbon Tetrachloride	<0.5 mg/l
Chlorobenzene	<100.0 mg/l
Chloroform	<6.0 mg/l
o-Cresol	<200.0 mg/l
m-Cresol	<200.0 mg/l
p-Cresol	<200.0 mg/l
1,4-Dichlorobenzene	<7.5 mg/l
1,2-Dichloroethane	<0.5 mg/l
1,1-Dichloroethylene	<0.7 mg/l
2,4-Dinitrotoluene	<0.13 mg/l
Hexachlorobenzene	<0.13 mg/l
Hexachloro-1,3-butadiene	<0.5 mg/l
Hexachloroethane	<3.0 mg/l
Methyl Ethyl Ketone	<200.0 mg/l
Nitrobenzene	<2.0 mg/l
Pentachlorophenol	<100.0 mg/l
Pyridine	<5.0 mg/l
Tetrachloroethylene	<0.7 mg/l
Trichloroethylene	<0.5 mg/l
2,4,5-Trichlorophenol	<400.0 mg/l
2,4,6-Trichlorophenol	<2.0 mg/l
Vinyl Chloride	<0.2 mg/l

If the waste has no free liquids, all parameters identified as TCLP analyses can be analyzed as "totals" instead of the TCLP. If the "totals" analysis for each parameter are < 20 times the TCLP acceptance limits, the TCLP need not be performed for the purposes of determining waste acceptance.

Veolias ES Emerald Park Landfill
W124 S. 10629 South 124th St.
Muskego, WI 53150
phone: 414-529-1360
fax = 414-529-1478

NR 347.05 Preliminary application and analytical requirements.

(1) Prior to submission of a formal application, anyone seeking to remove material from the beds of waterways shall provide the department with preliminary information including:

(a) Name of waterbody and location of project;

(b) Volume of material to be dredged;

(c) Brief description of dredging method and equipment;

(d) Brief description of proposed disposal method and location and, if a disposal facility is to be used, size of the disposal facility;

(e) Any previous sediment sampling (including field observations) and analysis data from the area to be dredged or from the proposed disposal site;

(f) Copy of a map showing the area to be dredged, the depth of cut, the specific location of the proposed sediment sampling sites and the bathymetry of the area to be dredged; and

(g) Anticipated starting and completion dates of the proposed project.