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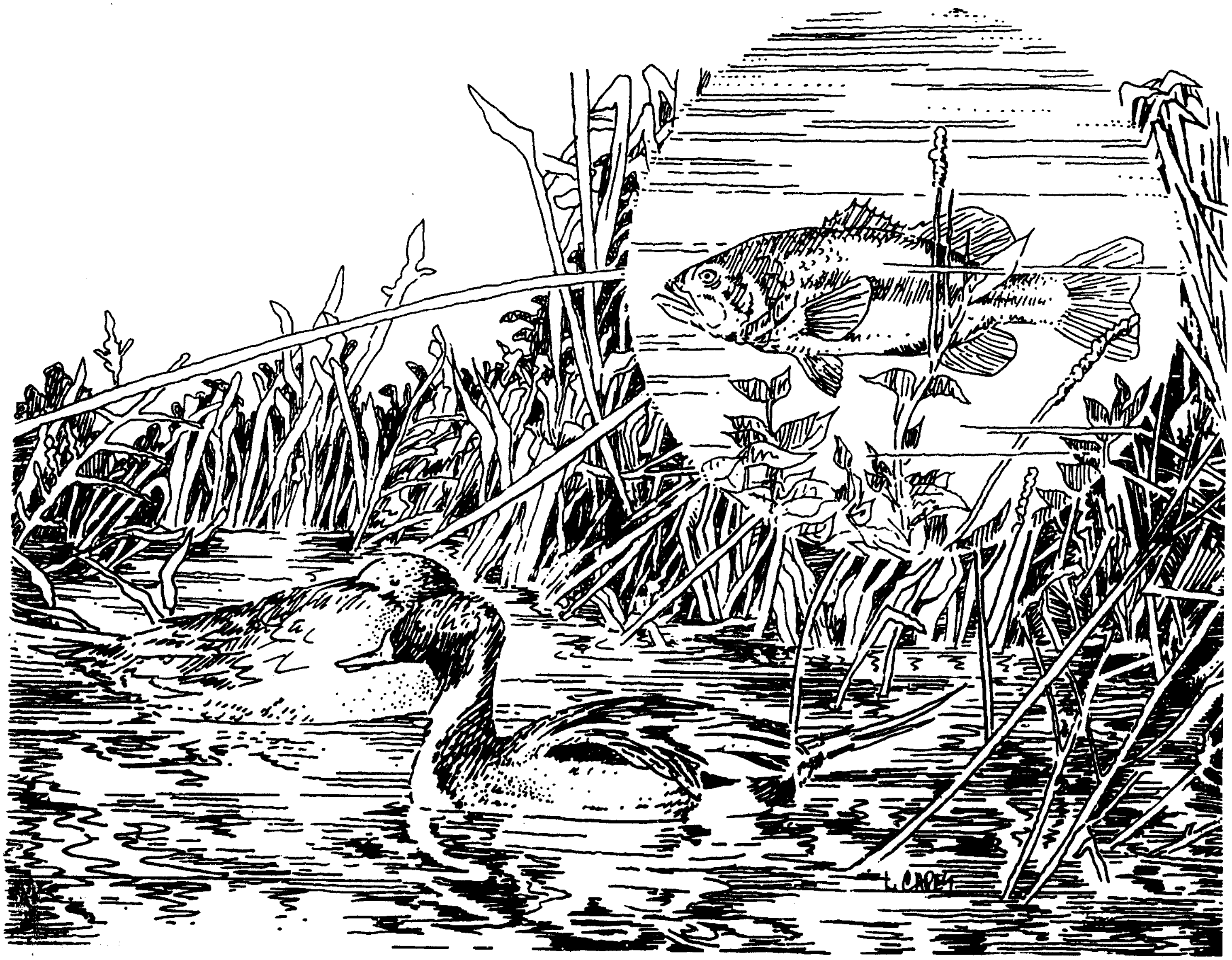
UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA

# REHABILITATION AND ENHANCEMENT

1996 FLOOD REPAIR



MARCH 1997

US Army Corps
of Engineers
Rock Island District

LA GRANGE POOL
ILLINOIS WATERWAY
MASON COUNTY , ILLINOIS

UPPER MISSISSIPPI RIVER SYSTEM
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# 1996 FLOOD REPAIR

MARCH 1997

LA GRANGE POOL
ILLINOIS WATERWAY
MASON COUNTY, ILLINOIS

# ACKNOWLEDGMENT

The following design team personnel are most familiar with the technical aspects of this study. Other US Army Corps of Engineers, Rock Island District and US Fish and Wildlife personnel also contributed to the project but are not listed.

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UPPER MISSISSIPPI RIVER SYSTEM
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LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR
LA GRANGE POOL, ILLINOIS WATERWAY, RIVER MILES 124-129
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2. Lake Chautauqua, Rehabilitation and Enhancement, Definite Project Report with Environmental Assessment, June 1991.

3. Lake Chautauqua, Environmental Management Program, Radial Gate Investigation, October 1996, (DRAFT).

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## 1. INTRODUCTION

a. Purpose. The purpose of this report is to present a detailed proposal for repairing 1996 flood damage consisting of the radial gate replacement and levee breach repair for the rehabilitation and enhancement of Lake Chautauqua's upper lake. This report analyzes various design options with a final recommendation and estimated cost. Final design and construction will proceed upon approval of this document.

b. Scope. This design memorandum is limited in scope to include only the radial gate replacement and the perimeter levee breach repair. This is located in the northeast corner of the refuge and the upper lake as shown on Plate 2. A future construction contract will be required to complete the perimeter levee, boat ramp, and cross dike berm. These items were deleted from the original contract.

c. Authority. The authority for this project is provided by the 1985 Supplemental Appropriations Act (Public Law 99-88) and Section 1103 of the Water Resources Development Act of 1986 (Public Law 99-662).

d. Project Location. Lake Chautauqua is a 4,200 acre waterfowl refuge located within the floodplain of the Illinois River north of Havana, Illinois, between river miles 124 and 130 as shown on Plate 1. Lake Chautauqua is formed by a 9 mile perimeter levee and a cross dike that divides the area into an upper lake and lower lake. The U.S. Fish and Wildlife Service (USFWS) operates the lakes for migratory waterfowl as part of the Chautauqua National Wildlife Refuge.

## 2. DESCRIPTION OF ORIGINAL PROJECT:

a. Goals and Objectives. The goal of the approved Lake Chautauqua project is to enhance waterfowl habitat with the objective of increasing submergent and emergent vegetation. A complete listing of proposed project goals, objectives, and enhancement potential for the original Lake Chautauqua Project can be found in the Lake Chautauqua, Rehabilitation and Enhancement, Definite Project Report, dated June 1991, Reference 2.

b. Original Project Description. A contract for the construction of the Lake Chautauqua Habitat Restoration and Enhancement Project (HREP) was awarded in

September 1992, Reference 1. Work items under contract include construction of a lower lake stop-log structure, lower lake drainage channels, pump station, cross dike and perimeter levee raise, access road, parking area, boat ramp, and modifications to an existing 60 year old gate in the upper lake.

c. Operating Details for the Upper Lake, Original Project. The upper lake has approximately 1000 acres of non-forested wetland (open water) and 130 acres of bottomland forest. The purpose of the original project is to raise the upper lake levees to elevation 449.1 feet (NGVD) which is approximately the 10 year flood event elevation. The levee protection and the new pump station will allow periodic dewatering of the upper lake, elimination of rough fish, and consolidation of bottom sediments. Following upper lake dewatering and elimination of rough fish, wetland vegetation shall begin to grow. Spring water will slowly fill the lake. The lake habitat with a high water quality, restocked beneficial fish species, and wetland vegetation will provide enhanced habitat for diving ducks and other waterfowl. The ten year event levee will provide sufficient protection to minimize sediment build-up and disruption to the habitat.

3. 1996 FLOOD INCIDENT AND DAMAGE DESCRIPTION.

a. During the course of the construction, the existing radial gate and adjacent levee section located at the northeast corner of the upper lake was damaged from flooding on June 1, 1996. The existing structure was displaced into an adjacent scour hole about 350 feet in diameter and 30 feet deep. As a result of the flood damage, the Corps of Engineers hired an architect-engineer firm to investigate the cause of failure and document the findings in a report, Reference 3. The report concluded that erosion may have occurred on the upstream face of the levee leading to levee failure and subsequent loss of the structure. The Corps of Engineers determined that the construction contractor's actions were not unreasonable prior to the flood event and therefore, the contractor was not held liable for the loss. Due to the loss of upper lake water control and changed site conditions, the Corps of Engineers terminated remaining work on the perimeter levee and other affected work which remained in the original contract.

b. The purpose of this design memorandum is to develop a cost effective solution to repair the levee breach and replace the damaged radial gates. Analysis of alternatives included both quantitative and qualitative methods. Sufficient design work was done on the alternative designs to adequately evaluate their strengths and weaknesses.

4. POST FLOOD PROJECT REQUIREMENT ANALYSIS. The US Army Corps of Engineers in coordination with USFWS and select members from the Environmental Management Program Coordinating Committee (EMPCC) have reviewed original project goals and objectives as well as design criteria in an effort to minimize the costs of completing this project and lessen the funding impact to other projects in the EMP program. The group reviewed a variety of alternatives presented in Section 6. Design criteria and considerations for the replacement structure and perimeter levee was reviewed and

coordinated with the USFWS. Primary functional criteria focused on the following five items:

a. Control of levee overtopping erosion damage. The initial DPR established a 3 day fill time starting at a river elevation of 446 feet and upper lake at elevation 435. In three days the river would be overtopping the levees at elevation 449 feet and the upper lake would have filled to elevation 448 feet. The head differential of 1 ft when the river overtops the levee will prevent significant erosion damage to the levees. In an effort to reduce costs, a higher overtop differential was considered. If the gates are not opened soon enough or if the river rises faster than 1 ft per day, a higher head differential when overtop occurs will increase flow velocities across the top of the levee. This will result in more potential for erosion and higher maintenance costs. However, a well maintained grass covered clay levee should sustain flow velocities with a 2 ft head differential, see Hydrology and Hydraulics Appendix.

b. Levee height, elevation 447.0 ft vs. 449.1 ft. In order to keep costs to a minimum, USFWS determined that protection to elevation 447.0 ft would give them sufficient protection from river overtopping and still meet their management objectives. When analyzed from a hydraulic perspective, new criteria were developed. Since the river historically rises at varying rates, the lower levee was analyzed from a lake fill time and overtop perspective. The average river rise was changed to 1.3 feet per day and the head differential of 1 foot when overtop occurs was studied. The 1 foot head differential may not be achieved under all circumstances. Velocity computations show that a well seeded levee should be able to withstand a 2 foot head differential. The ability to predict levee overtop was also studied. The lower levee requires new operating guidance to determine when to open the gates. This is included in the Hydrology and Hydraulics Appendix.

c. Allow fish passage out of the upper lake when dewatering the upper lake. A structure with an open top is preferable to a closed conduit. If the sill could be lowered to elevation 431.5 feet (Near lake bottom), more water and fish would escape the lake prior to pumping remaining water.

d. Incremental water level control. When the upper lake is managed for aquatic habitat, water levels will be maintained between elevation 434 and 436 feet during the spring and summer months. The level may be controlled to maximize the growth of emergent and submergent vegetation which is beneficial to migratory waterfowl. During the winter months, the water levels can be increased to provide better over-wintering habitat for fish species. Stop log water control best performs this function. If the lake is managed as moist soil unit habitat, water level control at lower levels is required. Gravity drainage and incremental control provided by stop logs is the most cost effective. If the river rises during operation of the upper lake making gravity drainage impossible, the pump station would be operated to provide the water control. The pump station should not be used to control the lake when gravity drainage is possible due to higher operating and maintenance costs.

e. USFWS requested that the structure allows passage of levee maintenance equipment which requires H 20 loading and 14 ft width. This criteria was used when designing and building the lower stop log structure and is consistent with the current project.

5. RECOMMENDED PLAN.

a. Gated Sheet Pile Cell Structure with 5 year Level of Protection. This option consists of 4 main sheet pile cells about 74 ft in diameter with a top elevation of 452.0 ft with 3 connecting intermediate cells. The 3 intermediate cells would have H-pile supported concrete caps at elevation 430.5 ft to support three 10 ft by 10 ft heavy duty sluice gates. This structure would be placed around the upstream side of the scour hole and effectively close the breach. The structure would be tied into the levee with approximately 50 ft of sheet pile cutoff.

b. Levee work. The levee would not be completed to the original design under this option. Cost savings result in elimination of most embankment construction on the perimeter levee. This option does require an additional 30,000 CY of embankment. Shaping the levee would require about 10,000 CY of cut material. The perimeter levee would be constructed to elevation 447.6 ft with a 3000 ft section at elevation 447.0 ft. The lower 3000 ft section will control the area of initial overtopping. To initiate early turf establishment, a cover crop will be planted on all levees. The refuge could later over-seed the levees if desired. Following flood events, reseeding the levees may be required. The levee connecting the structure to the parking area at the town of Goofy Ridge would be built to elevation 450.0 ft to provide more secure access to the structure for operation during a flood. This access route would be surfaced with granular material.

c. Operating Plan. A new gate operation plan was developed to predict when to open the new flood gates. This is required since the levee will only be built to elevation 447.0 ft and results in different river characteristics than the original project. The average rate of river rise for a levee crest of 447.0 ft is 1.3 ft per day. The DPR operational guidance was based on river rise rates of 1.0 ft per day for a levee crest of 449.1 ft. These rates are different because the river rises faster at lower elevations; the higher the rate of river rise, the more difficult it is to make an accurate prediction. The new operational plan to predict when to open the gates is located in the Hydrology and Hydraulics Appendix, Appendix A.

d. Sheet pile. The US Army Corps of Engineers, St. Louis District has used sheet pile from temporary cofferdam construction at Lock and Dam 26. Through inter-district coordination and cooperation during an EMP workshop, the availability of this sheet pile was discussed. St. Louis District agreed to provide the required amount of sheet piling. This will result in a $1,000,000 cost savings for the Lake Chautauqua project.

e. Site preparation. The existing displaced radial gate poses a safety hazard in its current condition. Following tie-in of the levee and new structure, the scour hole can be partially dewaterd to expose the displaced gate. The radial gates could be lowered or removed, the operators removed, and metal objects such as bolt stubs and fencing cut off flush with the top of concrete. Other site preparation includes removing some portions of concrete retaining wall, wood posts, and other displaced concrete. This debris can be placed in the scour hole to provide fish habitat and assist in water energy dissipation when equalizing the upper lake during a flood event. Each alternative requires this work.

6. ALTERNATIVES CONSIDERED. See summary, Table 6-1.

a. Alternative 1, Concrete Tainter Gate Structure with 10 year Level of Protection. This structure would replace the existing radial gate structure in kind but with current design standards. It consists of a concrete structure with two 9 ft wide by 18 ft high Tainter gates. The stilling basin is concrete and a standard design from Corps of Engineer design manual, EM 1110-2-1602. This alternative would require repair of the levee breach with a sand base below the water table and a clay levee. The perimeter levee would be constructed per the original contract. This alternative was rejected due to its high construction cost. The cost of the structure is estimated at $2,600,000. The cost to repair the breach is $690,000. The cost to raise the perimeter levee is $850,000.

b. Alternative 2, Gated Sheet Pile Cell Structure with 10 year Level of Protection. This option consists of 4 main sheet pile cells about 74 ft in diameter with a top elevation of 452.0 ft with 3 connecting intermediate cells. The 3 intermediate cells would have H-pile supported concrete caps at elevation 430.5 ft to support three 10 ft wide by 9 ft high heavy duty sluice gates. St. Louis District has used sheet pile from the construction of Lock and Dam 26 which is available to other government agencies. This sheet pile was used for temporary cofferdams. This option would close the breach, eliminate dewatering costs, and reduce risks due to flood damage during construction. This structure would be placed around the upstream side of the scour hole and effectively close the breach. The perimeter levee would be constructed per the original contract. This alternative was rejected due to the cost to raise the perimeter levee. The cost to raise the perimeter levee would be $600,000 more than the recommended option. The structure cost is about the same as the recommended option.

c. Alternative 3, Gated Sheet Pile Cell Structure with 5 year Level of Protection. This option consists of 4 main sheet pile cells about 74 ft in diameter with a top elevation of 452.0 ft with 3 connecting intermediate cells. The 3 intermediate cells would have H-pile supported concrete caps at elevation 430.5 ft to support three 10 ft by 10 ft heavy duty sluice gates. This structure would be placed around the upstream side of the scour hole and effectively close the breach. The perimeter levee would be constructed to elevation 447.6 ft with a 3000 ft section at elevation 447.0 ft. The estimated cost to complete the perimeter levee is $250,000. The estimated structure cost is $2,500,000. This is the recommended option.

d. Alternative 4, Uncontrolled Spillway with 2 year Level of Protection. This option consists of 5 main sheet pile cells and 4 connecting intermediate cells. The 2 outer cells would taper down to elevation 442.5 ft which would provide a 300 ft wide overflow structure. A stop log structure would be constructed between 2 main cells in order to allow lake dewatering and fish passage. The perimeter levee would be shaped and graded to provide the highest level of protection possible with existing levee material. This option was rejected due to the low level of protection and the lack of operating flexibility to provide lacustrine (aquatic) habitat. The lake could only be operated as a moist soil unit due to the low level of protection. The lower level of protection would result in increased sediment build-up. Future flexibility to raise the levee and the structure would be costly. This option was about $300,000 less than the recommended option.

e. Alternative 5, Abandon Project. This alternative would require cleaning up debris at the scour hole and reducing the safety hazards resulting from the relocated structure and steel appurtenances. This would require removal or lowering of the gates and cutting off fencing and sharp objects from the bridge deck. Removing the concrete structure is not economically feasible. The rest of the project would remain in its current state. The perimeter levee would not be finished, graded, or seeded. The cost of this option is roughly estimated at $100,000. This option was not selected due to a loss of project outputs and benefits.

f. Other alternatives. The design team explored the following alternatives in an effort to reduce costs: Overflow structure, fuse plug structure, box culverts or u-channels. These items are briefly discussed below.

(1). Modified Tainter Gate Structure. This structure is similar to alternative 1 in gate size and operation. The inlet wingwalls are constructed from Z-piling with tie backs. This would reduce dewatering costs and eliminate the requirement for H-pile foundation. The Z-piling would not last as long as the concrete; however, the piling should out live the 50 year design life of the project. Z piling could also be used to form the exterior surface of the structure. The "Z-pile box" provides structure protection and reduces dewatering costs. Another area to save costs would be to try and eliminate the costs of the stilling basin. A concept which would allow the lake bed to scour away from the levee creating a natural and inexpensive energy dissipater was pursued. A single sheet pile cutoff wall at the end of a concrete apron was not acceptable unless scour depth was limited to a design depth so that the sheet pile does not fail. Predicting scour characteristics is not an exact science and this method would require close monitoring, inspection, and maintenance. To overcome these deficiencies and take advantage of used sheet pile from St. Louis District, a cell could be used as an energy dissipation apron. The use of a sheet pile cell such as those used in cofferdam construction is self supporting and can withstand scour to a greater depth. Riprap is not required to prevent scour from occurring in the lake bed. This option greatly reduces the dewatering costs since a sheet pile or earthen cofferdam is no longer necessary. Approximate cost savings for this option is about $400,000 over alternative 1 but still higher than the recommended alternative.

(2). Overflow structure. An option to both close the breach and construct an overflow section was considered. The scour hole would be filled with sand, a sheet pile cut-off wall to prevent underseepage would be constructed and rock would be used to stabilize the sheet pile and prevent lake side scour. This design was not completed due to its high cost. The amount and size of riprap on the lake side would have been extensive in order to limit erosion during an overtopping event. In order to provide sufficient inflow capacity to limit overtopping erosion, the over-all level of protection would have to be lowered or the rest of the perimeter levee would have to be raised. Either project objectives would be compromised or the cost would be excessive.

(3). Fuse plug structure. This option utilizes an overflow section as a base for a sand "fuse plug". When the fuse plug is breached, water forces will scour away the sand and the result will be a designed overflow section. The use of this structure would allow protection to the full levee design height. The objectives of fish passage while dewatering would require a separate structure. A stop log structure could provide this service which is less expensive than the Tainter gate structure since the size of the structure would be smaller and the stop logs are less expensive than the Tainter gates. The base of the fuse would be constructed out of rock filled gabion mattresses, soil cement, or a surface such as fabri-form concrete. The length would depend on the fill time and the sill depth. A deeper sill would require a more elaborate energy dissipater. These dimensions are discussed in the Hydrology and Hydraulics Appendix. A disadvantage to the fuse plug is the maintenance costs to replace it whenever the levee is overtopped. Other disadvantages to the fuse plug are its vulnerability to sabotage and wave wash erosion. If a member of the public initiates the fuse plug when it is not required or waves erode the sand, the levee would overtop. The entire upper lake project would be in jeopardy. The costs of building this structure are similar to the overflow section and also have high maintenance requirements due to frequent replacement of the "fuse". Potential initial cost savings for this option are outweighed by the disadvantages and high maintenance costs.

(4). Box culverts or U-channels. A series of 3 box culverts or U-channels could be used in place of the radial gate structure. The gate size would be similar in size to the sluice gates used in Alternative 2 or 3, (9 ft high by 10 ft wide with a levee at 449.1 and 10 ft high by 10 ft wide with a levee at 447.0). The sluice gates are as expensive as the 9 ft by 18 ft Tainter gates even though they are smaller. Large concrete box culverts or large U-channel structures would have to be constructed through the levee. Concrete and riprap protection would be required at the outlet for energy dissipation. The fish passage and upper lake water control would require at least 1 U-channel. These structures would not require the pile foundation which the gate requires. This option was not selected due to its cost. The cost was roughly approximated at $1,900,000 with an estimated 1000 CY of reinforced concrete and $120,000 gate cost. An estimated $690,000 would be required to repair the levee breach.

TABLE 6-1, SUMMARY OF ALTERNATIVES AND CONSTRUCTION COSTS

| ALTERNATIVE | DISCUSSION | EST. CONST. COST |
|---|---|---|
| Alt. 1, Concrete Tainter Gate Structure with 10 year Level of Protection | Provides all required functions<br>Separate repair of levee breach<br>Restores levee to DPR level | $4,150,000 |
| Alt. 2, Gated Sheet Pile Cell Structure with 10 year Level of Protection | Provides all required functions at lower cost utilizing innovative design<br>Sheet pile cells close levee breach<br>Lower risk of flood damage during construction<br>Added savings of $1,000,000 by reusing sheet pile from St. Louis District<br>Restores levee to DPR level | $3,550,000 |
| **Alt. 3, Gated Sheet Pile Cell Structure with 5 year Level of Protection** | **Same structure as alternative 2**<br>**Lower level of flood protection**<br>**Upper lake can be operated as primarily aquatic or moist soil habitat** | **$2,750,000** |
| Alt. 4, Uncontrolled Spillway with 2 year Level of Protection | Upper lake could only be operated as moist soil habitat<br>More frequent river inundation<br>Objectives vary from DPR | $2,450,000 |
| Alt. 5, Abandon Project | Minimum work to eliminate safety hazards<br>Loss of project outputs and benefits | $100,000 |
| Modified Tainter Gate Structure | Innovative design and higher risk<br>Less cost than alternative 1 | $3,750,000 |
| Overflow structure at breach location | Fill Scour hole, Sheet pile cut off, Rock stabilization<br>High cost | >$3,000,000 |
| Fuse plug structure | Separate repair of levee breach<br>High maintenance costs<br>Frequent overtop makes less practical<br>Innovative and high risk | >$3,000,000 |
| Box culverts or U-channels | Separate repair of levee breach<br>Requires 3 ea 10 ft by 10 ft box culverts and gates<br>Cost and complexity is similar to alternative 1. | >$3,000,000 |

7. ENVIRONMENTAL EFFECTS

**a. Preferred Alternative 3, Gated Sheet Pile Cell Structure With 5 year Level of Protection.**

(1). This alternative most closely resembles the original project as described in the Definite Project Report (DPR). The main difference is construction of a lower elevation perimeter levee and replacement of the gate structure lost in the 1996 flood. The project goal of enhancing migratory waterfowl habitat by creating an open water habitat suitable for submergent, floating, and emergent vegetation growth remains the same. Secondary benefits to fish and other wildlife would also be provided.

(2). The lake will be dewatered during the first summer after completion of construction to compact the lake bottom, reduce turbidity, and increase the success of establishing submergent vegetation. Water levels of 2-6 feet will be maintained during the growing season and allow aquatic vegetation to become established. Fall and winter water levels will be raised to 4-8 foot depths in order to provide fish overwintering areas. The project also allows flexibility to manage for moist soil vegetation when it is beneficial to do so. Management is described further in the attached U.S. Fish and Wildlife Service, Draft Illinois River National Wildlife and Fish Refuges Management Plan and operational management plan (See Appendix E).

(3). The local breeding wood duck population will benefit from increased availability of brood habitat. Fish will benefit from the increased habitat diversity provided by open water habitat that includes submergent vegetation. The area will have conditions favorable for fish spawning, nursery, and overwintering. The gate structure will allow fish passage when the lake is raised and lowered. This benefits not only the local population but the fishery of the Illinois River. Many other non-target species will benefit from increased habitat diversity. This will include species such as rails, heron, raptors, song birds, small mammals, reptiles and amphibians.

**b. Alternative 1, Concrete Tainter Gate Structure With 10 year Level of Protection; and Alternative 2, Gated Sheet Pile Cell Structure With 10 year Level of Protection**

Alternatives 1 and 2 are both similar to the selected alternative described in the project DPR, June 1991 with the exception of different gate structures. Project impacts and outputs would be the same as those described in the original project. Open water habitat in the upper lake would include emergent and submergent vegetation and water levels during fall and winter would be raised to enhance migratory waterfowl habitat and fish overwintering.

**c. Alternative 4, Uncontrolled Spillway With 2 year Level of Protection**

(1). This alternative would provide migratory waterfowl habitat through creation of moist soil habitat. Moist soil management includes drawing down of water during the growing season to allow vegetation growth on mud flats. During fall and winter shallow water depths are maintained to increase waterfowl foraging suitability. Moist soil habitat is generally known to provide food preferred by dabbling ducks. Diving ducks are known to occasionally use moist soil habitat but it is not their preferred habitat.

(2). The 442 ft. levee may enhance waterfowl habitat by creating moist soil habitat but does not meet the DPR objective of increased submergent aquatic vegetation, nor would it meet Refuge management objectives. Moist soil habitat does not meet the project goals or objectives of providing suitable habitat for diving ducks. Fishery benefits achieved through this alternative would be restricted to potential spawning before water is drawn down for the growing season. With the lower levee, Upper Lake Chautauqua would continue to be frequently flooded and have sediment introduced with each flood. Moist soil habitat is abundant in the Illinois River Valley on State and Federal wildlife refuges and private land. The adjacent lower Lake Chautauqua includes approximately 2200 acres of moist soil habitat. The 442.5 ft. levee would not provide the management flexibility of Alternative 3 nor would it provide increased habitat diversity.

**d. Alternative 5, Abandon Project**

Alternative 5 would involve the clean up of existing debris and abandoning the project as is. It would not provide the habitat benefits as originally planned and would in fact, leave the area less suitable for migratory waterfowl and Refuge operation than when construction of the original project commenced.

**e. Compliance with Environmental Quality Statutes**

(1). Project impacts were originally evaluated and compliance with the various environmental statutes was documented in Upper Mississippi River System Environmental Management Program Definite Project Report with Integrated Environmental Assessment: Lake Chautauqua Habitat Rehabilitation and Enhancement, La Grange Pool, Illinois Waterway, River Miles 124-128 Mason County, Illinois, June 1991.

(2). The environmental impacts evaluated in the previous report considered the full range of potential impacts associated with the proposed project. It concluded that the project would provide a net gain in wildlife habitat and adverse impacts would be minimal. Environmental impacts by repair of the gate structure and levee raise as planned through the preferred alternative will not exceed those previously evaluated in the DPR and environmental assessment. The goals, objectives, and project benefits are all similar to the previous project. Therefore, the project remains in compliance with all applicable environmental statutes. It was coordinated with the appropriate Federal, State and local agencies and there were no objections to the project.

**f. Historic Properties**. As promulgated under Section 106 of the National Historic Preservation Act of 1966 (NHPA), as amended, and its implementing regulations 36 CFR Part 800: "Protection of Historic Properties," Federal agencies are required to determined the effects of their undertakings on all historic properties listed on, or eligible for, the National Register of Historic Places.

(1). The Corps and the U.S. Fish and Wildlife Service determined that no significant historic properties will be affected by the preferred design alternative repairs in response to the 1996 flood damage within the Lake Chautauqua Habitat Rehabilitation and Enhancement Project, a part of the Upper Mississippi River System - Environmental Management Program (UMRS-EMP). This determination was coordinated with the State Historic Preservation Officer (SHPO), Illinois Historic Preservation Agency, Springfield, Illinois. By letter dated February 19, 1997, the SHPO concurred with the Corps determination that no significant historic, architectural, archeological resources are located within the proposed project area (IHPA LOG# 970218007PMN).

(2). Although the Corps and the U.S. Fish and Wildlife Service are assured that no significant historic properties will be affected by the proposed Slide Gate and Steel Sheet Pile Cellular Structure with a 447.0 Levee, if any undocumented historic properties are identified or encountered during the undertaking, the Corps and its contractors will discontinue all construction activities and resume coordination with the Illinois Historic Preservation Agency and identify the significance of the historic property and any potential effects under Section 106 of the National Historic Preservation Act of 1966, and its implementing regulations, 36 CFR Part 800: "Protection of Historic Properties."

**g. Agency Comments**

U.S. Fish and Wildlife Service, Ecological Services, Rock Island Field Office- The Service agreed that the preferred alternative most closely resembles the original project and that it would provide benefits to a large range of species. They also commented that the lower levee elevation would increase the amount of rough fish in the lake and thus make it more difficult to manage for submergent vegetation. However, they believe this will be offset by the management flexibility provided through the Upper Lake Chautauqua Management Plan. The Service also stated that they do not see the need to reevaluate the habitat analyses and feel the changes in habitat suitability will be too subtle to be measured using Wildlife Habitat Appraisal Guide (WHAG).

Corps Response
In order to evaluate project benefits with the present baseline condition the WHAG will be conducted. The analyses will be performed to evaluate the habitat outputs of alternatives 3 and 4 as compared to the original project and to facilitate establishment of a baseline for project evaluation and monitoring. Biologists believe alternative 3 will produce similar results to the original project and accomplish the original goals and objectives of the original plan. Alternative 4, however, will not accomplish the intended objective of creating diving duck habitat.

Federal Emergency Management Agency, Region V, Chicago. -FEMA raised concerns regarding failure of the Draft Design Memorandum to state project compliance with

Executive Order 11988, Floodplain Management, and obtain local and State of Illinois floodplain development permits.

This concern was discussed with the FEMA office. The previous project complied with E.O. 11988 and State of Illinois floodplain development permit was obtained. Repair of the project with a lower levee does not constitute need for new documentation. FEMA concurred that the project is in compliance but would like to see that expressed with more detail in future documents.

Illinois Historic Preservation Agency, Springfield, Illinois. By letter dated February 19, 1997, the SHPO concurred with the Corps determination that no significant historic, architectural, archeological resources are located within the proposed project area (IHPA LOG# 970218007PMN).

U.S. EPA, Chicago.
By telephone EPA stated that they do not have concerns with or objection to the project.

Illinois EPA.
Illinois EPA was going to review the document and follow up with a telephone call but never did.

Illinois DNR Office of Water Resources
The Office of Water Resources will require a dam safety permit since the upper lake structure is classified as an intermediate-size class III dam. That permit is in the process of preparation. They also stated that they require no further activity for floodway construction permits. The previous permit is still valid (IDNR/OWR Permit 20937).

See Appendix E, Correspondence for the letters described above, the proposed Upper Lake Management Plan, and the USFWS Water Management Strategy for this reach of the river. The Finding of No Significant Impact follows this section.

FINDING OF NO SIGNIFICANT IMPACT
FOR REPAIRS TO LAKE CHAUTAUQUA HABITAT RESTORATION AND ENHANCEMENT PROJECT

In accordance with the National Environmental Policy Act, the Rock Island District, U.S. Army Corps of Engineers, has evaluated the environmental impacts of the above project. This project constitutes repair of a habitat restoration project under construction. It does not differ significantly from the original project for which an Environmental Assessment (EA)was prepared. Therefore, preparation of an Environmental Assessment is not required. Having reviewed the information contained in this Design Memorandum with incorporated environmental documentation, I find that the proposed project will have no significant adverse impacts on the environment..

This finding of no significant impact is based on the following factors:

a. The proposed construction is to replace a structure damaged during flooding at a project under construction and the project goals and objectives have remained the same. Environmental impacts from this project will be no more than those assessed in the original EA.

b. Implementation of the project will benefit nationally significant waterfowl and wetland resources.

c. The proposed action is complementary to the Lake Chautauqua National Wildlife Refuge goals and objectives.

d. There were no adverse comments received during public review.

e. Adverse effects to fish and wildlife from construction are temporary.

7 Mar 97
Date

Charles S. Cox
Charles S. Cox
Colonel, U.S. Army District Engineer

## 8. REAL ESTATE

No additional real estate is required to implement the recommended alternative in this report. The property line near the new structure is being investigated so that work limits can be established and laid out in the field during construction. USFWS is required to update the special use permit for the project.

## 9. COST ESTIMATE

a. General. This section contains the detailed cost estimate which was prepared for the Lake Chautauqua 1996 Flood Repair, Memorandum. It includes construction, planning, engineering and design, and construction management costs. The current working estimate (CWE) prepared for this Design Report (DR) was developed after review of project plans, discussions, with design team members, and review of costs for similar construction projects. Unit Price Book Items with associated Labor Costs and Equipment Cost (EP) were utilized to assemble and calculate project element costs. The Micro Computer Aided Cost Estimating System (MCACES) was utilized to assemble and calculate project element costs. These costs and appropriate contingencies, are presented in accordance with EC1110-2-538, Civil Works Project Cost Estimating - Code of Accounts.

b. Price Level. Project element costs are based on January 1997 prices. These costs are considered fair and reasonable to a well equipped and capable contractor and include overhead and profit. Calculation of the Fully Funded Estimate (FFE) was done in accordance with guidance from EC11-2-169 dated 31 March 1996. Appendix D, Table 1 shows the Project Cost Summary.

c. Presentation of Estimated Costs. The 1996 Flood Repair project consists of a series of sheet pile cells (4 each) with heavy duty sluice gates in the intermediate cells (3 each) with associated pile foundation, bridges, reinforced concrete caps, riprap and earthwork immediately adjacent to the repair area.

d. Contingency Discussion. After review of project documents and discussion with personnel involved in the project, cost contingencies were assigned which reflect the uncertainty associated with each cost item. Per EC1110-2-263, these contingencies are based on qualified cost engineering judgment of the available design data, type of work involved, and uncertainties associated with the work and schedule. Costs were not added to contingency amounts to cover items which are not identified project requirements. The following discussion of major project features indicates the basis for contingency selection and assumptions made. For other elements not addressed below, the assignment of contingencies was deemed appropriate to account for the uncertainty in design and quantity calculation and further discussion is not included.

e. Feature Discussion.

(1). Feature 06, Wildlife Facilities and Structures (Sheet Pile Cell Structure). This feature consists of 4 sheet pile main cells, 74 ft diameter each with a top elevation of 452.0

ft. Three 10 ft by 10 ft heavy duty sluice gates are located between the main cells on the arc cells with a sill elevation of 430.5 ft. Each of the gates is equipped with manually operated hoisting equipment, stoplogs for one gate with manually operated chain hoist, precast concrete bridges and associated foundations (H Piling), concrete caps on the "intermediate cells", riprap and earthwork associated with the structure. The cell structures are self supporting. The concrete caps, slide gates and bridges rests on H piling and adjacent sheet piling. Costs for the gates and precast portions of the bridges are based on market survey information.

- Although borings in the general location are available the actual foundation material may result in minor changes in the pile foundation design.

- Sluice gates are heavy duty with associated appurtenances with epoxy paint system, adjustable wedges and shims, rubber seals, stainless steel rub plates, adjustable bottom sill (stainless steel) and rubber seal, with associated lifting equipment. Each gate lift is independent and manually operated.

- Concrete is 4000 psi with #8, grade 60 deformed steel bars at 12" centers each way each face on all surfaces.

- Dewatering during the construction and installation of the slide gates and concrete associated with the gates is accomplished by installing the sheet piling of the intermediate cells to elevation 445.0 an when the gate and appurtenances are operational the sheet piling of the intermediate cells is cut off at elevation of 428.5 and removed.

- A concrete cap on the intermediate cells on the river and landside of the levee serves as the inlet and outlet structure. This concrete cap rests on H piling and sheet piling.

- Riprap and bedding is adjacent to the cell structures as a transition to the earthen levee sections.

- In general the design is considered complete and not expected to require any significant future modification. The overall design contingency is established as 19.06 %.

(2). Feature 30, Planning, Engineering and Design. The engineering and design for this project includes all planning and design work necessary to complete the Design Memorandum and prepare construction plans and specifications. This cost also includes engineering support during construction. The design effort for the construction was analyzed to determine the man-year effort required. This estimate is based on moneys expanded to date, discussions between the technical manager and project manager, and historical data and experience gained on other projects of similar nature. Contingencies are not applied to this feature.

(3). Feature 31, Construction Management. Construction management includes the following items: review of project reports, plans and specification, and conferences of construction staff to become familiar with design requirements; bidability, constructability, and operability reviews; pre-award activities to acquaint prospective bidders with the nature of work; administration of construction contracts; administration of A/E contacts which provide for supervision and inspection; establishment of benchmarks and baselines required for layouts of construction, reallocations, and clearing; review of shop drawings, manuals, catalog cuts, and other information submitted by the construction contractor; assure specifications compliance by supervision and inspection on construction work, conferences with the contractors to coordinate various features of the project and enforce compliance with schedules; sampling and testing during the construction phase to determine suitability and compliance with plans and specifications; negotiation with the contractor on all contract modifications, including preparation of all contract documents required therefore; estimate quantities, determine periodic payments to contractors, and prepare, review and approve contract payments; review and approve construction schedules and progress charts; prepare progress and completion reports; project management and administration not otherwise identified; and district overhead. These costs may be incurred at the job site, and area office, or at the District Office. Contingencies are not applied to this feature. For the construction of the Lake Chautauqua 1996 Flood Repair, the estimated cost of construction management is $225,000 for a construction contract with a 1.5 year duration and an estimated value of $2.50 million.

f. Detailed Estimate Both a hard copy and an electronic copy of the detailed MCACES estimate are available for review. To reduce reproduction requirements, a copy of the detailed MCACES estimate is not included in this design report or appendix. Copies of these documents will be provided upon request.

## 10. SCHEDULE FOR CONSTRUCTION

a. The following construction schedule was formulated to determine project completion.

| **Estimated Dates** | **Activity** |
|---|---|
| Mar 97 | Complete plans and specifications |
| Jun 97 | Contract award |
| Jul 97 | Begin construction |
| Dec 97 | Complete cellular construction and slide gates. Complete perimeter levee construction. |
| Nov 98 | Complete x-dike, boat ramp, etc. (Terminated features from original contract) |

b. The above schedule allows a 6 month time frame to construct the sheet pile cells and fill them with granular material. Only 1 pile driver was used since site access is restrictive from the perimeter levee side and vulnerable to flooding. The concrete caps, walls, and gates would be started as soon as the adjacent cells and intermediate cells were constructed. The last 2 gate structures would likely require cold weather concrete placement methods. This would add to the cost but allow earlier completion of the project and protect the levee from potential spring flooding.

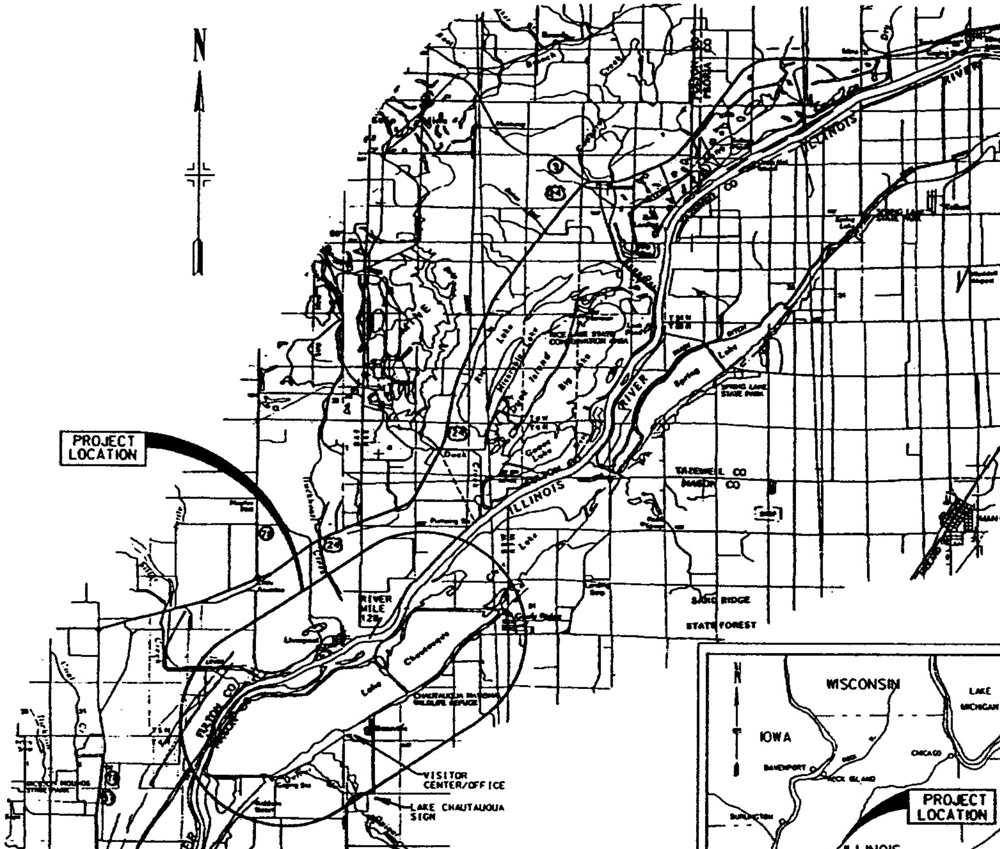
c. While the cellular structure is under construction, perimeter levee work would be ongoing. This assumes that water levels were below elevation 432.0 ft. Following completion of the structure and its tie into the levee, the system is well protected. At this time the upper lake would be dewatered with the pump station. Additional localized dewatering would be required while constructing the cross dike levee berm and boat ramp. The scour hole could be pumped down at this time to lower the existing gates and cut off sharp and protruding appurtenances from the gate so that it does not pose a safety hazard.

## 11. RECOMMENDATIONS

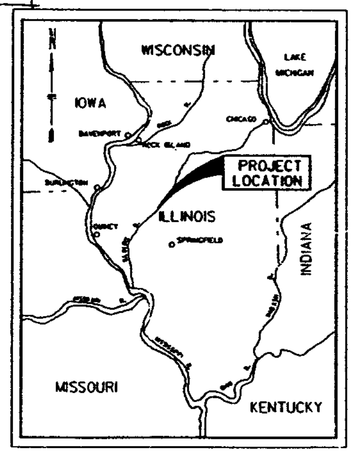
I recommend construction of alternative 3 as defined in this report for the repair of 1996 flood damage at Lake Chautauqua.

ORIGINAL SIGNED BY

Charles S. Cox
Colonel, U.S. Army
District Engineer




LOCATION PLAN




VICINITY MAP

US Army Corps of Engineers Rock Island District

| Symbol | Description | Date | Approved |
|---|---|---|---|

Revisions

| | | |
|---|---|---|
| Designed by: DRC | Date: XX XXX XX | |
| Drawn by: CAB - RLC | Scale: AS SHOWN | |
| Checked by: TGH | Drawing Code: XX-XXX | |
| Reviewed by: JJC | Solicitation Number: DACW25-97-B-XXXX | |

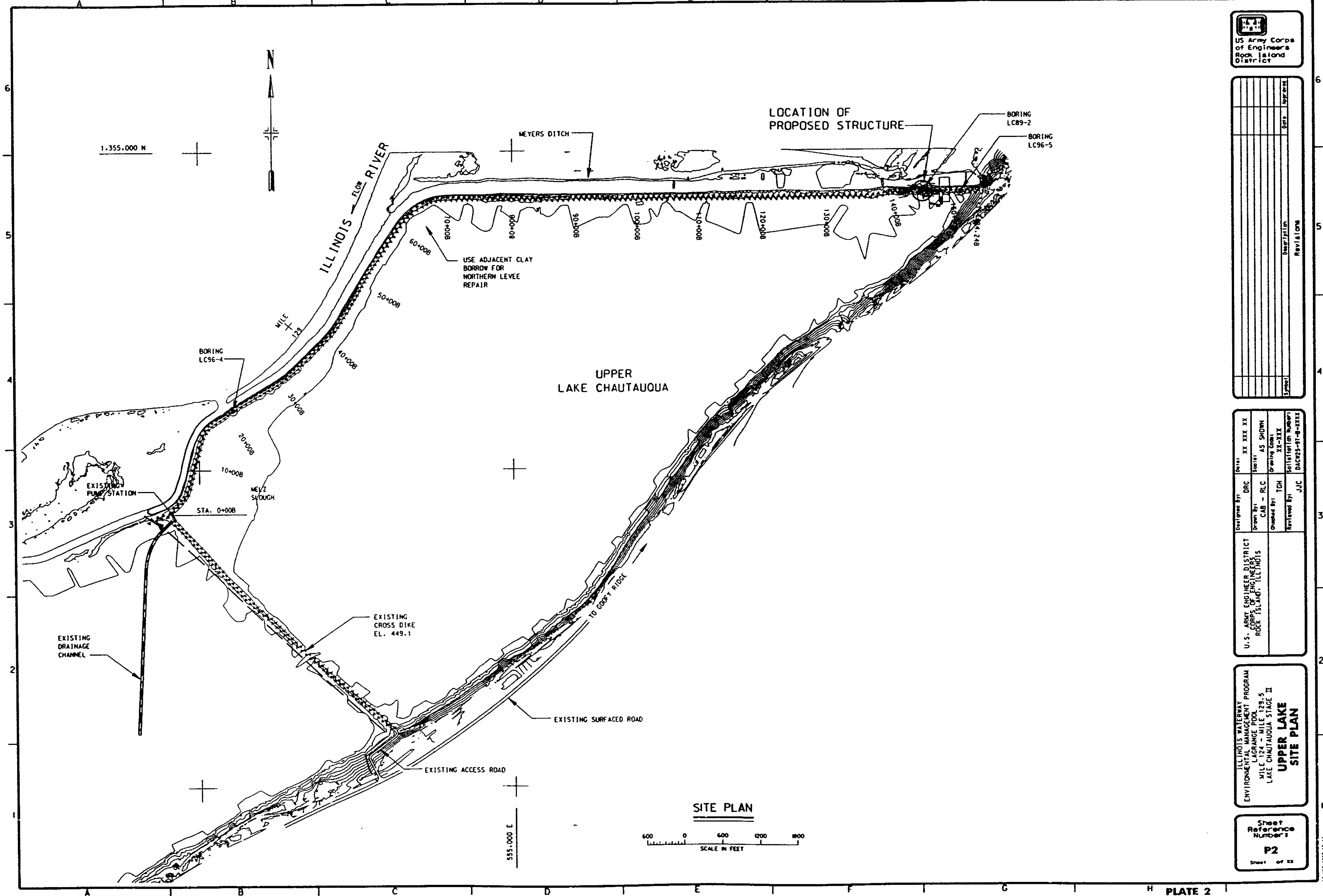
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ROCK ISLAND, ILLINOIS

ILLINOIS WATERWAY
ENVIRONMENTAL MANAGEMENT PROGRAM
LAGRANGE POOL
MILE 124 - MILE 129.5
LAKE CHAUTAUQUA STAGE II

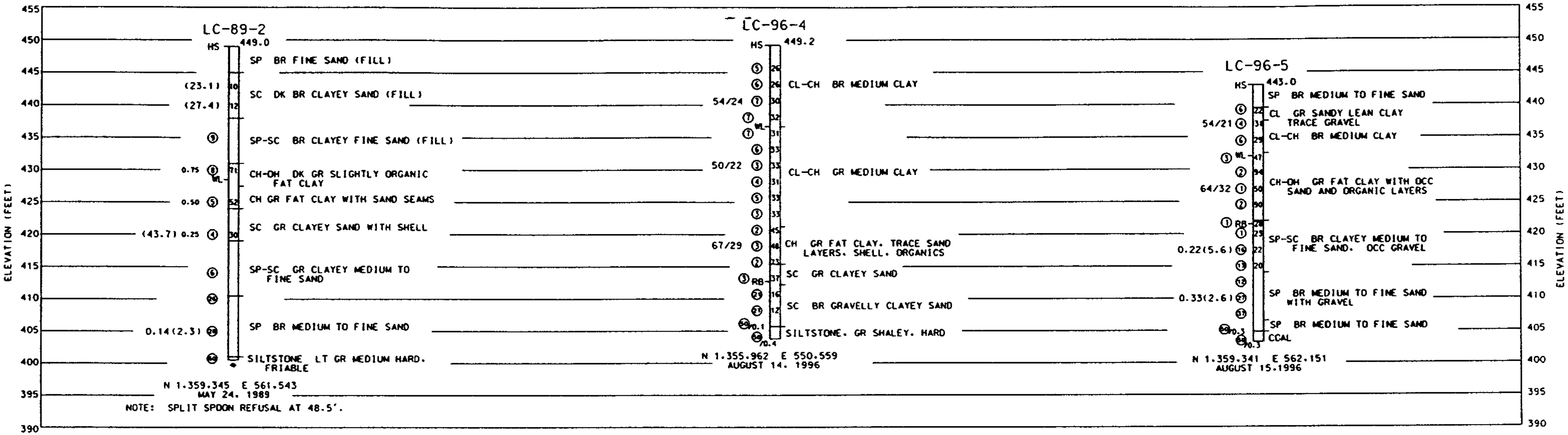
# LOCATION PLAN

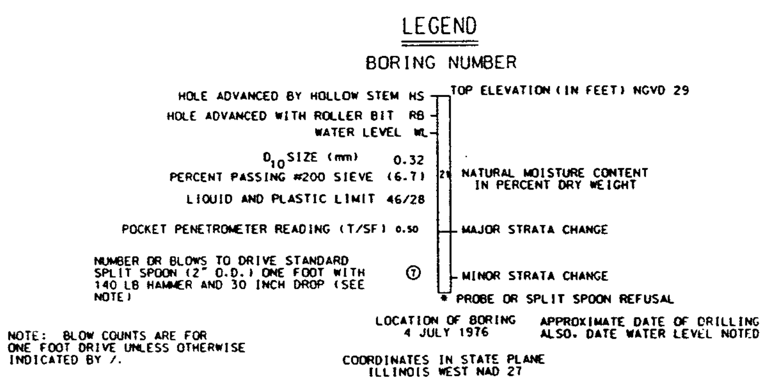
Sheet Reference Number:
**P1**
Sheet of XX

PLATE 1



US Army Corps of Engineers Rock Island District
LOCATION OF PROPOSED STRUCTURE
BORING LC89-2
BORING LC96-5
MEYERS DITCH
1.355.000 N
N
ILLINOIS RIVER
FLOW
USE ADJACENT CLAY BORROW FOR NORTHERN LEVEE REPAIR
MILE 128
BORING LC96-4
UPPER LAKE CHAUTAUQUA
EXISTING PUMP STATION
STA. 0+00B
MELZ SLOUGH
EXISTING DRAINAGE CHANNEL
EXISTING CROSS DIKE EL. 449.1
TO GOOFY RIDGE
EXISTING SURFACED ROAD
EXISTING ACCESS ROAD
555.000 E
SITE PLAN
SCALE IN FEET
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS ROCK ISLAND, ILLINOIS
Designed By: DRC
Drawn By: CAB - RLC
Checked By: TGH
Reviewed By: JJC
Date: XX XXX XX
Scale: AS SHOWN
Drawing Code: XX-XXX
Solicitation Number: DACW25-97-B-XXXX
ILLINOIS WATERWAY ENVIRONMENTAL MANAGEMENT PROGRAM LAGRANGE POOL MILE 124 - MILE 129.5 LAKE CHAUTAUQUA STAGE II
UPPER LAKE SITE PLAN
Sheet Reference Number: P2
PLATE 2




US Army Corps of Engineers Rock Island District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ROCK ISLAND, ILLINOIS

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| Reviewed By: GJM | Solicitation Number: DACW25-97-B-XXXX |

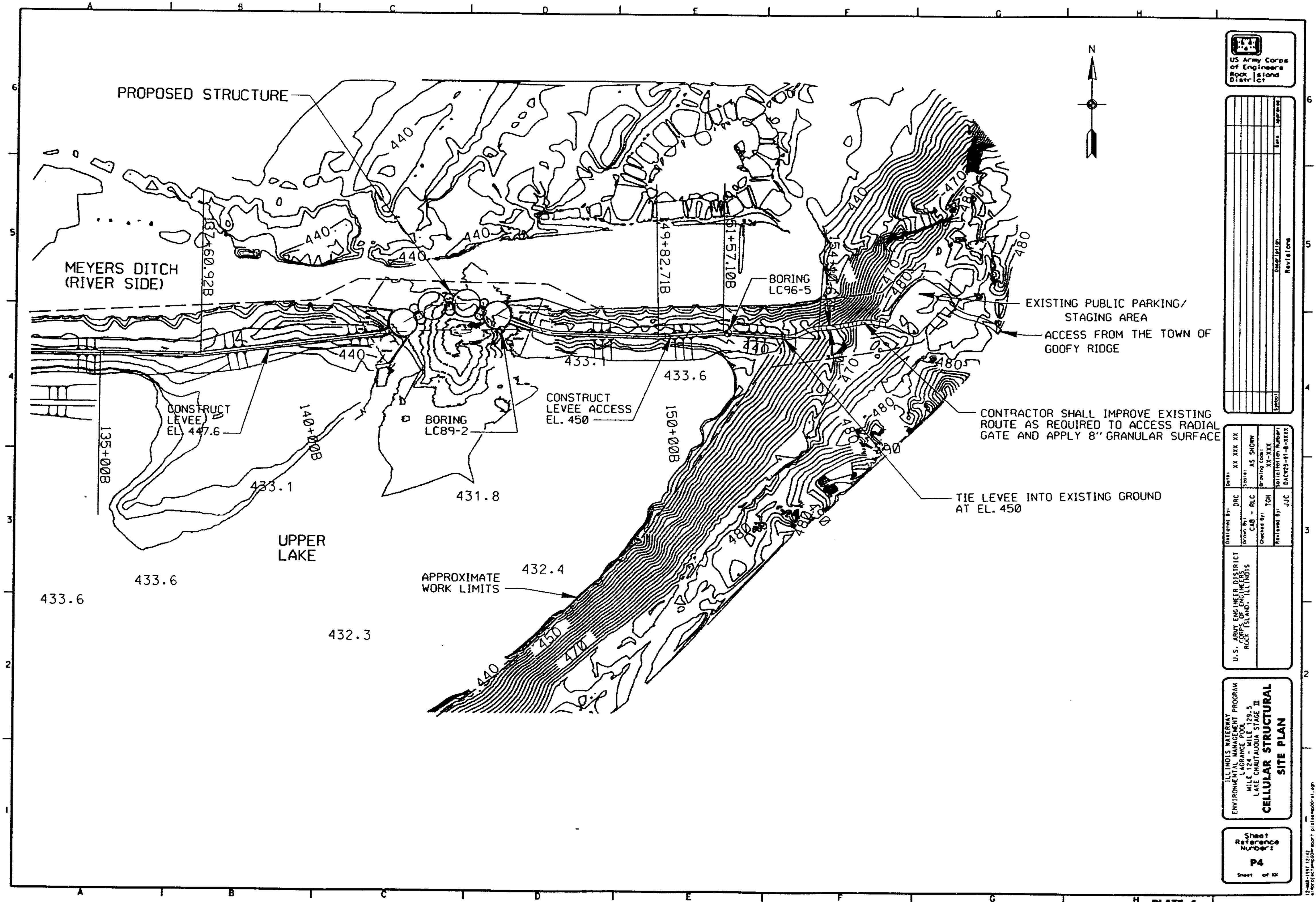
ILLINOIS WATERWAY
ENVIRONMENTAL MANAGEMENT PROGRAM
LAGRANGE POOL
MILE 124 - MILE 129.5
LAKE CHAUTAUQUA STAGE II

# BORING LOGS

Sheet Reference Number: P3

Sheet of XX

PLATE 3



PROPOSED STRUCTURE
MEYERS DITCH (RIVER SIDE)
137+60.92B
149+82.71B
151+57.10B
154+00.60B
135+00B
140+00B
150+00B
BORING LC96-5
BORING LC89-2
EXISTING PUBLIC PARKING/ STAGING AREA
ACCESS FROM THE TOWN OF GOOFY RIDGE
CONSTRUCT LEVEE EL. 447.6
CONSTRUCT LEVEE ACCESS EL. 450
CONTRACTOR SHALL IMPROVE EXISTING ROUTE AS REQUIRED TO ACCESS RADIAL GATE AND APPLY 8" GRANULAR SURFACE
TIE LEVEE INTO EXISTING GROUND AT EL. 450
APPROXIMATE WORK LIMITS
UPPER LAKE
433.1
433.6
431.8
432.4
432.3
440
450
470
480
490
N
US Army Corps of Engineers Rock Island District
Symbol
Description
Revisions
Date
Approved
Designed By: DRC
Drawn By: CAB - RLC
Checked By: TGH
Reviewed By: JJC
Date: XX XXX XX
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Solicitation Number: DACW25-97-B-XXXX
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ROCK ISLAND, ILLINOIS
ILLINOIS WATERWAY
ENVIRONMENTAL MANAGEMENT PROGRAM
LAGRANGE POOL
MILE 124 - MILE 129.5
LAKE CHAUTAUQUA STAGE II
CELLULAR STRUCTURAL
SITE PLAN
Sheet Reference Number:
P4
Sheet of XX
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**PLATE 4**

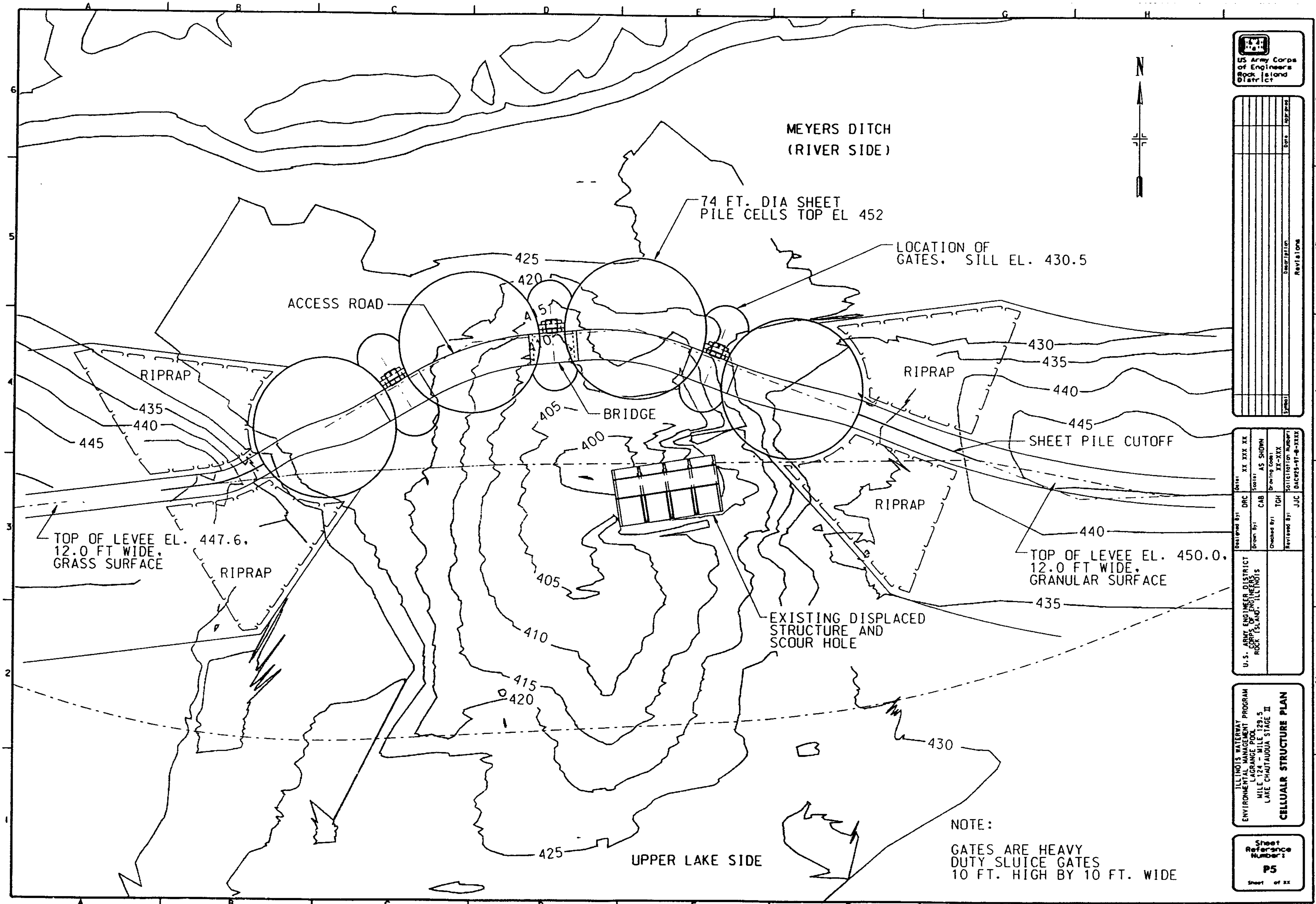
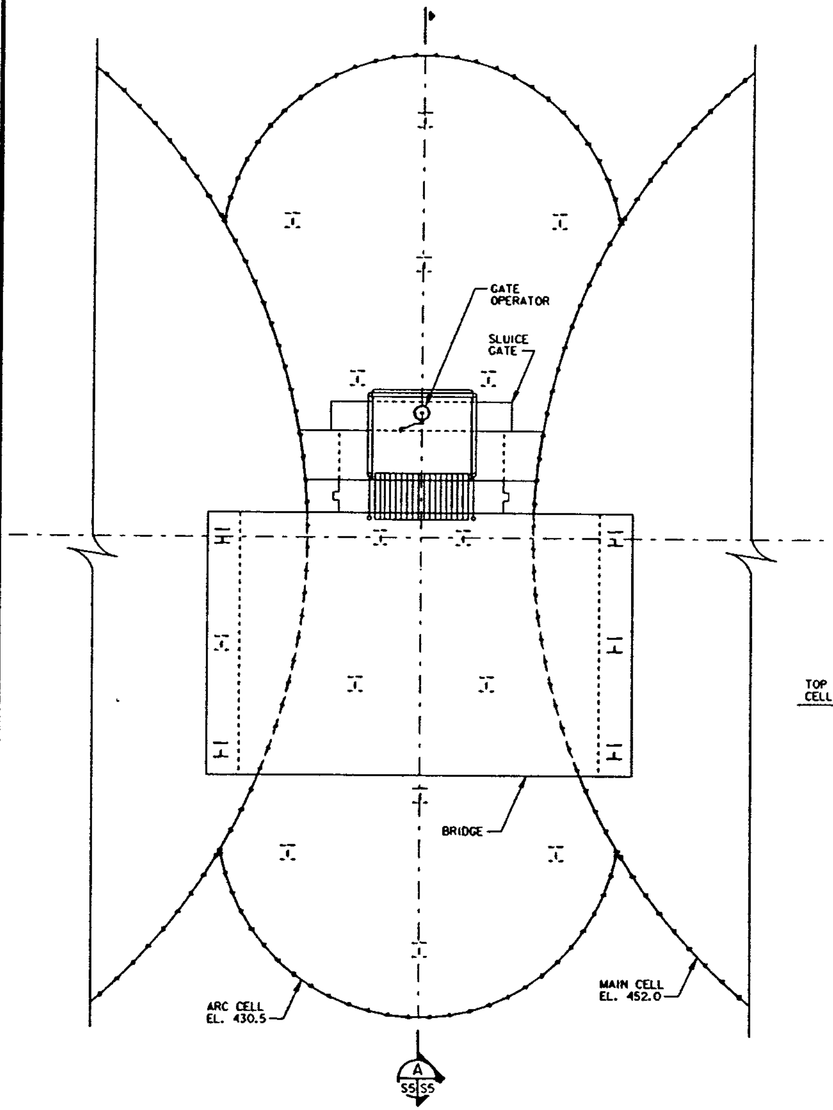

MEYERS DITCH
(RIVER SIDE)
N
74 FT. DIA SHEET
PILE CELLS TOP EL 452
LOCATION OF
GATES. SILL EL. 430.5
ACCESS ROAD
425
420
415
410
405
400
430
435
440
445
RIPRAP
BRIDGE
SHEET PILE CUTOFF
TOP OF LEVEE EL. 447.6,
12.0 FT WIDE,
GRASS SURFACE
TOP OF LEVEE EL. 450.0,
12.0 FT WIDE,
GRANULAR SURFACE
EXISTING DISPLACED
STRUCTURE AND
SCOUR HOLE
UPPER LAKE SIDE
NOTE:
GATES ARE HEAVY
DUTY SLUICE GATES
10 FT. HIGH BY 10 FT. WIDE
US Army Corps
of Engineers
Rock Island
District
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ROCK ISLAND, ILLINOIS
ILLINOIS WATERWAY
ENVIRONMENTAL MANAGEMENT PROGRAM
LAGRANGE POOL
MILE 124 - MILE 129.5
LAKE CHAUTAUQUA STAGE II
CELLUALR STRUCTURE PLAN
Sheet
Reference
Number:
P5

PLATE 5

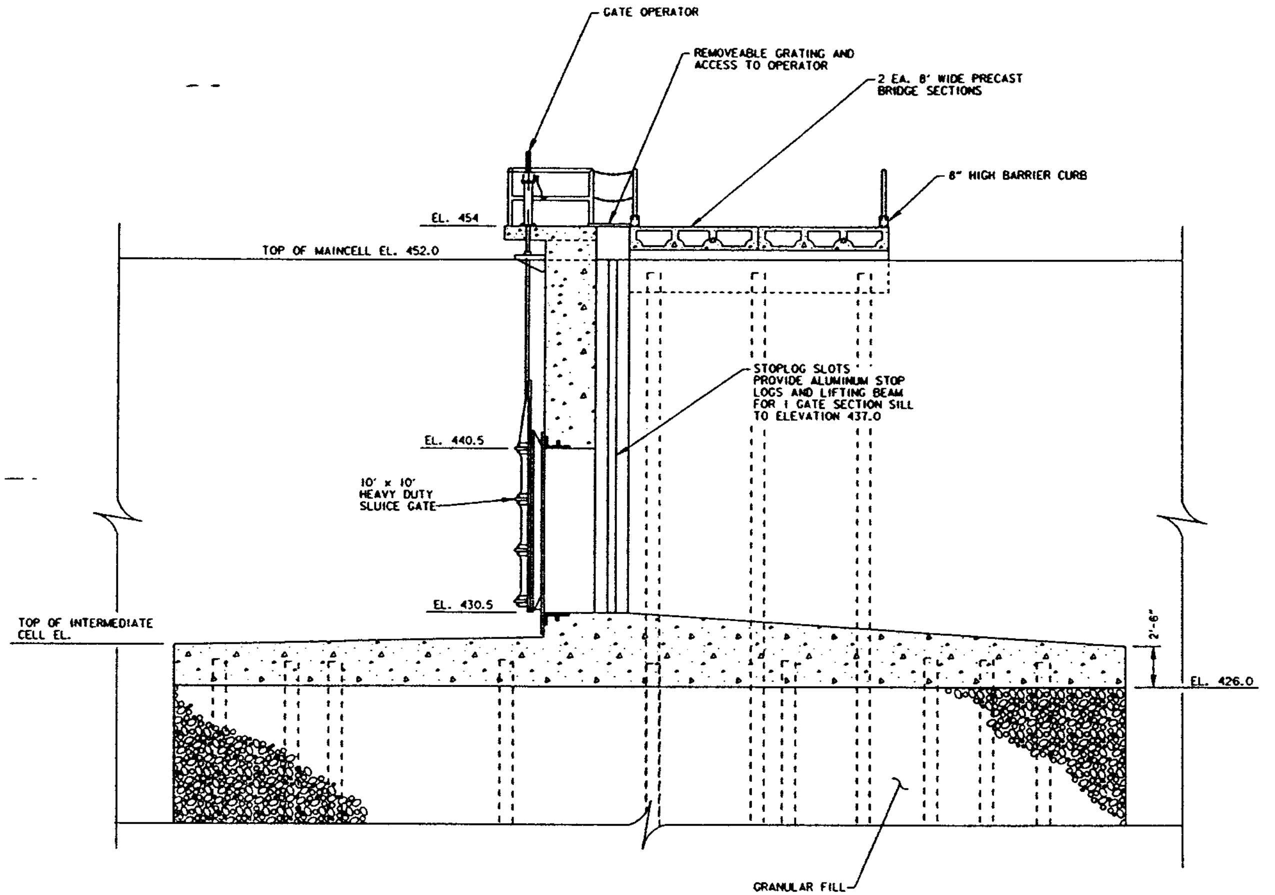



PLAN VIEW

## CELL FLOODWAY

(3 REQ'D)

12" 0 5'
SCALE IN FEET




## SECTION A

12" 0 5'
SCALE IN FEET

US Army Corps of Engineers Rock Island District

Revisions

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Date: XX XXX XX | Scale: AS SHOWN | Drawing Code: XXXXX

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ROCK ISLAND, ILLINOIS

ILLINOIS WATERWAY
ENVIRONMENTAL MANAGEMENT PROGRAM
LAGRANGE POOL

**LAKE CHAUTAUQUA CELL STRUCTURE DETAILS**

Sheet Reference Number: P6

**PLATE 6**

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR
LA GRANGE POOL, ILLINOIS WATERWAY, RIVER MILES 124-129
MASON COUNTY, ILLINOIS

# APPENDIX A

# HYDROLOGY AND HYDRAULICS APPENDIX

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR

# APPENDIX A
# HYDROLOGY AND HYDRAULICS

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UPPER MISSISSIPPI PROGRAM
DESIGN MEMORANDUM RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
RADIAL GATE REPLACEMENT AND LEVEE BREACH REPAIR

# APPENDIX A
# HYDROLOGY AND HYDRAULICS

## 1. PURPOSE

a. This appendix summarizes computations for various design alternatives for the replacement of the Lake Chautauqua control structure and levee remediation. The information is chronologically organized, providing a complete history of investigated alternatives leading up to the final (recommended) design. The recommended plan was the Gated Sheet Pile Cell Structure with 5 year Level of Protection which begins on page A-13 of this appendix.

b. Lake Chautauqua (about river mile 130) is on the east bank of the Illinois River between the gages at Copperas Creek and Havana. The levee originally protected the lake from flooding by the Illinois River with a design crest of 449.1 feet (all elevations referenced to NGVD). Due largely to cost considerations, the final recommendation was to replace the previous control structure with three 10 ft by 10 ft sluice gates, resurface the levee crest to an elevation of 447.5 ft, and have an overflow section with an elevation of 447.0 ft. The protection level of the recommended levee is 447.0 ft.

c. The main purpose of the control structure is to raise the interior lake level when river stages are predicted to exceed the levee crest; this would reduce erosion damage to the levee when water flows over the levee crest. Three secondary purposes are to allow drainage of Lake Chautauqua to elevation 431.5 feet when the water level on the Illinois River is low, to regulate the lake level, and to allow for fish passage. Inflow to the lake comes from rainfall and from springs within the lake. The drainage area of the lake is about the same size as the lake itself.

d. There were five occurrences from 1940 to 1996 where the river stages have been estimated to exceed the original levee crest of 449 ft. For two of these calendar years (1982 and 1985) the elevation was exceeded twice within the given year. These estimates were made by subtracting 0.7 feet from the observed stages at Copperas Creek. The value 0.7 was determined from published Illinois River flood profiles. Peak stages are shown on Plate A-1. However there are occasions when this gage was not working or other nearby gages were more accurate, in which case the more accurate results were used.

## 2. ANALYSIS OF ALTERNATIVES

The design of the tainter gates and sluice gates was examined as a direct replacement of the original project structure; these inlet structures use design criteria listed on Table A-1. The fuse plug spillway (p. A-11) was studied to replace or augment the inlet structure designs. The overflow section for the moist soil unit (p. A-12) was designed for a levee crest of elevation 446 feet using a different design criteria. The final recommended plan was designed for a levee crest of 447.5 ft (having an overflow crest of 447.0 ft), and uses design criteria located on p. A-13.

## 3. INLET STRUCTURES

The discharges necessary to fill the lake before levee overtopping are much larger than the discharges to drain the lake. As a result, the inlet was sized to raise the interior lake level prior to a flood. When the Illinois River rises from 446 ft to 449 ft, the fastest rate of rise is about one foot per day. All computations used the following hypothetical situation from earlier studies. The structure is opened when the Illinois River is at elevation 446 feet and the interior lake is at elevation 435 feet. The interior lake should be at elevation 448 feet three days later when the Illinois River is at elevation 449 feet and about to flow over the levee. Table A-1 summarizes this design criteria.

Table A-1
Design Criteria for Inlet Structures

| Description of Parameter | Elevation (Ft.) |
|---|---|
| Levee Crest Elevation | 449 |
| Level of Illinois River when gate is opened | 446 |
| Initial Lake Chautauqua Water Level | 435 |
| Final Lake Chautauqua Water Level | 448 |
| Crest of Regulating Weir if used | 437 |

a. Tainter Gate Inlet Structures

Three tainter gate alternatives were studied. Each of the alternatives consisted of two parallel, concrete channels. Each channel was rectangular with a bottom elevation of 431.5 feet and a length of about 53 feet. The gate was located at about midpoint of the 53 feet distance. Transition walls through the levee could make the structure longer.

Alternative 1 consisted of two gate bays, Alternative 2 consisted of two gate bays with a stop log weir at the entrance (river side) to each channel, and Alternative 3 was a hybrid (one channel had a stop log weir at the entrance while the other channel did not). Computations were made to determine the required gate width for each alternative. It was assumed that the gates would be opened within a short time period and that the gate bottoms would be completely out of the water. See Plate A-2 for a sketch of alternatives 1 and 2.

The gate width for each alternative is summarized in Table A-2. The widths are measured perpendicular to the direction of flow. After studying the alternatives the Design

Branch decided to use two 9 foot wide gates as the optimum tainter gate design. Stop logs would still be used to allow for easier regulation of lake levels, but they would have to be completely removed prior to opening the gates, otherwise they would block flow.

Table A-2
Tainter Gate Widths for Various Alternatives

| Description | Width of Each Gate (Ft.) | Total Gated Width (Ft.) |
|---|---|---|
| Alternative 1, Channel Only | 8.7 | 17.4 |
| Alternative 2, Channel w/stop logs | 14.7 | 29.4 |
| Alternative 3, Hybrid | 11.3 | 22.6 |

(1). Method of Analysis for Tainter Gate Widths

The solution was found by iteration until the gate width satisfied the design criteria. Computations were made on a Microsoft spreadsheet. At time equal to zero, the values for the river and lake were assigned and the inflow was set to zero. The volume of water in the lake at time equal zero was determined by the assigned lake elevation. For the next time, "n+1", the river stage was computed based on a constant rise. The lake level was based on the volume at time equal "n". The discharge at "n+1" was determined using the new "n+1" river and lake elevations. The inflow volume for the "n" to "n+1" time interval was the average of discharges at "n" and "n+1" multiplied by the time interval. The final volume at "n+1" was the volume at "n" plus the inflow volume for the "n" to "n+1" time interval. The computation interval could be set at any value; generally it varied from 15 minutes to 1 hour.

The iterations were repeated until 72 hours had passed. The discharge through the two open channels was computed using weir equations and coefficients from the Hydraulic Design Criteria (Reference 1). All computations were in English units. The elevation-volume relationship for Lake Chautauqua was taken from Reference 2 Plate F-9. Essentially, above elevation 434.5 feet the surface area is 1,200 acres.

Alternative 1, Channel Only

A sketch of this alternative is shown on Plate A-2. It was initially analyzed as a broad-crested weir. This width was then verified by comparing the discharge at various head water and tail water elevations with the discharge computed for two free flowing rectangular culverts.

The equation for free flow over a broad-crested weir (Reference 1 HDC 711) is:

$$Q = C_f (L-2KH) H^{3/2}$$

In this equation: $C_f$ is the empirical coefficient, L is the weir length (perpendicular to the direction of flow), K is the end contraction coefficient, and H is the head on the weir. The value of $C_f$ for the range of the H/B ratios (17.5/53) in this problem is constant and is 2.66 (HDC Chart 711, Figure a: Free Flow). B is the distance parallel to the direction of flow.

The value of K is initially taken as 0.1 as recommended. The free discharge for a broad-crested weir is not reduced until the depth of submergence is greater than 0.67 of the head on the weir. When this condition occurred the discharge computed from the free flow equation was reduced by the ratio Cs/Cf to account for submergence. This ratio is plotted in Reference 1 HDC Chart 711 Figure b: Submerged Flow. The ratio is a function of $H_2/H_1$ (the ratio of the tail water height above the weir crest to the head water height above the weir crest). In the spreadsheet, the value of $H_2/H_1$ was computed and used with the "lookup" function to choose the appropriate submergence factor.

Preliminary discharges using the broad-crested weir equation were about 20 to 30 percent smaller than discharges computed with a Corps of Engineers culvert rating program (HEC-IFH gravity outlet option). Parameters used in the culvert program included an "n" value of 0.012, an entrance loss coefficient of 0.2, and wing walls flared 30 to 75 degrees. The length was increased to 108 feet to include the part through the levee. The culvert program was believed to be more realistic since the weir equations were derived for large monolith structures with higher heads, longer breadths, and more uncertainty about the submergence coefficient. The end-contraction-reduction term (2KH) was eliminated from the discharge equation and the revised answers were then about 4 to 10 percent smaller than the culvert program. Using the culvert discharges would result in a slightly more narrow gate width design. Since the difference would be in inches, the weir spreadsheet was used.

Computations for the selected design of two nine foot gates appear on Plates A-3 to A-5. For comparison the rating table for two 9 foot wide culverts appear on Plates A-6 and the rating curve using the broad-crested weir equations appears on Plate A-7.

## Alternative 2, Channel and Stop Log Weirs

A sketch of this alternative appears on Plate A-2. It was analyzed as a sharp-crested weir to obtain the appropriate width. The sharp crest equation was selected because the stop log breadth was less than one half of the head (Reference 3 pp. 5-24). The approach recommended in HDC Sheet 122-1/2 was followed. The spillway design flow $Q_d$ was computed using the appropriate coefficient from Reference 1 HDC Chart 122-1/2 and abutment contraction coefficients from HDC Charts 111-3/1 and 122-2. The spillway design head $H_d$ and the computed design discharge $Q_d$ were then used as recommended in HDC Sheet 111-3/3 to compute the discharge for other depths.

The equation for the unit discharge of a sharp-crested weir in $ft^3/s$ (Reference 1 HDC 122-1/2) is:

$$q_d = C_d H^{3/2}$$

$C_d$ is the spillway design head discharge coefficient. H is the spillway design head in feet. The value of $C_d$ is a function of the distance from the crest of the weir to the floor of the channel (P) and the head on the weir (H). Reference 1 HDC Sheet 122-1/2 recommends using the USBR coefficients since they result in a conservative design. For a vertical weir with a P/H of (5.5/12) the value of $C_d$ is 3.78.

The effective length L was determined using HDC Sheet 111-3/1 to estimate abutment contraction and is:

$$L = L' - 2\,(N\,Kp + Ka)\,He$$

L' is the net length of crest, N is the number of piers, Kp is the pier contraction coefficient, Ka is the abutment contraction coefficient, and He is the energy head on the crest. The center wall was treated as a pier. If the pier is square Kp is 0.03 (Reference 1 HDC 111-5), if rounded it is 0.02 (Reference 1 HDC 122-2). A value of 0.03 was used for Kp. If the radius of the concrete abutment at the entrance is 2.4 feet, Ka can be as low as 0.08. However, the recommended value of 0.1 from the text was used.

Combining the two previous equations yields the equation for the design discharge:

$$Q_d = C_d\,\{L'-2(NKp+Ka)H\}\,H^{3/2}$$

Substituting the values this becomes: $Q_d = 3.78\,\{L'-2(1.03)H\}\,H^{3/2}$

Reference 1 HDC Sheet 111-3/3 recommends computing the discharge (Q) for other heads (He) using equation shown below:

$$Q = Q_d\,(He/Hd)^{1.6}$$

In this equation $Q_d$ is the design discharge and Hd is the design head. For each trial weir length $Q_d$ is computed once for Hd and the remaining discharges are found by solving the above equation for Q using the correct value for He. Taking the difference between the river elevation and the crest elevation of the weir, a maximum head of 12 feet was selected for the design head.

The discharge for a sharp-crested weir is reduced by submergence. When this condition occurred the discharge computed for the free flow equation was reduced by the sharp-crested ratio Cs/Cf to account for submergence. This sharp-crested ratio was from a different curve in Reference 1 HDC Chart 711 Figure b Submerged Flow. In the spreadsheet the value of $H_2/H_1$ was computed and used with the "lookup" function to choose the appropriate submergence factor for the sharp-crested weir.

Friction in the channel from the weir to the lake could result in a higher water level downstream of the stop logs than in the lake. If this occurred the method used to estimate discharge would overestimate the discharge over the weir. The significance of this head loss was evaluated using Chezy's equation (Reference 4 p. 92). The lake level was used to compute a channel velocity. This velocity was assumed to be constant in the channel. The equation for head loss is shown below:

$$\text{delta } h = (L\,V^2)\,/\,(C^2\,R)$$

Here L is the 53 feet channel length, V is the average water velocity, C is Chezy factor, and R is the hydraulic radius (cross sectional area divided by wetted perimeter). For most of the time the computed head loss due to friction was less than 0.1 feet. During the time when the lake level starts to submerge the weir the head loss is about 0.5 feet. However this small difference would not result in significant difference in the computed value of discharge.

### Alternative 3, Hybrid

In this analysis two discharges were computed. The broad-crested weir equation was used for one channel and the sharp-crested weir equation was used for the stop log channel. Then the average flows over the computation interval were added to compute the volume of water entering the lake during that time interval.

Since there was only one sharp-crested weir channel, the pier loss term was set to zero. However the abutment contraction coefficient of 0.1 was retained (see equations discussed under alternatives 1 and 2 for details).

### (2). Energy Dissipators Downstream of Structure

The discharge per unit foot width is quite large because the two channels are narrow and deep. Also velocities are high because the initial lake water level is below the critical depth for the expected river water level. Both of these factors indicate the potential for scour damage downstream of the outlet. Scour can occur even though the structure is only in operation for three day durations.

If the downstream exit consists of wingwalls with a concrete pad at elevation 431.5 feet the height of the water emerging from the 9-feet channels will drop which increases the average velocity of the discharge stream. Because of the high velocities, the flow is predicted to spread out at a small angle of about 13 degrees. If the energy equation is used and losses are ignored (Reference 5 page IV-A-6) the velocity could be as high as 26 feet per second at the end of the concrete pad.

It is difficult to predict the size of the scour hole formed in the sand downstream. An equation designed for small culverts (Reference 5 Chapter V) predicts a scour depth of 18 feet. An equation from an article on jet scour below flip buckets (Reference 6) predicts a depth of 22 feet. This range seems to be reasonable based on erosion measurements taken in 1996.

### Hydraulic Jump Stilling Basin

One design to reduce erosion damage is a hydraulic jump type stilling basin. The design followed guidance in Reference 7. A drawing of this type of basin appears on Plate A-8.

Widening the stilling basin reduces the excavation depth for the stilling pool floor. For this reason three sizes were examined. Table A-3 summarizes computed dimensions for the stilling basins. Two basins used discharges from the selected 9 foot gate plan. The no flair basin was designed as a continuation of the outer gate walls; the stilling basin width was the same as the two gates plus the dividing wall. The 1 on 6 basin used the maximum allowable angle in the transition section dropping to the stilling pool floor. The third stilling basin was designed for two 16.5 feet gates at a bottom elevation of 437.5 feet. This last option was studied to see if raising the gates would result in a more compact design. The stilling basin with a width of 22 feet was selected.

Table A-3
Dimensions for Stilling Basins

| Description of Item | Two 9 ft. Gates No Flair | Two 9 ft. Gates lon6 Flair | Two 16.5 ft Gates |
|---|---|---|---|
| Gate Height (ft.) . . . . . . . . . . | 18 | 18 | 12 |
| Transition Zone | | | |
| Entrance (elev.) | 431.5 | 431.5 | 437.5 |
| Angle | 0 | 9.46 | 0 |
| Horizontal run (ft.) | 13.3 | 15 | 11.3 |
| Vertical drop (ft.) | 9.2 | 7.9 | 11.3 |
| Stilling Pool Floor (elev.) . . . | 422.3 | 423.6 | 426.2 |
| Stilling Basin Width (ft.) | 22 | 27 | 37 |
| Stilling Basin Length (ft.) . . . | 44.6 | 40.3 | 31.1 |
| Pier Height (ft.) | 2.5 | 2.3 | 1.7 |
| Sill Height (ft.) . . . . . . . . . . | 1.2 | 1.1 | 0.9 |
| Top of Wall (elev.) | 442.5 | 442.5 | 445 |
| Stilling Wall Height (ft.) . . . | 20.2 | 18.9 | 18.8 |
| $d_1$ Depth (ft.) | 3.33 | 2.76 | 1.74 |
| $d_2$ Depth (ft.) . . . . . . . . . . . | 14.88 | 13.42 | 10.38 |
| Design discharge (cfs) . . . . . . | 2650 | 2650 | 2191 |
| Illinois River (elev.) | 446 | 446 | 446 |
| Tail Water in Lake (elev.) . . . | 435 | 435 | 435 |
| Total Width of Gates (ft.) | 18 | 18 | 33 |
| Wall Between Gates (ft.) . . . . | 4 | 4 | 4 |

The stilling pool will operate over a range of inlet discharges and tailwater conditions. Rather than choosing the conditions which would result in the deepest structure, the initial conditions were chosen in view of the short durations and infrequent periods of operation. However, the top of the wall was set at the elevation where supercritical flow no longer occurred within the inlet structure. As the lake level rises the hydraulic jump will be drowned out and the structure will operate as a channel connecting two reservoirs flowing at subcritical flow.

## Riprap Stilling Basin

Reference 5 contains equations for designing riprap stilling basins. However even with the maximum plunge pool (7.6 feet) this design required very large stone. The d50 stone size was between 1.45 and 2.0 feet and detailed computations were not made for this reason.

b. SLUICE GATE INLET STRUCTURES

Each sluice gate alternative consisted of gates placed in three foot thick concrete walls. These walls spanned the distance between sheet pile cells. Four standard sizes were evaluated. The same bottom gate sill elevation of 431.5 feet was used for all sizes.

It was assumed that all gates would be opened within a short period of time. However in this case the top of the gate opening would be under water. See Plate A-9 for a sketch. Leakage rates could be higher through the cells than with levees or concrete walls. And the submerged openings may be more prone to collect debris or ice.

Three gates (regardless of the size) were required for each alternative. Table A-4 lists the gate size and the predicted lake level after 72 hours. Computations were based on the criteria summarized in Table A-1. Design Branch selected the 9 ft. (vertical) by 10 ft. (horizontal) gate for the design.

Table A-4
Final Lake Level using Three Sluice Gates of Various Sizes

| Vertical Dimension (Feet) | Horizontal Dimension (Feet) | Final Lake Elevation (Feet) | Elev. used for flow Transition |
|---|---|---|---|
| 8 | 12 | 448.8 | 441.7 |
| 9 | 9 | 448.0 | 442. |
| 9 | 10 | 448.6 | 442. |
| 10 | 10 | 448.9 | 442.7 |

(1). Method of Analysis for Sluice Gates

Computations were made on a Microsoft spreadsheet. The routing method was identical to the arrangement discussed under Method of Analysis for Tainter Gate Widths. The computation interval could be set at any value; 1 hour was used.

Plate A-9 shows a sketch of a cross-section through one of the gates. As can be seen from this sketch when the gate is first opened the tailwater is low enough that it operates as a sluice gate (free controlled flow). As the lake level rises the flow through the gate enters a transition zone; then as the level continues to rise the gate operates as an orifice (submerged controlled flow). The two equations used to compute the instantaneous discharge for the spreadsheet are shown below:

Free Controlled Flow: $Q = C_g A (2gH)^{1/2}$

Submerged Controlled Flow: $Q = C A (2gh)^{1/2}$

In both of these equations Q is the discharge through the gate, C is the discharge coefficient, A is the area of the opening, and g is the acceleration of gravity. In free controlled flow, H is the difference between the upstream energy grade line and the gate sill. While in the submerged controlled flow, h is the difference between water levels in the Illinois River and the Lake Chautauqua.

The theoretical value of Cg for a vertical sluice gate computes to 0.49 using equations in Reference 4 (pp. 202-203). Values of Cg are also given in Reference 10 (chapter 5) for Tainter gates. These values are displayed as a function of the gate opening (G), gate radius (R), trunnion height above sill (a), and the gross head on the gate (H). Coefficients for the smallest "a/R' ratio (0.3) would approach a vertical gate. The applicable values of Cg for a gate 9 feet high at the site are in the range of 0.44 to 0.455. A constant value of 0.447 was used for Cg for all gate heights.

As the tailwater (lake level) approaches the conjugate depth for the discharge under the sluice gate the hydraulic jump is drowned. Conjugate depth calculations for conditions at the project site yield a tailwater elevation of about 443 feet. The transition range between free controlled flow and submerged controlled flow also appears in Reference 10 (figure 5-8). The range is plotted as H/G versus h/H. Values for the middle of the transition range from this reference were used to switch from the free controlled flow equation to the submerged flow equation. These tailwater elevations appear in Table A-4.

There was much variation among suggested values of C for submerged controlled flow. A value of 0.84 would produce discharges that are in agreement with culvert programs used by the district. Data from Reference 10 can be recombined to produce C values for Tainter gates that range from 0.8 to 1.0. Values of C from Reference 3 range from 0.6 to 0.83. Computations used values that ranged from 0.62 for a difference in water level of 5 feet to 0.83 for a difference of 0.02 feet (Reference 3 Table 4-9).

Computations using this procedure for the selected design of three 9 foot (Vertical) by 10 foot (Horizontal) gates appear on Plates A-10 to A-11. For comparison the rating curve using the same equations appears on Plate A-12.

(2). Energy Dissipation Downstream of Structure

Energy of the water flowing through the gates would be dissipated by forming a plunge pool on the downstream side of the sheet pile cells. As these cells would extend to bedrock the scour hole would not threaten the foundation.

c. CONSTRUCTION AND OPERATION (Levee Crest 449.0 ft)

Incomplete embankment structures which consist of zones of dissimilar materials are especially susceptible to erosion damage. Protection provided by a completed structure is not present during construction since exposed discontinuities provide the potential for greatly accelerated erosion during overtopping. Provision should be make to protect the incomplete structure with a temporary emergency spillway. Even after construction is complete a section of levee should not be overbuilt so as to act as an emergency spillway to control the location of overtopping (Reference 9).

Previous reports have stated that when the Illinois River is expected to exceed the top of the levee, Lake Chautauqua will be flooded to protect the levee. The following paragraphs attempt to convert this desire into some specific recommendations. In forming this plan, two kinds of floods were difficult to predict. One class occurred when the flood originated upstream with a predicted crest at the site very near the levee crest. If storage is available in Lake Peoria the observed stage was slightly lower than the estimated peak stage. In cases where the river rises slowly this storage is not as significant a factor and estimated and peak stages agree. While this small amount of storage may be modeled in the future, there is no easy way to cut it exactly in the proposed forecast plan. The second situation occurs when the flood is fed by tributaries that enter the Illinois River near the project site. This condition is sometimes accompanied by stage increases that exceed 1 foot per day or backwater effects that produce more erratic stage increases.

The forecast model makes use of stages measured at gages upstream on the Illinois River. Discharges on the Fox River at Dayton, Illinois, and on the Kankakee River near Wilmington, Illinois, should also be monitored since they provide an indication of discharges that will soon be coming down the Illinois River. Monitoring discharges on the Mackinaw (at Congerville) help determine if the total discharge in the Illinois River will approach 70,000 cubic feet per second. Discharges in this range produce stages that overtop the levee. Monitoring the Mackinaw River also provides information on flooding within the central part of the state. Partial filling to elevation 444 feet is made so that enough time is available to flood the entire levee.

## Operational plan for three 9'x 10' gates

The following target stages and actions make up the flood forecast plan (levee crest @ 449.0 ft):

1) When the tailwater stage downstream of Starved Rock Dam exceeds elevation 461.0 feet, open the emergency inlet gates and raise the lake to elevation 444 feet.

2) Even if condition 1 has not occurred: When the tailwater stage at Henry exceeds elevation 454.0 feet, open the emergency inlet gates and raise the lake to elevation 444 feet.

3) If the tailwater stage downstream of Peoria (ILO7) is at 450.0 feet and rising, and the Mackinaw River (CNGI2) is above 10,000 cubic feet per second, open the emergency inlet gates and raise the lake to elevation 444 feet.

4) When the tailwater stage downstream of Peoria (ILO7) Dam exceeds elevation 452.5 feet, open (or keep open) the gates and flood the lake to elevation 448 feet.

The flood forecast plan was made by studying a group of flood events. It was then tested on floods between 1940 and 1996 when the estimated stage at the project was above elevation 447 feet. Stages at Copperas Creek, Kingston Mines, and Havana were used to estimate the peak stage at Lake Chautauqua. Table A-5 summarizes the operation of the plan. The floods are divided into events that would have been below elevation 449 feet (-) and events that would have been above elevation 449 feet (+). Recommendations of the plan to keep gates closed or open are noted. The distance column shows the distance from the design levee crest (449 feet) to the estimated peak river stage.

Table A-5
Summary of Gate Operation Results

| | River Below El. 449 | | River Above El. 449 | | Distance |
|---|---|---|---|---|---|
| Mo.-Yr. | Closed | Opened | Closed | Opened | (Feet) |
| May 1943 | | | | Yes | +2.6 |
| Apr 1944 | Yes | | | | -1.5 |
| Apr 1950 | | Yes | | | -1.0 |
| Mar 1962 | Yes | | | | -2.0 |
| May 1970 | | Yes | | | -0.5 |
| Apr 1973 | Yes | | | | -1.2 |
| Jun 1974 | Yes | | | | -0.4 |
| Mar 1979 | | | | Yes | +2.3 |
| Mar 1982 | | | | Yes | +1.2 |
| Dec 1982 | | | | Yes | +1.7 |
| Apr 1983 | | Yes | | | -0.2 |
| Mar 1985 | | | | Yes | +2.3 |
| Nov 1985 | | | | Yes | +0.3 |
| Apr 1993 | Yes | | | | -1.3 |
| May 1995 | | | | Yes | +1.7 |

Following the plan would have opened the gates before floods that would have overtopped the levee. However, there were three events when the gates would have been opened but the Illinois River would not have overtopped the levee crest (elevation 449 feet). Following the plan during the 57 year period would have minimized overtopping damage of the Lake Chautauqua levee. This plan was prepared to determine the feasibility of forecasting. It is quite likely that refinements or even new plans may be developed later.

4. FUSE PLUG SPILLWAY SECTION

a. Lake Chautauqua could also be flooded by a fuse plug section or a combination of gates and fuse plug sections. The fuse plug is a sand section of the levee specifically designed to scour away prior to levee overtopping. This section would be hardened below the sill elevation and constructed of sand from the sill to the crest of the fuse section (448 feet). The back side of this section would be hardened with soil cement or gabions to resist erosion.

b. The fuse plug computations used the same maximum rise in the Illinois River of 1 foot per day. Computations started with the instantaneous failure of the fuse at an Illinois River elevation of 448 feet. Inflow continued until the Illinois River reached the crest of the levee (449 feet). Table A-6 shows the lengths of fuse plugs necessary to raise the interior lake from elevation 435 feet to elevation 448 feet within 24 hours. Inflow for the spreadsheet was determined using the weir equation:

$$Q = C_f \ L \ H^{3/2}$$

c. Submergence of the weir was accounted for by using the submergence adjustment for a broad-crested weir discussed in alternative 1 of the Tainter gate analysis. The weir coefficient for each run was constant and based upon the value of the initial head. Most of the coefficients were taken from Reference 8; this report modeled weir flow over levees. The coefficient for a sill elevation of 448 used the minimum broad-crested weir coefficient from Reference 1 HDC 711.

Table A-6
Lengths Recommended for Fuse Plug Reaches

| Sill Elevation (Ft.) | Reach Length (Ft.) | Initial Head (Ft.) | Weir Coef. |
|---|---|---|---|
| 448 | 8000 | 0 | 2.66 |
| 447 | 1670 | 1 | 2.72 |
| 446 | 725 | 2 | 2.92 |
| 445 | 410 | 3 | 3.14 |
| 444 | 270 | 4 | 3.26 |

5. OVERFLOW SECTION FOR MOIST SOIL OPERATION

a. A different approach and design purpose would be to operate the interior area protected by the upper levee as a moist soil unit. In this case there would be no lake, and the level of protection and levee crest would be lower. A weir section would flood the interior as the Illinois River exceeds the levee crest, and no manual operation would be required.

b. Elevation 431.5 feet was used as the interior water level at the start of the computations. The broken segment of levee would be rebuilt as a weir. It appears that the weir cold have a maximum length of 300 feet. The weir crest was assigned an elevation of 442.5 feet so that it could match the crest of the adjacent moist soil unit protected by the lower levee. In this situation it was the levee crest that was determined. The idea was to find the levee crest such that as the Illinois River rose at its maximum rate (1 foot per day) the difference between the lake level and the levee crest would approach 1 foot. The discharge used in the storage routing was determined by the equation for free flow over a broad-crested weir. The equation is shown below:

$$Q = C_f \ L \ H^{3/2}$$

A value of 2.72 was used for $C_f$ (Reference 8). 300 feet was used for the weir length (L). The submergence adjustment used the broad-crested weir table. However the weir was only submerged for the last hour when the interior was above elevation 444.6 feet. No adjustment was made for weir length contraction. After 85 hours the Illinois River had risen to 446.04 feet and the interior water level had reached elevation 444.94. From this routing a levee crest elevation of 446 feet was selected as the design levee crest. Either stop log sections or the culvert through the pump station could be used to drain the interior after a flood.

c. Historical data from the Havana gage (1943-1996) indicates a high frequency of overtopping for a weir crest of 442.5 ft at Lake Chautauqua. There have been 64 overtops in the 53-year period of record, 16 overtops have occurred during the growing season (Jun 15 - Oct 15). Overtopping during the growing season is detrimental to the successful operation of a Moist Soil Unit; therefore, this option may not be a proper solution to the wildlife needs of the area.

## 6. GATED SHEET PILE CELL STRUCTURE WITH 5 YEAR LEVEL OF PROTECTION

A final approach and design is to allow the interior area to be managed as either a moist soil unit or as a lacustrian environment. This can be accomplished by using three 10 ft x 10 ft sluice gates with a levee crest of 447.5 ft. A 3000 ft overflow section with crest of 447.0 ft is to be located between stations STA 25+00 and STA 55+00 to provide a control point for initial overtopping as well as to aid in the filling of the lake as the Illinois River threatens to overtop the entire levee. Positioning the overflow section between these two stations minimizes the potential for damage to the levee, the control structure, and the pumping station.

Other overflow section lengths and positions have been considered, but were found to be less beneficial to the project. Even though the levee crest is at 447.5 ft, the protection level offered by the levee is 447.0 ft due to the slightly lower crest of the overflow section.

Historical data from the Havana gage (1943-1996) indicates occasional overtopping for a levee crest of 447.0 ft at Lake Chautauqua. There have been 16 overtops in the 53-year period of record, 3 overtops have occurred during the growing season (Jun 15 - Oct 15). At the original levee's crest of 449.0 ft at Lake Chautauqua, there have been 9 overtops in the period of record, only 1 of which has occurred during the growing season.

### a. Model Assumptions

- 3000 ft Overflow Section with a crest of 447.0 ft
- Levee Crest at 447.5 ft
- Three 10' x 10' Sluice Gates
- Illinois River rises at 1.3 ft/day
- Bottom Sill of Gates at 431.5 ft
- Lake WSEL for Gate Submergence is 442.3 ft
- Initial Lake Level is at 436 ft (other starting levels are examined)

- Open Gates at 445.5 ft (other opening levels are examined)
- Pump Station is OFF during filling of lake
- Computational Time Step is 15 minutes
- Free Controlled Gate Coefficient is 0.45

The computational procedure is the same as that outlined in 'Method of Analysis for Sluice Gate Widths' on page A-8 of this appendix, using the parameters listed above.

b. Energy Dissipation

Energy dissipation is accomplished by directing the inflow through the control structure and into an existing 30 ft scour hole directly downstream of the inlet structure. The scour hole functions as a plunge pool energy dissipator. The larger the scour hole, the less erosion that would be expected. This approach relies upon the structural stability of the inlet structure. The inlet structure is constructed of four sheet pile cells driven to bedrock and is considered quite stable.

c. Construction and Operation (Levee Crest 447.5 ft)

(1). Levee damage is minimized by reducing the head difference between the Illinois River and the lake level at the time of overtopping. Past reports have indicated that a head difference of 1 ft will result in no damage to the levee; however, this figure is based on urban levee design where loss of life is the major factor if the levee happens to fail. Hydraulic analyses indicate that a 2 ft head differential will produce overtop velocities of about 6 ft/sec on the downstream slope of the overflow section. Hand calculations together with reference 11 verified this result. The type of grass planted on the overflow section is important in the prevention of scour on the levee. Bermuda Grass, for example, can resist velocities of up to 6 ft/sec on a 1:4 slope; other types of grasses may not perform as well (see Plate 20-A, from Reference 11). Grass that is deep-rooted and well maintained will perform better. **Careful maintenance and attention should be made for the 3000 ft overflow section, as this is where overtopping will begin and is the most likely location for levee damage to occur.**

(2). If the levee (crest 447.5) overtops with a head differential of 2 ft, it will take roughly 3 to 5 hours for the lake and river levels to equalize; damage to the levee during this time should be quite small if in fact there is damage. For head differentials larger than 2 ft the time to fill the lake is increased, and there will be a greater chance for scour to develop.

(3). There are several factors which affect the head differential at time of overtopping: gate sizing, pumping contributions, overflow section length, rate of river rise, time at which the gates are opened, and lake elevation at time of gate opening. All these factors contributed in the selection of the design parameters listed in the 'Model Assumption' section on page A-13 of this appendix. Choosing 10'x10' gates instead of

9'x10' gates will reduce the head differential by 3/4 ft at the time of overtopping, a noticeable change. Turning the 41000 GPM pump on when the gates are raised will result in less than 1/4 ft head difference reduction; this is a minor contribution that may result in siltation problems within the pump station. Adding a 3000 ft overflow section with a crest elevation of 447.0 ft provides greater than 1/2 ft head difference reduction (noticeable), while choosing a 1000 ft overflow section would provide less than 1/4 ft head difference reduction (minor).

(4). The rate of river rise was chosen after considering historical events on the Illinois River. Typically, river rises of 0.5 ft/day to 1.0 ft/day are appropriate for a levee crest of 449 ft. Faster rates of rise have been observed for levee crests below 449 ft. For all computations, a conservative value of 1.3 ft/day has been chosen for a levee crest of 447.5 ft; this rate of rise allows approximately 1.5 days to fill the lake before levee overtopping. Rates of rise greater than 1.3 ft/day have been observed on the Illinois River, although for shorter durations than 1.5 days.

(5). An operating plan was developed on page A-10 of this appendix for a levee crest of 449 ft. For crest elevations less than 449 ft, it becomes more difficult to forecast river peaks due to increasing probabilities of levee overtopping as well as less warning time before the gates should be opened. A levee crest of 447.5 ft (overflow section at 447.0) has roughly twice the chance of overtopping per year than a levee crest of 449.0 ft. Nevertheless, an operation plan has been developed for an overtopping crest of 447.0 ft using a 53-year period of record. Daily records for 29 flood events were examined during this period; 14 events did not overtop the levee, and 15 events did overtop the levee. A complete listing of the 29 events used to develop an operating plan is shown on Table A-7 on the following page.

Table A-7
29 Events Examined using a 53 Year Period of Record

| Year | Month | Would Levee have been Overtopped? | Difference (ft) from Overflow Crest (447.0) Gage HAV | Gage KNG | Action Taken (using Plan) | Desired Action | Results |
|---|---|---|---|---|---|---|---|
| Non-Overtop | | | | | | | |
| 1950 | Apr | ? | - 0.6 | + 0.7 | Gates Open | Gates Open | lake is filled before peak stage |
| 1960 | Apr | No | - 1.0 | - 1.5 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1962 | Mar | ? | - 0.1 | 0.0 | Gates Open | Gates Open | lake is filled before peak stage |
| 1974 | Jan | ? | 0.0 | - 0.1 | Gates Open | Gates Open | lake is filled before peak stage |
| 1974 | May | ? | - 0.1 | + 0.1 | Gates Open | Gates Open | lake is filled before peak stage |
| 1976 | Feb | No | - 1.4 | - 0.3 | Gates Open | Gates Open | lake is filled before peak stage |
| 1981 | May | No | - 2.2 | - 2.2 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1982 | Feb | No | - 1.4 | - 1.8 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1984 | Mar | No | - 1.2 | - 1.2 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1986 | Oct | No | - 2.0 | - 2.1 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1990 | Mar | No | - 2.3 | - 1.2 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1993 | Mar | No | - 1.6 | - 1.6 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1993 | Sep | No | - 1.1 | - 1.9 | Stay Closed | Stay Closed | lake is not filled, no overtopping |
| 1995 | Jan | No | - 0.7 | - 0.9 | Gates Open | Gates Open | lake is filled before peak stage |
| | | | | | | | |
| | | | | | | | |
| Overtopping | | | | | | | |
| 1943 | May | Yes | + 5.5 | + 5.2 | Gates Open | Gates Open | lake is filled before overtopping |
| 1944 | Apr | Yes | + 1.7 | + 0.5 | Gates Open | Gates Open | lake is filled before overtopping |
| 1970 | May | Yes | + 0.6 | + 1.7 | Gates Open | Gates Open | lake is filled before overtopping |
| 1973 | Apr | Yes | + 2.3 | + 1.5 | Gates Open | Gates Open | lake is filled before overtopping |
| 1974 | Jun | Yes | + 2.0 | + 1.4 | Gates Open | Gates Open | lake is filled before overtopping |
| 1979 | Mar | Yes | + 3.4 | + 4.4 | Gates Open | Gates Open | lake is filled before overtopping |
| 1982 | Mar | Yes | + 2.9 | + 3.5 | Gates Open | Gates Open | lake is filled before overtopping |
| 1982 | Dec | Yes | + 3.4 | + 3.6 | Gates Open | Gates Open | lake is filled before overtopping |
| 1983 | Apr | Yes | + 1.8 | + 2.2 | Gates Open | Gates Open | lake is filled before overtopping |
| 1985 | Mar | Yes | + 4.8 | + 4.9 | Gates Open | Gates Open | lake is filled before overtopping |
| 1985 | Nov | Yes | + 2.2 | + 2.0 | Gates Open | Gates Open | lake is filled before overtopping |
| 1993 | Apr | Yes | + 1.3 | + 0.7 | Gates Open | Gates Open | lake is filled before overtopping |
| 1993 | Jul | ? | + 1.8 | - 0.5 | Gates Open | Gates Open | lake is filled before peak stage |
| 1995 | May | Yes | + 4.6 | + 2.8 | Gates Open | Gates Open | lake is filled before overtopping |
| 1996 | May | Yes | + 0.3 | + 0.2 | Gates Open | Gates Open | lake is filled before overtopping |

(6). Using the developed operational plan (listed on the following page) on the above historical flood events has resulted in no levee damage as a result of overtopping; the gates would have been opened with adequate time (1.5 days) to allow the lake to fill to within 1 foot of the levee crest before levee overtopping for all fifteen overtopping events examined. Of the fourteen non-overtop events, there were six events when the gates would have been opened but the Illinois River may not have overtopped the levee overflow crest (elevation 447 feet); however, in each of these six events the peak river stage was within a foot of the levee crest.

Operational plan for three 10'x 10' gates

The following target stages and actions make up the flood forecast plan (levee crest @ 447.5 ft):

Step 1) Is the Stage at Havana greater than or equal to 443.4 ft ?
- if no: Gates remain closed
- if yes: Check step 2 below

Step 2) Is the Stage at Henry greater than or equal to 452.5 ft ?
- if yes: Open the gates at Chautauqua
- if no: Check step 3 below

Step 3) Is the Rate of River Rise at Havana for the last 24-hour period greater than or equal to 0.8 ft per day ?
- if no: Monitor the Stage at Havana several times a day. Open the gates if the stage is greater than 445 ft, otherwise remain closed
- if yes: Wait 8 hours, then Check step 4 below

Step 4) Is the Rate of River Rise at Havana for the last 8-hour period greater than or equal to 0.7 ft per day ?
- if yes: Open the gates at Chautauqua
- if no: Monitor the Stage at Havana several times a day. Open the gates if the stage is greater than 445 ft, otherwise remain closed

Stage information (hydrograph and 30 minute data) can be found at the Corps' Water Control web site:

**http://ncrbkp.ncr.usace.army.mil/docs/ill2_dsp.html**
**The Havana gage is designated HAVI2; the Henry gage is designated HNYI2.**

It was found that the preceding operational plan eliminated overtopping damage to the Lake Chautauqua levee for all 15 overtopping events analyzed. This operational plan was developed according to the assumptions specified on page A-13 of this appendix. **It is recommended that during construction conditions and when the lake is operated as a Moist Soil Unit (lake water surface elevation 431.5 ft) special attention be paid to the timing of gate opening, as the lake will require more than 1.5 days to fill.** If a flood is threatening or an intense period of rainfall is noted upstream, it may be wise to partially raise the lake level as a precaution during these low lake level conditions. Plates A-13 to A-18 provide information describing the response of Lake Chautauqua to various gate opening conditions; expected time to overtop and head differentials at time of overtopping are given for various initial lake and river levels. Plate A-19 shows a discharge rating curve for three 10' x 10' gates operating at various lake and river levels. These plates may serve as useful guidelines for planning gate operations when field conditions are other than those assumed on page A-13 of this appendix.

7. List of References

1. *Hydraulic Design Criteria*, US Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

2. *Lake Chautauqua Rehabilitation and Enhancement Technical Appendices*, US Army Corps of Engineers Rock Island District, June 1991.

3. *Handbook of Hydraulics*, Brater and King, McGraw-Hill Book Company, St. Louis, Mo., Sixth Edition, 1976.

4. *Open Channel Flow*, Henderson, Macmillan Company, New York, 1966.

5. *Hydraulic Design of Energy Dissipators for Culverts and Channels,* Hydraulic Engineering Circular No. 14, U.S. Department of Transportation, reprinted December 1978.

6. *Free Jet Scour Below Dams and Flip Buckets,* Journal of Hydraulic Engineering, Vol. 111, No. 2, February, 1985, ASCE, ISSN 0733-0929/85/0002-0220/, paper number 19494.

7. *Hydraulic Design of Reservoir Outlet Works,* EM 1110-2-1602, U.S. Department of Army Corps of Engineers, 15 October 1980.

8. *Stability of Riprap and Discharge Characteristics, Overflow Embankments, Arkansas River, Arkansas*, Technical Report No. 2-650, US Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi, June 1964.

9. *A Study of Embankment Performance During Overtopping*, by Gilbert and Miller, Technical Report GL-91-23, US Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi, Nov. 1991.

10. *Hydraulic Design of Navigation Dams,* EM 1110-2-1605, U.S. Department of Army Corps of Engineers, 12 May 1987.

11. *Minimizing Embankment Damage during Overtopping Flow,* Publication no. FHWA-RD-88-181, U.S. Department of Transportation, 11 Nov 1988.

# Peak Stages* at Copperas Creek Gage 1940 to 1996

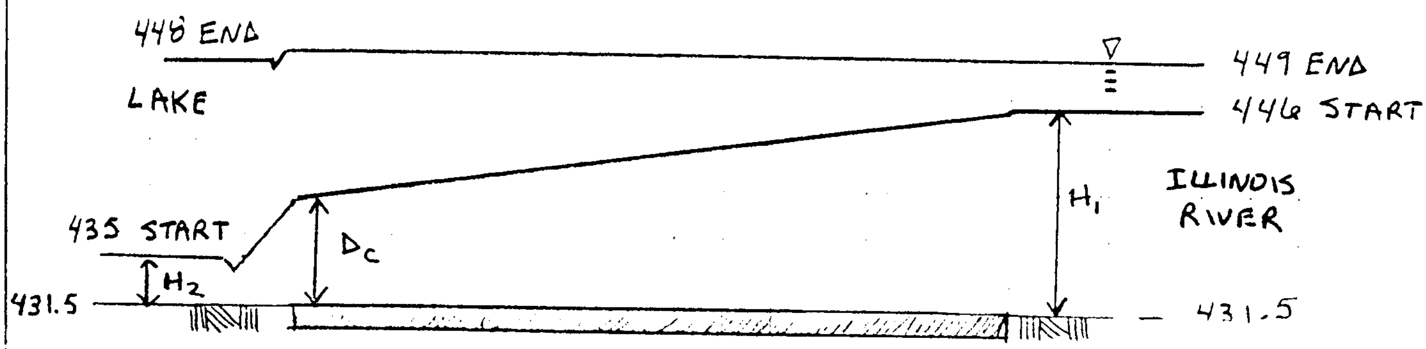
| YEAR | PEAK STAGE | PEAK DATE | OTHER PEAKS? |
|---|---|---|---|
| 1940 | 437.5 | | |
| 1941 | 441.5 | | |
| 1942 | 444.6 | | |
| 1943 | 452.7 | 25-May | |
| 1944 | 448.2 | 29-Apr | |
| 1945 | 444.2 | | |
| 1946 | 444.6 | | |
| 1947 | 444.4 | | |
| 1948 | 445.8 | | |
| 1949 | 442.6 | | |
| 1950 | 447.2 | 30-Apr | |
| 1951 | 445.1 | | |
| 1952 | 442.8 | | |
| 1953 | 439.5 | | |
| 1954 | 441.4 | | |
| 1955 | 440.7 | | |
| 1956 | 438.6 | | |
| 1957 | 443.7 | | |
| 1958 | 443.6 | | |
| 1959 | 443.8 | | |
| 1960 | 445.9 | | |
| 1961 | 443.9 | | |
| 1962 | 447.3 | 28-Mar | |
| 1963 | 440.5 | | |
| 1964 | 439.6 | | |
| 1965 | 443.8 | | |
| 1966 | 444.8 | | |
| 1967 | 443.6 | | |
| 1968 | 444.2 | | |
| 1969 | 442.9 | | |
| 1970 | 448.7 | 21-May | |
| 1971 | 441.3 | | |
| 1972 | 442.6 | | |
| 1973 | 449.5 | 28-Apr | |
| 1974 | 449.5 | 27-Jun | |
| 1975 | 443.7 | | |
| 1976 | 447.2 | 9-Mar | |
| 1977 | 441.4 | | |
| 1978 | 444.7 | | |
| 1979 | 452.0 | 25-Mar | |

| YEAR | PEAK STAGE | PEAK DATE | OTHER PEAKS? | DATE |
|---|---|---|---|---|
| 1980 | 445.2 | | | |
| 1981 | 446.4 | | | |
| 1982 | 451.5 | 10-Dec | 450.8 | 24-Mar |
| 1983 | 449.5 | 16-Apr | | |
| 1984 | 446.4 | | | |
| 1985 | 452.0 | 8-Mar | 450.8 | 25-Nov |
| 1986 | 445.8 | | | |
| 1987 | 441.6 | | | |
| 1988 | 443.0 | | | |
| 1989 | 441.8 | | | |
| 1990 | 445.8 | | | |
| 1991 | 445.4 | | | |
| 1992 | 443.1 | | | |
| 1993 | 448.4 | 20-Apr | | |
| 1994 | 443.0 | | | |
| 1995 | 451.7 | 31-May | | |
| 1996 | 448.0 | 5-Jun | | |

* Stages at Lake Chautauqua are approximately 0.7 ft lower than the stages at the Copperas Creek gage.

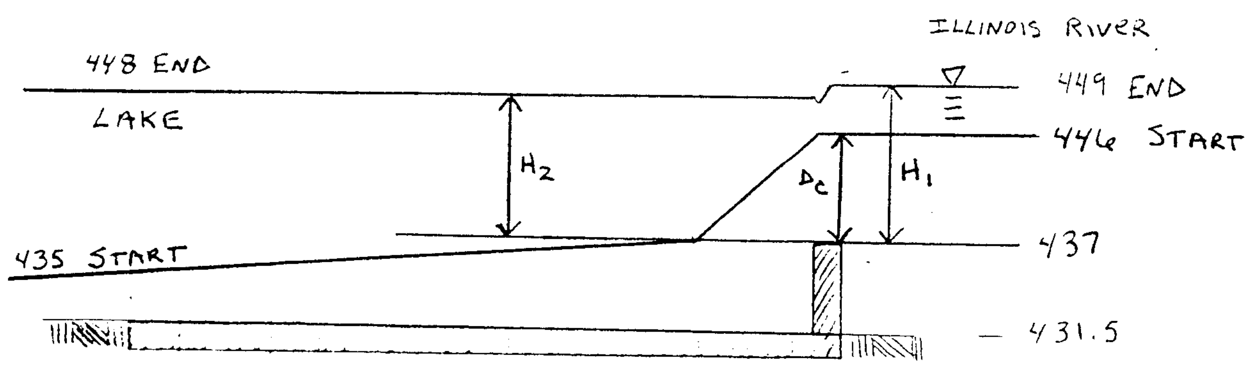
Plate A-1

# LAKE CHAUTAQUA




**ALTERNATIVE 1**

**BROAD-CRESTED WEIR**




**ALTERNATIVE 2**

**SHARP-CRESTED WEIR**

AUGUST 1996
LAKE CHAUTAQUA
SKETCH OF INLET STRUCTURE

Plate A-2

## Lake Chautauqua Control Structures -rectangualr channel modeled as a Broad Crested Weir without contraction coef.

"note: 1) no stop logs or stop log structure. 2) Q=C*L*H^1.5 "

| "H2/H1" | "Cs/Cf" |
|---|---|
| 0 | 1 |
| 0.6 | 0.97 |
| 0.73 | 0.925 |
| 0.78 | 0.875 |
| 0.82 | 0.825 |
| 0.85 | 0.775 |
| 0.875 | 0.725 |
| 0.89 | 0.675 |
| 0.91 | 0.625 |
| 0.93 | 0.575 |
| 0.94 | 0.525 |
| 0.95 | 0.475 |
| 0.96 | 0.425 |
| 0.97 | 0.35 |
| 1 | 0.3 |

**Rectangular Box Culvert**

Variables:

| | |
|---|---|
| 2 | Number of Box Culverts |
| 9.0 | Top Width of Box Culvert Opening (ft) |
| 53 | Length of Box Culvert (ft) |
| 2.66 | Coef. of Discharge see note or HDC711 |
| 435.0 | Starting Elevation of Lake Chautauqua (ft NG |
| 431.5 | Bottom Sill Elevation |
| 446.0 | Illinois River Elevation at time gates are raised |
| 0.00 | Time (hrs) to open gates |
| 1.0 | Illinois River Elevation Increase Rate (0.5-1.0 ft/day suggested) |
| 1.00 | Time Interval of Computation (hours) |
| 0 | Pumping Contribution (0-41000 gpm) |

**Computed:**

| | |
|---|---|
| 3088.3 | Maximum Flow (cfs) |
| 2710.7 | Average Flow (cfs) |
| 16061 | Sum inflow "col H" |
| 19061 | Compute total Lake (Ac-ft) |

| time (hrs) | ilinois Rive Elevation | Upper Lake Chautauqua Elevation | head differential (ft) | Head on Culvert (ft) | Submerged ondition | flow (cfs) | inflow volume (ac-ft) | storage (ac-ft) | | Percent Gate Opening | "H2/H1" | "Cs/Cf" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 446.00 | 435.00 | 11.00 | 14.50 | no | 0.0 | 0.0 | 3000 | 0.0 | 0.0 | 0.24 | 1.00 |
| 1.00 | 446.04 | 435.00 | 11.04 | 14.54 | no | 2655.1 | 109.7 | 3110 | 100.0 | 100.0 | 0.24 | 1 |
| 2.00 | 446.08 | 435.09 | 10.99 | 14.58 | no | 2666.5 | 219.9 | 3330 | 100.0 | 100.0 | 0.25 | 1 |
| 3.00 | 446.13 | 435.27 | 10.85 | 14.63 | no | 2677.9 | 220.8 | 3550 | 100.0 | 100.0 | 0.26 | 1 |
| 4.00 | 446.17 | 435.46 | 10.71 | 14.67 | no | 2689.4 | 221.8 | 3772 | 100.0 | 100.0 | 0.27 | 1 |
| 5.00 | 446.21 | 435.64 | 10.56 | 14.71 | no | 2700.8 | 222.7 | 3995 | 100.0 | 100.0 | 0.28 | 1 |
| 6.00 | 446.25 | 435.83 | 10.42 | 14.75 | no | 2712.3 | 223.7 | 4219 | 100.0 | 100.0 | 0.29 | 1 |
| 7.00 | 446.29 | 436.02 | 10.28 | 14.79 | no | 2723.8 | 224.6 | 4443 | 100.0 | 100.0 | 0.31 | 1 |
| 8.00 | 446.33 | 436.20 | 10.13 | 14.83 | no | 2735.3 | 225.6 | 4669 | 100.0 | 100.0 | 0.32 | 1 |
| 9.00 | 446.38 | 436.39 | 9.98 | 14.88 | no | 2746.9 | 226.5 | 4895 | 100.0 | 100.0 | 0.33 | 1 |
| 10.00 | 446.42 | 436.58 | 9.84 | 14.92 | no | 2758.4 | 227.5 | 5123 | 100.0 | 100.0 | 0.34 | 1 |
| 11.00 | 446.46 | 436.77 | 9.69 | 14.96 | no | 2770.0 | 228.4 | 5351 | 100.0 | 100.0 | 0.35 | 1 |

Plate A-3

| time (hrs) | Ilinois Rive Elevation | Upper Lake Chautauqua Elevation | head differential (ft) | Head on Culvert (ft) | Submerged ondition | flow (cfs) | inflow volume (ac-ft) | storage (ac-ft) | | Percent Gate Opening | "H2/H1" | "Cs/Cf" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12.00 | 446.50 | 436.96 | 9.54 | 15.00 | no | 2781.6 | 229.4 | 5581 | 100.0 | 100.0 | 0.36 | 1 |
| 13.00 | 446.54 | 437.15 | 9.39 | 15.04 | no | 2793.2 | 230.4 | 5811 | 100.0 | 100.0 | 0.38 | 1 |
| 14.00 | 446.58 | 437.34 | 9.24 | 15.08 | no | 2804.8 | 231.3 | 6042 | 100.0 | 100.0 | 0.39 | 1 |
| 15.00 | 446.63 | 437.54 | 9.09 | 15.13 | no | 2816.4 | 232.3 | 6275 | 100.0 | 100.0 | 0.40 | 1 |
| 16.00 | 446.67 | 437.73 | 8.94 | 15.17 | no | 2828.1 | 233.2 | 6508 | 100.0 | 100.0 | 0.41 | 1 |
| 17.00 | 446.71 | 437.92 | 8.79 | 15.21 | no | 2839.7 | 234.2 | 6742 | 100.0 | 100.0 | 0.42 | 1 |
| 18.00 | 446.75 | 438.12 | 8.63 | 15.25 | no | 2851.4 | 235.2 | 6977 | 100.0 | 100.0 | 0.43 | 1 |
| 19.00 | 446.79 | 438.31 | 8.48 | 15.29 | no | 2863.1 | 236.1 | 7213 | 100.0 | 100.0 | 0.45 | 1 |
| 20.00 | 446.83 | 438.51 | 8.32 | 15.33 | no | 2874.8 | 237.1 | 7451 | 100.0 | 100.0 | 0.46 | 1 |
| 21.00 | 446.88 | 438.71 | 8.17 | 15.38 | no | 2886.5 | 238.1 | 7689 | 100.0 | 100.0 | 0.47 | 1 |
| 22.00 | 446.92 | 438.91 | 8.01 | 15.42 | no | 2898.3 | 239.0 | 7928 | 100.0 | 100.0 | 0.48 | 1 |
| 23.00 | 446.96 | 439.11 | 7.85 | 15.46 | no | 2910.0 | 240.0 | 8168 | 100.0 | 100.0 | 0.49 | 1 |
| 24.00 | 447.00 | 439.31 | 7.69 | 15.50 | no | 2921.8 | 241.0 | 8409 | 100.0 | 100.0 | 0.50 | 1 |
| 25.00 | 447.04 | 439.51 | 7.53 | 15.54 | no | 2933.6 | 242.0 | 8651 | 100.0 | 100.0 | 0.52 | 1 |
| 26.00 | 447.08 | 439.71 | 7.37 | 15.58 | no | 2945.4 | 242.9 | 8894 | 100.0 | 100.0 | 0.53 | 1 |
| 27.00 | 447.13 | 439.91 | 7.21 | 15.63 | no | 2957.2 | 243.9 | 9138 | 100.0 | 100.0 | 0.54 | 1 |
| 28.00 | 447.17 | 440.11 | 7.05 | 15.67 | no | 2969.1 | 244.9 | 9382 | 100.0 | 100.0 | 0.55 | 1 |
| 29.00 | 447.21 | 440.32 | 6.89 | 15.71 | no | 2980.9 | 245.9 | 9628 | 100.0 | 100.0 | 0.56 | 1 |
| 30.00 | 447.25 | 440.52 | 6.73 | 15.75 | no | 2992.8 | 246.8 | 9875 | 100.0 | 100.0 | 0.57 | 1 |
| 31.00 | 447.29 | 440.73 | 6.56 | 15.79 | no | 3004.7 | 247.8 | 10123 | 100.0 | 100.0 | 0.58 | 1 |
| 32.00 | 447.33 | 440.94 | 6.40 | 15.83 | no | 3016.6 | 248.8 | 10372 | 100.0 | 100.0 | 0.60 | 1 |
| 33.00 | 447.38 | 441.14 | 6.23 | 15.88 | no | 3028.5 | 249.8 | 10622 | 100.0 | 100.0 | 0.61 | 0.97 |
| 34.00 | 447.42 | 441.35 | 6.07 | 15.92 | no | 3040.4 | 250.8 | 10872 | 100.0 | 100.0 | 0.62 | 0.97 |
| 35.00 | 447.46 | 441.56 | 5.90 | 15.96 | no | 3052.4 | 251.8 | 11124 | 100.0 | 100.0 | 0.63 | 0.97 |
| 36.00 | 447.50 | 441.77 | 5.73 | 16.00 | no | 3064.3 | 252.8 | 11377 | 100.0 | 100.0 | 0.64 | 0.97 |
| 37.00 | 447.54 | 441.98 | 5.56 | 16.04 | no | 3076.3 | 253.7 | 11631 | 100.0 | 100.0 | 0.65 | 0.97 |
| 38.00 | 447.58 | 442.19 | 5.39 | 16.08 | no | 3088.3 | 254.7 | 11885 | 100.0 | 100.0 | 0.66 | 0.97 |
| 39.00 | 447.63 | 442.40 | 5.22 | 16.13 | yes | 3007.3 | 251.9 | 12137 | 100.0 | 100.0 | 0.68 | 0.97 |
| 40.00 | 447.67 | 442.61 | 5.05 | 16.17 | yes | 3019.0 | 249.0 | 12386 | 100.0 | 100.0 | 0.69 | 0.97 |
| 41.00 | 447.71 | 442.82 | 4.89 | 16.21 | yes | 3030.6 | 250.0 | 12636 | 100.0 | 100.0 | 0.70 | 0.97 |
| 42.00 | 447.75 | 443.03 | 4.72 | 16.25 | yes | 3042.3 | 250.9 | 12887 | 100.0 | 100.0 | 0.71 | 0.97 |
| 43.00 | 447.79 | 443.24 | 4.55 | 16.29 | yes | 3054.0 | 251.9 | 13139 | 100.0 | 100.0 | 0.72 | 0.97 |

Plate A-4

| time (hrs) | llinois Rive Elevation | Upper Lake Chautauqua Elevation | head differential (ft) | Head on Culvert (ft) | Submerged ondition | flow (cfs) | inflow volume (ac-ft) | storage (ac-ft) | | Percent Gate Opening | "H2/H1" | "Cs/Cf" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 44.00 | 447.83 | 443.45 | 4.38 | 16.33 | yes | 2923.5 | 247.0 | 13386 | 100.0 | 100.0 | 0.73 | 0.925 |
| 45.00 | 447.88 | 443.66 | 4.22 | 16.38 | yes | 2934.7 | 242.1 | 13628 | 100.0 | 100.0 | 0.74 | 0.925 |
| 46.00 | 447.92 | 443.86 | 4.06 | 16.42 | yes | 2945.9 | 243.0 | 13871 | 100.0 | 100.0 | 0.75 | 0.925 |
| 47.00 | 447.96 | 444.06 | 3.90 | 16.46 | yes | 2957.2 | 243.9 | 14115 | 100.0 | 100.0 | 0.76 | 0.925 |
| 48.00 | 448.00 | 444.26 | 3.74 | 16.50 | yes | 2968.4 | 244.9 | 14360 | 100.0 | 100.0 | 0.77 | 0.925 |
| 49.00 | 448.04 | 444.47 | 3.58 | 16.54 | yes | 2818.6 | 239.1 | 14599 | 100.0 | 100.0 | 0.78 | 0.875 |
| 50.00 | 448.08 | 444.67 | 3.42 | 16.58 | yes | 2829.2 | 233.4 | 14832 | 100.0 | 100.0 | 0.79 | 0.875 |
| 51.00 | 448.13 | 444.86 | 3.26 | 16.63 | yes | 2839.9 | 234.3 | 15067 | 100.0 | 100.0 | 0.80 | 0.875 |
| 52.00 | 448.17 | 445.06 | 3.11 | 16.67 | yes | 2850.6 | 235.1 | 15302 | 100.0 | 100.0 | 0.81 | 0.875 |
| 53.00 | 448.21 | 445.25 | 2.96 | 16.71 | yes | 2697.8 | 229.3 | 15531 | 100.0 | 100.0 | 0.82 | 0.825 |
| 54.00 | 448.25 | 445.44 | 2.81 | 16.75 | yes | 2707.9 | 223.4 | 15755 | 100.0 | 100.0 | 0.83 | 0.825 |
| 55.00 | 448.29 | 445.63 | 2.66 | 16.79 | yes | 2718.0 | 224.2 | 15979 | 100.0 | 100.0 | 0.84 | 0.825 |
| 56.00 | 448.33 | 445.82 | 2.52 | 16.83 | yes | 2562.8 | 218.2 | 16197 | 100.0 | 100.0 | 0.85 | 0.775 |
| 57.00 | 448.38 | 446.00 | 2.38 | 16.88 | yes | 2572.3 | 212.2 | 16409 | 100.0 | 100.0 | 0.86 | 0.775 |
| 58.00 | 448.42 | 446.17 | 2.24 | 16.92 | yes | 2581.8 | 213.0 | 16622 | 100.0 | 100.0 | 0.87 | 0.775 |
| 59.00 | 448.46 | 446.35 | 2.11 | 16.96 | yes | 2424.2 | 206.9 | 16829 | 100.0 | 100.0 | 0.88 | 0.725 |
| 60.00 | 448.50 | 446.52 | 1.98 | 17.00 | yes | 2433.1 | 200.7 | 17030 | 100.0 | 100.0 | 0.88 | 0.725 |
| 61.00 | 448.54 | 446.69 | 1.85 | 17.04 | yes | 2273.7 | 194.5 | 17224 | 100.0 | 100.0 | 0.89 | 0.675 |
| 62.00 | 448.58 | 446.85 | 1.73 | 17.08 | yes | 2282.0 | 188.3 | 17412 | 100.0 | 100.0 | 0.90 | 0.675 |
| 63.00 | 448.63 | 447.01 | 1.61 | 17.13 | yes | 2290.4 | 188.9 | 17601 | 100.0 | 100.0 | 0.91 | 0.675 |
| 64.00 | 448.67 | 447.17 | 1.50 | 17.17 | yes | 2128.4 | 182.6 | 17784 | 100.0 | 100.0 | 0.91 | 0.625 |
| 65.00 | 448.71 | 447.32 | 1.39 | 17.21 | yes | 2136.2 | 176.2 | 17960 | 100.0 | 100.0 | 0.92 | 0.625 |
| 66.00 | 448.75 | 447.47 | 1.28 | 17.25 | yes | 2144.0 | 176.9 | 18137 | 100.0 | 100.0 | 0.93 | 0.625 |
| 67.00 | 448.79 | 447.61 | 1.18 | 17.29 | yes | 1979.6 | 170.4 | 18307 | 100.0 | 100.0 | 0.93 | 0.575 |
| 68.00 | 448.83 | 447.76 | 1.08 | 17.33 | yes | 1986.8 | 163.9 | 18471 | 100.0 | 100.0 | 0.94 | 0.575 |
| 69.00 | 448.88 | 447.89 | 0.98 | 17.38 | yes | 1820.5 | 157.3 | 18629 | 100.0 | 100.0 | 0.94 | 0.525 |
| 70.00 | 448.92 | 448.02 | 0.89 | 17.42 | yes | 1827.1 | 150.7 | 18779 | 100.0 | 100.0 | 0.95 | 0.525 |
| 71.00 | 448.96 | 448.15 | 0.81 | 17.46 | yes | 1659.0 | 144.1 | 18923 | 100.0 | 100.0 | 0.95 | 0.475 |
| 72.00 | 449.00 | 448.27 | 0.73 | 17.50 | yes | 1665.0 | 137.4 | 19061 | 100.0 | 100.0 | 0.96 | 0.475 |

Plate A-5

CSA 01.04.00

Study ID TEST

Struc.ID INLET

| Gravity Outlets (GRAVITY) |

View Computed Gravity Outlet Rating Table

| Flow Capacity (cfs) | Headwater Elevation (ft) No TailWater | TailWater Elev. 1 434.25 | TailWater Elev. 2 437.00 | TailWater Elev. 3 439.75 | TailWater Elev. 4 446.00 | TailWater Elev. 5 447.00 | TailWater Elev. 6 448.00 |
|---|---|---|---|---|---|---|---|
| 0.0 | * 431.50 | 434.25 | 437.00 | 439.75 | 446.00 | 447.00 | 448.00 |
| 150.0 | 433.67 | 434.45 | 437.05 | 439.77 | 446.00 | 447.00 | 448.01 |
| 300.0 | 434.88 | 435.04 | 437.19 | 439.83 | 446.02 | 447.02 | 448.02 |
| 450.0 | 435.90 | 435.90 | 437.43 | 439.94 | 446.06 | 447.05 | 448.05 |
| 600.0 | 436.81 | 436.81 | 437.76 | 440.08 | 446.10 | 447.09 | 448.08 |
| 750.0 | 437.64 | 437.64 | 438.18 | 440.27 | 446.16 | 447.14 | 448.13 |
| 900.0 | 438.43 | 438.43 | 438.68 | 440.49 | 446.23 | 447.21 | 448.18 |
| 1050.0 | 439.16 | 439.16 | 439.25 | 440.76 | 446.32 | 447.28 | 448.25 |
| 1200.0 | 439.87 | 439.87 | 439.88 | 441.07 | 446.42 | 447.37 | 448.33 |
| 1350.0 | 440.55 | 440.55 | 440.55 | 441.42 | 446.53 | 447.47 | 448.41 |
| 1500.0 | 441.20 | 441.20 | 441.20 | 441.80 | 446.66 | 447.58 | 448.51 |
| 1650.0 | 441.83 | 441.83 | 441.83 | 442.22 | 446.79 | 447.70 | 448.62 |
| 1800.0 | 442.44 | 442.44 | 442.44 | 442.67 | 446.95 | 447.83 | 448.73 |
| 1950.0 | 443.03 | 443.03 | 443.03 | 443.16 | 447.11 | 447.97 | 448.86 |
| 2100.0 | 443.61 | 443.61 | 443.61 | 443.66 | 447.29 | 448.13 | 449.00 |
| 2250.0 | 444.18 | 444.18 | 444.18 | 444.19 | 447.48 | 448.29 | 449.14 |
| 2400.0 | 444.73 | 444.73 | 444.73 | 444.73 | 447.68 | 448.47 | 449.30 |
| 2550.0 | 445.27 | 445.27 | 445.27 | 445.27 | 447.90 | 448.66 | 449.47 |
| 2700.0 | 445.80 | 445.80 | 445.80 | 445.80 | 448.13 | 448.86 | 449.64 |
| 2850.0 | 446.33 | 446.33 | 446.33 | 446.33 | 448.37 | 449.07 | 449.83 |

# Rating Curve for Two 9 foot wide Tainter Gates

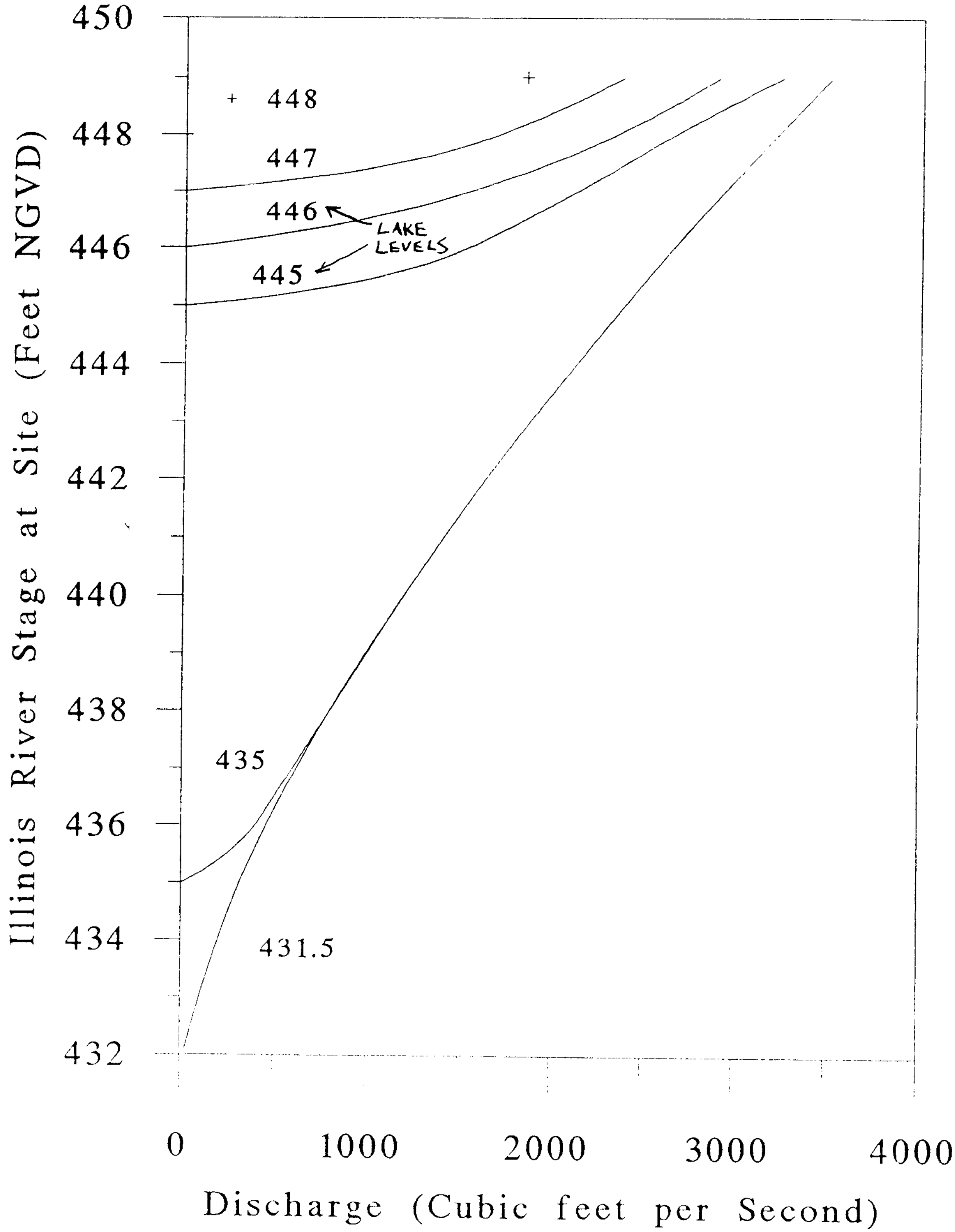
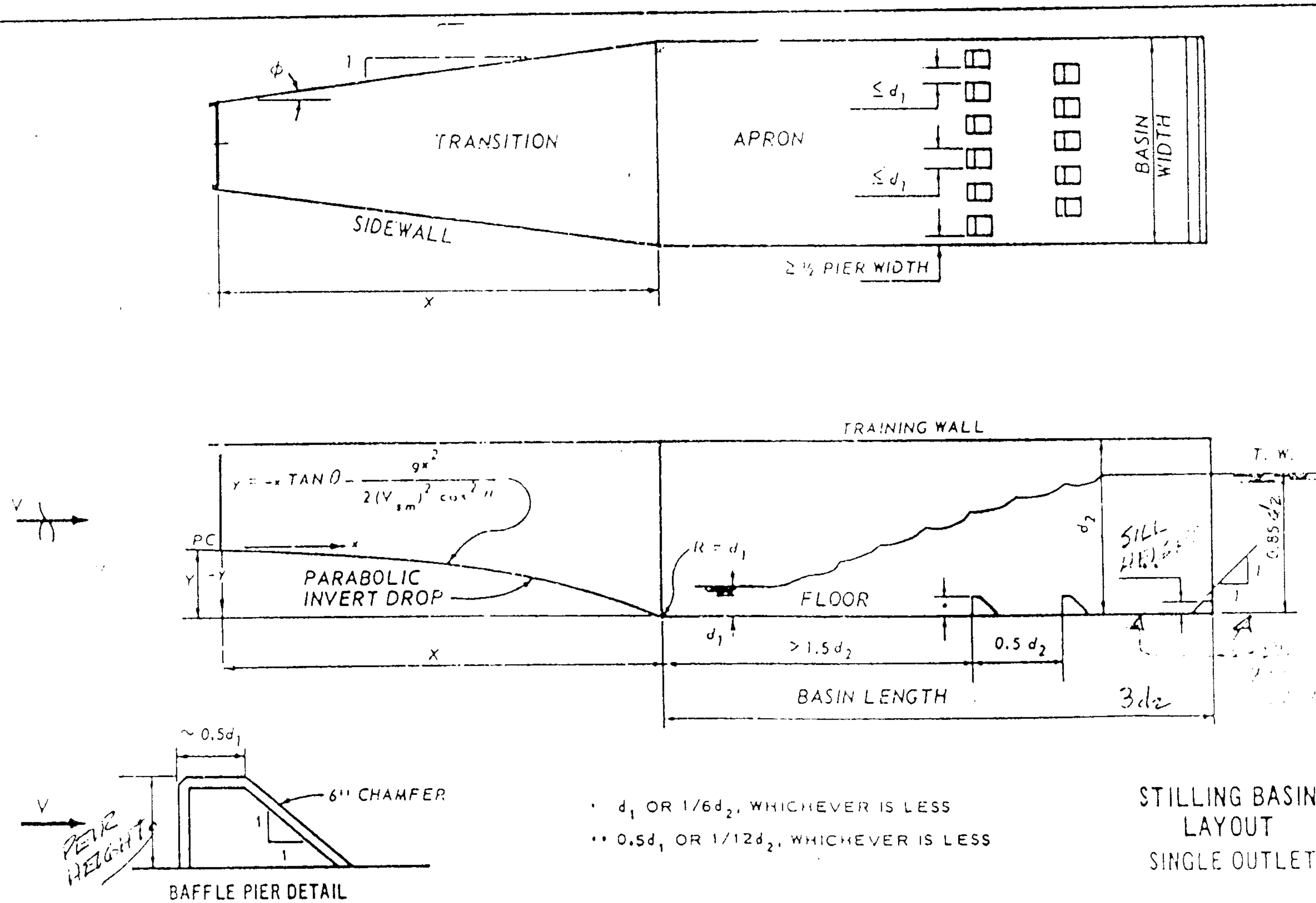



Plate A-7



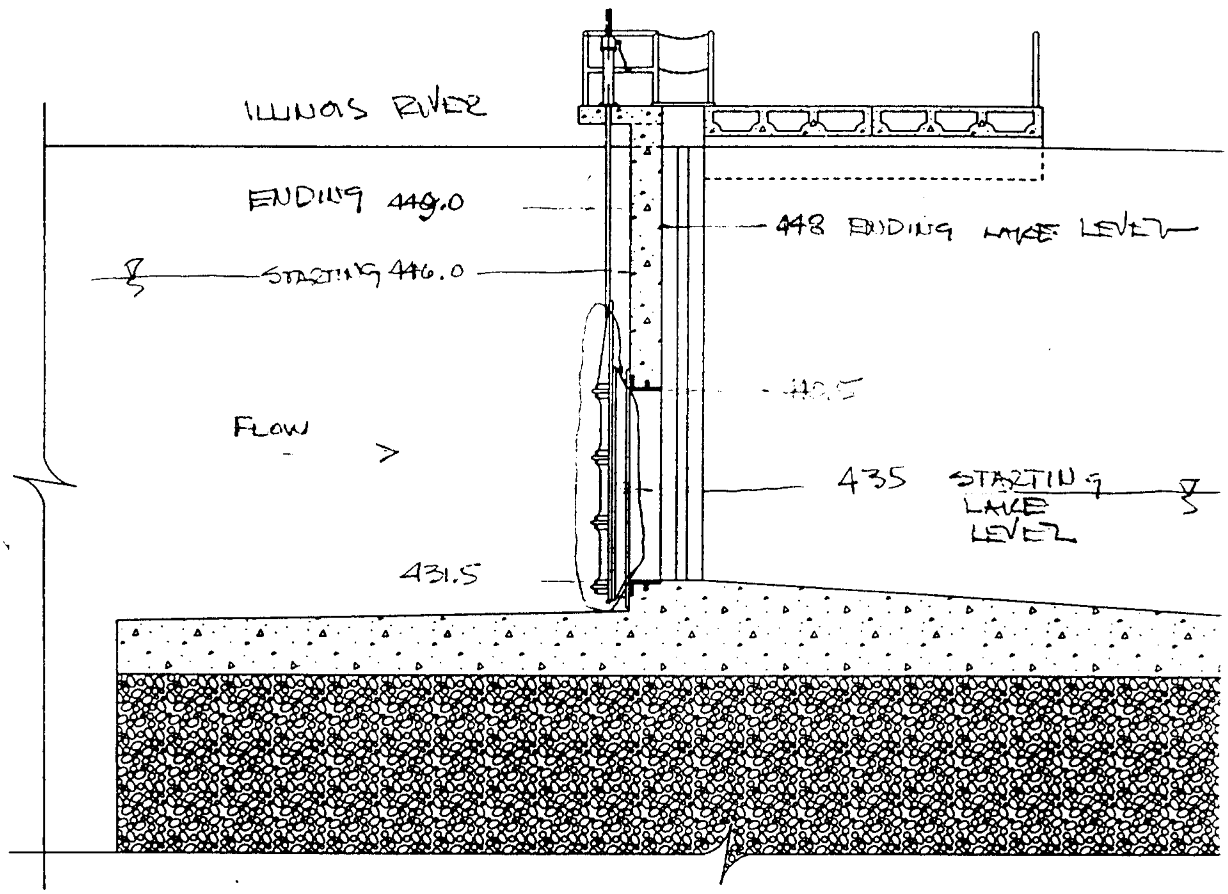

* $d_1$ OR $1/6 d_2$, WHICHEVER IS LESS

** $0.5 d_1$ OR $1/12 d_2$, WHICHEVER IS LESS

STILLING BASIN LAYOUT
SINGLE OUTLET

EM 1110-2-1602
15 Oct 80

CROSS SECTION THROUGH TYPICAL SLIDE GATE INLET STRUCTURE




SECTION A-A

9'x10' Slide Gate
449 ft Levee Crest

Plate A-9

## Lake Chautauqua Control Structures

## as a Gate, free controlled & submerged contolled (orifice)

Target wsel for submergence must be computed and entered

Computed values
Do not alter

90 Area of one Orifice (Sq. Ft.)
440.5 Top Sill of Gate

| Sub Coefficient Head | Cs |
|---|---|
| 0 | 0.83 |
| 0.1 | 0.82 |
| 0.5 | 0.77 |
| 1 | 0.73 |
| 2 | 0.67 |
| 3 | 0.64 |
| 4 | 0.63 |
| 5 | 0.62 |
| 12 | 0.62 |

Assigned Values

| | |
|---|---|
| 10 | Each Gate Width (horizontal in Feet) |
| 9 | Gate Height (vertical in Feet) |
| 3 | Number of Identical Gates |
| 435.0 | Starting Elevation of Lake Chautauqua (ft NGVD) |
| 431.5 | Bottom Sill of Gate |
| 446.0 | Illinois River Elevation at time zero |
| 1.0 | Illinois River Elevation Increase in feet per day (0.5-1.0 ft/day suggested) |
| 1.00 | Time Interval of Computation (hours) |
| 0 | Pumping Contribution (0-41000 gpm) |
| 0.447 | Free Controlled Gate Coefficient |
| 442 | Lake WSEL for submergence |

| time (hrs) | Illinois River Elevation | Upper Lake Chautauqua Elevation | Col B-C Delta WSEL (ft) | Head to bottom of Sill ( Ft.) | Orifice ? | flow (cfs) | inflow volume (ac-ft) | storage (ac-ft) | Orifice Vel (Ft/S) | Orifice C |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 446.0 | 435.0 | 11.0 | 14.5 | no | 0 | 0 | 3000 | | 0.62 |
| 1 | 446.0 | 435.0 | 11.0 | 14.5 | no | 3693 | 153 | 3153 | 13.7 | 0.62 |
| 2 | 446.1 | 435.1 | 11.0 | 14.6 | no | 3699 | 305 | 3458 | 13.7 | 0.62 |
| 3 | 446.1 | 435.4 | 10.7 | 14.6 | no | 3704 | 306 | 3764 | 13.7 | 0.62 |
| 4 | 446.2 | 435.6 | 10.5 | 14.7 | no | 3709 | 306 | 4070 | 13.7 | 0.62 |
| 5 | 446.2 | 435.9 | 10.3 | 14.7 | no | 3714 | 307 | 4377 | 13.8 | 0.62 |
| 6 | 446.3 | 436.1 | 10.1 | 14.8 | no | 3720 | 307 | 4684 | 13.8 | 0.62 |
| 7 | 446.3 | 436.4 | 9.9 | 14.8 | no | 3725 | 308 | 4992 | 13.8 | 0.62 |
| 8 | 446.3 | 436.7 | 9.7 | 14.8 | no | 3730 | 308 | 5300 | 13.8 | 0.62 |
| 9 | 446.4 | 436.9 | 9.5 | 14.9 | no | 3735 | 308 | 5608 | 13.8 | 0.62 |
| 10 | 446.4 | 437.2 | 9.2 | 14.9 | no | 3741 | 309 | 5917 | 13.9 | 0.62 |
| 11 | 446.5 | 437.4 | 9.0 | 15.0 | no | 3746 | 309 | 6227 | 13.9 | 0.62 |
| 12 | 446.5 | 437.7 | 8.8 | 15.0 | no | 3751 | 310 | 6537 | 13.9 | 0.62 |
| 13 | 446.5 | 437.9 | 8.6 | 15.0 | no | 3756 | 310 | 6847 | 13.9 | 0.62 |
| 14 | 446.6 | 438.2 | 8.4 | 15.1 | no | 3762 | 311 | 7157 | 13.9 | 0.62 |
| 15 | 446.6 | 438.5 | 8.2 | 15.1 | no | 3767 | 311 | 7468 | 14.0 | 0.62 |
| 16 | 446.7 | 438.7 | 7.9 | 15.2 | no | 3772 | 312 | 7780 | 14.0 | 0.62 |
| 17 | 446.7 | 439.0 | 7.7 | 15.2 | no | 3777 | 312 | 8092 | 14.0 | 0.62 |
| 18 | 446.8 | 439.2 | 7.5 | 15.3 | no | 3782 | 312 | 8404 | 14.0 | 0.62 |
| 19 | 446.8 | 439.5 | 7.3 | 15.3 | no | 3787 | 313 | 8717 | 14.0 | 0.62 |
| 20 | 446.8 | 439.8 | 7.1 | 15.3 | no | 3793 | 313 | 9030 | 14.0 | 0.62 |
| 21 | 446.9 | 440.0 | 6.8 | 15.4 | no | 3798 | 314 | 9344 | 14.1 | 0.62 |
| 22 | 446.9 | 440.3 | 6.6 | 15.4 | no | 3803 | 314 | 9658 | 14.1 | 0.62 |
| 23 | 447.0 | 440.5 | 6.4 | 15.5 | no | 3808 | 314 | 9973 | 14.1 | 0.62 |



Plate A-10

| time (hrs) | Illinois River Elevation | Chautauqua Elevation | WSEL (ft) | of Sill ( Ft.) | Orifice ? | flow (cfs) | volume (ac-ft) | storage (ac-ft) | Vel (Ft/S) | Orifice C |
|---|---|---|---|---|---|---|---|---|---|---|
| 24 | 447.0 | 440.8 | 6.2 | 15.5 | no | 3813 | 315 | 10287 | 14.1 | 0.62 |
| 25 | 447.0 | 441.1 | 6.0 | 15.5 | no | 3818 | 315 | 10603 | 14.1 | 0.62 |
| 26 | 447.1 | 441.3 | 5.7 | 15.6 | no | 3823 | 316 | 10919 | 14.2 | 0.62 |
| 27 | 447.1 | 441.6 | 5.5 | 15.6 | no | 3828 | 316 | 11235 | 14.2 | 0.62 |
| 28 | 447.2 | 441.9 | 5.3 | 15.7 | no | 3834 | 317 | 11551 | 14.2 | 0.62 |
| 29 | 447.2 | 442.1 | 5.1 | 15.7 | yes | 3028 | 284 | 11835 | 11.2 | 0.62 |
| 30 | 447.3 | 442.4 | 4.9 | 15.8 | yes | 3018 | 250 | 12085 | 11.2 | 0.63 |
| 31 | 447.3 | 442.6 | 4.7 | 15.8 | yes | 2966 | 247 | 12332 | 11.0 | 0.63 |
| 32 | 447.3 | 442.8 | 4.6 | 15.8 | yes | 2914 | 243 | 12575 | 10.8 | 0.63 |
| 33 | 447.4 | 443.0 | 4.4 | 15.9 | yes | 2862 | 239 | 12814 | 10.6 | 0.63 |
| 34 | 447.4 | 443.2 | 4.2 | 15.9 | yes | 2810 | 234 | 13048 | 10.4 | 0.63 |
| 35 | 447.5 | 443.4 | 4.1 | 16.0 | yes | 2759 | 230 | 13278 | 10.2 | 0.63 |
| 36 | 447.5 | 443.6 | 3.9 | 16.0 | yes | 2751 | 228 | 13506 | 10.2 | 0.64 |
| 37 | 447.5 | 443.8 | 3.8 | 16.0 | yes | 2698 | 225 | 13731 | 10.0 | 0.64 |
| 38 | 447.6 | 443.9 | 3.6 | 16.1 | yes | 2646 | 221 | 13952 | 9.8 | 0.64 |
| 39 | 447.6 | 444.1 | 3.5 | 16.1 | yes | 2594 | 217 | 14168 | 9.6 | 0.64 |
| 40 | 447.7 | 444.3 | 3.4 | 16.2 | yes | 2542 | 212 | 14381 | 9.4 | 0.64 |
| 41 | 447.7 | 444.5 | 3.2 | 16.2 | yes | 2490 | 208 | 14589 | 9.2 | 0.64 |
| 42 | 447.8 | 444.7 | 3.1 | 16.3 | yes | 2439 | 204 | 14792 | 9.0 | 0.64 |
| 43 | 447.8 | 444.8 | 3.0 | 16.3 | yes | 2500 | 204 | 14996 | 9.3 | 0.67 |
| 44 | 447.8 | 445.0 | 2.8 | 16.3 | yes | 2445 | 204 | 15201 | 9.1 | 0.67 |
| 45 | 447.9 | 445.2 | 2.7 | 16.4 | yes | 2389 | 200 | 15400 | 8.8 | 0.67 |
| 46 | 447.9 | 445.3 | 2.6 | 16.4 | yes | 2333 | 195 | 15595 | 8.6 | 0.67 |
| 47 | 448.0 | 445.5 | 2.5 | 16.5 | yes | 2278 | 191 | 15786 | 8.4 | 0.67 |
| 48 | 448.0 | 445.7 | 2.3 | 16.5 | yes | 2223 | 186 | 15972 | 8.2 | 0.67 |
| 49 | 448.0 | 445.8 | 2.2 | 16.5 | yes | 2169 | 181 | 16153 | 8.0 | 0.67 |
| 50 | 448.1 | 446.0 | 2.1 | 16.6 | yes | 2115 | 177 | 16331 | 7.8 | 0.67 |
| 51 | 448.1 | 446.1 | 2.0 | 16.6 | yes | 2061 | 173 | 16503 | 7.6 | 0.67 |
| 52 | 448.2 | 446.3 | 1.9 | 16.7 | yes | 2188 | 176 | 16679 | 8.1 | 0.73 |
| 53 | 448.2 | 446.4 | 1.8 | 16.7 | yes | 2128 | 178 | 16857 | 7.9 | 0.73 |
| 54 | 448.3 | 446.5 | 1.7 | 16.8 | yes | 2064 | 173 | 17030 | 7.6 | 0.73 |
| 55 | 448.3 | 446.7 | 1.6 | 16.8 | yes | 2001 | 168 | 17198 | 7.4 | 0.73 |
| 56 | 448.3 | 446.8 | 1.5 | 16.8 | yes | 1938 | 163 | 17361 | 7.2 | 0.73 |
| 57 | 448.4 | 447.0 | 1.4 | 16.9 | yes | 1877 | 158 | 17519 | 7.0 | 0.73 |
| 58 | 448.4 | 447.1 | 1.3 | 16.9 | yes | 1816 | 153 | 17671 | 6.7 | 0.73 |
| 59 | 448.5 | 447.2 | 1.2 | 17.0 | yes | 1756 | 148 | 17819 | 6.5 | 0.73 |
| 60 | 448.5 | 447.3 | 1.2 | 17.0 | yes | 1697 | 143 | 17961 | 6.3 | 0.73 |
| 61 | 448.5 | 447.5 | 1.1 | 17.0 | yes | 1639 | 138 | 18099 | 6.1 | 0.73 |
| 62 | 448.6 | 447.6 | 1.0 | 17.1 | yes | 1582 | 133 | 18232 | 5.9 | 0.73 |
| 63 | 448.6 | 447.7 | 0.9 | 17.1 | yes | 1610 | 132 | 18364 | 6.0 | 0.77 |
| 64 | 448.7 | 447.8 | 0.9 | 17.2 | yes | 1550 | 131 | 18495 | 5.7 | 0.77 |
| 65 | 448.7 | 447.9 | 0.8 | 17.2 | yes | 1488 | 126 | 18620 | 5.5 | 0.77 |
| 66 | 448.8 | 448.0 | 0.7 | 17.3 | yes | 1428 | 121 | 18741 | 5.3 | 0.77 |
| 67 | 448.8 | 448.1 | 0.7 | 17.3 | yes | 1370 | 116 | 18857 | 5.1 | 0.77 |
| 68 | 448.8 | 448.2 | 0.6 | 17.3 | yes | 1313 | 111 | 18967 | 4.9 | 0.77 |
| 69 | 448.9 | 448.3 | 0.6 | 17.4 | yes | 1258 | 106 | 19074 | 4.7 | 0.77 |
| 70 | 448.9 | 448.4 | 0.5 | 17.4 | yes | 1205 | 102 | 19176 | 4.5 | 0.77 |
| 71 | 449.0 | 448.5 | 0.5 | 17.5 | yes | 1229 | 101 | 19276 | 4.6 | 0.82 |
| 72 | 449.0 | 448.6 | 0.4 | 17.5 | yes | 1174 | 99 | 19375 | 4.3 | 0.82 |

# Rating Curve for Three Sluce Gates

Each 9 ft. vertical 10 feet horizontal

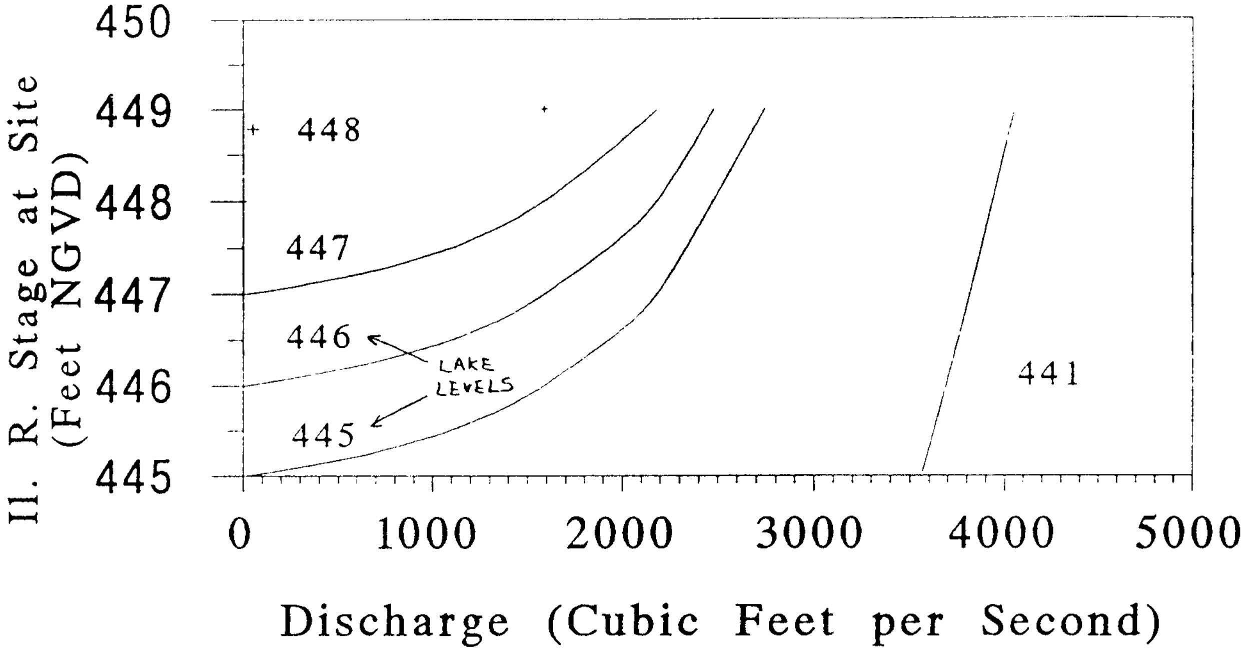



Plate A-12

**Lake Chautauqua**

Gates are opened at an Illinois River Elevation of 444.0 ft (Measured at Lake Chautauqua)

1) 3 10' x 10' Slide Gates
2) Levee Crest @ 447.5
3) 3000 ft Controlled Overflow Section Crest @ 447.0
(Located from STA 25+00 to STA 55+00)
4) No Pumping Contribution
5) Illinois River Rises at 1.3 ft/day assumed

| Raise Gates at River Elevation (ft NGVD) | Initial Lake Elevation when Gates Opened (ft NGVD) | Head Differential at River Elev: 447.0 (ft) | 447.25 (ft) | 447.5 (ft) | Time to Overtop Overflow Section (hrs) | Time to Overtop Levee (hrs) |
|---|---|---|---|---|---|---|
| 444 | 431 (Dry Lake) | 1.34 | 0.91 | 0.15 | 55.5 | 64.75 |
| 444 | 432 | 1.24 | 0.82 | 0.10 | 55.5 | 64.75 |
| 444 | 433 | 1.03 | 0.65 | 0.00 | 55.5 | 64.75 |
| 444 | 434 | 0.77 | 0.46 | 0.00 | 55.5 | 64.75 |
| 444 | 435 | 0.55 | 0.30 | 0.00 | 55.5 | 64.75 |
| 444 | 436 | 0.37 | 0.19 | 0.00 | 55.5 | 64.75 |
| 444 | 437 | 0.26 | 0.13 | 0.00 | 55.5 | 64.75 |
| 444 | 438 | 0.21 | 0.10 | 0.00 | 55.5 | 64.75 |
| 444 | 439 | 0.18 | 0.09 | 0.00 | 55.5 | 64.75 |
| 444 | 440 | 0.17 | 0.09 | 0.00 | 55.5 | 64.75 |
| 444 | 441 | 0.16 | 0.09 | 0.00 | 55.5 | 64.75 |

**Lake Chautauqua**
Gates are opened at an Illinois River Elevation of 444.5 ft (Measured at Lake Chautauqua)

1) 3 10' x 10' Slide Gates
2) Levee Crest @ 447.5
3) 3000 ft Controlled Overflow Section Crest @ 447.0
   (Located from STA 25+00 to STA 55+00)
4) No Pumping Contribution
5) Illinois River Rises at 1.3 ft/day assumed

| Raise Gates at River Elevation (ft NGVD) | Initial Lake Elevation when Gates Opened (ft NGVD) | Head Differential at River Elev: 447.0 (ft) | 447.25 (ft) | 447.5 (ft) | Time to Overtop Overflow Section (hrs) | Time to Overtop Levee (hrs) |
|---|---|---|---|---|---|---|
| 444.5 | 431 (Dry Lake) | 2.59 | 1.98 | 0.98 | 46.25 | 55.5 |
| 444.5 | 432 | 2.45 | 1.86 | 0.88 | 46.25 | 55.5 |
| 444.5 | 433 | 2.15 | 1.58 | 0.66 | 46.25 | 55.5 |
| 444.5 | 434 | 1.73 | 1.22 | 0.39 | 46.25 | 55.5 |
| 444.5 | 435 | 1.32 | 0.89 | 0.15 | 46.25 | 55.5 |
| 444.5 | 436 | 0.95 | 0.59 | 0.00 | 46.25 | 55.5 |
| 444.5 | 437 | 0.66 | 0.39 | 0.00 | 46.25 | 55.5 |
| 444.5 | 438 | 0.45 | 0.23 | 0.00 | 46.25 | 55.5 |
| 444.5 | 439 | 0.31 | 0.16 | 0.00 | 46.25 | 55.5 |
| 444.5 | 440 | 0.23 | 0.12 | 0.00 | 46.25 | 55.5 |
| 444.5 | 441 | 0.19 | 0.10 | 0.00 | 46.25 | 55.5 |

**' ake Chautauqua**

ɜates are opened at an Illinois River Elevation of 445.0 ft (Measured at Lake Chautauqua)

1) 3 10' x 10' Slide Gates
2) Levee Crest @ 447.5
3) 3000 ft Controlled Overflow Section Crest @ 447.0
(Located from STA 25+00 to STA 55+00)
4) No Pumping Contribution
5) Illinois River Rises at 1.3 ft/day assumed

| Raise Gates at River Elevation (ft NGVD) | Initial Lake Elevation when Gates Opened (ft NGVD) | Head Differential at River Elev: 447.0 (ft) | 447.25 (ft) | 447.5 (ft) | Time to Overtop Overflow Section (hrs) | Time to Overtop Levee (hrs) |
|---|---|---|---|---|---|---|
| 445 | 431 (Dry Lake) | 4.24 | 3.47 | 2.30 | 37 | 46.25 |
| 445 | 432 | 4.05 | 3.30 | 2.14 | 37 | 46.25 |
| 445 | 433 | 3.66 | 2.95 | 1.83 | 37 | 46.25 |
| 445 | 434 | 3.13 | 2.47 | 1.41 | 37 | 46.25 |
| 445 | 435 | 2.59 | 1.99 | 0.99 | 37 | 46.25 |
| 445 | 436 | 2.05 | 1.49 | 0.60 | 37 | 46.25 |
| 445 | 437 | 1.54 | 1.08 | 0.28 | 37 | 46.25 |
| 445 | 438 | 1.12 | 0.74 | 0.05 | 37 | 46.25 |
| 445 | 439 | 0.79 | 0.48 | 0.00 | 37 | 46.25 |
| 445 | 440 | 0.54 | 0.31 | 0.00 | 37 | 46.25 |
| 445 | 441 | 0.36 | 0.19 | 0.00 | 37 | 46.25 |

**ake Chautauqua**

Gates are opened at an Illinois River Elevation of 445.5 ft (Measured at Lake Chautauqua)

1) 3 10' x 10' Slide Gates
2) Levee Crest @ 447.5
3) 3000 ft Controlled Overflow Section Crest @ 447.0
(Located from STA 25+00 to STA 55+00)
4) No Pumping Contribution
5) Illinois River Rises at 1.3 ft/day assumed

| Raise Gates at River Elevation (ft NGVD) | Initial Lake Elevation when Gates Opened (ft NGVD) | Head Differential at River Elev: 447.0 (ft) | 447.25 (ft) | 447.5 (ft) | Time to Overtop Overflow Section (hrs) | Time to Overtop Levee (hrs) |
|---|---|---|---|---|---|---|
| 445.5 | 431 (Dry Lake) | 6.66 | 5.50 | 4.02 | 27.75 | 37 |
| 445.5 | 432 | 6.41 | 5.25 | 3.83 | 27.75 | 37 |
| 445.5 | 433 | 5.85 | 4.73 | 3.42 | 27.75 | 37 |
| 445.5 | 434 | 5.04 | 4.11 | 2.87 | 27.75 | 37 |
| 445.5 | 435 | 4.28 | 3.50 | 2.32 | 27.75 | 37 |
| 445.5 | 436 | 3.57 | 2.87 | 1.76 | 27.75 | 37 |
| 445.5 | 437 | 2.93 | 2.28 | 1.24 | 27.75 | 37 |
| 445.5 | 438 | 2.34 | 1.76 | 0.80 | 27.75 | 37 |
| 445.5 | 439 | 1.82 | 1.30 | 0.44 | 27.75 | 37 |
| 444.5 | 440 | 1.34 | 0.92 | 0.16 | 27.75 | 37 |
| 445.5 | 441 | 0.97 | 0.62 | 0.00 | 27.75 | 37 |

**ake Chautauqua**

Gates are opened at an Illinois River Elevation of 446.0 ft (Measured at Lake Chautauqua)

1) 3 10' x 10' Slide Gates
2) Levee Crest @ 447.5
3) 3000 ft Controlled Overflow Section Crest @ 447.0
   (Located from STA 25+00 to STA 55+00)
4) No Pumping Contribution
5) Illinois River Rises at 1.3 ft/day assumed

| Raise Gates at River Elevation (ft NGVD) | Initial Lake Elevation when Gates Opened (ft NGVD) | Head Differential at River Elev: 447.0 (ft) | 447.25 (ft) | 447.5 (ft) | Time to Overtop Overflow Section (hrs) | Time to Overtop Levee (hrs) |
|---|---|---|---|---|---|---|
| 446 | 431 (Dry Lake) | 9.76 | 8.11 | 6.41 | 18.5 | 27.75 |
| 446 | 432 | 9.02 | 7.86 | 6.16 | 18.5 | 27.75 |
| 446 | 433 | 8.45 | 7.29 | 5.60 | 18.5 | 27.75 |
| 446 | 434 | 7.65 | 6.44 | 4.83 | 18.5 | 27.75 |
| 446 | 435 | 6.76 | 5.60 | 4.10 | 18.5 | 27.75 |
| 446 | 436 | 5.76 | 4.66 | 3.36 | 18.5 | 27.75 |
| 446 | 437 | 4.76 | 3.92 | 2.70 | 18.5 | 27.75 |
| 446 | 438 | 3.99 | 3.24 | 2.09 | 18.5 | 27.75 |
| 446 | 439 | 3.31 | 2.63 | 1.55 | 18.5 | 27.75 |
| 446 | 440 | 2.69 | 2.08 | 1.07 | 18.5 | 27.75 |
| 446 | 441 | 2.14 | 1.58 | 0.66 | 18.5 | 27.75 |

**ake Chautauqua**
ates are opened at an Illinois River Elevation of 446.5 ft (Measured at Lake Chautauqua)

1) 3 10' x 10' Slide Gates
2) Levee Crest @ 447.5
3) 3000 ft Controlled Overflow Section Crest @ 447.0
   (Located from STA 25+00 to STA 55+00)
4) No Pumping Contribution
5) Illinois River Rises at 1.3 ft/day assumed

| Raise Gates at River Elevation (ft NGVD) | Initial Lake Elevation when Gates Opened (ft NGVD) | Head Differential at River Elev: 447.0 (ft) | 447.25 (ft) | 447.5 (ft) | Time to Overtop Overflow Section (hrs) | Time to Overtop Levee (hrs) |
|---|---|---|---|---|---|---|
| 446.5 | 431 (Dry Lake) | 11.92 | 10.68 | 9.06 | 9.25 | 18.5 |
| 446.5 | 432 | 11.67 | 10.43 | 8.81 | 9.25 | 18.5 |
| 446.5 | 433 | 11.10 | 9.87 | 8.25 | 9.25 | 18.5 |
| 446.5 | 434 | 10.30 | 9.66 | 7.45 | 9.25 | 18.5 |
| 446.5 | 435 | 9.41 | 8.18 | 6.56 | 9.25 | 18.5 |
| 446.5 | 436 | 8.41 | 7.18 | 5.56 | 9.25 | 18.5 |
| 446.5 | 437 | 7.41 | 6.18 | 4.64 | 9.25 | 18.5 |
| 446.5 | 438 | 6.41 | 5.18 | 3.84 | 9.25 | 18.5 |
| 446.5 | 439 | 5.41 | 4.33 | 3.12 | 9.25 | 18.5 |
| 446.5 | 440 | 4.46 | 3.64 | 2.48 | 9.25 | 18.5 |
| 446.5 | 441 | 3.74 | 2.98 | 1.90 | 9.25 | 18.5 |

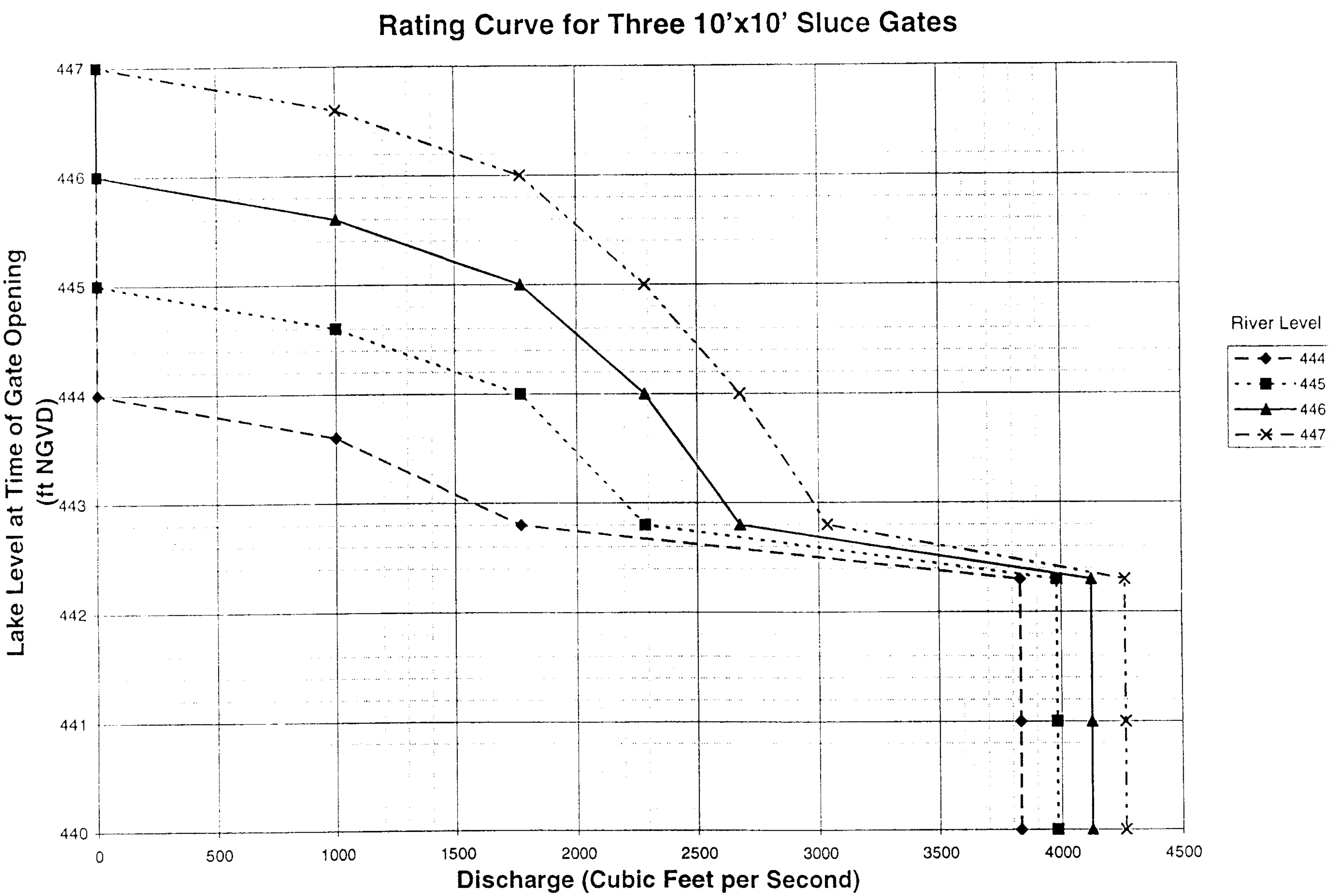

Rating Curve for Three 10'x10' Sluce Gates
Lake Level at Time of Gate Opening (ft NGVD)
447
446
445
444
443
442
441
440
0
500
1000
1500
2000
2500
3000
3500
4000
4500
Discharge (Cubic Feet per Second)
River Level
444
445
446
447

Plate A-19

Table 8. Maximum permissible velocities for vegetative linings.[27,28]

| Cover | Slope Range (%) | Permissible Velocity, Erosion-Resistant Soils ft/s | Permissible Velocity, Easily-Eroded Soils ft/s |
|---|---|---|---|
| Bermuda Grass | 0-5 | 8 | 6 |
| | 5-10 | 7 | 5 |
| | >10 | 6 | 4 |
| Buffalo Grass, Kentucky Bluegrass, Smooth Brome, Blue Grama | 0-5 | 7 | 5 |
| | 5-10 | 6 | 4 |
| | >10 | 5 | 3 |
| Grass Mixture | 0-5 | 5 | 4 |
| | 5-10 | 4 | 3 |
| | Do not use on slopes steeper than 10%. | | |
| Lespedeza Sericea, Weeping Love Grass, Ischaemum (yellow bluestem), Kudzu, Alfalfa, Crabgrass | 0-5 | 3.5 | 2.5 |
| | Do not use on slopes steeper than 5% except for sideslopes in a combination channel. | | |
| Annual-used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan Grass | 0-5 | 3.5 | 2.5 |
| | Use on slopes steeper than 5% is not recommended. | | |

ft/s x 0.3048 = m/s

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR
LA GRANGE POOL, ILLINOIS WATERWAY, RIVER MILES 124-129
MASON COUNTY, ILLINOIS

# APPENDIX B

# GEOTECHNICAL APPENDIX

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR
LA GRANGE POOL, ILLINOIS WATERWAY, RIVER MILES 124 -129
MASON COUNTY, ILLINOIS

# APPENDIX B
# GEOTECHNICAL CONSIDERATIONS

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## LIST OF PLATES

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR
LA GRANGE POOL, ILLINOIS WATERWAY, RIVER MILES 124 -129
MASON COUNTY, ILLINOIS

# APPENDIX B
# GEOTECHNICAL CONSIDERATIONS

## 1. PURPOSE AND SCOPE.

a. This appendix presents the general geology and specific geotechnical analysis pertinent to the project. This appendix will present the geotechnical information and analysis specific to the breach repair and water control structure replacement at the Chautauqua National Wildlife Refuge. The refuge is owned and operated by the U.S. Fish & Wildlife Service. The purpose of this report is to provide information on the repair of 1996 flood damage suffered at the Chautauqua National Wildlife Refuge. An analysis of several alternatives was completed to provide a cost effective repair that will perform the functions required by the U. S. Fish & Wildlife Service.

b. On June 1, 1996, during a flood event, the 60 year old radial gate structure failed. A contract was awarded to Rust Environmental & Infrastructure to perform an investigation and report on the cause of failure. The conclusion of the report is that riverside erosion near the structure led to levee failure adjacent to the radial gate structure. The breach caused erosional flow around and under the structure and caused the structure to fail. This led to erosion of the levee and foundation down to bedrock. The approximate size of the breach is 350 feet long and approximately 30 feet deep (from the existing ground surface). This report describes the proposed repairs that include a steel sheet pile cellular water control structure and breach repair.

## 2. LOCATION.

The Chautauqua National Wildlife Refuge, established in 1936 and administered by the U.S. Fish and Wildlife Service, Department of the Interior, is a wintering waterfowl refuge located within the Mississippi Flyway, which extends from Canada to the Gulf of Mexico. The refuge is situated in Mason County in central Illinois and contains 4,200 acres of land and water within the Illinois River floodplain. Lake Chautauqua impounds about 3,800 acres of water, while another 400 acres of water and timbered bottom land are located outside of the impounded area. The remaining acreage is composed of upland and timber.

The refuge is bounded on the West by the Illinois River from river miles 124 to 128. Adjacent on the north, south, and east ends are shallow floodplain lakes similar to Lake Chautauqua. The east boundary is a sandy bluff, rising 70 feet above the lake with wave-cut and nearly vertical faces.

3. PHYSIOGRAPHY.

a. The project is situated within the Central Lowland Province of the Galesburg Plain, a region of deeply dissected Illinoian glacial plains. The narrow, gentle, and wavelike appearance of the upland areas, interspersed by a maze of deep, sharp valleys, contrasts with the flat expanses of the Illinois Valley and its major tributary in this area, the Spoon Valley.

b. The most prominent topographic feature, the Illinois Valley, is 17 to 20 miles wide in the vicinity. This portion of the valley forms part of the Havana Lowland, a low, broad, and triangular alluvial plain that extends from Pekin to Beardstown, Illinois. The valley is bordered by steep, 80 to 150 foot-high bluffs on the Northwest. East of the river, the valley bottom is covered by sand ridges and dunes 20 to 40 feet high.

4. PLEISTOCENE AND RECENT DEPOSITS.

a. The area was glaciated during the Pre-Illinoian and Illinoian stages of the Pleistocene that took place approximately 10,000 to 900,000 years ago. Glacial deposits of till, sand, and gravel outwash average about 50 feet in thickness. Locally these deposits may be as thick as 150 feet over buried bedrock valleys.

b. The Pre-Illinoian glacier completely covered the area, and its deposits are widespread beneath younger drift and are rarely exposed. The Illinoian glacier deposited Illinoian drift during three separate advances that extensively underlie the uplands and are exposed in many places. Westerly winds, depositing loess during the Wisconsinan time and sand in recent times, formed surficial material in the bluffs throughout the area. Alluvial river and stream deposits of mostly clay and silt with some sand and fine gravels are the most recent deposits overlying glacial outwash. This material ranges from 15 to 20 feet in thickness.

5. BEDROCK.

The bedrock of the project area consists of layers, totaling approximately 4,500-feet of Paleozoic sedimentary rocks that range in age from late Cambrian to middle Pennsylvanian. The Cambrian rocks rest on an ancient erosion surface of Pre-Cambrian granite. Thick deposits of sedimentary rocks in the basin, consisting of Pennsylvanian age sandstone, shale, limestone, and coal, where deposited in the ancient shallow seas and marshes that periodically covered Illinois, including the Lake Chautauqua area, during the Paleozoic Era. The depth to bedrock in the project area ranges from 50 to approximately 150 feet and is of the Spoon Formation.

## 6. SUBSURFACE CONDITIONS

One boring (LC-89-2)was completed in 1989 to evaluate the foundation of the existing radial gate. Due to the failure of the radial gate, two new sites were investigated for construction of a new water control structure. Site One was the Melz Slough location and site Two was the Goofy Ridge location. The boring (LC-96-4) completed at the Melz Slough site shows a medium to fat (CL-CH to CH) clay down to elevation 416. Below this was a clayey sand (SC) to bedrock at elevation 405. The boring (LC-96-5) completed at the Goofy Ridge site shows a medium to fine sand (SP) down to elevation 439. Below this is 17 feet (elevation 439-422) of medium to fat (CL-CH to CH) clay with occasional sand and organic layers. From elevation 422 to 414 is a clayey medium to fine sand (SP-SC) with medium to fine sand (SP) to bedrock, encountered at elevation 405. Due to the design of the selected alternative (sheet pile founded on rock), either site One or site Two could be utilized. However the Goofy Ridge site was selected for hydraulic conditions and better access for operation. The selected plan provides a closure for the breach, eliminating the need for a specific breach closure plan as well as providing a gated water control structure. The boring logs are shown on plate 3 of the main report.

## 7. ALTERNATIVES CONSIDERED.

### a. BREACH REPAIR.

The general scheme to repair the breach would be to place sand below the water level and to elevation 435 MSL. The sand would be trucked in from off-site and end-dumped. Once the elevation of the sand fill is above the water elevation, pumps would be needed to keep the sand in a saturated condition. The sand would be "tracked in" and shaped with bulldozers. This would be used as a base to construct a clay embankment with a sand berm. The clay cap would be constructed on this sand base. The clay would come from on-site borrow and would be constructed in 6 inch lifts, compacted to 95% standard proctor dry density at 2% +/- of optimum moisture content. After the clay embankment was completed a sand berm would be constructed on the lake side of the clay embankment. The sand berm is needed to control uplift pressures. The sand berm would be 7.5 feet thick and extend a minimum of 30 feet from the clay embankment. The sand berm would be tied in a minimum of 50 feet past the existing embankment. This levee would be protected from wave erosion by riprap. The 18 inch thick riprap layer would be placed on a 6 inch layer of bedding stone. The levee repair in the breach area would be constructed to an elevation 2 feet higher than the rest of the levee. This would ensure overtopping will occur along the rest of the levee before the area of the breach repair.

### b. TAINTER GATE CONCRETE STRUCTURE.

(1). As mentioned above two possible locations were considered for the new Tainter gate structure. Site One at the Melz Slough location and site Two at the Goofy Ridge location which is adjacent to the location of the previous radial gate. Due to the decision to use a pile foundation driven to bedrock, the subsurface conditions had little impact on the site selection. The Goofy Ridge site was selected due to access requirements and hydraulic conditions.

(2). The structure would be founded on bearing piles, driven to bedrock to prevent any settlement. The structure would have a sheetpile cutoff driven to bedrock to prevent any underseepage related distress. This sheetpile cutoff would extend into the levee on either side to prevent seepage around the structure. The lakeside outlet for the structure would have a stilling basin with baffle piers and a riprap blanket past the end of the stilling basin to prevent erosional distress. All backfill for the structure would be clay compacted to 95% standard proctor, dry density at +/- 2% of optimum moisture content.

(3). A likely dewatering scenario for construction of the Tainter gate would be to construct a braced sheetpile cofferdam driven to bedrock. Sandpoints would most likely be used to control groundwater, although it maybe possible to control the groundwater with pumps set in pits in the bottom of the excavation. This option would be combined with the breach repair to provide water control. Due to the excessive cost this option was not selected.

c. CELLULAR WATER CONTROL STRUCTURE.

The selected plan is to build a sheet pile cellular structure with slide gates at the Goofy Ridge site. This option consists of 4 main sheet pile cells and 3 intermediate cells. The sheet pile will be driven to bedrock and the completed cells will be filled with coarse clean granular fill. The top elevation will be 452 MSL which will allow the levee to be raised in the future if needed. The intermediate cells will become the sill for the three 10 foot by 10 foot sluice gates. This option not only allows water control but also effectively closes the breach. A bridge will be constructed on top of the structure to allow access to the levee.

8. UNDERSEEPAGE.

a. The possibility of underseepage distress during high water events was investigated for the breach repair alternative. Three conditions were analyzed, as shown on plate B1, and labeled Condition 1 through 3. A five-foot-thick landside berm was also assumed for each of the three conditions. Due to the nature and extent of the scouring action during the overtopping event, all of the three conditions are thought to represent actual subsurface conditions existing at particular locations within the breach area. The first condition assumes that the clay layer shown in boring LC-96-5 was completely removed by the overtopping event. The second condition assumes that the clay layer remains between Elevation 420 and 430 and will be filled with clean sand between Elevation 430 and 435 during the breach repair. The third condition assumes that the clay layer lies between Elevations 425 and 435 and will be tied into the toe of the new clay levee.

b. All three conditions were analyzed using the computer program FastSEEP. The permeability of the sand foundation (and fill) was assumed to be 300 ft/day, or 0.1 cm/s, based on $D_{10}$ correlation's (Duck Island Sand Pit supplies Illinois gradation FA-1 having an average $D_{10}$ of .25 mm).

c. FastSEEP generated flow nets for all three conditions, as illustrated in plates B2 through B7. Seepage quantities were also computed using FastSEEP. For condition 1 the seepage quantity is 1012 cubic feet/day per foot of levee, which is the worst of all three conditions. For foundation Condition 1, computed factors of safety were 2.85 and 15.5 for uplift beneath the embankment and toe, respectively. For foundation Condition 2, the most critical factor of safety (1.80) was computed for overall uplift beneath the underlying clay layer. By far the most critical safety factors were computed at various locations for foundation Condition 3 (see plates B6 and B7). Factors of safety for uplift computed beneath the clay embankment and at the levee toe were both only slightly above 1.0. As a result of these findings for foundation Condition 3, it is recommended that the berm thickness be increased to 7.5 feet. This is sufficient to increase the safety factors to above 1.5, as required by EM 1110-2-1914, "Design, Construction, and Maintenance of Relief Wells". This is shown in the computations on plates B6 and B7. Due to the design of the selected alternative no underseepage related distress is not a concern.

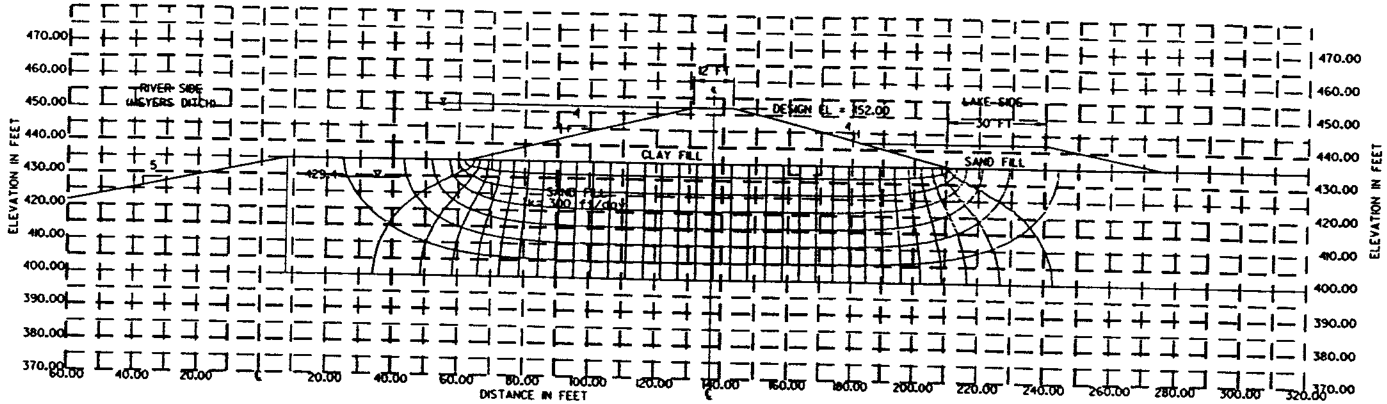
9. SLOPE STABILITY.

a. A slope stability study was completed to ensure that the breach repair would be safe from slope failure. The stability of the slope was analyzed by the Modified Swedish Method for a circular Arc Slope Stability Analysis in accordance with EM 1110-2-1902, "Engineering And Design Stability Of Earth And Rockfill Dams," dated April 1, 1970. All analysis was completed using the computer program Utexas3.

b. Conservative shear strengths were assumed for the most severe configuration of the embankment and foundation. These values are shown on plate B-8. Successive trials of various circular sliding surfaces were analyzed, and a determination of the critical failure arc having the lowest factor of safety was made. The computed minimum safety factor of 2.9 exceeds the 1.3 minimum required by EM 1110-2-1913, "Design And Construction Of Levees," dated March 31,1978. See plate B-8 for a graphic display of the slope stability. Due to the design of the selected alternative slope stability problems are not a concern.

10. SETTLEMENT.

Due to the cellular water control structure being founded on bedrock no settlement will occur below the sheet pile cells.

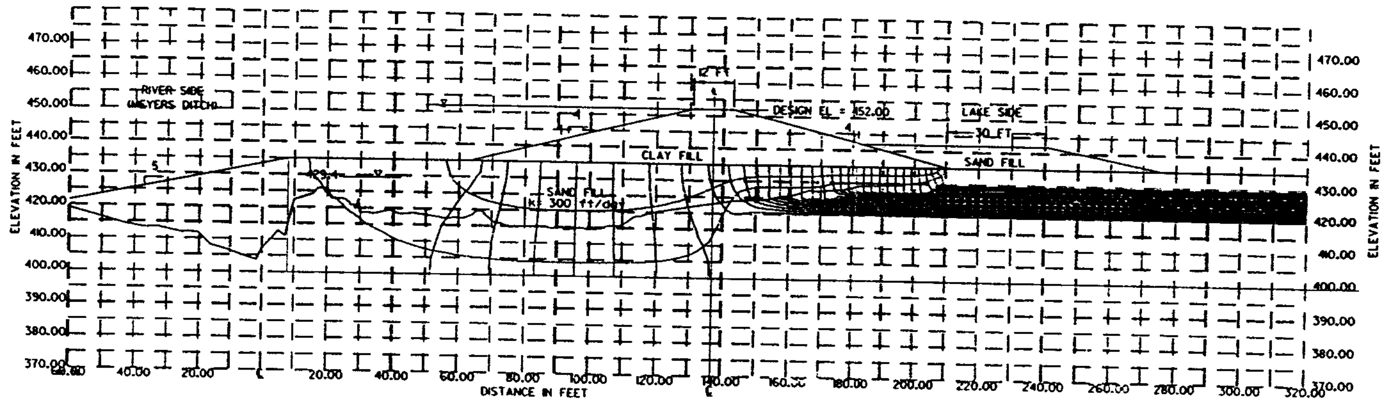



ORIGINAL ALIGNMENT STA 144+86

PROPOSED ALIGNMENT STA 145+00

Q=1012 ft^3/day/ft

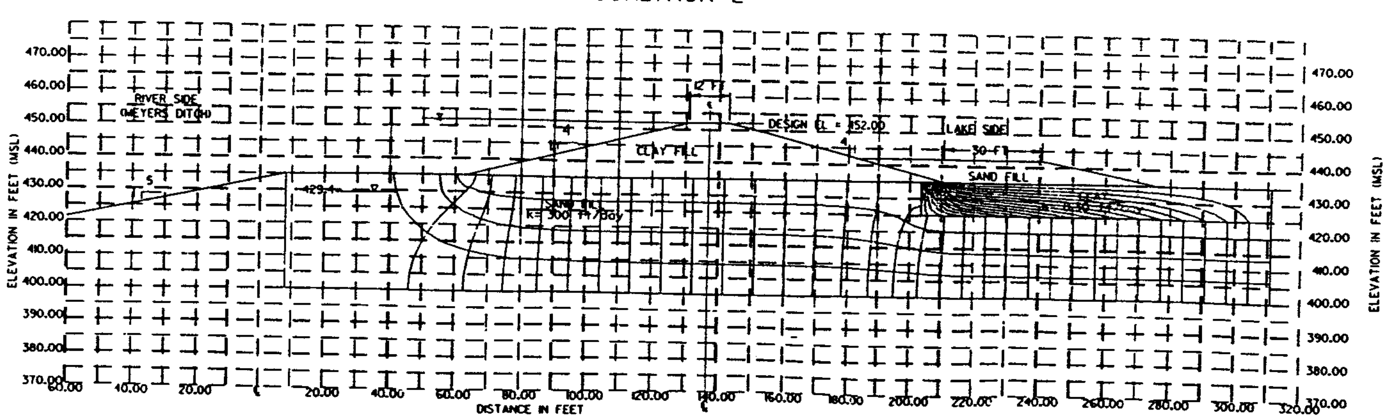
CONDITION 1




ORIGINAL ALIGNMENT STA 144+86

PROPOSED ALIGNMENT STA 145+00

CONDITION 2




ORIGINAL ALIGNMENT STA 144+86

PROPOSED ALIGNMENT STA 145+00

Q=585 ft^3/day/ft

CONDITION 3

**NOTES:**

1. OBTAIN SAND FROM OFF SITE SOURCE.
2. OBTAIN CLAY FILL FROM DESIGNATED BORROW AREAS.
3. FACTOR OF SAFETY AGAINST UPLIFT WITH SAND BERM 2.0, WITHOUT SAND BERM 1.3.

US Army Corps of Engineers Rock Island District

Revisions

Designed by: ECR | Drawn by: TEM | Checked by: RSK | Reviewed by: GJM

Date: XX XXX XX | Scale: AS SHOWN | Drawing Code: XX-XXX | Solicitation Number: DACW25-97-B-XXXX

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS ROCK ISLAND, ILLINOIS

ILLINOIS WATERWAY ENVIRONMENTAL MANAGEMENT PROGRAM LAGRANGE POOL MILE 124 - MILE 129.5 LAKE CHAUTAUQUA STAGE II

**Underseepage Analysis**

Sheet Reference Number:

Condition 1

Compute Factors of Safety at:

(A) uplift of embankment

(B) uplift at toe

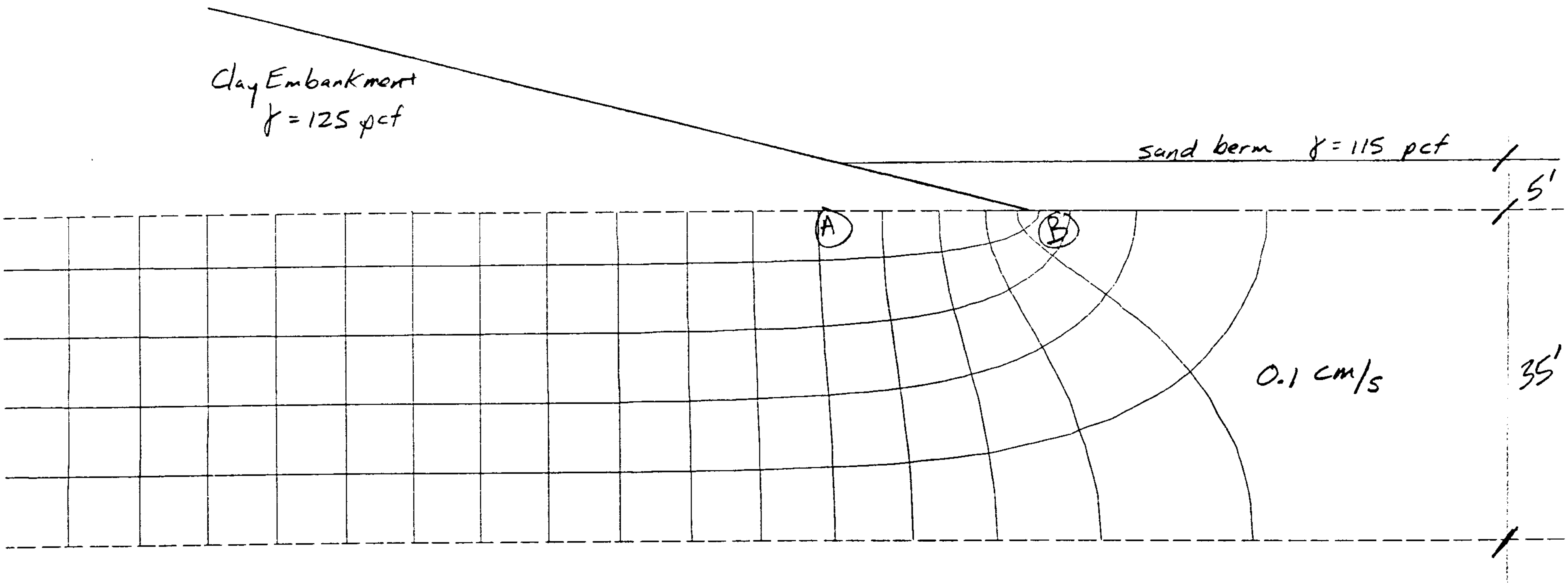
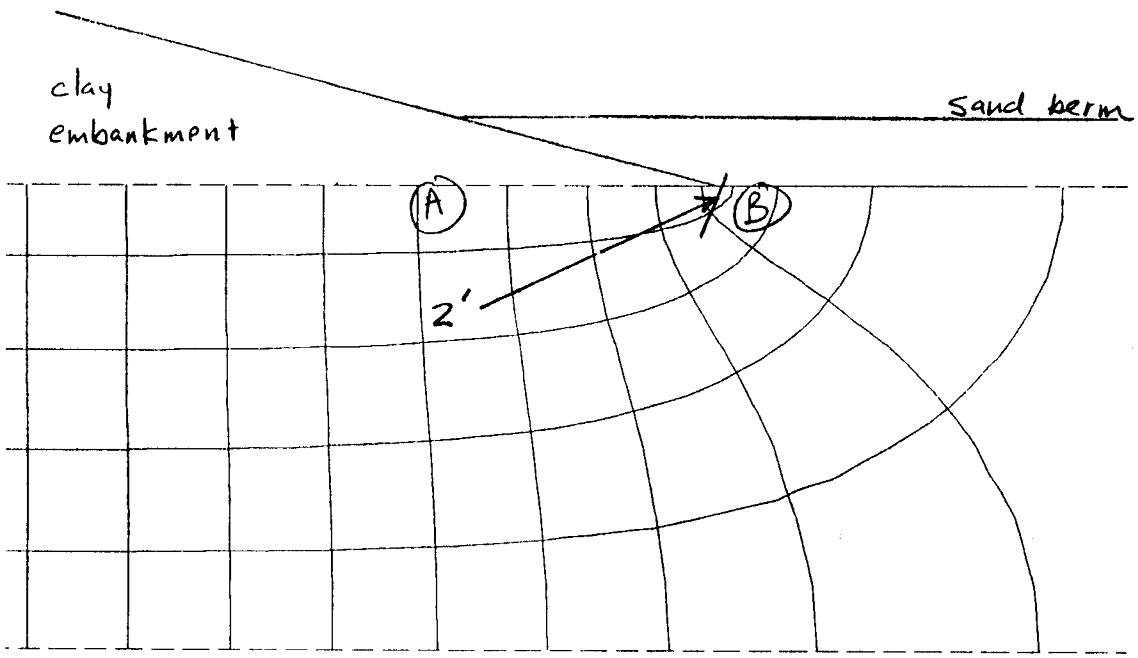



Plate B2

| Subject CHAUTAUQUA - condition 1 | | Date 11-19-96 |
|---|---|---|
| Computed by RSK | Checked by | Sheet of |




$$(A)\quad F.S. = \frac{i_{cr}}{i_o} = \frac{\gamma'/\gamma_w}{h_a/Z_t} = \frac{\gamma' Z_t}{\gamma_w h_a} = \frac{(125)(5)}{(62.5)(438.5-435)} = 2.85$$

$$(B)\quad F.S. = \frac{(115)(5)+(115-62.5)(2)}{(62.5)(435.7-435.0)} = 15.5$$

(without berm = 2.4)

NCR Form 381b
1 Aug 80

Condition 2

Compute Factors of Safety at;

(A) uplift of embankment

(B) uplift at embankment toe

(C) uplift overall

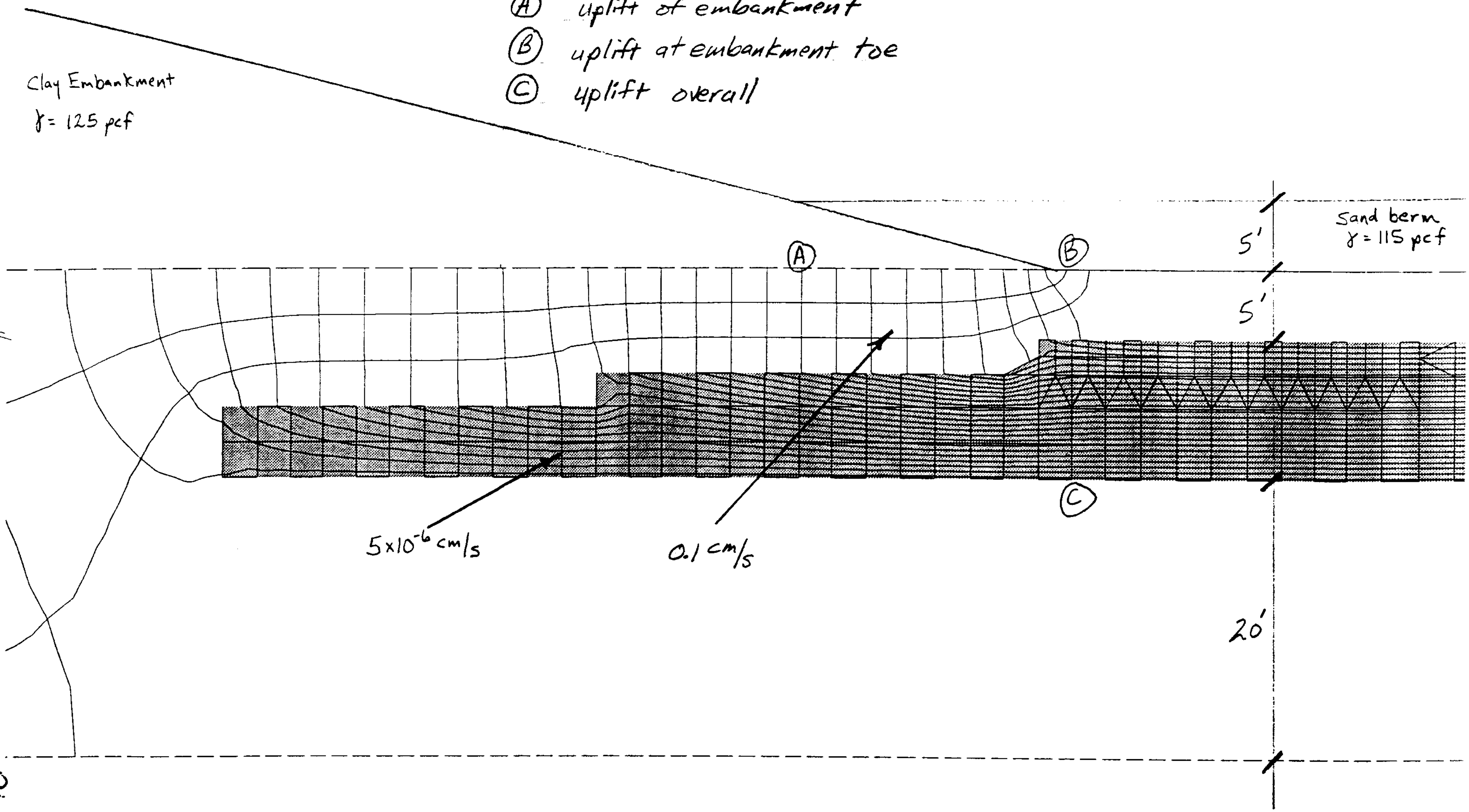
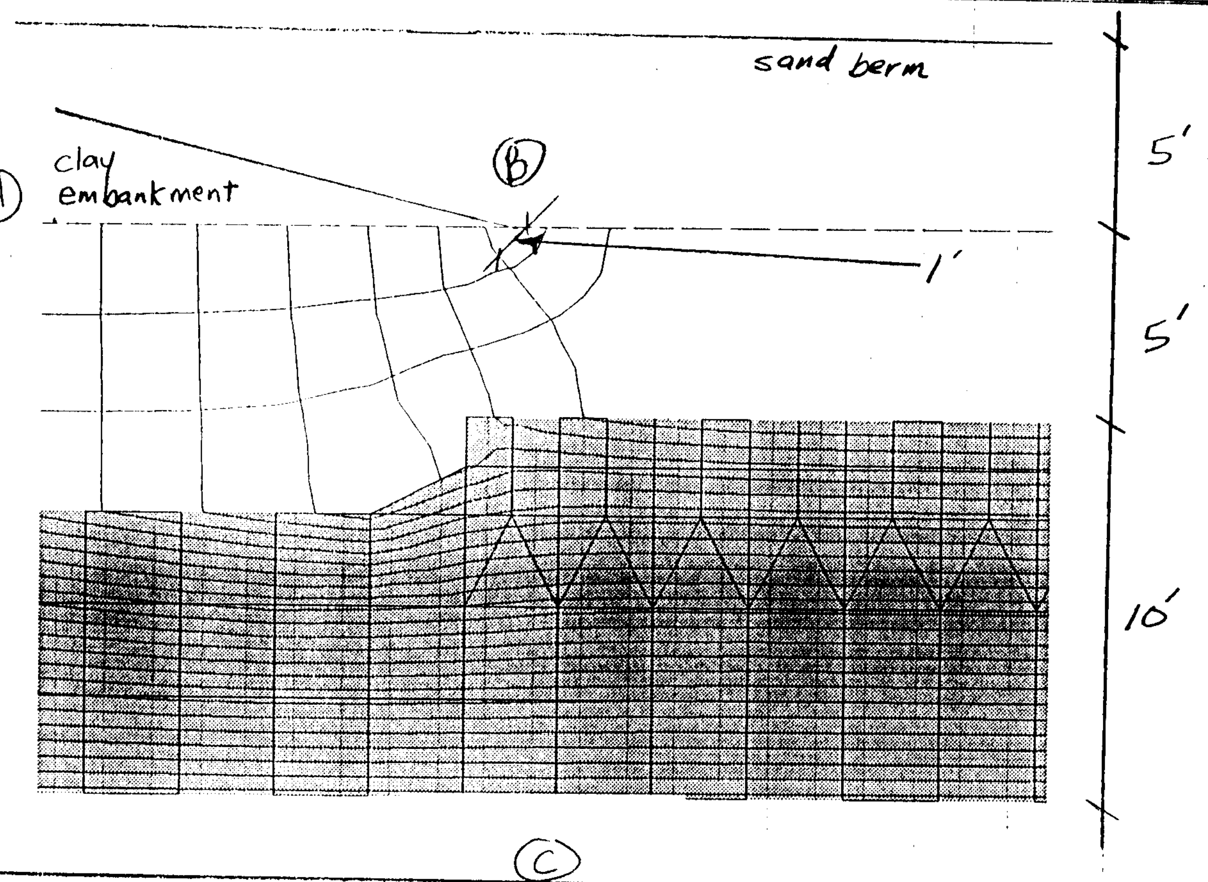



Plate B4

| Subject CHAUTAUQUA - condition 2 | | Date 11-19-96 |
|---|---|---|
| Computed by RSK | Checked by | Sheet of |




(A) $F.S. = i_{cr}/i_o = \dfrac{\gamma'/\gamma_w}{h_a/Z_t} = \dfrac{\gamma' Z_t}{\gamma_w h_a}$

$F.S. = \dfrac{(125)(5)}{(62.5)(439.65-435)} = \underline{2.15}$

(B) $F.S. = \dfrac{(115)(5)+(115-62.5)(1)}{(62.5)(435.50-435)} = \underline{20.0}$ (without berm = 1.68)

(C) $F.S. = \dfrac{(115)(5)+(115-62.5)(5)+(125-62.5)(10)}{(62.5)(448.0-435)} = \underline{1.80}$

NCR Form 381b
1 Aug 80



Plate B5

Condition 3

Compute Factors of Safety at;

(A) uplift of embankment

(B) uplift of embankment

(C) uplift beneath toe

(D) uplift beneath clay layer

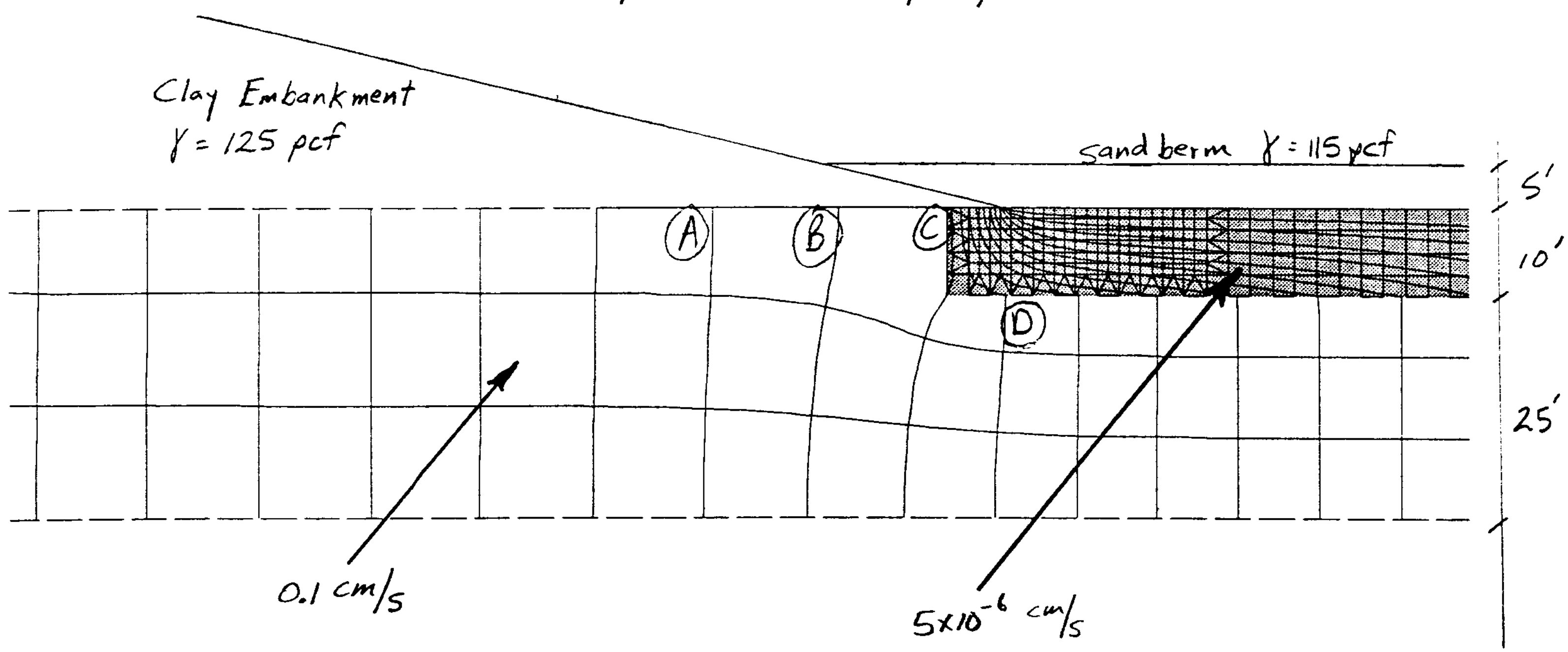
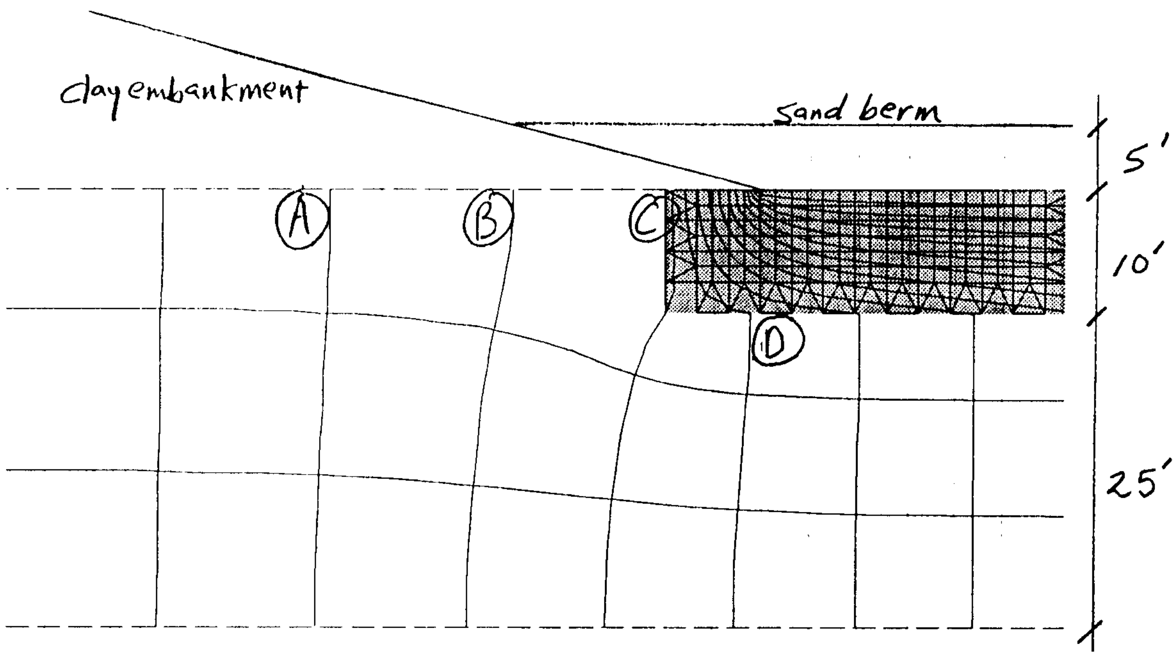



Plate B6

| Subject CHAUTAUQUA - condition 3 | | Date 11-19-96 |
|---|---|---|
| Computed by RSK | Checked by | Sheet of |




(A) $F.S. = \frac{(125)(7.5)}{(62.5)(445-435)} = 1.5$

(B) $F.S. = \frac{(125)(5)}{(62.5)(444.2-435)} = 1.08$ (5' thick berm)

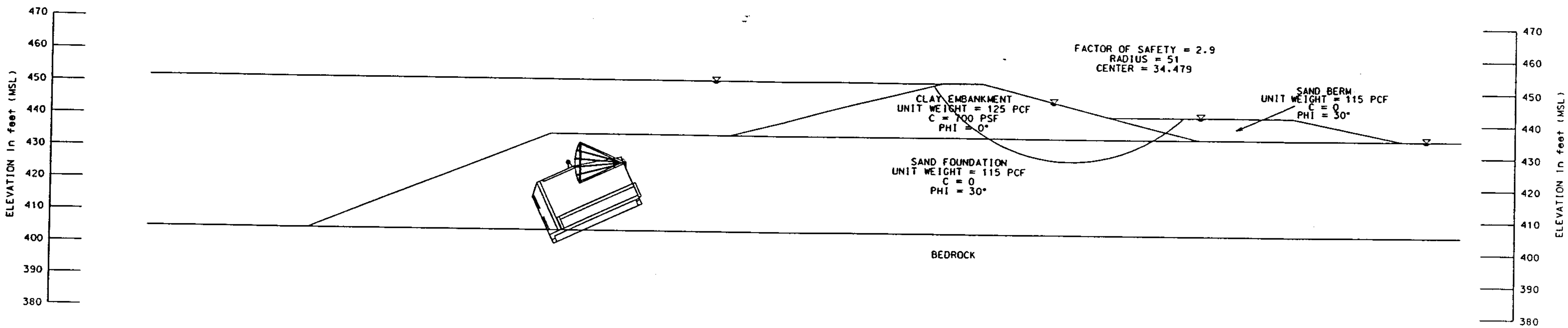
$F.S. = \frac{(125)(5)+(115)(2.5)}{(62.5)(444.2-435)} = 1.58$ (7.5' thick berm)

(C) $F.S. = \frac{(125)(2)+(115)(3)}{(62.5)(443.65-435)} = 1.1$ (5' thick berm)

$F.S. = \frac{(125)(2)+(115)(5.5)}{(62.5)(443.65-435)} = 1.63$ (7.5' thick berm)

(D) $F.S. = \frac{(125-62.5)(10)+(115)(5)}{(62.5)(442.8-435)} = 2.46$ (5' thick berm)

NCR Form 381b
1 Aug 80




US Army Corps of Engineers Rock Island District

| Symbol | Description | Date | Approved |
|---|---|---|---|
| Revisions | | | |

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ROCK ISLAND, ILLINOIS

| | | | |
|---|---|---|---|
| Designed By: EGR | Drawn By: CJK | Checked By: RSK | Reviewed By: GJM |
| Date: 06 NOV 96 | Scale: AS SHOWN | Drawing Code: XX-XXX | Solicitation Number: DACW25-97-B-XXXX |

ILLINOIS WATERWAY
ENVIRONMENTAL MANAGEMENT PROGRAM
LAGRANGE POOL
MILE 124 - MILE 129.5
LAKE CHAUTAUQUA STAGE II

**SLOPE STABILTY**

Sheet Reference Number:
**X1**
Sheet 1 of XX

**PLATE B-8**

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR
LA GRANGE POOL, ILLINOIS WATERWAY, RIVER MILES 124-129
MASON COUNTY, ILLINOIS

# APPENDIX C

# STRUCTURAL APPENDIX

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR

# APPENDIX C
# STRUCTURAL ANALYSIS

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UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR

# APPENDIX C
# STRUCTURAL ANALYSIS

1. Introduction.

a. This appendix is intended to describe the preliminary designs of the structural items required for the project. The structural items for the project are (1) the sheet-pile cells, (2) the floodway between the main cells, including the associated gate, and (3) the roadway bridge connecting main cells.

b. Sufficient design computations have been performed to establish accurate cost information. Final design computations will be performed during preparation of the final plans and specifications.

2. Materials.

a. Sheet pile for the main cells will be used hot rolled PSX32 and used hot rolled PS32 sheet piling will be used for the connecting cells. The used sheet piling is being provided by the St. Louis District, Corps of Engineers. The large diameter of the cells requires the use of high interlock strength (PSX32) sheet piling. The steel sheet piling conforms to ASTM-A328 and ASTM-A572, Grade 50.

b. Cast in place structural concrete will have a 28-day compressive strength of 4000 psi, with 60 ksi yield stress reinforcing steel conforming to ASTM-A615. Concrete for the precast, prestressed bridge deck beams will have a 28-day strength of 5000 psi. The prestressing steel tendons shall be 7 wire round stress relieved strands, 1/2 diameter, with an ultimate strength of 270,000 psi. Structural steel not otherwise noted will be ASTM-A36.

3. Main Cells.

The main cells were analyzed for stability using the Corps' computer programs EASY_CCELL and CSLIDE. Results of program runs for normal and extreme conditions are included. The design is in accordance with EM-1110-2-2503, "Design of Sheet Pile Cellular Structures, Cofferdams and Retaining Structures."

4. Floodway and Gate.

a. The floodway consists of a concrete slab supported on a pile foundation. A concrete wall and gate provide water control. A pile analysis to insure stability of the floodway slab and its gate was performed. Printouts for the pile analysis are included in this appendix. Wall size and reinforcing steel requirements were analyzed for overall stability and quantity calculations.

b. Three 10 ft by 10 ft heavy duty sluice gates shall provide inflow capacity to equalize the upper lake during a flood event. One gate shall be equipped with aluminum stop logs to allow incremental water level control in the upper lake. All gates shall have stop log slots. The gates and the stop logs are of standard design from the manufacturer.

5. Vehicular Bridges.

The vehicular bridges will be designed in accordance with Illinois Department of Transportation (IDOT) standards and the AASHTO Standard Specifications for Highway Bridges. The design live loading will be AASHTO HS20 truck loading. Bridge beam size and abutment details used for the cost estimate were obtained from the IDOT standards. A copy of the relevant pages is included in this appendix.

WES PROGRAM X0075

Checked HD 3-11-97

EM 1110-2-2503
29 SEP 89

PROGRAM EASY_CCELL - ANALYSIS OF CIRCULAR CELL
COFFERDAMS OR MOORINGS FOUNDED ON ROCK OR SOIL
(SAND OR HARD CLAY) USING CLASSICAL METHODS.
DATE: 11-MAR-1997 TIME: 14.39.59

1. INPUT DATA

1.1.--HEADING

Lake Chautauqua cell design - flood with scour EL 410

1.2.--FOUNDATION TYPE,CELL TYPE
FOUNDATION TYPE-ROCK
TYPE-CIRCULAR CELL WALL

1.3.--CELL DESCRIPTION

| | | |
|---|---|---|
| ELEVATION OF TOP OF CELL | = | 452.00 (FT) |
| ELEVATION OF LEFT-SIDE BOTTOM OF CELL | = | 400.00 (FT) |
| ELEVATION OF RIGHT-SIDE BOTTOM OF CELL | = | 400.00 (FT) |
| EFFECTIVE BASE WIDTH OF CELL | = | 60.87 (FT) |
| SLOPE OF THE ROCK FOUNDATION | = | .00 (DEG) |
| THE FRICTION COEFFICIENT BETWEEN FILL AND ROCK | = | .33 |
| CENTER TO CENTER OF CELL | = | 87.80 (FT) |
| ULTIMATE INTERLOCK TENSION | = | 252000.00 (LB/FT) |
| ELEVATION OF DREDGELINE ON INBOARD SIDE | = | .00 (FT) |

1.5.--CELL FILL MATERIAL DESCRIPTION

| | | |
|---|---|---|
| NUMBER OF FILL LAYERS | = | 1 |
| ELEVATION OF SURFACE OF CELL FILL | = | 452.00 (FT) |
| ULTIMATE BEARING CAPACITY OF FOUNDATION | = | 24000.00 (LB/SQ FT) |

CELL FILL MATERIAL LAYER DATA

| LAYER NO. | UNIT WEIGHT (LB/CU FT) | INTERNAL FRICTION ANGLE (DEG) | COHESION (LB/SQ FT) | WALL FRICTION ANGLE (DEG) | BOTTOM ELEV AT WALL (FT) |
|---|---|---|---|---|---|
| 1 | 125.00 | 30.00 | .00 | 17.00 | .00 |

COEFFICIENTS OF EARTH PRESSURE FOR CELL MATERIAL

CLKILT = K-VALUE FOR BURSTING
CLKSHC = K-VALUE FOR CENTER-PLANE SHEAR
CLKILR = K-VALUE FOR INTERLOCK SLIDING, PULLOUT, PENETRATION
CLKPEN = K-VALUE FOR PENETRATION RESISTANCE

| LAYER NO. | CLKILT | CLKSHC | CLKILR | CLKPEN |
|---|---|---|---|---|
| 1 | .400 | .400 | .300 | .200 |

1.6.--LEFT SIDE SOIL DESCRIPTION
NUMBER OF LEFT SIDE SURFACE POINTS = 1
NUMBER OF LEFT SIDE SOIL LAYERS = 1

LEFT SIDE SURFACE POINT COORDINATES

| POINT NO. | ELEVATION (FT) | X-COORD (FT) |
|---|---|---|

| | | |
|---|---|---|
| 1 | 431.50 | .00 |

LEFT SIDE SOIL LAYER DATA

| LAYER NO. | UNIT WEIGHT (LB/CU FT) | INTERNAL FRICTION ANGLE (DEG) | COHESION (LB/SQ FT) | WALL FRICTION ANGLE (DEG) | BOTTOM ELEV AT WALL (FT) | BOTTOM SLOPE (FT/FT) |
|---|---|---|---|---|---|---|
| 1 | 125.00 | 30.00 | .00 | 17.00 | ******* | 1: .0 |

1.7.--RIGHT SIDE SOIL DESCRIPTION
NUMBER OF RIGHT SIDE SURFACE POINTS = 1
NUMBER OF RIGHT SIDE SOIL LAYERS = 1

RIGHT SIDE SURFACE POINT COORDINATES

| POINT NO. | ELEVATION (FT) | X-COORD (FT) |
|---|---|---|
| 1 | 410.00 | .00 |

RIGHT SIDE SOIL LAYER DATA

| LAYER NO. | UNIT WEIGHT (LB/CU FT) | INTERNAL FRICTION ANGLE (DEG) | COHESION (LB/SQ FT) | WALL FRICTION ANGLE (DEG) | BOTTOM ELEV AT WALL (FT) | BOTTOM SLOPE (FT/FT) |
|---|---|---|---|---|---|---|
| 1 | 125.00 | 30.00 | .00 | 17.00 | ******* | 1: .0 |

1.8.--WATER DATA
ELEVATION OF WATER ON LEFT OUTSIDE CELL = 449.00 (FT)
ELEVATION OF WATER ON RIGHT OUTSIDE CELL = 434.00 (FT)
ELEVATION OF WATER ON LEFT INSIDE CELL = 449.00 (FT)
ELEVATION OF WATER ON RIGHT INSIDE CELL = 435.00 (FT)
UNIT WEIGHT OF WATER = 62.50 (LB/CU FT)

1.9.--SURCHARGE LOADS ON RIGHT SIDE
NUMBER OF CONCENTRATED LOADS = 0
NO DISTRIBUTED LOAD

1.10.--SURCHARGE LOADS ON LEFT SIDE
NUMBER OF CONCENTRATED LOADS = 0
NO DISTRIBUTED LOAD

1.11.-SURCHARGE LOAD ON CELL FILL
NO UNIFORM LOAD

1.12.--HORIZONTAL LOADS ON RIGHT
NUMBER OF HORIZONTAL CONCENTRATED LOADS = 0
NUMBER OF HORIZONTAL PRESSURE POINTS = 0

1.13.--HORIZONTAL LOADS ON LEFT
NUMBER OF HORIZONTAL CONCENTRATED LOADS = 0
NUMBER OF HORIZONTAL PRESSURE POINTS = 0

PROGRAM EASY_CCELL - ANALYSIS OF CIRCULAR CELL COFFERDAMS OR MOORINGS FOUNDED ON ROCK OR SOIL (SAND OR HARD CLAY) USING CLASSICAL METHODS.

DATE: 11-MAR-1997 TIME: 14.39.59

## 2. RESULTS -- FACTORS OF SAFETY

### 2.1 -- HEADING

Lake Chautauqua cell design - flood with scour EL 410

### 2.2 -- SUMMARY OF FACTORS OF SAFETY FOR A CIRCULAR CELL WALL ON ROCK

***** Indicates a very large value

| | | |
|---|---|---|
| BURSTING | = | 3.55 |
| SLIP FAILURE ALONG VERTICAL CENTER-PLANE | = | 1.53 |
| HORIZONTAL SHEAR (CUMMING'S METHOD) | = | 3.60 |
| PULLOUT OF OUTBOARD SHEETING | = | 1.97 |
| BEARING FAILURE OF THE FOUNDATION | = | 5.45 |
| SLIDING ON THE BASE | = | 1.96 |

## 3. -- SUMMARY OF INTERMEDIATE VALUES OCCURRING IN FACTOR OF SAFETY CALCULATIONS

( NA = not available -- Calculation too complex to print out in compact form)

### 3.1 -- F.S. -- BURSTING

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| p-max | 1616.7 LB/SQ FT | SEE SECTION 3.1.1 BELOW |
| L | 43.9 FT | = (CENTER-TO-CENTER DISTANCE)/2 |
| d' (left side) | 31.5 FT | NA |
| x' (left side) | 17.3 FT | =(Hfs + d')/3 |
| d' (right side) | 10.0 FT | NA |
| x' (right side) | 17.3 FT | =(Hfs + d')/3 |

### 3.1.1 -- PRESSURE DISTRIBUTION FOR p-max: CLKILT IS K-FACTOR FOR p-max

| PRESSURE POINT NO. | ELEVATION (FT) | PRESSURE (LB/SQ FT) |
|---|---|---|
| 1 | 452.0 | .0 |
| 2 | 442.0 | 500.0 |
| 3 | 442.0 | 500.0 |
| 4 | 400.0 | 4175.0 |

### 3.2 -- F.S. -- SLIP FAILURE ALONG VERTICAL CENTER-PLANE

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|

PLATE C-5

| | | | |
|---|---|---|---|
| PwL | 75031.3 | LB/FT | = RESULTANT WATER PRESS. ON LEFT |
| HwL | 49.0 | FT | WATER EL. - CELL TIP EL. ON LEFT |
| PwR | 36125.0 | LB/FT | = RESULTANT WATER PRESS. ON RIGHT |
| HwR | 34.0 | FT | WATER EL. - CELL TIP EL. ON RIGHT |
| Pa' | 9282.9 | LB/FT | WEDGE METHOD OR INPUT |
| Ha | 10.5 | FT | WEDGE METHOD OR INPUT |
| Pp* | 16832.9 | LB/FT | WEDGE METHOD OR INPUT |
| Pequil | 48189.2 | LB/FT | EQUILIBRIUM OF HORIZONTAL FORCES |
| Pp' | 16832.9 | LB/FT | INPUT OR SMALLER OF Pp* AND Pequil |
| Hp | 3.3 | FT | WEDGE METHOD OR INPUT |
| M | 857458.3 | FT-LB/FT | SEE SECTION 3.2.1 BELOW |
| Sm' | 26298.3 | LB/FT | SEE SECTION 3.2.2 BELOW |
| Tcw/L | 20064.1 | LB/FT | SEE SECTION 3.2.3 BELOW |
| Sm" | 6019.2 | LB/FT | = f * Tcw/L |

3.2.1 -- CALCULATION OF M:

M = (1/3)[PwL(HwL) - PwR(HwR)] + Pa'Ha - Pp'Hp + MOMENT DUE TO APPLIED FORCES AND PRESSURES

3.2.2 -- CALCULATION OF Sm'
CLKSHC IS K-FACTOR FOR Pc'

PRESSURE DISTRIBUTION

| PRESSURE POINT NUMBER | ELEVATION (FT) | PRESSURE (LB/SQ FT) | RESULTANT (LB/FT) | ANGLE OF INTERNAL FRICTION (DEG) | RESULTANT * TANGENT OF ANGLE (LB/FT) |
|---|---|---|---|---|---|
| 1 | 452.0 | .0 | | | |
| | | | 2500.0 | 30.0 | 1443.4 |
| 2 | 442.0 | 500.0 | | | |
| 3 | 442.0 | 500.0 | | | |
| | | | 43050.0 | 30.0 | 24854.9 |
| 4 | 400.0 | 1550.0 | | | |

3.2.3 -- CALCULATION OF Tcw/L
Tcw (left side) governed, values below apply to the left side.
Note: If applied pressures or loads act on the cell, or if water is present outside the cell, on this side then the resultants of these pressures must be subtracted from the resultants in the table below in order to calculate the value of Tcw/L.

PRESSURE DISTRIBUTION FOR PT (= RESULTANT OF SOIL AND WATER PRESSURES INSIDE THE CELL)
CLKILR IS K-FACTOR FOR PT

| PRESSURE POINT NO. | ELEVATION (FT) | PRESSURE (LB/SQ FT) | RESULTANT (LB/FT) |
|---|---|---|---|
| 1 | 452.0 | .0 | |
| | | | 1875.0 |
| 2 | 442.0 | 375.0 | |
| 3 | 442.0 | 375.0 | |
| | | | 8416.4 |
| 4 | 431.5 | 1228.1 | |

PLATE C-6

| | | | |
|---|---|---|---|
| 5 | 431.5 | 1228.1 | |
| | | | 19343.0 |
| 6 | 400.0 | .0 | |

RESULTANT OF APPLIED PRESSURES AND LOADS ON RIGHT = .0
RESULTANT OF APPLIED PRESSURES AND LOADS ON LEFT = .0

3.3 F.S. -- HORIZONTAL SHEAR (CUMMING'S METHOD)

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| M | 857458.3 FT-LB/FT | SEE SECTION 3.2.1 ABOVE |
| Tcw/L | 20064.1 LB/FT | SEE SECTION 3.2.3 ABOVE |
| Mf | 366389.8 FT-LB/FT | = f * b * Tcw/L |
| Mshear | 2716941.0 FT-LB/FT | NA |

3.4 F.S. -- PULLOUT OF OUTBOARD SHEETING

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| L | 43.9 FT | = (CENTER-TO-CENTER DISTANCE)/2 |
| M | 857458.3 FT-LB/FT | SEE SECTION 3.2.1 ABOVE |
| Pa'TAN(DELTLT) | 2838.1 LB/FT | NA |
| Ps'TAN(DELTC) | 10444.5 LB/FT | SEE SECTION 3.4.1 BELOW |

3.4.1 -- CALCULATION OF Ps'TAN(DELTC)
CLKILR IS K-FACTOR FOR Ps'

PRESSURE DISTRIBUTION

| PRESSURE POINT NUMBER | ELEVATION (FT) | PRESSURE (LB/SQ FT) | RESULTANT (LB/FT) | ANGLE OF WALL FRICTION (DEG) | RESULTANT * TANGENT OF ANGLE (LB/FT) |
|---|---|---|---|---|---|
| 1 | 452.0 | .0 | | | |
| | | | 1875.0 | 17.0 | 573.2 |
| 2 | 442.0 | 375.0 | | | |
| 3 | 442.0 | 375.0 | | | |
| | | | 32287.5 | 17.0 | 9871.3 |
| 4 | 400.0 | 1162.5 | | | |

3.5 F.S. -- PENETRATION OF INBOARD SHEETING

NO PENETRATION OF INBOARD SHEETING FOR ROCK FOUNDATION

3.6 F.S. -- BEARING FAILURE OF THE FOUNDATION

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| M | 857458.3 FT-LB/FT | SEE SECTION 3.2.1 ABOVE |
| Weffective | 235871.3 LB/FT | = EFFECT. WT. + SURCHARGE |
| e | 3.6 LB/FT | = M/Weffective |
| Qeffective | 4400.6 LB/SQ FT | = Weffective/(b - 2e) |

3.7 -- F.S. -- SLIDING INSTABILITY

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| Weffective | 235871.3 LB/FT | = EFFECT. WT. + SURCHARGE |
| PHI | 30.0 DEGREES | INPUT |
| Pp* | 16832.9 LB/FT | = RESULTANT PASSIVE PRESS. ON RIGHT |
| F applied-R | .0 LB | = APPLIED LOADS AND PRESS. ON RIGHT |
| Cohesion | .0 LB/FT | INPUT |
| B | 60.9 FT | = CELL WIDTH |
| PwL | 75031.3 LB/FT | = RESULTANT WATER PRESS. ON LEFT |
| PwR | 36125.0 LB/FT | = RESULTANT WATER PRESS. ON RIGHT |
| Pa' | 9282.9 LB/FT | WEDGE METHOD OR INPUT |
| F applied-L | .0 LB | = APPLIED LOADS AND PRESS. ON LEFT |

---

Input file (CHAUT.DAT) for this run follows.

```
30100 Lake Chautauqua cell design - flood with scour EL 410
30200 R C
30300   452.000      400.000      60.870     0.000
30400    87.800   252000.000       0.330
30500   C
30600     1          452.000   24000.000
30610   125.000       30.000       0.000    17.000
30700     0.400        0.400       0.300     0.200
30800     1            1
30810   431.500
30820   125.000       30.000       0.000    17.000
30900     1            1
30910   410.000
30930   125.000       30.000       0.000    17.000
31000   449.000      434.000     449.000   435.000
31100   0 N
31200   0 N
31300   N
31400   0   0
31500   0   0
```

------

Notes.

1. The following line is from Section 1.3 (Cell Description).

ELEVATION OF DREDGELINE ON INBOARD SIDE = .00 (FT)

This line is not meaningful in the current analysis, which is based on sheet piles driven to rock (as reflected in Section 1.2, Foundation Type). Apparently the program prints the li regardless of the conditions.

PLATE C-8

Checked HD

EM 1110-2-2503
29 SEP 89

PROGRAM EASY_CCELL - ANALYSIS OF CIRCULAR CELL
COFFERDAMS OR MOORINGS FOUNDED ON ROCK OR SOIL
(SAND OR HARD CLAY) USING CLASSICAL METHODS.
DATE: 11-MAR-1997 TIME: 14.28.13

1. INPUT DATA

1.1.--HEADING

Lake Chautauqua cell design - flood with severe scour EL 401

1.2.--FOUNDATION TYPE,CELL TYPE
FOUNDATION TYPE-ROCK
TYPE-CIRCULAR CELL WALL

1.3.--CELL DESCRIPTION

| | | | |
|---|---|---|---|
| ELEVATION OF TOP OF CELL | = | 452.00 | (FT) |
| ELEVATION OF LEFT-SIDE BOTTOM OF CELL | = | 400.00 | (FT) |
| ELEVATION OF RIGHT-SIDE BOTTOM OF CELL | = | 400.00 | (FT) |
| EFFECTIVE BASE WIDTH OF CELL | = | 60.87 | (FT) |
| SLOPE OF THE ROCK FOUNDATION | = | .00 | (DEG) |
| THE FRICTION COEFFICIENT BETWEEN FILL AND ROCK | = | .33 | |
| CENTER TO CENTER OF CELL | = | 87.80 | (FT) |
| ULTIMATE INTERLOCK TENSION | = | 252000.00 | (LB/FT) |
| ELEVATION OF DREDGELINE ON INBOARD SIDE | = | .00 | (FT) |

1.5.--CELL FILL MATERIAL DESCRIPTION

| | | | |
|---|---|---|---|
| NUMBER OF FILL LAYERS | = | 1 | |
| ELEVATION OF SURFACE OF CELL FILL | = | 452.00 | (FT) |
| ULTIMATE BEARING CAPACITY OF FOUNDATION | = | 24000.00 | (LB/SQ FT) |

CELL FILL MATERIAL LAYER DATA

| LAYER NO. | UNIT WEIGHT (LB/CU FT) | INTERNAL FRICTION ANGLE (DEG) | COHESION (LB/SQ FT) | WALL FRICTION ANGLE (DEG) | BOTTOM ELEV AT WALL (FT) |
|---|---|---|---|---|---|
| 1 | 125.00 | 30.00 | .00 | 17.00 | .00 |

COEFFICIENTS OF EARTH PRESSURE FOR CELL MATERIAL

CLKILT = K-VALUE FOR BURSTING
CLKSHC = K-VALUE FOR CENTER-PLANE SHEAR
CLKILR = K-VALUE FOR INTERLOCK SLIDING, PULLOUT, PENETRATION
CLKPEN = K-VALUE FOR PENETRATION RESISTANCE

| LAYER NO. | CLKILT | CLKSHC | CLKILR | CLKPEN |
|---|---|---|---|---|
| 1 | .400 | .400 | .300 | .200 |

1.6.--LEFT SIDE SOIL DESCRIPTION
NUMBER OF LEFT SIDE SURFACE POINTS = 1
NUMBER OF LEFT SIDE SOIL LAYERS = 1

LEFT SIDE SURFACE POINT COORDINATES

| POINT NO. | ELEVATION (FT) | X-COORD (FT) |
|---|---|---|

| | | |
|---|---|---|
| 1 | 431.50 | .00 |

LEFT SIDE SOIL LAYER DATA

| LAYER NO. | UNIT WEIGHT (LB/CU FT) | INTERNAL FRICTION ANGLE (DEG) | COHESION (LB/SQ FT) | WALL FRICTION ANGLE (DEG) | BOTTOM ELEV AT WALL (FT) | BOTTOM SLOPE (FT/FT) |
|---|---|---|---|---|---|---|
| 1 | 125.00 | 30.00 | .00 | 17.00 | ******* | 1: .0 |

1.7.--RIGHT SIDE SOIL DESCRIPTION
NUMBER OF RIGHT SIDE SURFACE POINTS = 1
NUMBER OF RIGHT SIDE SOIL LAYERS = 1

RIGHT SIDE SURFACE POINT COORDINATES

| POINT NO. | ELEVATION (FT) | X-COORD (FT) |
|---|---|---|
| 1 | 401.00 | .00 |

RIGHT SIDE SOIL LAYER DATA

| LAYER NO. | UNIT WEIGHT (LB/CU FT) | INTERNAL FRICTION ANGLE (DEG) | COHESION (LB/SQ FT) | WALL FRICTION ANGLE (DEG) | BOTTOM ELEV AT WALL (FT) | BOTTOM SLOPE (FT/FT) |
|---|---|---|---|---|---|---|
| 1 | 125.00 | 30.00 | .00 | 17.00 | ******* | 1: .0 |

1.8.--WATER DATA
ELEVATION OF WATER ON LEFT OUTSIDE CELL = 449.00 (FT)
ELEVATION OF WATER ON RIGHT OUTSIDE CELL = 434.00 (FT)
ELEVATION OF WATER ON LEFT INSIDE CELL = 449.00 (FT)
ELEVATION OF WATER ON RIGHT INSIDE CELL = 435.00 (FT)
UNIT WEIGHT OF WATER = 62.50 (LB/CU FT)

1.9.--SURCHARGE LOADS ON RIGHT SIDE
NUMBER OF CONCENTRATED LOADS = 0
NO DISTRIBUTED LOAD

1.10.--SURCHARGE LOADS ON LEFT SIDE
NUMBER OF CONCENTRATED LOADS = 0
NO DISTRIBUTED LOAD

1.11.-SURCHARGE LOAD ON CELL FILL
NO UNIFORM LOAD

1.12.--HORIZONTAL LOADS ON RIGHT
NUMBER OF HORIZONTAL CONCENTRATED LOADS = 0
NUMBER OF HORIZONTAL PRESSURE POINTS = 0

1.13.--HORIZONTAL LOADS ON LEFT
NUMBER OF HORIZONTAL CONCENTRATED LOADS = 0
NUMBER OF HORIZONTAL PRESSURE POINTS = 0

PROGRAM EASY_CCELL - ANALYSIS OF CIRCULAR CELL COFFERDAMS OR MOORINGS FOUNDED ON ROCK OR SOIL (SAND OR HARD CLAY) USING CLASSICAL METHODS.

DATE: 11-MAR-1997 TIME: 14.28.13

## 2. RESULTS -- FACTORS OF SAFETY

### 2.1 -- HEADING

Lake Chautauqua cell design - flood with severe scour EL 401

### 2.2 -- SUMMARY OF FACTORS OF SAFETY FOR A CIRCULAR CELL WALL ON ROCK

***** Indicates a very large value

| | | |
|---|---|---|
| BURSTING | = | 3.55 |
| SLIP FAILURE ALONG VERTICAL CENTER-PLANE | = | 1.44 |
| HORIZONTAL SHEAR (CUMMING'S METHOD) | = | 2.53 |
| PULLOUT OF OUTBOARD SHEETING | = | 1.85 |
| BEARING FAILURE OF THE FOUNDATION | = | 5.41 |
| SLIDING ON THE BASE | = | 1.62 |

## 3. -- SUMMARY OF INTERMEDIATE VALUES OCCURRING IN FACTOR OF SAFETY CALCULATIONS

( NA = not available -- Calculation too complex to print out in compact form)

### 3.1 -- F.S. -- BURSTING

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| p-max | 1616.7 LB/SQ FT | SEE SECTION 3.1.1 BELOW |
| L | 43.9 FT | = (CENTER-TO-CENTER DISTANCE)/2 |
| d' (left side) | 31.5 FT | NA |
| x' (left side) | 17.3 FT | =(Hfs + d')/3 |
| d' (right side) | 1.0 FT | NA |
| x' (right side) | 17.3 FT | =(Hfs + d')/3 |

### 3.1.1 -- PRESSURE DISTRIBUTION FOR p-max:
CLKILT IS K-FACTOR FOR p-max

| PRESSURE POINT NO. | ELEVATION (FT) | PRESSURE (LB/SQ FT) |
|---|---|---|
| 1 | 452.0 | .0 |
| 2 | 442.0 | 500.0 |
| 3 | 442.0 | 500.0 |
| 4 | 400.0 | 4175.0 |

### 3.2 -- F.S. -- SLIP FAILURE ALONG VERTICAL CENTER-PLANE

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|

PLATE C-11

| | | |
|---|---|---|
| PwL | 75031.3 LB/FT | = RESULTANT WATER PRESS. ON LEFT |
| HwL | 49.0 FT | WATER EL. - CELL TIP EL. ON LEFT |
| PwR | 36125.0 LB/FT | = RESULTANT WATER PRESS. ON RIGHT |
| HwR | 34.0 FT | WATER EL. - CELL TIP EL. ON RIGHT |
| Pa' | 9282.9 LB/FT | WEDGE METHOD OR INPUT |
| Ha | 10.5 FT | WEDGE METHOD OR INPUT |
| Pp* | 168.3 LB/FT | WEDGE METHOD OR INPUT |
| Pequil | 48189.2 LB/FT | EQUILIBRIUM OF HORIZONTAL FORCES |
| Pp' | 168.3 LB/FT | INPUT OR SMALLER OF Pp* AND Pequil |
| Hp | .3 FT | WEDGE METHOD OR INPUT |
| M | 913508.3 FT-LB/FT | SEE SECTION 3.2.1 BELOW |
| Sm' | 26298.3 LB/FT | SEE SECTION 3.2.2 BELOW |
| Tcw/L | 20064.1 LB/FT | SEE SECTION 3.2.3 BELOW |
| Sm" | 6019.2 LB/FT | = f * Tcw/L |

3.2.1 -- CALCULATION OF M:

M = (1/3)[PwL(HwL) - PwR(HwR)] + Pa'Ha - Pp'Hp + MOMENT DUE TO APPLIED FORCES AND PRESSURES

3.2.2 -- CALCULATION OF Sm'
CLKSHC IS K-FACTOR FOR Pc'

PRESSURE DISTRIBUTION

| PRESSURE POINT NUMBER | ELEVATION (FT) | PRESSURE (LB/SQ FT) | RESULTANT (LB/FT) | ANGLE OF INTERNAL FRICTION (DEG) | RESULTANT * TANGENT OF ANGLE (LB/FT) |
|---|---|---|---|---|---|
| 1 | 452.0 | .0 | | | |
| | | | 2500.0 | 30.0 | 1443.4 |
| 2 | 442.0 | 500.0 | | | |
| 3 | 442.0 | 500.0 | | | |
| | | | 43050.0 | 30.0 | 24854.9 |
| 4 | 400.0 | 1550.0 | | | |

3.2.3 -- CALCULATION OF Tcw/L
Tcw (left side) governed, values below apply to the left side.
Note: If applied pressures or loads act on the cell, or if water is present outside the cell, on this side then the resultants of these pressures must be subtracted from the resultants in the table below in order to calculate the value of Tcw/L.

PRESSURE DISTRIBUTION FOR PT (= RESULTANT OF SOIL AND WATER PRESSURES INSIDE THE CELL)
CLKILR IS K-FACTOR FOR PT

| PRESSURE POINT NO. | ELEVATION (FT) | PRESSURE (LB/SQ FT) | RESULTANT (LB/FT) |
|---|---|---|---|
| 1 | 452.0 | .0 | |
| | | | 1875.0 |
| 2 | 442.0 | 375.0 | |
| 3 | 442.0 | 375.0 | |
| | | | 8416.4 |
| 4 | 431.5 | 1228.1 | |

| | | | |
|---|---|---|---|
| 5 | 431.5 | 1228.1 | 19343.0 |
| 6 | 400.0 | .0 | |

RESULTANT OF APPLIED PRESSURES AND LOADS ON RIGHT = .0
RESULTANT OF APPLIED PRESSURES AND LOADS ON LEFT = .0

3.3 F.S. -- HORIZONTAL SHEAR (CUMMING'S METHOD)

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| M | 913508.3 FT-LB/FT | SEE SECTION 3.2.1 ABOVE |
| Tcw/L | 20064.1 LB/FT | SEE SECTION 3.2.3 ABOVE |
| Mf | 366389.8 FT-LB/FT | = f * b * Tcw/L |
| Mshear | 1947418.0 FT-LB/FT | NA |

3.4 F.S. -- PULLOUT OF OUTBOARD SHEETING

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| L | 43.9 FT | = (CENTER-TO-CENTER DISTANCE)/2 |
| M | 913508.3 FT-LB/FT | SEE SECTION 3.2.1 ABOVE |
| Pa'TAN(DELTLT) | 2838.1 LB/FT | NA |
| Ps'TAN(DELTC) | 10444.5 LB/FT | SEE SECTION 3.4.1 BELOW |

3.4.1 -- CALCULATION OF Ps'TAN(DELTC)
CLKILR IS K-FACTOR FOR Ps'

PRESSURE DISTRIBUTION

| PRESSURE POINT NUMBER | ELEVATION (FT) | PRESSURE (LB/SQ FT) | RESULTANT (LB/FT) | ANGLE OF WALL FRICTION (DEG) | RESULTANT * TANGENT OF ANGLE (LB/FT) |
|---|---|---|---|---|---|
| 1 | 452.0 | .0 | 1875.0 | 17.0 | 573.2 |
| 2 | 442.0 | 375.0 | | | |
| 3 | 442.0 | 375.0 | 32287.5 | 17.0 | 9871.3 |
| 4 | 400.0 | 1162.5 | | | |

3.5 F.S. -- PENETRATION OF INBOARD SHEETING

NO PENETRATION OF INBOARD SHEETING FOR ROCK FOUNDATION

3.6 F.S. -- BEARING FAILURE OF THE FOUNDATION

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| M | 913508.3 FT-LB/FT | SEE SECTION 3.2.1 ABOVE |
| Weffective | 235871.3 LB/FT | = EFFECT. WT. + SURCHARGE |
| e | 3.9 LB/FT | = M/Weffective |
| Qeffective | 4440.0 LB/SQ FT | = Weffective/(b - 2e) |

3.7 -- F.S. -- SLIDING INSTABILITY

| QUANTITY | VALUE | HOW VALUE WAS OBTAINED |
|---|---|---|
| Weffective | 235871.3 LB/FT | = EFFECT. WT. + SURCHARGE |
| PHI | 30.0 DEGREES | INPUT |
| Pp* | 168.3 LB/FT | = RESULTANT PASSIVE PRESS. ON RIGHT |
| F applied-R | .0 LB | = APPLIED LOADS AND PRESS. ON RIGHT |
| Cohesion | .0 LB/FT | INPUT |
| B | 60.9 FT | = CELL WIDTH |
| PwL | 75031.3 LB/FT | = RESULTANT WATER PRESS. ON LEFT |
| PwR | 36125.0 LB/FT | = RESULTANT WATER PRESS. ON RIGHT |
| Pa' | 9282.9 LB/FT | WEDGE METHOD OR INPUT |
| F applied-L | .0 LB | = APPLIED LOADS AND PRESS. ON LEFT |

---

Input file (CHAUT.DAT) for this run follows.

```
30100 Lake Chautauqua cell design - flood with severe scour EL 401
30200 R C
30300   452.000     400.000      60.870     0.000
30400    87.800  252000.000       0.330
30500   C
30600     1         452.000   24000.000
30610   125.000      30.000       0.000    17.000
30700     0.400       0.400       0.300     0.200
30800     1           1
30810   431.500
30820   125.000      30.000       0.000    17.000
30900     1           1
30910   401.000
30930   125.000      30.000       0.000    17.000
31000   449.000     434.000     449.000   435.000
31100   0 N
31200   0 N
31300   N
31400   0    0
31500   0    0
```

------

Notes.

1. The following line is from Section 1.3 (Cell Description).

ELEVATION OF DREDGELINE ON INBOARD SIDE = .00 (FT)

This line is not meaningful in the current analysis, which is based on sheet piles driven to rock (as reflected in Section 1.2, Foundation Type). Apparently the program prints the l. regardless of the conditions.

PLATE C-14

W.E.S. Program X0075D

Checked HD

PROGRAM CSLIDE - FINAL RESULTS

DATE: 96/12/04 TIME: 10.20.35

SLIDING ANALYSIS FOR LAKE CHAUTAUQUA CELL, LOAD CASE 1

MULTIPLE FAILURE PLANE ANALYSIS

SEEPAGE FORCE COMPUTED BY LINE OF CREEP

| WEDGE NUMBER | HORIZONTAL LOADS LEFT SIDE (KIPS) | HORIZONTAL LOADS RIGHT SIDE (KIPS) | VERTICAL LOAD (KIPS) |
|---|---|---|---|
| 1 | .000 | .000 | 27.405 |
| 2 | 9.570 | 29.070 | .000 |
| 3 | .000 | .000 | 3.315 |

WATER PRESSURES ON WEDGES

LEFTSIDE WEDGES

| WEDGE NO. | TOP PRESSURE (KSF) | BOTTOM PRESSURE (KSF) |
|---|---|---|
| 1 | 1.094 | 2.697 |

STRUCTURAL WEDGE

| X-COORD. (FT) | PRESSURE (KSF) |
|---|---|
| 100.00 | 2.697 |
| 161.80 | 1.980 |

RIGHTSIDE WEDGES

WEDGE NO. TOP PRESSURE (KSF) BOTTOM PRESSURE (KSF)

PLATE C-15

| | | |
|---|---|---|
| 3 | 1.906 | 1.980 |

| WEDGE NUMBER | FAILURE ANGLE (DEG) | TOTAL LENGTH (FT) | WEIGHT OF WEDGE (KIPS) | SUBMERGED LENGTH (FT) | UPLIFT FORCE (KIPS) |
|---|---|---|---|---|---|
| 1 | -51.5 | 40.250 | 51.302 | 40.250 | 76.292 |
| 2 | .000 | 61.800 | 417.768 | 61.800 | 144.535 |
| 3 | 29.9 | 2.006 | .113 | 2.006 | 3.898 |

| WEDGE NUMBER | NET FORCE ON WEDGE (KIPS) |
|---|---|
| 1 | -84.541 |
| 2 | 82.553 |
| 3 | 1.988 |

SUM OF FORCES ON SYSTEM ---- .000

FACTOR OF SAFETY ----------- 2.502

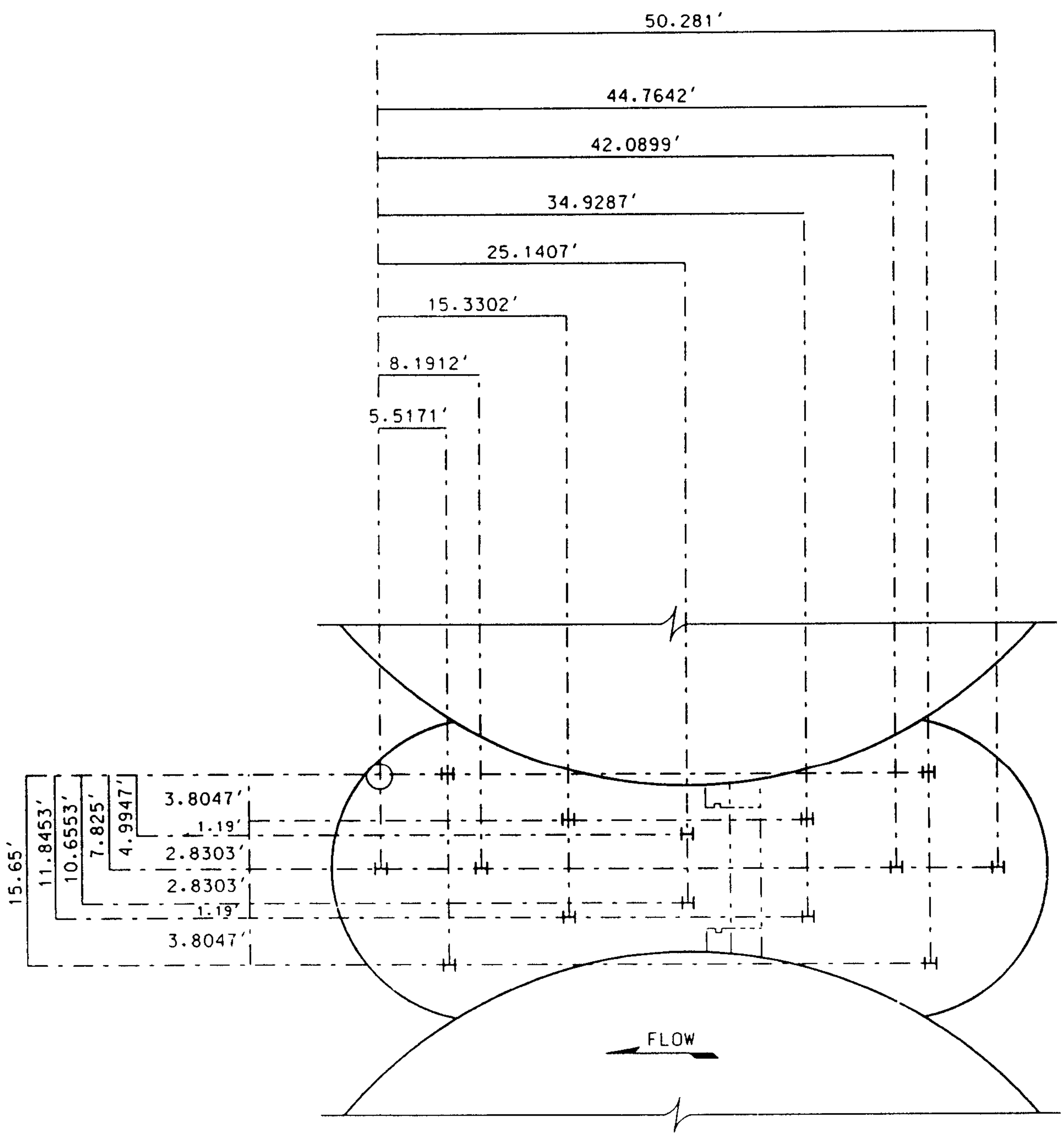
US Army Corps of Engineers
Rock Island District

# COMPUTATION SHEET

| | | |
|---|---|---|
| Page | of | Pages |
| Computed by JDP | | Date Dec96 |
| Checked by | | Date |

Project: Lake Chautauqua

Computation: ARC SLAB PILE LAYOUT



W.E.S. Program X0080                Checked HD

EM 1110-2-2906

```
*****************************
* CORPS PROGRAM  #  X0080   *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 86/09/02 *  RUN DATE 03-05-97     RUN TIME 12:42:00
*****************************
```

Chautauqua arc-cell H-pile group
for sluice gate pile cap between main cells.
Load case 1: flood conditions (FS=1.7).
Load case 2: drought conditions (FS=2.25).

THERE ARE  14 PILES AND
 2 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX

| | X | Y | Z |
|---|---|---|---|
| WITH DIAGONAL COORDINATES = | ( -25.00 , | -8.00 , | .00 ) |
| | ( 25.00 , | 8.00 , | .00 ) |

******************************************************************************

PILE PROPERTIES AS INPUT

| E | I1 | I2 | A | C33 | B66 |
|---|---|---|---|---|---|
| KSI | IN**4 | IN**4 | IN**2 | | |
| .29000E+05 | .25000E+03 | .75500E+03 | .25500E+02 | .10000E+01 | .00000E+00 |

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

ALL

******************************************************************************

SOIL DESCRIPTIONS AS INPUT

| NH | ESOIL | LENGTH | L | LU |
|---|---|---|---|---|
| | K/IN**3 | | FT | FT |
| | .34000E-01 | L | .29000E+02 | .00000E+00 |

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

ALL

******************************************************************************

PILE GEOMETRY AS INPUT AND/OR GENERATED

| NUM | X FT | Y FT | Z FT | BATTER | ANGLE | LENGTH FT | FIXITY |
|---|---|---|---|---|---|---|---|
| 1 | 25.00 | .00 | .00 | V | .00 | 29.00 | P |
| 2 | 20.00 | 8.00 | .00 | V | .00 | 29.00 | P |
| 3 | 20.00 | -8.00 | .00 | V | .00 | 29.00 | P |
| 4 | 17.00 | .00 | .00 | V | .00 | 29.00 | P |
| 5 | 10.00 | 4.00 | .00 | V | .00 | 29.00 | P |
| 6 | 10.00 | -4.00 | .00 | V | .00 | 29.00 | P |
| 7 | .00 | 3.00 | .00 | V | .00 | 29.00 | P |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 8 | .00 | -3.00 | .00 | V | .00 | 29.00 | P |
| 9 | -10.00 | 4.00 | .00 | V | .00 | 29.00 | P |
| 10 | -10.00 | -4.00 | .00 | V | .00 | 29.00 | P |
| 11 | -17.00 | .00 | .00 | V | .00 | 29.00 | P |
| 12 | -20.00 | 8.00 | .00 | V | .00 | 29.00 | P |
| 13 | -20.00 | -8.00 | .00 | V | .00 | 29.00 | P |
| 14 | -25.00 | .00 | .00 | V | .00 | 29.00 | P |
| | | | | | | ------ | |
| | | | | | | 406.00 | |

******************************************************************************

APPLIED LOADS

| LOAD CASE | PX K | PY K | PZ K | MX FT-K | MY FT-K | MZ FT-K |
|---|---|---|---|---|---|---|
| 1 | 459.0 | .0 | 714.3 | .0 | 4145.4 | .0 |
| 2 | .0 | .0 | 1416.6 | .0 | 1789.4 | .0 |

LOAD CASE 1. NUMBER OF FAILURES = 0. NUMBER OF PILES IN TENSION = 0.

LOAD CASE 2. NUMBER OF FAILURES = 0. NUMBER OF PILES IN TENSION = 0.

******************************************************************************

PILE CAP DISPLACEMENTS

| LOAD CASE | DX IN | DY IN | DZ IN | RX RAD | RY RAD | RZ RAD |
|---|---|---|---|---|---|---|
| 1 | .7028E+00 | .0000E+00 | .2401E-01 | .0000E+00 | .4247E-04 | .0000E+00 |
| 2 | .0000E+00 | .0000E+00 | .4762E-01 | .0000E+00 | .1833E-04 | .0000E+00 |

******************************************************************************

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

| PILE | F1 K | F2 K | F3 K | M1 IN-K | M2 IN-K | M3 IN-K | ALF | CBF |
|---|---|---|---|---|---|---|---|---|
| 1 | 32.8 | .0 | 23.9 | .0 | -1462.4 | .0 | .09 | .75 |
| 2 | 32.8 | .0 | 29.4 | .0 | -1462.4 | .0 | .12 | .76 |
| 3 | 32.8 | .0 | 29.4 | .0 | -1462.4 | .0 | .12 | .76 |
| 4 | 32.8 | .0 | 32.6 | .0 | -1462.4 | .0 | .13 | .77 |
| 5 | 32.8 | .0 | 40.2 | .0 | -1462.4 | .0 | .16 | .78 |
| 6 | 32.8 | .0 | 40.2 | .0 | -1462.4 | .0 | .16 | .78 |
| 7 | 32.8 | .0 | 51.0 | .0 | -1462.4 | .0 | .20 | .81 |
| 8 | 32.8 | .0 | 51.0 | .0 | -1462.4 | .0 | .20 | .81 |
| 9 | 32.8 | .0 | 61.9 | .0 | -1462.4 | .0 | .24 | .83 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 10 | 32.8 | .0 | 61.9 | .0 | -1462.4 | .0 | .24 | .83 |
| 11 | 32.8 | .0 | 69.4 | .0 | -1462.4 | .0 | .27 | .85 |
| 12 | 32.8 | .0 | 72.7 | .0 | -1462.4 | .0 | .29 | .85 |
| 13 | 32.8 | .0 | 72.7 | .0 | -1462.4 | .0 | .29 | .85 |
| 14 | 32.8 | .0 | 78.1 | .0 | -1462.4 | .0 | .31 | .86 |

LOAD CASE - 2

| PILE | F1 K | F2 K | F3 K | M1 IN-K | M2 IN-K | M3 IN-K | ALF | CBF |
|---|---|---|---|---|---|---|---|---|
| 1 | .0 | .0 | 89.5 | .0 | .0 | .0 | .35 | .19 |
| 2 | .0 | .0 | 91.8 | .0 | .0 | .0 | .36 | .20 |
| 3 | .0 | .0 | 91.8 | .0 | .0 | .0 | .36 | .20 |
| 4 | .0 | .0 | 93.2 | .0 | .0 | .0 | .37 | .20 |
| 5 | .0 | .0 | 96.5 | .0 | .0 | .0 | .38 | .21 |
| 6 | .0 | .0 | 96.5 | .0 | .0 | .0 | .38 | .21 |
| 7 | .0 | .0 | 101.2 | .0 | .0 | .0 | .40 | .22 |
| 8 | .0 | .0 | 101.2 | .0 | .0 | .0 | .40 | .22 |
| 9 | .0 | .0 | 105.9 | .0 | .0 | .0 | .42 | .23 |
| 10 | .0 | .0 | 105.9 | .0 | .0 | .0 | .42 | .23 |
| 11 | .0 | .0 | 109.1 | .0 | .0 | .0 | .43 | .24 |
| 12 | .0 | .0 | 110.5 | .0 | .0 | .0 | .43 | .24 |
| 13 | .0 | .0 | 110.5 | .0 | .0 | .0 | .43 | .24 |
| 14 | .0 | .0 | 112.9 | .0 | .0 | .0 | .44 | .25 |

---

Input file (C.I) for this run follows.
Output file (c.o) is named on line 8300.

```
1000  Chautauqua arc-cell H-pile group -
    - for sluice gate pile cap between main cells. -
    - Load case 1: flood conditions (FS=1.7). -
    - Load case 2: drought conditions (FS=2.25).
1100  PROP 29000 250  755    25.5  1.0    0.0         ALL
1200  SOIL NH 0.034 LEN 29 0 ALL
1300  PIN ALL
1320  TENSION 0.1 ALL
1410  ALLOW H     255   51.0 459.0  91.8 685.8 2106    ALL
3110  PILE  1  25.0  0.0 0
3120  PILE  2  20.0  8.0 0
3130  PILE  3  20.0 -8.0 0
3140  PILE  4  17.0  0.0 0
3150  PILE  5  10.0  4.0 0
3160  PILE  6  10.0 -4.0 0
3170  PILE  7   0.0  3.0 0
3180  PILE  8   0.0 -3.0 0
3190  PILE  9 -10.0  4.0 0
3200  PILE 10 -10.0 -4.0 0
3210  PILE 11 -17.0  0.0 0
3220  PILE 12 -20.0  8.0 0
3230  PILE 13 -20.0 -8.0 0
3240  PILE 14 -25.0  0.0 0
6110  LOAD 1  459.0  0.0  714.3 0.0 4145.4 0.0
6120  LOAD 2    0.0  0.0 1416.6 0.0 1789.4 0.0
8300  FOUT 1 2 4 5 c.o
8310  PFO ALL
```

------

Notes.

1. The pile is HP 13x87. The pile layout is from Joel's notes, rounded to the nearest foot.

2. The horizontal subgrade reaction constant (NH) was taken from two sources. A photocopy of a book left by Tom Wirtz suggested about 60 kips per cubic foot is a lower bound of appropriate values for dense sand. A chart that Dick Atkinson used suggested 40 tons per cubic foot. I used 30 tons per cubic foot, or 0.0347 kips per cubic inch, truncated to 0.034 kips per cubic inch, as shown on line 1200.

3. According to EM 1110-2-2906 of 910115, bending effects are greater near the top of the pile and damage is more likely near the bottom of the pile. Allowable axial loading at the bottom of the pile is 10 ksi, at the top of the pile it is 18 ksi, and allowable bending stress is 18 ksi (20 ksi for compact sections). A theoretical load without a load test should have the following safety factors: usual, 3.0; unusual, 2.25; extreme, 1.7. The allowable stresses were multiplied by the area or section modulus to obtain the allowable axial force or moment shown in line 1410.

4. Load calculations (unfactored) are summarized below.

| source of load | calc | load | arm | moment |
|---|---|---|---|---|
| concrete pile cap | 0.150*1034*3 | 465.3 | 0.0 | 0.0 |
| gate structure | 0.150*(25*15*3-300) | 123.8 | 5.0 | 619.0 |
| bulkhead slots | 0.150*3.7*2*23 | 25.5 | 2.5 | 63.8 |
| sluice gate (from Joel's notes) | | 15.0 | 7.5 | 112.5 |
| water | 0.0625*(517-75)*24 | 663.0 | 18.5 | 12265.5 |
| uplift | 0.0625*1034*27/2 | -872.4 | 9.7 | -8462.3 |
| horizontal water | 0.0625*24^2*15/2 | | 8.0 | -2160.0 |
| total | 270 horizontal | 420.2 | | 2438.5 |

5. The maximum flood is an extreme event; the loads were multiplied by the 1.7 safety factor. The drought condition is an unusual event; the loads were multiplied by 2.25 and the water loads were omitted.

| Subject: Chautauqua gate structure | Date: 970130 |
|---|---|
| Computed by: WM / Checked by: HD | Sheet 1 of |

ref. EM 1110-2-2104

ACI 318-89

Check for adequacy of dimensions.

Structure: Assume 23' cantilever, 5' deep, 1.5' wide.

The wall is actually taller than 23', but a water level of 21.5' would over top the main cell. The wall is actually wider than 1.5' for most of its depth. At the bulkhead slot, it is about 1.5'.

Loading: Assume 23' head of water, static.

$w = 62.5$ pcf

The cantilever wall (one on each side of the gate) must resist a 7.5' width of water.

$$M = (1.7)(1.3)\frac{Wl}{3}$$

1.7 live load factor

1.3 Corps hydraulic load factor

$W$ total load

$$W = 62.5 * 7.5 * 23 * 23/2$$

$$\simeq 124 \text{ kip}$$

$$M = (1.7)(1.3)(124)(23)/3$$

$$\simeq 2100 \text{ kip-ft}$$

| Subject | | Date |
|---|---|---|
| Chautauqua | | 970130 |
| **Computed by** | **Checked by** | **Sheet** 2 **of** |
| WM | HD | |

Section $b = 18''$
$h = 60'' \Rightarrow d = 54''$

$$m = f_y / 0.85 f'_c$$

$$= 60 / (0.85)(4) = 17.65$$

$$R_n = M_u / b d^2$$

$$= \frac{(2100)(12)}{(18)(54)^2} = 0.48 \text{ ksi}$$

$$\rho = \left(\frac{1}{m}\right)\left(1 - \sqrt{1 - 2 m R_n / f_y}\right)$$

$$= \left(\frac{1}{17.65}\right)\left(1 - \sqrt{1 - 2(17.65)(0.48)/60}\right) = 0.8\%$$

$\rho_{min} = 0.333\% < 0.8\%$ ok — ACI Sect. 10.5.1

$\rho_{max} = 0.75\, \rho_b$ — ACI Sect. 10.3.3

$$= (0.75)(0.85)\beta_1 (f'_c / f_y)\left(\frac{87}{87 + f_y}\right)$$

$$= (0.75)(0.85)(0.85)\left(\frac{4}{60}\right)\left(\frac{87}{87+60}\right) = 2.14\% > 0.8\% \text{ ok}$$

$A_s = (0.8\%)(18)(54) = 7.78 \text{ in}^2 \Rightarrow$

| | | |
|---|---|---|
| 10 | #8 | (7.[illegible] $in^2$) |
| [illegible] | #[illegible] | ([illegible]) |
| 7 | #10 | (8.89 $in^2$) |
| 5 | #11 | (7.8 $in^2$) |

Because the Tension side of the wall [illegible] the compression side, there will not be [illegible] the steel in the concrete.

NCR Form 381b
1 Aug 80

PLATE C-23

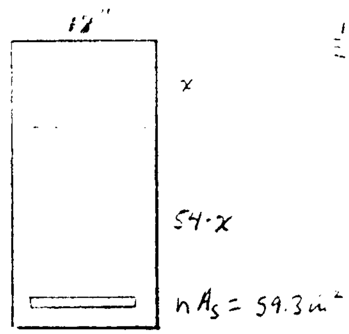
| Subject: Chautauqua | | Date: 970130 |
|---|---|---|
| Computed by: WM | Checked by: HD | Sheet 3 of |

estimate deflection

$$E_c = w_c^{1.5}\, 33 \sqrt{f'_c}$$

$$= (150)^{1.5}\,(33)\sqrt{4000} = 3.8 \times 10^6 \text{ psi}$$

$$n = \frac{E_s}{E_c} = 7.6$$




$$\tfrac{1}{2}(24)x^2 = (59.3)(54-x) \Rightarrow 0 = 12x^2 + 59.3x - 3202$$

$$\Rightarrow x = \frac{-59.3 + \sqrt{59.3^2 - 4(12)(3202)}}{2(12)}$$

$$= 14.05 \text{ in}$$

$$I_{cr} = \tfrac{1}{3} b x^3 + n A_s (54-x)^2$$

$$= \left(\tfrac{1}{3}\right)(12)(14.05)^3 + (59.3)(54-14.05)^2 = 111300 \text{ in}^4$$

deflection at top

$$\frac{W\ell^3}{15EI} = \frac{(124)(23 * 12)^3}{(15)(3.8\times10^6)(111300)} = \underline{\underline{0.4 \text{ in}}}$$

NCR Form 381b
1 Aug 80

| Subject: Chautauqua | | Date: 970130 |
|---|---|---|
| Computed by: WM | Checked by: HD | Sheet 4 of |

check for shear capacity

Total Load: $(1.3)(1.7)(124) = 274$ kip

Use $b = 24''$, which is more realistic. (The 18'' is only at the bulkhead slot)

ACI 318/318R p. 144
Sect. 11.3.2.1

$$V_c = \left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d$$

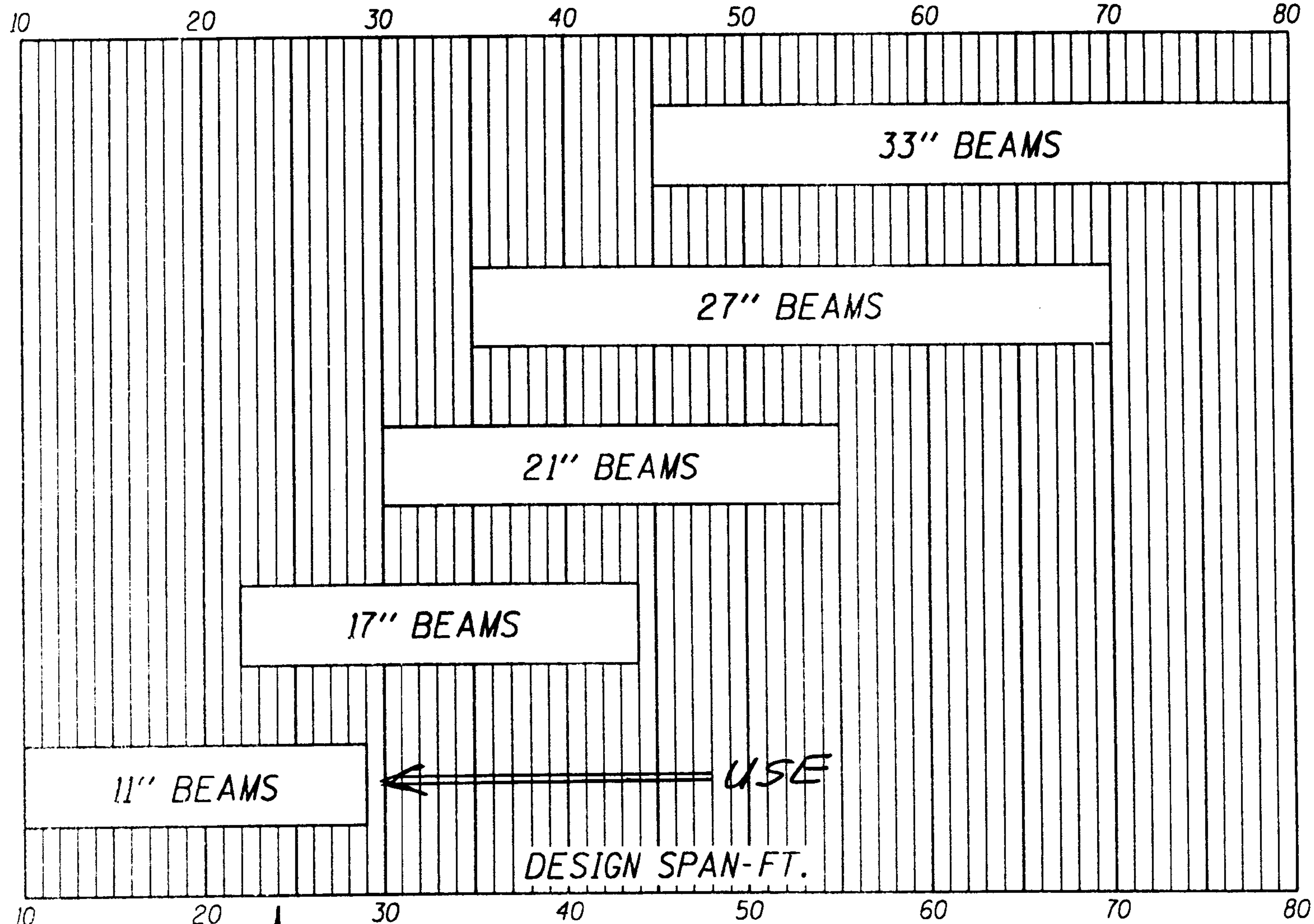
$$= \left(1.9\sqrt{4000} + 2500 \frac{7.8}{(24)(54)} \frac{(274000)(54)}{(2100)(12)}\right)(24)(54) = \boxed{167.2\,k}$$

~~kips.~~

but not more than $3.5\sqrt{f'_c}\, b_w d = (3.5)(\sqrt{4000})(24)(54) = \underline{287 \text{ kip}}$

Because $V_u > \frac{1}{2}\phi V_c$, shear reinforcement will be required.

With shear reinforcement, there should be no problem satisfying $\phi V_c > V_u$.




NOTE: For skews >0° and roadway widths <32'-0" reduce maximum spans shown.

24' SPAN

DECK BEAM-SPAN RANGE
HS 20 LOADING
f'c = 5000 p.s.i.

Figure 2.2.1

PLATE C-26

# 11" x 48" DECK BEAM
# MAXIMUM SPAN LENGTHS

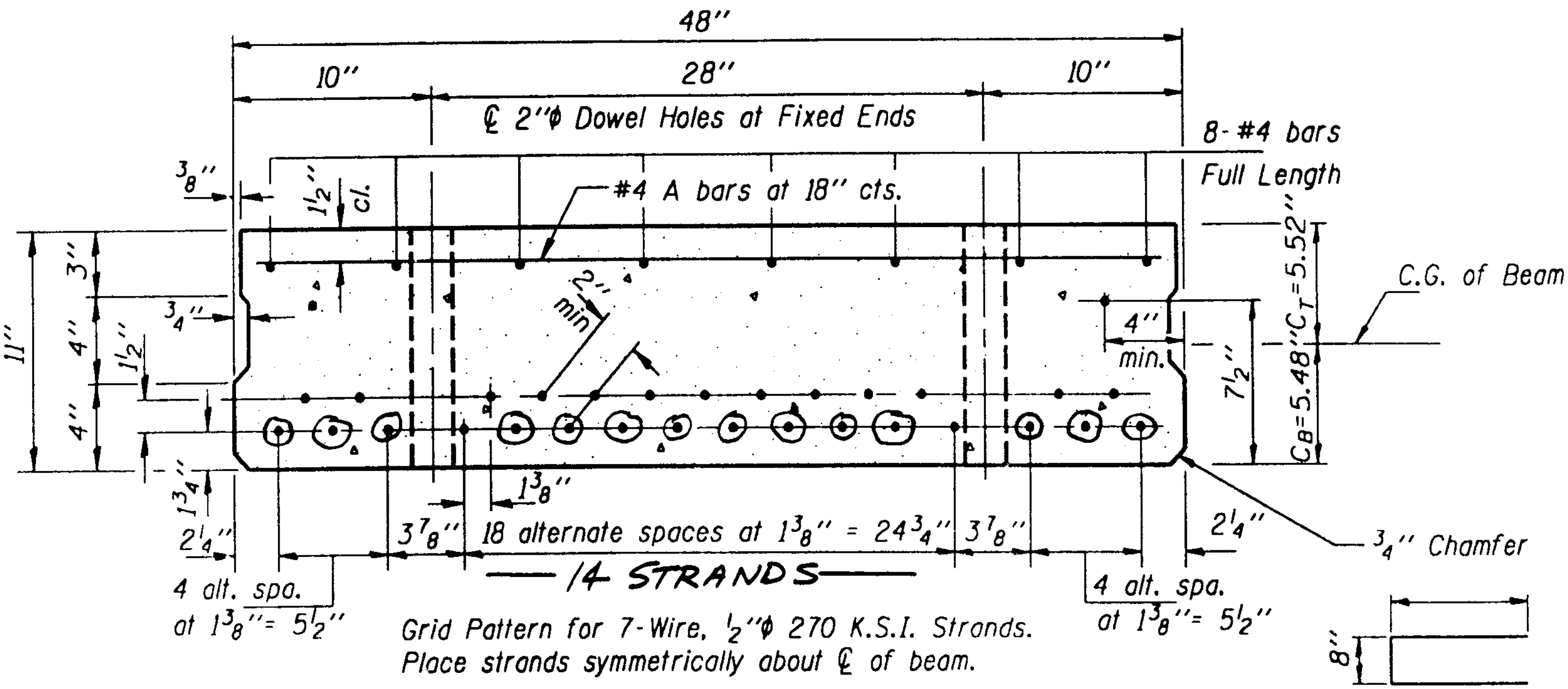
HS 20 Loading
50#/Sq. Ft. Wearing Surf.

f'c = 5,000 psi
½"ϕ Strands

| Strand Pattern | MAX. SPAN LENGTH - FT. Two Lanes L.L.+Imp. 24'-0" o.-o. Bms. | MAX. SPAN LENGTH - FT. Two Lanes L.L.+Imp. 32'-0" o.-o. Bms. | MAX. SPAN LENGTH - FT. Three Lanes L.L.+Imp. 44'-0" o.-o. Bms. |
|---|---|---|---|
| 4 | 9.5 | 12 | 12 |
| 5 | 12 | 14.5 | 14.5 |
| 6 | 14 | 17 | 16.5 |
| 7 | 16 | 19.5 | 19 |
| 8 | 18 | 21.5 | 21 |
| 9 | 19.5 | 23 | 22 |
| 10 | 20 | 23.5 | 23 |
| 11 | 21.5 | 24.5 | 24 |
| 12 | 22 | 25 | 25 |
| 13 | 23 | 25.7 | 25.5 |
| 14 | 24 | 26.5 | 26 |
| 15 | 24.5 | 27 | 26.6 |
| 16 | 25 | 27.7 | 27.2 |
| 17A | 25.5 | 28.2 | 27.7 |
| 18A | 26 | 28.8 | 28.2 |

PLATE C-27

Figure 2.2.2

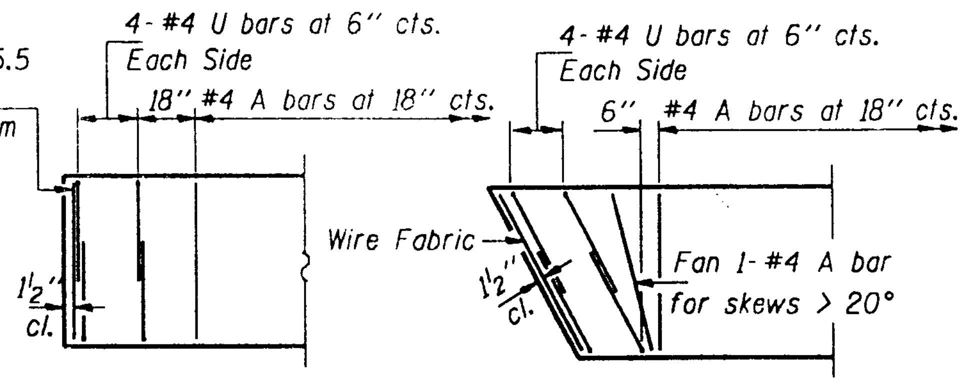



CROSS SECTION

BAR U

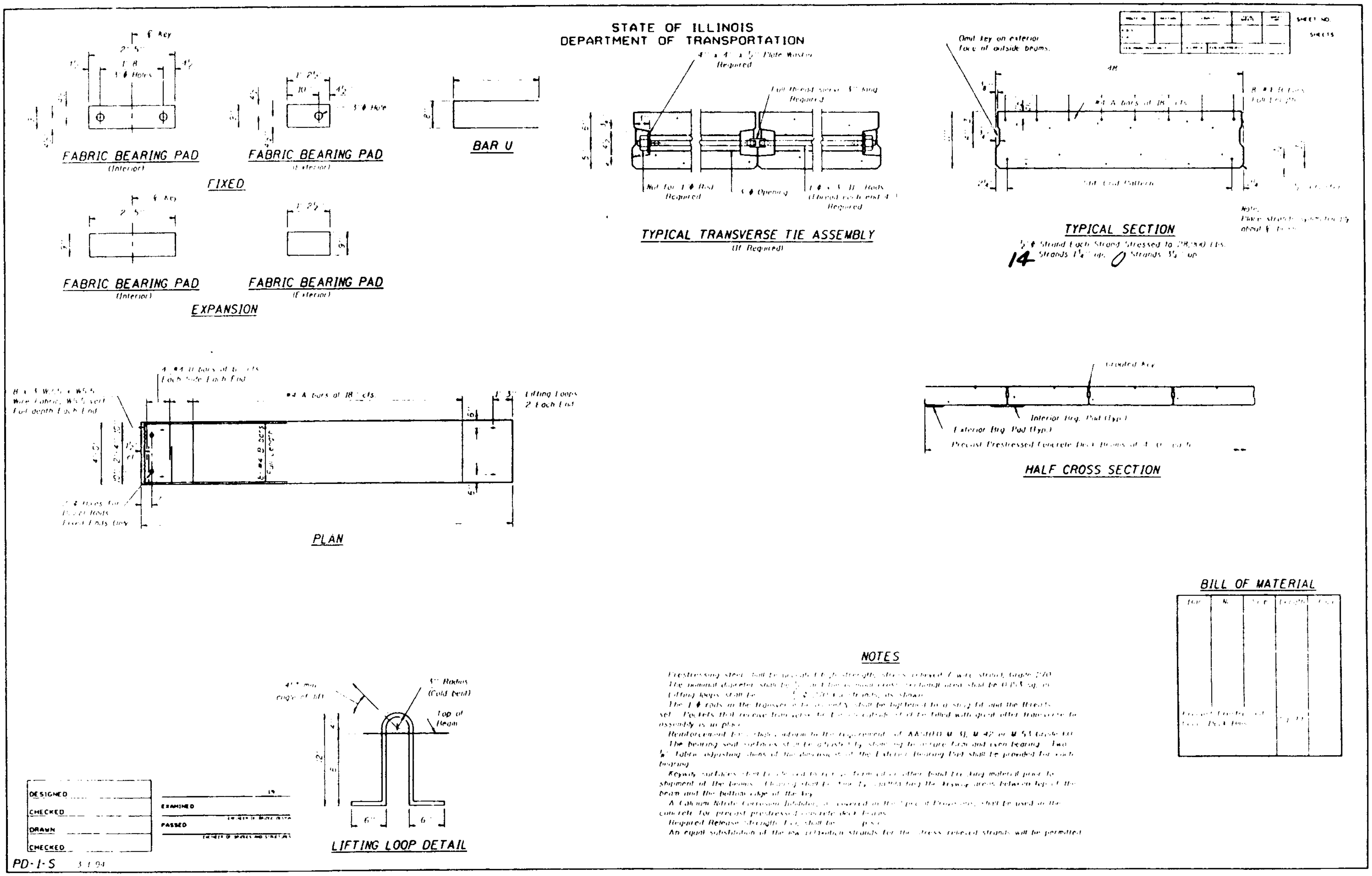
## BEAM PROPERTIES

Cross Sectional Area = 519.9 $in.^2$
Wt. per Lin. Foot = 542 Lbs.
Moment of Inertia = 5,260 $in.^4$
Section Modulus:
$S_B$ = 959.5 $in.^3$
$S_T$ = 953.2 $in.^3$




END REINFORCEMENT

PLATE C-29


STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION
FABRIC BEARING PAD
(Interior)
FABRIC BEARING PAD
(Exterior)
BAR U
FIXED
FABRIC BEARING PAD
(Interior)
FABRIC BEARING PAD
(Exterior)
EXPANSION
TYPICAL TRANSVERSE TIE ASSEMBLY
(If Required)
Omit key on exterior face of outside beams.
TYPICAL SECTION
14
0
#4 A bars at 18" cts.
Lifting Loops 2 Each End
PLAN
HALF CROSS SECTION
Grouted Key
Interior Brg. Pad (Typ.)
Exterior Brg. Pad (Typ.)
BILL OF MATERIAL
NOTES
3" Radius (Cold bend)
Top of Beam
6"
6"
LIFTING LOOP DETAIL
DESIGNED
CHECKED
DRAWN
CHECKED
EXAMINED
PASSED
PD-1-S
3-1-94

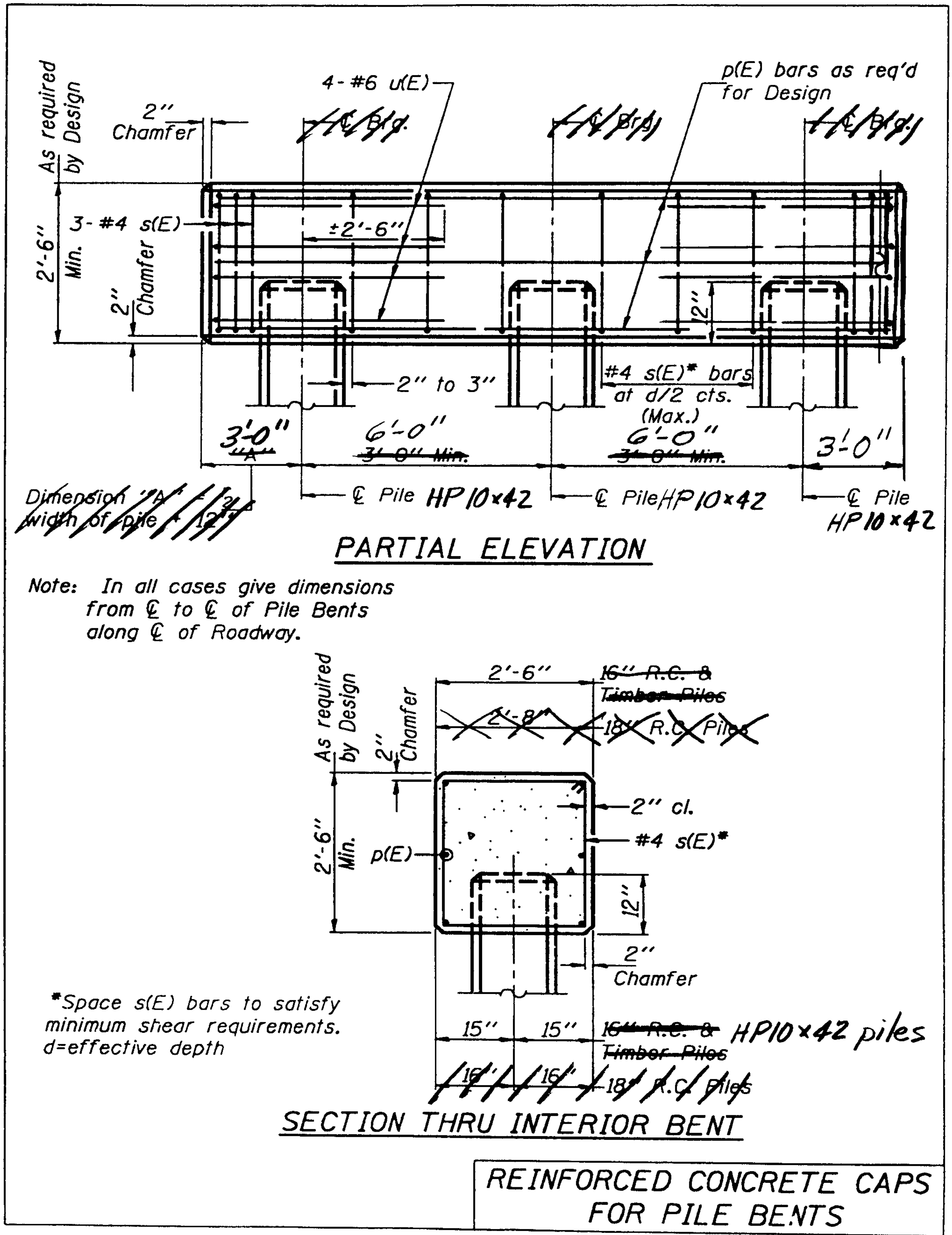

4-#6 u(E)
p(E) bars as req'd for Design
As required by Design
2" Chamfer
~~℄ Brg.~~
2'-6" Min.
3-#4 s(E)
±2'-6"
2" Chamfer
12"
2" to 3"
#4 s(E)* bars at d/2 cts. (Max.)
3'-0"
~~"A"~~
6'-0"
~~3'-0" Min.~~
6'-0"
~~3'-0" Min.~~
3'-0"
℄ Pile HP 10×42
℄ Pile HP 10×42
℄ Pile HP 10×42
~~Dimension "A" = 1/2 width of pile + 12"~~
PARTIAL ELEVATION
Note: In all cases give dimensions from ℄ to ℄ of Pile Bents along ℄ of Roadway.
2'-6"
~~16" R.C. & Timber Piles~~
~~2'-8" 18" R.C. Piles~~
As required by Design
2" Chamfer
2" cl.
#4 s(E)*
2'-6" Min.
p(E)
12"
2" Chamfer
*Space s(E) bars to satisfy minimum shear requirements. d=effective depth
15"
15"
~~16" R.C. & Timber Piles~~
HP10×42 piles
~~16" 16" 18" R.C. Piles~~
SECTION THRU INTERIOR BENT
REINFORCED CONCRETE CAPS FOR PILE BENTS
ABUTMENTS

Figure 3.7.8-1

WES program X0080 Checked, HP

EM 1110-2-2906

```
******************************     
* CORPS PROGRAM  #  X0080    *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 86/09/02  *  RUN DATE 03-04-97    RUN TIME 13:32:09
******************************
```

Chautauqua abutment H-piles
for roadway between main cells.
Load case 1: truck. (FS=3.0).

THERE ARE 6 PILES AND
1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX

| | X | Y | Z |
|---|---|---|---|
| WITH DIAGONAL COORDINATES = | ( -12.00 , | -6.50 , | .00 ) |
| | ( 12.00 , | 6.50 , | .00 ) |

******************************************************************************

PILE PROPERTIES AS INPUT

| E KSI | I1 IN**4 | I2 IN**4 | A IN**2 | C33 | B66 |
|---|---|---|---|---|---|
| .29000E+05 | .21000E+03 | .71700E+02 | .12400E+02 | .10000E+01 | .00000E+00 |

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

ALL

******************************************************************************

SOIL DESCRIPTIONS AS INPUT

| NH | ESOIL K/IN**3 | LENGTH | L FT | LU FT |
|---|---|---|---|---|
| | .34000E-01 | L | .50000E+02 | .00000E+00 |

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

ALL

******************************************************************************

PILE GEOMETRY AS INPUT AND/OR GENERATED

| NUM | X FT | Y FT | Z FT | BATTER | ANGLE | LENGTH FT | FIXITY |
|---|---|---|---|---|---|---|---|
| 1 | -12.00 | -6.50 | .00 | V | 90.00 | 50.00 | P |
| 2 | -12.00 | .00 | .00 | V | 90.00 | 50.00 | P |
| 3 | -12.00 | 6.50 | .00 | V | 90.00 | 50.00 | P |
| 4 | 12.00 | -6.50 | .00 | V | 90.00 | 50.00 | P |
| 5 | 12.00 | .00 | .00 | V | 90.00 | 50.00 | P |
| 6 | 12.00 | 6.50 | .00 | V | 90.00 | 50.00 | P |
| | | | | | | ------ | |
| | | | | | | 300.00 | |

************************************************************************

APPLIED LOADS

| LOAD CASE | PX K | PY K | PZ K | MX FT-K | MY FT-K | MZ FT-K |
|---|---|---|---|---|---|---|
| 1 | 9.6 | .0 | 192.0 | 576.0 | 960.0 | .0 |

LOAD CASE 1. NUMBER OF FAILURES = 0. NUMBER OF PILES IN TENSION = 1.

TENSION PILE ITERATION.

LOAD CASE 1. NUMBER OF FAILURES = 0. NUMBER OF PILES IN TENSION = 1.
IT TOOK 1 ITERATIONS.

************************************************************************

PILE CAP DISPLACEMENTS

| LOAD CASE | DX IN | DY IN | DZ IN | RX RAD | RY RAD | RZ RAD |
|---|---|---|---|---|---|---|
| 1 | .5722E-01 | -.1343E-08 | .5156E-01 | .5092E-03 | .1673E-03 | .0000E+00 |

************************************************************************

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO (F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

| PILE | F1 K | F2 K | F3 K | M1 IN-K | M2 IN-K | M3 IN-K | ALF | CBF |
|---|---|---|---|---|---|---|---|---|
| 1 | .0 | -1.6 | 21.5 | -55.3 | .0 | .0 | .17 | .31 |
| 2 | .0 | -1.6 | 45.3 | -55.3 | .0 | .0 | .37 | .42 |
| 3 | .0 | -1.6 | 69.1 | -55.3 | .0 | .0 | .56 | .53 |
| 4 | .0 | -1.6 | -.7 | -55.3 | .0 | .0 | .03 | .23 |
| 5 | .0 | -1.6 | 16.5 | -55.3 | .0 | .0 | .13 | .29 |
| 6 | .0 | -1.6 | 40.3 | -55.3 | .0 | .0 | .32 | .40 |

---

Input file (C.I) for this run follows.
Output file (c.o) is named on line 8300.

1000 Chautauqua abutment H-piles -
- for roadway between main cells. -

```
   - Load case 1: truck. (FS=3.0).
1100  PROP 29000 210   71.7  12.4  1.0    0.0         ALL
1200  SOIL NH 0.034 LEN 50 0 ALL
1300  PIN ALL
1320  TENSION 0.1 ALL
1410  ALLOW H    124   24.8 223.2 44.6  255.6  781.2  ALL
2500  ANGLE  90  ALL
3110  PILE  1 -12.0 -6.5 0
3120  PILE  2 -12.0  0.0 0
3130  PILE  3 -12.0  6.5 0
3140  PILE  4  12.0 -6.5 0
3150  PILE  5  12.0  0.0 0
3160  PILE  6  12.0  6.5 0
6110  LOAD 1  9.6 0.0 192.0 576.0 960.0 0.0
8300  FOUT 1 2 4 5 c.o
8310  PFO ALL
```

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Notes.

1. The pile is HP 10x42, because that's what the draft Design Memo said.

2. The horizontal subgrade reaction constant (NH) is the same as for the arc cell piles.

3. According to EM 1110-2-2906 of 910115, bending effects are greater near the top of the pile and damage is more likely near the bottom of the pile. Allowable axial loading at the bottom of the pile is 10 ksi, at the top of the pile it is 18 ksi, and allowable bending stress is 18 ksi (20 ksi for compact sections). A theoretical load without a load test should have the following safety factors: usual, 3.0; unusual, 2.25; extreme, 1.7. The allowable stresses were multiplied by the area or section modulus to obtain the allowable axial force or moment shown in line 1410.

4. Load calculations are summarized below. The bridge is too short for both trailer axels and the cab axel to be on the bridge at the same time. One trailer axel is directly over the abutment and the other is 14' away (My). One pair of wheels is on the centerline of the bridge and the other is 6' away (Mx). Thus there are applied moments in two directions. The deck itself is 542 plf for a 4' width.

| source of load | calc | Fx | Fz | Mx | My |
|---|---|---|---|---|---|
| bridge deck | 4*26*0.542 | | 56.4 | | |
| HS20-44 truck | 2*32 | 3.2 | 64.0 | 192.0 | 320.0 |
| total | | 3.2 | 120.4 | 192.0 | 320.0 |

5. This is the normal event; the loads were multiplied by the 3.0 safety factor.

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DESIGN MEMORANDUM

LAKE CHAUTAUQUA REHABILITATION AND ENHANCEMENT
1996 FLOOD REPAIR
LA GRANGE POOL, ILLINOIS WATERWAY, RIVER MILES 124-129
MASON COUNTY, ILLINOIS

# APPENDIX D

# COST ESTIMATES

# Appendix D, Table 1: Project Cost Summary

LAKE CHAUTAUQUA EMP PROJECT COST
PROJECT COST SUMMARY, JANUARY 1997

| ACCOUNT | FEATURE | CURRENT WORKING ESTIMATE (CWE) | FULLY FUNDED ESTIMATE (FFE) |
|---|---|---|---|
| | | FEDERAL | FEDERAL |
| 01. | LANDS AND DAMAGES | $0 | $0 |
| 02. | RELOCATIONS | $0 | $0 |
| 06. | FISH AND WILDLIFE FACILITIES | $2,498,070 | $2,617,977 |
| 30. | PLANNING, ENGINEERING AND DESIGN | $275,000 | $288,200 |
| 31. | CONSTRUCTION MANAGEMENT | $225,000 | $235,800 |
| | TOTAL PROJECT COST | $2,998,070 | $3,141,977 |

NOTES:

1. TOTAL PROJECT COST IS 100% FEDERAL COST; PROJECT LANDS WILL BE GOVERNMENT OWNED.
2. CONSTRUCTION FOR FLOOD REPAIR IS SCHEDULED FOR JULY 1997- NOVEMBER 1998 GIVING INFLATION FACTOR OF 1.048 (Mid Point of Construction 2nd Quarter of FY1998).
3. CONSTRUCTION MANAGEMENT COSTS HAVE INFLATION FACTOR OF 1.048

## Appendix D, Table 2: Project Cost Estimate Lake Chautauqua 1996 Flood Repair

LAKE CHAUTAUQUA 1996 Flood Repair, This Project will be Sheet Pile Cellular Structure with Appurtenances
PROJECT COST ESTIMATE, JANUARY 1997

| ACCOUNT CODE | ITEM | QUANTITY (English) | UNIT (English) | UNIT PRICE (English) | AMOUNT | CON % | CONTIN-GENCY | REASON |
|---|---|---|---|---|---|---|---|---|
| 06. | **FISH AND WILDLIFE FACILITIES** | | | | | | | |
| 06.3.-.- | Wildlife Facilities and Structures | | | | | | | |
| 06.3.0.- | Slide Gates | 3 | Each | $131,501.00 | $394,503.00 | 15 | $59,175.45 | 1,3,6,7 |
| 06.3.1.- | Large Cells 74' Dia., PSX 32's | 4 | Each | $125,422.00 | $501,688.00 | 15 | $75,253.20 | 1,2,4,5,7 |
| 06.3.2.- | Intermediate Cells PS 32's | 3 | Each | $40,314.00 | $120,942.00 | 15 | $18,141.30 | 1,2,4,5,7 |
| 06.3.3.- | H Piling @ HD Sluice Gates | 1470 | VLF | $33.15 | $48,730.50 | 20 | $9,746.10 | 1,4,5,6,7 |
| 06.3.4.- | Stop Logs & Accesories | 1 | EA | $34,004.00 | $34,004.00 | 20 | $6,800.80 | 3,4,6,7 |
| 06.3.5.- | 40 degree Extruded Y's | 660 | VLF | $25.69 | $16,955.40 | 15 | $2,543.31 | 1,3,4,5,7 |
| 06.3.6.- | Sheet Piling Extensions to Levee | 65.12 | Tons | $617.00 | $40,179.04 | 15 | $6,026.86 | 1,2,4,5,7 |
| 06.3.7.- | R/C Caps, Tops of Cells, 3.5' Thick | 3102 | SF | $34.50 | $107,019.00 | 20 | $21,403.80 | 4,5,6,7 |
| 06.3.8.- | Sheet Pile Patching @ $100/Sheet Bare Cost | 908 | Each | $130.00 | $118,040.00 | 30 | $35,412.00 | 4,6,7 |
| 06.3.9.- | R/C for Slide Gate Structures | 19.38 | Cubic Yards | $846.43 | $16,403.81 | 20 | $3,280.76 | 4,5,6,7 |
| 06.3.A.- | Road Surfacing to New Gate Structure | 635 | Tons | $32.26 | $20,485.10 | 30 | $6,145.53 | 1,3,4,5,7 |
| 06.3.B.- | Bridges @ Gates | 1620 | SF | $65.33 | $105,834.60 | 30 | $31,750.38 | 1,3,4,5,6,7 |
| 06.3.D.- | Fill inside of cells, No Removal | 15380 | Cubic Yards | $22.21 | $341,589.80 | 20 | $68,317.96 | 1,2,4,5,6,7 |
| 06.3.E.- | Levee Fill Remainder of Breach | 9000 | Cubic Yards | $13.63 | $122,670.00 | 20 | $24,534.00 | 1,2,4,5,6,7 |
| 06.3.R.- | Guardrail and Fence | 1 | Each | $17,630.00 | $17,630.00 | 30 | $5,289.00 | 1,3,4,5,6,7 |
| 06.3.W.- | Riprap and Bedding Protection | 2650 | Cubic Yards | $24.70 | $65,455.00 | 20 | $13,091.00 | 1,2,3,4,5,6,7 |
| 06.3.Y.- | Demolition of Existing Structure (Torch Cut Gate and Haul Away) | 1 | Lump Sum | $26,020.00 | $26,020.00 | 50 | $13,010.00 | 1,3,4,5,6,7 |
| | SUBTOTAL | | | | $2,098,149.25 | | $399,921.45 | |
| | TOTAL, LAKE CHAUTAUQUA RADIAL GATE REPAIR, WITH CONTINGENCY | | | | $2,498,070.70 | | | |
| | | | | | AVERAGE CONTINGENCY | | 19.06% | |

REASON FOR CONTINGENCIES: 1. UNKNOWN SITE CONDITIONS, 2. UNKNOWN HAUL DISTANCE, 3. UNIT PRICE UNKNOWN, 4. QUANTITY UNKNOWNS, 5. DIFFICULT SITE ACCESS, 6. UNKNOWN FINAL DESIGN, 7. QUALITY CONTROL BY CONTRACTOR