US Army Corps
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SYSTEM SIGNIFICANT COMPONENTS

ENGINEERING RELIABILITY MODELS REPORT

(A Stand Alone Report Compiling Backup Information)

Compiled in July 1997

Engineering Work Group

Engineering Divisions

St. Paul, Rock Island and St. Louis Districts

US Army Corps of Engineers

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1. **Objectives of This Report**. The UMR&IWW System Navigation Study included tasking the three-district Engineering Work Group to determine the expected investment costs to operate the overall navigation system at an acceptable performance level for the 2000 - 2050 planning study period. This tasking was categorized as determining the "Future Without-Project Condition". The expected investment costs for the without-project condition are derived from three contributing sources. The first investment cost source is derived from a projection of the historical Baseline Operation and Maintenance costs. The second investment cost source is derived from the expected costs associated with the engineering/economic reliability assessment analyses of the Future Without-Project Condition of the system significant components. The final investment cost source is derived from the expected costs associated with components not captured via the reliability assessments.

This Component Engineering Reliability Models Report is a compilation of the Engineering Work Group's reliability models for the system significant components. Summaries of these models and their results are contained in the UMR&IWW Navigation Study Feasibility Study - Engineering Appendix. This report serves as a backup information report and is not intended to be a part of the published Feasibility Study. The various Engineering Work Group disciplines along with the location within this report of the respective summary reports of their reliability modeling efforts and results are:

Discipline	Location in Report
Structural Steel	Green Tabs
Geotechnical Structures and Materials	Orange Tabs
Mechanical / Electrical Equipment	Yellow Tabs
Hydraulic Navigation Channel	Blue Tabs

A general description of the component engineering reliability assessment process follows.

2. **Engineering Reliability Model Methodology**. The UMR&IWW system component reliability studies were performed in accordance with the guidance provided in Engineering Technical Letter (ETL) 1110-2-532 "Reliability Assessment of Navigation Structures". This portion of the engineering appendix presents the basic methods and assumptions used to compute the probabilities of unsatisfactory performance of components and the results of the reliability analyses. The methodologies employed by the EWG for the UMR&IWW system reliability analyses are based on the guidance and practices in-place during the 1993-1994 timeframe. More specific information on the reliability analysis for each significant component is contained in the individual model reports.

In Corps of Engineer civil works applications, reliability, *R*, is defined as the probability that a structure, or some significant component of it, will perform satisfactorily at a certain time. The inverse of reliability is the probability that the structure will perform unsatisfactorily. Unsatisfactory performance happens when the limit state for a structure or component is exceeded and the structure or component is then unable to function as designed. In the case of steel structures for locks and dams, the steel structures are designed as movable damming surfaces so that a certain water elevation may be maintained for navigation. If a structure is unable to retain water or is unable to move, it is performing unsatisfactorily and consequences such as a lockage slow down or navigation stoppage could occur.

Several reliability methodologies were used by the EWG with the method applied dependent upon the component classification: structural steel, geotechnical structures/materials, mechanical/electrical equipment or hydraulic navigation channel.

a. Structural Steel. The method used to compute the reliability of structural steel components was developed in the report titled "Reliability Analysis of Hydraulic Steel Structures with Fatigue and Corrosion Degradation", March 1994, written by U.S. Army Corps of Engineers Waterways Experiment Station (WES) and the JAYCOR Company. This method uses the Taylor Series expansion method to compute reliability.

b Geotechnical Structures and Materials. The methods used to compute the reliability of geotechnical structures and materials were developed in the reports titled:

- "Probability Models for Geotechnical Aspects of Navigation Structures", Shannon & Wilson, Inc.

- "Reliability Assessments of Pile Founded Navigation Structures:, St. Paul District, U.S. Army Corps of Engineers

- "Geotechnical Time Reliability Model Report", UMR&IWW Engineering Geotechnical/Materials Work Group

- "Reliability Model of Concrete Deterioration of Lock Walls Due to Freeze-Thaw and Abrasion", Waterways Experiment Station, U.S. Army Corps of

Engineers

- Geotechnical/Materials Reliability Model, Objective 2A; UMR&IWW Navigation Study; U.S. Army Corps of Engineers, St. Louis/Rock Island/St. Paul Geotechnical Engineering Work Group, May 1997

These reports produced time-dependent reliability models. Past unsatisfactory performance events were tabulated into a data base for the geotechnical components. A three parameter Weibull distribution was used to represent unsatisfactory performance events. The data base is representative of the composite navigation system, not any single component.

c. Mechanical/Electrical Equipment. Mechanical and electrical components are typically complex and made up of many different parts, each with several modes of failure. These failure modes are associated with many variables such as operating environment, lubrication, corrosion, and wear. Historic performance data for lock and dam equipment is not usually available nor collected by controlled and tested means. Thus, the reliability analyses of mechanical/electrical equipment were completed through the use of data from larger systematic samples of similar equipment. The component's mean life and failure distribution were synthesized from generalized published failure rate data. The failure rate plotted as a function of time produces a *bathtub curve*[1] of unsatisfactory performance. This reliability curve is described by a two-parameter Weibull distribution.

d. Hydraulic navigation channel. The method attempted to compute the reliability of the hydraulic navigation channel was a dredge-capacity model developed and implemented in the July 1995 report, "Channel Reliability of the Navigation System in the Upper Mississippi River" developed by the University of Virginia for the Corps of Engineers. The model is a capacity-demand model, where the capacity and the demand are represented by probability distributions. The capacity distribution is a function of availability of the dredge(s) in the system. The demand distribution is a dredging demand for a navigation pool and is a function of flow. As discussed in the Engineering Appendix, the navigation channel reliability model was determined to only duplicate costs captured in the baseline operation and maintenance costs for the UMR&IWW system. Thus, the results of the hydraulic navigation channel model were not included in the Feasibility Study's Future Without-Project Condition investment costs summaries. The hydraulic model summary and results are included in this report for historical informational reference purposes.

[1] The bathtub curve can be distinguished by three conditions: early failures, random failures, and wear-out failures. While difficult to construct an actual bathtub curve for a given piece of mechanical/electrical equipment, the curve has been widely used to give an overall picture of the life cycle of many systems, particularly complex equipment systems.

3. **Reliability Assessment Implementation Plan**. The EWG future without-project condition system reliability implementation followed the general plan:

- Develop Component Reliability Model
- Identify UMR&IWW Sites for Model Application
- Determine Component Hazard Function
- Produce Component Consequences to Navigation System
- Construct Event Tree/Tables
- Establish Related Costs

a. Develop Component Reliability Model. The development of the individual component reliability models involved determining the critical members or sub-components for each component to be analyzed and the associated performance mode. Next, the primary failure modes, or limit states, for each critical member were determined. The developed reliability models calculate the probability of unsatisfactory performance for a component as a function of time. The individual reliability models are summarized later.

b. Identify UMR&IWW Sites for Model Application. The significant components selected for system reliability analysis are common to a majority of the 37 lock and dam sites in the UMR&IWW system. To determine the future without-project condition major rehabilitation system needs, a reliability analysis of each significant component at each site is required. The component hazard function and consequences at each site are needed to determine the optimal economic timing of rehabilitation. However, it was noted that the design, function, and usage of many of these components are very similar from site to site on the UMR&IWW system. Thus, to avoid duplication of effort, the EWG identified those sites where each of the significant components are similar. These sites were subsequently grouped under a common reliability analysis for that particular component. The site groupings are summarized later for each component.

c. Determine Component Hazard Function. The future without-project condition of the UMR&IWW system will vary over the next 50 years. Development of a component (structure or piece of equipment) hazard function is a key step in reliability assessment, which may lead to potential justification of major rehabilitation capital investment. Component hazard functions, which provide time dependent probability of satisfactory and unsatisfactory performance, were developed for each component under study. This function, h(t), represents the instantaneous hazard rate at which unsatisfactory performance occurs, given that unsatisfactory performance has not been demonstrated previously up to that point in time. Time dependency is addressed by defining the functions on a per annum basis. The hazard rates provide a present value and time functions for three cases; a normal O&M (unrehabilitated) hazard function, a hazard function after rehabilitation, and an enhanced maintenance[2] hazard function. Under the

[2] An enhanced maintenance study objective assesses the benefits and costs to the future condition of the navigation system given an increased level of maintenance. This enhanced level of maintenance assumes unconstrained funding, thereby allowing for meeting the needs of the O&M program to restore condition standards and performance levels.

enhanced maintenance condition, components are maintained on an augmented regular schedule in order to prolong their useful life. For example, miter gates or roller gates may be painted every 15 years instead of every 25 years to minimize the effects of corrosion. Only structural, mechanical and electrical components were considered for enhanced maintenance; geotechnical components were determined to receive no appreciable benefit from enhanced maintenance or maintenance thereof was not applicable.

d. Produce Component Consequences to Navigation System. A parameter included in the component ranking was the "system consequence". A primary factor in making a component significant in the overall UMR&IWW system is that a physical consequence results which has a significant adverse economic impact on navigation. Consequences include the cost of down time to navigation, the repair costs to remedy a component's unsatisfactory performance, along with other factors such as environmental impact costs. For navigation study purposes, consequences were considered to be constant with respect to time. Down time to navigation involves the number of hours or days that navigation will be delayed or be slowed down due to failure of a component. Navigation will be interrupted when lock components perform unsatisfactorily or when the navigation pool has been lost or significantly lowered due to a failure of a dam component. The EWG determined the time impact and repair/rehabilitation costs; the monetary costs to the navigation industry was determined as part of the economic model. The repair costs include labor cost of the repair crew, mobilization costs, material costs, and other miscellaneous costs.

Simple equations or methods to quantify the physical consequences do not exist. Therefore, UMR&IWW consequences were typically based on experience and engineering judgment. However, the EWG formulated several factors such that a consistent measure of consequences could be made. These factors are different for lock components and dam components. The lock components affect navigation directly and have an immediate impact. These impacts may have a long or short duration, but the component needs to be repaired or replaced before navigation can return to normal. Several of the consequence factors considered for locks include:

- Is the component redundant (internal and external)? If a component's structure has redundant elements, the chance of overall component failure due to failure of one of the elements is small. For instance, if a vertical beam on a miter gate reaches yield, the other beams may be able to carry some of the load. Another example is lock tainter valves. Generally, there are two culverts and two sets of tainter valves in a lock. If one tainter valve fails, the lock can operate with only the other set of valves, but at a slower rate.

- Will a full maintenance crew be needed to repair the component or can lock personnel repair the component? Typically, for major structural failures, a maintenance crew and floating plant with heavy equipment will be necessary.

- Are spare components available? Spare miter gates exist for most vertically framed miter gates on the UMR. Hence, downtime would likely be limited to the amount of time it takes for a maintenance crew to mobilize, pull the damaged gate, and install a spare gate.

- Is it likely lock personnel will notice the problem prior to an actual failure? Timely advance action may lessen the navigation downtime.

Dam components typically only affect navigation when the pool can not be maintained. Most often, navigation is not affected, but if pool is lost, navigation will be interrupted for an extended period of time. Several consequence factors considered for dams include:

- Can lock personnel install bulkheads before loss of pool?

- Can pool be maintained temporarily by adjusting other gates? For large rivers such as the Mississippi River, gate settings of non-affected gates can typically be changed to regulate the flow. For smaller rivers such as the Illinois River, the loss of a single gate during low flow may lead to a loss of pool.

- Is there a high probability of multiple gate failure? Under these conditions, loss of pool is more likely since there may not be enough bulkheads to block all failed gate bays.

- Is the component redundant? Similar to lock components, redundant structures can often survive when a single element fails.

- Could gate failure cause scour and eventual failure of the dam? Severe scour represents a worst case scenario. The navigation pool would be lost for an extended period.

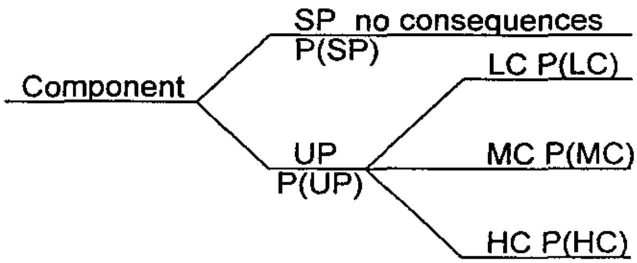
e. Construct Event Trees. In performing reliability analyses, it is desirable to consider different levels of consequences since the actual consequence of failure is unknown. By considering different levels, one can account for different outcomes if probabilities can be associated with each of the outcomes. Such probabilities are defined as conditional probabilities. In current lock and dam applications, these conditional probabilities are based solely on experience and engineering judgment. Conditional probabilities are the probabilities that a particular consequence occurs given that unsatisfactory performance has occurred. Typically, lower levels of consequences have a higher probability of occurring. Figure ENG-1 shows an event tree[3] with different levels of consequences. Development of a component event tree is a key step in the reliability assessment

[3] Event trees were first developed for identifying significant sequences associated with nuclear power plant accidents (circa 1975). Since the initial development, event trees have been used on other risk and uncertainty applications. In recent years, Corps of Engineers' major rehabilitation reporting guidance has required the use of event trees to describe events of unsatisfactory performance and resultant consequences.

process. An event tree provides a framework for economic analysis and defines the required input parameters. An event tree is a graphical device used to analyze risk, that is, the expected consequences based on one of several uncertain events. It allows one to follow the logic in the determination of a component's condition and the impact experienced if it performs unsatisfactorily or satisfactorily. The event trees in this study were compiled with input from many of the participating District's resources.

Figure ENG-1

Event Tree



where:
SP = Satisfactory Performance
UP = Unsatisfactory Performance
P(X) = Probability of X Event
LC = Low Consequence
MC = Medium Consequence
HC = High Consequence

Ideally, event trees show all possible combinations of events. Such comprehensiveness may reveal failure sequences that might otherwise have been overlooked. However, showing all combinations of events on the UMR&IWW system would result in an unmanageably large tree. Additionally, numerous unlikely or remote consequences divert attention from reasonable sequences of events. Thus, an important decision in an event tree development involves the level-of-detail, how many events should be included and what range of conditional probabilities. For the Navigation Study, the EWG typically established three levels of consequences and associated conditional probabilities for developing event trees. Event tree inputs include the probability of unsatisfactory performance at a given time and the likelihood and magnitude of potential consequences associated with satisfactory and unsatisfactory performance. The generic event tree is developed by assigning a branch for each potential event related to satisfactory and unsatisfactory performance. Generally, no consequences are associated with the satisfactory performance event branch. However, likely consequences must be considered for unsatisfactory performance. Potential consequences are typically placed into three categories: low, medium, and high, which result in three additional branches of the event tree. The likelihood of each consequence is then addressed and assigned a probability of occurrence.

These event trees are summarized in tabular format in this report. If event data could not be compiled to establish the conditional probabilities, the recommended consequence conditional probabilities are listed below.

P(LC) = Probability of a Low Consequence = 0.90
P(MC) = Probability of a Medium Consequence = 0.09
P(HC) = Probability of a High Consequence = 0.01

The assessment of consequences is undertaken for each branch of the event tree to address the likelihood and risk of satisfactory or unsatisfactory performance, physical consequences, and impacts to navigation. For example, if an engineering performance criterion based on the yield strength of a major structural steel member is exceeded, unsatisfactory performance occurs. Significant physical and navigation consequences may result, but depend on the likelihood (conditional probability) of subsequent events. If, however, the probability of unsatisfactory performance is zero, there is no chance of physical consequences and subsequent impacts to navigation. This indicates a reliable structure for that particular year.

f. Establish Related Costs. Finally, associated costs of each consequence were quantified by the Engineering and Economic Work Groups. These costs capture all pertinent repair costs and navigation delays incurred in order to return the component back to satisfactory performance, as well as the lost benefits to the navigation industry, if any. Thus, each branch was assigned a dollar value, which represents the total cost associated with that particular event. Historical costs were used to calibrate cost estimates for repairs. The cost values presented in the tables are in year 2000 dollars.

4. **Overview of Economic Models Related To Engineering Reliability Assessments**. The component hazard rates and event trees served as the input for the economic models. A Monte Carlo simulation was developed for each component to identify potential consequences to quantify rehabilitation and identify repair costs. Each economic model attempts to determine the level of repair that is warranted and when over the 50-year study period. The specifics of the economic models are detailed in the Economic Appendix, *Analysis of Future Investment Needs on the Upper Mississippi River and Illinois Waterway (Objective 2A).*

The economic models were based on simulation runs through the 50-year study period for each component studied. They essentially analyze a "built-up" component event tree developed by placing the start of the next year's event tree at each ending branch of the current year's event tree. The generic, four-branch event tree was stacked onto each terminal point of the cumulative event tree generated to date in order to address the potential events and consequences year after year. This represents 4^{50} possible end states that could be attained via a unique path through the component's event tree covering the 50-year study period. Only a sample of complete runs through the event tree was required to sufficiently converge on the expected hazard rate values for the entire study period.

As the model proceeded down a path for a simulation run, it generated unsatisfactory performance (failures) and consequences in any given year. Some measure of repair will be required if unsatisfactory performance occurs, and some cost will be incurred. Once a repair is made the hazard rates for the following years were adjusted, and the process continued. Hazard rate adjustment significantly lowers the likelihood or unsatisfactory performance. Finally, the end state at year 50 was reached and all costs related to consequences were discounted to present value. This process was repeated a sufficient number of times until convergence with respect to consequences occurred. This constitutes the Base Condition –Without Rehabilitation case. The same procedure was followed for the Project Condition – with Rehabilitation case. The only difference was that rehabilitation repairs were undertaken at selected times in the project life, which could be undertaken to prevent some or all of the consequences. The cost of the planned rehabilitation repairs was then added to consequences that could potentially occur before and after planned rehabilitation. The Base Condition's present value cost was then compared to the Project Condition's present value cost, which will have different values depending on when the planned rehabilitation was assumed to occur. When the Base Condition's present value cost is lower than Project Condition's present value costs throughout the study period, the project (planned rehabilitation) is not justified. If the opposite occurs, the project is justified, and benefits are maximized in the year where the Project Condition's present value cost minus the Base Condition's present value cost is the greatest. Hence, the simulation involves a whole continuum of time, and investment may justify early or late in the 50-year study period or not at all.

It is important to recognize the interpretations of the economic model output since it was capable of addressing expected consequences on a year by year basis. If the benefits/costs ratio does not exceed 1.0 at any point in the study, from an economic standpoint at least, it would be optimal to allow the component to reach the state of unsatisfactory performance and incur the physical problems, repair cost, and navigation consequences. Since one can predict through the model when these consequences are likely to occur, it implies, technically, that repair funds over and above Baseline O&M levels would be necessary at some point in the future, even though major rehabilitation capital investment would not have been justified (due to the B/C ratio being less than one). This implies a "fix as you go" type strategy that incorporates no preventative major capital improvement measures. Thus, future costs to ensure a given level of performance are still necessary. These costs, referred to as reliability repairs, were captured in the model and presented as present value life cycle costs in the Economic Appendix section titled *Analysis of Future Investment Needs on the Upper Mississippi River and Illinois Waterway (Objective 2A).*

strategy that incorporates no preventative large scale capital improvement measures. Thus, future costs to ensure a given level of performance are still necessary. These costs, referred to as reliability repairs, were captured in the model and presented as present value life cycle costs in the Economic Appendix section titled *Analysis of Future Investment Needs on the Upper Mississippi River and Illinois Waterway (Objective 2A).*

SYSTEM SIGNIFICANT COMPONENTS

ENGINEERING RELIABILITY MODELS REPORT

(A Stand Alone Report Compiling Backup Information)

RELIABILITY MODELS
FOR
STRUCTURAL STEEL STRUCTURES

Engineering Divisions

St. Paul, Rock Island and St. Louis Districts

US Army Corps of Engineers

Executive Summary

Purpose

1. As part of the Upper Mississippi River Navigation Study, Objective 2A, reliability of the steel structures on the Mississippi River and Illinois Waterway locks and dams were computed. This report will present the basic methods and assumptions used to compute these reliabilities and the results of the reliability analyses. This portion of the report contains a general description of the structure types that were investigated and the methods used to compute reliability. More specific information on the reliability analysis for each structure type is contained in the sections which follow.

Structure Types

2. There are five different types of steel structures that are present at locks and dams in the Upper Mississippi River. Structure types associated with the lock portion of the lock and dams are miter gates, tainter valves (culvert valves), and lift gates. Steel structures associated with the moveable dam portion of the locks and dams are tainter gates and roller gates. Although all locks have miter gates and all Mississippi River locks have tainter valves, at any given site, the other structure types may or may not be present and their numbers vary as well.

Reliability and Unsatisfactory Performance

3. For the purposes of this study, reliability, *R*, will be defined as the probability that a structure will perform satisfactorily at a certain time given that it has performed satisfactorily up to that time. The inverse of reliability is the probability that the structure will perform unsatisfactorily over a given time interval and in this report this will be called the hazard function. Unsatisfactory performance happens when the limit state for the structure, or some major component of it, is exceeded and the structure is then unable to function as designed. In the case of the steel structures for the locks and dams, the steel structures all are designed as movable damming surfaces so that a certain water elevation may be maintained for navigation. If the structures are unable to retain water or are unable to move, they have performed unsatisfactorily and consequences such as slow down or stoppage in navigation could occur.

Computation of Reliability

4. The basis of the method used to compute reliability for this study was developed in the report titled "Reliability Analysis of Hydraulic Steel Structures with Fatigue and Corrosion Degradation", March 1, 1994, written by U.S. Army Engineers Waterways Experiment Station (WES) and the JAYCOR Co. This method uses the Taylor Series - finite difference estimation method to compute reliability. The general procedure used will be explained in the paragraphs which follow. The exact procedures used can be found in this reference and it is beyond the scope of this

report to describe them in detail.

5. To compute reliability, first the critical members and limit states were identified for each structure. The parameters needed to compute the factor of safety of the members were then identified. Parameters for which values are uncertain, called random variables, were identified and a statistical distribution giving a mean and standard deviation was determined for each random variable. The distributions used for this study were determined from data published in the WES-JAYCORP report, from other published data, from data found from the site or records from the site, or from engineering judgement. Some random variables for which the statistical distributions could not readily be determined but which had little influence on the factor of safety were considered constants. The sections which follow explain in detail the random variables used for each structure type and how they were determined.

6. Next, factors of safety were determined for the critical members with each random variable varied individually one standard deviation above and below the mean value for that variable. From these factors of safety, a reliability index, β, was determined. The index β is the number of standard deviations between the average expected performance of a structure and its limit state. From β the reliability, R, was computed. As stated previously, R is the probability that the structure will perform satisfactorily in a given time period. Since some of the random variables usually vary with time, the β and its corresponding R were computed by year up to the year 2050. The next step was to convert R by year into a hazard function, the probability that the structure will have unsatisfactory performance in a given year.

7. The lock and dam structures have two basic limit state types for which final hazard functions were computed in different ways. The two limits states are the strength limit state and the fatigue limit state. The strength limit state occurs when the loading in a member, such as flexure, compression, or tension, is greater than its capacity in material strength or member stability. The fatigue limit state occurs when repeated load cycles in a member create a crack which weakens it and subjects it to further damage by fatigue or lowers its capacity for the strength limit state. All structural types are subject to potential unsatisfactory performance due to the strength limit state, but only structures seeing significant cyclic loadings, such as from lockages, are subject to the fatigue limit state. Therefore, dam structures were analyzed for the strength limit state and lock structures were analyzed for the both the strength and the fatigue limit states.

8. For the strength limit state, loadings used to compute the reliability index, β, were computed by finding the statistical distribution for the maximum loading that would occur in a year. The reliability, R, that was computed from this loading represents the probability that the structure will have satisfactory performance in the year for which the reliability is computed and is independent of loadings that occurred in previous years. The hazard function, or probability of unsatisfactory performance, is therefore equal to $1 - R$.

9. For the fatigue limit state, the reliability computed at a given year is dependent on the loadings in the previous years of the structures life since the fatigue limit state is dependent on the number and magnitude of the stress cycles experienced by the structure for its entire life up to that point in time. The reliability computed is therefore a cumulative probability that the structure will survive up to that point in time. The Weibull function was used to convert the cumulative reliabilities computed every year into a hazard function which gives the probability of unsatisfactory performance in a given year.

10. Reliability models were developed for each structure type. In some cases, more than one model was needed to analyze different components or different limit states of a single structure type. In other cases, several different models were needed to account for different types of structural systems used for the same structure type at different sites. The models that were developed are described in the sections that follow.

Results

11. Probabilities of unsatisfactory performance for each structure type are summarized in the sections which follow. Probability of unsatisfactory performance has been computed for the years 2000 to 2050 for a normal O&M case assuming that maintenance practices done in the past will be done also in the future and for an enhanced maintenance case where additional maintenance is done in the future. Reliability numbers have also been computed for the years after which the structure has been rehabilitated. Since there is similarity between structures at many sites, structures were grouped together where possible to reduce the amount of computations needed.

12. The consequences of each structure type having unsatisfactory performance are summarized in the section for each structure type. Consequences for navigation range from relatively little, such as if a tainter gate were to suffer minor damage from ice in the middle of winter during which navigation was shut down and the pool was not lost, to major, such as if a miter gate were to fail and repairs stopped navigation for many days during a busy navigation period or if a roller gate were to collapse and a pool was lost. Since what the actual consequences would be is uncertain, three possible scenarios of consequences have been developed with different probababilities of occurrence for any one instance of unsatisfactory performance. The scenarios listed below with their corresponding probability of occurrence are:

1. Probability of Low Consequence ~~Consequences with high probability of occurrence~~ - 90%.
2. Probability of Medium Consequence ~~Consequences with medium probability of occurrence~~ - 9%.
3. Probability of High Consequence ~~Consequences with low probability of occurrence~~ - 1%.

The consequences which correspond to any one of the three possibilities are listed in the sections for each structure type. but consequences with a higher probability of occurring are lesser than the consequences with a low probability of occurring.

13. Cost are listed in each following section for repair of a structure that has suffered unsatisfactory performance, for rehabilitation of the structures, and for enhanced maintenance.

14. For lock and dam sites where the reliability of the structure was very high until the year 2050 and the probability of unsatisfactory performance was considered to be insignificant, no results are given. For the economic analysis the reliability can be considered equal to one and the hazard function equal to zero. There are no costs associated with these structures.

Upper Mississippi River Navigation Study

Objective 2A

Structural Reliability

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SECTION 1 - Miter Gates

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Reliability Analysis of Miter Gates

I. Model Description

General

1. References The following publications were utilized in the probabilistic analysis of the miter gates.
a. WES & JAYCOR (March 1994): Reliability Analysis of Miter Gates
b. Ellingwood, B.R. (1993): Load and Resistance Factor Design for Steel Miter Gates, WES Report ITL-93-4.
c. Ellingwood, B.R. (July 1995): Engineering Reliability and Risk Analysis for Water Resources Investments; Role of Structural Degradation in Time-Dependent Reliability Analysis, WES Report ITL-95-3
d. EM 1110-2-2703 (1994): Lock Gates and Operating Equipment.
e. Lock Gate Operating Forces, Locks 2-5A & 10 (January 1989): St. Paul District, US Army Corps of Engineers.

2. There are many members or sub-components in a miter gate. It was decided that only those components which would either have a higher chance of unsatisfactory performance or major consequence would be analyzed in detail. After preliminary review and using engineering judgement by the objective 2A team members, it was concluded that the following members would be analyzed.

(i) Vertically Framed Miter Gates (VFMG):
Vertical beams, vertical girders, and top horizontal girder.
Gate anchorage

(ii) Horizontally Framed Gates (HFMG):
Horizontal Girders and Gate Anchorage.

Since there are many horizontal girders in a HFMG, only a few representative girders are analyzed. The girders above the upper pool level undergo atmospheric corrosion and resist lighter loads compared to the girders below the pool level which are subjected to submerged corrosion. The spacing of the girders is another variable that affects the loading on horizontal girders. A summary the miter gate data is given in the table "Miter Gates Data."

3. Reliability of members was calculated for various limit states and the limit state that produced the lowest reliability was assumed to control the member. Calculation of system reliability, such as the reliability of the total gate, was not attempted. After discussions with the economists involved in the study, it was concluded that determining the reliability of individual components together with appropriate consequences was sufficient to carry out the risk simulation model. Elements of a miter gate deteriorate with time due to fatigue damage accumulation and corrosion. Corrosion occurs when there is no protection by an effective paint coating.

Description of Models

4. Vertically Framed Miter Gate: Other than those at the ends, vertical beams and vertical girders carry the same amount of loading. However, vertical beams have smaller cross-sections and control the limit state. For this reason, only vertical beams were analyzed. In the horizontal girders, the bottom girder simply transfers the loads on to sill and does not undergo bending actions. In a VFMG, top horizontal girder carries the loads transferred by vertical beams. Top girder behaves like a member in a three-hinge arch and resists axial and flexural loads.

5. Horizontally Framed Miter Gate: In a HFMG, all horizontal girders ~~Except the bottom one~~ resist axial and flexural loads. Behavior of the girders is very similar to the top horizontal girder in a VFMG. Hydraulic loading on girders depends on the location (distance from top) and the spacing of girders.

6. Miter Gate Anchorage: Steel anchor bars transfer the gate reaction to anchorage channels which in turn transfer it to the concrete monoliths. Anchor bars are loaded in tension and unsatisfactory performance of these occurs due to fatigue cracking. Each miter gate leaf anchorage has two anchor bars. When the gate is open (i.e., in the recessed position), anchor force is taken by one of the two bars. When the gate is nearly closed (i.e., mitered position), anchor force is taken ~~almost~~ by the second ~~other~~ bar. The second ~~This~~ bar carries the higher load and therefore, the reliability was computed for the ~~this~~ bar which is loaded when the gate is closed. Cracking of a bar requires replacement of it with a spare bar. Permanent fix of replacing the bar would necessitate the shutdown of navigation for ~~a few~~ four days.

miter gate anchorage

Table 1 Miter Gate Data

Miter Gates	V.F.= vertically framed miter gate H.F.=horizontally framed miter gate			
Lock	Type of Framing	U.P.-T.W.(ft) (Design Head)	GATE HEIGHT (ft)	
			U.S.	D.S.
Upper Miss.				
2	V.F.	12.2	21	32
3	V.F.	8.0	25	30
4	V.F.	7.0	20	23
5	V.F.	9.0	20	23
5A	V.F.	5.5	27	27
6	V.F.	6.5	23	25
7	V.F.	8.0	23	25
8	V.F.	11.0	27	30
9	V.F.	9.0	27	33
10	V.F.	8.0	25	30
11	V.F.	11.0	25	30
12	V.F.	9.0	25	30
13	V.F.	11.0	25	30
14	V.F.	11.0	23	27
15	V.F.	16.0	32	32
16	V.F.	9.0	23	27
17	V.F.	8.0	25	30
18	V.F.	9.8	23	30
19	H.F.	36.2	lift gate	

20	V.F.	10.0	20	27
21	V.F.	10.5	27	33
22	V.F.	10.2	27	33
24	V.F.	15.0	25	33
25	V.F.	15.0	27	35
Mel Price Main	H.F.	12.5	lift gate	53
MP Auxlock	H.F.	12.5	57.5	53
27 Main Lock	H.F.	11.4	lift gate	70
27 Auxlock	H.F.	11.4	lift gate	70
IWW				
Lockport	H.F.	41.0	lift gate	58.6
Brandon Road	H.F.	23.0	19.5	47.5
Dresden Island	H.F.	14.0	19.5	34.8
Marseilles	H.F.	13.3	19.5	37.3
Starved Rock	H.F.	17.0	19.5	35.7
Peoria	V.F.	5.8	17.5	25.0
LaGrange	V.F.	5.8	17.5	25.0

Loads and Performance Modes

7. In both type of gates, hydraulic loading was considered. Impact load was not considered in the reliability analysis. Fatigue damage occurs in a steel component when it undergoes cyclic loading in tension. Other type of deterioration that occurs in steel structures, especially in a marine environment is corrosion. Degradation due to corrosion can be prevented by a periodic painting program, which has been the case with some sites. Corrosion was modeled as suggested in the WES report.

8. In vertical beams and horizontal girders, two performance modes (or limit states) were considered. One is the fatigue limit state and the other is the bending limit state. The limit state with the lower reliability controls a particular component. However, in cases where painting has been done on a reasonable frequency, fatigue limit state became the controlling case with time.

9. For both limit states, structural analysis was first done deterministically. Under the assumptions that vertical beams are pinned at the ends and the horizontal girders act as part of a three-hinge arch, closed form solutions were obtained for bending moments and axial forces. Reliability indices were calculated by the Taylor Series method using the random variables as described under Random Variables.

10. For anchor bars, loading (tensile reaction) comes due to the weight of a gate and the force in the strut arm. Gate weights were taken from the data in the as-built records. All the gates in the UMR were looked at and weights for similar gates were compared for accuracy. Strut arm forces were computed in accordance with the method shown in Ref. 1e.

Random Variables

11. Many random variables were used in determining the reliability of structural elements. Some of the variables are based on the report by WES & JAYCOR (1994).

12. Yield Strength of Steel: The ratio of Mean Value to Nominal Value(F_y), and the ratio of standard deviation to mean value (known as coefficient of variation, c.o.v.) depend on the limit state under consideration (Ellingwood, 1993). For the bending mode, mean value is 1.08*Fy and c.o.v. is 0.14.

13. Corrosion: The amount of corrosion, c, in mm was modeled as (WES & JAYCOR, 1994)

$$\log c = \log A + B \log t + \epsilon_c,$$

where ϵ_c has a mean value of 0. Values of A, B and standard deviation of ϵ_c depend on the environmental conditions as shown below.

	A	B	std. deviation of ϵ_c
Splash Zone	148.5	0.903	0.099
Submerged Zone	51.6	0.65	0.174
Atmospheric	23.4	0.65	0.219

Components of a gate were considered to corrode whenever the paint was not effective. Painting history was obtained from site specific records.

14. Fatigue Damage: Fatigue damage was evaluated using Miner's hypothesis as described in the WES-JAYCOR Report. There are two random variables associated with it: ϵ, fatigue strength correction and Δ, damage accumulation factor. Mean for ϵ and Δ is 0 and 1.0, respectively. Standard deviation for ϵ and Δ is 0.31 and 0.3, respectively.

15. Ratio of lockages to actual stress cycles (Kc): This variable enables the number of stress cycles to be computed from the number of lockages at a given lock site. Mean and standard deviation of Kc vary with site.

16. Hydraulic Loading: Loads on a miter gate is caused by the differential head between the upper pool and the lower pool. Upper pool was treated deterministic and the differential head was taken as a random variable. During part of a year (in winter), some locks are shutdown; however, fatigue damage depends on the head as well the number of cycles a gate undergoes at the particular head. Therefore, in evaluating the fatigue damage, weighted head (with respect to number of lockages) was used. In the flexural mode, the maximum head was used. The statistics of the maximum head is such that the probability of occurrence is one (100%) per year. Records of hydraulic data are available for all the sites for some extended period of time which makes the statistics very reliable. Records from daily/monthly were converted into yearly and the yearly values were used in the analysis because the time step used in the modelling is one year.

17. Stress Uncertainty Factor K_s (Ratio of actual force to computed force). This variable account for the modelling error in the structural analysis. It has been determined and reported by WES as follows.

Component	Mean	std. dev.
VFMG: Vertical Beam	0.964	0.120
VFMG: Top Girder	1.380	0.210
HFMG: Horizontal Girder	0.880	0.140

Reliability Analysis

18. Analysis of the gate elements was done on spreadsheet using Lotus 123. The model was developed such that most of the calculations are performed automatically. Different versions were developed for vertical beams (VVBEAM) and top horizontal girder (VHGIRD) in a VFMG and horizontal girders (HFGIRD) on a HFMG. VVBEAM, VHGIRD and HFGIRD are the spreadsheet programs that were developed.

19. First, all necessary data are input. This includes geometrical properties, material properties, etc. Also, lockage and head data are input into the spreadsheet by reading from external files. Data files contain the projected information for the future years until the study time period.

20. Forces in the components are calculated using the closed form solutions in the spreadsheet. Then the reliability for both limits are calculated at each time step. The limit state for bending was defined as the onset of extreme fiber yielding. The limit state for fatigue was defined as reaching the damage level as defined by Miner's rule.

21. Anchor Bars: Information was collected on geometry, gate weight, strut arm loading and lockage. Gate reactions from dead weight were computed and added to the strut arm force in order to determine the maximum tension in the anchor bar. The maximum force was assumed to be taken by only one of the anchor bars. Theoretical stresses gave low values in the anchor bar. Calibrating it with a finite element analysis, it was concluded that theoretical stress (given by formulas) should be multiplied by 1.5 to get a reasonable estimate. This is due to bushing attachment holes. Reliability index and a corresponding reliability were calculated at one-year time interval.

22. Hazard Function: Using the time dependent reliability, a regression analysis was done and a Weibull distribution was fit. Hazard function was then established using the Weibull parameters.

II. Site Selection

23. The study involves Locks 2 through 27 in the Upper Mississippi River (UMR) and 7 locks in the Illinois Waterway (IWW). In the UMR, Locks 2 through 10 are in the St. Paul District, Locks 11 through 22 are in the Rock Island District and the remaining four locks in the St. Louis District. All the seven locks in the IWW are in the Rock Island District. Five locks in IWW, and Melvin-Price and Lock 27 in UMR have horizontally framed miter gates. Others have vertically framed gates. Older gates are rivetted structures whereas the newer ones are welded structures.

24. Locks in the same District go through identical maintenance schedule; further, adjacent locks in a river system undergo similar lockage cycles. Locks in the very upper reach of Mississippi, such as those in the St. Paul District, have less number of load cycles compared to those in the lower reach. Therefore, in the upper reach of UMR only every other lock was evaluated in detail. Because the results showed that gates are reliable until 2050 under the present O&M pattern, remaining locks in the St. Paul District were not analyzed. Similar conclusion was reached for the UMR locks in the Rock Island District. All four UMR locks in the St. Louis District were evaluated.

25. Illinois Waterway: The locks from the upper reach of the river are Lockport (LP), Brandon Road (BR), Dresden Island (DI), Marseilles (MA), Starved Rock (SR), Peoria (PO) and LaGrange (LG). Peoria and LaGrange have almost identical, vertically framed miter gates and undergo similar loading. Therefore, only one (Peoria) lock was evaluated. BR, DI, MA and SR have identical horizontally framed miter gates (HFMG) at upstream with the same height and identical loading. Therefore, only one of these (BR) gates was evaluated. In the downstream side, only DI and SR have similar gates and loading. LP, BR, MA and SR were evaluated for the downstream HFMG. SR has rivetted HFMG and all others have welded gates.

	Lock Gates Investigated	Other Similar Locks
UMR		
VFMG:	2	-
	4	5
	6	7
	8	5A, 9

	10	3
	12	11, 13, 18
	15	14
	17	-
	20	16
	22	21
	24	-
	25	-
HFMG:	New 26 (Melvin-Price)	-
	27	-

<u>IWW</u>

VFMG:		
	Peoria	LaGrange
HFMG:		
	Lockport d/s	-
	Brandon Road u/s	DI, MA, SR (u/s)
	Brandon Road d/s	-
	Marseilles d/s	DI, SR (d/s)

26. Anchor Bars: All the anchor bars in the St. Paul and St. Louis Districts have been either replaced or planned to be replaced soon. When they are replaced, the bushing attachment holes are realigned and as such, anchor bars don't show any sign of unsatisfactory performance. Within the Rock Island, it is the same case on the IWW. In the UMR in the Rock Island District, Locks 12, 14, 17, 18, 19, 21 and 22 have the original anchor bars. Lock 12 was previously analyzed and the results are included in this report. Looking at the gate heights and the submergence of gates, it was observed that 17 and 18 are similar; 21 and 22 are similar. Therefore, 17, 20 and 22 were the three cases that were analyzed for anchor bars. Anchor bars at 14 are scheduled to be replaced in 1996 and are not analyzed here.

II. Hazard Functions

27. In the vertically framed miter gates, only those at the Locks 24 and 25 resulted in unsatisfactory reliability. At all other sites, the gates' probability of unsatisfactory performance (PUP) is very low until year 2050 for the performance

modes analyzed. Since the PUP is very low, hazard function is almost zero, or less than 0.000001. For within the accuracy of the effort, hazard may be considered zero. Lock 25 is slated for rehabilitation at present. Therefore, it will be satisfactory till 2050 once the rehabilitation is complete. Only lock 24 was analyzed for enhanced maintenance and rehabilitated conditions.

28. In a horizontally framed gate, representative number of horizontal girders were analyzed and the worst case was considered to control the reliability of the gate. Due to inappropriate weld details, some girders become less reliable very quickly.

29. Reliability indices for selected sites are listed in the following table. The reliability index is shown for comparison purposes at three time points. The time points shown are 1940 or the first year in service, year 2000 and year 2050.

Lock Site	Member		Reliability Index (beta) 1940(*)	2000	2050
4	Vert. Beam	u/s	11.2	5.66	4.55
		d/s	11.8	6.43	5.29
8	Vert. Beam	u/s	10.2	5.24	4.08
		d/s	10.5	5.45	4.30
12	Vert. Beam	u/s	10.6	5.20	3.84
		d/s	11.2	6.05	4.53
17	Hor. Girder	u/s	14.1	9.10	7.82
		d/s	12.8	7.81	6.52
20	Hor. Girder	u/s	10.4	6.18	4.94
		d/s	8.41	4.12	2.89
22	Vert. Beam	d/s	9.54	4.04	2.76
	Horiz.Girder	u/s	10.4	5.64	4.42
		d/s	8.83	4.00	2.79
24	Vert. Beam	u/s	7.92	1.09	0.00
	enhan. maint.			3.75	2.79
	Hor. Girder	u/s	7.92	3.12	2.05

25	Vert. Beam u/s	7.74	1.24	0.07
	Hor. Girder u/s	7.95	3.38	2.33
	d/s	13.3	8.18	7.92
M-P (26)	Hor. Girder d/s (cat. C weld)	11.11 (1990*)	7.51	4.75
27	H. Girder 10 d/s	14.7 (1963*)	9.73	7.74
Peoria	Vert. Beam u/s	9.78	3.75	2.53
	d/s	9.79	4.09	3.09
	Hor. Girder d/s	13.7	8.29	7.43
BR u/s	Girder 1	7.85(1935)	3.43	2.58
BR d/s	Girder 6 (cat C weld)	9.3(*1996)	6.76	2.49
	Girder 10 (cat E weld)	8.39	5.70	1.30
	Gir.10 (cat.C)	10.2	7.61	3.35
Marseilles d/s	Girder 8 (cat C)	11.8(*1996)	9.19	4.85
Lockport d/s	Horiz. Girder	10.0(*1985)	6.0	3.2
SR d/s	Girder 8	18.0(*1936)	8.0	7.0

30. As it can be seen from the reliability indices in paragraph 29, most sites had high reliability. Thus, hazard rate was zero for most sites. Only the sites where hazard rate was significant are listed below. Further, note that these values are per gate leaf and there are two gate leafs at each location (i.e., up stream and down stream).

Hazard Function		Lock 24 d/s vert.beam		
Year	Normal O&M	Enhanced	Rehabbed.	Year from rehab
2000	0.000008	0.000003	0.000000	1
2001	0.000008	0.000003	0.000000	2
2002	0.000009	0.000003	0.000000	3
2003	0.000010	0.000003	0.000000	4
2004	0.000010	0.000003	0.000000	5
2005	0.000011	0.000003	0.000000	6
2006	0.000011	0.000004	0.000000	7
2007	0.000012	0.000004	0.000000	8
2008	0.000013	0.000004	0.000000	9
2009	0.000014	0.000004	0.000000	10
2010	0.000014	0.000004	0.000000	11
2011	0.000015	0.000005	0.000000	12
2012	0.000016	0.000005	0.000000	13
2013	0.000017	0.000005	0.000000	14
2014	0.000018	0.000005	0.000000	15
2015	0.000019	0.000006	0.000000	16
2016	0.000020	0.000006	0.000000	17
2017	0.000021	0.000006	0.000000	18
2018	0.000022	0.000006	0.000000	19
2019	0.000023	0.000007	0.000000	20
2020	0.000024	0.000007	0.000000	21
2021	0.000025	0.000007	0.000000	22
2022	0.000027	0.000007	0.000000	23
2023	0.000028	0.000008	0.000000	24
2024	0.000029	0.000008	0.000000	25

2025	0.000031	0.000008	0.000000	26
2026	0.000032	0.000009	0.000000	27
2027	0.000034	0.000009	0.000000	28
2028	0.000035	0.000009	0.000000	29
2029	0.000037	0.000010	0.000000	30
2030	0.000038	0.000010	0.000001	31
2031	0.000040	0.000010	0.000001	32
2032	0.000042	0.000011	0.000001	33
2033	0.000044	0.000011	0.000001	34
2034	0.000046	0.000012	0.000001	35
2035	0.000048	0.000012	0.000001	36
2036	0.000050	0.000012	0.000001	37
2037	0.000052	0.000013	0.000001	38
2038	0.000054	0.000013	0.000001	39
2039	0.000056	0.000014	0.000001	40
2040	0.000058	0.000014	0.000002	41
2041	0.000061	0.000015	0.000002	42
2042	0.000063	0.000015	0.000002	43
2043	0.000065	0.000016	0.000002	44
2044	0.000068	0.000016	0.000002	45
2045	0.000071	0.000017	0.000003	46
2046	0.000073	0.000017	0.000003	47
2047	0.000076	0.000018	0.000003	48
2048	0.000079	0.000018	0.000003	49
2049	0.000082	0.000019	0.000004	50
2050	0.000085	0.000020	0.000004	51

Hazard Function			Peoria Lock u/s vert.beam	
Year	Normal O&M	Enhanced	Rehabbed.	Year from rehab
2000	0.000005	0.000003		
2001	0.000005	0.000003	0.000000	1
2002	0.000006	0.000003	0.000000	2
2003	0.000006	0.000003	0.000000	3
2004	0.000006	0.000003	0.000000	4
2005	0.000007	0.000004	0.000000	5
2006	0.000007	0.000004	0.000000	6
2007	0.000007	0.000004	0.000000	7
2008	0.000008	0.000004	0.000000	8
2009	0.000008	0.000004	0.000000	9
2010	0.000008	0.000004	0.000000	10
2011	0.000009	0.000005	0.000000	11
2012	0.000009	0.000005	0.000000	12
2013	0.000010	0.000005	0.000000	13
2014	0.000010	0.000005	0.000000	14
2015	0.000010	0.000006	0.000000	15
2016	0.000011	0.000006	0.000000	16
2017	0.000011	0.000006	0.000000	17
2018	0.000012	0.000006	0.000000	18
2019	0.000012	0.000006	0.000000	19
2020	0.000013	0.000007	0.000000	20
2021	0.000013	0.000007	0.000000	21
2022	0.000014	0.000007	0.000000	22
2023	0.000015	0.000007	0.000000	23

2024	0.000015	0.000008	0.000000	24
2025	0.000016	0.000008	0.000000	25
2026	0.000016	0.000008	0.000000	26
2027	0.000017	0.000009	0.000000	27
2028	0.000018	0.000009	0.000000	28
2029	0.000018	0.000009	0.000000	29
2030	0.000019	0.000010	0.000001	30
2031	0.000020	0.000010	0.000001	31
2032	0.000021	0.000010	0.000001	32
2033	0.000021	0.000011	0.000001	33
2034	0.000022	0.000011	0.000001	34
2035	0.000023	0.000011	0.000001	35
2036	0.000024	0.000012	0.000001	36
2037	0.000025	0.000012	0.000001	37
2038	0.000025	0.000012	0.000001	38
2039	0.000026	0.000013	0.000001	39
2040	0.000027	0.000013	0.000001	40
2041	0.000028	0.000014	0.000001	41
2042	0.000029	0.000014	0.000001	42
2043	0.000030	0.000014	0.000002	43
2044	0.000031	0.000015	0.000002	44
2045	0.000032	0.000015	0.000002	45
2046	0.000033	0.000016	0.000002	46
2047	0.000034	0.000016	0.000002	47
2048	0.000035	0.000017	0.000002	48
2049	0.000036	0.000017	0.000002	49
2050	0.000037	0.000018	0.000003	50

Hazard Function		Brandon Rd	d/s horiz. 10 (cat E weld)	
Year	Normal O&M	Enhanced	Rehabbed.	Year from rehab
2000	0.0000000	0.0000000	0.0000000	
2001	0.0000001	0.0000001	0.0000000	
2002	0.0000002	0.0000002	0.0000000	
2003	0.0000005	0.0000005	0.0000000	
2004	0.0000009	0.0000009	0.0000000	
2005	0.0000017	0.0000017	0.0000000	
2006	0.0000030	0.0000030	0.0000000	
2007	0.0000048	0.0000048	0.0000000	
2008	0.0000074	0.0000074	0.0000000	
2009	0.0000110	0.0000110	0.0000000	
2010	0.0000159	0.0000159	0.0000000	
2011	0.0000223	0.0000223	0.0000000	
2012	0.0000306	0.0000306	0.0000000	
2013	0.0000411	0.0000411	0.0000000	
2014	0.0000542	0.0000542	0.0000000	
2015	0.0000704	0.0000704	0.0000000	
2016	0.0000901	0.0000901	0.0000000	
2017	0.0001139	0.0001139	0.0000000	
2018	0.0001424	0.0001424	0.0000000	
2019	0.0001761	0.0001761	0.0000000	
2020	0.0002158	0.0002158	0.0000000	
2021	0.0002622	0.0002622	0.0000000	
2022	0.0003160	0.0003160	0.0000000	
2023	0.0003781	0.0003781	0.0000000	

2024	0.0004493	0.0004493	0.0000000	
2025	0.0005306	0.0005306	0.0000000	
2026	0.0006229	0.0006229	0.0000000	1
2027	0.0007273	0.0007273	0.0000000	2
2028	0.0008450	0.0008450	0.0000000	3
2029	0.0009770	0.0009770	0.0000000	4
2030	0.0011246	0.0011246	0.0000000	5
2031	0.0012891	0.0012891	0.0000001	6
2032	0.0014717	0.0014717	0.0000002	7
2033	0.0016741	0.0016741	0.0000005	8
2034	0.0018975	0.0018975	0.0000009	9
2035	0.0021436	0.0021436	0.0000017	10
2036	0.0024139	0.0024139	0.0000030	11
2037	0.0027101	0.0027101	0.0000048	12
2038	0.0030340	0.0030340	0.0000074	13
2039	0.0033874	0.0033874	0.0000110	14
2040	0.0037722	0.0037722	0.0000159	15
2041	0.0041903	0.0041903	0.0000223	16
2042	0.0046438	0.0046438	0.0000306	17
2043	0.0051347	0.0051347	0.0000411	18
2044	0.0056653	0.0056653	0.0000542	19
2045	0.0062377	0.0062377	0.0000704	20
2046	0.0068544	0.0068544	0.0000901	21
2047	0.0075178	0.0075178	0.0001139	22
2048	0.0082302	0.0082302	0.0001424	23
2049	0.0089944	0.0089944	0.0001761	24
2050	0.0098128	0.0098128	0.0002158	25

Anchor Bars

31. Existing anchor bars at Locks 12, 17, 18, 20, 21 and 22 have low reliability. Hazard functions listed in the following table are for existing anchor bars. If these anchor bars are rehabilitated with proper bushing hole arrangement, they will perform satisfactorily through the study period (2050).

Hazard Function		Lock 22	Anchor Bars	
Year	Normal O&M	Enhanced	Rehabbed.	Year From Rehab
2000	0.002705	0.002705	0.000000	1
2001	0.002807	0.002807	0.000000	2
2002	0.002910	0.002910	0.000000	3
2003	0.003016	0.003016	0.000000	4
2004	0.003124	0.003124	0.000000	5
2005	0.003233	0.003233	0.000000	6
2006	0.003345	0.003345	0.000000	7
2007	0.003459	0.003459	0.000000	8
2008	0.003575	0.003575	0.000000	9
2009	0.003693	0.003693	0.000000	10
2010	0.003813	0.003813	0.000000	11
2011	0.003936	0.003936	0.000000	12
2012	0.004060	0.004060	0.000000	13
2013	0.004187	0.004187	0.000000	14
2014	0.004315	0.004315	0.000000	15
2015	0.004446	0.004446	0.000000	16
2016	0.004579	0.004579	0.000000	17
2017	0.004715	0.004715	0.000000	18
2018	0.004852	0.004852	0.000000	19

2019	0.004992	0.004992	0.000000	20
2020	0.005133	0.005133	0.000000	21
2021	0.005277	0.005277	0.000000	22
2022	0.005423	0.005423	0.000000	23
2023	0.005572	0.005572	0.000000	24
2024	0.005722	0.005722	0.000000	25
2025	0.005875	0.005875	0.000000	26
2026	0.006030	0.006030	0.000000	27
2027	0.006187	0.006187	0.000000	28
2028	0.006347	0.006347	0.000000	29
2029	0.006509	0.006509	0.000000	30
2030	0.006672	0.006672	0.000000	31
2031	0.006839	0.006839	0.000000	32
2032	0.007007	0.007007	0.000000	33
2033	0.007178	0.007178	0.000000	34
2034	0.007351	0.007351	0.000000	35
2035	0.007526	0.007526	0.000000	36
2036	0.007703	0.007703	0.000000	37
2037	0.007883	0.007883	0.000000	38
2038	0.008065	0.008065	0.000000	39
2039	0.008250	0.008250	0.000000	40
2040	0.008436	0.008436	0.000000	41
2041	0.008625	0.008625	0.000000	42
2042	0.008817	0.008817	0.000000	43
2043	0.009010	0.009010	0.000000	44
2044	0.009206	0.009206	0.000000	45
2045	0.009404	0.009404	0.000000	46
2046	0.009605	0.009605	0.000000	47

2047	0.009808	0.009808	0.000000	48
2048	0.010013	0.010013	0.000000	49
2049	0.010221	0.010221	0.000000	50
2050	0.010431	0.010431	0.000000	51

Anchor bars at other sites (locks 12, 17, 18, 20 and 21) also gave similar values for hazard functions.

32. Hazard Functions After a Repair: Reliability of a structure after a repair depends on the type of component and the extent of the repair. Therefore, for lower level and medium level consequences, hazard rate after a repair is assumed to be the same as before the repair. Hazard rates after a high level consequence would be the same as the hazard rate after a rehabilitation.

V. Consequences

33. Consequences were assumed to fall into three categories. Low level (LC), Medium Level (MC) and High level (HC). Low level of consequence would include inspection and minor repair of gates. This could be done on a scheduled time during regular navigation shut down without loss of service. Medium consequence would be painting the gates, and repairing damaged members and welds. This again could be achieved during scheduled shutdowns without a loss of service. High level consequence would be replacing a gate leaf in the event of failure. It assumes the availability of a spare gate and mobilizing/demobilizing a plant to replace the damaged gate. The cost could vary slightly depending on the distance the plant has to travel to a given site. However, neglecting the cost variation due to travel time, the repair cost is the same for all sites.

For all sites:

a. Miter Gates

Level of Consequence	Probability	Nav. Shutdown Time (days)	Cost ($) Per Leaf
Low Level (LC)	0.90	0	40,000.
Medium Level (MC)	0.09	0	125,000.
High Level (HC)	0.01	14	825,000.

b. Anchor Bars

Level of Consequence	Probability	Nav. Shutdown Time (days)	Cost ($) Per Pair
Low Level (LC)	0.90	0	3,000.
Medium Level (MC)	0.09	2	15,000.
High Level (HC)	0.01	4	45,000.

Medium level would involve temporarily fixing any minor problems and waiting for the navigation season shutdown to do permanent repairs. High level would involve immediate fix.

IV. Costs for Rehabilitation

Miter Gates

34. Height of a gate doesn't vary the rehabilitation cost significantly. For example, in painting a miter gate, majority of the cost is due to mobilizing the equipment and labor. Rehabilitation could be either painting of a gate or replacement. Cost was estimated as follows:

Painting a miter gate leaf $125,000.
Replacing a miter gate $800,000.
(supply a new gate leaf, remove old & install new)

35. Painting cost estimated with the assumption that all four leafs are painted at the same time. Unit cost of replacing a gate would be little less if multiple leafs are done at a site at the same time.

Anchor Bars

36. In anchor bars, it assumes that a pair of spare bars are available and the bars are replaced without dewatering the lock. That is, the replacement is done with the help of a diving crew to jack-up the gate.

Replacing a pair of anchor bars $45,000.
(supply and install)

SECTION 2 - Lift Gates in St. Louis District

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Reliability Analysis for Lift Gates In the St Louis District

I. Model Description

General

1. The structural reliability model for lift gates was developed using methods outlined in the report written by WES and JAYCOR titled "Reliability Analysis of Hydraulic Steel Structures with Fatigue and Corrosion Degradation", March 1, 1994. For the lift gates, limits state for unsatisfactory performance due to both strength and fatigue were be examined.

2. The lift gates in the St. Louis District are steel structures composed of several members. An upstream skin plate is welded to the upstream flanges of horizontally framed plate girders. The top girder of a leaf forms a vertical damming surface. Vertical diaphragms on recently designed lift gates help distribute the hydrostatic loads to the plate girders. On older designs without diaphragms, there are adjacent downstream bracing members which serve the same function. For a given gate and loading, several members could be critical to the reliability of the gate. The most critical members are the downstream plate girder flanges which are tension members. The upstream skin plate is not critical unless a major portion of the skin plate failed, which has a very low probability. Therefore, a complete structural analysis was performed on the horizontally framed horizontal plate girders of the lift gates; critical loads, and the limit state of the members were identified and the member reliabilities computed. The overall reliability of the lift gate is determined from the reliability of the plate girders.

3. Because of the similarity to the reliability analysis on miter gate vertical beams, the structural reliability model used to analyze miter gates was adapted for lift gate reliability analysis. Both models are greatly based on the miter gate model presented in the WES-JAYCOR report. There are no significant deviations from the previously approved miter gate models.

4. The analysis of the lift gates for forces in the horizontal plate girders was done using conventional 2-dimensional modeling techniques. Appropriate loading diagrams were developed for each lift gate for the difference in head between the upper and lower pools carried by the upstream skin plate. This loading was then transferred to the horizontal plate girders using the appropriate contributory area of skin plate. Forces for individual girders were found based on equations for simply supported beams with uniform loading.

5. For the development of the lift gate reliability model, a copy of the spreadsheet developed by the St. Paul District for the structural reliability model for vertical

beams for miter gates was obtained and adapted for the lift gates at Locks No. 27 and Melvin Price Locks and Dam.

Definition of Unsatisfactory Performance

6. Although a lift gate is comprised of several components, any one of which could be loaded beyond its limit state and cause unsatisfactory performance of the gate, the most critical components are the horizontal plate girders. It is very unlikely that the loss of a lift gate leaf will result in a loss of the pool. If a lift gate leaf fails due to unsatisfactory performance of the plate girders, the possible consequences range from a single crack initiated in the compression flange of a plate girder, requiring a closure of the lock for repair scheduled within a two month window for a duration of ~~one week~~ two days, to multiple cracks initiated in several plate girders, requiring immediate closure of the lock for repairs for a duration of one month. The limit states for the plate girders are bending or shear. A review of the designs of the St. Louis District lift gates revealed that the factors of safety in shear were high and would yield high Betas. Therefore, only bending was incorporated into the lift gate reliability model.

7. For the skin plate, there do not seem to be any unsatisfactory performance modes which would affect the overall integrity of a gate short of failure of a significant portion of the entire upstream skin plate surface. The skin plate serves to contain water behind the gate and transfer loads to the horizontal plate girders. Because there is conservatism in the design method used for the skin plate and because its performance does not impact the overall structural capacity of the gate, the skin plate was not considered a critical member for the reliability analysis.

Random Variables

8. The random variables in the following paragraphs were used in the reliability model. They were derived from values listed in the WES-JAYCOR report:

9. <u>Corrosion Rate.</u> The random variable for corrosion is e_c and the amount of corrosion, C, is defined by:

$$\log C - \log A + B \log t + e_c.$$

The variables used for the corrosion equation were those variables given in the WES-JAYCOR report. Depending on which girder was analyzed, either the variables for atmospheric, splash zone, or submerged corrosion conditions were used. Therefore, the following variables were used in the reliability analysis for the lift gates to produce C in micrometers:

A = 23.4, B = 0.65, ec avg. = 0 with std. dev. = 0.219 (Atmospheric)
A = 148.5, B = 0.903, ec avg. = 0 with std. dev. = 0.099 (Splash Zone)
A = 51.6, B = 0.65, ec avg. = 0 with std. dev. = 0.174 (Submerged)

10. For the original lift gate leaves at Locks No. 27 and Melvin Price Locks and Dam, it was assumed the vinyl paint systems would prevent corrosion for 20 years, and that thereafter the gates would be painted on a regular maintenance schedule resulting in no further corrosion.

11. For the new lift gate leaves at Locks No. 27 and Melvin Price Locks and Dam, it was assumed the vinyl paint systems would prevent corrosion for 20 years, and that thereafter the gates would be painted on a regular maintenance schedule resulting in no significant corrosion of the gates.

12. Steel Yield Strength. The random variable is defined by LRFD research as follows:

Bending. Avg. = 1.08 Fy, Std. Dev. = 0.14

The yield strength, Fy, for the A7 steel used at Locks No. 27 on the original lift gate leaves is 33 ksi. The yield strength, Fy, for the new lift gate leaves at Locks No. 27 and Melvin Price Locks and Dam is 50 ksi.

13. Loading. The random variable used for loading is hydrostatic load. Since the reliability is computed on an annual basis, mean and standard deviation for loading should be for critical loading the gate experiences in a year. For a lift gate this is the maximum hydrostatic head it would see in a given year. A summary of the results for the strength and fatigue limit states are listed below. Discussion of load cases and additional loading information is found in the paragraph titles "Load Cases".

Strength Limit State
Avg. Maximum Yearly Head - 17.60 feet, std. dev. - 1.69 feet

Fatigue Limit State
Cumulative weighted average head, 1993, - 11.36 ft, std. dev. - 2.66 ft.

14. Ratio of actual to computed forces. This random variable is Ks and is the ration of actual to calculated stresses.

Avg. Ks - 1.02, Std. dev. - 0.10.

The numbers for Ks above were used for the development of the AISC LRFD code and therefore were developed from building construction. The values were taken from the WES-JAYCOR report and their applicability to lift gates may be questionable.

15. Fatigue. For fatigue, there are three variables which are used.

A) Ratio of lockages to actual stress cycles (Kc). This factor permits the number of stress cycles to be computed from the number of lockages. Kc is computed from the number of machinery hard cycles. For Locks No. 27:

Avg. Kc - 0.999, Std. dev. = 0.157.

B) Uncertainty in the fatigue life of the material (ϵ). This variable is applied in the equation which is used to compute the fatigue strength of the material:

Avg. ϵ = 0.0, Std. dev. = 0.31.

C) Damage accumulation factor (Δ)

Avg. Δ = 1.0, Std. dev. = 0.30.

Load Cases

16. One load case was selected for the reliability model. The lift gate was checked for both strength and fatigue. The load was for hydrostatic loading with headwater and tailwater due to maximum annual head for the strength limit state or due to average head for the fatigue limit state. Water loads on the gate were computed from daily records for pool and tailwater levels at Locks No. 27 from 1963 to 1993.

Procedure for Analyzing Reliability

17. The procedure used to compute reliability of lift gates in the St. louis District is described below:

A. Information was collected on the geometry, member properties, weight, loadings, and number of lockages for the lift gates.

B. Hydrostatic loads on the lift gate on the lift gate were computed. The loads, reactions, and all further analysis must be computed at the mean loading and one standard deviation above and below the mean.

C. Forces in the lift gate members were computed. Loading diagrams were developed to compute the hydrostatic forces on the various horizontal plate girders, which are the critical members in the lift gate reliability analysis. Moments were calculated by modeling the girders as simply supported beams with a uniform hydrostatic loading taking into account the effective width of the upstream skin plate.

D. Reliability was computed for limit states due to bending and shear loads in the gate members. From previous reviews of the design of the lift gates, it was determined that only bending was critical since the shear loads compute

yield high Factors of safety. several spreadsheets were the input required to be used by the reliability spreadsheet. As previously discussed, the spreadsheet used to calculate the reliability for the vertical beams in miter gates was adapted to calculate the reliability of the plate girders for the lift gates. The hazard function was computed based on the procedure shown in the WES-JAYCOR report.

II. Site Selection

18. Reliability analyses were performed for the lift gates at the following sites in the St. Louis District:

A) Locks No. 27, Main Lock, New Lift Gate, Upstream Leaf.

B) Locks No. 27, Main Lock, Old Lift Gate, Downstream Leaf.

C) Locks No. 27, Aux. Lock, Old Lift Gate, Downstream Leaf.

D) Melvin Price, Main Lock, Lift Gate, Middle Leaf.

It should be noted that Locks No. 27 has a lift gate in both the main and auxiliary lock chambers consisting of an upstream and downstream leaf (2-leaf configuration). Melvin Price Locks and Dam has a lift gate in the main lock chamber consisting of an upstream, middle, and downstream leaf (3-leaf configuration).

III. Parameters

Constants.

19. The following constants were used:

Gate Length = 112.5 at both Locks No. 27 and Melvin Price Locks and Dam.

Mean Upper Pool = 404.5 at Locks No. 27;

Mean Upper Pool = 419.0 at Melvin Price Locks and Dam.

20. The table on the next page lists random variables that were used in the analysis.

Random Variables							
Lock	RV	Kc	e	Ks	Delta	hi	ec
Locks 27	Mean	0.766	0.00	1.020	1.0	211.2	0.0000
Main Lock	StDv	0.032	0.06	0.100	0.3	20.279	0.0990
Upstream Leaf							
Locks 27	Mean	0.766	0.00	1.020	1.0	211.2	0.0000
Main Lock	StDv	0.032	0.10	0.100	0.3	20.279	0.0990
Downstream Leaf							
Locks 27	Mean	0.789	0.00	1.020	1.0	211.2	0.0000
Aux. Lock	StDv	0.126	0.10	0.100	0.3	20.279	0.0990
Downstream Leaf							
Mel Price	Mean	0.756	0.00	1.020	1.0	246	0.0000
Main Lock	StDv	0.030	0.10	0.100	0.3	12	0.0990
Middle Leaf							

List of Betas

21. The table on the following page lists Betas that were computed. The betas are listed for comparison purposes.

Betas				
	L&D 27	L&D 27	L&D 27	Mel Price
	Main Lock	Main Lock	Aux. Lock	Main Lock
	New Lift Gate	Old Lift Gate	Old Lift Gate	Lift Gate
	Upstream Leaf	Downstream Leaf	Downstream Leaf	Middle Leaf
Limit State	Bending	Bending	Bending	Bending
Year	Beta	Beta	Beta	Beta
1963		11.2	12.4	
1990		3.6	5.2	8.7
1994	13.6	3.3	4.9	5.6
1995	12.1	3.2	4.8	5.1
2005	8.1	2.3	3.9	2.6
2010	6.9	1.9	3.4	1.8
2015	6.1	1.5	3.0	1.2
2020	5.4	1.1	2.6	0.69
2025	4.9	0.77	2.3	0.23
2030	4.4	0.44	1.9	-0.19
2035	3.9	0.14	1.5	-0.56
2040	3.5	-0.15	1.1	-0.90
2045	3.2	-0.41	0.87	-1.2
2050	2.8	-0.65	0.59	-1.5

IV. Hazard Functions

22. The following tables list hazard functions that were computed for the lift gates.

Locks & Dam No. 27, Main Lock - New Lift Gate, Upstream Leaf				
Year	Current O&M Hazard Function	Enhanced Maintenance Hazard Function	Year	Rehabilitated Hazard Function
2000	0.000000	0.000000	0	0.000000
2001	0.000000	0.000000	1	0.000000
2002	0.000000	0.000000	2	0.000000
2003	0.000000	0.000000	3	0.000000
2004	0.000000	0.000000	4	0.000000
2005	0.000000	0.000000	5	0.000000
2006	0.000000	0.000000	6	0.000000
2007	0.000000	0.000000	7	0.000000
2008	0.000000	0.000000	8	0.000000
2009	0.000000	0.000000	9	0.000000
2010	0.000000	0.000000	10	0.000000
2011	0.000000	0.000000	11	0.000000
2012	0.000000	0.000000	12	0.000000
2013	0.000000	0.000000	13	0.000000
2014	0.000000	0.000000	14	0.000000
2015	0.000000	0.000000	15	0.000000
2016	0.000000	0.000000	16	0.000000

2017	0.000000	0.000000	17	0.000000
2018	0.000000	0.000000	18	0.000000
2019	0.000000	0.000000	19	0.000000
2020	0.000000	0.000000	20	0.000000
2021	0.000000	0.000000	21	0.000000
2022	0.000000	0.000000	22	0.000000
2023	0.000000	0.000000	23	0.000000
2024	0.000000	0.000000	24	0.000000
2025	0.000000	0.000000	25	0.000000
2026	0.000000	0.000000	26	0.000000
2027	0.000000	0.000000	27	0.000001
2028	0.000001	0.000000	28	0.000001
2029	0.000001	0.000000	29	0.000001
2030	0.000001	0.000000	30	0.000001
2031	0.000002	0.000000	31	0.000002
2032	0.000003	0.000000	32	0.000003
2033	0.000004	0.000000	33	0.000003
2034	0.000005	0.000000	34	0.000005
2035	0.000007	0.000000	35	0.000006
2036	0.000010	0.000000	36	0.000008
2037	0.000014	0.000000	37	0.000010
2038	0.000019	0.000000	38	0.000013
2039	0.000026	0.000000	39	0.000017
2040	0.000036	0.000000	40	0.000022

2041	0.000049	0.000000	41	0.000029
2042	0.000068	0.000001	42	0.000037
2043	0.000092	0.000001	43	0.000048
2044	0.000126	0.000001	44	0.000061
2045	0.000171	0.000002	45	0.000078
2046	0.000231	0.000003	46	0.000099
2047	0.000312	0.000004	47	0.000126
2048	0.000420	0.000006	48	0.000160
2049	0.000564	0.000009	49	0.000203
2050	0.000755	0.000013	50	0.000256

Locks & Dam No. 27, Main Lock - Old Lift Gate, Downstream Leaf				
Year	Current O&M Hazard Function	Enhanced Maintenance Hazard Function	Year	Rehabilitated Hazard Function
2000	0.001764	0.000741	0	0.000000
2001	0.001989	0.000817	1	0.000000
2002	0.002237	0.000900	2	0.000000
2003	0.002512	0.000990	3	0.000000
2004	0.002815	0.001087	4	0.000000
2005	0.003150	0.001192	5	0.000000
2006	0.003518	0.001305	6	0.000000
2007	0.003923	0.001428	7	0.000000
2008	0.004368	0.001559	8	0.000000
2009	0.004855	0.001701	9	0.000000
2010	0.005389	0.001853	10	0.000000
2011	0.005973	0.002016	11	0.000000
2012	0.006611	0.002191	12	0.000000
2013	0.007307	0.002379	13	0.000000
2014	0.008065	0.002580	14	0.000000
2015	0.008891	0.002794	15	0.000000
2016	0.009788	0.003024	16	0.000000
2017	0.010762	0.003269	17	0.000000
2018	0.011820	0.003530	18	0.000000

2019	0.012965	0.003809	19	0.000000
2020	0.014206	0.004105	20	0.000000
2021	0.015548	0.004421	21	0.000000
2022	0.016998	0.004757	22	0.000000
2023	0.018563	0.005114	23	0.000000
2024	0.020251	0.005492	24	0.000000
2025	0.022071	0.005894	25	0.000000
2026	0.024030	0.006320	26	0.000000
2027	0.026137	0.006772	27	0.000001
2028	0.028402	0.007250	28	0.000001
2029	0.030835	0.007756	29	0.000001
2030	0.033446	0.008291	30	0.000001
2031	0.036246	0.008857	31	0.000002
2032	0.039246	0.009454	32	0.000003
2033	0.042458	0.010085	33	0.000003
2034	0.045895	0.010750	34	0.000005
2035	0.049570	0.011452	35	0.000006
2036	0.053496	0.012192	36	0.000008
2037	0.057688	0.012970	37	0.000010
2038	0.062161	0.013790	38	0.000013
2039	0.066930	0.014653	39	0.000017
2040	0.072013	0.015560	40	0.000022
2041	0.077425	0.016514	41	0.000029
2042	0.083186	0.017516	42	0.000037

2043	0.089313	0.018569	43	0.000048
2044	0.095826	0.019673	44	0.000061
2045	0.102746	0.020832	45	0.000078
2046	0.110093	0.022047	46	0.000099
2047	0.117890	0.023321	47	0.000126
2048	0.126159	0.024655	48	0.000160
2049	0.134925	0.026053	49	0.000203
2050	0.144212	0.027516	50	0.000256

Locks & Dam No. 27, Aux. Lock - Old Lift Gate,Downstream Leaf				
Year	Current O&M Hazard Function	Enhanced Maintenance Hazard Function	Year	Rehabilitated Hazard Function
2000	0.000046	0.000005	0	0.000000
2001	0.000055	0.000005	1	0.000000
2002	0.000066	0.000007	2	0.000000
2003	0.000079	0.000008	3	0.000000
2004	0.000095	0.000010	4	0.000000
2005	0.000113	0.000011	5	0.000000
2006	0.000134	0.000014	6	0.000000
2007	0.000159	0.000016	7	0.000000
2008	0.000188	0.000019	8	0.000000
2009	0.000222	0.000023	9	0.000000
2010	0.000262	0.000027	10	0.000000
2011	0.000308	0.000032	11	0.000000
2012	0.000361	0.000037	12	0.000000
2013	0.000422	0.000044	13	0.000000
2014	0.000492	0.000051	14	0.000000
2015	0.000574	0.000060	15	0.000000
2016	0.000667	0.000070	16	0.000000
2017	0.000774	0.000081	17	0.000000
2018	0.000896	0.000094	18	0.000000

2019	0.001036	0.000109	19	0.000000
2020	0.001195	0.000127	20	0.000000
2021	0.001376	0.000146	21	0.000000
2022	0.001583	0.000169	22	0.000000
2023	0.001817	0.000194	23	0.000000
2024	0.002082	0.000223	24	0.000000
2025	0.002383	0.000256	25	0.000000
2026	0.002722	0.000294	26	0.000000
2027	0.003105	0.000336	27	0.000001
2028	0.003537	0.000384	28	0.000001
2029	0.004023	0.000438	29	0.000001
2030	0.004569	0.000498	30	0.000001
2031	0.005182	0.000567	31	0.000002
2032	0.005870	0.000644	32	0.000003
2033	0.006640	0.000730	33	0.000003
2034	0.007501	0.000827	34	0.000005
2035	0.008462	0.000935	35	0.000006
2036	0.009535	0.001056	36	0.000008
2037	0.010732	0.001192	37	0.000010
2038	0.012063	0.001343	38	0.000013
2039	0.013544	0.001512	39	0.000017
2040	0.015189	0.001699	40	0.000022
2041	0.017015	0.001908	41	0.000029
2042	0.019039	0.002140	42	0.000037

2043	0.021281	0.002398	43	0.000048
2044	0.023762	0.002684	44	0.000061
2045	0.026504	0.003001	45	0.000078
2046	0.029532	0.003351	46	0.000099
2047	0.032873	0.003739	47	0.000126
2048	0.036556	0.004167	48	0.000160
2049	0.040612	0.004640	49	0.000203
2050	0.045075	0.005161	50	0.000256

Melvin Price Locks & Dam, Main Lock - Middle Leaf				
Year	Current O&M	Enhanced	Year	Rehabilitated
	Hazard Function	Maintenance		Hazard Function
		Hazard Function		
2000	0.002052	0.001023	0	0.000001
2001	0.002380	0.001166	1	0.000001
2002	0.002752	0.001326	2	0.000002
2003	0.003176	0.001505	3	0.000002
2004	0.003658	0.001704	4	0.000003
2005	0.004202	0.001927	5	0.000004
2006	0.004819	0.002174	6	0.000005
2007	0.005514	0.002449	7	0.000006
2008	0.006297	0.002754	8	0.000008
2009	0.007177	0.003092	9	0.000010
2010	0.008166	0.003466	10	0.000012
2011	0.009274	0.003878	11	0.000015
2012	0.010513	0.004333	12	0.000019
2013	0.011898	0.004833	13	0.000023
2014	0.013444	0.005384	14	0.000029
2015	0.015165	0.005989	15	0.000036
2016	0.017080	0.006652	16	0.000044
2017	0.019207	0.007379	17	0.000054
2018	0.021567	0.008175	18	0.000066

2019	0.024181	0.009045	19	0.000081
2020	0.027074	0.009994	20	0.000098
2021	0.030271	0.011030	21	0.000119
2022	0.033800	0.012159	22	0.000144
2023	0.037690	0.013388	23	0.000173
2024	0.041974	0.014724	24	0.000209
2025	0.046686	0.016175	25	0.000250
2026	0.051863	0.017750	26	0.000300
2027	0.057545	0.019458	27	0.000358
2028	0.063775	0.021308	28	0.000427
2029	0.070598	0.023310	29	0.000507
2030	0.078062	0.025475	30	0.000602
2031	0.086222	0.027814	31	0.000712
2032	0.095132	0.030339	32	0.000841
2033	0.104853	0.033063	33	0.000992
2034	0.115448	0.035998	34	0.001167
2035	0.126985	0.039159	35	0.001370
2036	0.139538	0.042560	36	0.001605
2037	0.153183	0.046218	37	0.001878
2038	0.168003	0.050147	38	0.002192
2039	0.184087	0.054366	39	0.002555
2040	0.201526	0.058893	40	0.002973
2041	0.220421	0.063747	41	0.003454
2042	0.240877	0.068947	42	0.004005

2043	0.263006	0.074515	43	0.004637
2044	0.286926	0.080472	44	0.005360
2045	0.312764	0.086843	45	0.006187
2046	0.340652	0.093651	46	0.007130
2047	0.370733	0.100922	47	0.008204
2048	0.403155	0.108682	48	0.009427
2049	0.438077	0.116960	49	0.010816
2050	0.475666	0.125785	50	0.012393

Hazard Function After Repair

23. For miter gates which have required repair in the St. Louis District in the past, the repairs were not extensive and the gate was not in significantly different condition than it was before the unsatisfactory performance took place. For this reason, the same will be assumed for potential repairs of the lift gates. The hazard function after repair can be assumed to be the same as it was before repair unless the unsatisfactory performance falls under the category of high level of consequences in the section which follows. In this case, the hazard function after rehabilitation should be used.

V. CONSEQUENCES.

24. The members of the lift gates which are being investigated are fracture critical, meaning that a failure of these members could cause catastrophic failure of the entire gate. However, the model only predicts crack initiation and not propagation. Realistically, once a crack initiates, it may take numerous cycles before the crack reaches its critical crack size and finally fails. If lock personnel or periodic inspection teams notice the crack before the crack becomes critical, repairs can be scheduled and navigation downtime will not be severe. If the crack is not noticed, it may progress until the member fails suddenly resulting in unscheduled repairs and extended downtime.

25. Three levels of consequences were considered:

A) Low Level of Consequences. Cracks are found by lock personnel or periodic inspection team in the lift gate leaf at an early stage. Lock must be shut down for two days for in depth inspection and repair. Conditional probability is 84%; repair costs are $15,000.

B) Medium Level of Consequences. Cracks are found in the lift gate leaf before failure of the gate but they are of a more severe nature making it imperative to be repaired immediately. Lock chamber for the lift gate leaf in question is closed for a week (7 days) resulting in reduced lockage capacity and longer lockage times. Lock must be shut down for seven days for in depth inspection and repair. Conditional probability is 15%; repair costs are $53,000.

C) High Level of Consequences. Lift gate leaf fails while in use. Complete replacement of the leaf is required. Lock chamber for the lift gate leaf in question is closed for six months due to fabrication of a new lift gate leaf. Conditional probability is 1%; replacement cost is $800,000 per leaf at Locks No. 27 and $360,000 per leaf at Melvin Price.

VI. Cost of Rehabilitation and Enhanced Maintenance.

26. The table below lists cost for rehabilitating or for enhanced maintenance of the lift gates. The cost for enhanced maintenance is a per year cost based on a twenty year paint cycle.

Lock	Rehabilitation Cost	Enhanced Maint. Cost
Locks 27	\$800,000/Leaf or \$1,600,000 Both Leaves in Main or Aux. Lock	\$6,250/Leaf or \$12,500 Both Leaves in Main or Aux. Lock
Melvin Price	\$360,000/Leaf or \$1,080,000 Three Leaves in Main Lock	\$29,250/Leaf ? or \$8,775 Three Leaves in Main Lock

SECTION 3 - Lift Gates at Lockport and Lock 19 (Rock Island District)

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Reliability for Lift Gates at Lockport and Lock 19

I. Model Description

Loading Condition

1. The lift gates at Lockport and Lock 19 have essentially the same loading condition. This condition is a fairly constant upper pool hydraulic load with no tailwater load. Hence, the gate is subjected to a near constant amplitude load cycles. For the purpose of this study, the load will be treated as a constant.

Critical Members

2. The investigation was limited to fracture critical members of the main trusses for each gate. The vertical beams and the skin plate were not analyzed because both of these items are redundant in nature and not fracture critical. The lift gate at Lockport Lock, consists of four trusses. Only the third truss was investigated since it is the most heavily loaded of the four trusses. This gate has experienced cracking in the past and truss #3 had the most extensive cracking.

Limit State

3. Only the fatigue limit state will be investigated for this model. The strength limit state will not be investigated for two reasons. First, since the gate is subjected to near constant amplitude loading, the gate has shown repeatedly that yielding and buckling of its members is not a problem. The second reason for not investigating the strength limit state is that both of these gates have received adequate maintenance to keep corrosion and loss of section to a minimum. Hence, the strength of its members based on section properties has not significantly deteriorated. As in all of the reliability models, it is assumed that maintenance will continue at the same rate as has occurred in the past.

Performance Function

4. The performance function will be the same as described in Reference 1 for members subjected to fatigue.

$$FS=\frac{N}{N_{act}}$$

where N is the number of cycles that the member is capable of sustaining and is a function of the effective stress range (Se) and the stress category of the detail being investigated. Using Reference 1,

$$Log(N) = A - m \cdot Log(Se)$$

where A and m are values which correspond to a particular stress category. N_{act} is the number of cycles that the member has resisted to some point in time.

Method of Analysis

5. The gates will be analyzed using a frame analysis program to determine the axial force and bending moments in the critical tension members and will be used to establish the stress range of the critical members. Since these trusses resist only horizontal loads, the stress range for any member will be the stress under maximum load.

6. Each gate will have its own frame analysis since the trusses themselves are unique. Originally, these gates were designed as pure trusses and the bending moments were ignored. The inclusion of the bending moment increases the stresses at the extreme tension fiber of tension members significantly in some members and only marginally in others.

Random Variables

7. Uncertainty exists in both N and N_{act}. For N, there is uncertainty in the stress range due to uncertainties in the analysis. The parameter k_s will be used to express uncertainty in various aspects of the analysis which cannot be modeled well or to which exact values may not be known. For instance, the boundary conditions of the structural model may be slightly different than the simply supported condition used in the analysis. Also, not all connections are purely rigid and some eccentricities may exist at connections.

8. Another source of uncertainty is in the computation of N from the source data as described in Reference 1. For this reason the parameter ϵ_{SN} is added to equation 1. If k_s is also added to equation 1 it becomes:

$$Log(N) = A - m \cdot Log(Se \cdot k_s) + e_{SN}$$

9. The actual number of stress cycles is also uncertain. Stress cycle data has only been kept since 1987 while lock tonnage data has been recorded almost since the locks have been in place. Also, traffic projects for the future are based on tonnage rather than cycles or even lockages. Therefore, a conversion from tonnage to cycles is needed to determine the actual number of stress cycles in the future. To convert from tonnage to stress cycles, the random variable k_c is introduced. As a result,

$$N_{act} = k_c \cdot W_c$$

where W_c is the cumulative tonnage to a given point in time.

10. The amount of corrosion affects the stress range *Se*. The corrosion rate for bare steel is established in the WES-JAYCOR report and used in the other structural models. The uncertainty in this corrosion rate is given in the parameter ϵ_C.

II. Parameters

11. Four parameters were treated as random variables:

- k_s - stress concentration factor
- ϵ_{SN} - uncertainty in the SN curves
- k_c - load cycle per million ton of traffic
- ϵ_C - uncertainty in the corrosion rate

Lock	k_s		ϵ_{SN}		k_c		ϵ_C	
	μ	σ	μ	σ	μ	σ	μ	σ
Lockport	1.2	.15	0	.10	394	22	0	.099
Lock 19	1.0	.10	0	.10	165	9.3	0	.099

12. Several other parameters were treated as constants such as the geometry, the stress range, and the fatigue category.

	Stress Range	Fatigue Category	First Year in Service
Lockport	14.5	E	1968
Lock 19	15.45	E	1958

III. Hazard Functions

13. Three hazard functions are needed for the economic analysis. Normal O&M (unrehabilitated), ehabilitated, and enhanced maintenance. Under current O&M practices, both of these gates have been well maintained and show only slight section loss. Therefore, the normal O&M curve will assume that 95% of the time, an effective paint coating is in place. Hence, the splash zone corrosion rate established in Ref 1 will be multiplied by 0.05. The enhanced maintenance hazard function will reflect no section loss into the future. The condition of the gate after a rehabilitation cannot be accurately predicted, but it is assumed that rehabilitation will not fully restore the gate to a new condition. Rather, it is assumed that in the first year after a rehabilitation, the gate will have the same probability of unsatisfactory performance as a new gate after 10 years of service. This procedure effectively shifts the norm O&M hazard function over several years.

Lockport Lock Hazard Functions

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2000	0.06500	0.06000	0	0.00330
2001	0.06918	0.06258	1	0.00406
2002	0.07354	0.06535	2	0.00518
2003	0.07798	0.06819	3	0.00651
2004	0.08247	0.07107	4	0.00800
2005	0.08700	0.07400	5	0.00960
2006	0.09156	0.07696	6	0.01131
2007	0.09614	0.07994	7	0.01311
2008	0.09943	0.08400	8	0.01473
2009	0.10458	0.08657	9	0.01681
2010	0.11000	0.08900	10	0.01900
2011	0.11565	0.09134	11	0.02127
2012	0.12149	0.09359	12	0.02361
2013	0.12800	0.09413	13	0.02543
2014	0.13400	0.09688	14	0.02814
2015	0.14000	0.10000	15	0.03100
2016	0.14600	0.10345	16	0.03400
2017	0.15200	0.10719	17	0.03711
2018	0.15800	0.11200	18	0.04018
2019	0.16400	0.11600	19	0.04355
2020	0.17000	0.12000	20	0.04700
2021	0.17600	0.12400	21	0.05051
2022	0.18200	0.12800	22	0.05407
2023	0.18607	0.13200	23	0.05699
2024	0.19284	0.13600	24	0.06092
2025	0.20000	0.14000	25	0.06500
2026	0.20749	0.14400	26	0.06921
2027	0.21526	0.14800	27	0.07352
2028	0.22400	0.15200	28	0.07798
2029	0.23200	0.15600	29	0.08247
2030	0.24000	0.16000	30	0.08700
2031	0.24800	0.16400	31	0.09156
2032	0.25600	0.16800	32	0.09614
2033	0.26400	0.17200	33	0.09943
2034	0.27200	0.17600	34	0.10458
2035	0.28000	0.18000	35	0.11000
2036	0.28800	0.18400	36	0.11565
2037	0.29600	0.18800	37	0.12149
2038	0.30400	0.19200	38	0.12800
2039	0.31200	0.19600	39	0.13400
2040	0.32000	0.20000	40	0.14000

Lockport Lock Hazard Functions

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2041	0.32800	0.20400	41	0.14600
2042	0.33600	0.20800	42	0.15200
2043	0.34200	0.21200	43	0.15800
2044	0.35081	0.21600	44	0.16400
2045	0.36000	0.22000	45	0.17000
2046	0.36951	0.22400	46	0.17600
2047	0.37930	0.22800	47	0.18200
2048	0.38932	0.23200	48	0.18800
2049	0.39956	0.23600	49	0.19400
2050	0.41000	0.24000	50	0.20000

Lock 19 Hazard Functions

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2000	0.00580	0.00570	0	0.00010
2001	0.00623	0.00616	1	0.00010
2002	0.00706	0.00690	2	0.00010
2003	0.00816	0.00781	3	0.00010
2004	0.00948	0.00885	4	0.00010
2005	0.01100	0.01000	5	0.00010
2006	0.01270	0.01124	6	0.00010
2007	0.01455	0.01257	7	0.00010
2008	0.01595	0.01364	8	0.00010
2009	0.01835	0.01526	9	0.00010
2010	0.02100	0.01700	10	0.00010
2011	0.02386	0.01884	11	0.00010
2012	0.02690	0.02077	12	0.00010
2013	0.02900	0.02225	13	0.00010
2014	0.03282	0.02454	14	0.00010
2015	0.03700	0.02700	15	0.00011
2016	0.04149	0.02959	16	0.00012
2017	0.04626	0.03230	17	0.00015
2018	0.04980	0.03428	18	0.00022
2019	0.05566	0.03752	19	0.00030
2020	0.06200	0.04100	20	0.00039
2021	0.06875	0.04468	21	0.00050
2022	0.07585	0.04854	22	0.00063
2023	0.08154	0.05166	23	0.00069

Lock 19 Hazard Functions

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2024	0.08999	0.05619	24	0.00088
2025	0.09900	0.06100	25	0.00110
2026	0.10847	0.06604	26	0.00136
2027	0.11835	0.07128	27	0.00165
2028	0.12445	0.07618	28	0.00182
2029	0.13664	0.08199	29	0.00223
2030	0.15000	0.08800	30	0.00270
2031	0.16436	0.09416	31	0.00322
2032	0.17962	0.10046	32	0.00380
2033	0.19401	0.10399	33	0.00420
2034	0.21162	0.11164	34	0.00496
2035	0.23000	0.12000	35	0.00580
2036	0.24902	0.12899	36	0.00672
2037	0.26859	0.13853	37	0.00770
2038	0.28090	0.14797	38	0.00816
2039	0.30445	0.15879	39	0.00948
2040	0.33000	0.17000	40	0.01100
2041	0.35725	0.18152	41	0.01270
2042	0.38602	0.19331	42	0.01455
2043	0.41034	0.20213	43	0.01595
2044	0.44419	0.21568	44	0.01835
2045	0.48000	0.23000	45	0.02100
2046	0.51745	0.24498	46	0.02386
2047	0.55632	0.26053	47	0.02690
2048	0.59644	0.27658	48	0.03012
2049	0.63770	0.29308	49	0.03349
2050	0.68000	0.31000	50	0.03700

IV. Consequences

14. The members of the gate which are being investigated are fracture critical meaning that a failure of these member could cause catastrophic failure of the entire gate. However, the model only predicts crack initiation and not propagation. Realistically, once a crack initiates, it may take numerous cycles before the crack reaches its critical crack size and finally fails. If lock personnel or periodic inspection teams notice the crack before the crack becomes critical, repairs can be scheduled and navigation downtime will not be severe. If the crack is not noticed, it may progress until the member fails suddenly resulting in extended downtime. The three levels of consequences considered are:

> A. Low level of Consequences. Cracks are found by lock personnel or periodic inspection team at an early stage. Lock must be shutdown for two days for in depth inspection and repair. Conditional probability is 84%.
>
> B. Medium level of Consequences. Cracks are found before failure of the gate but they are of a more severe nature making it imperative to be repaired immediately. Emergency gates are used for a week resulting in a slow down. Shutdown during repair and inspection last for a week. Condition Probability is 15%
>
> C. High Level of Consequences. Gate fails while in use. Complete replacement of the gate is required. Emergency gates are used for six months during fabrication of new gates. Lock is completely shutdown for 3 weeks to remove old gate and install the new gate. Conditional Probability is 1%.

Lock	Low Level of Consequences (LC)				
	P(LC)	Nav. Down Time	Increased Lockage time	Length of slowdown	Repair costs ($1000)
Lockport	.84	~~7~~ 2 days	0	0	100
Lock 19	.84	~~7~~ 2 days	0	0	100

Lock	Medium Level of Consequences (MC)				
	P(MC)	Nav. Down Time	Increased Lockage time	slowdown period	Repair costs ($1000)
Lockport	.15	7 days	20 min	7 days	500
Lock 19	.15	7 days	20 min	7 days	500

Lock	High Level of Consequences (HC)				
	P(HC)	Nav. Down Time	Increased Lockage time	Length of slowdown	Repair costs ($1000)
Lockport	.01	21 days	20 min	180 days	5,000
Lock 19	.01	21 days	20 min	180 days	5,000

V. Rehabilitation and Enhanced Maintenance Costs

15. The table below lists costs estimated for rehabilitation and enhance maintenance of the gates. The exact work needed for the rehabilitation is unknown at this time, therefore engineering judgement was used to establish the amount of work needed for rehabilitation.

Cost Table

Lock	Rehabilitation Costs	Enhanced Maintenance Cost
Lockport	$800,000	$50,000
Lock 19	$800,000	$50,000

SECTION 4 - Roller Gates

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Reliability Analysis for Roller Gates

I. Model Description

Background

1 Roller gates are present at most Lock and Dam sites on the Upper Mississippi River but are absent on the Illinois Waterway. Roller gates are generally quite large in comparison to the tainter gates at the same site. Typically, roller gates will have a span of around 100 feet and have 20 or more feet of head at the upstream side. See the table on the following page for a list of roller gates on the Mississippi River locks nd dams. Because of their size, if a roller gate catastrophically were to fail, pool could easily be lost.

2. A roller gate is essentially a large tube with either one or two "aprons" attached. The tube consisting of skin plate is the main structural element in the gate. The gate is raised or lowered by rolling it up or down an inclined surface on piers at each end of the gate. It is rolled by a single chain located at one of its ends (driven end). Externally, the gate is statically determinant. Hence, moments, torques, and shears on sections transverse to the drum can be readily determined. At regular spacing within the drum and apron, there is internal framing. This framing braces the drum and aprons and allows the drum to act as a large beam subjected to biaxial moments and torsion. While the stresses in the drum can be readily determined, the load in the internal framing is highly indeterminant and difficult to analyze. The roller gate model will comprise of two models, a drum model and an internal framing model.

Drum Model

General

3. Roller gates act as large beams simply supported by the piers at each end. The piers prevent translation both vertically and horizontally at the point of contact but do not resist rotation. A chain is provided at one end of the gate to raise and lower the gate which also acts to resist rotation of the drum. Because the chain only supports the gate at one end, the gate also acts in torsion as well as in bending. The drum model considers nonsymetric bending of the gate combined with torsion. For flexural, section properties are computed including aprons and longitudinal channels. Torsion properties only include the drum of the gate since longitudinal channels and lower aprons are open sections and add very little torsional stiffness.

Roller Gate Data

Dam	Number of Gates	Design Head	Gate Length	Gate Height	Drum Diameter
3	4	8.0	80	20	14'-11"
4	6	7.0	60	20	14'-11"
5	6	9.0	60	20	14'-11"
5A	5	5.5	80	20	14'-11"
6	5	6.5	80	20	14'-11"
7	5	8.0	80	20	14'-11"
8	5	11.0	80	20	14'-11"
9	5	9.0	80	20	14'-11"
10	4	8.0	80	20	14'-8"
11	3	11.0	100	20	12.91'
12	3	9.0	100	20	12.91'
13	3	11.0	100	20	12.91'
14	4	11.0	100	20	12.91'
15	11	16.0	100	23.75	19.4'
16	4	9.0	80	20	14'-8"
17	3	8.0	100	20	12'-11"
18	3	9.8	100	20	12.91'
20	3	10.0	60	20	14'-8"
21	3	10.5	100	25	12.91'
22	3	10.2	100	25	15'-11"
25	3	15.0	100	25	17'-11"

Loads

4. Only hydraulic loads were considered. Since the upper pool is relatively constant, only the tailwater elevation is the only load parameter which was treated as a random variable. The maximum load against the gate occurs when the tailwater is a minimum. The annual minimum tailwater reading is used in the analysis.

5. Because the gate is comprised of a drum and one or two circular arcs, the computation of the forces acting on the gate are more involved than for simpler gates. It should be noted the loads on the aprons produce a torque on the gate. To determine the net torque acting on a section, it can be assumed that the load from either apron will pass through the center of the arc of the apron.

6. Fatigue was not considered in the analysis because the gates see relatively few loading cycles. While the settings on the gate change frequently, the amount of change is usually very small.

Corrosion

7. The model simulates the effects of corrosion. It first calculates the amount of section loss and its location and then recalculates the section properties to account for loss of section. The model uses the corrosion rate and procedure established in Reference (Fatigue and Corrosion). In general, the corrosion in the splash zone (the area near the water line) will suffer the most corrosion loss. While there will be some loss in areas that are constantly submerged or constantly above water, the magnitude of the corrosion in these areas will be much less than in the splash zone. For this reason, atmospheric and submerged corrosion can be ignored.

8. Splash zone corrosion occurs on both the upstream and downstream side of the gate. At the upstream side, splash zone corrosion only affects the top apron for a double apron gate. For the upstream side of a single apron gate, the corrosion will be near the top of the drum. For splash zone corrosion on the downstream side, both types of gates are affected in the same way. The corrosion will occur in several places.

Stress Computation

9. The model uses the general flexure equation to compute stresses as given in equation 1 below. The section properties computed are the area (A), center of gravity (cg_x and cg_y), moment of inertia about both X and Y axis (I_x and I_y) and the product of inertia (I_{xy}). While A, cg_x, and cg_y are not used directly in the general flexure equation, they are needed to compute I_x, I_y , and I_{xy}.

$$\sigma=\frac{M_xI_y-M_yI_{xy}}{I_xI_y-I_{xy}^2}y+\frac{M_yI_x-M_xI_{xy}}{I_xI_y-I_{xy}^2}x \qquad (1)$$

10. To find the point of maximum stress, the orientation of the neutral axis must be determined. To determine this the following equation was used:

$$\tan(\lambda)=\frac{M_xI_{xy}+M_yI_x}{M_xI_y+M_yI_{xy}} \qquad (2)$$

where λ is the angle measured from the x-axis to the neutral axis. Given the angle of inclination of the neutral axis the point of maximum and minimum bending stress can be easily determined. This point will lie on the drum on a line drawn through the center of the drum and perpendicular to the neutral axis. The values of Ix, Iy, and Ixy reflect amount of corrosion on the drums and aprons.

Limit States

11. Two limit states were considered: capacity of the tension face, and capacity of the compressive face. The capacity of the tension face is not affected by stability and is a function of the yield stress and applied stress. Because the tension face is generally farther away from the neutral axis than the compression face (due to location of the aprons), it will develop higher flexural stresses. In addition to the flexural stresses, torsional shear stresses are present. The torsional stresses are combined with flexural stresses using a Mohr circle analysis.

12. The capacity of the compression is affected by stability which means the critical compressive stress is reduced as the compressive face corrodes and looses section. First, a critical compressive stress is computed based on the thickness and yield strength of the plate. If this value is less than the yield strength, it will governs when computing the factor of safety of the compression face. Similar to the tension face, shear stresses due to torsion are combined with the flexural stresses using a Mohr circle approach.

Random Variables

13. Four variables were chosen to be treated as random variables. They include: tailwater elevation, splash zone corrosion rate, size of splash zone, and yield strength. These variables are used in a Taylor Series approach to establish a distribution of the factor of safety.

<u>Splash zone corrosion rate</u> - The splash zone corrosion rate is described in the WES report on Fatigue and Corrosion. It reflects the corrosion rate of bare steel in a riverine environment.

<u>Exposure Rate</u> - Since the tailwater elevation varies, the location of the splash zone varies from day to day and no particular spot is always in the splash zone. Hence, the splash zone corrosion rate suggested in the report was reduced to reflect the actual percent of time that a spot is in the splash zone.

Splash zone size - The size of the splash zone varies substantially due to wave size and turbulence from passing water. Also, the splash zone is different (smaller) inside the drum.

Yield Stress - The steel used to construct these gates is A7. The mean yield strength was taken as 33 ksi with a coefficient of variation (COV) of 10%. The COV used is similar to that used in the Fatigue and Corrosion Report.

Tailwater Elevation - Since the yearly maximum load occurs when the tailwater is a minimum, the yearly minimum tailwater elevation (both mean and standard deviation) were derived from historical data. The daily tailwater readings were not used because it was felt that their use would lead to erroneous results. On a daily basis, the tailwater elevation can vary dramatically. However, there is a very real lower limit on the tailwater elevation (upper limit on applied head). Basically, the tailwater will not fall below the pool elevation of the next downstream dam which is kept at a constant level. In other words, while there may be a lot of uncertainty about how high the tailwater can rise, there is very little uncertainty about how low the tailwater can fall.

Constants

14. All geometric parameters were treated as constants. This includes initial thickness, length of gate, radius and location of aprons and drum. Other constants include the life of a paint system, and frequency of painting in the future.

List of Random Variables

Splash Zone Corrosion Rate Variability (ϵ)
Splash Zone Size: (lc)
Yield stress (Fy)
Exposure (Kp)
Tailwater Elevation (tail)

List of Constants

Basic Geometry (drum radius and thickness, location of aprons, etc)
Corrosion Parameters A= 148.5 B= 0.903
Life of a Lead based paint system : 10 yrs
Life of a Vinyl Paint System: 20 yrs
Pool Elevation

Computation of Section Properties for the Drum Model

15. Because of the geometry of the roller gates, the computation of the section

properties is not as simple as picking a value out of a table. In fact, computation of the section properties are the most difficult task required by the drum model. The gates are comprised of a circular drum and several aprons which consists of circular arcs. Also, the majority of corrosion occurs in the splash zone which covers only a small portion of the gate. To compute the values needed in the general flexure equation, the center of gravity must first be computed (cg_x and cg_y).

$$cg_x=\frac{\sum Ax}{\sum A} \qquad cg_y=\frac{\sum Ay}{\sum A}$$

For circular arcs,

$$A=rt\alpha$$

where r is the radius of the arc and α is the angle subtended by the arc.

$$\alpha=\beta-\theta$$

To determine Ax and Ay,

$$Ax=\int_{\theta}^{\beta}(x_o+r\cdot\sin\phi)\cdot r\cdot t\cdot d\phi$$
$$Ax=r\cdot t[x_o\cdot\alpha-r(\cos\beta-\cos\theta)]$$

$$Ay=\int_{\theta}^{\beta}(y_o+r\cdot\cos\phi)\cdot r\cdot t\cdot d\phi$$
$$Ay=r\cdot t[y_o\cdot\alpha-r(\sin\beta-\sin\theta)]$$

where y_o and x_o are the x and y coordinates of the center of the arc respectively. These values are computed for the top apron, bottom apron, and the drum as well as the negative contributions made by corrosion.

16. The moments of inertia and product of inertia of the roller gates can be determined once the center of gravity of the gate is found. To determine these values, their definition must be employed.

$$I_x=\int y^2 dA \qquad I_y=\int x^2 dA \qquad I_{xy}=\int xy dA$$

For arcs, it is convenient to let

$$dA=r\cdot t\cdot d\phi$$

The values of "x" and "y" are the distances to the center of gravity and for a particular component can be given by:

$$x = x_o - cg_x + r\sin\phi$$
$$y = y_o - cg_y + r\cos\phi$$

For convenience, let

$$a_x = x_o - cg_x \qquad a_y = y_o - cg_y$$

Hence,

$$I_x = \int_\theta^\beta (a_y + r\cos\phi)^2 \cdot r \cdot t \cdot d\phi$$
$$I_x = r\cdot t\cdot[a_y^2\cdot\alpha + 2a_y\cdot r\cdot(\sin\beta - \sin\theta) + \frac{r^2}{2}(\alpha + \frac{\sin(2\beta)-\sin(2\theta)}{2})]$$

$$I_y = \int_\theta^\beta (a_x + r\sin\phi)^2 \cdot r \cdot t \cdot d\phi$$
$$I_y = r\cdot t\cdot[a_x^2\cdot\alpha - 2a_x\cdot r\cdot(\cos\beta - \cos\theta) + \frac{r^2}{2}(\alpha - \frac{\sin(2\beta)-\sin(2\theta)}{2})]$$

$$I_{xy} = \int_\theta^\beta (a_x + r\sin\phi)(a_y + r\cos\theta)\cdot r\cdot t\cdot d\phi$$
$$I_{xy} = r\cdot t\cdot[a_x\cdot a_y\cdot\alpha - a_y\cdot r\cdot(\cos\beta - \cos\theta) + a_x\cdot r\cdot(\sin\beta - \sin\theta) + \frac{r^2}{2}(\sin^2\beta - \sin^2\theta]$$

17. Once the moments of inertia about the centroid of the gate for each section is determined, they are added up to give the value for the entire section. At this point, the general flexure equation can be employed to determine the flexural stresses.

Internal Framing Model

General

18. The internal framing consists of beams and diagonals which brace the drum and allow it the act as a section. It appears that most of the hydraulic load imposed against the gate is carried by the arching action of the skin plate and is not directly resisted by the internal framing. Because it is bracing, it is difficult to determine the exact load carried by the different elements of the framing. Roller gates have been rehabilitated at several sites in the Rock Island District. Generally, the rehabilitation involves replacement of internal framing members due to excessive corrosion. But despite the corrosion levels, there have not been cases of unsatisfactory performance involving any of the internal framing members or the skin plate.

19. Generally, there is a lot of uncertainty in the stress levels of all the internal framing. During the 1930's, strain gages were installed on the roller gates at L/D 15 to measure the stress levels in the internal framing. The results varied greatly between frames. The difficulty in the analysis lies in the geometry of the roller gate. The internal framing transfers part of the load from one location on the skin plate to another spot on the skin plate. This makes a frame analysis difficult because not only is it uncertain how much of the load is transfer by the internal framing but it is also unclear what the boundary conditions would be. Also, the framing is highly redundant making the loss of a single diagonal relatively inconsequential.

20. The original design of the internal framing was very simplistic. Generally, the beams are treated as simply supported beams which carry half of the hydraulic load that the skin plate is subjected to. The diagonals then act as compression members to support the beam elements.

21. In most cases, the internal framing will govern over the drum as the probable cause of unsatisfactory performance because of the thickness of the elements. The diagonals and beams are typically made of 3/8 in thick steel whereas the drum is constructed of 5/8 or 3/4 in steel. Therefore, for the same amount of corrosion loss, the internal framing will lose a greater percentage of its strength.

Element Selection

22. Between the beams and diagonals, the latter is the more critical. The beams appear to be considerably overdesigned. This is especially true on the double apron gates where the beams are especially deep.

Analysis

23. The internal frame model will focus on the diagonals. The analysis of these members was done using frame analysis program STAAD III. As mentioned above, the exact load carried by the diagonals is not easily determined. To accurately determine the load in the framing members, a large finite element model would have to be developed which would be beyond the scope of this study.

24. The original analysis considers the segmental girders as simply supported beams which are supported by the diagonals in compression. This approach will be continued for this analysis except that a value of 30% of the load will be resisted by the members instead of the 50% assumed in the original design. The 30% value is the result of analysis of a curve plate supported at its ends given the basic dimensions of the drum.

Random Variables

25. The random variables used in the analysis are:

Yield Strength (Fy)
Analysis Uncertainty (ks) - Accounts for uncertainty in the analysis
Axial Load (P) from a frame analysis
Exposure Factor (kp) - Adjusts the corrosion rate based on the amount of time the steel is unpainted.

Constants

26. Items that were considered to be a constant in the analyis are the angle size and geometry of diagonals and the corrosion parameters (as for the drum model)

II. Site Selection

27. The model will not be implemented at each site since there are only a few different configurations of roller gates in the system. Also the heads are very similar among gates of the same configuration. Reliability of the drum will only govern at two sites, Dam 20 and Dam 25, which have 3/8 inch thick skin plates. The other sites have substantially thicker skin plate which gives them a higher initial factor of safety and makes them more tolerant of section loss due to corrosion. The following table shows the sites at which the model will be implemented and the site which will assumed to have the same hazard functions.

Site for Implementation	Model	Other Similar Sites
7	Diagonal	3, 4, 5, 6, 8, 9, 10
11	Diagonal	12, 13, 14, 17, 18, 21
15	Diagonal	16, 22
20	Drum	
25	Drum	

III Important Parameters

Drum Model
Random Variables

	Ks		ϵ		lc	
site	μ	σ	μ	σ	μ	σ
20	1.0	.2	0	.099	4	1
25	1.0	.2	0	.099	4	1

More Random Variables

	Fy		Kp		Tailwater	
site	μ	σ	μ	σ	μ	σ
20	33	3.3	.5	.1	-3.0	.98
25	33	3.3	.6	.1	-5.0	1.0

Some Constants

Site	Gate Span	Drum Radius	Skin Plate Thickness	Pool Elev.
20	100 ft	7.38 ft	.375	7
25	100 ft	9.0 ft	.375	9

Internal Framing
Random Variables

	Fy		kp		P	
site	μ	σ	μ	σ	μ	σ
3,4,5,6, 7,8,9,10	33	3.3	.4	.1	11.04	
11,12,13,14, 17,18,21	33	3.3	.4	.1	11.82	2
15,16,22	33	3.3	.4	.1	10.5	2

More Random Variables

	ϵ		ks	
site	μ	σ	μ	σ
3,4,5,6, 7,8,9,10	0	.099	.3	.1
11,12,13,14 17,18,21	0	.099	.3	.1
15,16,22	0	.099	.3	.1

IV. Hazard Functions

28. Three hazard functions are needed for the economic analysis. Normal O&M (unrehabilitated), rehabilitated, and enhanced maintenance. Under current O&M practices, the roller gates have been well maintained and show only slight section loss. Therefore, the normal O&M curve will assume that for 95% of the time an effective paint coating is in place. The splash zone corrosion rate established in the WES-JAYCOR report on fatigue and corrosion of hydraulic steel structures was multiplied by 0.05 to determine the actual section loss due to corrosion. The enhanced maintenance hazard function will reflect no section loss into the future. It is difficult to accurately access the condition of the gate after a rehabilitation but it is assumed that rehabilitation will not fully restore the gate to a new condition. Rather, it was assumed that in the first year after a rehabilitation, the gate will have the same probability of unsatisfactory performance as a new gate after 10 years of service.

29. Hazard functions are shown on the following tables. The numbers listed are for one roller gate only and must be multiplied by the number of gates per site listed in the table on page 2 to compute the probability of unsatisfactory performance of any one gate at a site in a given year.

30. The hazard function for the gate after an unsatisfactory performance would depend on the type of repair that was needed. For an unsatisfactory performance for the roller gate, the hazard function after repair can be assumed to be equal to the rehabilitated rate for the gate that is repaired.

Hazard Functions - Locks 3, 4, 5, 6, 7, 8, 9, and 10

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2000	0.00010	0.0001	0	0.00010
2001	0.00010	0.0001	1	0.00010
2002	0.00010	0.0001	2	0.00010
2003	0.00010	0.0001	3	0.00010
2004	0.00010	0.0001	4	0.00010
2005	0.00010	0.0001	5	0.00010
2006	0.00010	0.0001	6	0.00010
2007	0.00010	0.0001	7	0.00010
2008	0.00010	0.0001	8	0.00010
2009	0.00010	0.0001	9	0.00010
2010	0.00010	0.0001	10	0.00010
2011	0.00010	0.0001	11	0.00010
2012	0.00010	0.0001	12	0.00010
2013	0.00010	0.0001	13	0.00010
2014	0.00010	0.0001	14	0.00010
2015	0.00010	0.0001	15	0.00010
2016	0.00010	0.0001	16	0.00010
2017	0.00010	0.0001	17	0.00010
2018	0.00010	0.0001	18	0.00010
2019	0.00010	0.0001	19	0.00010
2020	0.00010	0.0001	20	0.00010
2021	0.00010	0.0001	21	0.00010
2022	0.00010	0.0001	22	0.00010
2023	0.00010	0.0001	23	0.00010
2024	0.00011	0.0001	24	0.00010
2025	0.00013	0.0001	25	0.00010
2026	0.00019	0.0001	26	0.00010
2027	0.00032	0.0001	27	0.00010
2028	0.00052	0.0001	28	0.00010
2029	0.00107	0.0001	29	0.00010
2030	0.00200	0.0001	30	0.00010
2031	0.00341	0.0001	31	0.00010
2032	0.00541	0.0001	32	0.00010
2033	0.00657	0.0001	33	0.00010
2034	0.01058	0.0001	34	0.00010
2035	0.01600	0.0001	35	0.00010
2036	0.02288	0.0001	36	0.00010
2037	0.03128	0.0001	37	0.00010
2038	0.03782	0.0001	38	0.00010
2039	0.05081	0.0001	39	0.00011

Hazard Functions - Locks 3, 4, 5, 6, 7, 8, 9, and 10

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2040	0.06600	0.0001	40	0.00013
2041	0.08322	0.0001	41	0.00019
2042	0.10232	0.0001	42	0.00032
2043	0.12147	0.0001	43	0.00052
2044	0.14503	0.0001	44	0.00107
2045	0.17000	0.0001	45	0.00200
2046	0.19615	0.0001	46	0.00341
2047	0.22333	0.0001	47	0.00541
2048	0.25142	0.0001	48	0.00810
2049	0.28034	0.0001	49	0.01160
2050	0.31000	0.0001	50	0.01600

Hazard Functions - Locks 11,12,13,14,17,18, and 21

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2000	0.00010	0.0001	0	0.00010
2001	0.00010	0.0001	1	0.00010
2002	0.00010	0.0001	2	0.00010
2003	0.00010	0.0001	3	0.00010
2004	0.00010	0.0001	4	0.00010
2005	0.00010	0.0001	5	0.00010
2006	0.00010	0.0001	6	0.00010
2007	0.00010	0.0001	7	0.00010
2008	0.00010	0.0001	8	0.00010
2009	0.00010	0.0001	9	0.00010
2010	0.00010	0.0001	10	0.00010
2011	0.00010	0.0001	11	0.00010
2012	0.00010	0.0001	12	0.00010
2013	0.00010	0.0001	13	0.00010
2014	0.00010	0.0001	14	0.00010
2015	0.00010	0.0001	15	0.00010
2016	0.00010	0.0001	16	0.00010
2017	0.00010	0.0001	17	0.00010
2018	0.00011	0.0001	18	0.00010
2019	0.00012	0.0001	19	0.00010
2020	0.00017	0.0001	20	0.00010
2021	0.00028	0.0001	21	0.00010
2022	0.00049	0.0001	22	0.00010
2023	0.00069	0.0001	23	0.00010
2024	0.00138	0.0001	24	0.00010

Hazard Functions - Locks 11,12,13,14,17,18, and 21

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2025	0.00250	0.0001	25	0.00010
2026	0.00415	0.0001	26	0.00010
2027	0.00643	0.0001	27	0.00010
2028	0.00811	0.0001	28	0.00010
2029	0.01245	0.0001	29	0.00010
2030	0.01800	0.0001	30	0.00010
2031	0.02477	0.0001	31	0.00010
2032	0.03276	0.0001	32	0.00010
2033	0.03915	0.0001	33	0.00010
2034	0.05076	0.0001	34	0.00010
2035	0.06400	0.0001	35	0.00010
2036	0.07870	0.0001	36	0.00010
2037	0.09474	0.0001	37	0.00010
2038	0.10918	0.0001	38	0.00011
2039	0.12892	0.0001	39	0.00012
2040	0.15000	0.0001	40	0.00017
2041	0.17221	0.0001	41	0.00028
2042	0.19542	0.0001	42	0.00049
2043	0.21788	0.0001	43	0.00069
2044	0.24356	0.0001	44	0.00138
2045	0.27000	0.0001	45	0.00250
2046	0.29706	0.0001	46	0.00415
2047	0.32466	0.0001	47	0.00643
2048	0.35271	0.0001	48	0.00943
2049	0.38117	0.0001	49	0.01326
2050	0.41000	0.0001	50	0.01800

Hazard Functions - Locks 15, 16, and 22

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2000	0.00010	0.0001	0	0.00010
2001	0.00010	0.0001	1	0.00010
2002	0.00010	0.0001	2	0.00010
2003	0.00010	0.0001	3	0.00010
2004	0.00010	0.0001	4	0.00010
2005	0.00010	0.0001	5	0.00010
2006	0.00010	0.0001	6	0.00010
2007	0.00010	0.0001	7	0.00010
2008	0.00010	0.0001	8	0.00010
2009	0.00010	0.0001	9	0.00010
2010	0.00010	0.0001	10	0.00010
2011	0.00010	0.0001	11	0.00010
2012	0.00010	0.0001	12	0.00010
2013	0.00010	0.0001	13	0.00010
2014	0.00010	0.0001	14	0.00010
2015	0.00010	0.0001	15	0.00010
2016	0.00010	0.0001	16	0.00010
2017	0.00010	0.0001	17	0.00010
2018	0.00010	0.0001	18	0.00010
2019	0.00010	0.0001	19	0.00010
2020	0.00010	0.0001	20	0.00010
2021	0.00010	0.0001	21	0.00010
2022	0.00010	0.0001	22	0.00010
2023	0.00041	0.0001	23	0.00010
2024	0.00085	0.0001	24	0.00010
2025	0.00160	0.0001	25	0.00010
2026	0.00274	0.0001	26	0.00010
2027	0.00436	0.0001	27	0.00010
2028	0.00547	0.0001	28	0.00010
2029	0.00871	0.0001	29	0.00010
2030	0.01300	0.0001	30	0.00010
2031	0.01837	0.0001	31	0.00010
2032	0.02483	0.0001	32	0.00010
2033	0.02959	0.0001	33	0.00010
2034	0.03946	0.0001	34	0.00010
2035	0.05100	0.0001	35	0.00010
2036	0.06408	0.0001	36	0.00010
2037	0.07859	0.0001	37	0.00010
2038	0.09099	0.0001	38	0.00010
2039	0.10968	0.0001	39	0.00010

Hazard Functions - Locks 15, 16, and 22

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2040	0.13000	0.0001	40	0.00010
2041	0.15173	0.0001	41	0.00010
2042	0.17471	0.0001	42	0.00010
2043	0.19987	0.0001	43	0.00041
2044	0.22475	0.0001	44	0.00085
2045	0.25000	0.0001	45	0.00160
2046	0.27555	0.0001	46	0.00274
2047	0.30136	0.0001	47	0.00436
2048	0.32739	0.0001	48	0.00655
2049	0.35361	0.0001	49	0.00940
2050	0.38000	0.0001	50	0.01300

Hazard Functions - Lock 20

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2000	0.00010	0.0001	0	0.00010
2001	0.00010	0.0001	1	0.00010
2002	0.00010	0.0001	2	0.00010
2003	0.00010	0.0001	3	0.00010
2004	0.00010	0.0001	4	0.00010
2005	0.00010	0.0001	5	0.00010
2006	0.00010	0.0001	6	0.00010
2007	0.00010	0.0001	7	0.00010
2008	0.00010	0.0001	8	0.00010
2009	0.00010	0.0001	9	0.00010
2010	0.00010	0.0001	10	0.00010
2011	0.00010	0.0001	11	0.00010
2012	0.00010	0.0001	12	0.00010
2013	0.00010	0.0001	13	0.00010
2014	0.00010	0.0001	14	0.00010
2015	0.00010	0.0001	15	0.00010
2016	0.00010	0.0001	16	0.00010
2017	0.00010	0.0001	17	0.00010
2018	0.00010	0.0001	18	0.00010
2019	0.00011	0.0001	19	0.00010
2020	0.00013	0.0001	20	0.00010
2021	0.00021	0.0001	21	0.00010
2022	0.00045	0.0001	22	0.00010
2023	0.00082	0.0001	23	0.00010
2024	0.00215	0.0001	24	0.00010

Hazard Functions - Lock 20

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2025	0.00480	0.0001	25	0.00010
2026	0.00938	0.0001	26	0.00010
2027	0.01663	0.0001	27	0.00010
2028	0.02779	0.0001	28	0.00010
2029	0.04359	0.0001	29	0.00010
2030	0.06300	0.0001	30	0.00010
2031	0.08587	0.0001	31	0.00010
2032	0.11209	0.0001	32	0.00010
2033	0.14843	0.0001	33	0.00010
2034	0.17897	0.0001	34	0.00010
2035	0.21000	0.0001	35	0.00010
2036	0.24142	0.0001	36	0.00010
2037	0.27317	0.0001	37	0.00010
2038	0.32187	0.0001	38	0.00010
2039	0.34685	0.0001	39	0.00011
2040	0.37000	0.0001	40	0.00013
2041	0.39179	0.0001	41	0.00021
2042	0.41252	0.0001	42	0.00045
2043	0.44804	0.0001	43	0.00082
2044	0.45973	0.0001	44	0.00215
2045	0.47000	0.0001	45	0.00480
2046	0.47925	0.0001	46	0.00938
2047	0.48774	0.0001	47	0.01663
2048	0.49563	0.0001	48	0.02735
2049	0.50302	0.0001	49	0.04248
2050	0.51000	0.0001	50	0.06300

Hazard Functions - Lock 25

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2000	0.00010	0.0001	0	0.00010
2001	0.00010	0.0001	1	0.00010
2002	0.00010	0.0001	2	0.00010
2003	0.00011	0.0001	3	0.00010
2004	0.00017	0.0001	4	0.00010
2005	0.00033	0.0001	5	0.00010
2006	0.00073	0.0001	6	0.00010
2007	0.00160	0.0001	7	0.00010
2008	0.00185	0.0001	8	0.00010
2009	0.00489	0.0001	9	0.00010
2010	0.01100	0.0001	10	0.00010
2011	0.02170	0.0001	11	0.00010
2012	0.03878	0.0001	12	0.00010
2013	0.07869	0.0001	13	0.00010
2014	0.11251	0.0001	14	0.00010
2015	0.15000	0.0001	15	0.00010
2016	0.19070	0.0001	16	0.00010
2017	0.23428	0.0001	17	0.00010
2018	0.29260	0.0001	18	0.00010
2019	0.33665	0.0001	19	0.00010
2020	0.38000	0.0001	20	0.00010
2021	0.42279	0.0001	21	0.00010
2022	0.46513	0.0001	22	0.00010
2023	0.51564	0.0001	23	0.00010
2024	0.55350	0.0001	24	0.00010
2025	0.59000	0.0001	25	0.00010
2026	0.62545	0.0001	26	0.00010
2027	0.66006	0.0001	27	0.00010
2028	0.71794	0.0001	28	0.00011
2029	0.74015	0.0001	29	0.00017
2030	0.76000	0.0001	30	0.00033
2031	0.77815	0.0001	31	0.00073
2032	0.79500	0.0001	32	0.00160
2033	0.81034	0.0001	33	0.00185
2034	0.82534	0.0001	34	0.00489
2035	0.84000	0.0001	35	0.01100
2036	0.85438	0.0001	36	0.02170
2037	0.86855	0.0001	37	0.03878
2038	0.88200	0.0001	38	0.07869
2039	0.89600	0.0001	39	0.11251
2040	0.91000	0.0001	40	0.15000

Hazard Functions - Lock 25

Year	Normal O&M	Enhanced Maintenance	Year from Rehabilitation	Rehabilitated
2041	0.92400	0.0001	41	0.19070
2042	0.93800	0.0001	42	0.23428
2043	0.97344	0.0001	43	0.29260
2044	0.97705	0.0001	44	0.33665
2045	0.98000	0.0001	45	0.38000
2046	0.98250	0.0001	46	0.42279
2047	0.98469	0.0001	47	0.46513
2048	0.98663	0.0001	48	0.50707
2049	0.98839	0.0001	49	0.54868
2050	0.99000	0.0001	50	0.59000

V. Consequences

31. No data exists on the consequences of unsatisfactory performance of roller gates on the Upper Mississippi River. While there have been internal framing members found which were extremely corroded, in no instance has there been any downtime to navigation. The consequences of failure are more severe if the drum fails rather than the internal framing. If the drum fails, there is a greater chance of losing the gate and the navigation pool. On the other hand, the consequences will probably be quite low if a diagonal fails since the internal framing is quite redundant.

Table of Consequences

Lock	Low Level of Consequences (LC)			Medium Level of Consequences (MC)			High Level of Consequences (HC)		
	P(LC)	Nav. Down Time	Repair costs	P(MC)	Nav. Down Time	Repair costs	P(HC)	Nav. Down Time	Repair costs
3,4,5,6,7,8,9,10	.9	0	$50,000	.09	2	$200,000	.01	7	$1,000,000
11,12,13,14,17, 18, 21	.9	0	$50,000	.09	2	$200,000	.01	7	$1,000,000
15,16,22	.9	0	$50,000	.09	2	$200,000	.01	7	$1,000,000
20	.9	2	$200,000	.09	7	$500,000	.01	14	$3,000,000
25	.9	2	$200,000	.09	7	$500,000	.01	14	$3,000,000

VI. Costs of Rehabilitation and Enhanced Maintenance

32. For the gates which are governed by the internal frame model,the cost of rehabilitation will be based on recent rehabilitation of roller gates in the Rock Island District. For the gates governed by the drum model (20, 25), no data exists on replacing large pieces of skin plate on the drum. Hence those values are based on engineering judgement.

Cost Table

	Number of Roller Gates	Rehabilitation Costs ($)	Enhanced Maintenance Costs ($/yr)
3, 4, 5	6	600,000	50,000
6, 7, 8, 9	5	500,000	42,000
10, 14	4	400,000	33,000
11, 12, 13, 14, 17, 18, 21	3	350,000	25,000
15	11	1,800,000	150,000
16, 22	4	400,000	33,000
20, 25	3	2,000,000	40,000

SECTION 5 - Tainter Gates

Reliability Analysis for Tainter Gates

I. Model Description

General

1. The reliability model for tainter gates was been developed using methods outlined in the report written by WES and JAYCOR titled "Reliability Analysis of Hydraulic Steel Structures with Fatigue and Corrosion Degradation", March 1, 1994; by ITL-95-3 written by Bruce R. Ellingwood titled,"Engineering Reliability and Risk Analysis for Water Resources Investments; Role of Structural Degradation in Time-Dependent Reliability Analysis"; and from "Probabilistic Structural Mechanics Handbook" by C. Sundararajan.

2. Tainter gates are steel structures composed of many members. For a given gate and loading, several members could be critical to the reliability of the gate. Unsatisfactory performance of the different critical members may have different consequences for the gate ranging from a simple repair of a bent member to complete loss of the gate and potential loss of the pool. Therefore, a complete structural analysis must be performed on the tainter gate so that the critical members, critical loads, and the limit state of the members can be identified and the member reliabilities computed. The overall reliability of the gate is determined from the reliability of the critical members.

3. The tainter gate reliability models were developed using conventional 2-dimensional modeling techniques such as those outlined in EM 1110-2-2702, "Design of Spillway Tainter Gates" or as described in design data on the original construction drawings. Models were developed for four different tainter gate types found on the Upper Mississippi River lock system.

4. For initial computation of reliability, the Taylor series method described in the WES JAYCOR report was used and the distribution for maximum yearly loading was included as a random variable. For the final computation of reliability, a method was used where the probability of load exceedence curve was combined with a fragility curve for the structure to compute the probability of unsatisfactory performance. The fragility of a structure is its probability of unsatisfactory performance under a given loading. To compute the fragility curve, the loading was considered a constant and the Taylor series method was used to compute probability of unsatisfactory performance due to a number of different loads.

5. The table on the following page lists general data for the tainter gates at each lock and dam site that has tainter gates.

Tainter Gate Data

Dam	Number of Gates	Design Head	Gate Height	Gate Width	Gate Radius
2	20	12.2	20	30	28
4	22	7.0	15	35	25
5	28	9.0	15	35	25
5A	5	5.5	15	35	25
6	10	6.5	15	35	25
7	11	8.0	15	35	25
8	10	11.0	15	35	25
9	8	9.0	15	35	25
10	8	8.0	20	40	32
11	13	11.0	20	60	30
12	7	9.0	20	64.17	30
13	10	11.0	20	64.17	30
14	13	11.0	20	60	27.5
15	0	16.0	-	-	-
16	15	9.0	20	40	30
17	8	8.0	20	64.17	32.5
18	14	9.8	20	60	30
19	0	36.2	-	-	-
20	40	10.0	20	40	30
21	10	10.5	20	64.17	32.5
22	10	10.2	25	60	40
24	15	15.0	25	80	33
25	14	15.0	25	30	40
26	-	-	-	-	-
Starved Rock	10	17.0	17	60	38
Marseilles	8	13.3	15	59	25
Dresden Island	9	14.0	14	60	25
Brandon Road	21	2.3	2.6	50	6

Definition of Unsatisfactory Performance

6. A tainter gate is comprised of many components, any one of which could reach its limit state and cause unsatisfactory performance of the gate. The expected results of each member type exceeding its limits state are as described in the following paragraphs.

7. For the skin plate on most gate types, there do not seem to be any unsatisfactory performance modes which would affect the overall integrity of the gate. The skin plate serves to contain water behind the gate and transfer loads to the ribs. It is designed as a 2-d plate, but may actually act as a 3-d plate or a diaphragm as well. Because there is conservatism in the design method used for it and because its performance does not impact the overall structural capacity of the gate, the skin plate was not considered a critical member for the reliability analysis except in the cases where it acted as a component of the main structural members.

8. Ribs carry load from the skin plate to the main girder-strut frames and the limit state is bending. Since there are many ribs parallel to each other, for there to be a significant problem with the gate, several ribs would have to reach their limit state at the same time. Yielding of just one rib would transfer load through the skin plate to adjacent ribs and would not result in complete collapse of the gate. This condition would be noticeable and the gate could be bulkheaded and repaired with no impacts on navigation.

9. Horizontal girders take load from the ribs to the strut arms. Unsatisfactory performance would be when the girder reaches its limit state due to bending or shear. When a girder reached its limit state, it could result in complete collapse of the gate, especially for tainter gates with just two girder-strut frames; but there would be some load redistribution between load frames and from the girders into the strut arms. Unsatisfactory performance of the girders could result in complete loss of the gate or the gate may jam in place and still be effective to dam water.

10. Strut arms take load from the girders to the trunnion and act as beam-columns. Unsatisfactory performance would result when the strut arm reach their limit states of yielding or buckling, depending on the combination of forces that act on them. Unsatisfactory performance in a strut arm could result in collapse of the gate but like the girders, there is some load redistribution between frames and from the strut arms into the girders.

11. Unsatisfactory performance for a trunnion pin would result when it reached its limit state for shear or, depending on the layout of the trunnion, bending. Unsatisfactory performance of a trunnion pin would result in loss of the gate since there is no redundancy in this member. Impacts would be the same as for the girders. Because of the large amount of conservatism in the design of trunnion pins, and the lack of corrosion in the greased trunnion which would lead to decreased

reliability with time, reliability for the trunnion pin wasn't computed.

12. The gate anchorage transfers the gate loads to the concrete piers. These members are embedded in the concrete piers and because there is no corrosion or fatigue mechanisms which could lead to decline in reliability of the anchors with time, reliability for these members was not computed.

Load Cases

13. Although there are many different load conditions that a tainter gate can experience, two or three cases representing the maximum expected loadings were evaluated for each site for which a reliability analysis was conducted. The basic load cases analyzed were as follows:

A. Headwater and tailwater which create the maximum force on the gate in a year. Gate resting on sill. (This case was not applicable for submergible gates).

B. Headwater and tailwater which create the maximum force on the gate in a year. Gate being lifted by chains with chain pull even on both sides of the gate.

C. Ice load on gate. Gate resting on sill for non-submergible gates and supported by chains for submergible gates.

Random Variables

14. Random variables are used to compute reliability by the Taylor Series method described in the WES-JAYCOR report. The following random variables are used in the reliability models and mostly come from the WES - JAYCOR report. Fatigue is not a concern for tainter gates.

15. <u>Corrosion Rate.</u> The random variable for corrosion is ϵ_c and the amount of corrosion, C, is defined by the equation:

$$\log C = \log A + B \log t + \epsilon_c.$$

For C in millimeters the variables for the corrosion equation for different conditions are:

Splash zone corrosion, A= 148.5; B=0.903; ϵ_c avg=0 with std. dev. = 0.099
Atmospheric corrosion, A= 23.4; B=0.650; ϵ_c avg=0 with std. dev. = 0.219
Submerged corrosion , A= 51.6; B=0.650; ϵ_c avg=0 with std. dev. = 0.174

For the reliability analysis, the gates were assumed to corrode at times when the paint is no longer effective. The original paint was assumed to be effective until 1948 and for 15 years after each subsequent painting.

16. <u>Steel Yield Strength.</u> The random variable is defined by LRFD research as follows:

Tension. Mean = 1.05 Fy, Std. Dev. = 0.11
Bending. Mean = 1.08 Fy, Std. Dev. = 0.14
Shear. Mean = 1.10 Fy, Std. Dev. = 0.15

17. <u>Loading.</u> Loads on the tainter gates are created by ice and water forces and vary by site. Random variables are used for uncertainty in the elevation of the water and the magnitude of the ice load. The critical loading for a tainter gate is the maximum water or ice load it would see in a given time period. Since the reliability is computed on an annual basis, mean and standard deviation for loading is the critical loading on the gate during a given year. Water elevations at each site were determined from existing records of pool and tailwater data. In most cases the pool was assumed to be a constant elevation.

18. For the initial computations of reliability, random variables for ice loadings were estimated using two methods. For tainter gates on the Mississippi River, ice loads are created by thermally expanding ice forces. Gates at Dam 5 have been damaged by ice loads and calculation of the ice loads necessary to cause the ice damage concluded that a maximum of about 3.7 k/ft was seen by the gates. But only a few gates were damaged, and therefore, most of the gates at the dam saw far less ice load. Based on the analysis, an average maximum yearly ice load of 2 k/ft was assumed with a standard deviation of 1 k/ft. Standard deviation for ice loading is expected to be quite high.

19. Several tainter gates on the Illinois Waterway have been severely damaged in the past by floating ice sheets. For the ice loads at these sites, the ice loading was estimated by assuming the design ice load for tainter gates suggested in EM 1110-2-2702 of 5 kip/ft is the 95th percentile of normally distributed ice loads on the gate and the variance is 25% of the mean ice load. The mean ice load assumed used was therefore 3.6 kip/ft with a standard deviation was 0.9 kip/ft.

20. <u>Ratio of actual to computed forces.</u> This random variable is Ks and is the ratio of actual to calculated stresses. The numbers for Ks that were used were determined in the research for the AISC LRFD code so their applicability to tainter gates is questionable.

Mean Ks = 1.02, Std. dev. = 0.10.

Procedure for Analyzing Reliability

21. The procedure used for computing reliability for tainter gates is described below. Much of the work in these steps was consolidated into a spreadsheet for the three tainter gate types on the Mississippi River. Reliability for tainter gate types which are found on the Illinois waterway were done without use of a spreadsheet to summarize information. All analysis was done for average loadings and for one standard deviation above and below the average load, for each load case.

A. Information on the geometry, member properties, weight, and loadings for the tainter gate was collected. Critical members were identified for which reliability would be computed.

B. The loads on the gate and the overall reactions of the gate at the trunnions, gate sill, and lifting chains were computed. The loads, reactions and all further analysis were computed at the average loading and one standard deviation above and below the average.

C. A structural analysis of the gate was performed in several steps in order to compute the forces in the critical members. For different gate types, some of the analysis was done by frame analysis computer programs, some was done by the use of standard formulas and some was done using formulas provided on the design data found on the as-built drawings.

D. Reliability for limit states due to bending, shear, and axial loads in the members was computed. The analysis in steps C produces average forces and forces one standard deviation above and below the average for use in computing the reliability using the Taylor Series method. Similar but different spreadsheets were prepared for computing reliabilities for several different limit states for the different types of critical members present. Equations in the AISC, LRFD manual of steel construction were used to compute a factor of safety for each limit state for each member type. The Taylor Series method was used to compute the reliability factor, Beta, and then a probability of unsatisfactory performance was computed from that. Since the loading were considered to be yearly, the resulting reliability is yearly and subtracting it from 1.0 would represent the probability of unsatisfactory performance in a given year. Information from this step was used to find critical members and loadings.

E. Final probability of unsatisfactory performance for the ice load cases, which were the critical cases for several dams on the Upper Mississippi and Illinois Waterway, were computed by using a method outlined in "Probabilistic Structural Mechanics Handbook" by C. Sundararajan. This method combines the curve for probability of load exceedence in a year with the fragility curve for the structure to calculate the probability of unsatisfactory performance.

This method was better for final computation of the probability of unsatisfactory performance because it provided more flexibility in the type of load distribution used. For the computation of unsatisfactory performance per year, an exponential distribution was used for the ice loading which seemed to provide a better prediction of unsatisfactory performance than by assuming the loading to be normally distributed. This method was checked by using a normal distribution for loading and the computed unsatisfactory performance was very close to that predicted when the loading uncertainty was included in the Taylor series method used to compute reliability described in paragraph D.

II. Site Selection

22. Because of the large amount of time that is required to compute the reliability for an individual tainter gate, gates from certain locks and dams were selected which would be representative of the remaining sites. This simplification could be done because the reliability for the tainter gates was for the most part very high due to conservative design criteria and design loadings that are much greater than expected actual loadings. Only ice loadings on a few gates created reliabilities low enough to be significant. The gates that were analyzed and the reasons for their selection are as follows:

A. Dam 10. The gates at this site were selected because Dam 10 was designed by the Rock Island District but is in the St. Paul District now. The tainter gates at Dams 16 and 20 are of the same type.

B. Dam 5. The gates at this site are identical to gates at Dam 4 and identical except for the top strut arms of the gates at dams 5A through 9. This dam was chosen because several gates at this dam have been slightly bent by ice loads. Results from this site can be estimated to be similar for other the sites.

C. Dam 13. The submergible gates at this site in the Rock Island District is typical of gates at Dams 12, 13, 17, 18 and 21 and has the most head differential.

D. Dam 22. The gates at this site are a type typical of gates at Dams 14 and 22.

E. Dam 24. The elliptical gates at this site in the St. Louis District are the only ones of their type.

F. Dam 25. The tainter gates at this site in the St. Louis District, similar in configuration to the type at Dam 13, have had limited past maintenance.

G. Dresden Island. The tainter gates at this site on the Illinois waterway were analyzed because two were heavily damaged by ice loads in the past and replaced. Only reliability for the existing gates was computed.

H. Starved Rock. The tainter gates at this site on the Illinois waterway are a type similar to the gates at Dresden Island, but of heavier construction.

III. Parameters

23. For all sites analyzed, random variables for corrosion rate and the ratio of actual to computed stresses, Ks, are as stated in the section above titled "Random Variables". Other variables are as stated in the table below.

Random Variables for Sites Analyzed

	Fyb	ksi	Fyv	ksi	Fh	ft.	Fi	k/ft	Fhi	ft
Dam	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
5	35.64	4.98	36.30	5.45	9.15	0.20	2.00	1.00	7.97	1.10
10	35.64	4.98	36.30	5.45	7.35	0.70	2.00	1.00	4.57	1.88
13	35.64	4.98	36.30	5.45	10.64	0.80	2.00	1.00	10.64	0.80
22	35.64	4.98	36.30	5.45	10.18	1.32	2.00	1.00	10.18	-
24	48.60	6.80	49.50	7.43	14.54	0.90	-	-	-	-
25	35.64	4.98	36.30	5.45	14.78	0.80	-	-	-	-
Dresden Island	35.64	4.98	36.30	5.45	14.00	-	3.60	0.90	14.00	-
Starved Rock	35.64	4.98	36.30	5.45	17.00	-	3.60	0.90	17.00	-

Fyb = Steel yield strength in bending
Fyv = Steel yield strength in shear
Fh = Maximum yearly head on gate.
Fi = Maximum yearly ice load on gate.
Fhi = Head on gate at time of maximum ice load.

For items in the table above where no standard deviation is given, the variable was assumed to be a constant. Where no values are listed, the gate was not evaluated for that condition.

IV. Hazard Functions

Summary of Reliability Results for Tainter Gates

24. Reliability indices (betas) were computed using the methods described in paragraph 21.D and are shown in the following table. These were produced for

comparison purposes and to find the critical gates and load cases. Where Betas of 3.0 or greater are computed, the probability of unsatisfactory performance will be small enough to be considered insignificant. The Betas indicate that reliabilities for the tainter gates will for the most part will be very high except for ice loadings on some gates.

Betas For Tainter Gates

Dam	Loading	Critical Member	Beta 1940	Beta 2000	Beta 2050
5	Water	Bottom Strut	4.38	4.13	3.99
	Ice	Top Strut	1.64	1.57	1.53
10	Water	Vertical Ribs	4.28	3.76	3.53
	Ice	Top Strut	3.22	3.18	3.16
13	Water	Horiz. Girder	6.26	6.04	5.98
	Ice	Top Strut	4.45	4.43	4.41
22	Water	Middle Girder	5.07	4.97	4.94
	Ice	Top Girder	3.65	3.43	3.33
24	Water	Shell Plate	4.02	3.19	3.01
25	Water	Horiz. Girder	4.28	4.18	4.16
Dresden Island	Water	End Strut	4.19	4.19	4.19
	Ice	Top Horiz. Beam	1.34	1.34	1.34
Starved Rock	Water	Bottom Horiz. Beam	6.85	6.85	6.85
	Ice	Top Horiz. Beam	2.28	2.28	2.28

Computation of Probability of Unsatisfactory Performance

25. Final computation of the unsatisfactory performance was done as described previously in paragraph 14.E for the tainter gates at Dams 4 - 9, Dresden Island, and Starved Rock. The probability of unsatisfactory performance will remain mostly constant with time for the tainter gates because the gates do not deteriorate very much when painted regularly. The hazard value was computed assuming that current, regular paint schedules used in the St. Paul and Rock Island Districts will be kept in the future. Enhanced maintenance would not improve on the reliability with time since it does not degrade significantly with the current maintenance schedule and therefore only one value for the hazard function was computed.

26. Although many tainter gates on Mississippi River dams show some very minor deformations caused by ice, the majority do not. Due to a lack of a design load case for ice and a bad connection detail, the gates at Dams 4 - 9 can be damaged by relatively small ice loads (about 2 kip/ft). The fact that many have not been damaged indicates that ice loads on the tainter gates do occur, but not frequently and usually not of great magnitude. The ice loads for the Mississippi River dams are caused by thermal ice loads. An analysis of ice loads done in the St. Paul district several years ago showed that when the ice expands thermally, most of the load is taken by the dam piers by arching across the tainter gate opening, except when the ice is very weak and/or thin relative to the loading that is produced. From the limited damage done to the tainter gates, it can be inferred that such conditions do not occur very often and that normally when ice loads occur, they are transferred to the dam piers without exerting much force on the tainter gates. Therefore, an exponential distribution for ice loading was assumed for the computation of the hazard function. The parameters for the loading distribution were selected so that it would approximately result in a load distribution indicated by the damaged gates.

27. The ice loads that would be produced on the Illinois Water Way dams at Dresden Island and Starved Rock are created by impact ice loads from floating ice sheets. Two tainter gates at Dresden Island Lock and Dam were severely damaged by floating ice at least two times in the past. An exponential loading distribution was used to compute the hazard on these gates as well since most years the gates experience very little ice loading. Parameters for the distribution were selected so that the design ice loading of 5 kips per foot is exceeded 5% of the time. The resulting reliability predicts approximately the same number of unsatisfactory performances that the gates have actually experienced.

28. Final computation of the probability of unsatisfactory performance per dam per year for the current gates are given on the table that follows. The reliability of the tainter gates is controlled by the strength of the top strut arm, which resists most of the ice loading. The tainter gates at Dams 5A through 9 have stronger strut arms than the gates at dams 4 and 5. Therefore, although the gates are identical otherwise, the gates at dams 5A through 9 have lower probabilities of unsatisfactory performance due to ice loading. Probability of unsatisfactory performance after a repair has been made to a damage gate can be assumed to be unchanged. The repair would not be likely to significantly strengthen the gate and the repair of one or two gates at a dam would not significantly change the probability of unsatisfactory performance for all of the gates at the site.

Hazard

Dam	Probability of Unsatisfactory Performance
4	0.0726
5	0.0924
5A	0.0025
6	0.0049
7	0.0054
8	0.0049
9	0.0039
Dresden Island	0.1530
Starved Rock	0.1600

V. Consequences

28. For the tainter gates at Dams 4, 5, 5A, 6, 7, 8, and 9, the loading which causes unsatisfactory performance is thermally expanding ice loading. This loading cannot cause impacts to navigation because it occurs in winter when no navigation is occurring and because the load is unable to follow the yielding structure in such a way as to cause the gate to be destroyed and the pool lost. The only consequence to this type of unsatisfactory performance is that bent members on the gate will need to be repaired or the gate replaced.

29. For the tainter gates at Starved Rock and Dresden Island, the loading is for impact loads from floating ice sheets. The consequences of this event occurring range from bending of the gate members requiring gate repair to damage of the gate sufficient to cause loss of pool. Pool can be restored by placing new bulk heads. If enough gates are destroyed, insufficient bulkheads will be available and bulkheads will need to be fabricated to restore the pool and a greater loss to navigation time will be incurred. The probability of unsatisfactory performance shown above for the Dresden Island gates is due to a vertical beam failing which will result in more gate damage and more severe consequences than the hazard number for the Starved Rock gate which is for a top horizontal beam.

30. The table which follows summarizes the consequences and costs for the tainter gates.

Table of Consequences

Lock	Low Level of Consequences (LC)			Medium Level of Consequences (MC)			High Level of Consequences (HC)		
	P(LC)	Nav. Down Time	Repair costs	P(MC)	Nav. Down Time	Repair costs	P(HC)	Nav. Down Time	Repair costs
4,5,5A 6,7,8,9	.9	0	$150,000	.09	2	$250,000	.01	7	$700,000
Dresden Island	.9	0	$300,000	.09	2	$500,000	.01	7	$1,000,000
Starved Rock	.9	0	$250,000	.09	2	$300,000	.01	7	$500,000

31. For the table above for Dams 4-9, Low Level of consequences assumes that the top strut arms for one gate require replacement. Medium level of consequences assumes that strut arms for three gates require replacement. The High level of consequences assumes that strut arms for two gates are replaced and also one gate is replaced.

32. For the gates at Dresden Island Dam, low level of consequences assumes that one gate is damaged and bulkheads are placed immediately. Medium level of consequences assumes that two gates are damaged and loss of pool is longer. High level of consequences assumes that 4 gates are damaged and bulkheads must be fabricated and therefore loss of pool is longer in duration.

33. For the gates at Starved Rock, the overall reliability for the gates is higher and a less critical member was used to compute probability of unsatisfactory performance. Low level of consequences would be if the gate was damaged but pool was not affected and navigation not lost. Medium level of consequences would be if a one gate was damaged enough to loose pool. High level of consequences would be if two gates were damaged.

VI. Rehabilitation of the Tainter Gates

34. For the tainter gates on the Mississippi River, top strut arms on gates in the St Paul District are the critical members and have shown deformations due to ice. For the rehabilitated case, it was assumed that the top strut arms would be replaced by new, welded plate members which would provide much more strength than the existing members.

35. For the Illinois Waterway gates, past damage has resulted in complete replacement of two gates. The new gates were designed to withstand ice loads and therefore by inspection have a high reliability. The probability of unsatisfactory performance for these gates after replacement was estimated to be the same as found for the Mississippi river gates, which had a Beta of about 3.5 after rehabilitation. The resulting probability is so small that inaccuracies in this assumption are insignificant.

36. For the rehabilitated gates, the probability of unsatisfactory performance would be as follows:

Dam	Probability of Unsatisfactory Performance
4,5,5A,6,7,8,9	0.0000007
Dresden Island	0.0000007
Starved Rock	0.0000007

37. Costs for Rehabilitation of the Tainter Gates is shown in the table below:

Dam	Rehabilitation Cost
4	$1,200,000
5	$1,500,000
5A	$ 350,000
6	$ 600,000
7	$ 650,000
8	$ 600,000
9	$ 500,000
Dresden Island	$2,100,000
Starved Rock	$2,340,000

SECTION 6 - Tainter Valves

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Reliability Analysis for Tainter Valves

I. Model Description

General

1. The reliability model for tainter valves has been developed using methods outlined in the report written by WES and JAYCOR titled "Reliability Analysis of Hydraulic Steel Structures with Fatigue and Corrosion Degradation", March 1, 1994. For the tainter valves, limit states for unsatisfactory performance due to both strength and fatigue were examined.

2. Tainter valves are steel structures composed of many members. For a given gate and loading, several members could be critical to the reliability of the valve. Unsatisfactory performance of the different critical members may have different consequences for the structure ranging from a simple repair of a bent member to complete loss of the tainter valve. Therefore, a complete structural analysis must be performed on the tainter valve so that the critical members, critical loads, and the limit state of the members can be identified and the member reliabilities computed. The overall reliability of the gate is determined from the reliability of the critical members.

3. The analysis of the tainter valve for forces in the structure members was done using conventional 2-dimensional modeling techniques. Since the valve is completely submerged when loaded, the net force on the gate can be represented as a uniform load on the valve face equal to the difference in head between the upper and lower pools.

4. On the following page is a table of general data for the tainter valves for the Mississippi River locks and dams

Definition of Unsatisfactory Performance

5. A tainter valve is comprised of many components, any one of which could be loaded beyond its limit state and cause unsatisfactory performance of the gate. The consequences of each type of member exceeding its limit state are described in the paragraphs below.

6. The skin plate serves to contain water behind the gate and transfer loads to the cross beams. It is designed as a 2-d plate, but may act as a 3-d plate or a diaphragm as well. Unsatisfactory performance would be created by fatigue cracking of the skin plate which would lead to leakage through the gate and possibly instability in the crossbeams. The valve would require repair but it could most likely be done at a convenient time to minimize affects on navigation.

Tainter Valve Data

Dam	Design Head, ft.	Valve Height	Valve Width	Valve Radius
2	12.2	12.5	12.5	17
3	8.0	12.5	12.5	17
4	7.0	12.5	12.5	17
5	9.0	12.5	12.5	17
5A	5.5	12.5	12.5	17
6	6.5	12.5	12.5	17
7	8.0	12.5	12.5	17
8	11.0	12.5	12.5	17
9	9.0	12.5	12.5	17
10	8.0	12.5	12.5	17
11	11.0	12.5	12.5	17
12	9.0	12.5	12.5	17
13	11.0	12.5	12.5	17
14	11.0	12.5	12.5	17
15	16.0	12.5	12.5	17
16	9.0	12.5	12.5	17
17	8.0	12.5	12.5	17
18	9.8	12.5	12.5	17
19	36.2	17.0	14.5	21
20	10.0	12.5	12.5	17
21	10.5	12.5	12.5	17
22	10.2	12.5	12.5	17
24	15.0	12.5	12.5	17
25	15.0	12.5	12.5	17
26	-	18.0	16.0	25.75
27	21.2	15.5	12.5	20

7. The cross beams carry load from the skin plate to the vertical end girders and the limit state for these member is bending or fatigue cracking. Although there are several beams parallel to each other and there would be some redistribution of load if one beam were to reach its limit state for strength or fatigue, the beams are far enough apart that there may be significant deflection of a portion of the gate. The valve would need to be repaired immediately.

8. The end girders take load from the cross beams to the strut arms. Limit states for the beams are bending, shear, or fatigue cracking due to bending. Unsatisfactory performance of a girder could likely result in complete failure of the gate and require immediate gate repair.

9. The strut arms take load from the girders to the trunnion and act as beam-columns. The limit state for these members is yielding or by buckling, depending on the combination of forces that act on them. Unsatisfactory performance of a strut arm could likely result in complete failure of the gate and require immediate gate repair.

Load Cases

10. Two load cases were selected for the reliability model. Each case was checked for both strength and fatigue. These cases are as follows:

A. Headwater and tailwater due to maximum head for the strength limit state or due to average head for the fatigue limit state. Valve resting on sill.

B. Headwater and tailwater due to maximum head for the strength limit state or due to average head for the fatigue limit state. Valve being lifted by chains with chain pull even on both sides of the tainter valve.

Random Variables

11. The random variables random variables in the following paragraphs were used in the reliability model. Most come from the WES - JAYCOR report.

12. Corrosion Rate. The random variable for corrosion is ϵ_c and the amount of corrosion, C, is defined by:

$$\log C = \log A + B \log t + \epsilon_c.$$

13. The variables for the corrosion equation were determined from thickness measurements conducted on a tainter valve at Lock and Dam 6. These measurements were compared to corrosion rates predicted by variables given in the WES-JAYCOR report. It was assumed that splash zone corrosion rates should be used for the tainter valves because the gate is almost completely lifted out of the water during every

lockage. The valves at Lock 6 showed corrosion rates about half those predicted by the splash zone corrosion equation given in the WES-JAYCOR report. Therefore, the following variables were used in the reliability analysis for the tainter valves to produce C in micrometers:

A = 74 ; B = 0.903; ϵ_c mean = 0 with std. dev. = 0.099

14. For the tainter valves, the paint was assumed to be effective for preventing corrosion for 15 years after each application of vinyl paints. The original lead paint was assumed to last until 1948.

15. <u>Steel Yield Strength.</u> The random variable is defined by LRFD research as follows:

Tension. Mean = 1.05 Fy, Std. Dev. = 0.11
Bending. Mean = 1.08 Fy, Std. Dev. = 0.14
Shear. Mean = 1.10 Fy, Std. Dev. = 0.15

The stated minimum yield strength, Fy, for all of the tainter valves analyzed is 33 ksi. The random variables for steel yield strength are therefore as follows:

Bending Strength. Mean: 35.64 ksi. Standard Deviation: 4.98 ksi.
Shear Strength. Mean: 36.30 ksi. Standard Deviation: 5.45 ksi.

16. <u>Loading.</u> The random variable used for loading is water load. The mean and standard deviation for water loads were computed as described in the WES-JAYCOR report. Since the reliability for the strength limit state is computed on an annual basis, mean and standard deviation for loading was for maximum loading on the gate during a given year.

17. <u>Ratio of actual to computed forces.</u> This random variable is Ks and is the ratio of actual to calculated stresses. The numbers for Ks that were used were those determined in the research for the AISC LRFD code and therefore their applicability to tainter valves is questionable.

Mean Ks = 1.02, Std. dev. = 0.10.

18. <u>Fatigue.</u> For fatigue, there are three variables which are used.

A) Ratio of lockages to actual stress cycles (Kc). This factor permits the number of stress cycles to be computed from the number of lockages. Kc is computed from the number of machinery hard cycles.

B) Uncertainty in the fatigue life of the material (ϵ). This variable is applied in the equation which is used to compute the fatigue strength of the material.

For riveted structures (which all of the tainter valves investigated were):

Mean $\epsilon = 0.0$, Std. dev. $= 0.31$.

C) Damage accumulation factor (Δ)

Mean $\Delta = 1.0$, Std. dev. $= 0.30$.

Procedure for Analyzing Reliability

19. The procedure used to compute reliability of tainter valves is described below:

A. Information on the geometry, member properties, weight, loadings, and number of lockages for the tainter valve was collected.

B. The loads on the tainter valve and the reactions on the valve at the trunnions, gate sill, and lifting chains were computed. The loads, reactions, and all further analysis were computed at the average loading and one standard deviation above and below the average.

C. Unit forces in the tainter valve members were computed. The tainter valves are always completely submerged so the net loading on it is a uniform load equal to the difference in head between the pool and tailwater. The member forces were computed for a unit load and a ratio of actual head to the unit head can be used to compute the actual member forces. The CORPS program X0030, CFRAME, was used to analyze the vertical frames on each side of the tainter valve. These frames are comprised of the end girder and the strut arms.

D. Reliability for limit states due to bending, shear, and axial loads in the gate members was computed. The analysis in step C produces average forces and forces one standard deviation above and below the average for use in computing the reliability using the Taylor Series method. The forces were input into appropriate spreadsheets for calculation of reliability. Several spreadsheets were prepared which compute reliability for the steel members that are present in the tainter valves.

II. Site Selection

20. Only tainter valves at a few locks on the Mississippi River were analyzed for reliability. This was done to save time because, except for a few sites, all of the tainter valves are of identical construction. This simplification could be done because the reliability for the tainter valves was in almost all cases very high due to conservative design criteria and design loadings that are outside of expected actual

loadings. The gates at Lock 26 and Lock 2 were not analyzed. These newer gates are assumed to have reliabilities similar to the other sites. The tainter valves for which reliability was computed and the reason that they were selected is as follows:

A. Lock 8. The valves at this site are identical to valves used at Locks 3 through 17 and 20 through 25. Lock 8 has the highest head in the St Paul District.

B. Lock 12. Reliability for the valves at this site was analyzed for an evaluation report and included in the Navigation Study.

C. Lock 15. This lock had the highest head of all dams with this common valve type.

D. Lock 19. The tainter valves at this site are of a different type than the other locks. Rather than being of riveted construction with the strut arms in compression, the tainter valves at dam 19 are of welded construction with the strut arms in tension. The head at this site is several times higher than head at other sites as well. Factors of safety were computed for the expected loadings by a simple and conservative analysis and were found to be quite high. By comparison with factors of safety from the other tainter valves, it appears that the reliability would be very high. Because of this and because of time considerations, the gates at this site were not formally analyzed for reliability.

E. Lock 24. The original valves at Locks 24 and 25 are the same and have had less maintenance and more lockage cycles than at other sites. The gates at Lock 25 were replaced in 1995 and the gates at Lock 24 were planned to be replaced although funding is uncertain at this time.

III. Parameters

21. For all sites analyzed, random variables for corrosion rate, the ratio of actual to computed stress (Ks), steel yield strength, and the fatigue parameters ϵ and δ are as stated in the section above titled "Random Variables". Other variables are as stated in the table below.

Random Variables for Sites Analyzed

Dam	Fhm (ft)		Fha (ft)		Kc	
	μ	σ	μ	σ	μ	σ
8	10.32	0.67	6.70	3.29	0.770	0.066
12	8.33	0.74	4.34	3.41	0.644	0.039
15	15.59	1.00	11.15	3.39	0.649	0.034
24	14.54	0.90	9.17	2.18	0.829	0.033

In the previous table:

Fhm = Maximum yearly head on valve.
Fhm = Average head on valve.
Kc = Ratio of number of hard cycles to number of lockages.

Summary of Reliability Results for Tainter Valves

22. The table which follows summarizes reliability indices (betas) for the tainter valve for which reliability was analyzed. The betas were used for comparison purposes and as an indication of which sites would be of concern for the economic analysis. Betas were computed for future dates assuming that maintenance would be done the same as has been done in the past. The beta listed for Dam 24 assumes that the existing gates are not replaced.

Betas

Dam	Limit State	Critical Member	Beta		
			1940	2000	2050
8	Strength	Cross Beam	9.22	8.10	7.51
	Fatigue	Skin Plate	11.84	6.55	4.88
12	Strength	Cross Beam	9.87	9.16	8.74
	Fatigue	Skin Plate	11.42	5.28	3.76
15	Strength	End Girder	7.13	6.49	6.10
	Fatigue	Cross Beam	13.14	6.06	4.80
24	Strength	Cross Beam	7.29	3.10	-
	Fatigue	Skin Plate	11.06	2.35	-

23. Almost all of the tainter valves have betas that indicate that reliability of the valves will be very high, provided that routine painting is done, for the life of the gate. Only the existing tainter valves at Lock 24 have betas that indicate that reliability will be significantly low in the future. The beta for strength of the cross beams becomes 1.0 in about 2025 and for fatigue in the skin plate it becomes 1.0 in about 2029. If these gates are replaced as planned, the reliability can be assumed to be very high until the year 2050.

IV. Hazard Functions

24. The probability of unsatisfactory performance by year for the existing tainter valves at Lock 24 are listed in the table which follows. These numbers were produced from a Weibull fit of computed data to form a hazard function. The Weibull equation is:

$$h(t) = \left(\frac{a}{b}\right) \cdot \left(\frac{t}{b}\right)^{a-1}$$

Where: h(t) is the probability of unsatisfactory performance
t is the year
a and b are variables defined below

25. The hazard function for the tainter valve given in the table which follows assume future maintenance the same as used to compute betas for the sites as listed above and also for a case which assumes enhanced maintenance in the future. For the tainter valves at Lock 24, painting in the future was assumed to take place in 1998, 2018, and 2038 for the enhanced maintenance condition. For a rehabilitation of the valves at Lock 24, the hazard function can be assumed the same as when the gates were new as shown in the third column.

Hazard Function Each Tainter Valve at Lock 24

Year	Current O & M Hazard Function	Enhanced Maintenance Hazard Function	Year	Rehabilitation Hazard Function
2000	0.00061	0.00007	0	0.000000
2001	0.00070	0.00007	1	0.000000
2002	0.00080	0.00008	2	0.000000
2003	0.00092	0.00009	3	0.000000
2004	0.00105	0.00010	4	0.000000
2005	0.00119	0.00012	5	0.000000
2006	0.00135	0.00013	6	0.000000
2007	0.00153	0.00014	7	0.000000
2008	0.00173	0.00016	8	0.000000
2009	0.00195	0.00017	9	0.000000
2010	0.00220	0.00019	10	0.000000
2011	0.00247	0.00021	11	0.000000
2012	0.00277	0.00024	12	0.000000
2013	0.00311	0.00026	13	0.000000
2014	0.00348	0.00028	14	0.000000
2015	0.00388	0.00031	15	0.000000
2016	0.00433	0.00034	16	0.000000

Year	Current O & M Hazard Function	Enhanced Maintenance Hazard Function	Year	Rehabilitation Hazard Function
2017	0.00483	0.00038	17	0.000000
2018	0.00537	0.00041	18	0.000000
2019	0.00597	0.00045	19	0.000000
2020	0.00662	0.00049	20	0.000000
2021	0.00734	0.00053	21	0.000000
2022	0.00812	0.00058	22	0.000000
2023	0.00898	0.00063	23	0.000000
2024	0.00991	0.00069	24	0.000000
2025	0.01093	0.00075	25	0.000000
2026	0.01204	0.00081	26	0.000000
2027	0.01325	0.00088	27	0.000000
2028	0.01456	0.00095	28	0.000000
2029	0.01599	0.00103	29	0.000000
2030	0.01754	0.00111	30	0.000000
2031	0.01922	0.00120	31	0.000000
2032	0.02103	0.00130	32	0.000000
2033	0.02300	0.00140	33	0.000000
2034	0.02513	0.00151	34	0.000000
2035	0.02743	0.00162	35	0.000000
2036	0.02991	0.00174	36	0.000000
2037	0.03258	0.00188	37	0.000000
2038	0.03547	0.00201	38	0.000000
2039	0.03857	0.00216	39	0.000001
2040	0.04192	0.00232	40	0.000001
2041	0.04551	0.00249	41	0.000002
2042	0.04937	0.00266	42	0.000003
2043	0.05352	0.00285	43	0.000004
2044	0.05797	0.00305	44	0.000006
2045	0.06275	0.00326	45	0.000009
2046	0.06786	0.00348	46	0.000014
2047	0.07334	0.00372	47	0.000020
2048	0.07921	0.00396	48	0.000030
2049	0.08548	0.00423	49	0.000043
2050	0.09219	0.00450	50	0.000061
Weibull Equation Variables				
a	9.27	7.97		19.14
b	108.9	-40.2		8.41

26. The hazard listed for the existing tainter valves for Lock 24 in the table above are for one tainter valve. The probability that any one of the four tainter valves at the site will have unsatisfactory performance can be found by multiplying the above hazard by four. The coefficients for the Weibull equation become as shown below:

Hazard Function for Tainter Valves at Lock 24

Year	Current O & M Hazard Function	Enhanced Maintenance Hazard Function	Rehabilitation Hazard Function
Weibull Equation Variables			
a	37.1	31.9	76.6
b	108.9	-40.2	8.41

Hazard Function After Repair

27. After a tainter valve is repaired following an unsatisfactory performance, the hazard function that the valve would have depends on the type of failure and the repair. Since the critical limit state for the tainter valve was fatigue cracking of the skin plate, it is assumed that after unsatisfactory performance, the gate would need to be replaced. The hazard function would be the same as for the rehabilitated case.

V. Consequences

28. It is almost impossible for the loss of a tainter valve to result in a loss of the pool. If a valve failed, the other valve in the culvert would still stop flow through the culvert. The impacts to navigation would be a possible slowing of lock operation while the valve was repaired and only one of the two culverts was operational. Navigation would be stopped for a few hours while the gate was removed and replaced.

29. The table which follows summarizes the consequences and costs for the tainter valves. The difference in consequences relate to the degree of failure and the amount of difficulty in removing the damaged tainter valve from the tainter valve pit. It is expected that normally it will be relatively simple to remove the tainter valve. The slow down time is related to how long the lock would be operated with just one on set of valves working. Lower probability of consequences assume that the gate cannot be operated while a new gate is fabricated.

Table of Consequences

Probability of Consequences P(LC)	Navigation Down Time	Navigation Slow Time	Repair Costs
0.90	4 hrs.	6 days	$175,000
0.09	1 day	20 days	$200,000
0.01	2 days	45 days	$250,000

VI. Rehabilitation and Enhanced Maintenance Costs

30. For rehabilitation, the gates at Lock 24 would require replacement identical to the replacement at lock 25. Costs for this for all four gates are $400,000 including mobilization and installation. For enhanced maintenance of the tainter valves, they would be painted in 2003, 2023, and 2043. The costs for each painting would be $175,000 and the yearly cost would therefore be $8,750.

SYSTEM SIGNIFICANT COMPONENTS

ENGINEERING RELIABILITY MODELS REPORT

(A Stand Alone Report Compiling Backup Information)

RELIABILITY MODELS
FOR
GEOTECHNICAL STRUCTURES & MATERIALS

Engineering Divisions

St. Paul, Rock Island and St. Louis Districts

US Army Corps of Engineers

UPPER MISSISSIPPI RIVER - ILLINOIS WATERWAY
NAVIGATION STUDY

GEOTECHNICAL/MATERIALS RELIABILITY MODELS

OBJECTIVE 2A

US Army Corps of Engineers
St. Louis District
Rock Island District
St. Paul District

March 1997

UPPER MISSISSIPPI RIVER-ILLINOIS WATERWAY
NAVIGATION STUDY

GEOTECHNICAL/MATERIALS RELIABILITY MODEL
OBJECTIVE 2A

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APPENDICES

UMR-IWW NAVIGATION STUDY
GEOTECHNICAL/MATERIALS RELIABILITY MODELS
OBJECTIVE 2A

1. Components. As part of Objective 2A of the Upper Mississippi River - Illinois Waterway Navigation Study (UMR-IWW) time-dependent reliability models were developed for all major components of the navigation system. Development of the time-dependent reliability models is documented in the report "Geotechnical Time Reliability Model." This report presents the results of these time dependent reliability models for all geotechnical/materials related components. The components of the UMR-IWW Navigation System, which are the responsibility of the geotechnical/materials work group, are listed in Table 1.

TABLE 1
GEOTECHNICAL/MATERIALS COMPONENTS

a. Through Seepage (Earth Embankments)

b. Slope Stability (Earth Embankments)

c. Gravity Structures (Sliding and Overturning)

(1) Guidewalls

(2) Lockwalls

(3) Dam Piers

d. Pile Foundations (Pile Capacity, Pile Stresses, and Pile Deformation)

(1) Guidewalls

(2) Lockwalls

(3) Dam Piers

e. Underseepage

(1) Locks

(2) Dam Piers

(3) Earth Embankments

f. Scour Protection Downstream of the Dam (Riprap)

g. Lockwall Concrete (Nonair-Entrained, Freeze-Thaw Damage)

h. Dam Pier Bridge Column Concrete (Nonair-Entrained, Freeze-Thaw Damage)

i. Concrete Spillway Fixed Crest (Nonair-Entrained, Freeze-Thaw Damage)

Table 2 shows the number and type of dam piers at each project.

TABLE 2
TYPE OF PIERS AT EACH DAM

NUMBER OF DAM PIERS			
Project	Tainter Gate Piers	Roller Gate Piers	Total
USAF	0	0	0
LSAF	4	0	4
1	0	0	0
2	21	0	21
3	0	5	5
4	22	6	28
5	28	6	34
5A	5	5	10
6	10	5	15
7	11	5	16
8	10	5	15
9	8	5	13
10	7	5	12
11	12	4	16
12	6	4	10
13	9	4	13
14	12	5	17
15	0	11	11
16	14	5	19
17	7	4	11

NUMBER OF DAM PIERS			
Project	Tainter Gate Piers	Roller Gate Piers	Total
18	13	4	17
19	0	0	0
20	39	4	43
21	9	4	13
22	9	4	13
24	15	0	15
25	13	4	17
Melvin Price	11	0	11
27	0	0	0
TJ O'Brien	0	0	0
Lockport	0	0	0
Brandon Road	21	0	21
Dresden Island	9	0	9
Marseilles	11	0	11
Starved Rock	10	0	10
Peoria	1	0	1
LaGrange	1	0	1

2. <u>Methods</u>. A three-parameter Weibull distribution was used to represent unsatisfactory performance events on the UMR-IWW Navigation System. The three-parameter Weibull distribution is defined by the three parameters b (the shape parameter), α (the characteristic life), and ν (the minimum life). Past unsatisfactory performance events were tabulated in a database for the components in Table 1. This database is representative of the navigation system as a whole, not any single component. The geotechnical and materials components listed in Table 1 are represented by nine different modes of performance for the navigation system as given in Table 3.

TABLE 3
PERFORMANCE MODES

a. Through Seepage

b. Slope Stability

c. Gravity Structures

d. Pile Foundations

e. Underseepage

f. Scour Protection Downstream of the Dam

g. Lockwall Concrete (Nonair-Entrained)

h. Dam Pier Bridge Column Concrete (Nonair-Entrained)

i. Concrete Spillway Fixed Crest (Nonair-Entrained)

3. <u>Equations</u>. The probability density function f(t) for the three-parameter Weibull distribution is:

$$f(t) = \frac{b}{\alpha}\left[\frac{t-\nu}{\alpha}\right]^{b-1}\exp\left[-\left[\frac{t-\nu}{\alpha}\right]^{b}\right] \tag{1}$$

where

b is the shape parameter.

α is the characteristic life, starting at time equal to the minimum life.

ν is the minimum life.

t is time. For all of the geotechnical/materials reliability models presented in this report, t is taken as zero in 1995, the year that all of the reliability analyses were performed.

F(t) is the cumulative distribution function, the probability that the system will fail by the time t or the probability of failure. F(t) is given as follows:

$$F(t) = 1 - \exp\left[-\left[\frac{t-\nu}{\alpha}\right]^{b}\right] \tag{2}$$

R(t) is the reliability function, the probability that the system will not fail by time t or the reliability of the system.

$$R(t) = \exp\left[-\left[\frac{t-\nu}{\alpha}\right]^{b}\right] \tag{3}$$

As can be seen from an examination of Equations 2 and 3, the reliability and probability of failure are related by the following equation:

$$R(t) = 1-F(t) \tag{4}$$

h(t) is the hazard function, the rate of failure at time t given that failure has not occurred at time t or **the probability of failure in any year,** given that failure has not occurred.

$$h(t) = \frac{b}{\alpha}\left[\frac{t-\nu}{\alpha}\right]^{b-1} \tag{5}$$

The Weibull distribution has the following characteristics: For b = 1, the Weibull distribution becomes the exponential distribution, which gives a constant hazard function with an equal rate of failure in any year. For b = 2, the Weibull distribution becomes the Rayleigh distribution, which gives a linearly increasing hazard function. For b < 1, the hazard function decreases with time, giving a decreasing rate of failure with time. For b > 1, the hazard function increases with time, giving an increasing rate of failure with time. A b value of 1 would be representative of the occurrence of a random event, such as scour occurring adjacent to a structure, erosion, or an accident. Deterioration of sheetpiling could be represented by a b value between 1 and 2. For any Weibull distribution, there is a 63.2 percent probability that failure will occur before the characteristic life and a 37.8 percent probability that failure will occur after the characteristic life. Put another way, 63.2 percent of the components will fail by the characteristic life and 37.8 percent will not fail.

4. Results. Each of the nine performance modes will be presented in an appendix of this report. Each appendix will contain the following information:

a. Model Description. All of the reliability models used are described in detail in one of the following reports prepared as a part of the UMR-IWW Navigation Study:

(1) "Probability Models For Geotechnical Aspects of Navigation Structures" by Shannon & Wilson, Inc.

(2) "Reliability Assessments of Pile Founded Navigation Structures" by the St. Paul District.

(3) "Geotechnical Time Reliability Model Report" by the Geotechnical/Materials Work Group.

(4) "Reliability Model of Concrete Deterioration of Lock Walls Due to Freeze-Thaw and Abrasion" by US Army Corps of Engineers Waterways Experiment Station, Draft.

b. Site Selection. An explanation is given if a reliability model was not implemented for each component at a lock and dam site. The component and site that was analyzed is given and the other components and sites that are similar and can be represented by that component and site are also given.

c. Important Deterministic and Random Variables. A listing of each random and deterministic variable used in the reliability model for each component of each lock and dam analyzed is given. The random variable is represented by a expected value or mean (μ) and a standard deviation (σ).

d. Weibull Distribution Parameters. The parameters needed for the three-parameter Weibull distribution are given for the current condition of each component and the condition of the component after it is rehabilitated. For geotechnical-materials components there is no enhanced maintenance distribution as there is for structural components. There exists no systematic maintenance system like painting to extend the usable life of the component. Using the three parameters given for the current condition or the rehabilitated condition of a component with the equations given in paragraph 3; the reliability (R), the cumulative distribution function (F), and the hazard function (h) of a component can be calculated at present and at any time (t) in the future. The cumulative distribution gives the probability of failure and the hazard function gives the probability of failure in a year, given that failure has not occurred. For all of the above functions, t is taken as zero in 1995.

e. Consequences. The consequences to the navigation system are given here for each component. These consequences consist of downtime to the system, operational slowdown of the system, and cost of repair if a component of the navigation system should experience unsatisfactory performance. The consequences are given in terms of a medium level (MC) and a high level consequence (HC) along with the probability of occurrence of a medium (P(MC)) and high level consequence (P(HC)). Low level consequences are not given because they were excluded from the database used to develop the Weibull distributions.

f. Cost of Rehabilitation. The cost of rehabilitating a component prior to an unsatisfactory performance event is given for each component.

g. Number of Components. For each performance mode, the number of components in the navigation system that are represented by that performance mode are given by District and the total number of components in the navigation system are given.

Appendix A - Through Seepage

T
H
R
O
U
G
H

S
E
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UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

THROUGH SEEPAGE

1. Model Description. The through seepage reliability model was based on the through seepage analysis for sand levees and dikes used in the Rock Island District. The method involves computation of two parameters, the maximum erosion susceptibility, M_r, and the relative erosion susceptibility, R, which are compared to critical combinations of values for which provision of toe berms is recommended. These parameters are functions of the embankment geometry and soil properties. The following deterministic parameters are required as input: the pool elevation, the tailwater elevation, and the height of the embankment. Five variables were treated as random. They include: the friction angle, the saturated density, Manning's coefficient, the traction stress, and the permeability.

2. Site Selection. Because of the large number of locks and dams in the Rock Island District, only selected sites were analyzed. Selection was made so that two sites with similar structural and foundation conditions were not both analyzed. There are cutoff walls in all overflow dikes, in the storage yard at Lock and Dam No. 18, and in all dikes at Locks and Dams Nos. 14, 16, 22, Brandon Road, Dresden Island, and in the non-overflow dike at LaGrange. There are no earth dikes at Locks and Dams Nos. 15, 19, TJ O'Brien, Marseilles, Starved Rock, and Peoria.

THROUGH SEEPAGE			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
11	Non-Overflow Dike	LSAF	NSP Dike
		2	Earth Dike
		3	Earth Dike
			Spot Dikes
		4	Earth Dike
		5	Earth Dike

THROUGH SEEPAGE			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
11	Non-Overflow Dike	5A	Earth Dike
		6	Earth Dike
		7	Earth Dike
		8	Earth Dike
		9	Earth Dike
		10	Earth Dike
		11	Storage Yard
		16	Storage Yard
		20	Storage Yard
		24	Storage Yard
		25	Storage Yard
		Melvin Price	Esplanade
		27	East Earth Embankment
			West Earth Embankment
12	Non-Overflow Dike	12	Storage Yard
		24	Auxiliary Lock Closure Dam
			Sny Levee
		25	Auxiliary Lock Closure Dam
			Sandy Slough Dike

THROUGH SEEPAGE			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
13	Non-Overflow Dike	13	Storage Yard
		17	Non-Overflow Dike
			Storage Yard
		21	Storage Yard
18	Non-Overflow Dike		

3. <u>Important Deterministic and Random Variables</u>.

THROUGH SEEPAGE														
Lock and Dam	Component	Random Variables										Deterministic Variables		
		φ (degrees)		γ (pcf)		m		τ (psf)		k (cm/sec)		Pool	TW	Height
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	(feet)	(feet)	(feet)
11	Non-Overflow Dike	32	2.0	120	3.6	0.02	0.002	0.03	0.003	0.055	0.0275	603	592	23
12	Non-Overflow Dike	32	2.0	120	3.6	0.02	0.002	0.03	0.003	0.10	0.05	592	583	18
13	Non-Overflow Dike	32	2.0	120	3.6	0.02	0.002	0.03	0.003	0.12	0.06	583	572	20
18	Non-Overflow Dike	32	2.0	120	3.6	0.02	0.002	0.03	0.003	0.10	0.05	528	518.2	25

THROUGH3.XLS

4. <u>Weibull Distribution Parameters</u>.

THROUGH SEEPAGE							
Cumulative Distribution Function		Current			Rehabilitated		
Lock and Dam	Component	b	α	ν	b	α	ν
11	Non-Overflow Dike	1.0	4000	0	1.0	4000	0
12	Non-Overflow Dike	1.0	4000	0	1.0	4000	0
13	Non-Overflow Dike	1.0	4000	0	1.0	4000	0
18	Non-Overflow Dike	1.0	4000	0	1.0	4000	0

THROUGH4.XLS

5. <u>Consequenses</u>.

THROUGH SEEPAGE										
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)					
		P(MC)	Nav. Down time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Slowdown		Repair Costs (million)
							Effect on Lockage Cycle	Slowdown Duration (days)	
LSAF	NSP Dike	0.9	1	$0.0	0.1	20	0	0	$0.0
2	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$4.2
3	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$5.2
	Spot Dikes	0.9	1	$0.5	0.1	20	0	0	$14.0[1]
4	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$6.5
5	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$16.4
5A	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$18.0
6	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$5.0
7	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$7.1
8	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$14.0
9	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$8.2
10	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$5.4

THROUGH5.WPD

[1]Rounded up from rehabilitation cost.

5. Consequences.

THROUGH SEEPAGE										
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)					
		P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Slowdown[2]		Repair Costs (million)
							Effect on Lockage Cycle	Slowdown Duration (days)	
11	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$4.740
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
12	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$6.640
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
13	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.632
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
16	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.332
17	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.476
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
18	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.760
20	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.180
21	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160

THROUGH5.XLS

[2]Slowdown time is added to navigation downtime.

6. Cost of Rehabilitation.

THROUGH SEEPAGE		
Lock and Dam	Component	Rehabilitation Cost (millions)
LSAF	NSP Dike	$0.000
2	Earth Dike	$2.200
3	Earth Dike	$3.200
	Spot Dikes	$13.130
4	Earth Dike	$4.500
5	Earth Dike	$14.400
5A	Earth Dike	$16.000
6	Earth Dike	$3.000
7	Earth Dike	$5.100
8	Earth Dike	$12.000
9	Earth Dike	$6.200
10	Earth Dike	$3.400
11	Non-Overflow Dike	$2.740
	Storage Yard	$0.160
12	Non-Overflow Dike	$4.640
	Storage Yard	$0.160
13	Non-Overflow Dike	$1.632
	Storage Yard	$0.160
16	Storage Yard	$0.332
17	Non-Overflow Dike	$1.476
	Storage Yard	$0.160
18	Non-Overflow Dike	$1.760
20	Storage Yard	$0.180
21	Storage Yard	$0.160

THROUGH6.XLS

7. Number of Components.

THROUGH SEEPAGE	
District	Earth Embankments
St. Paul	12
Rock Island	32
St. Louis	9
TOTAL	53

APPENDIX B - Slope Stability

S L O P E

S T A B I L I T Y

UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

SLOPE STABILITY

1. Model Description. UTEXAS2 was used to perform slope stability calculations. The reliability analysis was performed in the same manner as a deterministic analysis. The required inputs for this program are the embankment profile, material properties, location of the phreatic surface, and surface pressures. The Corps of Engineers' modified Swedish procedure was chosen to calculate the factor of safety. The reliability index, β, was determined by the Taylor's Series Method, with the soils strength parameters, phi angle, cohesion, unit weight, and depth of foundation parameters chosen as the random variables.

2. Site Selection. Because of the large number of locks and dams in the Rock Island District, only selected sites were analyzed. Selection was made so that two sites with similar structural and foundation conditions were not both analyzed. There are no earth dikes at Locks and Dams Nos. 15, 19, TJ O'Brien, Marseilles, Starved Rock, and Peoria.

SLOPE STABILITY			
Sites Investigated		Sites in the Same Bracket	
Lock	Component	Lock	Component
4-9	Non-Overflow Dike	3	Protection Dike
			Non-Overflow Dike
		5A	Protection Dike
11	Non-Overflow Dike	10	Non-Overflow Dike
		11	Storage Yard
		16	Storage Yard
			Overflow Dike
		20	Storage Yard

SLOPE STABILITY			
Sites Investigated		Sites in the Same Bracket	
Lock	Component	Lock	Component
11	Non-Overflow Dike	24	Storage Yard
			Sny Levee
			Overflow Dike
		25	Storage Yard
			Sandy Slough Dike
			Overflow Dike
		Melvin Price	Esplande
			Overflow Dike
12	Non-Overflow Dike	12	Overflow Dike
	Storage Yard	2-10	Storage Yard
17	Storage Yard	13	Storage Yard
			Overflow Dike
			Non-Overflow Dike
		17	Non-Overflow Dike
			Overflow Dike
18	Non-Overflow Dike		
	Overflow Dike	7	Overflow Dike
		8	Overflow Dike

SLOPE STABILITY			
Sites Investigated		Sites in the Same Bracket	
Lock	Component	Lock	Component
18	Overflow Dike	9	Overflow Dike
	Storage Yard		
21	Overflow Dike		
	Storage Yard		
22	Storage Yard	22	Overflow Dike
Lock-port	Right Side Dike		
Brandon Road	Non-Overflow Dike	14	Non-Overflow Dike
			Storage Yard
Dresden Island	Non-Overflow Dike		
La Grange	Non-Overflow Dike		

3. Important Deterministic and Random Variables.

SLOPE STABILITY																		
Variables		Random														Deterministic		
		Embankment						Foundation										
		Sand/Clay						Sand/Clay/Rock										
Lock & Dam	Component	ϕ (degrees)		c (psf)		γ (pcf)		ϕ (degrees)		c (psf)		γ (pcf)		d (feet)		Pool (ft)	Tail Water (ft)	Height (ft)
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ			
USAF	Stone Guard Wall	Assumed β greater than 4 by engineering judgement.																
LSAF	NSP Dike	Assumed β greater than 4 by engineering judgement.																
1	Crib Wall	Assumed Pr(u)=50% in 50 years, based on Design Analysis Report and present proposal for rehabilitation.																
2	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	8	0	20
3	Spot Dikes	Assumed Pr(u)=10% in 50 years, based on Design Analysis Report and present proposal for rehabilitation.																
4	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	11	0	20
5	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	11	0	20
5A	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	11	0	20
6	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	11	0	20
7	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	11	0	20
8	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	11	0	20
9	Non-Overflow Dike	30	3	0	0	120	0	30	3	0	0	120	0	7	2	11	0	20

SLOPE STABILITY																		
Variables		Random														Deterministic		
		Embankment						Foundation										
		Sand/Clay						Sand/Clay/Rock										
Lock & Dam	Component	φ (degrees)		c (psf)		γ (pcf)		φ (degrees)		c (psf)		γ (pcf)		d (feet)		Pool (ft)	Tail Water (ft)	Height (ft)
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ			
11	Non-Overflow Dike	32	3.2	--	--	120	3.6	34	3.4	--	--	125	3.75	--	--	603	592	23
12	Non-Overflow Dike	32	3.2	--	--	120	3.6	--	--	600	240	115	3.45	13	5.25	592	583	18
	Storage Yard	32	3.2	--	--	120	3.6	32	3.2	--	--	125	3.75	--	--	592	583	17
17	Storage Yard	32	3.2	0	0	120	3.6	34	3.4	0	0	125	3.75	37	12.95	536	528	27
18	Non-Overflow Dike	32	3.2	--	--	120	3.6	34	3.4	--	--	125	3.75	--	--	528	518	25
	Overflow Dike	32	3.2	--	--	120	3.6	34	3.4	--	--	125	3.75	--	--	540	540	13
	Storage Yard	32	3.2	--	--	120	3.6	34	3.4	--	--	125	3.75	--	--	528	515	30
21	Overflow Dike	32	3.2	0	0	120	3.6	--	--	--	--	--	--	--	--	474	474	13
	Storage Yard	32	3.2	--	--	120	3.6	34	3.4	--	--	125	3.75	--	--	470	459.5	25
22	Storage Yard	32	3.2	0	0	120	3.6	34	3.4	--	--	125	3.75	23	8.05	459.5	449	29
24	Auxiliary Lock Closure Dam	38	3.8	--	--	--	--	--	--	--	--	--	--	--	--	449	434	--
25	Auxiliary Lock Closure Dam	38	3.8	--	--	--	--	--	--	--	--	--	--	--	--	434	419	--
27	Earth Embankment	35	3.5	--	--	--	--	--	--	--	--	--	--	--	--	406	388.7	--
	Low Water Dam	40	4.0	--	--	--	--	--	--	--	--	--	--	--	--	400	400	--
Lock-port	Right Side Dike	--	--	800	320	120	3.6	40	4.0	--	--	120	3.6	--	--	--	--	--
Brandon Road	Non-Overflow Dike	0	0	1200	480	120	3.6	0	0	600	240	115	3.45	10	3.5	539	505	30
		--	--	--	--	--	--	60	6.0	0	0	165	4.95	--	--	--	--	--
Dresden Island	Non-Overflow Dike	0	0	1000	400	115	3.45	10	1.0	600	240	115	3.45	5	1.75	505	483	19.5
		--	--	--	--	--	--	34	3.4	0	0	125	3.75	--	--	--	--	--
La Grange	Non-Overflow Dike	Assumed β greater than 5 by engineering judgement.																

SLOPE3.XLS

Note: The two sets of foundation properties for Brandon Road and Dresden Island Locks & Dams indicate a slope failure plane through a layered foundation.

4. Weibull Distribution Parameters.

SLOPE STABILITY							
Lock and Dam	Component	Current			Rehabilitated		
		b	α	ν	b	α	ν
USAF	Stone Guard Wall	0.45	35,000	0	0.45	35,000	0
LSAF	NSP Dike	0.45	35,000	0	0.45	35,000	0
1	Crib Wall	0.45	35,000	-15450	0.45	35,000	0
2	Earth Dike	0.45	35,000	-0.03	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
3	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
	Spot Dikes	0.45	35,000	-186	0.45	35,000	0
	Protection Dike	0.45	35,000	0	0.45	35,000	0
4	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
5	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
5A	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
	Protection Dike	0.45	35,000	0	0.45	35,000	0
6	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
7	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
	Submersible Dam	0.45	35,000	0	0.45	35,000	0
8	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
	Submersible Dam	0.45	35,000	0	0.45	35,000	0
9	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
	Submersible Dam	0.45	35,000	0	0.45	35,000	0
10	Earth Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0

SLOPE4.WPD

SLOPE STABILITY							
Lock and Dam	Component	Current			Rehabilitated		
		b	α	ν	b	α	ν
11	Non-Overflow Dike	0.45	35,000	0	0.45	35,000	0
12	Non-Overflow Dike	0.45	35,000	-14.28	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
17	Storage Yard	0.45	35,000	0	0.45	35,000	0
18	Non-Overflow Dike	0.45	35,000	0	0.45	35,000	0
	Overflow Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
21	Overflow Dike	0.45	35,000	0	0.45	35,000	0
	Storage Yard	0.45	35,000	0	0.45	35,000	0
22	Storage Yard	0.45	35,000	0	0.45	35,000	0
24	Auxiliary Lock Closure Dam	0.45	35,000	-503.94	0.45	35,000	0
25	Auxiliary Lock Closure Dam	0.45	35,000	-503.94	0.45	35,000	0
27	Earth Embankment	0.45	35,000	0	0.45	35,000	0
	Low Water Dam	0.45	35,000	-6.13	0.45	35,000	0
Lockport	Right Side Dike	0.45	35,000	-0.17	0.45	35,000	0
Brandon Road	Non-Overflow Dike	0.45	35,000	-110.51	0.45	35,000	0
Dresden Island	Non-Overflow Dike	0.45	35,000	0	0.45	35,000	0
LaGrange	Non-Overflow Dike	0.45	35,000	0	0.45	35,000	0

SLOPE4.XLS

5. <u>Consequences</u>.

SLOPE STABILITY									
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)				
		P(MC)	Down Time (days)	Repair Costs (million)	P(HC)	Down Time (days)	Slowdown[2] Effect on Lockage Cycle	Slowdown[2] Slowdown Duration (days)	Repair Costs (million)
USAF	Stone Guard Wall	0.9	0[1]	0[1]	0.1	1[1]	0	0	$0.48[1]
LSAF	NSP Dike	0.9	1	$0.5	0.1	20	0	0	$0.00
1	Crib Wall	0.9	0[1]	$0.5	0.1	1[1]	0	0	$6.00[3]
2	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$4.20
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.04
3	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$5.20
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.24
	Spot Dikes	0.9	1	$0.5	0.1	20	0	0	$14.0[3]
	Protection Dike	0.9	0[1]	$0.5	0.1	1[1]	0	0	$1.12[1]
4	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$6.50
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.16
5	Earth Dike	0.9	1	$0.5	0.1	20	0	0	$16.40
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.08

[1]Structure does not retain pool.
[2]Slowdown time is added to navigation downtime.
[3]Rounded up from rehabilitation cost.

SLOPE STABILITY										
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)				
		P(MC)	Down Time (days)	Repair Costs (million)	P(HC)	Down Time (days)	Slowdown[2]		Repair Costs (million)
							Effect on Lockage Cycle	Slowdown Duration (days)	
5A	Earth Dike	0.9	1	0.5	0.1	20	0	0	$18.00
	Storage Yard	0.9	1	0.5	0.1	20	0	0	$2.12
	Protection Dike	0.9	0[1]	0.5	0.1	1[1]	0	0	$0.64[1]
6	Earth Dike	0.9	1	0.5	0.1	20	0	0	$5.00
	Storage Yard	0.9	1	0.5	0.1	20	0	0	$2.12
7	Earth Dike	0.9	1	0.5	0.1	20	0	0	$7.10
	Storage Yard	0.9	1	0.5	0.1	20	0	0	$2.12
	Submersible Dam	0.9	1	0.5	0.1	20	0	0	$5.00
8	Earth Dike	0.9	1	0.5	0.1	20	0	0	$14.00
	Storage Yard	0.9	1	0.5	0.1	20	0	0	$2.12
	Submersible Dam	0.9	1	0.5	0.1	20	0	0	$5.00
9	Earth Dike	0.9	1	0.5	0.1	20	0	0	$8.20
	Storage Yard	0.9	1	0.5	0.1	20	0	0	$2.12
	Submersible Dam	0.9	1	0.5	0.1	20	0	0	$5.00
10	Earth Dike	0.9	1	0.5	0.1	20	0	0	$5.40
	Storage Yard	0.9	1	0.5	0.1	20	0	0	$2.12

SLOPE5.WPD

[1]Structure does not retain pool.
[2]Slowdown time is added to navigation downtime.

SLOPE STABILITY									
		Medium Level of Consequences (MC)			High Level of Consequences (HC)				
							Slowdown[2]		
Lock and Dam	Component	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Effect on Lockage Cycle	Slowdown Duration (days)	Repair Costs (million)
11	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$4.720
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
12	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$6.640
	Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.152
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
13	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.632
	Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.464
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
14	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$2.928
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
16	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$2.584
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.332
17	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$2.184
	Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.476
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
18	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.760
	Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.768
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.156
20	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.180

[2]Slowdown time is added to navigation downtime.

SLOPE STABILITY									
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)				
		P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Slowdown[2]		Repair Costs (million)
							Effect on Lockage Cycle	Slowdown Duration (days)	
21	Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.344
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
22	Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$3.488
	Storage Yard	0.9	1	$0.5	0.1	20	0	0	$2.160
24	Auxiliary Lock Closure Dam	0.9	1	$0.5	0.1	20	0	0	$10.000
25	Auxiliary Lock Closure Dam	0.9	1	$0.5	0.1	20	0	0	$10.000
27	Earth Embankment	0.9	1	$0.5	0.1	20	0	0	$2.600
	Low Water Dam	0.9	1	$0.5	0.1	20	0	0	$10.000
Lockport	Right Side Dike	0.9	1	$0.5	0.1	20	0	0	$4.400
Brandon Road	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$2.656
Dresden Island	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$2.400
LaGrange	Non-Overflow Dike	0.9	1	$0.5	0.1	20	0	0	$2.312

SLOPES.XLS

[2]Slowdown time is added to navigation downtime.

6. Cost of Rehabilitation.

SLOPE STABILITY		
Lock and Dam	Component	Rehabilitation Cost (million)
USAF	Stone Guard Wall	$0.480
LSAF	NSP Dike	$0.000
1	Crib Wall	$4.530
2	Earth Dike	$2.200
	Storage Yard	$0.040
3	Earth Dike	$3.200
	Storage Yard	$0.240
	Spot Dike	$13.130
	Protection Dike	$1.120
4	Earth Dike	$4.500
	Storage Yard	$0.160
5	Earth Dike	$14.400
	Storage Yard	$0.080
5A	Earth Dike	$16.000
	Storage Yard	$0.120
	Protection Dike	$0.640
6	Earth Dike	$3.000
	Storage Yard	$0.120
7	Earth Dike	$5.100
	Storage Yard	$0.120
	Submersible Dam	$3.000
8	Earth Dike	$12.000
	Storage Yard	$0.120
	Submersible Dam	$3.000
9	Earth Dike	$6.200
	Storage Yard	$0.120
	Submersible Dam	$3.000
10	Earth Dike	$3.400
	Storage Yard	$0.120
11	Non-Overflow Dike	$2.720
	Storage Yard	$0.160
12	Non-Overflow Dike	$4.640
	Overflow Dike	$1.152
	Storage Yard	$0.160

SLOPE STABILITY		
Lock and Dam	Component	Rehabilitation Cost (million)
13	Non-Overflow Dike	$1.632
	Overflow Dike	$1.464
	Storage Yard	$0.160
14	Non-Overflow Dike	$0.928
	Storage Yard	$0.160
16	Non-Overflow Dike	$0.584
	Storage Yard	$0.332
17	Non-Overflow Dike	$0.184
	Overflow Dike	$1.476
	Storage Yard	$0.160
18	Non-Overflow Dike	$1.760
	Overflow Dike	$1.768
	Storage Yard	$0.156
20	Storage Yard	$0.180
21	Overflow Dike	$1.344
	Storage Yard	$0.160
22	Overflow Dike	$1.488
	Storage Yard	$0.160
24	Auxiliary Lock Closure Dam	$5.000
25	Auxiliary Lock Closure Dam	$5.000
27	Earth Embankment	$0.600
	Low Water Dam	$5.000
Lockport	Right Side Dike	$2.400
Brandon Road	Non-Overflow Dike	$0.656
Dresden Island	Non-Overflow Dike	$0.400
LaGrange	Non-Overflow Dike	$0.312

SLOPE6.XLS

7. <u>Number of Components</u>.

SLOPE STABILITY			
District	Earth Embankments	Crib Walls	Total
St. Paul	28	1	29 ~~28~~
Rock Island	32	0	32
St. Louis	13	0	13
TOTAL	73	1	74

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APPENDIX C - Gravity Structures

UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

GRAVITY STRUCTURES

1. Model Description.

a. Background. Gravity structures are present at rock founded lock and dam sites on the Upper Mississippi River. Gravity structures on rock can fail from sliding and overturning. The stability of lockwalls was analyzed for both the normal operating condition and the maintenance condition (unwatered lock chamber).

b. Overturning. The overturning model follows the guidance set in ETL 1110-2-321, "Guidance for Applying Reliability Analysis to the Stability of Gravity Structures." The stability of the structure is determined by analyzing the loads it is subjected to and finding the resultant location (X_R). The model determines these loads and their location from the height of soil, height of water, and earth pressure coefficient on either side of the structure. The weight of the structure, the center of mass, any applied load and its location, and the base width (B) are also required input. The factor of safety is then found from the equation:

$$C/D=B/(B-2X_R)$$

The reliability index, β, was determined by the Taylor's Series Method. Six variables were chosen to be treated as random variables. They include: water level left side of the structure, water level right side of the structure, earth pressure coefficient left side of the structure, earth pressure coefficient right side of the structure, wall friction angle, and any horizontal loading (used mainly in impact scenarios). The percentage of base in compression is determined for each case to be used in the corresponding sliding analysis.

c. Sliding Model. The sliding model was performed using program X0075, CSLIDE - Sliding Stability Analysis of Concrete Structures. The reliability index, β, was determined by the Taylor's Series Method. Eight variables were chosen to be treated as random variables. They include: soil phi angle, rock active phi angle, rock active cohesion, rock passive phi angle, rock passive cohesion, percentage of base in compression, backfill water level, and any horizontal loading. Where extensive testing of rock strengths had been performed, correlation coefficients between cohesion and phi angle were used.

2. Site Selection. Because of the large number of locks and dams in the Rock Island District, only selected sites were

analyzed. Selection was made so that two sites with similar structural and foundation conditions were not both analyzed. Locks and Dams Nos. 14, 15, 16, 19, 22, Lockport, Brandon Road, Dresden Island, Marseilles, and Starved Rock are founded on rock.

GRAVITY STRUCTURES			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
14	Landwall	15	Landwall
		16	Landwall
		19	Landwall
		20	Landwall
		22	Landwall
		Lockport	Landwall
		Brandon Road	Landwall
		Dresden Island	Landwall
		Marseilles	Landwall
		Starved Rock	Landwall
	I-Wall	15	I-Wall
		16	I-Wall
		19	I-Wall
		20	I-Wall

GRAVITY STRUCTURES			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
14	I-Wall	22	I-Wall
		Lockport	I-Wall
		Brandon Road	I-Wall
		Dresden Island	I-Wall
		Marseilles	I-Wall
		Starved Rock	I-Wall

3. Important Deterministic and Random Variables.

GRAVITY STRUCTURES OVERTURNING											
			Random Variables								Deterministic Variables	
Lock and Dam	Component	Load Condition	Wall Friction ϕ_w		Earth Pressure K_a		Impact Load (kips)		Water Level in Backfill		Pool (feet)	TW (feet)
			μ	σ	μ	σ	μ	σ	μ	σ		
USAF	Landwall	Unwatered	4	1	0.44	0.07	0	0	753	1.5	-	728.5
	Lock[2]	Unwatered	4	1	0.44	0.07	0	0	753	1.5	-	728.5
	DS Guidewall	Impact	8	2	0.44	0.07	6[1]	1.8[1]	-	-	750	750
	US Guidewall	Impact	8	2	0.44	0.07	8[1]	2.4[1]	-	-	800	800
LSAF	Lock[2]	Unwatered	8	2	0.44	0.07	0	0	728.5	6.25	-	725.5
	US Guidewall	Impact	8	2	0.44	0.07	8[1]	2.4[1]	-	-	750	750
	Dam Piers	Unwatered	8	2	0.44	0.07	0	0	750	2	-	725.5
	Non-Overflow	Ice Load	8	2	0.44	0.07	5[1]	1.5[1]	725.5	2	750.3	-
14	Landwall	Unwatered	8	2	0.44	0.07	-	-	-	-	572	547.5
	I-Wall	Unwatered	8	2	0.44	0.07	-	-	561	3.5	-	547.5
24	US Guidewall	Impact	8	2	1.1	0.165	322	97	-	-	449	-
	DS Guidewall	Impact	-	-	-	-	241	72	-	-	-	449
	Landwall	Normal	8	2	0.44	0.07	-	-	447	4	449	434
27	Eastwall	Normal	8	2	0.44	0.07	-	-	401.2	3.7	406	388.7
	Eastwall	Unwatered	8	2	0.44	0.07	-	-	401.2	3.7	406	388.7
	Auxiliary Lock Westwall	Normal	8	2	0.44	0.07	-	-	401.2	3.7	406	388.7
	Auxiliary Lock Westwall	Unwatered	8	2	0.44	0.07	-	-	401.2	3.7	406	388.7

GRAV3.XLS

[1]These loads are in kips/ft.

[2]Lock U-Structure

GRAVITY STRUCTURES SLIDING																		
		Random Variables															Deterministic Variables	
Lock and Dam	Component	Soil φ (degrees)		Rock φ Active Wedge (degrees)		Rock c Active Wedge (psf)		Rock φ Passive Wedge & Base (degrees)		Rock c Passive Wedge & Base (psf)		% Base Compression		Water Level in Backfill (feet)			Pool (feet)	TW (feet)
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ			
24	Landwall (Unwatered)	34	3.4	28.7	3.88	1.22	1.01	43.1	7.84	2.26	2.26	78.2	17.7	447	4		449	434
	I-Wall	-	-	28.7	3.88	1.22	1.01	43.1	7.84	2.26	2.26	100	0	-	-		449	434
27	I-Wall (Normal Operating)	-	-	40	4	32	12.8	40	4	32	12.8	100	0	-	-		406	388.7

GRAV3A.XLS

4. <u>Weibull Distribution Parameters</u>.

GRAVITY STRUCTURES							
Cumulative Distribution Function		Current			Rehabilitated		
Lock and Dam	Component	b	α	ν	b	α	ν
USAF	Landwall	1.2	470	-0.04	1.2	470	0
	Lock[1]	1.2	470	-70	1.2	470	0
	DS Guidewall	1.2	470	-0.8	1.2	470	0
	US Guidewall	1.2	470	-205	1.2	470	0
LSAF	Lock[1]	1.2	470	-1.5	1.2	470	0
	US Guidewall	1.2	470	-82	1.2	470	0
	Dam Piers	1.2	470	≈0	1.2	470	0
	Non-Overflow	1.2	470	-0.02	1.2	470	0
14	Landwall	1.2	470	0	1.2	470	0
	I-Wall	1.2	470	0	1.2	470	0
24	US Guidewall	1.2	470	-1.12	1.2	470	0
	DS Guidewall	1.2	470	-373.5	1.2	470	0
	Landwall (Normal Operating)	1.2	470	-1.29	1.2	470	0
	Landwall (Unwatered)	1.2	470	-2.87	1.2	470	0
	I-Wall	1.2	470	0	1.2	470	0
27	Eastwall (Normal Operating)	1.2	470	-2.85	1.2	470	0
	Eastwall (Unwatered)	1.2	470	-30.98	1.2	470	0
	I-Wall (Normal Operating)	1.2	470	0	1.2	470	0
	Auxiliary Lock Westwall (Normal Operating)	1.2	470	-0.08	1.2	470	0
	Auxiliary Lock Westwall (Unwatered)	1.2	470	-4.3	1.2	470	0

GRAV4.XLS

[1]Lock U-Structure.

5. Consequences.

GRAVITY STRUCTURES										
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)					
		P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Slowdown[2] Effect on Lockage Cycle	Slowdown[2] Slowdown Duration (days)	Repair Costs (million)
USAF	Landwall	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200
	Lock	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200
	DS Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$13
	US Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$13
LSAF	Lock	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200
	US Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$13
	Dam Piers	0.95	1	$2.0	0.05	20	0	0	$23[3]
	Non-Overflow	0.95	1	$2.0	0.05	20	0	0	$24[4]
14	Landwall	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200
	I-Wall	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200

[1] 30 days initial + 75 days during winter shutdown or low traffic period.

[2] Slowdown time is added to navigation downtime.

[3] $280 million x (200'/2400').

[4] $280 million x (208'/2400').

GRAVITY STRUCTURES										
		Medium Level of Consequences (MC)			High Level of Consequences (HC)					
							Slowdown[2]			
Lock and Dam	Component	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Effect on Lockage Cycle	Slowdown Duration (days)	Repair Costs (million)	
24	US Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$13	
	DS Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$13	
	Landwall (Normal Operating)	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	
	Landwall (Unwatered)	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	
	I-Wall	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	
27	Eastwall (Normal Operating)	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	
	Eastwall (Unwatered)	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	
	I-Wall (Normal Operating)	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	
	Auxilary Lock Westwall (Normal Operating)	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	
	Auxilary Lock Westwall (Unwatered)	0.85	3	$2.0	0.15	30+75[1]	double time to empty or fill	200	$200	

GRAV5A.XLS

[1]30 days initial + 75 days during winter shutdown or low traffic period.

[2]Slowdown time is added to navigation downtime.

6. Cost of Rehabilitation.

GRAVITY STRUCTURES		
Lock and Dam	Component	Rehabilitation Cost (millions)
USAF	Landwall	$5.8
	Lock	$5.8
	DS Guidewall	$5.8
	US Guidewall	$5.8
LSAF	Lock	$5.8
	US Guidewall	$5.8
	Dam Piers	$1.9[1]
	Non-Overflow	$2.0[2]
14	Landwall	$5.8
	I-Wall	$5.8
24	US Guidewall	$5.8
	DS Guidewall	$5.8
	Landwall (Normal Operating)	$5.8
	Landwall (Unwatered)	$5.8
	I-Wall	$5.8
27	Eastwall (Normal Operating)	$5.8
	Eastwall (Unwatered)	$5.8
	I-Wall (Normal Operating)	$5.8
	Auxiliary Lock Westwall (Normal Operating)	$5.8
	Auxiliary Lock Westwall (Unwatered)	$5.8

GRAV6.XLS

[1]23.2 million x (200'/2400').

[2]23.2 million x (208'/2400').

7. <u>Number of Components</u>.

GRAVITY STRUCTURES				
District	Lockwalls	Guidewalls	Pool Control Dams	Total
St. Paul	8	7	8	23
Rock Island	39	18	10	67
St. Louis	5	2	0	7
TOTAL	52	27	18	97

APPENDIX D - Pile Foundations

P I L E

F O U N D A T I O N S

UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

PILE FOUNDATIONS

1. Model Description. The general approach for calculation of the reliability index is covered in ETL 1110-2-354 - Reliability Assessment of Pile-Founded Navigation Structures. The pile foundation reliability model uses the methodology in the computer program X0080, CPGA - Pile Group Analysis. Based on the CPGA methodology, a spreadsheet was developed to analyze two-dimensional pile groups to determine their reliability. The following deterministic parameters are required as input: constant loading on pile cap (including weight of the structure), pile locations in the group, elastic modulus of pile, length of pile, axial pile stiffness, and allowable lateral pile deflection. Seven variables were chosen to be treated as random variables. They include: soil stiffness, earth pressure coefficient, impact loading, pile capacity, pile diameter, allowable compression stress, and allowable bending stress. The model calculates the reliability index for three different performance modes: pile capacity, pile stresses, and pile group deflection.

2. Site Selection.

PILE FOUNDATIONS			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
11	Landwall	12	Landwall
	I-Wall		I-Wall
	US Guidewall		US Guidewall
	DS Guidewall		DS Guidewall
	Dam Piers		Dam Piers

3. Important Deterministic and Random Variables.

PILE FOUNDATIONS																		
Variables		Random Variables															Deterministic Variables	
Lock & Dam	Component	n_h (kci)		Earth Pressure Coefficient		Impact Load (kips)		Pile/Soil Capacity (kips)		Pile Diameter (in)		Compression Stress (psi)		Bending Stress (psi)			Pool (feet)	TW (feet)
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ			
2	Landwall	0.002	0.0005	k[1]	0.15k	-	-	120	30	12	0.24	2528	455	5830	933		687.5	675
	I-Wall	0.002	0.0005	k[1]	0.15k	-	-	120	30	12	0.24	2528	455	5830	933		687.5	675
	US Guidewall	0.002	0.0005	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933		687.5	675
	DS Guidewall	0.002	0.0005	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933		687.5	675
	Dam Piers	0.002	0.0005	k[1]	0.15k	-	-	120	30	12	0.24	2528	455	5830	933		687.5	675
3	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		675	667
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		675	667
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933		675	667
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933		675	667
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		675	667
4	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		667	660
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		667	660
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933		667	660
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933		667	660
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		667	660
5	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		660	651
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		660	651
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933		660	651
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933		660	651
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		660	651
5A	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		651	645.5
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		651	645.5
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933		651	645.5
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933		651	645.5
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		651	645.5
6	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		645.5	639
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		645.5	639
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933		645.5	639
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933		645.5	639
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933		645.5	639

[1] k = as calculated.

PILE FOUNDATIONS																	
Variables		Random Variables														Deterministic Variables	
Lock & Dam	Component	n_h (kci)		Earth Pressure Coefficient		Impact Load (kips)		Pile/Soil Capacity (kips)		Pile Diameter (in)		Compression Stress (psi)		Bending Stress (psi)		Pool (feet)	TW (feet)
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ		
7	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	639	631
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	639	631
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	639	631
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	639	631
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	639	631
8	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	631	620
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	631	620
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	631	620
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	631	620
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	631	620
9	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	620	611
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	620	611
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	620	611
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	620	611
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	620	611
10	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	611	603
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	611	603
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	611	603
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	611	603
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	611	603
11	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	603	592
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	603	592
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	603	592
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	603	592
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	603	592
13	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	583	572
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	583	572
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	583	572
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	583	572
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	0	140	35	12	0.24	2528	455	5830	933	583	572

[1]k = as calculated.

PILE FOUNDATIONS																	
Variables		Random Variables														Deterministic Variables	
Lock & Dam	Component	n_h (kci)		Earth Pressure Coefficient		Impact Load (kips)		Pile/Soil Capacity (kips)		Pile Diameter (in)		Compression Stress (psi)		Bending Stress (psi)		Pool (feet)	TW (feet)
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ		
16	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	554	536
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	554	536
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	554	536
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	554	536
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	554	536
17	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	536	528
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	536	528
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	536	528
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	536	528
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	536	528
18	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	528	518.2
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	528	518.2
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	528	518.2
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	528	518.2
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	528	518.2
20	Dam Piers	0.01	0.0025	-	-	-	-	140	35	12	0.24	2528	455	5830	933	475.5	470
21	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	470	459.5
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	470	459.5
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	470	459.5
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	470	459.5
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	470	459.5
24	Dam Pier	0.009	0.002	-	-	-	-	100	25	13	0.35	2580	464	6225	996	449	434
25	US Guidewall	0.009	0.002	0.27	0.04	-	-	120	29	13	0.32	2580	464	6225	996	434	-
	DS Guidewall	0.009	0.002	-	-	241	72	120	29	13	0.58	2580	464	6225	996	-	434
	Landwall (Normal Operating)	0.009	0.002	0.27	0.04	-	-	120	29	13	0.33	2580	464	6225	996	434	419
	I-Wall (Normal Operating)	0.009	0.002	-	-	-	-	120	29	13	0.32	2580	646	6225	996	434	419
	Dam Pier Tainter Gate	0.009	0.002	-	-	-	-	120	29	13	0.38	2580	464	6225	996	434	419
	Dam Pier Roller Gate	0.009	0.002	-	-	-	-	120	29	13	0.33	2580	464	6225	996	434	419

[1]k = as calculated.

PILE FOUNDATIONS																		
Variables		Random Variables														Deterministic Variables	
Lock & Dam	Component	n_h (kci)		Earth Pressure Coefficient		Impact Load (kips)		Pile/Soil Capacity (kips)		Pile Diameter (in)		Compression Stress (psi)		Bending Stress (psi)		Pool (feet)	TW (feet)
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ		
MP	Auxiliary Lock US Guidewall	Assumed β greater than 4 by engineering judgement.															
	Auxiliary Lock DS Guidewall	Assumed β greater than 4 by engineering judgement.															
	Auxiliary Lock Monolith	Assumed β greater than 4 by engineering judgement.															
	Main Lock US Guidewall	Assumed β greater than 4 by engineering judgement.															
	Main Lock DS Guidewall	Assumed β greater than 4 by engineering judgement.															
	Main Lock Monolith	Assumed β greater than 4 by engineering judgement.															
	Dam Pier	Assumed β greater than 4 by engineering judgement.															
27	US Guidewall	0.009	0.002	-	-	322	97	310	77.5	11.8	0	37800	4158	38880	4277	406	-
	DS Guidewall	0.009	0.002	-	-	-	-	310	77.5	11.8	0	37800	4158	38880	4277	-	388.7
Peoria	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	440	429
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	440	429
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	440	429
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	440	429
La Grange	Landwall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	429	419
	I-Wall	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	429	419
	US Guidewall	0.01	0.0025	k[1]	0.15k	322	0	100	25	12	0.24	2528	455	5830	933	429	419
	DS Guidewall	0.01	0.0025	k[1]	0.15k	241	0	100	25	12	0.24	2528	455	5830	933	429	419
	Dam Piers	0.01	0.0025	k[1]	0.15k	-	-	140	35	12	0.24	2528	455	5830	933	429	419

PILE3.XLS

[1]k = as calculated.

4. Weibull Distribution Parameters.

PILE FOUNDATIONS							
Lock and Dam	Component	Current			Rehabilitated		
		b	α	ν	b	α	ν
2	Landwall	1.0	460	0	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0
3	Landwall	1.0	460	-0.2	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	-0.2	1.0	460	0
4	Landwall	1.0	460	0	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0
5	Landwall	1.0	460	-0.01	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0
5A	Landwall	1.0	460	-0.3	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0

PILE FOUNDATIONS							
Lock and Dam	Component	Current			Rehabilitated		
		b	α	ν	b	α	ν
6	Landwall	1.0	460	0	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	-0.01	1.0	460	0
7	Landwall	1.0	460	-0.01	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0
8	Landwall	1.0	460	-0.2	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0
9	Landwall	1.0	460	-0.01	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0
10	Landwall	1.0	460	-0.01	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	-0.03	1.0	460	0
11	Landwall	1.0	460	-1.2	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	-0.2	1.0	460	0

PILE FOUNDATIONS							
Lock and Dam	Component	Current			Rehabilitated		
		b	α	ν	b	α	ν
13	Landwall	1.0	460	-0.5	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
16	Landwall	1.0	460	-0.2	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	-0.02	1.0	460	0
17	Landwall	1.0	460	-0.2	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	-0.01	1.0	460	0
18	Landwall	1.0	460	-0.1	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	-0.2	1.0	460	0
20	Dam Piers	1.0	460	-0.2	1.0	460	0
21	Landwall	1.0	460	-0.5	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0
24	Dam Pier	1.0	460	0	1.0	460	0
25	Landwall	1.0	460	-0.26	1.0	460	0
	Intermediate Wall	1.0	460	-0.66	1.0	460	0
	US Guidewall	1.0	460	-3.39	1.0	460	0
	DS Guidewall	1.0	460	-612.03	1.0	460	0
	Dam Pier Tainter Gate	1.0	460	0	1.0	460	0
	Dam Pier Roller Gate	1.0	460	0	1.0	460	0

PILE FOUNDATIONS							
Lock and Dam	Component	Current			Rehabilitated		
		b	α	ν	b	α	ν
Mel Price	Aux. Lock US Guidewall	1.0	460	0	1.0	460	0
	Aux. Lock DS Guidewall	1.0	460	0	1.0	460	0
	Aux. Lock Monolith	1.0	460	0	1.0	460	0
	Main Lock US Guidewall	1.0	460	0	1.0	460	0
	Main Lock DS Guidewall	1.0	460	0	1.0	460	0
	Main Lock Monolith	1.0	460	0	1.0	460	0
	Dam Pier	1.0	460	0	1.0	460	0
27	US Guidewall	1.0	460	-6.28	1.0	460	0
	DS Guidewall	1.0	460	-2.35	1.0	460	0
Peoria	Landwall	1.0	460	-0.05	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
LaGrange[1]	Landwall	1.0	460	-0.01	1.0	460	0
	Intermediate Wall	1.0	460	0	1.0	460	0
	US Guidewall	1.0	460	-170	1.0	460	0
	DS Guidewall	1.0	460	-80	1.0	460	0
	Dam Piers	1.0	460	0	1.0	460	0

PILE4.XLS

[1]Flashboard Structure.

5. <u>Consequences</u>.

PILE FOUNDATIONS									
		Medium Level of Consequences (MC)			High Level of Consequences (HC)				
							Slowdown[2]		
Lock and Dam	Component	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Effect on Lockage Cycle	Slowdown Duration (days)	Repair Costs (million)
2, 3, 4, 5	Lockwalls	0.85	3	$2.0	0.15	30 + 75[1]	double time to empty or fill	200	$250
5A, 6, 7, 8	US Guidewalls	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
9, 10, 11, 13	DS Guidewalls	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
16, 17, 18, 20, 21	Dam Piers	0.95	1	$2.0	0.05	20	0	0	$350
24	Dam Pier	0.95	1	$2.0	0.05	20	0	0	$350
25	Landwall (Normal Operating)	0.85	3	$2.0	0.15	30 + 75[1]	double time to empty or fill	200	$250
	I-Wall (Normal Operating)	0.85	3	$2.0	0.15	30 + 75[1]	double time to empty or fill	200	$250
	US Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16

[1]30 days initial + 75 days during winter shutdown or low traffic period.

[2]Slowdown time is added to navigation downtime.

PILE FOUNDATIONS									
		Medium Level of Consequences (MC)			High Level of Consequences (HC)				
							Slowdown[2]		
Lock and Dam	Component	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Effect on Lockage Cycle	Slowdown Duration (days)	Repair Costs (million)
25	DS Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
	Dam Pier Tainter Gate	0.95	1	$2.0	0.05	20	0	0	$350
	Dam Pier Roller Gate	0.95	1	$2.0	0.05	20	0	0	$350
Melvin Price	Auxiliary Lock US Guidewall	0.75	3	$1.0	0.25	30 + 75[1]	double time to empty or fill	200	$16
	Auxiliary Lock DS Guidewall	0.75	3	$1.0	0.25	30 + 75[1]	double time to empty or fill	200	$16
	Auxiliary Lock Monolith	0.85	3	$2.0	0.15	30 + 75[1]	double time to empty or fill	200	$250
	Main Lock US Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
	Main Lock DS Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
	Main Lock Monolith	0.85	3	$2.0	0.15	30 + 75[1]	double time to empty or fill	200	$250
	Dam Pier	0.95	1	$2.0	0.05	20	0	0	$350
27	US Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
	DS Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
Peoria	Lockwalls	0.85	3	$2.0	0.15	30 + 75[1]	double time to empty or fill	200	$250
LaGrange	US Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
	DS Guidewall	0.75	5	$1.0	0.25	10	20 min. delay/tow	180	$16
	Dam Piers	0.95	1	$2.0	0.05	20	0	0	$350

PILES.XLS

[1]30 days initial + 75 days during winter shutdown or low traffic period.

[2]Slowdown time is added to navigation downtime.

6. Cost of Rehabilitation.

PILE FOUNDATIONS		
Lock and Dam	Component	Rehabilitation Cost (millions)
2, 3, 4, 5	Lockwalls	$3.0
5A, 6, 7, 8	US Guidewalls	$3.5
9, 10, 11, 13	DS Guidewalls	$3.5
16, 17, 18, 20, 21	Dam Piers	$12.0
24	Dam Pier	$12.0
25	Landwall (Normal Operating)	$3.0
	I-Wall (Normal Operating)	$3.0
	US Guidewall	$3.5
	DS Guidewall	$3.5
	Dam Pier Tainter Gate	$12.0
	Dam Pier Roller Gate	$12.0

PILE FOUNDATIONS		
Lock and Dam	Component	Rehabilitation Cost (millions)
Melvin Price	Auxiliary Lock US Guidewall	$3.5
	Auxiliary Lock DS Guidewall	$3.5
	Auxiliary Lock Monolith	$3.0
	Main Lock US Guidewall	$3.5
	Main Lock DS Guidewall	$3.5
	Main Lock Monoliths	$3.0
	Dam Pier	$12.0
27	US Guidewalls	$3.0
	DS Guidewalls	$3.5
Peoria LaGrange	Lockwalls	$3.0
	US Guidewalls	$3.5
	DS Guidewalls	$3.5
	Dam Pier	$12.0

PILE6.XLS

7. Number of Components.

PILE FOUNDATIONS				
District	Lockwalls	Guidewalls	Pool Control Dams	Total
St. Paul	32	21	16	69
Rock Island	29	24	10	63
St. Louis	5	8	3	16
TOTAL	66	53	29	148

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APPENDIX E - Underseepage for Lock Unwatering, Pool Control Dams & Earth Embankments

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UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

UNDERSEEPAGE

1. Model Description. Underseepage reliability analyses were accomplished by the finite element model (FEM), the method of fragments model, the Corps' method for levee underseepage analysis model, and the St. Paul District method.

a. FEM Model. Underseepage was analyzed at earth embankments that retain pool by the development of flow nets. These flow nets were developed with the finite element computer program, X8202 - A Plane and Axisymmetric Finite Element Program for Steady-State Seepage Problems. The program requires a profile of the embankment and foundation, the permeability of any materials in the profile, and the water pressure on regions where flow is permitted through the boundary of the profile. Resulting heads and corresponding exit gradients can be determined with the flow nets.

The reliability index, β, is determined by the Taylor's Series Method from the factor of safety against seepage failure. The only variable chosen as random was the ratio of horizontal permeability to vertical permeability. The actual value of permeability does vary, but only the ratio of horizontal permeability to vertical permeability affects the exit gradient.

b. Method of Fragments Model.

(1) Pool Control Dams.

(a) Underseepage was analyzed at pool control dams by the method of fragments. This method is presented in EM 1110-2-1901, Seepage Analysis and Control for Dams, Appendix B. A pool control dam consists of a number of concrete piers. Between each concrete pier is a steel tainter gate or roller gate. These gates are raised or lowered as necessary to maintain the pool at the required elevation. So, a pool control dam consists of concrete piers supporting either tainter gates or roller gates. When the term tainter or roller gate is used, it is referring to the pier that supports that type of gate.

(b) The geometry of the problem and water elevation at the boundaries are used to find the seepage gradient. High seepage gradients can form beneath or downstream of the dam piers, causing piping of the foundation material to occur. Piping is the removal of material from the foundation due to high seepage gradients. The material is either piped through dam pier monolith joints, weep holes, or occurs downstream of the dam. Removal of dam pier foundation material will lead to instability of the dam pier. The dam will slide or rotate downstream causing a break in the damming surface. A break in the damming surface will quickly lead to loss of pool and serious erosion of the foundation of the adjacent piers. The reliability index, β, is determined by the Taylor's Series Method from the exit gradients. Three variables were chosen to be treated as random variables: upstream sheetpile length, effective base length, and scour downstream of the dam. The deviation of the sheetpile length is to account for any windows that could be present in the cutoff. The effective base length varies as the ratio of the horizontal permeability to the vertical permeability varies.

(2) Lock Chambers. A method of fragments model was also used to determine the reliability index for unwatering of lock chambers. Seepage enters the foundation riverside of a lockwall, flows under a partially penetrating sheetpile cutoff wall under the lockwall, and flows upward to the lock floor. The geometry of the lockwall and the interior and exterior water elevations are used to find the seepage gradient. The seepage gradient below the lock floor can be so high that it causes sand boils to form at the lock floor joints and through the weep holes in the lock floor. High seepage gradient causes foundation material from beneath the lock floor and the lock walls to flow into the lock chamber. Sand boils left uncontrolled or uncontrollable will lead to the instability of the lockwall, which means the lockwall will collapse into the lock chamber. The reliability index, β, is determined by the Taylor's Series Method from the exit gradients. Two variables were chosen to be treated as random variables: the sheetpile cutoff wall length and the depth of the foundation material to bedrock. Table 1 lists each lock and the approximate time between unwatering events.

TABLE 1
TIME BETWEEN LOCK UNWATERING EVENTS

Lock	Approximate Time Between Unwatering Events (Years)
USAF	15
LSAF	15
Locks 1	15
Lock 2	15
Old Lock 2	Not Scheduled. Emergency Only.
Lock 3	15
Lock 4	15

Lock	Approximate Time Between Unwatering Events (Years)
Lock 5	15
Lock 5A	15
Lock 6	15
Lock 7	15
Lock 8	15
Lock 9	15
Lock 10	15
Lock 14	20
Lock 15	20
Lock 16	20
Lock 19	20
Lock 22	20
Lockport	15
Brandon Road	15
Dresden Island	15
Marseilles	15
Starved Rock	15
Lock 24	25
Lock 25	15
Melvin Price Auxiliary Lock	20
Melvin Price Main Lock	10
Locks 27 Main Lock	10
Locks 27 Auxiliary Lock	10

c. Corps' Method for Levee Underseepage Analysis. Underseepage analyses were performed on earth embankment sections that retain pool, in accordance with EM 1110-2-1913 - Design and Construction of Levees. The effective blanket thickness, blanket permeability, and uplift gradients at the earth embankment toe were computed based on the above guidance document. The pool entrance conditions were evaluated to determine the minimum effective entrance distance. The tailwater exit conditions assumed that the exit distances (X_3) were based on the effective thickness and permeability of the tailwater side blanket at the earth embankment toe. The equations and correlations presented in the above documents have been incorporated into a spreadsheet that determines the exit gradient (i_{exit}) at the tailwater side toe. The analyses to predict the probability of unsatisfactory performance were performed in accordance with ETL 1110-2-547 - Introduction to Probability and Reliability Methods for Use in Geotechnical Engineering. The Taylor's Series Method was used to obtain the expected value and standard deviation for the factor of safety against seepage failure. The probability of unsatisfactory performance was determined from the reliability index which was calculated from the expected value and standard deviation of the factor of safety against seepage failure. Three parameters required for the underseepage analysis were selected to be random: permeability of the aquifer (K_f), permeability of the top stratum (K_{bl}), and thickness of the top stratum (z_{bl}).

d. St. Paul Method. The general approach for calculation of the reliability index is covered in Chapter 4 of the Shannon and Wilson report dated 21 January 1994 (Contract DACW43-91-D-0503). This approach assumes a critical state failure consisting of uplift or piping of the landside blanket. Some modifications/additions to the Shannon and Wilson approach were made. A second order Taylor's series approximation for the second moment was used to reduce modeling errors associated with high variances. Since the uplift failure mode is hyper-sensitive to the landside blanket thickness, results were checked by analysis of slope stability effected by substratum pressure. Calculations and description of the method are discussed in the draft report titled "Piping/Uplift and Slope Stability of Earth Dikes Under Steady State Seepage" dated March 1995 by the St. Paul District. This model was only used by the St. Paul District.

e. Underseepage was subdivided into the following three separate models: underseepage for lock unwatering, underseepage for pool control dams, and underseepage for earth embankments. The results are presented as follows:

UNDERSEEPAGE FOR LOCK UNWATERING

2. Site Selection.

UNDERSEEPAGE FOR LOCK UNWATERING			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
11	Intermediate Wall	USAF	I-Wall
		LSAF	I-Wall
		1	I-Wall
		2	I-Wall
		3	I-Wall
		4	I-Wall
		5	I-Wall
		5A	I-Wall
		6	I-Wall
		7	I-Wall
		8	I-Wall
		9	I-Wall
		10	I-Wall
		Melvin Price	Main Lock
			Auxiliary Lock

3. <u>Important Deterministic and Random Variables</u>.

UNDERSEEPAGE FOR LOCK UNWATERING									
Lock and Dam	Component	Random							Deterministic
		Φ_A Approach Form Factor					i_{cr}		Differential Head (feet)
Probability		0.06	0.12	0.08	0.73	-0.01	μ	σ	
11	Intermediate Wall						0.843	0.08	16.0
12	Intermediate Wall						0.843	0.08	15.5
13	Intermediate Wall						0.843	0.08	17.0
16	Intermediate Wall						0.843	0.08	17.0
17	Intermediate Wall						0.843	0.08	18.0
18	Intermediate Wall						0.843	0.08	18.2
21	Intermediate Wall						0.843	0.08	17.0
25	Lock Chamber (Unwatered)	3.47	1.74	1.11	0.97	0.78			16.0
TJ O'Brien	Intermediate Wall	Assumed β greater than 5 by engineering judgement.							
Peoria	Intermediate Wall						0.843	0.08	15.0
LaGrange	Intermediate Wall						0.843	0.08	15.5

LOCKUW3.XLS

4. Weibull Distribution Parameters.

UNDERSEEPAGE FOR LOCK UNWATERING							
Cumulative Distribution Function		Current			Rehabilitated		
Lock and Dam	Component	b	α	ν	b	α	ν
11	Intermediate Wall	1.2	360	0	1.2	360	0
12	Intermediate Wall	1.2	360	-1.85	1.2	360	0
13	Intermediate Wall	1.2	360	0	1.2	360	0
16	Intermediate Wall	1.2	360	0	1.2	360	0
17	Intermediate Wall	1.2	360	0	1.2	360	0
18	Intermediate Wall	1.2	360	-3.61	1.2	360	0
21	Intermediate Wall	1.2	360	0	1.2	360	0
25	Intermediate Wall	1.2	360	-102	1.2	360	0
TJ O'Brien	Intermediate Wall	1.2	360	0	1.2	360	0
Peoria	Intermediate Wall	1.2	360	0	1.2	360	0
LaGrange	Intermediate Wall	1.2	360	0	1.2	360	0

LOCKUW4.XLS

5. <u>Consequences</u>.

UNDERSEEPAGE FOR LOCK UNWATERING

Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)				
		P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Slowdown[2] Effect on Lockage Cycle	Slowdown[2] Slowdown Duration (days)	Repair Costs (million)
11	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
12	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
13	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
16	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
17	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
18	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
21	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
25	Lock Chamber (Unwatered)	0.85	3	$0.5	0.15	30 + 75[1]	double time to empty or fill	200	$10.00
TJ O'Brien	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
Peoria	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64
LaGrange	Intermediate Wall	0.9	1	$0.5	0.1	20	0	0	$2.64

LOCKUW5.XLS

[1]30 days initial + 75 days during winter shutdown or low traffic period.

[2]Slowdown time is added to navigation downtime.

6. Cost of Rehabilitation.

UNDERSEEPAGE FOR LOCK UNWATERING		
Lock and Dam	Component	Rehabilitation Cost (millions)
11	Intermediate Wall	$0.64
12	Intermediate Wall	$0.64
13	Intermediate Wall	$0.64
16	Intermediate Wall	$0.64
17	Intermediate Wall	$0.64
18	Intermediate Wall	$0.64
21	Intermediate Wall	$0.64
25	Lock Chamber	$3.00
TJ O'Brien	Intermediate Wall	$0.64
Peoria	Intermediate Wall	$0.64
LaGrange	Intermediate Wall	$0.64

LOCKUW6.XLS

7. Number of Components.

UNDERSEEPAGE FOR LOCK UNWATERING	
District	Locks
St. Paul	13
Rock Island	17
St. Louis	3
TOTAL	33

UNDERSEEPAGE FOR POOL CONTROL DAMS

2. <u>Site Selection</u>.

a. <u>Rock Island</u>. Because of the large number of locks and dams in the Rock Island District, only selected sites were analyzed. Selection was made so that two sites with similar structural and foundation conditions were not both analyzed. Locks and Dams Nos. 14, 15, 16, 19, 20 (only lock structure), 22, Lockport, Brandon Road, Dresden Island, Marseilles, and Starved Rock are founded on rock. There are no earth dikes at Locks and Dams Nos. 15, 19, TJ O'Brien, Marseilles, Starved Rock, and Peoria.

b. <u>St. Paul</u>. Because of the large number of locks and dams in the St. Paul District, only the roller gate piers were analyzed, except at Dam 2, where only tainter gates exist. Preliminary analyses showed the roller gates piers were slightly more critical than the tainter gates piers.

<table>
<tr><th colspan="4">UNDERSEEPAGE FOR POOL CONTROL DAMS</th></tr>
<tr><th colspan="2">Sites Investigated</th><th colspan="2">Sites in the Same Bracket</th></tr>
<tr><th>Lock &Dam</th><th>Component</th><th>Lock & Dam</th><th>Component</th></tr>
<tr><td rowspan="4">11</td><td rowspan="4">Roller Gates</td><td>11</td><td>Tainter Gates</td></tr>
<tr><td rowspan="3">16</td><td>Tainter Gates</td></tr>
<tr><td>Roller Gates</td></tr>
<tr><td>Fixed Crest Dam</td></tr>
<tr><td rowspan="2">12</td><td>Tainter Gates</td><td>17</td><td>Tainter Gates</td></tr>
<tr><td>Roller Gates</td><td>17</td><td>Roller Gates</td></tr>
<tr><td rowspan="2">13</td><td>Tainter Gates</td><td></td><td></td></tr>
<tr><td>Roller Gates</td><td></td><td></td></tr>
</table>

UNDERSEEPAGE FOR POOL CONTROL DAMS			
Sites Investigated		Sites in the Same Bracket	
Lock &Dam	Component	Lock & Dam	Component
18	Tainter Gates		
	Roller Gates		
20	Tainter Gates		
	Roller Gates		
21	Tainter Gates		
	Roller Gates		
TJ O'Brien	Fixed Dam	TJ O'Brien	Control Dam
La Grange	Regulating Weir	Peoria	Regulating Weir
	Navigable Weir	Peoria	Navigable Dam

3. Important Deterministic and Random Variables.

UNDERSEEPAGE FOR THE POOL CONTROL DAMS																	
		Random													Deterministic		
Lock and Dam	Component	US Sheetpile Length (feet)		Dam Base Length (feet)			DS Depth of Scour (feet)			DS Sheetpile Length (feet)			Critical Gradient i_c		Depth of Acquifer (feet)	Pool (feet)	TW (feet)
Probability		0.9	0.1	0.2	0.6	0.2	0.5	0.4	0.1	0.2	0.6	0.2	μ	σ			
LSAF	Gravity Wall			37	37	37							1.1	0.1			
2	Tainter Gates	32	16	43.5	22	15				15	15	15	0.84	0.08	15		
3	Dam Pier	Reliability is high due to US cutoff and DS exit.															
4	Roller Gates	17.9	9	102	51	34				21	16	11	0.84	0.08	18		
5	Roller Gates	30	15	102	51	34				22	17	12	0.84	0.08	52		
5A	Roller Gates	40	20	105	52	35				21	16	11	0.84	0.08	60		
6	Roller Gates	45	22.5	102	51	34				23	17	12	0.84	0.08	92		
7	Roller Gates	42	21	105	52	35				21	16	11	0.84	0.08	50		
8	Roller Gates	40	20	105	52	35				21	16	11	0.84	0.08	60		
9	Roller Gates	38	19	105	52	35				21	16	11	0.84	0.08	69		
10	Roller Gates	31	16	120	60	40				23	18	13	0.84	0.08	91		
11	Roller Gates	35	18	105	53	35	0	6	12							603	592
12	Tainter Gates	35	18	80	40	27	0	6	12							592	583
	Roller Gates	35	18	105	53	35	0	6	12							592	583
13	Tainter Gates	33	17	78	39	26	0	6	12							583	572
	Roller Gates	34	17	102	51	34	0	6	12							583	572
18	Tainter Gates	33	17	73	37	24	0	6	12							528	518.2
	Roller Gates	32	16	98	49	33	0	6	12							528	518.2
20	Tainter Gates	31	16	75	38	25	0	6	12							480	470
	Roller Gates	31	16	123	62	41	0	6	12							480	470
21	Tainter Gates	35	18	82	41	27	0	6	12							470	459.5
	Roller Gates	35	18	107	54	36	0	6	12							470	459.5
24	Dam Piers	34	17	100	50	33	0	6	12							449	434
25	Dam Piers	41	21	102	51	34	0	6	12							434	419
Mel Price	Dam Piers	Assumed β greater than 4 by engineering judgement.															
O'Brien	Fixed Dam	Assumed β greater than 5 by engineering judgement.															
LaGrange	Regulating Weir	35	18	77	39	26	0	6	12							429	419
	Navigable Weir	30	15	77	39	26	0	6	12							429	419

POOL3.XLS

4. Weibull Distribution Parameters.

UNDERSEEPAGE FOR THE POOL CONTROL DAMS							
Cumulative Distribution Function		Current			Rehabilitated		
Lock and Dam	Component	b	α	ν	b	α	ν
LSAF	Gravity Wall	1.2	360	-0.2	1.2	360	0
2	Tainter Gates	1.2	360	0	1.2	360	0
3	Dam Pier	1.2	360	0	1.2	360	0
4	Roller Gates	1.2	360	0	1.2	360	0
5	Roller Gates	1.2	360	0	1.2	360	0
5A	Roller Gates	1.2	360	0	1.2	360	0
6	Roller Gates	1.2	360	0	1.2	360	0
7	Roller Gates	1.2	360	0	1.2	360	0
8	Roller Gates	1.2	360	0	1.2	360	0
9	Roller Gates	1.2	360	0	1.2	360	0
10	Roller Gates	1.2	360	0	1.2	360	0
11	Roller Gates	1.2	360	0	1.2	360	0
12	Tainter Gates	1.2	360	0	1.2	360	0
	Roller Gates	1.2	360	0	1.2	360	0
13	Tainter Gates	1.2	360	0	1.2	360	0
	Roller Gates	1.2	360	0	1.2	360	0
18	Tainter Gates	1.2	360	0	1.2	360	0
	Roller Gates	1.2	360	0	1.2	360	0
20	Tainter Gates	1.2	360	0	1.2	360	0
	Roller Gates	1.2	360	0	1.2	360	0
21	Tainter Gates	1.2	360	0	1.2	360	0
	Roller Gates	1.2	360	0	1.2	360	0
24	Concrete Dam	1.2	360	0	1.2	360	0
25	Concrete Dam	1.2	360	0	1.2	360	0
Mel Price	Concrete Dam	1.2	360	0	1.2	360	0
TJ O'Brien	Fixed Dam	1.2	360	0	1.2	360	0
	Control Dam	1.2	360	0	1.2	360	0
LaGrange	Regulating Weir	1.2	360	-0.64	1.2	360	0
	Navigable Weir	1.2	360	-1.23	1.2	360	0

POOL4.XLS

5. <u>Consequences</u>.

UNDERSEEPAGE FOR THE POOL CONTROL DAMS							
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)		
		P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Repair Costs (million)
LSAF	Gravity Wall	0.95	1	$2.0	0.05	20	$100.0
2	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
3	Dam Pier	0.95	1	$2.0	0.05	20	$350.0
4	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
5	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
5A	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
6	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
7	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
8	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
9	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
10	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
11	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
12	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
13	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
16	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
	Fixed Crest Dam	0.95	1	$2.0	0.05	20	$350.0
17	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
18	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
20	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
21	Tainter Gates	0.95	1	$2.0	0.05	20	$350.0
	Roller Gates	0.95	1	$2.0	0.05	20	$350.0
24	Concrete Dam	0.95	1	$2.0	0.05	20	$350.0
25	Concrete Dam	0.95	1	$2.0	0.05	20	$350.0
Mel Price	Concrete Dam	0.95	1	$2.0	0.05	20	$350.0
TJ O'Brien	Fixed Dam	0.95	1	$2.0	0.05	20	$350.0
	Control Dam	0.95	1	$2.0	0.05	20	$350.0
Peoria	Regulating Weir	0.95	1	$2.0	0.05	20	$350.0
	Navigable Dam	0.95	1	$2.0	0.05	20	$350.0
LaGrange	Regulating Weir	0.95	1	$2.0	0.05	20	$350.0
	Navigable Dam	0.95	1	$2.0	0.05	20	$350.0

POOL5.XLS

6. <u>Cost of Rehabilitation</u>.

UNDERSEEPAGE FOR POOL CONTROL DAMS		
Lock and Dam	Component	Rehabilitation Cost (millions)
LSAF	Gravity Wall	$4.0
2	Tainter Gates	$12.0
3	Dam Pier	$12.0
4	Roller Gates	$12.0
5	Roller Gates	$12.0
5A	Roller Gates	$12.0
6	Roller Gates	$12.0
7	Roller Gates	$12.0
8	Roller Gates	$12.0
9	Roller Gates	$12.0
10	Roller Gates	$12.0
11	Tainter Gates	$12.0
	Roller Gates	$12.0
12	Tainter Gates	$12.0
	Roller Gates	$12.0
13	Tainter Gates	$12.0
	Roller Gates	$12.0
16	Tainter Gates	$12.0
	Roller Gates	$12.0
	Fixed Crest Dam	$12.0
17	Tainter Gates	$12.0
	Roller Gates	$12.0
18	Tainter Gates	$12.0
	Roller Gates	$12.0
20	Tainter Gates	$12.0
	Roller Gates	$12.0
21	Tainter Gates	$12.0
	Roller Gates	$12.0
24	Concrete Dam	$12.0
25	Concrete Dam	$12.0
Mel Price	Concrete Dam	$12.0
TJ O'Brien	Fixed Dam	$12.0
	Control Dam	$12.0
Peoria	Regulating Weir	$12.0
	Navigable Dam	$12.0
LaGrange	Regulating Weir	$12.0
	Navigable Dam	$12.0

POOL6.XLS

7. <u>Number of Components</u>.

UNDERSEEPAGE FOR POOL CONTROL DAMS	
District	Pool Control Dams
St. Paul	23
Rock Island	10
St. Louis	3
TOTAL	36

2. <u>Site Selection</u>. Because of the large number of locks and dams in the Rock Island District, only selected sites were analyzed. Selection was made so that two sites with similar structural and foundation conditions were not both analyzed. Locks and Dams Nos. 14, 15, 16, 19, 22, Lockport, Brandon Road, Dresden Island, Marseilles, and Starved Rock are founded on rock. There are no earth dikes at Locks and Dams Nos. 15, 19, TJ O'Brien, Marseilles, Starved Rock, and Peoria.

UNDERSEEPAGE FOR EARTH EMBANKMENTS			
Sites Investigated		Sites in the Same Bracket	
Lock &Dam	Component	Lock& Dam	Component
11	Non-Overflow Dike	11	Storage Yard
		16	Storage Yard
		20	Storage Yard
		24	Storage Yard
		25	Storage Yard
		27	East Earth Embankment
			West Earth Embankment
12	Non-Overflow Dike	12	Storage Yard
13	Non-Overflow Dike	13	Storage Yard
		17	Non-Overflow Dike
			Storage Yard

UNDERSEEPAGE FOR EARTH EMBANKMENTS			
Sites Investigated		Sites in the Same Bracket	
Lock &Dam	Component	Lock& Dam	Component
13	Non-Overflow Dike	21	Non-Overflow Dike
			Storage Yard
18	Non-Overflow Dike	18	Storage Yard
		LaGrange	Non-Overflow Dike
24	Overflow Dike	24	Sny Levee
		25	Auxiliary Lock Closure Dam

3. Important Deterministic and Random Variables.

UNDERSEEPAGE FOR EARTH EMBANKMENTS															
Lock and Dam	Component	Random Variables													
				Top Blanket Thickness				Foundation							
		Seepage Length (feet)		Dbl (feet)		Dbr (feet)		Aquifer Thickness (feet)		Kf/Kbl		Kf/Kbr		Critical Gradient, ic	
		μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
LSAF	NSP Dike	Assumed β greater than 4 by engineering judgement.													
2	Non-Overflow Dike	800	300	7	2	7	2	35	15	300	150	300	150	0.92	0.37
3	Non-Overflow Dike													0.92	0.37
	Spot Dikes	Assume Pr(u)=10% in 50 years, based on Design Analysis Report and present proposal for Rehab.													
4	Non-Overflow Dike	800	400	7	2	7	2	50	25	200	100	200	100	0.92	0.37
5	Non-Overflow Dike	800	400	7	2	7	2	50	25	50	25	50	25	0.92	0.37
5A	Non-Overflow Dike	800	400	7	2	7	2	50	25	300	150	300	150	0.92	0.37
6	Non-Overflow Dike	800	400	7	2	7	2	50	25	100	50	100	50	0.92	0.37
7	Non-Overflow Dike	800	400	7	2	7	2	50	25	100	50	100	50	0.92	0.37
	Overflow Dike	Assumed β greater than 4 by engineering judgement.													
8	Non-Overflow Dike	800	400	7	2	7	2	50	25	100	50	100	50	0.92	0.37
	Overflow Dike	Assumed β greater than 4 by engineering judgement.													
9	Non-Overflow Dike	800	400	7	2	7	2	50	25	50	25	50	25	0.92	0.37
	Overflow Dike	Assumed β greater than 4 by engineering judgement.													
10	Non-Overflow Dike														
11	Non-Overflow Dike			6	2.4	6	2.4	188	37.6	200	100	800	400		
12	Non-Overflow Dike			13	5.2	13	5.2	125	25	200	100	800	400		
13	Non-Overflow Dike			8	3.2	8	3.2	100	20	200	100	800	400		
18	Non-Overflow Dike			6	2.4	6	2.4	127	25.4	200	100	800	400		
24	Overflow Dike									5	4	5	4		
25	Overflow Dike									5	4	5	4		
	Sandy Slough Dike	Assumed β = 2 by engineering judgement.													
MP	Sheet Pile Cell	Assumed β greater than 4 by engineering judgement.													
	Overflow Dike	Assumed β greater than 4 by engineering judgement.													
LaGrange	Non-Overflow Dike	Assumed β greater than 5 by engineering judgement.													

EARTH3W.XLS

4. Weibull Distribution Parameters.

UNDERSEEPAGE FOR EARTH EMBANKMENTS							
Cumulative Distribution Function		Current			Rehabilitated		
Lock and Dam	Component	b	α	ν	b	α	ν
LSAF	NSP Dike	1.2	360	0	1.2	360	0
2	Non-Overflow Dike	1.2	360	0	1.2	360	0
3	Non-Overflow Dike	1.2	360	0	1.2	360	0
	Spot Dikes	1.2	360	-5.19	1.2	360	0
4	Non-Overflow Dike	1.2	360	0	1.2	360	0
5	Non-Overflow Dike	1.2	360	0	1.2	360	0
5A	Non-Overflow Dike	1.2	360	0	1.2	360	0
6	Non-Overflow Dike	1.2	360	0	1.2	360	0
7	Non-Overflow Dike	1.2	360	0	1.2	360	0
	Overflow Dike	1.2	360	0	1.2	360	0
8	Non-Overflow Dike	1.2	360	0	1.2	360	0
	Overflow Dike	1.2	360	0	1.2	360	0
9	Non-Overflow Dike	1.2	360	0	1.2	360	0
	Overflow Dike	1.2	360	0	1.2	360	0
10	Non-Overflow Dike	1.2	360	0	1.2	360	0
11	Non-Overflow Dike	1.2	360	0	1.2	360	0
12	Non-Overflow Dike	1.2	360	-4.51	1.2	360	0
13	Non-Overflow Dike	1.2	360	-42.2	1.2	360	0
18	Non-Overflow Dike	1.2	360	-173.64	1.2	360	0
24	Overflow Dike	1.2	360	0	1.2	360	0
25	Sandy Slough Dike	1.2	360	-15.53	1.2	360	0
	Overflow Dike	1.2	360	0	1.2	360	0
MP	Sheet Pile Cell	1.2	360	0	1.2	360	0
	Overflow Dike	1.2	360	0	1.2	360	0
LaGrange	Non-Overflow Dike	1.2	360	0	1.2	360	0

EARTH4.XLS

5. Consequences.

UNDERSEEPAGE FOR EARTH EMBANKMENTS							
Lock and Dam	Component	Medium Level of Consequences (MC)			High Level of Consequences (HC)		
		P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Repair Costs (million)
LSAF	NSP Dike	0.9	1	$0.0	0.1	20	$0.00
2	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$4.20
3	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$5.20
	Spot Dikes	0.9	1	$0.5	0.1	20	$14.00
4	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$6.50
5	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$16.40
5A	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$18.00
6	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$5.00
7	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$7.10
	Overflow Dike	0.9	1	$0.5	0.1	20	$5.00
8	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$14.00
	Overflow Dike	0.9	1	$0.5	0.1	20	$5.00
9	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$8.20
	Overflow Dike	0.9	1	$0.5	0.1	20	$5.00
10	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$5.40
11	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$4.72
	Storage Yard	0.9	1	$0.5	0.1	20	$2.16
12	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$6.64
	Storage Yard	0.9	1	$0.5	0.1	20	$2.16
13	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$3.632
	Storage Yard	0.9	1	$0.5	0.1	20	$2.16
16	Storage Yard	0.9	1	$0.5	0.1	20	$2.332
17	Non-Overfow Dike	0.9	1	$0.5	0.1	20	$2.184
	Storage Yard	0.9	1	$0.5	0.1	20	$2.16
18	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$3.76
	Storage Yard	0.9	1	$0.5	0.1	20	$2.156
20	Storage Yard	0.9	1	$0.5	0.1	20	$2.18
21	Storage Yard	0.9	1	$0.5	0.1	20	$2.16
24	Overflow Dike	0.9	1	$0.5	0.1	20	$10.00
25	Sandy Slough Dike	0.9	1	$0.5	0.1	20	$5.80
	Overflow Dike	0.9	1	$0.5	0.1	20	$10.00
MP	Sheet Pile Cell	0.9	1	$0.5	0.1	20	$2.25
	Overflow Dike	0.9	1	$0.5	0.1	20	$3.60
LaGrange	Non-Overflow Dike	0.9	1	$0.5	0.1	20	$2.312

EARTH5.XLS

6. Cost of Rehabilitation.

UNDERSEEPAGE FOR EARTH EMBANKMENTS		
Lock and Dam	Component	Rehabilitation Cost (millions)
LSAF	NSP Dike	$0.00
2	Non-Overflow Dike	$2.20
3	Non-Overflow Dike	$3.20
	Spot Dikes	$13.13
4	Non-Overflow Dike	$4.50
5	Non-Overflow Dike	$14.40
5A	Non-Overflow Dike	$16.00
6	Non-Overflow Dike	$3.00
7	Non-Overflow Dike	$5.10
	Overflow Dike	$3.00
8	Non-Overflow Dike	$12.00
	Overflow Dike	$3.00
9	Non-Overflow Dike	$6.20
	Overflow Dike	$3.00
10	Non-Overflow Dike	$3.40
11	Non-Overflow Dike	$2.74
	Storage Yard	$0.16
12	Non-Overflow Dike	$4.64
	Storage Yard	$0.16
13	Non-Overflow Dike	$1.63
	Storage Yard	$0.16
16	Storage Yard	$0.33
17	Non-Overflow Dike	$1.48
	Storage Yard	$0.16
18	Non-Overflow Dike	$1.76
	Storage Yard	$0.16
20	Storage Yard	$0.18
21	Storage Yard	$0.16
24	Overflow Dike	$5.00
25	Sandy Slough Dike	$3.80
	Overflow Dike	$5.00
MP	Sheetpile Cell	$0.25
	Overflow Dike	$1.60
LaGrange	Non-Overflow Dike	$0.31

EARTH6.XLS

7. Number of Components.

UNDERSEEPAGE FOR EARTH EMBANKMENTS	
District	Earth Embankments
St. Paul	15
Rock Island	26
St. Louis	11
TOTAL	52

APPENDIX F - Scour Protection Downstream of the Dam (Riprap)

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UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

SCOUR PROTECTION DOWNSTREAM OF THE DAM

1. Model Description. Determination of present conditions/reliability was made by determining at what future date the probability of unsatisfactory performance would be 50%. These determinations were made using the present condition of the scour protection and engineering judgement.

2. Site Selection. This section includes scour protection downstream of the dam at all sites.

3. Important Deterministic and Random Variables.

SCOUR PROTECTION DOWNSTREAM	
Lock and Dam	Random Variables
USAF	Assumed β greater than 5 by engineering judgement.
LSAF	Assumed β greater than 5 by engineering judgement.
1	Assumed β greater than 5 by engineering judgement.
2	Assumed β greater than 5 by engineering judgement.
3	Assumed β greater than 5 by engineering judgement.
4	Assumed β greater than 5 by engineering judgement.
5	Assumed β greater than 5 by engineering judgement.
5A	Assumed β greater than 5 by engineering judgement.
6	Assumed β greater than 5 by engineering judgement.
7	Assumed β greater than 5 by engineering judgement.
8	Assumed β greater than 5 by engineering judgement.
9	Assumed β greater than 5 by engineering judgement.
10	Assumed β greater than 5 by engineering judgement.
11	Assumed β greater than 5 by engineering judgement.
12	Assumed β greater than 5 by engineering judgement.
13	Assumed β greater than 5 by engineering judgement.
14	Assumed β greater than 5 by engineering judgement.
15	Assumed β greater than 5 by engineering judgement.
16	Assumed β greater than 5 by engineering judgement.
17	Assumed β greater than 5 by engineering judgement.
18	Assumed β greater than 5 by engineering judgement.
19	Assumed β greater than 5 by engineering judgement.
20	Assumed β greater than 5 by engineering judgement.
21	Assumed β greater than 5 by engineering judgement.
22	Assumed β greater than 5 by engineering judgement.
24	Assumed β greater than 5 by engineering judgement.
25	Assumed β greater than 5 by engineering judgement.
Mel Price	Assumed Pr(u) = 50% in 100 years.
27	Assumed β greater than 5 by engineering judgement.
TJ O'Brien	Assumed β greater than 5 by engineering judgement.
Lockport	Assumed β greater than 5 by engineering judgement.
Brandon Road	Assumed β greater than 5 by engineering judgement.
Dresden Island	Assumed β greater than 5 by engineering judgement.
Marseilles	Assumed β greater than 5 by engineering judgement.
Starved Rock	Assumed β greater than 5 by engineering judgement.
Peoria	Assumed β greater than 5 by engineering judgement.
LaGrange	Assumed β greater than 5 by engineering judgement.

SCOUR3.XLS

4. Weibull Distribution Parameters.

SCOUR PROTECTION DOWNSTREAM						
	Current			Rehabilitated		
Lock and Dam	b	α	ν	b	α	ν
USAF	0.9	64	0	0.9	64	0
LSAF	0.9	64	0	0.9	64	0
1	0.9	64	0	0.9	64	0
2	0.9	64	0	0.9	64	0
3	0.9	64	0	0.9	64	0
4	0.9	64	0	0.9	64	0
5	0.9	64	0	0.9	64	0
5A	0.9	64	0	0.9	64	0
6	0.9	64	0	0.9	64	0
7	0.9	64	0	0.9	64	0
8	0.9	64	0	0.9	64	0
9	0.9	64	0	0.9	64	0
10	0.9	64	0	0.9	64	0
11	0.9	64	0	0.9	64	0
12	0.9	64	0	0.9	64	0
13	0.9	64	0	0.9	64	0
14	0.9	64	0	0.9	64	0
15	0.9	64	0	0.9	64	0
16	0.9	64	0	0.9	64	0
17	0.9	64	0	0.9	64	0
18	0.9	64	0	0.9	64	0
19	0.9	64	0	0.9	64	0
20	0.9	64	0	0.9	64	0
21	0.9	64	0	0.9	64	0
22	0.9	64	0	0.9	64	0
24	0.9	64	0	0.9	64	0
25	0.9	64	0	0.9	64	0
Mel Price	0.9	64	58.0	0.9	64	0
27	0.9	64	0	0.9	64	0
TJ O'Brien	0.9	64	0	0.9	64	0
Lockport	0.9	64	0	0.9	64	0
Brandon Road	0.9	64	0	0.9	64	0
Dresden Island	0.9	64	0	0.9	64	0
Marseilles	0.9	64	0	0.9	64	0
Starved Rock	0.9	64	0	0.9	64	0
Peoria	0.9	64	0	0.9	64	0
LaGrange	0.9	64	0	0.9	64	0

SCOUR4.XLS

5. Consequences.

SCOUR PROTECTION DOWNSTREAM						
Lock and Dam	Medium Level of Consequences (MC)			High Level of Consequences (HC)		
	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Repair Costs (million)
USAF	0.9	0	$0.5	0.1	0	$3.0
LSAF	0.9	0	$0.5	0.1	0	$3.0
1	0.9	0	$0.5	0.1	0	$3.0
2	0.9	0	$0.5	0.1	0	$3.0
3	0.9	0	$0.5	0.1	0	$3.0
4	0.9	0	$0.5	0.1	0	$3.0
5	0.9	0	$0.5	0.1	0	$3.0
5A	0.9	0	$0.5	0.1	0	$3.0
6	0.9	0	$0.5	0.1	0	$3.0
7	0.9	0	$0.5	0.1	0	$3.0
8	0.9	0	$0.5	0.1	0	$3.0
9	0.9	0	$0.5	0.1	0	$3.0
10	0.9	0	$0.5	0.1	0	$3.0
11	0.9	0	$0.5	0.1	0	$3.0
12	0.9	0	$0.5	0.1	0	$3.0
13	0.9	0	$0.5	0.1	0	$3.0
14	0.9	0	$0.5	0.1	0	$3.0
15	0.9	0	$0.5	0.1	0	$3.0
16	0.9	0	$0.5	0.1	0	$3.0
17	0.9	0	$0.5	0.1	0	$3.0
18	0.9	0	$0.5	0.1	0	$3.0
19	0.9	0	$0.5	0.1	0	$3.0
20	0.9	0	$0.5	0.1	0	$3.0
21	0.9	0	$0.5	0.1	0	$3.0
22	0.9	0	$0.5	0.1	0	$3.0
24	0.9	0	$0.5	0.1	0	$3.0
25	0.9	0	$0.5	0.1	0	$3.0
Mel Price	0.9	0	$0.5	0.1	0	$3.0
27	0.9	0	$0.5	0.1	0	$3.0
TJ O'Brien	0.9	0	$0.5	0.1	0	$3.0
Lockport	0.9	0	$0.5	0.1	0	$3.0
Brandon Road	0.9	0	$0.5	0.1	0	$3.0
Dresden Island	0.9	0	$0.5	0.1	0	$3.0
Marseilles	0.9	0	$0.5	0.1	0	$3.0
Starved Rock	0.9	0	$0.5	0.1	0	$3.0
Peoria	0.9	0	$0.5	0.1	0	$3.0
LaGrange	0.9	0	$0.5	0.1	0	$3.0

SCOUR5.XLS

6. Cost of Rehabilitation.

SCOUR PROTECTION DOWNSTREAM	
Lock and Dam	Rehabilitation Cost (millions)
USAF	$2.5
LSAF	$2.5
1	$2.5
2	$2.5
3	$2.5
4	$2.5
5	$2.5
5A	$2.5
6	$2.5
7	$2.5
8	$2.5
9	$2.5
10	$2.5
11	$2.5
12	$2.5
13	$2.5
14	$2.5
15	$2.5
16	$2.5
17	$2.5
18	$2.5
19	$2.5
20	$2.5
21	$2.5
22	$2.5
24	$2.5
25	$2.5
Mel Price	$2.5
27	$2.5
TJ O'Brien	$2.5
Lockport	$2.5
Brandon Road	$2.5
Dresden Island	$2.5
Marseilles	$2.5
Starved Rock	$2.5
Peoria	$2.5
LaGrange	$2.5

SCOUR6.XLS

7. Number of Components.

SCOUR PROTECTION DOWNSTREAM	
District	Area Downstream of Dam
St. Paul	18
Rock Island	13
St. Louis	4
TOTAL	35

APPENDIX G - Lockwall Concrete

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UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

LOCKWALL CONCRETE

1. Model Description. The reliability index of lockwall concrete was determined in the Rock Island District using the WES model and in the St. Paul and St. Louis Districts using the geotechnical time reliability model.

a. WES Model. A reliability model of concrete deterioration of lockwall concrete due to freeze-thaw action and abrasion was developed by the Waterways Experiment Station. This model calculates the reliability of the lockwall based on a barge in the lock hanging up on uneven lockwall surfaces. The following deterministic parameters are required as input: year under consideration, first year in operation, width above pool, width below pool, total number of lockages, lock length, top of lockwall, pool, mid-pool, tailwater, barge height, and barge depth. Seventeen variables were chosen to be treated as random variables. They are: tow weight, tow velocity, tow angle, structural interaction pressure constant, effective plate thickness, vertical impact distance, contact half-length, stress attenuation coefficient, uniaxial tensile strength, number of surface temperature cycles, strength degradation exponent, degradation coefficient, slope of the depth of critical saturation with time, dwell time, number of impacts per lockage, loss ratio, and coefficient of friction.

b. Geotechnical Time Reliability Model. Determination of present conditions/reliability was made by determining at what future date the probability of unsatisfactory performance would be 50%. These determinations were made using the present condition of the lockwall and engineering judgement.

2. Site Selection. Because of the large number of locks and dams in the Rock Island District, only selected sites were analyzed. Selection was made so that two sites with similar lockage patterns, environmental conditions, and concrete properties were not both analyzed. Locks and Dams Nos. 13, 19, 22, and Lockport have been rehabilitated with air-entrained concrete. TJ O'Brien lock and dam is constructed of diaphragm sheetpile cells.

LOCKWALL CONCRETE			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
1	Lockwall Concrete	Melvin Price	Main Lock Landwall
			Main Lock Riverwall
			Auxiliary Lock Landwall
			Auxiliary Lock Riverwall
		27	Eastwall
			I-Wall Main Lock Side
			I-Wall Auxiliary Lock Side
			Westwall
11	Lockwall Concrete		
12	Lockwall Concrete		
13	Lockwall Concrete	19	Lockwall Concrete
		22	Lockwall Concrete
		Lockport	Lockwall Concrete
		Brandon Road	Lockwall Concrete
		Dresden Island	Lockwall Concrete

LOCKWALL CONCRETE			
Sites Investigated		Sites in the Same Bracket	
Lock and Dam	Component	Lock and Dam	Component
13	Lockwall Concrete	Marseilles	Lockwall Concrete
		Starved Rock	Lockwall Concrete
		Peoria	Lockwall Concrete
15	Lockwall Concrete	14	Lockwall Concrete
		16	Lockwall Concrete
17	Lockwall Concrete		
18	Lockwall Concrete		
20	Lockwall Concrete	21	Lockwall Concrete
La Grange	Lockwall Concrete		

3. Important Deterministic and Random Variables.

LOCKWALL CONCRETE		
Lock and Dam	Component	Random Variables
USAF	Lock Chamber Concrete	Note 1
LSAF	Lock Chamber Concrete	Pr(u) = 50% in 50 years by engineering judgement
1	Lock Chamber Concrete	Note 1
2	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
3	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
4	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
5	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
5A	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
6	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
7	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
8	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
9	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
10	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement
24	Lock Chamber Concrete	Pr(u) = 50% in 20 years by engineering judgement
25	Lock Chamber Concrete	Pr(u) = 50% in 40 years by engineering judgement

LOCK3.XLS

[1]Air-entrained concrete. Assume no maintenance for next 50 years.

LOCKWALL CONCRETE										
Lock and Dam	Random Variables									
	W (kips)		f'_o (psi)		nt_o		t (hr)		r	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
11	17,500	5100	593.0	118.6	18.0	6.0	0.30	0.099	0.100	0.031
12	17,000	5100	593.0	118.6	10.0	3.0	0.26	0.086	0.100	0.031
13	Lock has been resurfaced with air-entrained concrete. Assume no maintenance for next 50 years.									
15	16,500	5100	536.0	107.2	11.6	3.0	0.25	0.083	0.100	0.031
17	17,500	5100	540.0	108.0	9.6	2.5	0.18	0.059	0.100	0.031
18	17,500	5100	543.0	108.6	7.7	2.0	0.18	0.059	0.500	0.031
20	16,500	5100	543.0	108.6	7.7	2.0	0.17	0.056	0.100	0.031
LaGrange	18,000	5100	589.0	117.8	1.9	0.6	0.46	0.149	0.200	0.031

LOCK3A.XLS

LOCKWALL CONCRETE												
Lock and Dam	Random Variables											
	V (ft/s)		θ (deg)		μ		λ (in)		s		h (in)	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
11,12,15,17,18 20, LaGrange	2.29	0.79	0.30	0.17	0.430	0.065	1.032	0.035	0.19919	0.0076	24.00	6.93

LOCKWALL CONCRETE												
Lock and Dam	Random Variables											
	a (in)		b		t_{eff} (in)		P_m (ksi)		C		ipl	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
11,12,15,17,18 20, LaGrange	2.0	0.6	0.82	0.044	1.19	0.216	13.7	4.93	0.002	0.00056	1.3	0.754

LOCK3B.XLS

LOCKWALL CONCRETE						
	Deterministic Variables					
Lock and Dam	Y_f (year)	Y_c (year)	w_1 (feet)	w_2 (feet)	Total # Lockages	Lock Length (feet)
11	1937	1995	2.00	2.00	158,688	600
12	1938	1995	2.00	2.00	182,866	600
15	1934	1995	1.12	1.44	195,688	600
17	1939	1995	2.00	2.00	268,561	600
18	1937	1995	2.00	2.00	271,734	600
20	1936	1995	2.00	2.00	280,546	600
LaGrange	1939	1995	2.00	2.00	284,640	600

LOCKWALL CONCRETE						
	Deterministic Variables					
Lock and Dam	Wall Top (feet)	Upper Pool (feet)	Lower Pool (feet)	Mid-pool (feet)	Height (feet)	Barge Depth (feet)
11	612.5	603.0	592.0	587.5	15.00	12.0
12	592.0	593.0	583.0	587.5	15.50	12.0
15	561.0	557.0	545.0	553.0	15.25	12.0
17	536.0	536.0	528.0	532.0	16.00	12.0
18	528.0	528.0	518.0	523.1	14.40	12.0
20	480.0	480.0	470.0	475.0	13.00	12.0
LaGrange	429.0	429.0	419.0	424.0	6.00	12.0

LOCK3C.XLS

4. Weibull Distribution Parameters.

LOCKWALL CONCRETE							
Lock and Dam	Component	Current			Rehabilitated		
		b	α	ν	b	α	ν
USAF	Lock Chamber Conc.	1.2	50	Note 1	1.2	50	32
LSAF	Lock Chamber Conc.	1.2	50	13.2	1.2	50	32
1	Lock Chamber Conc.	1.2	50	Note 1	1.2	50	32
2	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
3	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
4	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
5	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
5A	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
6	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
7	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
8	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
9	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
10	Lock Chamber Conc.	1.2	50	3.2	1.2	50	32
11	Lock Chamber Conc.	1.2	50	0	1.2	50	32
12	Lock Chamber Conc.	1.2	50	0	1.2	50	32
13	Lock Chamber Conc.	1.2	50	50	1.2	50	32
15	Lock Chamber Conc.	1.2	50	0	1.2	50	32
17	Lock Chamber Conc.	1.2	50	0	1.2	50	32
18	Lock Chamber Conc.	1.2	50	0	1.2	50	32
20	Lock Chamber Conc.	1.2	50	0	1.2	50	32
24	Lock Chamber Conc.	1.2	50	-16.94	1.2	50	32
25	Lock Chamber Conc.	1.2	50	3.0	1.2	50	32
LaGrange	Lock Chamber Conc.	1.2	50	-0.596	1.2	50	32

LOCK4.XLS

[1]Air-entrained concrete. Assume no maintenance for the next 50 years.

5. <u>Consequences</u>.

LOCKWALL CONCRETE									
		Medium Level of Consequences (MC)			High Level of Consequences (HC)				
							Slowdown[2]		
Lock and Dam	Component	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Effect on Lockage Cycle	Slowdown Duration (days)	Repair Costs (million)
USAF	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
LSAF	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
2	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
3	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
4	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
5	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
5A	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
6	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
7	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
8	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
9	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
10	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
11	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0
12	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0
13	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0
15	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0
17	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0
18	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0
20	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0
24	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
25	Lock Chamber Conc.	0.9	30 +75[1]	$0.5	0.1	30 +75[1]	0	0	$9.0
LaGrange	Lock Chamber Conc.	0.9	75	$0.5	0.1	75	0	0	$9.0

[1] 30 days initial + 75 days during winter shutdown or low traffic period.

[2] Slowdown time is added to navigation downtime.

LOCK5.XLS

6. Cost of Rehabilitation.

LOCKWALL CONCRETE		
Lock and Dam	Component	Rehabilitation Cost (millions)
USAF	Lock Chamber Conc.	$2.7
LSAF	Lock Chamber Conc.	$2.7
1	Lock Chamber Conc.	$2.7
2	Lock Chamber Conc.	$2.7
3	Lock Chamber Conc.	$2.7
4	Lock Chamber Conc.	$2.7
5	Lock Chamber Conc.	$2.7
5A	Lock Chamber Conc.	$2.7
6	Lock Chamber Conc.	$2.7
7	Lock Chamber Conc.	$2.7
8	Lock Chamber Conc.	$2.7
9	Lock Chamber Conc.	$2.7
10	Lock Chamber Conc.	$2.7
11	Lock Chamber Conc.	$2.7
12	Lock Chamber Conc.	$2.7
13	Lock Chamber Conc.	$2.7
15	Lock Chamber Conc.	$2.7
17	Lock Chamber Conc.	$2.7
18	Lock Chamber Conc.	$2.7
20	Lock Chamber Conc.	$2.7
24	Lock Chamber Conc.	$2.7
25	Lock Chamber Conc.	$2.7
LaGrange	Lock Chamber Conc.	$2.7

LOCK6.XLS

7. Number of Components.

LOCKWALL CONCRETE	
District	Concrete Lockwalls
St. Paul	28
Rock Island	46
St. Louis	12
TOTAL	86

APPENDIX H - Dam Pier Bridge Column Concrete

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UMR-IWW NAVIGATION STUDY
OBJECTIVE 2A
FINAL IMPLEMENTATION OF RELIABILITY MODELS
GEOTECHNICAL/MATERIALS MODELS

DAM PIER BRIDGE COLUMN CONCRETE

1. Model Description. Determination of present conditions/reliability was made by determining at what future date the probability of unsatisfactory performance would be 50%. These determinations were made using the present condition of the dam pier bridge columns and engineering judgement.

2. Site Selection. All sites were analyzed.

3. Important Deterministic and Random Variables.

DAM PIER CONCRETE												
Lock and Dam	Random Variables											
	Live Load (kips)		Dead Load (kips)		Impact Load (kips)		f'c (psi)		Crack Spacing (inch)		Eccentricity (inch)	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
USAF	Note 1											
LSAF	Pr(u)=50% in 50 years.											
2	Pr(u)=50											
3	Pr(u)=50% in 40 years.											
4	Pr(u)=50% in 40 years.											
5	Pr(u)=50% in 40 years.											
5A	Pr(u)=50% in 40 years.											
6	Pr(u)=50% in 40 years.											
7	Pr(u)=50% in 40 years.											
8	Pr(u)=50% in 40 years.											
9	Pr(u)=50% in 40 years.											
10	Pr(u)=50% in 40 years.											
11	Pr(u)=50% in 40 years.											
12	Pr(u)=50% in 40 years.											
13	Pr(u)=50% in 40 years.											
14	Pr(u)= 50% in 5 years.											
15	Note 1											
16	Pr(u)=50% in 40 years.											
17	Pr(u)=50% in 40 years.											
18	Pr(u)=50% in 40 years.											
20	Note 1											
21	Pr(u)=50% in 40 years.											
22	Pr(u)=50% in 40 years.											
24	278	55.6	42	4.2	44	.22	6,465	1,329	4.25	0.84	1.0	0.1
25	Assumed β greater than 5 by engineering judgement.											
Melvin Price	Note 1											
Brandon Road	Note 1											
Dresden Island	Note 1											
Marseilles	Note 1											
Starved Rock	Note 1											

PIER3.XLS

[1]Air-entrained concrete. Assume no maintenance for next 50 years.

4. <u>Weibull Distribution Parameters</u>.

DAM PIER CONCRETE						
Lock and Dam	Current			Rehabilitated		
	b	α	ν	b	α	ν
USAF	1.2	73	Note 1	1.2	73	30
LSAF	1.2	73	-3.8	1.2	73	30
2	1.2	73	-13.8	1.2	73	30
3	1.2	73	-13.8	1.2	73	30
4	1.2	73	-13.8	1.2	73	30
5	1.2	73	-13.8	1.2	73	30
5A	1.2	73	-13.8	1.2	73	30
6	1.2	73	-13.8	1.2	73	30
7	1.2	73	-13.8	1.2	73	30
8	1.2	73	-13.8	1.2	73	30
9	1.2	73	-13.8	1.2	73	30
10	1.2	73	-13.8	1.2	73	30
11	1.2	73	-13.8	1.2	73	30
12	1.2	73	-13.8	1.2	73	30
13	1.2	73	-13.8	1.2	73	30
14	1.2	73	-13.8	1.2	73	30
15	1.2	73	Note 1	1.2	73	30
16	1.2	73	-13.8	1.2	73	30
17	1.2	73	-13.8	1.2	73	30
18	1.2	73	-13.8	1.2	73	30
20	1.2	73	Note 1	1.2	73	30
21	1.2	73	-13.8	1.2	73	30
22	1.2	73	-13.8	1.2	73	30
24	1.2	73	-5.67	1.2	73	30
25	1.2	73	30	1.2	73	30
Melvin Price	1.2	73	Note 1	1.2	73	30
Brandon Road	1.2	73	Note 1	1.2	73	30
Dresden Island	1.2	73	Note 1	1.2	73	30
Marseilles	1.2	73	Note 1	1.2	73	30
Starved Rock	1.2	73	Note 1	1.2	73	30
Peoria	1.2	73	Note 1	1.2	73	30
La Grange	1.2	73	Note 1	1.2	73	30

PIER4.XLS

[1]Air-entrained concrete. There are not enough structures with air-entrained concrete to establish a separate Weibull curve. Assume no maintenance for next 50 years.

5. Consequences.

DAM PIER CONCRETE								
Lock and Dam	Medium Level of Consequences (MC)			High Level of Consequences (HC)				
	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Slowdown[2] Effect on Lockage Cycle	Slowdown[2] Slowdown Duration (days)	Repair Costs (million)
LSAF	0.9	0	$0.5	0.1	0	0	0	$10.0
2	0.9	0	$0.5	0.1	0	0	0	$10.0
3	0.9	0	$0.5	0.1	0	0	0	$10.0
4	0.9	0	$0.5	0.1	0	0	0	$10.0
5	0.9	0	$0.5	0.1	0	0	0	$10.0
5A	0.9	0	$0.5	0.1	0	0	0	$10.0
6	0.9	0	$0.5	0.1	0	0	0	$10.0
7	0.9	0	$0.5	0.1	0	0	0	$10.0
8	0.9	0	$0.5	0.1	0	0	0	$10.0
9	0.9	0	$0.5	0.1	0	0	0	$10.0
10	0.9	0	$0.5	0.1	0	0	0	$10.0
11	0.9	0	$0.5	0.1	0	0	0	$10.0
12	0.9	0	$0.5	0.1	0	0	0	$10.0
13	0.9	0	$0.5	0.1	0	0	0	$10.0
14	0.9	0	$0.5	0.1	0	0	0	$10.0

[2] Slowdown time is added to navigation downtime.

DAM PIER CONCRETE								
	Medium Level of Consequences (MC)			High Level of Consequences (HC)				
						Slowdown[2]		
Lock and Dam	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Effect on Lockage Cycle	Slowdown Duration (days)	Repair Costs (million)
16	0.9	0	$0.5	0.1	0	0	0	$10.0
17	0.9	0	$0.5	0.1	0	0	0	$10.0
18	0.9	0	$0.5	0.1	0	0	0	$10.0
21	0.9	0	$0.5	0.1	0	0	0	$10.0
22	0.9	0	$0.5	0.1	0	0	0	$10.0
24	0.9	0	$0.5	0.1	0	0	0	$10.0
25	0.9	0	$0.5	0.1	0	0	0	$10.0
Melvin Price	0.9	0	$0.5	0.1	0	0	0	$10.0

PIER5.XLS

[2]Slowdown time is added to navigation downtime.

6. <u>Cost of Rehabilitation</u>.

DAM PIER CONCRETE	
Lock and Dam	Rehabilitation Cost (millions)
LSAF	$3.4
2	$3.4
3	$3.4
4	$3.4
5	$3.4
5A	$3.4
6	$3.4
7	$3.4
8	$3.4
9	$3.4
10	$3.4
11	$3.4
12	$3.4
13	$3.4
14	$3.4
16	$3.4
17	$3.4
18	$3.4
21	$3.4
22	$3.4
24	$3.4
25	$3.4
Melvin Price	$3.4
Brandon Road	$3.4
Dresden Island	$3.4
Marseilles	$3.4
Starved Rock	$3.4
Peoria	$3.4
La Grange	$3.4

PIER6.XLS

7. Number of Components.

DAM PIER CONCRETE	
District	Concrete Dam Piers
St. Paul	11
Rock Island	18
St. Louis	3
TOTAL	32

APPENDIX I - Concrete Spillway Fixed Crest

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3. <u>Important Deterministic and Random Variables</u>.

CONCRETE SPILLWAY FIXED CREST	
Lock and Dam	Pr(u)=50% (years)
1	50
5A	50
6	50
7	Note 1
10	50
16	50
Dresden Island	50
Starved Rock	50

OVER3.XLS

[1]Air-entrained concrete. Assume no maintenance for next 50 years.

4. Weibull Distribution Parameters.

CONCRETE SPILLWAY FIXED CREST						
Lock and Dam	Current			Rehabilitated		
	b	α	ν	b	α	ν
1	1.2	42	19	1.2	42	45
5A	1.2	42	19	1.2	42	45
6	1.2	42	19	1.2	42	45
7	1.2	42	Note 1	1.2	42	45
10	1.2	42	19	1.2	42	45
16	1.2	50	13.2	1.2	50	45
Dresden Island	1.2	50	13.2	1.2	50	45
Starved Rock	1.2	50	13.2	1.2	50	45

OVER4.XLS

[1]Air-entrained concrete. Assume no maintenance for next 50 years.

5. <u>Consequences</u>.

CONCRETE SPILLWAY FIXED CREST								
	Medium Level of Consequences (MC)			High Level of Consequences (HC)				
						Slowdown[2]		
Lock and Dam	P(MC)	Nav. Down Time (days)	Repair Costs (million)	P(HC)	Nav. Down Time (days)	Effect on Lockage Cycle	Slowdown Duration (days)	Repair Costs (million)
1	0.9	0	$0.5	0.1	0	0	0	$8.0
5A	0.9	0	$0.5	0.1	0	0	0	$8.0
6	0.9	0	$0.5	0.1	0	0	0	$8.0
10	0.9	0	$0.5	0.1	0	0	0	$8.0
16	0.9	0	$0.5	0.1	0	0	0	$3.0
Dresden Island	0.9	0	$0.5	0.1	0	0	0	$3.0
Starved Rock	0.9	0	$0.5	0.1	0	0	0	$3.0

OVER5.XLS

[2]Slowdown time is added to navigation downtime.

6. <u>Cost of Rehabilitation</u>.

CONCRETE SPILLWAY FIXED CREST	
Lock and Dam	Rehabilitation Cost (millions)
1	$2.0
5A	$2.0
6	$2.0
10	$2.0
16	$2.0
Dresden Island	$2.0
Starved Rock	$2.0

OVER6.XLS

7. Number of Components.

CONCRETE SPILLWAY FIXED CREST	
District	Spillways
St. Paul	6
Rock Island	1
St. Louis	0
TOTAL	7

(GEOTRMOD.ECD)

SYSTEM SIGNIFICANT COMPONENTS

ENGINEERING RELIABILITY MODELS REPORT

(A Stand Alone Report Compiling Backup Information)

RELIABILITY MODELS
FOR
MECHANICAL & ELECTRICAL EQUIPMENT

Engineering Divisions

St. Paul, Rock Island and St. Louis Districts

US Army Corps of Engineers

SECTION 1 - Miter Gate Operating Machinery

RELIABILITY ANALYSIS FOR

MITER GATE OPERATING MACHINERY

I. MODEL DESCRIPTION:

This section explains the procedure used to find the Hazard Rates for critical components of the miter gate operating machinery on the Mississippi River and Illinois Waterway. The parameters and quantities that comprise the estimated Hazard Rates for selected sites are given in enclosed Tables. Most of the machinery at the locksites was replaced in the mid-late 80's and is relatively new, therefore the data starts with the systems in a new condition. Original machinery that has not currently been replaced was assumed to be installed in the year 2000.

a. Component Inspection: Site visits and review of the as-built drawings were conducted to assess the current general condition of the new lock operating machinery. The new operating machinery is similar in design and about the same age throughout the Upper Mississippi and Illinois Waterway, therefore machinery data can be referenced for lifespan judgments for a large sample size. The original machinery at the sites that have not had machinery replaced is approximately 60 years old. Inspection of gearing components at these sites was used for comparison purposes in determining the future condition of the new gear teeth. Visual inspection was considered a valid basis of engineering judgment since the major wear items are mostly the open and enclosed gearing components. The condition of the gears was judged by inspecting the gear tooth wear along the contact patterns.

b. Lock Gate Mechanical Items Considered Critical: The mechanical system consists of a large number of different mechanical components operating together, any one of which would fail according to its own unique failure distribution. However, the consequence of different failures vary from minor to major. Also, many parts do not "fail" but progressively get worse such as a plain bushing which exhibits gradual wear until the clearances become objectionable. Only components that could cause an extended unplanned outage were included in this model. The following describes the mechanical lock machinery components.

1) OPEN GEARING: GEARS
-SECTOR/BULL GEAR (critical component)

The sector/bull gears are large enough to require a barge crane to remove and reinstall them. Thus in considering the consequence of failure, however remote, the sector/bull gears have a high consequence due to the time involved in replacement.

There is slight to negligible indications of wear failure (spalling, fretting, pitting) on the sector gears. There is no visual evidence of fatigue failure (cracking at the tooth roots). The sector gears are not likely to fail in the predictable future.

The lack of wear is evidence that they have been properly lubricated and maintained and have adequate design capacity.

2) OPEN GEARING: PINION
-DRIVING PINION (critical component)

There is slight to negligible indications of wear failure (spalling, fretting, tooth deformation) of the miter gate pinions. There is no visual evidence of tooth cracking on any of the miter gate pinions. Miter gate pinion failure is not likely to occur in the predictable future.

Pinions are generally replaced whenever their mating gears are replaced since they wear together. The AGMA gear life analysis for the gear is identical to life analysis of the pinion so the gear and pinion of a pair were assigned the same meanlife, Weibull index, and other life factors. There are a total of four of gear/pinion pairs per lock.

3) ENCLOSED GEARING
-RIGHT ANGLE REDUCER (critical component)

The right angle gear reducers are relatively new and there are no reports documenting failure breakdowns. The gearboxes are large and require a barge mounted or truck mounted crane for removal. There would be significant consequences resulting from long downtimes should a failure occur.

For this analysis, only the last reduction gear set was examined since it carries the largest loads and would be expected to fail or reach unsatisfactory performance before the other reduction gear sets. This gearset rotates 2.7 revolutions for each direction of open and close.

4) STEEL CASTINGS: LINKAGE PARTS
-SECTOR ARM CASTING (critical component)
-SECTOR CONNECTION CASTING (critical component)
-SPRING CASING CASTING (critical component)

The sector arm casting is loaded multidirectionally with varying magnitude as the gate cycles. The spring casing and sector connection are loaded axially at various magnitudes.

There is a high consequence of failure should these cast steel linkage parts crack through their cross section. The connection on the gate would suffer considerable bending damage as the gate strut fell under its own weight. Replacement of these parts generally requires a barge crane. However, the probability of these parts failing by fatigue cracking is remote. It is probably more likely that these parts will fail from a towboat collision with the gate than from fatigue cracking.

5) BRONZE BUSHINGS:

-SECTOR BUSHING
-SECTOR ARM BUSHING
-DRIVING PINION BUSHINGS

These parts will wear gradually and predictably. Though replacement of these parts is difficult, this can be foreseen and scheduled to coincide with other work. The original bushings are in good condition. New bushings could be considered to last as long as the original bushings. These bushings were not considered in the model.

6) GEAR BASES:
-SECTOR BEARING

These large steel castings are cyclically loaded in a reversing direction. Their failure due to fatigue type cracking would be noticeable but would probably not stop the system as the castings have many redundant webs and gussets in case any one crack should occur. They were not considered in the model.

7) FORGED PINS:
-SECTOR PIN
-STRUT PIN
-BUFFER YOKE

These forgings are much less likely to suffer cracking failure than castings. They are not considered in the model.

8) SPRINGS:
-SPRING NEST

If the spring nest failed, the gate strut would still remain together and the system would function. Failure of the springs in the gate strut would not stop the system. Spring failure is not considered critical in the model.

9) PURCHASED COMPONENTS:
-DRIVE MOTORS
-BRAKES
-PILLOWBLOCK BEARINGS

These components are off the shelf type items and are relatively easy to replace. These items for the most part have reliable histories and have never broken down nor been rebuilt. The original purchased components observed appear in good condition. These parts are not considered in the model.

II. SITE SELECTION:

Three lock sites were selected for implementation of the model, Lock 11 and Lock 22 on the Mississippi River, and Dresden Island Lock and Dam on the Illinois Waterway. The miter gate machinery on both rivers is similar in arrangement and design so these sites were chosen as a representative sample of each river. The locks on the Mississippi were grouped according to the average annual number of lockages which increases slightly going downriver.

Lock Modeled	Locks that are similar
Lock 11	Locks 1-10,12-20
Lock 22	Locks 21,24,25,27
Dresden Island	Lockport, Brandon Rd, Marseilles, Starved Rock

III. RELIABILITY ANALYSIS:

There is little direct historical documentation describing failure modes for this type of equipment. The component meanlife and failure distribution must be synthesized from generalized textbook data tables on the subject. These textbook tables are reproduced and included as an attachment to this section. The Reliability model used is for individual components described by a two-parameter Weibull distribution (taken from Shannon and Wilson reference (c) page 2-26) is:

$$R(t) = \exp[-(t/a)^{b}]$$

where:
t = number of years the component has been in service.
a = meanlife of the component; the average number of years the component is expected to function.
b = Weibull index number (Table 7-2).
R(t) = reliability function for the component, probability of satisfactory performance at a given time t.

The hazard rate function h(t) of individual components for a two parameter Weibull distribution (taken from reference (c) page 2-26) is:

$$h(t) = (b/a)*(t/a)^{(b-1)}$$

where:
h(t) = the conditional probability that a component will fail in the next year given that it has not failed up to that point.

Table 7-2 (in Bloch and Geitner reference (d), attached) provides an estimate of the Weibull Index and part meanlife using the descriptive verb for the failure mode. For gear teeth,terms like “fretting” and “scoring” are used to describe the gear teeth wear failure and b=3 for these. Fatigue fracture describes the breakage of the steel castings and for this, b=1.1. Tables A-7 and A-8 (in Greene and Bourne reference (e) attached) also gives an estimate for the meanlife of a given component. The Table A-7 meanlife is weighted by dividing the life by an environmental factor K, from Table A-8 to account for outdoors operation. The textbooks thus give an estimate of the failure distribution for a given part if there is no actual failure history for the part.

The gear design capacities are estimated by the methods of AGMA 2001 and AGMA 6010 (references (g) and (h) as applied by Drago reference (f)). The theoretical number of lifetime cycles for the miter gate gears before pitting failure occurs is estimated to be approximately 2 million cycles for the open gears(based on a Life Factor of 1.1) and 7 million cycles for the enclosed gears(based on a Life factor of 1.0). The life factor for the open gearing was determined by comparing the predicted maximun loaded stress to the design capacity. The loading conditions for the miter gate operating machinery were computed in accordance with USACE Waterway Experiment Station Technical Report 2-651 "Operating Forces on Miter-Type Lock Gates." Bending failure was not considered critical in this analysis since the gears are underloaded in this regard. A load cycle for the miter gate machinery comprised of opening and closing all four gates. The gears were considered to be loaded once-per-cycle.

Meanlife for the part (in actual years) is the calculated number of lifetime cycles divided by the use rate or number of lockage cycles. It is further divided by the environmental factor (K=2) to account for being exposed outdoors. The meanlife for the components is the average number of years the component will be expected to function at the average use rate.

The steel castings in the four-bar linkage are assumed to have a lifespan of 5 million cycles, generally regarded as a life limit for fatigue analysis. Note that Table 7-2 (reference (d) shows infinite life for fatigue fracture by the simplification that anything with more than 16 year life is infinite life.

The final probability of unsatisfactory performance is computed by taking into account the number of items which comprise the component at each lock. For instance, if there are "n" open gearsets which each have a conditional probability of failure h(t)=P then the probability of failure (P_g) for all of the open gearsets is:

$$P_g = 1-(1-P)^n$$

Likewise, if the probability of failure of all the castings is P_c, the probability of failure of the open gears is P_g, and the probability of failure of the enclosed gears is P_e then the total probability of failure is:

$$P_T = 1-[(1-P_c)^n * (1-P_g)^n * (1-P_e)^n]$$

This total probability of failure assumes that there is no correlation between the different components or even among components.

IV. RESULTS OF RELIABILITY ANALYSIS:

a. Three hazard functions are required for three separate conditions; normal operation and maintenance (O&M), enhanced O&M, and rehabilitated. For the mechanical models, enhanced O&M was considered the same as normal O&M since there is little necessary maintenance beyond normal maintenance. Rehabilitation of these mechanical components would mean replacement of the components. For this condition, the rehabilitated hazard function will also be the same as normal O&M since the equipment was new at the start of this analysis. Tables 1,2, and 3 present the hazard function data and probabilities of unsatisfactory performance for the selected sites.

TABLE 1 Lock 11

HAZARD RATES (H(t))OF INDIVIDUAL COMPONENTS

Year	2000	2005	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060
Sector gear/pinion pair	0.0000	0.0001	0.0004	0.0008	0.0015	0.0023	0.0033	0.0045	0.0058	0.0074	0.0091	0.0110	0.0131
Steel Castings	0.0000	0.0032	0.0034	0.0036	0.0037	0.0038	0.0038	0.0039	0.0039	0.0040	0.0040	0.0041	0.0041
Spiral Bevel Reducer	0.0000	0.0000	0.0002	0.0004	0.0007	0.0010	0.0015	0.0020	0.0027	0.0034	0.0042	0.0051	0.0060

PROBABILITY OF UNSATISFACTORY PERFORMANCE - ITEM

Year	2000	2005	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060
Sector gear/pinion pair	0.0000	0.0004	0.0015	0.0033	0.0058	0.0091	0.0130	0.0177	0.0231	0.0292	0.0359	0.0433	0.0514
Steel Castings	0.0000	0.0377	0.0403	0.0419	0.0431	0.0441	0.0449	0.0456	0.0462	0.0467	0.0472	0.0476	0.0480
Spiral Bevel Reducer	0.0000	0.0002	0.0007	0.0015	0.0027	0.0042	0.0060	0.0082	0.0106	0.0135	0.0166	0.0201	0.0238

PROBABILITY OF UNSATISFACTORY PERFORMANCE - SYSTEM

Year	2000	2005	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060
Miter Gate Machinery	0.0000	0.0382	0.0423	0.0465	0.0512	0.0567	0.0630	0.0701	0.0781	0.0870	0.0967	0.1072	0.1185

TABLE 2 Lock 22

HAZARD RATES (H(t))OF INDIVIDUAL COMPONENTS

Year	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044	2049
Sector gear/pinion pair	0.0000	0.0002	0.0007	0.0015	0.0026	0.0041	0.0059	0.0080	0.0104	0.0132	0.0163	0.0197	0.0234
Steel Castings	0.0000	0.0040	0.0042	0.0044	0.0045	0.0046	0.0047	0.0048	0.0049	0.0049	0.0050	0.0050	0.0051
Spiral Bevel Reducer	0.0000	0.0001	0.0003	0.0007	0.0012	0.0019	0.0027	0.0037	0.0048	0.0060	0.0075	0.0090	0.0107

PROBABILITY OF UNSATISFACTORY PERFORMANCE - ITEM

Year	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044	2049
Sector gear/pinion pair	0.0000	0.0007	0.0026	0.0058	0.0104	0.0162	0.0232	0.0315	0.0410	0.0516	0.0635	0.0764	0.0904
Steel Castings	0.0000	0.0464	0.0497	0.0517	0.0531	0.0543	0.0553	0.0561	0.0568	0.0575	0.0581	0.0586	0.0591
Spiral Bevel Reducer	0.0000	0.0003	0.0012	0.0027	0.0048	0.0074	0.0107	0.0145	0.0190	0.0240	0.0295	0.0356	0.0423

PROBABILITY OF UNSATISFACTORY PERFORMANCE - SYSTEM

Year	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044	2049
Miter Gate Machinery	0.0000	0.0473	0.0533	0.0597	0.0674	0.0765	0.0871	0.0991	0.1126	0.1276	0.1439	0.1615	0.1804

TABLE 3 Dresden Island Lock

HAZARD RATES (H(t))OF INDIVIDUAL COMPONENTS

Year	1984	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044
Sector gear/pinion pair	0.0000	0.0000	0.0002	0.0003	0.0006	0.0010	0.0014	0.0019	0.0025	0.0031	0.0039	0.0047	0.0056
Steel Castings	0.0000	0.0023	0.0025	0.0026	0.0027	0.0027	0.0028	0.0028	0.0029	0.0029	0.0029	0.0030	0.0030
Gear Reducer	0.0000	0.0000	0.0001	0.0002	0.0003	0.0004	0.0006	0.0009	0.0011	0.0014	0.0018	0.0021	0.0026

PROBABILITY OF UNSATISFACTORY PERFORMANCE - ITEM

Year	1984	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044
Sector gear/pinion pair	0.0000	0.0002	0.0006	0.0014	0.0025	0.0039	0.0056	0.0076	0.0099	0.0125	0.0154	0.0186	0.0221
Steel Castings	0.0000	0.0277	0.0296	0.0308	0.0317	0.0324	0.0330	0.0335	0.0340	0.0343	0.0347	0.0350	0.0353
Gear Reducer	0.0000	0.0001	0.0003	0.0006	0.0011	0.0018	0.0026	0.0035	0.0045	0.0057	0.0071	0.0086	0.0102

PROBABILITY OF UNSATISFACTORY PERFORMANCE - SYSTEM

Year	1984	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044
Miter Gate Machinery	0.0000	0.0279	0.0305	0.0328	0.0352	0.0379	0.0408	0.0442	0.0478	0.0519	0.0563	0.0611	0.0663

The reliability model predicts relatively low hazard rates and probabilities of unsatisfactory performance for the miter gate operating machinery components.

V. CONSEQUENCE TABULATION

The consequences of three degrees of unsatisfactory performance are presented in Table 4. The navigation downtime is based solely on engineering judgment and is estimated by the expected repairs required to the components. The low level consequences were considered to result from a minor repair performed by lock personnel. The medium level consequences were considered to result from a repair that might require a scheduled shutdown of the lock. High level consequences were considered to result from catastrophic failure that would require major repair or replacement of an item. The estimated costs reflect the level of required repairs. It was assumed that lock personnel or hired labor forces would perform the repairs.

TABLE 4 Consequences Table

COMPONENT	LOW LEVEL OF CONSEQUENCES (LC)			MEDIUM LEVEL OF CONSEQUENCES (MC)			HIGH LEVEL OF CONSEQUENCES (HC)		
	P(LC)	NAVIGATION DOWN TIME (Days)	ESTIMATED REPAIR COST	P(MC)	NAVIGATION DOWN TIME (Days)	ESTIMATED REPAIR COST	P(HC)	NAVIGATION DOWN TIME (Days)	ESTIMATED REPAIR COST
Sector gear/Pinion Pair	0.9	0.08	$2,500	0.09	0.5	$10,000	0.01	3.0	$50,000
Steel Castings	0.9	0.08	$2,500	0.09	0.3	$10,000	0.01	3.0	$50,000
Gear Reducer	0.9	0.08	$2,500	0.09	0.3	$10,000	0.01	2.0	$50,000

VI. REHABILITATION COSTS

As noted previously, rehabilitation of the components would involve replacement of the items. Replacement of lock miter gate machinery has been performed on several past rehabilitation projects. The cost of replacement of each gate machinery is approximately $283,000 or $1,132,000 per site. Of the $283,000 approximately $70,000 is the cost of the gearbox, $100,000 is the cost of the sector/pinion pair and $30,000 is the cost of the castings/fabrications.

VII. REFERENCES

a. Operating Forces on Miter-Type Lock Gates (Technical Report 2-651 June 1964 USACE Waterworks Experiment Station, Vicksburg, MS)

b. CECW-ED ETL 1110-2-532 Reliability Assessment of Navigation Structures.

c. The Probability Models for Geotechnical Aspects of Navigational Structures, January 1994, Shannon and Wilson Inc, St. Louis, Missouri.

d. Practical Machinery Management for Process Plants (Bloch, Geitner). Gulf Publishing Company, Houston, Texas.

e. Reliability Technology (Greene, Bourne) John Wiley & Sons, New York, NY

f. Fundamentals of Gear Design (Drago, R.L.) Butterworths

g. Fundamental Rating Factors and Calculation Methods for Involute Spur and Helical Involute Gear Teeth (AGMA Standard 2001-1988)

h. Standard for Spur, Helical, Herringbone, and Bevel Enclosed Drives (AGMA Standard 6010-1988)

SECTION 2 - Lock (Tainter) Valve Operating Machinery

2) PURCHASED COMPONENTS:
- -DRIVE MOTORS
- -BRAKES
- -PILLOWBLOCK BEARINGS
- -CABLES

These components are off the shelf type items and are relatively easy to replace. These items for the most part have reliable histories and have never broken down nor been rebuilt. The original purchased components observed appear in good condition. These parts are not considered in the model.

c. Slide Valve Mechanical Items Considered Critical: The slide valve hoist is installed at five locks on the Illinois Waterway and consists of a self contained electric/hydraulic power unit with a vertically mounted hydraulic cylinder. Only components that could cause an extended unplanned outage were included in this model. The following describes the slide valve mechanical components.

1) HYDRAULIC CYLINDER (critical component)

The hydraulic cylinders are large and require a truck or barge mounted crane for removal. Other than some hydraulic oil leakage, there have been no reports of cylinder breakdowns.

2) POWER UNIT
- -MOTOR
- -PUMP
- -VALVES
- -PIPING

The hydraulic power units are well maintained. These items are purchased off the shelf and are easily replaced should an outage occur. These parts are not considered in the model.

II. SITE SELECTION:

Three locksites were selected for implementation of the model, Lock 11 and Lock 22 on the Mississippi River, and Dresden Island Lock and Dam on the Illinois Waterway. The ~~miter gate~~ lock valve machinery on both rivers is similar in arrangement and design so these sites were chosen as a representative sample of each river. The locks on the Mississippi were grouped according to the average annual number of lockages which increases slightly going downriver.

Lock Modeled	Locks that are similar
Lock 11	Locks 1-10,12-20
Lock 22	Locks 21,24,25,27
Dresden Island	Lockport, Brandon Rd, Marseilles, Starved Rock

III. RELIABILITY ANALYSIS:

There is little direct historical documentation describing the failure rates for this equipment. The component meanlife and failure distribution must be synthesized from generalized textbook data tables on the subject. These textbook tables are reproduced and included as an attachment to this section. The Reliability model used is for individual components described by a two-parameter Weibull distribution (taken from Shannon and Wilson reference (c) page 2- 26) is:

$$R(t) = \exp[-(t/a)^{b}]$$

where:

t = number of years the component has been in service.
a = meanlife of the component; the average number of years the component is expected to function.
b = Weibull index number (Table 7-2).
R(t) = reliability function for the component, probability of satisfactory performance at a given time t.

The hazard rate function h(t) of individual components for a two parameter Weibull distribution (taken from reference (c) page 2-26) is:

$$h(t) = (b/a)*(t/a)^{(b-1)}$$

where:
h(t) = the conditional probability that a component will fail in the next year given that it has not failed up to that point.

Table 7-2 was used (Bloch and Geitner reference (d), attached) to provide an estimate of the Weibull Index and part meanlife using the descriptive verb for each failure mode. For gear teeth, terms like "fretting" and "scoring" are used to describe wear failure and b=3 for these. Fatigue Fracture describes the breakage of the steel gears and b=1.1. A hydraulic cylinder typically experiences wear from pitting, rubbing or scoring of the cylinder rod. For this type of failure mode, b=3.0 was used. Tables A-7 and A-8 (in Greene and Bourne reference (e) attached) also gives an estimate for the meanlife of a given name component. The Table A-7 meanlife is weighted by dividing the life by an environmental factor K from Table A-8 to account for outdoor operation. The textbooks thus give an estimate of the failure distribution for a given part if there is no actual failure history for the part.

The gear design capacity is estimated by the method of AGMA 6010 (reference (g)) as applied by Drago (reference (f), attached). The theoretical number of lifetime cycles before pitting failure occurs for all the gears is approximately 7 million cycles based on Life factor of 1.0 used in the design of the gearbox. Bending Failure was not critical in this analysis since the gears are underloaded in this regard. The slide valve machinery lifespan is assumed to be 500,000 cycles which corresponds to a fifty year lifespan at the average use rate. The frequency of use is assumed to be the number of open and close operations equaling one cycle.

The meanlife for each component (in actual years) is the calculated from the number of lifetime cycles, divided by the use rate or number of lockage cycles. It is further divided by the environmental factor (K=2) to account for being exposed outdoors. The meanlife for the components is the average number of years the component will be expected to function at the average use rate.

The final probability of unsatisfactory performance is computed by taking into account the number of items which comprise the component at each lock. For instance, if there are "n" enclosed gearboxes which each have a conditional probability of failure h(t)=P then the probability of failure (P_e) for all of the enclosed gears is:

$$P_e = 1-(1-P)^n$$

In this case, there is only one component among each type of valve equipment therefore, the total probability of failure equals the probability of failure of the component. This total probability of failure assumes that there is no correlation between the different components or even among components.

IV. RESULTS OF RELIABILITY ANALYSIS:

a. Three hazard functions are required for three separate conditions; normal operation and maintenance (O&M), enhanced O&M, and rehabilitated. For the lock valve mechanical models, enhanced O&M was considered the same as normal O&M since there is little necessary maintenance beyond normal maintenance. Rehabilitation of these mechanical components would mean replacement of the components. For this condition, the rehabilitated hazard function will also be the same as normal O&M since the equipment was new at the start of this analysis. Tables 1,2, and 3 present the hazard function data and probabilities of unsatisfactory performance for the selected sites.

TABLE 1 Lock 11

HAZARD RATES [H(t)] OF INDIVIDUAL COMPONENTS													
Year	2000	2005	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060
Helical Reducer	0.0000	0.0000	0.0000	0.0001	0.0001	0.0002	0.0002	0.0003	0.0004	0.0005	0.0006	0.0008	0.0009
PROBABILITY OF UNSATISFACTORY PERFORMANCE - ITEM													
Year	2000	2005	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060
Helical Reducer	0.0000	0.0000	0.0001	0.0002	0.0004	0.0006	0.0009	0.0012	0.0016	0.0021	0.0025	0.0031	0.0036
PROBABILITY OF UNSATISFACTORY PERFORMANCE - SYSTEM													
Year	2000	2005	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060
Tainter Valve Machinery	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.002	0.002	0.003	0.003	0.004

TABLE 2 Lock 22

HAZARD RATES [H(t)]OF INDIVIDUAL COMPONENTS

Year	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044	2049
Helical Reducer	0.0000	0.0000	0.0000	0.0001	0.0002	0.0003	0.0004	0.0006	0.0007	0.0009	0.0011	0.0014	0.0016

PROBABILITY OF UNSATISFACTORY PERFORMANCE - ITEM

Year	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044	2049
Helical Reducer	0.0000	0.0000	0.0002	0.0004	0.0007	0.0011	0.0016	0.0022	0.0029	0.0037	0.0045	0.0055	0.0065

PROBABILITY OF UNSATISFACTORY PERFORMANCE - SYSTEM

Year	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044	2049
Tainter Valve Machinery	0.000	0.000	0.000	0.000	0.001	0.001	0.002	0.002	0.003	0.004	0.005	0.005	0.007

TABLE 3 Dresden Island Lock

HAZARD RATES [H(t)]OF INDIVIDUAL COMPONENTS

Year	1984	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044
Hydraulic cylinder	0.0000	0.0025	0.0099	0.0223	0.0396	0.0619	0.0892	0.1214	0.1585	0.2006	0.2477	0.2997	0.3567

PROBABILITY OF UNSATISFACTORY PERFORMANCE - ITEM

Year	1984	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044
Hydraulic cylinder	0.0000	0.0099	0.0390	0.0862	0.1494	0.2256	0.3118	0.4040	0.4986	0.5917	0.6797	0.7595	0.8287

PROBABILITY OF UNSATISFACTORY PERFORMANCE - SYSTEM

Year	1984	1989	1994	1999	2004	2009	2014	2019	2024	2029	2034	2039	2044
Slide Valve Machinery	0.000	0.010	0.039	0.086	0.149	0.226	0.312	0.404	0.499	0.592	0.680	0.760	0.829

The reliability model predicts relatively low hazard rates and probabilities of unsatisfactory performance for valve operating machinery components. It should be noted that the valves are redundant systems with four valves per lock. The locks can still operate with two functional valves(one upstream and one downstream). Failure of one valve system would not shutdown the lock. The resulting consequences would be the navigational delays experienced from the slow lockage times caused by operating with only two or three valves.

V. CONSEQUENCE TABULATION

The consequences of three degrees of unsatisfactory performance are presented in Table 4. The navigation downtime is based solely on engineering judgment and is estimated by the expected repairs required to the components. The low level consequences were considered to result from a minor repair performed by lock personnel. Lock downtime or delay is considered negligible. The medium level consequences were considered to result from a repair that would require use of only two valves. The medium level repairs would cause lockage delays because of slow emptying and filling times. High level consequences were considered to result from a breakdown of two or more valve systems that would require major repair or replacement of an item/s. In this case, the valves would be inoperable and lock downtime would occur. The estimated costs reflect the level of required repairs.

It was assumed that lock personnel or hired labor forces would perform the repairs.

TABLE 4 Consequences Table

COMPONENT	LOW LEVEL OF CONSEQUENCES (LC)			MEDIUM LEVEL OF CONSEQUENCES (MC)			HIGH LEVEL OF CONSEQUENCES (HC)		
	P(LC)	NAVIGATION DOWN TIME (Days)	ESTIMATED REPAIR COST	P(MC)	NAVIGATION DOWN TIME (Days)[1]	ESTIMATED REPAIR COST	P(HC)	NAVIGATION DOWN TIME (Days)[2]	ESTIMATED REPAIR COST
Helical Reducer	0.9	0	$2,500	0.09	0.08	$10,000	0.01	3.0	$50,000
Hydraulic Cylinder	0.9	0	$2,500	0.09	0.08	$10,000	0.01	3.0	$50,000

1 - Estimated lockage delay resulting from operating with two valves.

2 - Estimated lock downtime resulting from catastrophic failure that would prevent valve operation.

VI. REHABILITATION COSTS

As noted previously, rehabilitation of the components would involve replacement of the items. Replacement of lock valve machinery has been performed on several past rehabilitation projects. The cost of replacement of each valve machinery system is approximately $212,000 or $848,000 per site. The cost of the gearbox is approximately $40,000. The cost of the hydraulic cylinder is approximately $20,000.

VII. REFERENCES:

a. Operating Forces on Miter-Type Lock Gates (Technical Report 2-651 June 1964 USACOE Waterways Experiment Station, Vicksburg, MS)

b. CECW-ED ETL 1110-2-532 Reliability Assessment of Navigation Structures.

c. Probability Models for Geotechnical Aspects of Navigational Structures, January 1994, Shannon and Wilson Inc, St. Louis, Missouri.

d. Practical Machinery Management for Process Plants (Bloch,, Geitner). Gulf Publishing Company, Houston, Texas.

e. Reliability Technology (Greene, Bourne) John Wiley Sons, New York, NY

f. Fundamentals of Gear Design (Drago, R.L.) Butterworths

g. Rating Pitting Resistance and Bending Strength of Spur and Helical Involute Gear Teeth(AGMA Standard 218.01-1981)

SECTION 3 - Electrical Systems: Motor Control Center & Lock Control Cables

E L E C T R I C A L S Y S T E M S

RELIABILITY ANALYSIS
FOR LOCK ELECTRICAL SYSTEMS

I. Model Description

1. Assessment Of Reliability. Electrical reliability is based on the Probability of Unsatisfactory Performance of the systems components. The following paragraphs provide the assumptions, description of the model and the results from implementation of the reliability assessment.

2. Reliability Analysis. The following pages give a descriptive explanation of the equations and procedures used to find the Probability of Unsatisfactory Performance of each electrical sub-system. The attachments indicate the final results and quantities of the certain aspects needed to find the probability of unsatisfactory performance on a given year. The data shows the condition of the systems in new condition, present condition in 1995, and future condition. The data also show the trend of the condition of the systems between the years of new installation and future condition.

a. Sub-Systems Analyzed. Two main sub-systems of the lock electrical system were analyzed, the lock motor control center (or main switchboard) and the lock control cables. Only the major components that would produce an unsatisfactory performance and significant repair time were considered. The major components considered for the motor control center were circuit breakers and motor starters associated with tainter valves, miter gates, their control circuits, and the tow haul units and whose failure would cause inoperation of this machinery. The considerations for the lock machinery control cables included motor feeder cables, multi-conductor control cables, and groups of single conductor control cables in conduit. All other components that exist within the motor control center are either considered maintenance items or are easily replaced without considerable downtime.

b. Lock Motor Control Center.

(1) Only vital components of the motor control center that could cause an unsatisfactory performance were

chosen for analysis. These components were circuit breakers and motor starters for the lock operating machinery. The other components such as transformers, relays, pushbuttons, terminal blocks, fuses, etc. were considered non-vital because they were either considered to be maintenance items or they could be quickly replaced with minimal downtime.

(2) The meanlife, *a*, of the components is the expected average life or the average number of years the component will function. The meanlife is based on engineering knowledge of the existing components and from Table 10, IEEE STD 493-1990, "Design of Reliable Industrial and Commercial Power systems". From this table the failures per unit year of each component were determined. The reciprocal of the failures per unit year is the meanlife of the components. The meanlife was then divided by an environmental factor K, found in Table A.8, "Practical Machinery Management for Process Plants". The environmental factor was based upon the conditions under which the components operate. This process was used for both the circuit breakers and the motor starters. The meanlife of the cables was determined based upon engineering knowledge of the existing cables.

(3) The Reliability, $R(t)$, was found for individual components at certain years by using the Weibel distribution.

$$R(t) = e^{-(t/a)^{b}}$$

which gives the probability that the component will survive to time *t*, where *t* is the number of years the component has been in service, *a* is the meanlife of the component and *b* is the Weibel index number found in Table 7.2, "Practical Machinery Management for Process Plants". This index number was selected by the certain mode of failure each component possesses.

(4) The hazard rate function $h(t)$ for any given individual item is given by:

$$h(t) = b/a \cdot (t/a)^{b-1}$$

where b is the Weibel index, a is the meanlife of the component, and t is the number of years the component has been in service. The hazard rate of each component is the probability that a component will fail in the next year given that it has not failed up to that point.

(5) The system would have an unsatisfactory performance if any one of the components were to fail because all of the components are in series with each other. To compute the final probability of unsatisfactory performance, one has to take into account the number of items which comprise the component. For instance, if there are "n" cables which each have a probability of failure, P, then the probability of failure, P_b for all of the circuit breakers is:

$$P_b = 1 - (1-P)^n$$

Likewise, if the probability of failure of the motor starters is P_s and the probability of failure of the circuit breakers is P_b then the total probability of failure for the motor control center is:

$$P_t = P_s + P_b$$

b. Lock Control Cables.

(1)-(5) The analysis for the Lock Control Cables uses the same steps as the analysis for the Lock Motor Control Center to establish the Probability of Unsatisfactory Performance for system, except that the total probability of failure for the lock control cables is:

$$P_c = 1 - (1-P)^n$$

II. Site Selection. The model was implemented for each lock on both the Mississippi River and the Illinois Waterway.

III. Component Reliability constants and Variables.

1. The model was implemented with the following constants.

MEANLIFE OF INDIVIDUAL COMPONENTS				
	Failures per Unit Year	Environmental K Value	Meanlife of Part	B Value
(Motor Control Center)				
Circuit Breaker	0.0052	2	96	3
Motor Starter	0.01	2	50	3
Control Cables	---	2	90	2

2. Each site was investigated and found to have the following number of components for study in the model.

MISSISSIPPI RIVER LOCKS -- NUMBER OF ELECTRICAL COMPONENTS ANALYZED AND YEAR OF INSTALLATION

LOCK	USAF*	LSAF*	1 MAIN	1 AUX.	2	3	4	5	5A	6	7	8	9	10	11	12	13
(MOTOR CONTROL CENTER)																	
CIRCUIT BREAKER	11	7	6	6	11	11	11	11	11	11	11	11	11	11	11	11	11
MOTOR STARTER	10	4	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8
CONTROL CABLES	25	20	98	98	75	75	75	75	75	75	75	75	75	75	61	61	63
DATE EQUIPMENT WAS NEW	1965	1956	1995	1995	1995	1991	1990	1991	1992	1993	1994	1995	1996	1997	2000	2000	2000

* USAF AND LSAF REFER TO UPPER AND LOWER ST. ANTHONY FALLS LOCKS RESPECTIVELY.

MISSISSIPPI RIVER LOCKS -- NUMBER OF ELECTRICAL COMPONENTS ANALYZED AND YEAR OF INSTALLATION

LOCK	14	15 MAIN	15 AUX.	16	17	18	19	20	21	22	24	25	MELVIN PRICE MAIN	MELVIN PRICE AUX.	27 MAIN	27 AUX.
(MOTOR CONTROL CENTER)																
CIRCUIT BREAKER	11	11	12	11	11	11	24	10	10	10	33	33	69	41	69	41
MOTOR STARTER	8	8	8	8	8	8	11	8	8	8	20	20	21	14	21	14
CONTROL CABLES	61	74	75	63	63	63	35	65	70	70	32	32	34	13	34	13
DATE EQUIPMENT WAS NEW	1998	1994	1994	1993	1990	1992	1955	1988	1989	1989	2000	2000	1990	1990	2000	2000

ILLINOIS WATERWAY LOCKS -- NUMBER OF ELECTRICAL COMPONENTS ANALYZED AND YEAR OF INSTALLATION

LOCK	O'BRIEN	LOCKPORT	BRANDON ROAD	DRESDEN ISLAND	MARSEILLES	STARVED ROCK	PEORIA	LA GRANGE
(MOTOR CONTROL CENTER)								
CIRCUIT BREAKER	15	25	20	14	14	14	6	6
MOTOR STARTER	8	9	9	9	9	9	4	4
CONTROL CABLES	22	25	57	46	46	46	12	12
DATE EQUIPMENT WAS NEW	1960	1995	1984	1978	1977	1979	1988	1988

IV. Hazard Functions.

The study analyzes three separate hazard functions: Normal Operation and Maintenance(O&M), Rehabilitated, and Enhanced Maintenance. For the electrical models, enhanced maintenance is not applicable since there few maintenance items for electrical systems. For the system in question, "rehabilitation" means "replacement". The rehabilitated hazard function will be same as for a new component.

1. The following tabulated data shows the value of the normal O&M hazard function for the motor control center for the Mississippi River and Illinois Waterway locks respectively.

MISSISSIPPI RIVER - PROBABILITY OF UNSATISFACTORY PERFORMANCE - MOTOR CONTROL CENTER

YEAR	USAF	LSAF	1 (MAIN)	1 (AUX.)	2	3	4	5	5A	6	7	8	9	10	11	12	13
2000	0.253	0.359	0.007	0.003	0.006	0.018	0.023	0.018	0.015	0.011	0.008	0.023	0.004	0.002	0.000	0.000	0.006
2005	0.322	0.433	0.027	0.012	0.023	0.044	0.051	0.044	0.036	0.033	0.020	0.051	0.018	0.015	0.006	0.006	0.023
2010	0.395	0.511	0.061	0.027	0.051	0.081	0.089	0.081	0.073	0.065	0.058	0.089	0.044	0.038	0.023	0.023	0.051
2015	0.472	0.590	0.107	0.047	0.089	0.127	0.137	0.127	0.117	0.107	0.098	0.137	0.081	0.073	0.051	0.051	0.089
2020	0.551	0.670	0.163	0.073	0.137	0.181	0.193	0.181	0.170	0.159	0.148	0.193	0.127	0.117	0.089	0.089	0.137
2025	0.630	0.750	0.228	0.105	0.193	0.244	0.257	0.244	0.231	0.218	0.206	0.257	0.181	0.170	0.137	0.137	0.193
2030	0.709	0.828	0.303	0.141	0.257	0.313	0.327	0.313	0.298	0.284	0.271	0.327	0.244	0.231	0.193	0.193	0.257
2035	0.787	0.905	0.382	0.182	0.327	0.386	0.402	0.386	0.371	0.356	0.342	0.402	0.313	0.298	0.257	0.257	0.327
2040	0.862	0.979	0.465	0.228	0.386	0.464	0.480	0.464	0.448	0.433	0.417	0.480	0.386	0.371	0.327	0.327	0.386
2045	0.934	1.050	0.551	0.277	0.480	0.544	0.560	0.544	0.528	0.512	0.496	0.560	0.464	0.448	0.386	0.386	0.480
2050	1.002	1.117	0.637	0.330	0.560	0.625	0.641	0.625	0.609	0.592	0.576	0.641	0.544	0.528	0.440	0.480	0.560

MISSISSIPPI RIVER - PROBABILITY OF UNSATISFACTORY PERFORMANCE - MOTOR CONTROL CENTER

YEAR	14	15(MAIN)	15(AUX.)	16	17	18	19	20	21	22	24	25	MELVIN PRICE (MAIN)	MELVIN PRICE (AUX.)	27 (MAIN)	27 (AUX.)
2000	0.001	0.008	0.008	0.011	0.023	0.015	0.574	0.032	0.027	0.027	0.000	0.000	0.072	0.023	0.000	0.000
2005	0.011	0.028	0.028	0.033	0.051	0.038	0.678	0.064	0.057	0.057	0.015	0.015	0.159	0.067	0.018	0.012
2010	0.033	0.058	0.058	0.065	0.089	0.073	0.763	0.106	0.096	0.096	0.058	0.058	0.273	0.132	0.072	0.047
2015	0.065	0.098	0.098	0.107	0.137	0.117	0.845	0.156	0.145	0.145	0.128	0.128	0.408	0.216	0.159	0.104
2020	0.107	0.148	0.148	0.159	0.193	0.170	0.984	0.215	0.202	0.202	0.219	0.219	0.558	0.315	0.273	0.180
2025	0.159	0.206	0.206	0.218	0.257	0.231	1.079	0.280	0.266	0.266	0.328	0.328	0.715	0.427	0.408	0.274
2030	0.218	0.271	0.271	0.284	0.327	0.298	1.167	0.351	0.336	0.336	0.450	0.450	0.874	0.547	0.558	0.381
2035	0.284	0.342	0.342	0.356	0.402	0.371	1.250	0.426	0.410	0.410	0.578	0.578	1.027	0.672	0.715	0.498
2040	0.356	0.417	0.417	0.433	0.480	0.448	1.326	0.503	0.488	0.488	0.707	0.707	1.172	0.799	0.874	0.622
2045	0.433	0.496	0.496	0.512	0.560	0.528	1.395	0.583	0.567	0.567	0.834	0.834	1.304	0.924	1.027	0.748
2050	0.513	0.576	0.576	0.592	0.641	0.609	1.458	0.662	0.646	0.646	0.955	0.955	1.422	1.044	1.172	0.874

ILLINOIS WATERWAY - PROBABILITY OF UNSATISFACTORY PERFORMANCE - MOTOR CONTROL CENTER

YEAR	O'BRIEN	LOCKPORT	BRANDON ROAD	DRESDEN ISLAND	MARSEILLES	STARVED ROCK	PEORIA	LA GRANGE
2000	0.347	0.008	0.071	0.123	0.133	0.112	0.017	0.017
2005	0.427	0.030	0.121	0.181	0.194	0.169	0.033	0.033
2010	0.610	0.066	0.182	0.248	0.263	0.234	0.055	0.055
2015	0.596	0.117	0.253	0.323	0.338	0.307	0.083	0.083
2020	0.683	0.179	0.332	0.403	0.420	0.387	0.115	0.115
2025	0.769	0.252	0.418	0.487	0.505	0.470	0.153	0.153
2030	0.655	0.334	0.509	0.574	0.692	0.557	0.194	0.194
2035	0.917	0.424	0.603	0.662	0.680	0.645	0.240	0.240
2040	1.017	0.520	0.698	0.749	0.767	0.732	0.289	0.289
2045	1.093	0.619	0.793	0.835	0.852	0.818	0.342	0.342
2050	1.164	0.720	0.887	0.917	0.933	0.901	0.397	0.397

2. The following tabulated data shows the value of the normal O&M hazard function for the control cables for the Mississippi River and Illinois Waterway locks respectively.

MISSISSIPPI RIVER - PROBABILITY OF UNSATISFACTORY PERFORMANCE - CONTROL CABLES																	
YEAR	USAF	LSAF	1 (MAIN)	1 (AUX.)	2	3	4	5	5A	6	7	8	9	10	11	12	13
2000	0.487	0.578	0.030	0.024	0.088	0.154	0.169	0.154	0.138	0.122	0.105	0.088	0.071	0.054	0.000	0.000	0.075
2005	0.534	0.618	0.060	0.048	0.169	0.229	0.243	0.229	0.214	0.200	0.185	0.169	0.154	0.138	0.073	0.073	0.144
2010	0.577	0.654	0.089	0.072	0.243	0.297	0.310	0.297	0.284	0.271	0.257	0.243	0.229	0.214	0.140	0.140	0.208
2015	0.616	0.686	0.116	0.094	0.310	0.360	0.371	0.360	0.348	0.335	0.323	0.310	0.297	0.284	0.203	0.203	0.268
2020	0.651	0.716	0.143	0.116	0.371	0.417	0.427	0.417	0.406	0.394	0.383	0.371	0.360	0.348	0.261	0.261	0.323
2025	0.683	0.743	0.170	0.138	0.427	0.469	0.478	0.469	0.459	0.448	0.438	0.427	0.417	0.406	0.315	0.315	0.374
2030	0.712	0.767	0.195	0.159	0.478	0.516	0.525	0.516	0.507	0.498	0.488	0.478	0.469	0.459	0.365	0.365	0.421
2035	0.739	0.789	0.220	0.180	0.525	0.559	0.567	0.559	0.551	0.542	0.534	0.525	0.516	0.507	0.411	0.411	0.465
2040	0.763	0.809	0.244	0.200	0.567	0.599	0.606	0.599	0.591	0.583	0.575	0.567	0.559	0.551	0.454	0.454	0.505
2045	0.785	0.827	0.267	0.220	0.606	0.635	0.641	0.635	0.628	0.621	0.613	0.606	0.599	0.591	0.494	0.494	0.543
2050	0.805	0.844	0.290	0.239	0.641	0.667	0.674	0.667	0.661	0.655	0.648	0.641	0.635	0.628	0.531	0.531	0.577

MISSISSIPPI RIVER - PROBABILITY OF UNSATISFACTORY PERFORMANCE - CONTROL CABLES																
YEAR	14	15(MAIN)	15(AUX.)	16	17	18	19	20	21	22	24	25	MELVIN PRICE (MAIN)	MELVIN PRICE (AUX.)	27 (MAIN)	27 (AUX.)
2000	0.031	0.104	0.105	0.103	0.144	0.117	0.324	0.175	0.173	0.173	0.000	0.000	0.081	0.022	0.000	0.000
2005	0.103	0.182	0.185	0.171	0.208	0.183	0.363	0.239	0.242	0.242	0.039	0.039	0.119	0.038	0.041	0.016
2010	0.171	0.254	0.257	0.233	0.268	0.245	0.388	0.298	0.305	0.305	0.076	0.076	0.155	0.053	0.081	0.032
2015	0.233	0.319	0.323	0.290	0.323	0.301	0.407	0.353	0.363	0.363	0.112	0.112	0.190	0.068	0.119	0.047
2020	0.290	0.379	0.383	0.344	0.374	0.354	0.432	0.403	0.416	0.416	0.147	0.147	0.223	0.083	0.155	0.062
2025	0.344	0.434	0.438	0.393	0.421	0.403	0.457	0.449	0.465	0.465	0.180	0.180	0.256	0.098	0.190	0.077
2030	0.393	0.484	0.488	0.439	0.465	0.448	0.480	0.492	0.509	0.509	0.212	0.212	0.286	0.112	0.223	0.092
2035	0.439	0.529	0.534	0.481	0.505	0.490	0.503	0.532	0.550	0.550	0.243	0.243	0.316	0.127	0.256	0.107
2040	0.481	0.571	0.575	0.521	0.543	0.528	0.524	0.568	0.588	0.588	0.272	0.272	0.345	0.141	0.286	0.121
2045	0.521	0.608	0.613	0.557	0.577	0.564	0.545	0.602	0.623	0.623	0.301	0.301	0.372	0.155	0.316	0.135
2050	0.557	0.643	0.648	0.591	0.609	0.597	0.564	0.633	0.654	0.654	0.328	0.328	0.398	0.168	0.345	0.149

ILLINOIS WATERWAY - PROBABILITY OF UNSATISFACTORY PERFORMANCE - CONTROL CABLES								
YEAR	O'BRIEN	LOCKPORT	BRANDON ROAD	DRESDEN ISLAND	MARSEILLES	STARVED ROCK	PEORIA	LA GRANGE
2000	0.196	0.030	0.202	0.222	0.230	0.213	0.035	0.035
2005	0.218	0.060	0.256	0.265	0.273	0.256	0.049	0.049
2010	0.239	0.089	0.307	0.306	0.314	0.298	0.063	0.063
2015	0.260	0.116	0.355	0.344	0.352	0.337	0.077	0.077
2020	0.280	0.143	0.399	0.381	0.388	0.374	0.091	0.091
2025	0.299	0.170	0.440	0.415	0.422	0.409	0.104	0.104
2030	0.329	0.195	0.479	0.448	0.454	0.442	0.118	0.118
2035	0.337	0.220	0.514	0.479	0.485	0.473	0.131	0.131
2040	0.355	0.244	0.548	0.508	0.514	0.502	0.144	0.144
2045	0.373	0.267	0.579	0.536	0.541	0.530	0.156	0.156
2050	0.390	0.290	0.608	0.562	0.567	0.557	0.169	0.169

V. Consequence Tabulation.

1. The consequences of failure of the lock electrical components were investigated and include downtime to navigation and repair costs. The consequences are considered to be constant with respect to time and are based solely on engineering judgment. The table shown below indicates the conditional probability that a particular consequence will occur given that unsatisfactory performance has occurred.

CONSEQUENCES TABLE									
COMPONENT	LOW LEVEL OF CONSEQUENCES(LC)			MEDIUM LEVEL OF CONSEQUENCES(MC			HIGH LEVEL OF CONSEQUENCES(HC)		
	P(LC)	NAVIGATION DOWN-TIME (DAYS)	ESTIMATED REPAIR COST	P(MC)	NAVIGATION DOWN-TIME (DAYS)	ESTIMATED REPAIR COST	P(HC)	NAVIGATION DOWN-TIME (DAYS)	ESTIMATED REPAIR COST
(Motor Control Center)									
Circuit Breaker	0.9	0.08	$ 1,100.00	0.09	0.17	$ 1,500.00	0.01	0.79	$ 1,500.00
Motor Starter	0.9	0.08	$ 4,500.00	0.09	0.17	$ 5,000.00	0.01	0.79	$ 5,000.00
Control Cables	0.9	0.08	$ 600.00	0.09	0.08	$ 5,000.00	0.01	2.00	$5,000.00

Assumptions:

1. Cost for repair of components is based on replacement of the average lock component size.
2. Repair work performed by USACE electricians and mechanics.
3. Increased cost associated with acquisition of replacement parts NOT on-hand.
4. HC breakdown assumed to occur between 1700 and 1800 hours after normal business hours. This is included in the navigation downtime and the estimated repair cost.

VI. Rehabilitation Costs.

1. Rehabilitation of electrical equipment for the lock operating machinery has been conducted at several locks on both the Mississippi River and Illinois Waterway. A lock facility electrical rehabilitation has cost approximately $1.6 million. The electrical items of work included replacement of electrical equipment for the control station, the main lock, and all exterior electrical systems required to operate the entire facility. Rehabilitation of the dam is not included in this cost.

2. Approximately $350,000 of the $1.6 million was the cost for rehabilitation of critical electrical equipment that directly affects lock operability. Specific items included in the rehabilitation were replacement of the motor control center and two operator control stands. Power, control and instrumentation cables and their raceways were also replaced.

VII. References.

1. ANSI/IEEE STD. 493-1990 IEEE; Recommended Practice for Design of Reliable Industrial and Commercial Power Systems.

2. CECW-ED ETL 1110-2-532; Reliability Assessment of Navigation Structures.

3. Practical Machinery Management for Process Plants, Gulf Publishing Company, Houston, Texas.

4. The Probability Models for Geotechnical Aspects of Navigational Structures, January 1994, Shannon and Wilson Inc., St. Louis, Missouri.

5. Rehabilitation Evaluation Report, Lock and Dam 12, Upper Mississippi River, Bellevue, Iowa, March 1995, U.S. Army Corps of Engineers - St. Paul District (CENCS), St. Paul, Minnesota.

SYSTEM SIGNIFICANT COMPONENTS

ENGINEERING RELIABILITY MODELS REPORT

(A Stand Alone Report Compiling Backup Information)

RELIABILITY MODEL
FOR
HYDRAULIC NAVIGATION CHANNEL

Engineering Divisions

St. Paul, Rock Island and St. Louis Districts

US Army Corps of Engineers

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Dredge Capacity / Demand Navigation Channel Reliability Model

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Channel Reliability of the Navigation System in the Upper Mississippi River

Phase II Final Report

Submitted by the

Center for Risk Management of Engineering Systems
University of Virginia

to

The U.S. Army Corps of Engineers

St. Paul District
Rock Island District
St. Louis District

July 24, 1995

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Table of Contents

Glossary of Terms

disposal capacity	maximum volume of dredge material that can be disposed of in a year limited by the availability of disposal sites, for a system or a pool
dredge capacity	the maximum volume of dredge material that can be dredged in a year under current dredging practices, for a system or a pool
dredge demand	the dredge material volume needing removal in order to maintain safe navigation standards in a year, for a system or a pool
dredge system	one or more dredges operating on a group of pools and/or river reaches
dredge subsystem	a single dredge or a dredging contract
navigation structure	a structure constructed in divert river flow towards the main channel, for example, a wing dam or a closing dam
pool	the river reach between two consecutive navigation dams
pool failure	the joint occurrence in a year of the dredge demand exceeding the allocated dredge capacity for a pool and system failure
pool reliability	the probability of pool failure in a year
rehabilitation	substantial repair work to wing dams and other structures to maintain the navigation channel
safety margin	the difference between the dredge capacity and the dredge demand
setup time	the time between dredge arrival at a site and the initiation of dredging operations
system failure	the condition when the dredge demand exceeds the dredge capacity for a dredge system
system reliability	the probability of system failure in a year
travel time	the travel time between dredging sites

Notation

β_{ds} reliability index for the dredging system

β_i reliability index for pool i

C_{di} dredging capacity available for pool i (yd^3/year)

C_{ds} system dredging capacity (yd^3/year)

C_i capacity for pool i (yd^3/year)

C_{pi} dredging material placement capacity in pool i (yd^3/year)

D_i dredge demand in pool i (yd^3/year)

$D_{\neg i}$ system dredge demand exclusive of pool i (yd^3/year)

M maximum number of days in a dredging season based on the normal O&M budget (days/year)

$\mu_{C_{di}}$ mean of the dredge capacity distribution for pool i (yd^3/year)

$\mu_{C_{ds}}$ mean of the dredge capacity distribution for the dredging system (yd^3/year)

$\mu_{D_{ds}}$ mean of the dredge demand distribution for the dredging system (yd^3/year)

$\mu_{D_{di}}$ mean of the dredge demand distribution for pool i (yd^3/year)

$\mu_{SM_{ds}}$ mean of the safety margin distribution for the dredging system (yd^3/year)

p_{ds} probability of failure for the dredge system

p_i probability of failure for pool i

P dredge production rate (yd^3/day)

Q average daily flow (cfs)

θ_i percentage of system capacity allocated for pool i

R^2 coefficient of determination (a measure of the goodness of fit) for the linear regression model

S dredge setup time (days/site)

SM_{ds} safety margin for the dredging system (yd^3/year)

SM_i safety margin for pool i (yd^3/year)

$\sigma_{C_{di}}$ standard deviation of the dredge capacity distribution for pool i (yd^3/year)

$\sigma_{C_{ds}}$ standard deviation of the dredge capacity distribution for the dredging system (yd^3/year)

$\sigma_{D_{ds}}$ standard deviation of the dredge demand distribution for the dredging system (yd^3/year)

σ_{D_i} standard deviation of the dredge demand distribution for pool i (yd^3/year)

$\sigma_{SM_{ds}}$ standard deviation of the safety margin distribution for the dredging system (yd^3/year)

σ_{SM_i} standard deviation of the safety margin distribution for pool i (yd^3/year)

T dredge travel time between sites (days/site)

V dredge volume per site (yd^3/site)

Executive Summary

Overview

This report summarizes Phase II of the project "Channel Reliability of the Navigation System in the Upper Mississippi River." In Phase I of this project, two reliability models of the navigation channel were developed by characterizing the relationships among the dredging demand of the channel, the state of the navigation structures, and the river hydrograph. This report shows the application of one of the two models, the Dredge-Capacity Reliability Model developed in Phase I, to the Upper Mississippi River and the Illinois Waterway. The three Corps of Engineers Districts responsible for the operation and maintenance of the navigation system in this region are the St. Paul Corps District, the Rock Island Corps District, and the St. Louis Corps District. Appendix A gives the scope of work for Phase II of the project.

In part one of this report, we describe the navigation channel and summarize the results from Phase I of this project. We also introduce the framework for the capacity-demand model used for reliability modeling in Phase II.

In part two, we show how to evaluate the dredge demand for the capacity-demand model formulation. The demand is based on examination of the relationship between the historical dredging volumes and the hydrology for the river pools. The assessment of dredge demand is calibrated and validated with dredging and flow data extracted from historical data provided by the corps.

In part three, we describe the evaluation of the dredge capacity for the capacity-demand model. The system capacity is the sum of the dredging capacities of its component dredges. We show how to use the capacity and demand probability distributions to calculate the probabilities of the demand exceeding system-capacity and pool-capacity in a year.

In parts four, five, and six, we give the complete results for each of the three corps districts. Results are given for the current state of the system, the state prior to the rehabilitation performed between 1973-1993, and for scenarios of dredge-need reduction and dredge-capacity increase. In part seven, selected results from the three districts are given for comparison. The raw and calculated data sets and some representative calculations are given for the three districts in Appendices B, C, and D.

Tutorial of the Model

A. Dredge-Capacity Reliability

The basis of the Dredge-Capacity Reliability is a capacity-demand model, where the capacity and the demand are represented by probability distributions (Harr 1987; Ang and Tang 1984). In this formulation, the demand distribution is dredge demand for the dredging system or a pool and is a function of flow. The capacity distribution is a function of the availability of the dredge for the system or for that particular pool. A similar capacity-demand (loading-resistance) approach has been used for structural components of the navigation locks and dams (Shannon and Wilson 1994).

The dredging capacity, C_{ds},for the system and that the dredge demand, D_{ds}, of the system are measured in cubic yards per year as shown in Figure ES-1a. The likelihood of system failure, p_{ds}, is the probability that the pool dredge capacity (C_{ds}) is less than the pool dredge demand (D_{ds}), equal to the shaded area in Figure ES-1a.

An alternative formulation is the use of a safety margin for the dredge system, SM_{ds}. The safety margin is the difference between the system dredge capacity (C_{ds}) and the system dredge demand (D_{ds}). The likelihood of system failure, p_{ds}, is then determined by the probability that the safety margin is less than zero and is shown by the shaded area in Figure ES-1b.

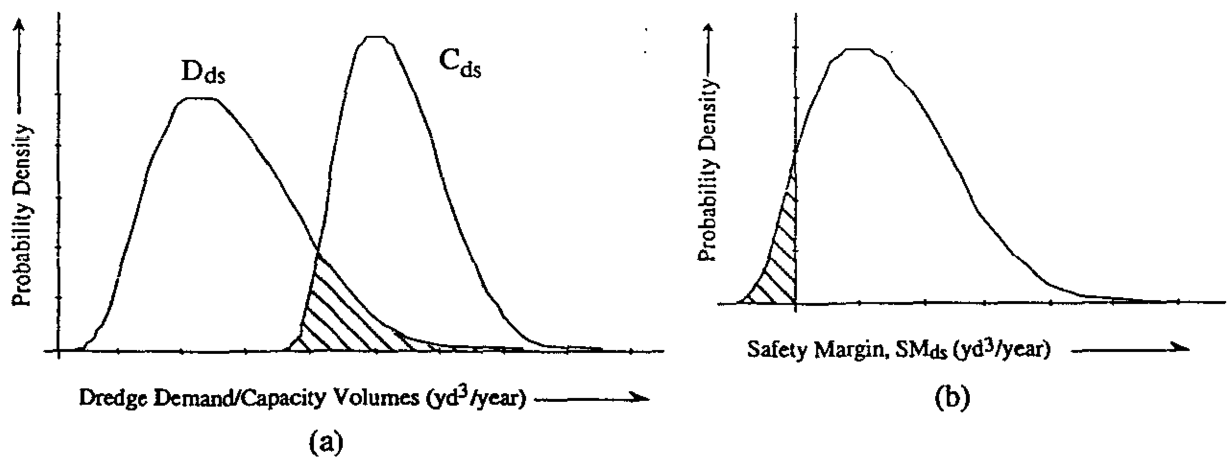

Figure ES-1 Capacity-Demand and Safety Margin Models for Waterway Navigation Systems

B. Assessment of Dredge Demand

The dredge-demand model relates the maintenance dredging performed at the end of the year (Figure ES-2) to the discharge hydrograph for the pool (Figure ES-3). The model demonstrates that structural rehabilitation changes the relationship between the discharge and the dredging. Results were developed for predicting the annual dredge volume for a pool from either:

(i) the year-average of daily discharge/stage in the pool; or

(ii) the number of daily falls in the pool hydrograph that exceeded the 95th percentile of magnitude.

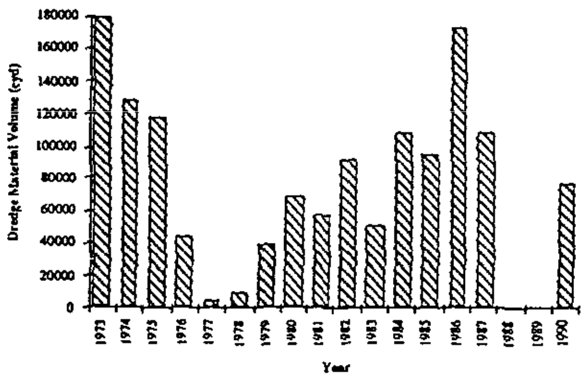

Figure ES-2 Quantity of Material Dredged from Pool 18 for the years 1973-1990

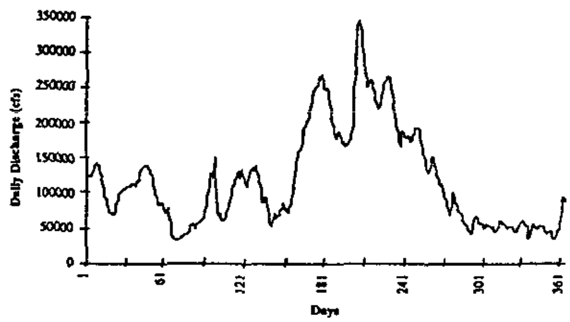

Figure ES-3 Daily Discharge in the year 1973 for the Upper Mississippi River at Keokuk

Notice in Figure ES-4 that the relationship between annual dredge volume and average daily discharge is linear, with the exception of three outlier points. A regression approach was used to relate the dredged material volume to the average daily discharge. This regression approach was applied to two cases: (1) pool/reach structures (wing and closing dams) in good condition, and (2) pool/reach structures in deteriorated condition.

In the period 1983-86, Pool 18 had significant (fifteen navigation structures) rehabilitation work. We used the two data sets 1973-83 and 1987-91 to develop two regression models for pre-rehabilitation and post-rehabilitation relationships. The regression model was then used to estimate the mean and standard deviation of the dredge demand distribution. Assuming that the dredge demand follows a normal distribution, we developed a distribution of dredge demand for the pool in the pre-rehabilitation condition as shown in Figure ES-5.

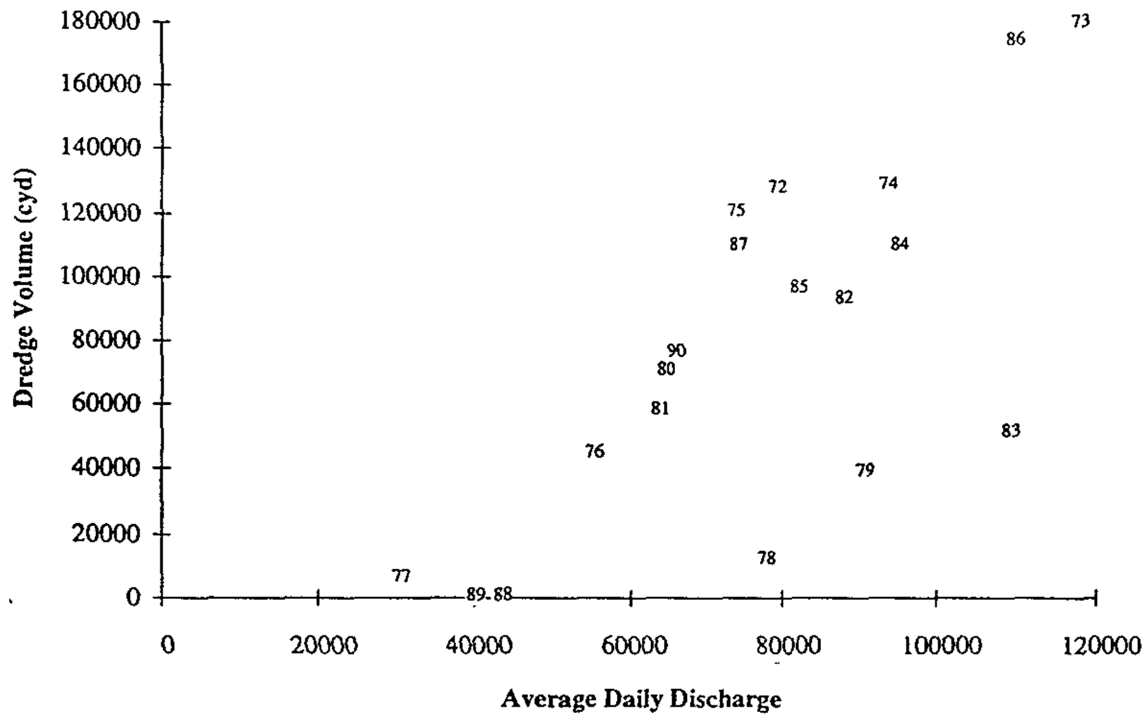

Figure ES-4 Plot of Annual Volume of Dredge Material vs. Average Annual Discharge for Pool 18 (1972-1990)

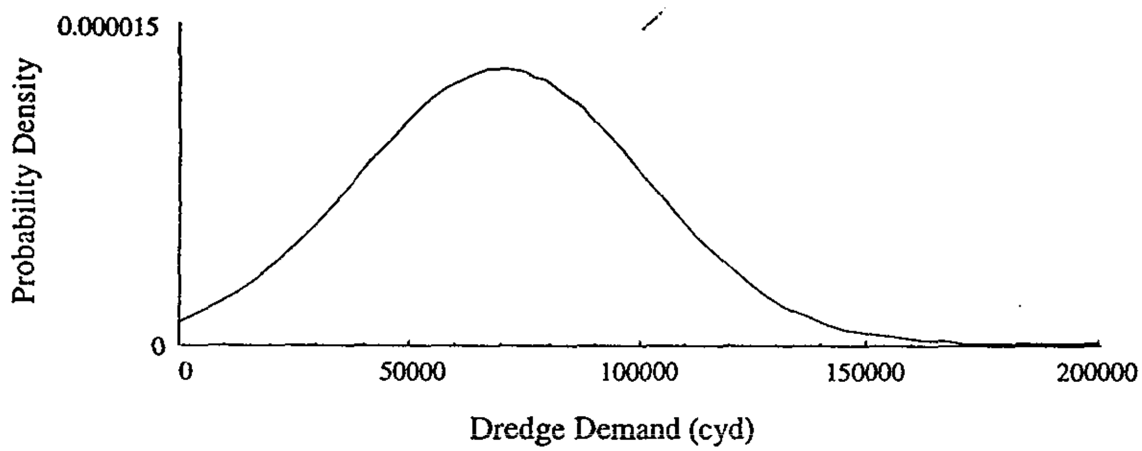

Figure ES-5 Pre-Rehabilitation Dredge Demand for Pool 18

C. Assessment of Dredge Capacity and Reliability

In order to evaluate the reliability of the channel, we need both the dredge demand and the dredge capacity for each pool. The distribution of dredge capacity (C_{ds}) for each dredging system was estimated from the number of days the dredge is used under the normal O&M budget, the dredge volume per site, and the time spent per site. The time spent per site is the sum of travel time, setup time, and production time. Dividing the number of days by the time per site gives an estimate of the upper limit on the number of sites that can be dredged in a year. This limit is then multiplied by the dredge volume per site to give a volumetric estimate of the dredge capacity.

Since the dredge capacity is an upper bound on the volume of material that can be dredged in any particular year, we assumed that the dredge capacity and demand are independent random variables. Figure ES-6 shows the plots of the Rock Island system's dredge capacity, dredge demand, and safety margin, assuming that these variables are normally distributed.

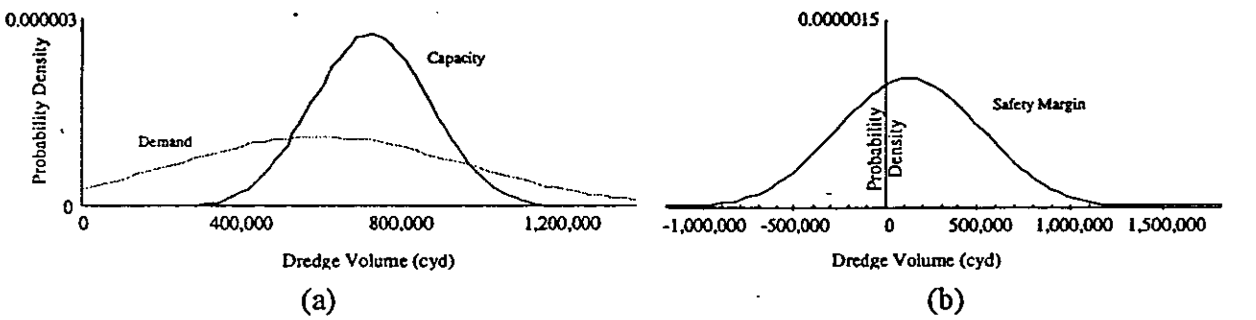

Figure ES-6 Pre-Rehabilitation Capacity, Demand, and Safety Margin Plots for Rock Island District

In order to evaluate the pool capacity, we defined a variable, θ_i, which allocated a percentage of the system capacity to each pool based on its historical dredge demand. This allocated pool capacity was then used to evaluate the probability of pool failure which is the joint occurrence of system failure and the dredge demand exceeding the allocated capacity for a pool.

Summary of Results

The dredge-capacity reliability model makes it possible to quantify the reliability of the navigation channel. It is also possible to quantify the impacts of rehabilitation of the navigation structures subject to the availability of sufficient dredging records after the completion of the rehabilitation effort. In addition, there exists a potential to further improve the navigation reliability from the current condition. The actual improvements are dependent upon the current condition of the channel and the dredging policy in each District.

Table ES-1 gives the values of the probability of system failure and the reliability index, β, for the three districts under various policy scenarios considered:

1. Pre-Rehabilitation is the condition prior to any historical rehabilitation work performed during the analysis period which is from 1973 to 1993.
2. Post-Rehabilitation (or Current) is the condition after the historical rehabilitation work and where sufficient post-rehabilitation records exist. For this reason, the impact of rehabilitation work performed during the last four to five years of the study period cannot be evaluated.
3. Enhanced Capacity is the condition where a twenty percent increase in the mean dredge capacity is assumed due to increased spending on dredging operations in the O&M budget..
4. Reduced Demand is the condition where a twenty percent reduction in dredge demand is assumed due to structural or other rehabilitation work performed on the navigation structures.

Table ES-1 Summary of System Reliabilities for the three Districts

Scenario	Results	St. Paul	Rock Island	St. Louis
Pre-Rehabilitation	Pr(failure)	0.1562	0.3745	
	β	1.01	0.32	
Post-Rehabilitation	Pr(failure)	0.1515	0.0233	0.4602
	β	1.03	1.99	0.10
Enhanced Capacity	Pr(failure)	0.0934	0.0015	0.2451
	β	1.32	2.96	0.69
Reduced Demand	Pr(failure)	0.1314	0.0048	0.2005
	β	1.12	2.60	0.84

The St. Paul District has maintained a high channel reliability primarily through the maintenance of a high dredge capacity by using contract dredges in addition to the government hydraulic and mechanical dredges. Results are also given for scenarios of dredge-need reduction and dredge-capacity increase. The scenario results show that an increase in the mean of the dredging capacity is much more effective in reducing the probability of failure than a reduction in dredge demand.

The Rock Island District has significantly improved its channel reliability through extensive and continued rehabilitation of navigation structures. This improvement illustrates the benefits of rehabilitation of the navigation structures and underscores the importance of continued maintenance of the navigation structures to avoid a return of the navigation channel to the less reliable pre-rehabilitated state. Potential for further improvement exists through dredge-need reduction and dredge-capacity increase.

In the St. Louis District, there is a potential to improve the channel reliability through increased capacity and/or rehabilitation of river training structures. The results of the scenario analysis show that demand reduction has a greater impact on the reduction of probability of system failure as compared to increases in dredge capacity.

Part I

Introduction and Background

1. Introduction

The Upper Mississippi River (UMR) begins at Lake Itasca in Minnesota and flows generally southwards, fed by several tributaries such as the Minnesota, St. Croix, Wisconsin, Rock, Des Moines, and Illinois Rivers. Just above St. Louis, the UMR meets the Missouri and then joins the Ohio River near Cairo, Illinois (Tweet 1983). The first five hundred miles of the river downstream from Lake Itasca to Minneapolis is not navigable and is thus not a part of this study. The reach from Minneapolis to Cairo, a distance of over eight hundred miles, forms the Upper Mississippi River Navigation System (Figure 1.1). Of these, the distance from St. Louis to Minnesota is navigable by means of a series of low navigation dams and associated locks which form a sequence of pools in the river. The reach from St. Louis downstream is navigable without requiring any locks and dams and forms the open river.

The Corps of Engineers has been responsible for the construction, operation, and maintenance of the Upper Mississippi River navigation system for more than a century. The Corps developed the original navigation system by making the Des Moines and the Rock Island rapids navigable. Moreover, the Corps was in charge of the construction of the 4 $^1/_2$-foot channel, the 6-foot channel, and the 9-foot channel projects. Currently, the navigation channel on the Upper Mississippi River is mandated by the U.S. Congress to be 300 feet wide and nine feet deep.

Sedimentation in the navigation channel reduces the depth available for navigation. The Corps of Engineers maintains the required navigation standard through the use of structural measures, such as wing dams and closing dams, as well as the use of maintenance dredging; however, these measures have an associated cost. In addition, there are several environmental concerns associated with the disposal of dredged material. Aside from the environmental concern, the physical deterioration of the various structures, including wing dams and closing dams, can also impact the need for dredging of the channel.

The Corps is currently undertaking a study to assess the future navigation needs and their economic, environmental, and other impacts. This project is part of that larger study. Examining the tradeoffs between costs, benefits, and reliabilities is a necessary part of the overall assessment that can eventually include uncertainties about the ecological impacts of navigation activities. Quantification of the navigation channel reliability is the first step in the development of a systematic framework for the management of the river navigation system that eventually includes examination of the tradeoffs among costs, benefits, and reliability.

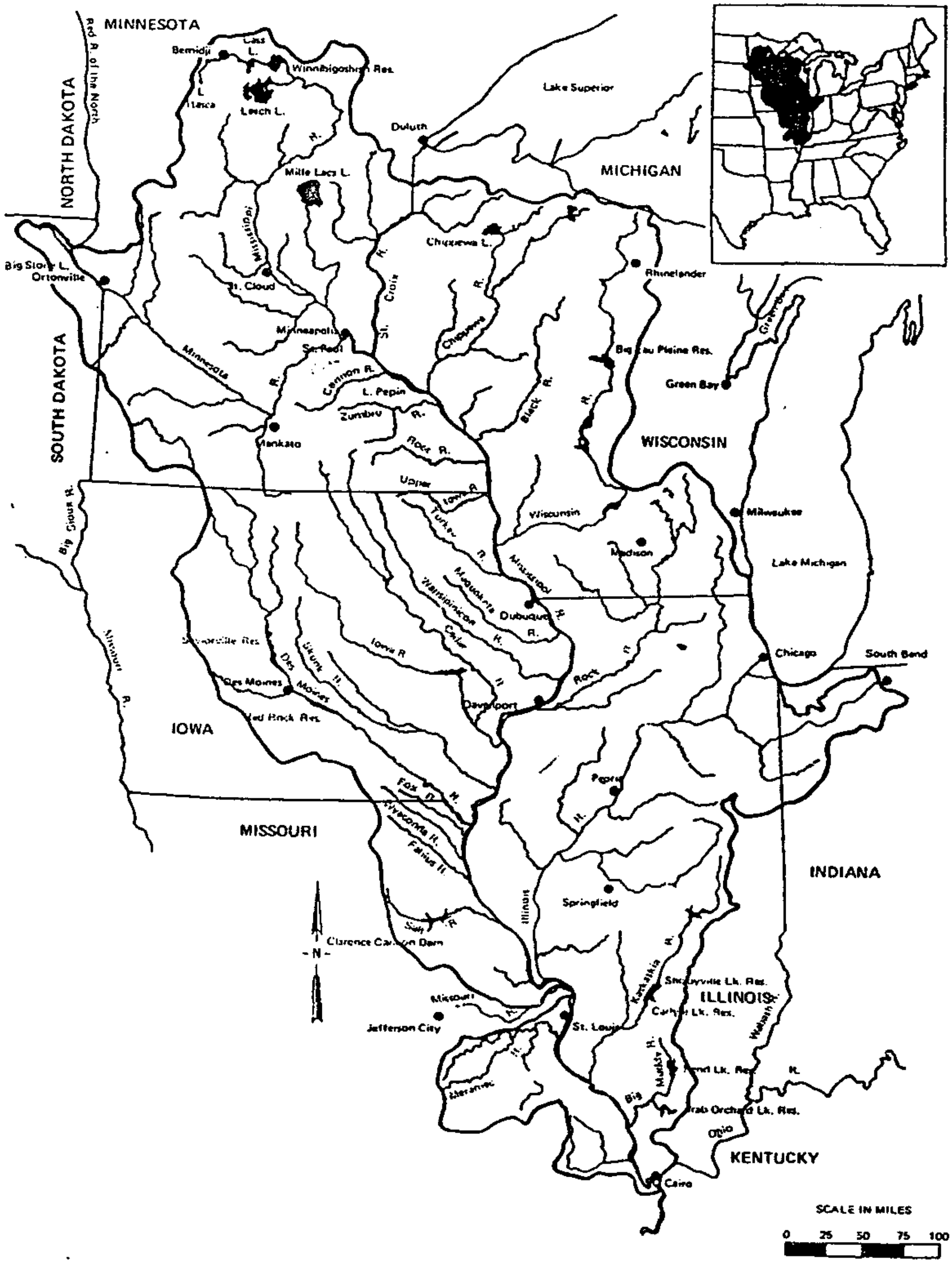

Figure 1.1 The Upper Mississippi River System

In order to develop this framework, we must first answer several questions: What exactly is the navigation channel reliability? Can we quantify it, that is, construct a reliability function for it? The answers to these questions will allow us to manage the navigation system effectively. In this project, we attempt to answer these questions and develop measures and models for the navigation channel reliability.

2. Previous Results

This section summarizes the work performed by the Center for Risk Management of Engineering Systems, University of Virginia, in Phase I of the project "Channel Reliability of the Navigation System in the Upper Mississippi River." This project has a heavy reliance on the shared documents and discussions among the Center and the Rock Island, St. Paul, and St. Louis Districts of the Corps of Engineers. In Phase I, two reliability models were developed for the navigation channel by characterizing the relationships among the annual dredge needs of the channel, the states of the navigation structures, and the river hydrograph. These models were applied to and evaluated in a representative sample of navigation pools of the Upper Mississippi River.

An important purpose of the reliability portion of the Upper Mississippi River navigation study is to project general funding requirements to maintain the navigation project in the future. The objective was not to decide exactly which projects should be built—a role which remains in the domain of professional engineering judgment, personal maintenance experience, and models of the physical processes involved—nor to give accurate forecasts of needed resources in the short term. Rather, the role of the models described in the Phase I report was to provide foundations upon which to quantify the benefits of increased rehabilitation funding for wings dams and closing dams on a system-wide basis and over a period of many years. There were two reliability models for the navigation channel discussed in that report—one associated with the need for dredging of the *pool* and the other with the dredging of the *reach*. These models are complementary approaches to demonstrating the reduced-dredging benefits associated with rehabilitation of channelization structures.

2.1 Dredge-Capacity Reliability Model

The dredge-capacity reliability model generates a probabilistic description of the annual dredge need for a given pool based on an assumed relationship between dredging and underlying features of the hydrograph. Underlying features that are considered include the average daily discharge through the pool, the number of falls of the hydrograph, and the number of flow peaks, where the hydrograph features are modeled as random variables. The dredge-capacity model also estimates a probability distribution of annual dredge need for the pool that is expected if *significant* rehabilitation is performed pool-wide. The two probability density functions, one of the unrehabilitated and one of the rehabilitated pool, are useful to characterize the variable cost of dredging the pool, or a system of pools that are similarly evaluated. A function relating the pool-dredging amount to the cost of dredging is required for this purpose.

In principle, one can use the probability function of dredge-need for the pool to evaluate the dredge-capacity reliability in the following two-step process: (1) define the annual capacity of the dredge operator(s) for the given pool; and (2) calculate the probability that the dredge need exceeds the capacity. A potential limitation of this approach that it is not common to fix the capacity of dredging for individual pools. In phase II, however, a capacity-based approach was used to allow economists to distinguish and characterize a failure of the system as an exceedance of the normal operating budget for dredging.

However, there is an intermediate result in this model that gives insight into the need for dredging on the pool-wide scale: the plot of annual dredge amount versus average daily discharge for the year (or versus some other alternate hydrograph feature, several of which were analyzed in Phase I of this project). From this curve it can be useful to study the impact of pool-wide rehabilitation on the relationship between dredging amount and the hydrograph in the year preceding dredging.

It is important that excessive dredging is not perceived as a failure by the channel user, but only by the channel operator. In real situations, the barge-and-tow operator is indifferent to the need for dredging and only learns of failure when the channel geometry is inadequate for the traffic. The *dredge-capacity reliability* is particularly relevant in planning for channel maintenance.

2.2 Reach Reliability Model

As a complement to the pool model described above, the evaluation of channel reliability can be extended down to the level of individual reaches to better understand and characterize the impacts on channel sedimentation associated with individual structural rehabilitation's. Thus, a reliability model that uses data on individual reaches as the statistical basis for an ideal characterization of sedimentation to the channel has been developed. This reach model can be used to generate a chart of the Upper Mississippi River on which the estimates of channel reliability are provided for all reaches together with the potential for improving the reach reliability by the rehabilitation of structures. Application of the reach model will yield a general picture of the benefits of rehabilitation—based on the identified significance of the parameters affecting sedimentation at the reach level—but the model is not able to recommend projects at specific reaches. The amount of dredging is not considered in this model because it is assumed that set-up costs dominate the cost differences (between dredging events) attributable to dredging volume for a particular reach.

The inter-dredge reliability model gives the probability that in some time interval no dredging is required in a particular reach. This probability is called the inter-dredge reliability. The inter-dredge model also estimates the improvement in inter-dredge reliability to be expected if the reach is rehabilitated. The inter-dredge model assumes that a reach can be characterized by a small set of parameters representing the channel morphology. A weighted sum of the parameter values, with weighting coefficients estimated from the real system, gives both the estimate of reliability and the expected improvement in reliability from rehabilitation. It is important to distinguish the outputs of this ideal-process model of inter-dredge reliability, which generates the frequency of the need to dredge expected from an idealized reach-by-reach model, from the observations of the real system, which are dredging records influenced by dredging policy shifts and other factors not related to the need to dredge.

Consider a group of reaches where dredging is performed. A plot can be generated of the inter-dredge reliability, for a given interval of time such as five years, as a function of river mile. Recall that inter-dredge reliability is generated, considering an idealized process of deposition to the channel reach, every 0.1 mile where dredging is performed. The potential improvement in inter-dredge reliability can be shown on the same plot. As a complement to the above plot of inter-dredge reliability and the potential for its improvement, a histogram can be generated showing the number of reaches in the group that fall in various ranges of inter-dredge reliability, both for unrehabilitated reaches and using the model of rehabilitated reaches. The results are useful as input to a model that evaluates the impact of dredging frequency on cost to the system operator. A function relating dredging frequency, for individual reaches, to the cost of dredging to the system operator would be required as a component of the cost analysis.

2.3 Comparison of Models

The reach model of reliability does not replace the predictive model of annual dredge volume for a pool. Rather, the two approaches are complementary and have different uses in assessing the state of the navigation channel. Both approaches are useful for demonstrating the benefits of rehabilitation to the channelization structures. It is important to consider that neither of the reliability measures, for the pool or for the reach, looks beyond the issue of dredging. For example, increased channel currents that hinder navigation and the environmental and other impacts of structures on the river are not accounted for in the models. These factors would have to be included in an engineering study of specific rehabilitation projects.

Table 2.1 compares the key features of the dredge-capacity and the inter-dredge models, each of which is described in detail in the Phase I Final Report (Center 1995). Used together, the models provide for a comprehensive evaluation of the reliability of the navigation channel from the viewpoint of the channel operator concerned with rehabilitation of channelization structures and maintenance dredging.

Table 2.1 Comparison of Dredge-Capacity and Inter-Dredge Reliability Models

Feature	*Dredge-Capacity Model*	*Inter-Dredge Model*
1. Spatial resolution of the process	Pool-based (~10-30 miles)	Reach-based (0.1 mile)
2. Formulation of reliability model	Loading-resistance model	Time-to-failure model
3. Considers frequency of dredging	No	Yes
4. Considers total volume of dredging	Yes	No
5. Quantifies impact of rehabilitation on dredging	Yes	Yes

Phase II of this project builds on the Dredge Capacity Reliability Model developed for the pool reach. It was decided that the Inter-dredge Reliability Model would not be further pursued in Phase II. Discussion of the two models is contained within the Final Report (Final Version dated January 31, 1995) for Channel Reliability of the Navigation System in the Upper Mississippi River, developed by the Center for Risk Management of Engineering Systems, University of Virginia (Center 1995).

3. The Capacity-Demand Model

In this section we will discuss the basis for the Dredge Capacity Reliability Model, which is problem formulation in terms of a capacity-demand model. The approach selected for the assessment of system reliability is in terms of a capacity-demand model, where the capacity and the demand are represented by probability distributions (Harr 1987; Ang and Tang 1984). A similar approach is used for the Corps maintained structural components of the navigation locks and dams in Shannon and Wilson (1994). In this formulation, the demand distribution is dredge demand for the dredging system or a pool and is computed using the model developed in Phase I. The capacity distribution is a function of the availability of the dredge for the system or for that particular pool. In this section, we illustrate the relationships required to obtain the reliability for the system and for each pool.

Let us assume that the dredging capacity available for pool i, C_{di}, is measured in cubic yards per year, and that the dredge demand of pool i, D_i, is also measured in terms of cubic yards per year as shown in Figure 3.1a.

The probability of failure for pool i, p_i, is then given by the probability that the pool dredge capacity (C_{di}) is less than the pool dredge demand (D_i). This probability of failure is shown by the shaded area in Figure 3.1a.

$$p_i = Pr(C_{di} < D_i) \tag{3.1}$$

An alternative formulation is the use of a safety margin for pool i, SM_i. The safety margin is defined as the difference between the pool dredge capacity (C_{di}) and the pool dredge demand (D_i).

$$SM_i = C_{di} - D_i \tag{3.2}$$

The probability of failure for pool i, p_i, is then given by the probability that the safety margin is less than zero and is shown by the shaded area in Figure 3.1b.

$$p_i = Pr(SM_i < 0) \tag{3.3}$$

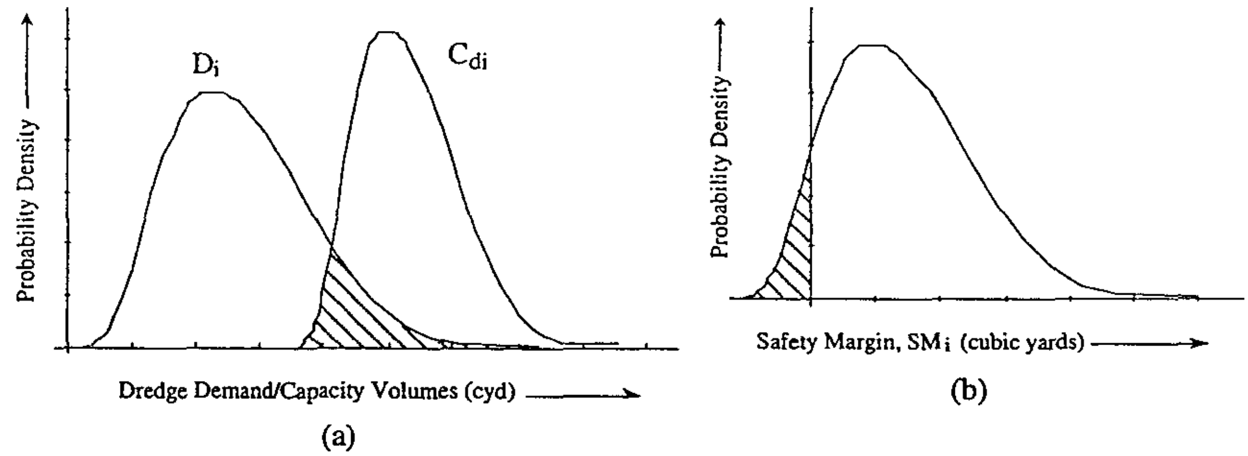

Figure 3.1 Capacity-Demand and Safety Margin Models for Waterway Navigation Systems

The reliability index (β_i) for a pool has been defined as the ratio of the mean of the safety margin to its variance (USACE 1992). The β-index will not be applied to the pools in this project.

$$\beta_i = \frac{E[SM_i]}{\sigma_{SM_i}} = \frac{E[C_{di} - D_i]}{\sqrt{\sigma^2_{C_{di}} + \sigma^2_{D_i}}} \tag{3.4}$$

where σ_{SM_i} is the standard deviation of safety margin in pool i,
$\sigma_{C_{di}}$ is the standard deviation of dredge capacity in pool i, and
σ_{D_i} is the standard deviation of dredge demand in pool i

3.1 System Reliability

Once the dredge demand and capacity distributions for the system have been obtained, we can obtain the distribution of the safety margin. This distribution can then be used to obtain the value of the probability of failure for the system. The moments of the safety margin can be computed given the moments of the dredge demand and the dredge capacity. Specifically, we have:

$$\mu_{SM_{ds}} = \mu_{C_{ds}} - \mu_{D_{ds}} \tag{3.5}$$

and

$$\sigma_{SM_{ds}} = \sqrt{\sigma^2_{C_{ds}} + \sigma^2_{D_{ds}}} \tag{3.6}$$

where $\mu_{SM_{ds}}$ is the mean safety margin for the dredge system,
$\mu_{C_{ds}}$ is the mean dredge capacity for the dredge system,

$\mu_{D_{ds}}$ is the mean dredge demand for the dredge system
$\sigma_{SM_{ds}}$ is the standard deviation of safety margin for the dredge system,
$\sigma_{C_{ds}}$ is the standard deviation of dredge capacity for the dredge system, and
$\sigma_{D_{ds}}$ is the standard deviation of dredge demand for the dredge system

assuming that the capacity and demand are independent. The probability of failure can be seen as the area under the curve in figure 3.1b that is less than zero. The value of the reliability index (β_{ds}) for the system can be computed using equation 3.4 as (USACE 1992)

$$\beta_{ds} = \frac{E[SM_{ds}]}{\sigma_{SM_{ds}}} \quad \text{for normal distributions} \tag{3.7}$$

3.2 Pool Reliability

The distribution of pool capacity (C_i) can be established based on the following two limit states:

3.2.1 Pool Dredging Capacity

System failure is the condition when the dredge demand exceeds the dredge capacity. However, pool failure would not necessarily occur in cases where the pool dredging capacity is exceeded, since excess dredging capacity could potentially be moved from other pools. In order to evaluate the pool reliability, we define a variable, θ_i, for each pool such that

$$\theta_i = \frac{\mu_{D_i}}{\mu_{D_{ds}}} \tag{3.8}$$

where μ_{D_i} is the mean dredge demand for pool i, and
$\mu_{D_{ds}}$ is the mean system dredge demand

The system dredging capacity is therefore allocated to each pool based on its expected demand, or

$$C_{di} = \theta_i \bullet C_{ds} \tag{3.9}$$

where C_{di} is the dredge capacity for pool i

3.2.2 *Pool Placement Capacity*

A second limit state for each pool reach will be imposed based on the availability of dredged material placement sites within each pool, (C_{pi}).

3.2.3 *Pool Capacity*

Exceedance of either the dredging capacity or the placement capacity defined above in undesirable condition. So that:

$$C_i = \min [C_{di}, C_{pi}]$$

where C_i = Minimum Capacity for Pool i based on dredging capacity and placement site availability.

For the current phase of this project it was determined by the Corps Districts that the placement capacity was not a limiting constraint in any of the three Districts and not anticipated through the 50 year planning horizon of this project. However, placement capacity could become a system constraint at some time in the future. As environmentally acceptable placement sites become more scarce and harder to find in the future, dredging costs will increase accordingly. Therefore the pool dredging capacity was not considered for further analysis.

Ignoring the pool placement capacity, we can then define the failure of a pool i to occur when its demand is more than its "fair share" of the system capacity (condition A) *and* the system demand is greater than the system capacity (condition B).

where ***A*** is the condition that $\theta_i C_{ds} < D_i$, and
B is the condition that $C_{ds} < D_s$

Then

$$\begin{aligned}
\Pr(A \text{ AND } B) &= \text{Probability of Failure} = \Pr(A) \bullet \Pr(B|A) \\
&= \Pr(\theta_i C_{ds} < D_i) \bullet \Pr(C_{ds} < D_s | \theta_i C_{ds} < D_i) \\
&= \Pr(\theta_i C_{ds} < D_i) \bullet \Pr((1-\theta_i) C_{ds} < D_{\neg i}) \qquad (3.10)
\end{aligned}$$

Equation 3.10 defines the condition of pool failure as the product of the probability that the pool demand is exceeding its fair share of the system capacity and the probability that the demand in the rest of the system is exceeding the capacity remaining in the system after allocating the capacity for pool i. Note that the reliability index (β_i) for each pool is not meaningful since the probability of pool failure relies jointly on two events: the allocated pool capacity exceedance and the rest-of-system capacity exceedance. Therefore the pool reliability index is not computed.

Part II

Dredge Demand

4. Assessment of Dredge Demand

The dredge-capacity reliability model developed in Phase I generates a probabilistic description of the annual dredge demand for a given pool based on an assumed relationship between dredging and underlying features of the hydrograph measured in terms of the discharge or the stage. Underlying features that were considered include the average daily discharge through the pool, the number of falls of the hydrograph, and the number of flow peaks, where the hydrograph features are modeled as random variables. The average daily discharge was the variable selected for use in the model. In addition to the annual dredge demand for the unrehabilitated pool condition, the dredge-capacity model also estimates a probability distribution of annual dredge demand for the pool that is expected if significant rehabilitation is performed pool-wide. By significant rehabilitation, we mean that four or more navigation structures have been rehabilitated in the time period under study. These two probability density functions, one of the unrehabilitated pool and one of the rehabilitated pool, are useful to characterize the variable cost of dredging the pool, or a system of pools that are similarly evaluated. A function relating the pool-dredging amount to the cost of dredging is required for this purpose.

The term "rehabilitated pool" thus means that a portion of the channel training structures within the pool have been rehabilitated during the study period (1973-1993) in an attempt to reduce the dredging. *It does not indicate that all of the training structures within the pool have been rehabilitated.*

In principle, one could use the probability function of dredge-demand for the pool to evaluate the dredge-capacity reliability in the following two-step process: (1) define the annual capacity of the dredge operator for the given pool; and (2) calculate the probability that the dredge demand exceeds the capacity. A potential limitation of this approach is that it is not common to fix the capacity of dredging for individual pools.

However, an intermediate result in this model gives an insight into the demand for dredging on the pool-wide scale: the plot of annual dredge amount versus average daily discharge for the year (or versus some other extremal-oriented hydrograph feature). From this curve the impact of pool-wide rehabilitation on the relationship between dredging amount and the hydrograph in the year preceding dredging is studied.

It is important that excessive dredging is not perceived as a failure by the channel user, but only by the channel operator. In real situations, the barge-and-tow operator is indifferent to the demand for dredging and only learns of failure when the channel geometry is inadequate for the traffic. The *dredge-capacity reliability* developed in this model is particularly relevant to the manager who is planning for channel maintenance.

Assumptions of the Pool-Based Model

The pool-based model is an attempt to relate the discharge hydrograph for a pool to the maintenance dredging performed at the end of a given year in the same pool. Importantly, the model demonstrates that structural rehabilitation changes the relationship between the discharge and the dredging on a pool-wide and on a year-to-year scale. Results were developed for predicting the annual dredge volume for an entire pool from either:

(i) the year-average of daily discharge/stage in the pool (equivalently, total discharge for the year) in the year preceding the dredging; or
(ii) the number of daily falls in the pool hydrograph that exceeded the 95th percentile of magnitude in the year preceding the dredging.

For a given pool, the pool-based model predicts the amount of dredged material in a year from the daily hydrograph over a period of a year preceding the dredging. Specifically, a regression approach is used to relate the dredging amount (cubic yards) to either the number of daily falls in river stage that exceed a given threshold of magnitude or the average daily discharge. This regression approach is tested and extended to examine the following two cases: (1) pool/reach structures (wing and closing dams) in good condition, and (2) pool/reach structures in deteriorated conditions. The underlying assumption was that abrupt falls in the river stage result in a net sediment deposition in the channel. This assumptions was subsequently confirmed in phase I of this study. In phase I, we discovered that the number of large daily falls of the hydrograph is strongly correlated with the number of large daily rises in the same year. The mechanism of sedimentation in the channel crossings is that increases in flow cause deposition in the channel bed which are then scoured out by falls in the flow. However, rapid falls in the flow prevent these deposits from scouring out, thus leading to *net* sediment deposition from the same river stage prior to the rise. In addition, a higher daily discharge increases the sediment carrying capacity of the river and thus more sediment is available for deposition. An example of this process is the 1993 flood when there was a high peak followed by a slow fall. This long fall scoured out the deposits on the crossings, leading to reduced dredging requirements for the channel than is expected from such a flood of that magnitude. Note that this sedimentation mechanism is valid for the pools of the Upper Mississippi River, the open river mechanism is different.

Several approaches for predicting the pool annual dredge volume were tried prior to that of using approaches (i) and (ii) described above. Some alternative approaches considered were the use of the peak annual hydrograph; the number of days discharge hydrograph is above a given threshold flow; and the number of rises together with the number of falls in up to 10 prespecified ranges of magnitude. These alternative approaches were tested with various time lags and averaging (smoothing) of the dependent and independent variables. It turned out that the simpler models gave predictive results that were equivalent in quality to the more complex approaches. In particular, a linear relationship between the year-average discharge and the annual dredge volume is found acceptable. Furthermore, results were obtained to suggest the impact of rehabilitating channelization structures—the linear dredge-discharge relationship showed a reduction in dredging from pre-rehabilitation to post-rehabilitation conditions for all magnitudes of discharge.

Since the probability distributions of the number of falls in a year (as well as the average daily discharge) can be estimated from the data, the functional relationship between either the average daily discharge or the falls and the dredged amount determines the probability distribution for the dredged amount. The distribution describing the dredged amount makes it possible to evaluate a measure of the reliability of the pool. The *dredge-capacity reliability* is defined as the probability that the dredging required in a pool or reach exceeds some predefined value in a year. In this model, a failure of the channel or pool is assumed to occur when the dredging required exceeded the capacity of the dredging system. This approach conforms to the standard formulation in reliability engineering in which both the potential "loading" on a design and the "resistance" of the design are considered. The reliability is then calculated as the probability that the resistance exceeds the loading for the planned lifetime of the design. Therefore, this model takes the annual dredge demand to be a "loading" on the channel system and the fixed capacity of the dredge providers for a given pool to be the "resistance." The pool model requires that any pool in which the model is to be applied be tested to estimate its own (potentially linear) relationship between dredge demand and discharge on the year-to-year scale. In the following section, we estimate these relationships for the pools of the Upper Mississippi River where historical dredging records are available.

5. Assessment of Pre-Rehabilitation Dredge Demand

Figure 5.1 depicts an example hydrograph for the Upper Mississippi River at Keokuk for the year 1973. Since we assume that the sedimentation rate is a function of the river discharge, we demand to relate discharge to sedimentation to the channel.

Dredging records thus help to establish a baseline for the dredge demand in a particular pool. Figure 5.2 shows the actual amount of dredging in Pool 18 during the period 1973-1990. This model is therefore meant to be applied to each complete pool.

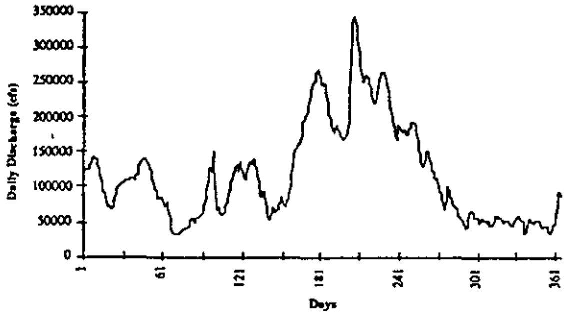

Figure 5.1 Example hydrograph for the Upper Mississippi River at Keokuk

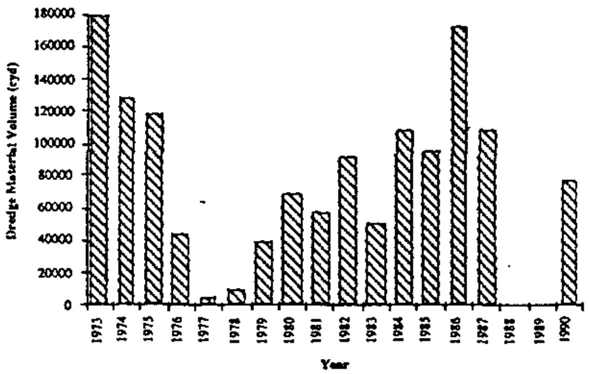

Figure 5.2 Quantity of material dredged from Pool 18 during 1973-1990

5.1 Prediction of Dredge Demand for a Pool: Regression Model

Once we obtain the plots shown in Figures 5.1 and 5.2, we can perform a regression of the volume of dredge material against the average daily water discharge. Such a plot of the data in Figures 5.1 and 5.2 is shown in Figure 5.3 for the period 1973-1990 (the year corresponding to each data point is indicated in the figure).

Notice in Figure 5.3 that the relationship between annual dredge volume and average daily discharge can be represented roughly by a linear function, with the exception of three outlier points.

Pool 18 underwent significant (15 navigation structures) rehabilitation work in the period 1983-86. This offers us the opportunity to analyze the impact of this historical rehabilitation work. Therefore we exclude the data for those years and use the two data sets 1973-83 and 1987-91 to develop two regression models for pre-rehabilitation and post rehabilitation relationships. Table 5.1 shows the average daily discharge at Keokuk and the annual dredging amount for the pre-rehabilitation period.

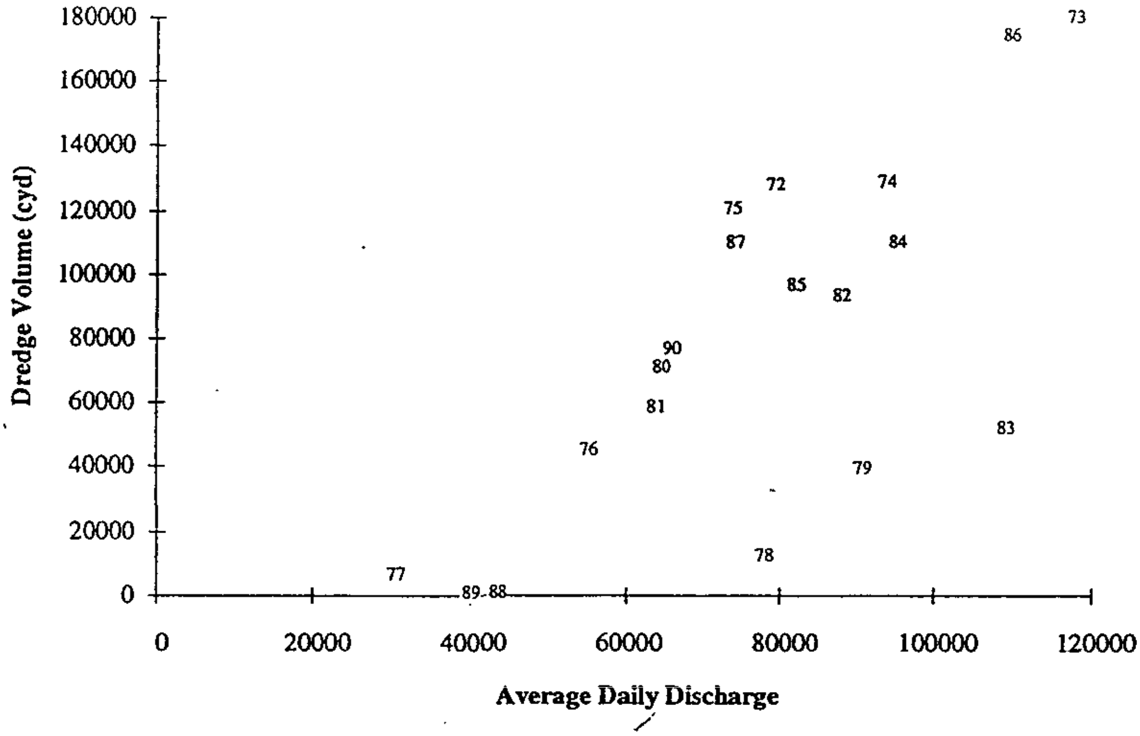

Figure 5.3 Plot of annual volume of dredge material vs. average annual discharge for Pool 18

Table 5.1 Pre-Rehabilitation Dredge and Discharge Data for Pool 18

YEAR	Actual Dredge Volume (yd^3/year)	Average Daily Discharge (cfs)	Predicted Dredge Volume (yd^3/year)
1973	178812	118553	124168
1974	127855	93714	91947
1975	119230	74066	66459
1976	44151	55340	42167
1977	5885	30382	9791
1978	11166	77995	71556
1979	38911	90534	87821
1980	70380	64423	53950
1981	57490	63877	53242
1982	92650	87934	84449
1983	50986	109147	111967

Equation 5.1 shows the regression models for the pre-rehabilitation data set. The regression results show a correlation of 0.617 between the dredge volume and the discharge and a value of R^2 of 0.38.

$$D_{pre\text{-}rehab} = 29620 + (1.297)(Q) \qquad (5.1)$$

where $D_{pre\text{-}rehab}$ is the dredge demand prior to the structural rehabilitation during 1983-86

Equation 5.1 is used to compute the predicted dredge demand for Pool 18 as shown in Table 5.1. These values for the predicted dredge demand are then used to compute the mean and standard deviation of the dredge demand distribution. Assuming that the dredge demand follows a normal distribution, we can then develop a distribution of dredge demand for the pool in the pre-rehabilitation condition as shown in Figure 5.4.

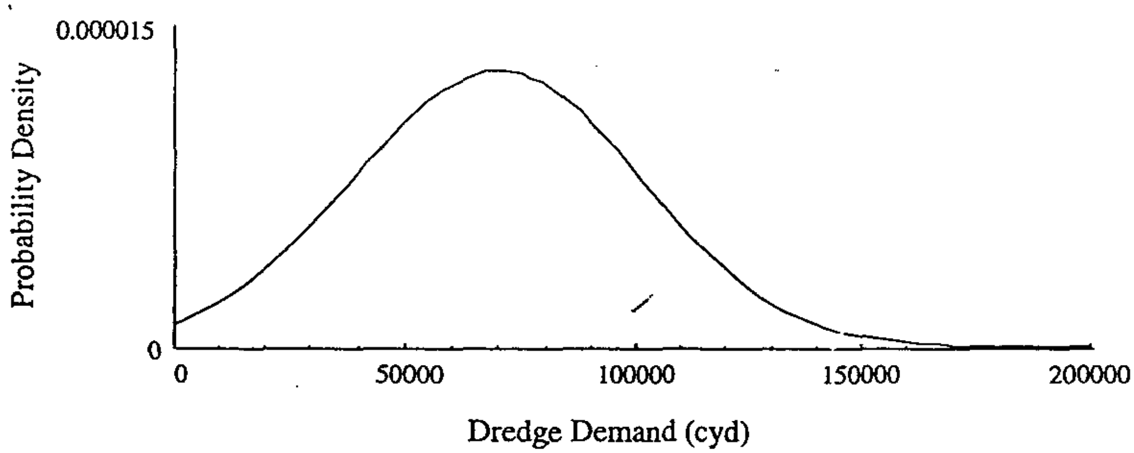

Figure 5.4 Pre-Rehabilitation Dredge Demand for Pool 18

5.2 Prediction of Dredge Demand for a System

From the study of the results for average daily discharge for the year and the number of daily falls in discharge in that year exceeding the 95th percentile, it was determined that the average daily discharge (or stage in cases where the discharge data was not available) was at least as good a predictor. Therefore, the average annual discharge/stage was used to compute the predicted dredge demands for the pools. These are summarized in Table C.2 for Rock Island District. The sum of the demands of the individual pools (D_i) gives us the system dredge demand (D_{ds}).

$$D_{ds} = \sum_i D_i \qquad (5.2)$$

The determination of the rest-of-system dredge demand, $D_{\neg i}$, is required for the computation of the pool reliability. This can be computed for each pool i by subtracting the pool demand, D_i, from the system demand, D_{ds}. The computed data for the rest-of-system demand is shown in Table C.3 in Appendix C for Rock Island District.

$$D_{\neg i} = D_{ds} - D_i \qquad (5.3)$$

Part III

Dredge Capacity

6. Assessment of Dredge Capacity

It is important that excessive dredging is not perceived as a failure by the channel user, but only by the channel operator. In real situations, the barge-and-tow operator is indifferent to the need for dredging and only learns of failure when the channel geometry is inadequate for the traffic. The *dredge-capacity reliability* developed in this model is particularly relevant to the manager who is planning for channel maintenance.

6.1 System Dredge Capacity

The distribution of dredge demand for each pool has been computed as shown in the previous section. Thus in order to compute the reliability for the navigation channel in each pool, we need to develop a dredge capacity for each pool. The distribution of dredge capacity (C_{ds}) for each dredging system is defined as:

$$C_{ds} = \left(\frac{1}{S + T + V/P}\right) \bullet M \bullet V \tag{6.1}$$

where C_{ds} = dredging capacity for the system (yd^3/year)
M = maximum number of days in a dredging season based on the normal O&M budget (days/year)
V = dredge volume per site (yd^3)
S = dredge setup time in days (e.g., 1-2 days/site)
T = dredge travel time in days (e.g., 1-2 days/site)
P = dredge production rate (yd^3/day)

The first part of equation 6.1 gives us an estimate of the time spent per site, that multiplied by the number of days in the dredging season (M), gives us the maximum number of sites dredged per year, which is then multiplied by the dredge volume per site (V) to obtain the system dredge capacity. Some of these variables are deterministic (point values) while others can be represented by distributions. We can therefore model these as random variables and calculate the means and standard deviations of these distributions either from historical data or subjectively from management information. Table 6.1 shows the data provided by the Corps for the computation of system dredge capacity. Given the values of the moments of the variables and the functional relationship between them (equation 6.1), we can calculate the approximate mean and standard deviation of the system dredging capacity using a Taylor Series expansion (Benjamin and Cornell 1970).

Table 6.1 Values of Capacity Variables for Rock Island District (Dredge Thompson)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	71	52,902	0.21	0.50	12,000	734,055
σ	0	39,544	0.00	0.00	0	145,905

6.2 Pool Capacity

In principle, one could use the probability function of dredge-need for the pool to evaluate the dredge-capacity reliability in the following two-step process: (1) define the annual capacity of the dredge operator for the given pool; and (2) calculate the probability that the dredge need exceeds the capacity. A potential limitation of this approach is that it is not common to fix the capacity of dredging for individual pools.

In order to evaluate the pool reliability, we define a variable, θ_i, for each pool which is computed according to equation 3.8. The system capacity is therefore allocated to each pool using equation 3.9. Table 6.2 shows the value of the capacity allocation variable (θ_i) and the resultant pool capacities for each of the pools of the Rock Island District.

Table 6.2 Allocated Dredge Capacity for the Pools of the Rock Island District (cyd/year)

Pool	11	12	13	14	15	16	17	18	19	20	21	22	System
θ_i	0.1505	0.0275	0.1007	0.0292	0.0012	0.0599	0.0238	0.1151	0.0357	0.2184	0.1115	0.1264	1.0000
μC_{di}	110,504	20,161	73,923	21,456	886	43,963	17,466	84,523	26,182	160,332	81,868	92,788	734,055
σC_{di}	56,610	24,180	46,302	24,945	5,070	35,707	22,506	49,510	27,556	68,189	48,726	51,875	145,905

7. Assessment of Probability of Failure

7.1 System Failure

System failure is defined as the condition when the dredge demand exceeds the dredge capacity for a dredge system. Assuming that the capacity and the demand are independent, equations 3.5 and 3.6 can be used to compute the moments (mean and standard deviation) of the safety margin. Since the dredge capacity we computed is an upper bound on the volume of dredge material that can be dredged in any particular year under the normal O&M budget, we can assume that the dredge capacity and demand are independent random variables. Figure 7.1 shows the plots of the system dredge capacity, system dredge demand, and the system safety margin assuming that these variables are normally distributed.

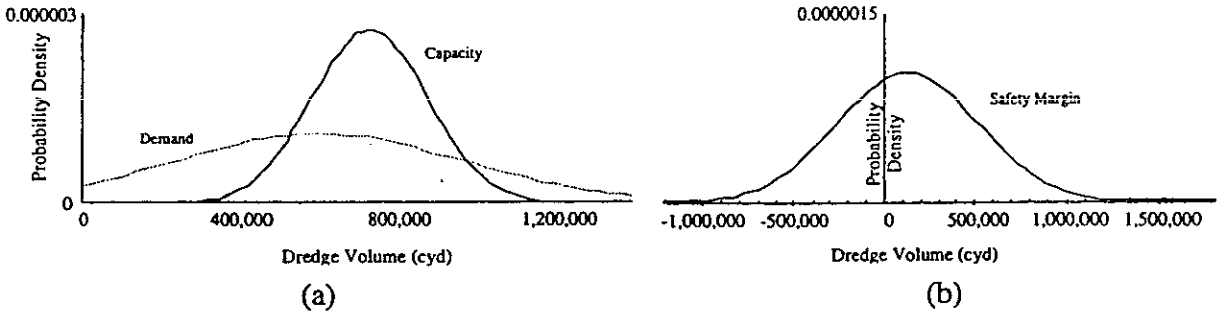

Figure 7.1 Pre-Rehabilitation Capacity, Demand, and Safety Margin Plots for Rock Island District

The probability of system failure can be seen as the area under the curve in figure 7.1b that is less than zero and can be evaluated from the normal probability tables as 0.3745. The value of the reliability index (β_{ds}) for the system can be computed using equation 3.4 as

$$\beta_{ds} = \frac{E[SM_{ds}]}{\sigma_{SM_{ds}}} = \frac{126,929}{393,073} = 0.32$$

7.2 Pool Failure

Note again that system failure is the condition when the dredge demand exceeds the dredge capacity. However, pool failure does not necessarily occur when the allocated pool capacity is exceeded, since excess capacity could potentially be moved from other pools. Pool failure is defined as the joint occurrence in a year of the dredge demand exceeding the allocated dredge capacity for a pool and system failure.

Given the dredge demand for each pool (Table C.3) and the allocated dredge capacities for the pools (Table 6.2), we can now compute the safety margin for the pools of the Rock Island District. Equation 3.10 is then used to compute the probability of pool failure. Table 7.1 gives these pool failure probabilities. Table C.5 shows the intermediate computational results for obtaining these probabilities for the Rock Island District dredging system.

Table 7.1 Probability of System and Pool Failure for Historical Pre-Rehabilitation Condition (Rock Island District)

Pool	11*	12	13*	14*	15	16*	17	18*	19	20*	21*	22*	System
p_i	0.1576	0.1681	0.1645	0.1679	0.1828	0.1577	0.1693	0.1533	0.1666	0.1533	0.1489	0.1589	0.3745

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the pre-rehabilitation data only

Part IV

Results for St. Paul District

8. Analysis of Dredging Operations in St. Paul District

8.1 Assessment of Pre-Rehabilitation Dredge Demand

Table B.1 shows the historical dredging data for and Table B.2 shows the average annual discharge for the pools of the St. Paul District. We use these data in the procedure developed in section 5 to compute the predictive dredging demand for the pools in the St. Paul District. These predictive dredging volumes are shown in Table B.3. Figure 8.1 shows the plots of the pre-rehabilitation dredging volumes vs. the average annual discharge or the stage for the 11 pools on the Upper Mississippi River in the St. Paul District. The 'x' marks shows the actual dredge-discharge data and the straight line shows the predictive linear regression fit to the data points.

Based on the regression data for the time period under consideration (1975-1993), we can calculate the means and standard deviations of the dredge demand for each pool as given in Table 8.1.

Table 8.1 Pre-Rehabilitation Dredge Demand for the Pools of the St. Paul District (cyd/year)

Pool	1	2	3	4	5 *	5A	6	7 *	8	9	10	System
μD_i	30,953	86,319	8,195	251,297	65,217	37,334	7,839	34,149	44,379	31,211	24,735	621,628
σD_i	2,330	32,427	3,500	82,089	52,905	4,259	1,131	11,965	13,128	6,286	3,762	201,061

* — rehabilitation was performed in pools 5 and 7 between 1975-1993 and the demand is calculated from the pre-rehabilitation data only

8.2 System Dredge Capacity

The St. Paul Corps District employs three different dredging systems; the hydraulic dredge Thompson, government mechanical dredge, and contract mechanical dredge. In order to compute the system dredge capacity we need to compute the dredge capacities of these individual dredge subsystems. We can use equation 6.1 to compute the mean and standard deviations of the dredge capacity for each of the dredge subsystems. Tables 8.2, 8.3, and 8.4 show the input data and the results of the three subsystems.

Table 8.2 Values of Capacity Variables for St. Paul District (Dredge Thompson)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	150	50,398	0.33	0.50	12,000	1,501,974
σ	0	55,416	0.00	0.00	0	1,349,215

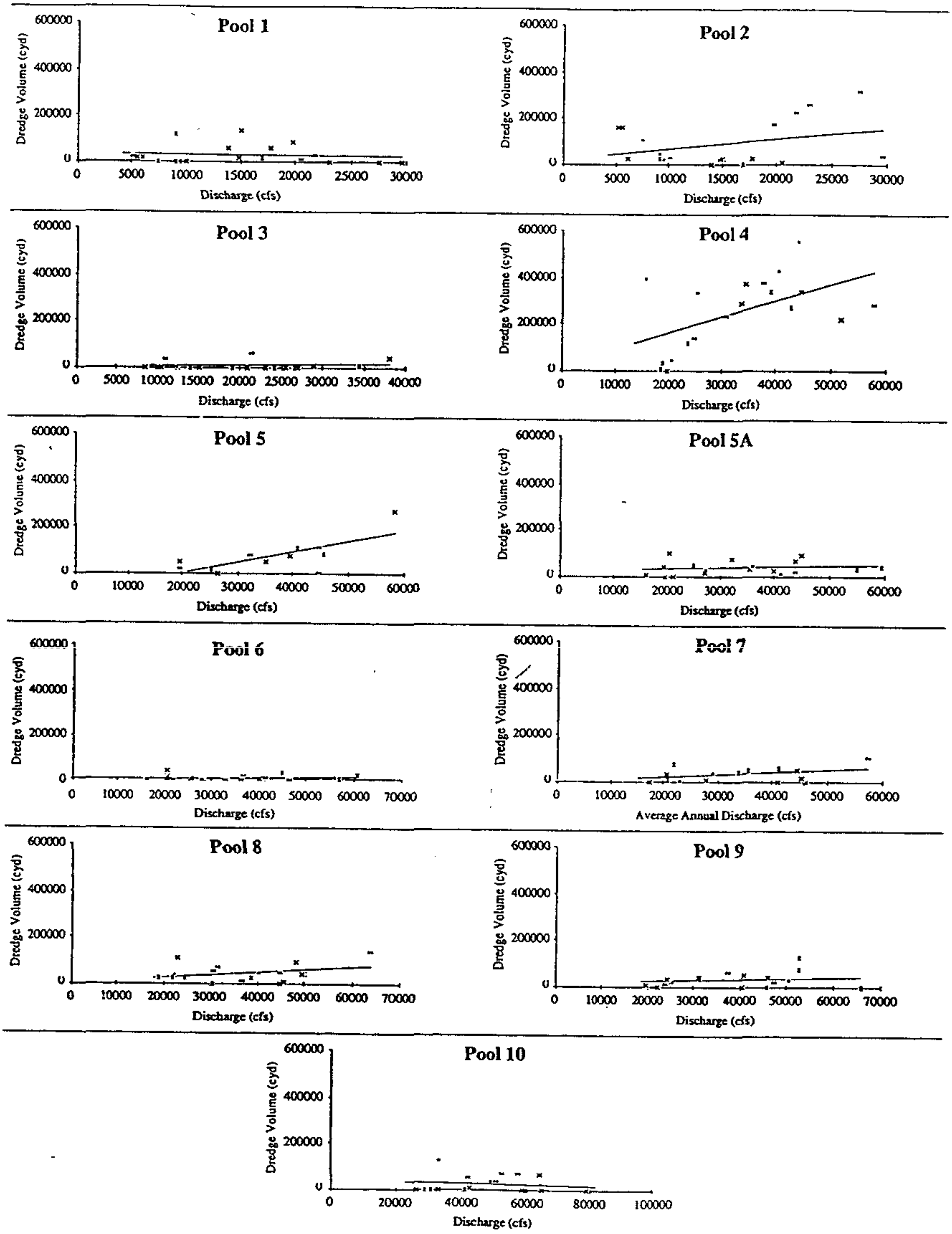

Figure 8.1 Summary Pre-Rehabilitation Dredge vs. Discharge Plots for St. Paul District

Table 8.3 Values of Capacity Variables for St. Paul District (Government Mechanical)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	120	10,402	0.04	0.50	2,340	250,299
σ	0	10,548	0.00	0.00	0	72,044

Table 8.4 Values of Capacity Variables for St. Paul District (Contract Mechanical)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	120	12,452	0.04	0.50	2,340	254,856
σ	0	10,540	0.00	0.00	0	37,684

The system dredge capacity is the sum of the three dredging subsystem capacities as calculated in Tables 8.2-8.4.

8.3 Pool Dredge Capacity

Using the approach outlined in section 6, we can now compute the allocated dredge capacities for the St. Paul pools based on the dredge demands from Table 8.1. These capacities are shown in Table 8.5.

Table 8.5 Allocated Pre-Rehabilitation Dredge Capacities for the St. Paul Pools (cyd/year)

Pool	1	2	3	4	5*	5A	6	7	8	9	10	System
θ_i	0.0498	0.1389	0.0132	0.4043	0.1049	0.0601	0.0126	0.0549	0.0714	0.0502	0.0398	1.0000
$\mu_{C_{di}}$	99,942	278,709	26,460	811,394	210,574	120,545	25,311	110,261	143,292	100,775	79,865	2,007,129
$\sigma_{C_{di}}$	301,616	503,682	155,195	859,403	437,808	331,250	151,787	316,805	361,154	302,870	269,624	1,351,663

8.4 Pool Reliability

The determination of the rest-of-system dredge demand, $D_{\neg i}$, is required for the computation of the pool reliability. This can be computed for each pool i by subtracting the pool demand, D_i, from the system demand, D_{ds}. The computed data for the rest-of-system demand is shown in Table B.4.

Given the dredge demand for each pool (Table 8.1) and the allocated dredge capacities for the pools (Table 8.5), we can now compute the probabilities of pool failure as given in Table 8.6.

Table B.5 shows the intermediate computational results for obtaining these probabilities for the St. Paul dredging system. The reliability index (β) for the system is computed as 1.01 and the pre-rehabilitation probability of system failure is 0.1562.

Table 8.6 Pre-Rehabilitation Probability of System and Pool Failure for St. Paul District

Pool	1	2	3	4	5 *	5A	6	7 *	8	9	10	System
Pi	0.0710	0.0725	0.0728	0.0786	0.0712	0.0707	0.0718	0.0714	0.0714	0.0710	0.0720	0.1562

* — rehabilitation was performed in pools 5 and 7 between 1975-1993 and the demand is calculated from the pre-rehabilitation data only

8.5 Impact of Historical Rehabilitation

During the period under study (1975-1993), Pools 5 and 7 in the St. Paul District have undergone rehabilitation of navigation structures. Note that the term "pool rehabilitation" does not mean that every channel training structure was rehabilitated, just that some of the structures which were determined to be in need of repair were. Data for Pool 5 where there was sufficient record of post-rehabilitation record was examined to see if there a trend of reduction in post-rehabilitation dredging volumes could be established. Figure 8.2 shows the plots of the dredge volumes vs. the average annual discharge for Pool 5 on the Upper Mississippi River in the St. Paul District. The '+' marks show the dredge-discharge data for the post-rehabilitation data. The straight lines show the linear regression fit to the data points with the solid line showing the pre-rehabilitated condition and the dashed line the post-rehabilitated condition.

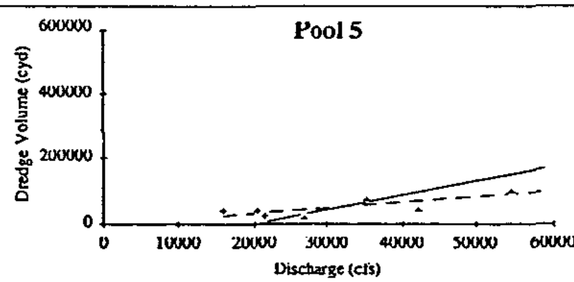

Figure 8.2 Summary Post-Rehabilitation Dredge vs. Discharge Plot for Pool 5

Based on this historical post-rehabilitation dredging volumes, we can update Table B.3 and B.4. The updated worksheets are shown in Table B.6 and B.7. The means and standard deviations of the post-rehabilitation dredge demand for each pool can now be calculated and these are given in Table 8.7.

Table 8.7 Post-Rehabilitation Dredge Demand for the Pools of the St. Paul District (cyd/year)

Pool	1	2	3	4	5*	5A	6	7	8	9	10	System
μ_{D_i}	30,953	86,319	8,195	251,297	54,570	37,334	7,839	34,149	44,379	31,211	24,735	610,981
σ_{D_i}	2,330	32,427	3,500	82,089	19,625	4,259	1,131	11,965	13,128	6,286	3,762	167,869

* — rehabilitation was performed in the pool between 1975-1993 and the demand is calculated from the pre-rehabilitation data only

Using the approach outlined in section 6, we can now compute the allocated post-rehabilitated dredge capacities for the St. Paul pools. These capacities are shown in Table 8.8.

Table 8.8 Allocated Post-Rehabilitation Dredge Capacities for the St. Paul Pools (cyd/year)

Pool	1	2	3	4	5*	5A	6	7	8	9	10	System
θ_i	0.0507	0.1413	0.0134	0.4113	0.0893	0.0611	0.0128	0.0559	0.0726	0.0511	0.0405	1.0000
$\mu_{C_{di}}$	101,683	283,566	26,921	825,534	179,267	122,646	25,752	112,183	145,789	102,531	81,257	2,007,129
$\sigma_{C_{di}}$	304,233	508,051	156,541	866,858	403,954	334,123	153,103	319,553	364,287	305,498	271,963	1,351,663

The determination of the rest-of-system dredge demand, D_{-i}, is required for the computation of the pool reliability. The computed data for the post-rehabilitation rest-of-system demand is shown in Table B.7.

Given the post-rehabilitation dredge demand for each pool (Table 8.7) and the allocated dredge capacities for the pools (Table 8.8), we can now compute the probabilities of pool failure as given in Table 8.9. Table B.8 shows the intermediate computational results for obtaining these probabilities for the St. Paul dredging system. The reliability index (β) for the system is computed as 1.03. Figure 8.3 shows a comparison of the pool reliabilities in the pre-rehabilitated and the post-rehabilitated conditions.

Table 8.9 Post-Rehabilitation Probability of System and Pool Failure for St. Paul District

Pool	1	2	3	4	5*	5A	6	7	8	9	10	System
P_i	0.0700	0.0708	0.0718	0.0777	0.0696	0.0690	0.0706	0.0703	0.0697	0.0700	0.0702	0.1515

* — rehabilitation was performed in pool 5 between 1975-1993 and the demand is calculated from the post-rehabilitation data only

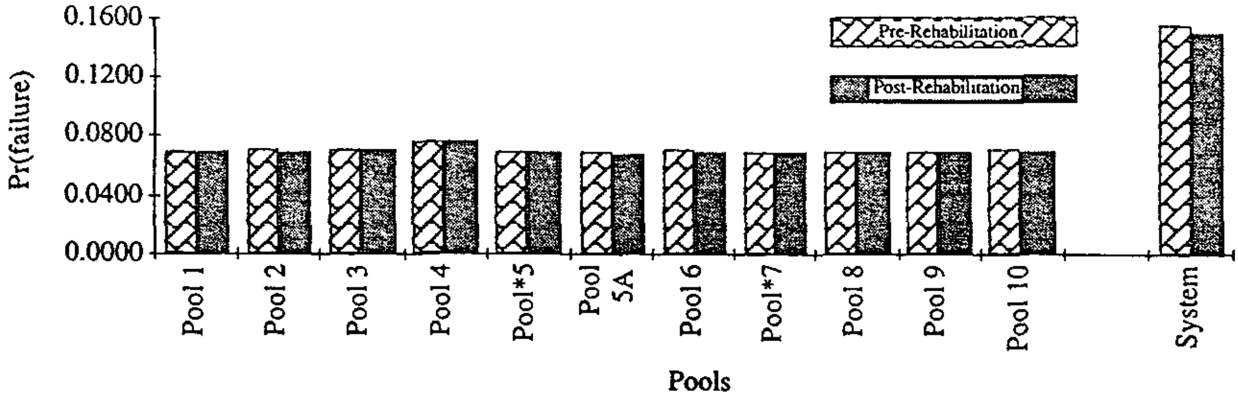

Figure 8.3 Comparison of Pre and Post Rehabilitation Reliabilities for St. Paul District for historical rehabilitation between 1975-1993

8.6 Impact of Hypothetical Increase in Capacity

Let us assume that we can increase the dredge capacity in the St. Paul District by increasing the mean of the dredge capacity of each of the three dredging subsystems by 20%. The enhanced system dredge capacities on the Upper Mississippi River in St. Paul District will then be the sum of the three dredging subsystem capacities as calculated in Tables 8.2-8.4 with the mean increased by 20%. Using the approach outlined in section 6, we can now compute the allocated post-rehabilitated dredge capacities for pools of the St. Paul District. These capacities are shown in Table 8.10.

Table 8.10 Allocated Enhanced Dredge Capacities for the St. Paul District Pools (cyd/year)

Pool	1	2	3	4	5	5A	6	7	8	9	10	System
θ_i	0.0507	0.1413	0.0134	0.4113	0.0893	0.0611	0.0128	0.0559	0.0726	0.0511	0.0405	1.0000
$\mu_{C_{di}}$	122,020	340,279	32,306	990,641	215,121	147,175	30,902	134,619	174,947	123,037	97,508	2,408,555
$\sigma_{C_{di}}$	304,233	508,051	156,541	866,858	403,954	334,123	153,103	319,553	364,287	305,498	271,963	1,351,663

Given the post-rehabilitation dredge demand for each pool (Table 8.7) and the enhanced dredge capacities for the pools (Table 8.10), we can now compute the probabilities of pool failure as given in Table 8.11. The probability of system failure is computed to by 0.0934 and the reliability index (β) for the system is computed as 1.32. Figure 8.4 shows a comparison of the pool failure probabilities for the current and the enhanced capacity scenarios.

Table 8.11 Probabilities of System and Pool Failure for Enhanced Capacity Scenario

Pool	1	2	3	4	5	5A	6	7	8	9	10	System
P_i	0.0425	0.0446	0.0434	0.0503	0.0431	0.0427	0.0434	0.0428	0.0428	0.0425	0.0423	0.0934

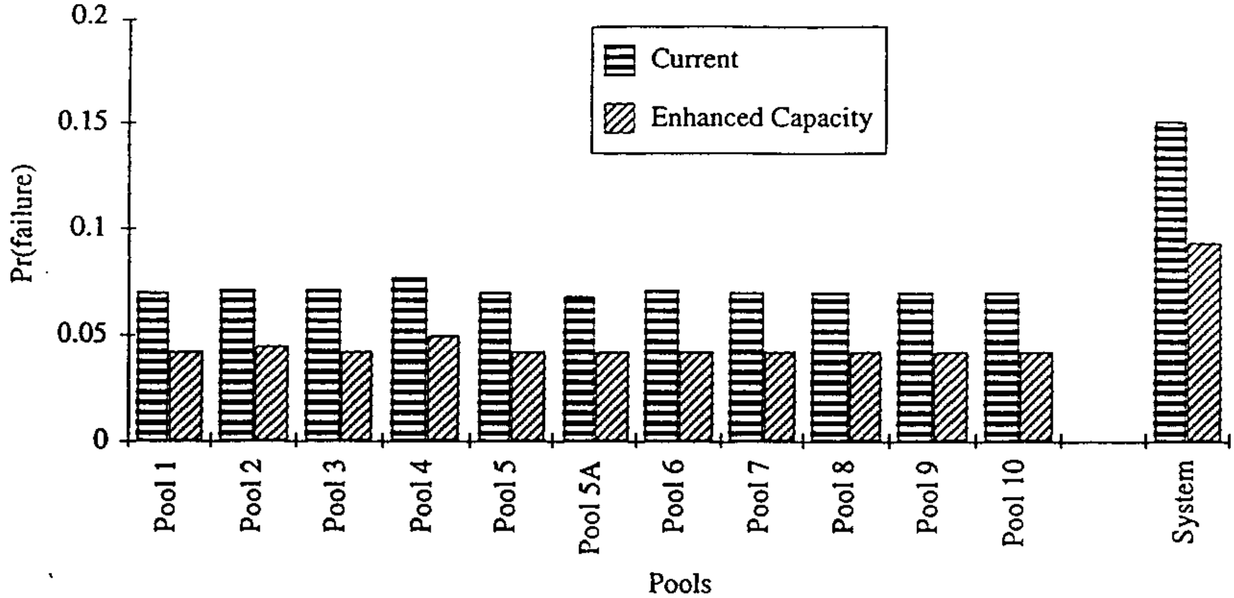

Figure 8.4 Comparison of Failure Probabilities for the Current and Enhanced Capacity Scenarios for St. Paul District

The results show that the 20% increase in the mean dredging capacity in the St. Paul District leads to a significant reduction in the failure probability by increasing the safety margin.

8.7 Impact of Hypothetical Reduction in Dredge Demand

Let us assume that rehabilitation of the navigation structures is done in the St. Paul District over the next twenty years and it leads to a 20% reduction in the dredge demand. Applying this reduction to the post-rehabilitation (current) data, we can calculate the means and standard deviations of the dredge demand for each pool as given in Table 8.12.

Table 8.12 Hypothetical Future Dredge Demand for the Pools of the St. Paul District (cyd/year)

Pool	1	2	3	4	5	5A	6	7	8	9	10	System
μ_{D_i}	24,762	69,055	6,556	201,038	43,656	29,867	6,271	27,319	35,503	24,969	19,788	488,785
σ_{D_i}	1,864	25,942	2,800	65,671	15,700	3,407	905	9,572	10,502	5,029	3,010	134,295

Since we are assuming that the dredge demand is reduced by 20% in each pool, the values of the capacity allocation variable (θ_i) remain the same. The allocated capacities for the pools thus remains the same as shown in Table 8.8.

Given the hypothetical dredge demand for each pool (Table 8.12) and the allocated dredge capacities for the pools (Table 8.8), we can now compute the probabilities of pool failure as given in Table 8.13. The reliability index (β) for the system is computed as 1.12. Figure 8.8 shows a comparison of the pool failure probabilities for the current and the enhanced capacity scenarios.

Table 8.13 Probability of Pool and System Failure for the Hypothetical Dredge Demand Reduction Scenario

Pool	1	2	3	4	5	5A	6	7	8	9	10	System
Pi	0.0687	0.0686	0.0711	0.0719	0.0675	0.0677	0.0700	0.0683	0.0683	0.0687	0.0689	0.1314

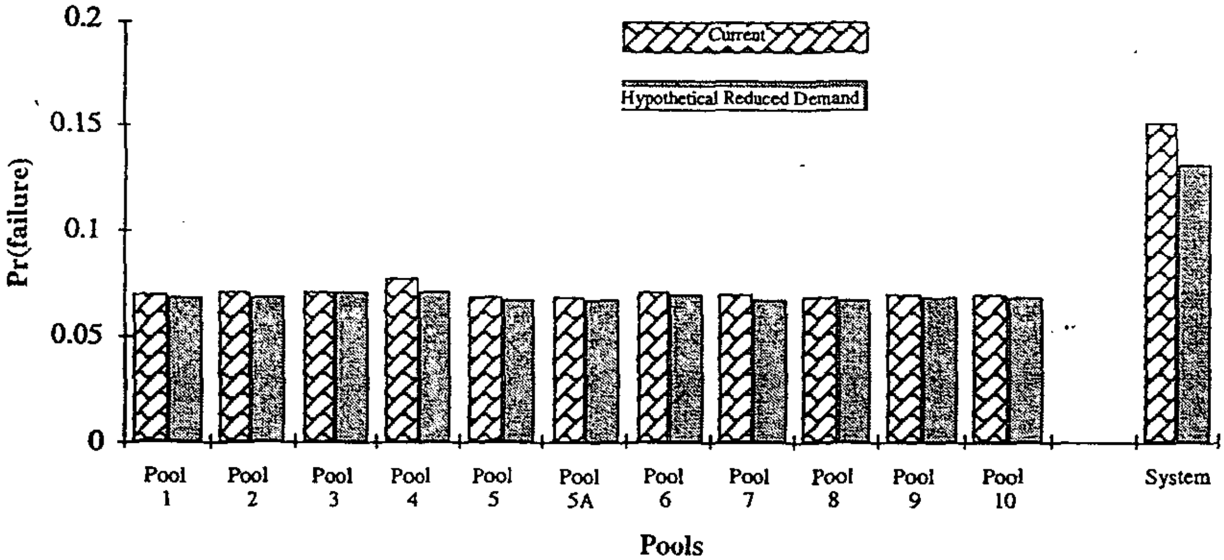

Figure 8.8 Comparison of Current and Hypothetical Dredge Demand Reduction Scenario in terms of the Failure Probabilities for St. Paul District

8.8 Conclusions

The dredge-capacity reliability model applied to the pools of the St. Paul District has shown that it is possible to quantify the impact of rehabilitation of river training structures and increasing funding for dredging operations. It also shows the importance of continued rehabilitation of the navigation structures to avoid increases in the dredge demand.

The St. Paul District has maintained a high channel reliability primarily through the maintenance of a high dredge capacity by using contract dredges in addition to the government hydraulic and mechanical dredges. Results are also given for scenarios of dredge-need reduction and dredge-capacity increase. The scenario results show that an increase in the mean of the dredging capacity is much more effective in reducing the probability of failure than a reduction in dredge demand.

Part V

Results for Rock Island District

9. Analysis of Upper Mississippi River Dredging Operations in Rock Island District

9.1 Assessment of Pre-Rehabilitation Dredge Demand

Table C.1 shows the historical dredging data and Table C.2 shows the average annual discharge for the pools of the Rock Island District. We use these data in the procedure developed in section 5 to compute the predictive dredging demand for the pools in the Rock Island District. These predictive dredging volumes are shown in Table C.3. Figure 9.1 shows the plots of the pre-rehabilitation dredging volumes vs. the average annual discharge or the stage for the 12 pools on the Upper Mississippi River in the Rock Island District. The 'x' marks shows the actual dredge-discharge data and the straight line shows the predictive linear regression fit to the data points.

Based on the regression data for the time period under consideration (1973-1991), we can calculate the means and standard deviations of the dredge demand for each pool as given in Table 9.1.

Table 9.1 Pre-Rehabilitation Dredge Demand for the Pools of the Rock Island District (cyd/year)

Pool	11*	12	13*	14*	15	16*	17	18*	19	20*	21*	22*	System
μ_{D_i}	91,396	16,675	61,141	17,746	733	36,361	14,446	69,908	21,655	132,608	67,712	76,744	607,126
σ_{D_i}	74,957	6,866	85,178	12,910	360	5,944	13,043	31,197	12,028	95,393	16,106	71,405	364,990

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the pre-rehabilitation data only

9.2 System Dredge Capacity

The Rock Island Corps District employs the hydraulic dredge Thompson for dredging in the pools on the Upper Mississippi River. In order to compute the system dredge capacity we need to compute the dredge capacity of the dredge subsystem. We can use equation 6.1 to compute the mean and standard deviations of the dredge capacity for the dredge Thompson. Table 9.2 shows the input data and the result.

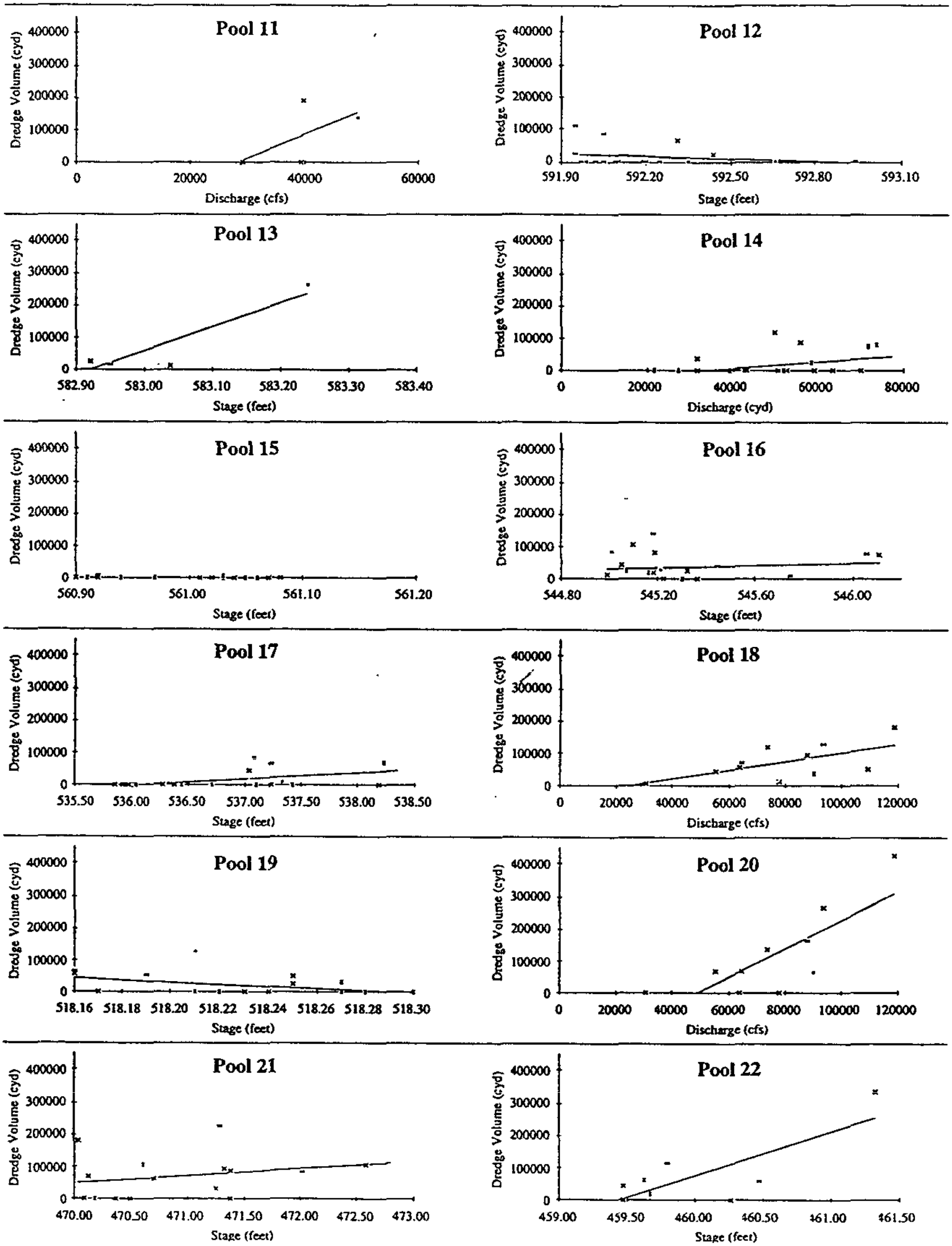

Figure 9.1 Summary Dredge vs. Discharge/Stage Plots for Rock Island District

Table 9.2 Values of Capacity Variables for Rock Island District (Dredge Thompson)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	71	52,902	0.21	0.50	12,000	734,055
σ	0	39,544	0.00	0.00	0	145,905

9.3 Pool Dredge Capacity

Using the approach outlined in section 6, we can now compute the allocated dredge capacities for the Rock Island pools based on the dredge demands from Table 9.1. These capacities are shown in Table 9.3.

Table 9.3 Allocated Pre-Rehabilitation Dredge Capacities for the Rock Island Pools (cyd/year)

Pool	11	12	13	14	15	16	17	18	19	20	21	22	System
θ_i	0.1505	0.0275	0.1007	0.0292	0.0012	0.0599	0.0238	0.1151	0.0357	0.2184	0.1115	0.1264	1.0000
$\mu_{C_{di}}$	110,504	20,161	73,923	21,456	886	43,963	17,466	84,523	26,182	160,332	81,868	92,788	734,055
$\sigma_{C_{di}}$	56,610	24,180	46,302	24,945	5,070	35,707	22,506	49,510	27,556	68,189	48,726	51,875	145,905

9.4 Pool Reliability

The determination of the rest-of-system dredge demand, D_{-i}, is required for the computation of the pool reliability. This can be computed for each pool i by subtracting the pool demand, D_i, from the system demand, D_{ds}. The computed data for the rest-of-system demand is shown in Table C.4.

Given the dredge demand for each pool (Table 9.1) and the allocated dredge capacities for the pools (Table 9.3), we can now compute the probabilities of pool failure as given in Table 9.4. Table C.5 shows the intermediate computational results for obtaining these probabilities for the Rock Island dredging system. The reliability index (β) for the system is computed as 0.32 and the probability of system failure is 0.3745.

Table 9.4 Pre-Rehabilitation System and Pool Failure Probabilities for Rock Island District

Pool	11*	12	13*	14*	15	16*	17	18*	19	20*	21*	22*	System
p_i	0.1576	0.1681	0.1645	0.1679	0.1828	0.1577	0.1693	0.1533	0.1666	0.1533	0.1489	0.1589	0.3745

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the pre-rehabilitation data only

9.5 Impact of Historical Rehabilitation

During the period under study (1973-1991), several pools in the Rock Island District have undergone rehabilitation of navigation structures. One objective of this study is to determine whether this rehabilitation work had any impact on the dredge demand. Note that the term "pool rehabilitation" does not mean that every channel training structure was rehabilitated, just that some of the structures which were determined to be in need of repair were. Data for five pools where there was sufficient record of post-rehabilitation record was examined to see if a trend of reduction in post-rehabilitation dredging volumes could be established. Figure 9.2 shows the plots of the dredge volumes vs. the average annual discharge or the stage for these five pools on the Upper Mississippi River in the Rock Island District. The '+' marks shows the actual data points for the post-rehabilitation data. The straight lines show the linear regression fit to the data points with the solid line showing the pre-rehabilitated condition and the dashed line the post-rehabilitated condition.

Based on these historical post-rehabilitation dredging volumes, we can update Table C.3 and C.4. The updated worksheets are shown in Table C.6 and C.7. The means and standard deviations of the dredge demand for each pool can now be calculated and are as given in Table 9.5.

Table 9.5 Post-Rehabilitation Dredge Demand for the RID Pools (cyd/year)

Pool	11*	12	13*	14	15	16	17	18*	19	20*	21	22*	System
μ_{D_i}	14,656	16,675	47,787	17,746	733	36,361	14,446	58,687	21,655	52,062	67,712	86,297	434,817
σ_{D_i}	7,646	6,866	5,708	12,910	360	5,944	13,043	31,881	12,028	241	16,106	37,646	36,702

* — rehabilitation was performed in these pools between 1973-1991 and the demand is calculated from the post-rehabilitation data only

Using the approach outlined in section 6, we can now compute the allocated post-rehabilitated dredge capacities for pools of the Rock Island District. These capacities are shown in Table 9.6.

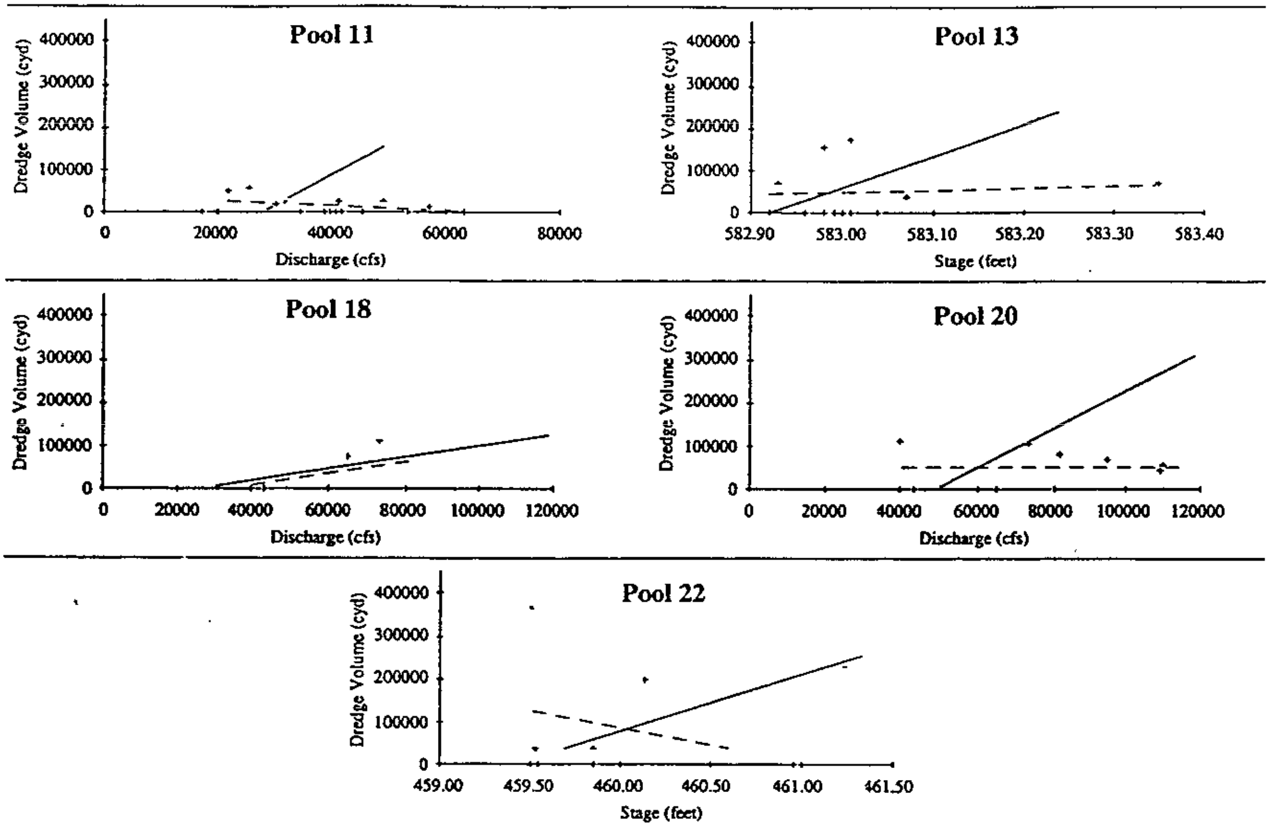

Figure 9.2 Post-Rehabilitation Dredge vs. Discharge/Stage Plots for Rock Island District

Table 9.6 Allocated Post-Rehabilitation Dredge Capacities for the Rock Island District Pools (cyd/year)

Pool	11	12	13	14	15	16	17	18	19	20	21	22	System
θ_i	0.0337	0.0383	0.1099	0.0408	0.0017	0.0836	0.0332	0.1350	0.0498	0.1197	0.1557	0.1985	1.0000
μC_{di}	24,742	28,151	80,674	29,959	1,237	61,384	24,388	99,075	36,558	87,891	114,311	145,686	734,055
σC_{di}	26,787	28,573	48,370	29,476	5,991	42,193	26,594	53,603	32,561	50,487	57,577	65,000	145,905

Given the post-rehabilitation dredge demand for each pool (Table 9.5) and the allocated dredge capacities for the pools (Table 9.6), we can now compute the probabilities of pool failure as given in Table 9.7. The reliability index (β) for the system is computed as 1.99. Figure 9.3 shows a comparison of the pool reliabilities in the pre-rehabilitated and the post-rehabilitated conditions.

Table 9.7 Post-Rehabilitation System and Pool Failure Probabilities for Rock Island District

Pool	11*	12	13*	14	15	16	17	18*	19	20*	21	22*	System
p_i	0.0110	0.0109	0.0113	0.0101	0.0112	0.0109	0.0101	0.0125	0.0105	0.0116	0.0122	0.0183	0.0233

* — rehabilitation was performed in these pools between 1973-1991 and the demand is calculated from the post-rehabilitation data only

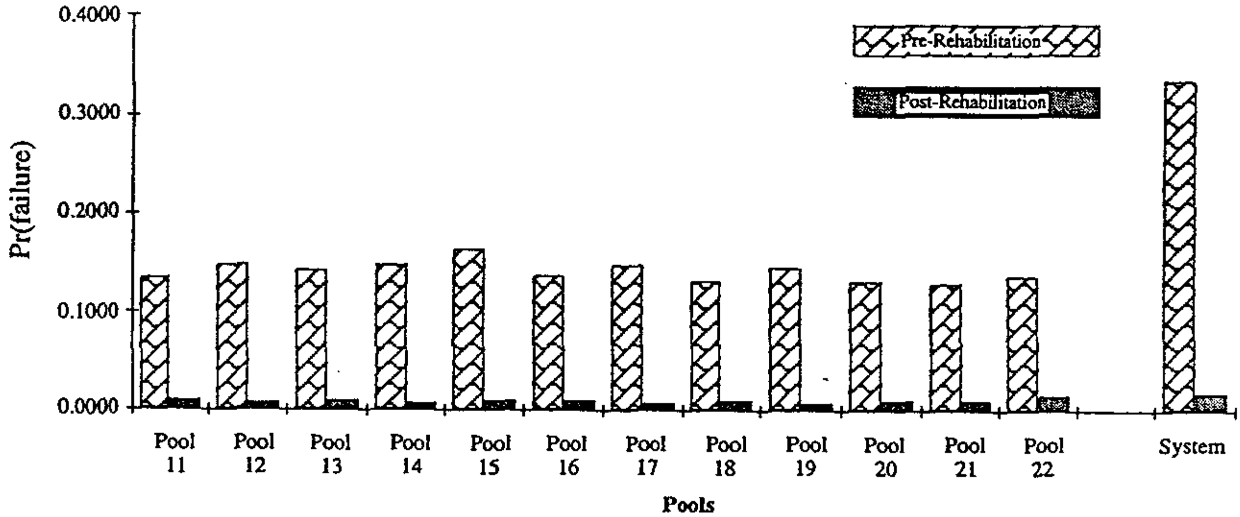

Figure 9.3 Comparison of Pre and Post Rehabilitation Reliabilities for Rock Island District for Historical Rehabilitation between 1973-1991

9.6 Impact of Hypothetical Increase in Capacity

Let us assume that we can increase the dredge capacity in the Rock Island District by increasing the mean of the dredge capacity by 20%. Using the approach outlined in section 6, we can now compute the allocated post-rehabilitated dredge capacities for pools of the Rock Island District. These capacities are shown in Table 9.8.

Table 9.8 Allocated Enhanced Dredge Capacities for the Rock Island District Pools (cyd/year)

Pool	11	12	13	14	15	16	17	18	19	20	21	22	System
θ_i	0.0337	0.0383	0.1099	0.0408	0.0017	0.0836	0.0332	0.1350	0.0498	0.1197	0.1557	0.1985	1.0000
$\mu_{C_{di}}$	29,691	33,781	96,808	35,950	1,485	73,661	29,265	118,890	43,869	105,469	137,173	174,823	880,866
$\sigma_{C_{di}}$	38,573	41,145	69,652	42,445	8,626	60,757	38,296	77,188	46,888	72,701	82,911	93,601	210,104

Given the post-rehabilitation dredge demand for each pool (Table 9.5) and the enhanced dredge capacities for the pools (Table 9.8), we can now compute the probabilities of pool failure as given in Table 9.9. The reliability index (β) for the system is computed as 2.96. Figure 9.4 shows a comparison of the system and pool failure probabilities for the current and the enhanced capacity scenarios.

Table 9.9 Probability of Pool Failure for Enhanced Capacity Condition

Pool	11*	12	13*	14	15	16	17	18*	19	20*	21	22*	System
p_i	0.0008	0.0007	0.0009	0.0007	0.0007	0.0008	0.0006	0.0011	0.0007	0.0010	0.0011	0.0024	0.0015

* — rehabilitation was performed in these pools between 1973-1991 and the demand is calculated from the post-rehabilitation data only

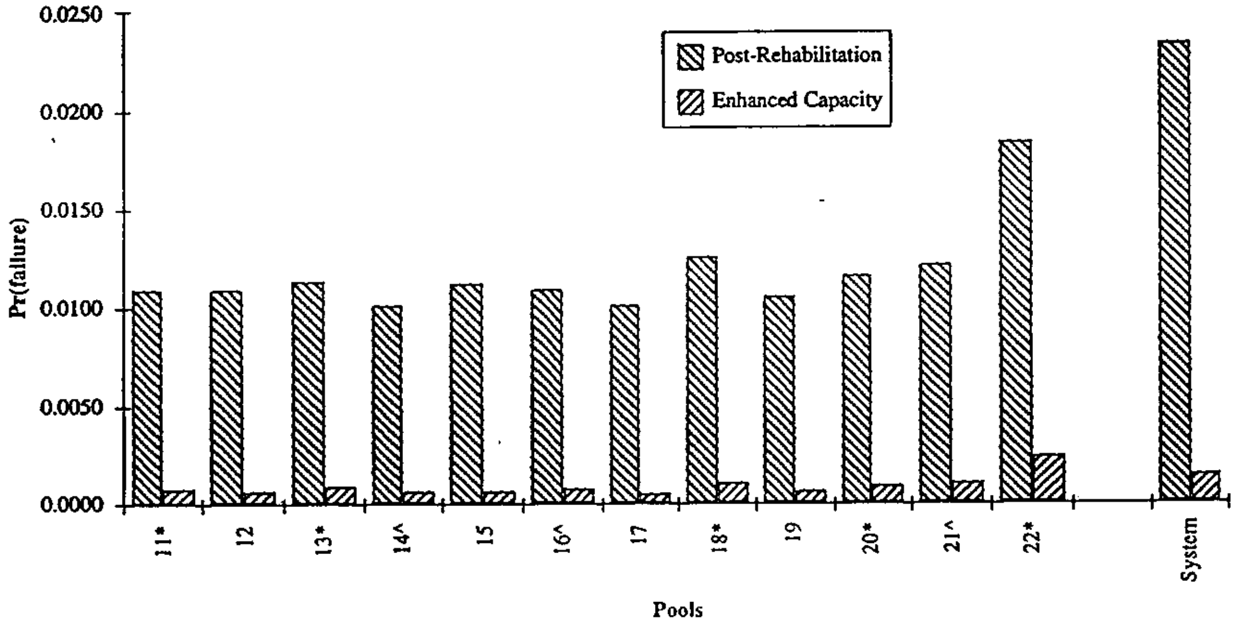

Figure 9.4 Comparison of Current and Enhanced Pool Failure Probabilities for Rock Island District

9.7 Impact of Hypothetical Reduction in Dredge Demand

Let us assume that further rehabilitation of the navigation structures is done in the Rock Island District over the next twenty years and it leads to a 20% reduction in the dredge demand. Applying this reduction to the post-rehabilitation (current) data, we can calculate the means and standard deviations of the dredge demand for each pool as given in Table 9.10.

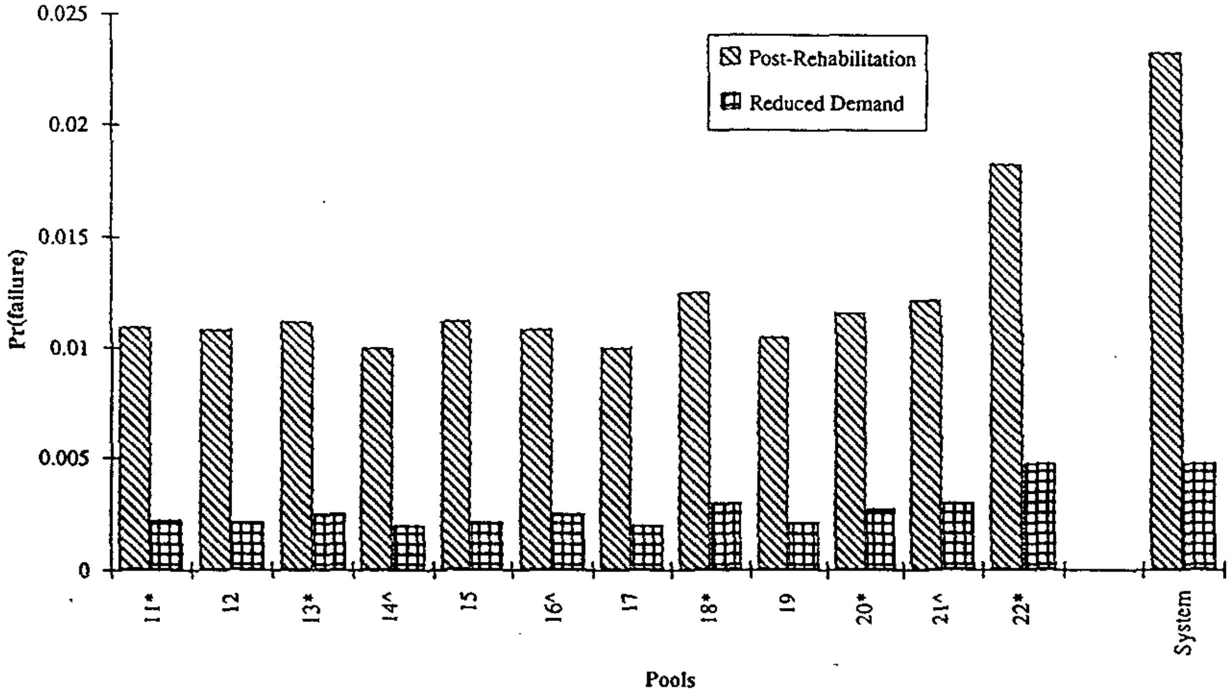

Figure 9.5 Comparison of Current and Hypothetical Dredge Demand Reduction Scenario in terms of the Failure Probabilities for Rock Island District

10. Analysis of Illinois Waterway Dredging Operations in Rock Island District

10.1 Assessment of Pre-Rehabilitation Dredge Demand

Table C.9 shows the historical dredging data and the average daily stage data for the LaGrange and Peoria Pools on the Illinois Waterway in the Rock Island District. We use these data in the procedure developed in section 5 to compute the predictive dredging demand for these two pools. These predictive dredging volumes are shown in Table C.10. Figure 10.1 shows the plots of the pre-rehabilitation dredging volumes vs. the average annual discharge or the stage for the LaGrange and Peoria pools. The 'x' marks shows the actual dredge-stage data and the straight line shows the predictive linear regression fit to the data points.

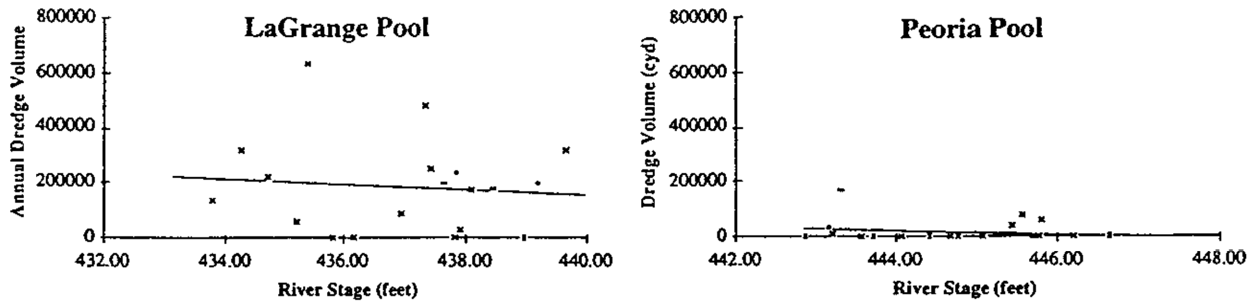

Figure 10.1 Summary Dredge vs. Stage Plots for Illinois Waterway in Rock Island District

Based on the regression data for the time period under consideration (1973-1991), we can calculate the means and standard deviations of the dredge demand for each pool as given in Table C.10.

10.2 System Dredge Capacity

The Rock Island Corps District employs three different dredging systems on the Illinois Waterway; the hydraulic dredge Thompson, contract mechanical dredge, and contract mechanical dredge from St. Louis District. In order to compute the system dredge capacity we need to compute the dredge capacities of these individual dredge subsystems. We can use equation 6.1 to compute the mean and standard deviations of the dredge capacity for each of the dredge subsystems. Tables 10.1, 10.2, and 10.3 show the input data and the results of the three subsystems.

Table 10.1 Values of Capacity Variables for the Illinois Waterway in the Rock Island District (Dredge Thompson)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	4	56,506	0.21	0.50	12,000	41,724
σ	0	42,831	0.00	0.00	0	399

Table 10.2 Values of Capacity Variables for the Illinois Waterway in the Rock Island District (Contract Mechanical Dredge)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	131	56,506	7.00	6.00	2,646	215,462
σ	0	42,831	0.00	0.00	0	89,170

Table 10.5 Probability of Failure for the Illinois Waterway Pools in the Rock Island District (cyd/year)

Pool	LaGrange	Peoria	System
Pi	0.0920	0.0840	0.1841

Part VI

Results for St. Louis District

11. Analysis of Upper Mississippi River Dredging Operations in St. Louis District

11.1 Assessment of Pre-Rehabilitation Dredge Demand

The Upper Mississippi River in the St. Louis District consists of a 194–mile stretch of open river and four pools (a total distance of about 300 miles). In order to apply the pool model to the open river, we proposed a division of that stretch into sections. For the open river, we had discharge data available at St. Louis, Chester, and Thebes from the U.S. Geological Survey. For dredge demand analysis, we therefore divided the open river into three sections corresponding to these three locations. For St. Louis, the stretch was from RM 194 to 126, for Chester from RM 126 to 76, and for Thebes from RM 76 to zero.

Table D.1 shows the historic dredging data and Table D.2 shows the average annual discharge/stage for the pools and sections of the river. Figure 11.1 shows the plots of the dredge volumes vs. the average annual discharge or the stage for the 4 pools on the Upper Mississippi River and the three open river sections in the St. Louis District. The 'x' marks shows the dredge-discharge/stage data and the straight line shows the linear regression fit to the data points.

Based on the regression data for the time period under consideration (1973-1993), we can calculate the means and standard deviations of the dredge demand for each pool as given in Table 11.1.

Table 11.1 Dredge Demand in the St. Louis District (cyd/year)

Pool	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
μ_{D_i}	197,931	666,624	551,788	67,395	1,594,003	1,049,296	2,752,477	6,879,515
σ_{D_i}	31,655	184,121	578,603	16,743	893,876	490,411	1,205,415	2,372,829

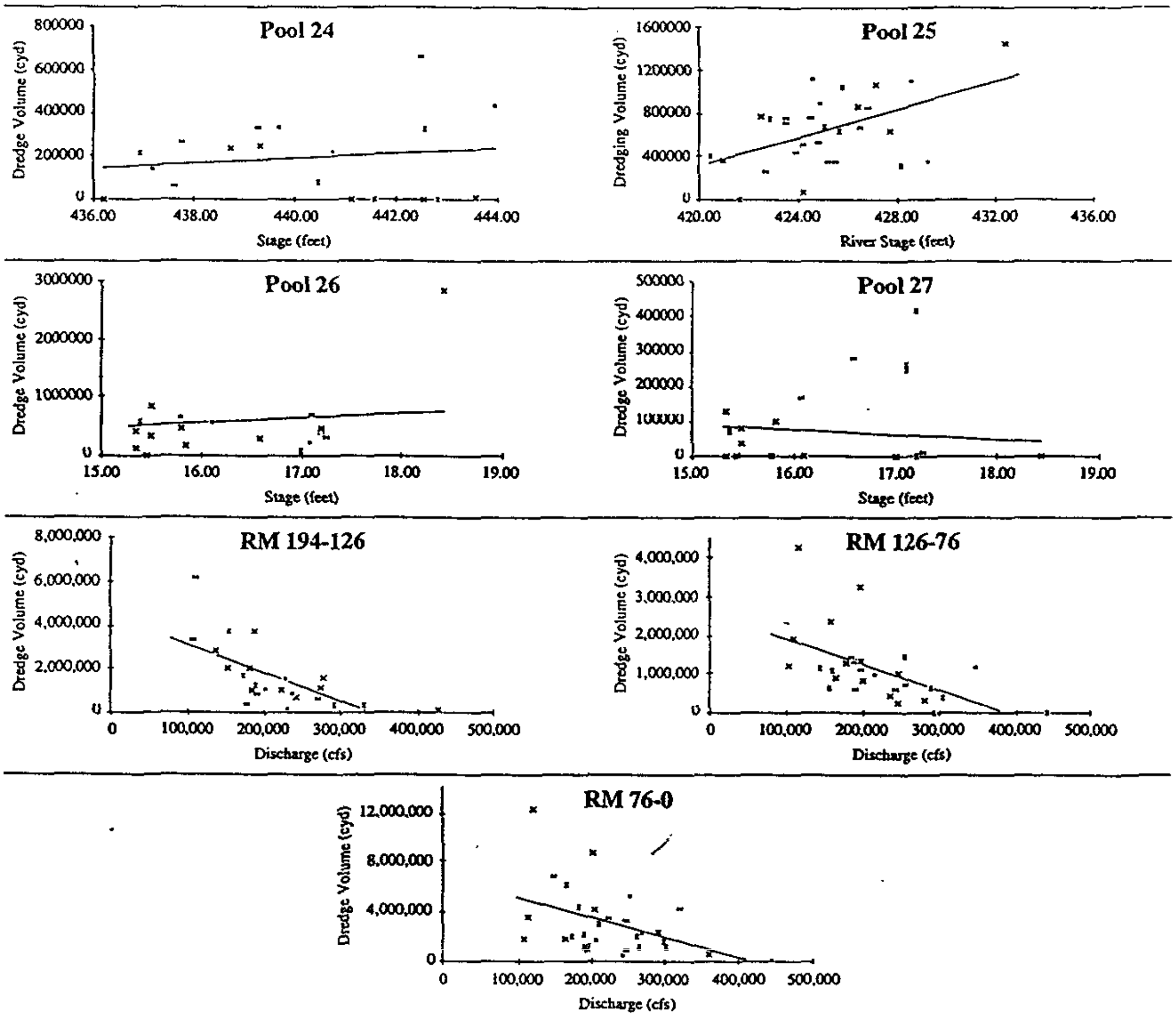

Figure 11.1 Summary Dredge vs. Discharge/Stage Plots for St. Louis District

11.2 System Dredge Capacity

The St. Louis Corps District employs two different dredging systems; the government dredge Potter and a contract mechanical dredge. In order to compute the system dredge capacity we need to compute the dredge capacities of these individual dredge subsystems. We can use equation 6.1 to compute the mean and standard deviations of the dredge capacity for each of the dredge subsystems. Tables 11.2 and 11.3 show the input data and the results of the two subsystems.

The system dredge capacity is the sum of the two dredging subsystem capacities as shown in Tables 11.2 and 11.3.

Table 11.2 Values of Capacity Variables for St. Louis District (Dredge Potter)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	130	176,509	0.08	0.08	50,000	6,206,956
σ	0	152,596	0.00	0.00	0	383,534

Table 11.3 Values of Capacity Variables for St. Louis District (Contract Dredge)

Variable	M (days/year)	V (cyd/site)	S (days/site)	T (days/site)	P (cyd/day)	C_{ds} (cyd/year)
μ	50	108,563	0.17	0.33	20,000	915,656
σ	0	44,787	0.00	0.00	0	22,665

11.3 Pool Dredge Capacity

Using the approach outlined in section 6, we can now compute the allocated dredge capacities for the St. Louis pools based on the dredge demands from Table 11.1. These capacities are shown in Table 11.4.

Table 11.4 Allocated Dredge Capacities for St. Louis (cyd/year)

Pool	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
θ_i	0.0288	0.0969	0.0802	0.0098	0.2317	0.1525	0.4001	1.0000
μC_{di}	204,487	688,703	570,063	69,627	1,646,797	1,084,049	2,843,640	7,107,366
σC_{di}	157,861	289,706	263,574	92,115	447,983	363,468	588,679	930,670

11.4 Pool Reliability

The determination of the rest-of-system dredge demand, $D_{\neg i}$, is required for the computation of the pool reliability. This can be computed for each pool i by subtracting the pool demand, D_i, from the system demand, D_{ds}. The computed data for the rest-of-system demand is shown in Table D.4.

Given the dredge demand for each pool (Table 11.1) and the allocated dredge capacities for the pools (Table 11.4), we can now compute the probabilities of pool/river section failure as given in Table 11.5. Table D.5 shows the intermediate computational results for obtaining these probabilities for the St. Louis District. The reliability index (β) for the system is computed as 0.10 and the probability of system failure is 0.4602.

Table 11.5 Probability of System and Pool Failure for St. Louis District

Pool	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
p_i	0.2118	0.2117	0.2265	0.2191	0.2153	0.2154	0.2135	0.4602

11.5 Impact of Hypothetical Increase in Capacity

Let us assume that we can increase the dredge capacity in the St. Louis District by increasing the mean of the dredge capacity of each of the two dredging subsystems by 20%. The enhanced system dredge capacities on the Upper Mississippi River in St. Louis District will then be the sum of the two dredging subsystem capacities as calculated in Tables 11.2 and 11.3 with the mean increased by 20%. Using the approach outlined in section 6, we can now compute the allocated post-rehabilitated dredge capacities for pools of the St. Louis District. These capacities are shown in Table 11.6.

Table 11.6 Allocated Enhanced Dredge Capacities in St. Louis District (cyd/year)

Pool	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
θ_i	0.0288	0.0969	0.0802	0.0098	0.2317	0.1525	0.4001	1.0000
μC_{di}	245,910	828,216	685,543	83,732	1,980,395	1,303,649	3,419,687	8,547,134
σC_{di}	93,843	172,220	156,686	54,759	266,311	216,069	349,950	553,252

Given the dredge demand for each pool or river section (Table 11.1) and the enhanced dredge capacities for the pools and river sections (Table 11.6), we can now compute the probabilities of pool failure as given in Table 11.7. The reliability index (β) for the system is computed as 0.69. Figure 11.2 shows a comparison of the probabilities of failure for the current and the enhanced capacity scenarios.

Table 11.7 Probability of System and Pool Failure for St. Louis District for the Enhanced Capacity Scenario

Pool	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
p_i	0.0640	0.0645	0.1068	0.0854	0.0705	0.0718	0.0677	0.2451

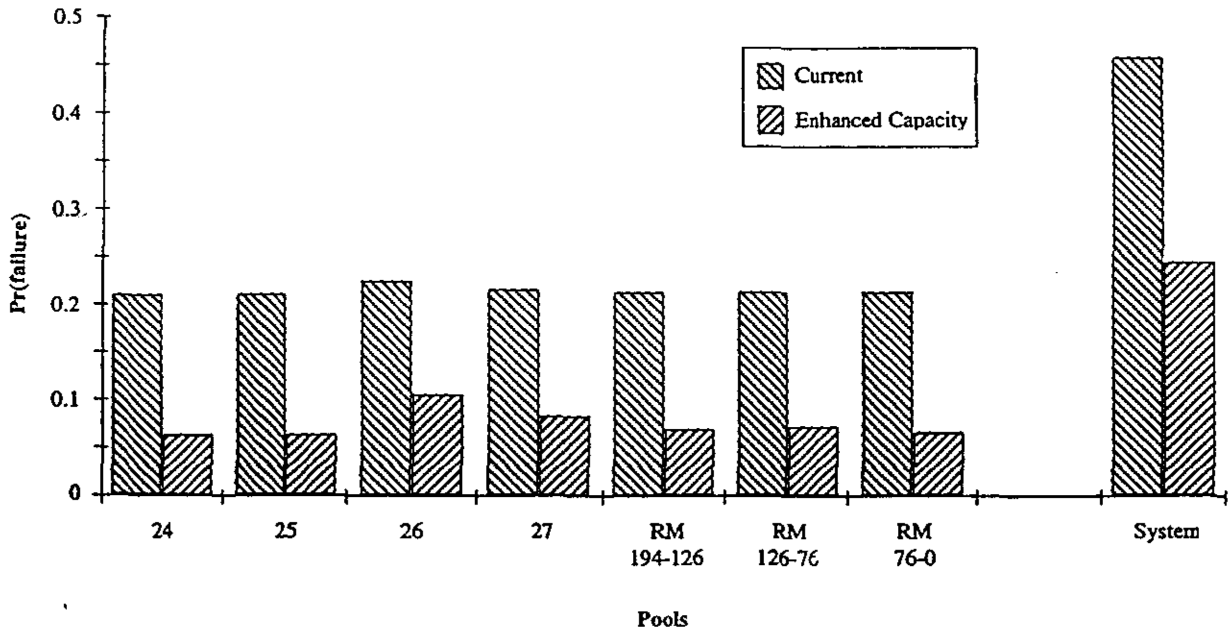

Figure 11.2 Comparison of Current and Enhanced Failure Probabilities for St. Louis District

11.6 Impact of Hypothetical Reduction in Dredge Demand

Let us assume that rehabilitation of the navigation structures is done in the St. Louis District over the next twenty years and it leads to a 20% reduction in the dredge demand. Applying this reduction to the current data, we can calculate the means and standard deviations of the dredge demand for each pool as given in Table 11.8.

Table 11.8 Hypothetical Future Dredge Demand for St. Louis District (cyd/year)

Pool	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
μD_i	158,345	533,299	441,430	53,916	1,275,202	839,437	2,201,982	5,503,612
σD_i	25,324	147,297	462,882	13,394	715,101	392,329	964,332	1,898,263

Since we are assuming that the dredge demand is reduced by 20% in each pool, the values of the capacity allocation variable (θ_i) remain the same. The allocated capacities for the pools thus remains the same as shown in Table 11.4.

Given the hypothetical dredge demand for each pool (Table 11.8) and the allocated dredge capacities for the pools (Table 11.4), we can now compute the probabilities of pool failure as given in Table 11.9. The reliability index (β) for the system is computed as 0.84. Figure 11.3 shows a comparison of the pool failure probabilities for the current and the reduced demand scenarios.

Table 11.9 Probability of Pool Failure for the Hypothetical Dredge Demand Reduction Scenario

Pool	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
Pi	0.0533	0.0492	0.0868	0.0708	0.0514	0.0534	0.0488	0.2005

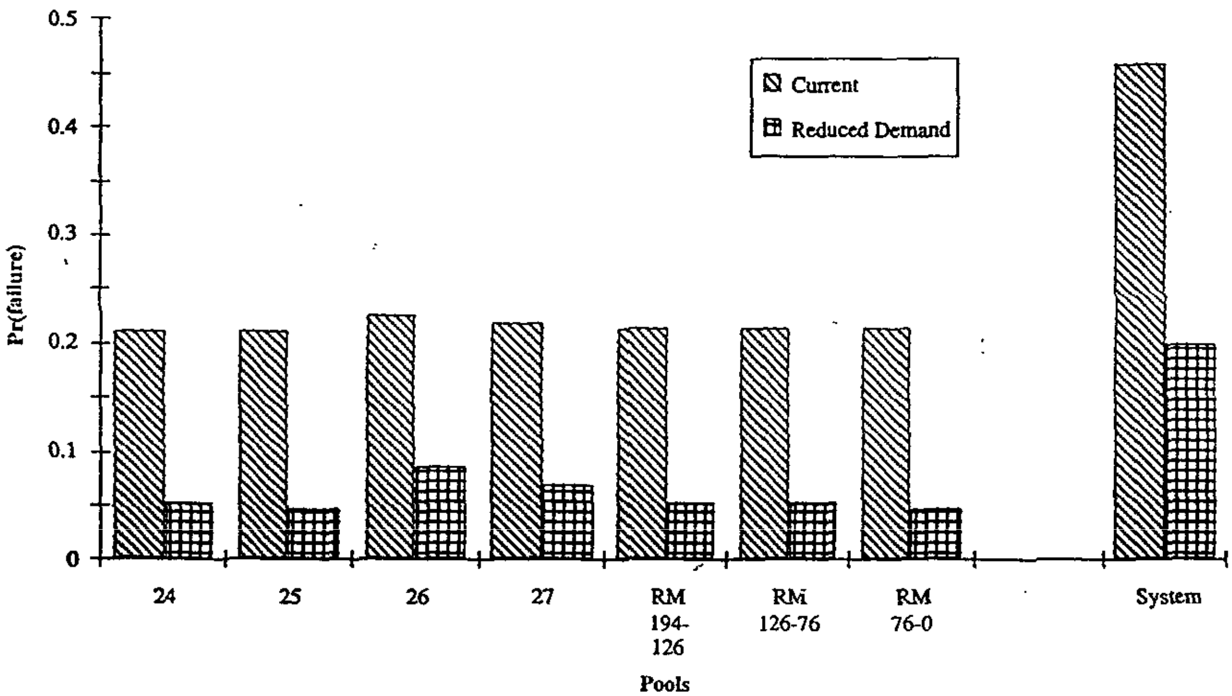

Figure 11.3 Comparison of Current and Hypothetical Dredge Demand Reduction Scenario in terms of the Failure Probabilities for St. Louis District

11.7 Conclusions

The dredge-capacity reliability model applied to the pools of the St. Louis District has shown that it is possible to quantify the reliability of the navigation channel. It is important to note the difference between the open river and the pools in the D versus Q model. There is a potential to improve the channel reliability through increased capacity and/or rehabilitation of river training structures. The results of the scenario analysis show that demand reduction has a greater impact on the reduction of failure probabilities than the increase in dredge capacity.

Part VII

Summary of Results and Conclusions

12. Summary of Results and Conclusions

12.1 System Level

The dredge-capacity reliability model makes it possible to quantify the reliability of the navigation channel. It is also possible to quantify the impacts of rehabilitation of the navigation structures subject to the availability of sufficient dredging records after the completion of the rehabilitation effort. In addition, there exists a potential to further improve the navigation reliability from the current condition. The actual improvements are dependent upon the current condition of the channel and the dredging policy in each District.

Table 12.1 gives the values of the probability of system failure and the reliability index, β, for the three districts under various policy scenarios considered:

1. Pre-Rehabilitation is the condition prior to any historical rehabilitation work performed during the analysis period which is from 1973 to 1993.
2. Post-Rehabilitation (or Current) is the condition after the historical rehabilitation work and where sufficient post-rehabilitation records exist. For this reason, the impact of rehabilitation work performed during the last four to five years of the study period cannot be evaluated.
3. Enhanced Capacity is the condition where a twenty percent increase in the number of available dredging days is assumed due to increased spending on dredging operations in the O&M budget.
4. Reduced Demand is the condition where a twenty percent reduction in dredge demand is assumed due to structural or other rehabilitation work performed on the navigation structures.

Table 12.1 Summary of System Reliabilities for the three Districts

Scenario	Results	St. Paul	Rock Island	St. Louis
Pre-Rehabilitation	Pr(failure)	0.1562	0.3745	
	β	1.01	0.32	
Post-Rehabilitation	Pr(failure)	0.1515	0.0233	0.4602
	β	1.03	1.99	0.10
Enhanced Capacity	Pr(failure)	0.0934	0.0015	0.2451
	β	1.32	2.96	0.69
Reduced Demand	Pr(failure)	0.1314	0.0048	0.2005
	β	1.12	2.60	0.84

12.2 St. Paul District

Table 12.2 summarizes the results for the pools in the St. Paul District for the four scenarios described earlier. Figure 12.1 shows the comparative probabilities of pool failure for the four scenarios analyzed for St. Paul District. Rehabilitation was performed only in pools 5 and 7 during 1975-1993

Table 12.2 Summary of Pool Reliabilities for Alternative Scenarios in the St. Paul District

Pr(failure)	1	2	3	4	5*	5A	6	7^	8	9	10	System
Pre-Rehabilitation	0.0710	0.0725	0.0728	0.0786	0.0712	0.0707	0.0718	0.0714	0.0714	0.0710	0.0720	0.1562
Post-Rehabilitation	0.0700	0.0708	0.0718	0.0777	0.0696	0.0690	0.0706	0.0703	0.0697	0.0700	0.0702	0.1515
Enhanced Capacity	0.0425	0.0446	0.0434	0.0503	0.0431	0.0427	0.0434	0.0428	0.0428	0.0425	0.0423	0.0934
Reduced Demand	0.0687	0.0686	0.0711	0.0719	0.0675	0.0677	0.0700	0.0683	0.0683	0.0687	0.0689	0.1314

* — rehabilitation was performed in pool 5 during 1975-1993, sufficient pre- and post-rehabilitation record
^ — rehabilitation was performed in pool 7 during 1975-1993, insufficient post-rehabilitation record

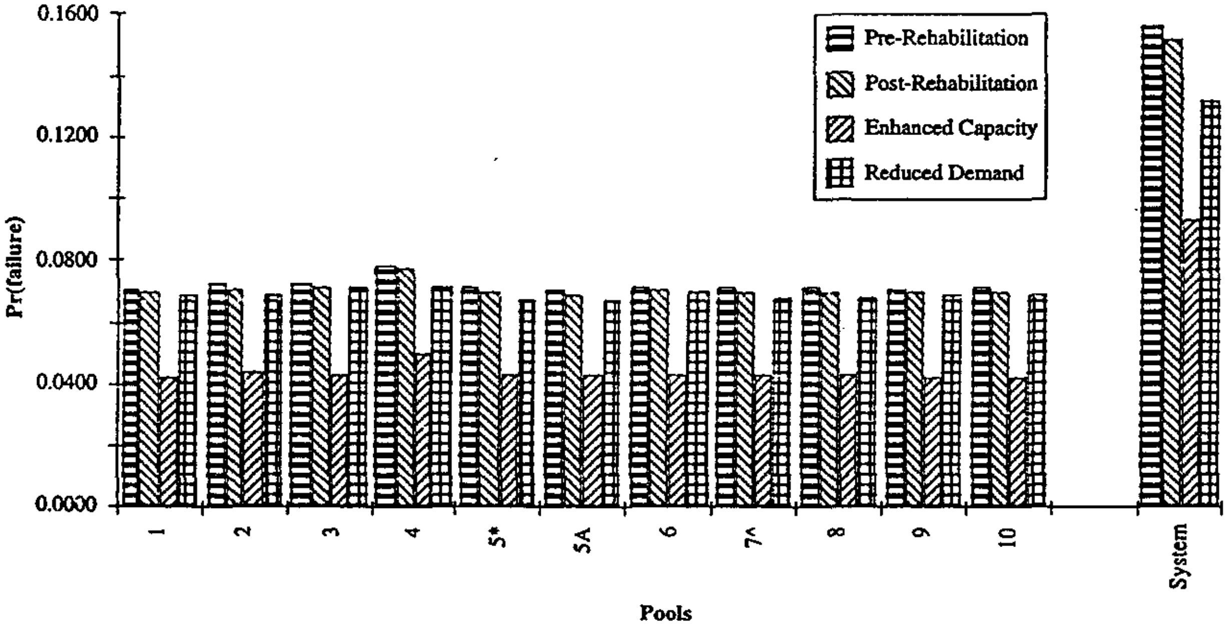

Figure 12.1 Comparison of Scenario Reliabilities for St. Paul District 1975-1993

The dredge-capacity reliability model applied to the pools of the St. Paul District has shown that it is possible to quantify the impact of rehabilitation of training structures and increasing dredging operations.

The St. Paul District has maintained a high channel reliability primarily through dredge capacity by using contract dredges in addition to the government hydraulic and mechanical dredges. The scenario results show that the specified addition of dredge capacity would be more effective in reducing the probability of failure than the specified reduction in dredging demand.

12.3 Rock Island District

Table 12.3 summarizes the results for the pools in the Rock Island District for the four scenarios described earlier. Figure 12.2 shows the comparative probabilities of pool failure for the four scenarios analyzed for Rock Island District.

Table 12.3 Summary of Pool Reliabilities for Alternative Scenarios in the Rock Island District

Pr(failure)	11*	12	13*	14^	15	16^	17	18*	19	20*	21^	22*	System
Pre-Rehabilitation	0.1576	0.1681	0.1645	0.1679	0.1828	0.1577	0.1693	0.1533	0.1666	0.1533	0.1489	0.1589	0.3745
Post-Rehabilitation	0.0110	0.0109	0.0113	0.0101	0.0112	0.0109	0.0101	0.0125	0.0105	0.0116	0.0122	0.0183	0.0233
Enhanced Capacity	0.0008	0.0007	0.0009	0.0007	0.0007	0.0008	0.0006	0.0011	0.0007	0.0010	0.0011	0.0024	0.0015
Reduced Demand	0.0023	0.0022	0.0025	0.0021	0.0022	0.0025	0.0021	0.0031	0.0022	0.0028	0.0030	0.0048	0.0048

* — rehabilitation was performed in these pools between 1973-1991, sufficient pre- and post-rehabilitation record
^ — rehabilitation was performed in these pools between 1973-1991, insufficient post-rehabilitation record

The dredge-capacity reliability model applied to the pools of the Rock Island District has shown that it is possible to quantify the impact of rehabilitation of river training structures and increasing funding for dredging operations. It also shows the importance of continued maintenance and rehabilitation of the navigation structures to avoid increases in the dredge demand.

From 1973 to the present, the Rock Island District significantly improved the channel reliability through extensive and continued rehabilitation of navigation structures. This improvement illustrates the benefits of rehabilitation of the navigation structures and underscores the importance of continued maintenance of the navigation structures to avoid a return of the navigation channel to the less reliable pre-rehabilitated state. Potential for further improvement exists through dredge-need reduction and dredge-capacity increase.

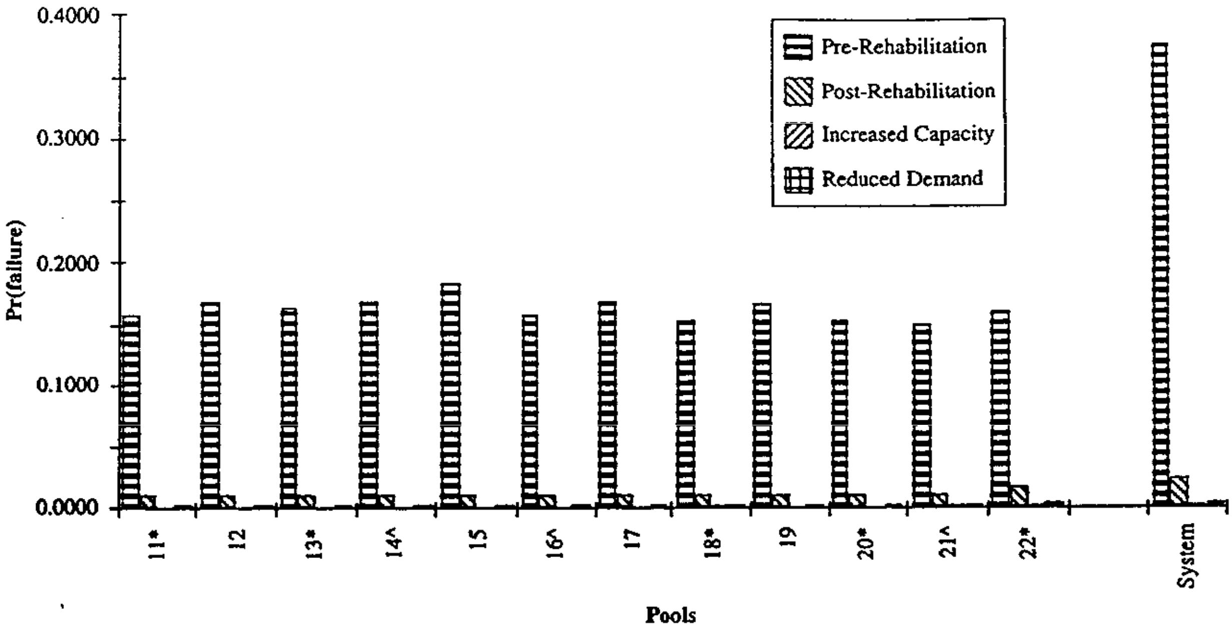

Figure 12.2 Comparison of Scenario Reliabilities for Rock Island District 1973-1991

12.4 St. Louis District

Table 12.4 summarizes the results for the pools in the St. Louis District for the scenarios. Figure 12.3 shows the comparative probabilities of pool failure for the scenarios analyzed for Louis District. There was no available record of rehabilitation in the study period 1973-1993.

Table 12.4 Summary of Pool Reliabilities for Alternative Scenarios in the St. Louis District

Pr(failure)	24	25	26	27	RM 194-126	RM 126-76	RM 76-0	System
Current	0.2118	0.2117	0.2265	0.2191	0.2153	0.2154	0.2135	0.4602
Enhanced Capacity	0.0640	0.0645	0.1068	0.0854	0.0705	0.0718	0.0677	0.2451
Reduced Demand	0.0533	0.0492	0.0868	0.0708	0.0514	0.0534	0.0488	0.2005

In the St. Louis District, there is a potential to improve the channel reliability through increased capacity and/or rehabilitation of river training structures. The results of the scenario analysis show that specified demand reduction would have a greater impact on the reduction of probability of system failure as compared to increases in dredge capacity.

Appendix A: Scope of Work

This Scope of Work for phase II of this project builds on the Dredge Capacity Reliability Model developed for the pool reach. The Inter-dredge Reliability Model will not be further pursued in Phase II of this project. Discussion of the two models is contained within the Final Report (Second Draft Version dated November 16, 1994) for Channel Reliability of the Navigation System in the Upper Mississippi River, developed by the Center for Risk Management of Engineering Systems, University of Virginia (Reference 1). The Scope of Work for phase I of this study is included as Appendix A in Reference 1. Following are the proposed work items for the phase II work.

A. Work Tasks

1. Task 1 - Develop Dredge Capacity Reliability Model parameters for each pool using F_{95} (number of falls in the hydrograph exceeding the 95% magnitude in the year) or Average Daily Discharge (whichever correlates the best), as the predictive variable for dredging.

2. Task 2 - Establish the distribution of dredging demand on both a system wide basis (D_s) and a pool basis (D_i). Those values are to be established for the rehabilitated and non-rehabilitated training structure condition.

3. Task 3 - Establish the distribution of system dredge capacity (C_{ds}) based on dredge travel time, setup time and dredging volumes of past operations. Data will be furnished by the Corps Districts for establishment of this distribution. There are three distinct dredging sub-systems on the Upper Mississippi River and Illinois Waterway. The Corps of Engineers Dredge Thompson is used primarily by the Rock Island and St. Paul Districts for dredging on the Upper Mississippi River. The St. Louis District uses the Corps of Engineers Dredge Potter and also has a contract dredge for dredging on the Upper Mississippi River and Illinois Waterway. The Rock Island District also has a contract dredge for dredging on the Illinois Waterway.

4. Task 4* - Establish the distribution of pool dredging capacity (C_i) based on the following limit states:

(a) Develop the limit state for dredge capacity as follows:

a. $$F = \frac{C_{di}}{D_i} = \frac{C_{ds} - D_s}{D_i}$$

where F = Performance Function (Safety Ratio) in Pool i
C_{di} = Dredging Capacity in Pool i (pool limit state)
C_{ds} = Dredging Capacity for the System (system limit state)
D_s = Dredging Demand of the System Exclusive of Pool i
D_i = Dredging Demand of Pool i

* Task 4 was revised as reflected in the work performed by:
(1) allocating dredge capacity to each pool based on the historical dredge demand and then calculating the probability of capacity exceedance, and
(2) ignoring for this phase of the project, the dredge material disposal constraints, which were not found to be a factor in any of the three Corps Districts.

(b) A second limit state for each pool reach will be imposed based on the availability of dredged material placement sites within each pool, (C_{pi}). Note that C_{pi} is time dependent and will be furnished by the Corps Districts.

Exceedance of either limit state (a) or the limit state (b) above would predict channel closure. So that:

$$C_i = \min [C_{di}, C_{pi}]$$

where: C_i = Minimum Capacity for Pool i based on dredging capacity and placement site availability.

5. Task 5 - Apply the Dredge Capacity Reliability Model and develop time dependent reliability indices (β) and the hazard function for each pool (12 pools in St. Paul District, 12 pools in Rock Island District, 4 pools in St. Louis District and 8 pools on the Illinois Waterway) in accordance with ETL 1110-2-532 (Reference 2).

6. Task 6 - Prepare a final report to document and summarize the results of the analysis and to describe a step-by-step procedure for applying the Dredge Capacity Reliability Model. Prior to submission of the final report a draft report shall be submitted to the Contracting Officer's Representative (COR) for review prior to production of the final report.

B. Project Review

Meet with the COR to review project status twice during commencement of the project. The University of Virginia team will be expected to travel for these two meetings.

C. Schedules

TASK	COMPLETION DATE
1	February 1, 1995
2	March 1, 1995
3	March 30, 1995
4	April 15, 1995
5	May 10, 1995
6	May 30, 1995

D. References

1. Center for Risk Management of Engineering Systems, *Channel Reliability of the Navigation System in the Upper Mississippi River*, Draft Final Report, University of Virginia, Charlottesville, VA, November 16, 1994.

2. Department of the Army, US Army Corps of Engineers, Washington, DC, *ETL 1110-2-532, Reliability Assessment of Navigation Structures*, May 1, 1992.

Appendix B: Worksheets and Data for St. Paul District

Table B.1 Historical Dredging Data for St. Paul District (cyd)

Year	Pool 1	Pool 2	Pool 3	Pool 4	Pool*5	Pool 5A	Pool 6	Pool*7	Pool 8	Pool 9	Pool 10	System
1975	135,922	26,630	0	290,235	52,149	36,112	15,345	49,025	23,411	0	38,322	667,151
1976	17,110	159,440	0	5,657	51,751	42,973	39,758	7,600	32,295	10,367	130,866	497,817
1977	14,563	23,027	0	32,918	19,740	0	0	35,515	23,973	0	0	149,736
1978	59,858	0	0	233,538	80,027	73,418	0	42,646	11,667	63,424	34,824	599,402
1979	83,200	173,500	0	340,500	110,700	17,000	500	0	0	0	0	725,400
1980	0	20,600	0	120,000	15,700	50,900	5,100	8,200	0	41,400	4,500	266,400
1981	0	28,000	0	140,700	0	28,500	0	37,600	69,600	29,900	52,900	387,200
1982	59,098	23,130	0	377,467	74,566	25,945	2,159	59,778	46,006	42,471	68,688	779,308
1983	13,302	7,808	0	270,210	113,854	94,380	6,755	0	39,873	73,841	0	620,023
1984	0	258,732	5,500	560,043	84,789	22,208	0	14,935	43,143	127,392	0	1,116,742
1985	29,079	225,793	0	338,323	1,957	66,700	29,613	47,986	90,568	27,509	67,095	924,623
1986	800	37,469	39,571	283,619	272,327	39,426	23,045	107,488	138,426	0	0	942,171
1987	119,410	47,306	3,174	43,545	24,622	0	760	2,409	18,153	21,101	0	280,480
1988	21,844	160,655	0	395,607	39,970	7,853	0	500	23,313	5,322	0	655,064
1989	0	103,547	37,857	0	40,306	99,916	14,542	73,594	107,702	31,939	0	509,403
1990	40	22,798	0	337,303	18,342	13,835	0	63,107	53,825	32,719	0	541,969
1991	16,204	1,250	0	427,827	43,809	11,469	9,063	30,569	4,660	19,586	0	564,437
1992	16,583	18,255	62,824	374,472	77,044	45,898	0	9,183	46,894	51,783	70,829	773,765
1993	0	317,232	8,113	218,762	101,233	33,465	2,613	85,320	68,813	13,000	3,047	851,598

* — rehabilitation was performed in pools 5 and 7 between 1975-1993

Table B.2 Average Annual Discharge Data for St. Paul District (cfs)

Year	Pool 1	Pool 2	Pool 3	Pool 4	Pool 5	Pool 5A	Pool 6	Pool 7	Pool 8	Pool 9	Pool 10
1975	15,092	15,092	21,046	33,765	35,130	35,486	36,718	35,735	38,604	40,738	50,388
1976	5,570	5,570	10,002	18,536	19,427	19,409	20,442	20,787	22,485	24,087	32,972
1977	6,067	6,067	10,573	18,948	19,479	19,906	20,461	20,512	22,041	22,316	28,713
1978	13,937	13,937	19,179	30,713	32,362	32,357	32,841	33,875	36,834	37,611	49,071
1979	19,812	19,812	25,676	39,024	40,748	40,875	40,910	41,113	44,752	46,006	58,406
1980	9,313	9,313	12,462	23,608	25,098	25,137	25,671	27,914	30,593	31,192	42,422
1981	10,143	10,143	15,212	25,039	26,402	27,328	28,245	29,145	31,578	31,492	41,725
1982	17,727	17,727	23,194	37,610	39,608	39,766	40,117	41,187	44,789	46,162	57,443
1983	20,550	20,550	26,736	42,641	44,755	44,962	46,286	46,082	50,078	52,604	64,503
1984	23,114	23,114	28,999	44,153	45,549	43,919	46,680	45,419	49,563	52,705	64,415
1985	21,831	21,831	27,087	44,502	44,505	43,924	45,102	44,671	48,130	50,541	64,064
1986	29,635	29,635	37,919	57,779	58,481	59,421	60,805	57,385	63,857	65,801	78,785
1987	9,079	9,079	12,272	20,826	21,405	21,390	21,814	22,941	24,598	25,150	32,692
1988	5,116	5,116	8,705	15,585	15,818	16,281	16,109	17,380	19,143	19,746	25,917
1989	7,461	7,461	11,100	19,879	20,256	20,475	20,423	21,830	23,236	24,365	30,502
1990	9,115	9,115	14,063	25,500	26,576	27,057	28,109	28,890	30,785	31,485	40,927
1991	16,937	16,937	24,282	40,342	42,059	41,127	41,477	42,594	45,689	47,329	59,707
1992	14,841	14,841	21,473	34,447	35,052	36,083	36,459	37,751	40,386	40,862	52,596
1993	27,621	27,621	34,461	51,877	54,419	54,970	56,907	55,940	59,678	64,351	81,589
Mean	14,893	14,893	20,234	32,883	34,059	34,204	35,030	35,324	38,254	39,713	50,360
Std. Dev.	7,457	7,457	8,631	12,196	12,508	12,431	12,848	11,945	13,069	13,918	16,464

Table B.3 Computed Pre-Rehabilitation Dredging Data for St. Paul District (cyd)

Year	Pool 1	Pool 2	Pool 3	Pool 4	Pool 5*	Pool 5A	Pool 6	Pool 7*	Pool 8	Pool 9	Pool 10	System
1973	31,167	83,340	8,397	270,179	80,651	38,490	8,124	39,757	50,569	34,756	21,913	667,343
1974	31,824	74,187	6,656	216,329	41,278	35,527	7,250	28,568	39,078	28,917	26,452	536,066
1975	30,830	88,024	8,611	258,365	69,338	37,828	8,011	34,577	44,700	31,627	24,787	636,698
1976	33,953	44,554	3,913	150,955	0	32,059	6,513	18,967	27,798	23,791	28,900	371,403
1977	33,790	46,822	4,156	153,860	0	32,238	6,514	18,680	27,332	22,958	29,905	376,257
1978	31,209	82,751	7,817	236,839	56,721	36,706	7,654	32,635	42,844	30,156	25,098	590,429
1979	29,282	109,572	10,580	295,457	94,945	39,762	8,397	40,193	51,147	34,107	22,893	736,335
1980	32,725	61,641	4,960	186,728	23,611	34,115	6,994	26,410	36,300	27,135	26,668	467,287
1981	32,453	65,431	6,129	196,821	29,555	34,901	7,231	27,695	37,333	27,276	26,832	491,657
1982	29,966	100,054	9,524	285,484	89,749	39,364	8,324	40,270	51,186	34,180	23,120	711,221
1983	29,040	112,942	11,031	320,968	113,209	41,229	8,891	45,382	56,732	37,212	21,453	798,088
1984	28,199	124,647	11,993	331,632	116,828	40,854	8,928	44,690	56,192	37,259	21,474	822,696
1985	28,620	118,790	11,180	334,094	112,070	40,856	8,782	43,908	54,689	36,241	21,557	810,787
1986	26,060	154,417	15,787	427,737	175,773	46,417	10,228	57,185	71,181	43,422	18,080	1,046,288
1987	32,802	60,573	4,879	167,106	6,778	32,770	6,639	21,217	30,013	24,292	28,966	416,035
1988	34,102	42,481	3,362	130,141	0	30,937	6,114	15,410	24,293	21,748	30,566	339,154
1989	33,333	53,187	4,380	160,427	1,541	32,442	6,511	20,057	28,585	23,922	29,483	393,867
1990	32,790	60,738	5,641	200,072	30,348	34,804	7,218	27,429	36,501	27,273	27,021	489,835
1991	30,225	96,447	9,987	304,753	100,921	39,853	8,449	41,740	52,129	34,729	22,586	741,818
1992	30,912	86,878	8,792	263,176	68,982	38,043	7,987	36,682	46,569	31,686	24,265	643,972
1993	26,721	145,223	14,316	386,110	157,258	44,820	9,869	55,676	66,798	42,740	17,418	966,950
Mean	30,953	86,319	8,195	251,297	65,217	37,334	7,839	34,149	44,379	31,211	24,735	621,628
Std. Dev.	2,330	32,427	3,500	82,089	52,905	4,259	1,131	11,965	13,128	6,286	3,762	201,061

* — rehabilitation was performed in pools 5 and 7 between 1975-1993 and the demand is calculated from the pre-rehabilitation data only

Table B.4 Computed Pre-Rehabilitation REST-OF-SYSTEM Dredging Demand for St. Paul District (cyd)

Year	Pool 1	Pool 2	Pool 3	Pool 4	Pool*5	Pool 5A	Pool 6	Pool*7	Pool 8	Pool 9	Pool 10
1973	636,176	584,003	658,946	397,164	586,692	628,853	659,219	627,586	616,774	632,587	645,430
1974	504,241	461,879	529,410	319,736	494,787	500,539	528,816	507,497	496,988	507,149	509,614
1975	605,868	548,674	628,088	378,333	567,361	598,870	628,688	602,121	591,998	605,071	611,912
1976	337,450	326,849	367,489	220,448	371,403	339,343	364,890	352,435	343,605	347,611	342,503
1977	342,467	329,435	372,101	222,397	376,257	344,019	369,743	357,577	348,925	353,299	346,352
1978	559,220	507,678	582,612	353,589	533,708	553,723	582,775	557,794	547,585	560,273	565,331
1979	707,053	626,762	725,755	440,878	641,390	696,573	727,938	696,142	685,188	702,228	713,442
1980	434,561	405,645	462,327	280,559	443,675	433,172	460,293	440,877	430,987	440,152	440,619
1981	459,204	426,226	485,528	294,837	462,102	456,756	484,426	463,962	454,324	464,381	464,825
1982	681,255	611,167	701,697	425,737	621,472	671,857	702,898	670,951	660,035	677,041	688,101
1983	769,048	685,147	787,057	477,120	684,879	756,860	789,197	752,706	741,356	760,877	776,635
1984	794,497	698,049	810,703	491,064	705,868	781,842	813,769	778,007	766,505	785,437	801,223
1985	782,167	691,997	799,607	476,693	698,717	769,931	802,004	766,878	756,098	774,546	789,230
1986	1,020,228	891,871	1,030,501	618,551	870,515	999,871	1,036,060	989,103	975,107	1,002,866	1,028,208
1987	383,233	355,462	411,156	248,929	409,257	383,265	409,396	394,818	386,022	391,744	387,069
1988	305,052	296,673	335,792	209,012	339,154	308,217	333,040	323,744	314,860	317,405	308,588
1989	360,534	340,681	389,487	233,440	392,326	361,425	387,356	373,811	365,282	369,945	364,384
1990	457,044	429,097	484,194	289,763	459,487	455,031	482,616	462,406	453,333	462,562	462,814
1991	711,593	645,371	731,831	437,065	640,898	701,966	733,370	700,079	689,689	707,089	719,233
1992	613,060	557,094	635,180	380,797	574,990	605,930	635,985	607,290	597,403	612,286	619,707
1993	940,229	821,727	952,633	580,840	809,692	922,130	957,081	911,273	900,151	924,210	949,532
Mean	590,675	535,309	613,433	370,331	556,411	584,294	613,789	587,479	577,248	590,417	596,893
Std. Dev.	203,370	168,979	197,572	119,070	148,265	196,810	199,932	189,169	187,999	194,815	204,763

* — rehabilitation was performed in pools 5 and 7 between 1975-1993 and the demand is calculated from the pre-rehabilitation data only

Table B.5 Intermediate Computational Results for Obtaining the Pre-Rehabilitation Probability of Pool Failure (St. Paul District)

	μ_{SM_i}	σ_{SM_i}	$Pr(SM_i < 0)$	$\mu_{SM_{\neg i}}$	$\sigma_{SM_{\neg i}}$	$Pr(SM_{\neg i} < 0)$	p_i
Pool 1	68,989	301,625	0.4090	1,316,512	1,399,759	0.1736	0.0710
Pool 2	192,390	504,725	0.3520	1,193,111	1,452,323	0.2061	0.0725
Pool 3	18,265	155,234	0.4522	1,367,236	1,374,814	0.1611	0.0728
Pool 4	560,097	863,314	0.2578	825,404	1,606,158	0.3050	0.0786
Pool 5*	145,357	440,993	0.3707	1,240,144	1,428,513	0.1922	0.0712
Pool 5A	83,211	331,277	0.4013	1,302,290	1,405,508	0.1762	0.0707
Pool 6	17,472	151,791	0.4522	1,368,029	1,374,774	0.1587	0.0718
Pool 7*	76,112	317,031	0.4052	1,309,389	1,401,122	0.1762	0.0714
Pool 8	98,913	361,392	0.3936	1,286,589	1,411,654	0.1814	0.0714
Pool 9	69,564	302,936	0.4090	1,315,937	1,398,812	0.1736	0.0710
Pool 10	55,130	269,651	0.4207	1,330,371	1,393,419	0.1711	0.0720
System	1,385,501	1,366,535	0.1562				

Table B.6 Computed Post-Rehabilitation Dredging Data for St. Paul District (cyd)

Year	Pool 1	Pool 2	Pool 3	Pool 4	Pool*5	Pool 5A	Pool 6	Pool 7^	Pool 8	Pool 9	Pool 10	System
1973	31,167	83,340	8,397	270,179	60,526	38,490	8,124	39,757	50,569	34,756	21,913	647,218
1974	31,824	74,187	6,656	216,329	46,336	35,527	7,250	28,568	39,078	28,917	26,452	541,124
1975	30,830	88,024	8,611	258,365	56,449	37,828	8,011	34,577	44,700	31,627	24,787	623,809
1976	33,953	44,554	3,913	150,955	30,653	32,059	6,513	18,967	27,798	23,791	28,900	402,056
1977	33,790	46,822	4,156	153,860	30,739	32,238	6,514	18,680	27,332	22,958	29,905	406,996
1978	31,209	82,751	7,817	236,839	51,902	36,706	7,654	32,635	42,844	30,156	25,098	585,610
1979	29,282	109,572	10,580	295,457	65,677	39,762	8,397	40,193	51,147	34,107	22,893	707,067
1980	32,725	61,641	4,960	186,728	39,969	34,115	6,994	26,410	36,300	27,135	26,668	483,645
1981	32,453	65,431	6,129	196,821	42,111	34,901	7,231	27,695	37,333	27,276	26,832	504,213
1982	29,966	100,054	9,524	285,484	63,805	39,364	8,324	40,270	51,186	34,180	23,120	685,277
1983	29,040	112,942	11,031	320,968	72,260	41,229	8,891	45,382	56,732	37,212	21,453	757,139
1984	28,199	124,647	11,993	331,632	73,564	40,854	8,928	44,690	56,192	37,259	21,474	779,432
1985	28,620	118,790	11,180	334,094	71,849	40,856	8,782	43,908	54,689	36,241	21,557	770,566
1986	26,060	154,417	15,787	427,737	94,807	46,417	10,228	57,185	71,181	43,422	18,080	965,322
1987	32,802	60,573	4,879	167,106	33,903	32,770	6,639	21,217	30,013	24,292	28,966	443,160
1988	34,102	42,481	3,362	130,141	24,725	30,937	6,114	15,410	24,293	21,748	30,566	363,878
1989	33,333	53,187	4,380	160,427	32,015	32,442	6,511	20,057	28,585	23,922	29,483	424,341
1990	32,790	60,738	5,641	200,072	42,397	34,804	7,218	27,429	36,501	27,273	27,021	501,884
1991	30,225	96,447	9,987	304,753	67,831	39,853	8,449	41,740	52,129	34,729	22,586	708,729
1992	30,912	86,878	8,792	263,176	56,321	38,043	7,987	36,682	46,569	31,686	24,265	631,311
1993	26,721	145,223	14,316	386,110	88,135	44,820	9,869	55,676	66,798	42,740	17,418	897,826
Mean	30,953	86,319	8,195	251,297	54,570	37,334	7,839	34,149	44,379	31,211	24,735	610,981
Std. Dev.	2,330	32,427	3,500	82,089	19,625	4,259	1,131	11,965	13,128	6,286	3,762	167,869

* — rehabilitation was performed in pool 5 between 1975-1993 and the demand is calculated from the post-rehabilitation data only

^ — rehabilitation was performed in pool 7 between 1975-1993, insufficient post-rehabilitation record

Table B.7 Computed Post-Rehabilitation REST-OF-SYSTEM Dredging Demand for St. Paul District (cyd)

Year	Pool 1	Pool 2	Pool 3	Pool 4	Pool*5	Pool 5A	Pool 6	Pool 7^	Pool 8	Pool 9	Pool 10
1973	616,052	563,878	638,821	377,039	586,692	608,728	639,094	607,462	596,649	612,462	625,305
1974	509,299	466,937	534,468	324,794	494,787	505,597	533,874	512,555	502,046	512,207	514,672
1975	592,979	535,785	615,199	365,444	567,361	585,981	615,799	589,232	579,109	592,182	599,023
1976	368,103	357,503	398,143	251,102	371,403	369,997	395,543	383,089	374,258	378,265	373,157
1977	373,206	360,173	402,840	253,135	376,257	374,758	400,481	388,316	379,664	384,038	377,091
1978	554,401	502,858	577,793	348,770	533,708	548,904	577,956	552,975	542,766	555,454	560,512
1979	677,785	597,495	696,487	411,610	641,390	667,305	698,671	666,874	655,920	672,961	684,174
1980	450,919	422,003	478,685	296,917	443,675	449,530	476,651	457,235	447,345	456,510	456,977
1981	471,760	438,783	498,084	307,393	462,102	469,312	496,983	476,518	466,881	476,937	477,381
1982	655,311	585,223	675,753	399,793	621,472	645,913	676,954	645,007	634,091	651,097	662,157
1983	728,099	644,197	746,108	436,171	684,879	715,910	748,247	711,757	700,407	719,927	735,686
1984	751,233	654,785	767,439	447,800	705,868	738,578	770,505	734,743	723,240	742,173	757,958
1985	741,947	651,777	759,386	436,472	698,717	729,710	761,784	726,658	715,877	734,326	749,010
1986	939,262	810,905	949,535	537,585	870,515	918,905	955,094	908,137	894,142	921,900	947,242
1987	410,357	382,586	438,281	276,054	409,257	410,389	436,521	421,943	413,146	418,868	414,194
1988	329,776	321,398	360,517	233,737	339,154	332,941	357,765	348,469	339,585	342,130	333,313
1989	391,009	371,155	419,961	263,915	392,326	391,900	417,831	404,285	395,756	400,419	394,859
1990	469,093	441,146	496,243	301,812	459,487	467,080	494,665	474,455	465,383	474,611	474,863
1991	678,504	612,282	698,742	403,976	640,898	668,876	700,280	666,989	656,599	674,000	686,143
1992	600,398	544,432	622,518	368,135	574,990	593,268	623,324	594,628	584,742	599,625	607,045
1993	871,106	752,603	883,510	511,716	809,692	853,006	887,957	842,150	831,028	855,086	880,408
Mean	580,029	524,662	602,786	359,684	556,411	573,647	603,142	576,832	566,602	579,770	586,246
Std. Dev.	170,179	135,782	164,380	85,871	148,265	163,618	166,740	155,976	154,808	161,625	171,570

* — rehabilitation was performed in pool 5 between 1975-1993 and the demand is calculated from the post-rehabilitation data only

^ — rehabilitation was performed in pool 7 between 1975-1993, insufficient post-rehabilitation record

Table B.8 Intermediate Computational Results for Obtaining the Post-Rehabilitation Probability of Pool Failure (St. Paul District)

	μ_{SM_i}	σ_{SM_i}	$Pr(SM_i < 0)$	$\mu_{SM_{\neg i}}$	$\sigma_{SM_{\neg i}}$	$Pr(SM_{\neg i} < 0)$	p_i
Pool 1	70,730	304,242	0.4090	1,325,417	1,395,891	0.1711	0.0700
Pool 2	197,247	509,085	0.3483	1,198,901	1,450,360	0.2033	0.0708
Pool 3	18,726	156,580	0.4522	1,377,422	1,370,591	0.1587	0.0718
Pool 4	574,237	870,737	0.2546	821,911	1,608,045	0.3050	0.0777
Pool 5*	124,697	404,430	0.3783	1,271,451	1,418,504	0.1841	0.0696
Pool 5A	85,312	334,150	0.3974	1,310,836	1,401,928	0.1736	0.0690
Pool 6	17,913	153,108	0.4522	1,378,235	1,370,487	0.1562	0.0706
Pool 7^	78,034	319,777	0.4052	1,318,114	1,397,654	0.1736	0.0703
Pool 8	101,410	364,523	0.3897	1,294,738	1,408,426	0.1788	0.0697
Pool 9	71,320	305,563	0.4090	1,324,828	1,395,150	0.1711	0.0700
Pool 10	56,522	271,989	0.4168	1,339,626	1,389,386	0.1685	0.0702
System	1,396,148	1,362,047	0.1515				

Appendix C: Worksheets and Data for Rock Island District

Table C.1 Historical Dredging Data for Rock Island District (cyd)

Year	Pool 11	Pool 12	Pool 13	Pool 14	Pool 15	Pool 16	Pool 17	Pool 18	Pool 19	Pool 20	Pool 21	Pool 22	System
1973	140,265	0	264,298	72,506	0	72,627	68,657	178,812	65,956	428,374	105,968	336,418	1,733,881
1974	192,845	0	24,443	0	0	0	45,028	127,855	55,244	267,143	227,148	0	939,706
1975	0	0	12,037	120,018	0	27,095	0	119,230	0	137,471	60,723	114,899	591,473
1976	0	0	16,497	0	0	17,841	0	44,151	0	68,075	0	60,310	206,874
1977	0	0	48,055	0	0	17,991	0	5,885	0	0	0	0	71,931
1978	27,018	0	0	0	0	11,576	0	11,166	0	0	0	18,800	68,560
1979	0	22,534	0	0	0	9,956	8,711	38,911	1,433	65,254	0	58,407	205,206
1980	0	0	73,300	0	0	83,137	0	70,380	0	68,996	69,090	44,436	409,339
1981	23,636	25,328	0	0	0	26,624	0	57,490	31,556	0	0	65,530	230,164
1982	0	0	0	0	0	0	66,120	92,650	0	164,791	33,964	242,216	599,741
1983	12,578	0	36,341	0	0	40,797	0	50,986	49,463	42,122	86,400	279,548	598,235
1984	0	66,051	0	0	7,507	18,825	0	108,956	24,676	67,631	94,354	124,459	512,459
1985	27,326	0	0	26,666	6,427	42,510	84,068	95,964	0	82,013	107,092	0	472,066
1986	0	0	68,271	79,672	0	77,511	0	173,242	0	59,000	85,111	0	542,807
1987	0	110,389	70,927	0	0	103,182	0	109,185	54,588	105,169	178,126	368,448	1,100,014
1988	49,851	0	0	0	0	850	0	0	0	0	45,166	0	95,867
1989	59,393	0	173,697	0	0	79,011	0	0	0	112,730	112,734	34,995	572,560
1990	18,991	87,582	0	38,444	0	27,348	0	77,025	0	0	102,450	39,635	391,475
1991	0	0	153,867	86,515	0	140,947	0	0	127,263	0	239,327	199,145	947,064

Table C.2 Average Annual Discharge or Stage Data for Rock Island District (cfs or feet)

Year	Pool*11	Pool 12	Pool 13	Pool*14	Pool 15	Pool 16	Pool 17	Pool*18	Pool 19	Pool*20	Pool 21	Pool 22
1973	49,352	592.66	583.24	71,848	560.91	546.11	538.23	118,553	518.16	118,553	472.58	461.33
1974	39,729	591.97	582.92	52,900	560.97	545.36	537.04	93,714	518.16	93,714	471.29	460.27
1975	39,298	592.25	583.04	50,130	561.01	545.32	536.71	74,066	518.17	74,066	470.71	459.80
1976	29,088	592.10	582.95	39,689	561.03	545.18	536.28	55,340	518.21	55,340	470.38	459.63
1977	17,399	592.02	582.97	21,809	561.07	545.18	535.93	30,382	518.22	30,382	470.09	459.47
1978	41,116	592.09	582.87	50,903	561.05	544.99	536.39	77,995	518.22	77,995	470.49	459.67
1979	45,319	592.44	583.01	59,637	560.94	545.75	537.33	90,534	518.30	90,534	471.38	460.47
1980	34,783	591.99	582.93	42,872	561.08	545.01	536.04	64,423	518.28	64,423	470.13	459.47
1981	32,073	591.95	582.98	43,737	561.06	545.07	536.00	63,877	518.27	63,877	470.19	459.52
1982	40,828	592.35	583.00	52,325	560.90	545.30	537.23	87,934	518.21	87,934	471.26	460.26
1983	57,294	592.35	583.07	70,261	560.90	545.44	537.42	109,147	518.25	109,147	471.38	460.38
1984	53,638	592.31	582.99	63,632	560.92	545.16	537.23	95,186	518.25	95,186	471.33	460.26
1985	49,120	592.19	582.92	58,889	561.03	545.05	537.08	82,159	518.24	82,159	470.61	459.86
1986	63,250	592.94	583.35	73,782	560.92	546.06	538.18	110,027	518.24	110,027	472.02	460.96
1987	39,012	591.95	582.93	52,216	561.04	545.10	535.86	73,956	518.19	73,956	470.04	459.51
1988	21,934	592.02	582.96	27,489	561.06	545.22	535.96	43,581	518.21	43,581	470.14	459.54
1989	25,767	592.05	583.01	31,811	561.05	545.19	535.98	40,006	518.23	40,006	470.12	459.53
1990	30,634	592.05	583.04	31,811	561.02	545.21	536.45	65,718	518.27	65,718	470.59	459.85
1991	41,806	592.20	582.98	56,347	561.06	545.18	537.10	81,169	518.21	81,169	471.03	460.14
Mean	39,549	592.20	583.01	50,110	561.00	545.31	536.76	76,725	518.23	76,725	470.83	460.00
Std. Dev.	11,935	0.26	0.11	14,963	0.07	0.32	0.74	24,049	0.04	24,049	0.72	0.53

* — discharge data in cfs

Table C.3 Computed Pre-Rehabilitation Dredging Data for Rock Island District (cyd)

Year	Pool*11	Pool 12	Pool*13	Pool*14	Pool 15	Pool*16	Pool 17	Pool*18	Pool 19	Pool*20	Pool*21	Pool*22	System
1973	154,728	3,184	238,904	37,679	1,237	51,072	40,327	124,168	41,844	313,376	107,022	256,148	1,369,690
1974	85,870	23,214	0	19,569	905	37,290	19,295	91,947	41,844	201,343	78,054	113,615	712,945
1975	82,786	15,086	81,289	16,921	684	36,555	13,463	66,459	38,765	112,723	65,030	50,417	580,176
1976	9,727	19,440	10,362	6,942	573	33,982	5,863	42,167	26,450	28,261	57,619	27,558	268,944
1977	0	21,762	26,124	0	352	33,982	0	9,791	23,371	0	51,107	6,043	172,533
1978	95,794	19,730	0	17,660	463	30,490	7,807	71,556	23,371	130,444	60,089	32,936	490,341
1979	125,870	9,571	57,647	26,008	1,071	44,457	24,421	87,821	0	187,000	80,075	140,508	784,447
1980	50,478	22,633	0	9,984	297	30,858	1,621	53,950	4,898	69,229	52,005	6,043	301,996
1981	31,086	23,794	34,004	10,811	407	31,960	914	53,242	7,976	66,766	53,353	12,767	327,081
1982	93,734	12,183	49,766	19,019	1,292	36,187	22,653	84,449	26,450	175,273	77,380	112,271	710,656
1983	211,558	12,183	104,931	36,162	1,292	38,760	26,011	111,967	14,134	270,951	80,075	128,407	1,036,431
1984	185,397	13,344	41,885	29,826	1,182	33,614	22,653	93,856	14,134	207,982	78,952	112,271	835,097
1985	153,068	16,827	0	25,293	573	31,593	20,002	76,957	17,213	149,225	62,784	58,485	612,021
1986	254,177	0	325,592	39,527	1,182	50,154	39,443	113,108	17,213	274,920	94,447	206,396	1,416,159
1987	80,739	23,794	0	18,915	518	32,512	0	66,316	32,608	112,226	49,984	11,422	429,034
1988	0	21,762	18,243	0	407	34,717	207	26,913	26,450	0	52,230	15,456	196,386
1989	0	20,891	57,647	0	463	34,166	561	22,276	20,292	0	51,781	14,111	222,187
1990	20,789	20,891	81,289	0	629	34,533	8,868	55,630	7,976	75,070	62,335	57,140	425,150
1991	100,732	16,537	34,004	22,863	407	33,982	20,356	75,673	26,450	144,760	72,215	96,135	644,115
Mean	91,396	16,675	61,141	17,746	733	36,361	14,446	69,908	21,655	132,608	67,712	76,744	607,126
Std. Dev.	74,957	6,866	85,178	12,910	360	5,944	13,043	31,197	12,028	95,393	16,106	71,405	364,990

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the pre-rehabilitation data only

Table C.4 Computed Pre-Rehabilitation REST-OF-SYSTEM Dredging Demand for Rock Island District (cyd)

Year	Pool*11	Pool 12	Pool*13	Pool*14	Pool 15	Pool*16	Pool 17	Pool*18	Pool 19	Pool*20	Pool*21	Pool*22
1973	1,214,962	1,366,506	1,130,787	1,332,011	1,368,453	1,318,618	1,329,363	1,245,522	1,327,846	1,056,314	1,262,668	1,113,542
1974	627,075	689,731	712,945	693,376	712,040	675,655	693,650	620,998	671,100	511,602	634,891	599,329
1975	497,391	565,090	498,887	563,255	579,492	543,621	566,713	513,717	541,411	467,454	515,146	529,759
1976	259,218	249,504	258,582	262,002	268,371	234,962	263,081	226,777	242,494	240,683	211,325	241,386
1977	172,533	150,770	146,409	172,533	172,180	138,551	172,533	162,741	149,162	172,533	121,426	166,489
1978	394,546	470,611	490,341	472,681	489,878	459,851	482,534	418,785	466,970	359,897	430,252	457,405
1979	658,578	774,877	726,801	758,440	783,376	739,991	760,027	696,626	784,447	597,448	704,372	643,939
1980	251,518	279,363	301,996	292,012	301,699	271,138	300,374	248,046	297,098	232,767	249,991	295,952
1981	295,995	303,287	293,077	316,270	326,674	295,121	326,167	273,840	319,105	260,315	273,729	314,315
1982	616,923	698,473	660,890	691,637	709,364	674,469	688,003	626,208	684,207	535,384	633,276	598,386
1983	824,873	1,024,248	931,500	1,000,269	1,035,139	997,671	1,010,420	924,465	1,022,297	765,480	956,356	908,025
1984	649,700	821,753	793,212	805,271	833,915	801,483	812,444	741,241	820,963	627,115	756,145	722,826
1985	458,953	595,194	612,021	586,728	611,448	580,428	592,019	535,064	594,808	462,796	549,237	553,536
1986	1,161,982	1,416,159	1,090,568	1,376,632	1,414,978	1,366,006	1,376,716	1,303,051	1,398,946	1,141,239	1,321,713	1,209,763
1987	348,295	405,240	429,034	410,119	428,516	396,522	429,034	362,718	396,427	316,808	379,050	417,612
1988	196,386	174,624	178,143	196,386	195,978	161,669	196,178	169,472	169,936	196,386	144,156	180,930
1989	222,187	201,296	164,540	222,187	221,724	188,021	221,626	199,911	201,895	222,187	170,406	208,076
1990	404,361	404,258	343,861	425,150	424,521	390,617	416,282	369,520	417,173	350,080	362,815	368,010
1991	543,383	627,577	610,110	621,251	643,707	610,133	623,759	568,442	617,665	499,355	571,899	547,980
Mean	515,729	590,451	545,984	589,379	606,392	570,765	592,680	537,218	585,471	474,518	539,413	530,382
Std. Dev.	299,295	371,170	301,566	353,099	364,688	360,111	352,428	336,061	363,539	274,537	349,659	297,432

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the pre-rehabilitation data only

Table C.5 Intermediate Computational Results for Obtaining the Pre-Rehabilitation Probability of Pool Failure (Rock Island District)

	μ_{SM_i}	σ_{SM_i}	$Pr(SM_i < 0)$	$\mu_{SM_{\neg i}}$	$\sigma_{SM_{\neg i}}$	$Pr(SM_{\neg i} < 0)$	p_i
Pool*11	19,108	93,932	0.4207	107,822	337,743	0.3745	0.1576
Pool 12	3,486	25,136	0.4443	123,442	399,550	0.3783	0.1681
Pool*13	12,782	96,949	0.4483	114,147	338,193	0.3669	0.1645
Pool*14	3,710	28,088	0.4483	123,220	382,870	0.3745	0.1679
Pool 15	153	5,082	0.4880	126,776	392,825	0.3745	0.1828
Pool*16	7,602	36,198	0.4168	119,327	390,184	0.3783	0.1577
Pool 17	3,020	26,013	0.4522	123,908	382,100	0.3745	0.1693
Pool*18	14,615	58,519	0.4013	112,313	369,698	0.3821	0.1533
Pool 19	4,527	30,066	0.4404	122,401	392,694	0.3783	0.1666
Pool*20	27,724	117,259	0.4052	99,205	318,290	0.3783	0.1533
Pool*21	14,156	51,319	0.3897	112,773	382,000	0.3821	0.1489
Pool*22	16,044	88,259	0.4286	110,884	335,328	0.3707	0.1589
System	126,929	393,073	0.3745				

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the pre-rehabilitation data only

Table C.6 Computed Post-Rehabilitation Dredging Data for Rock Island District (cyd)

Year	Pool*11	Pool 12	Pool*13	Pool 14	Pool 15	Pool 16	Pool 17	Pool*18	Pool 19	Pool*20	Pool 21	Pool*22	System
1973	8,273	3,184	59,419	37,679	1,237	51,072	40,327	114,614	41,844	52,482	107,022	0	517,154
1974	14,502	23,214	43,345	19,569	905	37,290	19,295	81,287	41,844	52,233	78,054	65,210	476,747
1975	14,780	15,086	49,373	16,921	684	36,555	13,463	54,926	38,765	52,036	65,030	100,132	457,750
1976	21,389	19,440	44,852	6,942	573	33,982	5,863	29,801	26,450	51,848	57,619	112,763	411,521
1977	28,954	21,762	45,857	0	352	33,982	0	0	23,371	51,597	51,107	124,651	381,633
1978	13,604	19,730	40,834	17,660	463	30,490	7,807	60,197	23,371	52,075	60,089	109,791	436,111
1979	10,884	9,571	47,866	26,008	1,071	44,457	24,421	77,021	0	52,201	80,075	50,349	423,922
1980	17,703	22,633	43,848	9,984	297	30,858	1,621	41,988	4,898	51,939	52,005	124,651	402,424
1981	19,457	23,794	46,359	10,811	407	31,960	914	41,255	7,976	51,933	53,353	120,936	409,157
1982	13,790	12,183	47,364	19,019	1,292	36,187	22,653	73,532	26,450	52,175	77,380	65,953	447,979
1983	3,133	12,183	50,880	36,162	1,292	38,760	26,011	101,994	14,134	52,388	80,075	57,037	474,049
1984	5,499	13,344	46,862	29,826	1,182	33,614	22,653	83,262	14,134	52,248	78,952	65,953	447,529
1985	8,424	16,827	43,345	25,293	573	31,593	20,002	65,784	17,213	52,117	62,784	95,674	439,629
1986	0	0	64,945	39,527	1,182	50,154	39,443	103,174	17,213	52,397	94,447	13,941	476,423
1987	14,966	23,794	43,848	18,915	518	32,512	0	54,778	32,608	52,034	49,984	121,679	445,635
1988	26,019	21,762	45,355	0	407	34,717	207	14,024	26,450	51,730	52,230	119,450	392,350
1989	23,538	20,891	47,866	0	463	34,166	561	9,227	20,292	51,694	51,781	120,193	380,671
1990	20,388	20,891	49,373	0	629	34,533	8,868	43,725	7,976	51,952	62,335	96,417	397,086
1991	13,157	16,537	46,359	22,863	407	33,982	20,356	64,456	26,450	52,107	72,215	74,869	443,759
Mean	14,656	16,675	47,787	17,746	733	36,361	14,446	58,687	21,655	52,062	67,712	86,297	434,817
Std. Dev.	7,646	6,866	5,708	12,910	360	5,944	13,043	31,881	12,028	241	16,106	37,646	36,702

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the post-rehabilitation data only

Table C.7 Computed Post-Rehabilitation REST-OF-SYSTEM Dredging Demand for Rock Island District (cyd)

Year	Pool*11	Pool 12	Pool*13	Pool 14	Pool 15	Pool 16	Pool 17	Pool*18	Pool 19	Pool*20	Pool*21	Pool*22
1973	508,881	513,970	457,735	479,476	515,918	466,082	476,827	402,541	475,310	464,672	410,132	517,154
1974	462,246	453,534	433,402	457,179	475,842	439,457	457,452	395,460	434,903	424,514	398,693	411,537
1975	442,969	442,664	408,377	440,829	457,066	421,195	444,287	402,824	418,984	405,714	392,720	357,618
1976	390,133	392,081	366,669	404,579	410,948	377,540	405,658	381,720	385,072	359,674	353,902	298,758
1977	352,679	359,871	335,776	381,633	381,281	347,651	381,633	381,633	358,262	330,036	330,526	256,982
1978	422,507	416,381	395,277	418,451	435,648	405,621	428,304	375,914	412,740	384,036	376,022	326,320
1979	413,039	414,352	376,056	397,914	422,851	379,465	399,502	346,901	423,922	371,721	343,847	373,573
1980	384,721	379,791	358,576	392,439	402,127	371,566	400,802	360,436	397,526	350,485	350,418	277,772
1981	389,700	385,362	362,797	398,346	408,749	377,196	408,242	367,901	401,180	357,223	355,804	288,220
1982	434,188	435,796	400,615	428,959	446,686	411,791	425,325	374,446	421,529	395,804	370,598	382,026
1983	470,915	461,866	423,169	437,887	472,756	435,289	448,037	372,055	459,914	421,661	393,974	417,012
1984	442,030	434,185	400,668	417,703	446,348	413,915	424,876	364,267	433,395	395,282	368,577	381,576
1985	431,205	422,801	396,284	414,336	439,056	408,036	419,627	373,845	422,416	387,512	376,845	343,955
1986	476,423	476,423	411,478	436,896	475,241	426,269	436,979	373,248	459,210	424,026	381,976	462,481
1987	430,670	421,841	401,788	426,720	445,117	413,124	445,635	390,857	413,028	393,601	395,651	323,956
1988	366,332	370,588	346,996	392,350	391,943	357,633	392,143	378,327	365,901	340,621	340,121	272,900
1989	357,134	359,780	332,805	380,671	380,209	346,506	380,111	371,444	360,379	328,978	328,891	260,478
1990	376,698	376,195	347,713	397,086	396,458	362,553	388,219	353,361	389,110	345,135	334,752	300,670
1991	430,601	427,221	397,399	420,895	443,351	409,777	423,403	379,303	417,309	391,652	371,543	368,890
Mean	420,162	418,142	387,031	417,071	434,084	398,456	420,372	376,131	413,163	382,755	367,105	348,520
Std. Dev.	43,060	41,564	34,161	26,060	36,445	33,345	26,942	14,831	32,628	36,482	24,907	70,632

* — rehabilitation was performed in the pool during 1973-1991 and the demand is calculated from the post-rehabilitation data only

Table C.8 Intermediate Computational Results for Obtaining the Post-Rehabilitation Probability of Pool Failure (Rock Island District)

	μ_{SM_i}	σ_{SM_i}	$Pr(SM_i < 0)$	$\mu_{SM_{\neg i}}$	$\sigma_{SM_{\neg i}}$	$Pr(SM_{\neg i} < 0)$	p_i
Pool*11	10,086	27,857	0.3594	289,150	154,467	0.0307	0.0110
Pool 12	11,476	29,386	0.3483	287,762	154,377	0.0314	0.0109
Pool*13	32,887	48,705	0.2483	266,350	157,464	0.0455	0.0113
Pool*14	12,213	32,179	0.3520	287,025	151,117	0.0287	0.0101
Pool 15	504	6,001	0.4681	298,733	150,507	0.0239	0.0112
Pool*16	25,023	42,609	0.2776	274,214	155,501	0.0392	0.0109
Pool 17	9,942	29,621	0.3669	289,295	150,737	0.0274	0.0101
Pool*18	40,388	62,367	0.2578	258,849	156,146	0.0485	0.0125
Pool 19	14,903	34,711	0.3336	284,334	153,014	0.0314	0.0105
Pool*20	35,829	50,487	0.2389	263,409	158,645	0.0485	0.0116
Pool*21	46,599	59,787	0.2177	252,639	158,820	0.0559	0.0122
Pool*22	59,389	75,115	0.2148	239,849	174,649	0.0853	0.0183
System	299,238	150,451	0.0233				

* — rehabilitation was performed in the pool between 1973-1991 and the demand is calculated from the post-rehabilitation data only

Table C.9 Historical Stage and Dredging Data for the Illinois Waterway in Rock Island District

Year	LaGrange Stage (feet)	Peoria Stage (feet)	LaGrange Dredging (cyd)	Peoria Dredging (cyd)	Illinois System (cyd)
1973	438.97	445.78	0	57,422	57,422
1974	438.45	445.70	179,611	0	179,611
1975	436.97	444.42	85,233	0	85,233
1976	435.21	443.72	55,690	6,605	62,295
1977	435.38	443.31	642,483	173,286	815,769
1978	436.17	444.06	0	0	0
1979	437.67	445.43	200,126	43,616	243,742
1980	435.84	443.59	0	0	0
1981	437.83	444.76	0	0	0
1982	439.67	446.63	322,808	0	322,808
1983	437.94	445.07	32,540	0	32,540
1984	437.35	444.76	482,665	0	482,665
1985	438.12	445.75	174,680	0	174,680
1986	437.43	444.67	253,700	0	253,700
1987	434.72	443.16	221,744	35,459	257,203
1988	434.26	443.21	320,671	12,813	333,484
1989	433.79	442.86	135,146	0	135,146
1990	439.20	446.19	200,735	0	200,735
1991	437.87	445.57	237,602	80,787	318,389
Mean	436.99	444.67	186,602	21,578	208,180
Std. Dev.	1.72	1.14	172,240	43,699	199,799

Table C.10 Computed Pool and REST-OF-SYSTEM Dredging Data for the Illinois Waterway in Rock Island District (cyd)

Year	LaGrange (cyd)	Peoria (cyd)	System (cyd)	Not LaGrange (cyd)	Not Peoria (cyd)
1973	167,168	14,728	181,896	14,728	167,168
1974	172,276	15,220	187,496	15,220	172,276
1975	186,814	23,085	209,899	23,085	186,814
1976	204,102	27,387	231,489	27,387	204,102
1977	202,432	29,906	232,339	29,906	202,432
1978	194,672	25,298	219,970	25,298	194,672
1979	179,938	16,879	196,817	16,879	179,938
1980	197,914	28,186	226,099	28,186	197,914
1981	178,366	20,996	199,362	20,996	178,366
1982	160,292	9,505	169,797	9,505	160,292
1983	177,285	19,091	196,377	19,091	177,285
1984	183,081	20,996	204,077	20,996	183,081
1985	175,517	14,913	190,430	14,913	175,517
1986	182,295	21,549	203,844	21,549	182,295
1987	208,916	30,828	239,743	30,828	208,916
1988	213,434	30,521	243,955	30,521	213,434
1989	218,051	32,671	250,722	32,671	218,051
1990	164,909	12,209	177,118	12,209	164,909
1991	177,973	16,019	193,992	16,019	177,973
Mean	186,602	21,578	208,180	21,578	186,602
Std. Dev.	16,882	6,979	23,724	6,979	16,882

Table C.11 Intermediate Computational Results for Obtaining the Probability of Pool Failure on the Illinois Waterway (Rock Island District)

	μ_{SM_i}	σ_{SM_i}	$Pr(SM_i < 0)$	$\mu_{SM_{\neg i}}$	$\sigma_{SM_{\neg i}}$	$Pr(SM_{\neg i} < 0)$	p_i
LaGrange	74,389	86,056	0.1949	8,602	122,995	0.4721	0.0920
Peoria	8,602	29,545	0.3859	74,389	95,153	0.2177	0.0840
System	82,991	92,274	0.1841				

Appendix D: Worksheet and Data for St. Louis District

Table D.1 Historical Dredging Data for St. Louis District (cyd)

Year	Pool 24	Pool 25	Pool 26	Pool 27	St. Louis	Chester	Thebes	System
1973	203,800	349,200	761,172	0	324,800	1,174,400	557,200	3,370,572
1974	218,100	1,071,100	647,980	0	1,120,200	638,400	1,156,300	4,852,080
1975	436,900	1,121,600	2,862,011	7509	1,086,600	984,300	3,397,600	9,889,011
1976	0	260,500	299,400	281,100	3,709,800	2,369,000	6,125,500	13,045,300
1977	334,300	402,100	178,610	96,900	3,359,200	1,918,100	3,524,500	9,813,710
1978	63,500	1,035,500	851,200	75,100	986,100	417,600	419,800	3,848,800
1979	211,500	867,500	116,800	125,600	1,519,000	599,200	842,300	4,281,900
1980	81,100	765,000	576,487	165,600	2,025,100	643,100	3,209,200	7,465,587
1981	0	440,400	84,352	0	1,681,800	1,256,400	4,309,200	7,772,152
1982	233,100	855,800	600,100	69,400	685,100	727,300	2,234,400	5,405,200
1983	332,500	1,099,700	467,444	0	344,000	363,000	4,204,900	6,811,544
1984	0	627,300	310,050	7,600	1,552,400	0	1,556,400	4,053,750
1985	666,900	625,400	233,845	263,000	149,600	240,000	1,974,600	4,153,345
1986	324,600	306,800	416,500	0	632,300	310,500	2,382,300	4,373,000
1987	0	536,866	458,800	420,414	847,879	1,014,758	5,229,726	8,508,443
1988	11,203	0	690,976	249,957	2,817,955	1,165,851	6,841,608	11,777,550
1989	263,724	365,278	335,499	33,722	6,201,892	4,302,089	12,222,792	23,724,996
1990	0	64,784	0	0	3,730,298	3,271,935	8,732,130	15,799,147
1991	139,315	346,805	426,331	0	847,460	807,081	2,957,525	5,524,517
1992	249,397	353,486	570,400	0	1,031,085	605,838	916,451	3,726,657
1993	0	1,457,631	699,598	67,923	57,789	0	0	2,282,941

Table D.2 Average Annual Discharge or Stage Data for St. Louis District (cfs or feet)

Year	Pool 24	Pool 25	Pool 26	Pool 27	St. Louis	Chester	Thebes
1973	443.92	429.25	18.42	18.42	332,150	347,512	360,125
1974	441.11	427.11	16.59	16.59	274,458	289,215	301,283
1975	439.66	424.57	15.84	15.84	201,452	214,427	223,961
1976	437.57	422.72	15.50	15.50	153,753	157,844	166,173
1977	436.93	420.47	15.35	15.35	105,842	109,507	113,623
1978	440.46	425.78	16.08	16.08	222,579	235,579	241,168
1979	441.56	426.43	16.99	16.99	227,781	241,752	247,147
1980	438.70	423.51	15.39	15.39	152,067	157,217	247,147
1981	439.24	423.91	15.80	15.80	172,415	178,764	182,157
1982	442.80	426.80	17.26	17.26	243,348	255,965	268,475
1983	442.48	428.55	17.10	17.10	293,667	305,502	320,567
1984	442.54	427.70	17.20	17.20	277,417	293,490	297,975
1985	442.51	425.64	17.20	17.20	231,283	245,874	261,633
1986	443.56	428.14	17.11	17.11	271,925	282,353	290,342
1987	437.72	424.80	15.50	15.50	237,833	246,484	252,650
1988	436.23	421.68	15.44	15.44	134,660	143,866	148,488
1989	437.17	421.04	15.35	15.35	109,490	116,834	121,613
1990	439.31	424.20	16.10	16.10	186,984	196,247	200,634
1991	442.41	425.49	16.30	16.30	190,825	200,610	208,700
1992	440.36	425.22	15.76	15.76	184,242	189,791	193,825
1993	446.49	432.38	21.19	21.19	428,058	441,701	444,392
Mean	440.61	425.49	16.55	16.55	220,582	230,978	242,480
Std. Dev.	2.69	2.87	1.36	1.36	77,026	79,902	79,321

* — discharge data in cfs

Table D.3 Computed Dredging Data for St. Louis District (cyd)

Year	Pool 24	Pool 25	Pool 26	Pool 27	St. Louis	Chester	Thebes	System
1973	236,942	907,433	761,172	44,337	32,181	248,630	853,487	3,084,181
1974	203,862	770,203	647,980	66,856	808,805	640,027	1,791,625	4,929,358
1975	186,793	607,323	2,862,011	76,085	1,791,580	1,142,142	3,024,396	9,690,330
1976	162,189	488,689	299,400	80,269	2,433,683	1,522,032	3,945,731	8,931,993
1977	154,655	344,405	178,610	82,114	3,078,640	1,846,559	4,783,554	10,468,537
1978	196,210	684,915	851,200	73,131	1,507,178	1,000,131	2,750,059	7,062,825
1979	209,160	726,597	116,800	61,933	1,437,151	958,686	2,654,734	6,165,062
1980	175,491	539,349	576,487	81,622	2,456,379	1,526,242	2,654,734	8,010,304
1981	181,848	564,999	84,352	76,577	2,182,464	1,381,578	3,690,892	8,162,711
1982	223,757	750,324	600,100	58,611	1,227,595	863,262	2,314,694	6,038,343
1983	219,990	862,545	467,444	60,580	550,222	530,678	1,484,173	4,175,633
1984	220,696	808,038	310,050	59,349	768,973	611,325	1,844,366	4,622,797
1985	220,343	675,938	233,845	59,349	1,390,008	931,012	2,423,779	5,934,274
1986	232,704	836,253	416,500	60,457	842,904	686,097	1,966,061	5,040,976
1987	163,955	622,072	458,800	80,269	1,301,835	926,916	2,566,998	6,120,844
1988	146,414	421,998	690,976	81,007	2,690,705	1,615,878	4,227,689	9,874,667
1989	157,480	380,957	335,499	82,114	3,029,532	1,797,367	4,656,166	10,439,116
1990	182,672	583,596	0	72,885	1,986,342	1,264,200	3,396,307	7,486,003
1991	219,166	666,319	426,331	70,424	1,934,636	1,234,908	3,267,708	7,819,492
1992	195,033	649,005	570,400	77,069	2,023,254	1,307,545	3,504,865	8,327,170
1993	267,196	1,108,148	699,598	10,250	0	0	0	2,085,193
Mean	197,931	666,624	551,788	67,395	1,594,003	1,049,296	2,752,477	6,879,515
Std. Dev.	31,655	184,121	578,603	16,743	893,876	490,411	1,205,415	2,372,829

Table D.4 Computed REST-OF-SYSTEM Dredging Demand for St. Louis District (cyd)

Year	Pool 24	Pool 25	Pool 26	Pool 27	St. Louis	Chester	Thebes
1973	2,847,239	2,176,747	2,323,009	3,039,844	3,052,000	2,835,551	2,230,694
1974	4,725,496	4,159,155	4,281,378	4,862,503	4,120,553	4,289,331	3,137,733
1975	9,503,537	9,083,007	6,828,319	9,614,245	7,898,750	8,548,188	6,665,933
1976	8,769,804	8,443,303	8,632,593	8,851,724	6,498,309	7,409,960	4,986,262
1977	10,313,883	10,124,132	10,289,927	10,386,423	7,389,897	8,621,978	5,684,984
1978	6,866,615	6,377,909	6,211,625	6,989,693	5,555,647	6,062,694	4,312,766
1979	5,955,902	5,438,464	6,048,262	6,103,128	4,727,911	5,206,375	3,510,328
1980	7,834,813	7,470,956	7,433,817	7,928,682	5,553,925	6,484,063	5,355,570
1981	7,980,862	7,597,711	8,078,359	8,086,134	5,980,247	6,781,132	4,471,818
1982	5,814,586	5,288,019	5,438,243	5,979,732	4,810,749	5,175,081	3,723,649
1983	3,955,643	3,313,088	3,708,189	4,115,053	3,625,411	3,644,954	2,691,459
1984	4,402,100	3,814,759	4,312,747	4,563,447	3,853,824	4,011,471	2,778,431
1985	5,713,931	5,258,336	5,700,429	5,874,925	4,544,266	5,003,262	3,510,495
1986	4,808,272	4,204,723	4,624,476	4,980,519	4,198,072	4,354,879	3,074,915
1987	5,956,890	5,498,772	5,662,044	6,040,576	4,819,009	5,193,928	3,553,846
1988	9,728,253	9,452,669	9,183,691	9,793,660	7,183,962	8,258,789	5,646,978
1989	10,281,636	10,058,159	10,103,617	10,357,002	7,409,584	8,641,749	5,782,950
1990	7,303,330	6,902,407	7,486,003	7,413,117	5,499,661	6,221,803	4,089,696
1991	7,600,326	7,153,173	7,393,161	7,749,067	5,884,855	6,584,584	4,551,784
1992	8,132,137	7,678,166	7,756,770	8,250,101	6,303,917	7,019,626	4,822,305
1993	1,817,997	977,045	1,385,595	2,074,943	2,085,193	2,085,193	2,085,193
Mean	6,681,583	6,212,890	6,327,726	6,812,120	5,285,511	5,830,219	4,127,038
Std. Dev.	2,400,065	2,547,724	2,373,558	2,358,793	1,529,597	1,904,428	1,264,394

Table D.5 Intermediate Computational Results for Obtaining these Probability of Pool Failure (St. Louis District)

	μ_{SM_i}	σ_{SM_i}	$Pr(SM_i < 0)$	$\mu_{SM_{-i}}$	$\sigma_{SM_{-i}}$	$Pr(SM_{-i} < 0)$	p_i
Pool 24	6,994	72,450	0.4602	236,104	2,431,496	0.4602	0.2118
Pool 25	23,556	219,554	0.4562	219,542	2,579,305	0.4641	0.2117
Pool 26	19,498	588,745	0.4880	223,600	2,406,913	0.4641	0.2265
Pool 27	2,381	41,550	0.4761	240,716	2,390,180	0.4602	0.2191
St. Louis	56,326	912,807	0.4761	186,772	1,587,917	0.4522	0.2153
Chester	37,078	512,852	0.4721	206,019	1,948,582	0.4562	0.2154
Thebes	97,263	1,229,668	0.4681	145,835	1,343,638	0.4562	0.2135
System	243,097	2,403,732	0.4602				

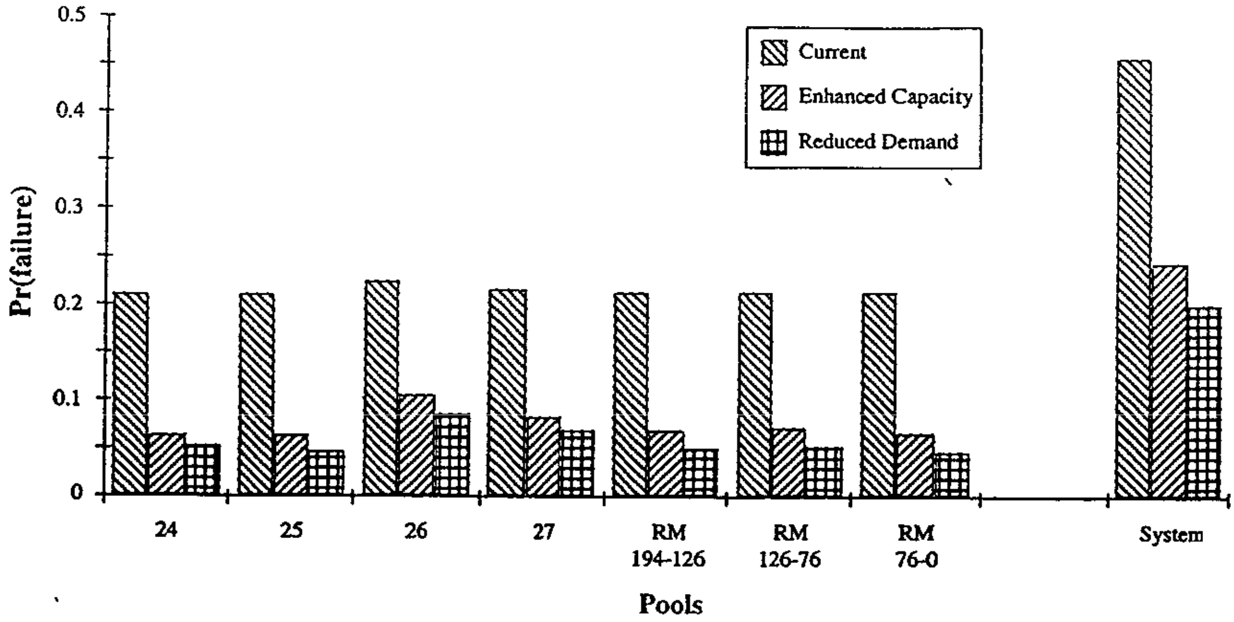

Figure 12.3 Comparison of Scenario Reliabilities for St. Louis District 1973-1993

References

Ang, Alfredo H-S., and Wilson H. Tang, *Probability Concepts in Engineering Planning and Design, Volume II: Decision, Risk, and Reliability*, John Wiley & Sons, New York, 1984.

Apogee Research, Inc. and Dames & Moore, *Risk-Based Analysis of Maintenance Dredging*, DRAFT Research Report, Risk Analysis Research Program, Institute for Water Resources, U.S. Army Engineer Water Resources Support Center, Fort Belvoir, Virginia, May, 1993.

Benjamin, Jack R., and C. Allin Cornell, *Probability, Statistics, and Decision for Civil Engineers*, McGraw Hill Book Company, New York, 1970.

Center for Risk Management of Engineering Systems, *Channel Reliability of the Navigation System in the Upper Mississippi River*, Phase I Final Report, Center for Risk Management of Engineering Systems, University of Virginia, January 31, 1995.

Chang, H., *Fluvial Processes in River Engineering*, Wiley, New York, 1988.

Fan, Shou-shan, Editor, *Twelve Selected Computer Stream Sedimentation Models Developed in the United States*, Federal Energy Regulatory Commission, December 31, 1988.

Graf, W. H., *Hydraulics of Sediment Transport*, Water Resource Publications, Littleton, Colorado, 1984.

Harr, Milton E., *Reliability-Based Design in Civil Engineering*, McGraw Hill Book Company, New York, 1987.

Nanda, S. K., and D. Baker, "Experience in Channel Maintenance in Upper Mississippi River," 1983.

Shannon and Wilson, Inc. and Thomas F. Wolff, *Probability Models for Geotechnical Aspects of Navigation Structures*, A report to the St. Louis District, U.S. Army Corps of Engineers, DRAFT, January 21, 1994.

Tweet, R. D., *History of Transportation on the Upper Mississippi and Illinois Rivers*, Navigation History NWS-83-6, National Waterways Study, Institute of Water Resources, U.S. Army Engineer Water Resources Support Center, Fort Belvoir, Virginia, January, 1983.

U.S. Army Corps of Engineers, *HEC-6: Scour and Deposition in Rivers and Reservoirs*, User's Manual, Hydrologic Engineering Center, Davis, CA, August 1993.

U.S. Army Corps of Engineers, *Reliability Assessment of Navigation Structures*, Engineer Technical Letter, No. 1110-2-532, Washington, DC 20314-1000, 1992.