EC 1110-2-6062
1 February 2011

US Army Corps
of Engineers®

ENGINEERING AND DESIGN

Risk and Reliability Engineering for Major Rehabilitation Studies

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EC 1110-2-6062

CECW-CE

Circular
No. 1110-2-6062

1 February 2011

EXPIRES 31 JANUARY 2013
Engineering and Design
RISK AND RELIABILITY ENGINEERING FOR MAJOR REHABILITATION STUDIES

1. Purpose. This Engineer Circular (EC) presents comprehensive guidance for engineering risk and reliability for Major Rehabilitation studies. This EC includes the methods for developing engineering reliability applications. It covers applications for multiple engineering disciplines. Although there is discussion of economic consequences from unreliable performance, the focus of this EC is on predicting engineering performance, not on the economics of investment decisions. A fuller treatment of risk assessment to inform the major rehabilitation investment decisions will be developed while this EC is used as interim guidance.

2. Applicability. This circular is applicable to all USACE commands having responsibility for the major rehabilitation studies.

3. Distribution Statement. Approved for public release; distribution is unlimited.

4. References. References are at Appendix A.

5. Discussion. The use of probabilistic analytical methods, including the development of hazard functions, is a relatively new concept within USACE. In the last 15 years, the use of probabilistic and risk-based methods has become an acceptable and required analysis technique for USACE studies. Most of the historical use of engineering reliability analysis within USACE has included the development and utilization of hazard functions for major rehabilitation studies, systems studies, and evaluation of the need for new navigation projects when the existing structure is in a deteriorated condition.

FOR THE COMMANDER:

JAME C. DALTON, P.E., SES
Chief, Engineering and Construction Division
Directorate of Civil Works

4 Appendices
Appendix A – References
Appendix B – Navigation Reliability
Appendix C – Flood Control Reliability
Appendix D – Hydropower Reliability

DEPARTMENT OF THE ARMY EC 1110-2-6062
U.S. Army Corps of Engineers
CECW-CE Washington, DC 20314-1000

Circular
No. 1110-2-6062 1 February 2011

EXPIRES 31 JANUARY 2013
Engineering and Design
RISK AND RELIABILITY ENGINEERING
FOR MAJOR REHABILITATION STUDIES

TABLE OF CONTENTS

CHAPTER 1
Introduction

1-1. Purpose. This Engineer Circular (EC) develops comprehensive, single-source guidance for reliability engineering for major rehabilitation studies related to existing USACE civil works infrastructure. This EC has material that is suitable to be developed as an Engineer Manual or an Engineer Pamphlet since it discusses the historical background that may aid new engineers in determining the best methods they can use to perform reliability analysis as part of a risk assessment. This EC focuses on the methods for developing engineering reliability applications. It covers applications for multiple engineering disciplines including structural, geotechnical, mechanical, electrical, coastal, and hydraulic engineering. Although there is discussion of economic consequences from unreliable performance, the focus of this EC is on predicting engineering performance, not on the economics of investment decisions. A fuller treatment of risk assessment to inform the major rehabilitation investment decisions will be developed while this EC is used as interim guidance.

1-2. Applicability. The use of probabilistic analytical methods, including the development of hazard functions, is a relatively new concept within the USACE. In the last 15 years, the use of probabilistic methods has become an acceptable and required analysis technique for USACE studies. Most of the historical use of engineering reliability analysis within USACE has included the development and utilization of the hazard functions for major rehabilitation studies, systems studies, and evaluation of the need for new navigation projects when the existing structure is in a deteriorated condition. Its use should be expanded to assist in making wise investment decisions associated with capital improvements when project deterioration, either in its current condition or as part of the future condition in the study period, plays a key role in the economic analysis. The same general process is applicable for all civil works infrastructure. This EC provides information for all engineering disciplines to use in evaluating engineering reliability of all types of existing USACE civil works infrastructure projects including navigation, flood control, hydropower, and coastal/harbor structures. The content of this EC is consistent with performance-based metrics in making investment decisions in a risk framework.

1-3. Distribution Statement.

This circulation is approved for public release: distribution is unlimited.

1-4. Definitions. A glossary follows Appendix D.

1-5. Past History and Limitations.

a. Risk and reliability methods have been a requirement in major rehabilitation guidance since 1991. There are several reasons that these methods have become more prevalent within USACE. First, the computational abilities of the modern desktop personal computer and availability of commercial Monte Carlo simulation software packages that link with traditional spreadsheet programs have made the use of these probabilistic methods easier from a computational and data collection standpoint.

b. Second, USACE navigation studies such as the Upper Mississippi River-Illinois Waterway (UMRIWW) and the Ohio River Mainstem Systems Study (ORMSS) have increased the knowledge of risk and reliability in the USACE through the development of engineering reliability models for use on all types of civil works infrastructure. The UMRIWW study was the first major systemwide study that utilized both engineering reliability with economic modeling to determine the future investment decisions for 37 locks and dams on the Mississippi River and Illinois Waterway. The UMRIWW was responsible for the development of state-of-the-art reliability models for hydraulic steel structures, concrete deterioration caused by abrasion, and freeze-thaw and mechanical/electrical reliability.

c. The ORMSS was a 10-year study evaluating the long-term performance (for the years 2010 through 2060) of 19 locks on the main stem of the Ohio River. It includes engineering reliability analysis for major components of the system (lock gates, culvert valves, operating machinery, etc,) and is integrated within a comprehensive economic analysis evaluating maintenance, major rehabilitations, and large-scale improvements. Many outputs are associated with the ORMSS, but one of the key outputs is a series of engineering reliability and economic models that required upfront research and development costs which would provide the critical tools to develop a long-term investment strategy in an adaptive management structure. A smaller scale study, such as a major rehabilitation study, could not have incurred these development costs that were required. Also as part of ORMSS, a series of economic models have been developed specifically for handling probabilistic engineering reliability input such as component-specific hazard functions and consequence event trees and using them in developing optimized investment decisions for capital improvements.

d. Finally, performance-based metrics have become the long-term goal of making investment decisions when developing USACE civil works budgets. This is particularly true for an aging infrastructure coupled with budget restrictions. The guidance provided in this EC fits perfectly into the performance-based metric framework in terms of making decisions that fully incorporate risk and reliability. The existing engineering reliability and economic models from ORMSS can periodically be updated with required information to adaptively manage that system in the future.

1-6. Past Guidance Documents. USACE has been developing reliability guidance over the past 20 years. The most current guidance document that uses risk and engineering reliability is the Major Rehabilitation guidance, EP 1130-2-500. This publication has been used by numerous Districts and Division to analyze existing infrastructure for risk and reliability of major rehabilitation purposes. Many USACE publications on risk and engineering reliability have been rescinded recently due to changes in the reliability methods used in some of those documents

1-7. Current Usage within USACE. As noted previously, probabilistic methods have been part of guidance within the USACE for several years, but have just started to be used successfully in the last 15 years. Major rehabilitation studies such as Hodges Village Dam (CENAE- dam seepage/piping and stability), Brazos River Floodgates (CESWG - steel structures) and Mississippi River Lock 15 (CEMVR - concrete lock structure) were among the first reliability studies to be undertaken by Districts. Time-dependent hazard functions were first successfully used in a USACE study as part of the Major Rehabilitation Report for the Cape Cod Canal Railroad Bridge completed by the New England District in 1996. The UMRIWW study advanced the ORMSS continued the development with additional research and further application of hazard rates for navigation structures in 1997 (Patev et al. 1997). These initial hazard functions were developed to get time-dependent probabilities of unsatisfactory performance for vertically framed miter gates. Since 1997, the analysis techniques have been expanded to develop time-dependent hazard functions for other components investigated under the ORMSS, as well as other navigation studies. The Markland Locks and Dam Major Rehabilitation Study (CELRL) built upon the reliability modeling efforts developed under ORMSS to incorporate hazard function analysis within a major rehabilitation study. The probabilistic and risk-based modeling methods completed as part of the Chickamauga Lock Replacement Project (CELRN – alkali aggregate reaction (AAR)) represented the first time hazard functions were developed for mass concrete deterioration within USACE. This is an example of a project for which the deteriorated condition of the existing project required a reliability analysis to estimate the remaining service life of the structure from an economic standpoint. This played a key role in justifying project improvements, which in the case of the Chickamauga study was a new lock chamber. Other more recent USACE studies have been completed using the tools and procedures developed as a part of ORMSS and Markland as a guide. This EC captures the methodology and procedures utilized in these studies and generalize them so they can be used as guidance when conducting reliability analyses for existing USACE civil works infrastructure.

1-8. Methods for Conducting Reliability Analysis: Preferences and Directives.

a. Currently, four approved and recognized methods can used to estimate the reliability of existing infrastructure for rehabilitation purposes:

(1) Historical frequency of occurrence

(2) Expert-Opinion Elicitation

(3) Time-independent methods

(a) First-order second-moment (Taylor series finite difference)

(b) Point estimate method

(c) Advanced second moment

(d) Monte Carlo simulation

(4) Time-dependent methods: Hazard functions (using Monte Carlo simulation)

b. These methods and their limitations are discussed in further detail in Chapter 3. The current directive for the utilization of these reliability methods is dependent upon the degree of non-linear behavior of the limit state and the need for a time-dependent reliability model. For analyzing non-time dependent reliability use the Monte Carlo Simulation or Advanced Second Moment methods. Only use hazard functions for time-dependent models. The use of either FOSM or ASM for time dependent reliability calculations is not recommended since these methods will produce reliability results only for a single instance in time.

1-9. Risk Assessment. Risk assessment is a systematic, evidence-based approach for describing the likelihood and consequences of any action, including no action. Risk assessment methodology is a technical and a scientific process by which the risks of a given situation for a system are modeled and quantified. A risk assessment asks the questions: What can go wrong? How can it happen? What is the likelihood it will go wrong? What is the consequence if it goes wrong? Risk assessment is a small part of the bigger process called risk management. Risk assessment provides a piece of the required data for the decision makers in the risk management process. Risk management includes the assessment, decision making, and communication aspects of risk. The domain of risk assessment is beyond engineering reliability, while reliability assessment is a significant activity in a risk assessment.

1-10. Utilization as an Investment Tool and Prioritization. Hazard functions are developed to estimate the remaining service life of the feature. The main reason that probabilistic modeling and hazard function analysis are utilized is for making investment decisions in a risk-based approach. This is increasingly important now that performance based metrics have become the long-term goal for making budget decisions involving USACE civil works infrastructure. As outlined by major rehabilitation guidance in EP 1130-2-500, an array of alternatives need to be evaluated to determine the plan or alternative that has the most economic merit. These alternatives have varying implications with respect to reliability analyses; therefore, the probabilistic modeling techniques allow a means to compare expected future performance of alternatives. Reliability analyses also provide critical information regarding the reliability of a particular feature given the limit state that is selected for unsatisfactory performance criteria. For example, the baseline condition is considered to be the case with which all other alternatives are compared. This typically involves a future condition with lower reliability that keeps the project serviceable. The next alternative typically improves the reliability of the project, although this could involve an interruption of service to construct the improvements. Other incremental improvements in reliability could reduce the chance of future unsatisfactory performance and determine the optimal time to make future investments that maximize the service life of the existing project. These alternatives are measured against one another through a comprehensive economic analysis accounting for the future reliability, investment costs, and impacts on the users of the project.

1-11. Risk Assessment for USACE Dam Safety Program.

a. USACE has an inventory of over 600 dams. These dams serve a wide variety of purposes such as flood control, navigation, water supply, hydropower, and recreation. Many projects serve multiple purposes. These purposes can play a critical role in the economic benefits of maintaining safe, continued operation of the projects. It is always important to make wise investment decisions when considering a portfolio of dams across the country. It becomes even more critical as infrastructure ages and budgets become tighter. Many USACE dams are quite old and represent an overall aging infrastructure across the inventory. In order to make wiser decisions with allotted budgets, USACE has initiated a multiple-phased risk assessment approach for their portfolio of dams. This is being handled through the USACE Dam Safety Program. This is an important step for USACE since individual projects with perceived deficiencies were previously evaluated separately by the District that owned, operated, and maintained the dam. Since USACE has 41 individual Districts and 8 Divisions, there was no consistent means of evaluating investment needs across the entire portfolio to determine the greatest need from a risk standpoint.

b. In order to improve the process of making risk-informed decisions across the entire spectrum of USACE dams, the Screening Portfolio Risk Analysis (SPRA) for the USACE Dam Safety Program was initiated during the summer of 2005. This effort represents the first level of a multiple-phased effort to bring full scale risk assessment to the decision-making process associated with dam safety by linking engineering reliability with economic and life loss impacts on a relative scale. The SPRA effort involved the development of a tool for evaluating the relative life and economic risk of dam failures for a variety of deficiencies across the inventory of USACE dams. The analysis tool uses relative engineering failure probabilities integrated with both potential life loss and economic damage estimates to determine the overall relative risk associated with a project. For more detail and discussion, reference is made to ER 1110-2-1156, DAM SAFETY- ORGANIZATION, RESPONSIBILITIES, AND ACTIVITIES.

CHAPTER 2
Engineering Reliability Considerations

2-1. Background. This chapter provides basic procedures and considerations when conducting engineering reliability analyses for existing USACE civil works infrastructure projects. This chapter includes selection of components for modeling, use of proper limit states, appropriate load cases, discussion on safety factors, time dependency of various components, calibration with field conditions, and finally, coordination with consequences analysis required within a risk assessment. Both this chapter and Appendices B, C and D of this EC provide detailed examples regarding many of the basic engineering reliability procedures and considerations for successful implementation of reliability procedures.

2-2. Selection of Critical Components for Reliability Modeling.

a. In general, numerous components are critical to the safe, successful operation of a USACE civil works facility. When reliability analyses are conducted for the evaluation of an existing USACE civil works project, it is important to determine which components are the most critical within the context of the evaluation being carried out. Typically, these will be components that are either very costly to replace and/or would potentially limit the project's intended service for a significant amount of time. For example, a multipurpose flood control dam requires the reliable operation of several components in order for the project to successfully carry out its intended mission. These components vary by dam type; for an embankment dam such components as embankment stability, foundation seepage/piping control, dam gates, operating equipment, and spillway structural capacity must perform satisfactorily over the range of loadings being considered for safe, reliable performance. Because major infrastructure components vary by project, determining which components are the most critical to the operation of a facility is site-specific. It is important to focus on the critical components when determining which ones will need to be analyzed with reliability techniques. Careful consideration must be given to both the current reliability as well as the potential for degradation during the economic analysis period of 50 years.

b. The project design team (PDT) must come up with a measurable way to determine the most critical infrastructure for the project being evaluated. As directed by major rehabilitation guidance, this includes all components that are critical, not just those with a perceived reliability problem. One relatively simple and definable way to determine the most critical infrastructure components is through some type of ranking analysis. The ranking analysis should be developed by a multidisciplinary team of engineers and operations specialists familiar with the project features, repair history, and overall operation of the facility. It is very important to document the methodology used to determine which components were selected for reliability modeling when multiple components are required for the operation of the project. The ranking analysis could possibly have multiple phases depending upon the number of overall components critical to the operation of the facility or facilities. The PDT must balance the number of components to be evaluated with reliability techniques within the budget and schedule for the overall project. The PDT should include experienced engineers, economists, and plan formulators early in the process. This will not only save considerable time and funding, but will also result in a higher quality product in the end. A good example of a fully developed ranking analysis that was used for the Markland Locks and Dam Major Rehabilitation Report is provided in Appendix B.

2-3. Types of Components Evaluated with Reliability Techniques. Engineering reliability models can be separated into basically two general categories: time-independent and time-dependent models. The time-independent models are assumed not to deteriorate significantly over time, whereas the time-dependent models degrade in reliability over time. These categories consider only the state of the feature being analyzed through time. Return period and other types of load cases such as seismic and barge impacts that are characterized by return periods are handled separately.

a. Time-independent reliability models. Time-independent components have the same reliability throughout the study period since their performance does not degrade through time. These components could have different probabilities of unsatisfactory performance values for varying load cases, i.e., seismic and flood, but within the same load case, the probabilities will not change as a function of time. The relative likelihood associated with the return period of the load case is handled through the frequency of the load case occurring within the study period. This is discussed in more detail in this chapter. For time-independent components, the probability of unsatisfactory performance is the same each year in the study period for a single load case. However, stability of gravity structures and embankment slopes are both dependent on uplift and seepage relief systems that will degrade over time unless they are properly maintained and rehabilitated. Gravity structures such as the Chickamauga lock walls are also subject to mass concrete deterioration as pointed out in Paragraph 1-7.

b. Time-dependent reliability models. Time-dependent components degrade in reliability with time because of their continued use and increasing service wear. Hydraulic steel structures are examples of time-dependent components because they are subject to fatigue and corrosion, which cause a decrease in reliability over time. Mechanical and electrical equipment are also time-dependent because these components become less reliable through time with increasing cumulative cycles and age. Another major issue for mechanical and electrical equipment is the availability of replacement parts as systems get outdated. Many times seepage and piping through soils can be considered a time-dependent problem because the loss of material over time gets worse until a pipe occurs. Careful consideration must be given to seepage and piping as it relates to the load case and evidence of piping, boils, and other hazards over time. Reliability analyses for time-dependent components have hazard functions developed for them. A hazard function is defined as the probability of unsatisfactory performance in a given year assuming it has survived up to that year, a truly time-dependent analysis.

2-4. Overall Risk Assessment Considerations.

a. An overall risk assessment must include several parameters in order to determine risk associated with the performance of a project and/or individual components. For an overall project, all components that are deemed critical enough to warrant reliability analyses are properly assembled to determine the overall risk associated with the project. Project components can be evaluated individually as well, although careful attention must be paid to the interdependency of components when determining risk associated with an individual component. A basic risk assessment includes the following parameters:

Risk = (Event Probability) (Conditional Probability of Failure) (Conditional Breach Probability) (Exposure) (Loss rate) (2-1)

or

$$R = H\,Pf\,Pb\,X\,L \tag{2-2}$$

where

- R = annual relative risk estimate (expected losses per year)
- H = hazard in the form of an initiating event rate, such as a flood or an earthquake (events per year)
- Pf = conditional probability of failure for a feature given the initiating event (conditional failure probability per event)
- Pb = conditional dam breach probability given the initiating event and feature failure (conditional breach probability per feature failure)
- X = conditional exposure of people and property caused by the initiating event, feature failure (or no failure), and breaching (or no breaching) of the dam (population or property at risk per breach)
- L = downstream loss rate for the exposed population and property (lives or dollars lost per population or property at risk).

The product of $X\,L$ is called the consequences associated with people and property caused by breaching of the dam and uncontrolled flooding (lives or dollars lost per breach).

b. Each of the following factors plays an integral part of the overall risk assessment:

(1) Frequency of load cases being evaluated

(2) Conditional probability of unsatisfactory performance

(3) Conditional consequences

c. Each portion is described in more detail in the following subparagraphs.

(1) Frequency of load cases being evaluated.

(a) This is typically the entry point into the overall risk assessment whether an individual component or an entire project is being evaluated. The frequency of the load cases being evaluated is a function of the overall type of analysis being carried out. This input needs to be structured to fit within the economic analysis. For example, most USACE studies require a study period extending 50 years into the future in order to evaluate a life cycle cost of both the existing project and any alternatives being considered. For the load cases being evaluated for the reliability analysis, the frequency of each particular load case occurring within the study period needs to be determined. This will vary by type of study and component being evaluated. Major rehabilitation studies generally do not include extreme events such as seismic and PMF flood load cases. However, these load cases are a critical piece when considering dam safety modifications for USACE flood control projects.

(b) The occurrence or nonoccurrence of unusual and extreme load cases like floods and earthquakes are commonly modeled as a Bernoulli sequence. The sequence consists of a series of discrete trials (usually annual) with two possible outcomes for each trial (occurrence or nonoccurrence of the event). The probability of occurrence (P) of the event for each trial must be constant and the trials must be statistically independent. If the probability of occurrence for each trial is p, the binomial distribution can be applied to compute the probability of x events occurring among n trials.

$$P = \binom{n}{x} p^x (1-p)^{n-x} \qquad \text{where} \quad \binom{n}{x} = \frac{n!}{x!(n-x)!} \tag{2-3}$$

(c) The concept of exceedence probability or frequency evolves from the application of the Bernoulli sequence model to natural events. For continuous random variables like flood discharge or ground acceleration, the discrete occurrence or nonoccurrence of an event is based on a particular value being exceeded or not exceeded during a trial. The probability that a particular value will be exceeded in any given trial can be defined as p. For a person buying a home in the 100-year floodplain (p = 0.01), the probability of experiencing flooding during a 30-year mortgage is calculated using the binomial distribution:

$$P = \binom{30}{x} 0.01^{30} (1-0.01)^{30-x} \qquad \text{for} \quad x = 1, 2, 3, \ldots, 30 \tag{2-4}$$

(d) Knowing the rules of probability, which require the sum of probabilities for all possible outcomes to equal one, the calculation can be greatly simplified:

$$P(\text{one or more events}) + P(\text{no event}) = 1.0$$
$$P(\text{one or more events}) = 1.0 - P(\text{no event})$$

The expression on the right side of the equation can be computed using the binomial distribution as

$$P(\text{one or more events}) = 1.0 - \binom{n}{0} p^0 (1-p)^{n-0}$$
$$= 1.0 - (1-p)^n$$

(e) Using the values from the flood example

$$P(one\,or\,more\,floods) = 1.0 - \binom{30}{0} 0.01^0 (1-0.01)^{n-0}$$
$$= 1.0 - (1-0.01)^{30}$$
$$= 0.26$$

These equations show that a person buying a home in the 100-year floodplain has a 26 percent chance of experiencing flooding during a 30-year mortgage period. This provides a good overall assessment of the flooding hazard. Caution must be exercised, however, when applying this method. Although the overall hazard of experiencing at least one flood is known, the number of times and sequence of when flooding might occur over the period are unknown. A more detailed analysis using Monte Carlo simulation would be appropriate if the number of events and temporal sequence of the events over the life of a project are important.

(f) A true risk assessment of the flooding or seismic risk requires knowledge of the relative likelihood (event rate) for these types of loads. The geometric distribution provides the basis for determining event rates. The probability of an event occurring for the first time during trial t must first be computed using the geometric distribution. Add in the following equation:

$$P = p(1-p)^{t-1} \tag{2-5}$$

(g) The average time between occurrences of the event (return period) is

$$\begin{aligned}\overline{T} &= \sum_{t=1}^{\infty} tp(1-p)^{t-1} \\ &= p[1 + 2(1-p) + 3(1-p)^2 + \ldots] \qquad (2\text{-}6) \\ &= \frac{1}{p}\end{aligned}$$

An event with an annual exceedence probability p will be equaled or exceeded at an average rate of once every $1/p$ years. Several events could occur in a few years or the project could go many years without seeing the event. The long-term average rate will be once every $1/p$ or T years. This concept is useful to estimate average annual risk or perform a Monte Carlo simulation over the life of a project.

(h) As an example of the geometric distribution, the return period for the Operating Basis Earthquake (OBE) (145 years) is defined in ER 1110-2-1806 Earthquake Design and Evaluation of Civil Works Projects, the earthquake that can reasonably be expected (50 percent chance) to be equaled or exceeded over the service life (100 years) of a project.

$$P = 1 - (1 - p)^n$$
$$0.5 = 1 - (1 - p)^{100}$$
$$p = 1/145$$
$$T = 145 years$$

(i) Once event rates are determined, they can be entered into the event tree for risk analysis. In order to estimate the risk over the full range of loading conditions, multiple event intervals must be applied. The estimate of risk is obtained from the numerical integration of the relationship between initiating event rate and expected value of the consequences. A conceptual model of this risk estimation procedure is provided in Figure 2-1. Numerical event rates for this example are provided in Table 2-1. Selection of the analysis points can vary and will depend on the type of analysis being performed. The accompanying example uses the geometric mean of the upper and lower range limits. The summation of rates over all events is equal to one, which satisfies the rules of probability. The summation is independent of the number of events being evaluated. As the number of events in the analysis increases, the accuracy of the risk estimate (area under the curve) should also improve. The key is to balance the desired level of accuracy with the time and cost required to perform the analysis.

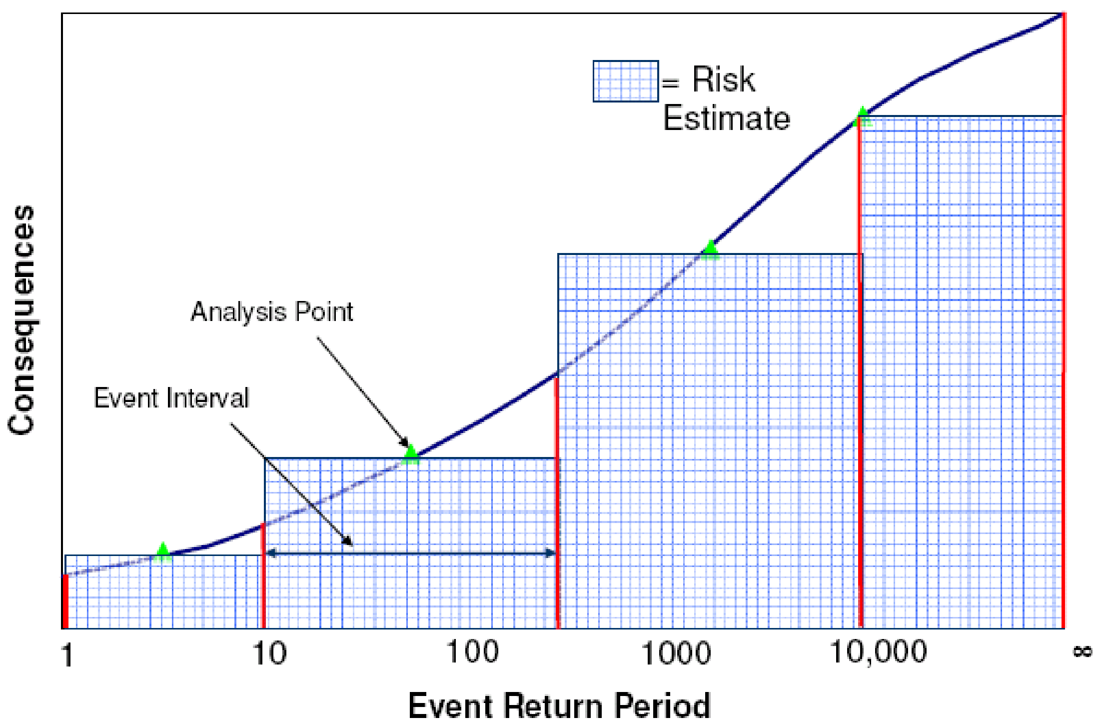


Figure 2-1. Example generic graph for consequences versus event return

Table 2-1. Values for Example Graph

Event Range, years	Analysis Point, years	Event Rate Calculation	Rate Events/Year
1 to 10	3.2	1/1 – 1/10	0.90000
10 to 750	54.8	1/750 – 1/10	0.09667
750 to 10,000	1,732.1	1/10,000 – 1/750	0.00323
> 10,000	10,000	1/10,000	0.00010

(j) Selection of the appropriate pool condition to combine with other load conditions requires careful consideration. Event combination analysis provides a means for considering the probability of multiple initiating events occurring concurrently. Both the rate of occurrence and duration of the events are considered. Not only do both events have to occur, they have to occur at the same time. The probability of unusual and extreme loads such as an earthquake occurring simultaneously with a flood is usually negligible and not necessary to evaluate in a risk analysis. Reference is made to Ellingwood (1995) for more details on load combinations for major rehab studies.

For many load conditions, an appropriate normal pool condition needs to be determined for the analysis. A pool elevation equal to the 50 percent annual duration exceedence would be appropriate for pools with relatively small fluctuations (e.g. upper pool at a gated navigation structure). When the pool elevation varies considerably (e.g. lower pool at a navigation structure), a range of normal pools should be considered to improve the accuracy of the analysis. An example of this application is presented in Table 2-2 and the associated graph illustrated in Figure 2-2. A potential branch of the event tree for analysis of an OBE event at this project follows the tabular values and is shown in Figure 2-3.

Table 2-2. Example Pool Elevation Exceedence Values

Lower Pool Range	Duration Range	Pool Level for Analysis	Duration for Analysis
> 731.8	0 – 20%	733.7	20%
729.7 – 731.8	20% - 40%	730.5	20%
728.6 - 729.7	40% - 60%	729.2	20%
727.7 – 728.6	60% - 80%	728.1	20%
< 727.7	80% - 100%	727.3	20%

(2) Conditional probability of unsatisfactory performance. The conditional probability of unsatisfactory performance is developed for individual components through engineering reliability analysis. It is computed for each load case being evaluated as part of the overall risk assessment. Actual methods of developing probabilities of unsatisfactory performance are covered in more detail in Appendices B, C and D, but this section focuses on considerations relative to selecting appropriate limit states consistent with both the load cases and measured consequences.

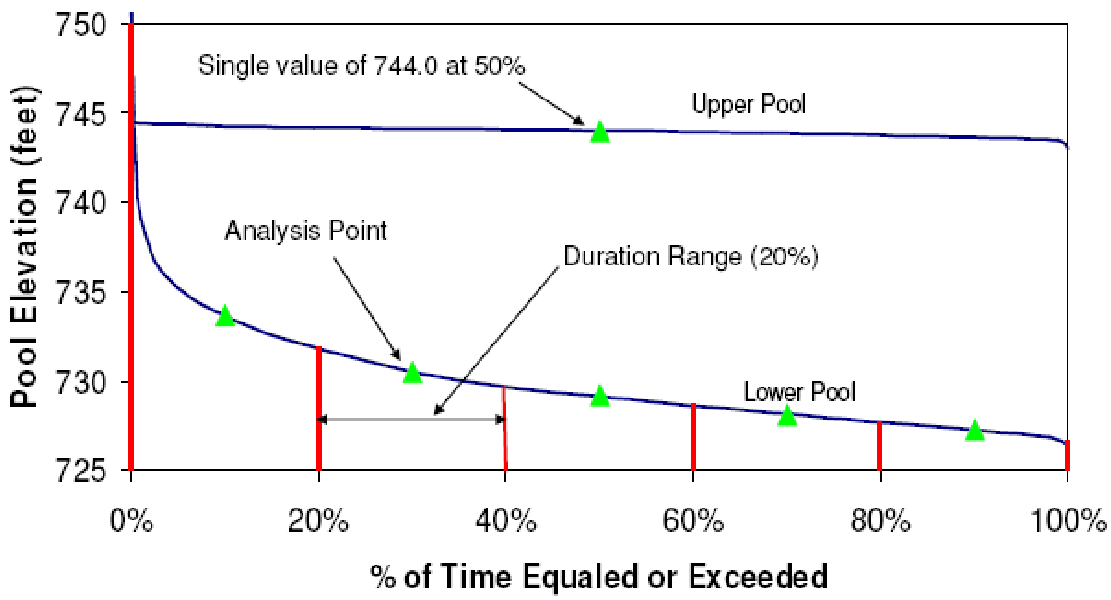


Figure 2-2. Example pool exceedance curve for navigation project

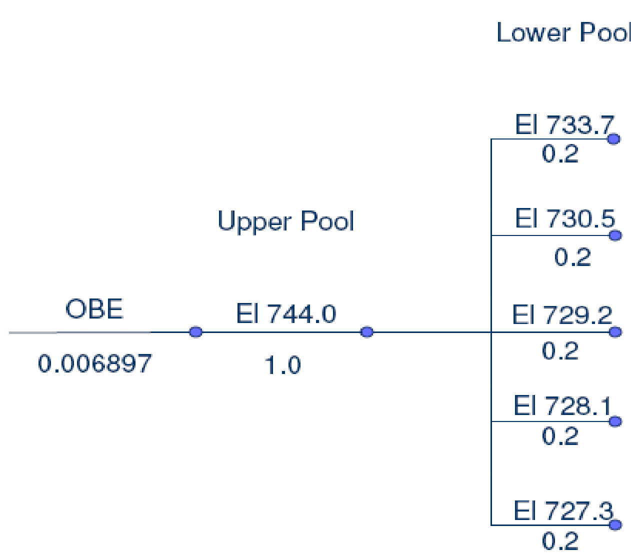


Figure 2-3. Example event tree branch

(3) Conditional consequences. Consequences can be measured through several metrics. Traditionally, these have included economic damages and loss of life. It is understood that there are additional consequences, but those are not captured in most USACE risk assessments. These could include such factors as impacts to regional economies, environmental issues, and other outlying metrics. The consequences that are included as part of the risk assessment must be consistent with the load cases being evaluated and limit state being analyzed as a part of the reliability analysis. Economists are responsible for developing this information but must work closely with the engineers developing the load case frequency values and engineering reliability analyses. Engineering and economic integration is covered in detail in Chapter 5.

2-5. Determining Appropriate Limit States for Reliability Modeling.

a. One of the key steps when developing reliability models is selecting an appropriate limit state for the component being evaluated consistent with field performance and anticipated performance for each load case to be evaluated. This will vary by component and essentially is an engineering analysis of "most likely" failure mode for a given component under each of the loads being evaluated in the risk assessment. As noted earlier, this is dependent upon the type of component being analyzed and ties directly to the load cases as well.

b. A good example of selecting appropriate limit states can be gathered from experience in modeling miter gates. Miter gates are critical components used in operating navigation locks. First-generation engineering reliability models originally developed for these structures focused on the structures as horizontally loaded beams as would be done with traditional hand calculations. The original output from the reliability models indicated that no performance problems would be expected during the 50-year study period. From a traditional design standpoint this probably makes sense given original safety factors in design. However, this was not consistent with what the Operations personnel were experiencing in the field. Recent inspections and repairs at several navigation projects indicated significant fatigue damage of the main load bearing girders near high compressive stress areas. This damage was not captured by traditional analysis since it was due to residual stress issues associated with original construction. The ORMSS Engineering Team redirected its analysis toward the actual cause of the fatigue cracking to be more consistent with field performance. This had to be done through finite element modeling that is calibrated with available instrumentation data. When the ORMSS Engineering Team completed its reliability analysis based upon this limit state, the results were much more consistent with the actual field performance.

c. This same general process is applicable to other components such as gravity structures. When appropriate limit states for stability analyses of gravity structures are being determined, the potential amount of movement of the structure and what that might mean to the overall effect on project performance must be considered. For example, a small amount of sliding or rotation may reduce at-rest earth pressure driving forces to active driving pressures, which could possibly stop movement. This small amount of movement required to lessen driving forces may not adversely affect the overall performance of the project, or it could adversely affect the project's performance. It needs to be evaluated on a case-by-case basis. The important point here is that a traditional stability analysis with a safety factor less than 1.0 does not necessarily represent the limit state that should be evaluated for every gravity structure. Each analysis needs to consider the function of the component and how much damage would be required to adversely affect the overall performance of the project.

2-6. Component Redundancy Considerations for Reliability Analysis.

a. Further consideration must be given to the individual component's redundancy in terms of performance and its effect on potential consequences. There are two types of redundancy that warrant discussion in this section: the redundancy of an individual component for the selected limit state and the redundancy of having multiple components on a project that can perform the same function, such as multiple dam gates to control water levels. Concentrating on the first type of redundancy, many infrastructure components are designed to be redundant meaning that if one part of the overall component fails then other sections can carry the load safely. This is an additional safety feature that is provided in many components, particularly hydraulic steel structures. For example, the limit state of a hydraulic steel structure, such as a miter gate, might involve propagation of a crack in the structural material to a specified length and not necessarily the initiation of the crack itself. That is because many structures have multiple levels of redundancy. The redundancy must be taken into account when determining the limit state for the reliability model. This was done for the miter gate analysis on the Ohio River. As noted earlier, actual field performance required the reliability analysis of the gates to be evaluated for fatigue of the gates caused by a concentration of residual stresses as opposed to main girder bending. Since many miter gates in service have existing fatigue cracks, just the initiation of the fatigue crack is not significant enough for the limit state in a risk analysis. The reliability analysis has to account for propagation of the crack growth to the point at which it threatens the overall structural integrity of the gates. This is where the redundancy of the structure comes into play. An engineering analysis was required to determine how long the cracks need to grow before they threaten the overall structural integrity. Examples of these types of limit states from actual USACE studies are detailed in Appendices B, C and D.

b. The second type of redundancy is associated with having multiple components on a project that can perform the same function. One excellent example relates to having enough electrical power to operate the facility. Generally, a mainline commercial or local source is used to power the facility. However, power outages and other actions can disrupt this main power source. Therefore, most projects have back-up emergency sources such as emergency generator(s) that are available for use when the main power source is unavailable. This means the main power source and back-up (emergency generator in this example) act in parallel to ensure the project has power to operate. This means that both sources must be out of service before the project loses power to operate. Redundancy issues and examples are covered in more detail in other areas of this document including the technical appendices.

2-7. Partial Safety Factor Considerations for Reliability Analysis.

a. In general, partial safety factors should not be utilized in reliability analysis since they induce conservatism in the probability of failure. Typically, partial safety factors and factored loads are used in design mode but should not be included in limit states used for reliability analyses. This is mainly because consequences associated with unsatisfactory performance are based upon the component actually reaching the limit state criteria where "failure" occurs. If partial safety factors were included in the analysis, this would skew the results to make the probabilities of unsatisfactory performance higher that what they are in reality. Therefore, limit states should be selected based upon actual expected field performance without an overriding emphasis on set design criteria.

b. However, partial factors of safety could possibly play a limited role in the selection of an applicable limit state, but only to the extent that some type of repair or operational restriction takes place to affect the benefits of the projects in an adverse way. For example, a district may have some type of policy in place where if a certain factor of safety is not met for some level of operation, then project restrictions may be enacted. While this may not represent an actual "failure," it would represent an action that could have a major impact on the benefits of operating the project. Thus, a physical failure may not occur, but an "economic failure" could occur in this situation. In this example, the use of the safety factor that triggers this situation would be warranted. Refer to the project examples in Appendices B, C and D to better understand how partial safety factors play such a limited role in reliability analysis.

CHAPTER 3
Basic Features and Methodologies for Reliability Analysis

3-1. Background on Probability and Statistics.

a. As discussed earlier, the probabilistic approach used in the reliability analysis uses engineering parameters that are treated as random variables. Random variables assume a range of values or a probability distribution with set statistical parameters such as a mean and standard deviation. Distributions are usually shown using a probability density function as shown in Figure 3-1 or cumulative distribution function (CDF) as shown in Figure 3-2. Typical distributions used in reliability analysis are normal (Figures 3-1 and 3-2), lognormal, uniform, and triangular. Although the value of a parameter is uncertain or variable, the probability density function quantifies the likelihood that the value of the random variable lies in any given interval. When parameters are defined as random variables, functions of these parameters such as safety factor also become random variables and can then be expressed in probabilistic terms for use in the reliability analysis.

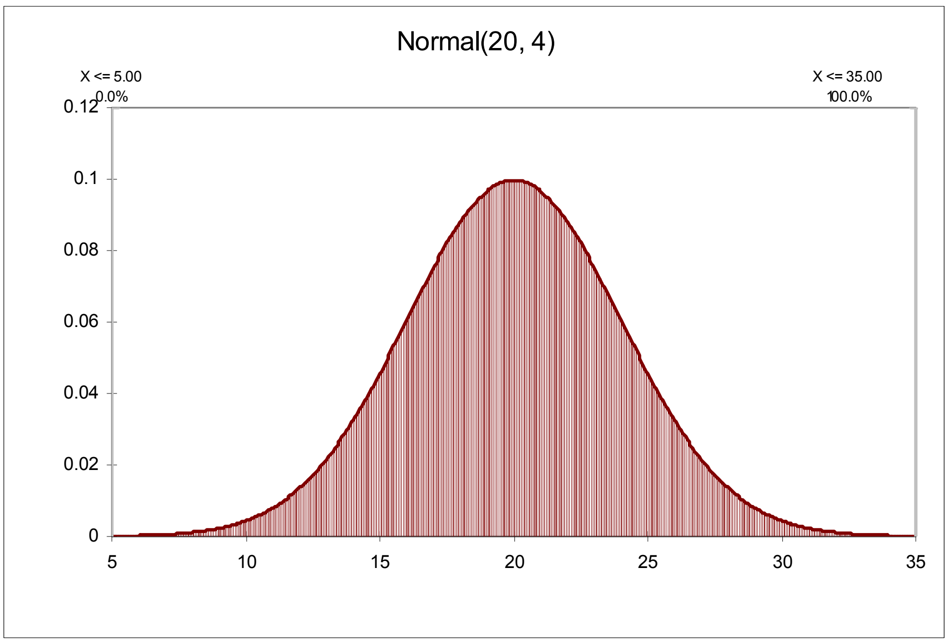


Figure 3-1. Example of a probability density function

b. Some structural examples of random variables are yield stress of steel, compressive strength of concrete, and corrosion rates of steel. Some geotechnical examples of random variables are angle of internal friction, cohesion, and the unit weight of soil. The correlation of random variables is also important in a consideration of random variables. For example, the angle of internal friction and cohesion for soils or rock are strongly correlated.

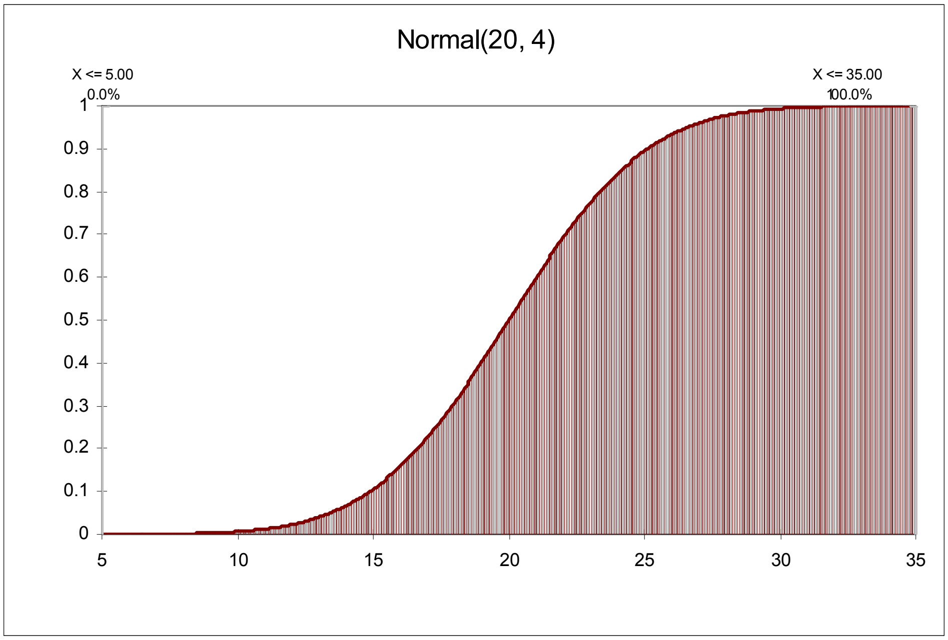


Figure 3-2. Example of a cumulative distribution function

c. Another issue that needs to be addressed with random variables is the upper or lower limits of the data. Typically distributions that are used in reliability models are developed based on a limited number data points. This makes them often hard to fit to those “simple” distributions where a mean and standard deviation can be interpreted and implemented into reliability models. However, during the design of USACE civil works structures minimum requirements are often specified for the materials and construction techniques that are used. For example, the yield strength of steel or the compressive strength of concrete is always specified as a guaranteed minimum strength. Therefore, if a structure was built with minimum yield strength of 36 ksi for the steel, then a normal distribution would not fit correctly if the mean was 44 ksi because the yield strength would obtain values less than 36 ksi.

d. Truncating a distribution is the proper way to handle this type of problem. Figure 3-3 shows a truncated normal distribution for the yield strength of 44 ksi truncated at 36 ksi. Truncation permits the calculated mean and standard deviation to be maintained but does not yield values below either set lower or upper limits. This truncation is easily handled with Monte Carlo simulation but, for other FOSM reliability techniques discussed later it is much harder to implement truncation since most of these use only the second moment to calculate the reliability.

e. Primarily two statistical distribution parameters are required to perform a reliability analysis: the mean, μ, and the variance, σ^2. of a distribution.

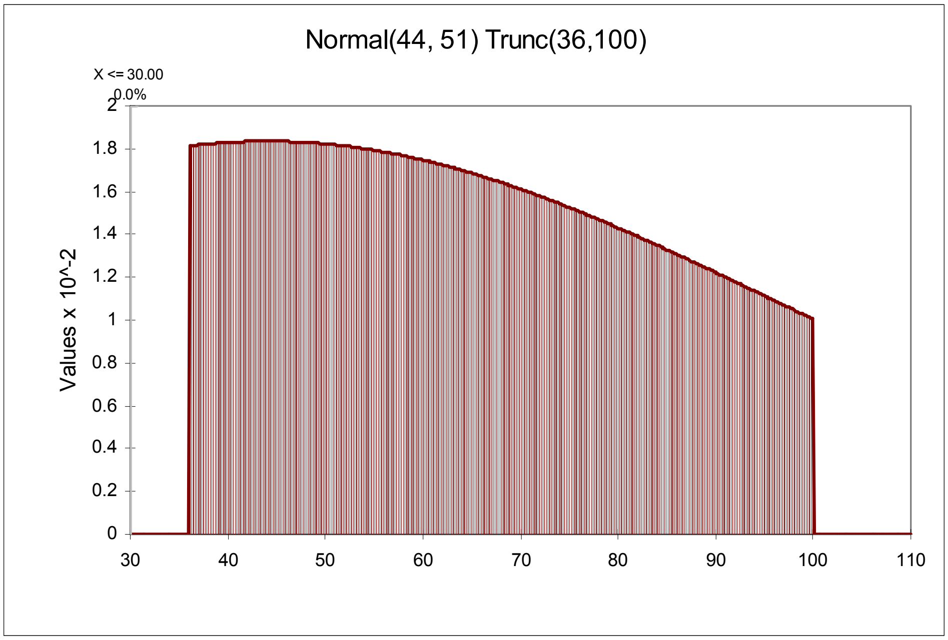


Figure 3-3. Example of truncated normal distribution

(1) The mean is the average of the data and is expressed as

$$\mu = \frac{1}{\mathrm{n}} \sum_{i=1}^{n} x_i \tag{3-1}$$

(2) The standard deviation, σ, is related to the variance (square root) but is more commonly used than variance in reliability analysis. The standard deviation is used since it directly reflects the dispersion of the data. It is expressed as

$$\sigma = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} (x_i - \mu)^2} \tag{3-2}$$

(3) Another term that is frequently used in reliability calculations is the coefficient of variation, ν, a dimensionless relationship of the standard deviation and mean. The coefficient of variation is expressed in percent as:

$$\nu = \frac{\sigma}{\mu} \tag{3-3}$$

(4) A term that is frequently used in expert opinion elicitation is the median. The median is a ranked order value and the 50 percentile value of a set of data. The median is not always equal to the mean value for a set of data.

3-2. Setting Up a Reliability Model – Basic Issues.

a. An engineering reliability analysis determines the probability of unsatisfactory performance P_{up}. This is defined as the probability that the value of a function that characterizes the unsatisfactory performance of the system exceeds some defined limit state. Limit states are typically quantitative in nature such as structural deflections or excessive stresses and do not have to define the total "failure" of a structure or system. Performance-based or serviceability limit states may be used to define the actual unsatisfactory performance of the structure. For example, the limit state for sliding of a monolith could be taken as the event representing excessive lateral movement that causes cracking, spalling, or binding of operating equipment. This limit state defines a state of unsatisfactory performance but does not reflect a state of total collapse or failure of the monolith.

b. The reliability R is the probability that the unsatisfactory performance will not occur. Reliability is the converse of P_{up} as shown in Figure 3-4 and is formally defined by the equation:

$$R = 1 - P_{up} \tag{3-4}$$

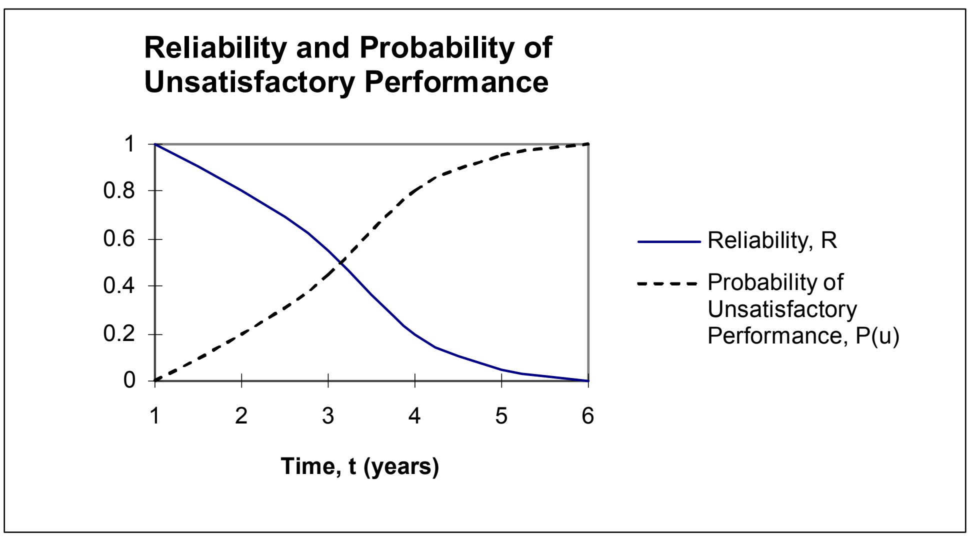


Figure 3-4. Relationships between reliability and probability of unsatisfactory performance

c. Two different methodologies are used to estimate the reliability for engineering components: time-independent reliability and time-dependent reliability. Time-independent reliability assumes that the component is not subjected to degradation or a similar mechanism. For example, such structures as gravity structures, T-walls, and levees would fall into these categories. Time-dependent reliability indicates that a structure is degrading over time. Some examples of these include hydraulic steel structures such as gates or valves, concrete deterioration caused by freeze-thaw, or concrete expansion caused by alkali-aggregate reaction. These concepts are discussed further in this chapter.

d. The primary concept used in reliability analysis is the definition of a limit state for the engineering component. Most limit states used in reliability models focus on a capacity (resistance) versus demand (load) relationship. Input parameters to both capacity and demand can be defined using random variables. This permits the total capacity *C* (resistance) and the total demand *D* (load) to be random variables also. Early reliability models used the quotient of capacity *C* and demand *D,* i.e., factor of safety or $C/D > 1$. This relationship is no longer used in reliability models because of the potential for compounding nonlinear problems. Therefore, the probability of unsatisfactory, performance P_{up} can then be expressed in terms of capacity *C* and demand *D* as:

$$P_{up} = P\,(C - D < 0) \qquad (3\text{-}5)$$

3-3. Random Variables, Constants and Utilization.

a. Random variables are the driving force behind any probabilistic model used in a reliability analysis. Random variables are defined using a distribution (often assumed if data is not available) with a set of statistical parameters, typically the mean and standard deviation, for the given distribution. Examples of distributions typically used in infrastructure reliability modeling are normal, lognormal, triangular, and uniform. More complex distributions such as Weibull or exponential are commonly used more in mechanical and electrical reliability models as shown in Chapter 7.

b. Constants are defined based on known information from either design or field data. Many constants can be determined by having either a small coefficient of variation (less than 2-3 percent) or little overall sensitivity to the reliability model (i.e., the model results change little if this is a random variable). Constants can be easily applied as a single point value in a reliability model.

c. The proper use of random variables is critical to the success of the outputs from the reliability model. Recognizing the limits of random variables where either truncation or the use of a specific distribution is important. Careful selection of distributions and parameters and examination of the sensitivity of the model to these inputs needs to be included for a formal review of a reliability model.

3-4. Selecting Appropriate Methods for Reliability Analyses.

a. Reliability methods are used to determine the safety of airplanes and space craft, electric power generation and distribution and the structural design of buildings as well as the reliability of household components such as light bulbs and appliances. These methods have been established in engineering practice for over 40 years and have evolved to different levels for different engineering disciplines, the basic principles remain the same.

b. This evolving process in the USACE began with the use of the FOSM technique called Taylor series finite difference, which was first used in early guidance for reliability estimations and has progressed to the use of Monte Carlo simulation to perform response surface modeling.

Currently, four approved methods can used to estimate the reliability for major rehabilitation purposes:

(1) Historical frequency of occurrence

(2) Expert-Opinion Elicitation

(3) Time-independent methods

(a) First Order Second Moment (FOSM)

(b) Point Estimate Method

(c) Advanced Second Moment (ASM)

(d) Monte Carlo simulation (MCS)

(4) Time-dependent methods or hazard functions

Each method has its own assumptions and limitations based on the type of problem and limit state that is being considered for major rehabilitation. These assumptions and limitations to each method are discussed in the sections below.

3-5. Historical Frequency of Occurrence.

a. This reliability method uses historical data and information from a project to estimate the probability of unsatisfactory performance of components, units or features. This method should be used only if it meets the following conditions: first, all the historical occurrences being documented should have the same mode or modes of unsatisfactory performance, and second, all the historical occurrences should have similar consequences due to the unsatisfactory performance mode for that component. This method is not really applicable when trying to compare different modes of unsatisfactory performance, different consequences, or different types of engineering systems. Such historical data and information is usually available to estimate the reliability of mechanical or electrical equipment but commonly available for structural or geotechnical components.

b. Survivorship curves are used to estimate reliability of components based on data from actual recorded field or laboratory testing performance. These types of curves do not exist for most USACE infrastructure except for hydropower components such as turbines and motors. Survivorships are generally maintained by commercial manufacturers of components for hydropower.

c. An example of historical frequency would be for a motor that failed because of bearing problems. If five of ten motors have failed because of the same unsatisfactory mode of performance over the past 20 years (10 motors x 20 years/motor = 200 years), then the historical frequency for the motor would be 5/200 or 0.025 failures/year. This would then indicate that 2.5

percent of the motors would see unsatisfactory performance in any given year. However, one of the problems with this method is that it does not truly reflect the time-dependent nature of the motor or the true mean time to failure for the component.

3-6. Expert-Opinion Elicitation.

a. EOE is a heuristic (through verbal discussion) process of estimating the probabilities of unsatisfactory performance using experts' opinions and knowledge of a particular subject matter. This process shall be established using consistent guidelines, and if conducted improperly can invoke substantial bias and subjectivity in the results. Therefore, caution should be exercised when using this process. The EOE process should be considered for use to fill in any gaps when reliability information or models are not readily available.

b. EOE is the synthesis of opinions of a panel of experts on a particular topic where little to no information is available to develop a reliability model or economic consequences for the event tree. Expert elicitation tends to be multidisciplinary as well as interdisciplinary, with practically universal applicability, and is used in a broad range of fields. EOE should not be relied upon to develop probability of failures and replace a physics based reliability model. EOE can be used to complement such models in defining both input data and calibration points for the limit state. EOE is discussed more in detail in Chapter 4.

3-7. Time-Independent Reliability. Reliability of a structure or component that does not degrade with time is considered to be time-independent. This method is used to estimate the reliability at a single evaluation point or time in the life of a structure. This makes the assumption that the reliability is constant in time and over the life of the structure. Three methods can be used to estimate the time-independent reliability, and they progress in sophistication and accuracy:

First Order Second Moment (FOSM) or Reliability Index (□ Method) (Baecher 2004)

Point estimate method (Rosenblueth 1969)

Advanced Second Moment (ASM) (Ayyub 1984)

Monte Carlo simulation (Palisades 2008)

a. Reliability index. The reliability index β provides a measure of reliability as a function of the means and standard deviations of capacity *C* and demand *D* where *C* and *D* are functions expressing the capacity and demand associated with the performance mode for the components. The reliability index is the distance or number of standard deviations from the mean to the limit state that has been set. Figure 3-5 shows graphically the meaning of the reliability index.

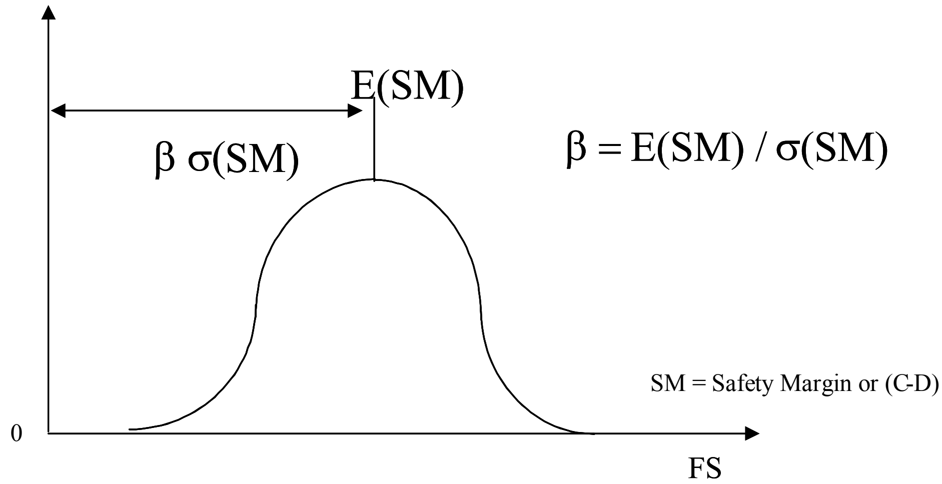


Figure 3-5. Reliability index relationships

The reliability index utilizes a limit state, which equates *C* and *D* into a safety margin that can be estimated assuming either a normal or lognormal distribution as follows.

(1) Normal distribution. If *C* and *D* are normally distributed, then reliability index is estimated in terms of the safety margin (SM) or *C* - *D*, as:

$$\beta = E[\mathrm{SM}]/\sigma[\mathrm{SM}] \tag{3-6}$$

$$\beta = E[\mathrm{C - D}]/(\sigma_C^2 + \sigma_D^2)^{1/2} \tag{3-7}$$

$$P_{up} \approx \Phi(-\beta) \tag{3-8}$$

where Φ is the standard normal deviate.

In the above formulations, if the distribution of safety margin or *C* - *D* is not exactly normal, β, still provides a good measure of the distance from the expected value of the safety margin to the limit state in normalized units of standard deviation.

(2) Lognormal distribution.

(a) If *C* and *D* are lognormal distributions and given their statistical parameters for the expected value of capacity $E[C]$, expected value of demand $E[D]$, standard deviation of capacity σ_C and standard deviation of demand, σ_D that describe the jointly distributed lognormal random variables, there is an equivalent joint normal distribution on the logarithms of *C* and *D* having mean values:

$$E[C]=\ln\,[\mathrm{C}] - \sigma_C^2/2 \tag{3-9}$$

$E[D]=\ln[D] - \sigma_D^2/2$ (3-10)

where

$\sigma_C = [\ln(1+V_C^2)]^{1/2}$

$\sigma_D = [\ln(1+V_D^2)]^{1/2}$

where V_C and V_D are the coefficients of variation (ratio of the standard deviation to the expected value). If the means are approximated as ln [C] and ln [D], and the standard deviations are approximated as V_C^2 and V_D^2, then the reliability index β becomes:

$\beta = \ln(E[C]/E[D]) / (V_C^2 + V_D^2)^{1/2}$ (3-11)

The above expression is for lognormal distribution of C and D and the probability of unsatisfactory performance is

$P_{up} = P(\ln C - \ln D) \leq 0$ (3-12)

Most importantly, the standard deviation of the transformed normal variate is used to scale the reliability index rather than the underlying lognormal variate. An approximate value for P_{up} can be estimated similar to the normal distribution through the use of the table for the standard normal variate. An example of the FOSM reliability method is discussed in detail in Appendix C.

(b) The benefits of using reliability index methods to estimate Pup are that the equations and methods are simple to use and can easily and quickly be developed into a spreadsheet model. The major limitation of using these methods is that they do not handle nonlinear limit states very well.

(3) Advanced Second Moment (ASM) - ASM is a first-order reliability technique that uses directional cosines (partial derivates of moments) and a Rosenblatt transformation procedure to determine the distance or reliability index β from the failure surface to the origin. This method is much more mathematically complex than the simple FOSM methods presented in paragraph 3.7a. This method generally requires the use of an advanced reliability program such as PROBAN or CALREL to perform the ASM calculations but some simple limit state problems (two or three random variables) can be programmed into a spreadsheet. However, one of the limitations to this method is that the types of probability density functions used in the program are somewhat restricted.

3-8. <u>Monte Carlo Simulation</u>. With the increase in computing power over the past 10 years, the Monte Carlo simulation can easily be used to determine the time-independent reliability of features. The Monte Carlo simulation is readily available as a commercial-off-the-shelf program for use as a spreadsheet add-in such as @Risk or Crystal Ball. These programs are easy to implement for reliability purposes. The Monte Carlo simulation method does not use the reliability index but calculates P_{up} using the probability density function of a random variable in combination for the input random variables on a limit state function. If convergence and number of simulations are proper, the resulting reliability calculations can be expressed:

P_{up} = Number of failures/Number of total iterations performed (3-13)

$R = 1 - P_{up}$ (3-14)

The result from the Monte Carlo simulation will be an exact answer if enough simulations are performed and convergence is obtained on the mean and variance of P_{up}. However, caution should be exercised and the input distribution data and output result distributions checked once the Monte Carlo simulation is complete.

3-9. Hazard Function Analysis. (Patev et al. 1997)

a. A conceptual illustration of the time-dependent reliability problem using the random functions for resistance R and load intensity S is shown in Figure 3-6.

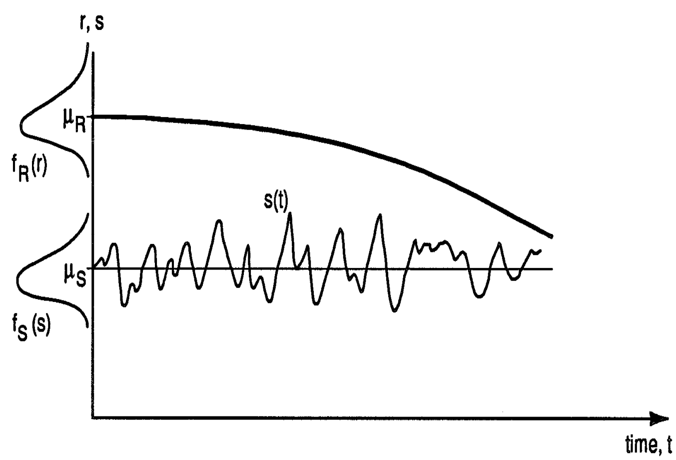


Figure 3-6. Schematic representation of load process and strength degradation

In this figure, the degrading phenomenon for strength over time is described by the density function $f_R(r)$, and the randomness in the loads is described by the density function $f_S(s)$. If both loads and strength are functions of time, the structure performs unsatisfactorily when $R(t) < S(t)$ for at least one of the load events in (0,t).

b. The limit state for any structure in time can be expressed by the equation;

$R(t) - S(t) < 0$ (3-15)

and the probability of unsatisfactory performance at time t is expressed by $P[R(t) < S(t)]$. Ellingwood and Mori (1993) have shown that equation for time-dependent reliability function $L(t)$, which is the probability a structure survives during the interval from $(0,t)$, can be written in terms of the random distribution functions of time for loads and strength. This equation is defined as

$$L(t) = \int_0^\infty e^{\left[-\lambda t\left[1-\frac{1}{t}\int_0^t F_S\{r.g(t)\}dt\right] f_{R0}(r)dr\right]}. \tag{3-16}$$

In this equation,

λ = mean rate of occurrence of loading

F_s = CDF of load S

$g(t)r$ = time-dependent degradation

$f_{R_0}(r)$ = pdf of the initial strength R

Because closed-form solutions for this equation are not readily available, Monte Carlo simulation must be used. Monte Carlo simulation can effectively be used for solution of this time-dependent reliability formulation at minimal expense in computational error invoked.

c. The probability of unsatisfactory performance $Pf(t)$ in the time interval $(0,t)$ is directly related to the $L(t)$ function as shown:

$$P_f(t) = 1 - L(t) \tag{3-17}$$

The conditional failure rate or hazard function describing the conditional probability that unsatisfactory performance occurs in the interval (t, t + dt), given that the structure has survived up to time t can be expressed as

$$h(t) = P[\text{fail in } (t, t + \mathrm{d}t)|\ \text{survived } (0,t)] \tag{3-18}$$

or

$$h(t) = - \mathrm{d}/\mathrm{d}t\ [\ln L(t)] \tag{3-19}$$

d. Functions $L(t)$ or $h(t)$ are the most informative descriptors of time-dependent reliability and are used to predict the service life of a structure. Typical reliability and hazard functions illustrated in Figure 3-7 are related to each other such that when one is known the other can be constructed using Equation 3-19. When the hazard function increases rapidly with time, structural performance becomes unpredictable and inspection and/or repair is indicated.

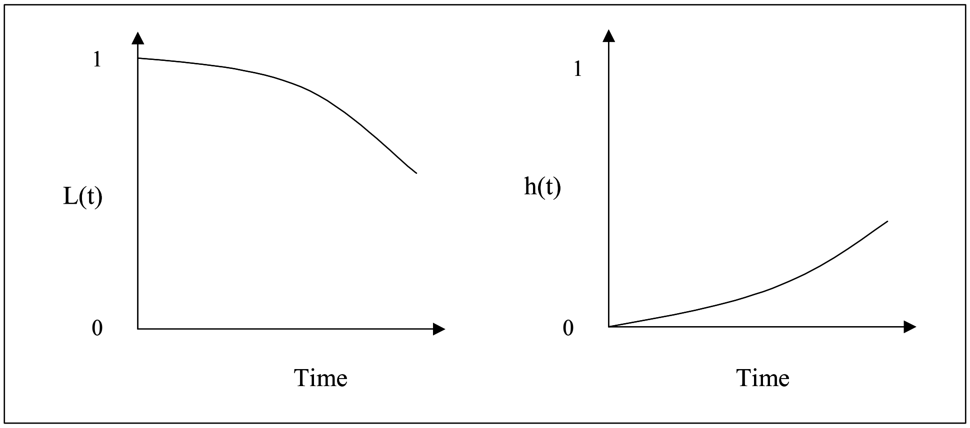


Figure 3-7. Time-dependent reliability and hazard functions

3-10. Hazard Function Example. This example will demonstrate how to apply the methods for time-dependent reliability and the hazard function described in Section 3-9 to a steel bar in tension subjected to corrosion. The steel bar and limit state are shown in Figure 3-8. The example utilizes both Microsoft Excel and @Risk, a commercially available Monte Carlo simulation package, to determine the time-dependent reliability and hazard function for the steel bar over a 50-year life cycle.

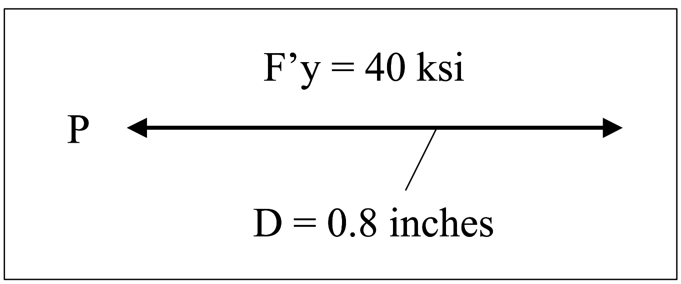


Figure 3-8. Steel bar example

a. Monte Carlo simulation.

(1) Example inputs.

(a) For this example, the steel bar is installed into a gate structure without any corrosion protection. The bar is subjected to uniform corrosion from submersion in an aquatic environment. The steel bar has an initial diameter (d) of 0.8 in. or an initial area (A) of 0.5 in.2 The anticipated service life for the steel bar in the structure is 50 years.

(b) The random variables for this problem are assigned to the yield stress of the steel f'_y, the corrosion rate μ, and annual extreme design load P. Table 3-1 shows statistical parameters and distributions for the random variables.

Table 3-1. Random Variables for Example

Random Variable	Notation	Distribution	Statistical Parameters
Yield stress (ksi)	f'_y	Lognormal	Mean = 40 ksi Std. Dev. = 4 ksi
Corrosion rate (inches per year)	M	Truncated normal	Mean = 0.001 in/year Std. Dev. = 0.0005 in/year Minimum = 0 Maximum = 0.02
Annual design load (kips)	P	Normal	Mean = 15 kips Std. Dev. = 3 kips

b. Limit state.

(1) The limit state for this example is based on the yield stress capacity in the bar compared to the yield stress demand over time from the corrosion in the bar. So the relationship for the decrease in cross-sectional area A(t) of the bar in time can be shown as

$d(t) = d(t - 1)\text{-}2^*\ \mu$ where $d(t = 0) = 0.8$ in. (3-20)

$\mathrm{A}(t) = \pi^* d(t)/4$ (3-21)

(2) The stress demand in the bar with time $D(t)$ can then be represented by

$D(t) = \sigma(t) = P/A(t)$ (3-22)

(3) The stress capacity of the bar over time C(t) can then be represented by the yield strength f'_y, so the limit state for this example can be shown as follows:

Limit state = $C(t) - D(t) = f'_y, - \sigma(t) < 0$ (3-23)

(4) Therefore, when C(t)-D(t) < 0, an unsatisfactory performance of the bar is assumed to occur at time t. Therefore the year t when unsatisfactory performance occurs becomes the critical information needed for calculating the time-dependent reliability and hazard function for the steel bar.

(5) A spreadsheet for the tension bar example is shown in Figure 3-9. The spreadsheet calculates the limit state defined above using the mean values of the random variables as shown in Table 3-1. It is interesting to note that there is no unsatisfactory performance for the given limit state at the end of the 50-year life cycle using the means for the random variables. This is

Tension Bar Example

Constants

Initial Diameter	0.8	in
Initial Area	0.50	in^2
Initial Stress	29.84	ksi

Variables

Yield Stress	40.000	ksi
Corrosion Rate (per year)	0.001	in/yr
Annual Extreme Design Load	15	kips

Year	**Diameter** (in)	**Area** (in^2)	**Load** (kips)	**Stress** (ksi)	**Design Stress** (ksi)	**Limit State** C - D < 0	Year
						0.9998	70
50	0.7000	0.38	15	38.98	40.00	1.02	50
49	0.7020	0.39	15	38.75	40.00	1.25	49
48	0.7040	0.39	15	38.54	40.00	1.46	48
47	0.7060	0.39	15	38.32	40.00	1.68	47
46	0.7080	0.39	15	38.10	40.00	1.90	46
45	0.7100	0.40	15	37.89	40.00	2.11	45
44	0.7120	0.40	15	37.67	40.00	2.33	44
43	0.7140	0.40	15	37.46	40.00	2.54	43
42	0.7160	0.40	15	37.25	40.00	2.75	42
41	0.7180	0.40	15	37.05	40.00	2.95	41
40	0.7200	0.41	15	36.84	40.00	3.16	40
39	0.7220	0.41	15	36.64	40.00	3.36	39
38	0.7240	0.41	15	36.44	40.00	3.56	38
37	0.7260	0.41	15	36.23	40.00	3.77	37
36	0.7280	0.42	15	36.04	40.00	3.96	36
35	0.7300	0.42	15	35.84	40.00	4.16	35
34	0.7320	0.42	15	35.64	40.00	4.36	34
33	0.7340	0.42	15	35.45	40.00	4.55	33
32	0.7360	0.43	15	35.26	40.00	4.74	32
31	0.7380	0.43	15	35.07	40.00	4.93	31
30	0.7400	0.43	15	34.88	40.00	5.12	30
29	0.7420	0.43	15	34.69	40.00	5.31	29
28	0.7440	0.43	15	34.50	40.00	5.50	28
27	0.7460	0.44	15	34.32	40.00	5.68	27
26	0.7480	0.44	15	34.13	40.00	5.87	26
25	0.7500	0.44	15	33.95	40.00	6.05	25
24	0.7520	0.44	15	33.77	40.00	6.23	24
23	0.7540	0.45	15	33.59	40.00	6.41	23
22	0.7560	0.45	15	33.42	40.00	6.58	22
21	0.7580	0.45	15	33.24	40.00	6.76	21
20	0.7600	0.45	15	33.07	40.00	6.93	20
19	0.7620	0.46	15	32.89	40.00	7.11	19
18	0.7640	0.46	15	32.72	40.00	7.28	18
17	0.7660	0.46	15	32.55	40.00	7.45	17
16	0.7680	0.46	15	32.38	40.00	7.62	16
15	0.7700	0.47	15	32.21	40.00	7.79	15
14	0.7720	0.47	15	32.05	40.00	7.95	14
13	0.7740	0.47	15	31.88	40.00	8.12	13
12	0.7760	0.47	15	31.72	40.00	8.28	12
11	0.7780	0.48	15	31.55	40.00	8.45	11
10	0.7800	0.48	15	31.39	40.00	8.61	10
9	0.7820	0.48	15	31.23	40.00	8.77	9
8	0.7840	0.48	15	31.07	40.00	8.93	8
7	0.7860	0.49	15	30.91	40.00	9.09	7
6	0.7880	0.49	15	30.76	40.00	9.24	6
5	0.7900	0.49	15	30.60	40.00	9.40	5
4	0.7920	0.49	15	30.45	40.00	9.55	4
3	0.7940	0.50	15	30.29	40.00	9.71	3
2	0.7960	0.50	15	30.14	40.00	9.86	2
1	0.7980	0.50	15	29.99	40.00	10.01	1
0	0.8000	0.50	15	29.84	40.00	10.16	0

70

Figure 3-9. Excel spreadsheet for steel bar example the reason for the placeholder of 0.9998 for Year 70 . This placeholder permits the simulation to sort out those iterations when bar did not have unsatisfactory performance in the study period of 50 years.

c. Calculating reliability and hazard rates using Monte Carlo simulation.

(1) Monte Carlo simulations are used to develop both the pdf (or probability mass function (PMF)) and resulting CDF for the years of unsatisfactory performance for the steel bar. The data contained in these distribution functions will be used to calculate both the reliability and hazard rate for this problem. A term called the life cycle concept has been adapted to establish these functions using Monte Carlo simulation (Patev et al. 1997). A flowchart defining the steps of the life cycle concept is shown in Figure 3-10. The process starts by using Monte Carlo simulation to select the random variables from the given distribution of strength and load. These values are then propagated for the entire life cycle (i.e., 50 years for this example). The year in which the limit state occurs is then recorded into an output file for post-processing of the reliability and hazard rate.

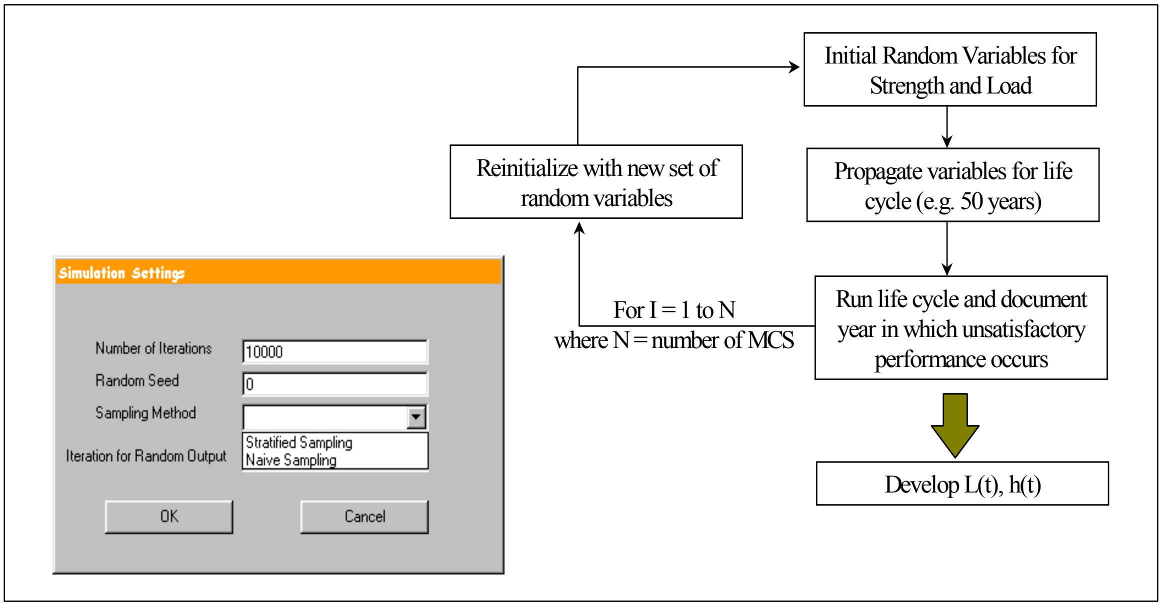


Figure 3-10. Monte Carlo simulation

(2) For this steel bar example, only 20,000 simulations were performed using @Risk. However, typically for reliability studies, the recommended values for convergence are between 20,000 and 50,000 simulations; but this depends directly upon the number of random variables in the reliability problem. A general rule of thumb is that the number of simulations should be around 10R, where R is the number of random variables.

(3) The output from @Risk gives 20,000 discrete values for the year of unsatisfactory performance for the given limit state. The PMF for the @Risk output is shown in Figure 3-11. The data can then be processed to determine the CDF as shown in Figure 3-12.

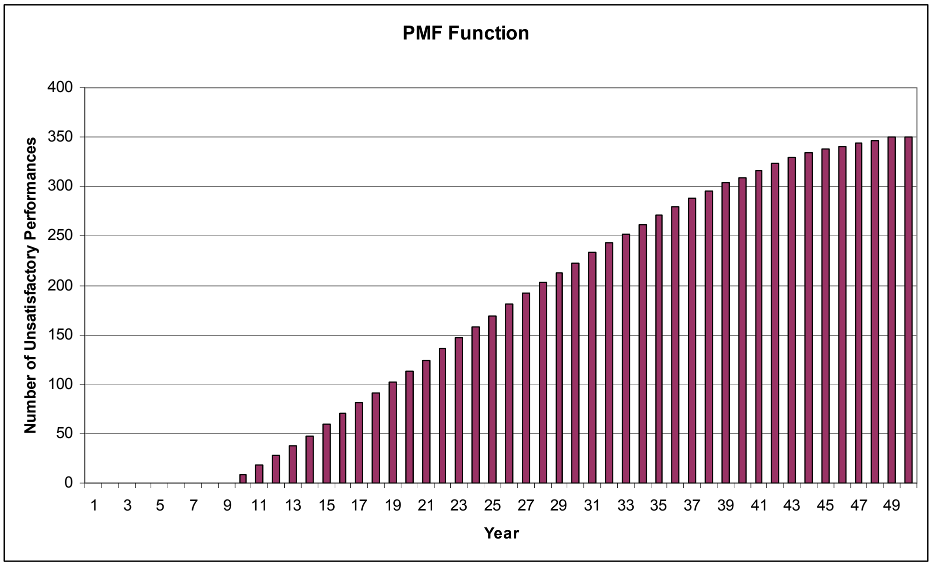


Figure 3-11 PMF for Year of Unsatisfactory Performance

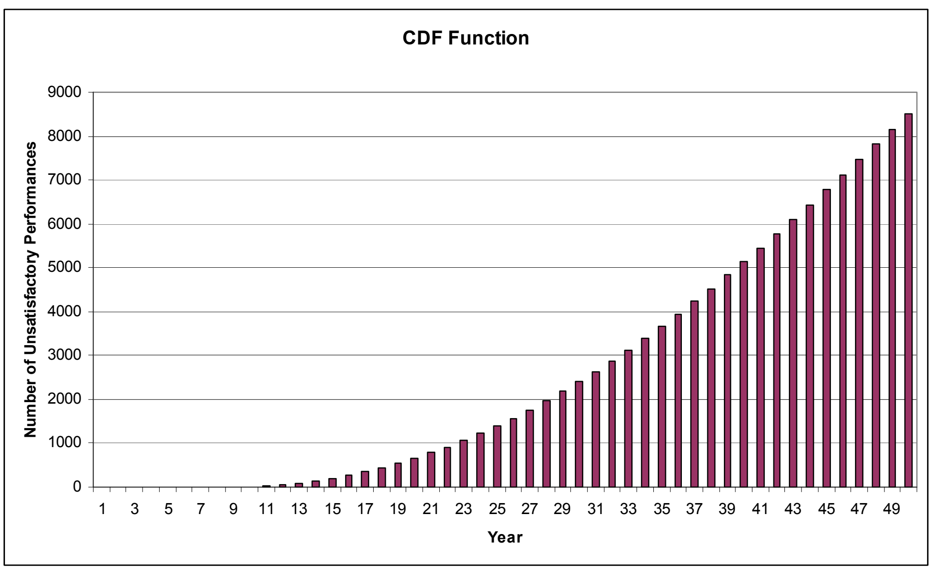


Figure 3-12 CDF for Year of Unsatisfactory Performance

(4) Using these results the cumulative reliability $L(t)$ can be determined by the following equation:

$$L(t) = \frac{\text{number of cumulative unsatisfactory performances up to time t}}{\text{total number of simulations}} \qquad (3\text{-}24)$$

(5) The hazard rate can be determined using the following equation:

$$\mathrm{h(t)} \approx \frac{\text{number of unsatisfactory performances in time t + dt}}{\text{number of survivors from 0 to t}} \qquad (3\text{-}25)$$

(6) The calculations of these items from the simulations are reflected in the spreadsheet shown in Figure 3-13.

Year	No. of UP	Cumult. UP	No. of Survivors	Reliability L(t)*	Hazard Rate h(t) No. of UP in t / No. of survivors in (t-1)
1	0	0	20000	1.00	0.0000
2	0	0	20000	1.00	0.0000
3	0	0	20000	1.00	0.0000
4	0	0	20000	1.00	0.0000
5	0	0	20000	1.00	0.0000
6	0	0	20000	1.00	0.0000
7	0	0	20000	1.00	0.0000
8	0	0	20000	1.00	0.0000
9	0	0	20000	1.00	0.0000
10	8	8	19992	1.00	0.0004
11	18	26	19974	1.00	0.0009
12	28	54	19946	1.00	0.0014
13	38	92	19908	1.00	0.0019
14	48	140	19860	0.99	0.0024
15	59	199	19801	0.99	0.0030
16	70	269	19731	0.99	0.0035
17	81	350	19650	0.98	0.0041
18	91	441	19559	0.98	0.0046
19	102	543	19457	0.97	0.0052
20	113	656	19344	0.97	0.0058
21	124	780	19220	0.96	0.0064
22	136	916	19084	0.95	0.0071
23	147	1063	18937	0.95	0.0077
24	158	1221	18779	0.94	0.0083
25	169	1390	18610	0.93	0.0090
26	181	1571	18429	0.92	0.0097
27	192	1763	18237	0.91	0.0104
28	203	1966	18034	0.90	0.0111
29	213	2179	17821	0.89	0.0118
30	223	2402	17598	0.88	0.0125
31	233	2635	17365	0.87	0.0132
32	243	2878	17122	0.86	0.0140
33	252	3130	16870	0.84	0.0147
34	262	3392	16608	0.83	0.0155
35	271	3663	16337	0.82	0.0163
36	280	3943	16057	0.80	0.0171
37	288	4231	15769	0.79	0.0179
38	295	4526	15474	0.77	0.0187
39	304	4830	15170	0.76	0.0196
40	309	5139	14861	0.74	0.0204
41	316	5455	14545	0.73	0.0213
42	323	5778	14222	0.71	0.0222
43	329	6107	13893	0.69	0.0231
44	334	6441	13559	0.68	0.0240
45	338	6779	13221	0.66	0.0249
46	341	7120	12880	0.64	0.0258
47	344	7464	12536	0.63	0.0267
48	347	7811	12189	0.61	0.0277
49	350	8161	11839	0.59	0.0287
50	350	8511	11489	0.57	0.0296

Figure 3-13. Calculations of reliability and hazard rate

(7) Another more complicated post-processing procedure can be also used to process the raw data from the Monte Carlo simulations. This technique calculates the hazard rate using the following equation:

$$h(t) = -\mathrm{d}/\mathrm{d}t\,[\ln L(t)] \tag{3-26}$$

This equation uses the natural logarithm of $L(t)$ determined from the simulations that can be fitted using nonlinear regression techniques. It is suggested that a minimum of a cubic equation be fit to the data and this fit should have a very high R squared value (greater than 0.98). The steps to determine the hazard rate by this method do take longer to process than the first method presented. While the second procedure is felt to be more accurate, if enough simulations are run, then the first procedure outlined previously generally converges very close to the more complex procedure.

d. Reliability and hazard rate results for example. The plots for results for the steel bar example are shown in Figures 3-14 and 3-15. These are taken from the data in Figure 3-13. The results indicate that the reliability for the bar at the end of the 50-year study period will be 0.6 or 60 percent. Conversely, a probability of unsatisfactory performance for the bar is 0.4 or 40%. The hazard rate would be approximately 0.03 at the end of the 50-year period, which means that out of 1000 similar bars, 30 bars would have unsatisfactory performance.

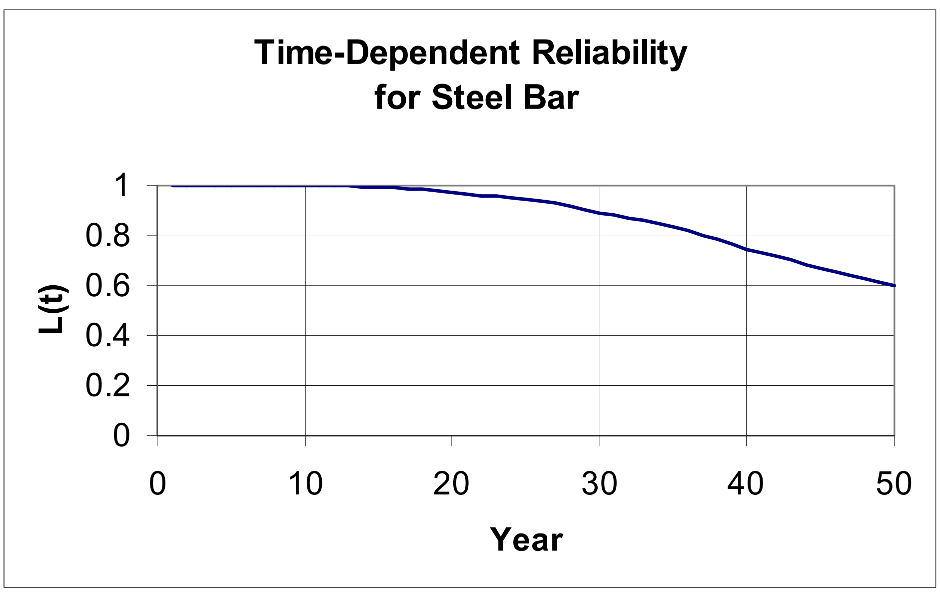


Figure 3-14. Time-dependent reliability for steel bar

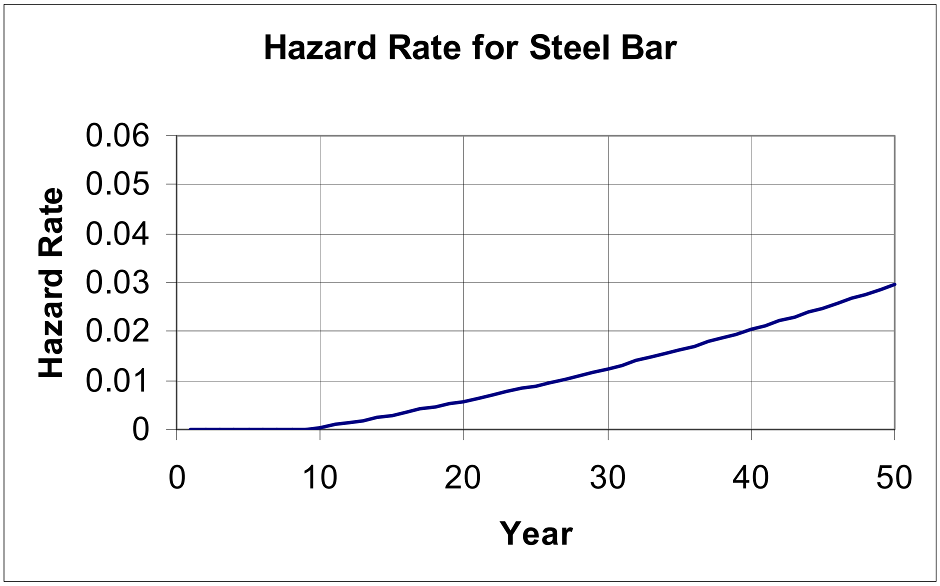


Figure 3-15. Hazard function for steel bar

3-11. Systems Reliability Applications. System reliability uses the reliability of individual components to estimate the overall reliability of a structure or project. For example, the reliability for a single leaf of a miter gate can be rolled into a system reliability for a pair of leaves or for the entire chamber depending upon how the event tree has been established. Because of redundancy of typical infrastructure components, it is often difficult to determine the exact system reliability. A process is typically established that considers bounding the overall reliability of a structure using two cases, the series system and the parallel system.

a. Series system. In a series system, the system will perform unsatisfactorily if any single component performs unsatisfactorily. This is the weakest link theory and will provide a lower bound to the overall system reliability. This also assumes there is no correlation between components. If a system has n components in a series, the probability of unsatisfactory performance of the i^{th} component is pi and its reliability, $Ri = 1 - pi$. Then the reliability of the system, or probability that all components will perform satisfactorily, is the product of the component reliabilities:

$$R = R1R2R3....Rn = (1-p1)(1-p2)(1-p3)...(1-pn) \quad (3\text{-}27)$$

b. Simple parallel system. In a parallel system, the system will perform unsatisfactorily only if all components perform unsatisfactorily. Thus, the reliability is unity minus the probability that all components perform unsatisfactorily, or

$$R = 1 - p1p2p3....pn \quad (3\text{-}28)$$

c. Parallel and series systems. Solutions are available for systems requiring r-out-of-n operable components, which may be applicable to problems such as dewatering with multiple pumps, or closing a gate bay with emergency bulkheads. Subsystems involving independent parallel and series systems can be mathematically combined by standard techniques.

d. Upper and lower bounds. Upper and lower bounds on system reliability can be determined by considering all components to form parallel and series systems, respectively; however, the resulting bounds may be so broad as to be unpractical. A number of procedures are found in the references to narrow the bounds (Ang and Tang 1984).

e. Performance modes. Real engineering systems such as locks and dams (or even building frames) are complex and may have many performance modes. Some of these may not be independent; for instance, several performance modes may be correlated to the occurrence of a high or low pool level. Earth pressures, sliding, and overturning performance are all correlated to shear strength. Rational estimation of the overall reliability of a lock and dam is a topic that is undergoing further research.

CHAPTER 4
Expert-Opinion Elicitation

4-1. Introduction.

a. The Expert-Opinion Elicitation (EOE) process is a formal (defined and set format), heuristic (through talking and discussion) process of obtaining information or answers to specific questions. The EOE was first developed by the RAND Corporation in the 1950's for use in civil defense strategic planning for the effects of thermonuclear war and the survival of the US population during a nuclear attack. The elicitation process was first called the Delphi Process or Scenario Analysis. These questions are defined in terms of what is referred to as issues. These issues can assist in defining such items as probabilities of failures or unsatisfactory performance and probabilities for event and fault trees and event timing such as time to repair or replace infrastructure components. EOE can also assist with defining economic consequences for event trees such as different levels of failure, unsatisfactory performance closure times and costs, and repair scenario costs. This EOE process should not be used in lieu of rigorous reliability and risk analytical methods, but should be used to supplement and complement them. The method is currently used by many engineering firms to assist with determining probabilities where there is lack of failure information. The EOE as a process has been outlined in Ayyub, Blair, and Patev (2000) and Baecher (2004).

b. An EOE should be performed as a face-to-face meeting of members of an expert panel that is developed specifically for the issues under consideration. Phone (including teleconferences) or e-mail surveys do not qualify as an EOE since there is no real heuristic exchange or stimulated dialog during the elicitation from the expert panel. The meeting of the expert panel should be conducted over a few days of closed session and may include a site visit to the project or component being considered in the EOE session. Site visits are good if the EOE panel members are not intimately familiar with the project for which they are being elicited. In advance of the meeting the experts must receive background information or a read-ahead package containing the objectives, the list of issues, and anticipated outcomes from the meeting. Ayyub, Blair, and Patev (2000) describe in detail the different phases of the expert-opinion elicitation process.

c. One of the primary reasons for using EOE is to assist in defining the probabilities of failure or unsatisfactory performance, closure time probabilities for event trees, and the years when long-term repairs for various components may be made. These probabilities may be utilized in the cost and closure matrices that are incorporated into the economic analysis. An EOE can be performed on these probabilities in the economic decision matrices due to time and budget constraints of this type modeling. It is warranted that some reliability models for infrastructure components are considered very difficult and impractical to develop and EOE is then a method for assisting in this case. Elicitations are performed in addition to and to complement the existing state-of-the-art reliability models that have already been developed for many components and features at CW projects.

4-2. Protocol for EOE Process. The panel of experts, observers, and the facilitator convened at a given facility for a set period to discuss and address the issues are part of the elicitation. The following protocol should be followed in the deliberation of the issues:

a. Training of the experts on probabilities and the elicitation process should be conducted using two different elicitation examples. This training is conducted to familiarize the experts with the type of questions that were forthcoming and to focus the experts on how to discuss and provide solutions to the issues that are forthcoming. The training is very helpful for making the experts more comfortable with their elicitation, and to gain their confidence in discussion with other panel members.

b. After an issue and question were presented, discussion of the issue should be encouraged to ensure that all experts clearly understood the question and event before answering. Assumptions and understandings should be also listed and agreed to by the participants. A general form with the issue should be given to each expert to record their evaluation or input. The experts' judgment along with their supportive reasoning should be recorded for the issues.

c. The collected assessments from the experts should be analyzed and aggregated quickly to obtain the first response from the experts about the issue. The medians and percentiles should be computed real time for the estimated value and discussed while being highlighted on a computer projection unit for the issue. Discussions should then ensue among the experts to explore and understand the range of their first response to the issue. The experts should be given the opportunity to revise their assessments at the end of discussion. The documentation of assumptions and opinions made by the experts for their first response should be recorded.

d. The experts should be then asked for their second responses after discussion is formally closed. The collected assessments from the experts shall be analyzed and aggregated quickly for review by the experts. This second response is shown to the experts, but no further changes can be made to these results as these are final. The experts should be also asked to give a qualitative response to their confidence in the final medians for the probabilities from the second response. This response in confidence is typically requested as high, medium, or low. These final medians become the values documented for the report and model. Most importantly, the names of the experts should be left off all elicitation documentation and just referred to as Expert #1 to N.

e. In addition, a comprehensive documentation of this process was essential in order to ensure acceptance and credibility of the elicitation results. The document should include complete descriptions of both the first and second responses and the confidence of the experts in the final median response.

4-3. EOE Process.

a. In summary, the process of elicitation of opinions is a formal process that should be performed systematically for each issue according to the following steps:

(1) Familiarization of experts with issue

(2) Discussion and agreement of initiating event and issue question

(3) First elicitation and collection of opinions

(4) Aggregation and presentation of results to experts

(5) Group interaction and discussion of first response

(6) Second elicitation and collection of opinions

(7) Aggregation and presentation of results to experts

The issues should consist of groups of questions concerning the probability of failure for multiple sequential years, the probabilities associated with input for event trees, and the year in which a component should be replaced. The questions should be assessed for the specified associated initiating events, failure scenarios, and consequences that are discussed at the beginning of each issue. The assumptions made and defined by the experts with each issue and event should be documented with the final results. These final responses should develop a cumulative probability curve that should be processed to determine the hazard function for that particular issue that was addressed.

b. The size of the expert panel should be large enough to achieve a needed diversity of opinion and credibility that will lead to resultant probabilities with minimal bias and robustness. Depending on the topics of interest it is recommended to have five to seven paneled experts for this type of study and analysis. A nomination process should be used to establish a list of candidates who could contribute best to the elicitation. From this list, formal nominations and a selection process should be established to define the candidates with the best background that closely fit the topics at hand. The panel members should be defined based on a comprehensive combined knowledge of the following:

(1) Design of locks and dams

(2) Construction of locks and dams

(3) Operating and maintenance for locks and dams

(4) Knowledge of similar structures

(5) Knowledge of method or process in the reliability models

(6) Traffic management during construction and operation of locks and dams

c. Observers should also be invited to participate in the elicitation process. The observers can contribute to the discussion, but not to the expert judgment and results. The observers can include the following:

(1) One or two observers with knowledge of lock and dam facilities including construction, operations and maintenance.

(2) One or two people with expertise in probabilistic analysis, probabilistic computations, consequence computations and assessment, and expert elicitation. This observer can be the technical facilitator or the technical integrator and facilitator.

d. EOE is a technique that uses a panel of individuals with various areas of specialized knowledge for estimating parameters or addressing issues of interest based on their expertise. Some examples of EOE were applied by CEMVK to examine three different construction alternatives for Lindy C. Boggs Lock and Dam, CELRP for concrete deterioration problems at Emsworth Lock and Dam, Mill Creek Flood Control Project by CELRL for mechanical and electrical gate closure components and ORMSS for economic cost and closures matrices. Other recent uses of EOE by the USACE include areas of navigation, dam safety, flood damage reduction, and economic studies.

CHAPTER 5
Engineering and Economic Integration

5-1. Background. Engineering reliability plays a key role in the overall economic analysis associated with USACE studies and projects. When properly implemented, engineering reliability serves as critical tool for investment decisions. Thus, the engineering reliability must be carefully integrated with the economic consequence modeling to ensure that the impacts associated with the reliability of all components being analyzed are accurately captured in the overall analysis. The reliability modeling must be consistent with the data upon which the development of economic consequences is based as well as the formulation of alternatives being evaluated. The primary format of linking the engineering reliability with the economic/life safety analysis is through consequence event trees.

5-2. Consequence Event Trees.

a. Consequence event trees are generally used to depict the applicable range of consequences associated with limit state evaluated in probabilistic modeling. The trees typically detail various levels of potential damage, possible repair scenarios, and/or replacement alternatives that may occur *given* an unsatisfactory event occurs. The event trees are used in combination with the probabilities of unsatisfactory performance as a way to determine the expected life safety and/or economic impacts associated with the performance of a particular component. Event trees can be set up to predict individual component performance or can be constructed to evaluate the reliability for all components across the project. It depends upon the type of analysis being undertaken as to how the event tree should be structured. Most importantly, the event tree data must be consistent with the limit state of the component(s) for which it is being developed. If this is not directly known for a particular study or there are multiple potential limit states associated with an individual component, a failure modes and effects analysis (FMEA) can be undertaken by a team of engineers to determine the most likely failure situations to be modeled. It is highly recommended that individuals familiar with the design, repair, and maintenance of the component be part of the team that develops the data that develops the framework for the event tree. The EOE is an excellent tool that can be used to develop the information required in an event tree when analytical modeling is not a viable option. The EOE process is described in detail in Chapter 4.

b. There is no standard of format for event trees that works for every situation. They are specific to the feature, load case, and project being evaluated. However, there are definitely some required elements of an event tree in order to carry out a risk analysis. For example, the levels of failure and associated repairs need to be consistent with the load cases being evaluated, limit state being modeled, and the formulation alternatives being analyzed as part of the overall study. In addition, each level of the tree must have branches that add up to 1.0 since all the event branches are considered mutually exclusive and collectively exhaustive. For example, given that an unsatisfactory performance occurs for a particular feature and load case and that it is determined that a range of three levels of failure fully covers the way the component can reasonably be expected to fail, the sum of the probabilities of each of these three failure levels must equal 100 percent because it is predicated upon the failure having already occurred. Consequences (potential life loss/property damage, etc..) are then determined for each level of failure in the event tree. A generic flow diagram that illustrates the basic parts of an event tree for a water resource system is shown in Figure 5-1. A description of each of the various aspects depicted in Figure 5-1 is provided following the diagram.

1. INITIATING HAZARD
Determined for each case being evaluated

↓

2. INFRASTRUCTURE PERFORMANCE
Given the event occurs, what is the probability of unsatisfactory performance (P_{UP})?

↓

3. LOSS OF CONTROL OR PROTECTION
Given limit state being modeled, what are possible levels of failure given that it occurred?

↓

4. EXPOSED POPULATION OR PROPERTY AT RISK
For each level of failure, what are expected consequences (LOL, $$$)?

↓

5. LOSSES AND FUTURE IMPROVEMENTS
Following the repair, what is the improvement in reliability for future operations?

Figure 5-1. General flow diagram for event trees

c. Reliability Analysis.

(1) Initiating Hazard. In general, this is the starting point of the consequence event tree. This represents the likelihood of the actual event or load case occurring for which the limit state is being analyzed. In some cases where the normal load case is the only case being evaluated on an annual basis, then the frequency can be set to 1.0 as the starting portion of the event tree. A good example of this situation is a fatigue analysis being carried out on a hydraulic steel structure that is subject to the same type of loading multiple times each year. The hazard rate can be developed to account for the multiple loadings within each year, and then the frequency is set equal to 1.0 since it is known that the load occurs every year. However, this is just one example and the frequency should be distributed according to their annual event frequency. There may be other cases where the probability of failure is developed for a particular seismic event. The frequency of that seismic event must then be determined and would serve as the first part of the event tree. Without that particular event, the frequency is just one minus that value.

(2) Infrastructure Performance. In general, the next branch of the event tree relates to the probability of failure for a specific event given that it occurs. This probability can be event driven or developed for some time period, which is usually annual. The probabilities covered in this category are developed directly from the reliability analysis regardless of the type of method used (e.g., hazard functions, survivorship curves, expert elicitation). These values could remain constant through the study period (time-independent) or could change for each required increment being evaluated, which is usually on an annual basis (time dependent). Generally speaking, only two branches are used for this portion of the event tree for each load case being evaluated: the probability of unsatisfactory performance (PUP) and the corresponding reliability (1-PUP). If multiple load cases are being evaluated within the same event tree (seismic, hydrologic) then several different sections of these branches may cover the various load cases.

(3) Loss of Control or Protection.

(a) This part of the event tree provides the range of expected failure assuming the component reaches its limit state as modeled in the reliability analysis. Since the engineering performance can vary so much depending upon the component, usually two to four levels of failure are used to cover the potential range of performance given that the component's limit state is reached. Many times failure levels such as catastrophic, major, and operational are used to describe the range of failures. In terms of their likelihood of occurrence, the sum of all the failure scenarios being considered in the event tree must equal a value of 1. Only "significant" levels of failure should be considered and thus selected as such for the limit state in the reliability model. The term significant level of failure implies that there is either a real potential for loss of life and/or economic consequences of considerable importance would occur if the limit state is reached. If "minor failures" are included, they most likely would have little impact on the overall analysis and therefore should not be considered. For example, if one of the consequences selected for a limit state would be to simply continue to monitor the project feature, this more than likely is not going to cause enough consequences to make any real difference in the overall life safety and/or economic analysis. In other words, limit states must be selected that cause a significant threat to life safety and/or economic damages that include the loss of project benefits associated with having the facility out of service as well as direct damages caused by the failure itself.

(b) The best means of obtaining the information regarding a realistic range of expected damages and repairs is from actual field experience with the component and limit state being modeled. However, many times this is not available because the component has not yet reached the limit state being modeled in the reliability analysis. When this is the case, an EOE is an excellent tool to develop the range of damages and repairs, along with the percentages associated with each portion of this range. Since the values within the event tree can have as much influence on the outcome of the analysis as the hazard rates themselves, it is important that a defendable method is used to develop these values. The EOE provides can provide that defendable method when analytical tools are not available.

(c) Economic and Social Impact.

(4) Exposed Population or Property at Risk. Directly related to the different levels of failure in the previous section of the event tree, the consequences are usually referenced to the loss of life and/or economic damages for each level of failure evaluated. They are used to determine the impact associated with the component reaching its limit state in the analysis. The economic damages include such losses of benefits as hydropower, flood damage reduction, loss of navigation, loss of recreation, or loss of irrigation for agriculture associated with the time the project is out of service. It also entails physical damage such as flood damages downstream of the project, damages to project, and the required repairs to get it back to fully repaired service. Loss of life is a function of the population at risk (PAR) and such characteristics as the project purpose, type of failure, terrain downstream of the project, or warning time. Consequences must be developed for each of the levels of failures used in the event tree. The consequences in the event tree need to be carefully integrated with the capabilities of the economic model to ensure that the overall costs associated with the failure event are included in the economic analysis such as loss of benefits associated with having a lock chamber closed due to failure and subsequent repair. Thus, for the example given, the economic model must be able to incorporate the economic impact to the navigation industry and any other impacts with the associated days of lock chamber closure. The preferred method of obtaining the consequences, both repair cost and component downtime, associated with each level of repair is through a formal EOE unless a historical database of repairs or other information can be used as a basis for the event tree information.

(5) Losses and Future Improvements.

(a) Once a repair, modification or rehabilitation has been completed after an unsatisfactory performance occurs, the future reliability of the component must be considered for situations where a life cycle analysis is being carried out such as a major rehabilitation evaluation. Some repairs such as an emergency action may not drastically improve the overall reliability of the feature, while other repairs may improve the component reliability for the limit state being evaluated that it can be assumed to be 100 percent reliable for the remainder of the study period. The future reliability must be consistent with the repair considered for each level of damage. Supporting engineering analysis or experience from similar historical repairs to determine the effectiveness of the repair should be used if possible as a basis for this upgraded reliability. The supporting engineering analysis may include advanced analytical techniques, such as finite element modeling, when warranted. This is a good idea if this type of analysis was used when developing the reliability model portion of the analysis. If supporting engineering analysis or historical information is not feasible or available, a formal EOE should be used to develop the upgraded reliability for the component.

(b) The event trees used for the economic analysis are dependent upon the component being analyzed, as well as the formulation of alternatives being investigated. The economic analysis typically encompasses a study period defined by a number of years. The economic analysis uses the component-specific event trees for each year of the analysis to determine the economic impact associated with its reliability on an annual basis. There are excellent examples of practical, well developed event trees in the technical appendices of this EC.

(c) The event tree is specific to the plan being evaluated. Thus, when such multiple scenarios as fix-as-fails or advance maintenance are being evaluated, the event trees are developed to be consistent with the scenario or alternative that is being analyzed. Plan formulation of the study usually requires multiple without-project scenarios to be investigated. For navigation studies, this typically means that a baseline condition (fix-as-fails) and advance maintenance, and possibly other scenarios, must be analyzed to determine the future without-project condition for the study. Each of these alternative scenarios can have a different impact on component reliability. Therefore, probabilities of unsatisfactory performance for each component must be developed for all without-project scenarios being investigated. An event tree must also be developed for the component that is consistent with each maintenance scenario being evaluated.

5-3. Consistency of Reliability Analysis with Component Performance.

a. As stated previously, one of the most important aspects to reliability modeling is ensuring that the limit state being analyzed is consistent with the various levels of performance including failure and subsequent consequences being used in the analysis. If this is not done, there is considerable probability that the results will be skewed. The skew can lead to overestimating or underestimating consequences. For example, a limit state was selected that evaluated whether safety factor criteria were exceeded. At the same time the consequences were set up to simulate a range of failures of the component. This situation would most likely overestimate the damages since it was based upon some type of safety criteria and not actual expected performance. This is the major reason that safety factors are not used in reliability analysis.

b. On the other hand, the results could be skewed such that they vastly underestimate the potential consequences associated with a structure's reliability as well. An excellent example of this was the process used for the reliability analysis conducted for the horizontally framed miter gates as part of the ORMSS Study (see Appendix B for the detailed example of the Markland Locks and Dam miter gate reliability analysis). An engineering reliability assessment of the miter gates on the Ohio River was required as a part of ORMSS. A known history of operational problems was associated with several sets of miter gates on the Ohio River that affected their ability to function properly without increased maintenance time, which put them out of service. When the ORMSS Engineering Team set out to do its original analysis, it took some previous examples of miter gate reliability modeling that focused on the fatigue analysis associated with bending of the main load-carrying girders. The original reliability analysis also considered corrosion effects through the entire study period, which went through 2050. Because of the original safety factors in the design of the structure with respect to bending, the original reliability analysis did not indicate any potential for cracking patterns until very late in the study period (beyond 2040). This information was not consistent with the field experience at several of the projects where such extensive cracking had recently been noted during maintenance inspections that emergency repairs had to be made. Therefore, the ORMSS Engineering Team refocused its reliability analysis to evaluate the cracking pattern occurring in the field. It was determined through advanced modeling that the cracks were indeed fatigue/corrosion related, but as a function of the residual stresses associated with original construction and continued operation. The gate was never originally designed for this situation. The reliability model was redirected to this type of limit state, and the results made much more sense to actual field experience.

c. Another example along these lines would be the analysis of a wall section for stability. Generally, the wall sections are analyzed for at-rest earth pressures on the driving and resisting sides of the wall section. However, once movement occurs, the at-rest earth pressures on the driving side generally turn to lower active pressures with little movement. Full passive resistance generally takes much more movement. The thing that must be considered is the amount of wall movement that causes equilibrium between the driving and resisting side. Additional consideration must be given to whether adjacent wall sections provide any frictional resistance to the movement as well. Regardless, if the calculated amount of wall movement to reach equilibrium would have little to no impact on the overall function of the facility, then a more stringent limit state such as additional movement or collapse of the wall needs to be selected. It depends upon the use of the wall within the context of the overall project and how much movement would adversely affect the project or cause a significant potential for loss of life or service of the project. Refer to the technical appendices for examples of limit states from actual USACE studies.

5-4. Methods to Develop Information Within Event Trees.

a. The best way to develop the necessary information for event trees is from historical experience; however, since limit states should be developed such that the consequences are not minor in nature, rarely is historical data available to support developing the event tree information. There are some mechanical and electrical components such as pumps or motors for which historical data may be available through manufacturer's survivorship curves. This would provide an excellent means to establish repair costs, service disruption time, and updated reliability if the repair requires a new pump or motor to be installed. In lieu of this type of data, basically only two other options are available to develop the information in a defendable manner: analytical methods or EOE.

b. Analytical methods are the preferred method particularly if an analysis is already set up to model the limit state and can easily be adapted to evaluate the effectiveness of repairs. A good example of the use of this process was for the Chickamauga Lock Replacement Study for the Nashville District. The main issue at this project was alkali aggregate reaction of the concrete, which causes it to expand and weaken internally through induced stress cracks associated with the expansion. This study required the use of reliability analysis to determine the long-term operating impacts associated with this degraded project. Engineering reliability models were developed for the lock walls at the project. The engineering basis for the reliability model was a finite element model of the lock walls that included the effects of alkali aggregate reaction through time. One of the short-term methods of repair evaluated for this structure was the installation of dowel bars to tie critical sections of the concrete lock walls together where they had cracked near embedded anchorages. Since a finite element model was already developed to establish the long-term reliability of the lock wall structure, the engineering team analyzing the structure modified the finite element analysis to look at the remaining service life of these dowel bars in terms of how long they could be expected to provide structural support for the monolith. This information was used to determine the most likely repair scenario for this lock wall for the advance maintenance scenario. It was also useful in helping establish the future reliability of the lock wall following the repair by determining the effectiveness of the dowel bars through time.

c. All reliability analyses will require a finite element model to determine the time-dependent reliability of the structure. Many times a traditional analysis can indicate how effective a repair might be over the long term depending upon the component and limit state being evaluated. Anchoring of a gravity structure for stability purposes could possibly fall in this category.

d. When analytical methods and historical data are not viable options to assist in developing event tree information, the best solution is the use of the EOE to gather the necessary information. The EOE is described in detail in Chapter 4. It can be a very useful tool to supplement analytical methods and cover the variety of information needed for the event tree.

5-5. Establishing the Base Condition and Alternative Without-Project Scenarios.

a. Determining the Base Condition represents a collaborative effort among Planning, Engineering, and Economic team members, and it represents one of the most critical steps in the whole analysis process. The Base Condition is the plan/scenario against which all other plans are measured; therefore, correctly establishing it is very important. Several considerations need to be determined when developing the Base Condition: determining operating trends, current and projected reliability of all critical components, and planned maintenance on the project. These must all be provided for through the entire study period. Once the Base Condition is established, one or more additional Without-Project (W/O) alternatives need to be evaluated to determine their merit in terms of risks. This is generally some level of advance maintenance in lieu of a full-scale major rehabilitation or project replacement. Once the Base Condition and the remaining W/O Project alternatives are developed, the overall W/O Project is considered optimized among the various W/O Project plans and is set for comparison to the With-Project alternative (generally a major rehabilitation or new project).

b. Typically, a reliability analysis is being carried out to evaluate an anticipated stream of services provided by a project for all W/O Project scenarios that are evaluated. Even projects with critical reliability issues can still provide positive benefits but at a reduced rate of return compared to its intended design. This simplified, generic representation is depicted in Figure 5-2 where the Base Condition net benefits (project benefits minus total operating costs) of the project are represented by the cross-hatched area between the curves. The project continues to provide benefits with reduced reliability. However, additional benefits are possible by improving the reliability of the project, which in a sense increases the operating capacity and makes the area between the curves larger, thus increasing project benefits. The most important question that must be answered through the analysis is “Are the funds required to improve the reliability project worth the investment?” This is where accurately establishing the Base Condition becomes critical to overall analysis. From there, other W/O Project alternatives can be developed and evaluated, but it starts with the Base Condition. Figures 5-3 and 5-4 show how including future reliability based performance increases the incremental benefits of the system.

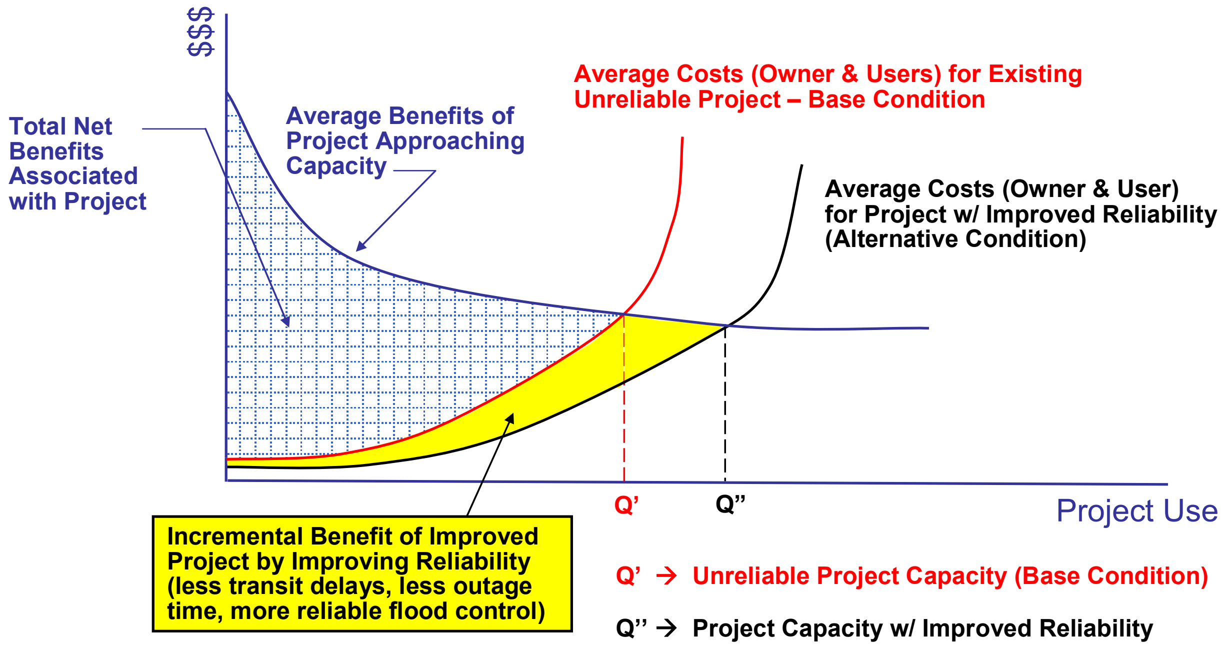


Figure 5-2. Graphical representation of added benefits of improved reliability

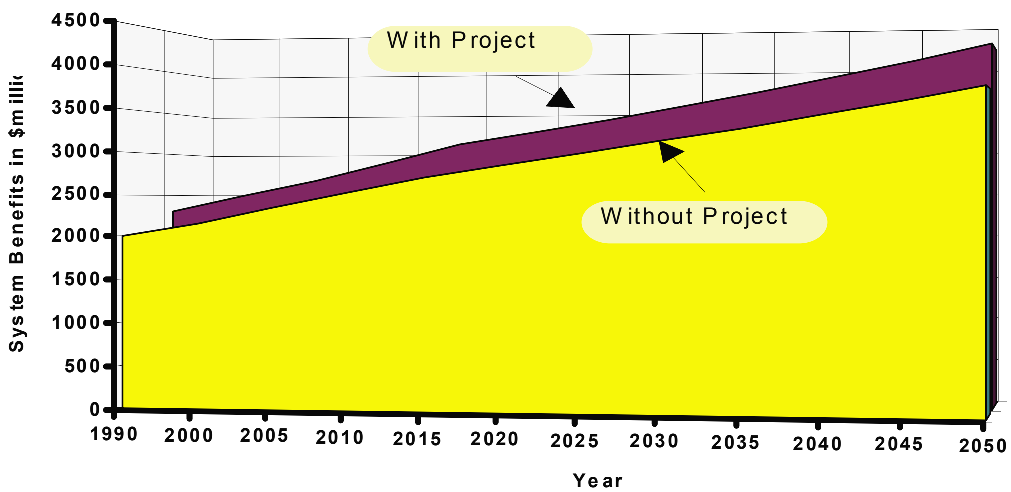


Figure 5-3. Incremental system benefits for 38 lock chambers assumed to be 100% reliable through study period for the Without Project condition

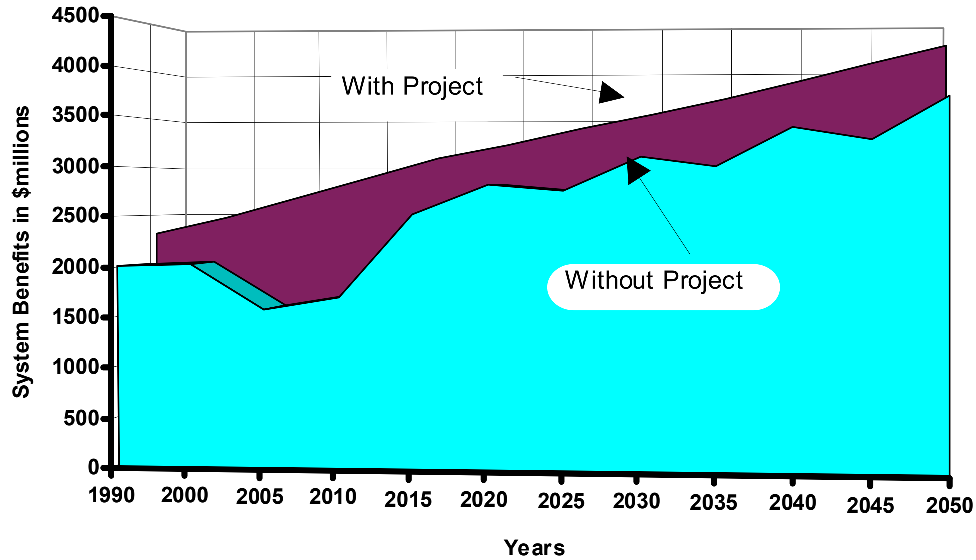


Figure 5-4 Incremental system benefits for 38 lock chambers accounting for future reliability-based performance of 15 lock and dam components

c. A starting point for establishing the Base Condition is the current situation with respect to funding, maintenance requirements, etc. This may not represent the Base Condition as some projects might already be in an aggressive or advance maintenance mode just to keep the project operational and safe, but it provides a good place to begin to establish what the Base Condition should be for the evaluation.

d. An example of how to establish a Base Condition and an Advance Maintenance W/O Project alternative can be taken from the Markland Locks and Dam Major Rehabilitation Evaluation. Markland Locks and Dam is one of seven Ohio River lock and dam projects on the Ohio River owned, operated, and maintained by the Louisville District. All of these projects have twin lock chambers. This example will focus only on the main lock chamber since this is the issue relative to this discussion. For all seven Louisville District Ohio River main lock chambers, a "typical" maintenance pattern entails dewatering and inspection of the main lock on a 5-year interval. Every third dewatering (15 years), the main lock chamber is dewatered for major repairs. In the mid-1990's during a routine inspection dewatering, major damage was found to the lock gates that are used to control pool levels for locking operations. It was subsequently determined that this was due to a fatigue issue related to the original construction procedures (residual stress build-up during welding) for which the gate was never originally designed. The level of damage and knowledge of the cause have led the Louisville District to go to a more frequent, aggressive maintenance and inspection program in order to keep the gates operational. Instead of every 5 years, the main lock chamber is now dewatered, inspected, and repaired on an annual or semi-annual basis until new gates can be procured and installed. This, of course, significantly drives up physical repair maintenance costs as well as having a major impact on shippers who use this facility. However, its impact is not nearly as bad as it would be if the gates were allowed to continue to deteriorate without the aggressive maintenance and a major or catastrophic failure were to occur. Note that this has not occurred at Markland, but these are considered stop gap measures to slow the rate of gate deterioration until new gates can be placed in the lock chamber.

e. Therefore, for the Markland example, the Base Condition was represented as the "business as usual" situation for typical Ohio River main lock chambers where the lock chamber is dewatered, inspected, and repaired on a 5-year recurring basis. The probability of unsatisfactory performance associated with this scenario was computed and linked to the potential consequences associated with this failure. In addition, the repair costs associated with the scheduled maintenance to determine the overall costs associated with this scenario were determined. The next step was to select an Advance Maintenance Scenario under the W/O Project. For the Markland example, the aggressive maintenance where the chamber is currently dewatered, inspected, and repaired annually because of its poor condition represents the Advance Maintenance Condition. This is then compared with to the Base Condition. This is done to determine the optimum W/O Project Scenario that is then compared to any With-Project Scenario (major rehabilitation, new project). Graphically, the hazard functions for the two W/O Project Scenarios (Base Condition and Advance Maintenance) for the Markland example is shown in Figure 5-5. This graph shows the improved reliability (lower failure rate) associated with the Advance Maintenance, but this comes at a higher "scheduled" maintenance cost since the lock chamber is dewatered more frequently.

5-6. Developing the With-Project Scenario and Comparing to W/O Project.

a. As noted earlier, an optimized W/O Project scenario is developed by first establishing the Base Condition Scenario. Next, one or more alternative W/O Project scenarios are then identified, and the total expected consequences (costs, loss of life, etc.) are calculated for each of these scenarios and then compared with the Base Condition. These alternative W/O Project scenarios should serve to improve the reliability, but generally come at the expense of additional service disruption time and/or additional maintenance costs. Once the life cycle projected reliability and total costs are developed for each of these alternatives, an optimized W/O Project is determined in much the same manner as specified in the previous paragraph. From an economic standpoint, the optimized W/O Project is the plan that maximizes National Economic Development NED benefits or minimizes overall total costs. For projects with life loss considerations, special consideration must be given to that aspect of the evaluation as many times it will drive the optimal W/O Project selection.

b. Once an optimized W/O Project is selected, it represents the plan against which any With-Project alternatives need to be compared to determine the economic merit. For major rehabilitation studies, this typically involves a major rehab of the project that meets the conditions (total rehab costs, multiple construction seasons, etc.) associated with a major rehab project (EP 1130-2-500). Then much like optimizing the W/O Project, the projected reliability and total costs associated with all of the With Project scenario(s) are determined and computed. Again, the With-Project plan that maximizes NED benefits will be the optimal With-Project plan for the project from an economics-only standpoint. This will be compared to the optimized W/O Project to determine its economic merit. As noted previously, projects with potential life loss issues need special consideration as that may drive the decision making process.

5-7. <u>How Economic Models Use Engineering Reliability Information</u>.

a. Generally speaking, the minimal engineering input into the consequence analysis are the probabilities of unsatisfactory performance through the study period (time-dependent components) or single probability of unsatisfactory performance (time-independent components) as well as the required repair costs and service disruption time associated with the range of failures given the limit state occurs. The expected improvement in reliability following the repair for each level of failure is also provided by the Engineering Team. While the Economics Team will be evaluating more aggressive maintenance scenarios such as a major rehabilitation of the project prior to failure, any rehabilitation cost and expected disruption time are also given to the economists for their analysis. This will be used to determine the economic feasibility and

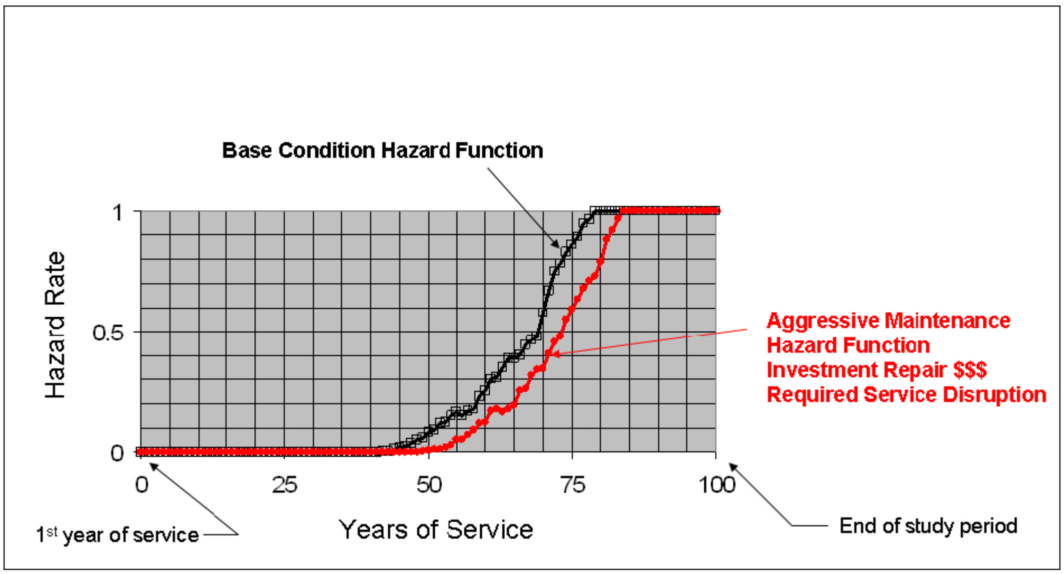


Figure 5-5. Hazard function comparison of W/O Project scenarios

optimal timing of the rehabilitation.

b. Depending upon the type of project and limit state being evaluated, characteristics of the type of failure may require the Engineering Team to provide how the structure might fail such as a breach of an embankment dam. The economists need to know how fast the breach might form to develop inputs such as warning time and other critical pieces of information for the loss of life and economic damages estimate. A similar situation may involve the structural failure of a dam spillway gate. The Engineering and Economics Teams must work together to establish the information to support the overall analysis.

CHAPTER 6
Risk and Reliability for USACE Studies

6-1. Introduction. The utilization of risk-based investment decision making is becoming more critical for USACE with an aging infrastructure coupled with tight budgets. A large backlog of critical maintenance remains unfunded, and this will continue to grow. Therefore, it is imperative that USACE make the wisest investment decisions possible with the best available data to draw upon when funding allocation decisions are required. This is one of the underlying goals of the dam safety portfolio risk assessment program (see Chapter 8 for further information regarding that effort). Risk-based methods are also now stretching into the funding allocation for "normal" maintenance as well in an attempt to use the limited funds on the highest return projects. One of the goals of developing this EC is to ensure that all districts across USACE are using the same analysis methods so similar project studies can be compared with one another based upon their economic merit and/or potential for life loss estimates. Previously there has not been a single source document for engineers conducting reliability analysis to use. It is hoped that this EC will go a long way in providing a solution to this problem by not only covering the engineering aspects, but also how it integrates with economic and life loss modeling.

6-2. Major Rehabilitation Evaluations and Other Risk-Based Applications.

a. The likely situation that will first expose a USACE engineer to conducting risk and reliability analysis will be for a Major Rehabilitation Evaluation of an existing project. The guidance for the major rehab program requires reliability analysis to be carried out on the major infrastructure of the project under study. The current guidance for the major rehab program is covered by Engineering Pamphlet (EP) 1130-2-500. This EP covers a lot of material beyond the major rehabilitation evaluation processes and procedures. However, Chapter 3 and Appendices B through H of this document cover major rehab evaluation requirements. Much of the engineering reliability aspects of EP 1130-2-500 were recently upgraded to be consistent with this EC. This EC is currently not referenced in EP 1130-2-500 because since it is an EC, it has a temporary life of 2 years. This will allow for initial field use and then the ability to make changes before it becomes an Engineering Manual (EM) after appropriate modifications are included.

b. EP 1130-2-500 was originally put out as guidance before computing processing speeds for individual PCs were not adequate for conducting Monte Carlo simulation analyses. In addition, commercial Monte Carlo simulation software programs were not readily available on the market. Therefore, many of the examples in EP 1130-2-500 focused on Reliability Index Methodology, which requires computations of a few different variations in key input values. This method is applicable for select situations, but it is not applicable for time-dependent reliability problems. It is a "snapshot in time" type of analysis. Other issues such as correlation of random variable and nonlinearity of the response make the use of Reliability Index Methods a good approach for only select situations. This is covered in more detail in Chapter 3 of this EC.

c. The requirements of Major Rehabilitation Evaluations are covered in EP 1130-2-500, but a few considerations are added here for clarity and reinforcement. When conducting a major rehabilitation evaluation is being conducted, the following steps should be followed to ensure that most critical elements of the analysis are captured in the study. Additional steps may be required based upon the characteristics of the study, but this provides a good starting point for consideration. This same set of general steps can be followed for other risk-based applications such as systems studies and new project evaluations based upon deterioration of the existing condition.

(1) Project Layout and Overview

(2) Problem Identification and Historical Operation Perspective

(3) Selection of Future Operating Scenarios for Major Rehab Evaluation

(a) Without Project (Base Condition, Advance Maintenance)

(b) With Project (Major Rehabilitation)

(4) Development of Full List of Operating Components

(5) Development of Criticality Ranking System for Operating Components

(6) "Short List" of Most Critical Operating Components from Ranking System

(7) Reliability Analyses for Most Critical Components for Each Scenario

(8) Event Tree Development for Most Critical Components for Each Scenario

(9) Development of Consequence Matrix/Schedule for Each Scenario

(a) Navigation Projects – repair cost and closure schedule projections through the study period, do these change for each scenario? For example, what costs and service disruptions are required for improved reliability associated with the Advance Maintenance scenario?

(b) Multipurpose Projects – repair cost and service disruptions projections for hydropower generation, loss of flood control capacity, etc.

(10) Economic/Life Safety Model Results for Reliability Analyses

(11) Summary of Results

d. Many of these steps for an individual component can be seen in the examples in Appendix B. Appendix B does not provide detailed information regarding Step 9 because of format and space constraints, but each of these individual examples required a consequence matrix for each scenario evaluated.

e. Some recent major rehabilitation evaluations have been done within USACE that use the proper methodology that can provide a good guide when starting this process. A partial list of these is provided as follows for reference along with the emphasis of the main components covered by the reliability analysis within the study. In addition, other recent risk-based studies as provided in the list provide additional examples to follow as they relate to recent successful reliability analyses applications within USACE.

(1) Markland Locks Major Rehab (CELRL)

(a) Horizontally Framed, Welded Miter Gates

(b) Horizontally Framed, Reverse Tainter Gate Culvert Valves

(2) John T. Myers Navigation Dam Major Rehab (CELRL)

(a) Scouring of Concrete Stilling Basin

(b) Stability of Dam Piers

(c) Mechanical and Electrical System for Dam

(3) Lower Monumental Lock Major Rehab (CENWW)

(a) Operating Machinery for Lock

(4) Ohio River Mainstem System Study (CELRD)

(a) 19 Lock Project Sites, 38 Lock Chambers

(b) Hydraulic Steel Structures (Miter Gates, Culvert Valves)

(c) Lock Operating Machinery

(d) Lock Electrical System

(e) Lock Wall Monolith Stability

(f) Approach Wall Stability

(5) Great Lakes and St. Lawrence Seaway System Study (CELRD)

(a) 15 Lock Project Sites, Multiple Bridges, Tunnels, etc…

(b) Bascule Bridge Roller Track Cracking

(c) Miter Gates

(d) Tunnel Structure Deterioration

(e) Alkali Aggregate Reaction of Mass Concrete

(f) Operating Machinery for Locks

(6) Chickamauga Lock Replacement Study (CELRN)

(a) Alkali Aggregate Reaction of Mass Concrete

(b) Riveted, Arched Miter Gates

(c) Anchored Lock Wall Stability

f. Systems studies refer to an inclusive study of multiple projects that are related to one another. Recent examples have been developed for navigation systems along major waterways. There has to be an economic model capable of handling the systematic application in terms of how project performance at one location relates to the performance at another location. A long levee system with multiple reaches and structures would be similar to a "systematic" evaluation where failures of various features could affect other areas downstream.

CHAPTER 7
Risk and Reliability Issues for Navigation Locks and Dams

7-1. An Overview of USACE Navigation Projects.

a. Constructing, operating, and maintaining the inventory of USACE navigation infrastructure is an enormous undertaking in terms of manpower and funding needs. USACE is responsible for maintaining over 12,000 miles of inland waterways to a minimum navigable depth of 9 ft. As a part of this inland navigation system, USACE has an inventory of approximately 192 navigation projects with 238 lock chambers (some projects have multiple lock chambers). These projects carry over 630 million tons of commodities on an annual basis. A map of the inland navigation system within the United States is shown in Figure 7-1. The majority of commodities shipped on the inland navigation system are primarily bulk in nature and include such goods as coal, petroleum, crude, grain, and aggregates. Many of the navigation projects also provide additional benefits beyond navigation such as hydropower, recreation, and water intakes for municipalities, businesses, and agricultural purposes.

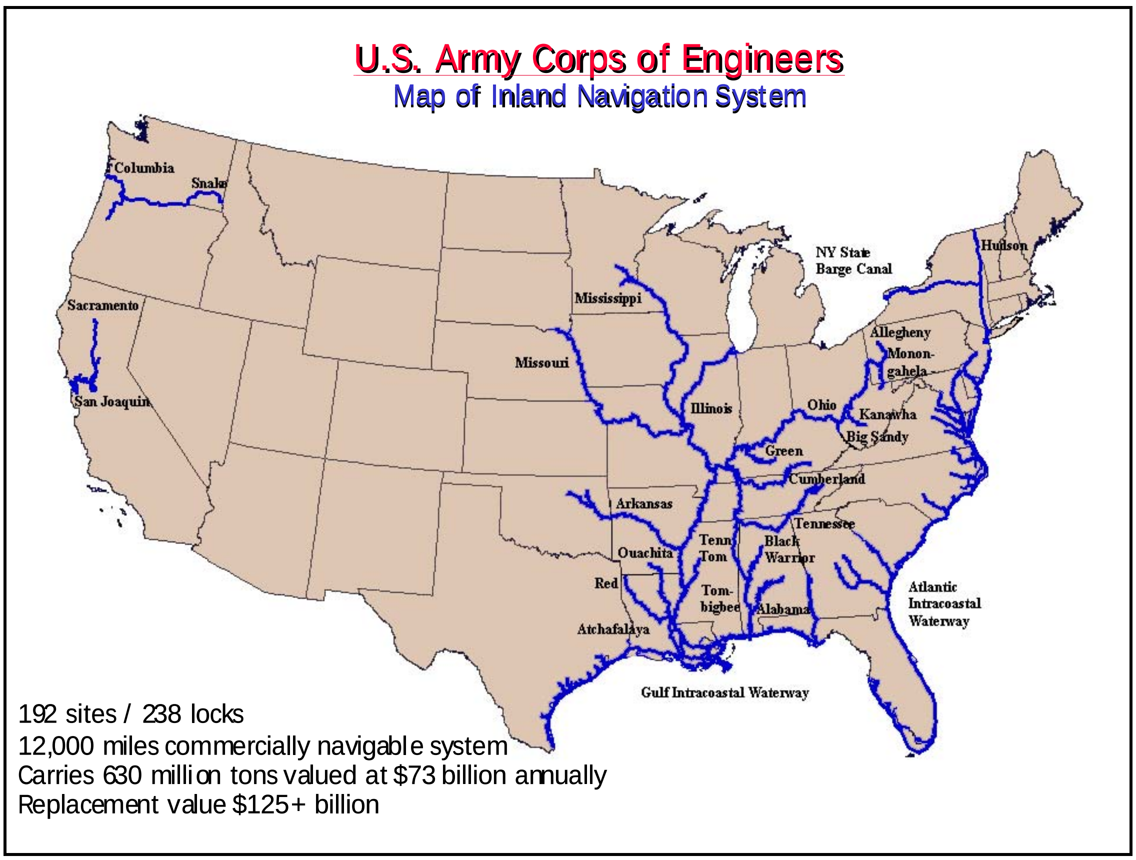


Figure 7-1. Map of the inland navigation system of the United States

b. USACE is also responsible for deep-draft navigation within United States waters. This includes projects such as the Soo Locks, which link Lake Superior to the remainder of the Great Lakes and St. Lawrence Seaway. In addition, USACE is responsible for maintaining approximately 300 commercial ports and more than 600 smaller harbors for smaller vessels and recreational craft.

c. One of the major issues facing USACE is its aging infrastructure including the system of navigation projects along the waterways shown in Figure 7-1. At the time this EC was published, the median age of all 238 lock chambers in the USACE inventory was 52 years, which is already beyond what is traditionally thought of as the original intended design life of 50 years. By the year 2015, approximately 60 percent of all navigation locks under the jurisdiction of USACE will exceed the 50-year threshold.

d. As with any aging infrastructure, increased funding is usually required to keep a project safely operating in an efficient manner. This usually requires additional maintenance funding. Recent history indicates that there are not enough funds to match the needs of the system. Currently over *$1 billion in critical maintenance backlog items* for USACE civil works infrastructure remains unfunded. This is up from roughly $200 million in 1998, an increase of over 400 percent in just 6 years. In addition to increasing maintenance needs with the inland navigation system, multiple recent emergency repairs associated with major operating component failures have caused projects either to shut down to users completely or cause major delay costs. It is important to note that systemwide lock closure time has doubled since 1990, another strong indication that aging infrastructure is beginning to affect project performance dramatically. Examples of these events are listed in Table 7-1, and this is just a small sample of the closures related to component failure across the inland system. It is evident that USACE needs to be able to prioritize the available funds that are provided so risks can be reduced in the best manner. That is the primary purpose of integrating risk and reliability into the analysis of existing lock and dam projects.

Table 7-1. Recent Inland Navigation Project Component Failures

Project	District	River System	Failure
John Day Lock	Portland	Columbia River	2002 gate failure, emergency dam repairs
McAlpine Lock	Louisville	Ohio River	2004 closure for emergency gate repairs
Greenup Lock	Huntington	Ohio River	52-day closure for gate repairs
L&D 27	St. Louis	Mississippi River	2004 emergency gate repairs
Smithland Lock	Louisville	Ohio River	1998 failures of culvert valves
Markland Lock	Louisville	Ohio River	Continual emergency gate repairs since 1994
Hannibal Lock	Pittsburgh	Ohio River	Valve and gate failures
Emsworth Dam	Pittsburgh	Ohio River	Dam gate failures, scour of dam apron

7-2. Selection of Critical Components for Reliability Analysis for Navigation Projects.

a. One of the first things that the Engineering Team must determine when conducting a risk-based evaluation of the existing infrastructure for any civil works project is the most important components that need to be evaluated through reliability analysis. These would most likely include the components that are having reliability issues, but components should also be considered that currently are not causing problems but have potential for major consequences should they fail and whose performance could be an issue during the study period, which is usually 50 years into the future. Determining the appropriate components to evaluate takes a combination of engineering judgment and discussion among the overall PDT to determine the potential effects on schedule, budget, etc. One of the best ways to determine which components should be evaluated through reliability analysis is to develop a criticality ranking system for the entire list of project infrastructure components. For navigation projects this list usually includes things such as lock gates, lock walls, approach walls, filling/emptying valves, electrical/mechanical operating machinery, dam gates, and dam monoliths to name a few. A good example of such a ranking system was used for the Markland Lock Major Rehabilitation Study (CELRL) and is described in detail in Appendix B. The context of the study and how reliability analysis will be integrated must also be taken into account when determining what to model. Generally, the ratings assigned to the criticality system are subjectively applied by reaching consensus among the Engineering Team (and possibly the overall PDT), and then a "reasonable" threshold is selected to determine what components should be modeled through probabilistic means for a reliability analysis.

b. Another good example to follow when developing a criticality ranking system for navigation projects can be taken from the recently completed Great Lakes and St. Lawrence Seaway (GLSLS) Navigation Study (CELRD). This study was a joint Canadian/United States evaluation of the lock and dam infrastructure making up the GLSLS system, which includes 17 lock chambers (13 Canadian and 4 U.S.) and an assortment of other infrastructure such as moveable bridges, hydropower facilities, tunnels, weirs, and other components. One of the primary objectives of the GLSLS Study was to determine the optimum funding requirements for maintenance/rehabilitation dollars for the system on a project-by-project basis over the next 50 years while taking into account reliability of the infrastructure during that time frame. In support of this effort, the GLSLS Study Engineering Team developed a global list of 160 operating components across the system and then developed a numerical, weighted ranking system to determine what the most critical infrastructure was in the context of the study. This list was developed through a combination of site visits, review of project plans, and discussions with project personnel. The methodology for the criticality ranking system is defined in GLSLS engineering reports located at Inland Waterway Navigation Center (CELRH). The categories used for ranking the infrastructure components are listed in Table 7-2.

c. Each of the 160 components was rated by the multidisciplinary GLSLS Engineering Team in consultation with Operations personnel from each project site. Since the thrust of the GLSLS Study was primarily navigation related and concentrated on future maintenance needs, two categories were weighted more highly ("Likelihood of Future Problems" and "Impact to Navigation if Fails") than the other categories. Once the categories were all appropriately rated and weighted properly, they were added together to get a combined criticality ranking. For the purposes of the GLSLS Study, the lower the aggregate value of the combined ratings, the more critical the component was with respect to the study. Lower ranked components were carried forward for detailed reliability analyses, while others were evaluated by other nonprobabilistic means.

Table 7-2. Ranking Criteria for Infrastructure Components

Category	Ranking Criteria
Recently Upgraded	Yes/No (yes, meant it could be screened out)
Component Redundancy	1 to 5 (lower value had less redundancy)
Current Condition	1 to 3 (lower value was poorer condition)
Likelihood of Future Problems	1 to 5 (lower value reflects expected problems)
Impact to Navigation if Fails	1 to 5 (lower value more significant impact)
Other Impacts if Fails (Rail, Car)	1 to 5 (lower value more impacts)

7-3. Economic Modeling Considerations for Navigation Projects. The careful and consistent collaboration of the Engineering and Economics Teams from the onset of the evaluation cannot be overemphasized. Both teams working with the overall PDT need to ensure everyone is "on the same sheet of music" and working together toward the same analysis goals. In the past, inconsistencies and inappropriate linkage of engineering and economic aspects of studies have led to the downfall of many USACE evaluations. It is always good to have team members from each team as an integral part of the other, for example, having Economic Team members sit in on the reliability modeling decision process meetings and vice versa, can go a long way to ensuring a smooth and consistent transition between the work tasks and information being developed by both teams. The Project Manager should help coordinate this effort when possible to be aware of the overall direction.

a. USACE evaluation procedures. There is clear guidance for evaluating navigation projects when determining future needs as they relate to infrastructure investments. The procedures as outlined in "Economic and Environmental Principles and Guidelines for Water and Related Land Resource Implementation Studies" must be followed. These include the following steps when conducting an evaluation of inland navigation projects:

(1) Identify problems and opportunities

(2) Inventory and forecast conditions

(3) Formulate alternative plans

(4) Evaluate alternative plans

(5) Compare alternative plans

(6) Select plan

These steps include formulating plans for both the Without Project Condition (WOPC) and With-Project Condition (WPC). Inventorying and forecasting conditions in Step 2 involve establishing the current conditions as they relate to such economic issues as traffic levels and transportation rates. The engineering aspects of such concerns as maintenance costs, frequency of repairs, and the reliability associated with these would also be required. As alternative plans are developed and evaluated, the changes in investment costs, service disruption times, reliability of the components, and any other issues that are dependent upon the maintenance scenario need to be evaluated. This is true when establishing the Base Condition and any other WOPC alternative. The same type issues must be addressed for the With Project alternatives as well in order to accurately assess and compare projects consistently. Generally speaking, the selection of the plan for navigation projects will be the plan that maximizes national economic development (NED) benefits.

b. Economic modeling inputs for navigation projects.

(1) The modeling inputs for the consequence side of the analysis are functions of the project itself. Most navigation dams within USACE are designed as flood control projects. Thus, when inflow increases, the dam gates are lifted and flow is allowed to pass through the project. These are referred to as run-of-the-river projects. The dams serve to retain pool during low to moderate inflow periods so a navigation pool can be maintained. All the locks and dams on the main portion of the Ohio River (20 projects, 40 lock chambers) and Mississippi River (30 projects, 33 lock chambers) are of this nature. For these projects, the primary benefits are related to navigation, but some additional economic issues are associated with retaining the pool for recreation and water intakes for power plants, municipalities, and industry. For navigation projects that are combined within a flood-control/hydropower project, additional project benefits are gained from these aspects and need to be considered in the analysis. These are referred to as high lift, combination projects.

(2) For either type of navigation project, one of the most important economic inputs is the level of navigation traffic that uses the facility. The level of usage projected for the future has a significant impact on the results of the study and has become a topic of controversy during recent USACE navigation studies. In response to this controversy one of the recent changes in projecting future traffic is the use of a range of future traffic projections. This lends itself well to risk analysis in that it can be integrated into the engineering reliability analysis to determine the effects of traffic on the project/system. This was recently done on the ORMSS. Five future traffic projections were predicted for the ORMSS. Every one of these projections was related to compliance with clean air regulations since the primary commodity on the Ohio River is coal for use in coal-fired power plants. The five traffic projections were developed to attempt to cover the reasonable range of future traffic on the Ohio River.

(3) Another major input to the economic analysis for navigation projects is the capacity of the lock chamber. This is modeled through a lock chamber tonnage-transit curve that account for congestion and delay. The less reliable the lock chamber, the less capacity it will have in terms of its ability to transit vessels. This is because as lock infrastructure deteriorates, it causes the chamber to be shut down for maintenance more frequently and for longer periods, increasing

transit costs to the user. Recent instances of these types of closures are described in paragraph 7-1.

(4) Another critical piece of information that economists must develop for navigation studies is transportation rates for the various commodities that are shipped through the lock facility. The transportation rates are combined with different forecast movements to form the demand side of the model. This reflects the willingness to pay by the users of the system. Vessel operating/cost parameters, lockage policies, and the river network itself represent other pieces of information that economists must develop for navigation studies. For combination projects, the benefits derived from hydropower and flood control must be gathered so that any scenario in which these project benefits would cease needs also to be integrated into the overall analysis.

(5) The engineering reliability failure rates and consequence event trees for the critical infrastructure being evaluated are developed outside the economic analysis to serve as input into the economics analysis. Component-specific hazard rates and event trees are developed to be consistent with each scenario being evaluated in the study. The process of developing hazard rates is covered in Chapter 3 in this EC and event tree chapter which is a work-in-progress. However, the importance of smoothly integrating risk-based engineering input into the economic systems analysis is stressed. This integration requires close and frequent coordination between the engineering team doing the reliability analyses and the economic team responsible for using the information in the study.

7-4. Developing Reliability Modeling for Navigation Lock and Dams. The principles and guidelines presented in Chapters 2 through 5 are generally applicable to components and features at navigation projects. This section discusses the basic issues, data needs, limit state selection and computing the probability of unsatisfactory performance that can be used to develop reliability models for navigation projects. This includes models for concrete hydraulic structures (time independent and time-dependent), hydraulic steel structures, and mechanical and electrical equipment. Appendix B presents detailed explanations and examples of reliability analyses for navigation projects.

7-5. Time-Independent Reliability for Concrete Hydraulic Structures. Several risk and reliability issues must be considered when evaluating mass concrete gravity structures on navigation projects. Foremost among these is the design of the structure, its intended function, and the potential for performance-related issues for the load cases to be evaluated in the study. The condition of most mass concrete structures on USACE navigation projects does not change significantly with time. This is not always the case as there are situations where the environment (heavy freeze/thaw), chemical reaction within the concrete, and other conditions cause the mass concrete to degrade with time. Also, most of these structures are founded on competent rock whose properties do not change with time. Therefore, it is safe to say that most mass concrete reliability-based evaluations will be independent of time with respect to deterioration of the structure.

a. Basic Issues.

(1) Independence of time such as “return period” types of loads, such as floods and earthquakes, are not considered for these structures. It may be necessary to consider these types of loads depending upon the analysis. It simply means that the concrete structure itself does not degrade with time for the limit state being evaluated. For example, most major rehabilitation studies do not consider these types of loads under the current guidance, opting to evaluate only for normal and maintenance types of load cases in order to evaluate the economics based upon nonextreme events. However, dam safety risk assessments consider these loads when evaluating the overall risk associated with the navigation dam. Under either case, the condition of the mass concrete will most likely not change, but different reliability analyses must be carried out for all the load cases (normal, maintenance, seismic, flood, barge/ship impact, etc.) to be considered in the analysis. The probabilities calculated for each of these load cases are then multiplied by the frequency of the event to determine the probability side of the risk equation. Thus, for a dam safety risk assessment, the probability of failure for the “normal” load case may be 0 percent because of original safety factors in the design; however, that same monolith may have a 10 percent probability of failure for the seismic load case. In both cases, the probabilities do not change with time, but the frequency of the event needs to be added into the analysis to determine the actual chance of getting that load case within the period of study.

(2) For the situation in which the mass concrete gravity structure does not change with time, the basic needs for the reliability analysis of these structures are dependent upon the load cases and potential failure modes being evaluated. For a stability analysis, this comes down to the driving loads and resisting forces at work to establish equilibrium for the structure for failure modes such as sliding, overturning, and bearing capacity. A free-body diagram of the global stability analysis should be drawn out to ensure all loads are properly being considered. An example of a free-body diagram for a navigation mass concrete gravity structure is shown in Figure 7-2.

(3) Examples of the driving and resisting loads working on different types of mass concrete structures on navigation projects used in the stability analysis include such loads as earth pressures (at rest, active, passive), hydrostatic (driving and resisting), uplift, hydrodynamic (seismic only), and concrete weights. Mass concrete structures such as approach walls are subject to other large forces such as barge/ship impact and hawser pull forces. The consideration of the weight of dam gates and their transfer of hydrostatic loads need to be considered for dam pier stability. As can be seen, the loads are highly dependent upon the concrete structure being evaluated and that is why a free-body diagram can help establish a clear picture of the loads acting on the structure to be considered for the reliability analysis. Once the loads acting on the structure are clearly identified, the next step is to determine which ones should be considered random variable and which ones can be considered as constants in the analysis.

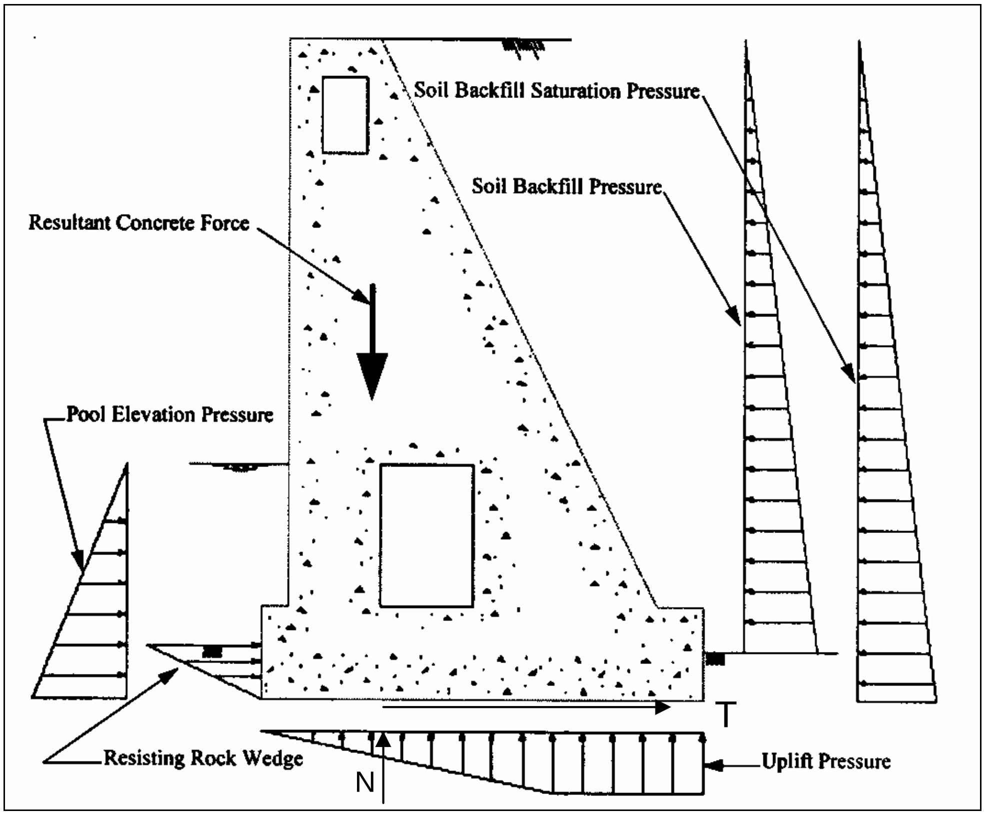


Figure 7-2. Typical free-body diagram for navigation mass concrete gravity structure

b. Formulation of limit states for concrete hydraulic structures. The three performance modes that describe stability are sliding, foundation bearing, and resultant location.

(1) The reliability analyses should use the performance function capacity *C*, divided by demand *D*. For sliding analyses, capacity will be the resistance along a sliding plane beneath the structure, plus the resistance of earth and rock in which the structure is embedded. Demand will be the applied loads on the structure tending to cause sliding. Sliding may occur at the base of a structure, but experience has shown that sliding along weak seams in the rock is more likely. For sliding along the base, the shear strength used in the analyses is the lesser of the contact strength of concrete on rock and the rock shear strength as defined by an angle of internal friction φ and cohesion *c*. Sliding may or may not involve a passive wedge, and will depend on embedment of the structure in rock and orientation of weak seams within the foundation. Where a passive wedge is considered, the magnitude of the passive force or factors affecting the force must be treated as random variables.

(2) For foundation bearing, capacity will be taken as the bearing capacity of the foundation. Demand will be the maximum bearing pressure at the toe of the structure. Foundation bearing failure for structures founded on rock is unlikely. Nevertheless, it is prudent that bearing capacity be evaluated as a performance mode. An appropriate method based on Corps criteria should be chosen to meet the condition of the foundation rock. Meyerhof's bearing capacity factors, including reductions for inclined loads, are used to assess the reliability of

gravity structures for the performance mode of foundation bearing. These bearing capacity factors were developed for homogeneous, isotropic soils but may be used for rock foundations with closed near-vertical joints and other conditions that approximate homogeneous isotropic conditions. In applying bearing capacity factors, it may be necessary to use the mean values of rock strength as constants. Bearing capacity factor equations are sensitive to large variations of φ, which result from the combination μ_φ plus or minus σ_φ.

(3) For resultant location, the ratio of capacity divided by demand will be taken as a function of the base dimensions and eccentricities of the resultant.

(4) For two-dimensional stability the resultant location is the performance mode selected herein to replace overturning analyses. Overturning is unlikely as a pure mode of failure, as foundation bearing and/or sliding failure would occur before the resultant reached the toe. In practice, the location of the resultant on the base is used to determine the percentage of the base in compression which is then used as a measure of stability. Reliability analyses, however, require that stability be expressed in terms of capacity divided by demand, i.e., a performance function. This ratio can conveniently be represented by the equation

$$\frac{C}{D} = \frac{B}{B - 2X_R}$$

(5) The derivation of this equation follows:

If moment M about the centroid of the base is taken as demand and Rv× B/2 is taken as capacity, the equation for capacity divided by demand can be derived as follows:

$$\frac{C}{D} = \frac{R_V \times \frac{B}{2}}{M} = \frac{R_V \times \frac{B}{2}}{R_V\left(\frac{B}{2} - X_R\right)} = \frac{B}{B - 2X_R}$$

where

M = summation of moments about centroid of base
R_V = resultant of vertical forces
R_H = resultant of horizontal forces
X_R = distance from toe to N
X_R = B/2 - e
e = eccentricity of resultant forces about centroid of the base
e=M/ R_V

(6) The limit states for resultant location analysis are defined in terms of the capacity-demand ratio as shown in Table 7-3.

Table 7-3. Capacity/demand limit states for resultant location

Load Category	Resultant Location	Capacity/Demand
Usual	Within the middle third	C/D >3
Unusual	Within the middle half	2< C/D >3
Extreme	Within the base	1<C/D/2

c. Three-dimensional stability. Structures such as dam piers and miter gate monoliths are normally analyzed for stability about two axes. The equation for capacity divided by demand for problems of three-dimensional stability can be expressed as follows:

$$\frac{C_{3D}}{D_{3D}} = \frac{B}{B - 2X_R + B\left(1 - \frac{2Z_R}{L}\right)}$$

where the eccentricities and base dimensions are as shown in Figure 7-4.

(1) The data needed for a reliability analysis of a concrete gravity structure on a navigation project relate to the forces at work that are generating the failure mode under evaluation. For a global stability analysis of a gravity monolith section, the data needed for the model center around such characteristics as geometry of the section, unit weights of soils, water levels, and foundation characteristics. The freebody diagram and resultant forces for a gravity monolith section are shown in Figure 7-2. Approach walls require information on these characteristics, but also information about the impact and line pull loads of ships/barges. Further consideration of all these loads is required to determine whether they should be random variables or constants within the analysis. Most of the time, foundation and soil properties, impact/line pull loads and water levels are considered random variables and their underlying distribution needs to be determined. The unit weight of water for hydrostatic forces and geometry of the concrete section would be good examples of constants in a reliability analysis of concrete monolith stability. Tables 7-4 and 7-5 are examples of "typical" random variables and constants, respectively, from a recent USACE major rehabilitation evaluation of the reliability analysis of global stability of lock wall monoliths on the project. As listed in Table 7-4, most of the random variables in the analysis related to soil and rock properties. The engineer conducting the reliability analysis will have to work closely with the geotechnical engineers and geologists familiar with the site to determine the appropriate distribution, ranges, and other critical information for the stability analysis. This is especially true if there is no recent boring data that can be used for the analysis. **Remember, all the values for the rock and soil characteristics should be what is expected in the field and <u>NOT</u> have any type of safety factor built into them**. This must be stressed to all those who are providing input information into the analysis. This is where reliability analysis differs from traditional design.

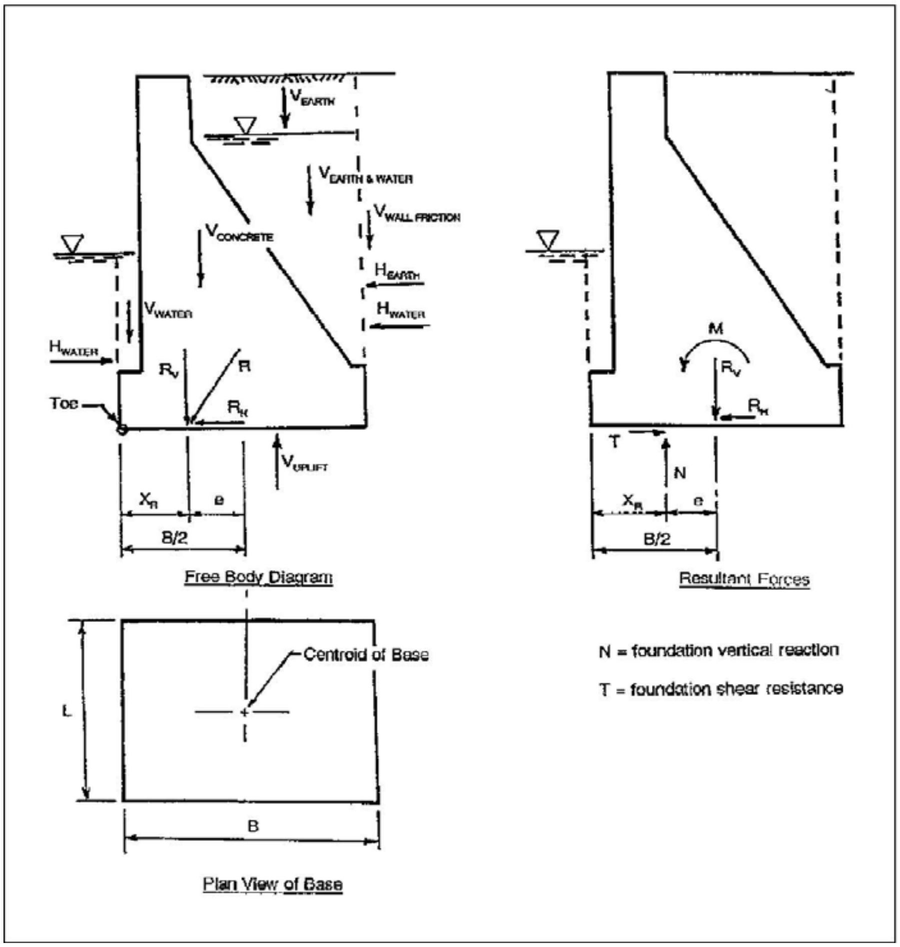


Figure 7-3. Resultant forces and freebody diagrams (from ETL 1110-2-321)

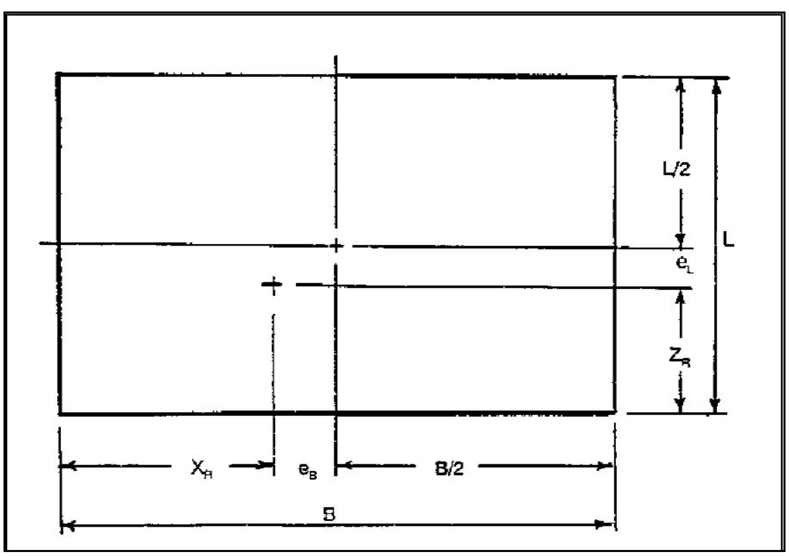


Figure 7-4. Plan view of base

(2) The constants shown in Table 7-5 are just that; they do not vary for any of the iterations. It is realized that some of the data shown in this table may have some small variation to it (such as the unit weight of concrete), but if it is minor and has negligible effect on the overall analysis, it can be assumed to be a constant for the stability analysis.

7-6. Limit State Selection.

a. Selecting an appropriate, well-defined limit state is one of the most critical steps of any reliability analysis. This is no different for mass concrete structures on navigation projects. A good general discussion of limit states in reliability modeling is provided in Section 2e . The limit state for the reliability analysis should not consider safety factors as part of the analysis unless the district goes into some repair or project restricted-use mode if a certain safety factor is not met. If this is the case, the consequence event tree must relate to the selected limit state for either the restricted use of the project or remediation actions and not the occurrence of a "failure" since that would be inconsistent with the limit state modeled in the reliability analysis. Consequence event trees are covered in detail in Section 6 .

b. Traditionally, mass concrete gravity structures are designed for global stability to meet or exceed certain safety factor criteria for various load cases. However, the threshold safety factor of 1.0 must be used with care in reliability analysis. Consideration must be given the potential amount of movement of the structure and what that might mean to the overall effect on project performance. For example, a small amount of sliding or rotation may cause at-rest earth pressure driving forces to reduce to active driving pressures, which could possibly stop movement. This small amount of movement required to lessen driving forces may adversely affect the overall performance of the project. It needs to be evaluated on a case-by-case basis. The important point here is that a traditional stability analysis with a safety factor less than 1.0 does not necessarily represent the limit state that should be evaluated for every gravity structure. Each analysis needs to consider the function of the component and how much movement would be required to adversely affect the overall performance of the project.

(1) Computing the probability of unsatisfactory performance.

(a) If the concrete structure does not degrade with time, the probability of unsatisfactory performance will be computed in the reliability analysis, and it will remain the same throughout the study period. Generally, this is handled in similar to a traditional stability analysis with input parameters for soil, rock, and water levels allowed to vary according to their underlying distribution. The probability of unsatisfactory performance is computed within the reliability model by taking the number of iterations where the limit state is exceeded divided by the total number iterations done for the analysis. Thus, if 100 iterations were computed in a simulation of the seismic load case and 20 of the iterations resulted in the limit state being exceeded, then the probability of unsatisfactory performance would be 20 percent for the seismic load case. This would then be multiplied by the event frequency to determine the overall probability of this occurring. Careful attention must be paid to how the iterations that exceed a limit state are accumulated when analyzing multiple limit states (e.g., sliding, overturning, bearing) within the

Table 7-4. Example Random Variables for Gravity Monolith Reliability Analysis

Variable	Mean	Standard Deviation	Maximum	Minimum	Distribution	Units	Description
Soil:							
Mst Unit Wt	0.115	0.003	0.124	0.106	Normal	kcf	Driving soil, unit weight, moist
Sat. Unit Wt	0.125	0.004	0.137	0.113	Normal	kcf	Driving soil, unit weight, saturated
Phi, internal	33	2	38	30	Normal	deg	Driving soil, internal friction angle
Rock:							
Phi, sliding	38	4	45	35	Normal	deg	Rock, sliding friction angle
c, sliding	20	20	25	0	Normal	psi	Rock, sliding shear strength
Phi crossbed	47	4.5	57	37	Normal	deg	Rock, cross-bed friction angle
c, crossbed	75	25	100	50	Normal	psi	Rock, cross-bed shear strength
Sat Unit Wt	0.1672	0.002	0.1697	0.1660	Normal	kcf	Rock, saturated unit weight
BrgCapacity	2083.3	208.3	2430.6	1736.1	Normal	psi	Rock, ultimate bearing capacity
Lower Pool					CDF [1/,2/]	n/a	Lower Pool elevation
Hawser Pull	57.5	11.5	80.5	34.5	Normal	kip	Hawser pull force, normal to face

[1/] Cumulative Density Function established for Lower Pool is used.
[2/] For river wall R-48, the maintenance condition, the maximum main chamber is flooded when the lower pool elevation exceeds EL 431.08.
n/a - Not applicable.

Table 7-5. Example Constants for Gravity Monolith Reliability Analysis

Constant	Value	Units	Description
Concentrated Unit Weight	0.1475	kcf	Concrete, unit weight
Water Unit Weight	0.0625	kcf	Water, unit weight
Saturation Level	455.0	ft	Water saturation level in backfill
Upper Pool	455.0[a]	ft	Upper pool elevation

[a]When the elevation of the lower pool is greater than that of the upper pool minus 1 ft, the elevation of the upper pool is equal to the elevation of the lower pool plus 1 ft.

same global analysis. The randomness of the input parameters may mean that a single iteration could exceed a limit state by more than one mode. However, this should be counted as only one event within the total number of iterations in order not to overestimate the probability of unsatisfactory performance.

(b) An elevation view of the concrete monolith for the simplified example is shown in Figure 7-5. The simple load case will determine the probability of unsatisfactory performance for two load cases: line pull (hawser) and seismic. This analysis for each case will check for stability against overturning and sliding. For the purposes of this simplified example only, a "failure" is encountered if the factor of safety for _either_ overturning or sliding falls below a value

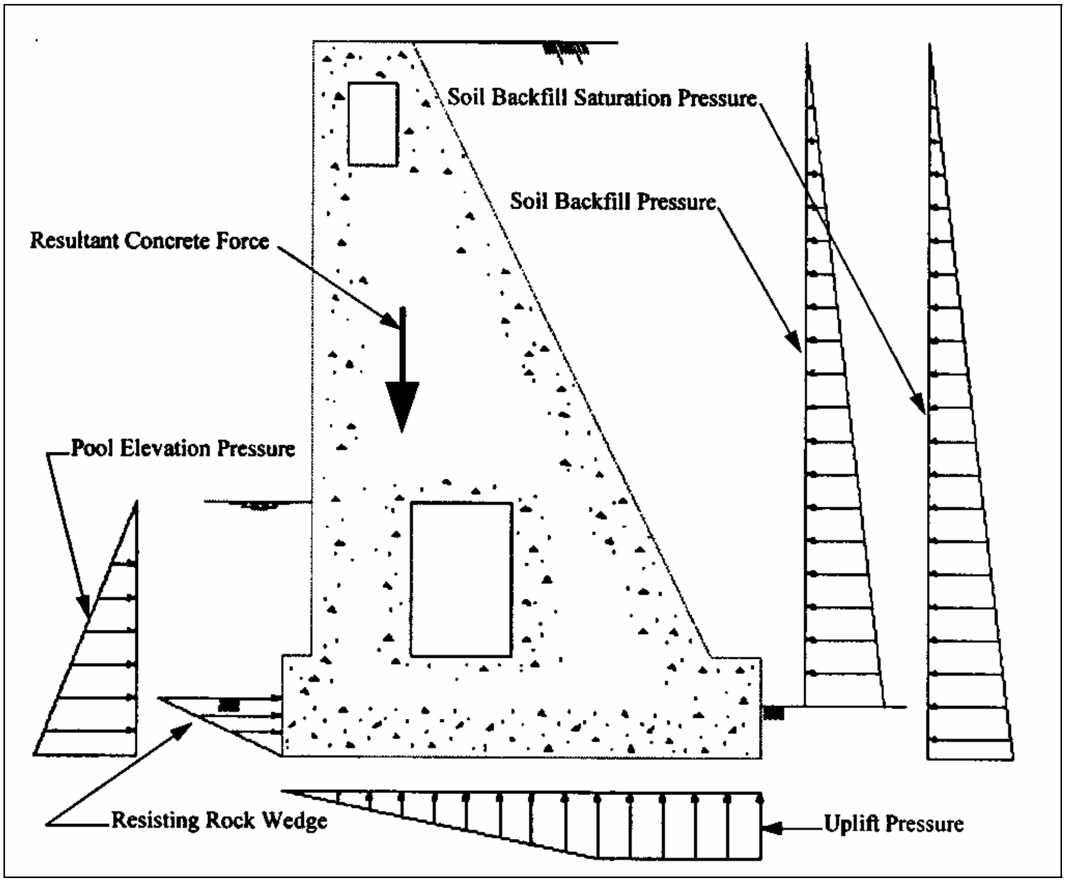


Figure 7-5. Typical free-body diagram for concrete gravity lock wall monolith

of 1. Again, it cannot be overstated that this may not reflect a “real” type of failure in an actual USACE study. Careful consideration must be given to what amount of movement or misalignment would induce significant consequences.

7-7. Time-Dependent Reliability Modeling Of Concrete Hydraulic Structures.

a. Basic issues.

(1) While most mass concrete structures on navigation structures do not degrade with time, there are situations that demand that degradation of the structure be considered. This would include a situation in which the concrete is expanding because of a chemical reaction called alkali aggregate reaction. This phenomenon causes the mass concrete to expand caused by a chemical reaction between the cement and aggregate in the concrete mix. Many times this problem does not surface until well after a project is constructed and in place. The expanding concrete can cause misalignment of critical operating equipment, mass cracking in the concrete, and a general weakening of the integrity of the supporting concrete in critical locations such as miter gate anchorages. When alkali aggregate reaction must be considered, the degradation of the structure should be considered a time-dependent evaluation. Instrumentation data of such

factors as misalignment, cracking patterns, and growth rates of the mass concrete, if it is available, can be very helpful when modeling this type of structure.

(2) Other types of mass concrete structures where degradation may need to be considered would be an anchored monolith where the corrosion and/or fatigue of the anchors may be an issue. Also, concrete scour of stilling basins, freeze/thaw applications on non-air-entrained concrete, and changing foundation conditions below a mass concrete structure may all be situations where the degradation of the structure would need to be considered in a time-dependent reliability analysis.

b. Data needs for time-dependent reliability analysis.

(1) The data needs for mass concrete gravity structures whose conditions degrade with time are somewhat different from those of the time-independent gravity structures. When alkali aggregate reaction is an issue that needs to be considered, stability is probably not going to be the critical issue that needs to be evaluated. It is most likely going to be some type of misalignment issue or deterioration of the internal support for embedded structures. This was the case from two recent USACE evaluations for the reliability analysis of mass concrete lock walls of navigation projects at Chickamauga Lock (CELRN) and the St. Lambert Lock on the St. Lawrence Seaway (GLSLS) (CELRD). The data required for this situation included the growth rate of the concrete caused by the chemical reaction, effectiveness of anchors/pins to help stabilize localized areas of the structure, and the compressive strength of the concrete itself. The Chickamauga example is detailed in Appendix B for reference.

(2) There can be cases where a stability analysis for mass concrete navigation structures can change through time. This is generally not the case unless the foundation or the structure itself physically changes. One recent example of this is the approach walls for one of the locks from the GLSLS Study. Ships with large bow thrusters have caused a gradual undermining of the foundation below the approach wall, degrading the stability of the structure with time. For this situation, the same load characteristics of the soil, rock, and water were required as they would have been for a time-independent analysis; but additional information regarding the erodibility of the foundation material was also required. The time-dependent aspects of this analysis had to account for the changing foundation conditions as they affected the global stability of the structure through time.

(3) The presence of anchors in a mass concrete structure may also warrant time considerations to evaluate their long-term performance. If it is expected that the anchors may degrade significantly with time caused by corrosion, then a time-dependent reliability analysis should be carried out for these structures. The properties of the anchors, including their variability, would be an additional data need for this analysis.

(4) Scouring of a concrete stilling basin also represents a time-dependent analysis for mass concrete structures on dams. Characteristics that affect the performance of scoured stilling basins generally include flow parameters, presence of debris, geometry of the structure/flow patterns, and concrete properties. The reliability analysis for the scouring of the J. T. Myers Navigation Dam (CELRL) is covered in detail in Appendix B.

c. Limit state selection.

(1) The limit state needs to relate to the function of the component and what would constitute a major economic consequence in terms of service disruption and/or repair cost. For the Chickamauga Project, the limit state related to continued concrete strain around the embedded miter gate anchorage was determined to be the best representation of the limit state where adverse impacts to the project operation could be reasonably be expected.

(2) For the GLSLS Study reliability evaluation of the St. Lambert Lock (CELRD), the limit state related to the inability to use both the miter gates and bulkheads because of misalignment of the recesses caused by the expansive concrete. Both of these represent "significant" limit states and not ones that really generate no serious economic consequence, such as just requiring continued monitoring or installing instrumentation to monitor. The time-dependent aspects of the eroding foundation below the approach wall on the GLSLS Navigation Study had a stability-based limit state controlled by damaged foundation. A traditional stability analysis was used to evaluate this structure for sliding, overturning, and bearing capacity. Each of these limit states could control with the damaged foundation so the model was set up to evaluate each failure mode.

7-8. Reliability Modeling for Hydraulic Steel Structures on Navigation Projects. Hydraulic steel structures for navigation projects represent some of the most critical infrastructure across the USACE inventory of locks and dams. They generally require the most maintenance and degrade through time because of the corrosion caused by operating in water and their cyclical use. Examples of critical hydraulic steel structures on navigation projects include lock gates of various types, culvert valves, stop logs or bulkheads, and dam gates of various types.

a. Basic issues. If a life cycle analysis is required, most, if not all, reliability analysis cases involving hydraulic steel structures will involve a time-dependent degradation aspect. The two primary drivers of degradation associated with hydraulic steel structures are usually fatigue and/or corrosion. Special attention must be paid to those maintenance situations that can affect fatigue and corrosion, primarily painting and historical repairs. These types of actions need to be accounted for in the analysis to ensure that the proper repair history is included and the model itself is calibrated properly when considering past actions. The redundancy aspects of these structures are also very important to the analysis when setting up the reliability analysis.

b. Data needs for reliability analysis.

(1) The data needs for the reliability analysis of navigation hydraulic steel structures include those needed to analyze the current load-carrying capacity of the structure. Therefore, as-built plans detailing the structure and various connections, as well as the material properties of the structure, are required. In addition, the modeler needs to know the repair and paint history on the structure. Historical painting details should be limited to either complete sand blasting and repainting of the entire structure or the detailed area under consideration. Field measurements from instrumentation or physical damages can also help establish the current condition of the structure. If there is existing damage on the structure, such as cracking, the location and extent of the cracks should be determined prior to the modeling effort.

(2) Besides the physical information about the structure itself, a multitude of other data requirements are necessary to do a detailed time-dependent reliability analysis. For components whose reliability is in part driven by cyclical use, the historic and projected future operating cycles are needed. A historical ratio of vessels transiting the system to number of cyclical operations might need to be established to determine what operating cycles will be anticipated based upon vessel forecasts. This is what was required for the ORMSS. The possible effects of corrosion need to be considered unless it is assumed that a protective paint system will be in place throughout the life of the structure (in which case the costs and timing of the painting investments also need to be included). The variability in the material properties will have to be established and defined. This is usually readily available for steel structures. Another important data requirement for navigation structures where the pool differential fluctuates is the percentage of time (or cycles) that lockages occur. This is important for the fatigue analysis since the initiation and growth of cracks is dictated by the number of operating cycles in various stress ranges. As you can see from the data, Markland has a maximum pool differential of approximately 35 feet. However, if the simplifying assumption was made that all operating cycles occurred at this "design" pool differential, it would grossly overestimate the fatigue damage to the structure. This information is readily available from the Lock Performance Monitoring System (LPMS) database and can be manipulated into this form with minimal effort.

(3) Each analysis will have its own data needs. It is important to collect the best data available for the analysis since this will serve as the backbone of the reliability analysis and subsequent estimate of failure probability. A simplified example of computing the hazard function for a beam is provided in d below, but the data needs associated with realistic examples (miter gates from the Markland Major Rehabilitation Study and miter gates from GLSLS Navigation Study) from recent USACE projects is provided in Appendix B for reference.

c. Limit state selection.

(1) The limit state selection ties back to consequences and consistency with the various scenarios being evaluated. The limit state has to be defined by some level of significant consequence. This consequence has to be severe enough to make a difference in the economic analysis. This is different from traditional design where safety factors and hand calculations can be used to size beam members. Safety factors are not to be included in reliability analyses unless they trigger some type of consequence in terms of service disruption to the project and a repair action. Continued monitoring by itself will not lend itself to any economic significance. Therefore, the limit state needs to be something that is realistic and tied to field performance if at all possible.

(2) A great lesson can be taken from the reliability analysis of the miter gates under the ORMSS. The ORMSS Engineering Team knew from recent field experience that there was significant cracking damage to the miter gates on several Ohio River projects. When the original reliability analysis was carried out by the ORMSS Engineering Team, it focused on traditional fatigue and corrosion analysis associated with the main load-bearing girders based upon midspan flexure. It was quickly realized from the initial modeling analysis that the results were not a good reflection of what was occurring in the field. The original analysis indicated that there should be no fatigue-induced cracks until well after 2050. The team knew that the main lock

chamber at several Ohio River sites had been closed for fatigue-related cracks to the gates. Therefore, a more detailed analysis was undertaken with the assistance of an A/E firm specializing in nonlinear finite element analysis to determine the cause of the cracking. It was determined that it was due to a residual stress issue during original construction that locked in tensile stresses at welded connections near the pintle area. These tensile stresses were then reversed during lockages into high compressive forces associated with the pool differential. The gates were never designed for this situation, and this led to the cracking of the gates initiating at girder/stiffener connections and then growing through the webs of the girder. Once this situation was determined, the ORMSS Engineering Team redirected its reliability model to reflect it. When this phenomenon was accounted for in the reliability analysis, much more reasonable results were encountered in terms of failure rates. The limit state itself was not the formation of the cracks since the gates were still in operation, but had suffered significant damage (see Appendix B). The limit state was the growth of the cracks through consecutive girders causing the overall integrity of the gate itself to be in question. A single gate leaf was modeled, and the time-dependent reliability associated with this limit state was computed. Since all four gate leaves were in the same general condition, the reliability of all four leaves were put in series to determine the reliability of the series of main chamber gates at Markland through time. However, once a failure occurred in the economic life-cycle simulation, the event tree was set up that immediate temporary repairs were undertaken for the failed gate and then the other gate was repaired soon thereafter. Therefore, it was not necessary to track individual gate leaves in the economic analysis. This analysis and other examples of limit state selection are covered in detail in Appendix B.

7-9. Reliability Modeling of Mechanical and Electrical Equipment on Navigation Projects.

a. Background.

(1) Navigational lock and dam facilities are an important link in the nation's transportation system. Their mission is to maintain the navigable waterways and allow both cargo transport and recreational traffic between adjacent segments of the waterways. The mechanical and electrical components at these facilities function as systems to operate the various gates and valves. Breakdowns and poor performance of these systems can cause delays to navigation and adversely affect the overall national economy.

(2) Lock and dam major rehabilitation projects began being budgeted under the Construction, General, and Flood Control, Mississippi River and Tributaries appropriation account in FY 93. To qualify as major rehabilitation projects, the work activities must extend over two full construction seasons and the total required implementation costs must be greater than a certain minimum threshold. The threshold amounts are adjusted annually for inflation as published in the Annual Program and Budget Request. To successfully compete as new starts, major rehabilitation proposals must be supported by the same level of economic analysis as new water resource projects. Chapter 3 of ER 1130-2-500 establishes policy for major rehabilitation at completed Corps projects. Chapter 3 of EP 1130-2-500 establishes guidance for the preparation and submission of Major Rehabilitation Project Evaluation reports for annual program and budget submissions.

(3) The rehabilitation of mechanical and electrical equipment is usually included as part of the overall project. Mechanical and electrical component rehabilitation may include replacement and/or reconditioning to restore or improve a system to a like-new condition. The rehabilitation may be considered from various perspectives. It may be necessary to restore existing equipment that has deteriorated with time or failed in service, or equipment may have become obsolete and replacement might be desired to upgrade the equipment to modern standards. The Major Rehabilitation Evaluation reports and supporting information will have to provide evidence of criticality with a certain level of detail based on specific uniform engineering criteria. Reliability assessments based on probabilistic methods provide more consistent results and reflect both the condition of existing equipment and the basis for design.

(4) Further guidance for the reliability evaluation of hydropower equipment has been published in Carderock Division, Naval Surface Warfare Center (1992) and Reliability Analysis Center (1995) .

b. Reliability concepts.

(1) Reliability function. The continuous probabilistic approach to item reliability is represented by the reliability function. It is simply the probability that an item has survived to time t. The mathematical expression can be summarized by:

$$R(t) = P(T \geq t) \qquad (7\text{-}2)$$

where

$R(t)$ = reliability of the item, i.e., probability of success

t = the designated period of time for the item's operation

T = time to item failure

$P(T \geq t)$ = probability that the time to failure of an item will be greater than or equal to its service time

Conversely, the probability of failure $F(t)$ is simply

$$F(t) = 1 - R(t) \qquad (7\text{-}3)$$

(2) Hazard function or failure rate.

(a) The failure rate or hazard function $h(t)$ represents the likelihood of failure of a component as a function of its age or time in operation. It reflects how the reliability of a component changes with time as a result of various factors such as the environment, maintenance, loading, and operating condition. From Modarres (1993) it can be shown that

$$h(t) = \frac{f(t)}{R(t)} \tag{7-4}$$

where *f(t)* is the pdf. This is a mathematical description for the curve approximation of the number of probable occurrences of a specific random variable (i.e., the failure of a component for use in this EC).

(b) The hazard function or instantaneous failure rate is the conditional probability of failure of an item in the next unit of time given that it has survived up to that time. The hazard function can increase, decrease, or remain constant. It has been shown that the failure rate behavior of most mechanical and electrical engineering devices follows that shown in Figure 7-6. This is known as the *bathtub curve*. Region A represents a high initial failure rate, which decreases with time to nearly constant. This is known as the infant mortality region and is a result of poor workmanship or quality control. Region B represents the useful life phase. Here, failures occur because of random events. Region C represents the wear-out phase where failures occur from complex aging or deterioration.

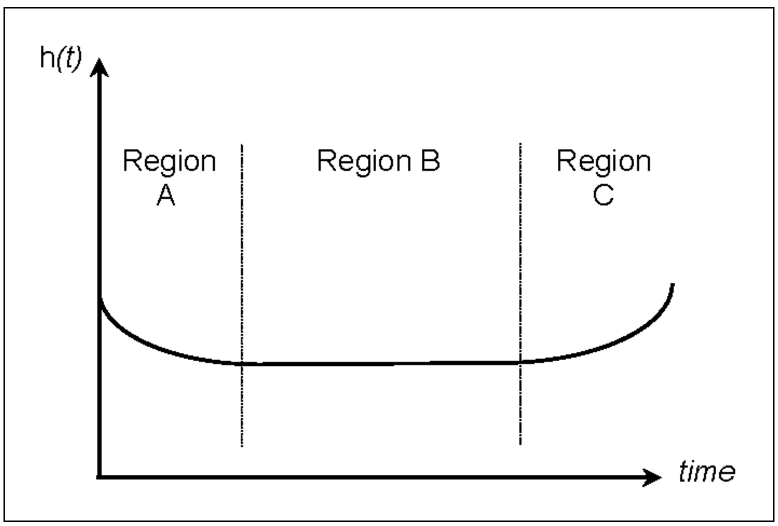


Figure 7-6. Typical bathtub curve

(c) The flat random or chance failure region (Region B) of the curve for electromechanical devices is much longer than the other two regions. Electrical devices exhibit a much longer chance failure period than do mechanical devices. Methods presented in this EC will attempt to determine reliability and predict the characteristics of Regions B and C of the bathtub curve for mature equipment using the common continuous distribution functions discussed in the next sections. The infant mortality region (Region A) will not be directly discussed in this EC since the equipment considered for major rehabilitation projects usually falls in Regions B or C.

(3) Exponential distribution.

(a) The exponential distribution is the most commonly used distribution in reliability analysis. The reliability function is

$$R(t) = e^{-\lambda t} \tag{7-5}$$

where

t = time

λ = failure rate

(b) This distribution can be used to represent the constant hazard rate region (Region B) of the bathtub curve. The hazard function for the exponential distribution remains constant over time and is represented as simply λ:

$$h(t) = \lambda \tag{7-6}$$

(c) Plots of the reliability and hazard functions for the exponential distribution are shown in Figures 7-7 and 7-8.

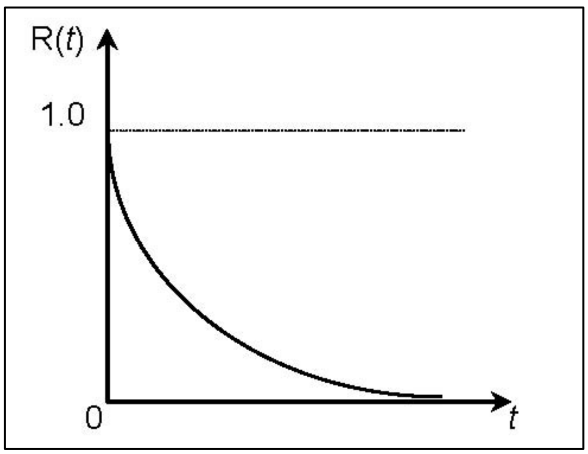


Figure 7-7. Reliability function for exponential distribution

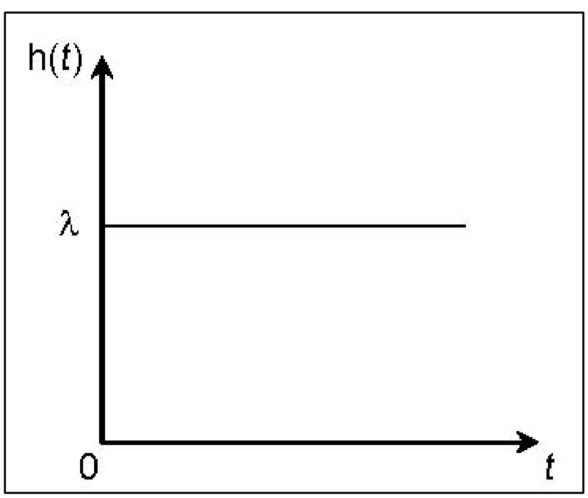


Figure 7-8. Hazard function for exponential distribution

(d) The average or mean of the exponential life distribution is the Mean Time to Failure (MTTF). It is the average length of life of all units in the population. It has significance in that the reciprocal of the hazard rate is equal to the MTTF:

$$\text{MTTF} = \frac{1}{\lambda} \tag{7-7}$$

(4) Weibull distribution.

(a) The Weibull distribution is a generalization of the exponential distribution. This distribution covers a variety of shapes, and its flexibility is useful for representing all three regions of the bathtub curve. The Weibull distribution is appropriate for a system or complex component made up of several parts. The Weibull reliability function is:

$$R(t) = \exp\left[-\left(\frac{t}{\alpha}\right)^{\beta}\right] \tag{7-8}$$

where

α = the scale parameter or characteristic life

β = the shape parameter

(b) For $0 < \beta < 1$, the Weibull distribution characterizes wear-in or early failures. For β=1, the Weibull distribution reduces to the exponential distribution. For $1 < \beta < \infty$, the Weibull distribution characterizes the wear-out characteristics of a component (increasing hazard rate).

(c) The Weibull hazard function is:

$$h(t) = \frac{\beta}{\alpha}\left(\frac{t}{\alpha}\right)^{\beta-1} \tag{7-9}$$

(d) Plots of the reliability and hazard functions for the Weibull distribution are shown in Figures 7-9 and 7-10.

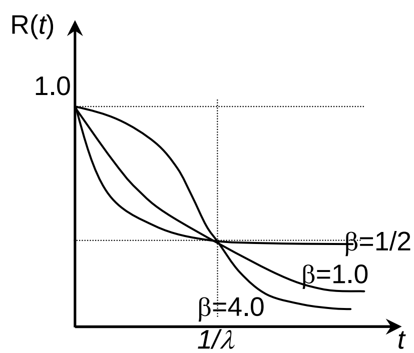


Figure 7-9. Reliability function for Weibull distribution

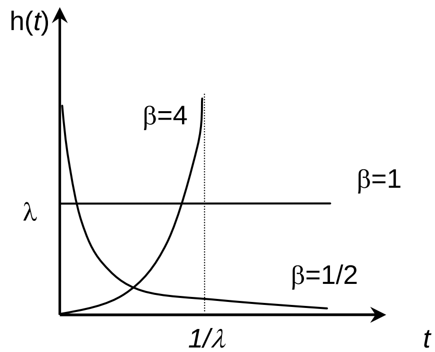


Figure 7-10. Hazard function for Weibull distribution

(5) General data required. Reliability analyses provide the best estimate of the reliability anticipated from a given design within the data limitations and to the extent of item definitions. The required data are dependent on the availability and depth of analysis required. 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 ambiguous variables such as operating environment, lubrication, corrosion, and wear. Historic data for lock and dam equipment have not usually been available. Lock and dam equipment for which data are not available requires the analysis to be completed through the use of data from larger systematic samples of similar equipment such as the published failure rate data source of Reliability Analysis Center (1995). Failure rate data can also be obtained by multivariate methods developed in EP 1130-2-500. Prior to any reliability determination, investigations should be conducted to gain a thorough knowledge of the mechanical and electrical requirements and layouts, identify equipment deficiencies, and learn the project history and future demands.

(6) Internet Web site. An Internet Web site has been established as a means to collect both historical and recent failure data for lock and dam mechanical and electrical equipment. It is intended that the data will be continually collected and compiled so that accurate failure rate tables can be developed. The data will better represent lock and dam equipment. The most important benefit is that the most current and up-to-date failure data for Corps mechanical and electrical equipment will be available to engineers doing the reliability work for future projects. In addition, it will provide a central reference source for operations and engineering personnel to check when failures occur to see if there are common problems with installed equipment. Engineering and operations personnel are encouraged to input available failure data. The Web site should be checked for the latest failure rate data when a reliability analysis is being developed.

7-10. Engineering Reliability Analysis. Assessment of the reliability of a system from its basic elements is one of the most important aspects of reliability analysis. As defined, a system consists of a collection of items (components, units, etc.) whose proper, coordinated function leads to its proper operation. In reliability analysis, it is therefore important to model the reliability of the individual items as well as the relationship between the various items to determine the reliability of the system as a whole. This EC applies the reliability block diagram (RBD) method as outlined in MIL-STD-756B to model conventional probability relationships of collections of *independent* components and systems.

7-11. System Reduction. The number of discrete mechanical and electrical components in a lock and dam requires system reduction to reduce the vast complexity of numerous components into smaller groups of critical components. The reliability models should be developed to the level of detail for which information is available and for which failure rate (or equivalent) data can be applied. Functional elements not included in the mission reliability model shall be documented, and rationale for their exclusion shall be provided.

7-12. Component Reliability. The failure distribution appropriate to the specific electronic, electrical, electromechanical, and mechanical items should be used in computing the component reliability. In most cases, the failure distribution will not be known and the exponential or the Weibull may be assumed. The α and β parameters of the Weibull equation are normally empirically determined from controlled test data or field failure data. This EC presents a procedure for estimating these values. If the β value in the Weibull function is unknown, a value of 1.0 should be assumed. The flat failure region of mechanical and electrical components is often much longer than the other two regions, allowing this assumption to be adequate. Once the component reliability values are determined, the RBD method is used to evaluate their relationship within the system to determine the total system reliability. Appendix B contains detailed information on determining component reliability.

7-13. System Risk Analysis Using Block Diagrams. The necessity for determining the reliability of a system requires that the reliability be considered from two perspectives, basic reliability and mission reliability. Both are separate but companion products that are essential to quantify the reliability of a system adequately. The incorporation of redundancies and alternate modes of operation to improve mission reliability invariably decreases basic reliability. A decrease in basic reliability increases the demand for maintenance and support. Basic reliability is normally applied to evaluate competing design alternatives.

a. Basic reliability–series system model. A basic reliability prediction is a simplified model intended to measure overall system reliability. It is used to measure the maintenance and logistics support burden required by the system. A basic reliability model is an all-series model. Accordingly, all elements providing redundancy or parallel modes of operation are modeled in series. In a series system, the components are connected in such a manner that if any one of the components fails, the entire system fails. Care should be taken when developing this type of model since the final value of the basic reliability of the system is inversely proportional to the number of components included in the evaluation, i.e., the more components there are, the lower the reliability. Such a system can be schematically represented by an RBD as shown in Figure 7-11.

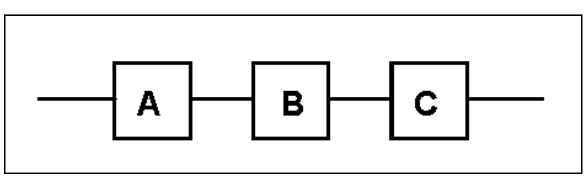


Figure 7-11. Series system

For a system with N mutually *independent* components, the system reliability R_S for time t is:

$$R_S(t) = R_A(t)*R_B(t)*R_C(t)*\ldots* R_N(t) \tag{7-10}$$

It can also be shown that if $h_s(t)$ represents the hazard rate of the system, then:

$$h_s(t) = \sum_{i=1}^{n} h_i(t) \tag{7-11}$$

The failure rate of a series system is equal to the sum of the failure rates of its components. This is true regardless of the failure distributions of the components.

b. Mission reliability. The mission reliability model utilizes the actual system configuration to measure the system capability to accomplish mission objectives successfully. The mission reliability model may be series, parallel, standby redundant, or complex.

(1) Parallel system model. In a parallel system, the system fails only when all of the components fail. Such a system is represented in Figure 7-12. In this configuration, the system will still perform if at least one of the components is working.

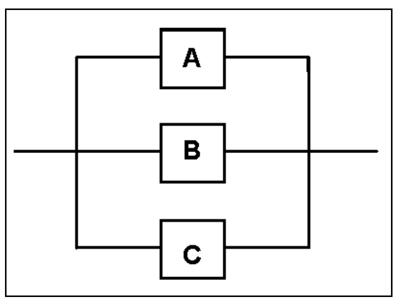


Figure 7-12. Parallel system

The reliability for the system is given by:

$$R_S(t) = 1 - [1 - R_A(t)][1 - R_B(t)][1 - R_C(t)] \tag{7-12}$$

or

$$R_S(t) = 1 - \prod_{i=1}^{N} [1 - R_i(t)] \tag{7-13}$$

$$R_S(t) = R_A(t) + \frac{\left[\lambda_A R_B(t)\right]}{(\lambda_A + \lambda_{SS} + \lambda'_B - \lambda_B)} \left\{1 - \exp\left[-(\lambda_A + \lambda_{SS} + \lambda'_B - \lambda_B) d\, t_i\right]\right\} \tag{7-14}$$

A more general form of a parallel system is the "*r* out of *n*" system. In this type of system, if any combination of *r* units out of *n independent* units arranged in parallel works, it guarantees the success of the system. If all units are *identical,* which is often the case, the reliability of the system is a binomial summation represented by:

$$R_S(t) = \sum_{j=r}^{n} \binom{n}{j} R(t)^j \, [1 - R(t)]^{n-j} \tag{7-15}$$

where

$$\binom{n}{j} = \frac{n!}{j!(n-j)!}$$

The hazard rate for parallel systems can be determined by using:

$$h_s(t) = \frac{-\,\mathrm{d} \ln R_S(\mathrm{t})}{\mathrm{dt}} \tag{7-16}$$

or

$$h_s(t) = \frac{-\,\mathrm{d} \ln\left\{1 - \prod_{i=1}^{N} ([1 - R_i(t)]\right\}}{\mathrm{dt}} \tag{7-17}$$

The result of $h_s(t)$ becomes rather complex, and the reader is referred to the reference literature.

(2) Parallel-series system. A parallel-series system is shown in Figure 7-13. This system contains equipment that is in primary use (e.g., electric utility) and also equipment ready to be used (e.g. a standby diesel generator). Upon failure of the primary equipment, the other connected equipment is immediately put into service and switchover is made by a manual or automatic switching device (SS).

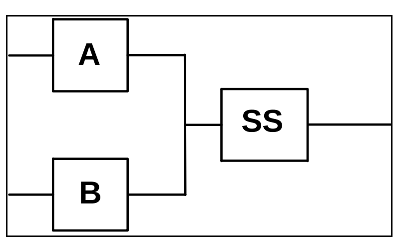


Figure 7-13. Parallel-series system

The system reliability function for the exponential distribution can be calculated for a two-component, standby redundant system using the following equation:

$$R_S(t) = (1 - \{[1 - R_A(t)]\ [1 - R_B(t)]\})\ *\ R_{SS}(t) \tag{7-18}$$

(3) Complex system models. Complex systems can be represented as a series-parallel combination or a non-series-parallel configuration. A series -parallel RBD is shown in Figure 7-14. This type of system is analyzed by breaking it down into its basic parallel and series modules and then determining the reliability function for each module separately. The process can be continued until a reliability function for the entire system is determined. The reliability function of Figure 7-14 would be evaluated as follows:

$$R_1(t) = (\ 1 - \{[1 - R_{A1}(t)]\ [1 - R_{B1}(t)]\ [1 - R_{C1}(t)]\})\ *\ \mathrm{R}_{D1}(t) \tag{7-19}$$

$$R_2(t) = (\ 1 - \{[1 - R_{A2}(t)]\ [1 - R_{B2}(t)]\})*\ R_{D2}(t) \tag{7-20}$$

$$R_S(t) = (\ 1 - \{[1 - R_1(t)]\ [1 - R_2(t)]\}) \tag{7-21}$$

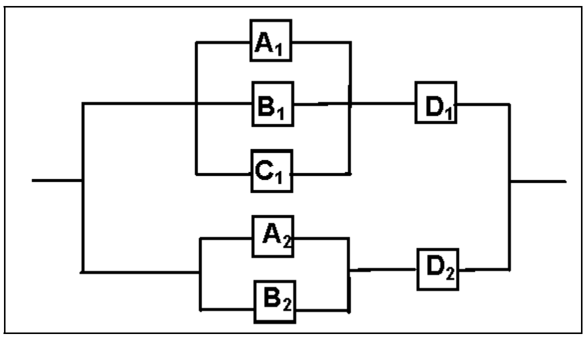


Figure 7-14. Series-parallel system

A non-series-parallel system is shown in Figure 7-15. One method of analyzing non-series-parallel systems uses the following general theorem:

$$R_S\,(t) = R_S\,(\text{if } X \text{ is working})\ R_X\,(t) + R_S\,(\text{if } X \text{ fails})\ [1 - R_X\,(t)] \tag{7-22}$$

The method lies in selecting a critical component X and finding the conditional reliability of the system with and without the component working. The theorem on total probability is then used to obtain the system's reliability.

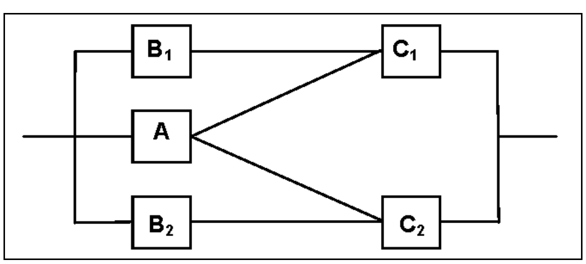


Figure 7-15. Non-series-parallel system

Select a critical component. In this case, select component A. The system can function with or without it and in each case the system resolves into a simpler system that is easily analyzed. If component A works, it does not matter if components B_1 or B_2 are working. The system can then be represented by the RBD in Figure 7-16.

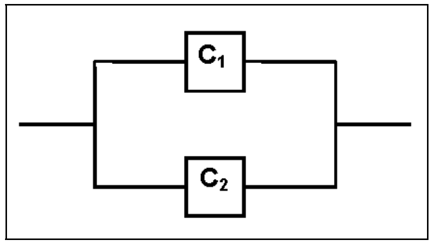


Figure 7-16.   Reduction of system with component A working

If component A does not work, the system can be reduced to Figure 7-17.

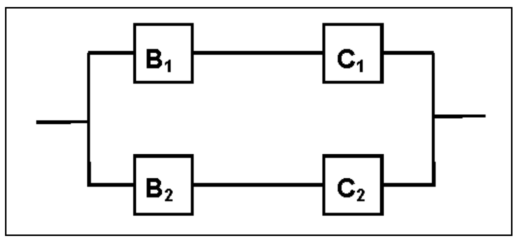


Figure 7-17.   Reduction of system with component A not working

Figure 7-17 is evaluated as follows:

$$R_S\,(\text{if A is working}) = 1 - \{[1 - R_{C1}(t)][1 - R_{C2}(t)]\} \quad (7\text{-}23)$$

Figure 7-17 is resolved as

$$R_S\,(\text{if A fails}) = 1 - (\{1 - [R_{B1}(t) * R_{C1}(t)]\}\{1 - [R_{B2}(t) * R_{C2}(t)]\}) \quad (7\text{-}24)$$

The total system reliability becomes

$$R_S\,(t) = R_S\,(\text{if A is working})\, R_A(t) + R_S\,(\text{if A fails})\,(1 - R_A(t)) \quad (7\text{-}25)$$

CHAPTER 8
Risk and Reliability Issues for Hydropower Projects

8-1. Overview of USACE Hydroelectric Projects.

a. USACE owns, operates and maintains 75 hydropower projects with a total nameplate capacity of 20,719 megawatts (MW). This represents approximately one quarter of all hydropower capacity in the United States. In terms of installed capacity, the Corps is effectively the fourth largest electrical utility in the United States.

b. The Corps is responsible for 349 main generating units with initial in-service dates from 1938 to 1988 with an average in-service date of approximately 1958. The service life of most of the major electrical and mechanical equipment in the powerhouse is between 25 and 50 years. Equipment at many of the projects has been replaced, rebuilt, or otherwise refurbished, thus effectively establishing a new in-service date. However, the rate of refurbishment has not kept pace with the rate of aging. While equipment is not rehabilitated based solely on age, at some point in time normal maintenance and repairs rise to uncomfortable levels. Equipment condition is a key element in the consideration of when to initialize rehabilitation efforts.

8-2. Scope.

a. This chapter covers the equipment and components of USACE-owned and -operated powerhouses that are related to the primary function of the facility, generation of electrical power. This includes all of the electrical and mechanical components of the facility. It also includes the structural components that are related to the power generation water passage including the penstock, scroll case, draft tube, gates, and surge tanks. The powerhouse structure itself should be handled using the methodologies described under the other chapters relating to navigation and flood-control structures.

b. The components covered by this chapter can be rehabilitated though various means including repair, rebuilding, refurbishment, and replacement. The term rehabilitation will be used in this chapter and is meant to cover all appropriate methods of extending and/or restoring the life span of the component or system being discussed. Where the term major rehabilitation is used, it is referring to the overall rehabilitation of the power-generating components of the powerhouse using the USACE Major Rehabilitation Program, described in paragraph 8-3.

8-3. Background.

a. In the mid-1970s USACE initiated a major rehabilitation program to provide a way to manage the large expenditures required to extend the life of its aging projects. Over the years the program and procedures have undergone many changes. In 1990, work began on improving the rehabilitation evaluation report process. The resulting process is well defined and comprehensive. It incorporates Project Management principles, analysis of operations and maintenance problems, engineering studies, economic analysis, and environmental reports.

b. Starting in FY 1992, Major Rehabilitation projects began being budgeted under Construction, General, and Flood Control, Mississippi River and Tributaries, appropriation accounts. Total implementation costs of hydropower rehabilitation projects have to be in excess of $5 million (adjusted for inflation) and the work must extend over two full construction seasons to qualify for the major rehabilitation program. Proposals for these projects are subjected to a much more rigorous economic analysis than in the past. Not only is it necessary to show that the monetary benefits of the major rehabilitation work exceed the cost, but it must also be demonstrated that each component in a rehabilitation plan is incrementally justified and that the combination of components proposed yields the maximum net benefits. In short, the same level of economic analysis as that for new water resources development projects must support proposals for major rehabilitation work.

c. Reliability is the key factor in determining whether there is a Federal interest in a proposed hydropower rehabilitation project. An increase in output that is primarily incidental to the reliability improvement may also be included in such a project. However, non-Federal funding is required to fund the project if there are no reliability problems and the project purpose is to improve output beyond the original design.

8-4. Hydropower Reliability.

a. The principles and guidelines presented in Chapters 2 through 5 are generally applicable to hydropower generating facilities. However, because a hydropower equipment reliability analysis requires a multifaceted approach, the following definition of reliability was developed.

b. Hydropower equipment reliability is defined as follows: "The extent to which the generating equipment can be counted on to perform as originally intended. This encompasses the confidence in soundness of the equipment based upon forced outage experience and maintenance costs, the output of the equipment in terms of measured efficiency and capacity, unit availability, and the dependability of the equipment in terms of remaining service life (retirement of the equipment)".

c. Appendix D presents detailed explanations and examples of hydropower reliability analyses. Also presented are explanations and examples of hydropower economic studies to show how the results from the reliability studies are used.

8-5. Funding Sources Effect on Risk and Reliability Methodologies. There are currently three sources of funding for rehabilitation of hydropower generating equipment at Corps owned facilities: Congressional appropriations, power marketing administration funding, and preference customer funding.

a. Congressional appropriations:

(1) The traditional method of funding hydropower rehabilitation is congressional funding. Funding can come from either the Operations and Maintenance (O&M) Appropriation or the Construction General (CG) Appropriation. The Major Rehabilitation Program described in para

8-3 is used to obtain funding through the Construction General Program. Major Maintenance and Major Rehabilitation Evaluation Reports (MRER) preparation uses O&M funding.

(2) CG funding is appropriated for each authorized hydropower project separately. This requires taking a comprehensive look at all aspects of the project and incorporating all rehabilitation needs into the proposed work package. This has the effect of creating large programs that incorporate the rehabilitation of numerous components. While this will bring all aspects of the power generation system in the powerhouse to like-new condition, it raises the project cost in an effort to maximize net benefits. Since Major Rehabilitation funding comes from congressional appropriations, the competition for funds is mostly from other water resources projects; however, the competition effectively extends to the full range of the Federal budget. When budgets are tight, projects tend to be shelved rather than split apart into smaller packages.

(3) The cost of preparing an MRER is quite high, often in the range of $1 million, and they take nearly 2 years to prepare. While the level of investment is quite good in respect to the potential return (Major Rehabilitation projects have range from $15 million to $120 million), it is often difficult for a district to program a $1 million study out of the O&M budget

b. Power marketing administration funding: The Bonneville Power Administration (BPA) has been given the authority to pay directly for the O&M costs of the USACE hydropower facilities for which they market the power that is generated. Since 1990’s, BPA and USACE have been jointly developing budgets and priorities for both annual O&M expenditures and capital investments.

(1) BPA takes risk and reliability into consideration when making their capital investment decisions, but they do not require the same level or type of analysis that the USACE Major Rehabilitation Program requires. Since they do not have to compare a hydropower rehabilitation project with a coastal harbor rehabilitation project, this is logical.

(2) Another major difference between BPA-funded projects and Congressionally funded projects is that BPA takes a power-system-wide look at their needs while Congress looks at each authorized project independently. This means that BPA can and has implemented systemwide replacements of certain components such as circuit breakers. This approach is very difficult under the Congressional appropriation process. BPA also tends to consider smaller projects with high benefits rather than taking a comprehensive look at each powerhouse. The cost and time required to develop decision documents for BPA is significantly less than what is involved in preparing a MRER.

c. Preference customer funding. At the time that this EC was prepared, the other three administrations that market the power generated at USACE projects did not have the same authority that BPA has to directly fund USACE. However, these organizations, Western Area Power Administration (WAPA), South Western Area Power Administration (SWAPA), and South Eastern Power Administration (SEPA) do have the authority to use funding from Preference Customers to pay for certain upgrades at USACE hydropower facilities.

8-6. Major Hydropower Equipment Components.

a. Chapter 2, Engineering Reliability Considerations of this EC, discusses in depth the process involved in selecting the critical components for reliability modeling. Although each hydropower facility is unique, the power generation system generally consists of the same components, so the selection process is fairly straightforward. The power generation system consists of all components involved in converting the energy in the water to usable electrical energy (water to wire).

b. The water side of the energy conversion process is considered in conjunction with the turbine. While the turbine does the actual energy conversion from the potential energy in the water to mechanical energy, the rest of the components in the water passage have an effect on the turbine in terms of efficiency, capacity, safety, and maintenance. The generator converts the mechanical energy from the turbine to electrical energy. Other electrical components convert the energy to power system voltage and otherwise connect the generator to the power grid. The other mechanical and electrical equipment in the powerhouse is necessary for operation, maintenance, and system protection.

c. Six major components are generally accepted as needing to be considered in an overall powerhouse rehabilitation: the turbine, governor, generator, exciter, transformer and main unit circuit breaker. These components work together to create a functioning generating unit. If any of the components fails, it takes one or more generating units out of service. Likewise, if the capacity or functionality of any of these components is limiting, the output of the generating unit is also limited.

d. A condition assessment and reliability analysis of the turbine, generator, and transformer are generally required for a MRER. The governor, exciter, and breaker should be the subject of a condition assessment, but the relatively low cost of replacement and improvements in technology generally negate the need for a reliability analysis. All of the powerhouse components should be subjected to a condition assessment or otherwise addressed when considering a project for Major Rehabilitation. The examples presented in Appendix D demonstrate the methods recommended for the reliability analysis.

8-7. Planning for Hydropower Rehabilitation.

a. Initial consideration of powerhouse rehabilitation is generally the result of problems or opportunities for significant improvements. Problems are normally classified as reliability or dependability issues and can be quantified as lost efficiency and/or capacity, frequent forced outages, high levels of unavailability, and high maintenance and repair costs. Also, as equipment ages and wears, the probability of failure increases to the point where it may be better to replace or rebuild the component in a planned manner rather than eking out the last bit of life and allowing a breakdown and potential collateral damage.

b. Improvement opportunities are generally classified as increased outputs beyond the initial or current output of the units or powerhouse. Turbines have frequently lost several percentage points of efficiency, and state-of-the-art turbines have efficiencies substantially higher than those built prior to the 1990s. Modern turbine design can frequently give a much broader band of high operating efficiency than the original turbines were capable of producing.

c. Generator and transformer design has also improved significantly since the initial installation of most of the USACE generating units. Modern materials and designs allow for higher outputs and a wider operating range in terms of temperature limitations. Governors and exciters now incorporate computer technology allowing more rapid response and better stabilization. Circuit breaker technology has also changed dramatically since the original construction of these facilities.

d. Design conditions have also been known to change. In one instance, the tailrace scoured away during operation, increasing the net head by several feet. In another instance, the project flood-control operation was limited because of recreational uses, and the pool fluctuations were significantly less than the original design, resulting in the design output being at a much higher head. New environmental considerations have also become a major factor in considering rehabilitation at numerous facilities.

e. The full powerhouse Major Rehabilitation process takes many years to complete. From inception to completion can easily take 5-10 years. For that reason, it is important to catch trends of declining reliability early and start the planning process. Condition assessment and trending are key to accomplishing this task.

8-8. Condition Assessment.

a. Deterioration with time and use is commonplace on most if not all mechanical and electrical components in a powerhouse. The rate at which this deterioration occurs is a function of how the equipment is used and how it is maintained. Decision makers have sought for a systematic method of assessing and rating condition in a uniform manner for a long time. An early attempt by USACE to incorporate hydropower condition assessment in the Repair, Evaluation, Maintenance and Rehabilitation (REMR) program of the 1990s was abandoned for numerous reasons, but the concept and need were not forgotten.

b. USACE and numerous other hydropower owners and operators were having informal discussions centered on the need for condition assessment tools. In 2002, USACE, the Bureau of Reclamation, Hydro Québec and BPA followed up on their informal discussions by creating the Hydropower Asset Management Partnership (HydroAMP). Representatives from the four organizations agreed to collaborate in the development of hydropower asset management tools related to equipment condition assessments, investment prioritization methods, and evaluation of business risks. The goal was to create a framework to streamline, simplify, and improve the evaluation and documentation of hydroelectric equipment condition to enhance asset and risk management decision making. The team recognized that equipment condition assessments support:

(1) Development of long-term investment strategies.

(2) Prioritization of capital investments.

(3) Coordination of O&M budgeting processes and practices.

(4) Identification and tracking of performance goals.

c. Technical teams comprising experts from the four HydroAMP organizations developed condition assessment guides for key hydroelectric powerhouse components, falling into two classes. The first equipment class includes major power train components, such as turbines, generators, transformers, governors, exciters, and circuit breakers. The second class consists of auxiliary components, including batteries, compressed air systems, cranes, emergency closure gates and valves, and surge arresters.

d. A two-tiered approach for assessing hydropower equipment condition was developed. Tier 1 of the assessment process relies on test and inspection results that are normally obtained during routine O&M activities. Equipment age, O&M history, and other relevant condition indicators are evaluated and combined with the test results to compute a Condition Index. An additional, standalone indicator is used to reflect the quality of the information available for scoring the condition indicators. The condition and data quality indicators and the condition index for each piece of equipment are easily tracked using a Computerized Maintenance Management System or other database tools.

e. The second, or Tier 2, phase of the condition assessment utilizes nonroutine tests and inspections to refine the condition index obtained during the Tier 1 assessment. Tier 2 tests often require specialized expertise or instrumentation, depending on the problem or issue being investigated. Typically, a low condition index or data quality indicator score from the Tier 1 assessment triggers the need for a Tier 2 evaluation.

f. A report was prepared by the USACE Hydroelectric Design Center (CENWP-HDC) for the hydropower asset management using condition assessments and risk-based economic analyses (HydroAMP). This document includes condition assessment guides for all of the types of equipment mentioned above, instructions on their use, and tools and case studies for using a risk analysis approach for hydropower investments.

CHAPTER 9
Guidelines for Report Writing Relative to Reliability Analysis

9-1. Introduction. When preparing a comprehensive report detailing a reliability analysis, it is important to focus on what the report intends to address. EP 1130-2-500 gives fairly clear directions for writing a report of a Major Rehabilitation Evaluation; however, the outline of what should go into a report is not clear on how to link the various pieces of the analysis. This chapter of the EC is intended to give the reader an overview of considerations when putting together the engineering reliability analysis portion of the report. Secondly, there are instances for which there is no specific guidance for evaluations (for example, systems studies) that utilize risk-based analysis. This portion of the EC can provide some good rules of thumb with respect to reporting the analysis, process, results, and conclusions. A comprehensive reporting of the engineering reliability analysis should include as a minimum the following sections: Project Information, Problem Identification, Critical Infrastructure Selection, Scenario Description, Component Reliability/Event Tree/Consequence Analysis, Other Life Cycle Operating Costs, and Summary of Overall Results.

9-2. Project Information. This section of the report should contain a brief overview of the project with information such as location, physical characteristics, overview of project use, year of construction, years of any rehabilitations and what was done, as well. After reading this section, the reader should have a fairly good handle on the overall aspect of the project. It is highly recommended that a location map and a recent color aerial photograph of the project be included as part of this section. One or two original construction photographs might prove beneficial if they are handy and of good quality. A quick overview of project benefits (for example, flood reduction damages, navigation tonnage, hydropower use, recreational use) would also help the reader identify with the importance of the project in terms of economics and/or the potential for life loss. If there is a Main Report for the study, then this information should be covered in detail there; however, it is a good idea to provide at least a concise version of this information at the beginning of the Engineering Appendix.

9-3. Problem Identification. This section is important to establish the issues on the project that will be significant in terms of reliability analysis. If there has been a history of problems at the site related to any of the critical operating components, then it should clearly be spelled out in this section. A couple of photos showing the damage are extremely helpful to the reader. In addition, consequences related to those historical damages are important to note. The consequences shown in the figure reflect those to both USACE (repair cost) and the navigation industry (delay cost). It is important to note the historical operating costs that would be associated with this feature if it were reliable. Other examples of consequences might be loss of hydropower generation or loss of recreational use, depending upon the project being evaluated. Keep in mind that the consequences need to relate directly to the problems identified within this section.

9-4. Critical Infrastructure Selection. This section details which components need to be evaluated using risk-based methods and the process by which they were selected. One requirement of Major Rehabilitation Evaluations is that the reliability of the entire infrastructure for the project be evaluated to prevent the features to be rehabilitated at a project from being evaluated in a piecemeal fashion. Thus, once rehabilitation of a project is completed, there should not be a need to go back and reevaluate that site for many years if all elements were properly evaluated the first time. Probably the best way to develop the list of components to be evaluated using risk-based methods would be to establish a criticality ranking system and apply it to all significant infrastructure on the project. This is applicable whether the study is for a single project or a systems study where several projects are evaluated. The ranking system must be set up to be relative to the study. An example of a ranking system for the GLSLS Study is provided in Chapter 7. Another example is provided in the Navigation Technical Appendix for the Markland Locks Major Rehabilitation Evaluation. When reporting the ranking system, make sure to include a detailed description of each category and how it was applied. The ranking results and final component list should be included in the report as well. Both of these can be used as examples of the type of information that should be placed in the report relative to the selection of critical infrastructure.

9-5. Scenario Description. The various scenarios to be evaluated in the study need to be detailed briefly in this section. Again, assuming there is a Main Report and/or Economics Appendix, these scenarios will be detailed in these parts of the report. It is also a good idea to give the reader a basic description of each scenario to be evaluated. This will help them as they continue further with the Engineering Appendix and there are differences in reliability analyses for the various scenarios being evaluated. Example descriptions should be provided for both the Without-Project (WOPC) scenarios (for example, Base Condition, Advance Maintenance) as well as any With–Project (WPC) scenarios (Major Rehabilitation, New Project). Make sure to reference where these scenarios are described in detail in other appendices and/or the Main Report so the reader can get more details if desired.

9-6. Component Reliability/Event Tree/Consequence Analysis.

a. This section should represent the substance of the Engineering Appendix as it relates to the major inputs to any risk-based study. Every component evaluated using risk-based methods should be detailed in this section with each broken out as a subsection under this part of the narrative. As an example, three critical components are to be evaluated on a flood-control project: Embankment Seepage/Piping, Spillway Monolith Stability, and Spillway Gates. The first subsection of the report should detail the reliability, event trees, and consequence (life loss/economic) analysis for each scenario for the Embankment Seepage/Piping. This should include the complete analysis from the development of the basic engineering analysis through the reliability model, the development of the event tree, and finally the economic analysis and results. The costs and any service disruption time placed in the event tree need to be clearly detailed in this section so the reader is aware of how the costs were developed. This procedure needs to be followed for each scenario where the reliability is affected by the scenario under consideration (which should be the case for all scenarios). The economic/life loss analysis associated with each scenario specific to this component should also be included in this subsection. The next subsection should follow the same process for the Spillway Monolith Stability. The final subsection will be the Spillway Gate reliability analysis. Any scenario specific maintenance and/or operating costs affected by the reliability analysis should clearly be defined within each component subsection as well.

b. A good example is from the Markland Major Rehabilitation Evaluation for the main chamber lock gates. Two WOPC scenarios were evaluated. The first was the Base Condition. The Base Condition represented the fix-as-fails scenario, and the gates were allowed to deteriorate with nothing more than minor maintenance. The hazard rates computed for the Base Condition were much higher than for the other scenarios, but no preventative maintenance costs were associated with this scenario. The second WOPC scenario evaluated was the Advance Maintenance scenario. The Advance Maintenance was the current method of maintaining the gates given their current level of damage. Therefore, preventative maintenance and inspections were scheduled every 2 years in an attempt to keep the gates serviceable and avoid a catastrophic failure. Under this scenario, the hazard rates were lower than for the Base Condition, but the preventive costs had to be included in the life cycle analysis. In addition, the event tree for the Advance Maintenance had less severe impacts than for the Base Condition because of the preventive maintenance. Finally, the WPC scenario (Major Rehabilitation) was the last scenario evaluated. This situation called for replacement of the gates prior to failure. This required the highest up-front cost, but provided the best reliability through the study period. The life cycle analysis included both the up-front cost and the improved reliability. The main emphasis of the discussion of this example is to ensure that the reader is aware that all life cycle costs need to be included with the reliability analysis so that each scenario can truly be compared against one another.

c. The Navigation Technical Appendix gives several good examples of the complete analysis for a single component. It covers all the aspects of the analysis for a single component including the engineering reliability analysis, event trees, and a summary of the economic results.

9-7. Other Life Cycle Operating Costs. This section is intended to report the life cycle operating costs for each scenario in an easily understandable format. Usually, a spreadsheet table detailing annual repair costs and service disruption times for each scenario is a suitable format. This information is required only if the maintenance on the critical features will change from scenario to scenario. This should be the case for any study where maintenance scenarios will be evaluated (as is required for Major Rehabilitation Evaluations). It is expected that fewer maintenance/repair funds will be required if critical components are replaced under a Major Rehabilitation than for a WOPC scenario. This change in future maintenance following the rehabilitation needs to be reflected in the analysis and shown in this section. The same is true for the differences in maintenance for other scenarios. Many studies are also interested in capturing the total life cycle costs including those that will not change from scenario to scenario, which is the case many times for such items as labor and utility costs. This information can also be presented within these tables to facilitate summarizing across all categories. Any maintenance costs that are related to the reliability analysis need to be separated so they can be connected with the reliability analysis for each component.

9-8. Summary of Overall Results. This section should summarize the final results for the study as they relate to the engineering analysis. A summary of the plan that optimizes the NED benefits and timing of any major replacement or rehabilitation work should be reported in this section. The outputs may vary by different economic scenarios (such as future traffic forecasts for navigation studies), and the results for each scenario should be detailed. This should be a concise version of results that are detailed in the Main Report and/or Economics Appendix.

APPENDIX A
References

MIL-STD-756B
Reliability Modeling and Prediction

ER 1105-2-100
Planning Guidance Notebook

ER 1130-2-500
Project Operations – Partners and Support (Work Management Policies)

ER 1110-2-1156
Dam Safety - Organization, Responsibilities, and Activities

EP 1130-2-500
Project Operations – Partners and Support (Work Management Guidance and Procedures)

EM 1110-2-2502
Retaining and Flood Walls

ETL 1110-2-547
Introduction to Probability and Reliability Methods for Use in Geotechnical Engineering

ETL 1110-2-561
Reliability Analysis and Risk Assessment for Seepage and Slope Stability Failure Modes for Embankment Dams

ETL 1110-2-556
Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies

ETL 1110-2-560
Reliability Analysis of Navigation Lock and Dam Mechanical and Electrical Equipment

ETL 1110-2-321 (rescinded)
Reliability Assessment of Navigation Structures and Stability of Existing Gravity Structures.

ETL 1110-2-354 (rescinded)
Reliability of Pile-Founded Navigation Structures.

ETL 1110-2-532 (rescinded)
Reliability Assessment of Navigation Structures.

ANSI/IEEE 2007
American National Standards Institute/Institute of Electrical & Electronics Engineers, Inc. 2007. Design of Reliable Industrial and Commercial Power Systems, ANSI/IEEE 493-2007, New York.

Ang and Tang 1984
Ang, A.H-S, and Tang, W.H. 1984. Probability Concepts in Engineering Planning and Design, Volumes I & II, John Wiley, New York.

ASME 1989
ASME Boiler and Pressure Vessel Code, 1989 Edition, American Society of Mechanical Engineers, New York

Ayyub and Haldar 1984
Ayyub, B.M. and Haldar, A. 1984. "Practical Structural Reliability Techniques, " Journal of Structural Engineering, ASCE, 110(8), 1707-1724.

Ayyub, Kaminsky 1985
Ayyub, B.M., Kaminskiy, M.P., Patev, R.C., 1995. "Loading Cycles for the Fatigue Reliability Analysis of Miter Gates". Technical Report ITL 95-12, U.S. Army Corps of Engineers Engineer Research and Development Center, Vicksburg, MS.

Ayyub, Blair, and Patev 2004
Ayyub, B.M., Blair, A.N., and Patev, R.C. 2002. "Expert-Opinion Elicitation for the Risk Analysis of Design-Improvement Alternatives to the Lindy Claiborne Boggs Lock and Dam". EDRC/ITL TR-02-2, U.S. Army Corps of Engineers Engineer Research and Development Center, Vicksburg, MS.

Ayyub, Blair, and Patev 2004
Ayyub, B.M., Blair, A.N., and Patev, R.C. 2002. "Risk Analysis of Design-Improvement Alternatives to the Lindy Claiborne Boggs Lock and Dam". EDRC/ITL TR-02-1, U.S. Army Corps of Engineers Engineer Research and Development Center, Vicksburg, MS.

Baecher and Christian 2004
Baecher, G and Christian, J. 2004. Geotechnical Risk and Reliability. John Wiley, New York

Bloch and Geitner 1994
Bloch, H. P., and Geitner, F. K. 1994. *Practical Machinery Management for Process Plants, Volume 2, Machinery Failure Analysis and Troubleshooting,* Gulf Publishing, .

Carderock Division, Naval Surface Warfare Center 1992
Carderock Division, Naval Surface Warfare Center. 1992. "Handbook of Reliability Prediction Procedures for Mechanical Equipment," NSWC-92/L01, West Bethesda, MD.

Ellingwood and Mori 1993

Ellingwood, B and Mori, Y. 1993. "Probabalisitic Methods for condition assessment and life prediction of concrete structures in nuclear plants." Nuclear Enginering and Design, 142:155-166.

Ellingwood (1995)
Look up and add ITL reference.

Green and Bourne 1972
Green, A. E., and Bourne, A. J. 1972. *Reliability Technology,* Wiley-Interscience, Hoboken, NJ.

Krishnamoorthi 1992
Krishnamoorthi, K. S. 1992. *Reliability Methods for Engineers*, Quality Press, New York.

Modarres 1993
Modarres, M. 1993. *What Every Engineer Should Know About Reliability and Risk Analysis,* Marcel Dekker, New York.

Palisades 2008
Palsides Corporation 2007. @Risk User's Guide. Newfield, NY

Patev, Riley, Schaaf and Kathir 1997
Patev, Robert C, Riley, Bruce, Schaaf, David, and Kathir Nathan. 1997. "Time-Dependent Reliability and Hazard Function Development for Navigation Structures in the Ohio River Mainstem System Study". ASCE Risk-Based Decision Making in Water Resources VIII, Santa Barbara, CA.

Reliability Analysis Center 1995
Reliability Analysis Center. 1995. *NPRD Nonelectronic Parts Reliability Data,* Rome, NY.

Rosenblueth 1975
Rosenblueth, E. 1975. "Point Estimates for Probabality Estimates." Proceeding of the National Acadamy of Sciences of the United States of America, Vol. 72 No. 10, 3812-3814.

Sadlon 1993
Sadlon, Richard J. 1993. "Mechanical Applications in Reliability Engineering," Richard Sadlon, Reliability Analysis Center, Rome, NY, 1993.

APPENDIX B
Navigation Reliability

Section I
Example 1
Issue: Lock Gravity Land Wall on Rock Foundation
Project: Ohio River Main Stem Systems Study

B-1. Background.

a. There are several basic types of lock walls, the most common type of which is the gravity lock wall. Gravity lock walls are susceptible to many forms of deterioration throughout their service life, and if the degradation changes the performance characteristics of the gravity lock wall over time, then the reliability is time dependent. Some examples of such deterioration are the expansion and disintegration of the concrete mass caused by alkali aggregate reaction (AAR) or the loss of subgrade material around foundation piles caused by scour or piping. An example of a reliability analysis for mass concrete deterioration is described in Section IV - Chickamauga Lock.

b. Another example of time-dependent reliability is an anchored lock wall. Usually the anchors are added as modification to an existing lock wall to correct a safety deficiency or to enhance performance. The reliability of anchored lock walls is time dependent because the anchor capacity may degrade over time from corrosion and fatigue. Only three project sites, out of a total of 20, on the Ohio River have anchored concrete monoliths for lock walls. All other remaining sites have unanchored concrete gravity monoliths for lock walls. The unanchored lock wall reliability analysis will be the example provided in this appendix.

c. Within the unanchored concrete gravity lock wall category, three types of monoliths were analyzed for reliability: a “typical” land wall, middle wall, and river wall within the limits of each lock chamber. Additionally, the lower middle wall auxiliary chamber miter gate monolith was analyzed. All of the unanchored lock wall sections that were analyzed as a part of the Ohio River Main Stem Systems Study (ORMSS) were concrete gravity structures founded on rock. Three walls are made of individual concrete, gravity monoliths that form the lock chamber. The land wall and one side of the middle wall form the auxiliary chamber. The river wall and other side of the middle wall form the main chamber. Since neither the time nor funding was sufficient to investigate every possible monolith cross section for reliability analyses, a typical monolith was selected to be representative for each wall.

B-2. Load Cases for Lock Wall Reliability.

a. Because the structures are massive concrete structures without anchors, they are not subject to fatigue and corrosion associated with steel structures. As a result, no significant deterioration over the operational life of the structure is considered, and the reliability of the structures is assumed to be independent of time. The reliability is assumed to be constant over the study period. This is consistent with HQUSACE reliability guidance for unanchored concrete gravity monolith structures. Since the reliability of the structure is based on limit states and not design values, unsatisfactory performance modes considered for the gravity monoliths are overturning, sliding, and bearing of the rock foundation without any safety factors applied to the analysis. The limit states established for the unsatisfactory performance modes are as follows:

(1) Sliding: the driving horizontal forces exceed the resisting horizontal forces.

(2) Bearing: the resultant monolith toe bearing pressure exceeds the maximum peak bearing strength of the foundation rock.

(3) Resultant location: the performance mode selected herein to replace overturning analysis. Overturning is unlikely as a pure mode of failure, as foundation bearing and/or sliding would occur before the resultant reached the toe. In practice, the location of the resultant on the base is used to determine the percentage of the base in compression, which is then used as a measure of stability. Reliability analyses, however, require that stability be expressed in terms of capacity versus demand, i.e., a performance function. This ratio can be conveniently represented by the equation

$$C/D = B/(B - 2Xr) \qquad \text{(B-1)}$$

where

C = *Capacity*

D = *Demand*

B = *Base width*

X = *Distance from toe*

r = *Resultant location*

b. These values were selected as a starting point for limit states to determine if any unsatisfactory performance would be encountered in the analysis. As it turns out, the original safety factors used in the design coupled with the competent foundation strengths meant that the ORMSS team did not have reliability issues with these structures. If the reliability analysis had indicated that any limit states were exceeded, careful attention to an appropriate limit state would have been necessary. This would have required the engineering team to investigate the amount of movement encountered before the structure would have become functionally deficient. For lock walls, this is a small amount of movement. For other concrete gravity structures, more movement might be allowed before a major problem occurs. In other words, using a safety factor value of 1.0 for these limit states does not necessarily reflect what the critical limit state should be for the structure. Limit states may be controlled by stability or service requirements.

c. For the ORMSS, two loading conditions are considered for the unanchored lock wall monoliths: the normal operating condition and the maintenance condition. The normal operating condition represents the usual daily cyclic loads experienced by the lock monoliths. Dewatering the chamber is the maintenance condition. Table B-1 depicts the loading conditions for both situations for all three types of lock chamber monoliths (land, river, and middle walls). As an example, the values and descriptions in the table are representative of the conditions at Markland Locks and Dam, located on the Ohio River at mile 531.5 below Pittsburgh, PA.. Normal upper pool at Markland is elevation (el) 455.0. For Ohio River navigation projects, the upper pool generally does not vary significantly and therefore is assumed to be constant in the model. Normal lower pool elevation is 420.0; however, the lower pool fluctuates and is a random variable in the reliability analysis. The major external loadings experienced by a land wall are lateral earth pressure, hydrostatic pressure from the saturation level of the backfill, uplift, hawser pull, and the fluctuating pool elevation in the lock chamber. The middle and river walls are subjected primarily to uplift, hawser pull, and fluctuating pool elevations in the chambers or river. Barge impact is excluded from the analysis since the lock chamber monoliths are not part of the navigational approach system.

Table B-1. ORMSS Lock Wall Stability Load Cases (Markland Project Values)

	Load Case	
Monolith	Normal Operating Condition	Maintenance Condition
Land	Backfill saturated to el 455.0 and fluctuating lower pool in main chamber.	Backfill saturated to el 455.0 and the main chamber dewatered, el 398.0.
Middle	Main chamber at upper pool, el 455.0, and auxiliary chamber at fluctuating lower pool.	Main chamber at upper pool, el 455.0, and auxiliary chamber dewatered, el 398.0.
River	Auxiliary chamber at upper pool, el 455.0, and the river at fluctuating lower pool.	River at fluctuating lower pool (<el 431.08) and auxiliary chamber dewatered, el 398.0.

d. For the analysis of all gravity structures, an external force resisting rotation was added to the model to account for rock embedment where appropriate. If the embedment was minimal, this external force was neglected in the analysis. The model calculates this force as the cross-bed shear resistance of the rock wedge on the chamber face of the monolith. This was handled on a case-by-case basis by project and monolith section.

B-3. Load Parameters.

a. The gravity loads considered in the analysis are due to the weights of the water and soil above the monolith, water within the culvert, and the concrete monolith. A typical free-body diagram of a land wall monolith is shown in Figure B-1 for reference. For an example of model input, the soil/rock random variables and constant values for the Markland project are provided in Tables B-2 and B-3, respectively. For the case where the moist soil unit weight exceeds the saturated soil unit weight, the moist soil unit weight is made equal to the saturated soil unit weight in the stability analysis. Lateral earth pressure of the backfill is computed using the at-rest pressure coefficient K_o that is calculated from Jaky's equation,

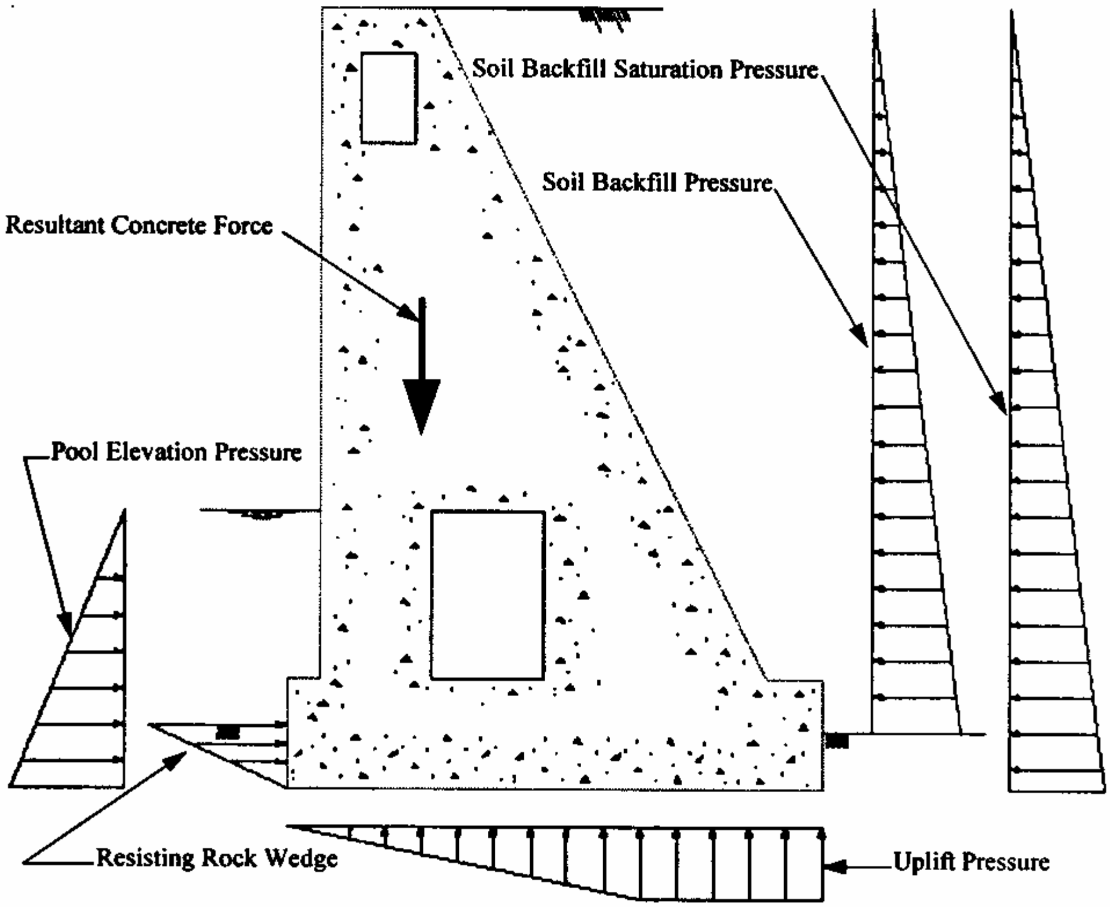


Figure B-1. Free-body diagram for typical land wall monolith

since the lock monoliths are founded on rock. For Markland, the saturation level in the backfill is assumed to be constant and equal to the normal upper pool elevation of 455.0. Uplift is assumed to be acting on the entire base of the monolith. The uplift pressure values are based on the varying lower pool elevation, constant upper pool elevation, and/or the saturation level in the backfill. The distribution of the uplift pressure was calculated using a derived solution for uplift that is a function of the overturning and resisting moments, uplift pressures at the toe and heel of the structure, and the resultant vertical load. The uniform uplift pressure equivalent to the maximum hydrostatic pressure at the heel of the base acts on the portion of the base not in compression. A hawser pull was applied to a structure under the normal operating condition for 20 percent of the Monte Carlo trials, for which 10,000 were run for the normal load case. It was estimated that roughly 20 percent of the lockages involved hawser pulls on typical land wall monoliths. The hawser pull force value normal to the face of a monolith is established from the guidance in ETL 1110-2-321, and the point of application is assumed to be 5 ft above the pool elevation. Vertical shear (downdrag), acting along the wall-soil interface caused by differential settlement of the backfill, is available in the model but was not utilized in the stability analyses since the lock monoliths are completely stable for both normal operating and maintenance conditions. Closer attention to this issue would have been required had this not been the case.

Table B-2. Geotechnical Random Variable Parameters (Markland Example)

Variable	Mean	Standard Deviation	Maximum	Minimum	Distribution	Units	Description
Soil							
Mst Unit Wt	0.115	0.003	0.124	0.106	Normal	kcf	Driving soil, unit weight, moist
Sat. Unit Wt	0.125	0.004	0.137	0.113	Normal	kcf	Driving soil, unit weight, saturated
Phi, internal	33	2	38	30	Normal	deg	Driving soil, internal friction angle
Rock							
Phi, sliding	38	4	45	35	Normal	deg	Rock, sliding friction angle
C, sliding	20	20	25	0	Normal	psi	Rock, sliding shear strength
Phi cross-bed	47	4.5	57	37	Normal	deg	Rock, cross-bed friction angle
c, cross-bed	75	25	100	50	Normal	psi	Rock, cross-bed shear strength
Sat Unit Wt	0.1672	0.002	0.1697	0.1660	Normal	kcf	Rock, saturated unit weight
Brg Capacity	2083.3	208.3	2430.6	1736.1	Normal	psi	Rock, ultimate bearing capacity
Lower Pool					CDF[a, b]	NA	Lower pool elevation
Hawser Pull	57.5	11.5	80.5	34.5	Normal	kip	Hawser pull force, normal to face

[a] Cumulative Density Function (CDF) established for Lower Pool is used.
[b] For river wall R-48, the maintenance condition, the maximum main chamber is flooded when the lower pool elevation exceeds 431.08.
NA = Not applicable.

Table B-3. Constants Used in ORMSS Lock Wall Analysis (Markland Values)

Constant	Value	Units	Description
Conc Unit Wt	0.1475	Kcf	Concrete, unit weight
Water Unit Wt	0.0625	Kcf	Water, unit weight
Saturation Level	455.0	Ft	Water saturation level in backfill
Upper Pool	455.0[a]	Ft	Upper pool elevation

[a]When lower pool el > upper pool el – 1 ft, upper pool el = lower pool el + 1 ft.

b. The tables and description of the conditions at Markland are included only to give the reader a flavor of the analysis and what type of information is required for the reliability analysis. Additionally, the random variables and constants are site-specific values but are input into the model the same as shown for Markland.

B-4. Random Variables and Constants in the Analysis. The geotechnical shear strength parameters for all sites are based on information obtained from the as-built drawings, design memoranda, foundation reports, periodic inspection reports, and reference material. Each district's geotechnical engineers provided the necessary data to complete the analysis. Cross-sections, boring logs, N-values, and laboratory test results are used to determine the range in strength values. Very limited test results are available for the majority of the sites. As a result, typical strength values are obtained from reference material and original design values. The probabilistic values used in the reliability analyses include the type of probability distribution function, mean, standard deviation, range, coefficient of variance, and correlation coefficient, and are provided in the following table. Unit weights, shear strength parameters, and ultimate bearing capacity values are provided for the soil and rock foundation. Cross-bed shear strengths are also provided for the monoliths embedded in rock.

B-5. Lock Wall Reliability Model Computations.

a. The Microsoft Excel spreadsheet and the @Risk add-on application consisted of six sheets (Input Parameters, Monolith Geometry, Soil Geometry, Water Elevation, Stability Analysis, and Stability Results) and two Visual Basic modules (Update and Visual Basic Program). @Risk is an add-on software application for Microsoft Excel that provides Monte Carlo simulation. The material properties and input data are represented by probability distribution functions instead of discrete values. For each Monte Carlo trial, material properties and input data are randomly selected according to their respective probability distributions for the stability analysis. The structure is analyzed for its stability in overturning, sliding, and bearing. Any unsatisfactory performance is tabulated for each trial. A sufficient number of trials, 10,000 iterations for each load case for this model, are required to achieve convergence and a particular level of confidence in the simulation results.

b. For the lock wall monolith reliability model, the probability distribution functions, parameters, and constants are provided in the Input Parameters sheet. The geometry, voids, and centroid computation of the monolith are provided in the Monolith Geometry sheet. Soil geometry is provided for one or two types of backfill, and the sheet calculates the moist and saturated soil layers, weights, and centroids using the Visual Basic Update functions. The lower pool cumulative density function and upper pool discrete value are provided in the Water Elevation sheet. Soil and rock elevations for computation of driving and resisting forces are provided in the Stability Analysis sheet. The stability calculations and results for overturning, sliding, and bearing are provided in the Stability Results sheet. A Visual Basic module is used to track unsatisfactory performances during the Monte Carlo trials. The respective unsatisfactory performances for each limit state and cumulative unsatisfactory performances are also tabulated on this sheet.

c. The stability analyses follow the guidance provided in Chapter 4 of EM 1110-2-2502. For the overturning stability analysis, the vertical and horizontal forces and the resultant moments are summed. The resultant moments are categorized as resisting or overturning moments. The effective base in compression and the uplift are solved for simultaneously using a closed-form solution. The closed-form solution is a function of the overturning and resisting moments, uplift pressures at the toe and heel of the structure, and the resultant vertical load. A negative effective base in compression indicates that the structure performs unsatisfactorily in overturning. Once the effective base and uplift are established, the sliding stability analysis is conducted. The passive resistance of the rock and structural wedge is computed, and the resisting forces are summed with the resultant net negative driving forces. If the sum of the resisting and driving forces is negative, the structure performs unsatisfactorily in sliding. The maximum bearing pressure is then calculated and compared to the ultimate bearing capacity for the rock foundation. If the bearing pressure exceeds ultimate bearing strength, the structure performs unsatisfactorily in bearing. Each mode of unsatisfactory performance is tabulated for each trial. However, any trial that results in a calculated unsatisfactory performance in any one or combination of the three performance modes will be counted for reliability purposes as one unsatisfactory performance for the structure.

B-6. Results and Conclusions. No unsatisfactory performances were calculated in 10,000 iterations for both the normal and maintenance load cases for any of the projects with unanchored monoliths. There were no unsatisfactory performance occurrences because of the original safety criteria used in design of the structures. Additionally, each site is founded on sound rock that resists all three possible failure modes. These results are reasonable and expected since no significant movement of the walls has been noted at any of the sites since construction. Since there were no unsatisfactory performances, the economists did not need to run their analysis for the lock wall monoliths where this analysis was completed.

Section II
Example 2
Issue: Reliability Analysis for Horizontally Framed Miter Gates
Project: Markland Locks Major Rehabilitation Study

B-7. Introduction. The horizontally framed miter gates at Markland are the major component that has caused considerable repair cost and closures of the main lock chamber, costing the navigation industry millions of dollars. This section will detail both the problems with the miter gates and with the reliability analysis associated with all three repair conditions associated with the Without-Project condition for both the main and auxiliary chambers.

B-8. Description of the Horizontally Framed Miter Gates. The miter gates at Markland are the same for both the main and auxiliary chambers. Additionally, there is no difference between the upper and lower miter gates within the same chamber; therefore, all sets are similar in terms of design and construction technique. The downstream elevation of the miter gates is shown in Figure B-2. Both the auxiliary and main chamber gates are of the same design and construction technique. A high quality photo of the entire leaf of the auxiliary chamber gates was available for inclusion in this report. The gates are approximately 55 ft tall from the center line of the lower girder to the center line of the highest girder. The gates are 61.5 ft wide. The main load-carrying members are 70-in.-deep, horizontally framed plate girders spaced about 44 in on center. Each leaf weighs about 440 kips.

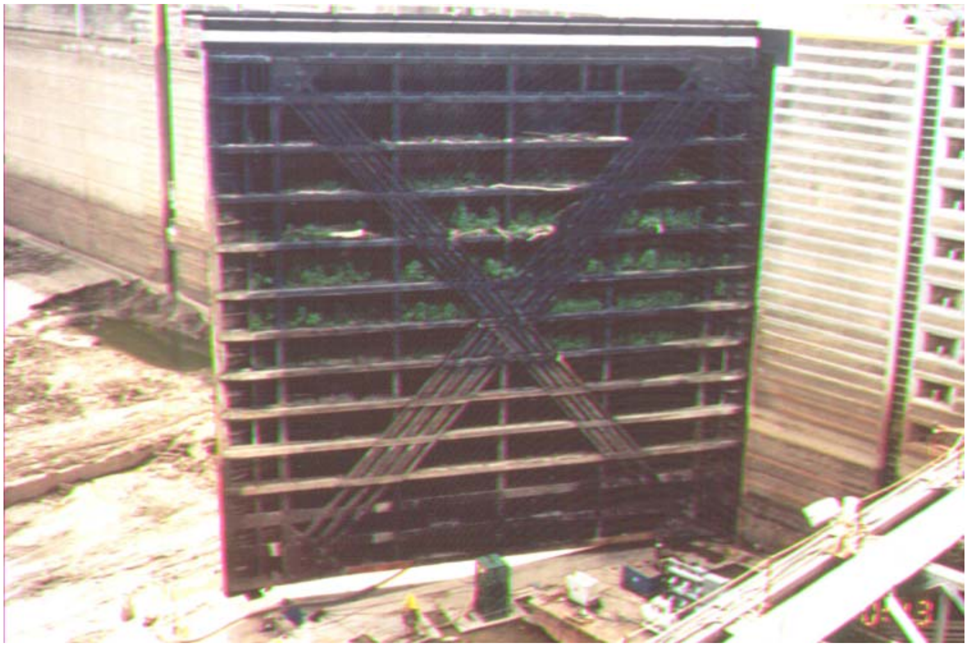


Figure B-2. Downstream elevation of auxiliary chamber miter gate leaf

B-9. Overview of the Problem with the Miter Gates.

a. Serious concern regarding the integrity of the miter gates arose during the 1994 dewatering of the main chamber at Markland Locks and Dam. This dewatering was scheduled to do major maintenance for the main chamber, including jacking the miter gates and replacing the pintle, seals, and other components. However, once the chamber was dewatered and the gates were inspected, severe cracking at several locations was noted. Some of the cracks were at welded connections of the main load-carrying members. In particular, the heaviest cracking occurred near the pintle area on the lower girders. It was determined that the extensive cracking was fatigue-related. Since the gates had seen less than 40 years of operation, the fatigue of the gates was considered to be an abnormal failure mode. In order to determine the cause for this type of extensive cracking, the Louisville District worked with an engineering consultant specializing in nonlinear finite element modeling to help determine the cause for the early fatigue cracking. It was determined that the early fatigue cracking was due to the original construction when the flanges and webs were welded together and subsequent repair methods when welding was used to repair smaller cracks throughout the history of operation. Because of the large number of structural members joining together in the pintle area, the entire region is highly constrained from movements caused by temperature fluctuations. During welding large stresses develop in the members near the weld joints. As the weld joint cools, large tensile stresses (termed residual stresses) are "locked" in place because of the restraints of the gate in the pintle area. The large tensile stresses then are subjected to normal operating loads from pool fluctuations as a chamber goes between upper and lower pool. When the gate is holding back the pool, then compressive stresses are applied to these areas where the tensile, residual stresses are locked in, thus causing a stress reversal during each operation. This large reversal, coupled with the historical number of load cycles, has caused the fatigue-related cracking on these of gates.

b. Figure B-3 depicts the widespread cracking present in the main chamber miter gates. The white arrows in the photo show areas where large cracks were found and in need of immediate repair. Note most of the cracking on this leaf is occurring where the vertical stiffeners are welded to the horizontal girders. Cracks initiate at that connection and grow through the girder flange.

Figure B-3. Main chamber miter gate cracking above pintle

c. Figure B-4 shows the repair technique on one of the miter gate leafs. Repair consisted of gouging out the entire length of the weld and rewelding material back together. Note that cracking on this leaf initiates at corners of small diagonal plates and girder/stiffener flanges and then proceeds through flange. Additionally, note extensive length of cracks.

d. Figure B-5 depicts cracking also prevalent near the pintle region where diagonal plate is welded to the gate. White arrows show positions of extensive cracking. Note the new flange for the lower girder for this leaf. This flange was added because of damage to the lower girder flange on this girder. This damage is also shown in Figures B-6 and B-7.

e. Figure B-6 shows main chamber miter gate damage to the lower girder downstream flange. Note damage to the lower girder flange plate caused by buckling of the web. The buckling of the web helped cause the connection between the web and flange plate to separate as shown in Figure B-7. Note that this portion of flange plate was replaced during the repair. This area is just past the location where the diagonal plate is welded to the girder.

f. Figure B-7 shows a close-up of the damage to the flange plate looking from inside the girder toward the downstream flange plate. Note the separation of the flange plate from the web of the girder.

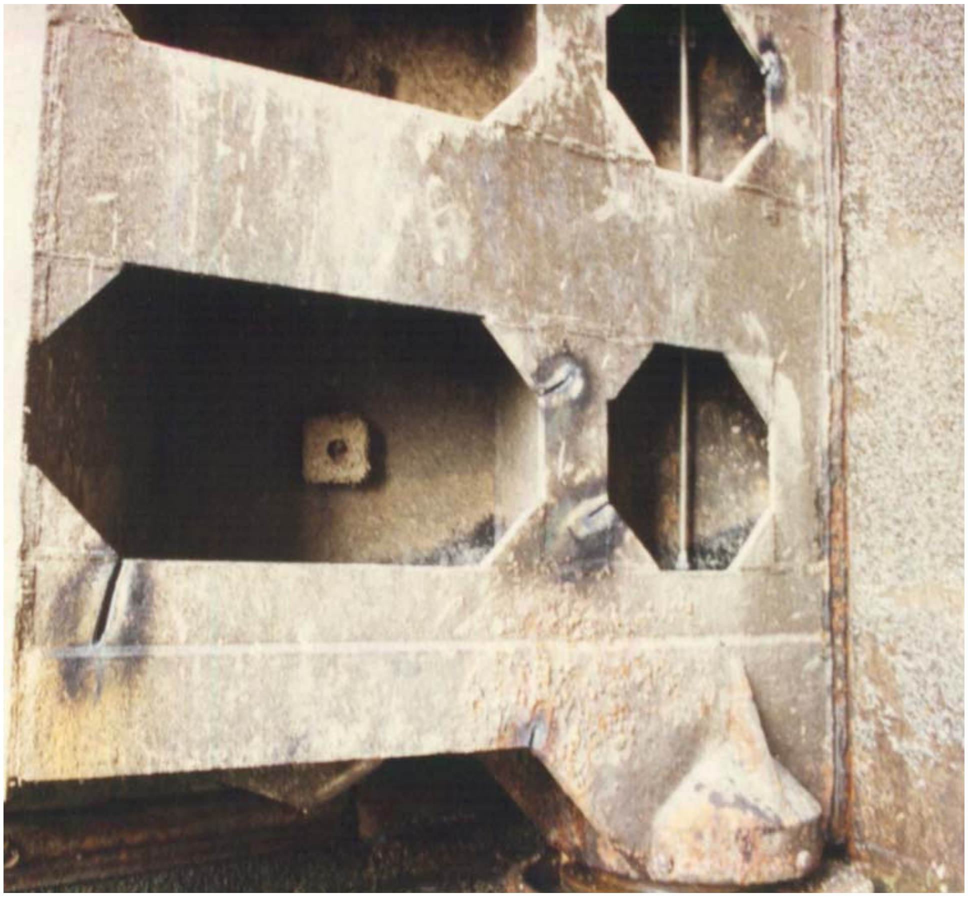


Figure B-4. Main chamber miter gate crack repair in pintle region

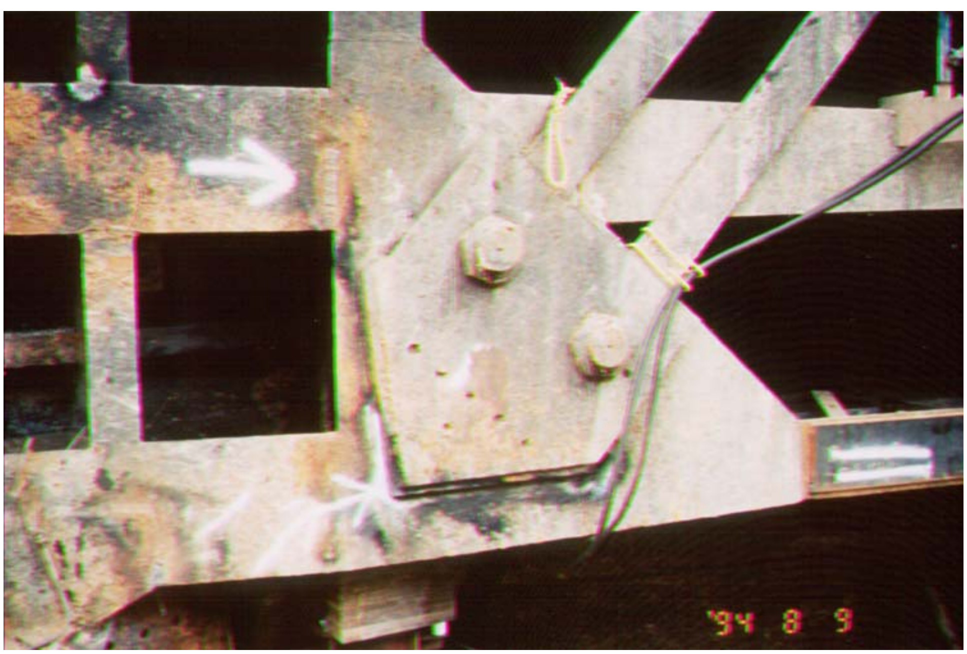


Figure B-5. Main chamber miter gate cracking near diagonal plate

Figure B-6. Miter gate damage to lower girder downstream flange

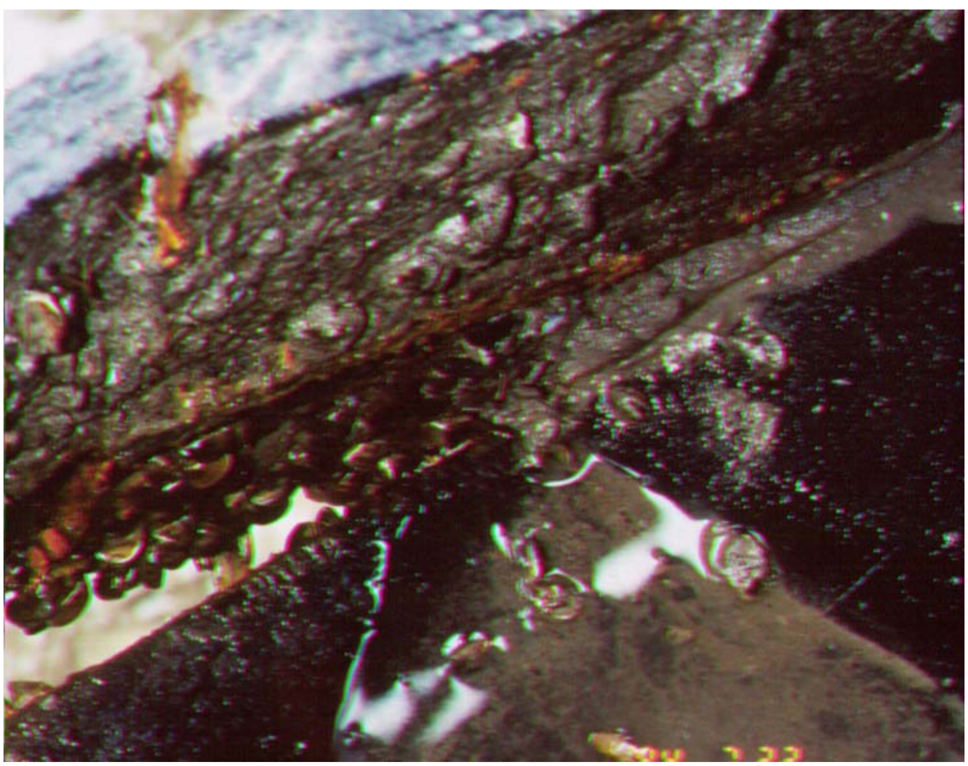


Figure B-7. Miter gate damage to lower girder downstream flange

B-10. Finite Element Modeling of the Miter Gates and Calibration.

a. The fatigue cracking problem at welded flange connections on the Markland miter gates was summarized in previous paragraphs. To evaluate the fatigue cracking problem from a reliability standpoint, the initiation and growth of the fatigue cracks must be characterized in terms of the variability of the parameters that control the fatigue cracking. The development of such a reliability model has three major components: to determine the characteristics and variability of the initiation of fatigue cracks, to establish the rate of crack growth and its variability, and to determine the limit state of the gate, which is defined as the extent of fatigue crack growth that will compromise the integrity of the gate. This determination of the limit state of the miter gate is described in paragraph B-11. The fatigue crack initiation and growth are influenced primarily by the residual stresses that develop during the welding of the girder flange and vertical stiffener flange. Large tensile residual stresses can develop in the flanges around the welded area caused by constraints against thermal expansion (and contraction) during the welding process. The arch action of the gate under hydrostatic operating loads develops compressive stress in the flanges in the pintle region. These compressive operating loads, which are exasperated by the geometric re-entrant corner at the welded flange connection and the usually rough surface at the weld bead, produce large stress cycles that initiate fatigue cracks.

b. A numerical study using detailed finite element modeling was conducted to evaluate the fatigue cracking at welded flange connections.[1] As depicted in Figure B-8, this study used global modeling of the gate leaf to define the range of compressive loads that develop in the girder flanges near the welded connections. Normal operating conditions as well as such factors as pintle wear and gate misalignment were considered. Detailed local models of the flange connection were used to establish the residual stress distributions by numerically simulating the weld process. This methodology was benchmarked against test data from the literature where stress magnitudes and distributions were measured around a weld on A36 steel as illustrated in Figure B-9. Once the residual stress field was established in the local model, the flange loads were applied consistent with the global operational loads. The stress range for a cycle of operation was determined from a gate-open condition, which includes gravity load, diagonal prestress, and residual stresses, to a gate-closed condition that adds the operational loads. This stress range was then used to evaluate the number of cycles for crack initiation based on the American Society of Mechanical Engineers' design fatigue curve for carbon steel. This calculation for fatigue crack initiation correlated very well with the observed cracking in the Markland miter gates during the 1994 and 1996 dewatering inspections.

c. The next step in the Markland study was to develop a method for evaluating the rate of fatigue crack growth. Typically, the linear elastic fracture mechanics (LEFM) based formulas for stress intensity as a function of stress level and crack length of the form,

$$K = Q\sigma\sqrt{\Pi a} \qquad \text{(B-2)}$$

is used to develop a relationship for the change in stress intensity versus crack length. This stress intensity relationship is then used with the Paris relation,

$$\frac{da}{dN} = C(\Delta K)^n \tag{B-3}$$

where *C* and *N* are material parameters (with variability) for integration to find the crack growth rate. This method is illustrated in the USACE procedure for structural inspection and evaluation of welded lock gates. However, these LEFM formulas are developed based on uniform far field

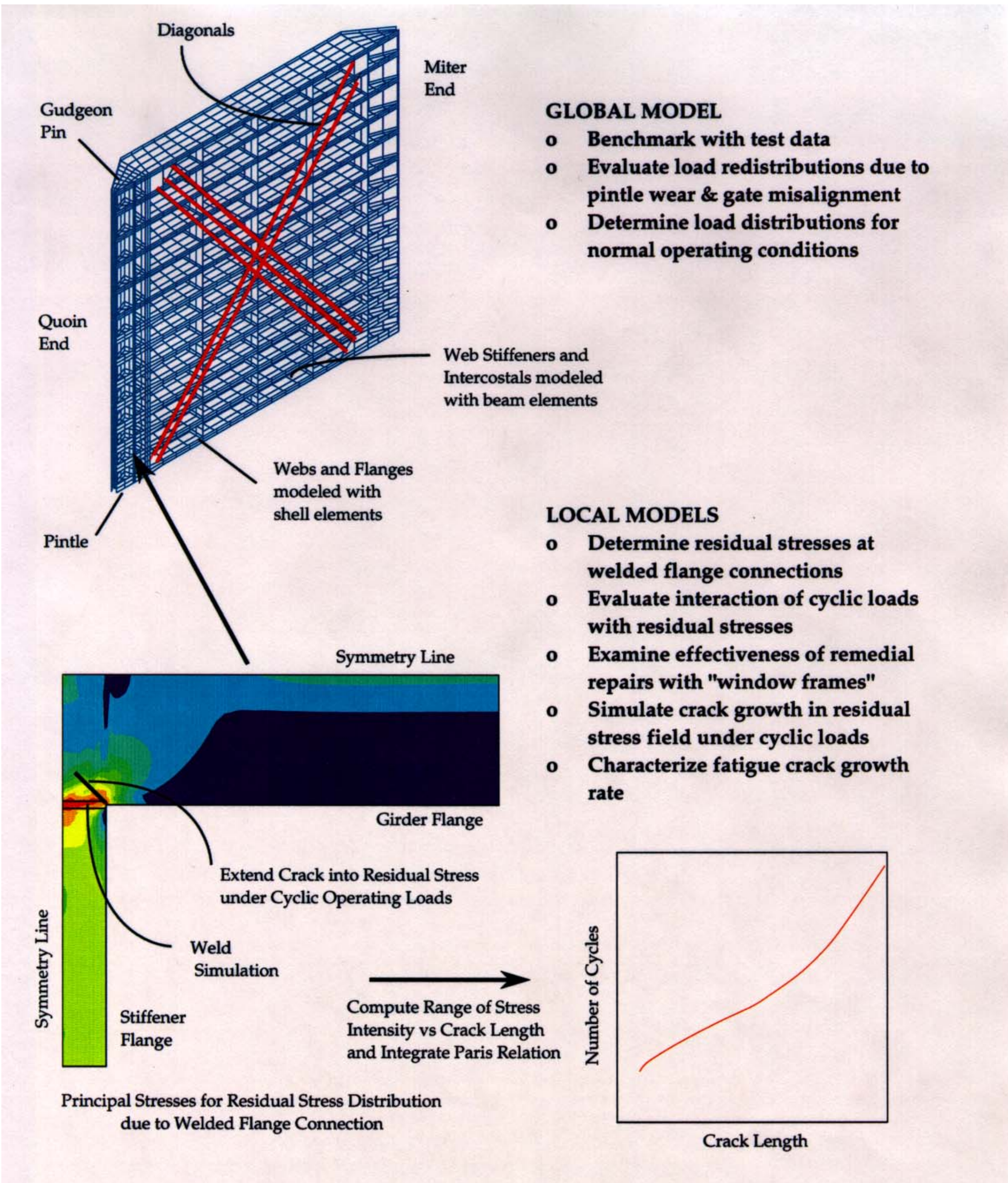


Figure B-8. Global finite element model of Markland miter gates

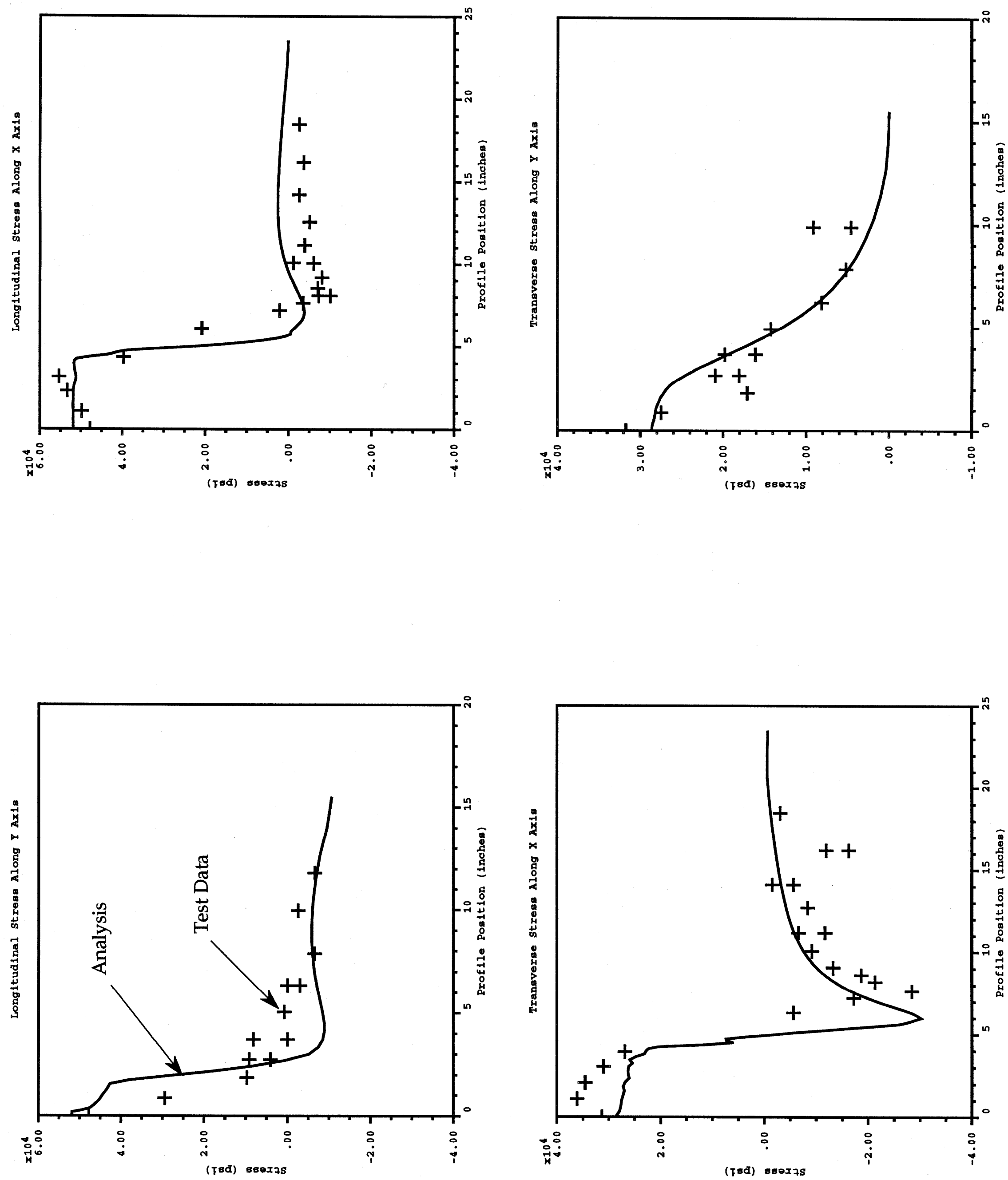


Figure B-9. Stress distribution around welds on A36 steel

stresses and, most often, Mode I crack growth. In this case, the driving stress for crack growth is the tensile residual stress distribution at the crack rather than the remote compressive flange stress. Moreover, these residual stresses change as the crack extends. Thus, another method for determining the rate of crack growth was required. The method that was developed in the

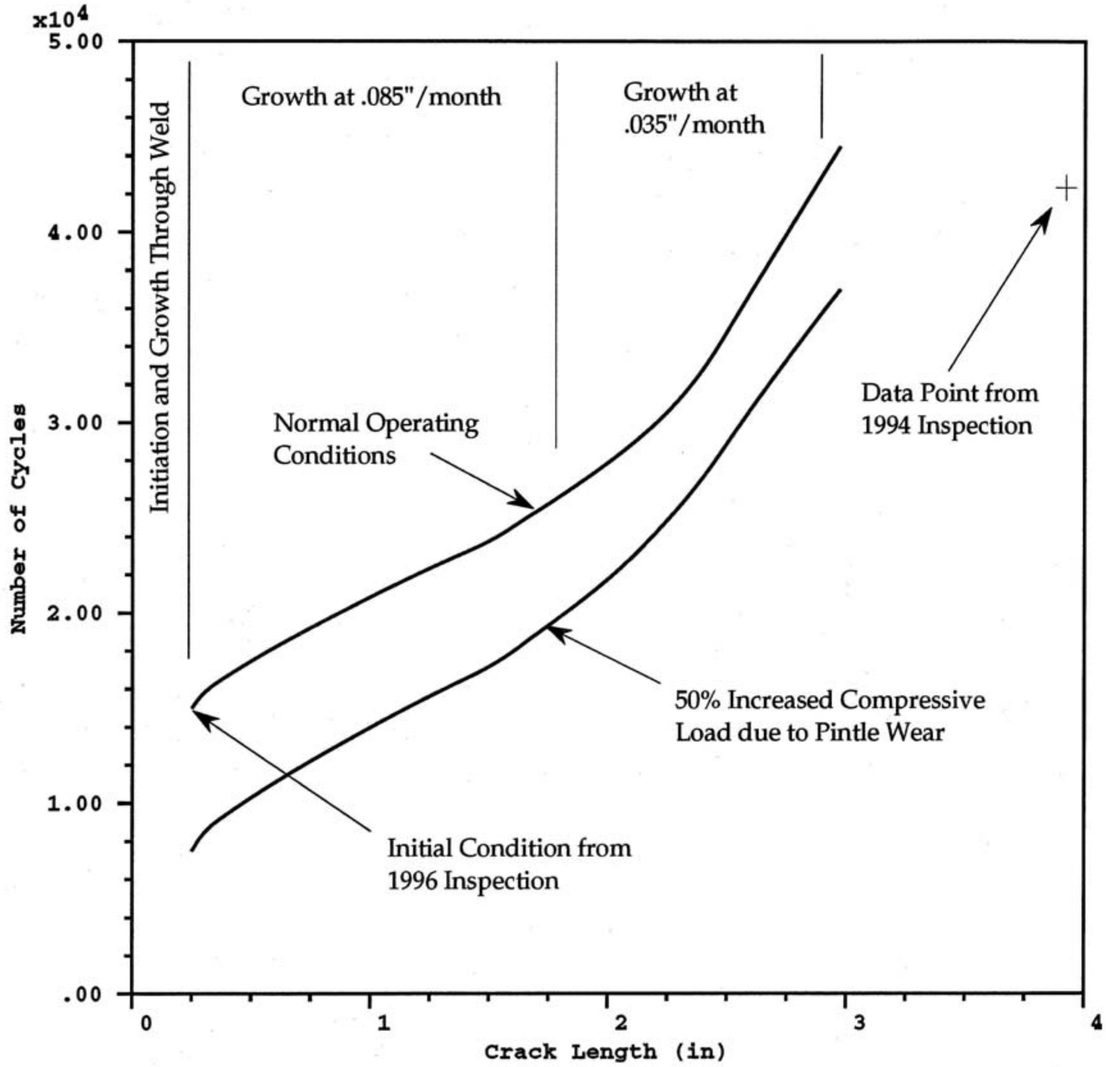


Figure B-10. Crack growth rate versus operating cycles

Markland study was to extend a crack within the residual stress field in the local finite element model and compute the resulting stress intensity value under gate-open and -closed conditions. This was accomplished using the J-integral method to calculate the energy release rate for an increment of crack extension. The stress intensity value was computed from the energy release rate using LEFM assumptions. This energy-based method also accounts for contributions to crack growth from all modes of crack extension. The Mode II or shear contribution was considered significant in this situation. Thus, a relation for stress intensity versus crack length was constructed by numerically extending a crack from the corner of the welded flange connection in the local model for gate-open and -closed conditions. The range of stress intensity versus crack length was then used to integrate the Paris relation to determine the crack growth rate of the fatigue cracks. As illustrated in Figure B-10, this calculated crack growth rate correlated very well with the observed crack lengths in the Markland gates during the 1994 and 1996 dewaterings.

d. The development of the reliability model for horizontally framed miter gates is based on this methodology. The intent of the model is to characterize the variability of the fatigue crack initiation and growth. The engineering team evaluated the importance of the parameters that influence fatigue cracking to establish the variables for characterization. A matrix of calculations is then performed with variations of these variables to develop relationships on the fatigue crack initiation and growth. The residual stress at a welded connection is influenced by many parameters, such as type of weld, number of passes, yield strength and strain hardening characteristics of the base metal and weld metal, and the degree of constraint during welding. The Markland Major Rehabilitation Study demonstrated that modeling the weld process in detail was not necessary to develop a reasonable residual stress distribution around the welded areas that govern the extended growth of fatigue cracks. Based on this work, the Engineering Team identified the material yield stress and the degree of constraint as the important random variables for developing the residual stress distribution at a welded connection. The temperature dependence and strain hardening variations are tied to the variation in yield stress. The degree of constraint is incorporated in the evaluation by considering three different types of welded connections. Thus, local models are developed for the stiffener flange to girder flange connection, the pintle casting to lower girder connection, and the diagonal anchor plate to girder flange connection. These connections represented areas of the miter gate where fatigue cracking has been observed and are considered likely to have serious reliability consequences for extended cracking.

e. The fatigue cracking is also governed by the compressive side of the stress cycle, so that the reliability model must be characterized in terms of operating stress on the connection, typically the girder flange stress, which can be related to the head variations. Finally, the crack growth is defined by the material constants in the Paris relation, and the material coefficient is also defined as a random variable. Thus, for each local connection model, analyses are conducted with material variation in yield stress to develop the resulting variations in residual stress distributions. Then variations of flange stress are applied to each variation of residual stress to develop combinations of stress ranges. That is, curves of peak tensile residual stress versus yield stress are constructed along with curves of peak compressive stress acting on the residual stress field versus nominal flange stress. These relations are then fit with equations for defining the reliability model. The variation in crack initiation is characterized by evaluating the variation in cyclic stress range for given values of the random variables and using the American Society of Mechanical Engineers fatigue design curve to define the allowable number of cycles for crack initiation. A variation on the fatigue design curve was not considered necessary since this curve has been adjusted for material variation and because the results using this method benchmarked very well with the observed crack initiation on the Markland gate.

f. The variability of the fatigue crack growth is developed in a similar manner. Cracks are extended in the variations of residual stress distributions for different variations of operating flange stresses to develop families of curves for stress intensity versus crack length. These variations are then used to integrate the Paris equation with variations in the material constant to develop families of curves for crack length versus number of cycles for the variations in yield stress, flange stress, and fatigue rate coefficient. An equation is then fit to these data and the incremental form used to return a increment in crack extension for a given number of cycles for current values of the random variables.

B-11. Limit State Selection for Miter Gates.

a. The methods and procedures used to characterize the initiation and growth of fatigue cracks at welded connections were described in paragraph B-10. The next component in the reliability model is to define the limit state of the gate, which is the extent of fatigue cracking that will compromise the integrity of the gate. As the fatigue cracks grow into the flanges, the effective area for compression loads and the effectiveness of the flange in preventing buckling of the webs are reduced. In the quoin region, where compressive loads are high, buckling of the girder webs could lead to progressive failure of the gate. The limit state of the gate is thus defined by considering the effect of the degradation on the buckling characteristics for the growth of fatigue cracks. A baseline for the margin against buckling under normal operating loads is first established for the undamaged gate. Fatigue cracks are then extended in the global model by disconnecting elements in the mesh. Buckling calculations are conducted for increasing levels of damage until the limit state is reached.

b. For these redundant structures, local buckling can be tolerated without seriously compromising the gate integrity. Local buckling of girder webs in diaphragm bays is known to occur without serious consequences. In the buckling calculations, an eigenmode method is used to find a factor (eigenvalue) on the operating loads to produce zero stiffness in the associated buckling shape (eigenvector). A sequence of buckling shapes and associated load factors is determined. A criterion must be established for the buckling characteristics that defines a limit state for the gate. The criterion defined for this study is that any of the following conditions warrants a limit state that compromises the integrity of the gate:

(1) A buckling mode that extends over more than one girder (global buckling).

(2) A buckling mode that extends over more than half of a girder.

(3) A load factor of less than 1.1 for the lowest buckling mode.

c. Since the buckling characteristics are highly dependent on initial imperfections and the buckling calculations consider only nominal (perfect) geometries, the last criteria for a 10 percent safety factor is deemed appropriate. The buckling calculations also do not consider the progressive nature of buckling in that each calculated buckling mode is independent of the previous modes occurring with smaller load factors.

d. For each type of connection, the limit states are determined by progressively incorporating fatigue cracking damage into the global model and evaluating the buckling characteristics against the above criteria. Table B-4 summarizes the levels of damage found to constitute limit states for the gate under fatigue cracking damage. The level of damage needed for failure caused by cracking at the pintle casting connection and for the diagonal anchor plate to girder flange connection was found to be much greater than for the stiffener flange to girder flange connection in the pintle region. In addition, the crack initiation phase is typically longer and the growth rate slower from lower compressive working stresses at these connections. The residual stresses are also lower because there is usually less constraint at these connections during the welding of the connection. In addition, the pintle casting weld is very redundant and

Table B-4. Levels of Damage for Limit State of Markland Miter Gate

Type of Connection	Level of Damage Required for Gate Instability
Girder flange to stiffener flange in quoin region	Separation of girder flange on bottom two girders
Girder flange to diagonal anchor plates at quoin region	Cracking through flanges and into girder web for one-eighth of web depth on bottom two girders
Welded pintle to bottom girder	Extensive cracking required. Will not govern fatigue life

also includes some bolting of the connection. Therefore, it was concluded by the Engineering Team that the stiffener flange to girder flange connection is the controlling case for reliability of the Markland gate for fatigue cracking. Figure B-11 illustrates the buckling mode for the undamaged Markland gate. Figure B-12 illustrates the level of damage needed to compromise the integrity of the gate from buckling of the girder webs in the quoin region. This level of damage basically renders the horizontal flanges completely ineffective in supporting the webs on the bottom two girders.

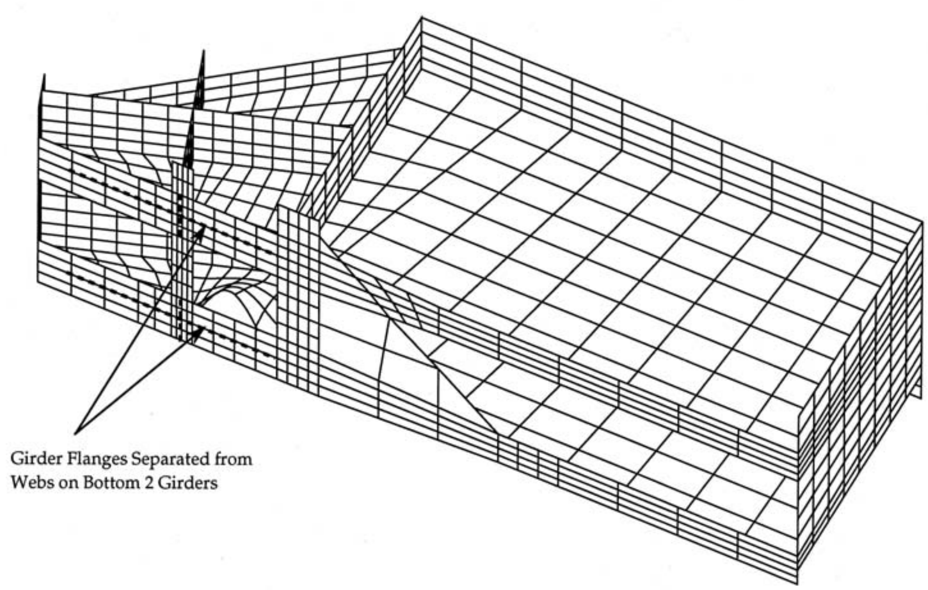


Figure B-11. Buckling damage of Markland gate
from finite element model

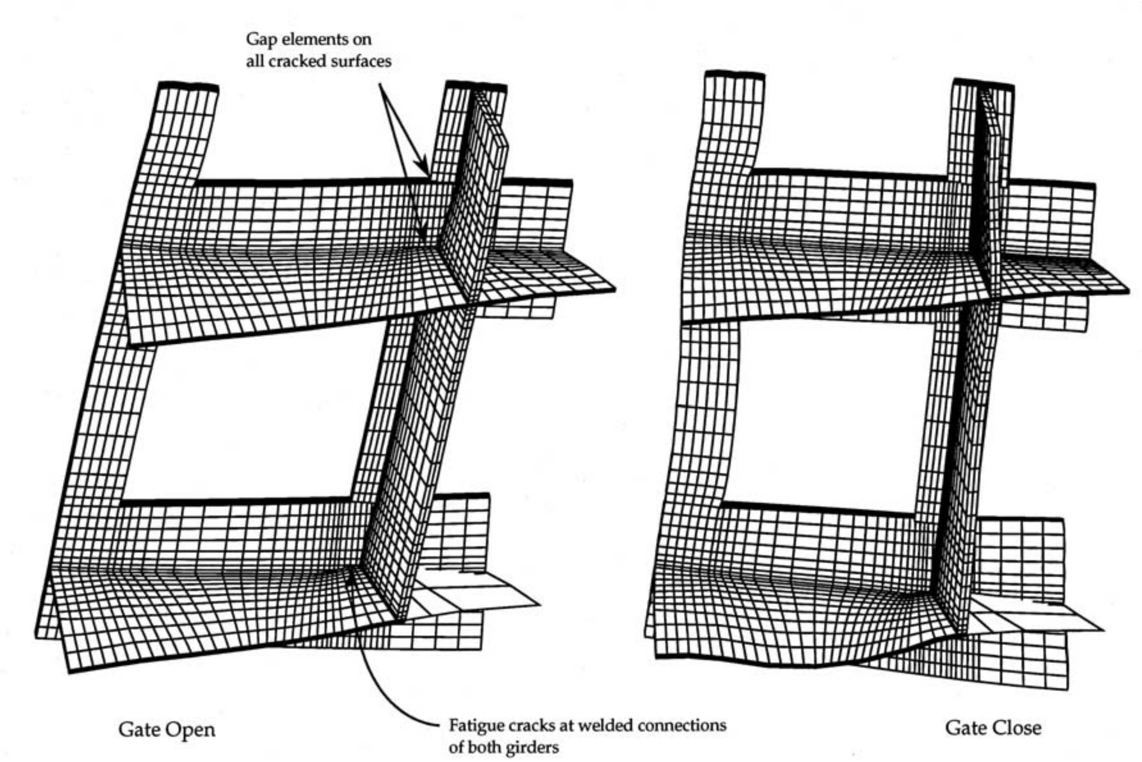


Figure B-12. Buckling damage required for major failure of the miter gates

e. The first scenario investigated involved cracks initiating in the girder flange at the corner of the connection and growing through the flange width to reach the web. Cracking needed to initiate and proceed from both the top and bottom of the flange and at the connections on both ends of the span along the web between stiffeners. However, as this type cracking developed, the global model showed that the resulting load redistribution in the gate would inhibit the continued crack growth at two of the opposite corners of the flange connections. The detailed local models also indicated that while the crack starts along a 45-deg angle from the corner of the connection, the residual stress field would cause the crack to turn horizontally toward the stiffener web. This led to the conclusion that the fatigue crack would turn and grow into the secondary residual stress field of the welded connection, joining the stiffener web on the underside of the girder flange. Because of the continuous tensile residual stress along the flange to girder connection, the fatigue crack is likely to have a fairly constant growth rate during this mode for very long crack lengths. As the cracking extends toward the girder web along the stiffener to girder flange connection, the large compressive loads in the girder will then cause the cracking to continue along the girder web to girder flange connection. This type of cracking at the girder web to girder flange connection has been observed in the Markland gate in the diaphragm bay next to the quoin region. This cracking will completely separate the flange from the web, leading to buckling of the web in the highly compressive load region. Because the local models of the welded flange connection considered only the growth of the fatigue crack in the girder flange, an additional local model was developed to define the growth rate of the crack along the flange to web connection. This model required three-dimensional finite element modeling because of the geometry involved.

B-12. Without-Project Condition Scenarios. Three Without-Project Conditions repair scenarios were considered for the miter gates at Markland: the baseline condition, advanced maintenance strategy, and finally, the scheduled repair strategy.

a. Baseline condition. The baseline condition represents the current method of operation for the Louisville District Operations personnel concerning lock maintenance. In general, the main chambers are dewatered at 5-year intervals for inspection and routine maintenance on the entire chamber. These dewaterings are usually 10 to 15 days in duration, and repair work consists of inspection of the miter gate, along with minor repairs. Additionally, an overall inspection of the chamber is completed including machinery and valves. However, every 15 years (or the third dewatering of the 5-year cycle) significantly more maintenance to the chamber is conducted. At this dewatering, the miter gates are jacked in place and pintles, seals, and quoin/miter blocks are reworked or replaced. Other chamber work also takes place during this dewatering such as culvert valve repair, gate and valve machinery work, and clearing the culvert of debris build-up. These larger dewaterings usually take anywhere from 30 to 45 days in length. Because the work involved with the normal maintenance schedule is generally for repair/replacement of maintenance items (seals, pintles, etc.), it is assumed that normal maintenance does not upgrade the overall reliability of the gate from fatigue and corrosion. Therefore, for the reliability assessment, the baseline condition is considered a "fix-as-fails" approach.

b. Advanced maintenance strategy. The advanced maintenance condition represents additional chamber closures to repair the miter gates for fatigue and corrosion damage. Because the miter gates are considered to be in critical condition, the Louisville District is already operating under this scenario. Additional dewaterings now are scheduled every 2 to 3 years between the scheduled normal maintenance dewaterings. These additional dewaterings are strictly for the inspection and repairs to the lower portion of the miter gates. The 1994 dewatering of Markland's main chamber was done as part of the normal, scheduled maintenance plan. However, as a precautionary measure, it was decided to check the gates again in 1996 with a complete dewatering of the chamber. During this dewatering it was found that the cracks had re-initiated at the same locations where they had been repaired during the 1994 dewatering. The Louisville District has decided that the best alternative is to dewater the main chamber every 2 to 3 years for miter gate inspections and repairs. This will be done until the gates can be replaced. For the advanced maintenance, the repair procedure consists of gouging out the cracks and rewelding in place. Based upon the follow-up dewatering in 1996, it appears that the repair techniques associated with the advanced maintenance condition are effective for a limited time in slowing down the crack propagation of the welded connections. The Engineering Team that developed the reliability and finite element model estimated that a total of three repairs would be the maximum that could be considered effective to improve gate reliability over a short time period. Therefore, field experience indicates that the first repair would be effective for approximately 3 years, the second repair for 2 years, and the final repair for 1 year. The three repair years assumed for the main chamber at Markland were 1994, 2000, and 2004. Remember, it is assumed that other dewaterings besides the ones in 1994, 2000, and 2004 are for normal chamber maintenance and significantly affect reliability since they are taking care of other issues not related to the fatigue cracking problem. It is assumed that each intermittent dewatering for miter gate repairs only is assumed to take 30 days for each set of miter gates.

c. Scheduled repair strategy. This scenario represents a high-level repair to the lower portion of the miter gates. It essentially assumes that the lower portion of the miter gate is removed and replaced with new members of the same size; i.e., the new plate girders would also be built-up sections that are 70 in. deep. This work would have to be accomplished in the chamber, thus causing a significant closure of the main chamber. It is assumed that this is a one-time fix and will significantly improve the overall reliability of the miter gate, but at an expensive cost and closure of the chamber. It is assumed that the repair will take 60 days for each set of miter gates.

B-13. Reliability Model Parameters. The reliability analysis for the horizontally framed miter gates was developed to focus specifically on the type of cracking and problems that were occurring in the field. In order to accomplish this goal, the team focused its effort toward developing a model based upon the finite element analysis of the Markland miter gates. It was learned from developing the vertically framed miter gate model for ORMSS that using the spreadsheet on time-dependent models was time consuming and often made tracking changes and output difficult. After initially developing a basic model with the spreadsheet, the Engineering Team decided to develop a Visual Basic coded model specifically for the ORMSS horizontally framed miter gates and use Markland as the basis for the analysis. Therefore, the team coded their own model focusing on the cracking of the miter gates near the pintle and used @Risk libraries for the Monte Carlo simulation within the reliability model. Immediately, it was determined that the coded model served the team's needs better for this component as it was easier to track changes and make calibration runs. The model was named HWELD since it was based upon the premise of crack initiation at welded connections. This portion of the appendix gives an overview of the HWELD regarding input, output, etc.

B-14. HWELD Reliability Model Input.

a. Lock information. The first portion of input being analyzed includes the project location, chamber, and girder. For Markland, the design and construction technique are the same for all of the miter gates for both the main and auxiliary chamber gates. However, because operating cycles and age are different for the chambers, each must be analyzed separately. The input menu from HWELD for the lock information is shown in Figure B-13.

b. Cross-section properties of miter gate. The properties of the miter gate girder are required in order to compute the operating stresses in the area where the gate is susceptible to cracking. The required input for cross-section properties of the miter gate in HWELD is for the web/flanges, thrust plate, and overall gate geometry. The values are treated initially as constants but decrease over time in thickness depending upon the paint life and corrosion rate. A series of input menus guide the user through the necessary property inputs for the girder properties, thrust plate properties, and finally, the overall gate geometry.

(1) Web/flanges. The inputs required for the upstream (u/s) and the downstream (d/s) web/flanges in HWELD are the thickness and width of the flange and the thickness and depth for the web in the quoin area. The x-distance is defined as the "section cut" from the quoin contact block to the critical point of interest where cracking of the welded connection is being considered. Since cracking for the Markland miter gates is widespread in the pintle region, the

average x-distance is used for the middle diaphragm location. The HWELD web/flange property input values for the web/flanges cross-section properties for Markland are shown in Figure B-14.

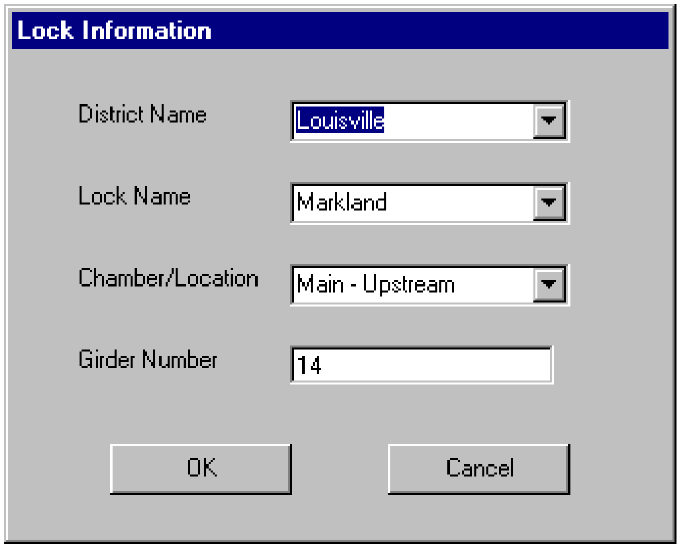


Figure B-13. Lock information input menu for
HWELD reliability model

(2) Thrust plate. The HWELD inputs for the thrust plate are the width, thickness, and the distance from the downstream (d/s) flange. The HWELD input for the thrust plate cross-section properties for the miter gates at Markland is shown in Figure B-15.

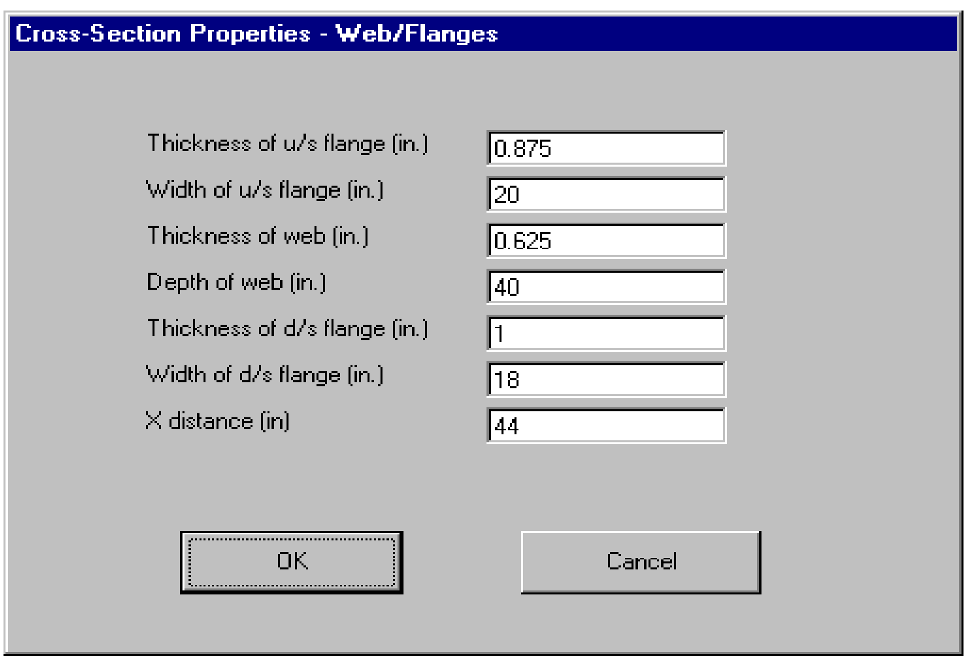


Figure B-14. HWELD web/flange properties input menu

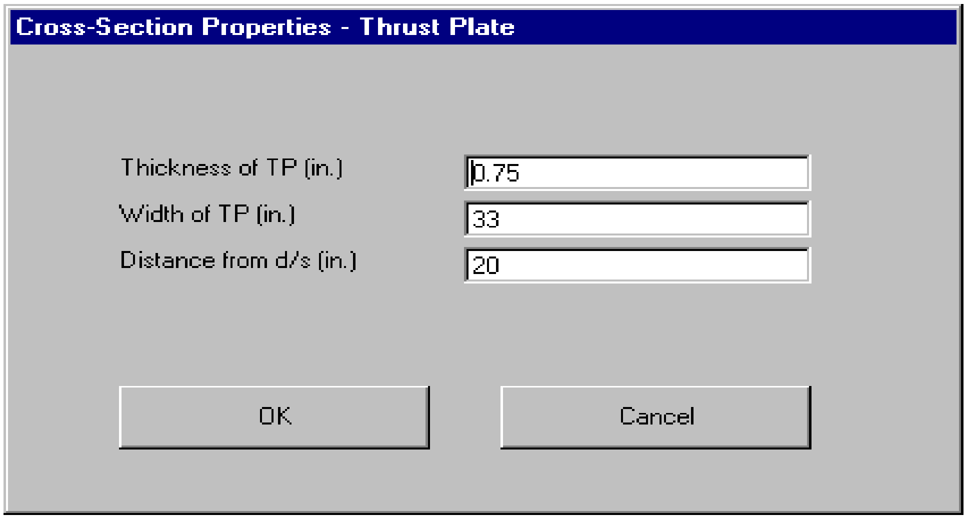


Figure B-15. HWELD thrust plate property input menu

(3) Geometry. The required inputs for the geometry of the horizontally framed miter gate are the gate height, spacing of girders, skin plate thickness, and working length. Other data are input into HWELD and not directly used in the reliability calculations. These data are used only for information and include such items as the gate height, length of girders, and tangent of angle of orientation of the girders. The HWELD input values for the Markland miter gates are shown in Figure B-16.

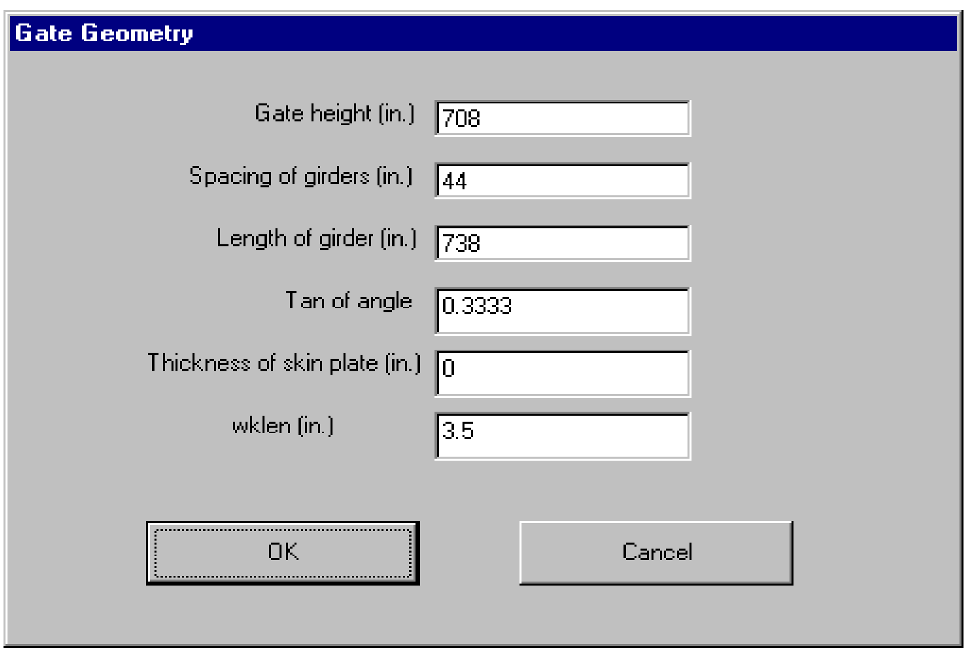


Figure B-16. HWELD gate geometry input menu

c. Crack parameters. The crack parameters required for the HWELD program are the initital crack length, the flange crack length, and the critical crack length. The initial crack length is set to a default value of 0.25 in. This value is based on the results from the finite element analysis discussed in the previous section. The flange crack length is the distance from the initial crack through the flange to the web. The critical crack length is defined as the critical distance along the web and flange welds to which the limit state buckling of the thrust plate occurs. The crack parameters input values for the Markland miter gates are shown in Figure B-17.

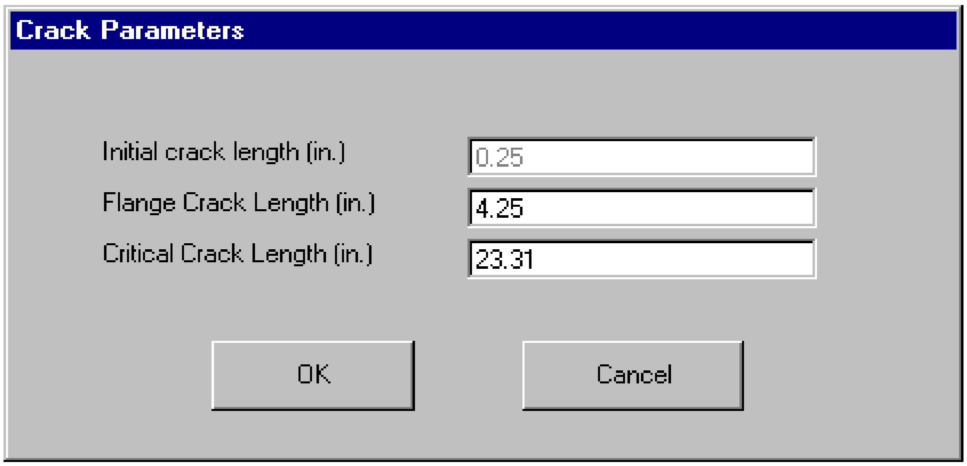


Figure B-17. HWELD crack parameters input menu

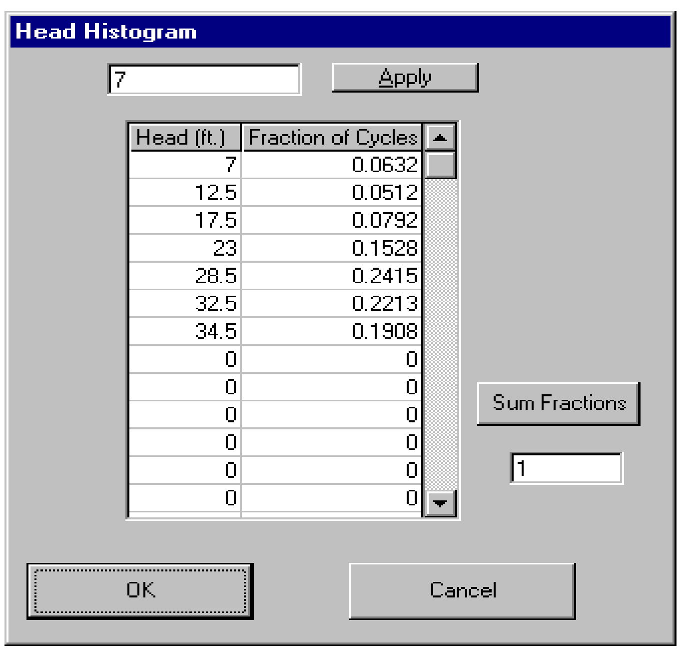


Figure B-18. HWELD head histogram input menu

d. Head histogram. The head histogram reflects the actual past distribution of head differential and hydraulic cycles for the Markland miter gates. This distribution is based on true daily lockage cycles available from the Lock Performance Monitoring System (LPMS) combined with the true head differential for each day. This distribution is valuable in determining the fraction of annual cycles versus the expected head differential that is used for fatigue analysis. The head histograms developed by the U.S. Army Engineer Waterways Experiment Station are based on data collected and analyzed for approximately 12 years (1984–1996) of lock operation. The HWELD program allows the input of up to 20 different blocks for head (at specified midpoints for ranges) and fraction of cycles from the histograms. This histogram is used in HWELD to parse the input annual cycles into the defined stress range blocks and number cycles for fatigue analysis. The head histogram input into HWELD for Markland is shown in Figure B-18.

e. Traffic cycles. The number of operating cycles for the gates is determined for each lock based on actual and predicted future cycles for the study period. The cycle information is used in fatigue analysis incorporated into the HWELD program. The cycles are input from the start of operation to the end of the study period. Operating cycles from the origination of the project in 1958 through 1984 were determined by going through the logbooks at Markland to determine the number of lockages in each chamber. From the LPMS data from 1984 through 1996, a ratio of lockages to operating cycles was determined and assumed to be the same in the past as well as for future projected cycles. Traffic cycles for 1985 through 1996 were determined using LPMS data. Finally, projected traffic through the end of the study period was determined by USACE Center of Expertise for Inland Navigation Planning in Huntington, WV. The traffic cycles input into HWELD for Markland Locks and Dam are shown in Figure B-19.

f. Paint history. The painting history of the miter gates can be incorporated in the reliability analysis. This directly affects the corrosion of the gate members based on the defined paint life. The input required is the specified paint life and the year in which the gates were painted. These paintings are assumed to be for the entire gate and not just spot painting of the gate. If a gate is painted after the initial paint life is exceeded, then corrosion is not invoked until the end of the paint life. Paint histories can be entered for up to three different years.

B-15. Random Variables. The random variables incorporated into the reliability analysis of the Markland miter gates are the yield strength of the steel, corrosion rate, stress concentration factors, and misalignment/pintle wear factors. These random variables are simulated using either direct Monte Carlo simulation or a modified simulation method called Latin Hypercube, and both methods are incorporated into the HWELD program. The Latin Hypercube method utilizes stratified sampling of the input distributions for quicker convergence. Pool level differential between the upper and lower pools (commonly referred to the head) is essentially a random variable because the actual histogram allows for heads in eight different ranges. Because the values are not chosen separately for each iteration, it represents a truer measure of the pool level distribution at Markland. The input distributions and statistical moments for the random variables are defined in this paragraph.

a. Yield strength. The distribution for yield strength is based on data from the published literature and previous Corps of Engineers reliability studies. The distribution is based on a truncated lognormal with a nominal yield stress of 38.88 ksi (i.e., mean yield strength times the strength ratio) and a standard deviation of 5.44. The lower limit for truncation is based on one standard deviation below the nominal (33.88 ksi), and the upper limit is based on approximately two standard deviations above the nominal (51 ksi). The distribution and statistical moments for yield strength input into HWELD for Markland are shown in Figure B-20.

b. Corrosion rate. The distribution for corrosion rate is based on the data from the published literature and previous Corps of Engineers reliability studies. Corrosion is based on a power law that has been fit to actual field data in various corrosive environments. The equation used for the corrosion is $C(t) = A{*}t^B$, where A is a random variable based on field measurements, B is generally a constant based on different corrosive environments, and $C(t)$ is the corrosion in micromils/year.[4] For this report, the mean value of A was selected based on submerged corrosion since the portion of the gate that was being investigated is always below lower pool. This distribution used for A was a truncated lognormal with a mean value of 77.33 and standard deviation of 24. The upper limit of the distribution was taken at 128 and the lower limit at 32. The value for B was a constant of 0.593. These limits and constants are based on actual field

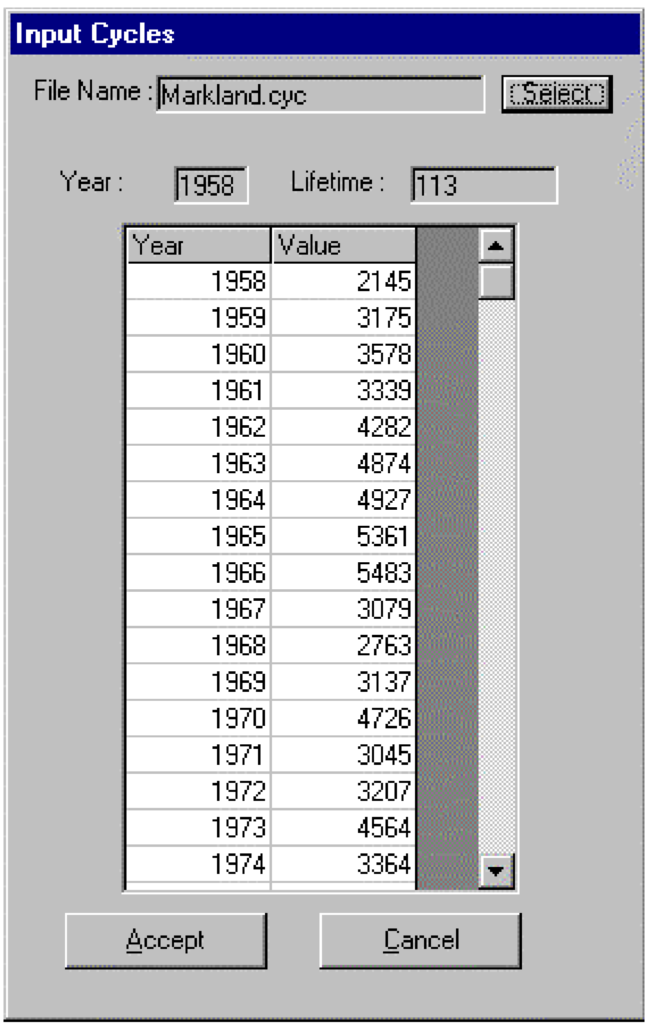


Figure B-19. HWELD operating cycles
input menu

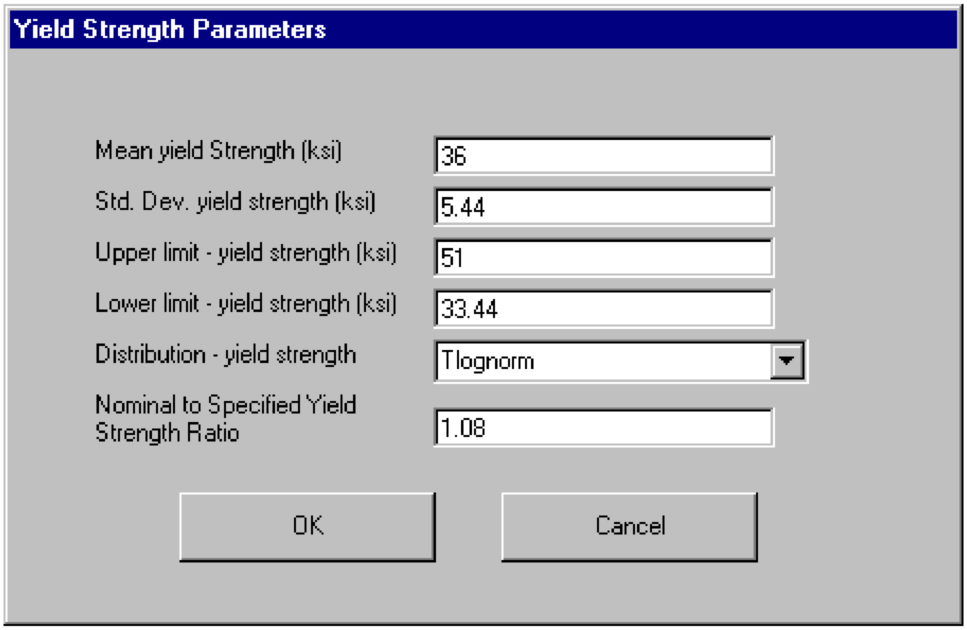


Figure B-20. HWELD yield strength input menu

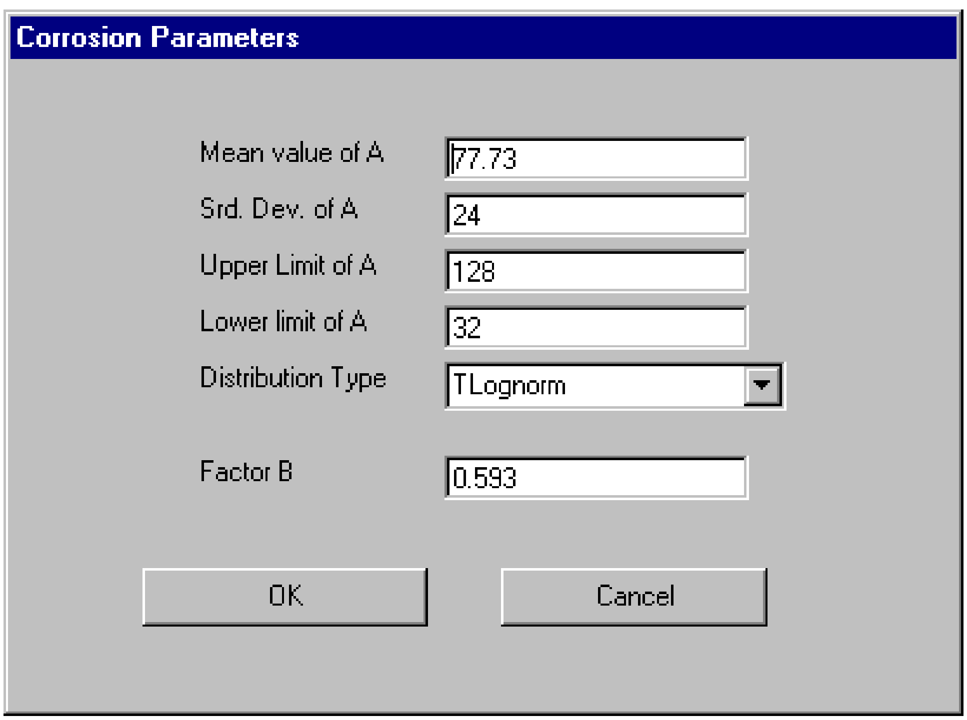


Figure B-21. HWELD corrosion parameters input screen measurement of submerged hydraulic steel structures. The distribution and statistical moments for corrosion input into HWELD for Markland are shown in Figure B-21.

c. Stress concentration and pintle misalignment/wear factors.

(1) Two types of factors are utilized in the reliability model to account for differences in stress values between traditional hand calculations and finite element analysis. One adjustment is the stress concentration factor for the intensification of the stress in the flanges near the pintle area. Additionally, a gate misalignment and pintle wear factor that accounts for an increase in stress in the girder flange during operation is provided in the analysis. The adjustment values for both the stress concentration and misalignment/pintle wear factors were based upon finite element modeling results and calibration with field test data at Markland.

(2) The distribution for the stress concentration factors was considered uniform since only the upper and lower limits can be well defined as well as the equal for the probabilities. The minimum value was determined to be 1.1 and the maximum value to be 1.4. The misalignment and pintle wear factors were determined on a percentage increase in the flange stress. A truncated lognormal distribution was selected with a mean of 20 percent with a standard deviation of 30 percent. The lower limit was 10 percent and the upper limit taken as 50 percent. The distribution and statistical moments for the adjustment factors input into HWELD for Markland are shown in Figure B-22.

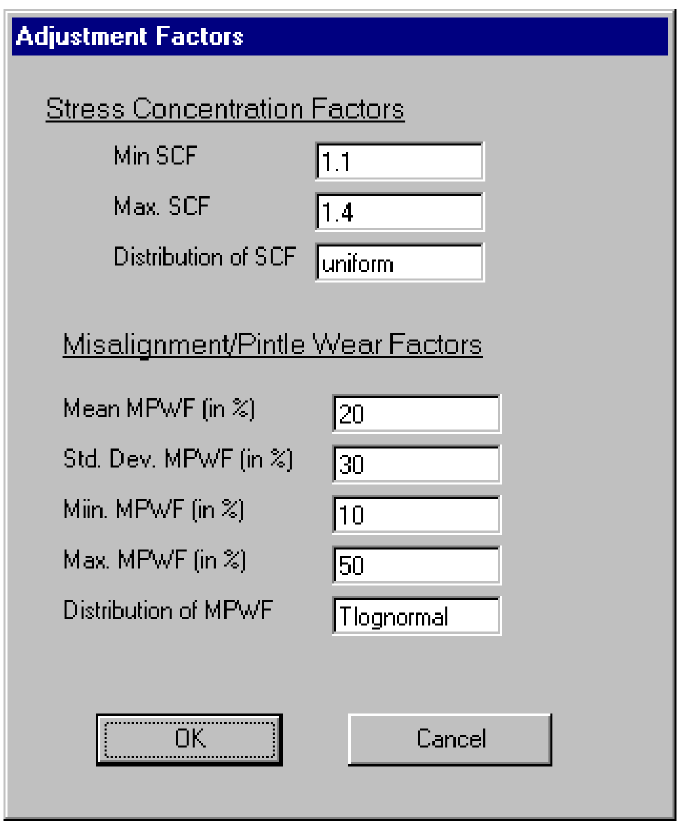


Figure B-22. HWELD analysis factors input menu

B-16. HWELD Reliability Model for Horizontally Framed Miter Gates. The computer program HWELD has been developed to complete a reliability analysis of the horizontally framed miter gates for Markland Lock and Dam. The model has been developed to test different maintenance and repair scenarios to determine their effect on the reliability. This capability permits the analysis of the gates under different levels of repair. The results will assist in selecting the preferred alternative based on cost and benefits. Additionally, the model is used to determine if it is better to replace the gates at some scheduled date as opposed to fixing them after they perform unsatisfactorily or maintaining them by a selected maintenance policy.

B-17. Reliability Analysis for Without-Project Scenarios.

a. The basis of the HWELD model is that it is a time-dependent reliability model for a structure subject to fatigue and corrosion. Therefore, input items such as paint history, corrosion rates, and other variables are used with the operating cycles to determine the time-dependent reliability of the structure. Using the analysis and limit state information from the finite element modeling, HWELD computes the time-dependent reliability of the miter gates given the input values. Two crack lengths are input into the menu shown in Figure B-17. The first is for the length required to crack through the flange, while the second is the distance from the end of the first crack length to the critical crack length for the Markland miter gates. For each iteration, the model determines the year in which a fatigue-related crack initiates and marks that year. Once the crack reaches the first length, the crack is allowed to grow relative to the operating cycles within the histogram for each year after it initiates. The crack then grows until it reaches the critical length input in the menu. Once the crack grows to the flange length, the growth rate is reset for the second growth rate associated with growth along the web/flange connection. Once the crack reaches the limit state crack length, the year is tracked, recorded and marked as the year of unsatisfactory performance. This is done for each iteration, and the results are tabulated in a separate file.

b. As noted previously, one of the capabilities of the HWELD program is that it allows the user to determine the reliability of the miter gates under different repair scenarios. This is required by guidance in order to investigate all possible solutions to the problem before replacing the miter gates. Three scenarios are measured in HWELD. The first is the baseline condition, which represents a fix-as-fails repair scenario. Repairs are not initiated on the miter gates until they fail. The second scenario is the advanced maintenance scenario, while the final scenario is the scheduled repair strategy. The implementation of each of these within the HWELD model is detailed in (1)-(3) below.

(1) Baseline condition. The baseline condition is generally the way that maintenance is performed at each project today. This is typically inspection and repair during scheduled dewaterings with no improvement to the overall reliability of the gates. The baseline condition for the miter gates assumes that the structure does not receive any major rehabilitation, painting, or repairs from the start of operation to the end of study period. The baseline condition also assumes a paint life of 20 years and that corrosion of the girder members occurs over the remaining study period. The corrosion rate is always assumed to be for a submerged structure since the portion of the gate that is being investigated is below lower pool.

(2) Advanced maintenance. The advanced maintenance strategy builds upon the baseline condition to include additional closures solely for repairs to the gates. These repairs are assumed to keep the reliability of the gates constant for a short specified period of time. This is in contrast to the baseline condition, for which the reliability continues to degrade each year. The advanced maintenance strategy in HWELD allows input of up to three repairs during the study period. The program shifts the time index 3 years for the first year of repair, 2 years for the second year of repair, and 1 year for the third repair year. This permits a slight extension in the life of the gate but at the expense of significant chamber downtime to complete repairs. The input menu for the advanced maintenance strategy in HWELD is shown in Figure B-23.

Figure B-23. Advanced maintenance strategy input menu

(3) Scheduled repair. The scheduled repair strategy is the baseline condition with a high level of repair that involves extensive reworking of the miter gates. This scenario provides the extension of gate life but at a lower level of reliability than for a new gate. Only one scheduled repair is permitted in HWELD. At the year selected for repair, the HWELD model loops back into the crack initiation phase for the gate. However, because these types of repairs are not considered to be to the reliability level of a new gate structure, the crack initiation will occur more rapidly than in a new gate. The HWELD program accounts for this through a modification to the cumulative damage factor in the program. For Markland, this value for the cumulative damage factor was calibrated and set to 0.5 instead of the original value of 1.0. The input menu in HWELD for the scheduled repair strategy is shown in Figure B-24.

Figure B-24. Scheduled repair strategy input menu for HWELD

B-18. <u>Calibration of HWELD Reliability Model</u>. The HWELD reliability models were calibrated based on field data of crack lengths for Markland. These measurements and repairs were taken at two points in time (1994 and 1996) during lock dewaterings to fix and repair cracks in the welds in the pintle area. In addition, instrumentation of these gates during this time was used to calibrate the loads calculated at critical locations in the finite element model. Since the HWELD program is based on the realistic flange stresses for the head values of the miter gates at Markland, the crack lengths and expected probability of failures determined from the model match well and support the field data.

B-19. <u>HWELD Reliability Model Results and Event Trees</u>. The Engineering Team is required to take the results from the reliability model, hazard functions for time-dependent components, and supply them to the Economics Team for their analysis. Additionally, the Engineering Team supplies an event tree for each component that is used in conjunction with the reliability analysis for the economists to measure the economic impacts associated with each component.

a. Baseline condition for miter gates.

(1) The baseline condition represents a fix-as-fails plan for the reliability analysis. It is assumed that any repairs to the miter gate during normal scheduled dewaterings do not upgrade the reliability of the miter gate because these repairs typically consist only of replacing pintles, miter and quoin blocks, etc. These repairs do not affect the reliability of the miter gate based upon the limit state set up in the reliability model. Therefore, the reliability of the structure is allowed to degrade through time without repairs under the baseline condition.

(2) For the purposes of this study, the hazard function *h* is defined as the probability of unsatisfactory performance in a given year assuming it has survived up to that year. The formula for this is depicted in Equation B-4:

$$h(t) = \frac{\text{number of failures in year t}}{\text{number of remaining survivors up to year t}} \qquad \text{(B-4)}$$

(3) The main chamber at Markland became operational in 1958. For the baseline condition, the hazard rate for the main chamber gates has an annual value of zero up through year 1986. The main chamber hazard rate quickly increases once initial failures begin. Initial failures begin in 1987. The hazard rate reaches 1.0 percent in 1992, 5.3 percent in 1996, and 10.7 percent in 1999. All iterations fail by the year 2015. The time from when failures initiate in 1987 to when all iterations fail, by year 2015, is 28 years.

(4) The auxiliary chamber at Markland became operational in 1959. The hazard rate for the auxiliary chamber miter gates first reached a non-zero value in 2007. The value reaches 1.3 percent in 2012, 5 percent in 2017, and 10 percent in 2021. All iterations fail by year 2043. Therefore, from the time failures initiated in 2007 to when all iterations fail is a total of 36 years. This compares to the 28 years versus the main chamber, again, a function of the number of operating cycles over time. As noted previously, the design and construction technique for the both the main and auxiliary chamber miter gates is the same. The only differences affecting the reliability are the age of each miter gate and the number of operating cycles, both historic and projected, that the gate will undergo.

(5) The baseline condition hazard functions for both the main and auxiliary chamber miter gates are provided in Figure B-25. As evidenced by the graph, the main chamber has a considerably higher hazard rate because both the historical and future projected number of operating cycles is much higher for the main chamber than for the auxiliary. The annual hazard rates (the overall hazard function) are provided to the Economics Team for the miter gates of each chamber, along with an event tree for the baseline condition, in tabular form for use in their economic models.

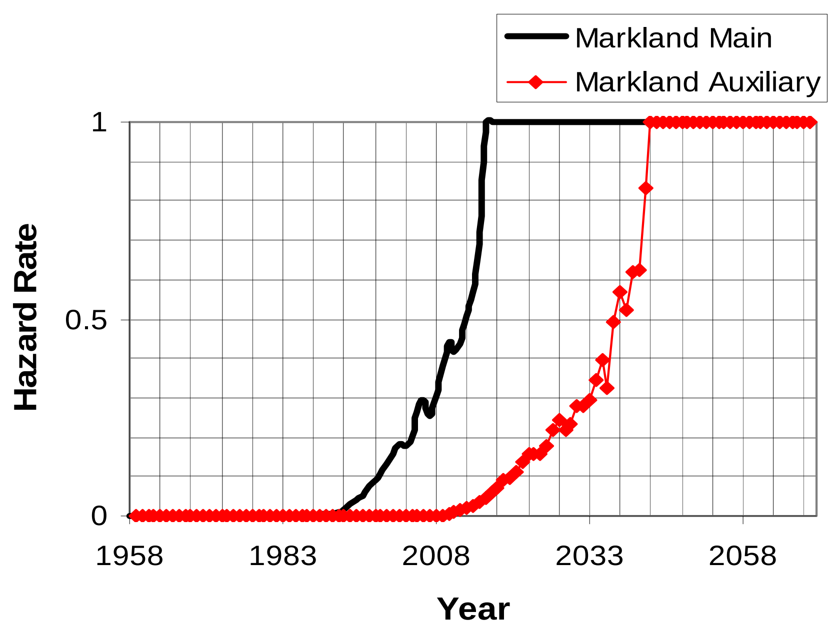


Figure B-25. Hazard rates for main and auxiliary horizontally framed miter gates

(6) As noted above, the baseline condition will be the scenario upon which all Without- and With-Project alternatives will be compared. Assuming the limit state for the miter gates as previously described, the event tree shown in Figure B-26 was developed for the horizontally framed miter gates. Regardless of the level of damage and repair option selected, the event tree represents a fix-as-fail scenario for the baseline condition. Thus, the repair is initiated in the economic model only once the gate “fails.” The same event tree is used for both the main and auxiliary chambers. The first branch of the event tree represents the annual hazard rate for the

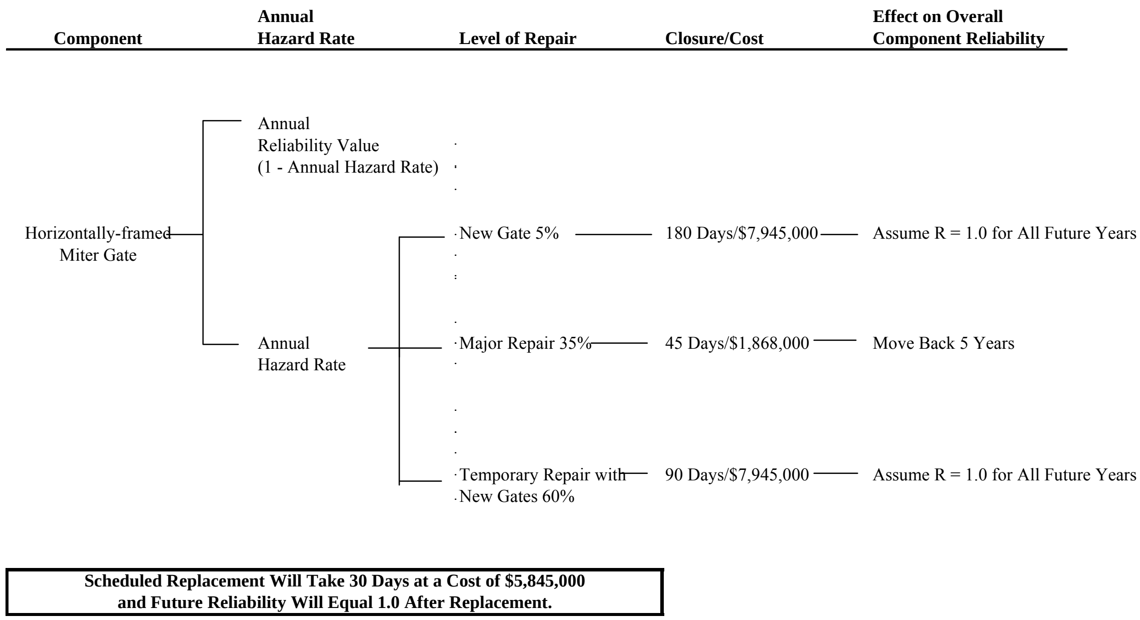


Figure B-26. Horizontally framed miter gate event tree

component. The hazard rate changes depending upon the chamber that is being investigated. The second branch is the various options associated with the level of repair for the miter gates. Since the limit state is based upon a major failure, minor repairs were neglected in the event tree. The group decided that minor repairs to the miter gates are taken care of during normal maintenance dewaterings and they do not affect the overall gate reliability. The percentages associated with each level of repair were determined from engineering judgment in consultation with Operations personnel. Associated with each of these repairs is a repair cost and chamber closure time. The loss of project benefits associated with the chamber closure IS modeled in the economic analysis. Finally, the last branch updates the reliability in the next year based upon the repair. A further breakdown of the event tree from the level of repair forward is provided in (a)-(c) below.

(a) Catastrophic failure, install new gates. This repair assumes the most catastrophic event, a total failure of one set of miter gates that is not repairable to the point that the chamber can be made operational. This repair assumes a single set of new gates are fabricated, delivered, and installed within 180 days. This would be possible only since the miter gates have already been designed and plans developed by the Louisville District (LRL). Additionally, a repair cost of $7,335,000 is assumed for this repair. It is known that the Louisville District Operations Repair Fleet costs on average about $35,000 per day including materials for repair work. The assumption is made that the repair fleet would need to be onsite for half the entire closure period. Additionally, the Great Lakes and Ohio River Division (LRD) gatelifter crane will cost about $6,500 per day. It would be required for only about 30 days. Therefore, the repair costs for the new gate repair level are determined as follows:

LRL Operations Repair Fleet Daily Cost:	$3,150,000 ($35,000 per day for 90 days)
LRD Gate Lifter Crane:	$ 195,000 ($6,500 per day for 30 days)
Assembly Area Construction	$ 600,000
New Set of Miter Gates Built and Delivered:	$4,000,000 (fabrication, delivery new gates)
Total for All Items:	$7,945,000

Because this is the most unlikely of the three chosen repair scenarios, the team placed only 5 percent on this level of repair. Future reliability of the miter gates would be considered to be 1.0, since the new gates would be installed by the next year and these would not be prone to the same type of problem as the present gates. In addition, safety factors would be applied in the design through a combination of factored loads and reduced capacity that is typical for designing new miter gates.

(b) Major repair. This repair assumes the gates have major damage, but can be repaired to the point that new gates are not immediately needed. Therefore, the closure time is reduced to 45 days with a repair cost of $1,868,000. This cost is developed from the repair fleet rate ($35,000 per day) plus the LRD gatelifter crane ($6,500) per day. Since the existing gates are placed back in service, it is assumed that the reliability has the net effect of pushing the hazard rate back to the value from 5 years previous to the unsatisfactory performance. This was an easy way for the economists to upgrade the reliability of the gates within their model based upon a lower level of repair than a new component. It was assumed that this level of repair is much more likely than a new set of gates, but less likely than the temporary repair with new gates in the following year. Therefore, it is assumed that this option would be selected 35 percent of the time.

(c) Temporary repair with new gates following year. The group envisioned the most likely repair scenario to be the one where the gates suffer major damage, but can be "patched up" to the point that the chamber is operational. However, the damage is too great to risk having the gates used for an extended period. Therefore, new gates are constructed and delivered to the site for installation by the following year. The repair cost associated with this alternative is assumed to be $7,945,000, but the chamber is closed only for 90 days. The closure is assumed to occur in two phases: an initial 45-day dewatering for the repair to the existing set of miter gates to get the chamber operational and another 45-day dewatering required later in the same year to install the new set of gates. Therefore, this scenario requires 90 days of repair fleet time at the lock at $41,500 per day including the LRD gatelifter crane. The team thought this was the most likely repair scenario given a "major" unsatisfactory performance event and placed 60 percent on this level of repair.

(7) A final piece of information the Engineering Team needed to supply in the event tree was the cost of a scheduled replacement for a set of miter gates ahead of any failure. Because the replacement is scheduled in advance and advance work is completed prior to dewatering the chamber, the chamber closure time and "repair" cost are reduced compared to replacing the gates only after they fail. The estimated cost of $5,845,000 includes $4,000,000 for a new set of gates and $1,245,000 to install them. Additionally, a cost of $600,000 is assumed for the assembly area. The economists will use the scheduled, advanced replacement cost and closure in their analysis to determine if it is more economical to replace the miter gates in advance before they perform unsatisfactorily. The scheduled replacement cost is shown in the event tree branches in Figure B-26.

b. Advanced maintenance condition.

(1) The basic information regarding the assumptions for the advanced maintenance condition was described previously in this example. To briefly review, additional chamber closures are scheduled strictly to upgrade the reliability of the miter gates by gouging out and rewelding heavily cracked areas of the gate. It is assumed that this can be done a maximum of three times during the life of the miter gate with each repair being less effective in halting crack growth. The three repairs are assumed to be between 4 and 6 years apart. The first repair is considered effective for 3 years, the second repair for 2 years, and the final repair for 1 year. The first repair of the main chamber gates at Markland took place in 1994. This sets the first year for the advanced maintenance condition. The second year was scheduled for the year 2000. The third and final repair was projected for 2004. The effectiveness of this repair method was based upon historical performance of this type of repair at Markland.

(2) The hazard function for the advanced maintenance condition for the main chamber miter gates is shown in Figure B-27. Also provided in this figure are the hazard functions for the baseline condition and scheduled repair strategy so a direct comparison can be made between the

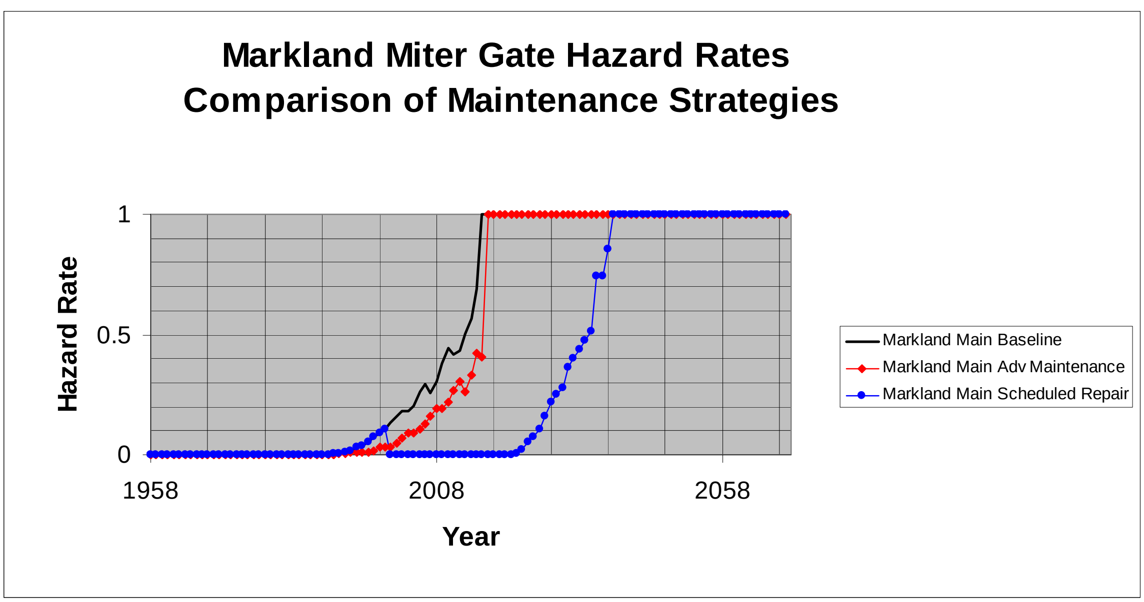


Figure B-27. Main chamber miter gate hazard rates for all without-project scenarios

scenarios. The values for the advanced maintenance condition are essentially the same as the baseline condition up to the year of the first repair in 1994. The first repair in 1994 has the effect of holding the hazard rate constant for three consecutive years, 1994 through 1996. The hazard rate begins to increase again in 1997 through 1999, until the second repair in the year 2000. The second repair holds the hazard rate value constant for 2 years, 2000 and 2001. The hazard rate again increases until the final repair in 2004, which has the effect of holding the rate constant for 1 year. For the advance maintenance condition, initial failures begin in the year 1988, very close to the same as the baseline condition. Minor differences are due only to the randomness associated with each run. The hazard rate reaches 1 percent in 1993, again very close to the baseline condition. However, the hazard rate does not reach 5 percent until the year 2002, which is 6 years later than the baseline condition. The hazard rate reaches 10 percent in 2005, again 6 years later than the baseline condition. All iterations reach the limit state by the year 2023. This is 35 years after the failures initiated. Therefore, the three repairs associated with the advanced maintenance condition have the net effect of "pushing" out the hazard rate by about 6 years, the total number of years that the repair are considered effective.

(3) The auxiliary chamber miter gate hazard functions for the advanced maintenance, scheduled repair, and baseline conditions are shown in Figure B-28. Because an advanced maintenance type of repair has not yet been done for the auxiliary chamber miter gates, an assumption had to be made regarding what year to have the first repair. The assumption was made that the initial repair would be made 2 years before initial failures were shown in the baseline hazard function. For the auxiliary chamber miter gates, the first failures show up in year 2007, therefore, the first repair for the advanced maintenance condition was set for 2005. The second repair was set for 2010, and the final repair for year 2015. The same pattern holds for the auxiliary chamber as for the main chamber regarding the effectiveness of the repairs. The first repair is effective in holding the hazard rate constant for 3 years, the second for 2 years, and the final one for a single year. Initial failures for the advanced maintenance condition show up in year 2013, 6 years later than the baseline condition. The hazard rate reaches 5 percent in year 2023 compared with 2017 for the baseline condition. A 10 percent hazard rate is not reached until the year 2027, again 6 years later than the baseline condition.

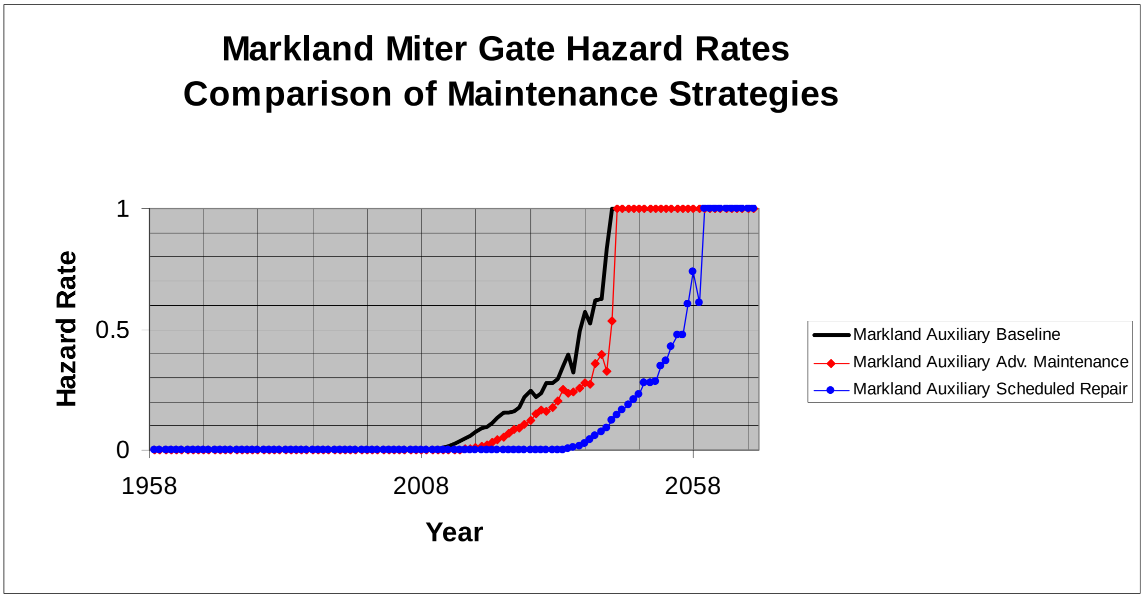


Figure B-28. Hazard rates for auxiliary miter gates for all maintenance scenarios

(4) The same event tree as provided for the baseline condition is used for the advance maintenance condition regarding disbenefits and costs associated with potential failures. However, additional information is provided to the economists regarding the three repair closures required to upgrade the reliability of the miter gates. Each of these closures is assumed to cause 30 days of chamber downtime. Additionally, the repair cost is assumed to be $1,050,000, which is equal to 30 days at $35,000 per day of Operations repair fleet time. Therefore, for the main chamber, a 30-day closure is required in the years 1994, 2000, and 2004. Also, each of these closures costs $1,050,000 for repairs only (not counting disbenefits to the navigation industry). The economists compare this alternative to the baseline to determine which is more economical. Additionally, it is compared to the option of installing new gates.

(5) The scheduled repair strategy was previously detailed for this example. To briefly review, this strategy represents a high-level repair that dramatically upgrades the reliability of the miter gates. It is assumed that the high-level repair can be done only once during the life of the miter gate.

c. Scheduled repair strategy hazard function.

(1) The hazard function for the scheduled repair strategy for the main chamber miter gates is shown in Figure B-27. Also provided in this figure are the hazard functions for the baseline and advance maintenance conditions so the three scenarios can be compared directly. The values for the scheduled repair strategy are essentially the same as the baseline condition up to the year of the high-level repair. For the main chamber, this repair is assumed to take place in the year 2000. Therefore, the hazard rate is assumed to be the same as the baseline condition up to the year 2000; then a new set of simulations is started for the repaired gate. This has the effect

of restarting the hazard rate at zero and the repair is effective for 20 years. For the repaired set of gates, failures initiate again in 2020. This dramatically upgrades the reliability of the miter gates.

(2) The hazard functions for all three scenarios for the auxiliary chamber are shown in Figure B-28. For the auxiliary chamber, the year 2005 was selected for the high-level repair. This is 2 years previous to any failures for the baseline condition. For the auxiliary chamber, the high-level repair is effective for almost 30 years because the operating cycles are much lower for the auxiliary chamber than for the main chamber.

(3) The event tree for the scheduled repair strategy is the same as the baseline condition; however, a long additional chamber closure is required to make the high-level repair to the gate. The additional closure for the scheduled repair strategy is considerably lengthier than the repairs associated with the advanced maintenance closures. The high-level repair associated with the scheduled repair strategy is assumed to be 60 days in length at a repair cost of $2,100,000. This is conservatively equal to the repair fleet daily cost of $35,000 per day times 60 days.

d. Scheduled replacement of a new set of miter gates. The final alternative is the option to replace the gates prior to failure. For the Markland Locks Rehabilitation Study, this scenario falls under the With-Project condition. As noted previously, the information required for the economic analysis for this alternative is supplied in the event tree so this can be tested and optimized within the analysis. The economic analysis uses this information to determine whether it is more economical to replace the gates or follow one of the maintenance strategies described above. Since any new gates are assumed to be designed with appropriate safety factors and incorporate improved design against the fatigue-related problems that plague the existing Markland miter gates, it is assumed that the future reliability is set equal to 1.0 for the remainder of the study period.

B-20. Economic Results for Miter Gates.

a. Using the miter gate hazard rates for each chamber for the various scenarios and the event tree depicted in Figure B-26, a direct comparison can be made of the alternative maintenance strategies for the miter gates. It is important to note that the additional closures for both the advanced maintenance condition and scheduled repair strategy are required in addition to any closures caused by unsatisfactory performance associated with the event tree. As shown in the hazard rate figures, Figure B-27 for the main chamber and Figure NAV B-28 for the auxiliary chamber, the hazard rate for the baseline condition is highest, followed by the advanced maintenance condition, then the scheduled repair strategy. As expected, there is an increase in reliability as the level of repair increases. However, serious consequences, both in chamber downtime and repair cost, are associated with both the advanced maintenance and scheduled repair strategy.

b. The economists use the data provided by the Engineering Team to determine average annual costs associated with each maintenance scenario. Additionally, the economists determine the average annual cost associated with replacing the miter gates prior to failure in various years within the study period. Each of the average annual costs associated with the maintenance scenarios is compared to different replacement dates to determine the lowest average annual cost.

The option with the lowest average annual cost sets the timed replacement of the miter gates. If the lowest average annual cost is associated with the baseline condition or any other maintenance scenario, then the replacement of the gates is not justified economically. This is done for each chamber independently. Table B-5 summarizes the average annual costs associated with the miter gates for both the main and auxiliary chambers. As shown in the table, replacing the main chamber miter gates in the year 2000 and auxiliary gates in 2025 is the optimal economic solution when considering the gates independently.

Table B-5. Average Annual Cost Associated with Miter Gates

Economic Analysis of Markland Miter Gates		
Description of Option	Chamber	Average Annual Cost
Fix-as-Fails	Main	$4,154,000
Advanced Maintenance in 1994, 2000, and 2004	Main	$5,240,000
Scheduled Repair in 2000	Main	$4,340,000
Replace in 2000	**Main**	**$1,178,000**
Replace in 2001	Main	$1,342,000
Replace in 2002	Main	$1,542,000
Replace in 2003	Main	$1,765,000
Replace in 2004	Main	$1,982,000
Replace in 2005	Main	$2,174,000
Replace in 2010	Main	$3,236,000
Fix-as-Fails	Auxiliary	$296,000
Advanced Maintenance in 2005, 2010, and 2015	Auxiliary	$526,000
Scheduled Repair in 2005	**Auxiliary**	**$257,000**
Replace in 2010	Auxiliary	$453,000
Replace in 2015	Auxiliary	$336,000
Replace in 2020	Auxiliary	$280,000
Replace in 2022	Auxiliary	$270,000
Replace in 2025	**Auxiliary**	**$267,000**
Replace in 2030	Auxiliary	$271,000

Section III
Example 3
Issue: Reliability Analysis for Vertically-Framed Reverse Tainter Culvert Valves
Project: Ohio River Main Stem Systems Study

B-21. Background. Reverse tainter culvert valves at Ohio River projects are used to control the filling and emptying of lock chambers at all sites with the exception of the upper three (Emsworth, Dashields, and Montgomery Locks and Dams). Emsworth, Dashields, and Montgomery (EDM) utilize butterfly valves for the operation of their filling and emptying systems. There are two types of reverse tainter culvert valves on the Ohio River: horizontally framed and vertically framed. Separate reliability models had to be developed for each of these reverse tainter culvert valves. In general, the older sites use horizontally framed culvert valves: Pike Island, New Cumberland, Greenup, Meldahl, Markland, and the existing main chamber at McAlpine. The newer projects have vertically framed valves: Willow Island, Belleville, Racine, Hannibal, R.C. Byrd, Cannelton, Newburgh, J.T. Myers, Smithland, and Olmsted. This example focuses on the vertically framed culvert valves.

B-22. Main Chamber versus Auxiliary Chamber Culvert Valves. The main chamber at all sites, with the exception of EDM, has a total of four reverse tainter culvert valves for filling and emptying the lock, two filling and two emptying valves. One filling and emptying valve is in the middle wall and the other set is in the river wall. They can be operated independently. Therefore, a repair to one of the main chamber culvert valves does not necessarily close the chamber. It is possible to dewater the area around the valve only, leaving the other filling and emptying set to operate the chamber. Filling and emptying time is roughly double that of normal operation. Normal filling and emptying time for a typical 1,200-ft lock on the Ohio River is approximately 8 minutes each. For the auxiliary chamber, two valves control filling and emptying operations, one filling and emptying valve each. Therefore, a problem with one of the valves on the auxiliary chamber closes the entire chamber while necessary repairs are made. The impact of closing the auxiliary chamber is considerably less than for the main chamber; however, loss of navigation benefits associated with a lengthy auxiliary chamber closure can become large.

B-23. Grouping of Vertically Framed Reverse Tainter Culvert Valves.

a. The valves are termed vertically framed since the main load from the skin plate is transferred to large horizontal plate girders by a series of vertical curved ribs. The large horizontal plate girders transfer the load to a series of axially loaded strut arms that connect the body of the valve to a pin plate casting, which transfers the load to the valve's trunnion beam. The trunnion beam then transfers the load to the concrete monolith. The valves act in tension since the tainter gate is reversed to the direction of flow. Nine projects on the Ohio River system use vertically framed culvert valves. These valves can be broken into four separate groups. The groups are classified as follows:

(1) Group 1. Group 1 vertically framed culvert valves include those found at Willow Island, Belleville, Racine, and Hannibal Lock and Dams. These valves typically have curved vertical ribs that are approximately 11 in. deep and 1/2 in. thick. The flanges are roughly 6 in. wide and 1 in. thick. Most of the horizontal plate girders are 13-1/2 in. deep by 1-1/2 in. thick with flanges that measure 12 in. wide by 1-1/4 in. thick. Additionally, all four normally operate at a head of 20 to 22 ft.

(2) Group 2. Sites considered for group 2 are Cannelton, Newburgh, and J.T. Myers. Each of these has vertical curved ribs that measure approximately 8 in. deep by 1/2 in. thick. The flanges typically measure 8 in. x 1 in.. The horizontal girders measure approximately 28 in. deep by 5/8 in. thick. All these were built in the early 1970's. It is important to note that one of the valves at Cannelton in 2002 suffered a serious failure after approximately 30 years of service.

(3) Group 3. The valves at Smithland are the only ones in this group. This is due mainly to the small flange size on the vertical curved ribs. These ribs have flanges that measure only 4 in. wide by 1-1/4 in. thick. It should be noted that there was a major failure of one of the Smithland valves in 1998 at the connection of the vertical curved rib and lower horizontal girder. At the time of the failure, the other valves at Smithland were inspected and found to have the same deteriorated condition; thus, they were also on the verge of failure. This is similar to the type of failure encountered at Cannelton.

(4) Group 4. R.C. Byrd represents the only site with valves in this group. This is because the valves are the newest ones on the Ohio River system (1993) and do not fit well within other categories for member sizes.

b. Since all the vertically framed valves on the Ohio River system are similar in construction type and operation, it was decided to develop the reliability model based upon field experience at Smithland and Cannelton. Therefore, global and local finite element models for the Smithland culvert valves were made in order to develop a time-dependent reliability model for all Ohio River Main Stem Systems Study (ORMSS) vertically framed valves. The limit state of the vertically framed valves was based upon the type of failure that occurred at the Smithland and Cannelton projects. From the global and local finite element models, appropriate adjustments were made to determine group specific load factors for such things as stress concentration factors associated with different member sizes.

c. Figure B-29 shows the vertically framed reverse tainter culvert valves being painted outside the chamber at Smithland. Note that all of the vertically framed culvert valves are of similar general design and construction technique; thus, setting up the reliability model based upon experiences at Smithland is valid. Figures B-30 through B-34 depict the damage at Smithland from the 1998 failure and the limit state selected for the valves.

Figure B-29. Smithland Culvert valve being painted (typical framing and construction of other Ohio River vertically framed culvert valves)

Figure B-30. Side view of failed Smithland valve. Note sheared rib at strut arm and offset of curved ribs above and below horizontal girder

Figure B-31. Side view of failed curved ribs at bottom horizontal girder. Note the failure of the weld at horizontal girder in second rib from end. Same weld failure occurred at second rib from other end as well. All other ribs failed in shear

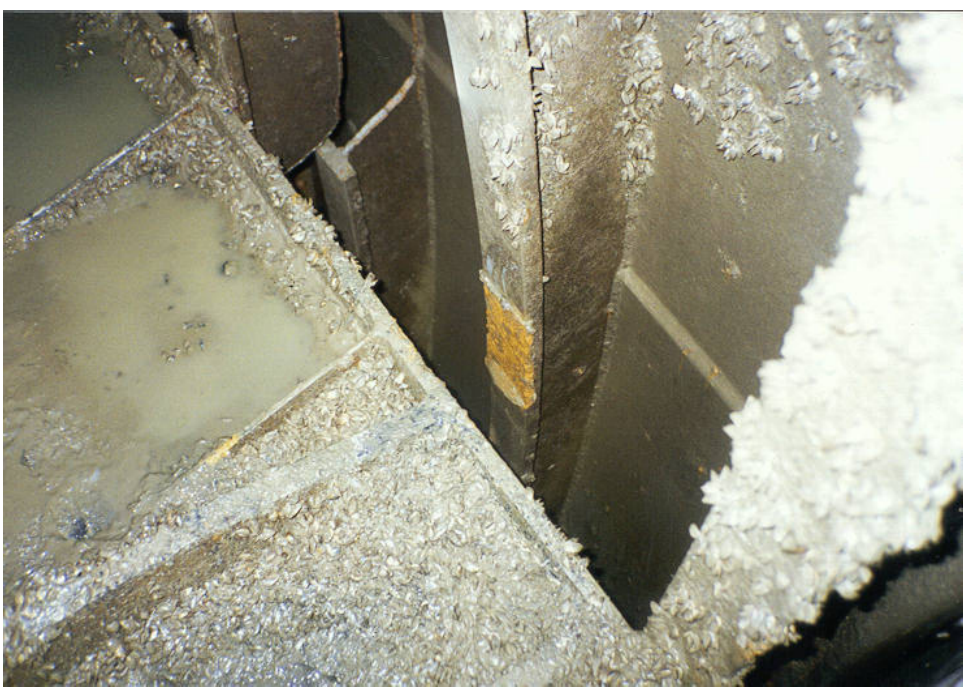


Figure B-32. Failure of weld at second vertical rib. Note weld material left on rib after it separated from horizontal girder.

Figure B-33. Shear failure of vertical ribs in middle of valve. Note vertical rib on far right where initial weld failed

Figure B-34. Shear failure through vertical rib

B-24. <u>Finite Element Model and Calibration of Vertically Framed Reverse Tainter Culvert Valves.</u>

a. Finite element modeling is used to develop reliability models for fatigue cracking at welded connections for vertically framed culvert valves based on analyses and experience gained from reliability modeling for fatigue cracking at welded connections on miter gates detailed in the previous example. In addition, recent field experience involving welded connection failures on vertically framed culvert valves at the Smithland and Cannelton projects was used to guide the analysis and benchmark the reliability model. On each of these damaged culvert valves, the weld attaching a vertical rib to the main horizontal load beam failed, which separated the rib from the load beam. As the load transferred to adjacent connections, subsequent connections failed, both at the welded connections and from complete fracture through the vertical ribs. This sequential failure is diagnosed to have progressed in a fairly rapid manner relative to a reliability study for fatigue cracking. Thus, once a crack initiated at the first welded connection, the operational failure of the valve developed within a relatively few additional cycles of operation. Therefore, for this reliability modeling, the limit state can be considered the initiation of fatigue cracking at the critical connection of the vertical rib to the horizontal load beam, and the finite element modeling concentrated on characterizing the fatigue failure of this connection.

b. A symmetrical, global finite element model of half of the Smithland lock culvert valve, as illustrated in Figure B-35, was developed to identify the local areas that are more susceptible to cracking caused by elevated stress concentration factors. The Smithland design was used as a surrogate for the finite element modeling since field data were available for benchmarking and calibrating the reliability model. The global model indicated that the connection between the vertical rib and the horizontal load beam near the edge of the valve would develop the highest stress concentration under the normal operating head. This is the connection that was determined to have failed first in the Smithland culvert valve.

c. More detailed modeling of this critical connection was then implemented into the global model, as illustrated in Figure B-36, to characterize the fatigue cracking at this connection. At this type of connection, the top of the flange plate of the vertical rib is welded directly to the bottom of the flange plate on the horizontal load beam using a fillet weld around the perimeter of the contacting plate areas. In the detailed modeling, the plate elements are constructed along the center lines of the respective flanges. The two flanges are then connected together with plate elements around the perimeter representing the weld. The thickness of these weld elements is taken as the ligament thickness across the throat of the weld. The membrane stress in these weld elements, which acts through the depth of the weld, is used to establish the stress level for the fatigue cracking evaluations. Figure B-36 also illustrates the maximum principal stress distribution in the weld at this connection caused by the nominal operating head on the valve. As in the reliability modeling for the horizontally framed culvert valves, a tensile residual stress is assumed to exist in the welded area. Because the connection failure is due to cracking along the weld, the residual stress can be assumed to be constant during the extension of the crack. This is consistent with the field evidence that the fatigue crack extends relatively fast once it initiates. However, since limited funding and time constraints did not allow for detailed modeling of the distribution of residual stresses, a larger variation for the level of residual stress is also assumed in the reliability calculations. The stress level calculated at the connection under the operational

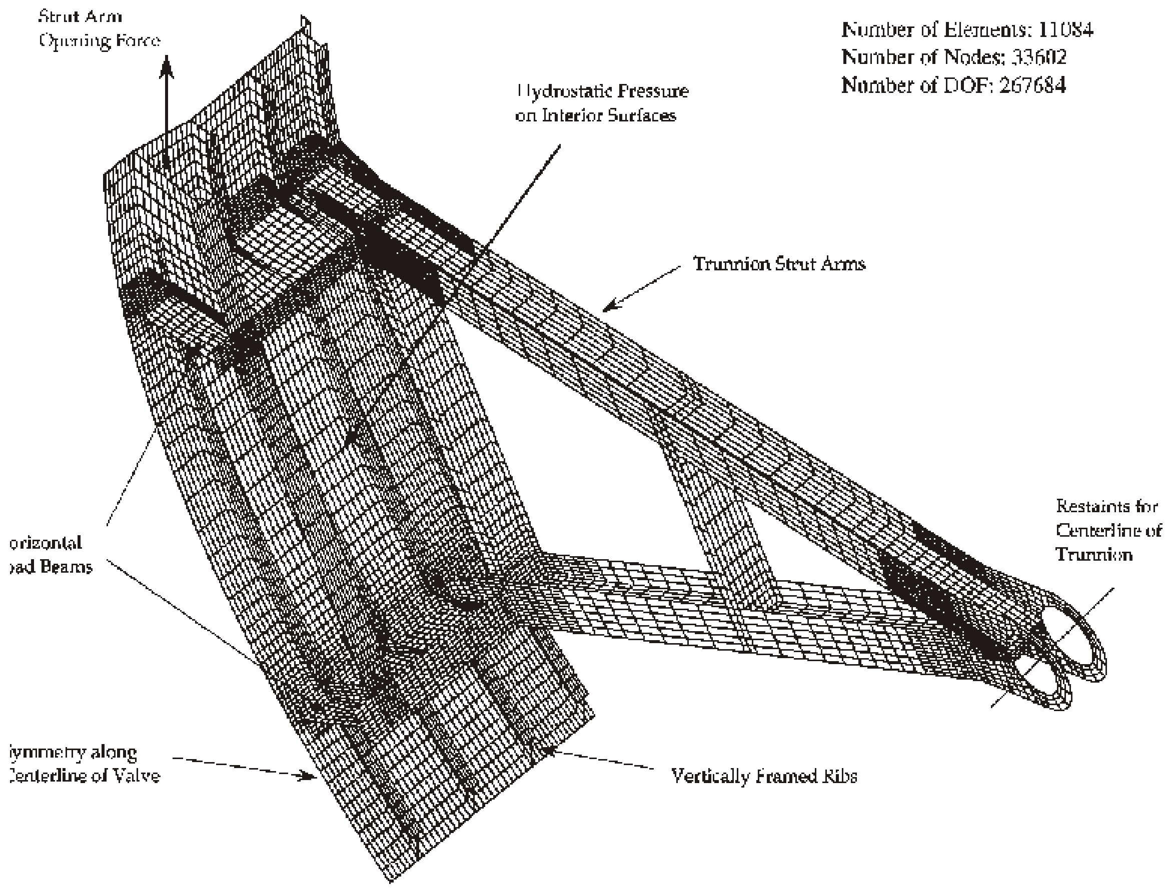


Figure B-35. Free-body diagram for typical land wall monolith

loads becomes the stress range for the fatigue cracking since these operational loads are imposed on top of the residual stresses. However, because the stress is cycling about a mean tensile value because of the residual stress, the effective alternating stress for determining the allowable fatigue cycles is adjusted using the Goodman relation.

d. The calculated peak membrane stress in the welded connection is used to establish a stress concentration factor that can be applied to the design-based calculation for the average stress in the weld. The flange sizes are adjusted in the global model to account for the differences between site-specific culvert valve designs. For example, Figure B-37 shows the principal stress contours for the geometry of Ohio River main chamber culvert valves (J.T. Myers is shown as an example in the figure) to illustrate the stress concentrations present through the depth of the weld material. The stress concentration is then characterized for variations in operating head and thickness reduction caused by corrosion.

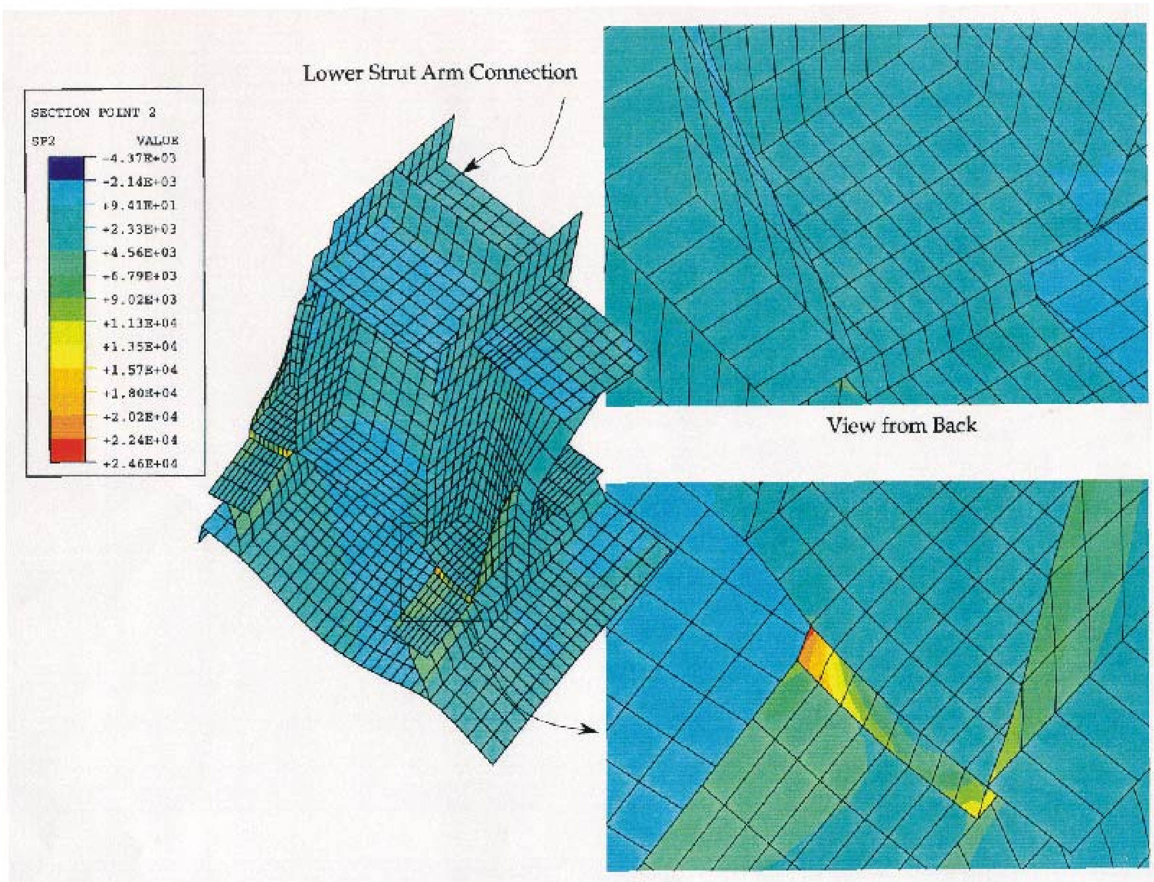


Figure B-36. Maximum principal membrane stress refined modeling of welded connection

e. The dynamic amplification factor of 1.3 on the nominal pressure head is also used to account for the hydrodynamic loading during opening of the valve. This factor was developed based on fluid flow modeling for a horizontally-framed culvert valve. Since this effect is a function of the general shape of the valve and culvert, rather than the details of the construction, this factor is also used for the vertically framed culvert valve reliability model. For further details regarding the fluid-flow interaction, please refer to the horizontally framed culvert valve narrative in Section 5.4 of this appendix. Figure B-38 illustrates the principal stress for crack initiation characterized as a function of head and thickness reduction developed for the reliability model.

f. As mentioned previously, the limit state of the vertically framed culvert valve is defined to be the initiation of fatigue cracking at the welded connection between the vertical rib and the horizontal load beam. Field experience indicates that this cracking will rapidly propagate because of the reduction in area resisting the cyclic tensile loads. The cracking will completely separate the vertical rib from the horizontal load beam. As the load is transferred to the adjacent connections, similar failures will propagate until the valve has an operational failure. The failure hazard caused by this limit state was benchmarked successfully with the Smithland and Cannelton field experience. Thus, for this reliability modeling, the initiation of fatigue cracking at the first connection is considered sufficient to establish a failure of the valve.

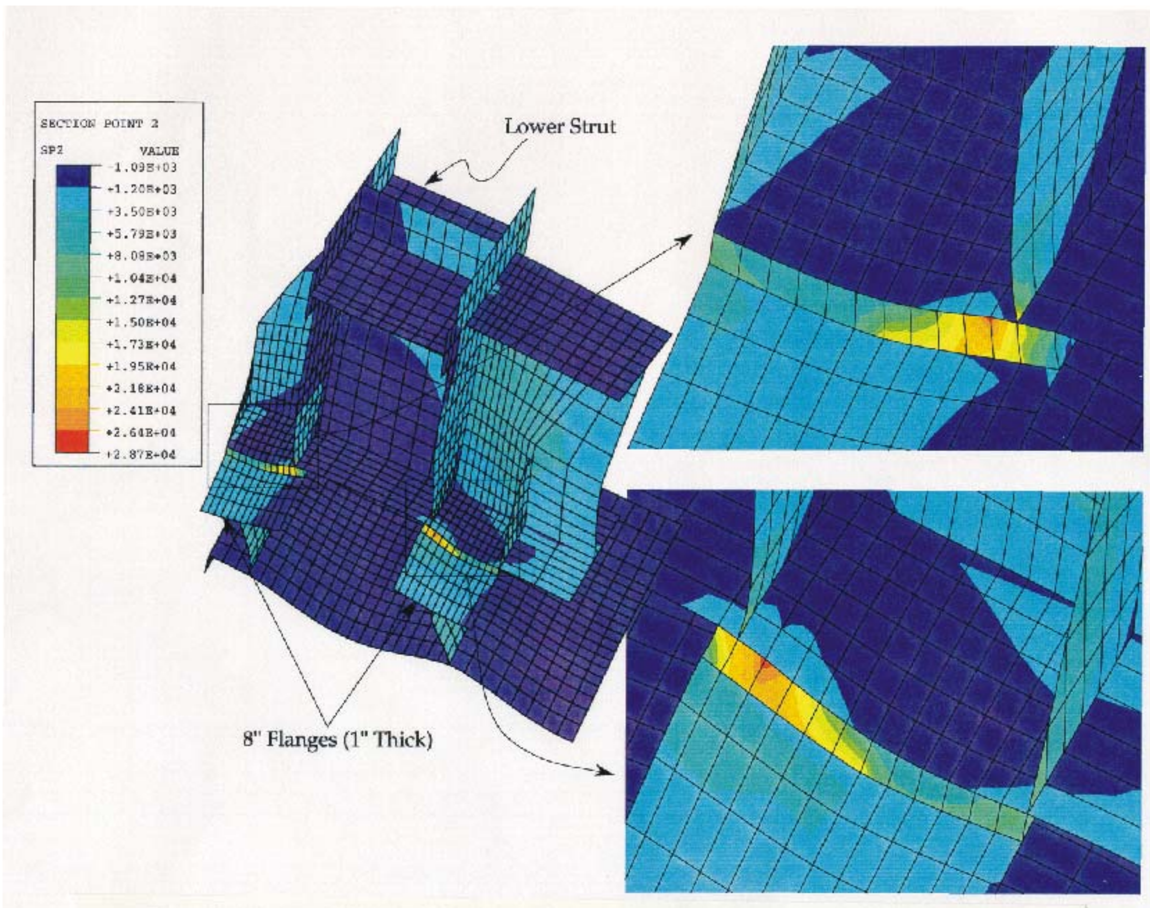


Figure B-37. Maximum principal membrane stress at welded connections

B-25. <u>Reliability Model Parameters</u>. The time-dependent reliability analysis for the vertically framed reverse tainter culvert valves was developed to estimate the hazard rate for these structures. Similar to the miter gate analysis in the previous example, the reliability analysis for vertically framed valves incorporates both the fatigue and corrosion of the welds at the girder/rib connections. Additionally, the ORMSS Engineering Team performed a range of three-dimensional finite element analyses of the valves to investigate the potential modes of failure of the valve, redistribution of loads upon failure, and the realistic values of stresses (residual, static, and dynamic) to utilize in the reliability model. The limit state incorporated into the reliability model is based on the initiation of a crack at the girder/rib weld interface that causes a failure of the welds at the rib, which causes a redistribution of loads to the welds at the adjacent ribs. As evidenced from the valves at Smithland and Cannelton, actual field experience was used in the modeling effort to calibrate the timing of the limit state for the valves. For this model, the ORMSS Engineering Team decided to develop a Visual Basic coded model specifically for the

ORMSS vertically framed reverse tainter culvert valves that was modeled similarly to one developed for the miter gates. The Visual Basic model was named VFCVWELD for the reliability of vertically framed reverse tainter culvert valves.

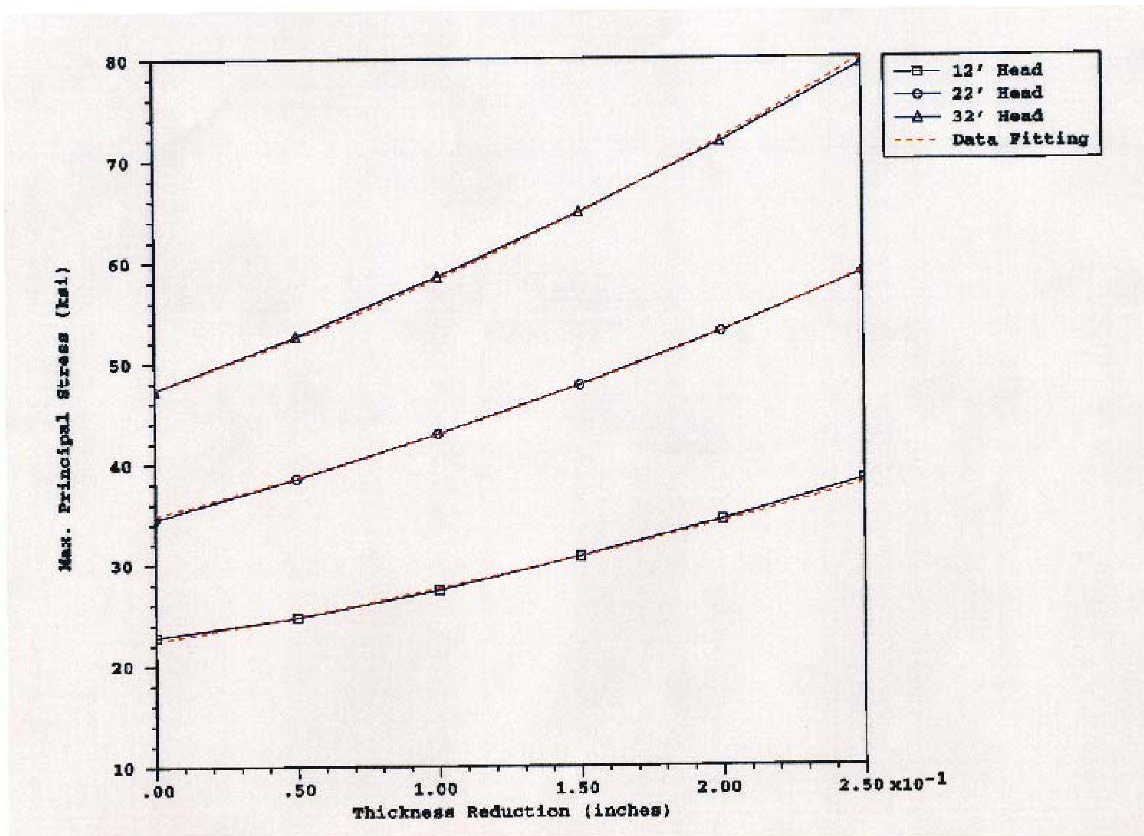


Figure B-38. Principal stress at welded connection as function of head and thickness reduction caused by corrosion

B-26. VFCVWELD Reliability Model.

a. The computer program VFCVWELD was developed to complete a reliability analysis of the vertically framed culvert valves for ORMSS lock projects. The model was developed to measure the future performance of the valves over time relative to the selected limit state. Additionally, the model is used to make the decision about whether to replace the valves at some scheduled date as opposed to fixing them after they perform unsatisfactorily.

b. The basis of the model is determining the time-dependent reliability for the valve structure subjected to fatigue and corrosion. Therefore, factors such as paint history, corrosion rates, historical operating head with cycle information, and other random variables are input in the model. Using the analysis and limit state information defined from the finite element modeling, VFCVWELD computes the time-dependent reliability of the vertically framed culvert valves given the input parameters. For each iteration, the model determines the year in which a fatigue-related crack initiates and marks that year as the time of unsatisfactory performance. The results are then tabulated for the hazard function in a separate file.

(1) Lock information. The first portion of input is the project name and chamber that is being analyzed. For each of the ORMSS locks, both the main chamber and auxiliary chamber valves are of the same design and construction technique. However, because operating cycles and age are different for the chambers, each must be analyzed separately. The input menu requests the district, lock project, and chamber being analyzed.

(2) Rib/girder properties. The VFCVWELD program requires the input of rib and girder properties for the valve. Since the original model was calibrated to the performance at Smithland, most of the figures will reference Smithland vertically framed valve properties. The input menu for the valve properties includes the vertical spacing between ribs, the length of the valve, the top dimension distance to the horizontal girder, which defines the positions of both the top and bottom girders on the vertical ribs (for simplicity, the top and bottom ribs were assumed to be equidistant from both ends since all differences are very minor), the rib flange width, the horizontal girder flange width, and finally both the horizontal and vertical weld thickness at the rib/flange connection. The input for these properties in VFCVWELD for the Smithland valves are shown in Figure B-39.

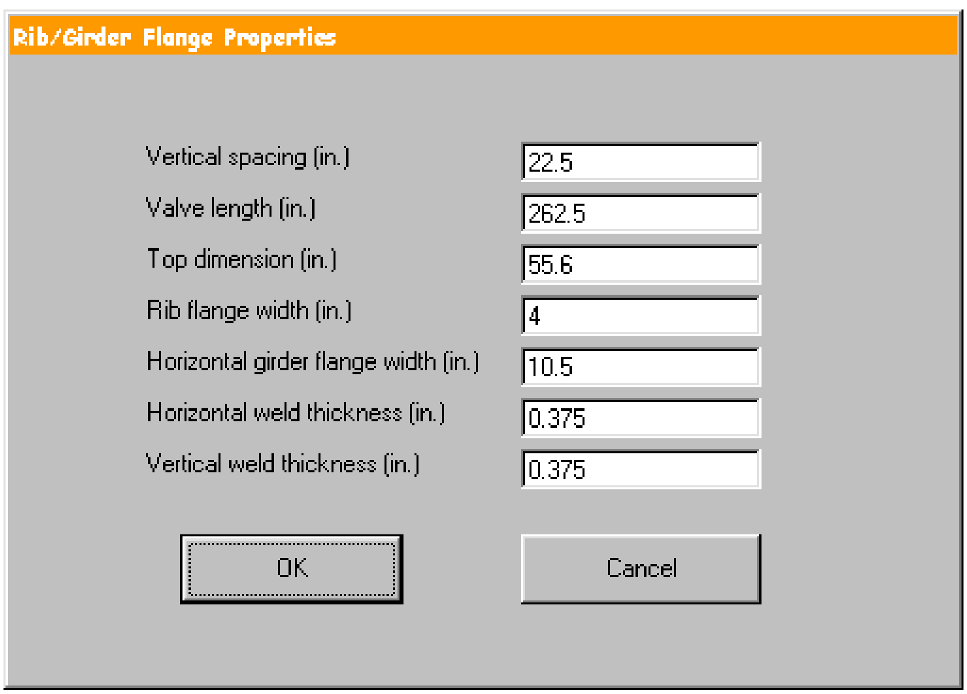


Figure B-39. Rib/girder flange properties input menu

(3) Crack parameters. The only crack parameter required for the VFCVWELD is the initital crack length. This is because the reliaibility model accounts for only the crack initiation and not crack propagation because of the anticipated brittle failure mode that was evidenced at Smithland. The initial crack length is set to a default value of 0.25 in., the same as the miter gate initial crack length. In essence, there is much less redundancy with these structures than with the

ORMSS miter gates since the limit state for that model required a significant crack growth to occur before “failure” was encountered in the analysis.

(4) Head histogram. The head histogram reflects the actual past distribution of head differential and hydraulic cycles for the reverse tainter valves. This distribution is based on true daily lockage cycles available from the LPMS combined with the true head differential for each day. This distribution is very valuable in determining the fraction of annual cycles versus the expected head differential that can be used for fatigue analysis. The head histograms developed are based on data collected and analyzed for approximately 21 years inclusive (1984–2004) of lock operation. The VFCVWELD program allows the input of up to 20 different blocks for head (at specified midpoints) and fraction of cycles from the histograms. This histogram is used in VFCVWELD to parse the input annual cycles into the defined stress range blocks and number cycles for fatigue analysis. An example head histogram is shown in Figure B-50 for Markland Lock and Dam (even though Markland valves are horizontally-framed, the histograms are similar in nature).

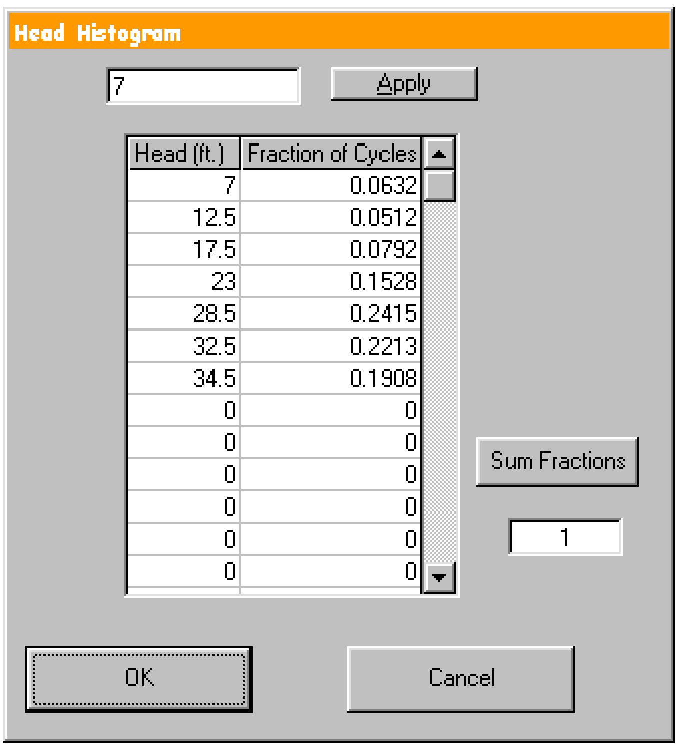


Figure B-40. Example of head histogram

(5) Traffic cycles. The number of operating cycles for the vertically framed valves is determined for each lock based on actual and predicted future cycles for the study period. The cycle information is used in fatigue analysis incorporated into the VFCVWELD program. The cycles are input from the start of operation to the end of the study period. Operating cycles from the origination of the project through 1984 were determined by going through the log books at various ORMSS sites to determine the number of lockages in each chamber. From the LPMS data from 1984 through 2004, a ratio of lockages to operating cycles was determined and assumed to be the same in the past as well as for future projected cycles. Traffic cycles for 1985 through 2004 were determined using LPMS data. Finally, projected traffic through the end of the study period was provided by the USACE Planning Center of Expertise for Inland Navigation in the Great Lakes and Ohio River Division (LRD). The input traffic cycles for one of the Smithland 1200-ft chambers are shown in Figure B-41.

B-27. <u>Random Variables Used in VFCVWELD</u>.

a. The random variables incorporated into the VFCVWELD analysis are the yield strength of A36 steel, corrosion rate, residual stress factor, stress concentration factor, and the dynamic amplification factor. The values and ranges for the yield strength used for the vertically framed valve analysis are the same as applied to the miter gates and horizontally framed culvert valves. The corrosion rate selected was for a structure subjected to wet/dry applications because the valves are constantly in and out of the water during operation, again the same as the horizontally framed culvert valves. This rate is termed in the "splash zone" and has a higher corrosion rate than a submerged structure. Additionally, it was assumed the valves had an initial effective paint life of only 5 years because of turbulent water flow impacting the valve during filling and emptying operations. This was based upon engineering judgment. However, sensitivity analyses were conducted varying the "effective" paint life from 0 to 20 years, and it did not turn out to be a controlling variable. Therefore, the 5-year life was used to be consistent with the analysis for the horizontally framed culvert valves. It is important to note here that the finite element modeling and reliability analysis for the horizontally framed culvert valves was undertaken first for the ORMSS. Therefore, many of the characteristics such as corrosion and paint life assumptions were carried forward to the analysis of the vertically framed valves.

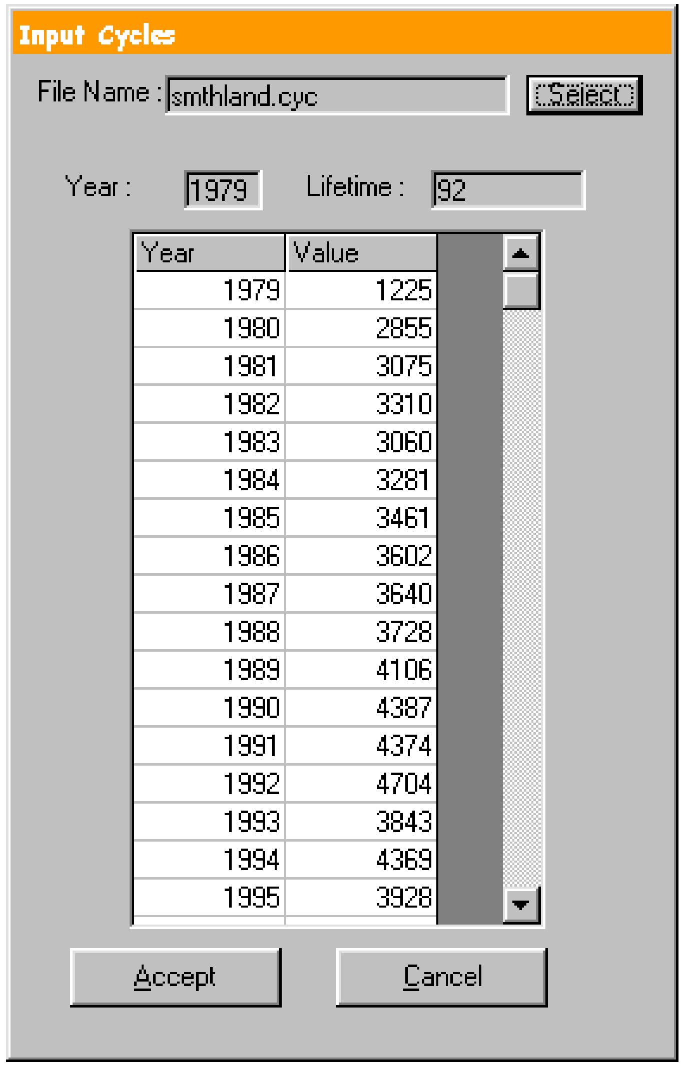


Figure B-41. Example input traffic cycles

b. Because a detailed residual stress analysis was not possible for this model because of funding and schedule constraints, a residual stress factor and stress concentration factor were created to attempt to measure the randomness associated with the residual stress analysis required for this model. The factor was based upon the residual stress analysis completed for the Markland miter gates. This is also consistent with the analysis for the horizontally framed culvert valves. Finally, a dynamic amplification factor was needed to measure the increase in load on the valve from the high velocities that occur during filling and emptying operations. This value (along with appropriate range) was determined by using a steady-state fluid-structure interaction finite element model. This model is described in the horizontally framed valve narrative. Again, all random variables were selected using Monte Carlo simulation techniques.

(1) Yield strength. The distribution for yield strength is based on data from the published literature and previous Corps of Engineers reliability studies. The distribution is based on a

truncated lognormal with a nominal yield stress of 38.88 ksi (i.e., mean yield strength times the strength ratio) and a standard deviation of 5.44. The lower limit for truncation is based on one standard deviation below the nominal (33.44 ksi), and the upper limit is based on approximately two standard deviations above the nominal (51 ksi). The distribution and statistical moments for yield strength of the steel are the same as used for the miter gates and horizontally framed culvert valves.

(2) Corrosion rate. The distribution for corrosion is based on the data from the published literature and previous Corps of Engineers reliability studies. Corrosion is based on a power law that has been fit to actual field data in various corrosive environments. The equation used for the corrosion is $C(t) = A{*}t^{B}$, defined in paragraph B-15b. For this report, the mean value of *A* was selected based on splash zone corrosion. This distribution used for *A* was a truncated lognormal with a mean value of 140 and standard deviation of 42. The upper limit of the distribution was taken at 224 and the lower limit at 56. The value for *B* was a constant of 0.667. These limits and constants are based on actual field measurement of hydraulic steel structures.

(3) Residual stress, stress concentration, and dynamic amplification factors.

(a) Three types of factors are utilized in VFCVWELD to account for the major differences in stress values between traditional hand calculations and the more sophisticated finite element analysis. The residual stress factor represents the tensile stresses that are created during the heating and subsequent cooling of the welds at the time of construction. The second factor is the dynamic amplification factor, which represents increased load on the valve that is created by the vortex flow and pressure differential of the water around the valve upon opening. This quick change in pressure increases the stresses on the strut arms during valve operation. The third factor is the stress concentration factor, which tries to account for local stress increases caused by fabrication confinements that occur in welded structures. An extensive literature search for field measurement data on these factors was conducted. No data are available to assist in better defining any of these parameters for the reliability of the valve. Therefore, these adjustments were determined based on various finite element analyses to determine the range of values that may be exhibited in these random variables.

(b) The distribution for the residual stress model factors was considered to be a Gaussian distribution since the limits were defined by a concentration about a certain percent ratio. The mean value for the residual stress was 0.35 with a standard deviation of 0.05. The dynamic amplification factor was also determined to be a normal distribution with a mean of 1.25 (25 percent increase) and a standard deviation of 0.025 (2.5 percent). As an example, the stress concentration factor for the Group 2 valves was determined to be a uniform distribution with an upper limit of 2.1 and a lower limit of 1.5.

B-28. VFCVWELD Reliability Model Results and Event Trees. The output from the VFCVWELD reliability model is a hazard function giving the annual probability of unsatisfactory performance of the culvert valve over time. For simplicity, it was decided to look at the reliability associated with only a single valve rather than numerous ones for the main chamber. The consequence event tree is structured around the failure of a single valve and how repairs would be initiated following a valve failure.

a. Main chamber results and event tree.

(1) Similar to other sections, a single project site (Cannelton in this case) will be detailed to clearly illustrate the process of modeling carried out for vertically framed culvert valves and the subsequent output. As an example of the output provided by the VFCVWELD program, the hazard rates for the vertically framed culvert valves of the Cannelton main chamber are shown in Figure B-42 for the high (NAAQS), middle (Utility High) and low (Clear Skies) future traffic projections. As shown in the figure, the variability between the hazard rates for the main chamber vertically framed culvert valves does not differ greatly. This is primarily due to the fact that the cumulative future traffic cycles are fairly similar across all traffic scenarios for the Cannelton project, unlike other Ohio River project sites where there are large difference between future traffic scenarios. For example, at Cannelton the difference between the lowest traffic scenario (Clear Skies) cumulative traffic cycles and the highest scenario (NAAQS) was roughly 23 percent.

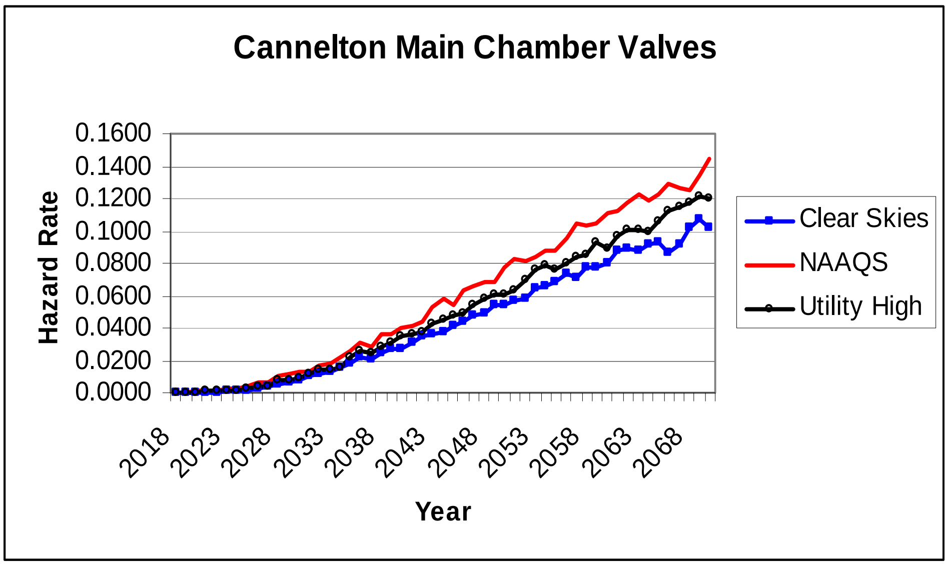


Figure B-42. Cannelton vertically framed culvert valve hazard rates

(2) The hazard rates shown in Figure B-42 are for a single valve and were given to the economists along with chamber-specific event trees. The event tree for the main chamber was formulated within the context of how repairs to valves have been made historically with the chamber operating at half filling/emptying speed. This is different from valve performance for Ohio River auxiliary chambers where only a single filling and single emptying valve is used for the smaller 600-ft chamber. The doubling of filling and emptying time does not begin to compare to the navigation loss of benefits associated with having the main chamber closed and needing to move large tows through the smaller auxiliary chamber. Therefore, separate event trees were needed for the two chambers. The event tree for the main chamber is shown in Figure B-43. A format similar to that used for the miter gate event tree was used for the valves. Assuming an unsatisfactory performance of the culvert valve based upon the mode selected in the reliability model, three possible repair scenarios were chosen along with a replacement ahead of failure scenario, which is used to time individual component replacements when economically justified.

A breakdown of these repair scenarios for the main chamber culvert valves along with their costs and closures is provided.

b. Catastrophic failure, install four new valves.

(1) This repair assumes the worst situation, a catastrophic failure of a culvert valve. It is assumed the damage and potential problems associated with the failure are enough to warrant a short closure of the main chamber to determine the problem and do a brief inspection of the other valves since they should all be of the same general condition. The duration of this closure is estimated to be 5 days, and then the main chamber can be put back into service by operating at half speed. The overall repair effort is spread over 3 years since the damage caused by the failure is done during the first year and then four new valves are fabricated, delivered to the site, and installed over the next 2 years. Future reliability is assumed to be equal to 1.0 for the remainder of the study since new valves are installed. The repair cost associated with this repair is estimated at $10,714,660. The estimate is not accurate to that degree, but the values are based upon average historical valve repair rates completed in the last 20 years by the LRL repair fleet. Several emergency valve repairs have been made by LRL during this time frame, and the average fleet rate for these repairs is $32,836 per day in FY05 dollars. The rate per day is lower because fewer personnel are required than for full chamber dewaterings for miter gates where main chamber closure time must be kept to a minimum. It was also determined that a rate of approximately $22,000 per day is applicable for "normal" valve repairs from historic records. This rate will be used for work that can be planned in advance. A breakdown of the costs for this repair scenario is supplied below.

Repair fleet closes chamber for 5 days for inspection (year 1)	$164,180 ($32,836/day for 5 days)
Full inspection and valve repairs (45 days each valve, year 1)	$5,910,480 ($32,836/day for 180 days)
Design, fabricate, and deliver four new valves (year 2)	$2,000,000
Installation of four new valves in chamber (year 3)	$2,640,000 ($22,000/day for 120 days)
Total for all items for catastrophic repair	$10,714,660

(2) It is assumed that the chance of a catastrophic failure of this magnitude is quite low; therefore, it was decided to place only about a 5 percent chance of this occurrence in the first 30 years of service on this branch. This was increased to 10 percent for valves in service between 31 and 50 years, and finally, to a maximum of 15 percent for valve service life of over 50 years.

ANNUAL HAZARD RATE	LEVEL OF FAILURE & SUBSEQUENT REPAIR			CLOSURE TIME	CHAMBER @ 1/2 SPEED	REPAIR COST	DESCRIPTION OF REPAIR SEQUENCE	EFFECT ON FUTURE COMPONENT RELIABILITY
Annual Reliability Value (1 - Annual Hazard Rate)								
	Catastrophic Failure of a Single Valve. Close Chamber to Inspect/Repair Other Valves.	5%	0-30 years of service	15 days in year 1	180 days in year 1	Cost $6,402,985	Fleet on-site for a total of 15 days for closure and 45 days per valve for emergency repairs	Assume R = 1.0 for All Future Years
	Design New Valves. Operate Chamber @ 1/2 Speed Until New Valves Installed.	15%	31-50 years of service	0 days in year 2	365 days in year 2	Cost $2,000,000	Costs associated with design of new valves and fabrication cost	
	New Valves Fabricated and Installed in Chamber.	30%	Over 50 years of service	0 days in year 3	180 days in year 3	Cost $2,640,000	Fleet requires 120 days to install new valves (30 days per valve on average)	
Annual Hazard Rate	Major Repair Required but All Valves Still Serviceable for Immediate Future	35%	0-30 years of service	0 days in year 1	90 days in year 1	Cost $2,955,224	90 days to repair filling valves	Move Back 5 Years
		30%	31-50 years of service	0 days in year 2	90 days in year 2	Cost $2,955,224	90 days to repair emptying valves	
		20%	Over 50 years of service					
	Failure of a Single Valve. Close Chamber to Inspect/Repair Other Valves.	60%	0-30 years of service	10 days in year 1	180 days in year 1	Cost $6,238,806	Fleet on-site for a total of 10 days for closure and 45 days per valve for emergency repairs	Assume R = 1.0 for All Future Years
	Temporary Repairs Make Chamber Fully Operational. However, New Valves Required.	55%	31-50 years of service	0 days in year 2	0 days in year 2	Cost $2,000,000	Costs associated with design of new valves and fabrication cost	
	Design, Fabricate, and Install New Valves	50%	Over 50 years of service	0 days in year 3	180 days in year 3	Cost $2,640,000	Fleet requires 120 days to install new valves (30 days per valve on average)	

REPAIRS COST INFORMATION

$32,836	LRL Fleet Rate for Emergency Repairs to Valves (Factored Cost from ORMSS 1/2 Speed Historical Repair Work)
$200,000	Engineering Cost for Design of New Valves and Contract Award Effort (Markland Rehab Esimate)
$22,000	LRL Fleet Rate for Normal Valve Repairs (ORMSS Historical Database)
$1,800,000	Cost to Fabricate and Deliver 4 New Culvert Valves

REPLACEMENT OF MAIN CULVERT VALVES PRIOR TO FAILURE

Year 1 - Fabricate 2 Culvert Valves
Year 2 - Install 2 New Valves. Fabricate Other 2 Valves.
Year 3 - Install 2 Remaining Valves.

	Chamber 1/2 Speed	Cost of Repairs
Year 1	0	$900,000
Year 2	60	$2,220,000
Year 3	60	$1,320,000

There is no closure time required when valves are replaced ahead of failure.
With proper preparation, it will take about 30 days per valve to switch out the damage valve with the new one.

Figure B-43. Main chamber culvert valve event tree for ORMSS

c. Temporary repair with new valves following year.

(1) This repair assumes that the major damage has occurred to at least one of the four valves and it is out of service. The chamber is assumed to be shut down while engineers travel to the site to assess the situation. After a brief inspection, the chamber is opened back up for traffic but only at half speed while detailed inspection and repairs are made to all four valves. This would be done for each of the four valves. Emergency repairs would be made and then new valves would be designed and fabricated the following year. The four valves would be installed in year 3 of the repair. Repair, fabrication, and installation times similar to those for the catastrophic repair are assumed for this repair scenario. The only difference is that the chamber is closed only 1 day in the year of the failure. The overall cost for this repair is estimated at $10,583,326 with chamber closure time of 1 day and total half speed time of roughly 2 years. Future reliability is assumed to be equal to 1.0 for the remainder of the study since new valves are installed the third and last year of this repair scenario. A breakdown of the costs for this repair is supplied below.

Repair fleet closes chamber for 1 day for inspection (year 1)	$32,836 (emergency repair rate)
Full inspection and valve repairs (45 days each valve, year 1)	$5,910,480 ($32,836/day for 180 days)
Design, fabricate, and deliver four new valves (year 2)	$2,000,000
<u>Installation of four new valves in chamber (year 3)</u>	<u>$2,640,000</u> ($22,000/day for 120 days)
Total for all items for temporary repair	$10,583,326

(2) It was agreed by the Engineering and Operations Teams that this scenario was more likely to occur than the catastrophic repair scenario; however, it is not the most likely repair option. Thus, 20 percent was placed on this branch for up to 30 years of service, 25 percent between 31 and 50 years, and 30 percent for valves in service longer than 50 years. It is believed that the repair fleet would do everything possible to get the chamber operational again; however, major damage would prompt the district to obtain the funds to procure new valves.

d. Major repair, leave existing valves.

(1) This repair assumes the least damage to the culvert valves so they are repairable and can continue in service. For this situation, the main chamber does not require a closure, but is operated at half speed for repairs to each of the valves. It is estimated that each valve will require 45 days to repair. The cost associated with this alternative is estimated to be $5,910,480 over 2 years assuming a daily repair fleet rate of $32,836 per day for emergency valve repairs. It is assumed that the valves are repaired well enough that they can continue to provide adequate service in the future. Thus, the hazard rate after the repairs is lowered by “moving back” the hazard rate to what the value was 15 years prior to the failure and resetting the curve. Again, this is the easiest way to improve the reliability of a component after a repair in the economic model. A breakdown of costs associated with this repair is provided below.

Emergency repairs to two valves (45 days per valve, year 1)	\$2,955,240 (\$32,836/day for 180 days)
<u>Emergency repairs to two valves (45 days per valve, year 2)</u>	<u>\$2,955,240</u> (\$32,836/day for 180 days)
Total for all items for major repair	\$5,910,480

(2) It was agreed that this scenario represented the most likely repair given the limit state modeled in the reliability analysis. Therefore, 75 percent was placed on this branch for valves with less than 30 years of service, 65 percent for valves with 31 to 50 years of service, and 55 percent for valves in service more than 50 years.

e. Scheduled replacement of culvert valves. The other piece of information the economists need is the cost and chamber closure or filling/emptying effect associated with the scheduled replacement of the valves before failure. There are four valves for the main chamber, and it can be operated at half-speed in the event of repair or replacement work to one of the valves. The simplifying assumption is made that new valves will be 100 percent reliable for this limit state for the remainder of the study period once they are installed. A well-planned and executed replacement should save considerable time compared with the fix-as-fails scenarios evaluated with the hazard rate and event tree repair scenarios. Therefore, this option is reduced to \$4,440,000, and the total required time of the chamber operating at half speed is 120 days over a 2-year period. The cost and closure breakdown associated with a scheduled replacement of the main chamber culvert valves is provided below.

Fabrication of two new valves (year 1)	\$900,000
Fabrication of remaining two new valves (year 2)	\$900,000
Installation of two new valves (30 days per valve, Year 2)	\$1,320,000 (\$22,000/day for 60 days)
<u>Installation of remaining two new valves (year 3)</u>	<u>\$1,320,000</u> (\$22,000/day for 60 days)
Total for all items for replacement ahead of failure	\$4,440,000

f. Auxiliary chamber hazard rates and event tree. As noted previously, the failure of a single culvert valve in a main chamber with four valves is considerably different from a failure of a single valve in an auxiliary chamber. The auxiliary chambers on the Ohio River have only two valves (one filling and one emptying). Thus, a failure of an auxiliary chamber valve shuts that chamber down totally until the valve can be repaired. The same type of engineering reliability analysis is carried out for the auxiliary chamber culvert valves as was done for the main chamber culvert valves; however, chamber-specific information such as operating cycles and paint history is applied to determine the hazard rate. Since the auxiliary chambers on the Ohio River are used primarily for recreational lockages and when the main chamber is closed for maintenance, they have seen significantly lower operating cycles to date. After the engineering reliability analysis was conducted for the auxiliary chamber culvert valves, only one project (Markland) had enough operating cycles and stresses where predicted damage was high enough to generate a positive hazard rated during the study period. The analysis was carried out for the auxiliary chamber culvert valves, but no failures were encountered for the selected limit state. Since only one auxiliary chamber horizontally framed culvert valve was analyzed in the economic analysis, no significant detail is provided on the event tree. However, the following general information applies. The same format was used; however, chamber closures were specified for each of the three levels of repair (catastrophic, temporary, and major) as opposed to half-speed operating days for main chamber culvert valves. In addition, replacement costs are roughly half of the main chamber since there are only two valves for an auxiliary chamber.

B-29. Economic Results for Vertically Framed Culvert Valves.

a. The hazard rates shown in Figure B-42 are combined with consequence event trees for the vertically framed culvert valves, as depicted in Figure B-43. It is important to note that the same event trees for the main chamber vertically framed culvert valves are applicable for the horizontally framed culvert valves. Although there are structural differences between the two types of valves, they are not great enough to warrant using different event trees.

b. The Ohio River Navigation Investment Model (ORNIM) was the economic systems model used to evaluate the future reliability of major infrastructure in terms of projected replacement needs. ORNIM was used to compute average annual costs for two repair options (fix-as-fails or replace ahead of failure) for the main chamber culvert valves at Cannelton. The average annual costs include both navigation delay and physical repair costs. Improved reliability after the repair is also included in the analysis, as noted in the event tree for differing levels of repairs. The costs of each alternative, fix-as-fails versus selected replacement dates, are compared to determine the best economic option. The replacement year with the lowest average annual cost sets the timed replacement of the culvert valves with fix-as-fails costs added to the economic analysis up to the year of scheduled replacement. If the lowest average annual cost is associated with the baseline condition (fix-as-fails), then the replacement of the culvert valves is not justified economically and the fix-as-fails costs are embedded into the overall economic analysis for every year of the study period. ORNIM produces a graphical output depicting the results and optimal timing of this process for each component and each traffic scenario. The ORNIM output for the Cannelton main chamber culvert valves is shown for the Utility High traffic forecast in Figure B-44.

c. As noted earlier, the culvert valves at Cannelton were evaluated for each traffic scenario. Using the same methodology, optimized replacement dates were determined for each traffic scenario. As shown in Figure B-42, there is a relatively small difference in the hazard rates between all three traffic scenarios for the main chamber culvert valves at Cannelton. Thus, the replacement dates will vary only slightly across the different traffic projections for this particular component.

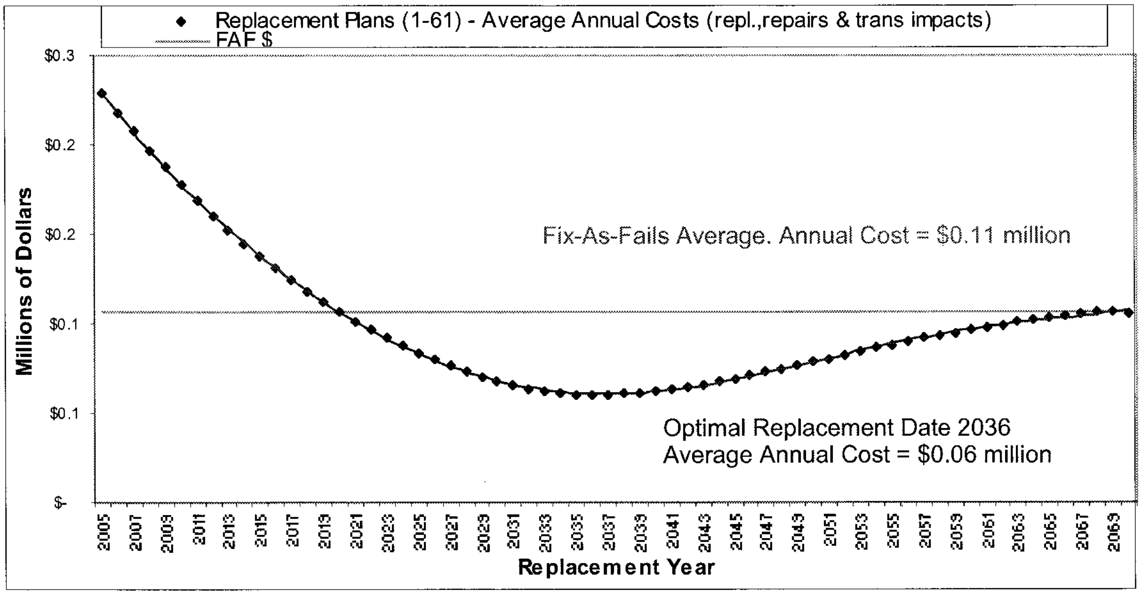


Figure B-44. Economic results for Cannelton main valves for Utility High forecast

Section IV
Example 4
Issue: Reliability Analysis for Mass Concrete Lock Deterioration
Project: Chickamauga Lock Replacement Project

B-30. General Background Information Regarding Project. The economic analysis for the Chickamauga Lock Replacement Project evaluation required several key pieces of information to be developed by the Engineering Team. Most notably, the Engineering Team provided schedules of lock chamber costs and closures for Chickamauga from the years 2000 through 2060. The schedules included yearly estimates of costs associated with operations and maintenance, along with closures of the lock chamber. These schedules were developed for all scenarios being evaluated under both the Without- and the With-Project conditions. It is well understood that the need for improvements at Chickamauga is based upon the deteriorated condition of the lock and its ability to provide adequate service in the future. The root cause of the deterioration at the project can be traced to AAR (alkali aggregate reaction). AAR is a chemical reaction between the aggregate and the alkali in the cement that causes the concrete to expand as it ages. This volumetric expansion in the concrete walls of the lock causes concrete cracking when restrained and also develops misalignment in mounted equipment, such as miter gates and culvert valves. In order to address the deterioration in the economic analysis, the Engineering Team used a combination of state-of-the-art mathematical reliability modeling and expert elicitation to develop time-dependent hazard functions of critical components at Chickamauga. Repair event trees were used with the component- and maintenance-specific hazard functions to determine their overall effect in the economic analysis. These probabilistic methods attempt to capture the uncertainty associated with the future condition of the lock and mesh well with the economic analysis.

B-31. Background Information Regarding Lower Miter Gate Monolith. The lower, riverward miter gate monolith at Chickamauga Lock Project is commonly called Block 47. This monolith is critical to the safe operation of the project. Not only does it distribute the loads from the lower miter gates when the upper pool is in the chamber, but it also forms part of the continuous damming surface that separates upper and lower pools. In addition to these reasons, this component is considered the most critical because of the current condition of the structure. Block 47 has cracking damage through its cross section at several locations, and the top of the block has moved several inches upward and downstream from concrete expansion over the years. The miter gates are anchored to the top section of this monolith, and substantial adjustments, including rebuilding the gudgeon pin connection, have been required over the years to keep the miter gates in alignment. There is significant concern regarding the condition of the concrete around the embedded miter gate anchorage. This concrete must be sound for the continued safe function of the miter gates. Misalignment of the miter gates caused in part by the expansion of the concrete can induce additional stress in the gates, leading to accelerated fatigue cracking.

B-32. Finite Element Modeling of Block 47. The lower riverward miter gate monolith is a gravity structure subject to three-dimensional loading. The AAR problem has caused significant damage and deformations to the structure. While the root cause of the AAR condition is understood, the structural effects are very complex since the expansive growth is a function of the restraint and induced stresses. Because of the uncertainties in this AAR expansion mechanism and in the material properties during construction, a probabilistic-based reliability model was developed for this structure. Traditional hand calculations cannot accurately capture the structural response of Block 47 through time. Therefore, nonlinear finite element modeling was used to characterize the structural response for variations in parameters, such as concrete strength and AAR expansion rates, as a basis for the reliability models. An isometric view of the global finite element modeling of the lower portion of the river wall is shown in Figure B-45. The intent of the finite element modeling is to characterize the structural performance caused by AAR expansion for variations in problem parameters so that subsequent probabilistic-based reliability evaluations can be performed. The finite element model simulated the complete history of the structure, including repairs, and projected the future structural performance. The cracking throughout the section, periodic repairs, and continued expansion of the concrete required a time-dependent analysis of the loads on the dynamic structure. As shown in Figure B-45, the global finite element modeling included the supporting structures surrounding Block 47. These included the founding rock, deteriorated lower approach wall, and the monoliths upstream of Block 47. All of these items were modeled to understand their effects on the structural response of Block 47. A coarser mesh was used for the surrounding structures since only a global determination of their effects on Block 47 was required. A much more refined mesh was provided for a three-dimensional slice of Block 47. This slice made up the critical portion of Block 47 where the miter gate anchorage is embedded in the concrete monolith. A close-up view of the refined mesh is shown in Figure B-46. The refined local mesh allowed a more accurate determination of future cracking patterns, load and stress redistribution after cracking, and deflections. As evidenced by Figure B-46, the expansion rate of the concrete was set up as a function of the restraint surrounding each finite element block. For example, the expansion rate of the concrete was greater at the top where there is less restraint than at the base where the structure is supported by rock on all sides. Time-dependent pressure on the upstream face represents the expansion of the surrounding concrete toward the downstream end. Support from the downstream end comes from the lower approach wall, which is represented by time-dependent spring elements. Because the condition of the lower approach wall has deteriorated from cracking over the years, the spring elements vary in stiffness through time. Finally, the critical area of the embedded anchorage was modeled in the structure. In addition, the anchors that were installed in the 1982 and 1996 are also embedded in this model.

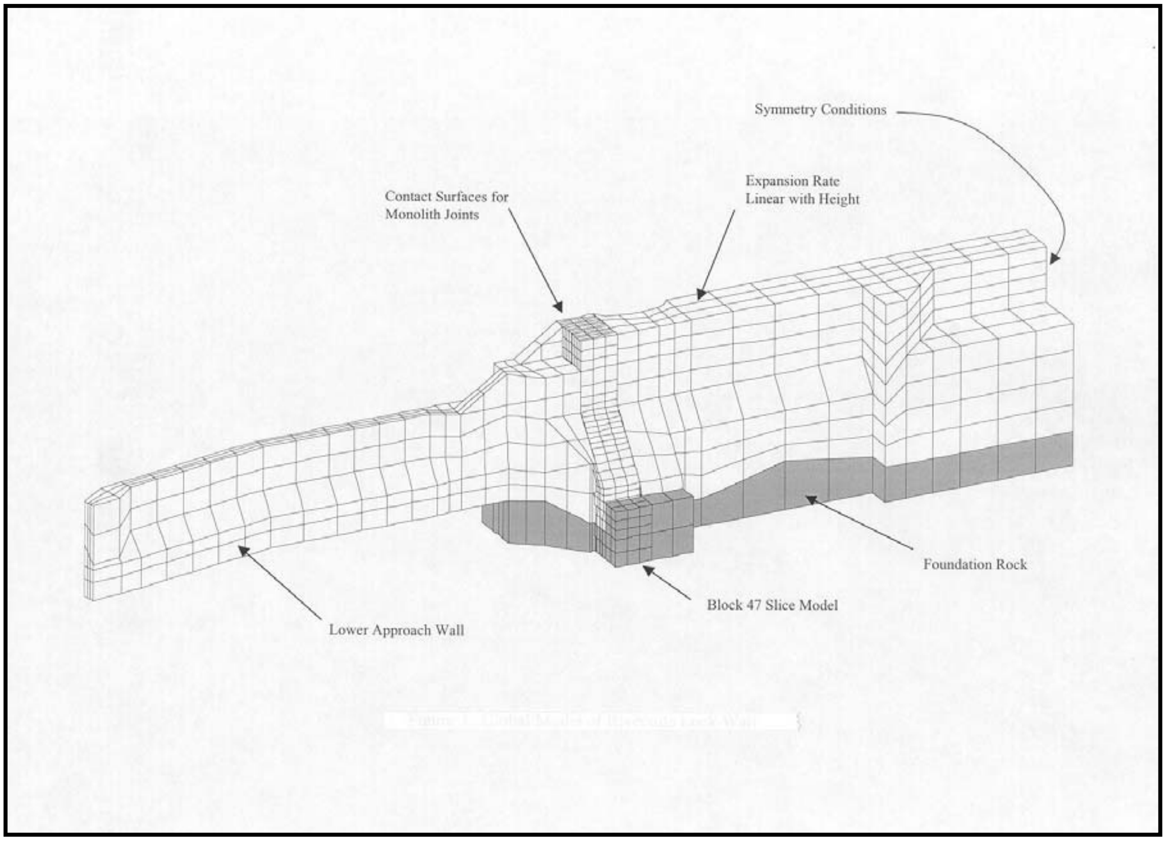


Figure B-45. Global finite element model of lower river wall

B-33. <u>Calibration of Block 47 Finite Element Model</u>. A matrix of calculations were needed for problem variations with each calculation covering a 120-year history (installation date of 1940 through end of the study period 2060). A local three-dimensional slice finite element model was used to characterize the structural performance of Block 47 over this time frame. Because of a lack of data for describing a constitutive model for AAR growth relative to stress buildup and the uncertainties in the distribution of material properties during construction, the effects of the AAR expansion had to be "reverse engineered" by defining expansion rates and distributions until field data were approximated by model results. The field data consisted of survey data for lock wall movement since 1982 and historical observations and repair for cracking, both in time of development and extent of damage. Figure B-47 illustrates the observed cracking damage that was repaired from 1982 through 1984 and again in 1997. The first significant cracking of Block 47 was noted during inspections of the dewatered lock chamber during the 1970's. The baseline FEM was first calibrated to the observed cracking patterns and lock wall displacements, and the corresponding structural repairs were simulated in the model. The future structural performance was then evaluated by continuing the analyses through time. Since the response of the local slice model shown in Figure B-46 was dependent upon the expansion of the concrete in the upstream lock wall and the support from the wall downstream of the slice, the global model, shown in Figure B-45, was used to define the boundary conditions for the local slice model. The pressure distribution on the upstream face of the local model was extracted from the results of the global model after several years of expansive growth. The spring support on the downstream side of the local model was based on the stiffness provided by the global model. This was allowed to vary in time according to the known damage to the structures. Figure B-48 illustrates the cracking damage that developed in the model at four points in time: 1977, 1982, 1997, and 2012 for the base case of 5000-psi compressive strength concrete. It is important to note that the base

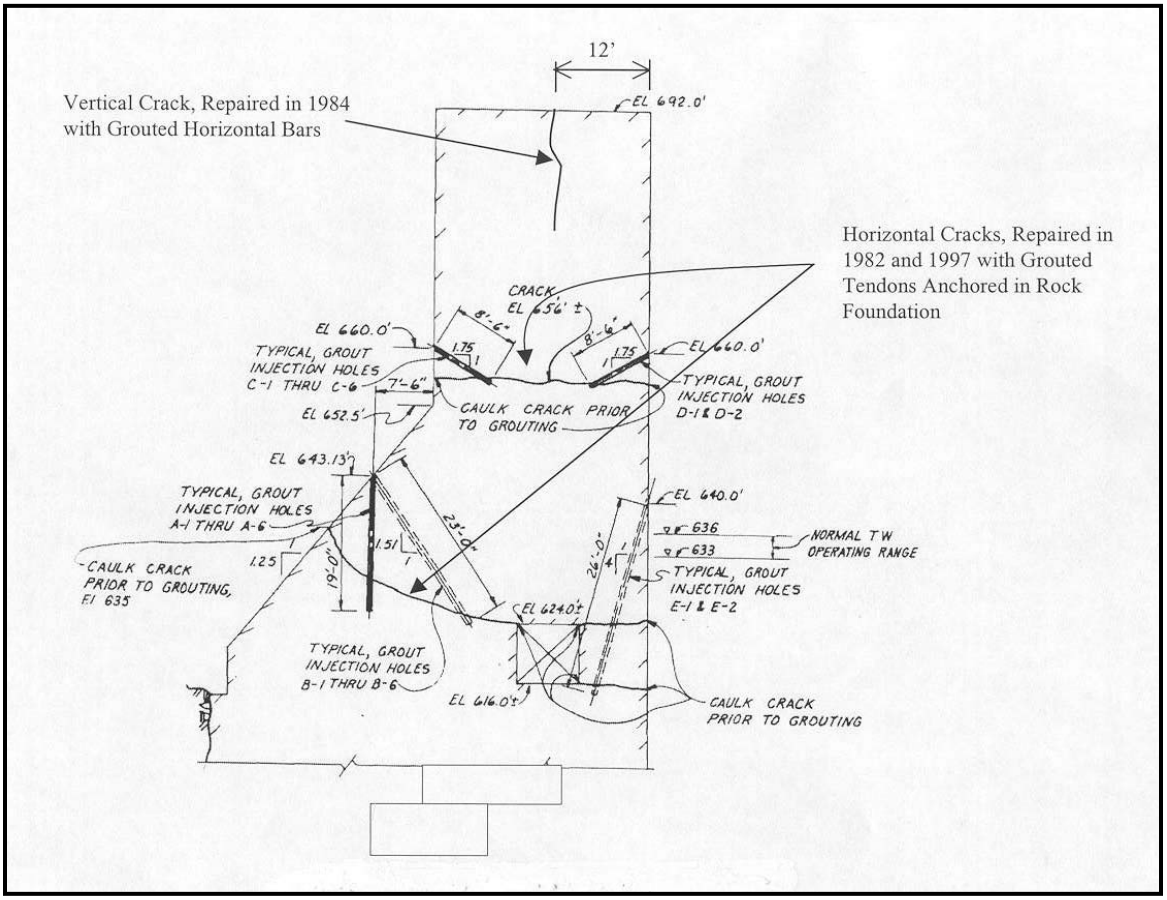


Figure B-47. Crack patterns and damage areas in Block 47

case was calibrated to match known cracking and deformations before any parameters, such as concrete compressive strength, were varied for the necessary finite element model runs to obtain input for the reliability model. Also, the cracking pattern for 1982 was used as the baseline benchmark since that was when the current survey data were set. The cracking continually worsens in the model into the future as shown in Figure B-48, especially at the base and top of the structure. These cracking patterns seem to match well with the known damage to the structure. The next calibrating step was to approximate the displacements to those from field survey data. Accurate survey data on the displacements were available from 1982 to the present. Figure B-49 compares the displacement at the top of the lock wall in the model with the survey data for the baseline benchmark. The model was constructed so that the measured rate of displacement from 1982 to 1999 is continued into the future, and the concrete strains and

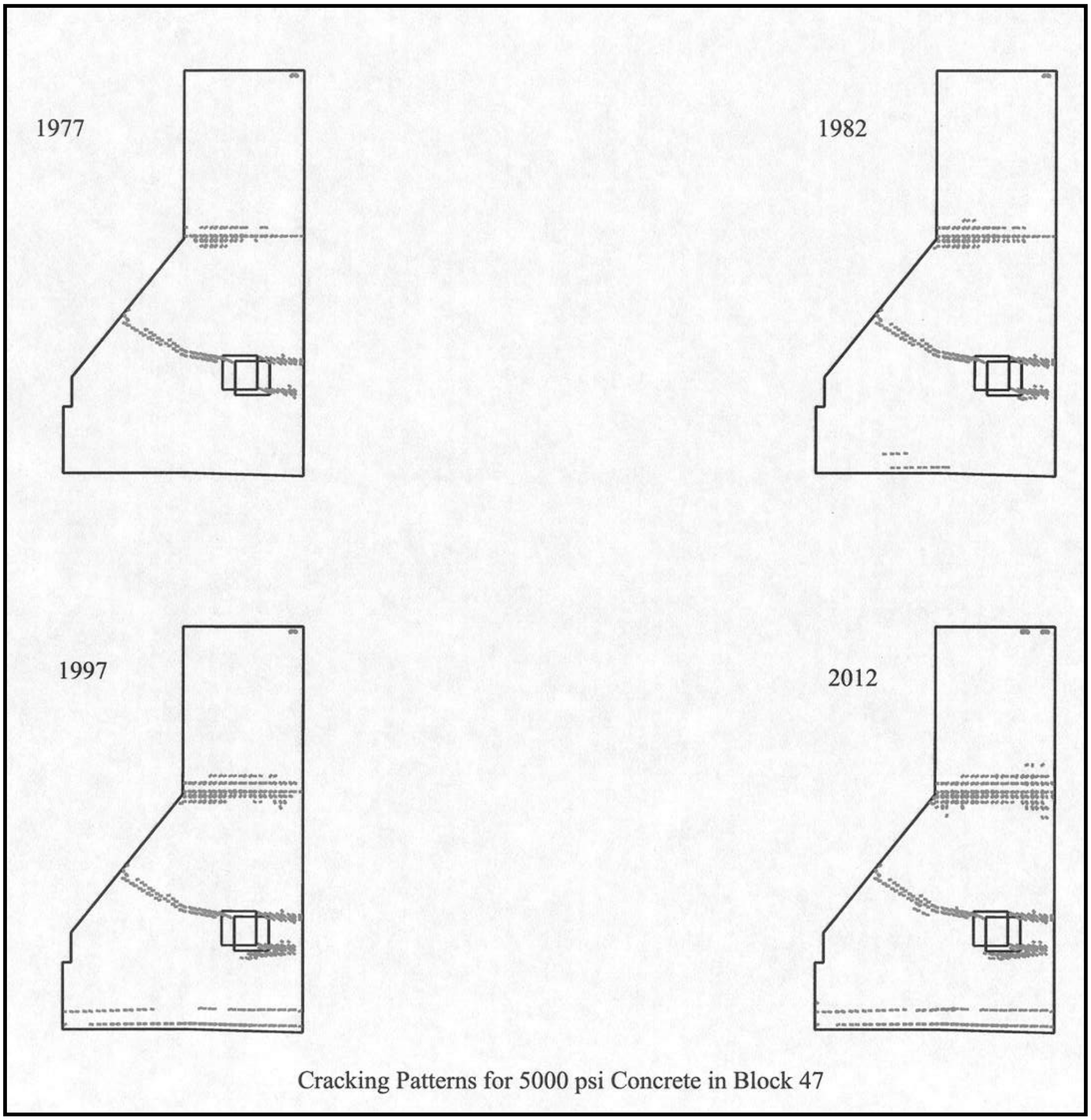


Figure B-48. Block 47 cracking pattern through time for base case

cracking form the basis for evaluating the structural performance. The projections for both the vertical and transverse deformations follow the trend from available survey data.

B-34. <u>Block 47 Limit State for Reliability Modeling</u>.

a. After the cracking patterns from the finite element model results were reviewed, possible limit states were identified from the baseline projections. After much discussion among the team, excessive cracking (strain) of the concrete in the region of the monolith where the miter gate anchorage is embedded became the main focal point regarding a limit state. This concrete cracking is due to horizontal strain differentials, as evidenced by the repaired vertical cracking in 1982, and is exasperated by the normal compressive stress from tendon loads and

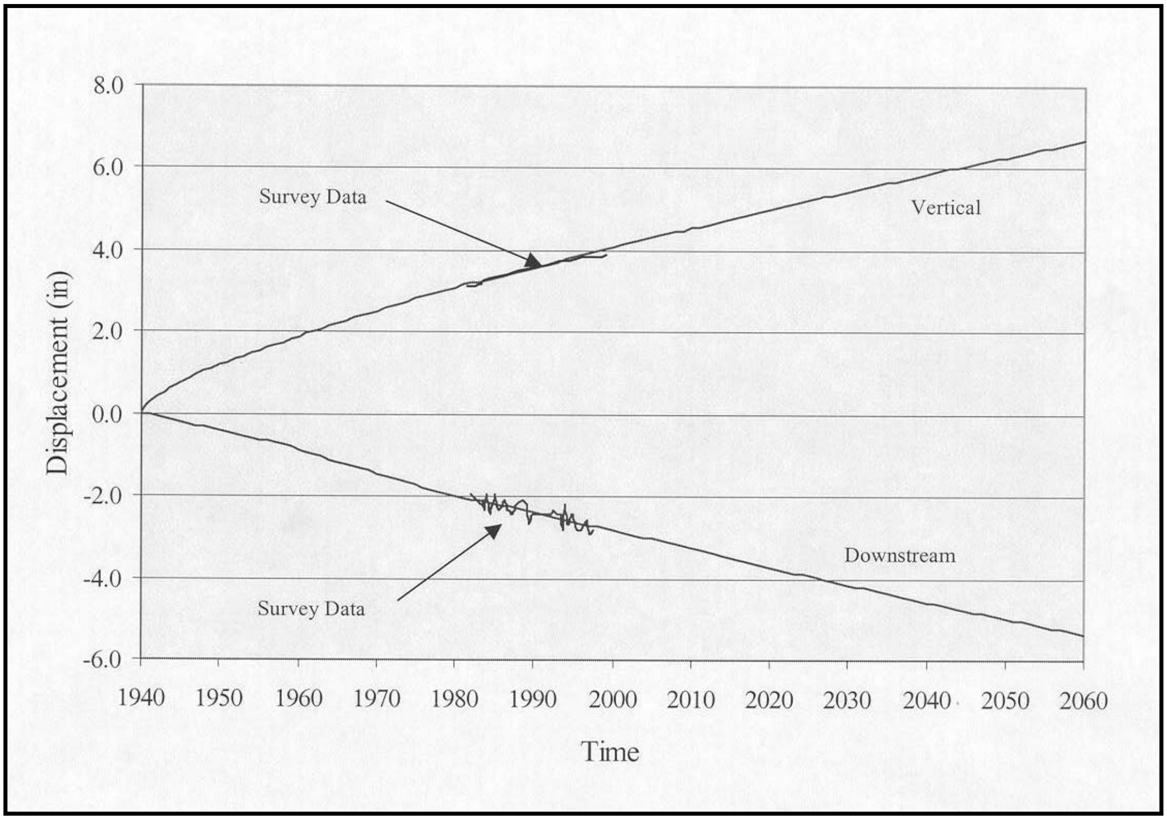


Figure B-49. Survey displacements versus finite element model deformations

longitudinal stress from pressure from the upstream lock wall growth. This cracking is influenced by the concrete properties (strength and stiffness), concrete expansion rate caused by

AAR growth, support provided by the lock wall downstream of Block 47, and the effectiveness of the repairs. Variations on the baseline analyses for these parameters were used to quantify the variation in the structural performance as required for input into the reliability model. Figure B-50 plots the concrete strain in the top section for variations in the concrete compressive strength. It should be noted that the concrete strains varied within the nonlinear finite element model and were within the bounds of the existing test data of concrete specimens at Chickamauga. Also, note that the concrete modulus and tensile strength also vary when the compressive strength varies. For the baseline results with 5000-psi concrete, the repairs were made when the strain measurement was about 0.35 percent, and the effects of the grouted horizontal pins in mitigating this strain or cracking is evident in Figure B-50 where the curves change to a much flatter slope.

b. It should be noted that time = 0 references the year 1940 in the graph, the year the project became operational. The lines in Figure B-50 represent curve fits that were used to develop the formulas that served as the engineering basis for the reliability model from the finite element model analysis. The graph clearly displays the effect the concrete compressive strength has on the level of concrete strain around the miter gate anchorage. The weaker the concrete, then the higher the strain level at any particular time. Thus, the concrete compressive stress became a key random variable in the reliability model for Block 47. Another key input variable is the installation and effective repair life of the horizontal pins to stabilize the upper portion of the monolith.

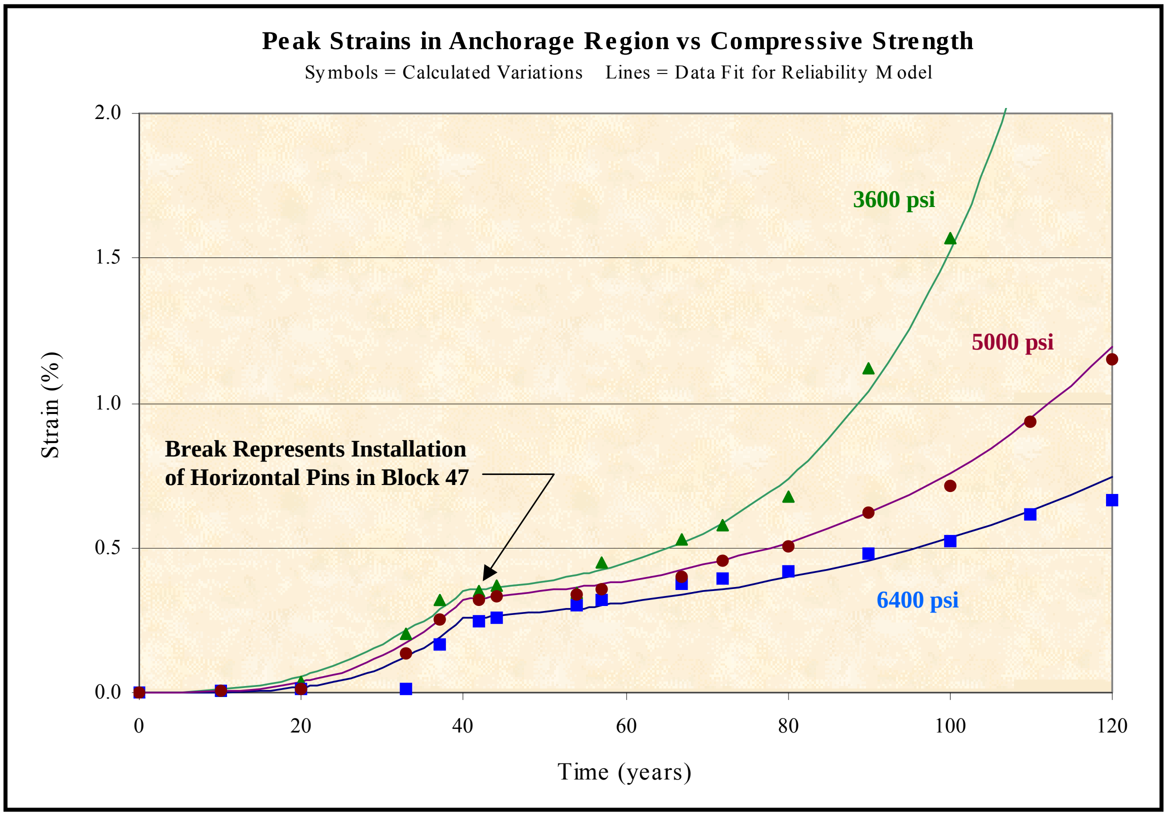


Figure B-50. Block 47 peak strain in anchorage region versus concrete strength

c. The next decision involved what concrete strain should be used as the maximum value for the limit state of each maintenance scenario. Typical American Concrete Institute (ACI) guidelines limit the design strain of the concrete to 0.3 percent. Since reliability models typically are based upon capacity without safety factors, a higher level of strain was used for the baseline, fix-as-fails limit state. Based upon the finite element modeling, ACI guidelines for design, and engineering judgment, the reliability team decided to use a strain level slightly higher than traditional concrete design for the baseline condition since the cracking was in the critical section of the miter gate anchorage region. Therefore, a limit state of 0.5 percent concrete strain was selected for the baseline, fix-as-fails condition. A repair event tree was developed for the baseline condition according to the 0.5 percent strain level in the region of the embedded anchorage. With the decision regarding the baseline criteria made, the reliability team next focused on how to address the advance maintenance scenario for Block 47 in terms of developing a hazard function and repair event tree. There is more than one way to handle the effect on the hazard function for the advance maintenance scenario where repairs are initiated prior to the strain reaching a level 0.5 percent. One way to do this is to use additional finite element modeling techniques to determine the long-term effectiveness of a repair and assume it must be undertaken when the strain reaches a particular level. This is the preferred method from an analytical perspective, but funding and schedule constraints did not allow for this approach. Therefore, the team adopted a simplified way to achieve the same general effect. Instead of developing new formulas for the reliability model specific to a repair, the same model was used with a lower limit state for concrete strain. A lower criterion of 0.3 percent strain, to match the maximum ACI design guideline, was used for the limit state for the advance maintenance condition. Thus, the strain level was reached sooner, meaning the advance maintenance hazard function has higher values than the baseline scenario for the same point in time. However, the repair event tree consequences are greatly reduced compared with those of the baseline condition. The goal was to initiate repairs before the strain reaches the baseline level of 0.5 percent, and that will help prolong the life of the structure and minimize the chance of a catastrophic failure. Thus, the overall result for the advance maintenance condition for Block 47 was a higher hazard rate, but significantly reduced consequences associated with repair cost and required chamber closure time.

B-35. <u>Block 47 random variables for probabilistic analysis</u>. The basic premise behind any reliability analysis is to determine which variables control the limit state and use the proper range and distribution in the probabilistic analysis. For Block 47, the parameters controlling the concrete strain had to be determined using the finite element model analytical tools. Once the base case was established and calibrated, the reliability team focused its efforts on determining which parameters had a significant impact on the concrete strain. Several iterations with the finite element model allowed the team to focus on three variables: concrete compressive strength, expansion rate of the concrete, and the "effective repair life" of the horizontal pins used to repair the vertical crack damage at the top of Block 47 (see Figure B-47). Other variables were evaluated in the finite element model but were found to have minimal effect on the concrete strain in the miter gate anchorage region. These other variables were left in the reliability model as random variables, but their effect on the Block 47 concrete strain was minimal. The variables found to have little effect on the concrete strain for Block 47 included the downstream approach wall spring stiffness, corrosion of the anchors, and yield strength of the anchors. The anchors were not key factors to the analysis of Block 47 because the concrete strain reaches its critical strain limit prior to the degradation of the anchors. The deterioration of the downstream approach wall concerned some of the engineers about its ability to provide sufficient restraint to Block 47 in the downstream direction. To consider the effect of variations in the downstream wall support on Block 47, parametric analyses using ±20 percent factors on the spring supports were considered. Significant cracking occurs in the monoliths just downstream of Block 47, and to account for continued cracking in these blocks, the downstream support was degraded in time in the base case model. The ±20 percent bands for the downstream support were applied in the finite element model by degrading the stiffness shape with respect to time. The results of this parametric variation indicated that the strains in the top section were not very sensitive to this effect, and the response was not characterized for this variable. Therefore, even though the downstream wall stiffness and anchors were included as random variables in the Block 47 reliability analysis, only the three critical variables will be detailed in this narrative.

a. Concrete compressive strength As previously noted (see Figure B-50), the compressive strength of the concrete played a key role in the determination of peak concrete strains in Block 47. Fortunately, concrete compressive strength data were available from testing of samples that were cored at Chickamauga. This information was used to develop the range and distribution of concrete strengths used in the reliability analysis. Based upon test data from core samples at Chickamauga, a truncated lognormal distribution was used for this variable with a mean of 4,500 psi. A lower limit of 3,000 psi and upper limit of 8,400 psi were used for the concrete compressive stress. Sensitivity runs indicated that the truncation limits did not have much effect on the overall analysis. A truncated normal distribution was used in the reliability model based upon the available test data.

b. Expansion rate of concrete. The only truly reliable test data available were associated with the concrete compressive strength. However, the expansion rate of the concrete also had a significant impact on the concrete strain. Variations in the AAR growth rate were characterized by using ±20 percent bands on the imposed expansion rates defined in the base case model, which was calibrated to current and historical conditions. Since the pressure load on the local finite element model was due to the expansion rate in the lock wall, the pressure amplitude was also varied by the corresponding ±20 percent factor. From these analyses, a factor was defined as a function of time and expansion rate that was applied to the concrete strains for the reliability analysis where the expansion rate was a random variable. The base case used available survey data available only from 1982 to establish the base case expansion rate. The ±20 percent band surrounding the base case value gave the team confidence that this factor would capture any amount of expansion the monolith would see within the study period. The Engineering Team agreed that the expansion rate factor should be correlated to the compressive strength of the concrete. The expansion rate was inversely correlated in the reliability model with whatever concrete compressive strength was selected for a particular iteration. Thus, if a high concrete compressive strength was randomly selected in the reliability model, a low expansion rate was also used for that particular iteration.

c. Effective repair life of horizontal pins. The last parameter of the analysis that significantly affected the concrete strain was the effective repair life of the horizontal pins across the vertically cracked section at the top of the structure. These pins were installed in the early 1980's as a means to stabilize the upper portion of Block 47 across the series of vertical cracks. The effect of this repair on the concrete strain in the upper portion of that monolith is evident from the peak strain graph shown in Figure B-50. The point at which the concrete strain curves flatten out indicates the effectiveness of this repair. The tricky part is to determine how long the repairs will be effective given the continued expansion of the concrete, especially at the upper portion of the monolith where it is free from restraint in the transverse direction. Further detailed modeling was not available given the funding and schedule constraints. Therefore, a factor as a function of time after grouted pin failure that considered the "effective repair life" as a random variable was developed for use in the reliability model. Given the base case for the expansion rate of the concrete, the reliability team did not anticipate the pins to be functional beyond 2020. Therefore, the upper limit on the effective repair life of the pin was set at 35 years. The lower limit was set for 20 years since the pins appear to be effective at the current time. Given these limiting values and lack of available data, the reliability team used 30 years as a mean value and decided to use a triangular distribution weighted towards the later part of the distribution (30 to 35 years). Again, sensitivity runs through both the finite element model and reliability model tended to verify this to be a reasonable range for these parameters.

B-36. Block 47 Time-Dependent Reliability Model. A state-of-the-art, analytical reliability model for the deterioration of the concrete around the miter gate embedded anchorage of Block 47 was specifically coded to compute the time-dependent probabilities of unsatisfactory performance (hazard functions) from 1940 through 2060. The model was used to compute hazard functions for the baseline fix-as-fails scenario, as well as the advance maintenance scenario. The reliability model used the analysis from the finite element modeling of this structure as a basis for the computations of the hazard functions. The model was written in Visual Basic software using @Risk libraries. @Risk is a software package that interfaces with the Visual Basic language to allow the use of Monte Carlo simulation of random variables for the probabilistic analysis. The opening reliability model interface menu for the Block 47 reliability model is shown in Figure B-51. The opening interface menu requests the user to input information such as the district, project, and monolith being analyzed. The reliability model requires several input menus to be entered to enable it to compute the hazard function: the random variable input menus for the concrete compressive strength, expansion rate of the concrete, and effective repair life of the horizontal pins. Input menus for the downstream wall stiffness and anchor properties must also be input, although their effect on the limit state for this component is minimal. A typical Block 47 reliability model input menu for the concrete compressive strength is shown in Figure B-52. Similar input menus were used for the other random variables in the analysis. Once the input menus were entered with the appropriate data, a

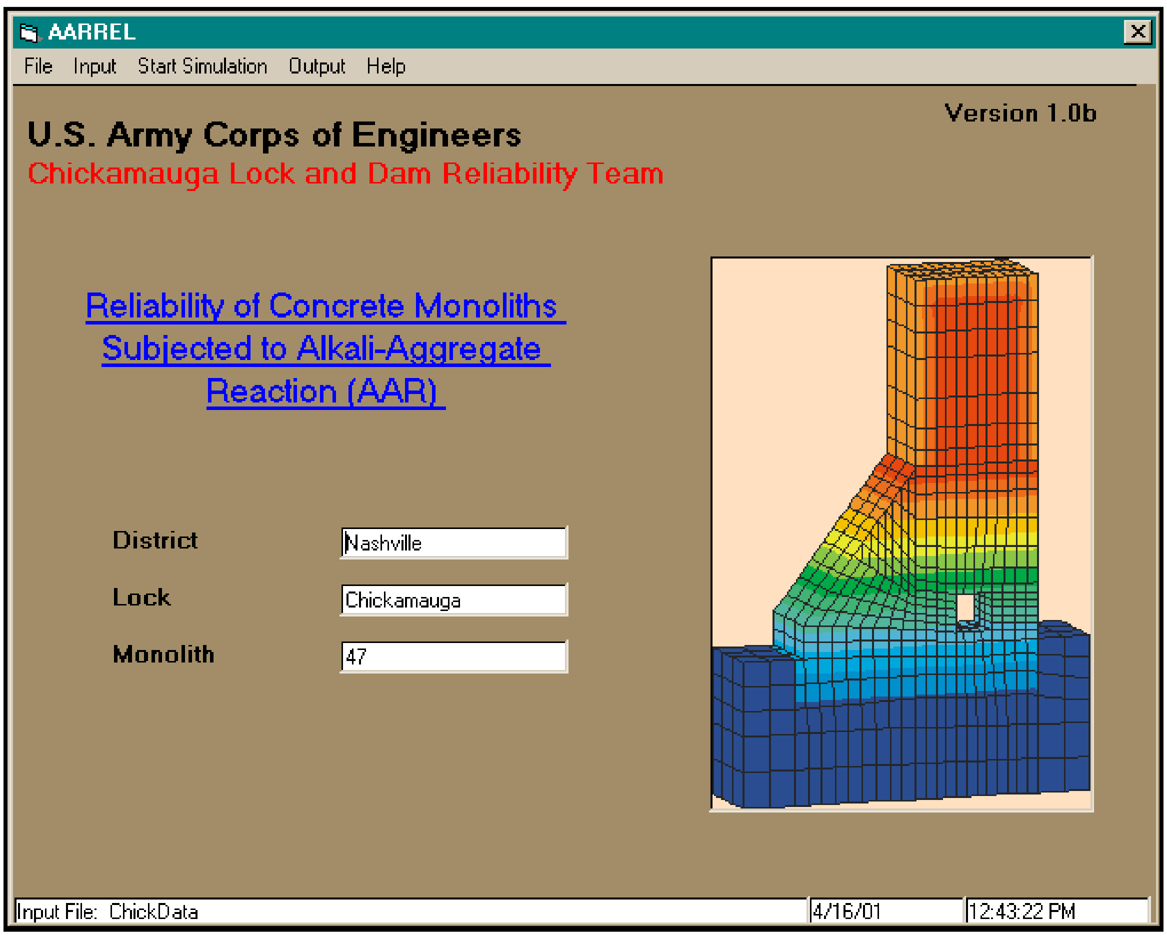


Figure B-51. Chickamauga Block 47 reliability model interface preset number of iterations was selected and the model run. A total of 50,000 iterations for each simulation were run for the Block 47 reliability model. This amount allowed for a full range of random variables to be selected. The efficiency of the model allowed for the high number of iterations, which tends to smooth out the hazard rate through time and capture a full range of combinations of random variables.

B-37. Block 47 Reliability Model Hazard Functions. For the purposes of this study, the hazard function was defined as the probability of unsatisfactory performance in a given year assuming it has survived up to that year. The formula for this is depicted in Equation B-5:

$$h(t) = \frac{Number\ of\ failures\ in\ year\ t}{Number\ of\ remaining\ survivors\ up\ to\ year\ t} \qquad \text{(B-5)}$$

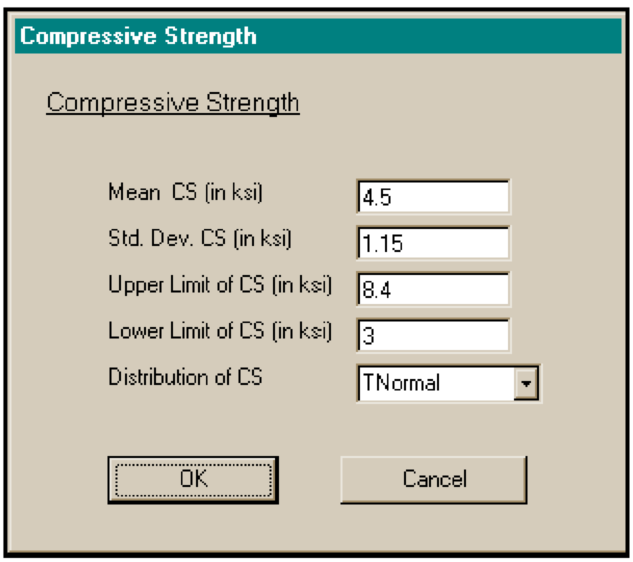


Figure B-52. Block 47 reliability model concrete compressive strength menu

The hazard functions were computed for both the baseline and advance maintenance scenarios. As noted earlier, the limiting criterion for each scenario was a maximum concrete strain in the miter gate embedded anchorage region of Block 47. The limiting criteria were 0.5 percent and 0.3 percent strain for the baseline and advance maintenance scenarios, respectively. The hazard functions computed by the reliability model for both scenarios are shown in Figure B-53. The hazard functions were based upon 50,000 iterations run through the reliability model for each maintenance scenario. Once a failure was reached in the model, the iteration was stopped and a new one was begun since multiple failures were not allowed within the same iteration according the definition of the hazard function. A new set of values were randomly selected using Monte Carlo simulation according to the distribution, mean, and truncated maximum and minimum values for each iteration. The sharp reduction in the advance maintenance hazard rate curve in 1982 reflects the installation of the horizontal pins across the damaged vertical cracks, which caused a temporary increase in the reliability of the structure under the advance maintenance case. Because the limiting strain was lower for the advance maintenance condition, the hazard rates for that scenario are higher than the baseline, fix-as-fails condition. However, the overall economic impact was less for the advance maintenance condition because the repair event tree has considerably less chamber closure time and repair cost.

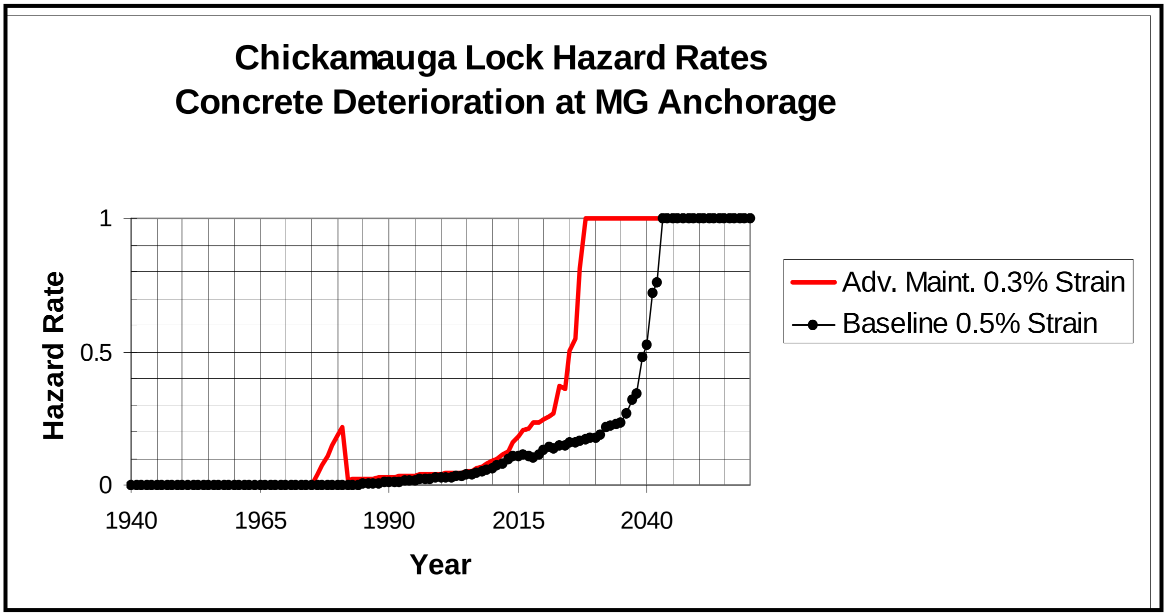


Figure B-53. Block 47 hazard functions for both maintenance scenarios

B-38. Block 47 Consequence Event Trees. The event tree is where the engineering reliability analysis merges with the economic analysis. There are several parts of the consequence event trees. The event trees were used in conjunction with the hazard rates for each maintenance scenario. The event tree format used for both maintenance scenarios was the same. However, the values were considerably different. The leading branch of the event tree is the hazard rate for the year being analyzed within the economic model. The output from the reliability model provided the annual hazard rates for the first branch of the event tree. The remaining branches were developed using engineering judgment by the Reliability and Engineering Teams. The second branch of the event tree is the level of failure and corresponding repair. This branch is broken into three levels corresponding to various amounts of damage. Three levels of unsatisfactory performance were used for this event tree: catastrophic, major failure, and an operational failure. The third branch is the estimated repair cost associated with each level of failure. The fourth branch represents the number of days the lock chamber is closed for repairs. It is very important to note that these repairs are reactive as opposed to preventive. Thus, no preparation time or readiness is assumed prior to the failure. The final branch is the repair effectiveness in terms of upgraded reliability. The reliability is upgraded by "sliding back" the hazard rate values a select number of years according to the level of repair. The "sliding back" technique is the easiest way for the economic model to handle upgrading the reliability of components after they have been repaired. For the lock components at Chickamauga, it does not make sense to reset the future reliability equal to 1.0 even if a totally new component is part of the repair: the surrounding concrete will continue to expand over time, thus degrading the new component after it has been replaced.

a. Baseline, fix-as-fails event tree. The baseline, fix-as-fails event tree is shown in Figure B-54. The costs, closure times, and other data are consistent with the hazard function limit state of 0.5 percent concrete strain in the miter gate anchorage region. Again, the repair cost and chamber closure times represent emergency, reactive types of repair. The Nashville District Operations repair fleet is assumed to make most repairs. The daily cost for the full fleet including materials is approximately $30,000 per day. This information was obtained by reviewing recent major dewatering jobs done by the Nashville District on navigation locks. The repair fleet rate includes all labor, equipment, and material cost for repairs not requiring extensive material costs. For repairs with extensive material costs, such as a repair involving fabrication of a new set of miter gates, the material and fabrication costs are added separately to the repair. The repair cost generally equals the number of days the fleet is onsite plus any cost of major materials. This daily fleet rate compares well with other districts within the Ohio River and Great Lakes Division. A detailed breakdown of each level of repair is provided.

(1) Baseline scenario event tree–catastrophic failure branch.

(a) This branch is assumed to be the least likely to occur. A 5 percent chance of occurrence is applied to this branch. This failure assumes a catastrophic failure of the miter gate anchorage caused by the deterioration of the concrete. A 5 percent chance might be considered high for a catastrophic event; but USACE typically does not let a component degrade until failure occurs, which is what the baseline, fix-as-fails scenario is predicated upon. The catastrophic failure assumes that a miter gate anchorage failure causes a detrimental failure of the miter gates, causing them to need replacement. It should be noted that there are no spare miter gates for Chickamauga. A new set of lower miter gates would need to be fabricated under this type of failure. The chamber is assumed closed for 365 days while new miter gates are fabricated under emergency conditions. The repair fleet is assumed to be onsite for the time to repair the initial damage, prepare the new gates/anchorage, and install the new gates. This time is estimated to be 90 days. A breakdown of the repair cost for this event is detailed below:

Fabricate and deliver a new set of lower miter gates		$4,500,000
Materials for repair of anchorage/concrete		$ 500,000
Repair fleet time at site (90 days plus 5 days transport)		$2,850,000
	Total	$7,850,000

(b) The repair assumes that the section of the monolith around the miter gate anchorage is replaced when the new anchorage is placed in the monolith. Future reliability is assumed to be greatly improved for this limit state. Therefore, the hazard rate is “slid back” to the value it was 50 years prior to this failure.

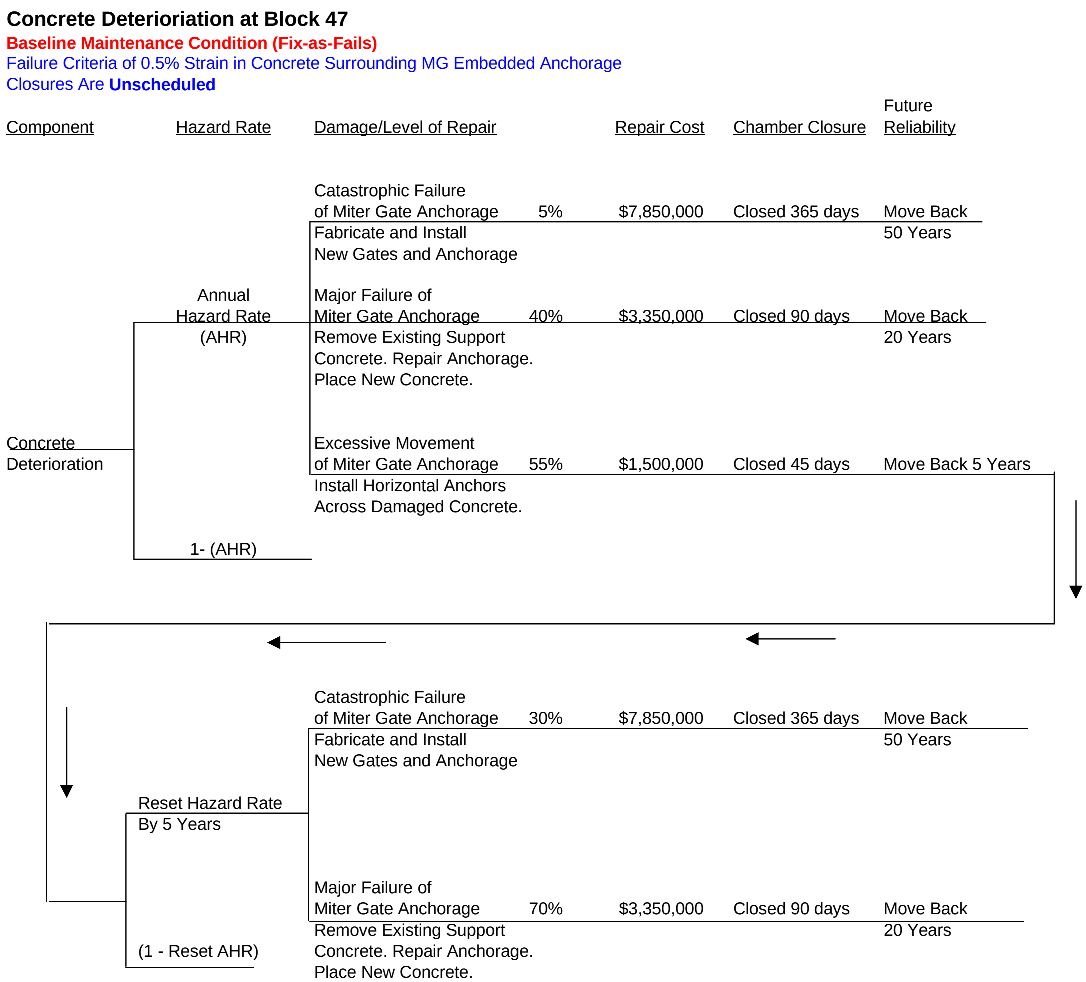


Figure B-54. Block 47 baseline, fix-as-fails repair event tree

(2) Baseline scenario event tree–major failure branch.

(a) This second most likely branch is assumed 40 percent of the time when a concrete strain failure in the fix-as-fails maintenance mode is encountered. This failure assumes less damage to the anchorage and surrounding structure than with the catastrophic failure. For this repair scenario, the anchorage is damaged, but the miter gates are salvaged and repaired. However, immediate repairs to the anchorage are needed. The repair fleet is assumed to be onsite for the entire duration of chamber closure, which is 90 days. A breakdown of the repair cost for this event is detailed below:

Materials for repair of anchorage/concrete		$ 500,000
Repair fleet time (90 days plus 5 days transport)		$2,850,000
	Total	$3,350,000

(b) This repair is not as extensive as that of the catastrophic scenario. There is no replacement of the miter gate or anchorage. Therefore, the improved reliability is less than that of the catastrophic repair. The hazard rate for the major failure branch is slid back 20 years.

(3) Baseline scenario event tree–operational failure branch.

(a) This scenario is considered the most likely under the baseline, fix-as-fails maintenance scenario. It is assumed 55 percent of the time. The failure assumes excessive movement of the miter gate anchorage. The need for repair is immediate and calls for installation of horizontal anchors across the damaged section of concrete. The chamber is assumed closed for 45 days for this repair work. Cost breakdown is shown below:

Repair fleet time (45 days plus 5 days transport)	$1,500,000

(b) The unique thing about this event branch is that it is assumed to be a one-time temporary repair. It is assumed that the horizontal anchors can be installed only once and the repair only slides the hazard rate back 5 years under the baseline maintenance scenario. Once the temporary repair is made and the hazard rate is reset, two repair options exist for the economic model to select in future years: the catastrophic and major failure branches. The same costs and chamber closures assigned previously are assigned to this new branch off the operational failure branch. The percentages have been changed to 30 percent and 70 percent for the catastrophic and major failure branches, respectively.

b. Advance maintenance event tree. The advance maintenance repair event tree has the same format as the baseline, fix-as-fails event tree. There are three levels of repair options depending upon the severity of the “failure”. The event tree for this scenario is shown in Figure B-55. The major differences between the two scenario event trees are the percentage, costs, and chamber closure times of the different repair options. It is also important to note that the limit state for the advance maintenance scenario is a concrete strain that is 0.3 percent compared to 0.5 percent for the baseline condition. The advance maintenance scenario is a preventive approach: repairs are initiated prior to any failures. Thus, damage at the time of repair is assumed to be less than the baseline, fix-as-fails scenario. Less damage assumes less repair cost and time, and less chance of a catastrophic failure occurring.

(1) Advance maintenance scenario event tree–catastrophic failure branch.

(a) This branch is assumed to be the least likely to occur. The chance of this repair occurring has been reduced from 5 percent under the baseline to only 1 percent in the advanced maintenance condition. Additionally, the severity of the failure has been reduced under the advance maintenance condition since a less damaging limit state occurs under the advance maintenance scenario. The failure for this branch is also less devastating. It assumes that the miter gate anchorage fails, but the miter gates themselves are salvageable.

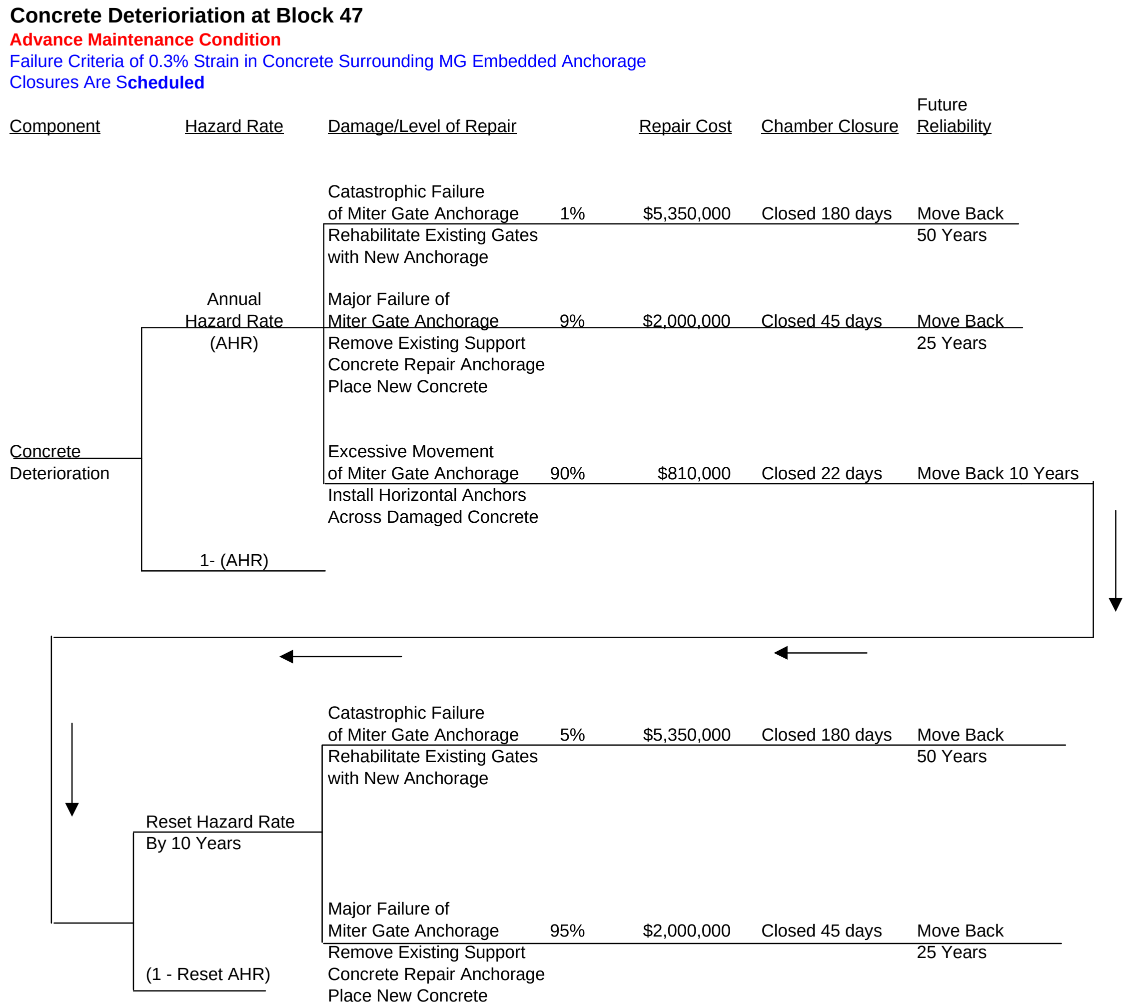


Figure B-55. Block 47 advance maintenance repair event tree

Therefore, new gates are not fabricated, thus reducing chamber closure time. Chamber closure time is reduced from 365 days under the baseline to 180 days under the advance maintenance condition. The cost breakdown for this branch is shown below:

Materials for repair of anchorage/concrete	$2,500,000
Repair fleet time at site (90 days plus 5 days transport)	$2,850,000
	$5,350,000

(b) Future reliability is assumed to be greatly improved for this limit state. Therefore, the hazard rate is moved back to the value it was 50 years prior to this failure.

(2) Advance maintenance scenario event tree–major failure branch.

(a) This second most likely branch is assumed 9 percent of the time when an advance maintenance concrete strain failure is encountered in the model. This is sharply reduced from the 40 percent chance assigned to this branch under the baseline scenario. For this repair scenario, the anchorage is damaged, but the miter gates receive negligible damage. However, immediate repairs to the anchorage are needed. The repair fleet is assumed to be onsite for the entire duration of chamber closure, which is 45 days. This again is a significant reduction from the 90 days of closure required under the baseline event tree for Block 47. The cost breakdown for this branch is shown below:

Materials for repair of anchorage/concrete	$ 500,000
Repair fleet time (45 days plus 5 days transport)	$1,500,000
	$2,000,000

(b) Future reliability is upgraded by sliding the following year hazard rate back by 25 years.

(3) Advance maintenance scenario event tree–operational failure branch.

(a) This scenario is again considered the most likely result of a concrete strain failure. Since it is the repair with the smallest consequences, it receives the highest chance of occurrence under the advance maintenance scenario. A 90 percent chance of occurrence is assigned to this repair branch. This compares with 55 percent assigned to it under the baseline scenario. The failure assumes movement of the miter gate anchorage. The need for repair is immediate and calls for installation of horizontal anchors across the damaged section of concrete. Since damage is less than that under the baseline condition, the chamber closure time is reduced from 45 days in the baseline event tree to 22 days for the advance maintenance event tree. The cost breakdown for the advance maintenance repair is shown below:

Repair fleet time (22 days plus 5 days transport)	$810,000

(b) This repair is again assumed to be a one-time temporary repair. However, it is considered much more effective under the advance maintenance condition than under the baseline, fix-as-fails scenario. The hazard rate is reset by 10 years under the advance maintenance versus 5 years for the baseline condition. Once the temporary repair is made and the hazard rate is reset by 10 years, two repair options exist for the economic model to select in future years: the catastrophic and major failure branches. The same costs and chamber closures are assigned as previously to this new branch of the Advance Maintenance Scenario Operational Failure branch. The percentages have been changed to 5 percent and 95 percent for the catastrophic and major failure branches, respectively. As shown in Figure K10, these same branches were set at 30 percent and 70 percent, respectively, under the baseline scenario event tree.

B-39. Economic Results of Block 47 Engineering Reliability Analysis.

a. Using the hazard functions and event trees specific to each maintenance scenario, the economists analyzed the impact of each scenario for Block 47 using the Life Cycle Component Model (LCCM). The LCCM is a spreadsheet-based analytical model that was developed to analyze engineering hazard functions and event trees. The economic model produces expected costs as its end product. The expected annual costs from the model represent a culmination of costs from repairs, navigation delay, and all other categories that are being modeled. The alternative with the lowest expected annual cost is the preferred economic alternative since this is a cost-basis model. Three scenarios were tested in the WOPC analysis: baseline (fix-as-fails), advance maintenance, and replacement-in-kind.

b. An excellent summary of the economic analysis relative to the Block 47 engineering reliability analysis is shown in Table B-6. The table gives a clear breakdown of the expected average annual costs associated with the reliability of Block 47. The results are presented in year 2000 dollars. The dollar amounts reflect the average annual expected costs for the period of 2000 through 2060, all relative to a present worth date consistent with the year 2000. It is evident that the baseline, fix-as-fails scenario has the highest average annual expected cost of all WOPC scenarios, $1,240,252. The average annual expected cost drops 46 percent to $673,281 under the advance maintenance scenario. This is directly attributable to the reduced consequences associated with the repair options in the event tree. Since the limit state for the advance maintenance scenario is less than that for the baseline, fix-as-fails condition, the consequences are much lower in terms of repair cost and chamber closure time. Replacement-In-Kind (RIK) is under the umbrella of the WOPC analysis, but it truly is different from the baseline and advance maintenance scenarios. The RIK alternatives test the replacement of the chamber with another chamber of exactly the same size at different dates, whereas the baseline and advance maintenance scenarios consider keeping the existing project operational through the study period. Once the new chamber is complete in the RIK scenario, the reliability of Block 47 becomes unimportant. It is considered 100 percent reliable in the future since the old chamber is no longer in service. The reason average annual expected costs are associated with the RIK alternatives is that advance maintenance is assumed for the existing chamber until the new lock chamber is completed. Therefore, the average annual expected cost is lowest for the RIK in 2010 and gets continually higher the later the RIK is completed. As expected, all RIK scenarios have lower average annual expected costs for the performance of the component than those for the baseline and advance maintenance scenarios.

Table B-6. Economic Results from Block 47 Reliability Analysis

Block 47 Average Annual Present Worth Expected Costs				
WOPC Scenario	**Repair Cost**	**Transportation Impacts**	**External and Recreational**	**Total**
Fix-As-Fails	$259,000	$448,926	$532,326	$1,240,252
Advance Maint.	$202,207	$251,966	$219,108	**$673,281**
R-I-K in 2010	$99,232	$113,249	$45,287	**$257,768**
R-I-K in 2015	$139,671	$162,765	$80,902	$383,338
R-I-K in 2020	$166,025	$198,019	$118,575	$482,619

Section V
EXAMPLE 5
Issue: Reliability Analysis for Mechanical Operating Equipment
Project: Lower Monumental Lock Major Rehabilitation Study

B-40. Background. This example is based on the mechanical appendix of the Lower Monumental Lock and Dam Major Rehabilitation Evaluation Report. The mechanical analysis for this report was completed in 2003. The mechanical analysis followed the procedures outlined in ETL 1110-2-560, referred to herein as the ETL. The Markland Locks and Dam Major Rehabilitation Evaluation Report, March 2000, was used as an example for preparing the Mechanical Appendix for the Lower Monumental Major Rehab Evaluation Report. Assistance in preparing Appendix was also received from others with experience in preparing rehabilitation reports. In this example, the indented text is taken from the Mechanical Appendix of the Lower Monumental Lock and Dam Major Rehabilitation Evaluation Report to show how the finished product was presented in the report. This example is presented in a step-by-step format showing the process used to complete the analysis. ***Please keep in mind that this example references the failure rate data from the guidance as it existed in 2003. The failure rates in the guidance at that time are not from navigation lock and dam equipment, and updated failure rates are being established as a part of an updated ETL*** 1110-2-***560. The process is applicable, but the updated failure rates and other associated updates should be used as opposed to the ones provided in this example.***

B-41. Step 1.

a. Determine the features and their functions as part of the facility for which the reliability analysis will be performed. Focus on systems and subsystems that must be operational for the lock to actually allow vessels to be locked through. This is necessary because system and subsystem failure must create an economic impact in order to justify their modification or replacement. This is done by reviewing the facility as-built drawings and by site visits to discover any changes and upgrades following original construction. Although other systems support lock operations, it was determined that those systems did not cause much impact on lock operation. This is also a good time to get acquainted with facility staff, brief them on the analysis you will be performing, and learn the facility operating procedures, constraints, and equipment problem areas. Equipment repair and replacement records should also be reviewed at this time.

> The Lower Monumental lock is 86 feet wide by 666 feet long. The lift of the lock is approximately 100 feet. The lock has one upstream and one downstream lift gate and two filling and two emptying tainter valves. The lift gates are raised and lowered to allow vessels to enter and exit the lock. The tainter valves are used to fill and empty the lock. Each lift gate is raised and lowered with dual hydraulic systems. Each tainter valve is opened and closed by a hydraulic cylinder and associated hydraulic system. Except for the hydraulic power units and some other miscellaneous items replaced during the course of normal maintenance, the lock machinery is the original machinery installed when the lock was completed in 1969. The lift gate hydraulic power units were replaced in 1989,

and the tainter valve hydraulic power units were replaced in 1992. A downstream lift gate cable drum bearing was replaced in 2000, and new upstream lift gate cables were installed in 1992. In 2004, the lift gate cables for both gates were replaced after it was discovered the cables had a significant loss of cross section due to corrosion. The project currently has replacement bearings for each lift gate cable drum and the old main speed reducers for the lift gate machinery on hand.

b. The mechanical analysis for the Lower Monumental Lock included the upper and lower lift gate operating machinery systems and the tainter valve operating machinery systems. These systems were further broken down into the discrete subsystems making up these systems. For the most part, the systems have been reliable with most equipment breakdowns being minor in nature.

B-42. Step 2. Determine the number of lockages the lock operating equipment has experienced. The number of past lockages can be determined using the lock historical records and LPMS data. Some of the data for Lower Monumental were given in tonnage rather than the actual number of lockages. These data were converted to lockages by using the typical barge tonnage data of tows on the river. Because additional lockages are performed by lock operators for such things as maintenance and debris passage, some additional lockages were added to those recorded.

The number of lockages occurring in the years 1969 through 1988 was not recorded, but tonnage data was recorded for these years. Based on historical lockage and tonnage data, the total number of lockages through the end of 1989 was estimated to be 25,075. Actual recorded lockage data from 1990 through 2001 is available from the Lock Performance Monitoring System (LPMS). In addition to these lockages, the various machinery subsystems are also cycled to varying degrees for maintenance purposes by project operations. For purposes of this study, the number of lockages recorded in the LPMS data was increased slightly each year to account for these additional equipment cycles.

B-43. Step 3.

a. Project the future lockages during the length of the study. This will require a study of the transportation and commodity environment for the region. This was done by our economists.

For future projections, the number of lockages is based on the projected shipping tonnage. The projected tonnage through Lower Monumental lock was estimated from economic modeling based on historical tonnage data. The economic model projected the number of lockages per year will stay constant for future years of the study period.

b. Lockage data for Lower Monumental Dam is shown below in Table B-7 and Figure B-56. Data for years up to 1989 were grouped together because the form of the data changed that year and because there were major equipment replacements that year.

B-44. Step 4. Select the alternatives to be covered in the analysis. The various Without-Project scenario (fix-as-fails, status quo, etc.) alternatives must be analyzed to have a basis by which to judge the other alternatives selected. The other alternative usually selected for analysis is the

With-Project (planned replacement or major rehabilitation) alternative. Other alternatives, such as partial rehabilitation and furnishing additional spare parts, may also be considered.

Table B-7. Operating Cycles for
Lower Monumental Lock

Year	Cycles
through 1989	25,075
1990	1,616
1991	1,702
1992	1,724
1993	1,689
1994	1,622
1995	1,732
1996	1,477
1997	1,630
1998	1,671
1999	1,598
2000	1,747
2001	1,574
2002 and beyond	1,730

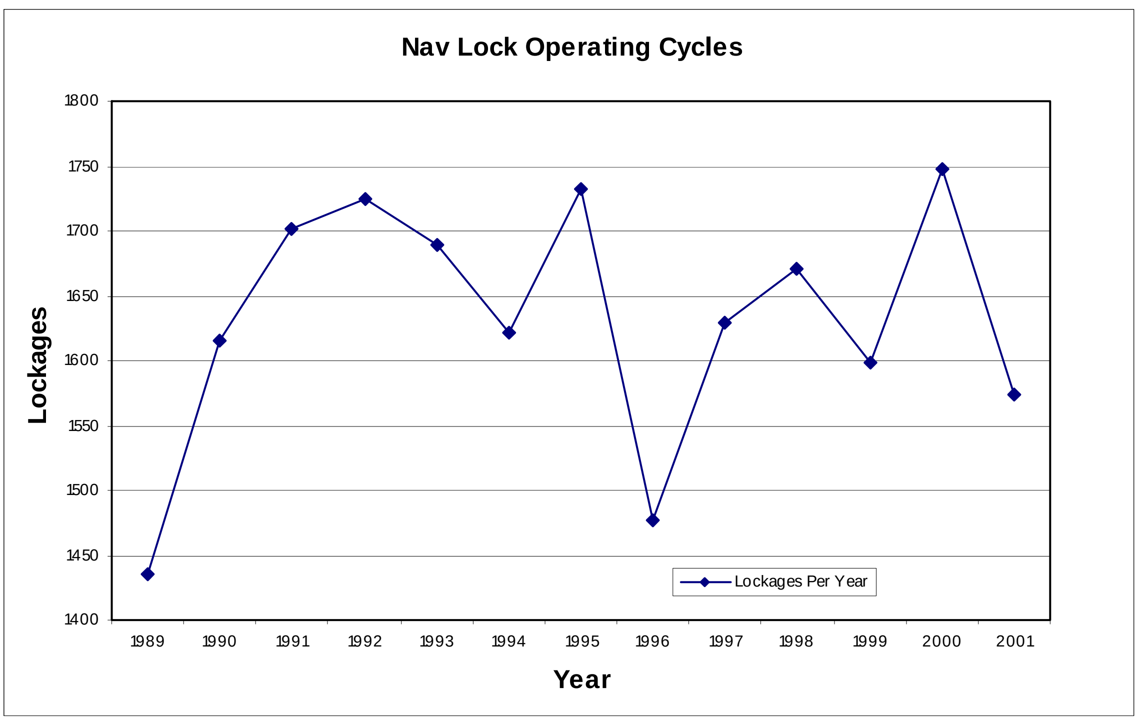


Figure B-56. Graphical representation of historical lock operating cycles

B-45. Step 5. Determine whether a target year for the major rehabilitation construction activity will be used or if the analysis is intended to determine the most economical year when major rehabilitation construction activities should occur. Because of scheduled work at other dams on the lower Snake and lower Columbia Rivers, the Lower Monumental Lock and Dam Major Rehabilitation study was based on a specific target year for the major rehabilitation construction activities.

Two alternative strategies are presented in the order of scheduled work requirements for the Without Project Condition. The alternative with the minimum scheduled work requirement is the "fix-as-fails" strategy that would only replace or repair components after a failure. In this scenario, the repair is only initiated in the economic model once the machinery "fails." This scenario is the starting point of the analysis. The other alternative is termed "spare parts." This scenario is based on immediate procurement of spare parts for one complete set of operating machinery for each of the lift gate and tainter valve machinery subsystems.

The With Project Condition considered in this report is "planned replacement." It is referred to as planned replacement rather than major rehabilitation since the cost of the work on each system separately does not necessarily exceed the minimum cost threshold necessary to qualify for major rehabilitation funds. All mechanical machinery used to operate the lift gates and tainter valves would be scheduled for replacement in 2009. This analysis assumed a start year of 2008. Following completion of the analysis, it was decided to delay the scheduled lock outage for rehabilitation until 2009 to allow a complete system-wide outage for lower Snake and lower Columbia River locks.

B-46. Step 6. Create the Reliability Block Diagrams (RBDs) for the subsystems that will be analyzed.

a. This is done using the information gained from the as-built contract drawings and field observations. To assist in this process, sketch the machinery arrangements to show the components and their arrangement in the subsystems. Next, list the components and their number present in the subsystem. Because of limited failure rate data, similar types of components, even though different in size, are considered to be more of the same component as far as failure rate data are concerned. For example, in the RBD shown in Figure B-57, even though the subsystem contains reducers that vary in size, the same failure rate will be used for each one and the reducer quantity is listed as 3. Each component for which failure rate data are available is assigned a letter and is used to create the RBD by connecting blocks containing the letter of the components as dictated by the machinery arrangement. In this case, the components are connected end-to-end because this is a series subsystem and all the components must function in order for the subsystem to operate. Components for which failure rate data are not available are not included in the RBD because they will not be included in the reliability calculations. Failure rate data for components are listed in the ETL and in Reliability Analysis Center's publication Nonelectronic Parts Reliability Data (Reliability Analysis Center (RAC) 1995). ***Keep in mind that the failure rate data for this type of equipment are being revised for an updated*** ETL 1110-2-560 ***to reflect rates that are more in line with navigation mechanical***

and electrical equipment. Other sources of data are being developed or may be available for use with permission of Headquarters, USACE.

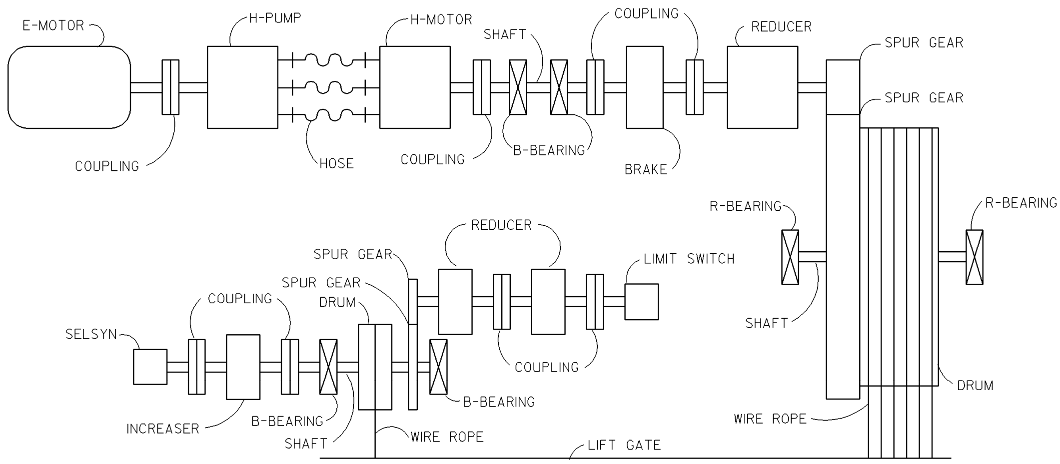
> The purpose of the machinery is to operate the lift gates and tainter valves. The major components required for mission success were defined and organized into RBDs. There are no parallel or redundant items in these mechanical subsystems; therefore, the RBDs were arranged as series system models. In this analysis, the structural supports and anchorages were not included in the model. They are unique to the system, and there is no published failure rate data available.

b. For the study, an RBD will need to be prepared for each subsystem that will be analyzed. If the With-Project alternative involves upgrading or replacing components with an arrangement different from the existing one, RBDs will need to be prepared for the new systems. The Lower Monumental study was based on equipment replacement in like kind.

c. Two subsystem RBDs are included for this example. The first RBD, illustrated in Figure B-57, is for the lower (downstream) lift gate mechanical machinery in the North Tower, and the second RBD is for the tainter valve mechanical machinery (Figure B-58). At Lower Monumental Dam, each of the four lift gate mechanical machinery subsystems is unique, partly because the upper lift gate is much smaller than the lower lift gate and partly because of differences in the positioning system components in the north side and south side systems. (The lock axis is in an east-west orientation.) The four tainter valve mechanical machinery subsystems are identical. In operation, the duty factors for the filling tainter valve machinery systems are different from those of the emptying tainter valve machinery systems, and the duty factor for the upper lift gate machinery system is different from the lower lift gate machinery system because of different machinery run time during lockages.

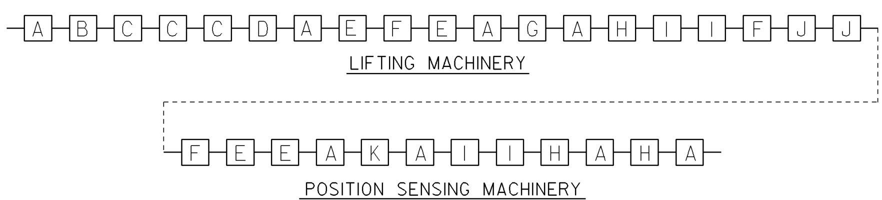
B-47. <u>Step 7</u>. Determine the duty factor *d* for each of the systems.

a. For the Fix-as-Fails alternative, this should be based on actual field measurements taken as equipment is being cycled. Some as-built and O&M data may contain this information; but to ensure accuracy on this point, actual field measurements should be used. The operating time measured in minutes or seconds is multiplied by the number of cycles made each year. Because this number is calculated for each year of the study, convert the number calculated to units of years. Consider both downstream and upstream lockages to get the correct duty factor. In the example spreadsheets that follow, the cycle duration is shown as 789 seconds. The accumulated number of cycles through the year 1989 is shown as 25075, so the duty factor for the year 1989 is 789 x 25075 / 3153600 = 0.6274. (The conversion factor for converting seconds to years is 3153600.) Likewise, for the year 1990, the cycle duration is again 789 seconds and the number of cycles shown for that year is 1616, so the duty factor for the year 1990 is 789 x 1616 / 31536000 = 0.0404. Because these duty factors are in units of years, the accumulated length of time the equipment has been in actual operation can be determined by adding the duty factor for the preceding years, so for the year 1990 the calculation for the accumulated years of actual operation is 0.6274 + 0.0404 = 0.6678. Take care to determine the duty factor on a year-by-year basis because the calculations for reliability and hazard rate are accumulative through the study.



MACHINERY ARRANGEMENT

COMPONENT	BLOCK	QUANTITY
E-MOTOR (ELECTRIC MOTOR)	**	1
COUPLING (SHAFT)	A	8
H-PUMP (HYDRAULIC, PISTON, AXIAL)	B	1
HOSE (PRESSURE)	C	3
H-MOTOR (HYDRAULIC, PISTON, AXIAL)	D	1
B-BEARING (BALL BEARING)	E	4
SHAFT	F	3
BRAKE (ELECTROMECHANICAL)	G	1
REDUCER (GEAR BOX)	H	3
SPUR GEAR	I	4
R-BEARING (ROLLER BEARING)	J	2
INCREASER (GEAR BOX)	K	1
LIMIT SWITCH	**	1
SELSYN	**	1
DRUM	*	2
WIRE ROPE	*	13
LIFT GATE	***	1



RELIABILITY BLOCK DIAGRAM

*** EVALUATED IN STRUCTURAL RELIABILITY ANALYSIS
** EVALUATED IN ELECTRICAL RELIABILITY ANALYSIS
* ITEMS NOT EVALUATED DUE TO LIMITED FAILURE RATE INFORMATION
STRUCTURAL SUPPORTS AND VARIOUS ANCHOR BOLTS WERE NOT EVALUATED

LOWER MONUMENTAL LOCK AND DAM
SNAKE RIVER, OREGON, WASHINGTON AND IDAHO
NAV LOCK REHABILITATION REPORT
MECHANICAL SYSTEMS - RELIABILITY BLOCK DIAGRAM
LOWER LIFT GATE MECHANICAL MACHINERY (LLGM)
NORTH TOWER

Figure B-57. RBD for lower lift gate machinery (North Tower)

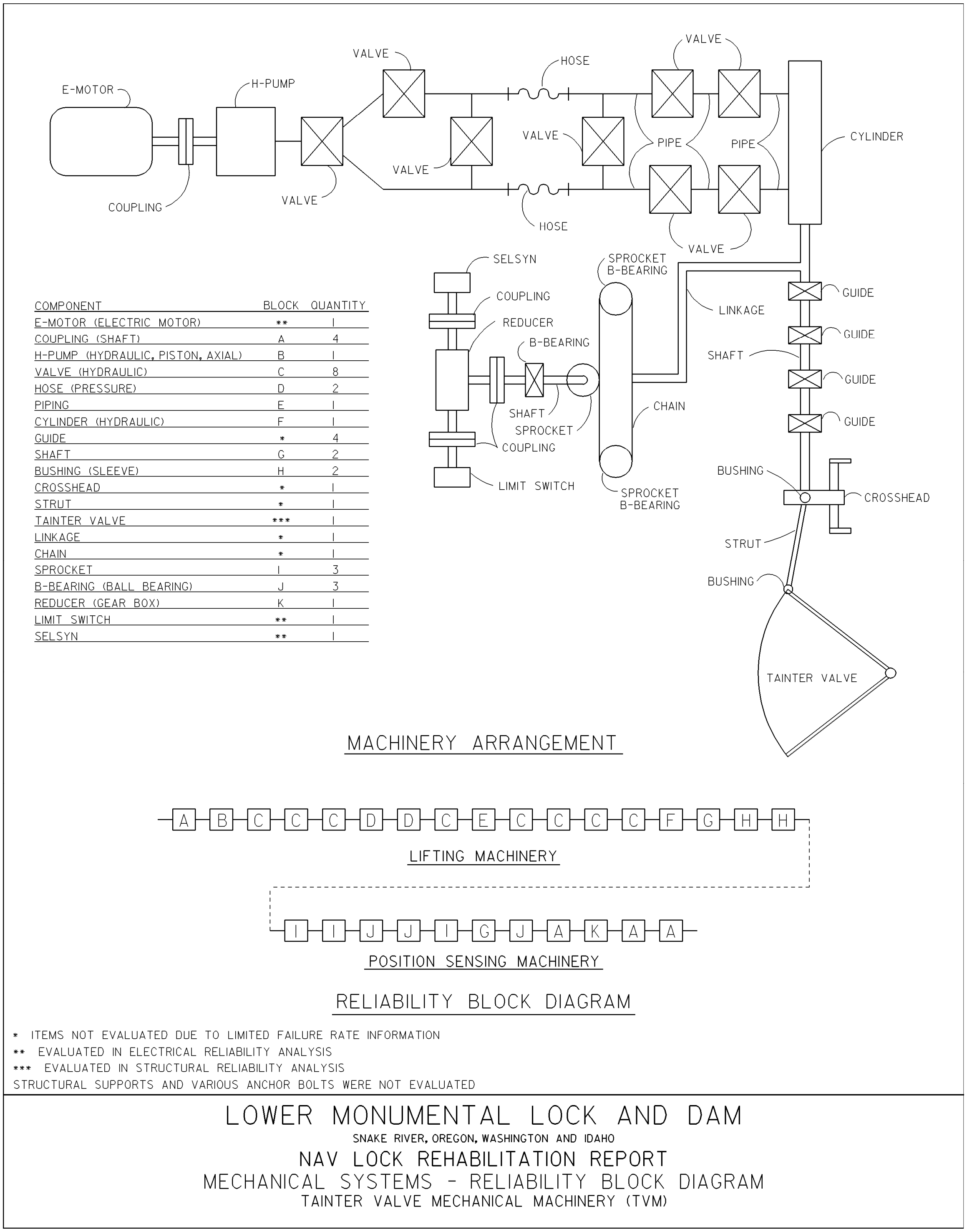


Figure B-58. RBD for tainter valve machinery

b. In the example spreadsheets in Figures B-59 and B-60, there is an input column on the far right with the heading "New." This column allows for components to be "new" or replaced in various years of the study. Because the reliability and hazard rate are affected only by components that are currently installed in the system, i.e., components that are no longer part of the system cannot have an effect on the system as it currently exists, the reliability and hazard rates are not calculated for the individual components prior to the year in which they were most recently installed and N/A is displayed for these components in those years. Therefore, the subsystem reliability and hazard rates are displayed as N/A for those years of the study, even though reliabilities and hazard rates for the original equipment are calculated for all years. The spreadsheet formulas for the reliability and hazard rates use IF and SUMIF functions to incorporate this feature. Because the duty factor varies for each year of the study and is the only changing variable, given a specific component, used in the equations for determining reliability and hazard rate, extreme care must be taken to ensure that the correct duty factor is used in these calculations.

> The lift gate and tainter valve machinery were considered to have a negligible failure rate during periods of non-operation (ignoring barge impact). The duty factor *d* is the ratio of actual operating time to total mission time *t*. The failure rate can be modified by the duty factor. The lock machinery operates a certain number of open/close cycles per year. When looked at on a yearly basis, the annual duty factor reduces to the length of service each year converted to units of years. The reliability analysis calculations take into account the varying lengths of service of the subsystem components.

B-48. Step 8. Select the environmental factors for the subsystem components. Assess the system operational environment to select these factors.

> The environmental conditions of the lift gate machinery were defined for the ambient service in a vibration-free, controlled environment. For this machinery, K_1 was set at 0.5. This machinery is located in equipment rooms that have some climate control. The environmental conditions of the tainter valve machinery were defined for the ambient service in a vibration-free, controlled environment for the nonsubmerged components and for the ambient service in a ship environment for the submerged components. For the tainter valve analysis, K_1 was either 0.5 or 2.0. Because the machinery design stresses are typically below 75 percent of the yield stress for the material, K_2 was set at 0.6 for all components except the hydraulic system components. The K_2 was set at 1.0 for the hydraulic system components because these items are typically rated on the operating pressure. Because ambient temperatures are typically in the 20^{o} Celsius (C) or lower range, K_3 was set at 1.0 for all machinery.

B-49. Step 9. Determine the Weibull distribution shape parameter β based on the dominant failure mode for each component. In the example spreadsheets (Figures B-59 and B-60), these values are mostly a 3 (wear) with some being a 1 (failure other than from wear). In the equation for calculating the component hazard rate, when β is 1, it causes the time factor of the equation to equal 1 and the component hazard rate ends up being a constant for the study period. In the equation for calculating the component reliability, when β is 1, the equation reduces to the exponential distribution.

Lower Monumental Nav Lock Rehabilitation Report
Mechanical System Reliability Analysis
Lower Lift Gate Mechanical Machinery (LLGM) North Tower
Fix-as-Fails

Notes:
1. Failure data is from ETL 1110-2-560 and RAC Automated Databook, CD-ROM.
2. NA is displayed for Reliability of new components for years before their installation.
3. NA is displayed for Hazard Rate of new components for years before their installatio
4. Cycles for 1989 is the total of all cycles prior to and through 1989.

INPUT DATA												
Cycle Duration (sec) =	789								Environmental K Factors			
Component/Block	Quantity	Failure Rate	Failure Mode	Weibull Index	Lambda '	MTTF (hrs)	Alpha/MTTF	Alpha (yrs)	K_1	K_2	K_3	New
Coupling	8	1.0038	misalignment	1	0.30114	3320715	1	379.0770	0.5	0.6	1.0	1989
H-Pump	1	7.7220	wear	3	3.861	259000	1.1	32.5229	0.5	1.0	1.0	1989
Hose	3	3.9300	bulging	1	1.965	508906	1	58.0943	0.5	1.0	1.0	1989
H-Motor	1	7.7220	wear	3	3.861	259000	1.1	32.5229	0.5	1.0	1.0	1989
B-Bearing	4	1.6445	wear	3	0.49335	2026959	1.1	254.5268	0.5	0.6	1.0	1989
Shaft	3	0.9298	fracture	1	0.27894	3585000	1	409.2466	0.5	0.6	1.0	1969
Brake	1	10.6383	jam/misalign	1	3.19149	313333	1	35.7686	0.5	0.6	1.0	1969
Reducer	3	5.0000	wear	3	1.5	666667	1.1	83.7139	0.5	0.6	1.0	1989
Spur Gear	4	3.2232	wear	3	0.96696	1034169	1.1	129.8614	0.5	0.6	1.0	1969
R-Bearing	2	2.8201	wear	3	0.84603	1181991	1.1	148.4236	0.5	0.6	1.0	1969
Increaser	1	7.1193	wear	3	3.55965	280926	1.1	35.2762	0.5	1.0	1.0	1995

DUTY FACTOR

Year	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Cycles per Year	25075	1616	1702	1724	1689	1622	1732	1477	1630	1671	1598	1747
Duty Factor (d)	0.6274	0.0404	0.0426	0.0431	0.0423	0.0406	0.0433	0.0370	0.0408	0.0418	0.0400	0.0437
Accumulated (yrs)	0.6274	0.6678	0.7104	0.7535	0.7958	0.8363	0.8797	0.9166	0.9574	0.9992	1.0392	1.0829

RELIABILITY [R(t)] OF INDIVIDUAL COMPONENTS

Year	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Component/Block												
Coupling	NA	0.9999	0.9998	0.9997	0.9996	0.9994	0.9993	0.9992	0.9991	0.9990	0.9989	0.9988
H-Pump	NA	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Hose	NA	0.9993	0.9986	0.9978	0.9971	0.9964	0.9957	0.9950	0.9943	0.9936	0.9929	0.9922
H-Motor	NA	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
B-Bearing	NA	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Shaft	0.9985	0.9984	0.9983	0.9982	0.9981	0.9980	0.9979	0.9978	0.9977	0.9976	0.9975	0.9974
Brake	0.9826	0.9815	0.9803	0.9792	0.9780	0.9769	0.9757	0.9747	0.9736	0.9725	0.9714	0.9702
Reducer	NA	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Spur Gear	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
R-Bearing	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Increaser	NA	NA	NA	NA	NA	NA	NA	1.0000	1.0000	1.0000	1.0000	1.0000

HAZARD RATES [h(t)] OF INDIVIDUAL COMPONENTS

Year	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Component/Block												
Coupling	NA	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026
H-Pump	NA	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Hose	NA	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172
H-Motor	NA	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
B-Bearing	NA	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Shaft	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024
Brake	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280
Reducer	NA	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Spur Gear	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
R-Bearing	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Increaser	NA	NA	NA	NA	NA	NA	NA	0.0000	0.0000	0.0000	0.0000	0.0000

RELIABILITY OF SUBSYSTEM [R_{subsys}(t)]

Year	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
	NA	NA	NA	NA	NA	NA	NA	0.9480	0.9438	0.9396	0.9355	0.9311

HAZARD RATE OF SUBSYSTEM [H_{subsys}(t)]

Year	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
	NA	NA	NA	NA	NA	NA	NA	0.1081	0.1081	0.1081	0.1081	0.1081

Figure B-59. Reliability analysis for North Tower downstream lift gate machinery (fix-as-fails alternative 1989-2000)

Lower Monumental Nav Lock Rehabilitation Report
Mechanical System Reliability Analysis
Lower Lift Gate Mechanical Machinery (LLGM) North Tower
Planned Replacement

Notes:

1. Failure data is from ETL 1110-2-560 and RAC Automated Databook, CD-ROM.

INPUT DATA												
Cycle Duration (sec) =		789							Environmental K Factors			
Component/Block	Quantity	Failure Rate	Failure Mode	Weibull Index	Lambda '	MTTF (hrs)	Alpha/MTTF	Alpha (yrs)	K1	K2	K3	New
Coupling	8	1.0038	misalignment	1	0.30114	3320715	1	379.0770	0.5	0.6	1.0	2008
H-Pump	1	7.7220	wear	3	3.861	259000	1.1	32.5229	0.5	1.0	1.0	2008
Hose	3	3.9300	bulging	1	1.965	508906	1	58.0943	0.5	1.0	1.0	2008
H-Motor	1	7.7220	wear	3	3.861	259000	1.1	32.5229	0.5	1.0	1.0	2008
B-Bearing	4	1.6445	wear	3	0.49335	2026959	1.1	254.5268	0.5	0.6	1.0	2008
Shaft	3	0.9298	fracture	1	0.27894	3585000	1	409.2466	0.5	0.6	1.0	2008
Brake	1	10.6383	jam/misalign	1	3.19149	313333	1	35.7686	0.5	0.6	1.0	2008
Reducer	3	5.0000	wear	3	1.5	666667	1.1	83.7139	0.5	0.6	1.0	2008
Spur Gear	4	3.2232	wear	3	0.96696	1034169	1.1	129.8614	0.5	0.6	1.0	2008
R-Bearing	2	2.8201	wear	3	0.84603	1181991	1.1	148.4236	0.5	0.6	1.0	2008
Increaser	1	7.1193	wear	3	3.55965	280926	1.1	35.2762	0.5	1.0	1.0	2008

DUTY FACTOR

Year	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
Cycles per Year	1730	1730	1730	1730	1730	1730	1730	1730	1730	1730	1730	1730
Duty Factor (d)	0.0433	0.0433	0.0433	0.0433	0.0433	0.0433	0.0433	0.0433	0.0433	0.0433	0.0433	0.0433
Accumulated (yrs)	0.0433	0.0866	0.1299	0.1732	0.2165	0.2598	0.3031	0.3463	0.3896	0.4329	0.4762	0.5195

RELIABILITY [R(t)] OF INDIVIDUAL COMPONENTS

Year	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
Component/Block												
Coupling	0.9999	0.9998	0.9997	0.9995	0.9994	0.9993	0.9992	0.9991	0.9990	0.9989	0.9987	0.9986
H-Pump	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Hose	0.9993	0.9985	0.9978	0.9970	0.9963	0.9955	0.9948	0.9941	0.9933	0.9926	0.9918	0.9911
H-Motor	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
B-Bearing	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Shaft	0.9999	0.9998	0.9997	0.9996	0.9995	0.9994	0.9993	0.9992	0.9990	0.9989	0.9988	0.9987
Brake	0.9988	0.9976	0.9964	0.9952	0.9940	0.9928	0.9916	0.9904	0.9892	0.9880	0.9868	0.9856
Reducer	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Spur Gear	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
R-Bearing	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
Increaser	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

HAZARD RATES [h(t)] OF INDIVIDUAL COMPONENTS

Year	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
Component/Block												
Coupling	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026	0.0026
H-Pump	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Hose	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172	0.0172
H-Motor	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
B-Bearing	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Shaft	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024
Brake	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280	0.0280
Reducer	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Spur Gear	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
R-Bearing	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Increaser	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

RELIABILITY OF SUBSYSTEM [$R_{subsys}(t)$]

Year	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
	0.9953	0.9907	0.9861	0.9815	0.9769	0.9723	0.9678	0.9633	0.9588	0.9543	0.9498	0.9454

HAZARD RATE OF SUBSYSTEM [$H_{subsys}(t)$]

Year	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
	0.1080	0.1080	0.1080	0.1080	0.1080	0.1081	0.1081	0.1081	0.1081	0.1081	0.1081	0.1081

Figure B-60. Reliability analysis for North Tower downstream lift gate machinery (planned replacement alternative 2009-2020)

B-50. Step 10. Determine the failure rate λ for each component. Failure rates can be selected from the table in the ETL or from RAC (NPRD-951995). This information is also available on the Automated Databook compact disc-read only memory (CD-ROM). NPRD-95 contains additional information on the failure rates that is helpful in selecting the components that most closely match the application of those used in lock mechanical systems. The failure rate data tabulated in these publications are listed as per million operating hours. The calculations in paragraphs B-51 through B-58 convert this to a per-year basis. Additional sources of failure rate data are being investigated and developed because the current sources are generally not a close match to the equipment used in lock mechanical systems. Alternate sources of failure rate data should be approved by Headquarters, USACE, prior to use.

B-51. Step 11. Calculate the adjusted failure rate λ' for each component.

$$\lambda' = \text{Adjusted failure rate} = \lambda * K_1 * K_2 * K_3$$

In the example spreadsheets in Figures B-59 and B-60:

$$\lambda' \text{ (for H-Pump)} = 7.7220 \times 0.5 \times 1.0 \times 1.0 = 3.861 \text{ (per million operating hours)}$$

B-52. Step 12. Calculate the mean time to failure (MTTF) for each component:

$$\text{MTTF} = 1000000 / \lambda' \quad \text{(hours)}$$

In the example spreadsheets in Figures B-59 and B-60:

$$\text{MTTF (for H-Pump)} = 1000000 / 3.861 = 259000 \text{ (hours)}$$

B-53. Step 13. Determine γ for each component.

$$\gamma = \alpha/\text{MTTF ratio from the ETL}$$

In the example spreadsheets in Figures B-59 and B-60:

$$\gamma = \alpha/\text{MTTF ratio (for H-Pump)} = 1.1$$

B-54. Step 14. Determine the characteristic life parameter α for each component:

$$\alpha = \gamma \times \text{MTTF}$$

In the example spreadsheets in Figures B-59 and B-60:

$$\alpha \text{ (for H-Pump)} = 1.1 \times 259000 / 8760 = 32.5229 \text{ (years)}$$

B-55. Step 15. Calculate the reliability of each component for each year of the study using the following equation:

$R(t) = \exp [-(t * d/\alpha)^{\beta}]$

In this example, the product of $t * d$ has already been calculated in the formulation of the duty factor outlined in Step 7. It is simply the actual equipment run-time. So the formula used reduces to

$R(t) = \exp [-(d/\alpha)^{\beta}]$

B-56. Step 16. Calculate the hazard rate of each component for each year of the study using the following equation:

$h(t) = (\beta/\alpha) * [(t * d)/\alpha]^{(\beta-1)}$

In this example, as in the reliability calculations, the product of $t * d$ has already been calculated in the formulation of the duty factor in Step 7. So the formula used reduces to

$h(t) = (\beta/\alpha) * [(d)/\alpha]^{(\beta-1)}$

B-57. Step 17. Calculate the subsystem reliability $R_S(t)$for each alternative by combining the subsystem component reliabilities using the following equation:

$R_S(t) = R_A(t)*R_B(t)*...*R_N(t)$

The lift gate mechanical subsystem was considered to begin at the coupling between the motor and the hydraulic pump. The subsystem reliability for the lift gate machinery model shown above at time t is determined from the individual reliability of each component as follows:

$$R_{SUBSYS}(t) = R_A(t)^8 * R_B(t) * R_C(t)^3 * R_D(t) * R_E(t)^4 * R_F(t)^3 * R_G(t) * R_H(t)^3 * R_I(t)^4 * R_J(t)^2 * R_K(t)$$

Where,

$R_A(t)$ = Reliability of the shaft coupling

$R_B(t)$ = Reliability of the hydraulic pump

$R_C(t)$ = Reliability of the hydraulic hose

$R_D(t)$ = Reliability of the hydraulic motor

$R_E(t)$ = Reliability of the ball bearing

$R_F(t)$ = Reliability of the shaft

$R_G(t)$ = Reliability of the brake

$R_H(t)$ = Reliability of the speed reducer

$R_I(t)$ = Reliability of the spur gear

$R_J(t)$ = Reliability of the roller bearing

$R_K(t)$ = Reliability of the speed increaser

Likewise for the tainter valves.

The tainter valve mechanical subsystem was also considered to begin at the coupling between the motor and hydraulic pump. The subsystem reliability for the tainter valve machinery model shown above is calculated as follows:

$R_{SUBSYS}(t) = R_A(t)^4 * R_B(t) * R_C(t)^8 * R_D(t)^2 * R_E(t) * R_F(t) * R_G(t)^2 * R_H(t)^2 * R_I(t)^3 * R_J(t)^3 * R_K(t)$

Where,

$R_A(t)$ = Reliability of the shaft coupling

$R_B(t)$ = Reliability of the hydraulic pump

$R_C(t)$ = Reliability of the hydraulic valve

$R_D(t)$ = Reliability of the hydraulic hose

$R_E(t)$ = Reliability of the hydraulic piping

$R_F(t)$ = Reliability of the hydraulic cylinder

$R_G(t)$ = Reliability of the shaft

$R_H(t)$ = Reliability of the bushing

$R_I(t)$ = Reliability of the sprocket

$R_J(t)$ = Reliability of the ball bearing

$R_K(t)$ = Reliability of the speed reducer

B-58. Step 18.

a. Calculate the subsystem hazard rate $h_{SUBSYS}(t)$ for each alternative by combining the subsystem component hazard rates using the following equation:

$h_{SUBSYS}(t) = \sum h_i(t)$

The subsystem hazard rate for the lift gate machinery model shown above at time *t* is determined from the individual hazard rate of each component as follows:

$h_{SUBSYS}(t) = h_A(t) * 8 + h_B(t) + h_C(t) * 3 + h_D(t) + h_E(t) * 4 + h_F(t) * 3 + h_G(t) + h_H(t) * 3 + h_I(t) * 4 + h_J(t) * 2 + h_K(t)$

Where,

$h_A(t)$ = Hazard rate of the shaft coupling

$h_B(t)$ = Hazard rate of the hydraulic pump

$h_C(t)$ = Hazard rate of the hydraulic hose

$h_D(t)$ = Hazard rate of the hydraulic motor

$h_E(t)$ = Hazard rate of the ball bearing

$h_F(t)$ = Hazard rate of the shaft

$h_G(t)$ = Hazard rate of the brake

$h_H(t)$ = Hazard rate of the speed reducer

$h_I(t)$ = Hazard rate of the spur gear

$h_J(t)$ = Hazard rate of the roller bearing

$h_K(t)$ = Hazard rate of the speed increaser

b. Likewise for the tainter valves.

The subsystem hazard rate for the tainter valve machinery model shown above is calculated as follows:

$$h_{SUBSYS}(t) = h_A(t) * 4 + h_B(t) + h_C(t) * 8 + h_D(t) * 2 + h_E(t) + h_F(t) + h_G(t) * 2 + h_H(t) * 2 + h_I(t) * 3 + h_J(t) * 3 + h_K(t)$$

Where,

$h_A(t)$ = Hazard rate of the shaft coupling

$h_B(t)$ = Hazard rate of the hydraulic pump

$h_C(t)$ = Hazard rate of the hydraulic valve

$h_D(t)$ = Hazard rate of the hydraulic hose

$h_E(t)$ = Hazard rate of the hydraulic piping

$h_F(t)$ = Hazard rate of the hydraulic cylinder

$h_G(t)$ = Hazard rate of the shaft

$h_H(t)$ = Hazard rate of the bushing

$h_I(t)$ = Hazard rate of the sprocket

$h_J(t)$ = Hazard rate of the ball bearing

$h_K(t)$ = Hazard rate of the speed reducer

c. The example spreadsheets in Figures B-59 and B-60 are portions of a couple of the Microsoft Excel workbooks used in the analysis for the Lower Monumental Lock mechanical equipment. The fix-as-fails and spare parts alternatives used the same spreadsheet analysis and the planned replacement alternative used a different spreadsheet analysis. The Microsoft Excel workbooks were set up so that for each subsystem, a separate workbook is used with different sheets in the workbook corresponding to the fix-as-fails and the planned replacement alternatives.

d. The first example spreadsheet (Figure B-59) shows the input data and calculation results for the reliability analysis on the North Tower downstream lift gate mechanical machinery subsystem for the years 1989 through 2000 for the fix-as-fails alternative. The spreadsheets for each subsystem include calculations for each year up to 2060.

e. The reliability and hazard rates were computed for the years 2009 through 2060 for the With-Project condition. The second example spreadsheet (Figure B-60) shows the input data and calculation results for the reliability analysis on the North Tower downstream lift gate mechanical machinery subsystem for the years 2009 through 2020 for the planned replacement alternative.

B-59. <u>Step 19</u>. Create the system RBDs for each alternative by combining the subsystem blocks into the system block diagrams. For Lower Monumental, the systems were the downstream lift gate mechanical machinery, upstream lift gate mechanical machinery, and tainter valve mechanical machinery systems.

> All the mechanical machinery for both of the lift gates must operate properly for satisfactory performance of the lock system. The downstream lift gate machinery is larger than the upstream lift gate machinery. As a result, the lift gate (LG) subsystem blocks (LG1, LG2, LG3, and LG4) were organized in two simple series systems. All four tainter valve (TV) machinery systems are essentially identical. At least one of two filling tainter valves and one of two emptying tainter valves must be operational at all times for satisfactory performance of the lock system. Therefore, the tainter valve machinery blocks (TV1, TV2, TV3, and TV4) were modeled as a complex series-parallel system.

LG1 = Downstream north lift gate machinery

LG2 = Downstream south lift gate machinery

LG3 = Upstream north side lift gate machinery

LG4 = Upstream south side lift gate machinery

TV1 = North emptying tainter valve machinery

TV2 = North filling tainter valve machinery

TV3 = South emptying tainter valve machinery

TV4 = South filling tainter valve machinery

The reliability block diagrams for the systems are shown in Figure B-61.

B-60. <u>Step 20</u>. Calculate the system hazard rates for each alternative.

a. Because these are calculated on a yearly basis, they are called the Annual Hazard Rates (AHR). This is done by combining the subsystem reliabilities and hazard rates as required to match the system configuration. Only the system hazard rates were calculated because these values are what are used in the event trees and in the economic analysis.

The AHR for the downstream lift gate mechanical machinery model was calculated as follows:

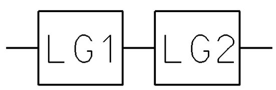
$h_{DLGSYS}(t) = h_{LG1}(t) + h_{LG2}(t)$

Where,

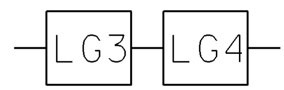
$h_{LG1}(t)$ = Hazard rate of the downstream north lift gate machinery

$h_{LG2}(t)$ = Hazard rate of the downstream south lift gate machinery

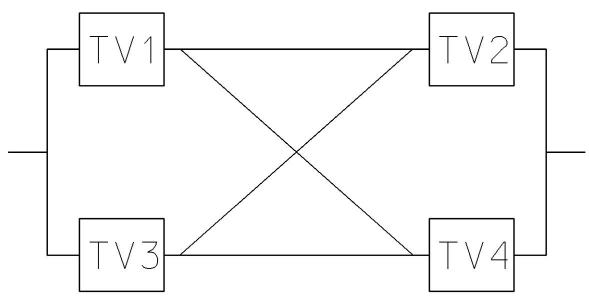
The AHR for the upstream lift gate mechanical machinery model was calculated as follows:



Downstream lift gate system



Upstream lift gate system



Tainter valve system

Figure B-61. Reliability block diagrams

$h_{ULGSYS}(t) = h_{LG3}(t) + h_{LG4}(t)$

Where,

$h_{LG3}(t)$ = Hazard rate of the upstream north side lift gate machinery

$h_{LG4}(t)$ = Hazard rate of the upstream south side lift gate machinery

The AHR for the tainter valve mechanical machinery model was calculated as follows:

$$h_{TVSYS}(t) = [4 * (h_{TV1}(t) + h_{TV2}(t)) - 2 * (h_{TV1}(t) + 2 * h_{TV2}(t)) * R_{TV2}(t) - 2 * (2 * h_{TV1}(t) + h_{TV2}(t)) * R_{TV1}(t) + (2 * h_{TV1}(t) + 2 * h_{TV2}(t)) * R_{TV1}(t) * R_{TV2}(t)] / [4 - 2 * R_{TV2}(t) - 2 * R_{TV1}(t) + R_{TV1}(t) * R_{TV2}(t)]$$

Where,

$R_{TV1}(t)$ = Reliability of the north emptying tainter valve machinery

$R_{TV2}(t)$ = Reliability of the north filling tainter valve machinery

$R_{TV3}(t)$ = Reliability of the south emptying tainter valve machinery

$R_{TV4}(t)$ = Reliability of the south filling tainter valve machinery

$h_{TV1}(t)$ = Hazard rate of the north emptying tainter valve machinery

$h_{TV2}(t)$ = Hazard rate of the north filling tainter valve machinery

$h_{TV3}(t)$ = Hazard rate of the south emptying tainter valve machinery

$h_{TV4}(t)$ = Hazard rate of the south filling tainter valve machinery

$R_{TV1}(t) = R_{TV3}(t)$

$R_{TV2}(t) = R_{TV4}(t)$

$h_{TV1}(t) = h_{TV3}(t)$

$h_{TV2}(t) = h_{TV4}(t)$

b. This computation can take three forms. The first form, which is a simplified version of the equation in a above, is usable if all the subsystem hazard rates are equal and if all the subsystem reliabilities are equal for any given year of the study. The form of the computation shown in a above is usable if the following conditions are met:

$R_{TV1}(t) = R_{TV3}(t)$

$R_{TV2}(t) = R_{TV4}(t)$

$h_{TV1}(t) = h_{TV3}(t)$

$h_{TV2}(t) = h_{TV4}(t)$

This is the case for the Lower Monumental Lock.

c. The most complex form of computation must be used if the subsystem reliabilities and hazard rates are all different for any given year of the study. The most complex form of the equation was used in the spreadsheet calculations in order to allow for "what-if" scenarios wherein various subsystem components or entire subsystems were replaced in any year of the study. The derivation of these formulas is included at the end of this example.

d. The fix-as-fails hazard rates for the lift gate and tainter valve mechanical systems for Lower Monumental lock are shown in Figure B-62. These hazard rates are used with the fix-as-fails and spare parts alternatives for original components.

e. The planned replacement hazard rates for the lift gate and tainter valve mechanical systems for Lower Monumental Lock are shown in Figure B-63. These hazard rates are used with the planned replacement alternative for all components.

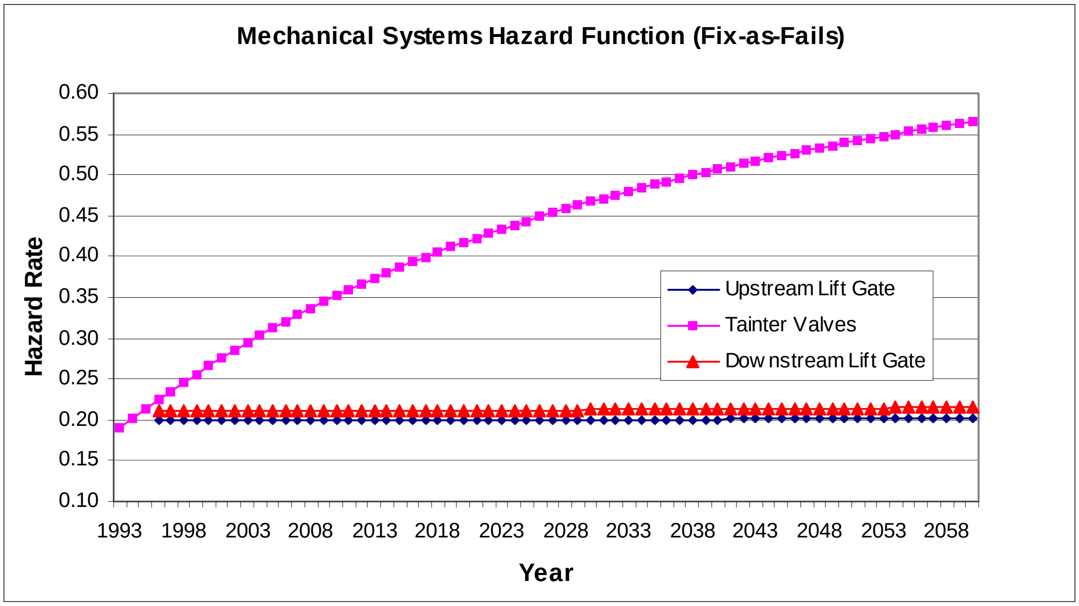


Figure B-62. Fix-as-fails hazard rates for mechanical machinery

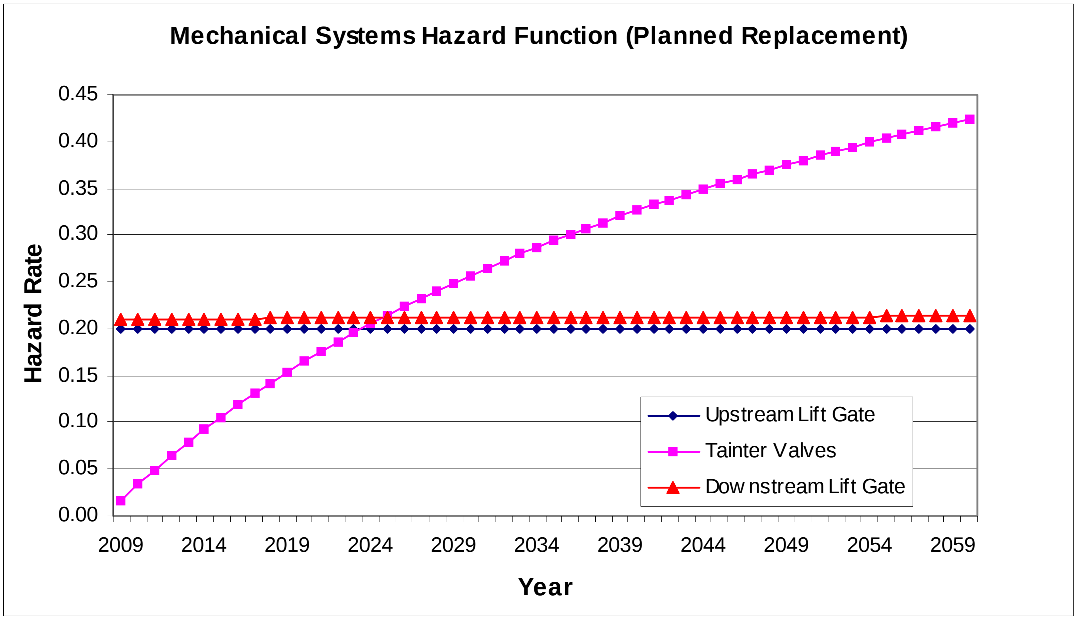


Figure B-63. Planned replacement hazard rates for mechanical machinery

f. These graphs show that the lift gate machinery is still in the bottom of the "bathtub" curve typical for a Weibull distribution with the hazard rate slowly increasing over time. Because the new lift gate mechanical equipment installed in the planned replacement alternative closely matches the existing arrangement and the hazard rate is increasing slowly, the hazard rate for the new mechanical equipment is about the same as for the existing lift gate machinery. The tainter valve machinery hazard rate increases over time at a greater rate with the hazard rate for the new mechanical equipment installed in the planned replacement starting out and ending the study at lower values than the fix-as-fails alternative.

Published failure rate data was used to develop the mechanical hazard functions, because there is a general lack of available historic lock and dam failure data. The U.S. Army Corps of Engineers (Corps) has only emphasized compiling and trending accurate maintenance data over the service life of a lock and dam in the last several years. The available records have not proven useful for trending purposes from a completely historical point of view. The Walla Walla District recognizes improvement in the area of lock and dam operation, maintenance cost, and failure data tracking and trending needs to occur. The Corps of Engineers Financial Management System (CEFMS) will provide the vehicle to improve this effort in the future. However, because CEFMS has only been in operation for a few years, not enough data is available to significantly improve cost tracking from a service life view of the lock.

All items with published failure rate data were included in the analysis. Some of the items are not represented in the reliability analysis because published failure rate data was not available for them. A qualitative judgment was made to determine which items were the most important; however, items left out of the reliability analysis can affect the overall AHR and reliability.

B-61. <u>Step 21</u>. Develop the event tree format for each of the systems for each alternative.

a. This will involve determining the failure repair levels for each event tree. In the example event trees included in paragraphs B-62 through B-67, the first branch lists the component or system the event tree is for. The second branch shows the paths for failure (AHR path) and nonfailure (1-AHR path). This represents that in any year of the analysis, there are two options, either one or more failures, or no failures. The third branch shows the repair levels. Three repair levels are shown following the AHR path. Because the equipment is aging and is anticipated to require more maintenance than new equipment, the branch following the 1-AHR path includes additional maintenance for the Without-Project alternatives. The fourth branch shows the conditional probabilities for the various repair levels. The fifth branch shows the repair costs associated with each failure branch. The sixth branch shows the number of closure days for each failure branch. The seventh branch shows the anticipated impact on the system reliability following each of the repair levels.

b. In these example event trees, there is an additional branch for the tainter valves. This is because lock operation is still possible, but at a slower speed, under some failure scenarios.

For the economic analysis, the overall mechanical system model was broken into three separate systems: downstream lift gate machinery, upstream lift gate machinery, and tainter valve machinery. This was necessary for the development of separate event trees for each system. Additionally, the repair histories, duty factors, and system replacement costs are different for each of the different systems. Therefore, each system was analyzed individually for the purposes of this study.

B-62. Step 22. Develop the repair levels for the event trees.

a. In the example event trees that follow, there are three failure levels.

(1) Because the existing equipment is aging and would be expected to incur annual maintenance costs above those of having new equipment, an additional annual maintenance cost for preventative maintenance to prevent an unscheduled lock outage is included in the fix-as-fails and spare parts event trees. This additional annual maintenance cost was estimated to be $2,500 for the downstream lift gate machinery, $2,000 for the upstream lift gate machinery, and $3,600 for the tainter valve machinery. These costs do not include the costs associated with the scheduled annual lock outage. It is anticipated that the scheduled annual lock outage will continue whether or not the planned replacement option is chosen. Because of the difficulty in quantifying the effect of repairs made during the scheduled annual lock outage on the mechanical system reliability and hazard rate, it was assumed that these repairs do not affect the reliability or hazard rate of the mechanical systems.

(2) Coincident failures of subsystems are not addressed in the mechanical machinery event trees. A sensitivity analysis was performed using the reliability spreadsheet model to determine the effect of increasing the conditional probabilities and repair costs in the high level repair branch of the mechanical event trees. In the sensitivity analysis for the tainter valve machinery, the repair costs and conditional probabilities were increased fourfold and the slow speed operating days were changed to lock closure days. In the sensitivity analysis for the lift gate machinery, the repair costs were increased twofold and the conditional probabilities were increased fourfold. The probability of coincident failures is small. With the small conditional probability associated with the high level repair branch, the sensitivity analysis showed that coincident failures do not change the outcome of the economic analysis or the recommendations of this study.

b. The probability diagram shown in Figure B-64 for the tainter valve system was not included in the Lower Monumental Lock and Dam Major Rehabilitation Evaluation Report, but the results of the diagram were used to assist in evaluating the chances of multiple coincident valve failures. The diagram is based on a target year of 2008 and uses the fix-as-fails reliability data for the valve subsystems for that year. The diagram was generated using the laws of probability. The analysis makes the following assumptions:

(1) The system cannot have more than three valves out of service, i.e. when the third valve fails, lock operation stops.

(2) When a sister valve fails, lock operation stops (sister valves are the two filling and two emptying valve pairs).

(3) Reliabilities are complete system reliabilities or "black box" reliabilities, any failure of the "black box" means it must be replaced in its entirety.

(4) Outage duration is 50 days, so additional failures must occur within 50 days of the first failure in order for the failures to be coincident.

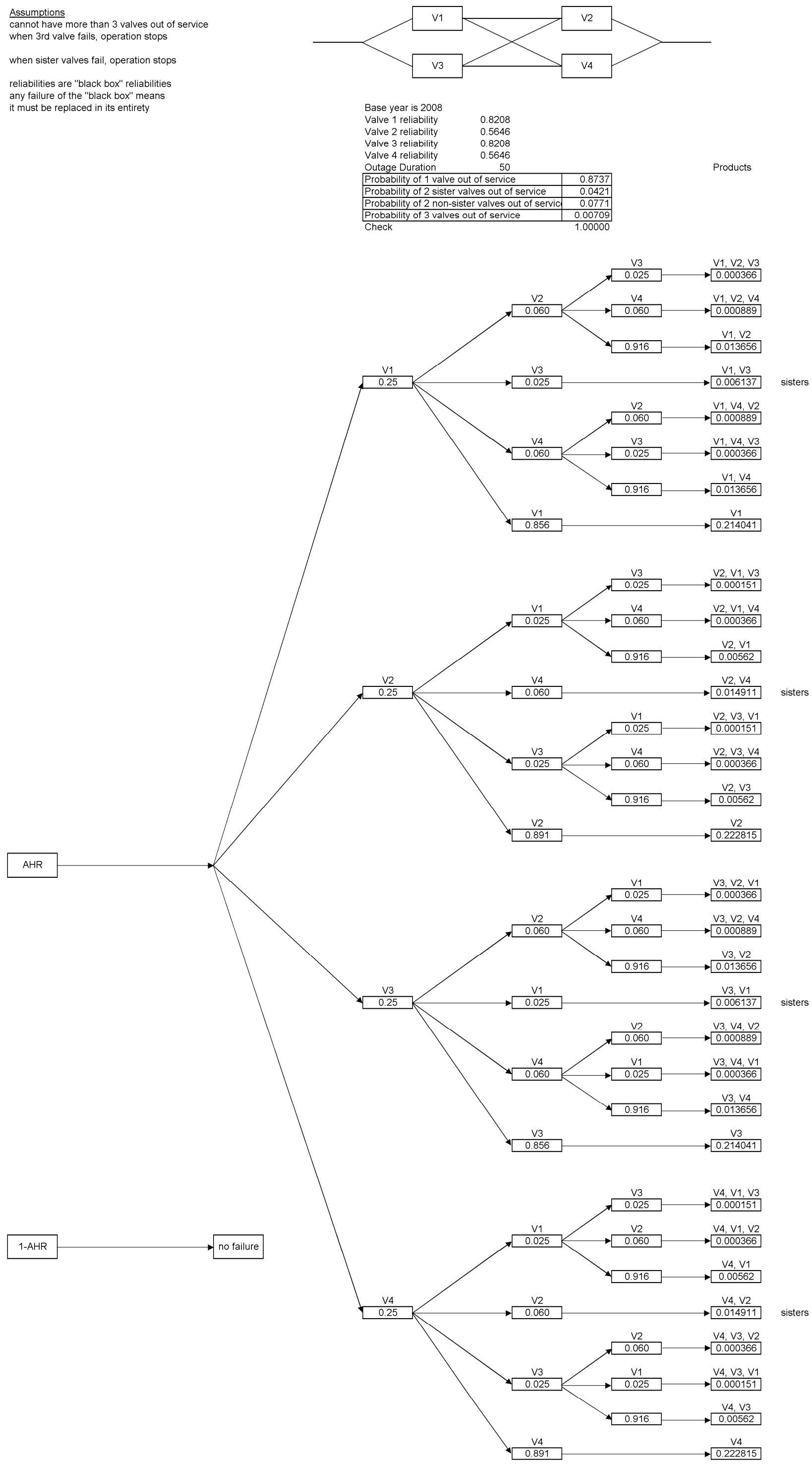


Figure B-64. Probability diagram for the tainter valve system

c. The results show that given there was a catastrophic failure, i.e., we went down the AHR path to the high level failure branch in the event tree, the probability of a single valve failing is about 87 percent, the probability of two sister valves failing is about 4 percent, the probability of two non-sister valves failing is about 8 percent, and the probability of three valves failing is less than 1 percent. Due to the size of the probability diagram, it is best printed on 11- by 17-in. paper.

B-63. Step 23. Determine the conditional probabilities for each repair level. These probabilities must sum to 1 or 100 percent. These can be determined by expert elicitation and by studying other rehabilitation reports covering similar equipment used in similar applications. In the Lower Monumental study, the conditional probabilities were determined mostly from studying other rehabilitation reports covering similar equipment used in similar applications. In general, the higher the level of failure, the lower the conditional probability of its occurrence.

> Conditional probabilities are used to approximate the physical consequences of the performance predicted by the hazard rates. Conditional probabilities were generally determined by historic data, likelihood of the consequence occurring, severity of consequence, and engineering judgment. The relative magnitude of the consequences is dependent on when the symptoms of a problem are noticed or when the component fails to operate. Historically, most repairs have been minor in nature so a conditional probability of from 0.70 to 0.85 was used for the low level failure. Most of the operating machinery is readily accessible for inspection, maintenance, and repair. Less accessible components (e.g., some parts of the tainter valve operating machinery) have not experienced failures. Very few failures of the moderate level have occurred, so a conditional probability of from 0.14 to 0.30 was used. A high level (catastrophic) failure, as defined in this study, has not been experienced by the project, but is possible. A conditional probability of from 0.002 to 0.0025 was used for high level failures. These conditional probabilities are consistent with those used in other major rehabilitation reports for similar systems.
>
> In a risk management approach, risk is defined as the product of the AHR and the conditional probability, which is the probability of a particular consequence given that the unsatisfactory performance has occurred. Hence, a reasonable estimate of the conditional probabilities is important for obtaining accurate results. The conditional probabilities, by their nature, are subjective. Historical experience assisted in the development of these probabilities, but it did not serve as the entire basis for subjective probability determination. Certain realistic events, which have not occurred, may occur at some time in the future. For instance, problems exhibited by the lift gate and tainter valve mechanical machinery have been minor in nature; however, there is a real probability, albeit small, of major or catastrophic consequences occurring. These potential consequences were considered during the development of conditional probabilities. A relatively larger subjective probability value is given to the minor consequences of an event, and a relatively smaller value is given to the catastrophic consequences. The inherent uncertainty of these conditional probabilities necessitates this conservative approach.

Very real consequences, which have not been addressed, are those attributable to unsafe conditions. The costs due to liability from death and injury can be severe, yet they have not been quantified in the reliability analysis.

B-64. Step 24. Determine the repair costs for the repair levels. For the Lower Monumental study, the high and moderate repair level costs were determined by Walla Walla District Cost Engineering Branch. The low-level failure costs were determined from project repair cost records in conjunction with assistance from Cost Engineering.

B-65. Step 25. Determine the length of closure for the repair levels. For the Lower Monumental Study, the high and moderate repair level closure lengths were determined by Cost Engineering based on procurement and repair activity durations. The low level failure closure lengths were determined from project repair records in conjunction with assistance from Cost Engineering.

Costs and closures associated with different levels for repair are provided in the event tree along with the effect on future reliability based upon the type of repair. Detail estimating methods were not employed because adequate design information is unknown and cannot be reasonably assumed. Historical cost data from previous similarly designed projects and past Government estimates were used for the development of costs. Parametrics were used to adjust pricing for similar equipment of differing physical size.

All cost estimate elements in the event trees were indexed to the effective price level date of October 1, 2002. Escalation is not included in this analysis. Cost estimates are for comparative purposes only and should only be used for ranking of the specific alternatives discussed in this report. Costs presented herein are not applicable for funding request or budgeting purposes.

Contingency is included in all estimates. Each assigned contingency identifies the uncertainty and cost risk associated with that item of work. Where normal design variances are expected, a 25 percent contingency value is used. Work items that appear to have a greater cost risk use a 30 percent contingency.

The costs presented for each event tree include construction costs, replacement parts, planning and engineering, engineering during construction, and construction management. Each event representing an unplanned replacement or repair includes a cost allowance for additional resources (e.g., emergency actions and overtime). These costs do not include the economic costs of having the lock out of service. Event trees were prepared for the fix-as-fails, spare parts, and planned replacement alternatives for both the lift gate and tainter valve operating machinery.

B-66. Step 26. Determine the effect on reliability for the various repair levels. For the Lower Monumental Study, these were determined by replacing various components or subsystems in the spreadsheet models to see the effect on the resulting hazard rates for the systems. This is depicted in the event trees by indicating how far back in history the various repairs affected the hazard rates.

The effect of new components on the overall system reliability and hazard rate following repairs was determined by simulation of various failure scenarios using the reliability analysis spreadsheet model. The effect of new components on the overall system reliability and hazard rates following repairs is portrayed as resetting the reliability and hazard rates to their value a specified number of years prior to the repair. In most instances, replacement of a single component, whether minor or major, had little or no effect on the overall system reliability and hazard rates following a repair. Because the hazard rates for the lift gate machinery change only slightly over the study period for each alternative, the economic model assigned the "current" state to the lift gate machinery following repairs in all levels. In most instances, the state following repairs of the tainter valve machinery does not change, so the economic model also assigned the "current" state to the tainter valve machinery for low and moderate level repairs. The economic model assigned the "new" state to the tainter valve machinery following high level repairs.

B-67. Step 27. Determine the cost and closure period for the With-Project alternatives. For the Lower Monumental study, these were determined by Cost Engineering based on procurement and repair activity costs and durations. This information is shown in the lower left corner of the example event trees that follow.

The event trees included in this example are representative of those contained in the Lower Monumental Lock and Dam Major Rehabilitation Evaluation Report. The event trees included in this example are for the downstream lift gate machinery Fix-as-Fails, Spare Parts, and Planned Replacement alternatives and for the tainter valve Fix-as-Fails alternative. The event tree for the tainter valves is similar to those for the lift gate system, except it shows the incorporation of the slow speed operation branch.

The fix-as-fails baseline event tree for the downstream lift gate machinery is shown in Figure B-65. There are three levels of repair assumed: one for high level (catastrophic) repairs; one for moderate repairs; and one for minor repairs. A breakdown of the costs and closures associated with the lift gate machinery fix-as-fails baseline event tree is as follows.

Lift Gate Machinery High-Level Failure, Unplanned New Lift Gate Machinery. This repair level assumes a catastrophic failure of one of the downstream lift gate machinery subsystems to the extent that it is not repairable. New machinery would be fabricated and installed for the failed lift gate machinery subsystem. Lock closure time was estimated to be 140 days with a repair cost of $2,529,000. This repair level is assumed to be the least likely of all the options. A 0.25 percent chance of occurrence was assigned to this repair level. Based on simulation of various failure scenarios using the reliability spreadsheet model, future reliability and hazard rates were reset back 13 years, or to the initial state, whichever occurs the latest.

Lift Gate Machinery Moderate-Level Failure, Replace Major Component. This repair level assumes a moderate failure of a downstream lift gate machinery subsystem;

requiring the replacement of a major component. Lock closure time was estimated to be 21 days at a repair cost of $48,000. This repair level is assumed to occur 29.75 percent of the time. Based on simulation of various failure scenarios using the reliability spreadsheet model, future reliability and hazard rates were reset back 4 years, or to the initial state, whichever occurs the latest.

Lift Gate Machinery Low-Level Failure, Replace Minor Component. The most likely repair level assumed is for the replacement of a minor component. A 70 percent chance of occurrence was assigned to this repair level. The repair cost is estimated to be $6,000 and a lock closure time of 7 days. Based on simulation of various failure scenarios using the reliability spreadsheet model, future reliability and hazard rates were reset back 2 years, or to the initial state, whichever occurs the latest.

The spare parts baseline event tree for the downstream lift gate machinery is shown in Figure B-66. There are three levels of repair assumed: one for high level (catastrophic) repairs; one for moderate repairs; and one for minor repairs. A breakdown of the costs

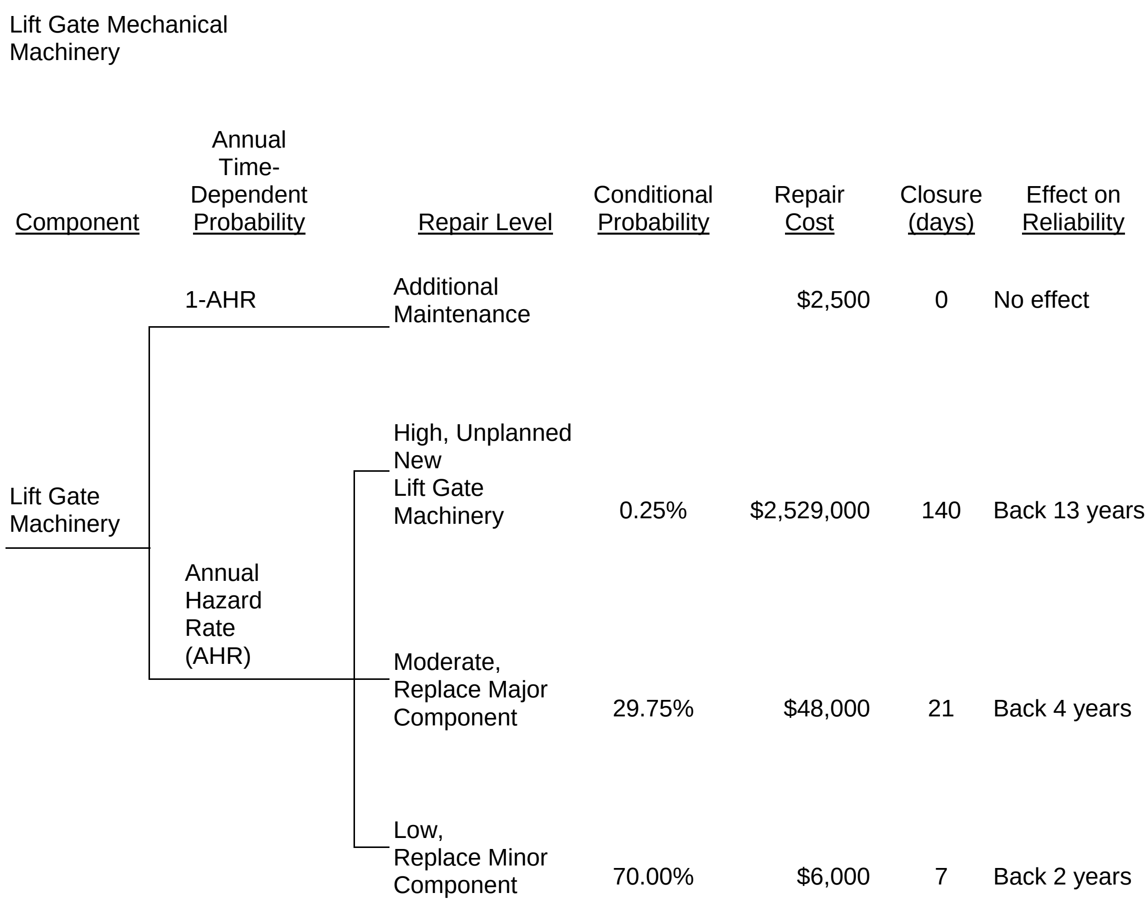


Figure B-65. Fix-as-Fails event tree for lift gate mechanical system

Lift Gate Mechanical
Machinery

Component	Annual Time-Dependent Probability	Repair Level	Conditional Probability	Repair Cost	Closure (days)	Effect on Reliability
Lift Gate Machinery	1-(AHR)	Additional Maintenance		$2,500	0	No effect
	Annual Hazard Rate (AHR)	High, Unplanned New Lift Gate Machinery	0.25%	$2,297,000	28	Back 13 years
		Moderate, Replace Major Component	29.75%	$48,000	4	Back 4 years
		Low, Replace Minor Component	70.00%	$6,000	0.2	Back 2 years

Spare parts for one lift gate machinery subsystem cost $1,167,000 at the beginning of the study period.

Figure B-66. Spare part event tree for lift gate mechanical machinery

and closures associated with the downstream lift gate machinery spare parts baseline event tree is provided below.

Lift Gate Machinery High-Level Failure, Replace Lift Gate Machinery. This repair level assumes a catastrophic failure of one of the downstream lift gate machinery subsystems to the extent that it is not repairable. Because there would be a complete set of replacement spare parts available, the lock closure duration would only be for 28 days and only the failed set of equipment would be replaced. The repair cost was estimated to be $2,297,000. This cost includes the replacement of the failed spare parts, so a complete set of spare parts for one machinery subsystem is again available following the repairs. This repair level is assumed to be the least likely of all the options. A 0.25 percent chance of occurrence was assigned to this repair level. Future reliability and hazard rates were reset back 13 years, or to the initial state, whichever occurs the latest.

Lift Gate Machinery Moderate-Level Failure, Replace Major Component. This repair level assumes a moderate failure of a downstream lift gate machinery subsystem; requiring the replacement of a major component. Because there would be a spare replacement part available, the lock closure time is estimated to be 4 days at a repair cost of $48,000. This cost includes the replacement of the failed spare part, so a complete set of spare parts for one machinery subsystem is again available following the repairs. This repair level is assumed to occur 29.75 percent of the time. Future reliability and hazard rates were reset back 4 years, or to the initial state, whichever occurs the latest.

Lift Gate Machinery Low-Level Failure, Replace Minor Component. The most likely repair level assumed is for the replacement of a minor component. A 70 percent chance of occurrence was assigned to this repair level. The repair cost is estimated to be $6,000 and a lock closure time of 0.2 day. This cost includes the replacement of the failed spare part, so a complete set of spare parts for one machinery subsystem is again available following the repair. Future reliability and hazard rates were reset back 2 years, or to the initial state, whichever occurs the latest.

Up-Front Procurement of Lift Gate Machinery Spare Parts. The estimated cost of one complete set of downstream lift gate operating machinery is $1,167,000. The cost of the spare parts is shown in the lower left of the event tree as illustrated in Figure B-66.

The planned replacement alternative assumes the lift gate machinery has not failed up to this point and the lock is operational when the machinery is replaced. The estimated cost of planned replacement is $4,158,000 and covers the replacement of both downstream lift gate mechanical machinery subsystems. The planned replacement cost is shown in the lower left of the event tree in Figure B-67. With all new downstream lift gate machinery subsystems, an updated reliability of 1.0 was assigned for the first year following the replacement, and the hazard rate was reset to the value for new equipment.

The planned replacement baseline event tree for the downstream lift gate machinery is shown below. Because there is still the risk of equipment failure even though it is new, there are also three levels of repair assumed for the planned replacement alternative: one for high level (catastrophic) repairs; one for moderate repairs; and one for minor repairs. A breakdown of the costs and closures associated with the downstream lift gate machinery planned replacement baseline event tree is provided below. Because the equipment would have been recently replaced, the conditional probabilities for the high and moderate level failures were reduced somewhat and the conditional probability for the low level failure was increased somewhat.

Lift Gate Mechanical Machinery

Component	Annual Time-Dependent Probability	Repair Level	Conditional Probability	Repair Cost	Closure (days)	Effect on Reliability
	1-(AHR)	None		$0	0	
Lift Gate Machinery	Annual Hazard Rate (AHR)	High, Unplanned New Lift Gate Machinery	0.2%	$2,529,000	140	Back 8 years
		Moderate, Replace Major Component	19.8%	$48,000	21	Back 1 year
		Low, Replace Minor Component	80.00%	$6,000	7	Back 1 year

Planned replacement of lift gate machinery subsystem will cost $4,158,000. Reliability is reset to 1.0.

Figure B-67. Planned replacement event tree for lift gate mechanical machinery

Lift Gate Machinery High-Level Failure, Replace Lift Gate Machinery. This repair level assumes a catastrophic failure to one of the downstream lift gate machinery subsystems to the extent that it is not repairable. Lock closure time was estimated to be 140 days with a repair cost of $2,529,000. This repair level is assumed to be the least likely of all the options. A 0.2 percent chance of occurrence was assigned to this repair level. Based on simulation of various failure scenarios using the reliability spreadsheet model, future reliability and hazard rates were reset back 8 years, or to the initial state, whichever occurs the latest.

Lift Gate Machinery Moderate-Level Failure, Replace Major Component. This repair level assumes a moderate failure of a downstream lift gate machinery subsystem; requiring the replacement of a major component. Lock closure time is estimated to be 21 days at a repair cost of $48,000. This repair level is assumed to occur 19.8 percent of the time. Based on simulation of various failure scenarios using the reliability spreadsheet model, future reliability and hazard rates were reset back 1 year.

Lift Gate Machinery Low-Level Failure, Replace Minor Component. The most likely repair level assumed is for the replacement of a minor component. An 80-percent chance of occurrence was assigned to this repair level. The repair cost is estimated at $6,000 and a lock closure time of 7 days. Based on simulation of various failure scenarios using the reliability spreadsheet model, future reliability and hazard rates were reset back 1 year.

Because there are two filling and two emptying tainter valves, the lock can be operated at reduced speed if the mechanical machinery for one valve fails. As recorded by project personnel, it takes 3 additional minutes per downstream lockage with an emptying tainter valve out of service and 8 additional minutes per upstream lockage with a filling tainter valve out of service. Again, there are three levels of repair assumed: one for high level (catastrophic) repairs; one for moderate repairs; and one for minor repairs. A breakdown of the costs and closures associated with the tainter valve machinery fix-as-fails baseline event tree is provided below and depicted in FigureB-68.

Tainter Valve Machinery High-Level Failure, Unplanned New Tainter Valve Machinery. Similar to the lift gate machinery high-level failure, this repair level assumes a catastrophic failure to one of the tainter valve machinery subsystems to the extent that it cannot be repaired. New machinery would be fabricated and installed for the failed tainter valve machinery subsystem. Lock reduced speed operating time is estimated to be 50 days with a repair cost of $394,000. Additionally, there would be a 7-day initial complete lock outage to assess damages and return the lock to reduced speed operation, so the total repair duration would be 57 days. This repair level is assumed to be the least likely of all the options. Only 0.25 percent chance of occurrence was assigned to this repair level. Based on simulation of various failure scenarios using the reliability spreadsheet model, future reliability and hazard rates were reset back 14 years, or to the initial state, whichever occurs the latest.

Tainter Valve Machinery Moderate-Level Failure, Replace Major Component. A 19.75 percent chance of occurrence is assigned to this repair level. The failure assumes a single tainter valve machinery subsystem needs a major component replaced. Reduced speed operation of the lock is estimated to be for 44 days. Again, there would be an additional 7-day initial complete lock outage to assess damages and return the lock to reduced speed operation, so the total repair duration would be 51 days. Overall repair cost is estimated to be $134,000. Future reliability and hazard rates are not improved under this scenario because only a single component would be replaced.

Tainter Valve Machinery Low-Level Failure, Replace Minor Component. This is considered to be the most likely repair scenario. An 80 percent chance of occurrence is assigned to this repair level. The failure assumes a single valve machinery subsystem needs a minor component replaced. Because there is already a spare hydraulic system available at the project, reduced speed operation of the lock is assumed to be for 1 day to exchange the failed component with the spare. There would not be a short, total outage period under this scenario. Overall repair cost is estimated to be $7,000 and includes the replacement cost of the spare component. Future reliability and hazard rates are not improved under this scenario.

Tainter Valve Mechanical Machinery

Component	Annual Time-Dependent Probability	Repair Level	Conditional Probabilities	Repair Cost	Closure (days)	Slow Speed (days)	Effect on Reliability
Valve Machinery	1-(AHR)	Additional Maintenance		$3,600	0	0	No effect
	Annual Hazard Rate (AHR)	High, Unplanned New Valve Machinery	0.25%	$394,000	7	50	Back 14 years
		Moderate, Replace Major Component	19.75%	$134,000	7	44	No effect
		Low, Replace Minor Component	80.00%	$7,000	0	1	No effect

Figure B-68. Fix-as-fails event tree for tainter valve machinery

B-68. Step 28. Furnish the event trees and system hazard rates to the economists working on the study so they can run the economic model to select the most economic course of action.

The hazard rates, event trees, and associated descriptions were given to the economists for use in the economic model to determine if replacing the components ahead of time or purchasing spare parts ahead of time were justified as opposed to a fix-as-fails approach. The reason for preparing event trees is so annual probable costs can be calculated. This is done in the economic model and can also be approximated by a spreadsheet that does the calculations using the hard numbers in the event trees and using the appropriate compound amount factors. The cost per day of lock outage or delay will need to be determined. This was done by our district economists. The annual probable costs can then be calculated by multiplying across the branches of the event tree and summing the products. These values are then combined with any other scheduled costs using the appropriate compound amount factors to determine the present value or average annual cost for each alternative. These costs can then be compared between alternatives to calculate benefit cost ratios and thus the most economic course of action. The spreadsheet calculations are a useful tool for screening which systems may warrant the full-blown economic modeling process and subsequent full-blown rehabilitation study and which ones will not. An example of one of the spreadsheets used to perform these calculations for the fix-as-fails alternative for the downstream lift gate at Lower Monumental dam is shown in Figure B-69.

The outline of the spreadsheet model structure used in preparing the analysis for the mechanical machinery for the Lower Monumental Lock and Dam Major Rehabilitation Evaluation Report is shown in Figure B-70.

Microsoft Excel workbook calculations are listed:

lmanlcpyaton.xls	lock cycles per year data
lmallgmnt.xls	lower lift gate machinery north tower subsystem
lmallgmst.xls	lower lift gate machinery south tower subsystem
lmaulgmns.xls	upper lift gate machinery north side subsystem
lmaulgmss.xls	upper lift gate machinery south side subsystem
lmaetv1m.xls	emptying tainter valve # 1 machinery subsystem
lmaetv2m.xls	emptying tainter valve # 2 machinery subsystem
lmaftv3m.xls	filling tainter valve # 3 machinery subsystem
lmaftv4m.xls	filling tainter valve # 4 machinery subsystem
lmamsras.xls	combines the subsystems into the systems
lmamshr.xls	summary of the annual hazard rates of the systems
lmallgea.xls	lower lift gate system economic analysis
lmaulgea.xls	upper lift gate system economic analysis
lmatvea.xls	tainter valve system economic analysis

The economic analysis spreadsheets are for preliminary purposes only. The economic results presented in the final study were from a full economic study model.

Lower Monumental Nav Lock Rehabilitation Report
Mechanical Systems Reliability Analysis
Lower Lift Gate Economic Analysis

Sensitivity	
Percentages	0
Repair Costs	1
Closure Days	0

Repair Levels	High	Moderate	Low
Percentages	0.0025	0.2975	0.7
Closure Days	140	21	7
Repair Cost	$2,529,000	$48,000	$6,000
Closure Cost/Day	$62,641	$62,641	$41,445
Delay Days	0	0	0
Delay Cost/Day	$0	$0	$0

Interest Rate 5.875%
Base Year 2009.5
Study Length 50 years

Note: This sheet has calculations for Fix-as-Fails scenario.
Note: Annual cost includes additional cost of maintaining old equipment of: $2,500.00

Year of Study	Calendar Year	AHR	Closure Days				Repair Cost				Delay Cost				Total Annual Cost	PV Factor	PV in Base Year
			High	Moderate	Low	Total	High	Moderate	Low	Total	High	Moderate	Low	Total			
-1.5	2008	0.2111	0.07	1.32	1.03	2.43	$1,335	$3,014	$887	$5,236	$4,628	$82,610	$42,868	$130,106	$137,314	1.0894	$149,590
-0.5	2009	0.2111	0.07	1.32	1.03	2.43	$1,335	$3,015	$887	$5,236	$4,629	$82,623	$42,875	$130,127	$137,336	1.0290	$141,312
0.5	2010	0.2112	0.07	1.32	1.03	2.43	$1,335	$3,015	$887	$5,237	$4,630	$82,637	$42,882	$130,149	$137,359	0.9719	$133,493
1.5	2011	0.2112	0.07	1.32	1.03	2.43	$1,335	$3,016	$887	$5,238	$4,630	$82,652	$42,890	$130,173	$137,383	0.9179	$126,108
2.5	2012	0.2112	0.07	1.32	1.04	2.43	$1,336	$3,016	$887	$5,239	$4,631	$82,668	$42,898	$130,198	$137,409	0.8670	$119,133
3.5	2013	0.2113	0.07	1.32	1.04	2.43	$1,336	$3,017	$887	$5,240	$4,632	$82,685	$42,907	$130,224	$137,436	0.8189	$112,544
4.5	2014	0.2113	0.07	1.32	1.04	2.43	$1,336	$3,018	$888	$5,241	$4,633	$82,702	$42,916	$130,251	$137,464	0.7734	$106,321
5.5	2015	0.2114	0.07	1.32	1.04	2.43	$1,336	$3,018	$888	$5,243	$4,634	$82,720	$42,925	$130,280	$137,494	0.7305	$100,443
6.5	2016	0.2114	0.07	1.32	1.04	2.43	$1,337	$3,019	$888	$5,244	$4,635	$82,739	$42,935	$130,310	$137,525	0.6900	$94,891
7.5	2017	0.2115	0.07	1.32	1.04	2.43	$1,337	$3,020	$888	$5,245	$4,636	$82,759	$42,945	$130,340	$137,557	0.6517	$89,646
8.5	2018	0.2115	0.07	1.32	1.04	2.43	$1,337	$3,021	$888	$5,246	$4,637	$82,779	$42,956	$130,372	$137,590	0.6155	$84,692
9.5	2019	0.2116	0.07	1.32	1.04	2.43	$1,338	$3,021	$889	$5,248	$4,639	$82,800	$42,967	$130,406	$137,624	0.5814	$80,013
10.5	2020	0.2116	0.07	1.32	1.04	2.43	$1,338	$3,022	$889	$5,249	$4,640	$82,822	$42,978	$130,440	$137,660	0.5491	$75,592
11.5	2021	0.2117	0.07	1.32	1.04	2.43	$1,338	$3,023	$889	$5,250	$4,641	$82,845	$42,990	$130,476	$137,697	0.5187	$71,417
12.5	2022	0.2117	0.07	1.32	1.04	2.43	$1,339	$3,024	$889	$5,252	$4,642	$82,868	$43,002	$130,513	$137,735	0.4899	$67,473
13.5	2023	0.2118	0.07	1.32	1.04	2.44	$1,339	$3,025	$890	$5,253	$4,644	$82,892	$43,015	$130,551	$137,775	0.4627	$63,747
14.5	2024	0.2119	0.07	1.32	1.04	2.44	$1,340	$3,026	$890	$5,255	$4,645	$82,917	$43,028	$130,590	$137,815	0.4370	$60,227
15.5	2025	0.2119	0.07	1.32	1.04	2.44	$1,340	$3,027	$890	$5,257	$4,647	$82,943	$43,041	$130,631	$137,857	0.4128	$56,903
16.5	2026	0.2120	0.07	1.32	1.04	2.44	$1,340	$3,027	$890	$5,258	$4,648	$82,969	$43,055	$130,672	$137,901	0.3899	$53,762
17.5	2027	0.2121	0.07	1.32	1.04	2.44	$1,341	$3,028	$891	$5,260	$4,650	$82,997	$43,069	$130,715	$137,945	0.3682	$50,795
18.5	2028	0.2121	0.07	1.33	1.04	2.44	$1,341	$3,029	$891	$5,262	$4,651	$83,025	$43,083	$130,759	$137,991	0.3478	$47,992
19.5	2029	0.2122	0.07	1.33	1.04	2.44	$1,342	$3,031	$891	$5,264	$4,653	$83,053	$43,098	$130,804	$138,038	0.3285	$45,345
20.5	2030	0.2123	0.07	1.33	1.04	2.44	$1,342	$3,032	$892	$5,266	$4,654	$83,083	$43,114	$130,851	$138,086	0.3103	$42,843
21.5	2031	0.2124	0.07	1.33	1.04	2.44	$1,343	$3,033	$892	$5,267	$4,656	$83,113	$43,129	$130,899	$138,135	0.2930	$40,480
22.5	2032	0.2125	0.07	1.33	1.04	2.44	$1,343	$3,034	$892	$5,269	$4,658	$83,144	$43,145	$130,947	$138,186	0.2768	$38,248
23.5	2033	0.2125	0.07	1.33	1.04	2.44	$1,344	$3,035	$893	$5,271	$4,660	$83,176	$43,162	$130,998	$138,238	0.2614	$36,139
24.5	2034	0.2126	0.07	1.33	1.04	2.44	$1,344	$3,036	$893	$5,273	$4,662	$83,208	$43,179	$131,049	$138,291	0.2469	$34,147
25.5	2035	0.2127	0.07	1.33	1.04	2.45	$1,345	$3,037	$893	$5,276	$4,663	$83,242	$43,196	$131,101	$138,345	0.2332	$32,265
26.5	2036	0.2128	0.07	1.33	1.04	2.45	$1,345	$3,039	$894	$5,278	$4,665	$83,276	$43,214	$131,155	$138,401	0.2203	$30,487
27.5	2037	0.2129	0.07	1.33	1.04	2.45	$1,346	$3,040	$894	$5,280	$4,667	$83,311	$43,232	$131,210	$138,458	0.2081	$28,807
28.5	2038	0.2130	0.07	1.33	1.04	2.45	$1,347	$3,041	$894	$5,282	$4,669	$83,346	$43,250	$131,266	$138,516	0.1965	$27,220
29.5	2039	0.2131	0.07	1.33	1.04	2.45	$1,347	$3,043	$895	$5,285	$4,671	$83,383	$43,269	$131,323	$138,575	0.1856	$25,720
30.5	2040	0.2132	0.07	1.33	1.04	2.45	$1,348	$3,044	$895	$5,287	$4,673	$83,420	$43,288	$131,382	$138,636	0.1753	$24,304
31.5	2041	0.2133	0.07	1.33	1.04	2.45	$1,348	$3,045	$896	$5,289	$4,675	$83,458	$43,308	$131,441	$138,697	0.1656	$22,965
32.5	2042	0.2134	0.07	1.33	1.05	2.45	$1,349	$3,047	$896	$5,292	$4,678	$83,496	$43,328	$131,502	$138,760	0.1564	$21,701
33.5	2043	0.2135	0.07	1.33	1.05	2.45	$1,350	$3,048	$897	$5,294	$4,680	$83,536	$43,349	$131,564	$138,825	0.1477	$20,506
34.5	2044	0.2136	0.07	1.33	1.05	2.46	$1,350	$3,050	$897	$5,297	$4,682	$83,576	$43,369	$131,627	$138,890	0.1395	$19,378
35.5	2045	0.2137	0.07	1.33	1.05	2.46	$1,351	$3,051	$897	$5,299	$4,684	$83,617	$43,391	$131,692	$138,957	0.1318	$18,311
36.5	2046	0.2138	0.07	1.34	1.05	2.46	$1,352	$3,053	$898	$5,302	$4,687	$83,658	$43,412	$131,758	$139,025	0.1245	$17,303
37.5	2047	0.2139	0.07	1.34	1.05	2.46	$1,352	$3,054	$898	$5,305	$4,689	$83,701	$43,434	$131,824	$139,094	0.1176	$16,351
38.5	2048	0.2140	0.07	1.34	1.05	2.46	$1,353	$3,056	$899	$5,307	$4,692	$83,744	$43,457	$131,893	$139,165	0.1110	$15,452
39.5	2049	0.2141	0.07	1.34	1.05	2.46	$1,354	$3,057	$899	$5,310	$4,694	$83,788	$43,480	$131,962	$139,237	0.1049	$14,602
40.5	2050	0.2142	0.07	1.34	1.05	2.46	$1,354	$3,059	$900	$5,313	$4,697	$83,833	$43,503	$132,032	$139,310	0.0991	$13,799
41.5	2051	0.2143	0.08	1.34	1.05	2.46	$1,355	$3,061	$900	$5,316	$4,699	$83,878	$43,526	$132,104	$139,384	0.0936	$13,040
42.5	2052	0.2144	0.08	1.34	1.05	2.47	$1,356	$3,062	$901	$5,319	$4,702	$83,925	$43,550	$132,177	$139,460	0.0884	$12,323
43.5	2053	0.2146	0.08	1.34	1.05	2.47	$1,357	$3,064	$901	$5,322	$4,704	$83,972	$43,575	$132,251	$139,536	0.0835	$11,646
44.5	2054	0.2147	0.08	1.34	1.05	2.47	$1,357	$3,066	$902	$5,325	$4,707	$84,019	$43,600	$132,326	$139,614	0.0788	$11,006
45.5	2055	0.2148	0.08	1.34	1.05	2.47	$1,358	$3,068	$902	$5,328	$4,710	$84,068	$43,625	$132,403	$139,694	0.0745	$10,401
46.5	2056	0.2149	0.08	1.34	1.05	2.47	$1,359	$3,069	$903	$5,331	$4,712	$84,117	$43,650	$132,480	$139,774	0.0703	$9,830
47.5	2057	0.2151	0.08	1.34	1.05	2.47	$1,360	$3,071	$903	$5,334	$4,715	$84,167	$43,676	$132,559	$139,856	0.0664	$9,290
48.5	2058	0.2152	0.08	1.34	1.05	2.47	$1,361	$3,073	$904	$5,337	$4,718	$84,218	$43,703	$132,639	$139,939	0.0627	$8,779
49.5	2059	0.2153	0.08	1.35	1.06	2.48	$1,361	$3,075	$904	$5,341	$4,721	$84,270	$43,730	$132,720	$140,023	0.0593	$8,297

PV of Without Project conditions at Base Year $2,567,082
Fix-as-Fails AAE COST USING PV @ Base Year @ 2003 price level $160,032

lmallgea.xls   sheet FAF

Figure B-69. Example economic screening calculations for fix-as-fails alternative

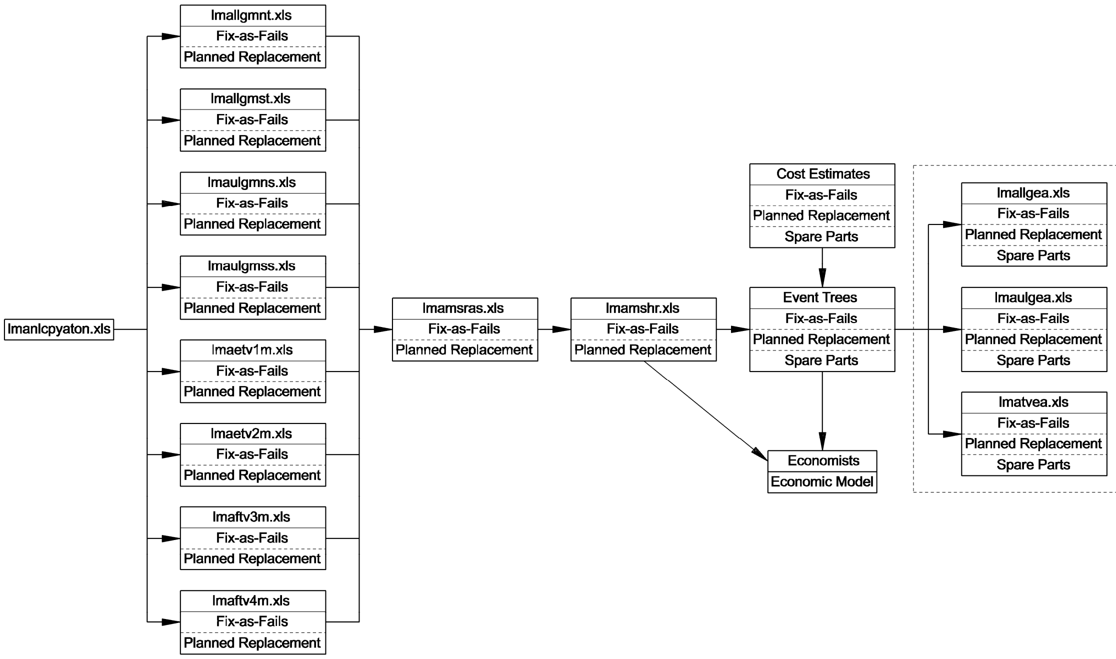
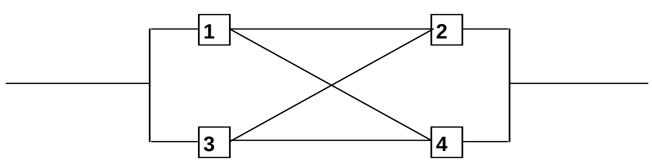


Figure B-70. Spreadsheet model structure for
Lower Monumental rehabilitation study

<u>HAZARD MATH</u> The following derivative results in the hazard rate for a lock with four valves arranged in the following manner.



$$h(t) = \frac{1}{R(t)}\left[\frac{-dR(t)}{dt}\right]$$

$$\frac{-d}{dt}R_t = h(t)R(t)$$

$$R_t(t) = [1-(1-R_2)(1-R_4)] * R_1 + [R_3 * [1-(1-R_2)(1-R_4)]] * (1-R_1)$$

$$=\left[1-(1-R_4-R_2+R_2R_4)\right]R_1+\left[R_3(1-1+R_4+R_2-R_2R_4)\right](1-R_1)$$
$$=\left[1-1+R_4+R_2-R_2R_4\right]R_1+\left[R_3R_4+R_2R_3-R_2R_3R_4)\right](1-R_1)$$
$$=\left[R_1R_4+R_1R_2-R_1R_2R_4\right]+\left[R_3R_4-R_1R_3R_4+R_2R_3-R_1R_2R_3-R_2R_3R_4+R_1R_2R_3R_4\right]$$
$$=R_1R_4+R_1R_2-R_1R_2R_4+R_3R_4-R_1R_3R_4+R_2R_3-R_1R_2R_3-R_2R_3R_4+R_1R_2R_3R_4$$
$$=\frac{-d}{dt}\left[R_1R_4+R_1R_2-R_1R_2R_4+R_3R_4-R_1R_3R_4+R_2R_3-R_1R_2R_3-R_2R_3R_4+R_1R_2R_3R_4\right]$$
$$=\frac{-d}{dt}(R_1R_4)+\frac{-d}{dt}(R_1R_2)-\frac{-d}{dt}(R_1R_2R_4)+\frac{-d}{dt}(R_3R_4)-\frac{-d}{dt}(R_1R_3R_4)+\frac{-d}{dt}(R_2R_3)-$$
$$\frac{-d}{dt}(R_1R_2R_3)-\frac{-d}{dt}(R_2R_3R_4)+\frac{-d}{dt}(R_1R_2R_3R_4)$$
$$=A+B-C+D-E+F-G-H+I$$

$$A=[(\frac{-d}{dt}R_1)R_4+(\frac{-d}{dt}R_4)R_1]=(h_1R_1)R_4+(h_4R_4)R_1$$
$$B=[(\frac{-d}{dt}R_1)R_2+(\frac{-d}{dt}R_2)R_1]=(h_1R_1)R_2+(h_2R_2)R_1$$
$$C=[(\frac{-d}{dt}R_1)R_2R_4+(\frac{-d}{dt}R_2)R_1R_4+(\frac{-d}{dt}R_4)R_1R_2]=[(h_1R_1)R_2R_4+(h_2R_2)R_1R_4+(h_4R_4)R_1R_2]$$
$$D=[(\frac{-d}{dt}R_3)R_4+(\frac{-d}{dt}R_4)R_3]=(h_3R_3)R_4+(h_4R_4)R_3$$
$$E=[(\frac{-d}{dt}R_1)R_3R_4+(\frac{-d}{dt}R_3)R_1R_4+(\frac{-d}{dt}R_4)R_1R_3]=[(h_1R_1)R_3R_4+(h_3R_3)R_1R_4+(h_4R_4)R_1R_3]$$
$$F=[(\frac{-d}{dt}R_2)R_3+(\frac{-d}{dt}R_3)R_2]=(h_2R_2)R_3+(h_3R_3)R_2$$
$$G=[(\frac{-d}{dt}R_1)R_2R_3+(\frac{-d}{dt}R_2)R_1R_3+(\frac{-d}{dt}R_3)R_1R_2]=[(h_1R_1)R_2R_3+(h_2R_2)R_1R_3+(h_3R_3)R_1R_2]$$
$$H=[(\frac{-d}{dt}R_2)R_3R_4+(\frac{-d}{dt}R_3)R_2R_4+(\frac{-d}{dt}R_4)R_2R_3]=[(h_2R_2)R_3R_4+(h_3R_3)R_2R_4+(h_4R_4)R_2R_3]$$
$$I=[(\frac{-d}{dt}R_1)R_2R_3R_4+(\frac{-d}{dt}R_2)R_1R_3R_4+(\frac{-d}{dt}R_3)R_1R_2R_4+(\frac{-d}{dt}R_4)R_1R_2R_3]$$
$$=[(h_1R_1)R_2R_3R_4+(h_2R_2)R_1R_3R_4+(h_3R_3)R_1R_2R_4+(h_4R_4)R_1R_2R_3]$$

$$\frac{-d}{dt}R_t(t)=[(h_1R_1)R_4+(h_4R_4)R_1]+[(h_1R_1)R_2+(h_2R_2)R_1]-[h_1R_1R_2R_4+h_2R_2R_1R_4+h_4R_4R_1R_2]+$$
$$[(h_3R_3)R_4+(h_4R_4)R_3]-[(h_1R_1)R_3R_4+(h_3R_1)R_3R_4+(h_4R_1)R_3R_4]+[(h_2R_2)R_3+(h_3R_3)R_2]-$$
$$[(h_1R_1)R_2R_3+(h_2R_1)R_2R_3+(h_3R_1)R_2R_3]-[(h_2R_2)R_3R_4+(h_3R_2)R_3R_4+(h_4R_2)R_3R_4]+$$
$$[(h_1R_1)R_2R_3R_4+(h_2R_1)R_2R_3R_4+(h_3R_1)R_2R_3R_4+(h_4R_1)R_2R_3R_4]$$
$$=(h_1+h_4)R_1R_4+(h_1+h_2)R_1R_2-(h_1+h_2+h_4)R_1R_2R_4+(h_3+h_4)R_3R_4-(h_1+h_3+h_4)R_1R_3R_4+$$
$$(h_2+h_3)R_2R_3-(h_1+h_2+h_3)R_1R_2R_3-(h_2+h_3+h_4)R_2R_3R_4+(h_1+h_2+h_3+h_4)R_1R_2R_3R_4$$

If the hazard rates and reliability values are considered equal, the equation above can be reduced to:

$$= 2hR^2 + 2hR^2 - 3hR^3 + 2hR^2 - 3hR^3 + 2hR^2 - 3hR^3 - 3hR^3 + 4hR^4$$
$$= 8hR^2 - 12hR^3 + 4hR^4$$

Also:

$$R_t(t) = R^2 + R^2 - R^3 + R^2 - R^3 + R^2 - R^3 - R^3 + R^4$$

$$= 4R^2 - 4R^3 + R^4$$

Therefore:

$$h(t) = \frac{8hR^2 - 12hR^3 + 4hR^4}{4R^2 - 4R^3 + R^4}$$
$$h(t) = \left(\frac{8h - 12hR + 4hR^2}{4 - 4R + R^2}\right)\frac{R^2}{R^2}$$
$$h(t) = \frac{8h - 12hR + 4hR^2}{4 - 4R + R^2}$$

If the hazard rates and reliability values are not all equal, then some form of the more complex equations will have to be used.

This completes this example.

List of Tables

List of Figures

APPENDIX C
Flood-Control Reliability

Section I
Example 1
Issue: Slope Stability and Non-Time Dependent Seepage Through an Earthen Embankment
Project: Hodges Village Dam, New England District

C-1. Introduction.

a. The Hodges Village Dam is a part of an existing USACE New England District flood-control project located on the French River in Oxford, Massachusetts. The dam is an embankment dam constructed in 1959 on a foundation of glacial outwash consisting of deposits of very pervious gravel with numerous open-work gravel and cobble strata interconnected throughout the foundation.

b. The project has experienced severe seepage and internal erosion problems on three occasions at only moderate flood events (flood frequency of 40-year interval and less) during 1968, 1987, and 1993. The problems required costly emergency and remedial repairs to maintain the dam's integrity. In view of these seepage and erosion problems, it became necessary to provide a permanent solution to the recurring seepage and internal erosion problems by providing a concrete cut-off wall through the central portion of the dam.

c. A reliability analysis of the main dam embankment and reservoir dikes considering seepage, piping, and stability as failure modes was performed in 1995. Seepage, piping, and stability conditions of the dam were studied extensively, incorporating the effects of remedial repairs carried out in the past to evaluate the existing condition of the dam and develop remedial measures to correct the problem. These conditions were also studied with a concrete cut-off wall (selected remedial measure) through the center of the dam. The results of the analysis indicated a low reliability level for the existing project, indicating a hazardous condition. With the concrete cut-off wall in-place, the reliability of the project increased to an acceptable level.

C-2. Background.

a. The Hodges Village Dam project is a single-purpose flood-control dam project constructed and operated by USACE. The project is normally operated as a dry-bed reservoir, impounding water only during flood events. The dam is 2140 ft long and has a maximum height of 54.5 ft above the river channel. The layout of the dam and typical sections of the dam embankment are shown in Figure C-1. The foundation deposits beneath the dam embankment consist mainly of glacial outwash materials. To the right of the old river channel the foundation materials consist mainly of sandy gravel with traces of silt and numerous boulders overlying bedrock. To the left of the old river channel, a deep preglacial valley is present below the dam embankment and left abutment. These foundation materials consist of clean stratified gravelly sands and sandy gravels with numerous highly pervious open-work gravel strata overlying bedrock. The embankment of the main dam consists of homogeneous pervious fill with a

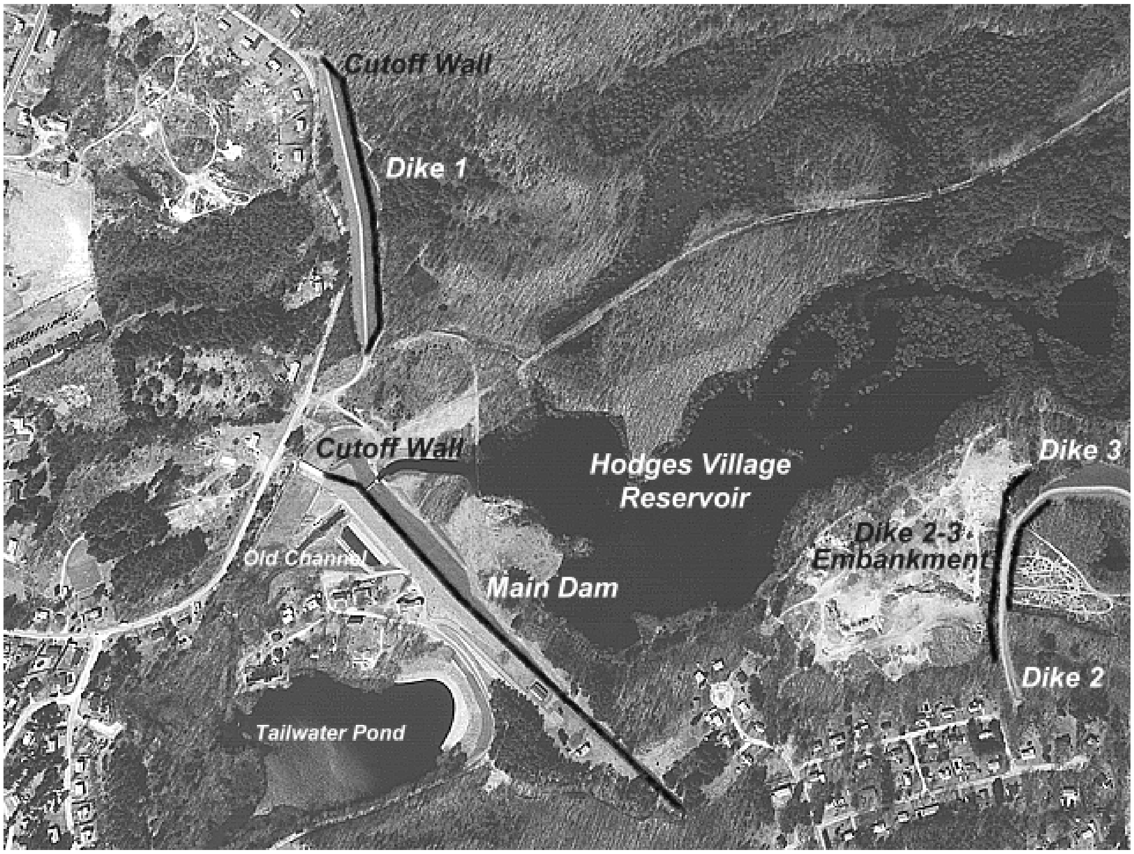


Figure C-1. Layout of Hodges Village Dam and typical sections of the dam embankment

downstream rockfill shell. The permeability parameters for the foundation and the embankment materials are shown in Table C-1.

b. Although the dam was founded on a highly pervious foundation, no provisions were made for a impervious foundation cut-off to prevent underseepage or for an impervious core through the embankment to control seepage. Because of the normal dry-bed reservoir condition and the anticipated short duration of a flood pool, seepage control measures were assumed not to be necessary during the initial project design in 1956.

c. Since the Hodges Village Dam designers did not incorporate seepage control measures in the dam's highly pervious foundation, seepage and internal erosion had become a recurring problem even during the flood events of a frequency as low as 10 years. High exit gradients during flood events caused sand boils, internal erosion and loss of foundation materials caused by piping, and erosion of the access road caused by excessive seepage.

Table C-1 - Seepage Model Permeability Parameters, ft./min

Materials	K_h	K_v	K_h/K_v	K_{eff}
Foundation				
Gravels, GP	0.35	0.00035	1000	0.01
Sands, SP	0.07	0.00007	1000	0.002
Gravels, GP-GM w/Boulders	0.25	0.002	150	0.02
Embankment				
Drains	0.1	0.1	1	0.1
Rockfill	1.0	1.0	1	1.0
Embankment	0.02	0.005	4	0.01
Concrete	0.00001	0.00001	1	0.0001

Note: K_h = horizontal permeability
K_v = vertical permeability
K_{eff} = effective permeability

d. First evidence of seepage and piping at the dam site was observed during a 1968 flood event (20-year flood) when the reservoir pool reached a level 12.5 ft below the spillway crest. Seepage emerged from the slopes of the left abutment embankment, causing considerable damage to the access road. The second flood event exceeding the 10-year frequency occurred in 1987 with the reservoir pool reached a level 8.1 ft below the spillway crest. Seepage emerged in several areas at the toe of the dam, on the slopes of the left abutment, and through pavement cracks in the access road. Several sand boils developed along a 600-ft reach around the perimeter of the tailwater pond (Figure C- 1) downstream of the left abutment, and two small sinkholes developed on the surface of the access road. Remedial work completed included construction of a gravel berm with 12 relief wells along the edge of the tailwater pond and a deep foundation drain beneath the access road on the slope of the left abutment.

e. The worst situation developed during a 1993 flood event, a 10-year flood event. The reservoir pool reached a level 13.2 ft below the spillway crest. Outlets of the foundation drains installed after the 1987 flood event were running full with muddy water, and a large delta of sand was formed at the mouth of the one of the outlet pipes in the tailwater pond. A depression measuring about 12 ft in diameter appeared on the surface of the access road above the foundation drains. In a few hours, the depression developed into a large sinkhole about 50 ft long threatening the integrity of the entire left abutment. Emergency action was called in, and steel sheet piles were driven surrounding the sinkhole to cut off the seepage water and to provide stability to the walls of the sinkhole. This action prevented a complete failure of the dam. A complete history of the seepage and piping problems at the Hodges Village Dam has been provided in "Major Rehabilitation Report, Hodges Village Dam" completed in June 1995, by the New England Division of the Corps of Engineers.

f. In view of the recurring problems that developed during the moderate flood events, USACE was very concerned about the ability of the dam to safely impound the floods for which the project was designed and provide protection to the downstream communities. An extensive study to evaluate the reliability of the dam in its existing condition and with a new concrete cut-off wall through the center of the dam was completed.

C-3. <u>Seepage Analysis of Existing Condition</u>.

a. The first goal of the seepage analysis was to reproduce observed behavior of the dam and predict future conditions at various loading conditions the dam is expected to carry. The second goal was to perform the seepage analysis by varying the materials properties within reasonable limits and performing reliability analysis. The material properties used in the analysis were determined by field observations and laboratory tests, and reasonableness of the model calibration limits compared to actual experiences. Two cross-sections, one representing left abutment and the other representing the main dam across the old river channel, were selected. The locations of these cross-sections were chosen where seepage problems during the previous flood events were the most severe. A two-dimensional finite element model was built for each cross-section. The final finite element model for the left abutment is shown in Figure C-2**.** The accuracy of the models was verified by comparing the model outputs with the observed seepage during the past flood events and actual instrumentation data from foundation piezometers. Some adjustments were made in the permeability parameters obtained from the laboratory analysis to account for the presence of the openwork gravel strata. The final models incorporated all the toe drains, deep foundation drains, and the relief wells installed as remedial measures after the past flood events. The models were run for six reservoir pool elevations: 1987 flood elevation (el) 493.8, 1993 flood el 486.8, spillway crest el 501.0, el 505.0 and 510.0, and surcharge pool el 515.0.

b. The seepage flow vectors and pressure heads through the dam and the foundation for the left abutment are shown is shown in Figure C-3, the exit gradients and total seepage flow are shown in Table C-2.

c. Table C-2 clearly shows that for pre-1990 conditions exit gradients higher than the critical gradient (0.825) will develop leading to a development of piping conditions in the dam at reservoir pool elevations as low as 495.0 (1.2 ft above the 1987 flood event). This corroborates the development of sand boils in 1987 flood event. The large flow vectors in Figure C-3 strongly support the possibility of seepage breakouts on the downstream side of the dam at a pool el of 488.6 (1968 flood event).

C-4. <u>Stability Analysis</u>.

a. Detailed slope stability analysis of the natural slope above the access road along the left abutment was performed to determine the extent of adverse slope stability conditions that will develop in the event of high reservoir pool elevations.

Typical Seepage Model - FE Grid

FastSeep Analysis - Current Conditions

(incl. proposed cutoff wall)

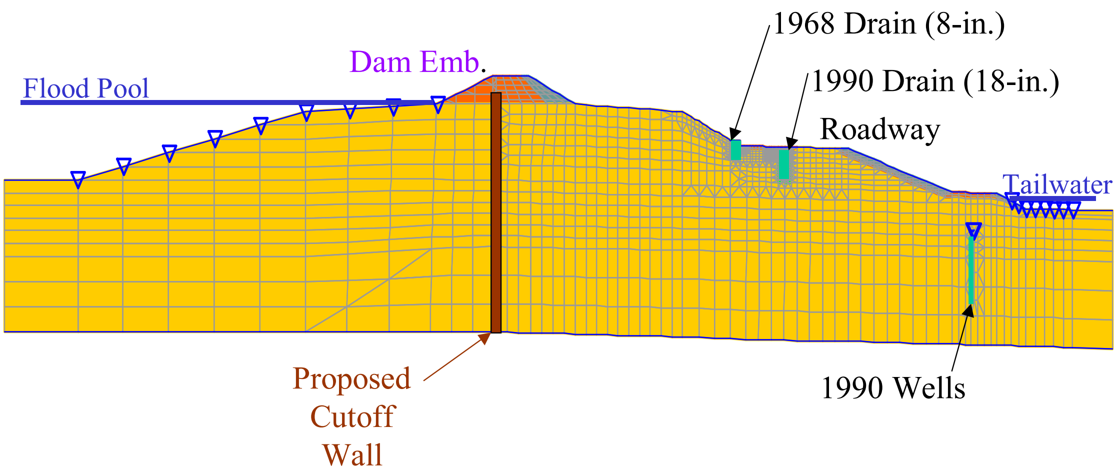


Figure C-2. Final finite element model for the left abutment of Hodges Village Dam

FastSeep Analysis - Current Conditions

Reoccurrence 1955 Record Flood (39 ft. pool)

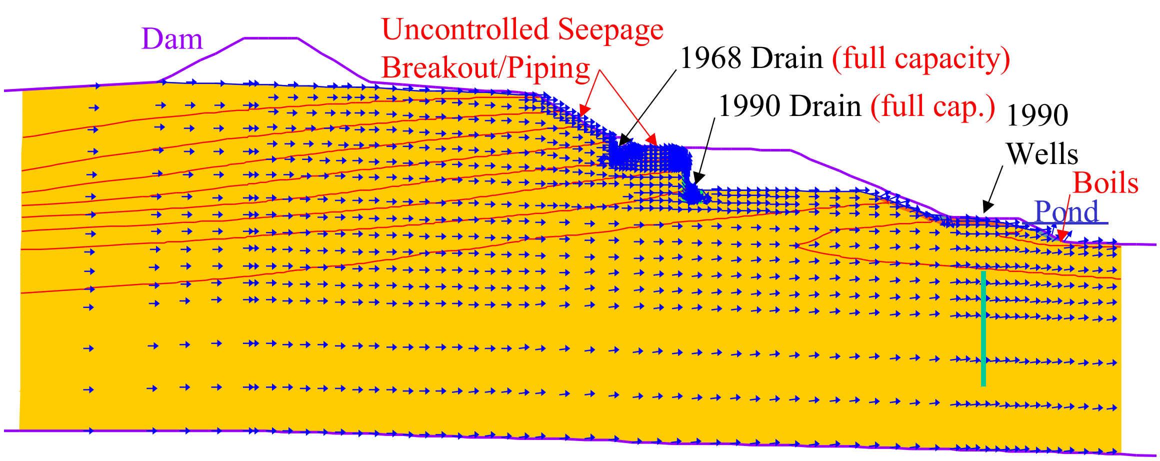


Figure C-3. Seepage flow vectors and pressure heads through the dam and the foundation for the left abutment

Table C-2 - Seepage Model

Pool El	Exit Gradient		Factor of Safety		Seepage Flow, cfs/lf	
	Left Abutment	Main Dam	Left Abutment	Main Dam	Left Abutment	Main Dam
486.6	0.54	0.59	1.53	1.40	0.45	0.25
493.8	0.63	0.69	1.31	1.20	0.68	0.35
501.0	0.72	0.79	1.15	1.05	1.00	0.45
505.0	0.79	0.84	1.05	0.98	1.15	0.52
510.0	0.85	0.91	0.97	0.91	1.45	0.62
515.0	0.92	0.99	0.90	0.84	1.81	0.71

b. A dam embankment and foundation cross-section that is typical of the left abutment section (Figure C-4) was selected as a critical section for the analysis. The shear strength parameters used for analysis are shown in Table C-3. The results of the slope stability analysis are shown in Table C-4.

Typical Cross Section - Sta. 13+00

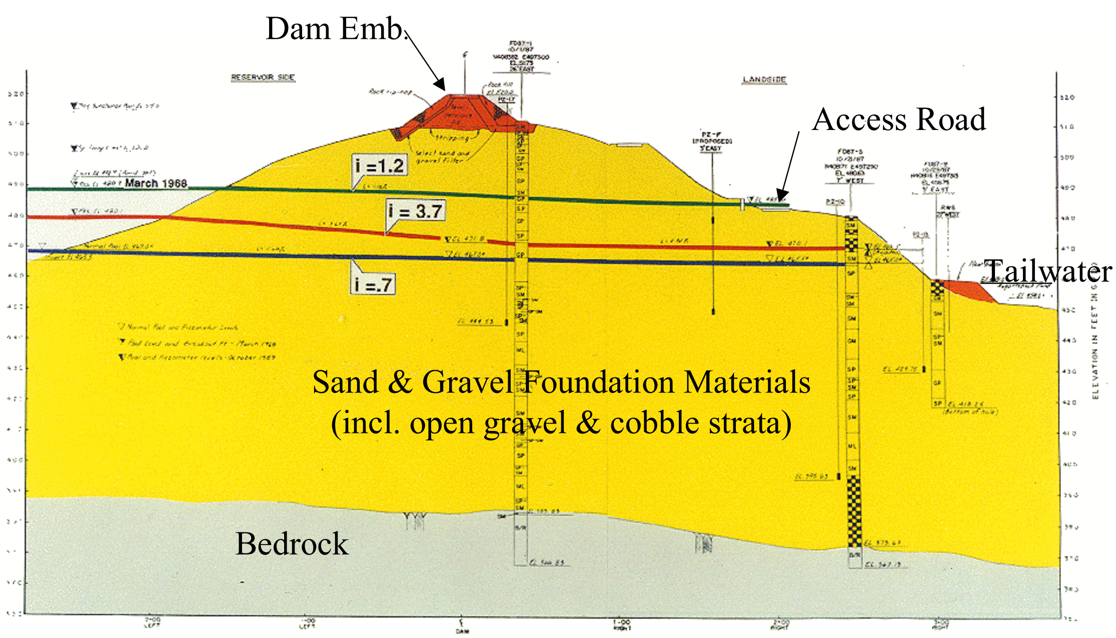


Figure C-4. Dam embankment and foundation cross-section typical of the left abutment section

Table C-3 - Stability Model Shear Parameters

Materials	Saturated Unit Weight pcf	Moist Unit Weight, pcf	φ Angle deg	Cohesion psf
Foundation Gravels, GP	125	120	35	0
Foundation Gravel, GP-GM	130	130	35	0
Embankment Fill	125	125	40	0
Rock fill	130	130	45	0
Filter	125	125	40	0

Table C-4 - Stability Analysis Results

Pool El	Factor of Safety
486.60	1.51
493.80	1.51
501.00	1.11
505.00	0.87
510.00	0.69
515.00	0.62

c. The model results indicate that low factors of safety will develop for the natural slope located above the access road caused by horizontal seepage force imposed by the rising phreatic surface and from the seepage breakout on the slope as the reservoir pool elevation increases. Table C-4 clearly shows a drop in factor of safety with the rising pool. At pool el 501.0 (spillway crest), the factor of safety reaches the critical value of 1.0.

C-5. Seepage and stability with Remedial Measures (Concrete Cut-off).

a. Because of the adverse seepage flow, seepage breakouts, and piping conditions predicted by the analysis, remedial measures were considered necessary to enhance the structural integrity of the dam to carry the design load safely. A concrete cutoff wall was selected as the most practical and reliable alternative based on the specific conditions at the dam site. The cutoff wall was modeled at both spillway and surcharge pool conditions and with various cutoff wall depths to determine the most effective depth to prevent the development of high exit gradients that exceed the critical gradient, and to prevent failure of the dam from potential piping condition. The results of the analysis are presented in Figs. 2 & 3. Fig. 2 shows the flow vectors and pressure heads through the dam with a full-depth cut-off wall, and Fig. 3 shows variations in exit gradients and seepage flows for various cut-off wall depths.

b. The results of the analysis indicate that a concrete cutoff wall will effectively reduce the phreatic surface within the foundation that will prevent both uncontrolled seepage breakouts on the downstream slope of the dam and development of large flow and high exit gradients along the downstream tailwater pond. The wall must extend to a minimum depth of 102 ft (el 418.0) below the top of the dam to limit the exit gradient to 0.825 (factor of safety of 1.0). In traditional geotechnical design, however, a factor of safety of 1.0 against piping is not considered adequate. A desirable value is 2.0. Analysis indicates that to achieve this, the cutoff wall must be extended to a depth of 135 ft (el 385.0) to the top of the surface of the bedrock (Fig. 4).

C-6**.** <u>Non- Time Dependent Reliability Analysis</u>**.** In 1992, USACE introduced reliability assessments as a method to help prioritize major rehabilitation project funding. The objectives of the assessment are to quantify the reliability of civil works projects, determine the probabilities and consequences of dam failure, assist in formulating a rehabilitation plan, and provide input parameters into the economic justification analysis. Probability of Unsatisfactory Performance is used as a relative measure of reliability or confidence in the ability of a structure to perform in a satisfactory manner. Probabilistic methods are used to systematically evaluate uncertainties in parameters that affect performance.

C-**7.** <u>Probabilistic Parameters</u>**,** The probabilistic parameters for each of the selected random variables (horizontal permeability k_h, vertical permeability $k_{v,}$, saturated weight $Y_{sat.,}$ coefficient of friction φ, k_h/k_v) including expected values *E,* standard deviations σ, and coefficient of variations *V* used in the various analysis are shown in Table C-5. These values were obtained by extensive model calibrations in order to duplicate the observed performance during the past flooding, and from field and instrumentation data.

Table C-5 - Probabilistic Parameters

Material Type	Random Variables		Expected Value *E*		Standard Deviation σ		Coefficient of Variation *V*, %	
GP	k_h	φ	0.35 fpm	35 deg	0.1 fpm	3 deg.	28.6	8.6
GP	k_h/k_v	$Y_{sat.}$	1000	125 pcf	100	5 pcf	10	4
GP-GM	k_h	φ	0.25	--	0.05 fpm	--	20	--
GP-GM	k_h/k_v	$Y_{sat.}$	150	--	15	--	10	--
Critical Gradient	i_{cr}	--	0.825	--	0.06	--	7.3	--

C-8. <u>Probabilistic Analysis (Existing Conditions)</u>.

a. A probabilistic analysis was made for the three failure conditions at the dam as well as for two additional failure conditions at the adjacent dike embankment. An event tree of various modes of failures is shown in Figure C-5**.** The goal of the analysis was to evaluate the probability of unsatisfactory performance for possible seepage and stability failure modes.

Taylor Series methods were used to calculate probability of failures for the analyses using lognormal distributions of the performance functions. A typical reliability plot for Taylor Series Analysis is shown in Figure C-6; and the results are summarized in Table C-6. The plots of the probability of failures versus pool elevations are shown in Figure C-7.

b. The five analyzed failure conditions are assumed to compose a representative model for the entire project. The total project probability of failure was expressed as the sum of the probability of failure of each component. The results of the reliability analysis clearly show that a hazardous condition develops as soon as the reservoir pool reaches el 493.8 (1987 flood event).

Reliability Analysis - Project Event Tree

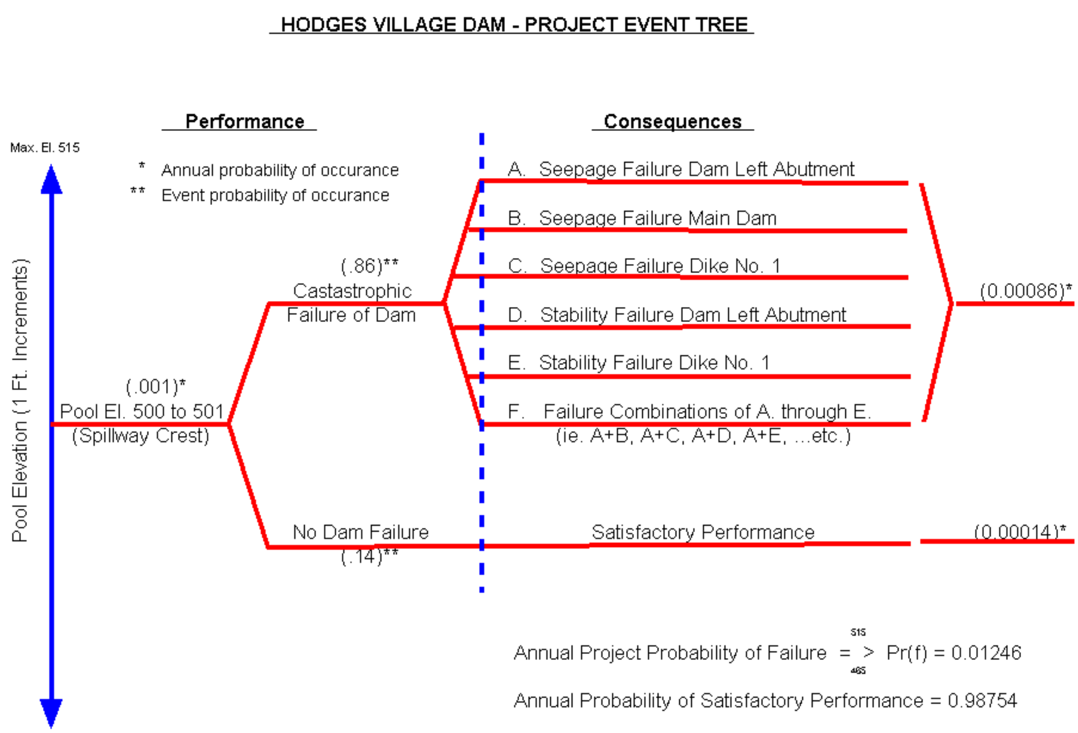


Figure C-5. Event tree of various modes of failures

Capacity / Demand Reliability Model

Hodges Village Dam
Typical Reliability Plot for Taylor Series Analysis

Probability of Underseepage Failure
Dam Left Abutment Peninsula Area
Pool at el. 505 - Current Conditions, Post 1993
(Reoccurance of Pre-Construction 1955 Record Event)

Taylor's Series Analysis

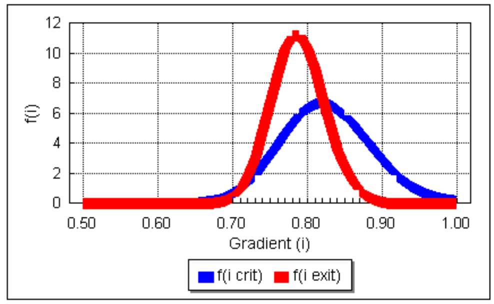
Run No.	kh	kh/kv	i (exit)	Var
1	0.35	1000	**0.787**	
2	0.25	1000	0.764	
3	0.45	1000	0.787	0.00013225
4	0.35	900	0.809	
5	0.35	1100	0.742	0.00112225
			Sum	0.00125450

E[i crit] = 0.825
sigma[i crit]= 0.06
V(i crit)= 0.07272727
E[ln i crit]= -0.1950096
sigma[ln i crit]= 0.07263138

E[i exit] = 0.787
sigma[i exit] = 0.03542
V(i exit) = 0.04500498
E[ln i exit] = -0.2405387
sigma [ln i exit] = 0.04498222

E[FS]= 1.04658154
sigma[FS]= 0.08957551
V[FS]= 0.08558865
E[ln FS]= 0.04187982
sigma[ln FS]= 0.08543253

Beta = 0.533
Pr(f) = 0.29704
1 in 3.37



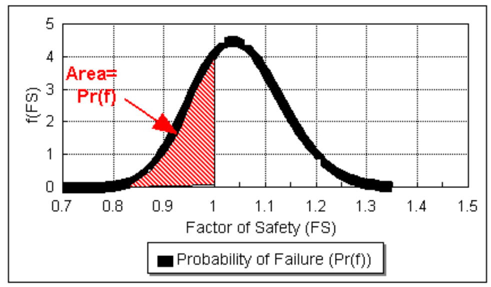


Figure C-6. Typical reliability plot for Taylor Series analysis

Table C-6- Summary of Probability of Failure

	Seepage			Stability		
Pool El	Left Abutment	Main Dam	Dike 1	Left Abutment	Dike 1	Total
486.80	0	0	0	0	0	0
493.80	0	0.023	0.105	0	0	0.127
501.00	0.060	0.318	0.770	0.189	0.126	0.895
505.00	0.297	0.584	0.947	0.910	0.554	0.999
510.00	0.668	0.860	0.995	0.999	0.917	1
515.00	0.913	0.977	1	1	0.987	1

Model Results: Potential Failure Modes

HODGES VILLAGE DAM - PROBABILITY OF FAILURE

Pool Elevation	Seepage Pr(f) Left Abut.	Seepage Pr(f) Main Dam	Seepage Pr(f) Dike No.1	Stability Pr(f) Left Abut.	Stability Pr(f) Dike No. 1	Combined Pr(f) Total Project
486.6	0.0	0.00004	0.00005	0.00012	0.0	0.00021
493.8	0.00044	0.02298	0.10456	0.00012	0.00002	0.12564
501	0.06044	0.31796	0.76966	0.18911	0.12583	0.89537
505	0.29704	0.58410	0.94336	0.90969	0.55362	0.99933
510	0.66799	0.86014	0.99530	0.99897	0.91673	1.0
515	0.91305	0.97691	0.99954	0.99924	0.98693	1.0

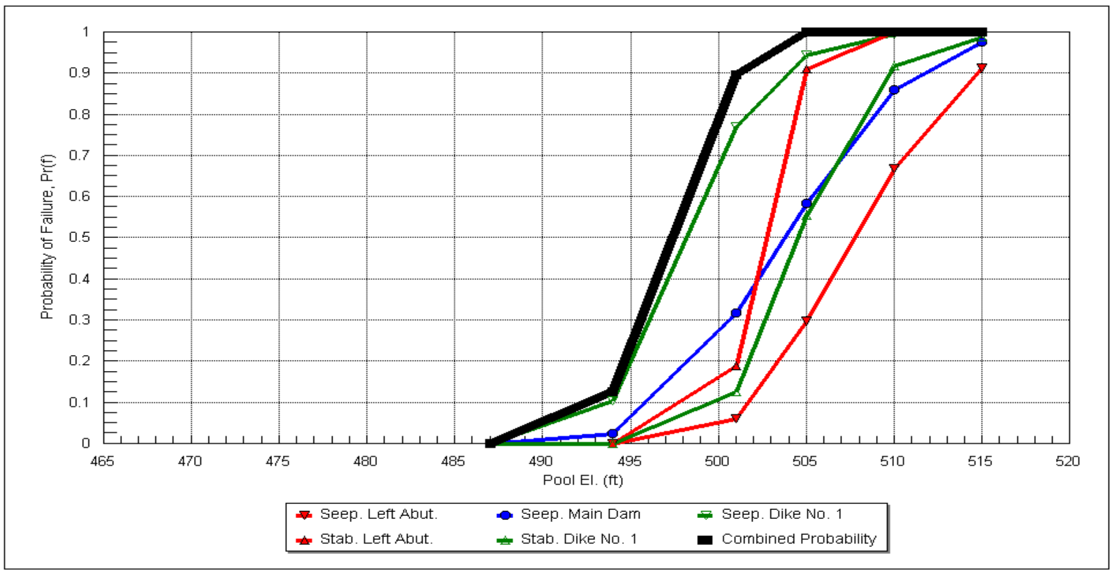


Figure C-7. Plots of the probability of failures versus pool elevations

C-9. Reliability analysis with Concrete Cutoff Wall.

a. Probabilistic analyses were performed for the dam with concrete cutoff walls with their tops at el 515.0 (surcharge pool elevation) and the bottoms at depths varying from el 445.0 to 385.0, el 385.0 being the top of the foundation bedrock. The cutoff wall with bottom at el 385.0 is considered a full-depth cutoff wall. Variations in exit gradients and total seepage flow through the dam with the varying depth of the cutoff walls are shown in Fig. C-3. Results of the probabilistic analysis for the full-depth cutoff wall are presented in Table C-7 and compared with those for the prerehabilitation conditions (at surcharge pool).

Table C-7. Summary of Cutoff Wall Analysis

Location of Analysis	Prerehabilitation Conditions		After Rehabilitation with Concrete Cutoff Wall		Remarks
	i_{ex} or FS	$P_r(f)$	i_{ex} or FS	$P_r(f)$	
Left Abutment	0.92	0.913	0.09	$6x10^{-6}$	Seepage analysis
Main Dam	0.99	1.000	0.05	$3x10^{-8}$	
Dike	1.23	1.000	0.19	$5x10^{-15}$	
Left Abutment	0.62	0.999	1.51	$1x10^{-4}$	Stability analysis
Main Dam	0.63	0.987	1.86	$1x10^{-14}$	

b. The results show that the worst probability of failure decreased from 100 percent to almost zero percent with the introduction of the concrete cutoff wall.

C-10. Conclusions.

a. Hodges Village Dam, in operation since 1959, required three major projects to repair the dam after seepage problems occurred during 1968, 1987, and 1993 after flood events of relatively low frequencies (16, 40, and 10 years). Seepage problems at low-frequency flood events indicated that future floods of greater magnitude could seriously impact the safety of the dam.

b. Finite element analysis was used to reproduce the behavior of the dam observed during the past flood events and to predict the future behavior at floods of higher magnitude. Reproduction of observed behavior of the dam served to validate the analytical model used to predict the response of the dam at higher flood frequencies. Seepage analysis using the model with variation in permeability and shear strength parameters served as a basis for assessing reliability of the dam at various reservoir pool elevations. Behavior of the model with a concrete cutoff wall helped confirmed the selection of the concrete cutoff wall as an appropriate remedial measure.

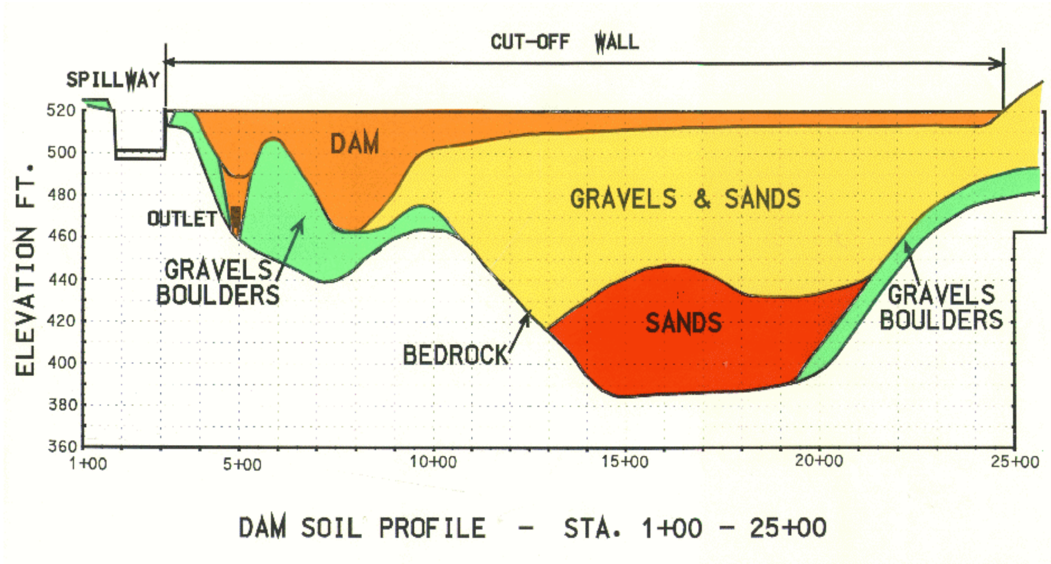
Hodges Village Dam - Annual Probability of Failure

[Total Annual Probability of Failure, Pr(f) = $\overset{515}{\underset{465}{>}}$ (Prob. Failure)(Delta Prob.Stage)(d pool)]

Pool Elevation	Pool Stage	Pool Stage Exceedance Probability	Delta Stage Exceedance Probability	Pool Stage (@mid point)	Prob. Failure (@mid point.)	Combined Probability (per 1 ft. incr.)
486	20.5	0.11700				
			0.02200	21.0	0.0002	0.000004
487	21.5	0.09500				
			0.01800	22.0	0.0007	0.000013
488	22.5	0.07700				
			0.01500	23.0	0.0022	0.000033
489	23.5	0.06200				
			0.01200	24.0	0.0055	0.000066
490	24.5	0.05000				
			0.01000	25.0	0.0135	0.000135
491	25.5	0.04000				
			0.00800	26.0	0.0300	0.000240
492	26.5	0.03200				
			0.00600	27.0	0.0600	0.000360
493	27.5	0.02600				
			0.00500	28.0	0.1100	0.000550
494	28.5	0.02100				
			0.00500	29.0	0.1800	0.000900
495	29.5	0.01600				
			0.00300	30.0	0.2900	0.000870
496	30.5	0.01300				
			0.00250	31.0	0.4000	0.001000
497	31.5	0.01050				
			0.00230	32.0	0.5400	0.001242
498	32.5	0.00820				
			0.00170	33.0	0.6600	0.001122
499	33.5	0.00650				
			0.00110	34.0	0.7700	0.000847
500	34.5	0.00540				
			0.00100	35.0	0.8600	0.000860
501	35.5	0.00440				
			0.00090	36.0	0.9300	0.000837
502	36.5	0.00350				
			0.00070	37.0	0.9800	0.000686
503	37.5	0.00280				
			0.00050	38.0	0.9930	0.000497
504	38.5	0.00230				
			0.00030	39.0	0.9986	0.000300
505	39.5	0.00200				
			0.00050	40.0	0.9997	0.000500
506	40.5	0.00150				
			0.00030	41.0	1.0000	0.000300
507	41.5	0.00120				
			0.00030	42.0	1.0000	0.000300
508	42.5	0.00090				
			0.00020	43.0	1.0000	0.000200
509	43.5	0.00070				
			0.00020	44.0	1.0000	0.000200
510	44.5	0.00050				
			0.00010	45.0	1.0000	0.000100
511	45.5	0.00040				
			0.00010	46.0	1.0000	0.000100
512	46.5	0.00030				
			0.00010	47.0	1.0000	0.000100
513	47.5	0.00020				
			0.00005	48.0	1.0000	0.000050
514	48.5	0.00015				
			0.00005	49.0	1.0000	0.000050
515	49.5	0.00010				

Total Annual Probability of Failure = 0.012461

Soil Profile of Main Dam



Glacial outwash foundation deposits contain numerous interlaced open work gravel and cobble strata!

Figure C-8. Soil Profile of Main Dam

Section II
Example 2
Issue: Time-Dependent Analysis of Seepage Through a Karst Foundation
Project: Wolf Creek Dam, Nashville District

C-11. Brief Summary of Past and Current Performance of Wolf Creek Dam.

a. Wolf Creek Dam has a wealth of historical information and instrumentation data to assist with understanding the past and current performance of the project. The construction of the dam was started in 1941 and abruptly stopped in 1943 because of World War II. The construction of the dam was restarted in 1946 and finally completed in 1951. After about 16 years of operation, wet areas and sinkholes were discovered at the downstream toe of the embankment in 1967 and 1968. At that time emergency grouting was conducted during which approximately 290,000 cu ft of grout was placed over a 200-ft length in the embankment near the concrete section of the dam.

b. Diaphragm walls were designed and constructed in 1975 to 1979 knowing the emergency grout placement in 1968 would be only a temporary fix to the seepage problems in the foundation. This diaphragm wall consisted of two sections: an embankment diaphragm wall that was 2250 ft in length and a switchyard diaphragm wall that was 590 ft in length. Both of the walls were constructed of primary and secondary elements centered on 24-in. spacing. Portions of the embankment wall were placed through the weak Liepers limestone formation into the more durable Catheys limestone formation in order to cut off the seepage under the structure. However, as a cost savings for the project, other portions of the diaphragm wall did not reach to this critical formation contact.

c. Unfortunately, given time, these seepage paths have moved along the joint patterns, and similar distress indicators to the 1967 and 1968 events have emerged. Significant settlement of the crest of the embankment started in 1993, and this settlement still continues today. Numerous wet spots in areas downstream of the toe similar to 1968 and some wet areas close to 1968 sinkholes in the switchyard have reappeared, causing significant concern. In addition, increases in piezometric readings within the embankment have continued to rise as much as 11 ft over the past 20 years. Some of the high piezometric pressures found in the embankment fluctuate with headwater and are nearly 40 to 60 percent of the headwater readings.

C-12. Distress Indicators for Wolf Creek Dam**.** Wolf Creek Dam has shown a number of distress indicators, now and in the past, that have documented poor and unacceptable performance within the karst foundation and dam embankment. These distress indicators have manifested themselves visually as sinkholes and large wet areas downstream of the toe of the embankment. Other key distress signals are found within the instrumentation data for the project. These signals are shown in the form of significant rises in piezometric pressure, increased number of artesian piezometers, and continuous settlement of the embankment crest. These indicators, whether visual or from instrumentation, are critical in understanding the foundation and embankment deficiencies that currently exist at Wolf Creek Dam. Figure C-9 compares distress indicators at Wolf Creek Dam between the conditions in 1968 and 2004.

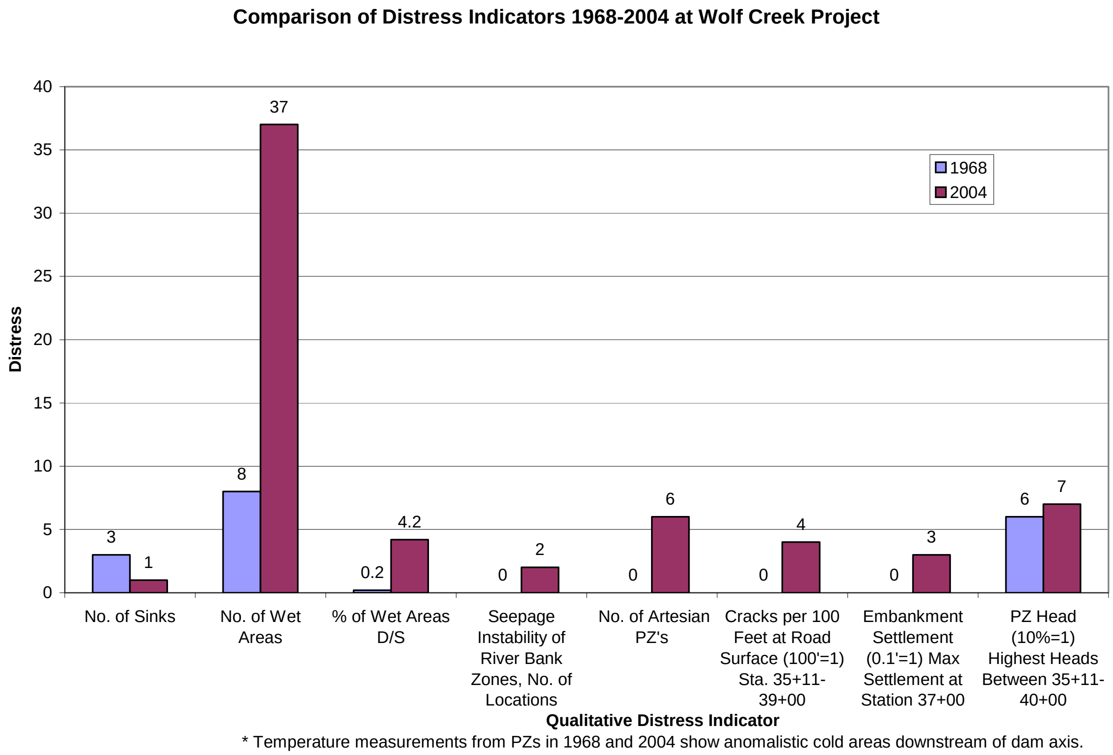


Figure C-9. Comparison of distress indicators for Wolf Creek Dam

C-13. Reliability Modeling Concepts for Wolf Creek Dam.

a. In general, reliability models can be developed for dam projects where sufficient and reliable instrumentation data such as piezometric head or settlement data exist. These data can be used to examine and establish significant trends of unsatisfactory performance over time. The use of such historical data permits time to be an important factor in developing the concepts for a reliability model. The development of a time-dependent model to determine the hazard function or rate is important to assist with the inputs to the economic modeling required for the Major Rehabilitation process.

b. However, in the development of the reliability models, even when sufficient data exist, the limit states are still hard to define because of the complexities found in modeling the karst foundation. For Wolf Creek Dam various distress indicators and the data as defined in previous section were evaluated for use in the development of the reliability model. From this detailed evaluation, it was determined that the pressure data from the piezometers and settlement data from the crest were the most reliable and most representative of the problems developing in the foundation and embankment. From a quick preview of the data, the first concept for a reliability model of the foundation was developed to show the changes over time for the selected data and the tie into both the performance (past and present) and repairs based on some an arbitrarily defined limit state. The reliability model for Wolf Creek Dam would need to examine more closely the piezometric head and crest settlement data over time to establish the trends for the model. These trends for the dam and thus the reliability analysis would assist in the development of defining a realistic limit state to be used in the generation of the results from the reliability model. Figure C-10 shows the first cut at the reliability model concept developed for the foundation conditions at Wolf Creek Dam.

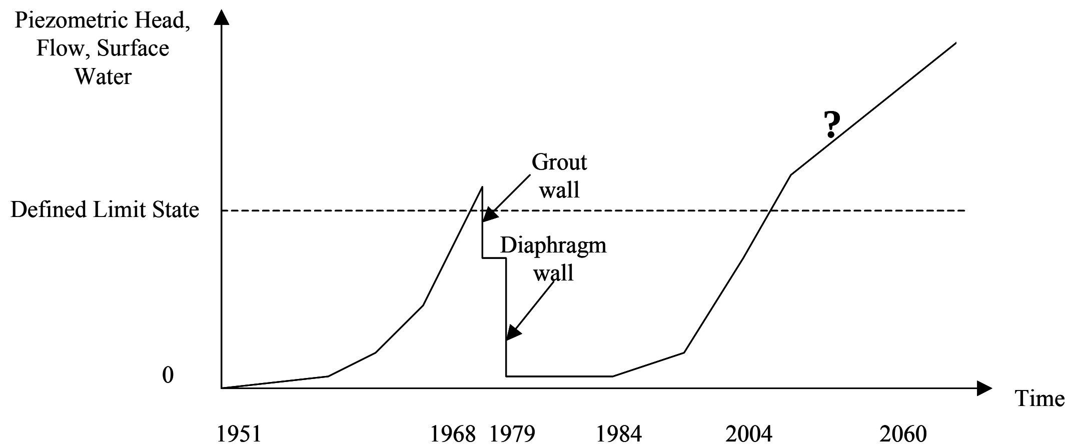


Figure C-10. Reliability concepts for Wolf Creek Dam

C-14. <u>Processing of Instrumentation Data</u>.

a. The instrumentation data were collected from representative piezometers within three sections of the dam. Three piezometers were selected for each section that best reflected a good, average, and worst condition of the piezometric pressures within the embankment zone. The piezometers also were selected based on their positions ranging from center line of the dam to the downstream toe. The spatial variability of the piezometric data will be accounted for as a random variable in the development of the reliability model. A summary of information for the piezometers is shown in Table C-8.

b. Instrumentation data from 1984 to 2004 for the selected piezometers were processed and plotted to establish a trend line for the suite of piezometers in each section. An average trend line with a standard error was established for use in development of the reliability model. The time plots for the piezometric data sets with the changes in headwater for each section are shown in Figures C-11 through C-13. The processed data from the trend line for each section are shown in Table C-9 and the dam sections are shown in Plates G-1 through G-5 in Section 6 of the Main Report.

Table C-8. Piezometer Information

Piezometer	Installation Date	Type
D-321		Single
D-322		
D-323		
DC-258		
DC-254R	Nov 1979	
WA-25		
D-275A		
WA-59A		
WO-35		

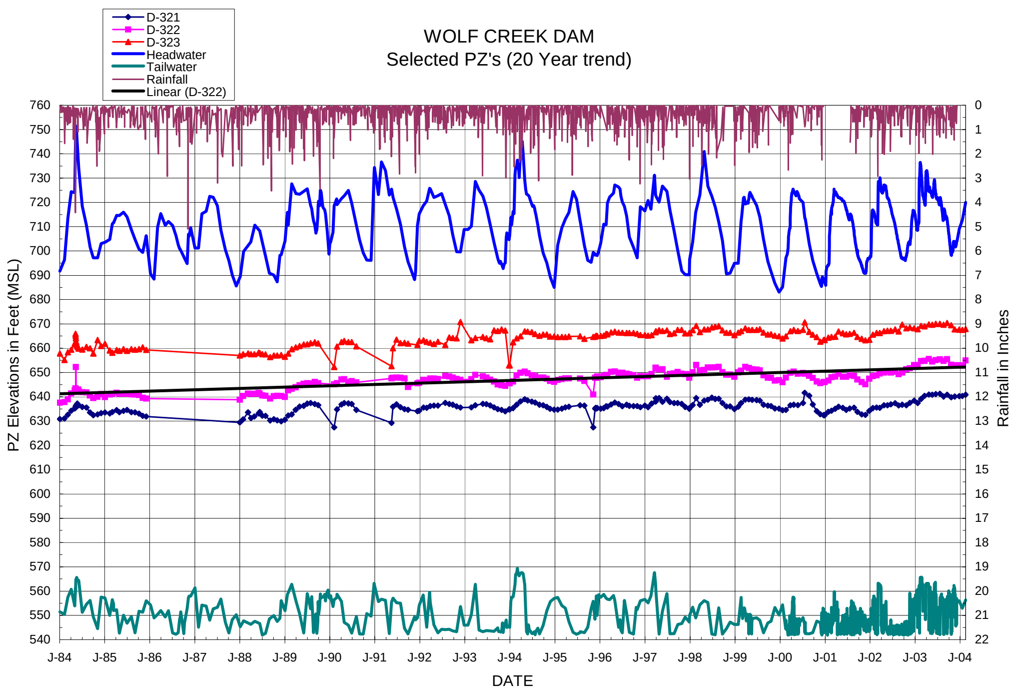


Figure C-11. Piezometric data for Section 1 (Note: Headwater is shown as blue line above piezometric data for reference)

c. Another important indicator of the foundation problems at Wolf Creek is the crest settlement data. The variable rate of settlement may be the result of inconsistent erosion in the foundation or the translation of the settlement through 200 ft of embankment material. Erosion of the embankment foundation may not occur at the same rate because the flow of water may shift locations in the interconnected karst system underlying the embankment. Erosion of the base of the embankment must be translated through the height of the embankment to affect the

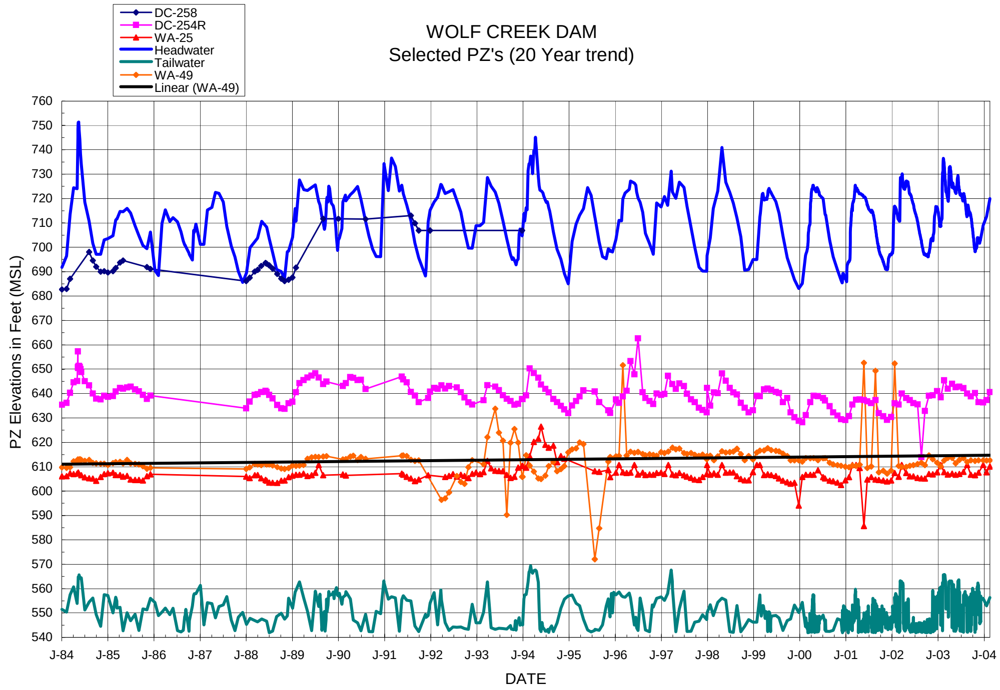


Figure C-12. Piezometric data for Section 2 (Note: Headwater is shown as blue line above piezometric data for reference)

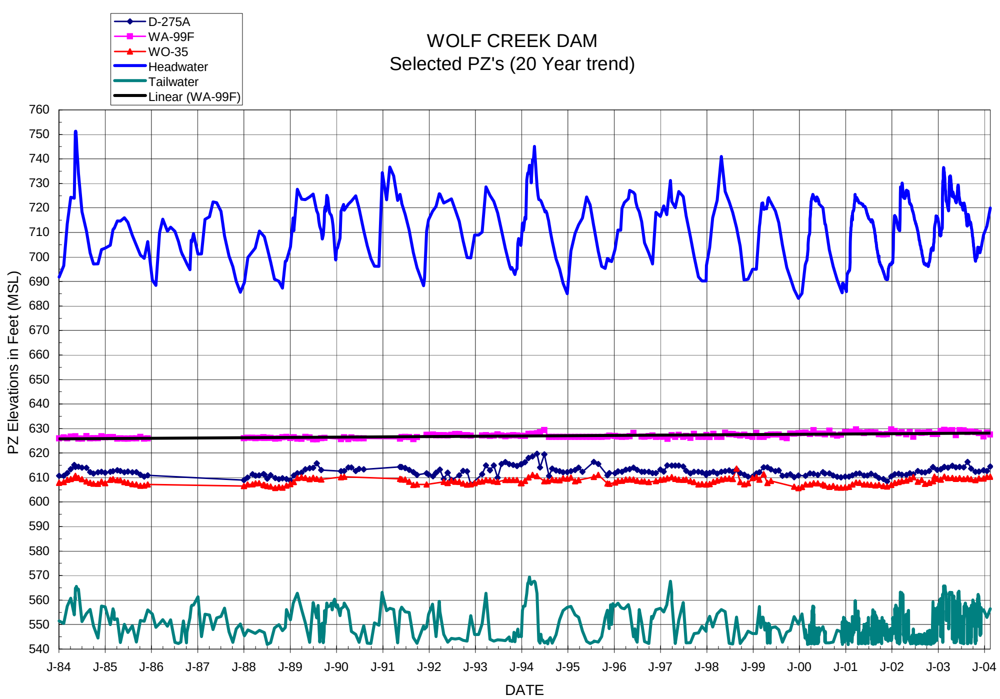


Figure C-13. Piezometric data for Section 3 (Note: Headwater is shown as blue line above piezometric data for reference)

Table C-9. Trends in Piezometric Pressure Rise at Wolf Creek (1984-2004)

Section	Average Trend Rise in Piezometer, ft	Standard Error ft
1	11.3	1.7
2	6.1	1.4
3	4.2	1.2

surface monumentation in a series of relatively small settlements. As the clay layers settle above the erosion, a small gap is created between the eroding layers and the overlying clay. The strength of the overlying clay layer is initially sufficient to create an arch. However, because the clay is plastic, it eventually fails under the overlying load of the embankment. This progressive failure moves upward through the embankment until it ultimately reaches the surface, causing the surface monument to settle. This type failure could take place over a considerable amount of time and could explain the somewhat sporadic settlement of the surface monuments.

d. These data give valuable information to the reliability model by establishing a time period during which the foundation performance started to change significantly. The settlement data were available from crest monument surveys conducted from 1985 to 2005. The results from the data show that the embankment has had two significant periods of settlement, 1991 to 1993 and 1998 to present. The first settlement period resulted in a maximum settlement of the embankment of 0.17 ft near the concrete dam section. This initial settlement slowed over the period from 1994 to 1998 where a continued period of settlement currently exists with the maximum settlement of the crest at 0.3 ft. Figure C-14 shows the time trend of the settlement data for all the crest monuments. Note that the monuments are numbered EM-1 to EM-26 with EM-1 starting near the concrete dam section and progressing in 100-ft stations across the dam. Figure C-15 shows the trend of the settlement across the crest of the dam, i.e., from EM-1 to EM-26, for each year.

C-15. Time-Dependent Reliability Model Development for Wolf Creek Dam.

a. Sections defined for reliability model. For the reliability modeling purposes, the embankment dam has been divided into three different sections based on differences in the existing embankment structure and past performance. Section 1, Station 31+11 to Station 40+00, contains the diaphragm wall and a portion of the concrete dam, and is perpendicular to the original core cutoff trench. Section 2, Station 40+00 to 57+50, contains the diaphragm wall through its entire section and is parallel with the core trench. Section 3, Station 57+50 to 74+00, is founded only on existing alluvium and no diaphragm wall was installed. The defined sections are shown in Plates G-1 through G-5 in Section 6 of the Main Report.

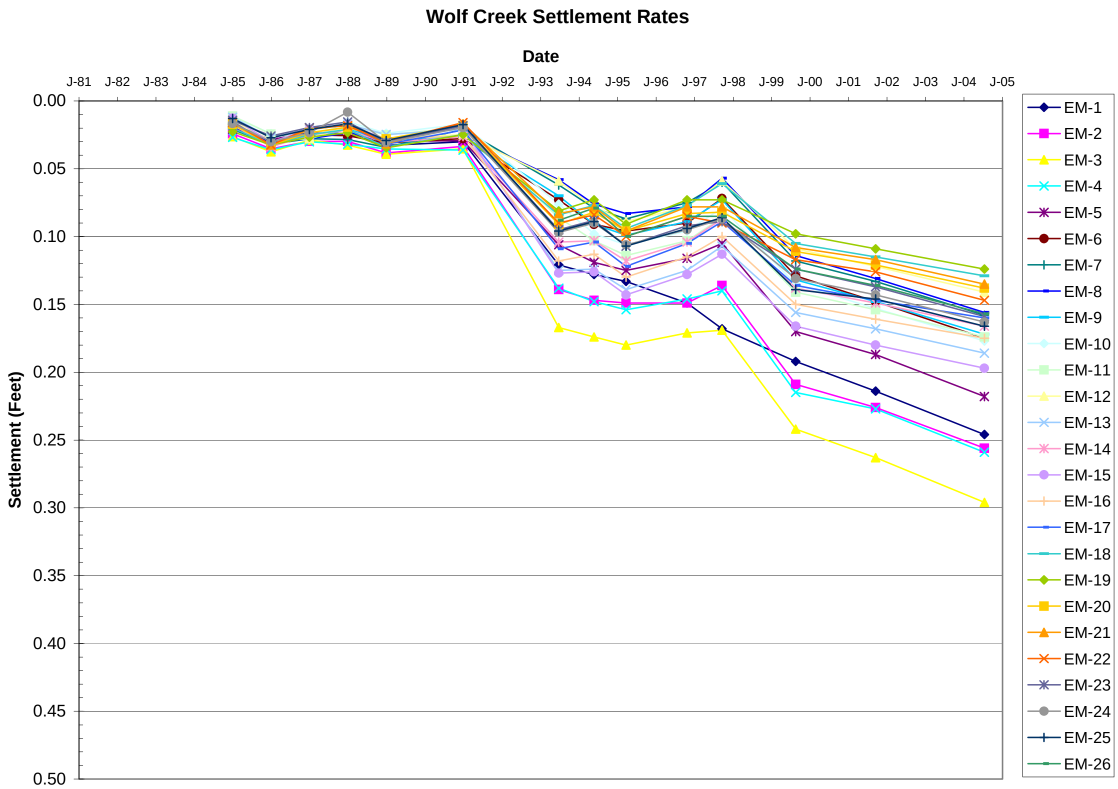


Figure C-14. Time settlement of crest monuments at Wolf Creek Dam

b. Damage Accumulation Model.

(1) A Damage Accumulation Model (DAM) was selected for incorporation into the reliability analysis based on historical headwater data. DAMs are frequently used in both fatigue analysis of steel structures and wear rate analysis of highway pavements as well as many other engineering applications. A DAM model was utilized to account for the annual cyclic nature of the headwater that has occurred and still occurs at the project. This cyclic variation has caused an accumulated increase in piezometric pressures in the foundation caused by both the intensity (annual maximum pool) and duration. Figures C-11, C-12, and C-13 show the accumulated affects of the cyclic headwater (blue lines at the top of the figures) on the piezometric pressures in Sections 1, 2, and 3 respectively.

(2) Annual headwater intensities and duration were determined from the headwater data for Wolf Creek from 1950 to 2004. The intensity and duration were used to define the shape of the annual loading curve in the damage accumulation model. This shape was simplified to a triangle with the annual peak of the intensity and a width in days of the duration at el 710. This elevation was used since it is tied to the median, the lowest hydropower pool, and little or no piezometric damage occurred at headwaters lower than this elevation. A more detailed analysis of the headwater data using a sinusoidal shape function was developed but was not felt as

accurate as the triangular shape load model. Figure C-16 shows an example of the concept of duration and annual peak intensity.

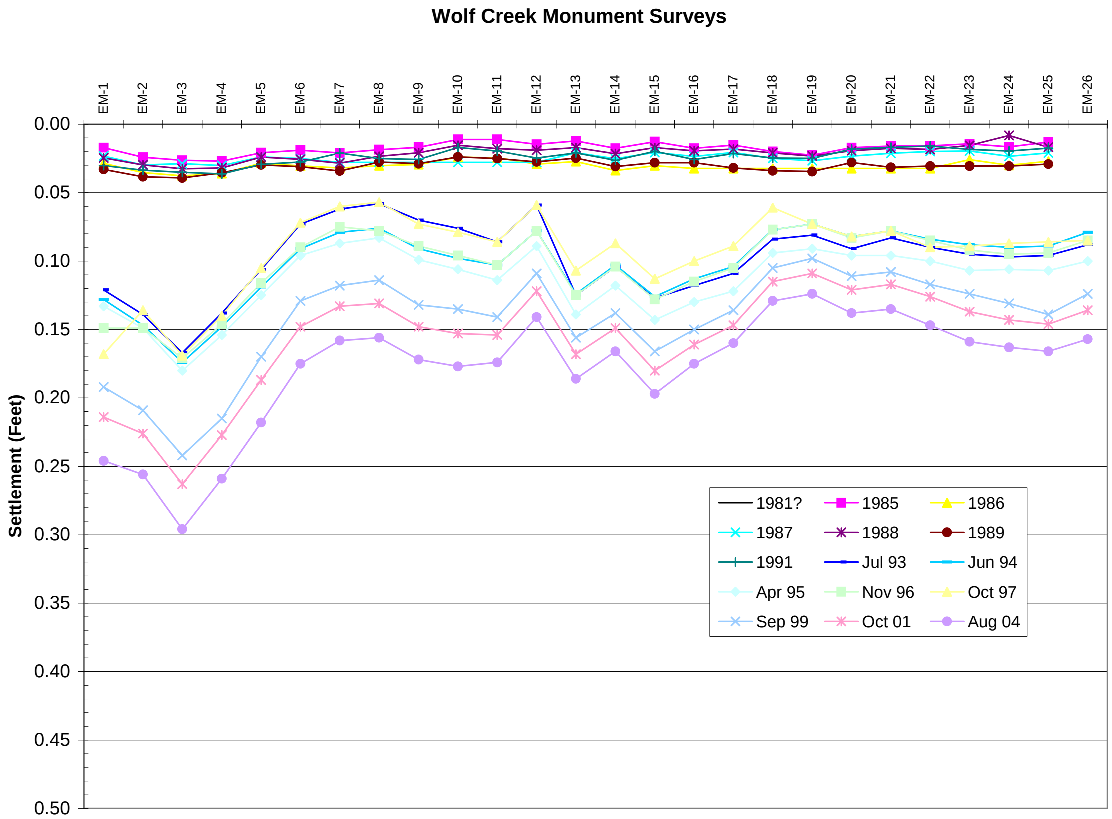


Figure C-15. Settlement of crest monuments across dam center line

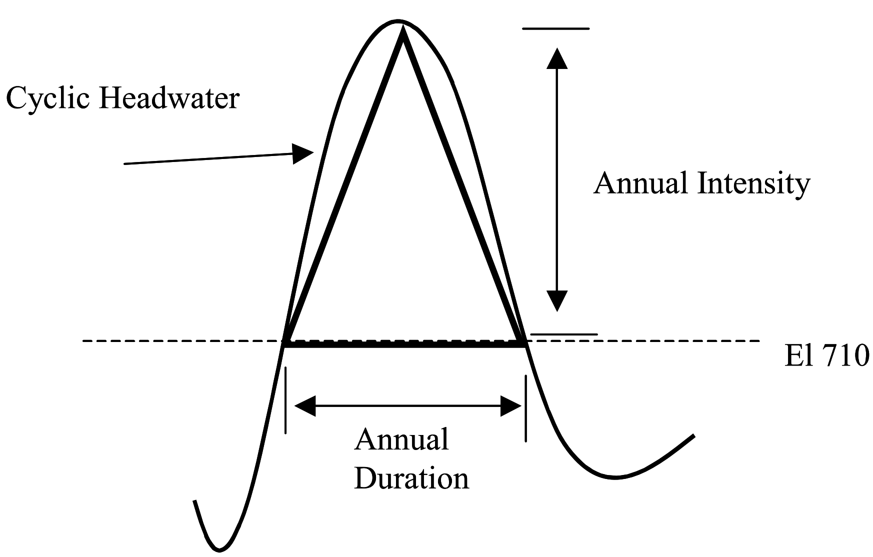


Figure C-16. Relationship of annual intensity and duration

(3) The intensity was taken as the maximum value of the annual cyclic peak headwater. Figure C-17 shows the annual peak intensity values used in the calibration of the DAM. The duration was determined using the median pool elevation of 710 that also corresponds to the hydropower pool at the project. Figure C-18 shows the annual duration values used in the DAM. Note that there is a significant dip in Figures C-17 and C-18 for intensity and durations, which reflects the years during the diaphragm wall construction in which lower pools were maintained. From these data, a set of nonlinear equations was developed based on pool intensity and duration as they relate to the actual annual damage for piezometric rise. These equations permitted the DAM model to account for increases in annual damage with a higher intensity and longer duration and vice versa for lower duration and intensity events. These equations were then extrapolated into the future using statistical distributions for headwater to predict the annual and accumulated damages within the reliability model.

c. Reliability model calibration.

(1) The reliability model was calibrated for each section to account for the proper accumulated damages as shown in Table B2.1. This calibration also included, in Sections 1 and 2, an arbitrary decrease in the piezometric rise to reflect the grouting in 1968 and installation of the diaphragm wall in 1979. Section 3 was calibrated without any changes in piezometric rise. The calibration was made using the data and nonlinear equations for the historical intensity and duration from 1950 to 2004 as discussed above. The DAM determines both the annual piezometric rise and then sums for an accumulated rise in pressures. This calibration was performed to ensure that the model would correctly predict the past and current performance and repairs to the dam.

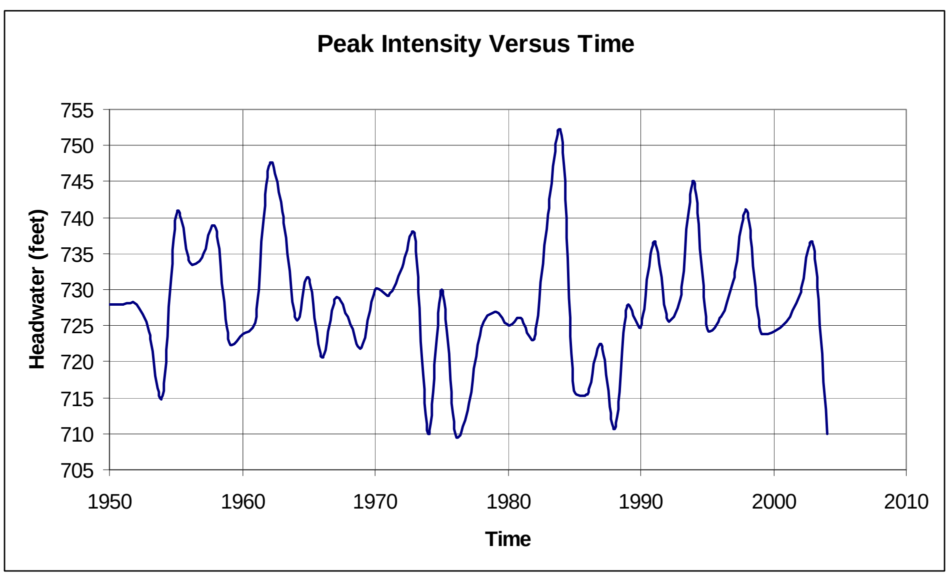


Figure C-17. Annual peak intensity for headwater

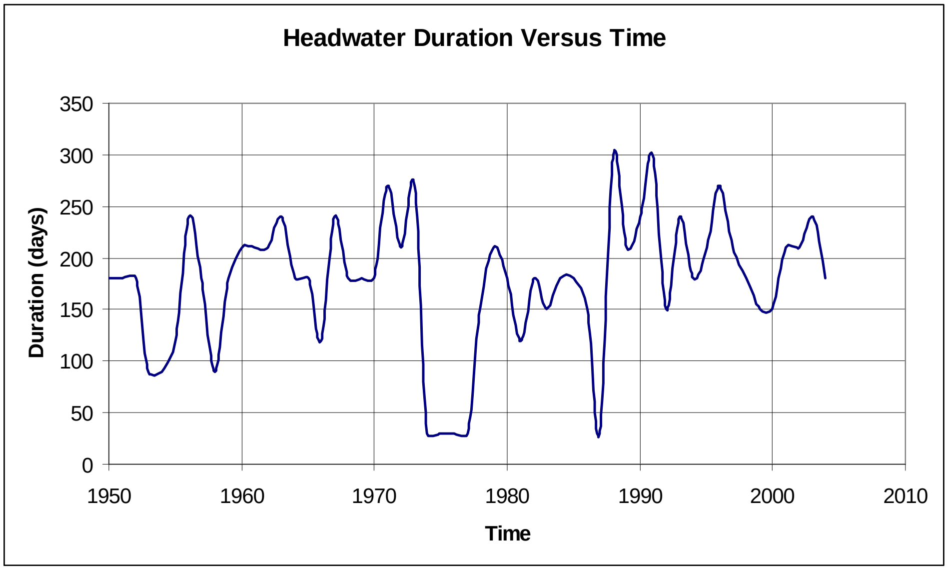


Figure C-18. Annual headwater duration

(2) In addition, the calibration of the DAM model was also made to reflect the settlement changes in both 1991 and 1998 and their current changes in slope over time. This has been matched to the changes reflected in the piezometric pressure data and is correlated with the accumulated damages in the DAM model. The settlement in the foundation is an important supporting and initiating event where changes in piezometric rise have been fully documented and incorporated into the reliability model. Figure C-19 shows an example of the calibration of the DAM.

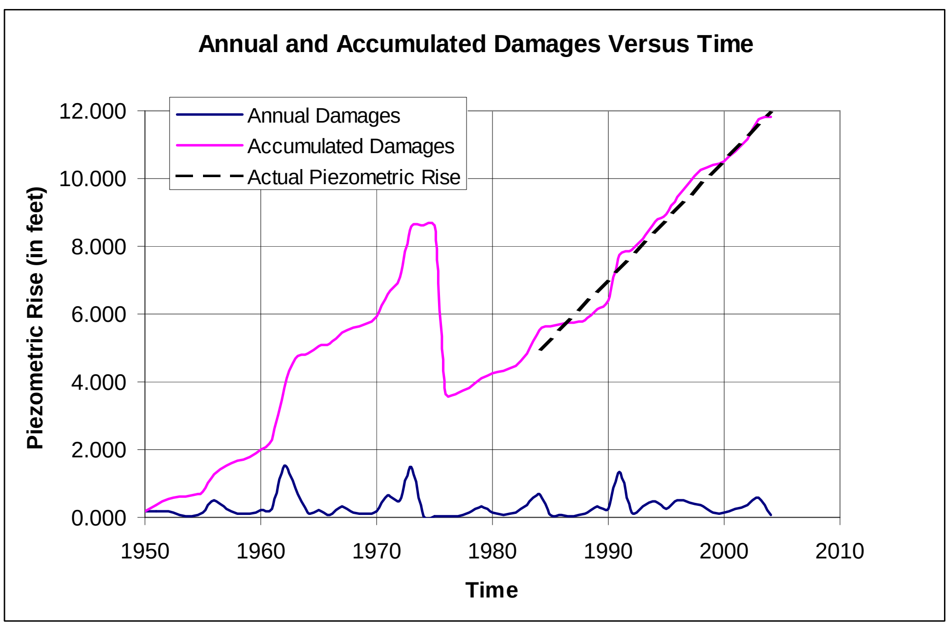


Figure C-19. Calibration of DAM

d. Random variables in reliability model.

(1) The reliability model uses only three random variables in the model: annual peak intensity, annual duration, and a model factor to account for the spatial variability in piezometric data. The distributions for annual intensity and duration were taken from the headwater data for the project from 1950 to 2004 as previously discussed. The resulting distribution for the peak intensity was lognormal with a mean value of el 727 ft with a standard deviation of 10 ft. The lognormal distribution was truncated at a minimum of 710 and a maximum of el 751. The distribution for duration was determined to be lognormal with a mean value of 180 days and a standard deviation of 60 days. The normal distribution was truncated at a minimum of 30 days and at a maximum of 300 days.

(2) A model factor for spatial variability in the piezometers was accounted for by examining the piezometric data from 1984 to 2004 within each section. These data were used to determine the differences and estimate the uncertainties in areas within each section that does not have piezometric data. Variations in pressures were accounted for in the factor in both the lateral and longitudinal directions for each section. A quadratic equation was developed and implemented in the reliability model from 1984 to the end of study. The quadratic equation was used as a multiplier to the annual damages to reflect the variability within the foundation. This permitted the increase in variation to range from a straight line to an exponential curve. The upper limits for the model factor were different for each section since there are fewer piezometers and more uncertainty to model in Section 3 than in Sections 1 or 2. Table C-10 summarizes the random variables used in the reliability analysis.

Table C-10. Summary of Random Variables

Random Variable	Distribution	Mean	Standard Deviation	Truncation (min, max)
Annual Peak Intensity	Lognormal	727 ft	10 ft	710 ft, 751 ft
Annual Duration	Normal	180 days	60 days	30 days, 300 days
Spatial Variation	Uniform	Lower	Upper limit	
	Section 1	1.0	1.7	
	Section 2	1.0	2.0	
	Section 3	1.0	2.5	

e. Limit states for reliability model.

(1) The limit state for the reliability model is based on a set increase or rise in piezometric pressure in the foundation. This limit state was established using both the input from the Expert-Opinion Elicitation (EOE) and the identification of a set point in time when the foundation started to experience recent distress. While it is recognized that a drop in piezometric pressure indicates a severe problem within the foundation, the aim of the limit state that was selected for the reliability model is to define the unsatisfactory performance of the foundation. This unsatisfactory performance assumes that a more critical limit, such as piezometric drop, will not be reached prior to the limits set for the rise in foundation pressures.

(2) Based on the results from the EOE, the experts felt that a 5-ft rise in piezometric pressure would necessitate unsatisfactory performance of the foundation. The elicitation results

showed this 5-ft rise to be true for all the sections. However, based on a review of the past history of wet areas and sinkholes downstream and with the lack of a diaphragm wall in Section 3, it was felt more realistic to use a lower value of rise for this section based on the lack of the confidence in the median value by the experts. The limit state for Section 3 was redefined as a 3-ft piezometric rise.

(3) The other critical piece of information required is the point of initiation of unsatisfactory performance of the foundation. This information would confirm the piezometric pressure rise limits defined by the experts. The crest settlement data provide a critical time when significant changes and unsatisfactory performance in the foundation were initiated. The year at which this initiation occured is indicated by the settlement data in 1994. This year coincides with the end of the initial settlement in the crest.

(4) The average value for piezometric rise from 1984 to 2004 was determined as described earlier. The value for the rise in 2004 with the unacceptable rise defined by the experts now becomes the limit state established in the reliability model. These limiting values agree well with the opinions of the experts who indicated that the structure has already seen initiation of unsatisfactory performance in the past and expect the unsatisfactory performance of the structure in the near future. This also relates to the District's Emergency Action Plan for Wolf Creek Dam where specific actions will be taken upon certain levels of piezometric rise or drop. A summary of the limit states for each section is shown in Table C-11.

Table C-11. Summary of Limit States for Reliability Model

Section	Average Piezometer Value in 2004, ft	Rise of Piezometer from EOE ft	Unsatisfactory Performance Limit State. ft
1	9.3	5	14.3
2	4.1	5	9.1
3	1.2	3	4.2

f. Time-dependent reliability and hazard functions.

(1) The time-dependent reliability *L(t)* and hazard functions *h(t)* for each section were determined using the Monte CarlosSimulation (MCS) package called @Risk Version 4.5. @Risk is an add-in to Microsoft Excel that is used to perform the MCS for reliability modeling. The MCS utilizes the DAM model in combination with the random variables to calculate the accumulated damages up to the year in which the unsatisfactory performance occurs. This year for the unsatisfactory performance is used to develop the reliability and hazard functions for use in the economic analysis. The MCS reliability models developed for these analyses were run for 50,000 iterations.

(2) The cumulative time-dependent reliability is calculated using the following equation:

$$L(t) = \frac{\text{Cumulative number of unsatisfactory performances up to time, t}}{\text{Total number of iterations}}$$

(3) The hazard rate or functions is calculated by using the following equation:

$$h(t) = \frac{\text{Number of unsatisfactory performances in time, } t+1}{\text{Number of survived up to time, } t}$$

(4) The baseline condition assumes that no rehabilitation or repairs have been made to the dam since the installation of the diaphragm wall in 1979. The cumulative time-dependent reliability and hazard functions for the baseline condition of the three sections are shown in Figures C-20 and C-21, respectively.

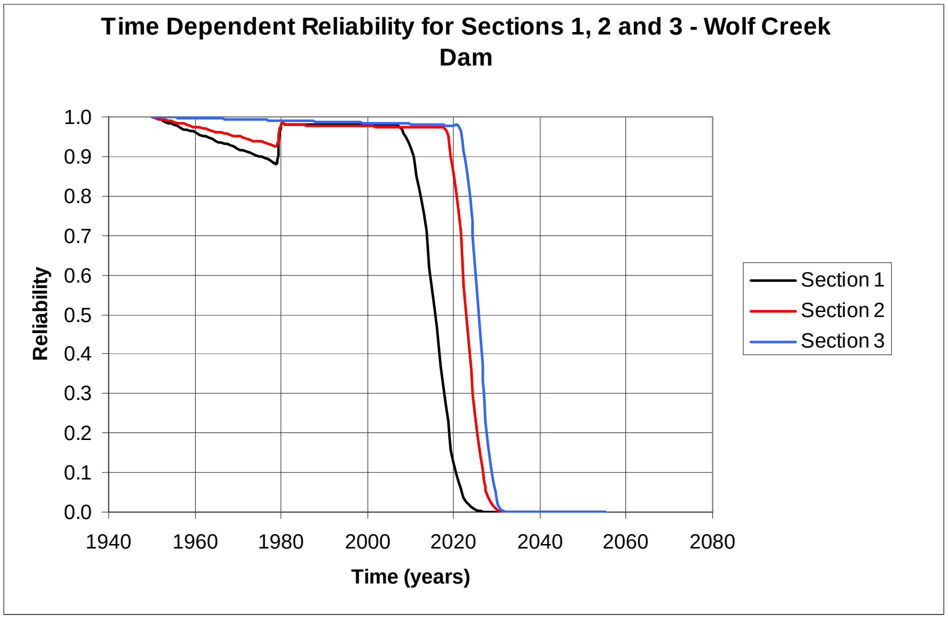


Figure C-20. Baseline time-dependent reliability

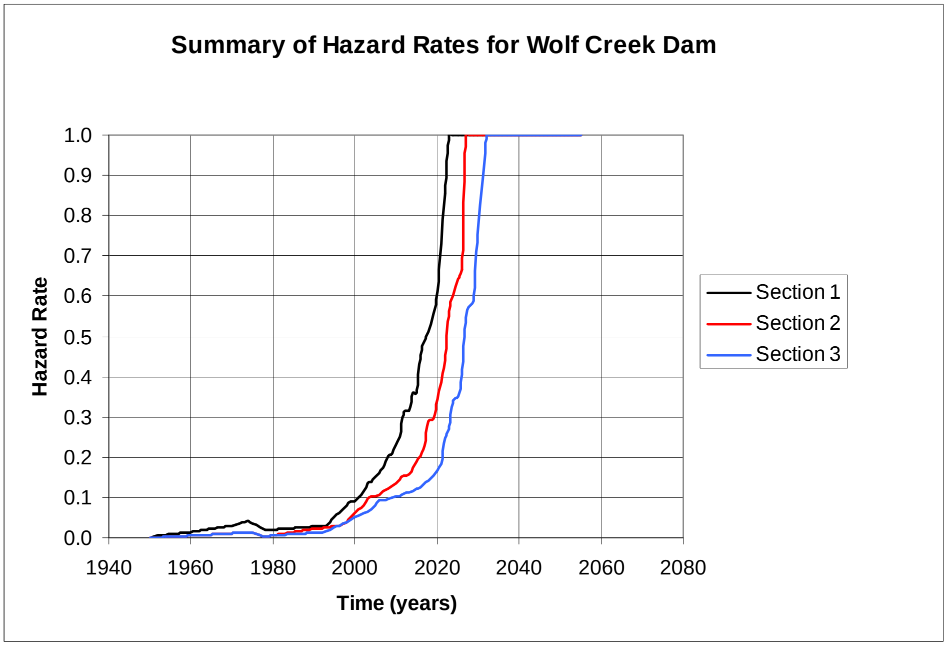


Figure C-21. Baseline hazard rates

(5) Two alternatives were selected for the permanent repairs of the dam: cutoff wall and grouting. The cumulative time-dependent reliability and hazard rates for these alternatives were determined using the results from the EOE. The expected life for each alternative was determined from the EOE. Grouting had an expected life of 12.5 years, and a new cutoff wall had an expected life of 50 years. Figures C-22 through C-25 show the time-dependent reliability and hazard rates for the two alternatives determined from the EOE.

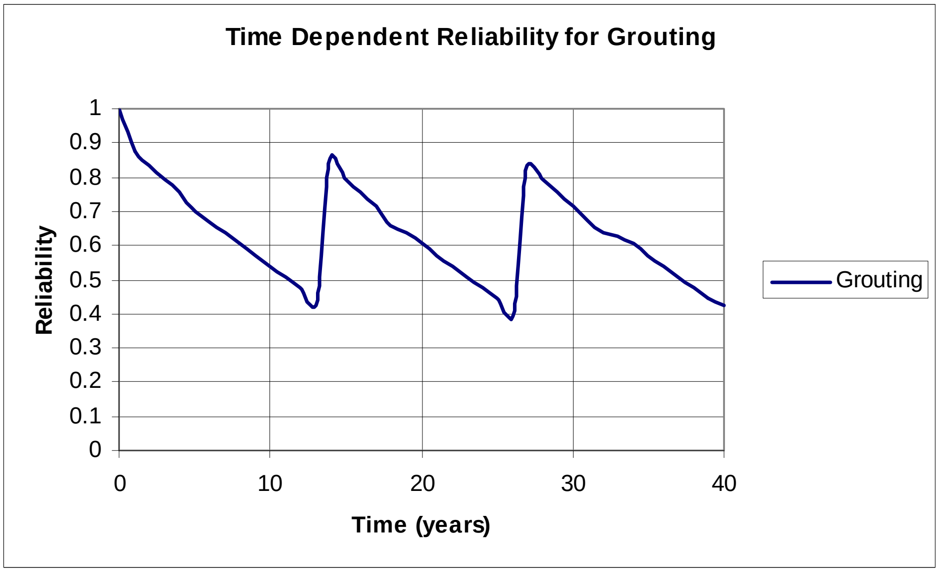


Figure C-22. Time-dependent reliability for grouting alternative

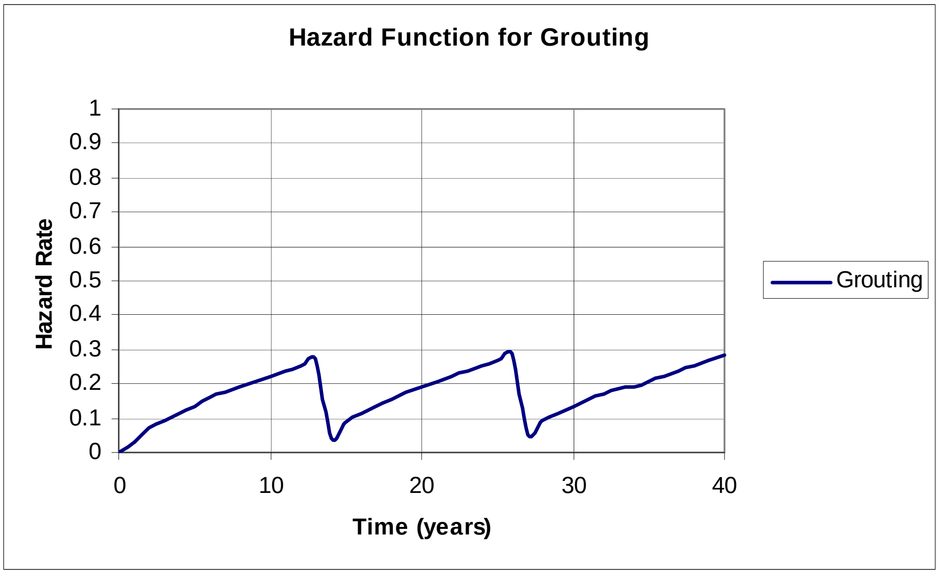


Figure C-23. Hazard rate for grouting alternative

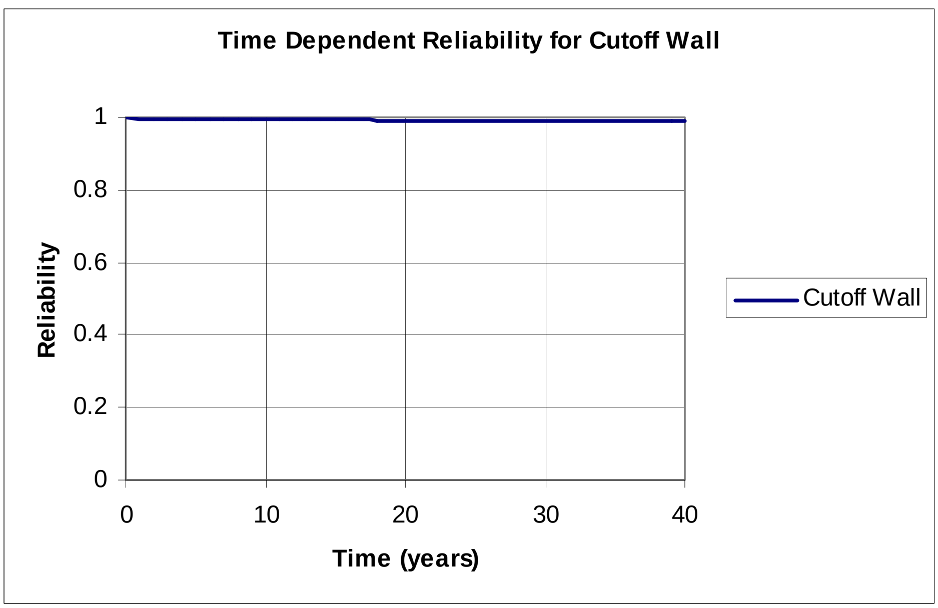


Figure C-24. Time-dependent reliability for cutoff wall alternative

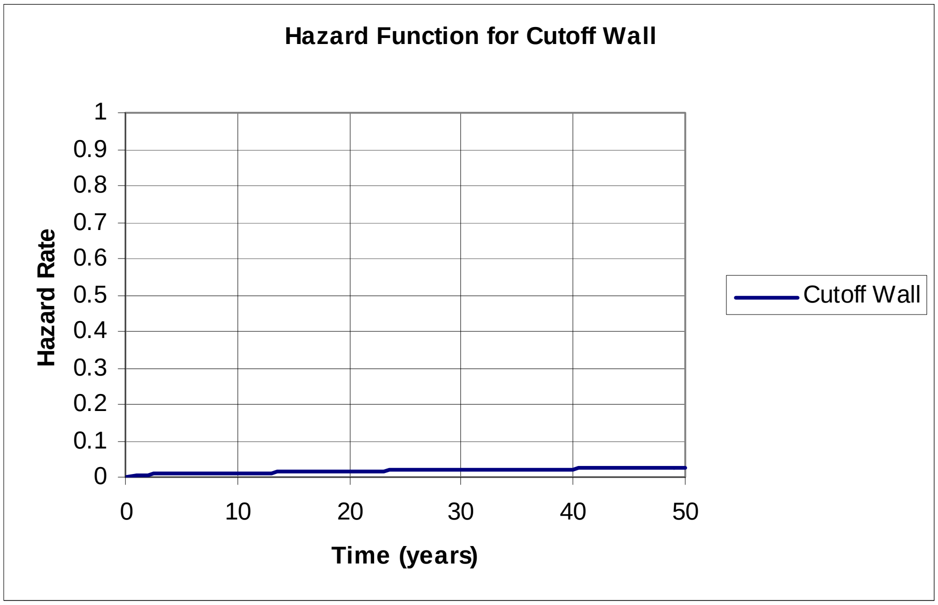


Figure C-25. Hazard rate for cutoff wall alternative

g. Sensitivities to MCS reliability model. The MCS results were examined to define the sensitivies that are calibrated into the reliability model. The sensitivities are ranked by correlation and regression to the year of unsatisfactory performance. They are also ranked in order of importance with values from −1 or +1 (sensitive) to 0 (negligible). Therefore for these analyses, the reliability model is most sensitive to the spatial variability factor that was to be expected due to differences in piezometric pressures throughout the foundation. Table C-12 shows the sensitivity to the results from the MCS.

Table C-12. Sensitivities to MCS

Random Variable	Sensitivity (regression and correlation)
Spatial Variability Factor	-0.88
Peak Intensity	-0.127
Duration	-0.114

C-16. Event Trees for Wolf Creek Dam. Event trees were developed for the baseline condition that incorporates the annual hazard rates from the reliability analysis and expert elicitation and the probabilities defined by the experts for the unsatisfactory events. Figure C-26 through C-28 show the baseline event tree. Similar event trees were developed for the grout and cutoff wall alternatives. The probabilities for these event trees were not developed as part of the elicitation but by the District Product Delivery Team with inputs and guidance from some the experts on the panel. Figures C-29 and C-30 show the event tress for the grout and cutoff wall alternatives, respectively.

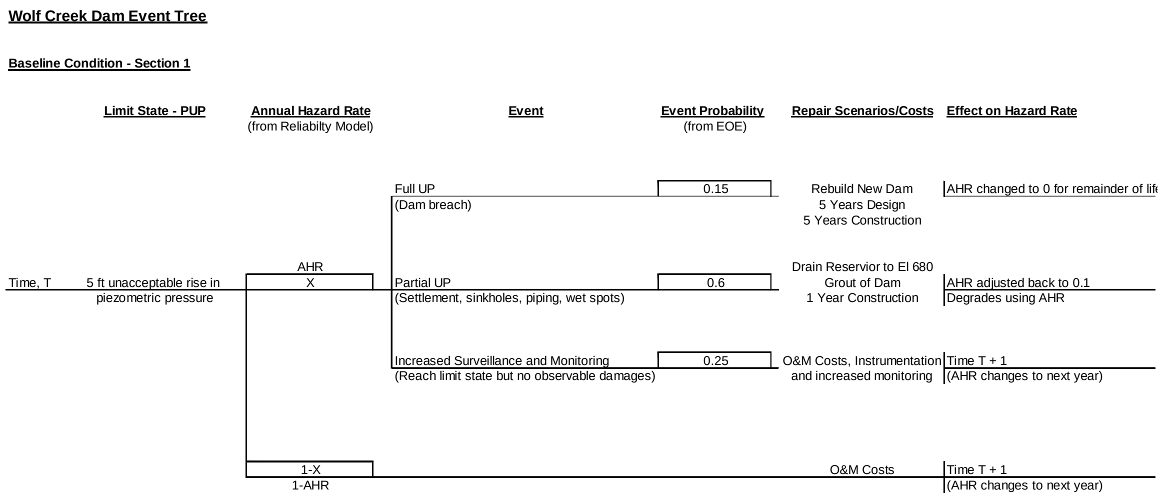


Figure C-26. Event tree for baseline condition – Section 1

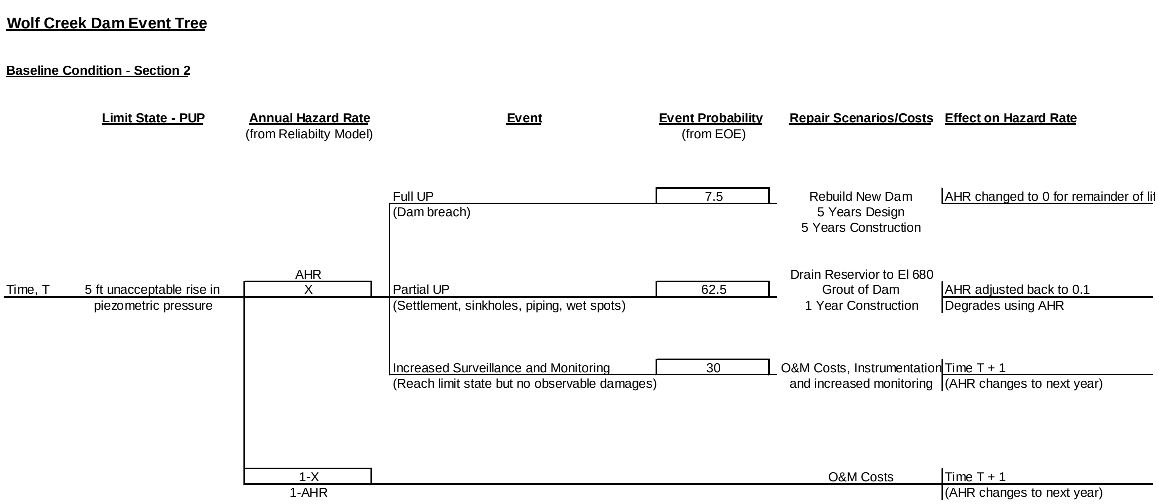


Figure C-27. Event tree for baseline condition – Section 2

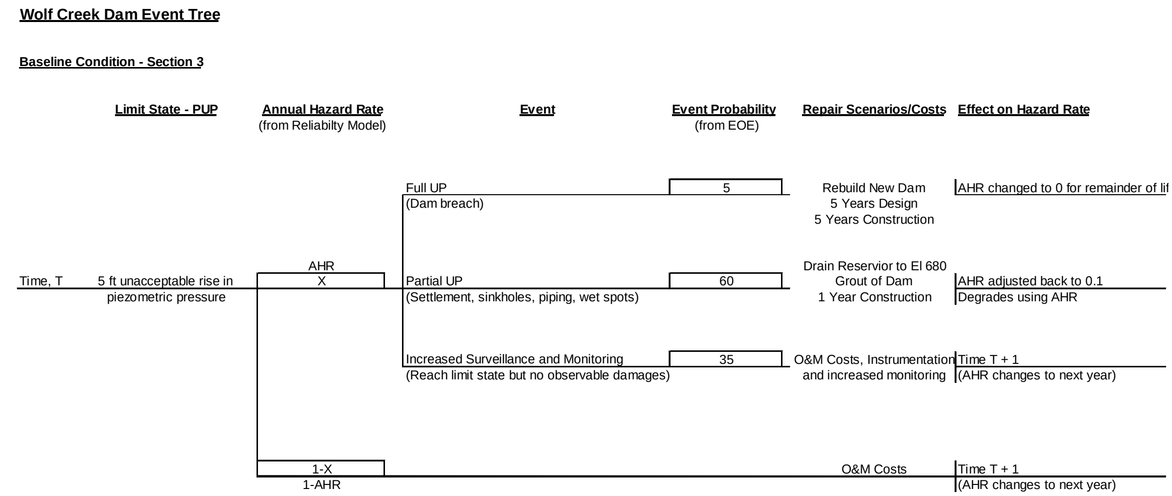


Figure C-28. Event tree for baseline condition – Section 3

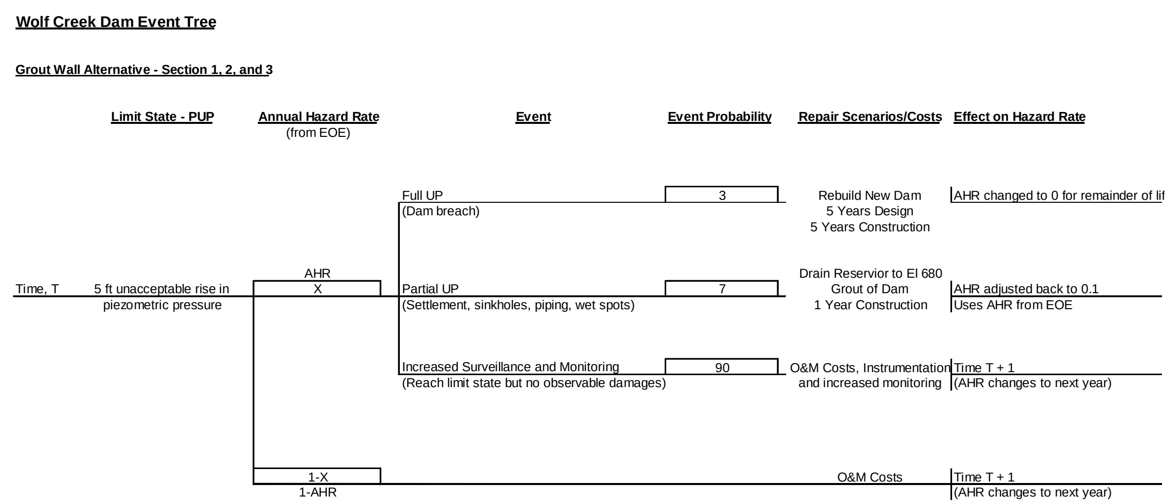


Figure C-29 Event tree for grout wall alternative

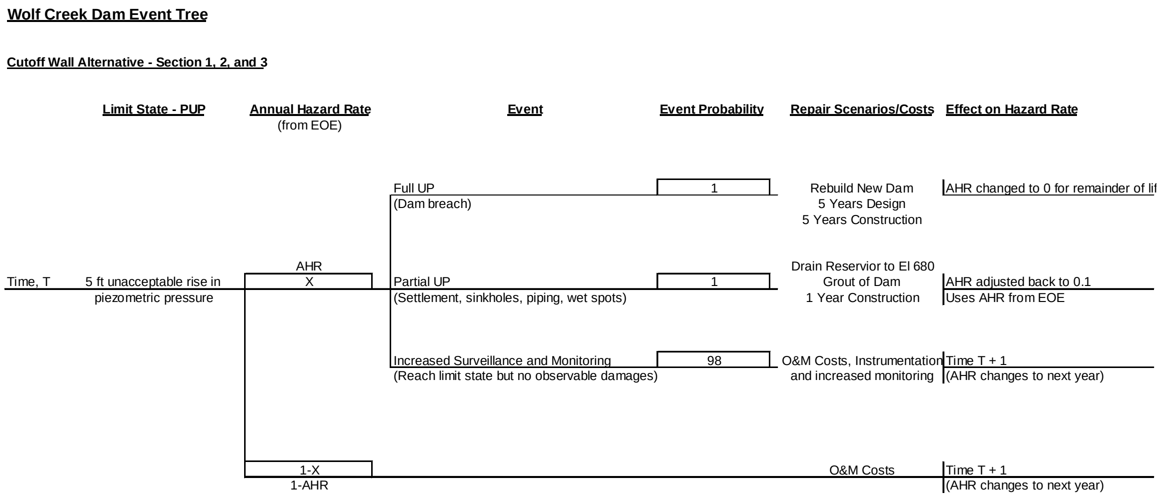


Figure C-30. Event tree for cutoff wall alternative

APPENDIX D
Hydropower Reliability

D-1. Reliability Methodology – Major Rehabilitation Evaluation Reports**.** This Appendix discusses the many facets of reliability of hydropower equipment in relatively broad terms. It goes into further detail by exploring a theoretical project and applying an analysis to that project. The overall engineering reliability analysis for Major Rehabilitation projects consists of four independent analyses that align with the four facets of the definition of hydropower reliability. These analyses determine the following equipment reliability factors: (a) forced outage experience and maintenance costs; (b) efficiency and capacity; (c) availability; and (d) dependability. The life-cycle costs of each facet are compiled for use in the economic analysis. Benefits for each alternative are calculated by subtracting the average annual equivalent life-cycle costs for the study alternatives from the average annual equivalent life-cycle costs for the base condition. The following paragraphs briefly summarize each facet of the reliability analysis.

a. Forced outage experience and maintenance costs

(1) A forced outage occurs when a power plant component fails to perform satisfactorily and causes an interruption in power production. A planned outage occurs when a unit is intentionally taken out of service to perform planned repairs, replacements, routine inspections, and rehabilitations.

(2) The life-cycle cost of equipment maintenance and repair includes labor and material costs as well as lost energy and capacity benefits associated with forced or planned outages. Therefore, reliability is a determining factor in estimating life-cycle costs. Decreased reliability may be represented by a large increase in labor and materials costs over time. Certainly, increasing maintenance costs and unit outage hours can both be used to indicate a need for equipment replacement or rehabilitation. Project records for the equipment in question can be used to document trends and as a basis to make future projections.

(3) Caution must be exercised when relying on maintenance costs alone as an indicator of reliability. Explanations of costs and maintenance efforts should be presented in the evaluation reports. Maintenance and repair records should be tabulated and charted to show the trends over the past few years. Projections for future years can be made using sound engineering judgment to extrapolate these costs and should be made for each of the alternatives being considered. Lost energy and capacity are discussed in c below under the topic of availability.

b. Efficiency and capacity. This portion of the reliability analysis can be applied to any piece of equipment that has an effect on the ability of the generating unit to produce electrical energy. However, this approach is applicable primarily to the turbines, generators, and transformers. Turbines will be used as an example in the following explanation.

(1) Part of the aging process of turbines is the development of cracks, corrosion, erosion, scaling, and cavitation damage. Damage is often repaired by welding, which induces stresses and changes the shape of the turbine components. This has the effect of lowering the efficiency of

the turbine. Thus, turbine performance degrades as a result of the aging process and can be exacerbated by repairs that are necessary to keep the turbine operational.

(2) The first step in quantifying the performance degradation is to determine the current and original levels of performance. Current efficiency and power output must be determined by field testing at settings similar to those used in the original field tests. The current performance must then be compared with the original level of performance to establish the amount of performance degradation that has occurred. Original levels of performance can be established from model tests and acceptance test data. It is important to fully investigate the calibrations and calculations of the data in order to truly compare the original and current performance.

(3) The information derived from this testing and analysis is provided as input to the hydroelectric power benefits analysis, which is discussed later in this Appendix. The benefits analysis estimates the power system production costs using a full range of unit availability that can be applied to the base case and each alternative.

c. Availability. Availability is the annual percentage of time that the generating equipment is available for power production. Records of availability are maintained by each project on a unit-by-unit basis. The current level of availability must be compared with previous data to establish the extent of degradation. Historical trends can be extrapolated to project future changes in the unit availability rate. Availability data are also used as input to the hydroelectric power benefits analysis.

d. Dependability.

(1) The final area of consideration concerning equipment reliability is dependability. Dependability is ascertained by a risk analysis that determines the probability that the equipment will not perform satisfactorily in any given year. The output from this risk analysis is used in the probabilistic life-cycle cost analysis. One way to graphically represent the probabilistic life-cycle cost model is with event trees, which were discussed in Chapter 6. A discussion of event tree models as used in hydropower studies is presented in Chapter 5. Two methods of probabilistic risk analysis are frequently used. The first method uses historical data and an evaluation of the condition of the equipment to determine a statistical distribution of age at retirement. This method is characterized by the use of reliability curves. The second method is similar to that used in structural evaluations. It extends the safety factor concept by using a probabilistic approach to determine a reliability index. The method that is most appropriate depends upon the type of equipment being evaluated and the specific situation.

(2) Hydropower equipment is typically operated until it fails or is retired for some other reason. Failure means that it ceases to function properly under the stresses applied. Replacement and refurbishment are both considered as constituting the effective retirement of a piece of equipment.

(3) Equipment may be retired for two primary reasons. The first major reason for equipment retirement is physical condition, which includes deterioration with time; wear from use, and failure in service. The second reason for retirement is related to functional situations,

which include inadequacy to perform required functions, potential for improvement (uprating), and obsolescence. These may occur from a change in environment, operating conditions, or load requirements.

(4) The first category, physical condition, is the primary reason that USACE developed the Major Rehabilitation Program. This program establishes a standardized method of considering and evaluating the deterioration and wear of equipment in an effort to optimize rehabilitation actions. Failures in service are generally not evaluated under the Major Maintenance and Rehabilitation Programs, but are funded through reprogramming Operation and Maintenance funds.

(5) Reliability is the key factor in determining whether there is a Federal interest in a proposed rehabilitation. As previously stated, if there are no reliability problems and the sole proposed project purpose is to improve output beyond the original design (improvement in functional situations), non-Federal funding is required to fund the project. However, Federal funding can be obtained for a rehabilitation that is reliability driven, even if there is increased output, so long as the increase is primarily incidental to the reliability work.

D-2. Risk Analysis Using Reliability Curves.

a. Historically, engineering judgment was used to predict remaining equipment life and determine the probability that a given hydroelectric generating unit will perform unsatisfactorily (fail). Since the early 1990s, USACE has made a concerted effort to improve on engineering judgment by using statistical methods and risk analysis to quantify these predictions in terms of probabilities. Methods of determining reliability are well established for many types of physical properties. A useful way of expressing reliability is the annual probability that a piece of equipment will fail to perform satisfactorily. The following discussions explain the terms and their applications used in this process.

b. The reliability of equipment can be considered a continuous variable with a probability density function (pdf) of *f*. This is defined further in Chapter 3. A pdf is a theoretical model for the frequency distribution of a population of measurements. In this case regarding reliability, the pdf is the rate of change of the equipment dependability. The following two equations are used in the development of reliability curves.

(1) Therefore, if the dependability of the equipment at age t is defined as:

$$D(t) = P(A > t) \tag{D-1}$$

where A is the age of the equipment at retirement and $P(A > t)$ is the probability that $A > .t$

Then the pdf of $D(t)$ is

$$f(t) = dD(t)/dt = D(t) \tag{D-2}$$

This simply states that the dependability of a piece of equipment is equal to the probability that the equipment is still functioning at age *t*.

(2) The hazard function H(t), or incremental failure rate associated with the random variable A, is given by:

$$H(t) = d \ln D(t)/dt \qquad \text{(D-3)}$$

That is, the incremental failure rate is equal to the probability of the equipment life being age *t* divided by the probability of the equipment surviving to age *t* in the first place. It is the probability that the failure occurs at age *t*.

c. The Corps has assembled databases of equipment histories in an attempt to establish the reliability characteristics of various categories of equipment. These databases include generator stator windings, turbines, and main transformers. The historical databases include many attributes such as year installed, age at failure, and rated capacity. The raw data have been compiled and reduced into annual summaries of exposures and failures.

d. The raw retirement data can be curve fitted using any number of means. One early method is the application of Iowa Curves developed in the 1930s by the Engineering Experiment Station at what was then Iowa State College (Winfrey 1935). Other distribution functions that may be used include normal, exponential, lognormal, and Weibull. The Weibull distribution is one of the most widely used reliability functions. It has been shown that the differences between the Iowa curves and a Weibull distribution are statistically insignificant. The Weibull distribution is much easier to adapt to computer analysis techniques and is preferred for that reason.

e. The practice in USACE evaluation reports has been to use the hazard function directly if the condition of the specific equipment in question is considered average. If, however, the equipment has exhibited signs of premature or accelerated deterioration, the hazard function has been adjusted to account for the evident higher probability of failure. Similarly, the hazard function can be modified to account for lower failure probabilities for equipment that is in better condition than average.

f. Figure D-1 is a plot of turbine raw data showing the number of units performing satisfactorily given years in service or age and the number of retirements at a given age. Figure D-2 shows these data plotted as a reliability curve, with percent in service as the ordinate. Figures D-3 and D-4 then show these data fitted to a Weibull curve and the resultant hazard function, respectively.

g. The factor being used by USACE to evaluate equipment condition and modify the frequency curve data is the condition index (CI). Condition assessment methods developed by USACE in 1993 for the Repair, Evaluation, Maintenance and Rehabilitation (REMR) program have been replaced by the methods developed by the Hydropower Asset Management Partnership (hydroAMP). The CI is a screening tool that provides a uniform method of evaluating conditions through testing and inspections. Inspection and test data are gathered and CI numbers assigned for each unit in accordance with the latest guidance. Equipment with CI values from 7 to 10 is considered to be in good condition. CI values in this range, when applied to the survivor curve, will tend to show increased reliability. Equipment with CI values in the midrange, from 3 to 7, is considered fair. The best prediction of this equipment's reliability is the statistical baseline data of similar equipment. Therefore, there is no cause to adjust the baseline frequency curve for equipment that falls into this category. Equipment with a CI below 3 is considered to be in poor condition. CI values below 3 will tend to increase the probability of failure, and the baseline frequency curve is adjusted accordingly. It is important to note that the methodology to be used in applying CIs to the reliability analysis is continuing to be developed. Current guidance should be sought by contacting HDC.

D-3. Hydropower Reliability Study Process for Major Rehabilitation Reports. A reliability analysis of hydropower plant equipment requires the following three basic steps: data collection and investigations, identification of specific reliability issues, and calculations and evaluation. Figures D-5 and D-6 show the basic steps in a reliability study and the typical hydropower equipment analyzed for reliability.

a. Step 1. The data collection and investigations are extensive and need to cover all aspects of the equipment design, use, history, and future demands that will be placed on the equipment. The data collected should include historical unit availability and operation, any equipment derating, accident reports, operation and maintenance records, equipment performance tests (original, interim, and current), periodic inspection reports, condition assessment reports, design and construction reports, the operation and maintenance manual, and turbine model test reports. During this step it is also important to identify the priorities and concerns of the project personnel. A thorough site investigation should be conducted by hydropower technical experts and should include equipment inspections and project personnel interviews.

b. Step 2. The data should be compiled and the primary equipment weaknesses and project concerns identified. The equipment condition should be quantified with the CI value as defined in the hydroAMP Condition Assessment Guides. In addition to the CIs, the equipment operation, demands, and maintenance practices should be considered in evaluating the reliability

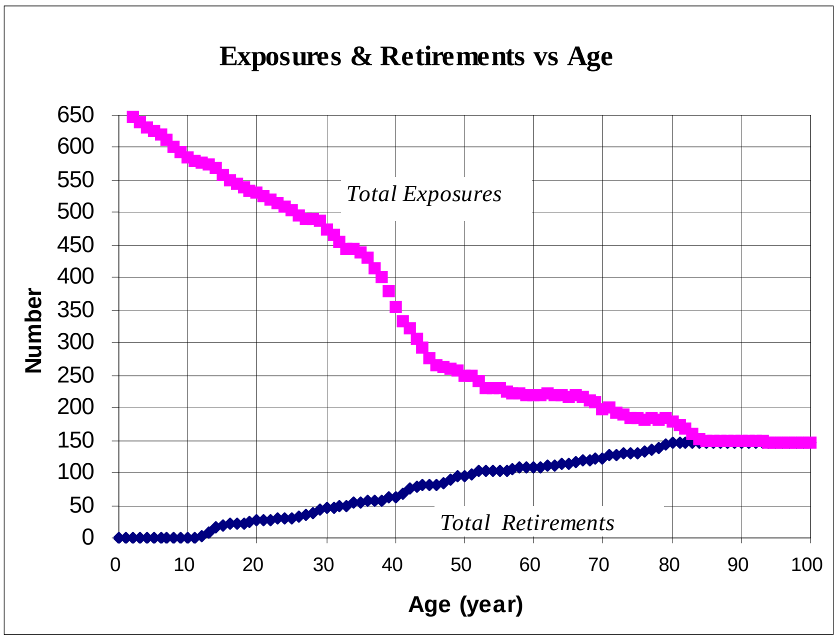


Figure D-1. Plot of turbine life data

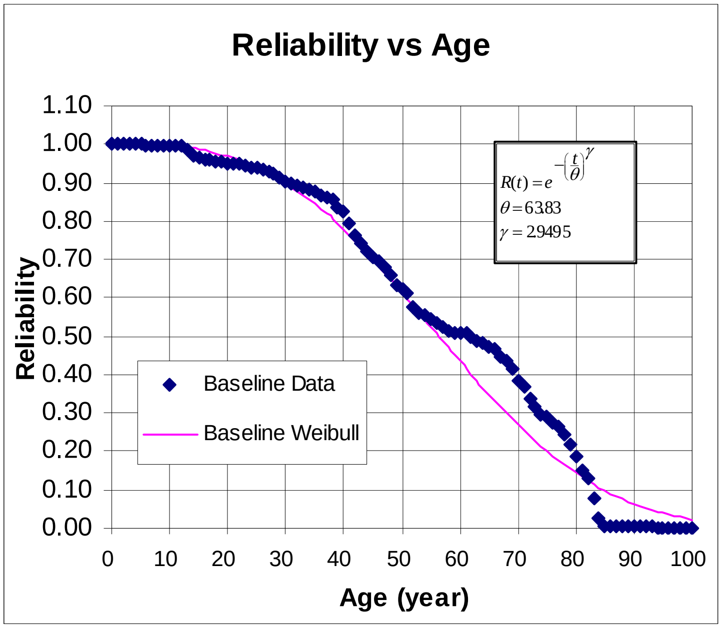


Figure D-2. Reliability curve

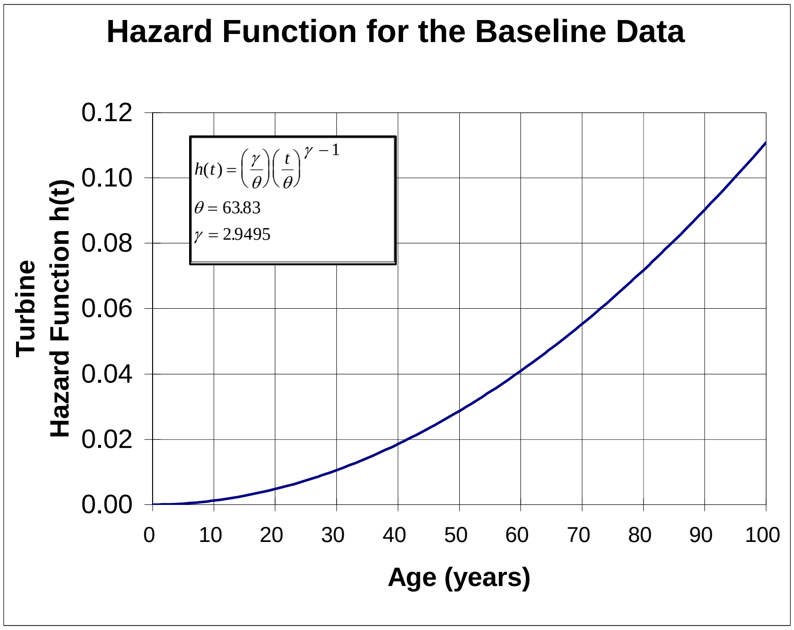


Figure D-3. Hazard function for turbine data

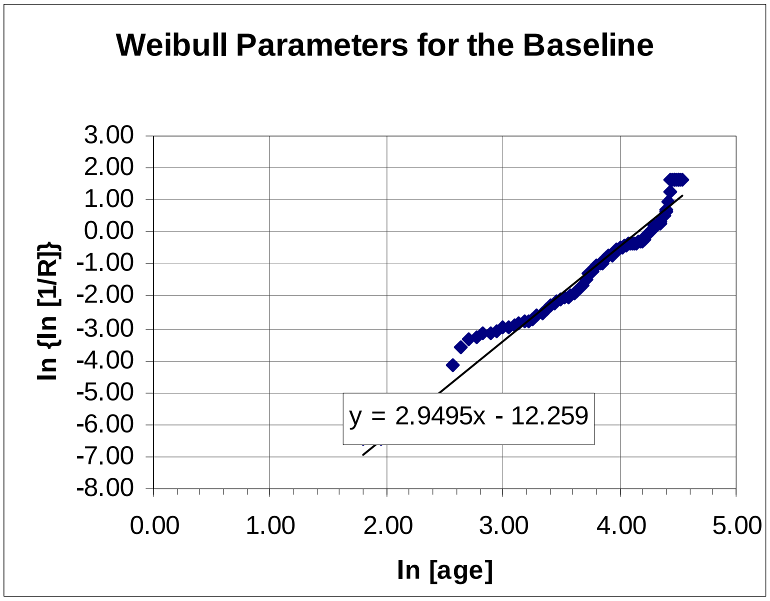


Figure D-4. Weibull parameters for turbine data

of the equipment. Experience and historical failure information of similar equipment can be utilized in the determination of the future reliability of the equipment.

c. Step 3. Once the condition of the equipment has been determined, the calculations and evaluation should be performed. For equipment with extensive life databases, such as generators, turbines and transformers, standard time-dependent reliability and hazard functions should be used. Any of the weaknesses and concerns identified in the previous steps should be fully explained and addressed separately if required.

D-4. Hydropower Benefits Calculations.

a. Background.

(1) In order to discuss the engineering reliability analysis, the hydroelectric power benefits analysis, and the economic modeling process, a brief overview of a fictional example rehabilitation project is warranted. The Chapman Hydroelectric Power Project consists of a single powerhouse with four Francis turbines, which were placed into service in 1947. The total rated capacity is 200 megawatts (MW). There are two three-phase generator step-up transformers, each serving two generating units. The plant is a storage project located in the southeast portion of the United States. There is a relatively small variation in lake elevation caused by seasonal flows and the need for flood protection. The storage in the lake is very large in relation to the flow in the river. Therefore, all of the flow into the lake, except during extreme flood events, either evaporates or passes through the turbines for power production. The spillway has rarely been used. The plant factor is 25 percent.

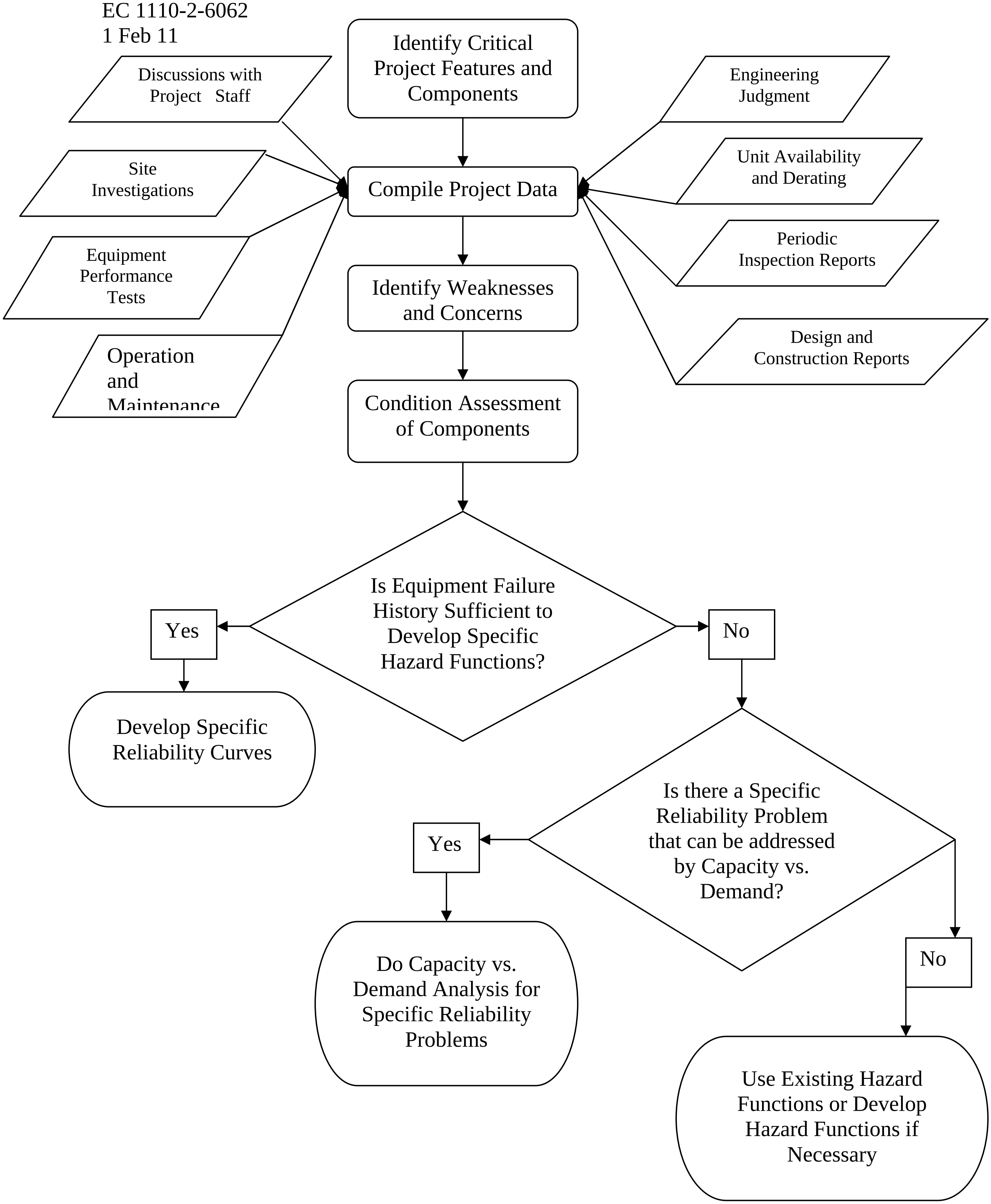


Figure D-5. Steps in reliability analysis of hydropower equipment

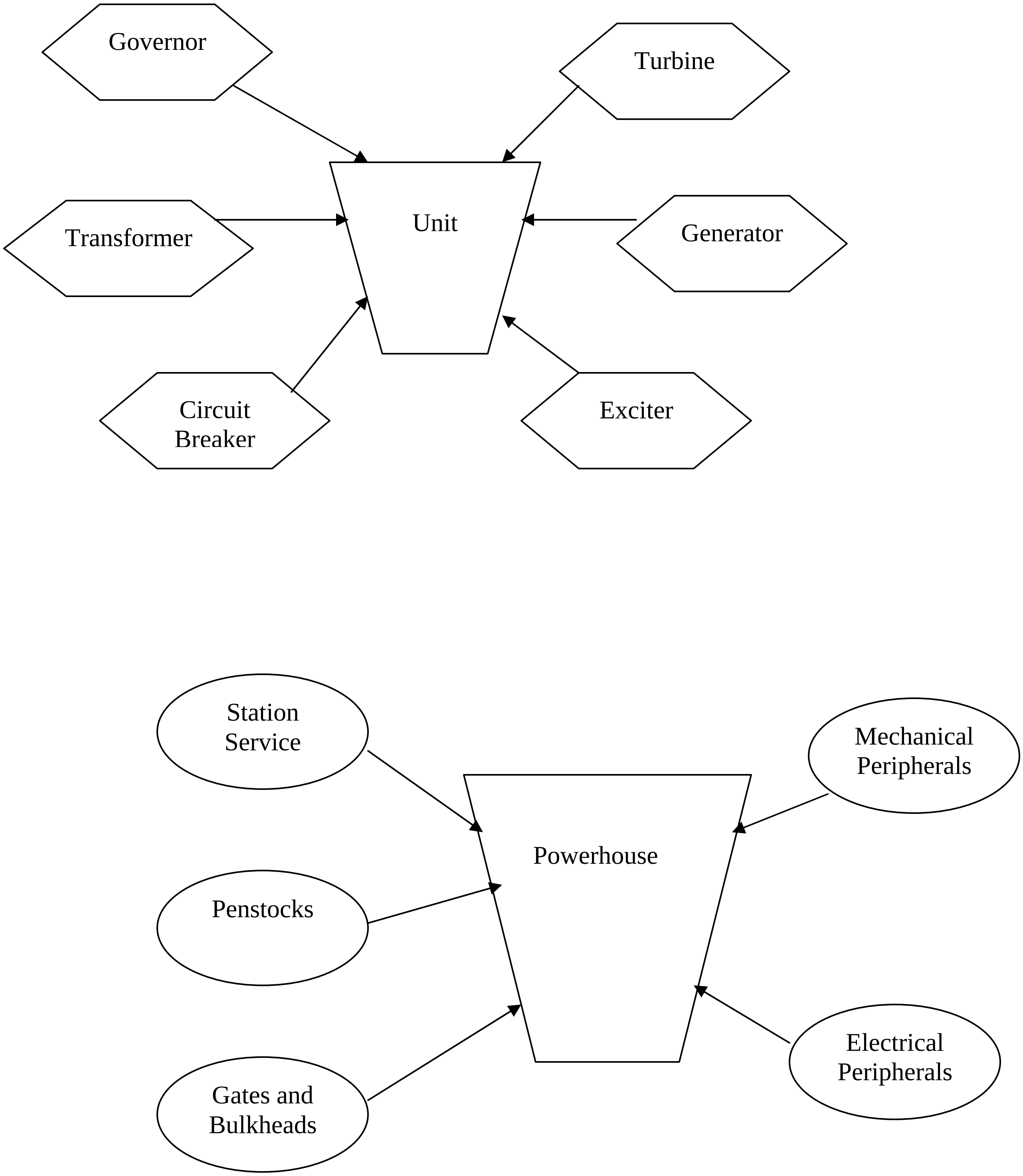


Figure D-6. Hydropower components for reliability

(2) Problems at the Chapman plant include turbine runner cracking, severe cavitation damage, generator coil degradation, and deterioration of the generator step-up transformers. Over the past 10 years, the turbine runners have exhibited increased cracking. On three separate occasions, pieces of the buckets have broken off. An enhanced maintenance program has been instituted. This program, which includes more frequent inspections and welding repair, has prevented further breakage. However, cracking and cavitation damage continue to increase at an accelerating rate. Deterioration of coil insulation has caused coil failures in three of the four generators in the last 2 years. Spare generator coils are not available, and there is no spare transformer. Unsatisfactory performance (failure) of either the generator or turbine runner will cause a unit outage. Unsatisfactory performance (failure) of a transformer will cause an outage of two units. Field testing has shown that the units have experienced an efficiency loss from their original condition. Average unit availability has also deteriorated from 95 percent 10 years ago, to 93 percent 5 years ago, and to 88 percent last year.

b. Hydropower economic benefits. Traditionally, the economic feasibility of a hydroelectric project is determined by comparing the cost of the hydroelectric project to the cost of the most likely thermal alternative. In other words, is the cost of constructing and operating a hydroelectric project less than the cost of obtaining the power from the thermal power plant(s) that would be the most likely source of that power if the hydroelectric plant were not built?

c. Energy and capacity benefits

(1) These two parameters define hydroelectric project output: energy (the total amount of generation over a given time period, expressed in megawatt-hours (MWh)); and capacity (the maximum amount of power that can be delivered at any given moment, expressed in megawatts).

(2) Energy benefits are measured by the cost of producing an equivalent amount of generation in the electrical power system with the hydroelectric plant replaced by the most likely thermal alternative. The energy benefits represent the variable costs associated with producing the alternative thermal generation, which are primarily fuel costs.

(3) Capacity benefits are measured as the cost of constructing an equivalent amount of thermal power plant capacity. The capacity benefits represent the capital costs and other fixed costs associated with the most likely thermal alternative.

d. Gain in output resulting from rehabilitation projects

(1) The first step in estimating benefits is to determine the gain in power output that will be realized from the proposed rehabilitation plan. Rehabilitation measures can be grouped into five categories, based on the way in which they increase hydroelectric power project output:

(a) Those that restore lost efficiency.

(b) Those that restore lost capacity.

(c) Those that restore lost availability.

(d) Those that increase the remaining service life (reduce the probability of retirement).

(e) Those that increase a plant's operating flexibility.

(2) Replacing the damaged and worn turbine runners is a measure that primarily restores lost efficiency. The major benefit of this type of rehabilitation is increased energy production. Increasing efficiency beyond that of the original equipment can be part of a rehabilitation project provided it is incidental to improving reliability. New turbine runners can also contribute to an increase in capacity.

(3) Rewinding the generators with state-of-the-art materials often permits the units to operate at higher output levels. This is an example of a capacity-increasing measure. The incremental costs of improving generator capacity beyond the original project level are often small and in many cases are incidental to the reliability improvement. A generator rewind often also makes a small contribution to an efficiency improvement of the unit.

(4) Replacing runners and rewinding the generators will also improve the unit availability and increase remaining service life. All of these benefits should be taken into consideration.

(5) Replacing a Kaplan Unit having an unreliable blade adjustment mechanism can improve the unit's response to changes in load and increase plant flexibility. Governor and exciter upgrades also contribute significantly to the improved flexibility of the powerhouse. Transformers and circuit breakers often limit capacity.

e. Project example. The easiest way to describe the benefit evaluation process is to walk through an example of a typical rehabilitation project. The proposed plan for the fictional Chapman project includes replacing all four worn turbine runners with new runners and rewinding the generator stators. When the original runners were new, the units had an average overall efficiency of 87 percent based upon field acceptance test reports. Recent tests have shown that, in their current condition, the overall average efficiency has dropped to 84 percent. With new runners, it is estimated that an average efficiency of 89 percent can be achieved. However, the rated capacity of the turbines remains the same. The rated capacity of the original generators is 50 MW. By rewinding the generator stator with state-of-the-art materials, the rated capacity of the generators can be increased to 60 MW, which now matches more closely the maximum capability of the turbines.

(1) Generation duration curve.

(a) To graphically display the amount of energy that can be gained from a rehabilitation measure, a generation duration curve will be used. The curve could be developed using historical records or output from a sequential streamflow routing model such as HEC-5.

(b) Table D-1 shows the output of the plant by unit, and Figure D-7 shows the annual generation duration curve for the example plant for the available period of streamflow record based on the existing condition of the plant. The duration curve in this case is based on weekly average streamflow data from a 60-year simulated operation study. Since this is a weekly average, it does not reflect the effect of peaking operation. This would require an hourly generation duration curve, which would have the same area under the curve, but would show more operation at or near full output and less operation at low output levels.

Table D-1. Plant Output

Unit	Unit Capacity MW	Cumulative Capacity MW	Unit Energy MWh	Cumulative Energy MWh
1	50	50	412,000	412,000
2	50	100	254,000	666,000
3	50	150	112,000	778,000
4	50	200	23,000	801,000

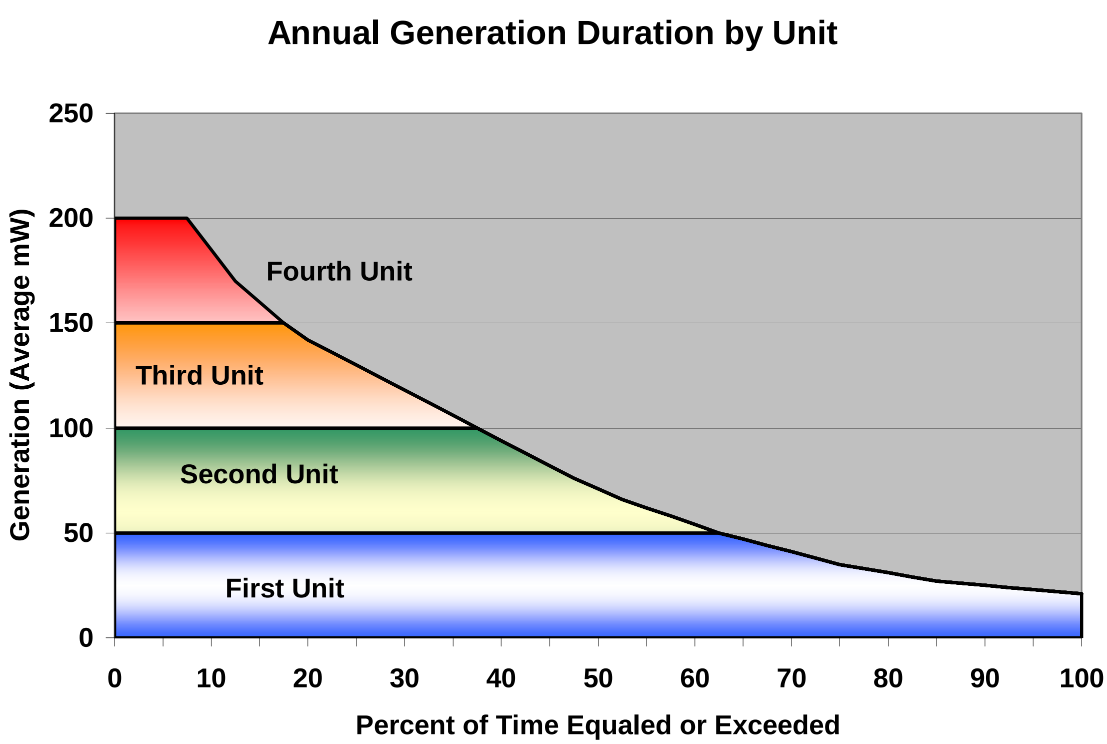


Figure D-7. Plant generation

(c) However, for purposes of estimating energy output, a curve based on average daily, weekly, or monthly output should be used rather than an hourly curve. The use of average values is necessary to measure the amount of energy that would otherwise be spilled if the rehabilitation measure were not implemented.

(d) The horizontal line at the top of the duration curve defines the maximum capacity of the plant, which in this case is 200 MW, the combined capacity of the four existing generators.

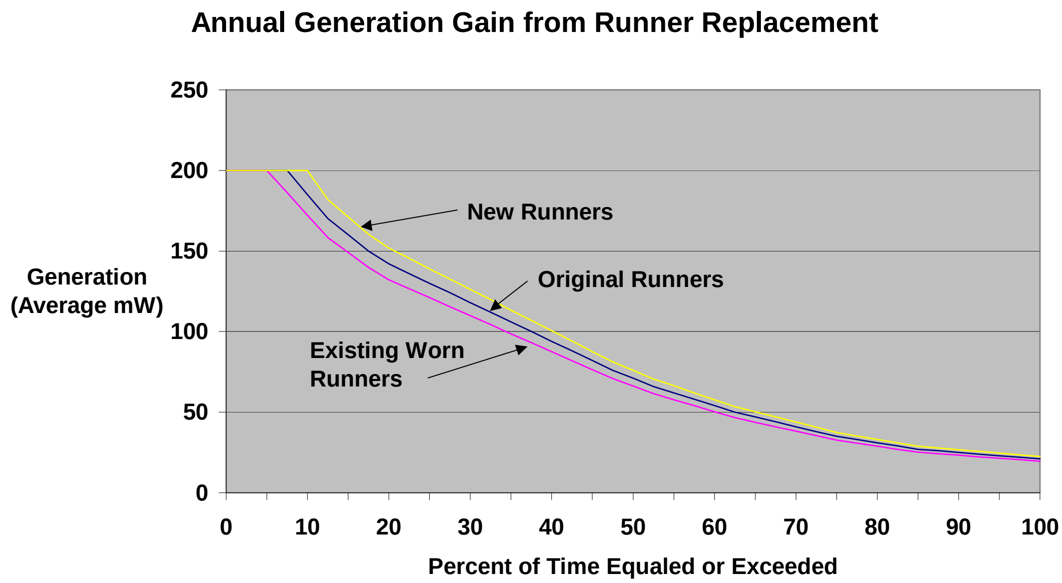


Figure D-8. Generation gain from runner replacement

(2) Energy gained by new runners. Figure D-8 graphically shows the gain in energy achieved by replacing the worn existing turbine runners with new state-of-the-art runners. The middle curve shows the output when the original runners were new (overall efficiency of 87 percent), and the lower curve shows the output with the original runners in their existing, worn condition (overall efficiency of 84 percent). The area between the middle and lower curves represents the amount of energy lost from deterioration of the existing turbine runners. The upper curve shows the output with new runners (overall efficiency of 89 percent). The area between the upper and middle curves represents the increase in energy creditable to the new runner design. The area between the upper and lower curves represents the amount of energy that can be obtained from a rehabilitation that includes new runners. The upper and middle curves are derived by applying efficiency adjustment factors to the existing case (Figure D-7) generation duration curve. In summary:

Energy output with original runners when new	828,000 MWh
Energy output with new runners	845,000 MWh
Energy output with existing original runners	801,000 MWh
Gain in energy output	44,000 MWh

Note that the capacity of the existing generators limits the total output of the powerhouse to a maximum of 200 MW. So, even if the new runners have the power to drive a generator with a greater megawatt capability, it would not be possible to take advantage of that capability without improving the generators and perhaps other components in the power train such as transformers and circuit breakers.

(3) Energy gained by new generator windings.

(a) Figure D-9 shows the gain in energy that is achieved by rewinding the stators with state-of-the-art design and insulation materials. The new designs and materials make it possible to place more copper in the windings, which increases the capacity of the generators. In this example, it is assumed that the new runners are in place and the

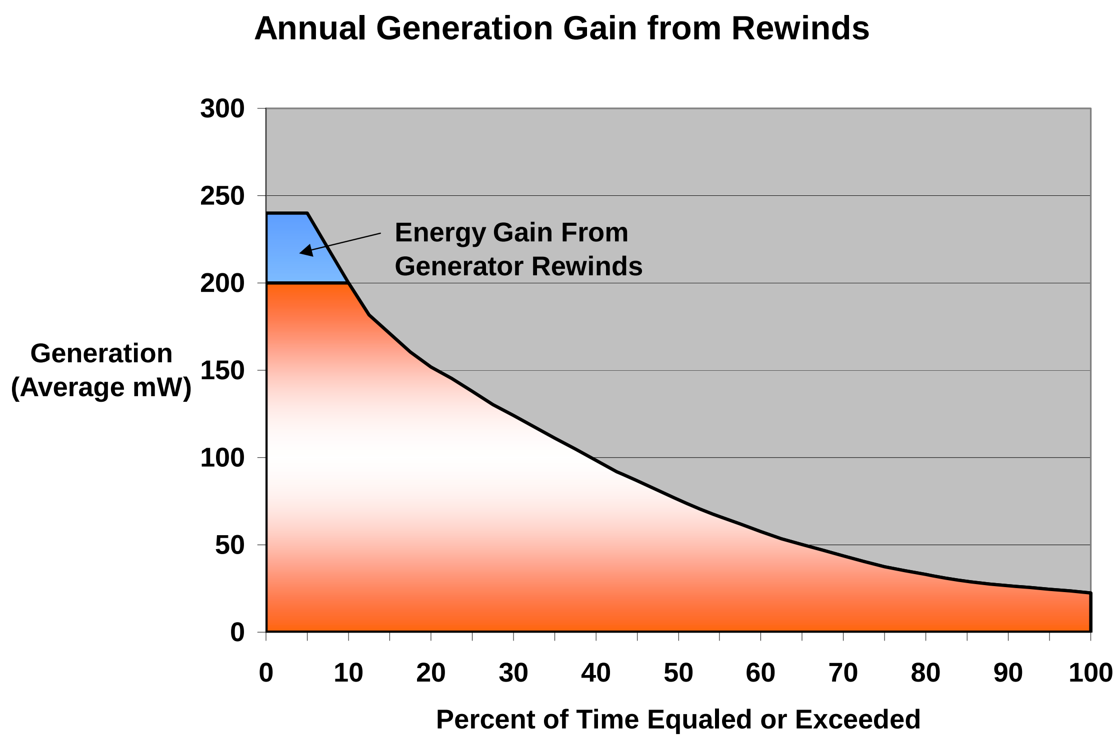


Figure D-9. Generation gain from rewind replacement

capacity of the generators can be increased to match the full output of the new runners. As a result, the capacity of the plant is increased to (4 units × 60 MW) = 240 MW.

(b) The upper limit (which truncates the duration curve) is increased from 200 MW to 240 MW. This allows the generation duration curve to be extended to the new upper limit. The upper, hatched area in Figure D-9 defines the gain in energy output realized from adding a generator rewind to the turbine runner replacement. In summary:

Energy output with existing generators	845,000 MWh
Energy output with generator rewind	861,000 MWh
Gain in energy output	16,000 MWh

(c) Note that rewinding the generators but retaining the existing turbines would also result in a gain in generation. The hatched area would be smaller, being defined by an extension of the lower curve in Figure D-8 rather than the upper curve. The gain in energy for this scenario would be 4,000 MWh instead of 16,000 Mwh.

(4) Energy Gained by Improved Availability.

(a) The key elements in the availability analysis are the assumptions that are used to define the base condition, or the "without major rehabilitation" condition. As stated in Chapter 1, the baseline condition is considered the low-level funding plan against which all other alternatives are compared. This represents the lowest level of planned future investments to keep the project serviceable. Should the project experience unsatisfactory performance (e.g., a generating unit outage), it is assumed that emergency funds will be made available to fix the problem. The timing, frequency, and consequences of system disruptions are all unknown and must be estimated for both with- and without-project conditions.

(b) Both the new runners and the generator rewind contribute to improved availability for the plant. Replacing additional old, failure-prone components with new components will also reduce the amount of time the generating units are out of service from forced outages. This in turn increases the amount of generation the plant can produce.

(c) Figure D-10 illustrates the concept of generation loss caused by forced outages. The shaded area represents the generation that would be lost if forced outages kept one unit out of service one third of the time. A rehabilitation measure that reduces the outage rate will reduce the size of this area, thus increasing energy output. The process of computing the loss in energy from outages is complex because it is necessary to account for the combined probability characteristics of multiple components (turbine runners and generator windings, for example), the combined probabilities of different numbers of units being out of service, and the fact that component reliability tends to decrease with age. In addition, it is necessary to account for the length of the outage and the cost of repair. In order to account for all of these factors, event tree models have been developed for estimating the energy benefits attributable to reliability improvements. This topic is discussed in more detail in Appendix C Chapter 5. However, for purposes of illustration, it is assumed that the combined gain in average annual energy benefits from improvement in the availability of the turbines and generators is $750,000.

(5) Computation of energy benefits.

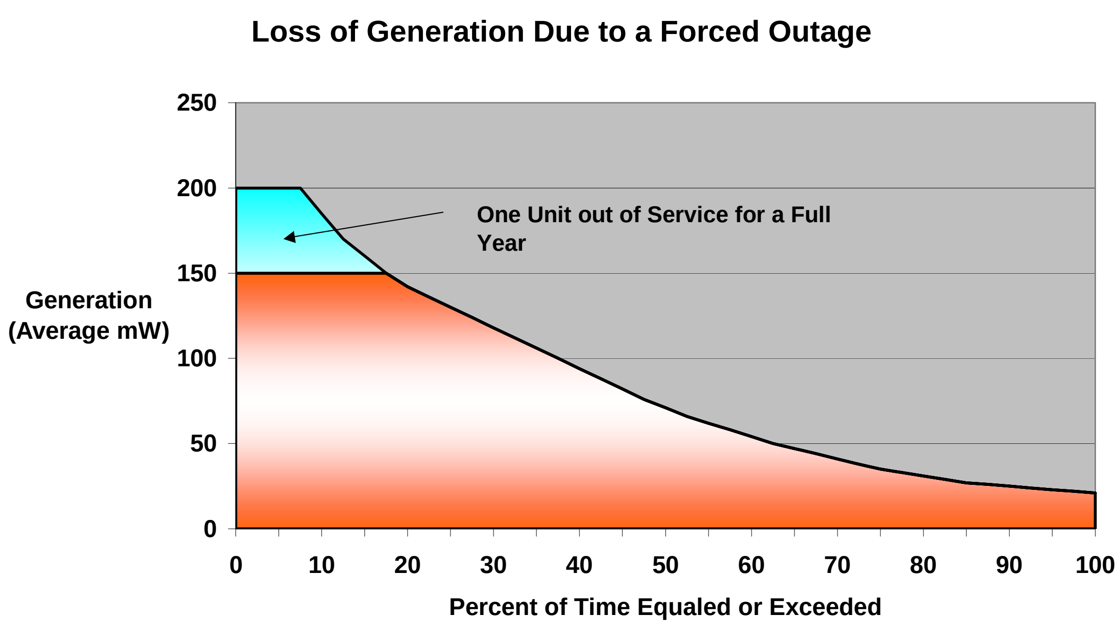


Figure D-10. Lost generation caused by force outage

(a) The average annual gain in energy benefits that accrues to a rehabilitation plan is computed by applying a unit energy value to the gain in energy creditable to that plan. Assuming an energy value of $35/MWh, the gain in energy benefits for the runner replacement and generator rewind measures would be as follows:

Runner replacement benefits	44,000 MWh @	$35 /MWh	=	$1,540,000
Generator rewind benefits	16,000 MWh @	$35 /MWh	=	$560,000
Availability benefits			=	$1,000,000
Total energy benefits			=	$3,100,000

(b) The unit energy values represent the energy cost associated with producing the generation with the most likely thermal alternative or alternatives. The energy value is based on the energy values provided from the Federal Energy Regulatory Commission for coal-fired steam and gas-fired combustion turbines and combined cycle plants. The value is based on weighted national values by fuel source and inclusion of estimated real fuel cost escalation. The Corps usually develops these values using a system production cost model, simulating the operation of a particular power system twice: once with the hydroelectric plant in the system, and once with the hydroelectric plant replaced with an equivalent number of megawatts of thermal capacity. The different nature of power systems, loads, and fuel costs throughout the nation requires site-specific evaluation for each Major Rehabilitation study.

(6) Dependable Capacity

(a) The dependable capacity of a hydroelectric power plant is an estimate of the amount of thermal generating capacity that would be required to carry the same amount of peak load in a power system as the hydroelectric power plant. It is intended to account for the variables that affect the amount of hydroelectric power capacity that can be used effectively in the system load, including the following:

- The variability in the maximum capacity that a hydroelectric power plant can deliver caused by variations in head.

- The variability in usable capacity caused by variations in the availability of streamflow, which in turn causes variations in the amount of energy available to support the capacity.

(b) A variety of different techniques are used to estimate dependable capacity. USACE presently uses the average availability method for projects that operate in thermal-based power systems and the critical month method for projects in hydroelectric-based power systems.

(c) For this example, the average availability method was used. Space does not permit a detailed discussion of the procedure, but, in brief, it involves computing the amount of capacity that can be supported with the available energy for each week in the peak demand months for each year in the hydroelectric period of record. The average capacity that can be supported over that period defines the project's dependable capacity.

(d) Supportable capacity is defined as the amount of capacity that can be supported for a specified number of hours per week. The required number of hours varies from project to project and from system to system, depending on the system resource mix and hourly load shape. A typical example might be 4 hours per day, 5 days per week (or 20 hours per week).

(e) Some examples will illustrate this concept. Taking the 200-MW example project and using the 20-hour/week criterion, assume that in a particular month, sufficient streamflow is available to produce 5,000 MWh/week. Applying the 20-hour criteron, (5,000 MWh)/(20 hours/week) = 250 MW could theoretically be supported. However, the installed capacity of the plant is only 200 MW, so the supportable capacity for that month is limited to 200 MW. However, if the generators were rewound to 240 MW, the supportable capacity would increase to 240 MW. Assume that in another month, 3,000 MWh/week can be generated. In this month, only (3,000 Mwh)/(20 hours/week) = 150 MW can be supported, either with or without the rewind.

(7) Dependable capacity gained by new runners. The amount of energy produced in each week will be increased because of the higher runner efficiency. In some weeks, sufficient energy is already available to support the existing capacity. But in some of the lower flow weeks, this additional energy will permit more capacity to be supported. The average gain in capacity over all of the peak demand weeks in the period of record defines the gain in dependable capacity attributable to the new runners. Typically, this gain is relatively small for runner replacement. For this example, the new runners increase the dependable capacity from 185 MW to 190 MW (compared with an installed capacity of 200 MW).

(8) Dependable capacity gained by generator rewind. The generator rewind increases the maximum capacity of the plant. This in turn permits more capacity to be supported in those weeks where more energy is available than is needed to support the existing capacity. In the example case, the generator capacity is increased 40 MW, and the dependable capacity increases from 190 MW to 226 MW (compared with the new installed capacity of 240 MW).

(9) Computation of capacity benefits.

(a) The average annual gain in capacity benefits that accrues to a rehabilitation plan is computed by applying a unit capacity value to the gain in dependable capacity creditable to the plan. Assuming a capacity value of $180/kW-year, the gain in capacity benefits for the runner replacement and rewind measures would be:

Runner replacement benefits	5000 kW	@	$180 /kW	=	$900,000
Generator rewind benefits	36000 kW	@	$180 /kW	=	$6,480,000
Total capacity benefits				=	$7,380,000

(b) The unit capacity value represents the investment cost associated with delivering the replacement capacity with the most likely thermal alternatives. The capacity value is based on a mix of coal-fired steam plants, gas-fired combined cycle plants, and gas-fired combustion turbine plants, weighted by the Energy Information Administration's projections of future capacity additions nationwide. The Corps usually obtains these values from the Federal Energy Regulatory Commission, although they can be developed from data published by other sources.

(10) Increase in capacity benefits realized by increased availability.

(a) Although improving the electrical-mechanical reliability of hydroelectric generating units clearly increases the peak load-carrying capability of the units, it has proven difficult to quantitatively estimate the benefits realized from this gain. However, a relationship of generating unit average availability to effective load-carrying capability has been developed.

$$ELCC = C - \{M * \ln [(1 - R) + (R * eC/M)]\} \qquad \text{(D-4)}$$

where

ELCC = effective load-carrying capability of unit, MW
C = rated capacity of that unit, MW
M = system characteristic (typically, 3 percent of system capacity), MW
R = unit's equivalent forced outage rate, percent
e = 2.718

(b) Using this equation, ELCCs can be developed for each unit size and each forced outage rate associated with the different proposed rehabilitation measures or plans. Ratios of ELCC are developed by dividing the ELCC for a proposed measure by the ELCC for the capacity value developed by the Federal Energy Regulatory Commission. The ratios of ELCC can then be applied to the unit capacity values to estimate the gain in capacity benefits that apply to the proposed rehabilitation measure or plan. The capacity values, as developed by the Federal Energy Regulatory Commission, already include a factor that accounts for the average availability of a typical hydropower unit compared with that of a thermal generating unit. For the example study, assume that the $180/kW-year Federal Energy Regulatory Commission capacity value is based on a typical hydro unit availability of 93 percent, and the availability of the units in the existing condition is only 91 percent. Assume that the turbine runner replacement increases the availability to 93 percent, and adding the generator rewind increases it further to 95 percent. These availability values would be obtained from reliability studies.

(c) While these capacity value adjustments are small, they apply to the entire dependable capacity of the plant, so they result in substantial benefits. Table D-2 summarizes the calculation of the increase in capacity unit values based on the ELCC ratios. The table also provides total benefits attributable to both the increases in dependable capacity and increases in reliability.

Table D-2. Increase in Capacity Benefits

Case	Dependable Capacity. MW	Capacity Value $/kW-year	Total Benefits $1,000	Incremental Benefits $1,000
Existing	185	175	32,375	-
New Runners	190	180	34,200	1,825
+ Rewind	226	185	41,810	7,610

(d) Subtracting out the previously calculated benefits for the gains in dependable capacity, the gain in capacity benefits as a result of improved reliability is $925,000 ($1,825,000 - $900,000) for the new runners alone, and $230,000 ($7,610,000 - $7,380,000) for the combined plan of new runners plus rewind.

(11) Benefits from increasing remaining service life. The hydroelectric power benefits accruing from replacing equipment before it fails are limited to the differences in unit outage times. A planned rehabilitation program will substantially reduce the time that a unit is out of service compared with experiencing a major equipment failure in service.

(12) Flexibility Benefits.

(a) An additional area in which benefits might accrue to power plant rehabilitation is in the area of flexibility—the ability of a power plant to come on-line quickly and to respond rapidly to changes in load. An example might be a plant with aging Kaplan Units that have deteriorated to the point at which the turbine blade adjustment mechanism can no longer be operated reliably. In such cases, the blades may have to be welded in a fixed position so that they lose their ability to follow load. Rehabilitating the units would restore the load following capability to the blade operation . This in turn would generate some benefits that could be used to help support the investment in the rehabilitation work.

(b) Unfortunately, while it is widely agreed that flexibility benefits are an important hydroelectric project output, it is difficult to quantify such benefits. The Electrical Power Research Institute (EPRI) and others have done some work in this area, but so far an accepted procedure for quantifying flexibility benefits does not exist. However, if a proposed rehabilitation project does improve a project's flexibility, this should at least be addressed qualitatively in the rehabilitation project feasibility report.

(13) Total gain in benefits. The total annual power benefits attributable to the combined runner replacement/stator rewind plan would be as follows:

Energy benefits	$3,100,000
Capacity benefits	$7,610,000
Total benefits	$10,710,000

(14) Last-added test.

(a) Standard economic practice requires that separable components of multicomponent plans be incrementally justified on a last-added basis. For instance, the example rehabilitation plan includes two components. For the plan to be economically feasible, both runner replacement and generator rewind would have to be individually justified on a last-added basis. This assures that the plan with the highest net National Economic Development benefits (i.e., benefits-costs) is identified, as called for in ER 1105-2-100.

(b) Last-added analysis refers to a comparison of the incremental benefits gained by one component of a plan on a last-added basis, with the incremental costs of including that component in the plan. The last-added benefits for a component are determined by deducting the benefits of a plan with that component excluded from the benefits of the plan with all components included. Again referring to the example, the last-added benefits of the generator rewind would be the benefits of the total plan minus the benefits of runner replacement alone. A similar process would be followed to determine the incremental benefits of the runner replacement. Once incremental benefits are determined, they are compared to the incremental costs of including the component. If the incremental benefits exceed the incremental costs, the component is justified on a last-added basis.

(15) Analysis tools. Various computer analysis tools have been developed to assist in the evaluation of Major Rehabilitation and Operations and Maintenance repair projects. One example is the Hydroelectric Power REPAIR model developed through the U. S. Army Corps of Engineers Institute of Water Resources. Several districts have developed other life-cycle risk models for evaluation of Major Rehabilitation projects. These models are conceptually described in the following section. Assistance in evaluation of the potential project benefits can be received from the Hydroelectric Design Center, associated administratively with the U.S. Army Engineer District, Portland.

D-5. <u>Hydropower Economic Modeling using Event Trees</u>.

a. Introduction. Engineering reliability analysis coupled with traditional engineering judgment offers a more effective and objective way of identifying future events and consequences than engineering judgment alone. Economic studies including risk and uncertainty analysis provide decision makers with a more comprehensive picture of the range and likelihood of the economic consequences of any particular project proposal. This paragraph provides guidance for the use of event trees and incorporation of engineering reliability and hydropower benefits studies in the economic analysis of hydropower rehabilitation projects.

b. Event trees for hydropower projects.

(1) An event tree is simply a diagram of the potential events and outcomes that could occur to a given component or group of components in one time period or in subsequent time periods. Event tree diagrams are used to identify possible occurrences of satisfactory or unsatisfactory performance and their consequences, given specific events.

(2) For example, during any time period a mechanical/electrical component such as a turbine runner or a generator may be fully operational, out of service from a prior period, or exhibiting unsatisfactory performance. These possible events or branches of the tree identify all of the pathways that may occur during each time period. The event tree is developed for each component to be evaluated for each time period of the analysis.

(3) The consequences of each pathway are also identified. The consequences may consist of changes in system hydropower generation costs caused by unit outages or changes in unit

generating efficiencies, increases or decreases in operation and maintenance costs, or changes in repair or replacement costs.

(4) The event tree also facilitates coordination of the engineering reliability analysis with the economic evaluation and assists in developing a clear definition of the without-project condition. The without-project condition is a description and evaluation of the consequences that are expected to occur during the period of analysis in the absence of an intervention of some sort. The use of event trees requires planners and project engineers to graphically depict what is expected to happen to various components in any given time period. This process helps clarify critical elements and possible solutions. It highlights any apparent data gaps and serves as a road map for building the economic model.

c. The economic model.

(1) In its simplest form, the economic model that is developed for hydropower rehabilitation analysis can be described as a basic accounting spreadsheet. In its final evolution it can be very large and complex. The Institute for Water Resources has developed a PC-based program that will handle hydropower economic modeling requirements faster and easier than creating custom models using spreadsheet-based software. The basic spreadsheet model is described in d-h below because it is relatively easily understood.

(2) The spreadsheet model is first created to mirror a single unit event tree diagram for the without-project condition. This incorporates both the physical and economic consequences of possible events and the results of the engineering reliability analysis for each component. A Monte Carlo simulation procedure is used to calculate variance and expected values.

(3) Monte Carlo simulation is a process in which random numbers are generated from a range of possible values, usually between zero and one, with any number in the range having an equal likelihood of occurrence. Each random value is input into the spreadsheet, and the spreadsheet is recalculated to arrive at an associated outcome. Each random trial or iteration of the spreadsheet represents an independent what-if game. By generating hundreds, or in some cases, thousands, of what-if games, Monte Carlo sampling will generate the input distribution and the entire range of potential outcomes. Monte Carlo simulation is described in detail in Chapter 3.

d. Model requirements. Basic functional requirements are established for the model. These requirements allow for flexibility in the analysis, incorporation of basic assumptions, and the ability to change parameters as needed. Some of these requirements are described in (1)-(10) below:

(1) The model must accurately reflect the without-project condition. The without-project condition establishes a base condition from which all other alternatives are to be evaluated.

(2) The model must be flexible enough to evaluate a full range of alternatives. Alternatives considered in the analysis often include enhanced maintenance, use of spare parts, a

full array of rehabilitation scenarios, and, subsequently, appropriate timing of any rehabilitation strategy.

(3) The model must distinguish between individual operating components, economic consequences of various alternatives, and the timing of events.

(4) The model must be able to incorporate incremental analysis of each unit and its separable components.

(5) The model must account for a project life (35 years is recommended) and for near-term events that could impact future rehabilitation strategies.

(6) The model must be able to incorporate the engineering reliability and risk and uncertainty analysis results for each time period and each functional component under evaluation.

(7) For each alternative, the model must be able to incorporate routine and non-routine Operation and Maintenance costs for each component over the period of analysis.

(8) The model must be able to account for changes in generating unit efficiencies with various rehabilitation scenarios.

(9) The model must be able to incorporate the consequences of events and repair/rehabilitation scenarios in terms of changes in hydropower system benefits and alternative construction costs. Each alternative produces different hydropower outputs, system benefits, and Operation and Maintenance costs.

(10) The model must be able to accommodate other economic calculations such as present valuation and amortization of costs and incorporation of interest during construction.

e. Model operating characteristics.

(1) For each alternative considered, the spreadsheet is modified to simulate the specific engineering, operational, and economic consequences relative to the alternative. Monte Carlo simulation techniques are incorporated into the spreadsheet. This approach uses random number generation to compute an expected result given a combination of probabilities and events. The program sums the results of multiple iterations of the simulation and produces expected values and variance. Each simulation should include a minimum of 300 iterations. Up to 5,000 iterations may need to be computed in some simulations.

(2) Separate simulations are conducted for the without-project and for each alternative considered in the analysis. Simulations for the Chapman Powerhouse example include: rehabilitation of one to four turbines; rehabilitation of one to four generators; rehabilitation of one or two transformers; and all reasonable combinations of these alternatives. The appropriate timing for rehabilitation should also be evaluated. Another alternative that should be considered is one that uses an enhanced maintenance strategy. In many cases this may already be

implemented in the without-project condition. A spare parts alternative should also be considered where reasonable. Incremental analysis of the alternatives should be performed to allow for optimization of the number of components to be rehabilitated.

(3) This process permits consideration of the physical condition of the individual components and the potential sequencing of repairs. Each simulated outage incorporates consequences, in the form of cost resulting from increased frequency of repair, increased maintenance effort, and having to resort to more expensive means of energy production (hydropower benefits calculations).

f. Incorporation of physical and economic consequences.

(1) Several columns of the spreadsheet model are needed to account for the results of the engineering reliability analysis. The engineering reliability analysis establishes the probability of unsatisfactory performance for each component for current and future conditions. This probability, over time, is inserted for each year in the modeling sequence. Current conditions and probabilities of unsatisfactory performance vary for each individual turbine, generator, and transformer.

(2) Within each iteration, a random number is generated for each component in a given time period. Based on the probability of unsatisfactory performance in that time period, the unit either incurs an outage or continues to operate. For example, if the probability of unsatisfactory performance for turbine unit number one in the first year of the study is 2.19 percent, then any random number generated between 0 and 1 that is less than 0.0219 will cause an outage to occur; any number greater than 0.0219 will indicate that the unit is still available for operation. If the unit remains operational, then the probability of unsatisfactory performance in the next time period increases. A random number is generated for each successive time period, and the consequences are recorded. Should a unit incur an outage, depending on the alternative being modeled and the type of outage, the unit will either be repaired or rehabilitated. If the unit is repaired, then the probability of unsatisfactory performance in each successive time period continues to increase. If a unit is rehabilitated, then the probability of unsatisfactory performance is returned to a new condition as the equipment is considered restored.

g. Types of unsatisfactory performance.

(1) The analysis can include multiple types of unsatisfactory performance with different probabilities of occurrence. Continuing with the Chapman hydropower example, one type of unsatisfactory performance would be a catastrophic outage. For a generator stator, this type of outage could occur if a significant number of coils failed, and a rewind was the only possible repair. The second type of outage is less debilitating. This outage mode consists of a repairable coil failure.

(2) For each type of unsatisfactory performance, outage times and costs for repair are computed. For the Chapman generators, a repairable coil failure may cause an outage of 1 month at an estimated repair cost of $50,000. For a catastrophic outage, the Chapman unit is estimated to be out of service for a period of 36 months at a repair cost of $2,500,000.

(3) For each alternative considered, routine annual O&M costs are also estimated. Under existing conditions, the Chapman turbine units are dewatered, inspected, and repaired once every 6 months. If a unit is rehabilitated, inspections are assumed to significantly decrease in frequency with a resulting reduction in O&M costs. The time associated with inspections and routine maintenance must also be accounted for in each iteration.

(4) Subsequent columns in the spreadsheet sum all unit outages for a given year. Subroutines should be incorporated in the model to prevent double counting of outage time if two interrelated components are out concurrently. If the unit is considered to be out of service in excess of 12 months, outage times must be carried over into the next time period.

(5) Additional columns are required to sum O&M, repair, and rehabilitation costs for any given year. Again, subroutines must be used to prevent double counting of normal maintenance costs if the unit is considered to be out of service for an extended period of time.

(6) Columns must be added to the spreadsheet to account for specific alternatives and conditions. For example, in an alternative that includes a planned sequence of rehabilitation, if a unit outage occurs within a year of the planned rehabilitation, the unit would not be repaired or returned to service prior to the rehabilitation. This would be the proper sequence of events assuming that it is more cost effective to leave the unit off-line than to return it to service and then shut it down later for a permanent rehabilitation.

(7) Another column needs to account for whether or not existing spare parts are available for a given unit. In any simulation, if a unit with spare parts experiences a catastrophic outage, the existing spare parts should be assumed to be put into service.

h. 5-8 Cost of replacement power – hydropower benefits.

(1) The without-project condition must first be modeled as discussed previously. This produces an annual system production cost assuming all four of the Chapman powerhouse units are available for production. Next, the without-project condition is modeled assuming that only three units are available. Subsequent scenarios are run removing a unit at a time until all four units are considered to be off-line. This process results in construction of a system production cost curve assuming a full range of unit availability in the without-project condition. This production cost curve is then used in the economic model to quantify the production cost consequences of unit availability for any potential combination of randomly generated unit outages.

(2) Additional system production cost curves are constructed to assist in modeling the alternative rehabilitation and repair scenarios. As units are rehabilitated, unit efficiencies increase, hydropower production increases, and system production costs decrease.

(3) Once all of the separate cost curves and previously described input values are established, the without-project and all of the with-project conditions are simulated. For each iteration of a simulation, potential outages are generated; O&M, repair, and rehabilitation costs are calculated; and system production costs are estimated. The economic consequences for each

alternative over the period of analysis are summed and described in present value terms. Net benefits are computed for each alternative, and the plan that maximizes net benefits is identified. Additional statistics are generated to describe the range and distribution of values for each component.

D-6. Major Rehabilitation Example.

a. Background. There may be a clear failure history of specific equipment which warrants a unique reliability analysis. For example, the generators at The Dalles powerhouse demonstrated a specific failure mode (coil failure from turn-to-turn faults) and a severe decline in reliability after fifteen years of age. Hazard functions were developed specifically for these generators since the historical data of the fourteen units, for which there had been thirteen coil failures, constituted a statistically sufficient database. Specific equipment hazard functions can be developed by adjusting the standard hazard functions equipment demonstrates accelerated degradation, such as was found at the Buford powerhouse. A reliability study of the Buford turbines found that the condition of the main unit turbines was typical for their age, but the station service unit showed severe degradation (USACE 1996). Therefore, it was reasonable to use the standard hazard functions for the main units and adjust these functions to reflect the poor state of the station service unit. If the equipment has a specific reliability problem but lacks a statistically significant base of data, a capacity versus demand analysis may be done. This approach was appropriate for the reliability analysis of the Walter F. George powerhouse. The turbines were found to have two areas of specific concern, the turbine shaft sleeve for the packing gland and turbine hub cavitation. Both of these concerns involved loss of material in an area that could not be repaired without total disassembly of the turbine. After a detailed study, the turbine hub cavitation was determined not to be a significant problem, but the shaft sleeve was wearing at a rate that would have required a complete unit disassembly fairly early in the study period.

b. Examples.

D-7. Condition Assessments and Indexes.

a. Background

(1) Successful strategic planning for capital investments in existing hydropower facilities requires consideration and balancing of many factors, including the risks and consequences of equipment failure. The hydropower community has long recognized the importance of assessing the condition of existing equipment in order to make informed and sound business decisions for the replacement of that equipment. Early attempts to develop condition assessment tools, however, were not completely successful.

(2) One formal approach for assessing the condition of hydroelectric equipment existed in the Corps of Engineers' (REMR) Research Program undertaken in the early 1990s. Prior to the REMR guidance there were no industry standards for consistent evaluation of the numerous tests and inspections that were performed on hydropower equipment. REMR was intended to provide guidance and a standard methodology for making condition assessments, and to

consolidate the assessments into a uniform format. However, there was dissatisfaction with the REMR program for the following reasons:

(a) The equipment evaluation processes tended to be unwieldy, requiring too many tests, inspections, and measurements.

(b) The evaluation procedures and results were not properly validated and calibrated.

(c) There was no convenient and consistent method to capture, retrieve, and utilize the data being collected.

(3) As a result of this feedback, there were many discussions within the Corps concerning the need to continue the development and improvement of the REMR guidance. Concurrent with these discussions within the Corps of Engineers, other industry leaders were wrestling with this same issue. In 2002, the Bureau of Reclamation, Hydro-Québec, the Corps of Engineers and the Bonneville Power Administration formed the Hydropower Asset Management Partnership (hydroAMP) primarily out of the need to establish an objective, consistent, valid and inexpensive condition assessment process.

b. Principles. The hydroAMP-working group agreed that the Condition Assessment process should:

(1) Be guided and managed through a collective team effort.

(2) Be designed for fair and equitable application to all hydroelectric projects.

(3) Result in an objective and repeatable assessment of the major equipment and critical systems in the generation power train.

(4) Start small and grow over time as experience is gained.

(5) Be streamlined to minimize the time and expense required for testing, evaluating data, developing conclusions, and record keeping.

(6) Rely on existing O&M records and routine inspections and tests applied at regular intervals.

(7) Be technically sufficient although not necessarily “perfect.”

(8) Be field-tested and assessed periodically

.

(9) Be open to continuous improvement.

(10) Be adaptable for different users, purposes, and situations.

c. Intended users. The hydroAMP Condition Assessment process was designed to work with existing maintenance, planning, budgeting, and decision-making structures. It is intended to serve multiple users who may have distinct roles and responsibilities for hydropower asset management. Typical users include the following:

(1) On-site plant staff. The individuals who work with the equipment on a daily basis and will have a direct role in performing the equipment condition assessments. The information provided by the on-site staff is the foundation of the asset management process. Plant staff will typically:

(a) Perform Tier 1 equipment condition assessments.

(b) Record the data in the maintenance management system.

(c) Collaborate with technical specialists conducting Tier 2 tests or inspections.

(d) Use equipment condition information to manage their operation and maintenance activities.

(2) Plant or facility managers. These individuals will use the hydroAMP processes to:

(a) Support plant maintenance, rehabilitation, or replacement decisions.

(b) Evaluate equipment condition assessment data and trends, in conjunction with other business decisions factors to recommend additional analyses for certain components or systems.

(3) Technical staff. This group consists of engineers, economists, environmentalists, biologists, and other staff and technical specialists who are responsible for preparing detailed evaluations and justifications for larger, more complex decision packages. They use equipment assessments to:

(a) Support economic analyses.

(b) Support risk analyses.

(c) Support regional or multiple project analyses.

(4) Asset managers. These individuals may use the hydroAMP processes to:

(a) Prioritize competing investment needs.

(b) Analyze various business cases or justifications for investment decisions.

(c) Support decisions that consider tradeoffs between competing needs or conflicting requirements.

d. HydroAMP condition assessment process.

(1) The methodology outlined in the hydroAMP condition assessment guides is divided into two tiers or levels. A Tier 1 assessment relies on test and inspection results that are normally obtained by on-site staff as part of routine operation and maintenance or by examination of existing data. Generally, the following condition indicators are used to evaluate the equipment condition:

(a) Physical Inspection

(b) Tests and Measurements

(c) Operation & Maintenance History

(d) Age or Number of Operations

(2) Numerical scores are assigned to each condition indicator using the guidelines provided. The scoring criteria may refer to conditions such as “normal” and “degraded.” These relative terms are intended to reflect industry-accepted levels for equipment of similar design, construction, or age operating in a similar environment, or to baseline or previous (acceptable) levels. In some situations, determination of the condition indicator scores is subjective and must rely on the experience and opinions of personnel conducting the maintenance or inspection.

(3) Weighting factors are applied to the condition indicator scores, which are then summed to compute the Condition Index. Weighting factors account for the fact that certain condition indicators reflect the actual equipment condition more than other indicators. The weighting factors also normalize the Condition Index to a score between 0 and 10 and result in a rating system as shown in the following table:

Table D-3. Condition Index Ratings of Equipment

Condition Index (CI)	
$7 \leq CI \leq 10$	Good
$3 \leq CI < 7$	Fair
$0 \leq CI < 3$	Poor

(4) An additional stand-alone indicator is used to denote the quality of the information available for scoring the condition indicators. Although reasonable efforts should be made to perform the Tier 1 tests and inspections, in some cases, data may be missing, out-of-date, or of questionable integrity. Any of these situations could affect the accuracy of the associated condition indicator scores as well as the validity of the overall Condition Index. Given the potential impact of poor or missing data, a Data Quality Indicator is assigned a value of 0, 4, 7, or 10 as a means of recording confidence in the final Condition Index. The more current and complete the assessment information, the higher the rating for this indicator.

(5) Tier 1 tests may indicate abnormal conditions that must be addressed immediately or that can be resolved via standard corrective maintenance solutions. To the extent that Tier 1 tests lead to immediate corrective actions being taken, appropriate adjustments to the condition indicator scores should be made and the new results used to compute a revised Condition Index. The Data Quality Indicator score may also be updated to reflect the availability of additional information or test data.

(6) As a result of the Tier 1 assessment, additional information may be required to improve the accuracy and reliability of the Tier 1 Condition Index or to evaluate the need for more extensive maintenance, rehabilitation, or equipment replacement. Therefore, each condition assessment guide describes a "toolbox" of Tier 2 inspections, tests, and measurements that may be performed, depending on the specific issue or problem being pursued. A Tier 2 assessment is considered non-routine. Tier 2 inspections, tests, and measurements generally require specialized equipment or expertise, may be intrusive, or may require an outage to perform.

(7) For certain types of equipment, there are many tests that can provide information about different aspects of component condition. The choice of which tests to apply should be made based on the Tier 1 assessment as well as information obtained via review of O&M history, physical inspection, other test results, and company standards. Results of the Tier 2 analysis may either increase or decrease the Condition Index. In some cases, more than one Tier 2 test may be available to detect or confirm a single defect or state of deterioration. It is important to avoid over-adjusting the Condition Index simply because two or more tests confirm or disprove the same suspected problem. In the event that multiple tests are performed to assess the same problem or concern, the test with the largest adjustment would normally be used to recalculate the Condition Index. Since the Tier 2 tests are being performed by and/or coordinated with knowledgeable technical staff, the decision as to which test is more significant and how different tests overlap is left to the experts.

(8) An adjustment to the Data Quality Indicator score may be appropriate if additional information or test results were obtained during the Tier 2 assessment.

D-8. <u>Condition-Based Risk Mapping</u>.

a. Introduction.

(1) Appendix C Chapter 7 covers the Condition Assessment process for hydropower equipment. These assessments are a critical factor for planning maintenance and capital investments. But there are other factors including cost, consequence, and risk.

(2) A cost-effectiveness analysis of rehabilitating a specific piece of equipment at a hydro plant is a complex undertaking. Benefit is derived from actions that lead to efficiency improvements (reduction in losses) and cost savings, or that avoid lost revenues. For reliability investments, the first two areas of benefit can be easily determined. The third area is more difficult to calculate. In the case of lost revenues, benefit is derived only from making the piece of equipment in question more reliable than the next least-reliable piece of equipment of the

power train. Making this calculation and determining how to allocate the benefit among multiple investments in the power train is complicated.

(3) A cost-effectiveness analysis on an entire generating unit or plant can more easily be done. An analyst can compare the expected future investments on all equipment components of a generating unit to the future avoided lost-revenue benefits to determine whether the investments would be cost effective overall. If so, investments when needed for individual equipment components can be deemed cost effective, as long as they are consistent with the expected future investments that were analyzed. This is the Major Rehabilitation process described in Appendix C Chapters 1 thru 6.

(4) However, there are other, relatively simple and low cost processes for rating equipment condition and prioritizing investments using risk-management tools. These processes can be applied to a single powerhouse component in-lieu of the comprehensive Major Rehabilitation approach.

b. Risk mapping.

(1) For relatively expensive pieces of equipment where there are several investment alternatives for improving reliability, several factors that relate to the consequence of undertaking or not undertaking a rehabilitation action can be considered when setting priorities. These factors, which may not be appropriate to every situation, are as follows:

(a) Total Cost: Cost to repair or replace the equipment, including engineering, administration, and commissioning costs.

(b) Current-Year Cost: Portion of investment cost incurred in the current year.

(c) Incremental Annual Maintenance: The increase or decrease in maintenance provided by the investment dollars.

(d) Achievability: Ability to undertake the project in the immediate timeframe.

(e) Phase of the Project: Study (S), engineering (E), procurement (P), or construction (C).

(f) Condition Index.

(g) Marginal Value of Generation: Annual value attributed to the piece of equipment.

(h) Total Outage Duration: The length of time (in years) to restore a unit to service after failure, including both the time required to procure and to install equipment.

(i) Revenue at Risk: Marginal value of generation times the total outage duration.

(j) Risk Map Score: A score (explained below) that measures the relative risk for a piece of equipment given its condition rating and the consequence associated with its failure.

(k) Other Business Factors: Factors important to the decision, including environmental, legal, and safety considerations.

(2) The risk map shown in Table D-4 is a tool that helps prioritize a portfolio of investment needs. As stated above, it measures the relative risk of a piece of equipment given its condition rating and the consequence associated with its failure. The consequence used here is loss of revenue, but it could include other business factors.

(3) The map is laid out in a grid, with the condition index on one axis and the consequence of failure on the other. Values in the grid are the sum of the corresponding beta values for condition and consequence shown in Table D-5. The values in this table are for illustration only and can be changed to meet the specific needs of the analysis.

Table D-4. Risk Map

Condition Index		Low		Medium-Low		Medium		Medium-High		High	
Poor	0 to 0.9	11	12	13	14	15	16	17	18	19	20
Poor	1 to 1.9	10	11	12	13	14	15	16	17	18	19
Poor	2 to 2.9	9	10	11	12	13	14	15	16	17	18
Fair	3 to 3.9	7	8	9	10	11	12	13	14	15	16
Fair	4 to 4.9	6	7	8	9	10	11	12	13	14	15
Fair	5 to 5.9	5	6	7	8	9	10	11	12	13	14
Fair	6 to 6.9	4	5	6	7	8	9	10	11	12	13
Good	7 to 7.9	3	4	5	6	7	8	9	10	11	12
Good	8 to 8.9	2	3	4	5	6	7	8	9	10	11
Good	9 to 10	1	2	3	4	5	6	7	8	9	10

Consequence

Risk Level Results (Map #)
High 17 - 20
Medium-High 13 - 16
Medium 9 - 12
Medium-Low 5 - 8
Low 1 - 4

Table D-5. Beta Tables

Condition Lookup	
CI	Beta
0	10
1	9
2	8
3	6
4	5
5	4
6	3
7	2
8	1
9	0

Consequence Lookup Type 2 Analysis	
Rev @ Risk	Beta
-	1
100	2
200	3
400	4
600	5
800	6
1,000	7
2,000	8
3,000	9
4,000	10

(4) In this example, the value of the condition beta is inversely related and proportional to the Condition Index. It takes on the assumption that the likelihood of failure increases directly with the decrease in condition. Generally, this is a valid argument although the exact relationship is probably not linear for most pieces of equipment.

(5) The value of the consequence beta is related in an approximated exponential increasing fashion. The consequences for this example are limited to revenue at risk. This is the amount of revenue that would be lost if the component were to fail and need to be rehabilitated in an emergency rather than planned program.

(6) This method could be used to compare totally unrelated potential work items at different powerhouses. Two simple examples follow.

(a) Example 1.

(1) Two transformers are being considered for replacement. Both have a Condition Index of 4, which gives a beta value of 5. Transformer A. is at a relatively small powerhouse and the revenue at risk is $1 million, giving a beta of 7. Transformer B is part of a bank of single-phase transformers that will take out several generating units if it fails. The revenue at risk is in excess of $4 million giving a beta of 10.

Transformer	CI beta	Cons beta	Total beta	Cost to Rehab
A	5	7	12	$800,000
B	5	10	15	$2,000,000

(2) Using the risk map, Transformer B is in a medium high-risk category while Transformer A is in a medium risk category. All else being equal, it would be logical to rehabilitate Transformer B first. However, what if these two work items were fairly far down the overall priority list for the year, and there was only $1 million left in the budget for the year? In this case, it may make sense to rehabilitate Transformer A in the current year and hope that Transformer B would high enough on the list the following year that the $2 million was available. The other alternative is to present a case to management in hopes of increasing the budget in the current year.

(b) Example 2.

(1) Suppose a powerhouse has six circuit breakers that are in poor condition with CI's averaging 2. Replacement is the only viable option with the total cost being $3 million for all six breakers. If one breaker fails, the revenue at risk is $400,000. If two breakers fail in close succession, the revenue at risk is $1 million. The same powerhouse also has a generator that is showing some signs of age with a CI of 6. The cost of rehabilitation is also $3 million but the revenue at risk is $2 million.

Work Item	CI beta	Cons beta	Total beta	Cost to Rehab
Breakers	8	4	12	$3,000,000
Generator	3	8	11	$3,000,000

In this case, both work items fall into the medium risk category. However, the relatively poor condition of the breakers should make it the more viable project. The poor condition increases the likelihood of multiple breaker failures that would increase the consequences. Conversely, even though the consequences of a generator failure are greater than the breaker failure, the relatively fair condition of the generator makes the potential for failure lower.