REMR TECHNICAL NOTE CS-ES-2.4

ESTIMATION OF RELIABILITY OF CIVIL WORKS STEEL STRUCTURES

PURPOSE: To develop a general procedure for the estimation of the reliability of Civil Works steel structures.

BACKGROUND: The reliability of a structure is the probability that, when operating under stated environmental conditions, the structure will perform its intended function adequately for a specified interval of time (Ref a). This function thus has all the characteristics one desires for an index of a structure's present condition. As a mathematical "probability," it is a numerically ordered measure with unity representing the best condition of performance and zero representing the worst condition or no performance. Through the "stated environmental conditions," the effects of imposed service loadings that were not contemplated during design are explicitly incorporated. The "specified interval of time" quantifies any deterioration in strength capacity that a structure experiences. Finally, by the "intended function" it is possible to address operational and maintenance problems that are manageable.

It is realized that structural design has traditionally been accomplished in the Corps without recourse to probabilistic methods. However, the fundamental differences between the evaluation of an existing structure and its original design mandate the adoption of some probabilistic concepts. For example, consider a lock chamber with sheet pile cells as walls. During both the design of a new project and the evaluation of an existing one, significant uncertainties arise with respect to operational loadings and sheet pile strength characteristics. A prudent designer assumes conservative values for both and possibly overdesigns the feature, but realizes an insurance benefit from any additional cost that is usually a small fraction of the total construction cost. On the other hand, the cost of rehabilitation, including the temporary loss of function, represents a substantial operational and maintenance cost that should be avoided if possible. Some confidence is required to capitalize such a cost avoidance since the failure to make a necessary repair can be even more dramatic.

PROCEDURE: In the REMR Program a general procedure for the estimation of the reliability of existing Corps structures was developed. Simply stated, the procedure consists of the following three steps:

a. Selection of the deterministic design model for the relevant mode of failure.

b. Explicit quantification of the certainty about the variables in the deterministic design model.

c. Estimation of the reliability of the structure with the use of the deterministic design model.

In the initial work, the deterministic model for the mode of operational impairment threatening a particular existing structure was formulated as a margin of safety (Resistance - Load), and the margin of safety (MS) was approximated using the method of point estimates with a normal distribution (Ref b). This method has been revised and is described in the following.

a. Step 1: Deterministic model: In the first step of reliability estimation, the engineer must select a deterministic model for the mode of operational impairment threatening a particular existing structure. In most cases, this model can be the same analytical procedure employed to design the structure for this contingency. On the simple extreme, it can be an equilibrium equation written from a free body diagram of a statically determinate component. On the other hand, the model could be a comprehensive finite element stress analysis of a complex, indeterminate system. As in the design situation, one is well advised to use the simplest model that adequately incorporates the essential factors of the mode of failure. For subsequent reliability estimation, it is convenient to formulate the model as a factor of safety (FS), which is the resistance of the component divided by the effect of the loading, i.e.

$$FS = Resistance/Load$$

b. Step 2: Certainty of variables: In the second step of reliability estimation, the engineer quantifies his uncertainty about the actual values of the variables in the deterministic model. Some of these may be known with great confidence and can be assigned a single value as in the design process. Other variables may be less precisely defined and should be considered random variables. For these factors, a mean and a standard deviation are assigned.

The engineer can draw upon extensive sources of information to assign values for these parameters. In some cases, a sample of repeated observations may have been collected having descriptive statistics that can be useful. In many cases, data collected for similar problems can be considered to apply. The expert judgment of an experienced engineer can even be quantified in selecting the mean and the standard deviation for random variables. This step is different from the design situation in which a single conservative value is wisely assigned to uncertain factors. However, in evaluating existing structures, it is important to explicitly quantify what is known about such factors through the mean as well as what is not known about them through the standard deviation.

A sensitivity study for each of the variables in the deterministic model will be performed to advise the engineer as to which of the variables should be paid the most attention.

c. Step 3: Reliability estimation: In the last step of the reliability estimation, the engineer first computes approximations for the mean and the standard deviation for the FS. The mean and standard deviation are estimated by expansion of a Taylor Series about the mean. From the mean and standard deviation of the FS, a further approximation of reliability is calculated.

APPLICATIONS: The procedure for reliability estimation is applied to three different structural features in this section. The procedure, based on the FS, the Taylor Series expansion, and a lognormal distribution, is applied to: (a) a vertical lift gate at Emsworth Dam, (b) the sheet pile interlock strength at Ohio River Lock and Dam 53, and (c) a lift gate at John Day Lock and Dam. These applications, collectively with the previous case studies, encompass the prevalent problems of the Corps civil works structures documented in the previous statistical study (Ref c).

VERTICAL LIFT GATE AT EMSWORTH DAM: Emsworth Locks and Dams were constructed on the Ohio River 6 miles below Pittsburgh and have been operational since 1921. The dams, which were converted from fixed crest structures between 1935 through 1938, now control the navigation pool with vertical lift gates in both the main and back channels. As indicated in Figure 1, the lift gates are composed of the following elements: two horizontal trusses, two vertical girders, seven diaphragms, and two end frames. The horizontal trusses, one top and bottom, transmit horizontal loads to the vertical girders, which transmit the loads to the end frames, lifting mechanisms, and piers. The first periodic inspection of the gates in 1971 revealed severe deterioration, with corrosion of individual members up to 35 percent of original thickness. Further inspection and analysis by the Pittsburgh District led to emergency repairs and eventual conversion of the gates to a nonoverflow operation. This rehabilitation is described in detail in Ref d.

A critical element in the structural integrity of the lift gate is a diagonal member in the top truss. Although failure of a diagonal might not cause collapse of the gate, it would certainly impair gate operation. Based on deterministic calculations by District engineers, the critical diagonal indicated in Figure 2 was selected for reliability estimation.

The reliability of the diagonal member is estimated by following the three step procedure previously described.

a. Deterministic model: The mode of failure for the Emsworth Lift Gate is taken to be yielding of the critical diagonal member in the top horizontal truss. Assuming the critical truss member carries no appreciable bending moment, the acting stress in the double-angle member is simply:

$$f_a = \frac{P}{A} \tag{1}$$

where

f_a = acting stress, ksi

P = member axial load, kips

A = member cross-sectional area, in.^{2}

The deterministic equation for the FS is then

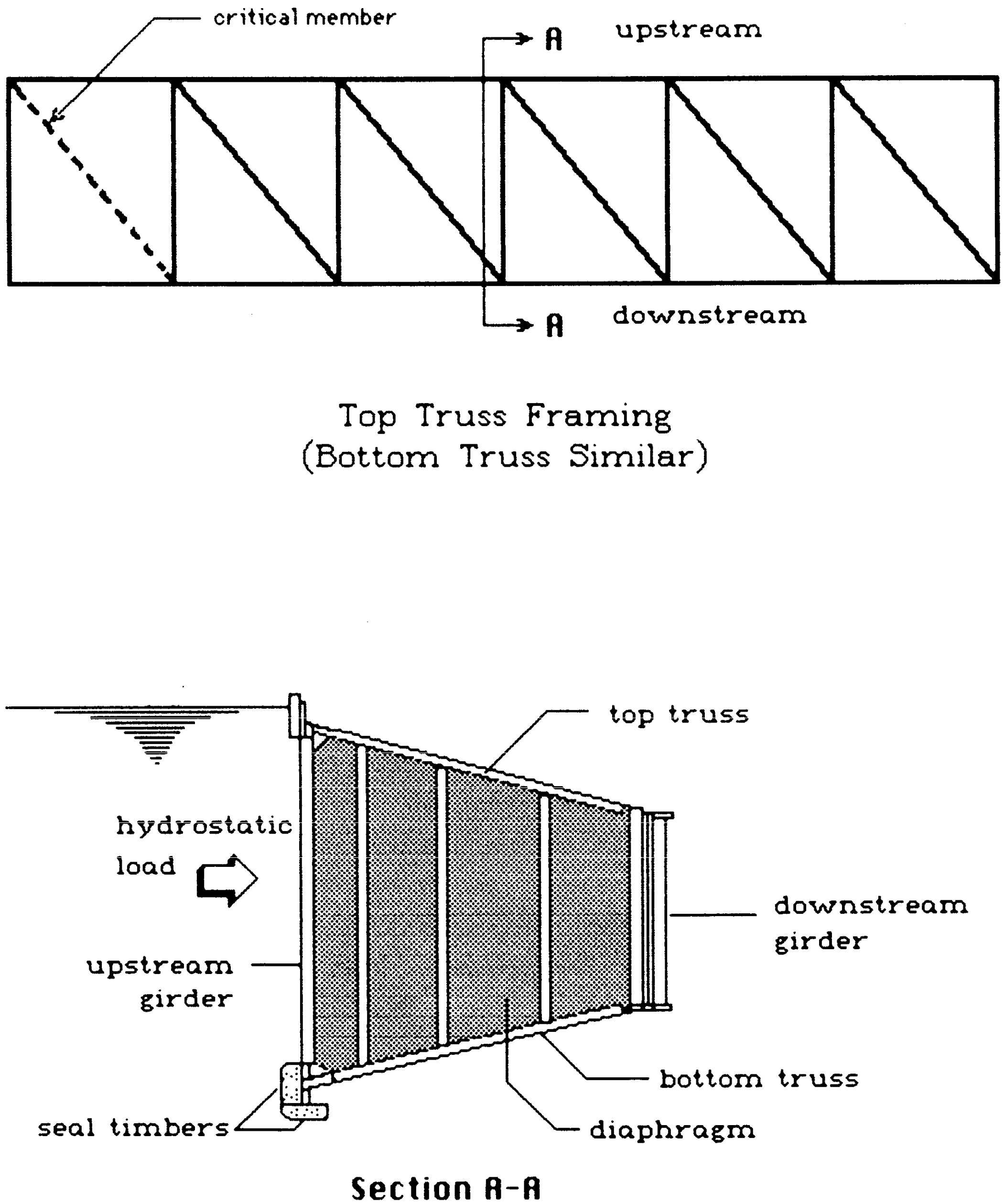


Figure 1. Emsworth vertical lift gate

$$Y = FS = \frac{F_y}{f_a} \tag{2}$$

where F_y is the material yield strength, ksi.

b. Randomness of variables: In the second step of reliability estimation, the engineer quantifies his uncertainty about the actual variables in the deterministic model by assigning mean values and standard deviations. The principal inputs for reliability of the diagonal are shown in Table 1 and include axial load, member cross-sectional area, and the material yield strength. The member axial load is a function of structure dead weight (known with confidence) and hydrostatic load, which is not as well known. It is assumed to be a random variable with a mean value of 144 kips (as taken from Corps' calculations) with a coefficient of variation (COV) of 20 percent, i.e. a standard deviation of $0.2 \times 144 = 28.8$ kps. The 20-percent COV represents the uncertainty believed associated with the hydrostatic load. The material yield strength is also a random variable, having a mean value of 36 ksi (as measured by the Corps in coupon tests) and a COV of 10 percent, consistent with previous observations for F_y. The member cross-sectional area is assumed to be known with confidence, i.e. the mean value = A with a zero COV. Corrosion of the diagonal is assumed to uniformly reduce the thickness of each angle composing the member such that

A = reduced member cross-section area, in.^{2}

$= (1 - \varepsilon/t) \times A_o$

ε = total corrosion, in.

t = original angle thickness, in.

A_o = original member cross-sectional area, in.^{2}

c. Probabilistic calculation: For the above-stated conditions, failure of the truss member occurs when the FS is less than unity; i.e., the reliability function is

$$R(\varepsilon) = P[FS > 1] \tag{3}$$

where ε denotes the amount of corrosion and $R(\varepsilon)$ indicates the reliability for that amount of corrosion. This function has been computed using the refined procedure described previously and is presented in Figure 2.

CONCLUSIONS: As can be seen in Figure 2, the FS for the truss diagonal drops below 1.0 (reliability = 0.5) when corrosion exceeds a total of 0.35 in., a finding consistent with the Corps' deterministic calculations. The reliability calculations further show that the brace reliability actually begins to decline significantly when corrosion is about 0.2 in. Assuming a corrosion rate of 0.005 in./year (consistent with past experience at this location), the member would remain reliable until about 1975. The rehabilitation and member

Table 1

Data for Emsworth Dam Lift Gate

Constants

Name	Description, in.	Value
A	Initial member area	10.62
t	Initial member thickness	0.5625

Random Variables

Name	Description	Mean	Standard Deviation
P	Member axial load, lb	144,000	28,800
f_y	Steel yield strength, psi	36,000	3,600

replacement were carried out between 1974 and 1980 for the different gates; thus, the gates were replaced before a serious decline in reliability. The probabilistic findings are more informative than the deterministic assessment since they indicate the gradual deterioration as a function of corrosion and time.

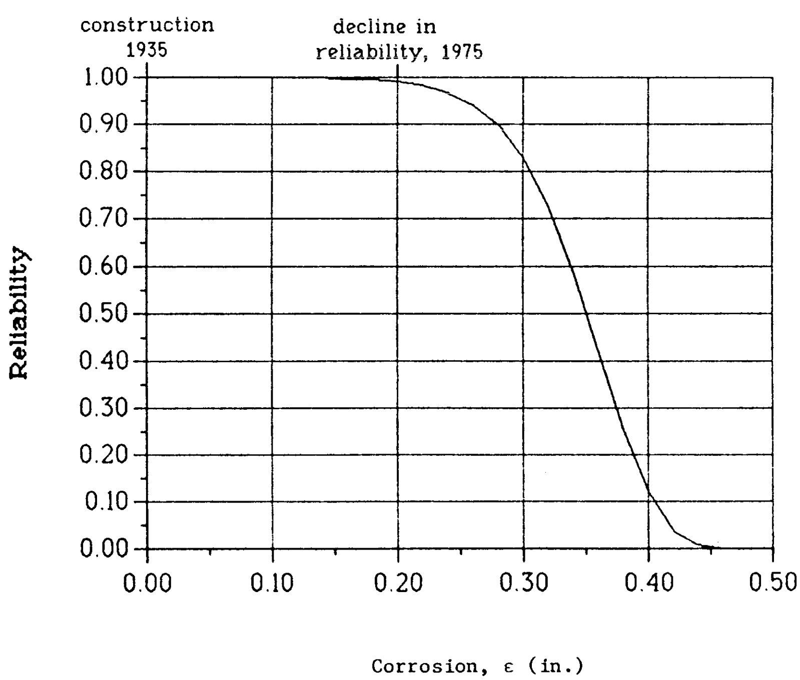


Figure 2. Reliability of Emsworth lift gate

OHIO RIVER LOCK AND DAM 53: A temporary 1,200-ft lock was constructed between 1975 and 1980 beside the existing lock at Ohio River Lock and Dam 53. The temporary lock walls, completed in October 1980, are composed of interconnecting circular sheet pile cofferdam cells that form the flume, land, and river walls, as shown in Figure 3. An inspection of the lock in August 1986 revealed severe corrosion of the sheet piling with spot corrosion rates as high as 0.02 in./year and an average corrosion rate of 0.007 in./year. A critical element of the cell integrity, sheet pile web strength, was reviewed by the Corps in 1986. These deterministic calculations indicated required cell wall thicknesses for normal and earthquake loading conditions.

a. Deterministic model: The mode of failure considered for the lock walls in the Corps' calculations was tensile failure of the sheet pile web. The maximum acting tensile load in the web (N) is given by

$$N = R[K_a \gamma (H - s) + (K_a \gamma + \gamma_w) (s - d) - \gamma_w (h - d)] \qquad (4)$$

where these terms are defined in Table 2. The tensile load computed acts at the bottom of the cell wall, and lesser values act throughout the wall height. The web strength, N_w, is simply calculated as

$$N_w = t\sigma_u \qquad (5)$$

where

t = web thickness, in.

σ_u = steel ultimate strength, ksi

The deterministic equation for the FS is then

$$Y_w = FS = \frac{N_w}{N} \qquad (6)$$

An additional mode of failure considered for the lock walls is failure of the sheet pile interlock. The interlock strength, N_u, is defined as

$$N_u = 2N_1 + N_2 \qquad (7)$$

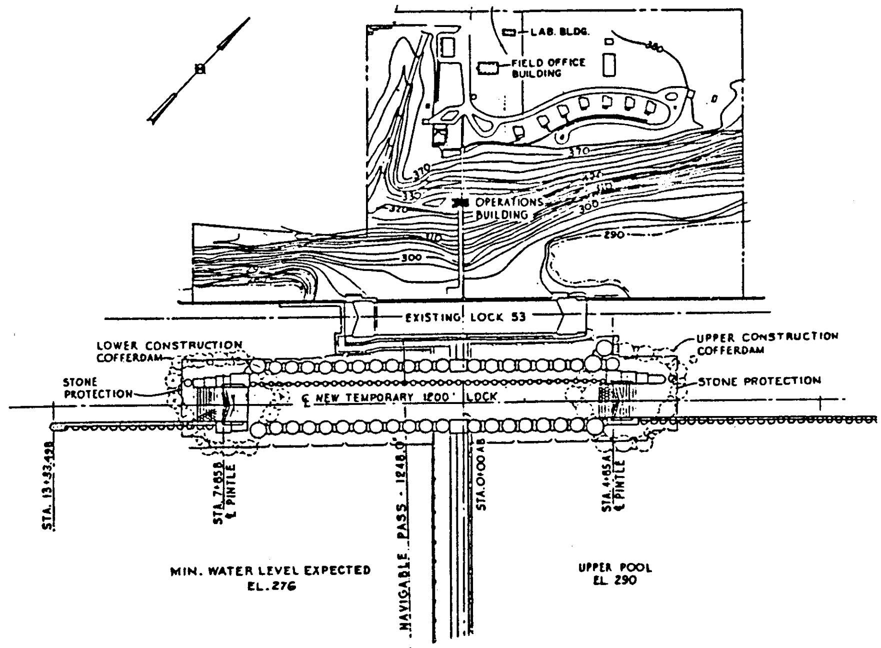
where

N_1 = force per unit length between finger and thumb

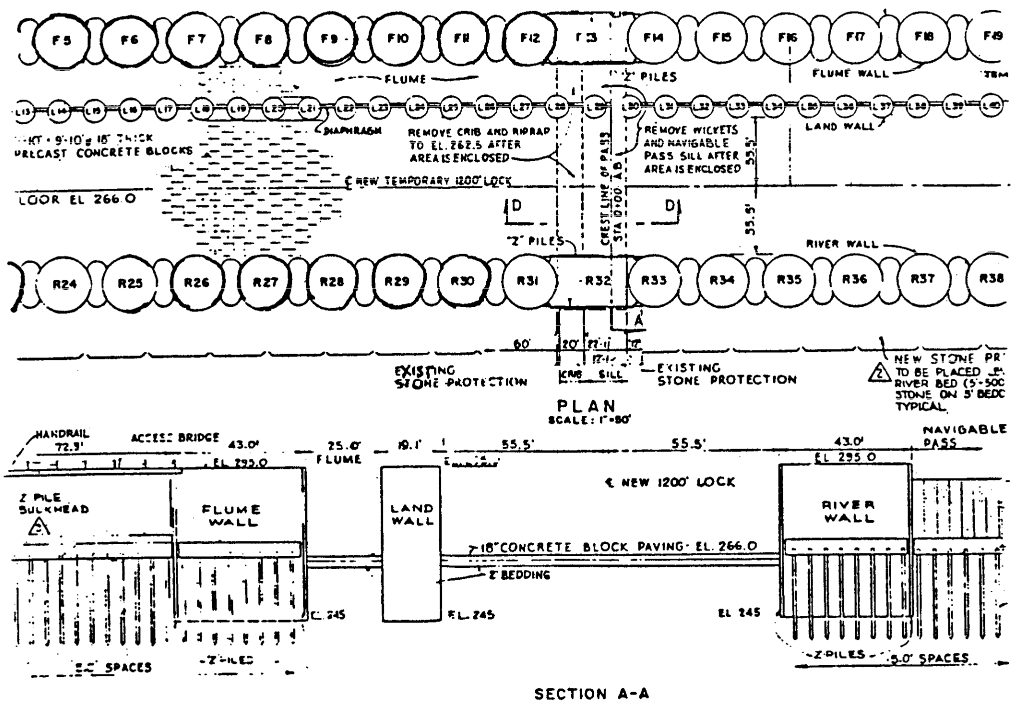
N_2 = force per unit length between interlocked thumbs

These two forces are found from the following two equations

$$N_1 = 2\, e_3\, \sigma_u \left(-1 + \sqrt{1 + t_2^2 / 4e_3^2} \right) \qquad (8)$$



a. General arrangement



b. Partial plan and section

Figure 3. Ohio River Lock and Dam 53

Table 2

Data for Interlock Reliability at Lock and Dam 53

Constants

Name	Description	Value
t	Web thickness, in.	0.375
t_1	Thumb thickness	0.35
t_2	Finger thickness, in.	0.500
e_1,e_2	Eccentricity of N_1,N_2, in.	0.48
e_3	Eccentricity of finger, in.	0.56
R	Radius of circular cell, in.	258
d	Elevation at bottom of chamber, in.	3,120
h	Elevation of water in chamber, in.	3,312
H	Elevation at top of cell, in.	3,504
σ_u	Steel yield strength, psi	38,500

Random Variables

Name	Description	Mean	Standard Deviation
s	Elevation of backfill water table, in.	360	24
γ	Unit weight of soil, lb/in.	0.0579	0.0029
K_a	Coefficient of active earth pressure	0.60	0.125
N_u	Interlock strength, lb/in.	*	$0.11 \times N_u$

* Found using Equation $N_u = 2N_1 + N_2$.

$$4(N_2\ e_2 - N_1\ e_1) = \frac{t_1^2\ \sigma_u - \left(N_1 + N_2\ ^2\right)}{\sigma_u} \tag{9}$$

where

e_3 = eccentricity of finger, in.

σ_u = steel yield strength, psi

t_2 = finger thickness, in.

e_2 = eccentricity of N_2, in.

e_1 = eccentricity of N_1, in.

t_1 = thumb thickness, in.

The deterministic equation for the FS for interlock is similarly:

$$Y = FS = \frac{N_u}{N} \tag{10}$$

Inputs to the resistance include sheet pile geometry and steel ultimate strength. The acting load (N) is a function of cofferdam cell radius; unit weights of soil and water; coefficient of lateral earth pressure; and elevations at chamber bottom, river water, backfill water table, and top of cell.

b. Randomness of variables: As in the previous interlock calculations, elevation of the water table, unit weight of soil, coefficient of lateral soil pressure, and interlock strength were assumed to be random variables. Geometry of the sheet pile and cofferdam and steel strength were assumed to be known with confidence and thus were taken as constants. The values of the constants and random variable means and standard deviations are presented in Table 2. The constants and mean values are based on the Corps' calculations and lock drawings, and the standard deviations are based on previous interlock experience. It was assumed that corrosion uniformly reduces the sheet pile thicknesses.

c. Probabilistic calculation: For the above-stated conditions, failure of the web or interlock occurs when the FS is less than unity, i.e., the reliability function is

$$R(\varepsilon) = P[FS > 1] \tag{11}$$

where ε denotes the amount of corrosion and $R(\varepsilon)$ indicates the reliability for that amount of corrosion. The reliability functions were computed for normal conditions using the refined procedure, and the results for web failure and interlock failure are compared in Figure 4.

CONCLUSIONS: As seen in Figure 4, sheet pile web reliability begins to decline at about 0.23 in. of corrosion, and interlock reliability begins to decline at about 0.20 in. of corrosion. Assuming an average corrosion rate of 0.007 in./year, the sheet piling should remain quite reliable until 2010. After that time, further corrosion at the 1980 to 1986 rate will significantly reduce the condition of this structural feature in 6 to 10 years. The Corps thus need not initiate emergency repairs to the sheet pile at this time. They should, however, immediately plan for the orderly rehabilitation or replacement of this feature while closely monitoring its performance. While these remarks are supported by this probabilistic assessment, the deterministic evaluation of this structure's condition does not forecast the temporal deterioration of condition that justifies such remarks.

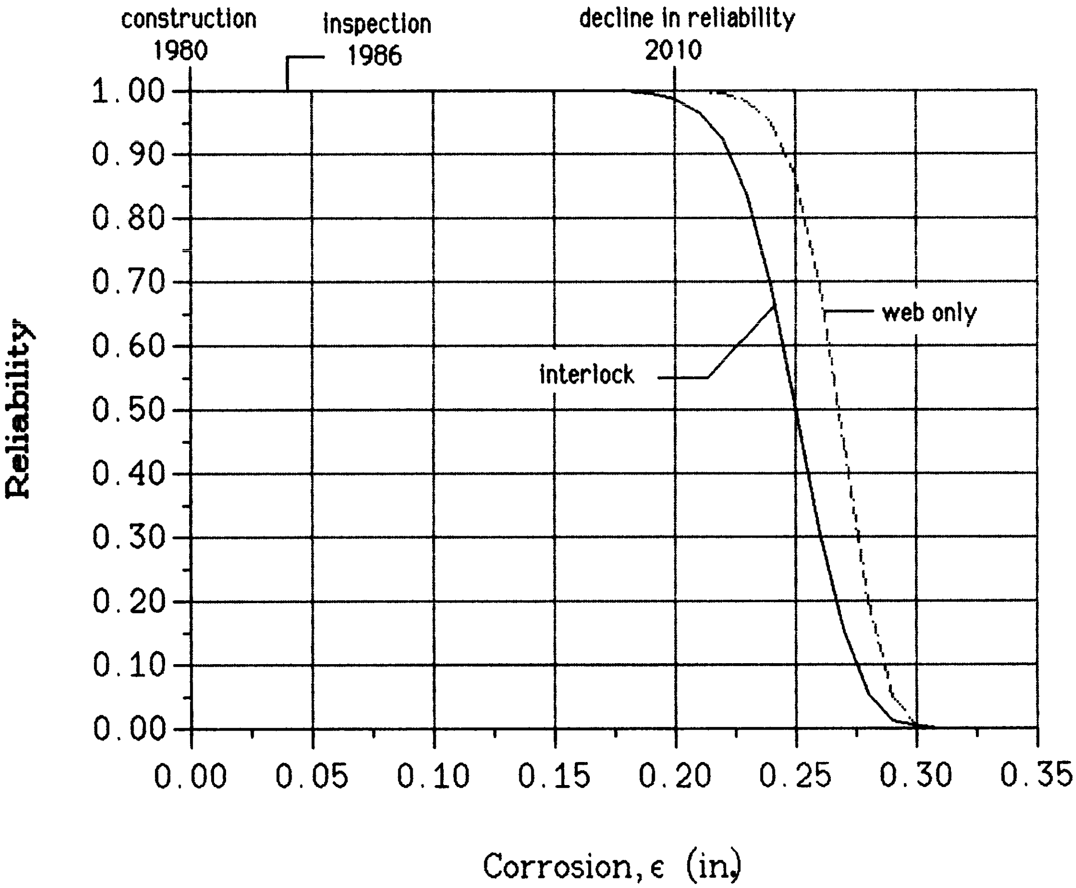


Figure 4. Reliability of temporary lock 53

VERTICAL LIFT GATE AT JOHN DAY NAVIGATION LOCK: John Day Lock and Dam is located on the Columbia River about 110 miles upstream from Portland, OR. The downstream lock gate is a vertical lift gate 88 ft wide and 113 ft high (Figure 5). The all-welded steel structure consists of a series of horizontal tied arches made up of a 1-1/8-in.-thick curved skinplate, a curved compression member (WT18 by 150), and a tension tie girder (W33 by 240) with the web

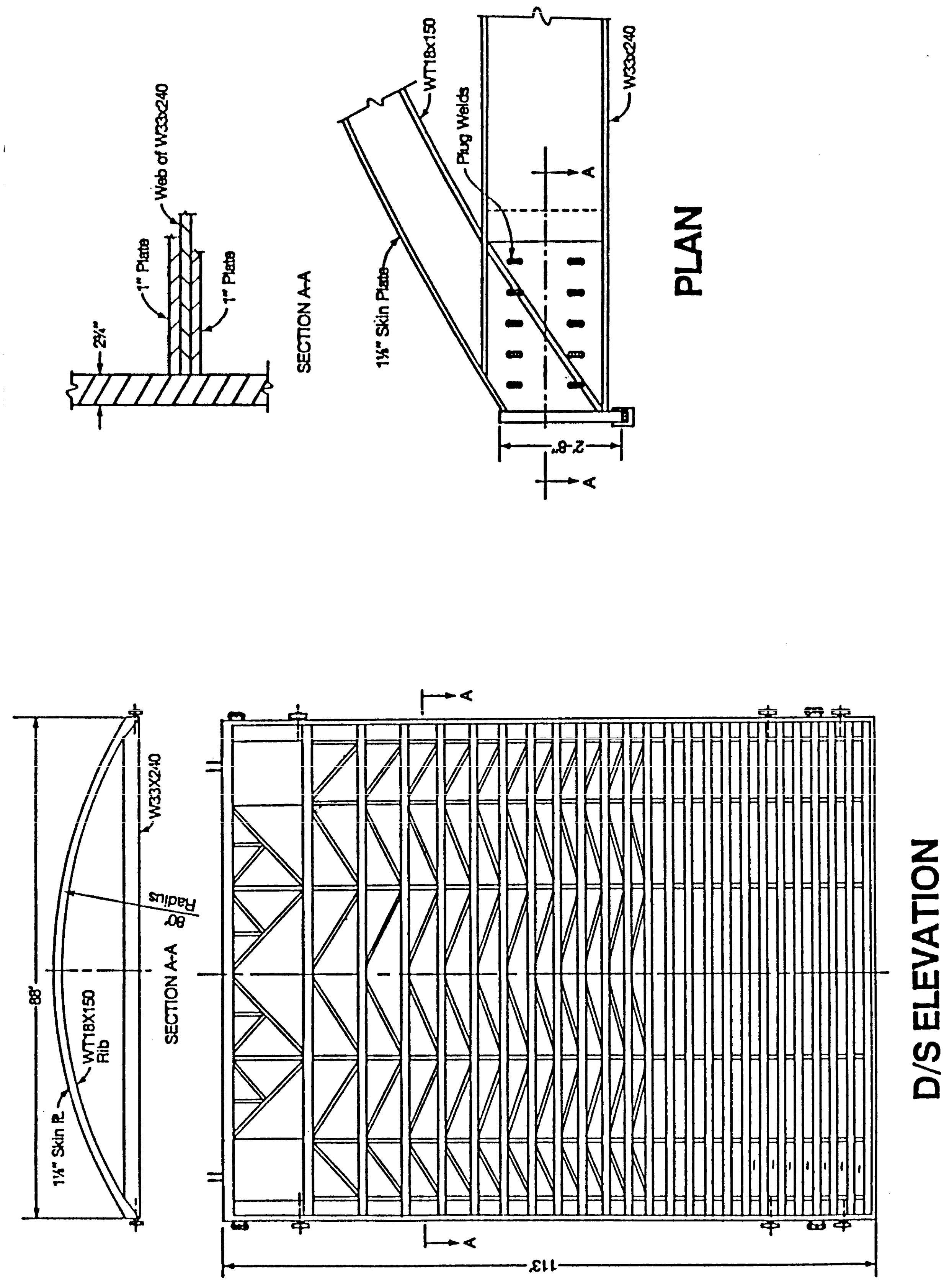


Figure 5. John Day vertical lift gate (Ref e)

in a horizontal plane. After detection and repair of numerous defects in the joints, the gate was accepted and put into operation in 1963.

An annual inspection program for this gate was initiated in 1980 following serious cracking of tension tie members in a gate of almost identical design at the Ice Harbor Project on the Snake River. During the annual inspection in November 1982, major cracks were discovered in both ends of the bottom six arches at the junction of the inner flanges of the tension ties and the curved compression members. Examples of the cracking, which began at the inner flange of the tie member propagating to the outer flange, are shown in Figure 6. The gate was repaired and strengthened in December 1982, after cracks up to about 30 in. were discovered in the tie girder of the lowest arch. Fortunately, load and stress redistribution to other girders prevented catastrophic failure. Possible causes for the cracking included (a) impact caused by hard stops or bumping the sill, (b) shock load induced by slipping at the gate to lock bearing during filling, (c) vibration caused by water rushing past poorly seated seals during filling, (d) stress risers at notch producing details, and (e) low winter temperatures causing the steel to be more brittle and crack sensitive.

The tie member is an important member of the gate, and failure of one of these members would seriously compromise the operation of the gate. An estimate of the reliability of the gate as a function of the crack length in the lower tie girder is presented in the following sections. The reliability of the tie girder as a function of the crack length is estimated by the procedure described previously.

a. Deterministic model: The mode of failure for the gate is taken to be yielding of the tie girder at the cracked section when the lock is filled to maximum lift. The acting stress (ignoring stress concentrations) in this section is

$$f_a = \left(\frac{P}{A}\right) + \left(\frac{M}{S}\right) \tag{12}$$

where

f_a = acting stress, ksi

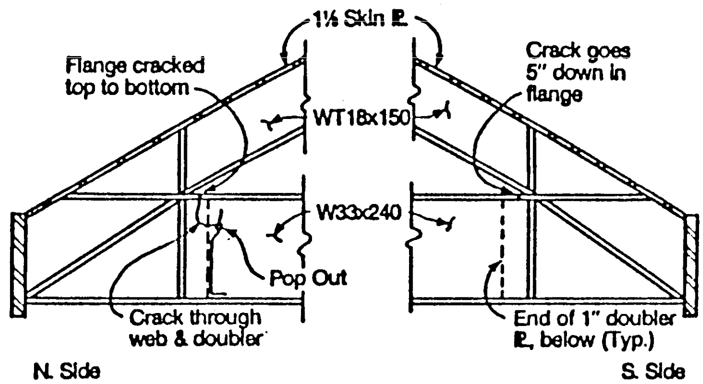
P = axial load in the girder, kips

A = area of the cracked section, $in.^2$

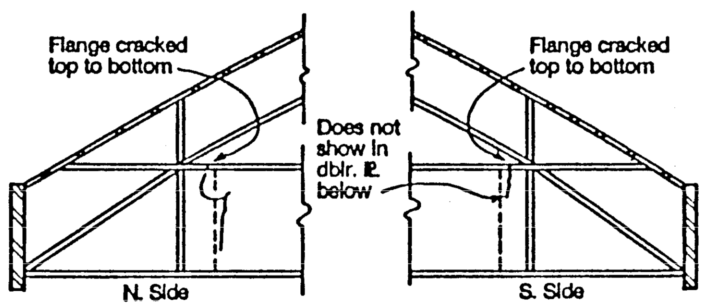
M = bending moment, in.-kips

S = section modulus of the cracked section, $in.^3$

For this indeterminate structure, the axial load and bending moment at the cracked section are functions of member stiffness and must be computed for each crack length, δ . For this analysis, a simple finite-element beam model of one tied arch, as indicated in Figure 7, was developed to compute P and M as a function of crack length, δ . In this model, a short section of the tie beam at the crack (member 2-4) was assigned properties and location of



GIRDER NO. 1



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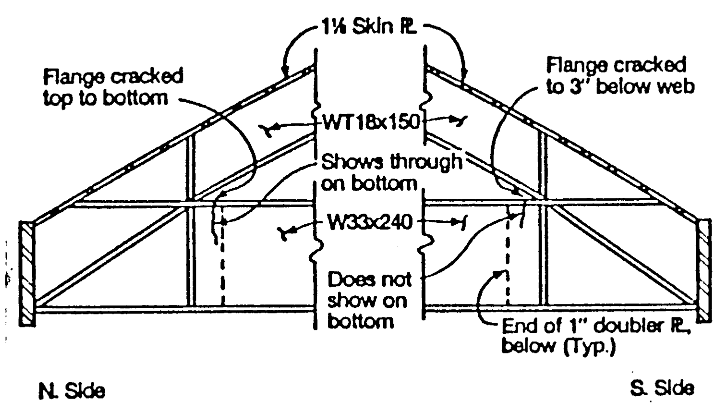
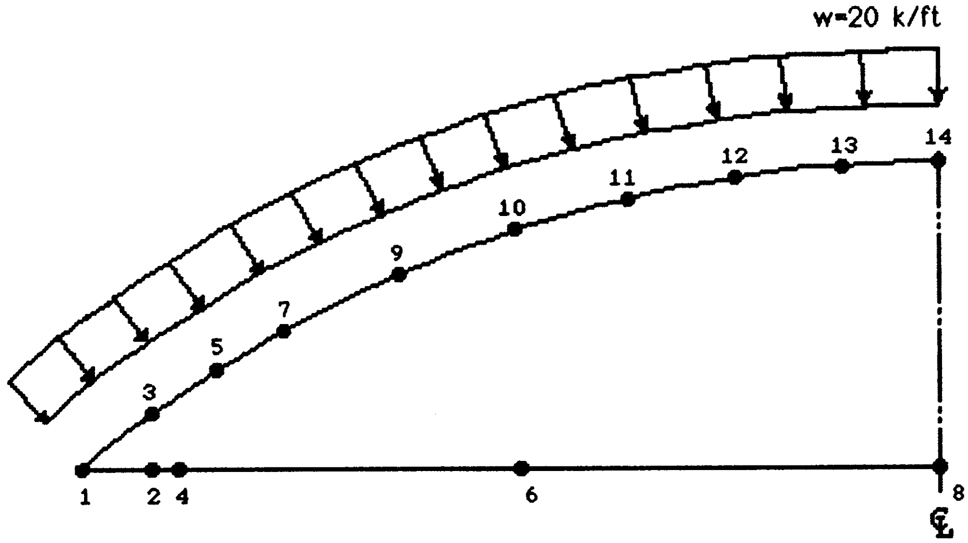
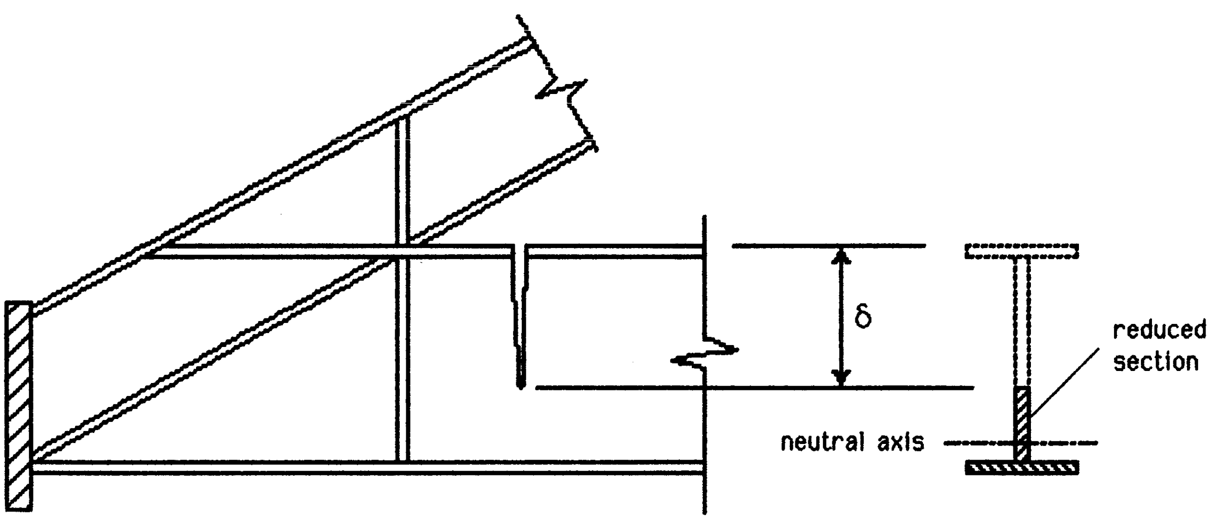


GIRDER NO. 3

Figure 6. John Day tension tie member cracking (Ref e)



a. Beam element model



b. Cracked section

Figure 7. Finite-element model of John Day vertical lift gate

neutral axis consistent with the cracked cross section, shown in Figure 7b. The deterministic equation for the FS is then

$$Y = FS = \frac{F_y}{F_a} \tag{13}$$

where F_y is the material yield strength, ksi.

b. Randomness of variables: It is assumed that the geometry of the structure is known with confidence. Two inputs to the analysis, lock water elevation and steel yield strength, are considered to be less well known and are taken as random variables. The mean hydrostatic load on the section was taken to be 20 kips/ft with a COV of 5 percent. This corresponds to an uncertainty of about 5 ft in water elevation and also reflects other service loadings such as debris and impact. The mean steel yield strength was taken as 46 ksi (Ref f) with a COV of 10 percent (consistent with previous observations). Another source of uncertainty lies not in the input variables, but in the analysis method used to compute the axial load and bending moment. Analysis methods used for previous cases in this report are thought to represent the actual stress state with confidence and were not assumed to add to the overall dispersion in reliability. As the beam model used here provides results with lesser confidence, an analysis factor was introduced to represent this random effect. This random effect on axial load (P) may be illustrated by writing the deterministic equation as follows

$$P = \lambda_a \; Cw \tag{14}$$

where

P = axial load

λ_a = axial load analysis factor

C = transfer function (finite element analysis)

w = water pressure load

Thus, the axial load analysis factor, λ_a , can be introduced into the reliability estimation just as any random variable. A bending moment analysis factor is similarly illustrated.

$$M = \lambda_b \; Cw \tag{15}$$

where

M = internal bending moment

λ_b = bending moment analysis factor

C = transfer function (finite-element analysis)

w = water pressure load

Based on engineering judgment, the load results from the finite-element analyses were taken as mean values, and a COV of 10 percent on axial load and of 15 percent on bending moment were applied, respectively. Note that a more "accurate" analysis, e.g. using a detailed plate element model, would warrant lower COV on analysis factors.

c. Probabilistic calculation: Failure of the gate is assumed to occur when acting stresses reach yield (when the FS is 1); thus the reliability function is

$$R(\delta) = P[FS > 1] \tag{16}$$

where δ denotes the crack length and $R(\delta)$ indicates the reliability for that crack length. This function was computed using the above-described equations and input by the refined procedure, with the results presented in Figure 8.

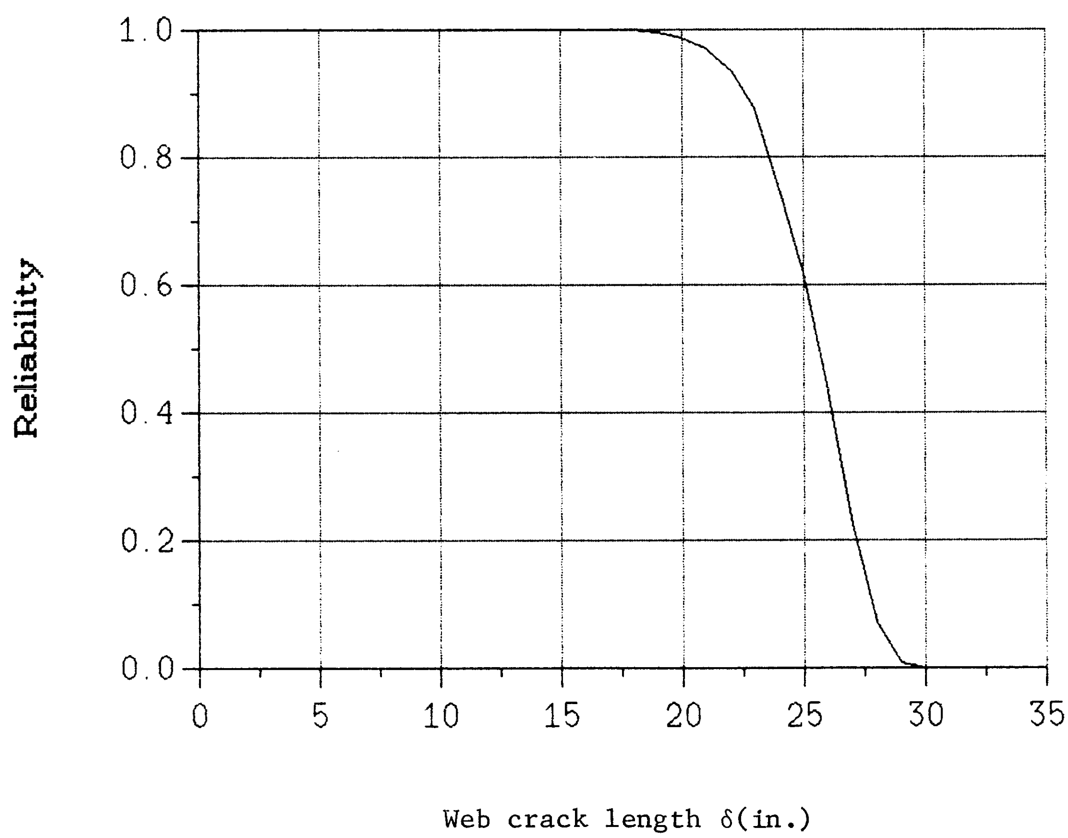


Figure 8. Reliability of John Day lift gate

CONCLUSIONS: As can be seen in Figure 8, the FS drops below 1.0 (reliability = 0.5) when the crack length exceeds about 26 in., as can be shown by deterministic calculations. The results further show that reliability begins to decline when the crack length is about 18 in. Unlike corrosion degradation, it is difficult to associate a time scale with crack length since crack propagation occurs sporadically and at various rates. This estimation does, however, provide an indication as to the safety of the gate for different crack conditions. The repair of the gate in 1982 was certainly appropriate according to both deterministic and probabilistic calculations.

REFERENCES:

a. Kapur, K. C., and Lamberson, L. R. 1977. Reliability in Engineering Design, John Wiley and Sons, New York.

b. Benjamin, J. R., and Cornell, C. A. 1970. Probability, Statistics, and Decision for Civil Engineers, McGraw-Hill Book Company, New York.

c. Mlakar, P. F. 1984 (Dec). "A Statistical Study of the Condition of Civil Works Steel Structures," Report No. J650-84-001, prepared by JAYCOR, Structures Division, Vicksburg, MS, for US Army Construction Engineering Research Laboratory, Champaign, IL.

d. Ardine, E. A. 1975 (Jun). "Rehabilitation of Vertical Lift Gates - Emsworth Dam," US Army Corps of Engineers District, Portland, OR.

e. Kearney, F. W. 1986 (Sep). "Life-Cycle Analysis of Corroding Sheet Pile Structures for a Lock and Dam," Technical Report M-86/12, US Army Construction Engineering Research Laboratory, Champaign, IL.

f. Wheeler, W. 1985 (Jun). "Investigation and Repair of John Day Navigation Lock Downstream Lift Gate," presented by US Army Engineer District, Portland, OR, at the 1985 Corps of Engineers Structural Engineering Conference.

g. Mlakar, P. F., and Toussi, S. "Reliability of Civil Works Structures; General Methodology and Application to Sheet Piles," REMR Technical Report in preparation, US Army Construction Engineering Research Laboratory, Champaign, IL.

h. Bryant, L. M., and Mlakar, P. F. "Evaluation of Civil Works Steel Structures," REMR Technical Report in preparation, US Army Construction Engineering Laboratory, Champaign, IL.

i. National Bureau of Standards (NBS). 1980 (Jun). "Development of a Probability Based Load Criterion for American National Standard A58, Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," NBS Special Publication 577, US Department of Commerce, Washington, DC.

j. Julian, O. G. 1957 (Jul). "Synopsis of First Progress Report of Committee on Factors of Safety," Journal of the Structural Division, American Society of Civil Engineers, Vol 83, No. ST 4, pp 1316-1-1316-22.

k. Liu, T. C. 1980 (Jul). "Strength Design of Reinforced Concrete Hydraulic Structures; Report 1, Preliminary Strength Design Criteria," Technical Report SL-80-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

l. Moses, F. 1980. Utilizing a Reliability-Based API RP2A FORMAT," API PRAC Project 79-22, submitted to the American Petroleum Institute, Dallas, TX.