CECW-ED

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

Technical Letter
No. 1110-2-551

31 August 1998

Engineering and Design
IDENTIFICATION, INSPECTION, AND EVALUATION OF FRACTURE CRITICAL MEMBERS OF IN-SERVICE BRIDGES

1. Purpose

This Engineer Technical Letter (ETL) provides guidance in the identification, inspection, and evaluation of fracture critical members of in-service bridges owned and operated by the U.S. Army Corps of Engineers (USACE) on Civil Works projects. This ETL is not intended to provide guidance on analysis and design of bridges.

2. Applicability

This ETL applies to all USACE Commands having responsibilities for planning, inspecting, evaluating, and documenting the safety of in-service bridges.

3. References

References are listed in Appendix A.

4. Distribution Statement

Approved for public release; distribution is unlimited.

5. Background

a. The national average for bridge failures per year is 150 collapses resulting in the death of 12 people. Nationally, bridge collapses are not now as frequent as they were in the nineteenth century; however, they still occur. It is extremely important that fracture critical members on bridges be identified, properly inspected, and evaluated.

b. As noted in Appendix A, a significant amount of information is currently published on inspecting and evaluating fracture critical members. A methodology for identifying fracture critical members is explained in this ETL. Information pertaining to state-of-the-art techniques for real-time damage assessment of bridge structures is provided in Appendix B.

6. Summary

This ETL summarizes procedures for the identification, inspection, and evaluation of fracture critical members of USACE in-service bridges on public roads. The ETL is not intended to provide guidance on how to develop a numerical model, apply loads and boundary conditions, or develop load combinations. However, once a structural model has been developed, this ETL will provide guidance on identifying, inspecting, and evaluating fracture critical members of in-service bridges. Two bridges, Summit Inland Waterway Bridge crossing the Delaware River and Chesapeake Bay and St. George's Highway Bridge located in Delaware and Maryland crossing the Chesapeake Bay and Delaware Canal, are analyzed using the finite element method to demonstrate a procedure of locating fracture critical members. The structural degradation process resulting from fracture and fatigue is presented to provide background for critical assessment and inspection planning. A state-of-the-art review of new techniques in structural damage monitoring and structural integrity assessment methodology is presented in Appendix B. This review summarizes information pertaining to new methodology and technology available for more effective inspection and evaluation of bridges. The reader should be aware that the information in Appendix B is new technology and may not apply to all conditions. CECW-ED should be contacted if there is a concern about applicability.

7. Objectives

The objective of this ETL is to provide information on the identification, inspection, and evaluation of fracture critical members on in-service bridges. In addition, this ETL provides information pertaining to state-of-the-art review of new techniques for real-time damage assessment of bridge structures.

8. Action

The guidance in this ETL should be used to identify, inspect, and evaluate fracture critical members on bridges owned and operated by USACE on Civil Works projects.

FOR THE COMMANDER:

CARL F. ENSON, P.E.
Chief, Engineering Division
Directorate of Civil Works

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Chapter 1 Introduction

1-1. Overview

a. As of November 1991, 35 percent of approximately 590,000 bridges in the United States were considered structurally deficient or functionally obsolete (Bagdasarian 1994). Many bridges have become deficient due to aging and heavier than expected service loads. In particular, some highway and railroad bridges ranging from 50 to more than 100 years old are still performing their intended functions in spite of excessive use (Scalzi 1988). The recent collapse or near-collapse of some bridges has resulted in the development of extensive inspection programs and engineering assessment methods to ensure that highway bridges are safe for public use.

b. Highway bridges are subjected to a wide range of vehicular loads. As vehicles cross, the live loads produce changing stresses which cause a wide range of strain or deformation in the members. The impact of a vehicle also contributes to the changing stresses. The relatively large range of repeated elastic strain or deformation places greater demands on the material properties of critical members and increases the probability of damage. In addition, bridges are relatively unprotected from the environment. Bridge members are exposed to water, debris, and contaminants such as deicing salts, and they must resist freeze/thaw damage and accommodate significant thermal movement.

c. Bridge deterioration typically occurs at specific locations related to deck drainage, debris accumulation, and exposure. Cracks can initiate at stress concentrations caused by certain framing details and fabrication defects. To evaluate the degree to which a deficiency effects safety often requires an appraisal of that specific deficiency's significance on the structural stability of the bridge. Locating the fracture critical members of the bridge, as well as assessing the criticality of deficiencies in the fracture critical members (FCMs), is necessary to determine if the bridge should remain open. An effective inspection plan must contain information helpful in locating problems on members with potentially high-risk modes of failure. Unless the inspector understands where to look and what to look for when inspecting bridges, the inspection activity will be ineffective. Cracks frequently start at stress concentrations and out-of-place stresses due to connections of transverse members. Additional information on structural inspection can be found in Chapter 2 of the AASHTO (1983) Manual for Maintenance Inspection of Bridges, and Chapter 18 of the FHWA (1991) Bridge Inspector's Training Manual 90.

1-2. Organization

This report summarizes the procedures for identification, inspection, and evaluation of FCMs of USACE in-service bridges on public roads. In Chapter 2, two bridges that cross the Chesapeake Bay to the Delaware River canal, Summit Inland Waterway Bridge and St. George's Highway Bridge, are analyzed using the finite element method to demonstrate a procedure of identifying FCMs. In Chapter 3, the structural degradation process due to fracture and fatigue is presented to provide background for critical assessment and inspection planning. A review of state-of-the-art techniques in structural damage monitoring and structural integrity assessment methodology is presented in Appendix B. This review summarizes information pertaining to new methodology and technology available for more effective inspection and evaluation of bridges. This report is not intended as a stand-alone technical resource on fracture critical members. However, several references are included to provide the reader with additional information. Information provided in this report and other referenced documents is in a mixture of SI metric units and inch kip units. A more consistent set of equations will be developed in a future Engineer Manual.

Chapter 2
Locating Fracture Critical Members

2-1. Fracture Critical Members

a. The AASHTO (1996) Guide Specification for Fracture Critical Bridge Members states that "Fracture Critical Members or member components are tension members or tension components of members whose failure would be expected to result in collapse of the bridge." To qualify as a FCM, the member must be a nonredundant member subject to tensile force. There must not be any other member or system of members which will serve the functions of the member in question should it fail. This has also been interpreted to include bending members which experience tensile forces over part of their cross section, whose failure would be expected to result in collapse of the bridge. Compression members or components are not considered fracture critical. Since it is considered undesirable from an operation and maintenance standpoint to have a bridge member yield, collapse is taken to mean yielding has occurred. This is consistent with the approach used by the Federal Highway Administration. The FCM can be identified by removing the member in tension and checking the remaining members in the bridge to see if any members have yielded. Information on redundancy in bridge framing systems and of tension members, along with the necessary definitions, are included in Chapter 2 of the Federal Highway Administration's "Bridge Inspector's Training Manual 90" (Hartle et al. 1991). In addition, pertinent articles on FCMs have been published in Civil Engineering (1987).

b. To locate the FCMs in a bridge, both dead and live loads must be considered in the structural analysis. As defined by AASHTO Standard Specifications for Highway Bridges (1996), dead loads are the weight of the complete structure, including the roadway, sidewalks, car tracks, pipes, conduits, cable, and other public utility services. Dead loads do not change with time and need to be considered as permanent loads acting on the structure. Live loads consist of the weight of applied moving loads such as vehicles and pedestrians. Live loading on the roadway of bridges or incidental structure shall consist of standard trucks or lane loads which are equivalent to truck trains. Two systems of loading, the H loading and the HS loading, are defined by AASHTO specifications (1996). Standard truck loads, wheel spacing, weight distributions, and clearances for standard H and HS truck loading can be obtained from the specification. H20-44 and HS20-44 standard truck loads that will be applied in the examples discussed later in this section are shown in Figure 2-1.

c. The lane loads consist of uniform load per linear foot of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans) so placed on the span as to produce maximum stress. The

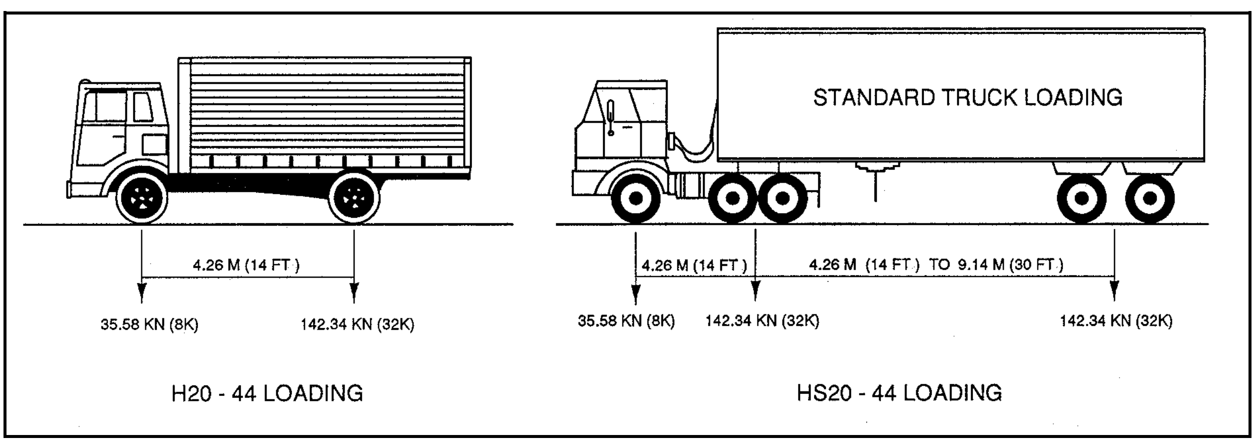


Figure 2-1. Standard truck loading

concentrated load and uniform load shall be considered as uniformly distributed over a 3-m (10-ft) width on a line normal to the center line of the lane. Figure 2-2 shows the lane loading for H20-44 and HS20-44. For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 2-2.

d. For continuous spans, the lane loading shown in Figure 2-2 needs to be modified by the addition of a second equal-weight concentrated load placed in one other span in the series in such position as to produce the maximum negative moment. Live load stresses produced by H or HS loading shall be increased for bridge superstructures and the portion of concrete or steel piles above the groundline which are rigidly connected to the superstructure as in rigid frames or continuous designs to account for impact effects. The amount of this allowance or increment should be calculated in accordance with AASHTO design specifications (1996).

2-2. Analysis Procedure for Locating FCMs of Non-Truss Bridges

a. Figure 2-3 shows flowcharts for locating FCMs in non-truss bridges. Dead loads and live loads must be applied to the bridge according to AASHTO requirements. A structural analysis is performed to determine the member forces. To locate FCMs, each tension member is removed on an individual basis to determine if its removal and the redistribution of forces cause any of the remaining members to yield. If yielding develops, the removed tension member is a FCM. The tested tension member is then reinstalled; the next tension member is removed, and the remaining members are again checked for yielding. This tension removal procedure continues until each tension member has been individually removed and the remaining members have been checked for yielding. After each tension member has been checked, a new live load condition is applied, and the tension member testing procedure is repeated. The FCMs for the entire bridge can be obtained utilizing this process.

b. Because this repeated analysis procedure can be very tedious and time consuming, the structural analysis can be performed by using a finite element structural program. ANSYS program (ANSYS 1992) is used for the example cases presented in paragraph 2-3.

2-3. Analysis Procedure for Locating FCMs of Truss Bridges

a. For truss bridges, the first step of the analysis is to decide the degree of indeterminacy. For a

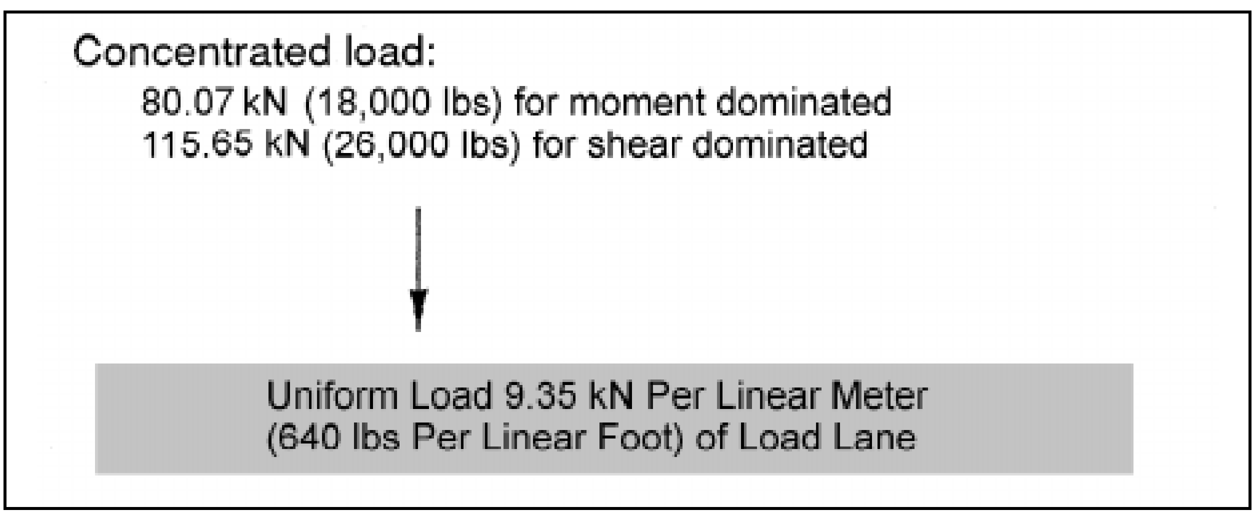


Figure 2-2. H20-44 lane loading and HS20-44 lane loading

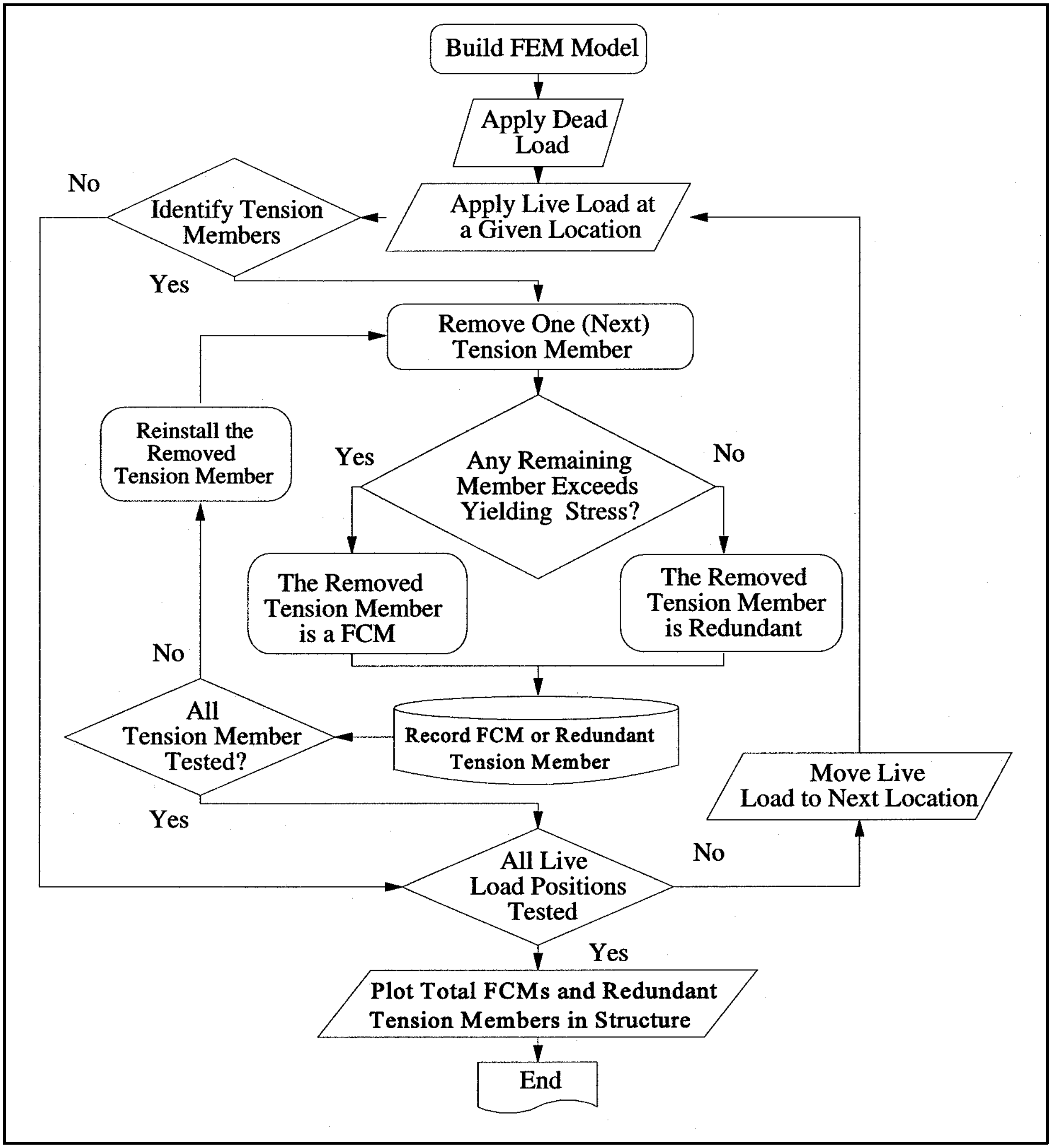


Figure 2-3. Flowchart for locating FCMs of non-truss bridges and indeterminate truss bridges using linear elastic and perfectly plastic model

determinate truss bridge all tension members are FCMs. The flowchart for determinating FCMs of determinate truss bridges is presented in Figure 2-4. For an indeterminate truss bridge, the procedure is similar to a non-truss bridge as plotted in the flowchart in Figure 2-3.

b. Example 1 is Summit Bridge, an inland waterway bridge (627.28 m (2,058 ft) total span) crossing the Delaware River and the Chesapeake Bay. The bridge approach is via several simple supported girders, followed by a 76.2-m (250-ft) single-span deck truss (Figure 2-5), and then onto

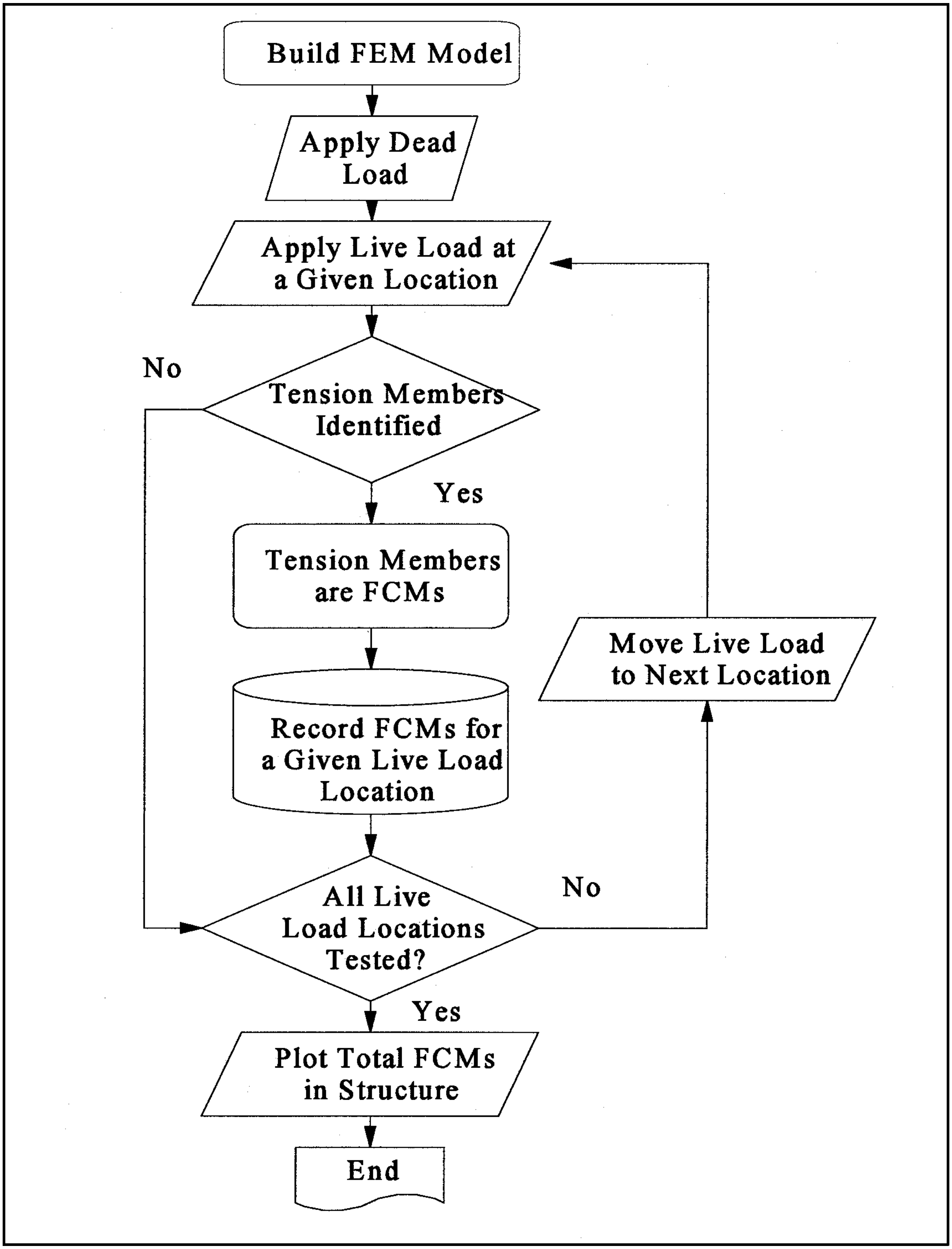


Figure 2-4. Flowchart for locating FCMs of statically determinate truss bridges

the 91.44-m (300-ft) anchor arm span and 182.88-m (600-ft) main span (Figure 2-6). The main span in the middle of the bridge (Figure 2-6) can be further divided into a suspended span and two cantilever spans. Figure 2-5 shows the finite element model of the deck truss. The deck truss system is a determinant (nonredundant) structure. Figure 2-6 shows the finite element model of the anchor arm and main spans. This bridge has four traffic lanes. The dead loads of each bridge member were applied according to the design data (USACE 1940). The design live load is a HS20-44 loading (Figure 2-2) plus an impact load of 111.2 kN (25 kips) (USACE 1940), except for the deck slab which is designed for 142.34 kN (32 kips) per axle load. The 9.35 kN per linear meter (640 lb per linear foot) of lane load was applied as a distributed load to the truss

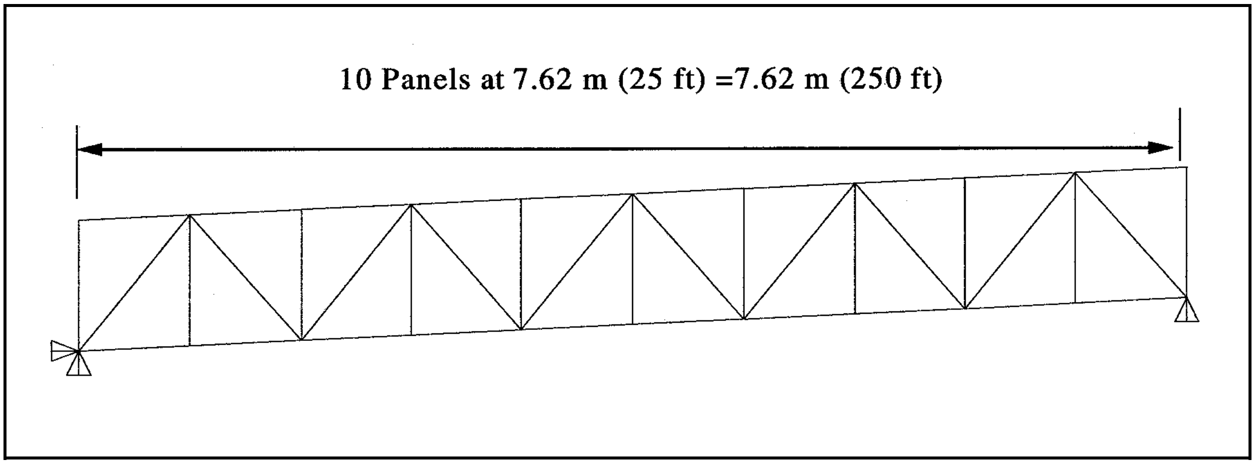


Figure 2-5. Summit Bridge (single-span deck truss)

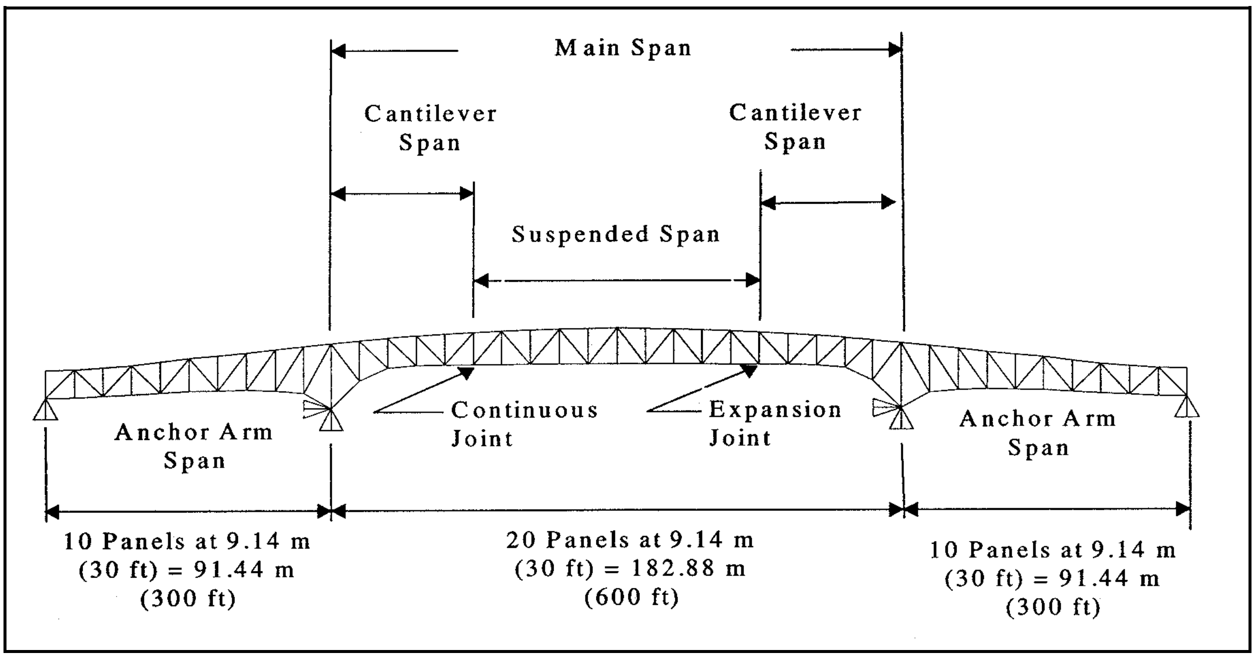


Figure 2-6. Summit Bridge (mid spans)

joints connected to the bridge deck. A concentrated live load of 115.65 kN (26 kips) plus the (111.2-kN (25-kip)) impact load were positioned at one truss joint connecting to the deck and were moved from one end of the bridge to the other end; then, the tension members were recorded. Since this is a determinate bridge, all the tension members recorded are FCMs. The results are shown in Figures 2-7 and 2-8.

c. Example 2 is St. George's Highway bridge, a tied-arch single span (164.59 m (540 ft)) bridge located in Delaware and Maryland crossing the Chesapeake and Delaware Canal. Since this bridge is not a truss bridge, Equation 2-1 does not apply. The finite element model is shown in Figure 2-9. St. George's Highway bridge is designed for H20-44 standard loading. As used for the Summit Bridge analysis, the same lane loading

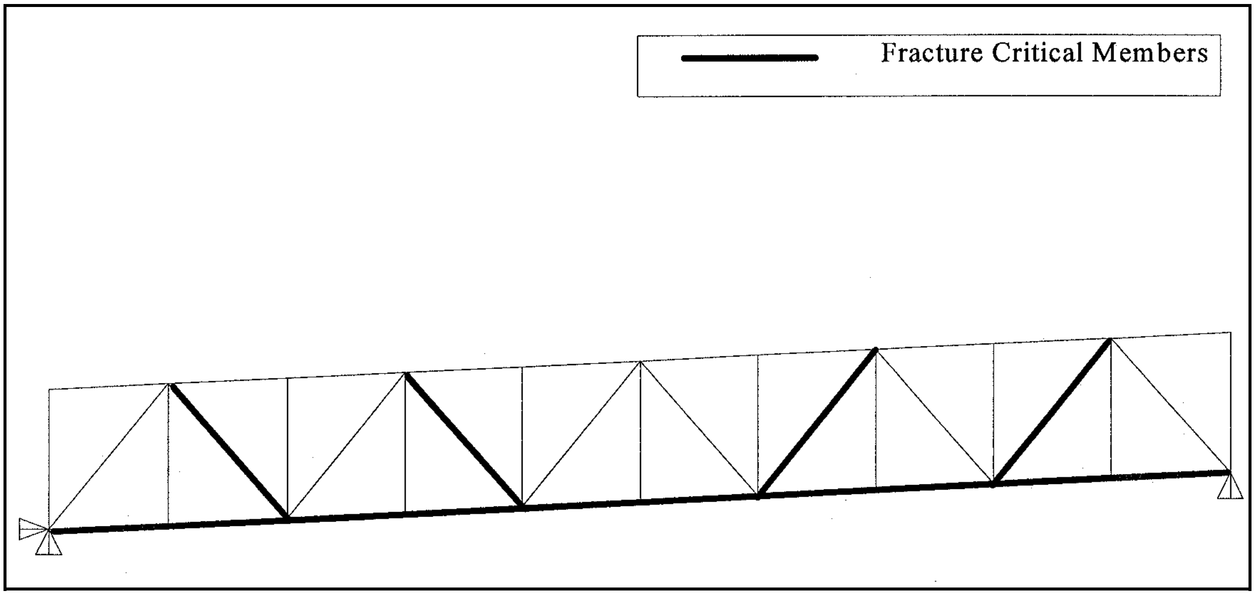


Figure 2-7. Summit Bridge (FCM in the simple span)

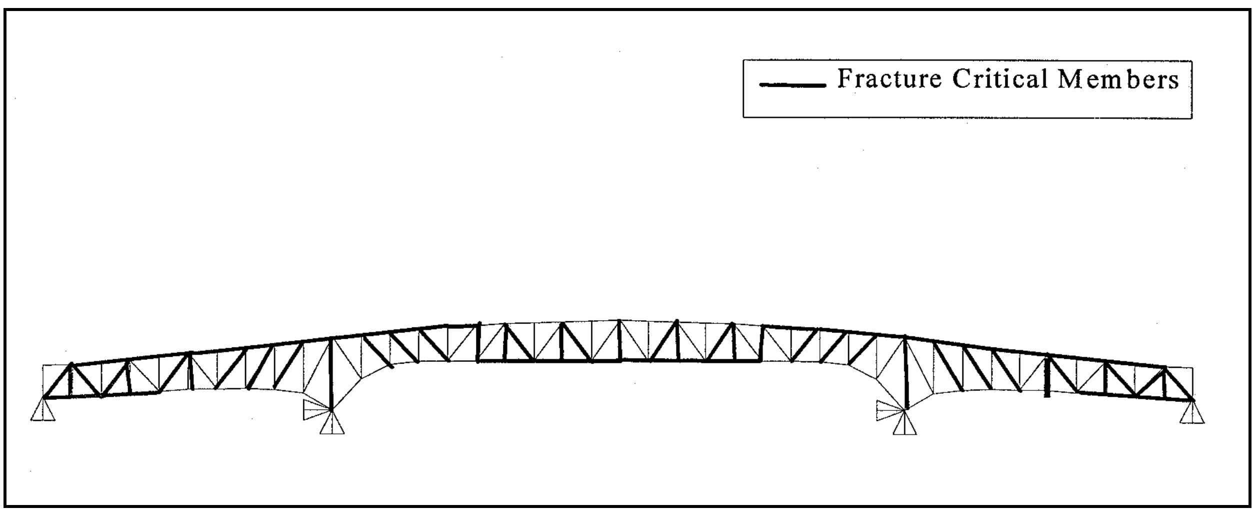


Figure 2-8. Summit Bridge (FCM in the mid spans)

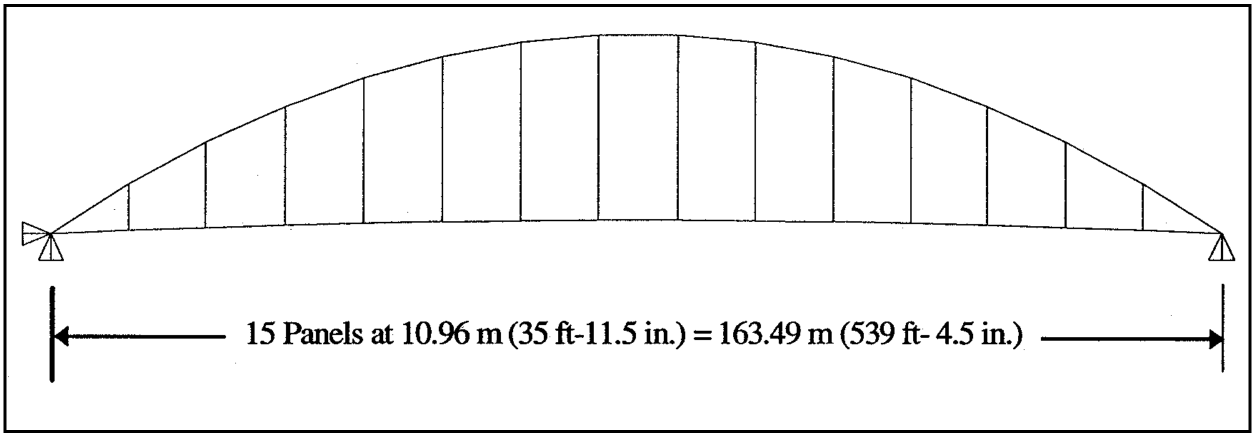


Figure 2-9. St. George's Highway Bridge (tied arch span)

(HS20-44) was applied to this bridge. After the dead loads were applied to each bridge member and the concentrated load was moved to all the deck joints, the tension members were identified (USACE 1956). For a concentrated loading position, each tension member was individually removed to determine if the redistribution of the load caused any remaining members to reach yield stress. If yielding occurred in the remaining members, the tension member removed was considered to be a FCM. If yielding did not occur, the removed tension member was considered a redundant member. The concentrated live load was then moved to the next deck joint. The process was repeated until all the FCMs were identified. The FCMs for the St. George's Highway Bridge are shown in Figure 2-10.

2-4. Guidance for Locating FCMs

a. From the results shown for the simple-span deck truss, it can be observed that the bottom chord must be composed of tension members because it stretches as the span bends. The diagonal truss members may be in tension or compression. Harland et al. (1986) proposed that, for controlling loads uniformly distributed across the span length, diagonals pointing upward towards the truss mid-span are subject to compression, while diagonals pointing upward away from the mid-span are subject to tension. The results from the Summit Bridge analysis shown in Figure 2-7 support Harland's proposition.

b. From the results shown in Figure 2-8, the top chord is in tension in the area over the piers. In the area near the end support (abutment), the truss is similar to the simple spans; therefore, the bottom chord is in tension. However, when using visual inspection of the framing arrangement, there are transient zones in which it is not obvious if the members are in tension or compression. The FCMs in these zones become obvious by analysis using the procedure outlined in Figure 2-3.

c. The suspended span for Summit Bridge acts as a simple span; therefore, the same principles as noted in paragraph 2-4*a* above apply as shown in Figure 2-8.

d. The tied-girder prevents the separation of supports; therefore, it is in tension. Any fracture in the girder will cause partial or total collapse of the bridge; therefore, the tied girder is a FCM. The members suspended from the arch are also subject to tension; however, they must be investigated to see if the failure of one suspension member could cause the remaining members to yield.

e. The guidelines set forth in this ETL can help bridge engineers to generally locate FCMs using visual observation. However, it is suggested that the procedures shown in Figures 2-3 and 2-4 be used to specifically identify the FCMs. FCMs should be identified during initial bridge design and documented as part of the permanent design file.

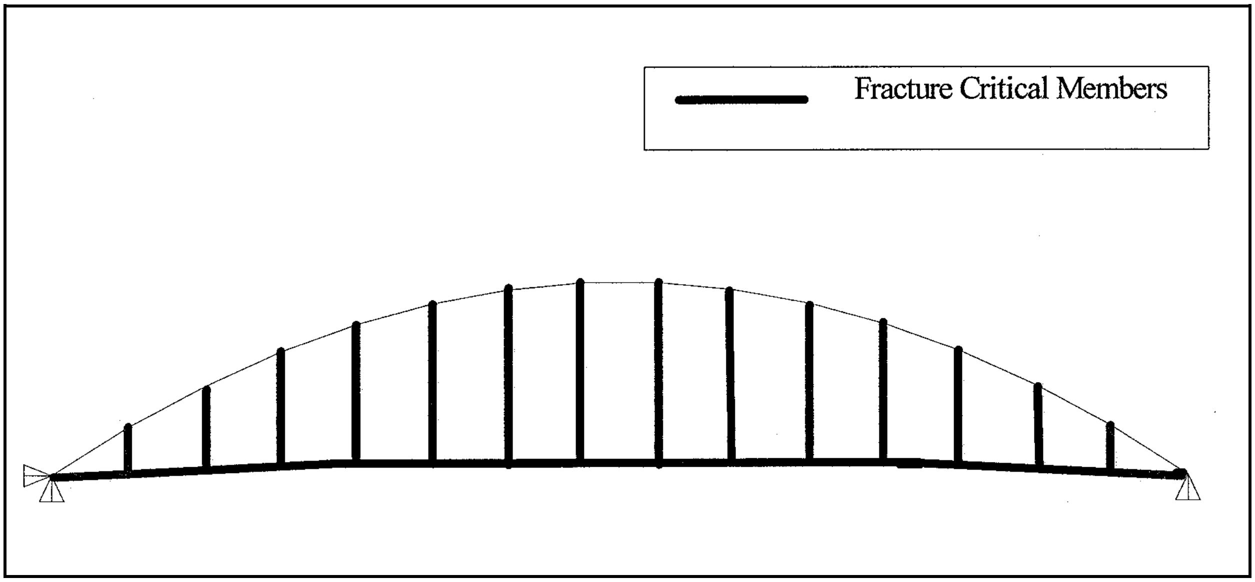


Figure 2-10. St. George’s Highway Bridge (FCM in the tied arch span)

Chapter 3
Inspection Planning and Quality Control of Fracture Critical Members

3-1. Overview

The inspection of FCMs should receive the highest priority in any bridge inspection program. Some FCMs may have details that are highly susceptible to damage due to repeated loading (i.e., fatigue), or others may be in poor condition due to corrosion or damage. Repairs and modifications can influence the likelihood of problems. The inspector should recognize that age and heavy traffic, particularly trucks, can compound problems. Inspection planning should consider the age of the bridge and traffic information if available.

3-2. Inspection Planning and Quality Control

a. Inspection planning involves having the appropriate equipment available to permit a hands-on inspection. Factors such as location, capacity, traffic, roadway width, height, and water depth must be considered in selecting access equipment. The special equipment may also require more elaborate traffic control provisions or staging.

b. The level of inspection should be tailored appropriately for the bridge being inspected. When establishing priorities for bridge inspection, consideration should be given to the age of the bridge, number of cycles since last inspection, fatigue category for connections and attachments, and extent of nondestructive testing (NDT) during the original fabrication and subsequent repairs. The bridges should be categorized and ranked in order of criticality so that the resources available for the inspections are used to provide the highest degree of safety.

(1) If it becomes necessary to establish bridge inspection priorities, a structural engineer with experience in both load rating and evaluating the types of bridges being considered should be involved in the process. Several things influence relative criticality:

(a) The degree of redundancy.

(b) The live load member stress.

(c) The propensity of the material to crack or fracture.

(d) The condition of specific FCMs.

(e) The existence of fatigue-prone design details.

(f) The previous and predicted number and size of loads.

(2) Stress analysis using the finite element method, coupled with fracture mechanics analysis and materials testing may have to be pursued to identify the structural criticality if such condition is not easily determined.

3-3. NDT and Evaluation

a. There are a number of NDT methods available for quantifying the distressed condition of a FCM. No single test will meet all the needs for a given circumstance, and in many cases it will be necessary to use one or more of these tests in conjunction with another. When NDT is required, the testing must be performed by a person fully qualified in its use (e.g., ASNT Certified inspector). NDT can be conducted using the appropriate process and procedure applicable to the specific conditions being evaluated. The NDT processes commonly used for bridge inspection include visual testing (VT), dye penetrant testing (PT), magnetic particle testing (MT), and ultrasonic testing (UT). Radiographic testing (RT) and eddy current testing (ET) are not common for field applications. These test processes and procedures are covered in detail in American Welding Society (AWS 1985).

b. Serious problems discovered in FCMs must be addressed immediately. This should include closing the bridge if the condition warrants. Less serious problems may require repair, retrofit, or partial closure of the bridge. The inspection results may find that the distress condition of a FCM is subcritical. However, problems may develop slowly over a period of time. The subcritical cracks may grow to a critical length (as discussed in Chapter 4), at which time catastrophic structural failure may occur suddenly. Therefore, periodic inspections and evaluations of FCMs are directed at determining the overall condition of the bridge and identifying potential problem areas before they reach a critical level. To ensure bridge safety, it is important that periodic inspections be performed to ensure that cracks are detected before reaching critical size. Periodic inspections should correlate with expected crack growth rates.

3-4. Guidance for Field Inspection

a. In general, field inspections can be divided into two stages, a scheduled visual inspection and a detailed inspection for structural evaluations. Distressed FCMs or an open surface crack length at least twice the joint thickness can usually be detected by visual inspection without using a magnifying glass or removing the surface coating. Intervals for scheduled visual inspections are in accordance with ER 1110-2-111.

b. If distress indications are found in FCMs by initial VT inspection, detailed inspections must be performed. Paint, corrosive oxides, dirt, debris, grease, and other surface materials on the member must be removed before more detailed PT, MT, or UT inspections can be scheduled to determine additional information pertaining to the conditions of the distress members. A fracture and fatigue analysis can also be performed at this stage to help evaluate how fit the bridge is for service.

c. Retrofitting or replacement of the distressed FCMs must be scheduled immediately if the analysis results indicate bridge failure is imminent.

Chapter 4
Engineering Critical Assessment Procedures

4-1. Overview

When inspections reveal cracks, it is necessary to establish acceptance levels to determine if immediate repairs are needed to prevent fracture. The critical crack size may be determined through a fracture mechanic's evaluation for a given set of loads, environmental factors, geometry, and material properties. If the crack size is less than the critical dimension, the expected remaining life and rate of crack propagation may be determined by a fatigue analysis. The engineering decision on appropriate repair or planned maintenance is based on the concept of fitness-for-service of the distressed bridge (International Institute of Welding 1990). These analysis procedures are called Engineering Critical Assessment (ECA) procedures.

4-2. Fracture Behavior of Steels

a. The service temperature under which a steel bridge operates has a significant effect on the fracture behavior of the steel. The critical fracture stress remains unchanged by temperature for a given crack size if the service temperature is below a transition temperature, called nil ductility transition temperature. The critical stress decreases as the crack size increases and is inversely proportional to the square root of the crack size. Above the transition temperature, fracture stress of steels becomes less dependent on the crack size. As the material temperature increases, the fracture stress eventually reaches the yield strength, and then the ultimate strength, regardless of the crack size.

b. As the service temperature decreases, for low and intermediate strength steels, the material changes from ductile fracture behavior to brittle fracture behavior at the nil ductility transition temperature. Considering constraint, the appropriate fracture parameter, K_{Ic} (critical stress intensity factor under plane strain condition) or crack tip opening displacement (CTOD) can be selected for evaluating the fracture behavior of the bridge material. Those fracture parameters are defined and discussed in detail by Barsom and Rolfe (1987).

4-3. Fracture Analysis Procedure

a. For bridges containing cracks and operating below the nil ductility transition temperature, linear elastic fracture mechanics analysis can be used to assess the cracks revealed from inspections. For bridges with cracks operating at temperatures above the transition temperature, elastic-plastic fracture analysis must be conducted. Fatigue growth rates must also be considered when developing the inspection and maintenance scheduling for distressed bridges. This section presents a procedure for fracture analysis of FCMs.

(1) For brittle fracture analysis, the stress intensity factor (K_I) shall always be less than the critical stress intensity factor (K_{Ic}). The critical crack size (a_{cr}) is related to material fracture toughness (K_{Ic}) for a given applied load and loading rate at the minimum service temperature as follows:

$$a_{cr} = [K_{Ic} \ / \ (F.S. \ \beta \ \sigma)]^2 \qquad (4\text{-}1)$$

where

a_{cr} = critical crack size in inches

K_{Ic} = fracture toughness of the bridge material in ksi times square root of inches

$F.S.$ = appropriate factor of safety (e.g., 2)

β = constant which is a function of crack and joint geometry, loading type, and welding-induced residual stress

σ = applied nominal stress in ksi

(2) For ductile fracture analysis, CTOD is usually used to calculate crack criticality. An effective crack parameter, equivalent to the through thickness dimension of the joint which would yield the same stress intensity factor as the actual crack under the same load, is used to compare with the critical CTOD values of the bridge material. This effective crack parameter shall not be greater than the critical CTOD.

b. The procedure for fracture assessment of cracks is discussed by Tsai and Shim (1992) and is summarized below:

(1) Determine the actual shape, location, and size of the discontinuity by NDT inspection.

(2) Determine the effective crack dimensions to be used for analysis. Cracks are classified as through thickness (may be detected from both surfaces), embedded (not visible from either surface), or surface (may be observed on one surface). Through thickness cracks may be detected and defined by visual, dye penetrant, magnetic particle, or ultrasonic methods. Embedded cracks may be detected by ultrasonic and possibly radiographic methods. To determine the effective dimensions of a single crack or multiple cracks:

(a) Resolve the crack(s) into a plane normal to the principle stresses.

(b) Determine the effective dimensions for various isolated cracks. Check interaction with neighboring cracks to obtain the idealized crack dimensions.

(c) For surface or embedded cracks (idealized or actual), check their interaction with surfaces by recategorization.

(d) Determine final idealized effective dimensions for fracture analysis.

(3) Determine material properties including yield stress, Young's Modulus and K_{Ic} or CTOD. K_{Ic} may be estimated from Charpy V-Notch test (CVN) by Barsom's two-stage transition method (Barsom and Rolfe 1987) if direct K_{Ic} test data is not available.

(4) Idealize the total stresses by dividing them into primary stress, σ_p, and secondary stress, σ_s. The primary stress consists of membrane stresses, σ_m, and bending stress, σ_b, which include the effect of stress concentration imposed by geometry of the detail under consideration. Examples of the secondary stress include stress increase at re-entry angles in the joint, thermal, and residual stress. For cracks in welds, the residual tensile stress should be taken as yield stress. An estimate of the residual stress should be appropriate for post heat-treated weldments.

(5) Perform fracture assessment to determine the critical crack size. If applied stress is greater than the yield stress, CTOD must be employed. If applied stress is less than the yield stress and the plane strain factor $\beta_{Ic} < 0.4$ (Irwin's plane strain condition for brittle fracture), analysis must be based on K_{Ic}. When applied stress is less than the yield stress and $\beta_{Ic} > 0.4$, K_c should be used instead of K_{Ic}.

(6) If the crack is subcritical, determine the remaining life using a fatigue analysis procedure. The upper limit for the brittle fracture behavior (plane strain behavior) is:

$$K_{Ic}/\sigma_{ys} \; = \; (t/2.5)^{0.5} \tag{4-2}$$

c. When this upper limit is exceeded, extensive plastic deformation occurs at the crack tip (crack tip blunting), and a nonlinear elastic plastic analysis must be used to assess the crack. CTOD is appropriate for this type of fracture analysis. The CTOD analysis procedure can be summarized as follows:

(1) Determine the effective crack parameter ($\bar{a}$).

(a) For through thickness crack (= ι/2) where ι is the crack size.

(b) For surface crack, $\bar{a}$ is determined from Figure 4-1.

(c) For embedded crack, $\bar{a}$ is determined from Figure 4-2.

(2) Determine allowable crack parameter a_m, which can be calculated by

$$\bar{a} \; \bar{a}_m \; = \; c \left(\frac{\delta_{crit}}{\varepsilon_y} \right) \tag{4-3}$$

where

δ_{crit} = critical CTOD

ϵ_y = material yield strain

The constant *c* is determined from Figure 4-3. In determining *c*, if the sum of the primary and secondary stresses, excluding residual stress, is less than $2\sigma_{ys}$, the stress ratio $(\sigma_p + \sigma_s)/\sigma_{ys}$ is used as the abscissa in Figure 4-3. If this sum exceeds $2\sigma_{ys}$, an elastic plastic stress analysis should be carried out to determine the maximum equivalent plastic strain which would occur in the region containing the crack if the crack was not present. The value of *c* may then be determined using the strain ratio, $\varepsilon/\varepsilon_y$, as the abscissa in Figure 4-3.

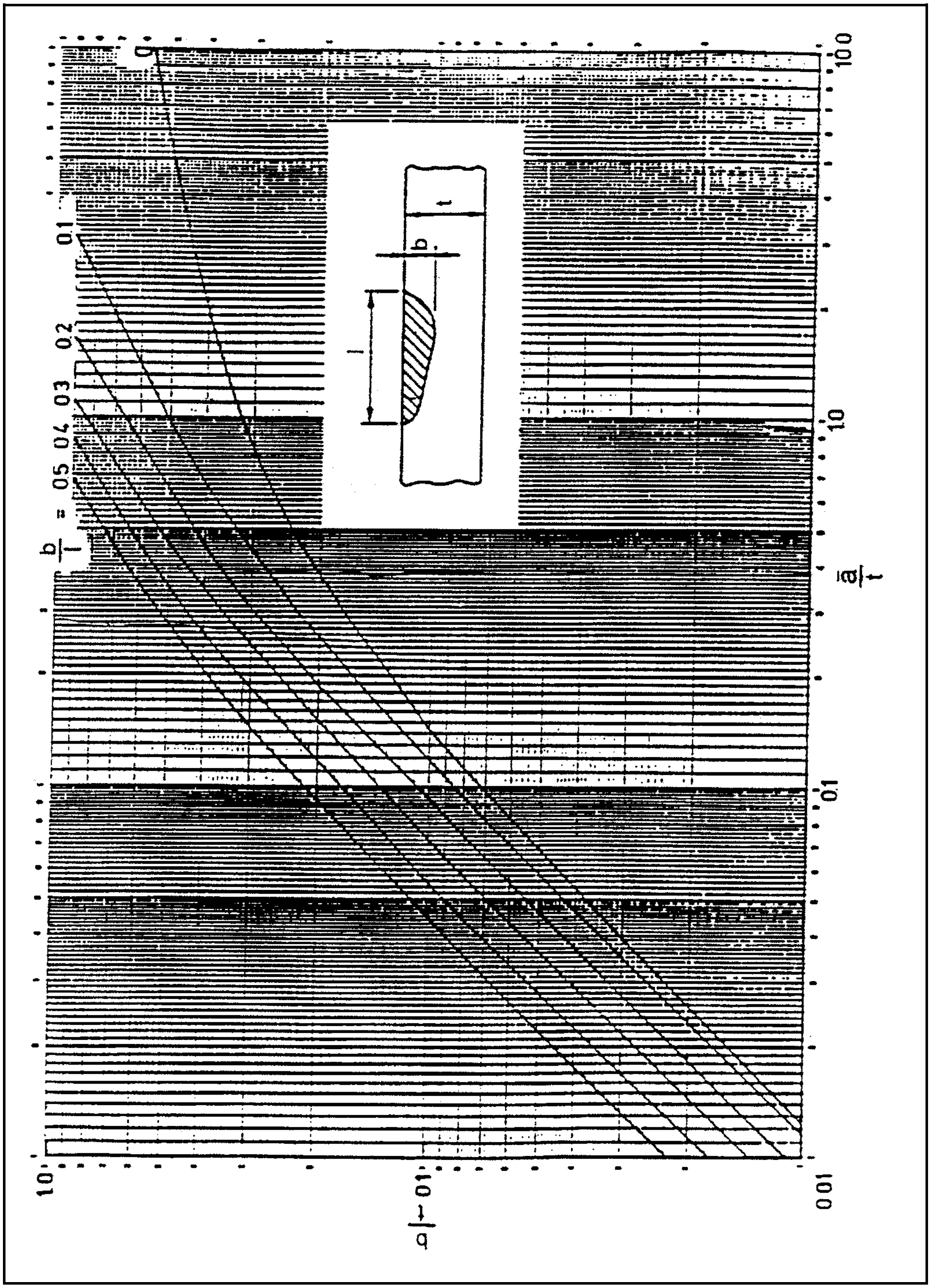


Figure 4-1. Relation between dimensions of a discontinuity and the parameter *a* for surface discontinuities

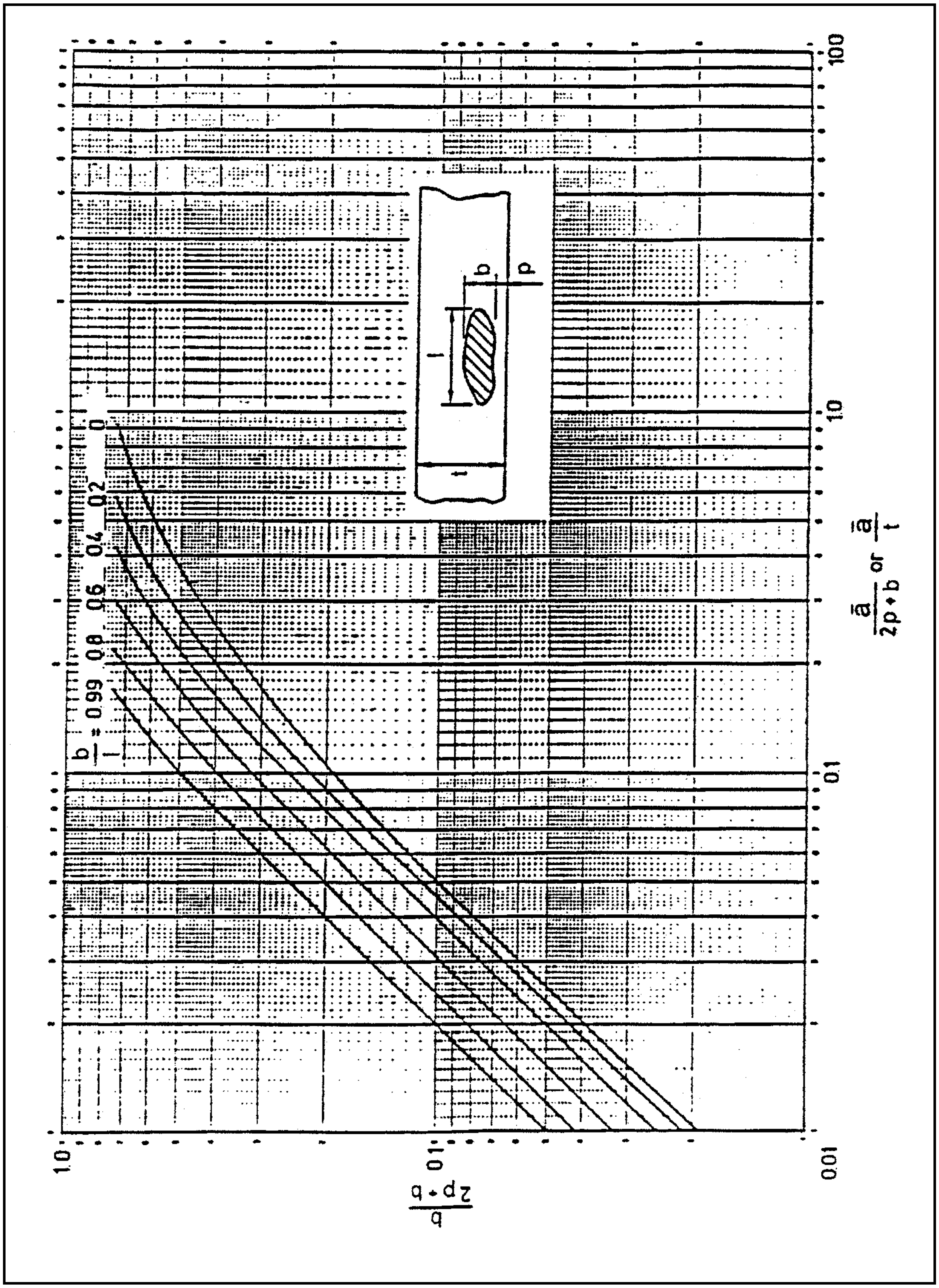


Figure 4-2. Relation between dimensions of a discontinuity and the parameter for embedded discontinuities

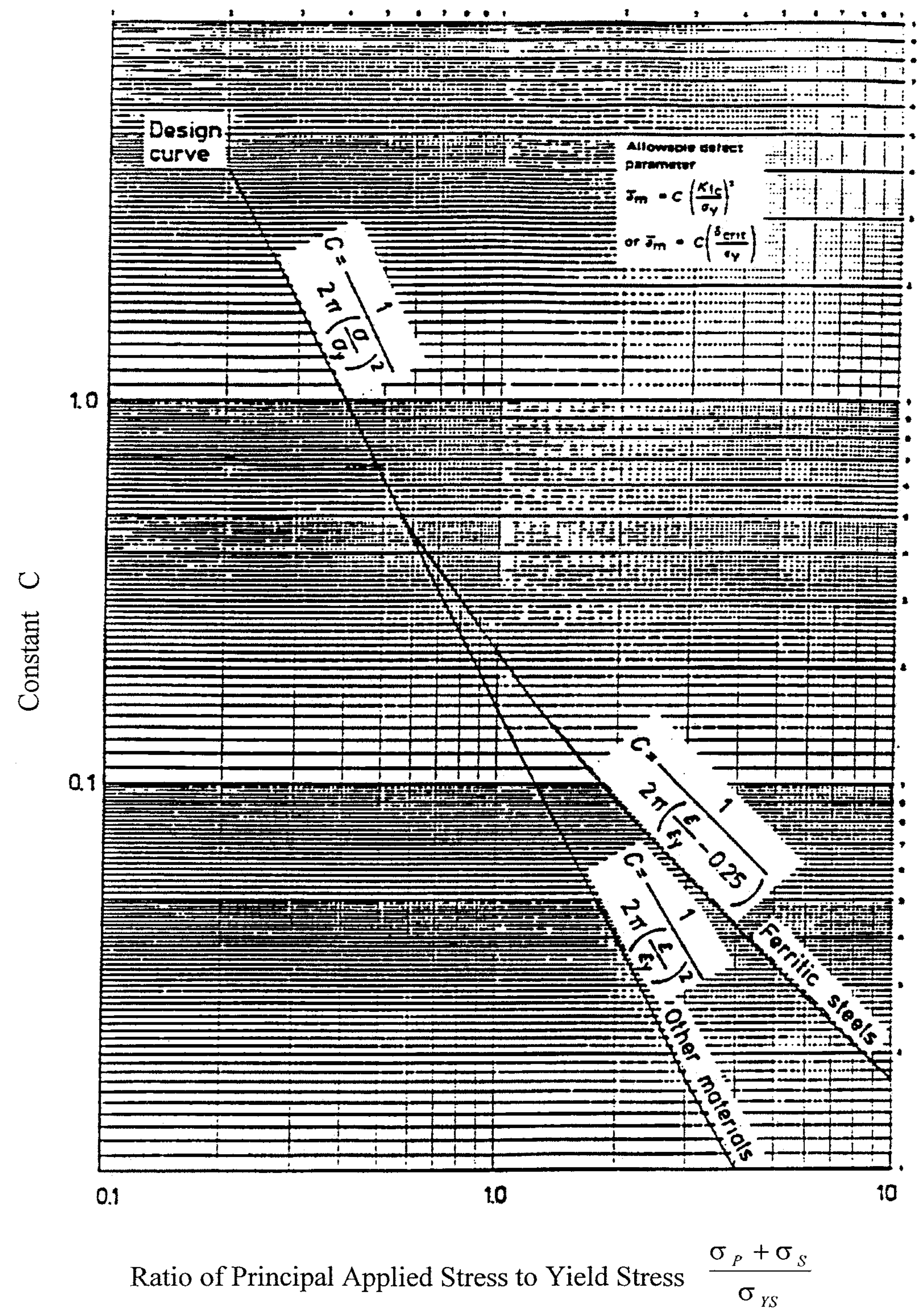


Figure 4-3. Values of constant *c* for different loading conditions

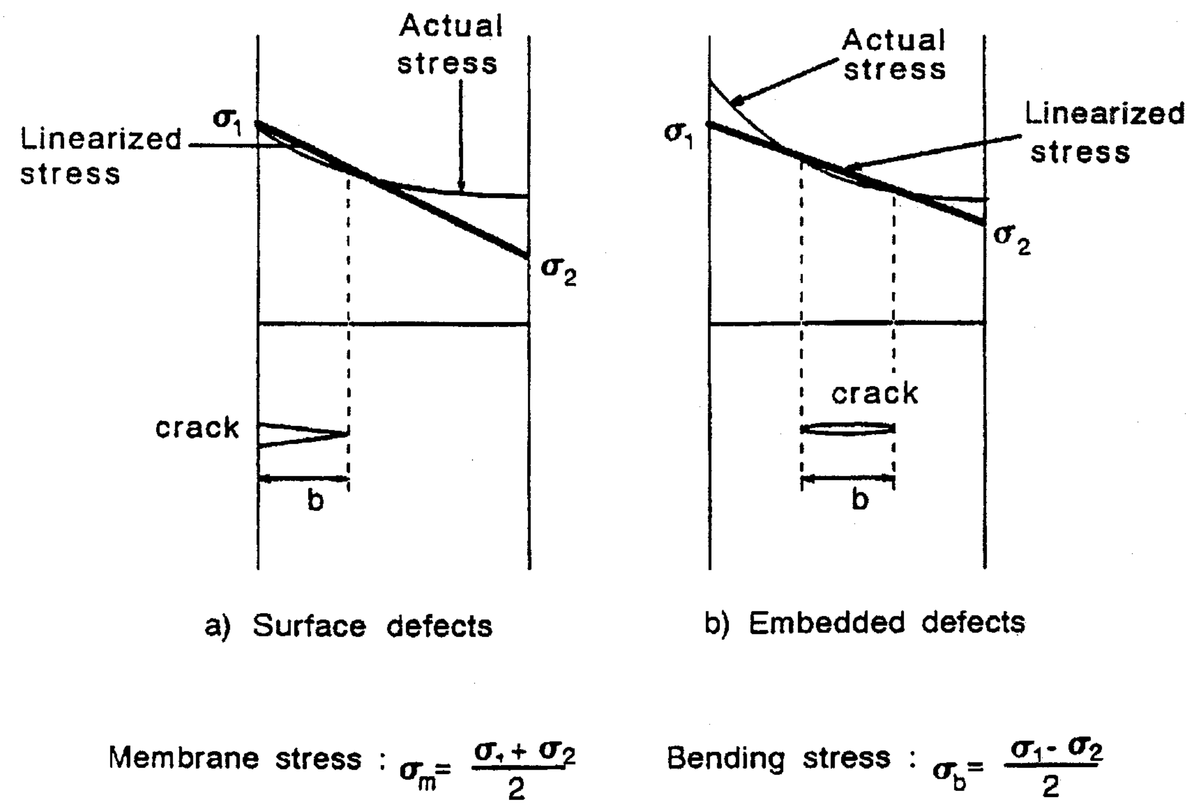


Figure 4-4. Linearization of stresses

(3) If the effective crack parameter, *a*, is smaller than the allowable crack parameter, $\bar{a}_m$, then the crack is considered stable under static loading. Using the procedure described in the second step above results in a safety factor of approximately 2.5 in determining $\bar{a}_m$. Therefore, the calculated critical crack size would be equal to 2.5 $\bar{a}_m$.

4-4. Fatigue Analysis Procedure

a. Dependent on the nature and fabrication of the joint detail, the joint fatigue characteristics are represented by the S-N curve of the appropriate category. While S-N curves are referenced to constant amplitude stress cycles, the stress cycles experienced by actual bridge structural details vary insignificantly in normal bridge operations. An equivalent constant stress range would cause the same damage and fatigue life as the actual stress range spectrum experienced in the field.

b. To evaluate the fatigue safety of an existing bridge structure, the maximum stress range should first be compared with the fatigue limit for the detail in question. Fatigue limit is defined as the constant amplitude stress range with which the detail can endure an unlimited number of cycles without developing fatigue cracks. If the maximum stress range is greater than the fatigue limit, fatigue cracking is expected after a number of stress cycles. The total fatigue life of the detail may be tens of millions of stress cycles, but not unlimited. Further application of loading after crack initiation would cause the crack to extend. Only after significant crack growth is the situation likely to become critical to the extent that the crack would become unstable and failure would occur. In general, fatigue cracks usually exist in structural members adjacent to weld toes.

c. The design S-N curves for various steel joint details are specified in the AWS D1.1 Structural Welding Code (American Welding Society 1992). Figure 4-5 shows a summary of fatigue categorization for various details of nonredundant structures used by the AWS welding code. To ensure conservative fatigue assessments, the code uses a mean minus 2 standard deviations (i.e., 97.7 percent survival probability) as the lower bound S-N curves for design purpose. The design S-N curves can be expressed as

$$\log N = \log A + m \log S \tag{4-4}$$

or

$$N = A\,S^{m} \tag{4-5}$$

where

m = inverse negative slope of the S-N curve

$\log A$ = intercept of the $\log N$ axis

S = full stress range in ksi (i.e., applied maximum nominal stress minus applied minimum nominal stress)

N = fatigue life in number of cycles

d. Redundant structures are those structures using redundant structural members. Failure of these members will not cause catastrophic structural failure. Therefore, the S-N design curves use a mean minus one standard deviation (i.e., 84.1 percent survival probability) as the lower bound for design purpose. Secondary bridge members, such as stiffeners, may apply the redundant structure S-N curves to assess the connection fatigue categories. However, fatigue cracks do not usually occur in these secondary stiffening members. For application simplicity, the nonredundant structure S-N curves are used for assessing the entire bridge structures.

e. Six fatigue categories are defined by the AWS D1.1-Structural Welding Code (American Welding Society 1992) for different joint details and stress types. Category F is for shear stress only and in most cases is used to categorize fillet welds. The AWS fatigue categories for redundant and nonredundant structures are shown in Figure 4-6 and the constants are summarized in Table 4-1.

f. The S-N curve design procedure is relatively simple to apply. However, this approach has some disadvantages. For example, S-N curves do not separate the stages of crack initiation and crack growth, the plasticity effects cannot be quantified, although they are included in the test data; and the local stress-strains at the weld toe are unknown where fatigue crack will inevitably initiate. Therefore, the S-N curve design procedure is used to plan a strategy for scheduled inspection and evaluation only. For those members found with cracks during the scheduled inspection, fracture mechanics and fatigue theory must be applied to estimate the remaining life of the distressed bridge members.

g. An accurate estimation of the number of stress cycles experienced to-date by a bridge structure requires knowledge of the operating history of the bridge. An average daily operation (ADO) curve may be established based on the bridge operating history. A possible source for historical operating information is an onsite operational control device, if one exists. The area under the ADO represents the total number of stress cycles experienced by the bridge within a specified period of time. Dependent on the operational characteristics of the bridge, each event may cause one or more stress cycles at a given detail. For example, vibration of the bridge may occur if it is harmonic with the vehicle crossing frequency. This vibration may induce more than one stress cycle at joint details. The natural frequency of the bridge should also be considered when estimating the number of stress cycles.

h. Occasionally, due to collision, (e.g., barge impact, falling ice or debris) during bridge operation, overload may occur in the bridge. This occasional overload may cause brittle fracture of cracked members. Therefore, brittle fracture should be considered when infrequent overload is possible. For frequent overload occurrence, the cumulative damage must be considered in the fatigue analysis. The root-mean-cube effective stress range may be used to estimate the total fatigue life using the constant-amplitude S-N design curves. To estimate the effect of known overloading history, Barsom's root-mean-square crack propagation model (Barsom and Rolfe 1987) may be used to estimate the remaining life of the cracked members.

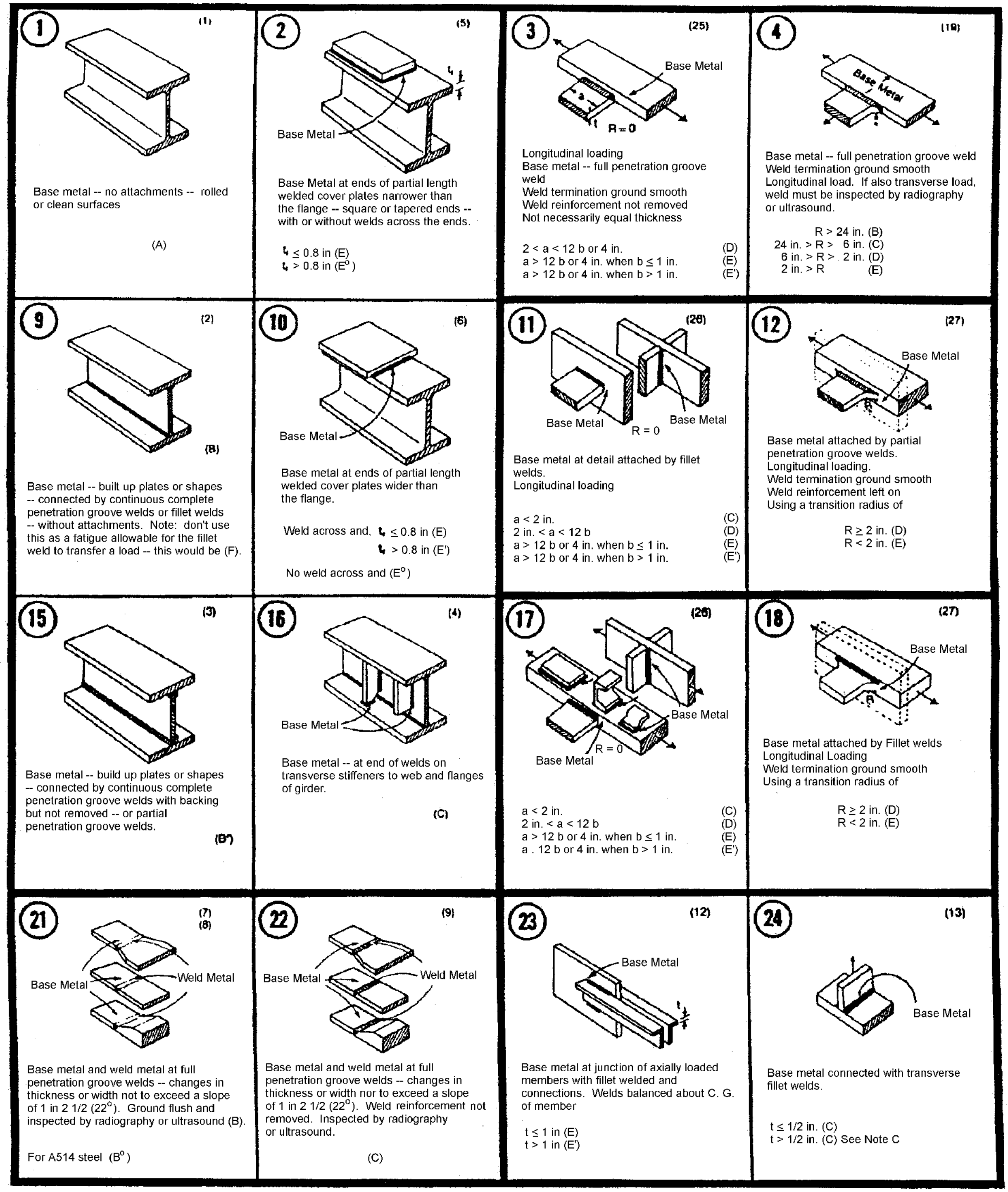
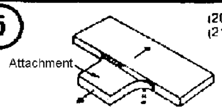


Figure 4-5. Summary of fatigue categorization for nonredundant structure details. Personal Communication from Omer W. Blodgett to Dr. Chon Tsai, Ohio State University, Columbus, OH (continued)

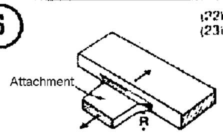
5 (20) (21)



Transverse loading
Attachment -- Full penetration groove weld
Weld termination ground smooth
Equal Thickness

	Weld Reinforcement Removed	Weld Reinforcement Not Removed
R < 24 in.	B	C
24 in. < R < 6 in.	C	C
6 in. < R < 2 in.	D	D
2 in. < R	E	E

6 (22) (23)

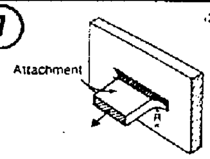


Transverse loading
Attachment -- full penetration groove weld
Weld termination ground smooth
Unequal thickness
Weld reinforcement removed
R > 2 in. (D)

R ≤ 2 in. (E)

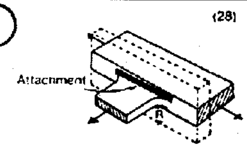
Weld reinforcement not removed
any R (E)

7 (24)



Transverse loading
Attachment — full penetration groove weld
Weld termination ground smooth
Weld reinforcement not removed
R > 6 in (C)
6 in > R ≥ 2 in (D)
R < 2 in (E)

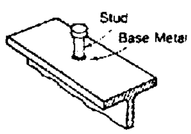
8 (28)



Attachment — Transverse load
Fillet welds
Weld termination ground smooth
Main material longitudinal loaded
Using a transition radius of

R ≥ 2 in (D)
R < 2 in. (E)

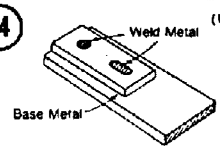
13 (29) (30)



Base metal at stud — type shear connector — fillet weld attachment or automatic end weld (C)

Shear stress on nominal area of stud-type shear connectors (F)

14 (15)



Base metal at plug or slot welds
(E)

Shear on plug or slot welds
(F)

Allowable Stress Range, Ksi

Category (from Table A-K4.2)	20,000 to 100,000 ~	100,000 to 500,000 ~	500,000 to 2×10^6 ~	Over 2×10^6 ~
A	63	37	24	24
B	49	29	18	16
B'	39	23	15	12
C	35	21	13	10[a]
D	28	16	10	7
E	22	13	8	5
E'	16	9	6	3
F	15	12	9	8

[a] Flexural stress range of 12 ksi permitted at toe of stiffener welds on flanges.

allowable fatigue stress $\boxed{\sigma_{max} = \frac{\sigma}{1 - K}}$ for normal stress

$\boxed{\tau_{max} = \frac{\tau}{1 - K}}$ for shear stress

but shall not exceed steady allowables

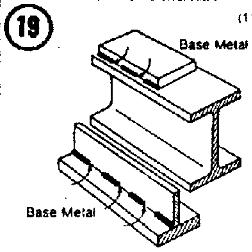
σ_{max} or τ_{max} = maximum allowable fatigue stress

σ_{sr} or τ_{sr} = allowable range of stress from table

$$K = \frac{\sigma_{min}}{\sigma_{max}} = \frac{M_{min}}{M_{max}} = \frac{F_{min}}{F_{max}} = \frac{\tau_{min}}{\tau_{max}} = \frac{V_{min}}{V_{max}}$$

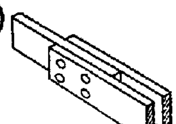
S = shear
T = tension
R = reversal
M = stress in metal
W = stress in weld

19 (11)



Base metal at intermittent fillet welds
(E)

20 (16) (17) (18)

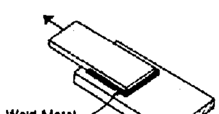


Base metal at *gross section* of high strength bolted *slip-critical* connections — No out of plane bending (B)

Base metal at *net section* of fully tensioned high strength bolted *bearing* connections (B)

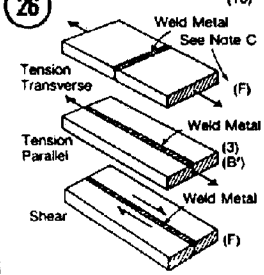
Base metal at net section of other mechanical fastened joints (D)

25 (14)



Weld metal of continuous or intermittent longitudinal or transverse fillet welds
(F)

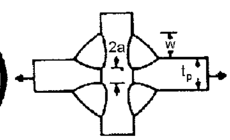
26 (10)



Partial penetration groove welds

NOTE C

$$\sigma_{sr} = \tau_{sr}^{c}\left(\frac{0.71 - 0.65\frac{2a}{t_p} + 0.79\frac{W}{t_p}}{1.10\,t_p^{1/6}}\right)$$



for fillet welds $\frac{2a}{t_p} = 1.0$

$$\left(\frac{0.06 + 0.79\frac{W}{t_p}}{1.10\,t_p^{1/6}}\right)$$

but $\sigma_{xr} \le \tau_{xr}^{c}$

σ_{sr} = stress range for this condition
τ_{sr}^{c} = stress range for category (C)
W = leg size of fillet weld
2a = lack of penetration into joint
t_p = plate thickness

Figure 4-5. (Concluded)

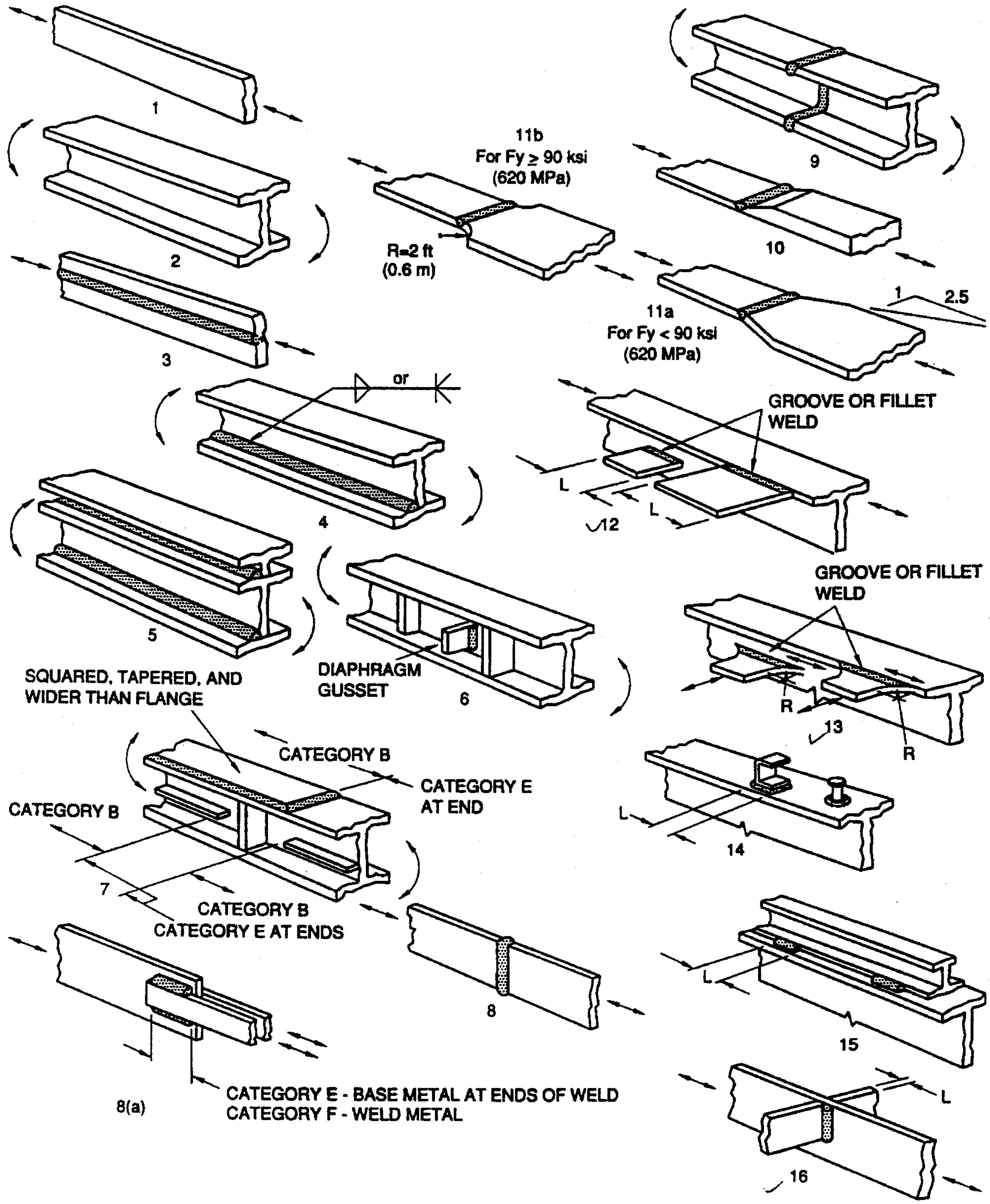


Figure 4-6. Fatigue categorization for nonredundant and redundant structure detail

Table 4-1
Fatigue Categories for Redundant and Nonredundant Structures

Category	Constant m	Constant A, cycles	Fatigue Limit, ksi
Nonredundant Structures			
A	-4.76	1.800×10^{12}	22.0
B	-3.73	2.044×10^{10}	15.8
C (stiffeners)			
12.5 > S > 10.9	-18.18	3.666×10^{25}	10.9
19.0 > S > 12.5	-5.93	1.706×10^{12}	10.9
C (other attachments)	-5.93	1.706×10^{12}	9.0
D	-3.75	2.945×10^{9}	4.7
E	-3.85	1.392×10^{9}	2.4
F 9.0 > S > 7.0	-9.88	1.359×10^{15}	7.0
F S > 9	-6.74	1.359×10^{12}	7.0
Redundant Structures			
A	-3.32	8.070×10^{10}	21.7
B	-3.11	1.466×10^{10}	15.5
C (stiffeners)	-3.29	7.729×10^{9}	11.3
C (other attachments)	-3.29	7.729×10^{9}	10.0
D	-2.98	1.914×10^{9}	7.0
E	-2.99	9.817×10^{8}	5.0
F	-5.68	4.840×10^{11}	8.0

i. The cumulated fatigue damage degree (i.e., without crack found in the member) is estimated by comparing the cycles to date with the total fatigue life. The difference between these two values is the remaining fatigue life. The remaining fatigue life converted into a length of time is dependent upon the projected ADO curve. A scheduled inspection and evaluation plan for the bridge can be developed based on the projected remaining fatigue life.

j. A practical procedure for the estimation of fatigue life can be summarized as follows:

(1) Examine the structural detail in question and determine its fatigue category.

(2) Estimate the maximum full stress range, which must reflect the extreme stress values caused by overloads.

(3) If the maximum full stress range does not exceed the fatigue limit of the structural detail in question, fatigue cracking is unlikely to occur. The fatigue life is taken as infinite. Additional assessment is unnecessary at this time.

(4) If the maximum full stress range exceeds the fatigue limit of the structural detail in question, the fatigue life is not infinite, and the risk of fatigue cracking must be assessed. The total fatigue life is determined using the appropriate S-N relationship.

(5) Use the ADO information to determine the stress cycles to date and the remaining fatigue life. Use projected ADO information to convert the remaining life cycles to number of years. If the remaining fatigue life is judged to be inadequate, retrofitting or strengthening measures should be considered to extend the bridge life.

4-5. Prediction of Crack Growth

a. Fatigue is a process causing cumulative damage from repeated loading. Fatigue damage occurs at stress concentrated regions where the localized stress exceeds the material yield stress. After a certain number of load cycles, the accumulated damage results in crack initiation, as well as propagation. Fatigue life is the sum of the total number of cycles required to initiate a crack and propagate the crack to failure.

$$N_T = N_i + N_p \tag{4-6}$$

where

N_T = total number of life cycles

N_i = initiation life

N_p = propagation life

Fatigue assessment is performed to determine the remaining life of a bridge.

b. A crack under repeated loading could be a nonpropagating crack. Tensile plastic strains developed at the crack tip during the initial tensile loading can result in compressive residual stresses upon unloading. If subsequent tensile loading is not sufficient to reopen this closed crack tip, the crack will not grow. Therefore, for a crack to propagate, the stress intensity factor must exceed a threshold value. The threshold values given below are applicable to martensitic, bainitic, ferrite-pearlite, and austenitic steels, which are the primary bridge steels (Barsom and Rolfe 1987).

$$\Delta K_{th} = 6.4(1 - 0.85R)\,\text{ksi sqt(in) for } R > 0.1 \tag{4-7}$$

$$\Delta K_{th} = 5.5 \text{ ksi sqt(in) for } R < 0.1 \tag{4-8}$$

where

R = the fatigue ratio which can be defined as the ratio of minimum stress to the maximum stress

ΔK_I = stress intensity factor range which is determined using the full applied stress range (i.e., the maximum stress minus the minimum stress) for welded structures

c. The crack will propagate according to Paris's power law of propagation if the stress intensity factor range is greater than the threshold value (Barsom and Rolfe 1987). Ferritic-pearlitic steels such as ASTM A36 and A572 Grade 50 steels are commonly used in bridge construction. For welded steel bridges fabricated with this type of material, the following crack growth rate equation has been developed:

$$da/dN = 3.6 \times 10^{-10} (\Delta K_I)^3 \tag{4-9}$$

d. Crack growth rate accelerates as the subcritical crack approaches its critical dimension. Catastrophic fracture of the distressed bridge structural member will occur when the stress intensity factor at the maximum load reaches the critical fracture toughness value (i.e., $K_I = K_{Ic}$).

4-6. Fracture and Fatigue Assessment Procedures

a. The following fracture and fatigue procedures have been used for assessing a bridge's fitness for service (Barsom and Rolfe 1987).

(1) On the basis of the inspection data, determine the maximum initial crack size a_o present in the distressed connections and calculate the associated K_I.

(2) Knowing K_{Ic} for the material and the nominal maximum design stress, calculate the critical crack size (a_{cr}) that would cause failure by brittle fracture.

(3) Determine fatigue crack growth rate using Paris's power law.

(4) Determine K_I using the appropriate equation, the estimated initial crack size a_o, and the range of live load stress.

(5) Integrate the crack growth rate equation between the limits of a_o (at the initial K_I) and a_{cr} (at K_{Ic}) to obtain the life of the structure prior to failure. To identify inspection intervals, integration may be applied with the upper limit being the tolerable size (a_t). A safety factor of 2 may be appropriate for some applications. Another consideration to specifying a tolerable crack size is the crack growth rate (*da/dN*). The tolerable crack size (a_t) should be chosen such that the crack growth rate (*da/dN*) is relatively small and a reasonable length of time remains before the critical size is reached.

b. Large embedded cracks or surface cracks may be recategorized into an equivalent surface crack or a through-thickness crack, respectively. The crack recategorization procedure is as follows:

(1) For embedded cracks, assume that the crack grows until it reaches a circular shape. Subsequently, it grows radially and eventually protrudes a surface at which time it is treated as a surface crack.

(2) For surface cracks, the initial propagation will result in a semi-circular shape. Further propagation will result in the crack reaching the other surface, at which time it is treated as a through thickness crack.

4-7. Development of Inspection Schedule

Inspection schedules can be developed from number of cycles versus crack size curves. Figure 4-7 shows a schematic curve of the number of cycles versus crack size, which can be obtained from integrating the crack growth rate equation (West 1982). The critical crack size is determined by equating the maximum K_I to K_{Ic}. Repair will be needed before the crack grows to the critical dimension (a_{cr}). For some applications, repair might be made when the crack reaches one half the critical crack length (i.e., factor of safety 2). Inspection intervals may be determined by dividing the remaining life cycles into several intervals.

4-8. Fitness-for-Service Assessment Procedure

A bridge is fit for service when it performs the intended structural functions satisfactorily in service during its lifetime without reaching any serious limit state. Fitness-for-service is the concept of developing a maintenance schedule to ensure structural reliability for the lifetime of the structure. Some essential constituents to be considered when determining a structure's fitness-for-service include design, materials, welding, fatigue, codes and standards, reliability analysis, fracture control plans, failure modes, and the effectiveness of the quality assurance program. The fitness-for-service assessment procedure presented in this section addresses the evaluation of distressed existing bridges. The procedure consists of the following five steps:

- *Description of general concerns*. The general concerns include structural performance of the distressed bridge, consequences of failure, political and economic impact, costs for further inspection and repair, interruption of bridge operation due to further inspection or repair, and operation scheduling.

- *History review of the bridge and preliminary analysis*. This would include reviewing the design, drawings, performance functions, loading history, environmental conditions, properties of structural materials, welding procedures used, fracture control plan, and quality control documentation. Fatigue categorization of various joint details may also be necessary to select the appropriate S-N curve for life assessment, along with information pertaining to the location of FCMs.

- *Fracture and fatigue analysis*. After the bridge inspection has been performed, it may be necessary to perform fracture and fatigue analysis to determine if discontinuities are defects. The appropriate fracture criterion must be selected; idealization of the total stresses must be considered, and it may be necessary to recategorize the discontinuities identified in the field inspection. It may become necessary to calculate stress intensity factors and perform material testing to obtain information on the mechanical and chemical properties of the bridge members. For fatigue life estimation it may be necessary to use S-N curves and the Paris crack propagation law.

- *Fitness-for-service assessment*. With the analysis results and information obtained from the preceding steps, the life expectancy of the distressed bridge can be assessed based on the service requirements, as well as other considerations, such as, failure consequences and economic and scheduling impact due to repair or replacement of the distressed members. A fracture control plan can be developed at this time if one does not already exist.

- *Repair and damage control*. If the discontinuities are determined to be defects, a repair procedure must be developed to restore the distressed bridge to a level fit-for-service. A maintenance schedule must be developed based on the fracture and fatigue analysis to restore the bridge. If the discontinuities are determined to be noncritical at this time, then an inspection and evaluation schedule must be developed considering the estimated remaining bridge life and the calculated crack growth rate.

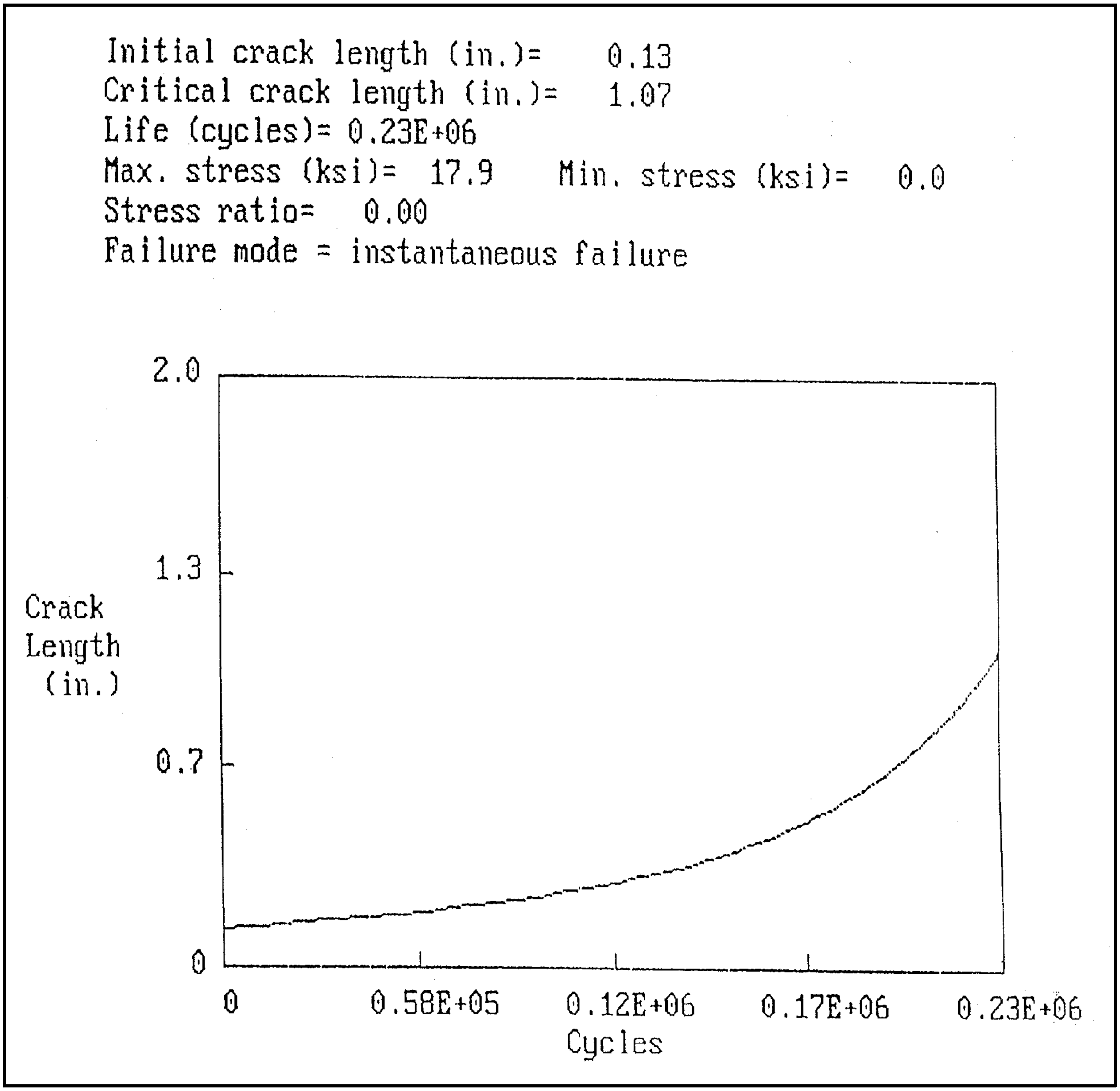


Figure 4-7. Relation between number of cycles and crack size

Chapter 5
Conclusion

This report provides information on how to identify fracture critical members in steel bridges. In addition, this report also provides guidance for effective inspection and evaluation of the fracture critical members. The engineering critical assessment procedure presented in this report can be applied to assess the overall condition of a bridge and its fitness-for-service. Appropriate application of these procedures can assure public safety of in-service bridges.