Fracture Control Considerations for Steel Bridges

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ABSTRACT
This paper presents the factors that the authors consider of primary importance in obtaining reliable steel bridges. The paper suggests that, to minimize cracking and fracture of bridge components, primary focus must be given to proper design and selection of details, and to proper exercise of quality control and assurance in fabrication and erection. The authors consider the present AASHTO fracture-toughness requirements to be adequate and that over-emphasis on fracture toughness has distracted the attention from the other factors, that are significantly more important in preventing fracture of bridge components.

The paper contains several appendices that briefly review the information presented by the FHWA in the development of their proposed fracture toughness requirements. The appendices show that inconsistent interpretation of test results, inconsistent application of fracture-mechanics concepts and incorrect conclusions on causes of actual failures in various types of steel structures were used in the FHWA - Fracture Control Plan technical notes.
INTRODUCTION

The term, Fracture Control Plan, can be defined in a general manner as an operational plan which is thought to provide adequate fracture control for certain conditions. The structures of interest here are steel bridges. Obviously, the nature of a fracture control plan for steel bridge members should be closely related to our understanding of cracking in steel bridges based upon previous experience. Much depends upon judgments as to the degree of fracture control which is adequate.

In order to specify a fracture control plan in a manner which permits judgment of its suitability, statements of objectives are the first requirement. These objectives will depend upon what is accepted as an appropriate perspective on cracking in steel bridge members. The statements of objectives should be quantitative where possible. Where complexities interfere with quantitative statements, an outline of a cost-benefit analysis which could be used as a replacement would be helpful. It would be natural to expect subdivisions of the objectives which would deal more specifically with design, fabrication, materials, inspection and maintenance.

Currently the AASHTO Specifications include two fracture control plans. The plan which applies to redundant members includes all parts of the AASHTO Standard Specifications for the Design of Highway Bridges, the AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges, and the AASHTO
Materials Specifications. The "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members" is the second plan and is a supplement to all parts of the plan covering redundant members.

A path for revisions to the two plans on an annual basis has been provided. This is through the annual meeting of the AASHTO Committee on Bridges and Structures. As new information and knowledge is gained in research, practical applications or other, it is the responsibility of the appropriate subcommittee to bring such information to the attention of the Bridge Committee for its consideration and action.

Several illustrations are desirable at this point with regard to the importance of having consensus agreement on fracture control objectives.

(1) A recent document from FHWA (SES No. 5) provides a fracture control plan for non-redundant bridge members. The fracture control methods are restricted to fabrication by welding and to material toughness requirements. With regard to fabrication by welding, the specified operational procedures are intended to minimize weld related discontinuities which might cause crack extension in the bridge members. Granting that poor control of welding has caused many cracking problems in bridge members, the value of the proposed remedies cannot be judged unless the purpose of each welding control element is clearly stated in relation to the fracture control objectives. Similar comments apply to the specified toughness requirements.
(2) One might reasonably assume that our present difficulties related to cracks in redundant steel bridges are mainly the costs of bridge repair. A hazard to public safety is, of course, involved due to increase of road accidents when traffic is restricted or re-routed. The costs of a bridge repair in terms of public inconvenience should not be ignored. On the other hand, there are those who assume the dominant current problem is primarily the hazard to public safety of bridge collapse due to fracturing. This second viewpoint is responsible, in part, for a major emphasis on a fracture control plan related to non-redundant members. However, when cracks develop in redundant members, it is rarely prudent to leave them unrepaired. Balanced and objective judgments on the importance of certain features of each viewpoint are essential in order to establish goals for an improved fracture-control plan.

(3) Many operational features of fracture control should be decided with as much assistance from cost-benefit analysis as is possible. For example, assume that flange material for certain bridge members can be purchased either as-rolled or, to meet higher toughness requirements, can be purchased heat treated. In judging the value of the higher toughness, one must bear in mind that size limitations of heat treatment facilities will reduce the plate lengths which can be purchased and this will increase the required number of welded connections in the bridge members. One can readily estimate the costs due to heat treatment and additional welding. With regard to benefits, these must be adjusted to allow for the added potential for weld
related cracking. Whether a significant benefit would remain would depend upon how much value could be assigned to enhanced toughness of portions of the flange not modified by welding. Further progress with assignment of the benefit value would require close attention to perspective considerations as represented in the fracture control plan statements of objectives.

Granting that further study permits development of acceptable statements of fracture control objectives, the operational elements of the plan can then be stated. Furthermore the purpose of each element in relation to the objectives can be given. These purpose statements are important. Without them, those concerned may not understand either the necessity or the relative value of any given procedural element of the fracture control plan.

The present AASHTO Guide Specifications For Fracture Critical Non-Redundant Steel Bridge Members is a modest but substantially helpful fracture control plan. The published text has only 34 pages. The proposed Fracture Control (SES 5) recently distributed by FHWA, even though incomplete in its coverage of essential topics, is an order of magnitude larger in terms of length of text. This plan also reflects differences of opinion and viewpoint from the AASHTO plan. Without dealing with these differences, such an extensive set of specifications seems to demand statements of objectives and, for each element of the plan a clear statement of its purpose. Resolution of the above differences of opinion and viewpoint should be possible and deserves first priority at the present time.
Statements of fracture control plan objectives for steel bridge members will not be attempted here. However, as an aid to their development, several comments of perspective nature can be offered. With regard to the relative importance of (a) bridge repairs due to cracking and (b) the hazard of fracture induced collapse of a bridge, the relevant occurrences during the past 30 years indicate the following conclusions. The public safety hazard is probably larger from (a) than from (b). The financial cost and public inconvenience cost are obviously larger by orders of magnitude from (a) as compared to (b). It follows that fracture control methods should give highest priority to ensure that hazards and costs of bridge repairs due to cracking in noncritical members can be held within reasonable control. Procedures to deal with these two problems move along similar lines. Thus we can give serious attention to both aspects. The requirements for adequate fracture control may differ considerably in different bridges due to such factors as design, location, volume of traffic, and cost of repair or replacement. Purpose and value uncertainties in the operational elements of the fracture control plan should be clarified to the degree possible by cost-benefit considerations. In these as well as in other aspects of the plan, the role of fatigue in causing development and extension of cracks is a factor of major importance.
Welded Details

The primary cause of cracking in the bridge structures reviewed in FHWA Structural Research Series No. 5, Vol. III and Structural Research Series No. 7 has been the development of fatigue crack propagation from either very low fatigue resistant welded connection details (Yellow Mill Pond, Dan Ryan) or from very large initial discontinuities perpendicular to the cyclic stress (i.e. Kings Bridge, Lafayette Street Bridge, I-79 Bridge and Quinnipiac River Bridge). In several instances very severe welded connection details also contributed to fracture during construction (i.e. Bryte Bend Bridge and the Fremont Bridge).

Most of these members experienced fracture as a result of fatigue crack propagation that sharpened and enlarged the crack until instability developed when the fracture toughness of the member was exceeded.

Often very large initial discontinuities have been inadvertently fabricated into bridge members because the welded connection detail was not considered to be very important. This was the case at the Lafayette Street Bridge where lateral connection plates were connected to the web and transverse stiffeners and resulted in lack of fusion in the gusset-stiffener connection. At least three other bridges have experienced this same type of cracking. A related condition developed in the Quinnipiac River Bridge and at least four other bridge structures when continuous longitudinal stiffeners were attached to girder webs in tension regions. Groove welds in splices of longitudinal stiffeners had large lack of fusion conditions which resulted in
fatigue crack growth had initiated at the longitudinal-stiffener splices and propagated into the web and flange. In all but one case the crack was detected before the tension flange was cracked through.

Service experience and research results have demonstrated that cracking will not be prevented by material considerations alone. Changes in the fracture toughness of the steel would not have stopped the development or altered the rate of growth of fatigue cracks in any of the structures mentioned. To minimize cracking in bridges and more specifically in fracture critical members, primary focus must be given to techniques and ways to overcome the design, details and quality control problems that have developed in the past.

Among the most obvious steps that can be taken is the elimination of as many details as is feasible. The use of any unnecessary welded connection detail should be avoided. Very low fatigue resistance welded details should also be avoided. Cover-plated beams, and other web and flange attachments that are Category E or E' welded details would best be eliminated from fracture critical members. This decreases the possibility of crack growth under normal use of the structure including permit overloads.

Care should also be exercised when using details that are different or innovative and about which nothing is known. Although our understanding about the fatigue behavior of welded details has improved, it is not always possible to predict the behavior of new and different connections. This can be readily
seen in the Dan Ryan Elevated Transit Structure where girder flanges intersected and passed through the webs of the support bents. The significance of the lack of fusion in the web of the pierced girders was not fully understood. This resulted in a detail that has very low fatigue resistance and can be expected to crack under the levels of stress range that most bridges experience in service. In retrospect, it should not have been used.

The application of fracture control plans to recent structures has not prevented poor conditions from being fabricated into fracture-critical members (FCM) such as passing drainage pipes through tie girder boxes and seal welding them to the flange plates. These type of conditions should be avoided in fracture critical members.

Preferably, all welded details should be fabricated so that they are inspectable. This will permit better quality control to be achieved at time of fabrication and will also permit inspection of the detail during its service life. Any unforeseen or undetected discontinuity will be more likely detected at subsequent inspections.

Greater emphasis must be placed on minimizing fabrication and weld discontinuities that are susceptible to fatigue crack propagation. Such discontinuities are going to enlarge with the accumulation of cyclic stress. Eventually, such cracks will reach a point where crack instability develops. Increasing the levels of fracture toughness does not have a significant influence on the enlargement of the discontinuity. If crack
propagation develops it is necessary to repair the resulting crack. An increased level of fracture toughness will not change this fact. The expense associated with repair of all of the examples cited will still exist.

**FABRICATION AND INSPECTION**

Fabrication and the inspection of the fabrication are extremely important in a fracture control plan. It has been traditional to allow the fabricator to propose the fabrication procedures he will employ in making the structure. This tradition allows the fabricator to select the most economical methods and, consequently, to produce the structure with a minimum cost. These procedures must be within the bounds set by the specifications and, if not generally accepted, be subjected to qualification testing and approval.

The present welding specification, as well as the modifications contained in the AASHTO and FHWA plans, are a mixture of performance and procedure specifications. Performance specifications are typically applied to the mechanical properties of the weld; for example, minimum yield strength or Charpy V-notch performance. Procedure specifications are normally used to cover weld heating and inspection requirements. The purpose of these specifications should be to minimize harmful fabrication practices which can lead to failure of the structure before it reaches its design life. The procedure specifications should be balanced between 1) restricting practices that are known to be detrimental and 2) allowing sufficient latitude to permit the weldment to be economically fabricated. An example would be weld
preheat and interpass temperatures. Too high a temperature causes welder discomfort and makes welding difficult and can lead to deviations from established procedures. Too low a temperature can also lead to cracking problems. The present AWS and AASHTO heating requirements appear to be reasonable. The small number of problems which have occurred are due to improper implementation of the present requirements rather than to the inadequacy of the requirements.

The single most important element with respect to fracture control which needs to be controlled during fabrication is the size of imperfections introduced by the fabrication process. The fatigue life of a structure is controlled by joint geometry, stress range, and initial imperfection size. The first two variables are the responsibility of the designer. The latter is controlled by the quality of fabrication of the structure. It should be noted that the AASHTO fatigue specification categories imply an associated allowable initial imperfection size. Longitudinal fillet welds, Category B, have an associated internal initial imperfection (i.e. gas pocket) size of 0.1 in., while the transverse filet weld categories, C, D, and E, have an implied initial toe imperfection (i.e. slag inclusion) of the order of 0.025 in. One major goal of a fracture control plan should be to ensure that the fabricated structure does not contain imperfections larger than assumed by the fatigue specifications.

A review of the bridge cracking problems indicates that most are associated with deviations from the specifications in force at the time of fabrication which resulted in fabrication of
large defects in the structure. The role of quality assurance and control emphasized in both the AASHTO and FHWA Plans are a recognition of this problem. However, to ensure conformance with the specifications, the designer should avoid the use of weldments that are difficult to fabricate and inspect. The responsibility and knowledge of the inspection personnel employed must be addressed. The AASHTO specification provides for qualifications of personnel and delineates responsibility. These requirements, along with the added emphasis provided by the identification of critical weldments, should minimize the occurrence of the imperfections responsible for many of the current problems.

The emphasis placed upon repair procedures in both the FHWA and AASHTO specifications should serve to minimize the occurrence of cracking from unsuitable weld repairs. Two major structures have sustained cracks which occurred from weld repairs of questionable quality. The repair provisions along with the provisions for more rigorous inspection should alleviate this problem.

**FRACTURE TOUGHNESS**

Fracture of structural components has never been just a material problem. Design, fabrication, material, inspection, and operating conditions are all factors that affect the susceptibility of a structure to fracture. When one or more of these factors is significantly more severe for a particular structure, compared with conditions for other structures of the same generic class, the possibility of fracture is increased. Analysis of failures discussed in Appendix C show that this is the case and
that in most of these failures, the particular level of notch toughness of the material was not the primary cause for failure. Furthermore, a significant increase in the notch toughness would not have prevented the failure. In fact, in several of the failure cases, the steels also would have met the more stringent notch toughness requirements proposed in SES No. 7, and yet brittle fractures still occurred. Appendices A, B, and C show that unstable crack propagation for constructional steels in the temperature range of interest is brittle. However, the present AASHTO Fracture Control Plan, in conjunction with other AASHTO specifications, should eliminate the possibility of unstable brittle-fracture initiation in those cases where good design and fabrication practices are followed.

In those rare cases where good design, fabrication, inspection, and loading practices are not followed, merely increasing the material toughness requirements to test at the least anticipated service temperature (LAST) will not be sufficient to eliminate brittle fracture initiation and propagation in bridge members.

**Fracture-Toughness Evaluation.** ASTM has recognized the variability inherent in measuring fracture toughness of materials. The variability may be related to details in the testing procedure and/or to variability in the material properties. It is widely recognized that the variability may be larger in the temperature range where transition from brittle (cleavage) to ductile (fibrous) fracture occurs. The variability may also be larger for materials that are strain-rate sensitive than for those that are not strain-rate sensitive.
Given the uncertainties that currently pertain to the value of fracture toughness, evaluation of generalized procedures, such as the temperature shift, should be based on a cumulative average behavior. Otherwise, evaluation of such procedures that are based on extreme (worst-possible) values require extensive testing and the assignment of probabilistic values for the occurrence of such behavior. Given the uncertainties that currently pertain to the value of fracture toughness in comparison to other factors (such as proper design and selection of details and proper fabrication and inspection) that have significantly higher cost benefits for steel bridges, it is pointless to devote an intensive discussion here to the many complexities of fracture toughness and fracture-toughness evaluation methods.

The temperature-shift concept has been discussed in the commentary to the Guide Specifications For Fracture Critical Non-Redundant Steel Bridge Members. This proven concept was used in the development of the fracture testing required by the AASHTO fracture-control plan. Thus, rather than testing fracture-mechanics type specimens under conditions that simulate those for actual bridges (i.e. intermediate loading rates), the concept permitted screening and qualifying bridge steels by using the low-cost, standardized Charpy V-notch impact specimen. A high degree of accuracy in the application of the temperature shift to a wide range of steels is not warranted unless the present uncertainties that currently pertain to the value of fracture toughness are resolved. If the existing uncertainties are regarded as
dangerous, we first need to know the nature of danger associated with fracture-toughness uncertainty. Moreover, we should establish the cost benefits that would result from resolving these uncertainties including changes in life expectancy. At the present time these uncertainties do not appear to be significant because, as discussed in Appendix C, field fractures were caused by factors other than fracture toughness. In several cases, the steel would have met the proposed FHWA CVN impact requirements and yet the failures still occurred. At the present time, over emphasis on fracture toughness has distracted the attention from other factors, such as selection of structural details and fabrication, that are significantly more important in preventing fracture of bridge components.

Under conditions of interest relative to crack propagation in bridge components, fracturing by dominant cleavage (brittle fracture) from a fatigue pre-crack is the expected mode of crack extension (see Appendix A). Also, one might suppose that nearness of the precrack to a serious local discontinuity or embrittled zone might initiate such cracking (pop-in) with an unexpectedly low indication of fracture toughness. This is possible but, in practice, is highly improbable as research and case histories have shown. Efforts to observe pop-in type cracking as an interruption to regular fatigue crack growth were made at Lehigh University using A36, A588, and A514 steels in large flange thicknesses and with a variety of welded details and at low temperatures. Pop-in was never observed.
FRACTURE TOUGHNESS IN RELATION TO TWO BRIDGE FRACTURE EXAMPLES

The collapse of the old Silver Bridge in 1967 started an intensive USA study of the integrity of steel bridges. Subsequently cracking has been discovered in a number of bridges revealing faults in design, fabrication, material and welding-process control that deserved attention. Although the explanations were occasionally similar, the circumstances varied widely. The old Silver Bridge incident itself was completely non-typical with regard to subsequent examples of cracking in steel bridges. However, it provided the only example of a USA bridge collapse due to fracture.

The old Silver Bridge would currently be ruled unsafe on the basis of design and material properties. The crevice situation at the pinned connection between eye-bars was conducive to corrosion and to corrosion fatigue. At the time of failure the crack which became unstable was only one-eighth inch deep. However, due to stress concentration influences of the eye-bar hole, the stress was nearly as high as the material yield strength and, at the temperature of interest, the static fracture toughness acting to restrain crack propagation was about 40 ksi $\sqrt{\text{in}}$. At the same temperature, the Charpy V-notch energy absorption was 1.5 ft-lb. Had the fracture toughness been larger, there was a nearby satellite crack which would soon have joined with its neighbor, causing a doubling of the crack size. Since the growth rate of fatigue and corrosion fatigue cracks accelerates rapidly, it is doubtful that a significant increase of toughness would have added more than one year to the life of the bridge.
As a second example, consider the fracture which occurred during final stages of construction of the two-cell Bryte Bend box-girder bridge near Sacramento. The highest stressed regions of the bridge were regions of negative moment over the piers. In these locations, the three top flanges of the box girder were fabricated using 2-1/4 inch thick plates of non-specification material meeting the chemistry of ASTM A517H steel. The design required welded connections of intermediate cross frames to the box girder flanges. However, the design specifically omitted the connection of the diaphragms over the piers to the box girder flanges. In actual fabrication, a lateral connection was made at the pier locations and this contributed directly to the fracture. There were other contributing factors. The 2-1/4 inch box-girder flanges were chamfered vertically and longitudinally by gouging to reduce the flange thickness to one inch at the weld connecting the 1 inch diaphragm flanges to the box-girder flanges. Chamfers and tapers did not conform to AASHTO standards. Careful inspection of this weld would have been difficult. A one-quarter inch deep welding crack was not detected at one end of the welded lateral connection at the point of fracture failure. At the time of fracture, the concrete deck was being placed on the adjacent span of the bridge.

It was assumed during discussions of this fracture that the fracture occurred in two stages. First, a "pop-in" crack through the region of high welding residual stress, then final failure during increases of dead weight load during
construction. Examinations of the fracture surface in the laboratory did not provide convincing evidence for the supposed arrest of the crack at a depth of about one inch. This does not mean that such an arrest did not occur. It means that, if a sudden crack extension (pop-in) did occur, the crack arrest and start of final crack propagation occurred at nearly the same driving value of $K$.

The measured value of $K_{IC}$ using material from the broken flange was about 55 ksi $\sqrt{\text{in}}$ at the temperature of interest. At the same temperature, the impact Charpy V-notch energy absorption was about 6 ft-lb. If the fracture toughness had been substantially larger, the bridge would surely have been completed and placed into service with welding cracks at the ends of unwanted lateral connections in the regions of highest stress. Since the expected additional loads due to traffic were relatively low in comparison to dead load, it is difficult to say how much service life might have occurred prior to development of enough fatigue cracking to start rapid crack propagation.

Since the actual fracture stopped after several feet of travel in the ASTM A517 web plate, it seems that bridge collapse would not have occurred in any case. However, the sudden disruption of traffic if the event occurred after the bridge was in service might have caused serious traffic accidents.

These two fractures have been briefly reviewed because the circumstances of these examples are widely known. In both examples low toughness, in itself, added nothing to the general cost hazard of a fracture occurrence either in terms of public safety or financial loss. The first example, although completely
non-typical, is our only USA bridge collapse due to fracture. In the second example, more careful attention should have been given to ensure that the fabrication was in accordance with the design drawings and that all aspects of welding which might result in the development of cracking during service were considered. Clearly the extreme lack of toughness of the material may, in itself, have contributed to welding problems reported by the fabricator. As in most examples of bridge fractures, it is difficult to establish in any clear way the advantage of increases of fracture toughness beyond some low value which might be reasonable associated with poor weldability. However additional fracture toughness would not have prevented the fracture of either bridge.

**PROPOSED FHWA FRACTURE CONTROL PLAN**

The proposed FHWA-FCP does not discuss general objectives, fatigue cracking in relation to design, specific purpose of FCP elements, and cost-benefit considerations. As such, the current FHWA-FCP can be considered only as a set of suggestions for addition to the present AASHTO-FCP. These suggestions can be divided into two groups, those dealing with welding practice and those dealing with required values of fracture toughness. The suggestions dealing with welding practice and inspection of welds deserve careful study. If the general FCP objectives are clearly stated and the purpose of each proposed practice (related to welding) is clearly stated, it should be possible to estimate costs and benefits and determine which of the proposed FCP elements should be added to the AASHTO-FCP. We must emphasize that clear statements of FCP objectives and element purposes are
essential prerequisites. Our present economical situation does not permit us to do everything that might seem helpful to quality without determining that the benefits are large enough to justify such cost increases as are required. Cost-benefit considerations will remain uncertain and ambiguous so long as general objectives and element purposes are not clearly stated.

The suggestion in the FHWA-FCP related to toughness fall into a special second category, because even without full clarification of general objectives, it seems clear that fracture toughness is of secondary importance to fatigue cracking. It is unfortunate that general and specific relationships of toughness to probability of welding flaws are not discussed in the FHWA-FCP. It is not clear if large increases of fracture toughness can be justified on that basis. However, as a toughness consideration, the topic deserves careful review. In the absence of demonstrated support for large increases of required toughness, the moderate levels of fracture toughness currently specified in the AASHTO-FCP seem adequate. For this reason, approximate methods of estimating toughness seem to be satisfactory and the toughness topic need not be discussed in great detail. On the other hand, such estimates as are used should follow a consistent plan. The toughness estimation methods employed in the FHWA-FCP have departures from consistency which are undesirable. For this reason appendices dealing with the FHWA-FCP have been added to this commentary. These appendices show that inconsistent interpretation of test results, incorrect application of fracture-mechanics concepts, and
incorrect conclusions on causes of actual failures in various
types of steel structures were used in the FHWA-FCP technical notes.

The motivation of the proposed FHWA-FCP to improve
welding quality is commendable. However, every practice thought
to be beneficial to quality cannot be accepted as a steel-bridge
construction requirement without careful assessment of costs and
benefits.

As noted above, much depends upon perspective and judg-
ment regarding the present problems in steel bridges and the
corresponding fracture control objectives. If there are remain-
ing uncertainties with regard to these objectives, special
studies and discussions should be arranged to clarify these as
promptly as possible. It would appear that we should give
primary emphasis to reducing the causative factors which have led
to excessive bridge repairs due to cracking.
Appendix A

FRACTURE BEHAVIOR OF BRIDGE STEELS

1. Fracture Toughness

The fracture-toughness behavior of bridge steels and the basic principles used in the development of the AASHTO fracture-toughness requirements have been presented in the Commentary to the Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members.

The AASHTO fracture-toughness requirements were based on data that were valid plane-strain fracture-toughness values according to ASTM requirements. Deviations from the ASTM requirements for validity can result in apparent fracture-toughness behaviors that are strongly affected by specimen size and geometry and that do not represent the inherent behavior of the material. This section presents a brief discussion of some problems caused by such deviations. These problems are of primary importance for evaluating data presented in the FHWA documents on fracture control and temperature shift.

One of the requirements for valid $K_{IC}$ testing is restriction on the minimum thickness, $B$, for the test specimen. This restriction was established by ASTM to ensure plane-strain conditions at the crack tip. The minimum thickness requirement for valid $K_{IC}$ tests is given by the equation

$$B = 2.5 \left( \frac{K_{IC}}{F_{TY}} \right)^2 \quad (1-A)$$
where $FTY$ is the static yield strength of the steel at the test temperature.

A second ASTM requirement for valid $K_{IC}$ tests is a restriction on the ratio of the stress-intensity for maximum load, $K_{max}$, and $K_{IC}$ (or $K_Q$). This ratio, $K_{max}/K_Q$, must not exceed 1.10.

There are few instances where deviations from the preceding requirements result in fracture-toughness values that are equal to $K_{IC}$ for the plate being investigated. However, this observation is the exception rather than the rule. For example, the data presented in Figure 1A for a 2-inch-thick plate of HY-80 steel show a problem that frequently occurs when the preceding requirements are violated. The data show the transition for valid $K_{IC}$ with temperature that occurred at temperatures below -275°F. Above this test temperature the results do not meet the ASTM requirements for validity on thickness and $K$ ratio. This data cannot be analyzed by using linear-elastic fracture-mechanics concepts, but might be analyzed by using elastic-plastic fracture-mechanics test methods, such as $J$-integral concepts.

The data in Figure 1-A also show that an invalid static test conducted at -170°F, if considered valid, would have the same value as for an impact test. Thus, deviation from ASTM requirements for validity can lead to the conclusion that the material does not exhibit a temperature shift. Similarly, invalid static tests conducted at temperatures between -275 and -175°F incorrectly predict a temperature shift that is less than
the inherent temperature shift between static and impact fracture-toughness curves for the tested steel plate.

Many examples used by the FHWA to show that the temperature shift "is not a reliable basis for a materials specification" (SES No. 5, Vol. III page 27) contain data that are not valid in accordance with ASTM Standard Specification E-399. For example, consider "the following example [that] was selected [by FHWA] from several similar examples because it is a case that is meticulously documented by CALTRANS" (SES No. 5, Vol. III, page 27). None of the four tests, Table 1-A, conducted by CALTRAN had sufficient thickness to satisfy the ASTM requirements for validity. Moreover, two of the four tests had a $K_{\text{max}}/K_Q$ ratio that was higher than the 1.10 maximum value required by ASTM for a valid test result. Based on the preceding discussion, use of these invalid test results caused the FHWA to underestimate the magnitude of the temperature shift. Consequently, the reliability of the temperature-shift concept was incorrectly questioned.

2. Cleavage Crack Initiation and Propagation

The FHWA documents on fracture control and temperature shift discuss brittle (cleavage) fractures of test specimens and structures. This mode of fracture is used by the FHWA to suggest that the AASHTO fracture-toughness requirements are not adequate.

Cleavage mode of crack initiation and crack propagation can be understood best by considering the fracture-toughness transition behavior of steels under static and impact (dynamic) loading, Figure 2-A. The static fracture-toughness transition
curve determines the mode of crack initiation at the crack tip. The dynamic fracture-toughness transition curve determines the mode of crack propagation.

The fracture-toughness curve for either static or dynamic loading can be divided into three regions as shown in Figure 2-A. In region I_s for the static curve, the crack initiates in a cleavage mode from the tip of the fatigue crack. In region II_s the fracture toughness to initiate unstable crack propagation increases with increased temperature. This increase in the crack-initiation toughness corresponds to an increase in the size of the plastic zone and in the zone of ductile tear (shear) at the tip of the crack prior to unstable crack extension. In region III_s the static fracture toughness is quite large and somewhat difficult to define, but the fracture initiates by ductile tear (shear).

The ductile-tear zone at the tip of a statically loaded crack is confined to the zone of plastic deformation along the crack front. The maximum size for this plastic zone is restricted by ASTM requirements to three percent of the specimen thickness in order to ensure valid plane-strain test results. Deviations from this requirement for elastic plane-strain conditions toward elastic-plastic conditions usually result in high initiation fracture-toughness values. The plastic-zone size and the ductile tear zone at the tip of the crack under elastic-plastic conditions is usually very small and is difficult to delineate by visual examination.
Once a crack initiates under a static load, the morphology (cleavage or shear) of the fracture surface for the propagating crack is determined by the dynamic behavior and degree of plane strain at the temperature. Regions I_d, II_d, and III_d in Figure 2-A correspond to cleavage, increasing ductile-tear (shear), and full-shear crack propagation, respectively. Thus, at temperature A, Figure 2-A, the crack initiates and propagates in cleavage. At temperatures B and C the crack exhibits ductile initiation but propagates in cleavage. The only difference between the behaviors at temperatures B and C is that the ductile-tear zone for crack initiation is larger at temperature C than at B. At temperatures D, Figure 2-A, cracks initiate and propagate in full shear. Consequently, full-shear fracture occurs only at temperatures when the static and dynamic (impact) fracture behaviors are on the upper shelf. Because the static upper-shelf behavior for bridge steels always starts at lower temperatures than for the dynamic upper-shelf behavior, the requirement of a dynamic upper-shelf behavior at a given temperature always ensures a static upper-shelf behavior at the same temperature.

The morphology of a running crack in steel is influenced by constraint (degree of plane-strain) and temperature. Loss of cleavage from the appearance of a Charpy V-notch test specimen may not mean that cleavage will be absent at that temperature under conditions of greater constraint.

In conclusion, fracture of a structural component whose dynamic (impact) fracture toughness is not on the dynamic upper-shelf
(that is, about 100% shear fracture appearance) will exhibit regions of cleavage crack propagation. This observation is supported by the examples of field failures discussed in the FHWA documents. For example, the casualty plate for the Martha R. Ingram barge met the proposed FHWA fracture-toughness (ft-lb) requirement but did not meet their requirement of an upper-shelf (80% shear) behavior at the fracture temperature. The fracture of the casualty plate was cleavage as should be expected from the preceding discussion. This example as well as other examples in the FHWA documents, are discussed further in Appendix C.

Photographs of fracture surfaces for 1.5-inch-thick specimens of an A572 grade 50 steel have been published by Novak,* Figure 3-A. The specimens tested at room temperature exhibited CVN and $K_C$ values that were twice and about twelve times, respectively, higher than the requirements proposed by the FHWA. Despite these large toughness values at the test temperature, the specimens exhibited ductile crack initiation and brittle (cleavage) unstable crack extension, Figure 3-A.

Thus, ductile crack propagation occurs only when the test temperature is above that for dynamic upper-shelf behavior (i.e. 80% shear fracture appearance for an impact CVN specimen). Data presented in Appendix B show that upper-shelf dynamic behavior occurs at significantly higher impact CVN ft-lb values than those proposed by the FHWA for A36, A572, and A588 steels.

---

Table 1-A

(From FHWA Table Cl.5.4.1.b of Reference 1-A, page 29)

COMPACT TENSION TEST RESULTS FOR A36 STEEL

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Rate, (1/2) ksi-in./sec.</th>
<th>(A_{\text{avg.}}), inch</th>
<th>(P_{Q'}), kips</th>
<th>(P_{\text{max'}}), kips</th>
<th>(K_{Q'}), ksi-in.(^{1/2})</th>
<th>(K_{\text{max'}}), ksi-in.(^{1/2})</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>1.08</td>
<td>1.94</td>
<td>26.0</td>
<td>27.5</td>
<td>59.7</td>
<td>63.1</td>
</tr>
<tr>
<td>B2</td>
<td>1.11</td>
<td>1.98</td>
<td>22.5</td>
<td>28.3</td>
<td>53.1</td>
<td>66.8</td>
</tr>
<tr>
<td>B3</td>
<td>111.2</td>
<td>1.98</td>
<td>24.5</td>
<td>33.0</td>
<td>57.8</td>
<td>77.9</td>
</tr>
<tr>
<td>B4</td>
<td>110.5</td>
<td>1.97</td>
<td>20.8</td>
<td>20.8</td>
<td>48.8</td>
<td>48.8</td>
</tr>
</tbody>
</table>

Test Temperature = +32°F

Yield Strength at +32°F = 44.5 ksi (ref. 1-A, page 27)

NOTE: 1. None of the tests met the ASTM thickness requirement for valid test, \(B \geq 2.5 \left( \frac{K_Q}{FTY} \right)^2\).

2. The results for test specimens B2 and B3 did not meet the ASTM ratio requirement of \(K_{\text{max}}/K_Q \leq 1.10\).
\( K_{ic} \) - Static
\( K_{Q} \)
\( K_{max} \)
\( K_{id} \) - Dynamic

Data above line correspond to invalid data

**Figure 1-A**
Figure 2-A

The graph illustrates the fracture toughness as a function of temperature and shear. The graph is divided into four regions:

- **Region I_s**: Cleavage Initiation
- **Region II_s**: Increasing Shear
- **Region III_s**: Full-Shear Initiation

For degradation:
- **Region I_d**: Cleavage Propagation
- **Region II_d**: Increasing Shear
- **Region III_d**: Full-Shear Propagation

The graph shows how fracture toughness changes with temperature and shear, with distinct transition points between the regions.
Fracture surfaces of full-thickness (B = 1.5 inches) 4T compact-tension (CT) specimens of A572 Grade 50 steel tested under load-control conditions using a total-unload/reload loading sequence (U. S. Steel procedure).
Appendix B

THE FHWA PROPOSED FRACTURE-TOUGHNESS REQUIREMENTS
FOR FRACTURE-CRITICAL MEMBERS\(^1-4\)

The FHWA fracture-toughness requirements for fracture-critical members were based on various assumptions that are contradictory. To understand these contradictions and the resulting fracture-toughness requirements, the procedure used in the development of the FHWA fracture-toughness requirements are presented in the following sections. The proposed FHWA fracture-toughness requirements include impact Charpy V-notch (CVN) energy absorption (ft-lb) and 80 percent shear fibrous fracture at the least anticipated service temperature (LAST). The proposed requirements also include a nil-ductility transition (NDT) at 30°F below the LAST that is permitted only when retested CVN specimens fail to meet 80 percent shear at the LAST. The following discussion shows that:

1. The FHWA procedure which is based on through-thickness yielding concepts in combination with CVN-\(K_{IC}\) correlations were incorrectly used to calculate the CVN ft-lb requirements;

2. The upper-shelf CVN-\(K_{IC}\) correlation is valid only when the CVN impact specimen exhibits 80 percent or more shear fracture. Consequently, the 80 percent or more shear requirement is fundamental to the development of the FHWA proposed requirements and cannot be relaxed or eliminated.

3. The FHWA proposed ft-lb requirements do not contribute to the acceptance or rejection of a steel because the 80 percent requirement is the most severe requirement and, therefore, governs the selection process;
4. The proposed 80 percent shear requirement would eliminate the use of A36, A572, and A588 steels for fracture-critical members of steel bridges;

5. The requirement of NDT at 30°F below the LAST which appears to be incidental to the proposed FHWA requirements is the only surviving requirement.

**Development of the Proposed FHWA Fracture-Toughness Requirements**

The procedure used to develop the FHWA fracture-toughness requirements are scattered throughout four documents issued by the FHWA. These documents are Structural Engineering (SES) No. 5, Volumes I, II, and III and SES No. 7. The primary assumptions used to develop these requirements are stated in Paragraph 1.5.4 of SES No. 5, Vol. III on "Plate Toughness." The FHWA fracture-toughness requirements were based on "through-thickness-yielding concept" (SES No. 5, Vol. III, page 15) and correlation equations for CVN ft-lb and $K_{IC}$ fracture-toughness values.

$$\left(\frac{K_{IC}}{FTY}\right)^2 \quad B = 1$$  \hspace{1cm} (1-B)

where $K_{IC}$ = critical stress-intensity factor under plane-strain conditions and static loading

$FTY$ = static yield strength

$B$ = plate thickness

Equation 1-B was used to calculate a required static $K_{IC}$ value for a given plate thickness, $B$, and static yield
strength, FTY. Correlations between static fracture toughness, $K_{IC}$, and impact Charpy V-notch (CVN) energy absorption were used to establish the FHWA CVN ft-lb requirements.

When the CVN-impact test results are not on or close to the upper shelf (as demonstrated by 80% or more shear in the fracture surface), the transition-temperature correlations from Barsom should be used

$$K_{IC}^2 = 5 \times (CVN) \times E$$


Equation 2-B was developed by Barsom and Rolfe to predict $K_{IC}$ values at a given temperature from CVN ft-lb values at the same temperature in the toughness-transition region when both tests are conducted at the same loading rate. To calculate impact CVN ft-lb values, dynamic fracture toughness, $K_{ID}$, values must be substituted in Equation 2-B. Conversely, to calculate $K_{ID}$ values in the toughness-transition region, impact CVN ft-lb values must be substituted in Equation 2-B. Because the toughness-transition region for impact-loading rates occurs at higher temperatures than for static-loading rates (i.e., temperature shift), Equation 2-B cannot be used directly to predict impact fracture-toughness behavior from the static behavior without also accounting for the temperature shift.

The FHWA, having determined the static $K_{IC}$ requirements for a given plate thickness and yield strength by using Equation 1-B, substituted this static $K_{IC}$ value in Equation 2-B
to obtain their proposed impact CVN requirements (Table Cl.5.4.a, SES No. 5, Vol. III, page 22) shown in Table 1-B of this document. This violation of the basic requirement for validity of Equation 2-B resulted in the very severe ft-lb requirements of Table 1-B. The FHWA recognized the severity of these requirements when they realized that, "unfortunately most heats of domestic steel will not meet the requirements" (SES No. 5, Vol. III, page 19). Thus, "because of the anticipated extreme difficulty in meeting the requirements... (in Table 1-B), the transition range CVN-\(K_I\) (i.e., Equation 2-B) relationship was not used in the fracture control plan." (SES No. 5, Vol. III, page 23)

To obtain "more realistic" impact CVN toughness requirements, the static \(K_I\) values obtained by using Equation 1-B were substituted in an upper-shelf correlation between static \(K_I\) and impact CVN. This upper-shelf correlation was established by Barsom, Rolfe and Novak as

\[
\left( \frac{K_I}{F\text{TY}} \right)^2 = \frac{5}{F\text{TY}} \left( \frac{CVN - F\text{TY}}{20} \right) \quad (3-B)
\]

Thus, having determined the static \(K_I\) requirement for a given plate thickness and yield strength by using Equation 1-B, this value for static \(K_I\) was substituted in Equation 3-B to obtain impact CVN ft-lb energy-absorption requirements. The use of Equations 1-B and 3-B resulted in the proposed FHWA impact CVN requirements (Table 1.5.4, SES No. 5, Vol. I, page 5) shown in Table 2-B of this manuscript. To ensure the applicability of the
upper-shelf correlation (Equation 3-B), it was necessary to add footnote (a) in the table which requires that the fracture appearance in any one specimen must be 80 percent shear (fibrous) or higher. This requirement is essential to ensure the applicability of Equation 3-B.

Because the proposed FHWA CVN requirements shown in Table 2-B were developed by using Equations 1-B and 3-B, the technical errors made by the FHWA in applying these equations are of primary importance and, therefore, are discussed in the following section.

Technical Errors in the Development of the Proposed FHWA Fracture-Toughness Requirements

The correlation between upper-shelf impact CVN energy absorption (Equation 3-B) and static $K_{IC}$ was developed by Barsom, Rolfe and Novak to predict $K_{IC}$ values. Thus, given an impact CVN ft-lb value obtained for a specimen that exhibited 100 percent shear (fibrous) fracture surface, an estimate of an "upper-shelf" $K_{IC}$ value can be calculated by using Equation 3-B. Initially, Barsom et al restricted the use of this correlation to high-strength steels having yield strengths equal to or higher than 110 ksi which are relatively insensitive to loading-rate effects. The static and dynamic fracture-toughness curves for these steels are displaced by less than 50°F and reach an upper-shelf behavior within a small temperature region. Research results by several investigators have been published in support of the upper-shelf correlation.
More recent data by Iwadate, Kuraushi and Watanabe* show that this correlation may be valid for the impact CVN upper-shelf region of low-strength steels having yield strengths higher than 59 ksi, Figure 1-B. This figure is the same as FHWA Figure C1.5.4.1 that they used to justify the use of the upper-shelf correlation for low strength steels. Iwadate, et al data also support the temperature-shift concept.

The static $K_{IC}$ and impact CVN ft-lb data published by Iwadate, et al show that the static $K_{IC}$ upper-shelf behavior for low strength steels extends to significantly lower temperatures than the impact CVN ft-lb upper-shelf behavior. This observation in itself supports the temperature-shift concept. Hence, an impact CVN upper-shelf behavior always coincides with a static $K_{IC}$ upper-shelf behavior. The temperature region defined by coincident impact and static upper-shelf behaviors, as shown in Figure 2-B, is the region of validity for Equation 3-B. However a static $K_{IC}$ upper-shelf behavior does not necessarily coincide with an impact CVN upper-shelf behavior. Thus, it follows that in the region where Equation 3-B is not valid, the static $K_{IC}$ value can reach an upper-limit value for that steel while at the same temperature the impact CVN behavior may exhibit low ft-lb values with less than 80 percent shear.

Research results by several investigators have been published in support of the upper-shelf correlation. More recent

---

data by Iwadate, Karashui and Watanabe show that this correlation may be valid for the CVN upper-shelf region of low strength steels having yeild strengths higher than 59 ksi, Figure 1-B.

The region of applicability of the upper-shelf correlation is shown schematically in Figure 2-B. Thus, given any point A on the upper-shelf impact CVN energy absorption, one can use Equation 3-B to calculate an equivalent $K_{IC}$ value (point A') at the same temperature. Equation 3-B is not valid and, therefore, should not be used when the CVN energy-absorption value is not on the upper shelf. Similarly, given a $K_{IC}$ value for any point B' that is not in the region of applicability of Equation 3-B cannot be used to calculate an equivalent upper-shelf CVN specimens are on the upper shelf, Equation 3-B cannot be used to calculate an equivalent upper-shelf CVN impact-energy-absorption value. Unfortunately, the FHWA used this procedure to develop their proposed fracture-toughness requirements. Equation 3-B was developed to characterize the behavior of steels in the region of applicability shown in Figure 2-B and cannot be used, as was done by the FHWA, to dictate and modify the inherent behavior of steels.

**Significance of the FHWA Proposed Fracture-Toughness Requirements**

Because of the preceding errors made by the FHWA in the development of their proposed fracture-toughness requirements, the significance and consequences of their requirements, shown in Table 2-B, need to be established.

According to the FHWA requirements, Table 2-B, a bridge steel plate having a given thickness and yield strength must be
the appropriate impact CVN energy-absorption value specified in Table 2-B at the LAST and, as required in footnote (a) of Table 2-B the fracture appearance of each of the CVN specimens should exhibit 80 percent shear or more.

On the surface, the FHWA requirements suggest that the impact CVN energy absorption (ft-lb) values are the primary requirements, and that the 80 percent shear requirement is a control parameter that is a by-product of the procedure used to establish the FHWA ft-lb requirements. In reality, the 80 percent shear requirement is the primary requirement because the upper-shelf correlation (Equation 3-B) is valid only when the CVN specimen exhibits 80 percent shear fracture appearance or higher regardless of its energy absorption (ft-lb) value. The FHWA impact CVN ft-lb requirements and the requirement of 80 percent shear at the LAST are not compatible.

A requirement of 80 percent shear at a given temperature will always ensure a CVN ft-lb value at that temperature greater than required in Table 2-B for low strength steels such as A36, A572 and A588. For example, consider the 1-inch-thick plate of A36 steel whose energy absorption and shear (fibrous) fracture are presented in Figure 3-B. This steel plate would meet the proposed FHWA fracture-toughness requirement of 25 ft-lb at 60°F. Thus, according to the FHWA ft-lb requirement, this plate is acceptable for use when the least anticipated service temperature (LAST) is equal to 60°F or higher. However, this steel plate exhibits 80 percent shear at 175°F. Consequently, according to the FHWA 80 percent shear requirement, this plate is
acceptable for use when the LAST is equal to 175°F or higher. In this example, the LAST defined by the 80 percent shear requirement is 115°F higher than the LAST defined by the ft-lb requirement. Examination of available data, some of which are presented in Table 3-B, shows that this inconsistency is the rule rather than the exception. Examination of the available data for plates up to 2 inches thick of steels up to 60 ksi yield strength shows that the average and mean values of the LAST defined by the 80 percent shear are 115°F and 110°F, respectively, higher than the LAST defined by the proposed FHWA requirement of 25 ft-lb. Thus, A36, A572 and A588 steel plates up to 2 inches thick and up to 60 ksi yield strength must be produced to a 25-ft-lb requirement at 110°F lower than the LAST to ensure that only 50 percent of the product would meet the proposed FHWA requirement of 80 percent shear. A 50 percent product acceptability is totally unsatisfactory from both cost and production standpoints. A36, A572 and A588 steels cannot be produced to this requirement regardless of the melting or processing (including heat treatment) procedures.

The FHWA proposed impact CVN requirements at the LAST increase with increased yield strength such that for plates up to 2 inches thick, the requirement is 25 ft-lb for steels up to 60 ksi yield strength and 50 ft-lb for steels with yield strength of 100 ksi to 110 ksi. Because A514 and A517 steels exhibit an upper-shelf CVN impact toughness in the neighborhood of 50 ft-lb, these steels come closer to satisfying the 80 percent shear requirement than A36, A572 or A588 steels. Consequently, the
FHWA proposed requirements are much more severe for the low-strength steels than for the high-strength steels, which is contrary to fracture-prevention procedures to ensure safety and reliability of structures.

The impact CVN energy absorption (ft-lb) corresponding to 80 percent shear for the steels shown in Table 3-B are presented in Table 4-B. The data in Table 4-B show that the impact CVN energy absorption corresponding to 80 percent shear are significantly higher than the proposed FHWA impact CVN energy absorption requirements for A36, A572 and A588 steels. In other words, to ensure 80 percent shear at the LAST, the proposed FHWA impact CVN requirements for these steels, Table 2-B, must be increased substantially. The impact CVN energy absorption necessary to ensure 80 percent shear for these steels are significantly higher than the requirements of Table 1-B which were discarded by the FHWA "because of the anticipated extreme difficulty in meeting the requirements."

The data in Table 4-B also show that A514 and A517 steels may exhibit 80 percent shear at the last but may be rejected because the upper-shelf impact CVN energy absorption values for these steels is lower than the proposed FHWA impact CVN requirements.

The 80 percent shear requirement at the LAST is fundamental to the development of the proposed FHWA proposed requirements and should be the primary controlling requirement for accepting or rejecting the steel. Equations 1-B and 3-B were misused by the FHWA to calculate the impact CVN ft-lb
the acceptance or rejection of a steel because the 80 percent requirement is the most severe requirement and, therefore, governs the selection process. The proposed FHWA requirements would eliminate the use of A36, A572 and A588 steels and possibly A514 and A57 steels for fracture-critical members of steel bridges.

**Proposed Nil-Ductility-Transition (NDT) Temperature Requirement**

The preceding discussion shows that A36, A572 and A588 steels cannot meet the 80 percent shear requirement at the LAST. The proposed FHWA requirements include a nil-ductility transition (NDT) at 30°F below the LAST that is required for acceptability of the steel only when the impact CVN specimens do not meet the 80 percent shear requirement at the LAST. Consequently, these steels will be acceptable if they meet the proposed impact CVN ft-lb requirements of Table 2-B, and if they also exhibit an NDT at 30°F below the LAST.

The proposed impact CVN ft-lb values have already been shown to be in error. The justifications given by the FHWA (see SES No. 5, Vol. III, page 34) for requiring an NDT at 30°F below the LAST are:

1. It is "listed as a Supplementary Requirement in several ASTM Standard Specifications (See 1.5.1)."

2. It is "needed in addition to the standard Charpy V-notch impact test because the latter is sometimes not reliable when the heat of steel is highly sensitive to notch acuity."

3. "With the requirement that the LAST be no lower than 30°F above the NDT temperature, one can expect some degree of crack-arrest capabilities."
The alleged justifications have no relationship to the procedure used by the FHWA to develop their proposed requirements. Moreover, the FHWA references section 1.5.1 of SES No. 5, Vol. III in support of their statement that the NDT test is "listed as a Supplementary Requirement in several ASTM Standard Specifications." Generally, ASTM Standard Specifications state that the supplementary requirements "shall not apply unless specified" and "the required acceptance criteria shall be as agreed upon between purchaser and manufacturer. Moreover, section 1.5.1 of reference 3 discusses ASTM steels A633-75 Grade D, A678-75 Grade A, A537-76 Class 2 and A588-75. The only supplementary requirement for A633 Grade D, A678 Grade A and A588 that ASTM supplementary specifications "consider suitable for use" for these steels is the bend test. There is no mention of NDT test for these steels. However, it is common knowledge that the purchaser can purchase any steel to any requirement, including NDT, if agreed upon between purchaser and manufacturer. For the A537 Class 2 steel, the ASTM supplementary requirements list NDT as one of ten standardized tests that are considered suitable for use for this steel. Moreover, no values for acceptability are assigned to the data obtained by using any of the standardized tests. These values must be agreed upon between the purchaser and the manufacturer.

The FHWA states that the NDT test "is needed in addition to the standard Charpy V-notch impact test because the latter is sometimes not reliable when the heat of steel is highly sensitive to notch acuity." This statement is inconsistent with
the proposed FHWA requirements (see Table 2-B in this document and section 1.5.4 of SES No. 5, Vol. I) Section 1.5.4 of Volume I of the FHWA SES No. 5 documents states that "when retesting (impact CVN specimens) fails to produce 80 percent shear fracture as an indicator of upper-shelf performance [see footnote (a) of Table 1.5.4] the steel in FCM's shall have a nil-ductility transition temperature (NDTT) at least 30°F below the LAST." Consequently, if the steel meets the proposed FHWA impact CVN ft-lb and shear requirements, the steel is acceptable without an NDT test. Thus, the NDT is a proposed FHWA requirement in lieu of rather than in addition to the 80 percent shear requirement.

The third alleged justification by the FHWA for the proposed NDT requirement is that "the requirement that the LAST be no lower than 30°F above the NDT temperature, one can expect some degree of crack-arrest capabilities." The nil-ductility-transition test defines the highest temperature at which the NDT specimen fractures brittly in two pieces under impact loading and small displacement. Crack propagation at 30°F above NDT is primarily brittle and the specimens would fracture in two pieces if the small displacements in the test procedure are increased slightly. Ductile crack propagation under impact loading occurs only when the steel is tested at temperatures corresponding to the impact upper-shelf behavior. Consequently, negligible crack-arrest capabilities would be expected at 30°F above NDT because; 1) unstable crack extension at this temperature is primarily brittle, and 2) arrest of unstable-crack propagation cannot be assured by high fracture toughness unless the forces that drive
the unstable extension of the crack are eliminated or decreased to a level lower than the steel's capabilities to resist unstable crack extension (i.e., resistance force).

Despite the preceding observations regarding the FHWA's justification for the NDT requirement, its effect in combination with the proposed impact CVN ft-lb requirements, Table 2-B, on the selection of bridge steels must be examined.

To properly evaluate the proposed FHWA fracture-toughness requirements, complete impact CVN energy-absorption curves and percent shear curves as a function of temperature and NDT must be available. Unfortunately, most published data for steels do not include these properties. NDT data were available for only two of the bridge steels shown in Table 3-B. Those two steels are shown in Table 5-B along with the proposed FHWA impact CVN ft-lb requirements at the LAST, the temperature corresponding to the CVN ft-lb requirement (i.e., the LAST), the FHWA proposed NDT temperature requirement (i.e., 30°F below the LAST), and the actual NDT temperature for each steel. Because, as discussed earlier, these steels failed to meet 80 percent shear at the LAST, these steels can be accepted only if they exhibit an NDT temperature that is 30°F below the LAST. A comparison of the proposed NDT temperature requirement and the actual NDT temperature for these steels, Table 4-B, shows that the two steels would not be acceptable because they do not meet the FHWA proposed NDT requirement.

To further support the preceding observations that bridge steels will not be acceptable according with the proposed
FHWA requirements, data from U. S. Steel Corporation on A588 steel are presented in Table 6-B. The data include plates 5/8 to 2-1/2 inches thick in the hot-rolled and heat-treated (normalized condition. The data show that none of the plates in the hot-rolled or normalized conditions meet the proposed FHWA 80 percent shear requirement at the LAST and that their NDT temperature was always higher than the proposed FHWA NDT temperature requirement at 30°F below the LAST. Thus the steels, even in the normalized condition, do not meet the proposed NDT requirement.

The preceding discussion shows conclusively that the proposed FHWA fracture-toughness requirements are technically unfounded and extremely excessive.
Table 1-B
(Same as FHWA Table C1.5.4.a)

BASE METAL\(^a\)
CHARPY V-NOTCH IMPACT REQUIREMENT
for
FRATURE-CRITICAL MEMBERS

<table>
<thead>
<tr>
<th>YIELD(^b) STRENGTH (KSI)</th>
<th>MINIMUM CVN-IMPACT(^c)(FT-LB) AT THE LAST(^d) for specified thickness ranges (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>up to 1 1/2</td>
</tr>
<tr>
<td>from 36 to 60</td>
<td>35</td>
</tr>
<tr>
<td>over 60 to 70</td>
<td>50</td>
</tr>
<tr>
<td>over 70 to 80</td>
<td>65</td>
</tr>
<tr>
<td>over 80 to 90</td>
<td>80</td>
</tr>
<tr>
<td>over 90 to 100</td>
<td>100</td>
</tr>
<tr>
<td>over 100 to 110</td>
<td>120</td>
</tr>
<tr>
<td>over 110 to 120</td>
<td>145</td>
</tr>
</tbody>
</table>

NOTES:
(a) The CVN-impact testing shall be "P" (plate) frequency testing; when more than one flange or web is stripped from a larger plate, only the larger plate need be tested. The Charpy test pieces shall be coded with respect to heat/plate number and that code shall be recorded on the mill-test report of the steel supplier with the test result.

(b) The yield strength is the value given in the certified MILL TEST REPORT.

(c) Average of three (3) tests. If the energy value for more than one of the three test specimens is below the minimum average requirement, or if the energy for one of the three specimens is less than 75 percent of the specified minimum average requirement, a retest shall be made and the energy value obtained from each of the three retest specimens shall equal or exceed the specified minimum average requirement.

(d) The lowest anticipated service temperature (the LAST) shall be based on the isoline in Figure 1.5.4.1 nearest the geographical location of the structure.

(e) Plate in excess of 2-inch thick shall not be used in FCMs when the yield strength exceeds 90 ksi.
Table 2-B
(Same as FHWA Table 1.5.4)

CHARPY-IMPACT REQUIREMENT(a) for FRACTURE CRITICAL MEMBERS

<table>
<thead>
<tr>
<th>YIELD(b) STRENGTH (ksi)</th>
<th>MINIMUM CVN-IMPACT (ft-lb)(c) AT THE LAST(d) FOR SPECIFIED THICKNESS RANGES (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>up to 2</td>
</tr>
<tr>
<td>from 36 to 60</td>
<td>25</td>
</tr>
<tr>
<td>over 60 to 70</td>
<td>30</td>
</tr>
<tr>
<td>over 70 to 80</td>
<td>35</td>
</tr>
<tr>
<td>over 80 to 90</td>
<td>40</td>
</tr>
<tr>
<td>over 90 to 100</td>
<td>45</td>
</tr>
<tr>
<td>over 100 to 110</td>
<td>50</td>
</tr>
<tr>
<td>over 110 to 120</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>over 2 1/2 to 3</td>
</tr>
</tbody>
</table>

NOTES:  
(a) The CVN-impact testing shall be "P" (plate) frequency testing; when more than one flange or web is stripped from a larger plate, only the larger plate need be tested. The Charpy test pieces shall be coded with respect to heat/plate number and that code shall be recorded on the mill-test report of the steel supplier with the test result. The fracture appearance at the LAST shall be no less than 80 percent shear (see ASTM A370-75, Section 23.2.2.1). If the fracture appearance in any one specimen is less than 80 percent shear (fibrous), a retest shall be made and the fracture appearance of each of the three retest specimens shall equal or exceed the 80 percent shear requirement. If the retest specimens fail to meet the fractureappearance, ASTM E208 testing is required.

(b) The yield strength is the value given in the certified MILL TEST REPORT.

(c) Average of three (3) tests. If the energy value for more than one of the three test specimens is below the minimum average requirement, or if the energy value for one of the three specimens is less than 75 percent of the specified minimum average requirement, a retest shall be made and the energy value obtained from each of the three retest specimens shall equal or exceed the specified minimum average requirement.

(d) The lowest anticipated service temperature (the LAST) shall be based on the isoline in Figure 1.5.4.1 nearest the geographical location of the structure.

(e) Plate in excess of 2-inch thick shall not be used in FCMs when the yield strength exceeds 90 ksi.
Table 3-B*

<table>
<thead>
<tr>
<th>Steel</th>
<th>Yield Strength, ksi</th>
<th>FHWA Impact CVN Requirements, ft-lb</th>
<th>LAST Temp. Corresponding to FHWA ft-lb Requirement, °F</th>
<th>LAST Temp. Corresponding to 80 Percent Shear Requirement, °F</th>
<th>Difference in Temp. for the Two Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>A36</td>
<td>36.0</td>
<td>25</td>
<td>+35</td>
<td>+110</td>
<td>75</td>
</tr>
<tr>
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<td>43.6</td>
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* These data are for a given heat and plate thickness and are not necessarily representative of the product.
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* These data are for a given heat and plate thickness and are not necessarily representative of the product.
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* These data are for given heat and plate thicknesses and are not necessarily representative of the product.
Relationship Between $K_{IC}$ and Impact Charpy V-Notch Energy Absorption in the Upper-Shelf Region (Iwadate, et al). This Figure is FHWA Figure Cl.5.4.f.

Figure 1-B
Schematic Representation of the Region for Which the Super-Shelf Correlation Can be Used to Predict $K_{IC}$ Values from Charpy V-Notch Energy Absorption.

Figure 2-B
Impact Charpy V-Notch Energy Absorption and Fibrous Fracture for a 1-Inch-Thick Plate of A36 Steel.

Figure 3-B
Appendix C

DISCUSSION OF FRACTURE EXAMPLES

Several examples of brittle fractures are presented in SES No. 7. In the analysis of these failures, the CVN impact values of the steels are compared with the CVN impact values from the AASHTO material toughness requirements. In most cases the same general incorrect conclusion is reached, namely: because the steels would have met the AASHTO material toughness requirements, and yet a brittle fracture still occurred, then the AASHTO requirements are not satisfactory.

If brittle fracture were solely a material problem, then this conclusion might be valid. However, as has been demonstrated time after time, very often there are other factors (e.g., design of details, fabrication and inspection, loading) that actually have a greater contribution to a brittle fracture than the material toughness.

In fact, a simple engineering mechanics analysis of a steel member loaded such that the principle stresses ($\sigma_1$, $\sigma_2$, and $\sigma_3$) are equal shows that tensile failure can occur with no apparent yielding. In this example, the failure would appear to be a brittle one with no apparent ductility regardless of the level of notch toughness because the fracture strength would be reached before any yielding occurs. That is, the failure would result from the severe constraint (loading and detail geometry that resulted in $\sigma_1 = \sigma_2 = \sigma_3$) not because of the particular level of notch toughness or ductility.
Accordingly, the failures described in SES No. 7 are analyzed in light of the primary causes of failure to illustrate the point that other factors contributed to these failures than just material toughness. In fact, in several cases, the steels would have met the CVN impact requirements proposed in SES No. 7 and yet the failures still occurred (although none would have met the inappropriate near upper-shelf requirement of 80% shear at the LAST). This observation further substantiates the importance of factors other than material toughness in preventing brittle fractures.

Merchant Ship Failures

Between 1942 and 1952, more than 200 ships had sustained fractures classified as serious and at least nine T-2 tankers and seven Liberty ships had broken completely in two as a result of brittle fractures. Although steel quality later was found to be an important factor in these failures, the immediate solution to the problem was achieved by design changes and better quality fabrication. It was not until the 1950's that changes in material toughness were made.

In spite of these changes that have been made in material toughness, brittle fractures still have occurred in ships. One of the more widely quoted recent ship failures is that of the Ingram Barge, which has been used as an example to prove many different viewpoints. However a proper analysis of the Ingram Barge failure shows that a specific detail and overload were the primary causes of failure, not the level of notch toughness of the material.
Ingram Barge

On January 10, 1972, the 584-ft long Ingram Barge fractured in a brittle manner. At the time the air temperature was 45°F and the ship was turning slowly in calm seas at the harbor. Failure originated at an unusually high stress level for this type of structure (2.5 times design load or a nominal stress level of about 24 ksi. The ship had been in service for only 9 months.

The fracture initiated in a region of very high constraint at a doubler ring welded to the deck plate with a king post welded to the doubler. Also, four gusset plates were welded to the king post and deck as stiffeners.

No pre-existing flaw was observed, even though chevron markings on both sides of the king post pointed toward the fracture origin at the king post. The local geometry was such that a high triaxial state-of-stress existed and, thus, failure occurred at very high loads even in the absence of a flaw. A finite element stress analysis verified that the local stress level was well above the yield stress level. Because there was no pre-existing crack, the notch toughness level did not have as significant an effect on the initiation as did the local geometrical constraint and loading. Once a brittle fracture had initiated in the deck plating, the loading conditions (essentially constant load) were such that the cracks would be expected to propagate in both directions until the potential energy was dissipated by a complete fracture. That is, even steels with very high toughness levels probably would not have arrested the running crack.
The steel had reasonably good notch toughness as measured by the Charpy V-notch impact test specimen (55 ft-lb at the service temperature of 45°F). The notch toughness as measured with one of the most severe notch toughness tests available (i.e., the DT test), was near the lower end of the DT transition range. However, the NDT of this steel was 10-20°F, approximately 30°F below the service temperature. Thus this steel would have met the FHWA NDT requirement. It should be emphasized that this steel also would have easily met the CVN impact requirements proposed in SES No. 7.

Overall analysis of this failure leads to the conclusion that the very large sustained loading at a highly restrained detail (i.e., a "hard spot") caused by the particular condition of ballasting was the primary cause of the failure. It is also possible that a superimposed dynamic loading occurred because of premature separation of the stiffener from the deck plate.

The notch toughness of the steel at the service temperature as measured by the CVN impact test was very good, and certainly better than that found in many other types of structures. Also, as stated above, it would have met the toughness requirements proposed in SES No. 7.

This particular brittle fracture emphasizes the fact that brittle fracture is not, and never has been, just a material problem. Design, fabrication, materials, inspection, and operation (i.e., loading) are all factors that affect the susceptibility of a structure to brittle fracture. When one or more of these factors is significantly more severe in a particular structure,
compared with conditions in other structures of the same generic class, then the possibility of brittle fracture obviously is increased. In the case of the Ingram Barge, the sustained loading of 2.5 times the design load in the presence of a severe geometrical "hard spot" was significantly different than usually is found in this type structure. Therefore, this loading at the particular king post detail was the dominant factor leading to the brittle fracture, not the particular level of material toughness.

**I-79 Bridge - Pittsburgh**

The brittle fracture that occurred in the I-79 bridge near Pittsburgh initiated in a weld repair of an electro-slag weldment in the tension flange. Such weld repairs are clearly poor fabrication practice and resulted in an initial defect size of about 3-inches. This defect then grew by fatigue until the time of failure, which occurred during a very severe and sudden decrease in the temperature. Until more is known about the general behavior, including fatigue, of electro-slag weldments, they are not being used in bridge members. However, the key point to be made in an analysis of this failure is that the primary cause was the presence of a large defect in the electro-slag weld as a result of a weld repair, not the particular notch toughness of the A588 base metal. In this failure, the toughness level of the steel had nothing to do with the failure.

**Lafayette Street Bridge**

The fracture of a main girder of the Lafayette Street Bridge was due to the formation of a fatigue crack in the lateral
bracing gusset to transverse stiffener weld. This fatigue crack originated from a large lack of fusion region that existed near the back-up bars. At the gusset where failure initiated, this groove weld intersected the transverse stiffener-web weld and thus provided a direct path for the fatigue crack to follow into the girder web. After the fatigue crack had nearly propagated through the girder web, it precipitated a brittle fracture of part of the web and all of the tension flange. Subsequent cyclic load permitted further fatigue crack growth over a limited time interval, which eventually resulted in substantial extension of the web crack. Even had the toughness level of this steel been higher than it was, failure still would have occurred in a relatively short period of time because the primary problem was the initial defect and the fact that there was a continuous path for the fatigue crack to grow into the girder web.

A36 CAP Girder

A detailed examination and analysis of the fracture surfaces removed from pier box girders of the CTA Dan Ryan elevated structure has shown that these fractures all originated from fatigue cracks. The cracks developed at the tips of the stringers that pierced the steel box pier bents.

The fabrication of the structure resulted in slots for the stringer flanges being flame cut into the box girder webs. The stringer flange was inserted through these slots and then connected to the web by groove welds all around the flange surface. At the stringer flange tips an interior lack of fusion region was produced at every joint. The high stress concentration that
existed at the exterior weld surface adjacent to the stringer flange tips and the interior lack of fusion region resulted in a joint that was very susceptible to fatigue crack propagation at very low levels of stress range. An Analysis of the joint geometry showed that fatigue crack growth would occur at stress ranges between 1 and 2 ksi. The striation spacing detected on the fatigue crack surface also confirmed that very low stress range levels were propagating the cracks.

The combination fatigue crack - lack of fusion region at the stringer flange tips resulted in cracks that reached a critical condition at the low temperatures that were experienced in Chicago in late December 1977 and early January 1978. All of the conditions necessary for fracture of the box bent existed at that time. First, large crack as a result of fatigue crack growth between the outside of the weld and the lack of fusion region in the box web at each stringer flange tip. Second, a high tensile stress field at the crack tips in the pier box girder web primarily from the weld restraint conditions where the welds connecting the stringer flange to the pier box webs intersect. This resulted in yield point residual tensile stress at the crack tip when these cracks propagated into these regions.

The combination crack size and high stress field resulted in a stress intensity factor estimated to be between 77 ksi√in and 89.6 ksi√in at -5°F. Hence brittle fracture was very likely under the combination of conditions that existed when the cracks were first observed and the level of material toughness was not one of the primary factors responsible for the failure.
Fort Howard Paper Company

The girders in the Fort Howard Paper Company building were fabricated from coiled steel formed into channels by press braking. As would be expected, the toughness of this steel did not meet the AASHTO toughness requirements. However the primary cause of this failure was a hot crack that occurred at the ends of discontinuous back-up bars. The severity of this detail has been recognized by the American Welding Society and is specifically not allowed by the American Welding Society Structural Welding Code (AWS 01.1-72) Section 4.7.3 that states:

"4.7.3 Steel backing shall be made continuous for the full length of the weld. All necessary joints in the steel backing shall be complete joint penetration butt welds meeting all workmanship requirements of Section 3 of this Code".

Given the extremely severe stress concentration, failure of these girders at low temperatures might well be expected regardless of whether or not the steel met either the AASHTO toughness requirements or the ones proposed in SES No. 7.

ASTM A572-65 Steel

The case of brittle fracture involving 1-inch-thick ASTM A572-65 steel was discussed on page 15 of SES No. 7. This steel exhibited a CVN impact value of about 26 ft lb at -20°F. Because the service temperature was about 0°F, the steel easily met the AASHTO notch toughness requirement of 25 ft lb at +70°F. It also met the proposed FHWA impact requirements at the
LAST. Failure occurred at a nominal stress level of 21 ksi (minimum yield strength was 65 ksi for the steel) and originated from a discontinuous back-up bar. AWS specifications specifically prohibit the use of discontinuous back-up bars, as given above in Section 4.7.3 of the AWA Structural Welding Code. Thus, even though this steel met the AASHTO and proposed FHWA toughness specifications as presented in SES No. 7, and the stress level was within the design limits, failure occurred because of the severe stress concentration at the discontinuous back-up bar. This failure once again illustrates that poor details and fabrication procedures can result in brittle fractures in steels with relatively good notch toughness, and which are subjected to normal design loadings.

Summary

In summary, analysis of these service failures emphasizes the fact that brittle fracture is not, and never has been just a materials problem. Design, fabrication, materials, inspection, and operation (i.e., loading) are all factors that affect the susceptibility of a structure to brittle fracture. When one or more of these factors is significantly more severe in a particular structure, compared with conditions in other structures of the same generic class, then the possibility of brittle fracture obviously is increased. Analysis of the failures discussed above shows that this is the case and that in most of these failures, the particular level of notch toughness of the material was not the primary cause for failure. In fact, in several cases, the steels also would have met the more stringent
notch toughness requirements proposed in SES No. 7, and yet brittle fractures still occurred. This observation dramatically illustrates the fact that the overall service experience with bridges has been most satisfactory, particularly when compared with large number of bridges in service. The present AASHTO Fracture Control Plan, in conjunction with other AASHTO specifications, should minimize the possibility of brittle fracture initiation in those cases where good design and fabrication practices are followed.

In those rare cases where good design, fabrication, inspection, and loading practices are not followed, merely increasing the material toughness requirements to test at the LAST will not be sufficient to always prevent brittle fracture initiation and propagation in bridge members. Analysis of service failures substantiates this fact.