Fracture and Fatigue Control in Steel Structures

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CONSIDERABLE effort has been devoted to the prevention of brittle fracture* in manufactured structures such as aircraft and pressure vessels, where large numbers of essentially identical structures are fabricated under closely controlled conditions. For example, the emphasis on safety and reliability of nuclear pressure vessels and the ensuing extensive research, as well as stringent controls, have led to a situation where the probability of a brittle fracture in a nuclear pressure vessel is virtually zero. For other types of manufactured structures, the causes of field failures usually can be remedied by changes in design of subsequent units.

In contrast, other types of structures, such as bridges and buildings, are often individually designed for a specific function and location. The overall service experience of steels in these structures has been excellent, so that the designer in the past has seldom concerned himself with notch-toughness as a design parameter. However, the trend in structural design has been such that the following changes have occurred.

1. Structural engineers and architects are designing more complex structures than in the past.
2. There is increased use of high-strength, thick, welded steel members, as compared with lower-strength, thinner, riveted or bolted steel members.
3. The choice of construction practices has become increasingly dependent on minimum cost.
4. The magnitude and number of types of loadings considered in design have increased.

Because of the above noted changes, the increasing number of structures subjected to severe loadings (such as offshore drilling rigs), the use of more precise methods of analysis, and the explicit recognition of inelastic behavior in the design process, the probability of brittle fracture incidence in structures of many types would appear to be increasing. Therefore, the designer should become more aware of the conditions under which brittle fracture may occur and the available methods for preventing brittle fractures, particularly in view of the current AISC *Code of Standard Practice,* which assigns responsibility for the suitability, adequacy, or legality of a design.

Almost all large complex steel structures are designed using structural steels that have yield strengths ranging from 36 to 100 ksi. These steels have inherent levels of notch toughness that generally are adequate for most structural applications. However, the fracture behavior of these structural steels and weldments can be affected significantly by temperature, loading rate, stress level, and flaw size, as well as by plate thickness or constraint, joint geometry, and workmanship. The effect of temperature on notch toughness is generally well known, but the roles of stress (or strain), flaw size, loading rate, and thickness are less well known. In addition, it is possible for the inherent notch toughness of these steels to vary depending upon manufacturing variables (thermo-mechanical history), even though the steel may meet an existing chemistry or tensile test specification.

From a fracture control viewpoint, therefore, the basic problems are as follows. Is it necessary to specify notch toughness for the steels and weldments used in a particular class of structures, based on the specific design, fabrication, and service conditions to which the structures will be subjected? Furthermore, if notch toughness requirements are necessary, what notch toughness level should be specified to ensure satisfactory performance at reasonable cost. Also, what joining techniques and fabrication controls are required, consistent with the overall service conditions and consequences of failure. It should be noted that notch-toughness requirements often are developed to be used in conjunction with good design, fabrication, and inspection procedures, without being specific as to how "good" procedures are defined.

Because the cost of structural steels generally increases with their ability to perform satisfactorily under more severe operating conditions, the designer should not arbitrarily specify more notch toughness than is required. How much notch toughness is sufficient for a particular structural application is a difficult question to answer, and es-

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* *Brittle fracture is a type of catastrophic failure that usually occurs without prior plastic deformation and at extremely high speeds (crack speeds as high as 7,000 fps or possibly more). The fracture is usully characterized by a flat fracture surface (cleavage) with little or no shear lips and at average stress levels below those of general yielding. Brittle fractures are not so common as fatigue, yielding, or buckling failures, but when they do occur they may be more costly in terms of human life and/or property damage.*

tablishing the fracture-toughness requirements and the concomitant quality control and inspection requirements for various structural applications should be an important design consideration. As with most other aspects of design, it is as much an economic matter as a technical one.

Over the years many different tests have been used to evaluate the notch toughness of steels. These include the Charpy V-notch (CVN) impact test, the drop-weight nil-ductility transition (NDT) test, the dynamic tear (DT) test, the wide-plate test, the Battelle drop-weight tear (DWTT) test and many others.[1-7] Generally, these tests were developed for a specific purpose. The CVN test is widely used as a screening test in alloy steel development as well as a quality-control test. In addition, because of correlations with service experience, the CVN impact test often is used in specifications for alloy steels for various structural and pressure-vessel applications. The NDT test is used to establish the minimum service temperature for various Navy and structural applications, whereas the Battelle DWTT test was developed to relate the fracture appearance of line-pipe steels to temperature.

All these tests generally have one thing in common, namely, to produce fracture in steels under carefully controlled laboratory conditions. Hopefully, the results of the tests can be correlated with service behavior to establish levels of performance for various steels being considered for specific applications. However, even if correlations are developed for a class of materials and structures, they do not necessarily hold for other designs, new operating conditions, or new materials, because the results, which are expressed in terms of energy, fracture appearance, or percentage deformation, cannot be translated into normal structural design and inspection parameters, namely, stress and flaw size. Fortunately, recent advances in the fracture mechanics field have led to techniques and concepts which permit a more rational approach to fracture as a part of the design process than was possible in the past.

FRACTURE MECHANICS AND DESIGN

As a general rule the designer must properly proportion his structure to prevent failure by tensile overload (yielding or ductile fracture), compressive instability, and by stable crack growth (for example, arising from fatigue or stress corrosion) or unstable crack growth (brittle fracture). Design to prevent brittle fracture usually refers to using a relatively low allowable stress level, as well as to the elimination (as much as possible) of those structural details that act as stress raisers that can be potential fracture initiation sites, e.g., certain weld joint details, holes, intersecting plates, arc strikes, etc. Actually, large complex structures (welded or bolted), cannot be designed or fabricated without some discontinuities, although good design and fabrication practices can minimize the original size and number of these discontinuities. It is realized that stress concentrations or discontinuities will be present, but the designer assumes that his structural materials will yield locally and redistribute the load in the vicinity of these stress concentrations or discontinuities. The selection of structural materials and allowable stress levels is based on the appropriate realization of the fact that crack-like discontinuities in large complex structures may be present or may initiate under cyclic loading or stress corrosion, and that some level of notch toughness is desirable.

"Fracture mechanics" is a term commonly used to describe a method of characterizing fracture toughness, fatigue crack growth, or stress-corrosion crack growth behavior in terms of structural design parameters familiar to the engineer, namely, stress and flaw size.[7] Fracture mechanics commonly is subdivided into two general categories: linear-elastic and elastic-plastic* fracture mechanics. Although linear-elastic fracture mechanics techniques are established reasonably well as compared with elastic-plastic fracture mechanics, most commonly used structural metals do not behave elastically to fracture and thus linear-elastic fracture analysis techniques are not directly applicable to most structural steels. This is good, because obviously the engineer wants his materials to exhibit gross structural general-yielding behavior rather than a brittle type (linear-elastic) behavior.

Elastic-plastic fracture mechanics approaches are not yet well-defined and, in fact, no widely accepted simple analysis technique for this type of behavior is available to the engineer. Considerable research on elastic-plastic fracture mechanics is underway. However, the research based approaches are yet to be simplified to the point where they can be widely used by engineering designers, although Crack-Opening-Displacement (COD) test methods have been used in some areas of fracture analysis for large structures, for example the Alaska pipeline.

Although research has shown that numerous factors can contribute to brittle fractures in large welded structures, the recent development of fracture mechanics has shown that there are three primary factors (conceptually) that control the susceptibility of a structure to brittle fracture. These three primary factors are:

1. **Material Toughness**—Material toughness can be defined as the resistance to unstable crack propagation in the presence of a notch. For linear-elastic behavior the material toughness is measured in terms of a static critical stress-intensity factor under conditions of plane stress (K_c), of plane strain (K_{Ic}), or for dynamic loading (K_{Id}). For elastic-plastic fracture behavior, the material toughness may be measured in terms of ductility related parameters such as J_{Ic}, R-curve, COD, and Equivalent Energy Approaches as defined below:

   J-Integral Technique—A path-independent integral which is an average measure of the elastic-plastic stress/strain field ahead of a crack. For elastic conditions, $J_{Ic} = K_{Ic}^2/E(1 - \nu^2)$. A test method for this approach is currently in development.

* *Sometimes referred to as "general yielding", particularly in the British literature. The term "elastic-plastic" connotates the situation where a significant yield zone relative to plate thickness of inelastic straining occurs near the crack tip such that the linear-elastic analyses are not applicable.*

Resistance-Curve (R-Curve) Analysis—A procedure used to characterize the resistance to fracture of materials during incremental slow-stable crack extension, K_R. At instability $K_R = K_c$, the plane stress fracture-toughness which is dependent upon specimen thickness, as well as temperature and loading rate.

Crack-Opening Displacement (COD) Technique—Toughness evaluation in terms of the pre-fracture deformation at the tip of a sharp crack that shows considerable potential as a fracture criterion; a proposed test method has been developed by the British Standards Institution.

Equivalent Energy Approach—An energy approach based on using test results to predict failure, primarily of thick walled pressure vessels.

2. **Flaw Size**—Brittle fractures initiate from flaws or discontinuities of various kinds. These discontinuities can vary from extremely small cracks, for example, from within a weld arc strike (as was the case in the brittle fracture of a T-2 tanker during World War II), to much larger weld or fatigue cracks. Even though only small flaws may be present initially, repeated loading (fatigue), or stress corrosion can cause them to enlarge, possibly to a critical size where brittle fracture can occur.

3. **Stress Level**—Tensile stresses (applied, residual, or both) are necessary for brittle fractures to occur.

Engineers have known the foregoing facts for many years and have reduced the susceptibility of structures to brittle fractures by applying these concepts to their structures, *qualitatively*. That is, good design (the use of lower stress levels and the minimizing of discontinuities) and sound fabrication practice (decreased flaw size through use of proper welding procedures and control), as well as the use of materials with good notch toughness levels (e.g., as measured with a Charpy V-notch impact test), have minimized the probability of occurrence of brittle fractures in structures. However, the engineer has not had techniques available to permit evaluation of the relative performance and economic trade-offs between design, fabrication, and materials in a *quantitative* manner prior to the development of fracture mechanics.

The fundamental concept of linear-elastic fracture mechanics is that the stress field ahead of a sharp crack can be characterized in terms of a single parameter, K_I, the stress intensity factor for flat crack propagation (usually referred to as opening mode), having units of ksi$\sqrt{\text{in.}}$ This single parameter K_I is related to both the stress level, σ, and the flaw size, a. When the particular combination of σ and a leads to a critical value of K_I, called K_{Ic} or K_c, unstable crack growth occurs. The equations that describe the elastic-stress field in the vicinity of a crack tip in a body subjected to tensile stresses normal to the plane of a simple crack are presented in Fig. 1. These stress-field equations define the distribution of the elastic-stress field in the vicinity of the crack tip, and can be used to establish the

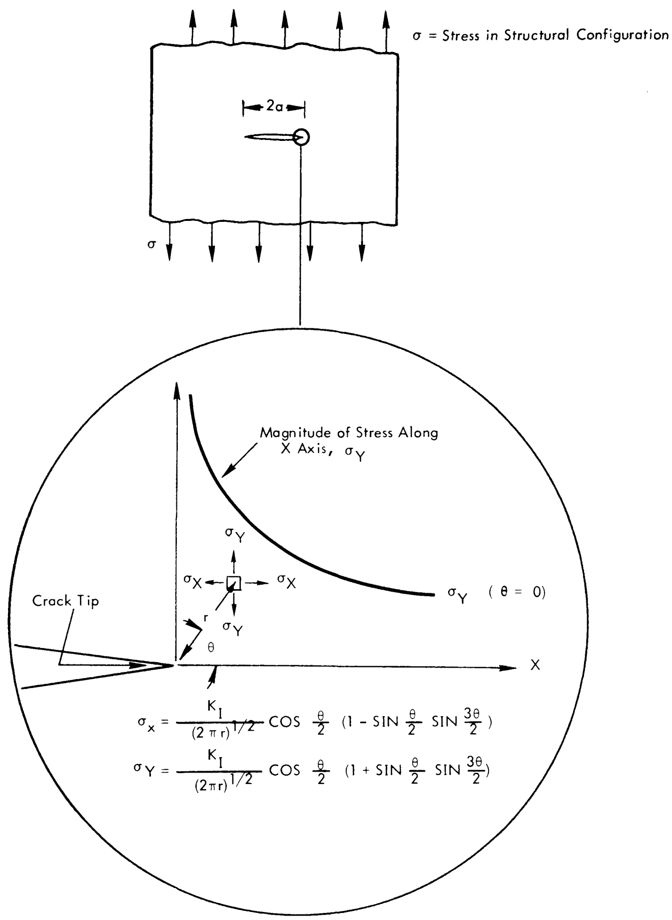

Fig. 1. Elastic-stress-field distribution ahead of a crack

relation between K_I, σ, and a for different structural configurations, as shown in Fig. 2.[8] Other crack geometries have been analyzed for different structural configurations and are published elsewhere.[9,10]

If the critical value of K_I at failure (K_c, K_{Ic}, or K_{Id}) can be determined for a given metal of a particular thickness and at a specific temperature and loading rate, the designer can determine theoretically the flaw size that can be tolerated in structural members for a given design stress level. Conversely, he can determine the design stress level that can be safely used for a flaw size that may be present.

This general relationship between material toughness K_{Ic} or K_c, nominal stress σ, and flaw size a is shown schematically in Fig. 3. If a particular combination of stress and flaw size in a structure (K_I) reaches the K_{Ic} or K_c level, fracture can occur. Thus, there are many combinations of stress and flaw size (e.g., σ_f and a_f), that may cause fracture in a structure that is fabricated from a steel having a particular value of K_{Ic} or K_c at a particular service temperature, loading rate, and plate thickness. Conversely, there are many combinations of stress and flaw size (e.g., σ_o and a_o), that will not cause failure of a particular steel, i.e., below the K_{Ic} or K_c line.

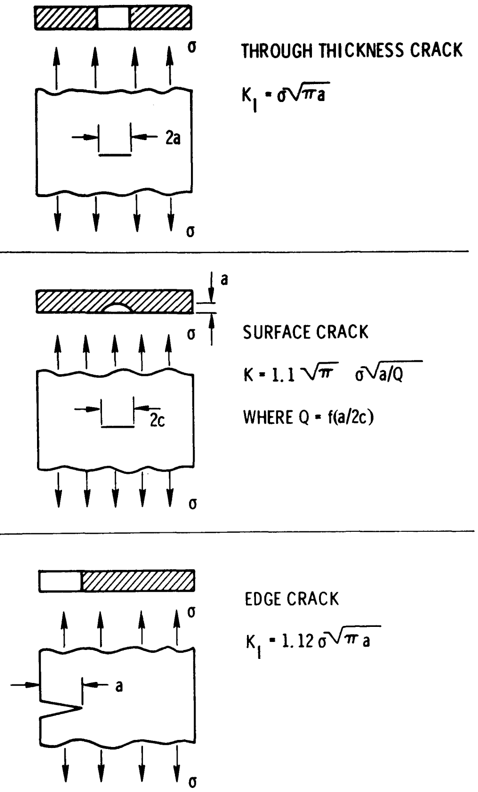

Fig. 2. K_I values for various crack geometries

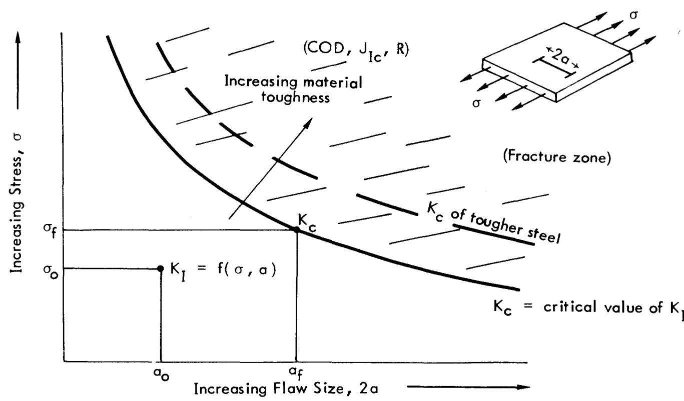

Fig. 3. Schematic relationship between stress, flaw size, and material toughness

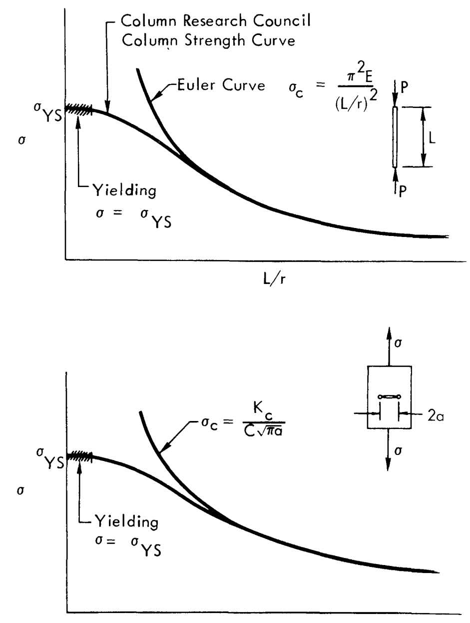

Fig. 4. Column instability and crack instability (after Madison, Ref. 12)

A useful analogy for the designer is the relation between applied load P, nominal tensile stress σ, and yield or limit stress σ_{ys} in an unflawed structural member, and between applied load P, stress intensity K_I, and critical stress intensity for fracture K_c, K_{Ic}, or K_{Id} in a structural member with a flaw. In an unflawed structural member, as the load is increased, the nominal stress increases until a limit loading (yielding) occurs. As the load is increased in a structural member with a flaw (or as the size of the flaw grows by fatigue or stress corrosion), the stress intensity K_I increases until a limit condition (fracture at K_c, K_{Ic}, or K_{Id}) occurs. Thus, the K_I level in a structure should always be kept below the appropriate K_c value, just as the nominal design load is always kept below the limit loading.

Another analogy that may be useful in understanding the fundamental aspects of fracture mechanics is the comparison with the Euler column instability curve, Fig. 4.[11,12] The stress level required to cause instability in a column (buckling) decreases as the L/r ratio increases. Similarly, the stress level required to cause instability (fracture), in a flawed tension member decreases as the flaw size a increases. As the stress level in either case approaches the yield strength, both the Euler analysis and the K_c analysis are invalidated because of yielding. To prevent buckling, the actual stress and L/r values must be below the Euler curve. To prevent fracture, the actual stress and flaw size a must be below the K_{Ic} or K_c level shown in Fig. 4. Obviously, using a material with a high level of notch toughness will increase the possible combinations of design stress and flaw size a structure can tolerate without fracturing.

At this point, it should be emphasized that the K_c levels for most common structural steels are so high that they cannot be measured directly using existing ASTM standardized test methods.[13] Thus, although concepts of fracture mechanics can be used to develop fracture-control guidelines and desirable toughness levels, the state-of-the-art is such that actual K_{Ic} or K_c values cannot be measured for most commonly used structural metals at service temperatures. Therefore, traditional notch toughness tests (e.g., CVN, NDT, etc.) are widely used at the present time to specify the notch toughness requirements for various structural applications. Examples of the use of

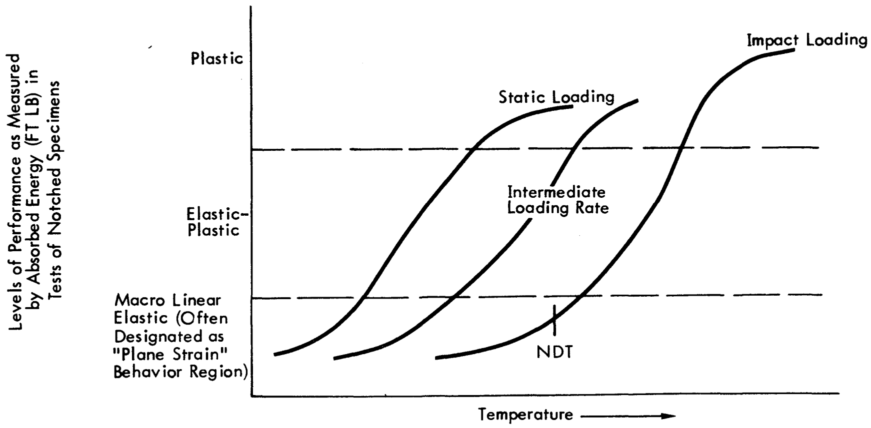

Fig. 5. Schematic showing relationship between notch toughness test results and levels of structural performance for various loading rates

such test methods in specifications are the recently developed AASHTO material toughness requirements for bridge steels and the ASME toughness requirements for steels for nuclear vessels. In both of these cases, concepts of fracture mechanics were used to develop the desired toughness requirements, but the actual material toughness requirements are in terms of CVN or NDT values based on empirical correlations.[7]

FRACTURE CRITERIA

A fracture criterion is a standard against which the expected fracture behavior of a structure can be judged. In general terms, fracture criteria are related to the three levels of fracture performance as shown in Fig. 5, namely macro linear-elastic (often referred to as "plane strain" in the fracture mechanics literature), elastic-plastic, or fully plastic. Although it would appear desirable to specify fully plastic behavior, this is rarely done because it is almost always unnecessary, as well as being economically undesirable in many cases.

For most structural applications, some moderate level of elastic-plastic behavior at the service temperature and loading rate constitutes a satisfactory performance criterion. While there may be some cases where considerable inelastic behavior is necessary (e.g., dynamically loaded military protective structures), or where low toughness level behavior can be tolerated, (e.g., certain short-life aerospace applications where the loading and fabrication can be precisely controlled), for the majority of large complex structures such as bridges, ships, buildings, pipelines, offshore drilling rigs, etc., some moderate level of elastic-plastic behavior at the service conditions is satisfactory. The question arises then as to what level of elastic-plastic behavior is required and how this level of performance can be insured.

Unfortunately the selection of a fracture criterion is often quite arbitrary and based on service experience for other types of structures that may have no relation to the particular structure an engineer may be designing. Also, selection of a fracture criterion alone, without considering the other factors involved in fracture control, will not necessarily result in a structure with the desired margin of safety. An example of the use of a fracture toughness criterion developed for one application but also widely used in many other situations is the 15 ft-lb CVN impact criterion at the minimum service temperature, which was established on the basis of the World War II ship failures. This criterion has been widely used for various types of structures, even though the material, service conditions, structural redundancy, etc., may be quite different from those of the World War II ships for which the criterion was established.

Criteria selection should be based on a careful study of the particular performance requirements for a given structure. The factors involved in the development of criteria commonly include:

1. Service conditions (loadings, temperature, controlling stress and strain levels, loading rate, cyclic loading, etc.) to which the structure will be subjected.
2. Desired level of performance and margin of safety of the structure under both normal service and extreme loading conditions.
3. Possible modes and consequences of failure.

There is no single fracture criterion that can be applied to all structures, because optimum design involves economic considerations as well as technical trade-offs.

At the present time it is difficult to establish notch toughness criteria for the following reasons:

1. Establishment of the specific level of required notch toughness (i.e., the required CVN, K_{Ic}, or K_{Id} value at a particular test temperature), is costly and time consuming, and is a subject unfamiliar to engineers. Furthermore, it depends on many factors such as the particular service loadings, design of structural details, quality of fabrication, inspection, etc., which are difficult to establish.

2. There is no well-recognized single "best" approach. Therefore, different experts will have different opinions as to the "best" approach, although the science of fracture mechanics is slowly helping to overcome this difficulty.
3. The cost of structural materials increases with increasing levels of inherent notch toughness. Thus economic considerations as well as technical ones must be included when establishing a toughness criterion. However, any design criterion includes economic as well as technical considerations.

A general fracture criterion defined in terms of the levels of performance (linear-elastic, elastic-plastic, and plastic), as described in Fig. 5, must be translated into some specific fracture test requirement that insures the desired level of performance. For example, a general requirement that a structural material exhibit elastic-plastic behavior at service temperatures is a general criterion that is useful to the engineer. However, because of ambiguity and differences in opinion, this general criterion must be made specific in terms of a fracture test specimen and some specified index value. An example of a general toughness criterion might be that a low level of elastic-plastic behavior is required under the most severe expected service conditions. The specific toughness criterion for this example might be that "all structural steels and weldments used in this assemblage must exhibit 21 ft-lb energy absorption as measured in a standardized longitudinal Charpy V-notch impact test specimen tested at 32°F." Hopefully, this particular criterion would have been based on sufficient laboratory results, service experience, and fracture mechanics analysis to insure that the desired structural behavior is consistent with economic considerations. The criterion would then be specified for purchase of materials and quality control during fabrication.

As a result of several large structural failures in the period 1967–1972,[14–18] as well as a growing concern with the overall reliability and safety of structures, many specifications are now beginning to include specific minimum toughness requirements. This trend is expected to grow as regulatory governmental agencies become increasingly active in the development of mandatory fracture prevention criteria. Recent examples are the ASME Nuclear Code,[19] AASHTO Material Toughness Requirements,[20] and the floating nuclear power plant hull structure toughness requirements[7] imposed by the United States Coast Guard and the Nuclear Regulatory Commission.

There are two general parts to a fracture criterion:

1. **General Test Specimens to Categorize the Material Behavior**—Throughout the years, various fracture criteria have been specified using notch toughness tests such as CVN impact, NDT, DT, and, more recently, the fracture mechanics test specimens used to measure K_{Ic} and K_{Id}. Test specimens currently used as research tools and expected to be used more extensively in the future for metals in the lower yield strength category are J_{Ic}, COD, and R-curve specimens. The specimen used for a particular application should be that which most closely models actual structural behavior. However, commonly the selection of the test specimen is based on past experience as well as economics of testing.
2. **Specific Notch Toughness Value or Values**—The second and more difficult part of establishing a fracture criterion is the selection of the specific level of performance using a particular test specimen. The specified values in any criterion should be an optimization of both safe structural performance and cost, and depend to a large degree on the design, quality of fabrication, inspection, and loading for the particular structure.

EFFECTS OF TEMPERATURE, LOADING RATE, AND THICKNESS

In general, the notch toughness of most structural steels increases with increasing temperature and decreasing loading rate. The effect of temperature is well known and has led to the transition-temperature approach to designing to prevent fracture. However, the effect of loading rate may be equally as important, not only in designing to prevent fracture, but in understanding the satisfactory behavior of many existing structures built from materials that have low impact toughness values at their service temperatures.

The general effects of temperature and loading rate on K_{Ic} and Charpy V-notch behavior are shown schematically in Figs. 6 and 7. The toughness of most structural steels

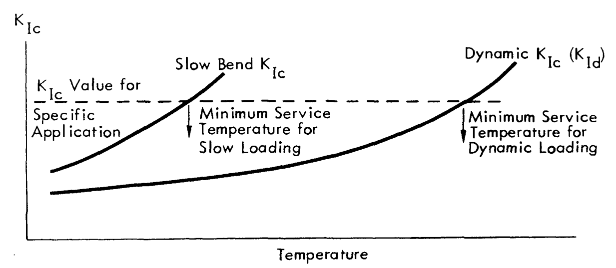

Fig. 6. Schematic showing effect of temperature and loading rate on K_{IC}

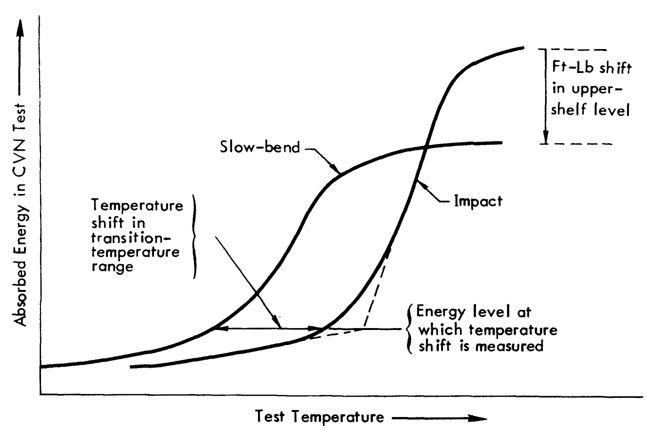

Fig. 7. Schematic representation of shift in CVN transition temperature and upper-shelf level due to loading rate

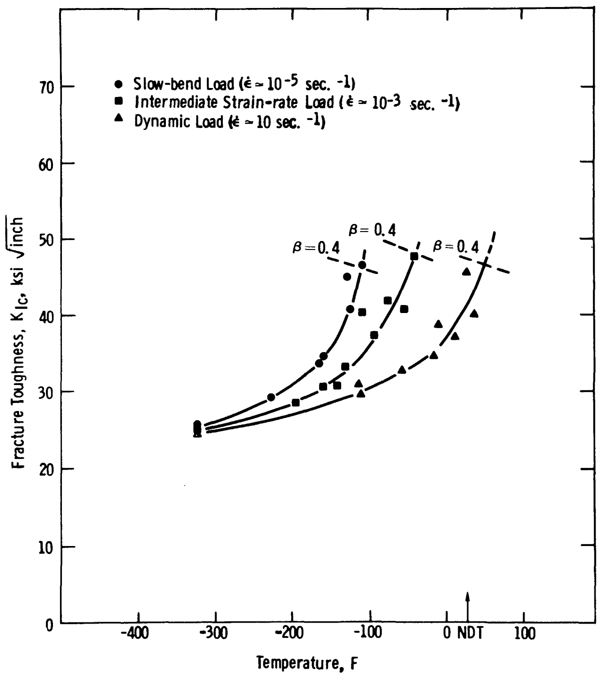

Fig. 8. Effect of temperature and loading rate on fracture toughness of A36 steel

tested at a constant loading rate undergoes a significant increase with increasing temperature. Thus, the general effect of a slow loading rate, compared with impact loading rates, is to shift the fracture-toughness curve to lower temperatures, regardless of the test specimen used. Examples of this shift in behavior with loading rate are presented in Figs. 8 and 9 for an A36 structural steel and an A572 Grade 50 structural steel, respectively.

The magnitude of the temperature shift between slow-bend loading and very rapid dynamic loading in steels of

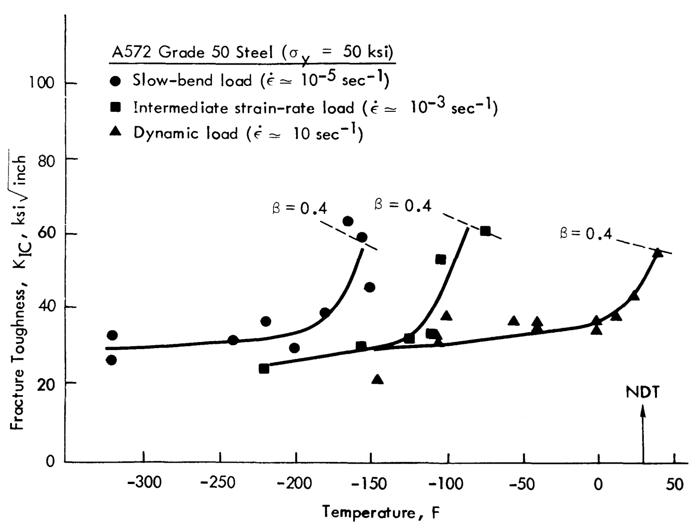

Fig. 9. Effect of temperature and loading rate on fracture toughness of A572 Grade 50 steel (σ_{ys} = 50 ksi)

various yield strengths has been related to the room-temperature yield strength of the steel and can be approximated by the following equations:

$$T_{shift} = 215 - 1.5\,\sigma_{ys} \quad (\text{for } 36 \text{ ksi} < \sigma_{ys} < 140 \text{ ksi})$$
$$T_{shift} = 0.0 \quad (\text{for } \sigma_{ys} > 140 \text{ ksi})$$

where

T_{shift} = absolute magnitude of the shift in the transition temperature between slow-bend loading and rapid dynamic loading, degrees F

σ_{ys} = room-temperature yield strength, ksi

Because of this shift, increasing the loading rate can decrease the fracture-toughness value at a particular temperature for steels having yield strengths less than 140 ksi. The change in fracture toughness values for loading rates varying from slow-bend to dynamic rates is particularly important for those structural applications that are loaded slowly, such as bridges.

As a specific example of the use of the loading-rate shift in the development of fracture criteria, assume that a structure is loaded at a slow-loading rate of 10^{-5} in./in./sec and that the fracture toughness of the material is as shown in Fig. 9. If stress-flaw size calculations show that a K_{Ic} value of about 60 ksi$\sqrt{\text{in.}}$ would insure satisfactory structural performance, the results presented in Fig. 9 show that this behavior can be obtained at about +40°F dynamically ($\dot{\epsilon}$ = 10 in./in./sec), at about −90°F at an intermediate loading rate, and at about −150°F for a slow-loading rate.

Since it is usually much easier and less expensive to conduct impact (dynamic) tests than intermediate-loading rate tests, criteria can be established on the basis of one loading rate, and the results "shifted" on the basis of a laboratory test conducted at a different loading rate. The recently developed American Association of State Highway and Transportation Officials (AASHTO) material toughness requirements were based on this reasoning.[21] It should be emphasized that this criterion can only be used with those materials that exhibit a shift in transition behavior with changes in loading rate. The magnitude of this shift can be considerable and helps to explain why many structures operate successfully at service temperatures well below their "dynamic" transition temperature.

Qualitatively, the effect of increasing specimen or plate thickness is to promote a more severe state-of-stress, namely, plane strain. A triaxial state-of-stress occurs at the tip of a sharp discontinuity in a thick plate and this reduces the apparent ductility of the material to a lower-bound value. Conversely, the apparent fracture toughness of materials can increase with decreasing plate thickness, as a result of the relaxation of the lateral constraint in the vicinity of the notch tip. This apparent increase in toughness is controlled solely by the thickness of the plate, even though the inherent metallurgical properties of the material remain unchanged. Thus, the minimum toughness of a particular material occurs at specimen thicknesses large enough so that the state-of-stress is plane strain.

FRACTURE CONTROL

The objective in structural design of large complex structures such as bridges, ships, pressure vessels, aircraft, etc., is to optimize the desired performance requirements relative to cost considerations (i.e., the overall cost of materials, design, fabrication, and operation), so that the probability of failure (and its economic consequence) is low. To achieve these objectives, engineers make predictions of service loads and conditions, calculate stresses in various structural members resulting from these loads and service conditions, and compare these stresses with the critical stresses in the particular modes that may lead to failure of the structure. Various criteria are then selected so that failure does not occur by any of the pertinent failure modes.

Possible failure or limit modes usually considered are:

1. General yielding or excessive plastic deformation (straining)
2. Buckling or general instability, either elastic or plastic
3. Sub-critical crack growth (through fatigue, stress corrosion or corrosion fatigue), leading to loss of section or unstable crack growth
4. Fracture, either ductile or brittle, leading to either partial or complete failure of a member

Although other failure modes exist, such as general corrosion or creep, the above mentioned failure modes are the ones that usually receive the greatest attention. Furthermore, of these four failure or limit modes, structural engineers usually concentrate on only the first two and assume that proper selection of materials will prevent the other two failure modes from occurring. This reasoning is not always true and has led to several large structural failures. In a complete structural design, all possible failure modes should be considered.

In the case of brittle fracture or fatigue, many of the fracture-control guidelines that have been followed to minimize the possibility of brittle fractures in structures are familiar to structural engineers. These guidelines include the use of structural materials with good notch toughness, elimination or minimization of stress raisers, control of welding procedures, proper inspection, etc. When these general guidelines are integrated into specific requirements for a particular structure, they become part of a fracture-control plan. A fracture-control plan is therefore a specific set of recommendations developed for a particular structure and should not be indiscriminately applied to other structures.

The four basic elements of a fracture-control plan are:

1. *Identification* of the factors that may contribute to the brittle fracture of a structural member or to the failure of an entire structure; includes description of service conditions, loadings, and/or deformations.
2. *Establishment* of the relative contribution of each of these factors to a possible brittle fracture in a member or to the failure of the structure.
3. *Determination* of the relative efficiency and trade-offs of various design methods to minimize the possibility of brittle fracture in a member or failure of the structural system.
4. *Recommendation* of specific design considerations to ensure the safety and reliability of the structure against brittle fracture. This would include recommendations for desired levels of material performance, as well as material selection, design stress levels, weld performance, design of details, fabrication, inspection, and maintenance.

For those cases where crack growth is a possibility, the total useful design life of a structural component can be estimated from the time necessary to initiate a crack plus the time to propagate the crack from sub-critical dimensions to the critical size. The life of the component can be prolonged by extending the crack-initiation life and/or the sub-critical-crack-propagation life. Consequently, the crack-initiation, sub-critical-crack-propagation, and unstable-crack-propagation characteristics of structural materials, as well as their fracture behavior, are primary considerations in the formulation of fracture-control guidelines for structures. Unstable crack propagation is the final stage in the useful life of a structural component subject to failure by the fracture mode. This stage is governed by the material toughness, the crack size, and the stress level. Consequently unstable crack propagation cannot be attributed only to low material toughness, or only to high stress levels, or only to poor fabrication, but rather to particular combinations of all the above factors. However, if any of these factors is significantly different than that which is normally found in a particular type of structure, experience has shown that for most structures the possibility of failure is generally increased.

Figure 10 is a schematic showing the three stages of total life behavior (crack-initiation, sub-critical-crack-propagation by fatigue, and unstable-crack-propagation, either

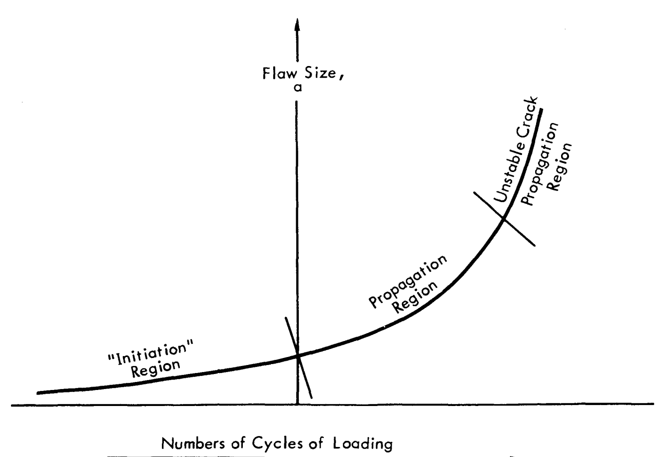

Fig. 10. Schematic showing three regions in the total life of a structure subjected to fatigue loading

by rapid fatigue crack growth, ductile tearing, or fracture). The question of when does a crack "initiate" to become a "propagating" crack is somewhat philosophical and depends on the level of observation of a crack, i.e., crystal imperfection, dislocation, microcrack, macrocrack, etc. In an engineering sense, the initiation stage is that region in which a very small initial discontinuity or crack grows to become a measurable propagating crack in fatigue. The sub-critical-crack-growth stage is that region in which a propagating fatigue crack follows one of the existing crack-growth laws,[7] e.g., $da/dN = A(\Delta K)^m$. The unstable crack-growth stage is that region in which either fatigue crack growth is *very* rapid, or a brittle fracture occurs, or ductile tearing occurs. All three situations in the unstable crack-growth stage result in loss of section and failure occurs very quickly, although failure by ductile tearing is usually preceded by large deformations.

Figure 11 illustrates the effect of tensile stress level, flaw size, and material toughness (the three primary factors that control susceptibility to brittle fracture) on the life of a structure subject to fatigue loading. Note that these factors are related to the three levels of performance, i.e., plane strain, elastic-plastic behavior, and plastic behavior, discussed earlier (see Fig. 5). The following observations may be made concerning the effectiveness of these control factors in improving service life:

Reduce Tensile Design Stress—Large effect on life (Region I) because the rate of fatigue crack growth is decreased significantly as the applied stress range is decreased (σ_1 curve compared with σ_2 curve). Design stress range ($\sigma_{max} - \sigma_{min}$) is the primary factor to control.

Reduce Initial Flaw Size—Large effect on life (Region II) because the rate of fatigue crack growth for *small* flaws is very low. Quality of fabrication and inspection is the primary factor to control.

Increase Material Toughness—

a) Large effect on life in moving from plane strain behavior to elastic-plastic behavior (Region III). The AASHTO Material Toughness Requirements for bridge steels insure this level of performance under intermediate rates of loading.

b) Small effect on life in moving from elastic-plastic behavior to plastic behavior (Region IV), because the rate of fatigue crack growth becomes so large that even if the critical crack size (a_{cr}) is increased significantly, the effect on the remaining fatigue life is small. Failure mode may change, however.

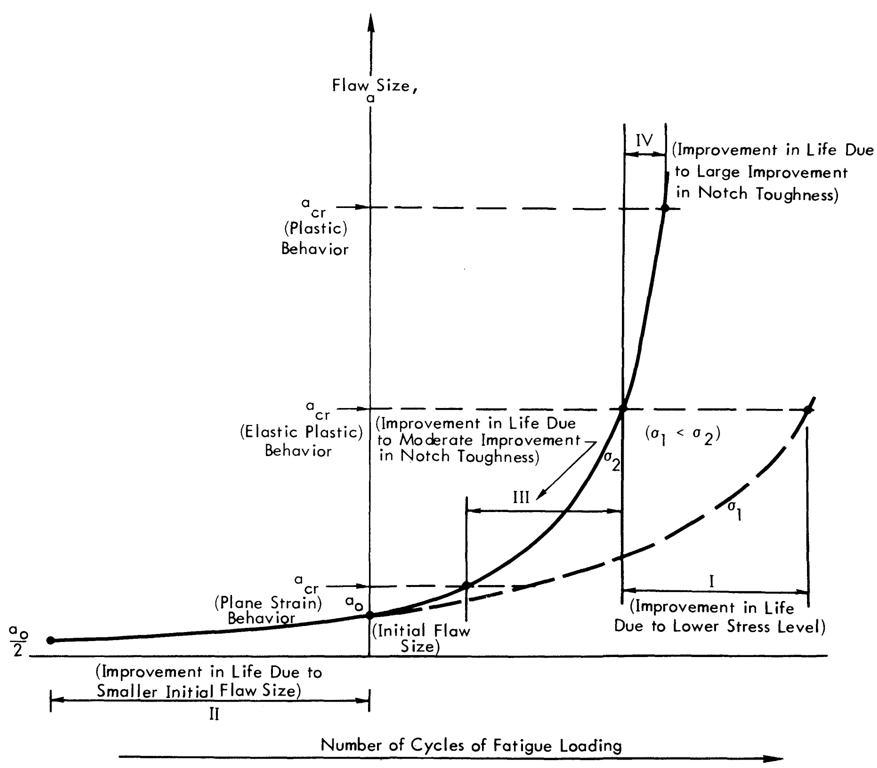

Fig. 11. Schematic showing effect of notch toughness, stress range, and flaw size on improvement of life of a structure subjected to fatigue loading

For most structural applications subject to fatigue, some moderate level of elastic-plastic behavior at service temperature and loading rate constitutes a satisfactory performance criterion. However, unusually severe service requirements may require special material toughness standards. In extreme cases, structural material is required whose notch toughness is such that the material does not fail by brittle fracture even under the most severe operating conditions to which the structure may be subjected. The use of HY-80 or HY-130 steels for submarine hull structures is an example of this method. However, as shown in Fig. 11, this method is not very effective in increasing the life of structures subjected to fatigue loading, such as bridges. However, the increased notch toughness certainly is a desirable property and does result in the change of failure mode from brittle to ductile fracture.

Although the above three methods are the basic design approaches to the control of brittle fracture in most structural applications, there are other design methods which can minimize susceptibility to, or the consequences of, brittle fracture (should the phenomenon occur). These other design methods include:

A) Provide multiple-load paths or structural redundancy, so that a single fracture cannot lead to complete failure of the structure. From a fracture behavior viewpoint, multiple-load path structures are different from redundant structures. A redundant structure is one in which the laws of statics are insufficient to solve for the loads and stresses, and thus the structure is indeterminant from an analysis viewpoint. If a single member fractures, one degree of redundancy may be removed (e.g., an effective hinge may be formed), but the structure is still stable.

A *multiple-load-path* structure is defined by the particular geometry of the members used to make up the structure. For example a simply-supported single-span bridge structure is determinant (non-redundant) because the reactions can be determined by the laws of statics. If the geometry of this single-span determinant structure is a single wide-flange shape such that failure of the single tension flange leads to collapse of the bridge, then the structure is also a *single-load-path* structure. However, if the geometry consists of eight independent wide-flange shapes with a concrete deck, then the structure is a multiple-load-path structure and is much more resistant to complete fracture than the single member structure. Lateral bracing of girders and trusses also provide multiple-load paths in the event of failure of a primary structural member. Another example is a truss member composed of multiple shapes (e.g., 4 to 10 eye-bar members parallel to each other), as opposed to one structural shape (e.g., a wide-flange shape in tension). The former is a multiple-load-path member, while the latter is a single-load-path member.

The distinguishing feature is whether or not, in the event of fracture of a primary structural member, the design load can be transferred to other members, whether they are initially classified as primary or secondary. If so, the structure is multiple-load-path; if not, it is single-load-path. In this sense, multiple-load-path structures are usually more resistant to failure than single-load-path structures. For example, if a single member fails in a single-load-path structure, the entire structure may collapse, as occurred with the Silver Bridge at Point Pleasant, W. Va.[14] Conversely, if a single member fails in a multiple-load-path structure, the entire structure probably will not collapse. This type of behavior was demonstrated in the failure of the Kings Bridge in Australia.[12] At the instant of failure, the failed span in the Kings Bridge contained three cracked girders. One girder had cracked while still in the fabrication shop. A second failed during the first winter the bridge was opened to traffic, a full 12 months before the failure of the bridge. Failure of a third girder led to final failure, although architectural concrete sidewalls (which added to the multiple-load-paths of the overall structure) prevented complete collapse. Similar examples of the importance of multiple-load-paths can be cited.

Therefore, lower notch toughness can be used in members of multiple-load-path structures than in members of single-load-path structures if a constant factor of safety is to be provided for the structure. Moreover, fatigue-crack propagation in multiple-load-path or redundant structures occurs essentially under constant maximum deflection, which corresponds to a decreasing stress-field intensity.

Thus, cracks propagating in multiple-load-path or redundant structures may eventually arrest and, although individual structural components will have to be replaced or repaired, complete failure of the structure is not expected to occur as long as sufficient redistribution of load can occur.

B) Provide crack arresters so that, in the event that a crack should initiate, it will be arrested before catastrophic failure occurs. Crack arresters or a fail-safe philosophy (i.e., in the event of "failure" of a member, the structure is still "safe") have been used extensively in the aircraft industry, as well as in the shipbuilding industry.

C) Insuring that the loading rate is slow is an effective method of fracture control. Many structures are loaded at slow to intermediate loading rates, where their notch toughness is quite satisfactory on the basis of the loading-rate shift. This leads to an understanding of why there are so few brittle fractures in older structures, even though the notch toughness of the steels in these structures would be considered to be very low on the basis of impact loading-rate tests. Thus, if the structure can be designed such that it is loaded *slowly,* so that the controlling toughness parameter is K_{Ic} rather than K_{Id}, the possibility of fracture is reduced considerably.

AASHTO MATERIAL TOUGHNESS REQUIREMENTS

The recently developed AASHTO material toughness requirements for bridge steels are based on the observation that the maximum loading rates in bridges[22,23,24] are closer to slow-bend loading rates than to impact loading rates.

In fact, the loading times in bridges are greater than 1 second, which corresponds to a strain rate of less than 10^{-3} sec^{-1} on the elastic-plastic boundary in the vicinity of a crack tip. Thus, a strain rate of 10^{-3} sec^{-1} (intermediate loading rate) is used as a conservative measure of the maximum strain rate for bridges.

The AASHTO toughness requirements for bridge steels, which are based on testing at temperatures above minimum service temperatures and at impact CVN strain rates, are such that the transition from plane strain to elastic-plastic behavior under intermediate loading rates will occur below the minimum service temperature.

The K_{Ic} data obtained by testing A36 and A572 Grade 50 steels at a strain rate of 10^{-3} sec^{-1} (intermediate loading rate) indicate that fracture does not occur under plane-strain conditions when the test temperature is greater than about $-80°F$ (see Figs. 8 and 9).[25,26] Because K_{Ic} tests are expensive and difficult to conduct, and because of the apparent correspondence between K_{Ic} test results and CVN test results,[27,28,29] the CVN test was selected as the reference test for the AASHTO fracture-toughness requirements. The fracture-toughness transition temperature is the temperature at which the fracture toughness of the steel begins to increase rapidly from plane-strain behavior to fully ductile behavior. The CVN test results, Fig. 12, show that this transition behavior at 15 ft-lbs under intermediate rates of loading at $-80°F$ should ensure a non-plane-strain fracture behavior at a minimum operating temperature of $-30°F$ for a 50 ksi yield strength steel.

Although the intermediate-loading-rate test is the test that more properly describes the expected service performance of bridge steels, the standard impact-loading-rate CVN test is much easier to conduct and analyze and is less expensive than an intermediate-loading-rate CVN test. Consequently, the difference in fracture-toughness behavior at the two strain rates was used to develop the toughness values in terms of the impact test rather than an intermediate-loading-rate test. The temperature shift between the CVN (and K_{Ic}) curves of a 50 ksi yield strength steel tested at a strain rate of 10^{-3} sec^{-1} and at an impact strain rate of 10 sec^{-1} (10,000 times greater) was on the order of 120°F (see Fig. 9). Consequently, the requirement of a 15 ft-lb CVN impact value at $+40°F$ corresponds to a 15 ft-lb CVN value under an intermediate strain-rate at $-80°F$, which in turn corresponds to a non-plane-strain fracture behavior at an assumed minimum operating temperature of $-30°F$. Thus, a CVN fracture-toughness requirement of 15 ft-lbs at $+40°F$ was imposed on all primary member components in tension and of 50 ksi yield strength steels for bridge applications. This same requirement was also imposed on all primary member components in tension for bridge steels having yield strengths less than 50 ksi, which is a conservative requirement for these steels.

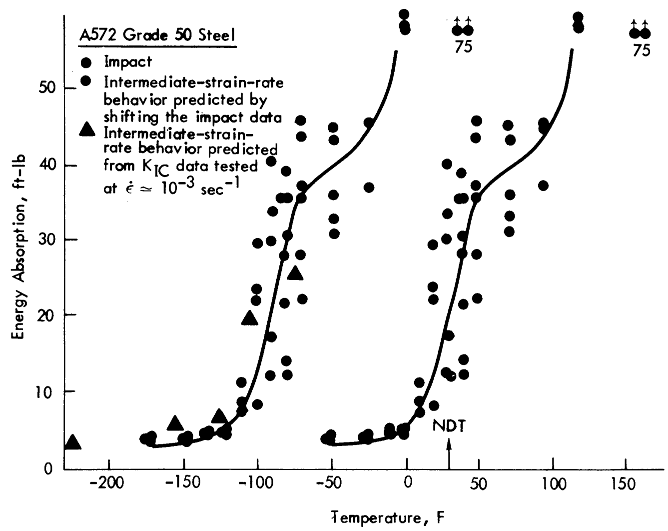

Fig. 12. Charpy V-notch energy absorption behavior for impact loading and intermediate rate loading of standard CVN test specimens

The 15-ft-lb CVN impact toughness requirement at $+40°F$ for steels of 50 ksi yield strength or less was based on a $-30°F$ minimum operating temperature. The preceding procedure can be used to develop toughness requirements for any minimum operating temperature. The resulting toughness requirement for 50 ksi yield strength steels is 15 ft-lbs in an impact test at a test temperature that is 70°F higher than the specified minimum operating temperature. Thus, the CVN test temperatures and the minimum operating temperatures are linearly related. To minimize the proliferation of a variety of testing temperatures, and the resulting problems in the design and fabrication of steel bridges, the variable testing temperatures were comprehended by establishing three zones of service temperatures and providing temperatures and CVN impact values for each zone. The three zones of service temperatures and the corresponding test temperatures and minimum toughness values for bridge steels are presented in Table 1.

The general relationships between service temperatures and test temperatures for A36 steel satisfying the requirements of each of the three service-temperature zones are shown in Fig. 13. These results show that, because of the loading-rate shift, CVN-toughness levels greater than 15 ft-lbs are expected at intermediate loading rates approximately 70°F below the impact testing temperature. In terms of the NDT temperature measured using drop weight test specimens subjected to impact loading, the minimum service temperature is approximately 70°F below NDT.

The specifications of the American Society for Testing and Materials (ASTM) for A572 Grade 50 and A588 steels require a minimum yield strength value of 50 ksi. Consequently, these steels as actually produced may have

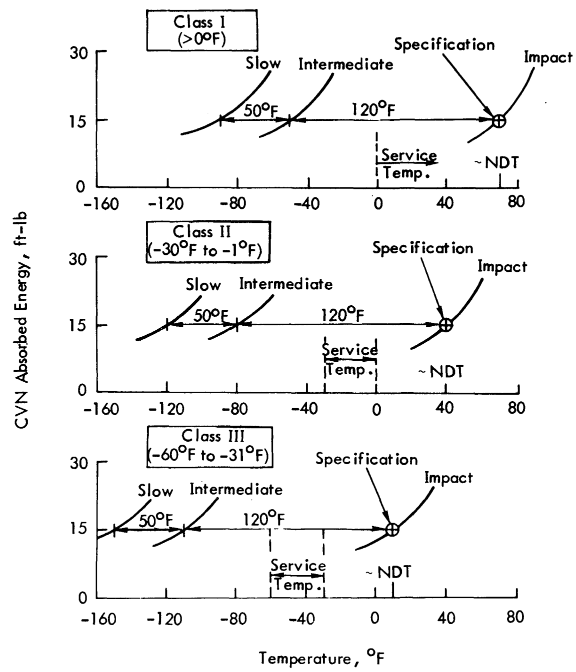

Fig. 13. AASHTO material toughness specifications for A36 steel

yield strengths that are higher than 50 ksi. The data in Fig. 14 show that the magnitude of the temperature shift between static and impact loading rates decreased with increased yield strength. The magnitude of the decrease in the temperature shift is about 15°F for every 10 ksi increase in yield strength. To ensure the same fracture behavior for A572 Grade 50 and A588 steels having yield strengths significantly greater than 50 ksi, the CVN test temperature was decreased incrementally as the yield strength increased. The CVN requirements of 15 ft-lbs at +70°, +40° and +10°F for Zones 1, 2 and 3, respectively, were restricted to A572 Grade 50 and A588 steels having yield points between 50 and 65 ksi, inclusive. When the yield strength of these steels exceeds 65 ksi, the temperature for the CVN value for acceptability was reduced by 15°F for each increment of 10 ksi above 65 ksi.

The above philosophy, which is based on fracture-mechanics concepts, was used to develop toughness requirements for bridge steels of 100 ksi yield strength (ASTM A514 and A517). These steels show a temperature shift of 60°F between static and impact loading rates (Figs. 14 and 15). Moreover, increasing the design stress (which generally requires a higher yield strength steel) results in more stored elastic energy in a structure. Thus, the fracture toughness of the steel should also be increased to ensure the same degree of safety against fracture as the structure with the lower design stress. The resulting fracture-toughness requirements for high-strength bridge steels also are presented in Table 1.

In summary, the basis for the AASHTO material-toughness specification, which is in the recently developed ASTM A709 Standard Specification for Structural Steel for Bridges, is fracture mechanics. However, the desired level of performance is outside the range of linear-elastic fracture-mechanics behavior at the service temperatures and loading rates for bridges. Thus, K_{Ic} values cannot be measured directly by existing fracture-mechanics tests, and correlations between K_{Ic} and CVN test results were used to establish the material-toughness requirements shown in Table 1. These toughness requirements depend on the

Table 1. AASHTO Fracture-Toughness Specifications for Bridge Steels*

ASTM Designation	Thickness	Energy Absorbed (ft-lbs) Zone 1**	Zone 2**	Zone 3**
A36		15 @ 70°F	15 @ 40°F	15 @ 10°F
A572†	Up to 4 in. mechanically fastened	15 @ 70°F	15 @ 40°F	15 @ 10°F
	Up to 2 in. welded	15 @ 70°F	15 @ 40°F	15 @ 10°F
A440		15 @ 70°F	15 @ 40°F	15 @ 10°F
A441		15 @ 70°F	15 @ 40°F	15 @ 10°F
A242		15 @ 70°F	15 @ 40°F	15 @ 10°F
A588†	Up to 4 in. mechanically fastened	15 @ 70°F	15 @ 40°F	15 @ 10°F
	Up to 2 in. welded	15 @ 70°F	15 @ 40°F	15 @ 10°F
	Over 2 in. welded	20 @ 70°F	20 @ 40°F	20 @ 10°F
A514	Up to 4 in. mechanically fastened	25 @ 30°F	25 @ 0°F	25 @ −30°F
	Up to 2-1/2 in. welded	25 @ 30°F	25 @ 0°F	25 @ −30°F
	Over 2-1/2 in. to 4 in. welded	35 @ 30°F	35 @ 0°F	35 @ −30°F

*See also ASTM A709-75 Standard Specification for Structural Steel for Bridges.

**Zone 1: Minimum service temperature 0°F and above. Zone 2: Minimum service temperature from −1 to −30°F. Zone 3: Minimum service temperature from −31 to −60°F.

†If the yield point of the material exceeds 65 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 65 ksi.

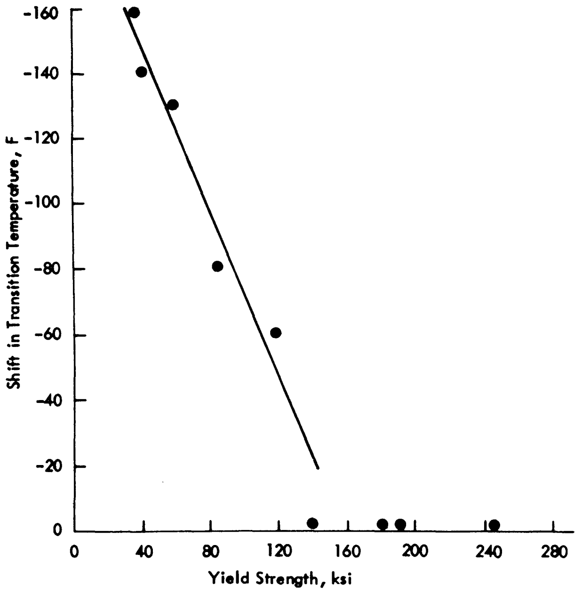

Fig. 14. Effect of yield strength on shift in transition temperature between impact and slow-bend CVN test results

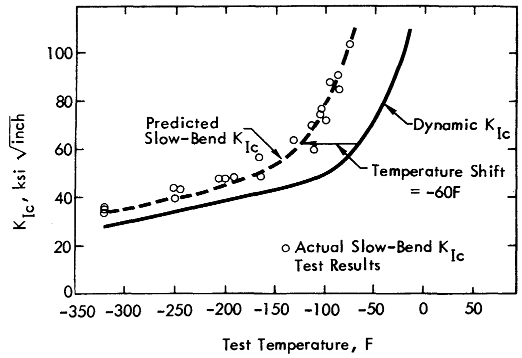

Fig. 15. Effect of temperature and loading rate on fracture toughness of A517 steel

existence of a strain-rate shift[7] to obtain the desired toughness levels at a service loading rate (intermediate) that is 70°F below the actual specification loading rate (impact). In addition, these toughness requirements are also based on the assumption that the design, fabrication, erection, and inspection procedures for bridges are satisfactory, although an exact definition of satisfactory is difficult to obtain. If this is the case, as it is for most bridges, then the AASHTO material toughness requirements presented in Table 1 generally are quite adequate. If this is not the case, then even making the material toughness requirements more stringent may not achieve the desired result of a fracture-resistant structure because of fatigue, as was illustrated in Fig. 11.

FRACTURE TESTS OF WELDED BEAMS

To verify the adequacy of the toughness requirements presented in Table 1, beam specimens of A36 steel and A572 Grade 50 steel were used to study the fracture behavior of simulated bridge members under extreme service conditions.[30] The specimens were designed to include two common structural details that adversely affect fatigue and fracture strength: (1) a cover plate end and (2) a transverse stiffener. Fatigue tests have shown that the cover plate end is one of the most severe common structural details with respect to fatigue[31] and probably also fracture.[23]

The beam specimens of A36 and A572 Grade 50 steels were subjected to a cyclic stress range of 21 ksi for 100,000 cycles or more. This loading corresponds to the maximum allowable fatigue loading specified by AASHTO for cover plate ends in either steel, but is much more severe than the cyclic loadings measured in actual bridges.[22] After the specimens had been subjected to cyclic loading, some were cooled to −30°F and then loaded under an impulse load to a maximum bending stress of 20 ksi for A36 steel and 27 ksi for A572 Grade 50 steel. The specimens were then subjected to impulses of either 36 ksi or 50 ksi, the specified minimum yield strengths of the two steels, respectively. The total time for the 20 ksi impulse was approximately 1 second — about the same time as for the impulses observed in field measurements of truck loadings on short span bridges.[22] Since the stress levels in the field measurements were generally below 6 ksi, the strain rates in the test impulse were well above the strain rates observed in the field.

In summary, the test temperature was below the minimum temperature expected to occur in actual highway bridges in the continental United States, the loading rates were well above the loading rates observed in field measurements, and the beams were subjected to the highest allowable cyclic stresses and the maximum number of cycles specified by AASHTO for cover plate ends at that stress level. Thus, the fracture stress was determined for non-redundant (single-load-path) bridge members containing the most severe common structural detail and for the most severe combination of temperature, strain rate, and prior fatigue loading that could reasonably be expected to occur in actual highway bridges in the continental United States, and they still did not fracture until stress levels about 70 percent higher than the maximum design stress permitted by AASHTO were reached.

Even though the principles involved in the development of the AASHTO material toughness requirements for bridge steels can be used to develop toughness requirements for other types of structures, these specific AASHTO toughness requirements are not recommended for direct use in other types of structures. As in the development of fracture criteria for any type of structure, the particular criterion is dependent on the overall service behavior and experience, loadings, strain rate, design, and details, redundancy, consequences of failure, etc., and not just the fracture characteristics of the materials.

ACKNOWLEDGMENTS

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In addition, much of the information also has been published in Refs. 21, 32, and 33, and the assistance of W. J. Hall and J. M. Barsom is gratefully acknowledged.

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