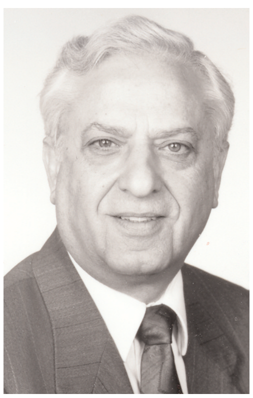
2003 T.R. HIGGINS LECTURE

STRUCTURAL FAILURES: EFFECTS OF JOINT DESIGN

The T.R. Higgins Award Jury selected John M. Barsom as the winner of the 2003 award. The paper upon which the selection was made is "Development of Fracture Toughness Requirements for Weld Metals in Seismic Applications" from the *Journal of Materials in Civil Engineering*, Vol. 14, No. 1, February 1, 2002, pp. 44–49. ©ASCE 2002. This paper is reprinted in these proceedings with permission from the publisher, ASCE (www.pubs.asce.org).

In order to expand on that paper and to provide updated information in his presentation, a summary is provided in these proceedings for the presentation that will be given as the T.R. Higgins Lecture, "Structural Failures: Effects of Joint Design."



John M. Barsom

John M. Barsom is president of Barsom Consulting, Ltd., Pittsburgh, PA, a forensic engineering company. He is a member of several AISC committees including the Committee on Specifications and is the recipient of the AISC Lifetime Achievement Award.

Barsom is a Fellow of the American Society of Metals (ASM International), the American Society of Mechanical Engineers (ASME), the American Welding Society (AWS) and the American Society for Testing and Materials (ASTM International). Barsom is the recipient of the Edgar C. Bain Award in metallurgy from the Pittsburgh chapter of ASM International and of the Fracture Mechanics Medal from ASTM Committee E08 on Fracture and Fatigue. He received the medal for having exerted a profound and positive effect on the development of the scientific development of fracture mechanics and "in recognition of his outstanding contribution to application of fracture mechanics and its usefulness to the practicing engineer."

Most of Barsom's distinguished career was spent at U.S. Steel in Pittsburgh, where he was named Research Fellow and Director of the Material Technology Division. He is a specialist in properties and behavior of steels and weld metals, fracture mechanics, failure analysis of structures and equipment including metallographic and fractographic investigations, accident reconstruction, integrity and life extension of structures and equipment, and behavior of fabricated components under slow and rapid loading conditions. Barsom is also an adjunct professor in the Civil and Environmental Engineering Department at the University of Pittsburgh.

Barsom received BS in Physics, an MS in Mathematics, and a Ph.D. in Mechanical Engineering from the University of Pittsburgh.

T.R. HIGGINS LECTURE SUMMARY

Fracture behavior of structural components is governed by several factors including material properties, design, fabrication, inspection and usage. Failure of otherwise properly designed structures, frequently, is caused by the design configuration and geometry of the joints. Joint design can have a significant effect on the deformation and fracture toughness of steels and weld metals. The presentation describes the basic properties of steels and weld metals and the effects of joint design on these properties. Several examples of structural failures are presented to demonstrate the effects of joint design on the fracture behavior of structural components.

Development of Fracture Toughness Requirements for Weld Metals in Seismic Applications

John M. Barsom[1]

Abstract: Fracture of unreinforced welded moment frame connections subjected to simulated seismic loads was caused by the initiation of fatigue cracks and their propagation to critical size. The fatigue cracks initiated at the web-to-flange intersection at the weld access hole, the valleys on the flame-cut weld access hole surface, the weld toe, and weld imperfections. Final fracture occurred when the fatigue crack extended unstably either in the base metal or in the weld metal. Final fracture is determined by the size of a crack, the stresses and strains acting on the crack, and the fracture toughness of the material. This paper presents the methodology used to establish the necessary and sufficient fracture toughness requirement for weld metal used in seismic applications. The methodology was based on fracture mechanics principals and on empirical correlations. The proposed Charpy V-notch (CVN) toughness is 40 ft-lb at 70°F and 20 ft-lb at 0°F for components subjected to +50°F and higher. This CVN requirement should preclude weld metal toughness from being a contributing factor to the fracture of unreinforced moment frame connections. Further improvements in the fracture performance of the connections must be accomplished by changes in design, detailing, fabrication, and inspection.

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Introduction

Full-scale welded moment frame connections have been tested under simulated seismic loads developed by the SAC Steel Project (Goel et al. 1999; Fry et al. 2000; Ricles et al. 2000). These were conducted to determine if newly detailed unreinforced fully restrained connections can behave satisfactorily in future earthquakes. The base metal for beams and columns were produced to ASTM A572 Grade 50 specifications. The specimens were welded with either E70TG-K2 or E70T-6 electrodes. All tests were conducted at room temperature.

The simulated seismic loads induce high-strain low-cycle fatigue deformation in the welded joint. A failure analysis (Barsom and Pellegrino 2000) of the specimens tested at the Univ. of Michigan (Goel et al. 1999) and at Lehigh Univ. (Ricles et al. 2000) demonstrated that fatigue cracks initiated and propagated in all the tested connections.

One of the many SAC Steel Project objectives was to develop fracture toughness requirements for weld metals to be used in welded moment frame connections. The following sections describe the methodology used to develop the SAC-recommended Charpy V-notch (CVN) toughness requirements for weld metals in seismic applications.

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Development of Fracture-Toughness Requirements

The primary objective in the structural design of large complex structures such as bridges, ships, pressure vessels, aircraft, and buildings is to optimize the desired cost, performance, and safety requirements. This objective is achieved by considering the relationships among design, material properties, fabrication, inspection, operation, and maintenance, and the contribution of each of these factors to the performance of the structure. Several fracture control guidelines minimize the possibility of fracture in structures: proper design; the use of materials with adequate strength, ductility, and fracture toughness; elimination or minimization of stress-raisers; proper inspection; and the like. When these general guidelines are integrated into specific requirements for a particular structure, they become part of a fracture-control plan. Therefore, a fracture control plan is a specific set of recommendations developed for a particular structure and should not be applied indiscriminately to other structures.

The magnitude and fluctuation of the applied stresses, the geometry of the structural details, constraint, fabrication, and inspection affect material performance. For example, ductile materials may behave in a nonductile manner when the structural details are highly constrained and/or contain severe stress raisers such as notches, cracks, or fabrication defects. Fracture toughness is one of several properties that may affect the performances of the material and the structural connection. Fracture toughness of steels is a function of constraint, temperature, and loading rate; high constraints, low temperatures, and rapid loading rates decrease the fracture toughness value. Requiring high fracture toughness does not ensure adequate structural performance when the stresses and stress ranges are high or the structural details are highly constrained or contain severe geometric stress-raisers (e.g., notches, cracks, or fabrication defects). The safety and reliability of cost-effective structures and/or structural components depend on the contribution of, and trade-off between, many factors, including fracture toughness.

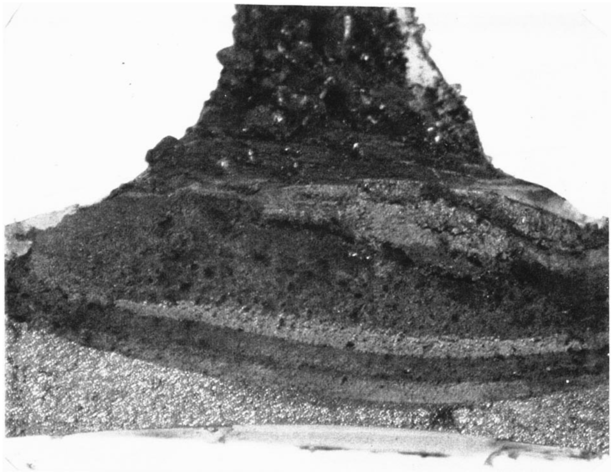

Fig. 1. Initiation and propagation of fatigue cracks in the Univ. of Michigan beam-to-column test welds

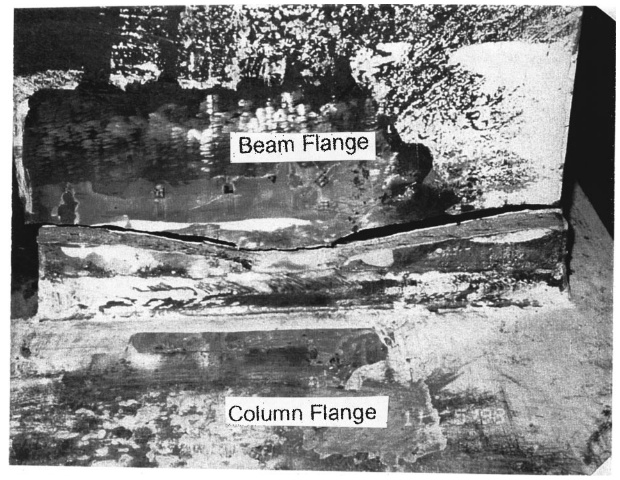

Fig. 2. Stable crack growth (center) prior to unstable fracture

This paper presents the development of CVN fracture toughness requirements for weld metals in seismic applications. These requirements were developed in the absence of knowledge of these factors: seismic demands (e.g., loads and deformations) for different building configurations and connection geometries; fracture-mechanics type fracture toughness (e.g., critical stress-intensity factors, K_{IC}, or crack-tip-opening displacements, δ_c) for the base metals, heat-affected zones, or weld metals; and fabrication and inspection requirements. At the time the fracture-toughness development was performed, only three room-temperature δ_C values were available for one (E70TG-K2) of several filler metals. Consequently, the development of fracture-toughness requirements within the context of a fracture-control plan was not possible. The proposed fracture-toughness requirements for weld metals in seismic applications were therefore based on the following:

1. fundamental fatigue-crack-propagation and elastic-plastic fracture-toughness behaviors of steels and weld metals;
2. the δ_C value above which negligible fatigue life remains under full-reversal seismic deformations, and thus the crack driving force, δ, should be kept below the δ_C value; and
3. the δ_C value was converted to an equivalent CVN value.

The methodology used to develop the CVN fracture-toughness requirements for weld metals in seismic applications is detailed in the following sections. Future technical developments and an improved understanding of the factors that are integral parts of a fracture control plan for buildings subjected to seismic loads and deformations may modify, augment, or replace the methodology and/or the proposed requirements.

Fatigue Crack Propagation Behavior

Failure analysis of unreinforced welded moment frame connections subjected to simulated seismic loads during testing at the Univ. of Michigan showed that fracture was caused by the initiation and propagation of fatigue cracks, as shown in Fig. 1 (Barsom and Pellegrino 2000).

The fatigue cracks initiated at the web-to-flange intersection at the weld access hole, the valleys of the flame cut weld access hole surface, the weld toe, and weld imperfections. The applied cyclic loads increased the size of the fatigue crack until it reached a critical dimension where unstable crack extension severed the beam flange (Fig. 2).

The fatigue cracks in all the tested specimens exhibited stable ductile tearing under the applied cyclic loads. Subsequent unstable crack extension was ductile in some specimens and brittle in others. Regardless of the mode of unstable crack extension, the critical crack size at fracture was large and the remaining fatigue life under the simulated seismic loads was negligible.

The fatigue-crack-propagation behavior for metals can be divided into three regions (Fig. 3) (Barsom and Rolfe 1999). The behavior in region I exhibits a ‘‘fatigue-threshold’’ cyclic stress-intensity-factor fluctuation, ΔK_{th}, below which cracks do not propagate under cyclic-stress fluctuations. Region II represents the fatigue-crack-propagation behavior above ΔK_{th}, which can be represented by

$$da/dN = A(\Delta K)^m \tag{1}$$

where a = crack length; N = number of cycles; ΔK = stress-intensity-factor fluctuation; and A and m = constants.

In region III, the fatigue-crack growth per cycle is higher than predicted for region II. Experimental data show that the rate of fatigue-crack growth increases and that, under zero-to-tension loading (that is, $\Delta K = K_{\max}$), this increase occurs at a constant value of crack-tip displacement, $\Delta\delta_T$, and at a corresponding stress-intensity-factor value ΔK_T, given by Eq. (2) (Barsom and Rolfe 1999):

$$\Delta\delta_T = (\Delta K_T)^2/E\sigma_{ys} = 1.6\times 10^{-3}\ \text{in.}\ (0.04\ \text{mm}) \tag{2}$$

where ΔK_T = stress-intensity-factor-range value corresponding to onset of acceleration in fatigue-crack-growth rates; E = Young's modulus; and σ_{ys} = yield strength (0.2% offset). (The available data indicate that the value of ΔK_T can be predicted more closely by using a flow stress, σ_f, rather than σ_{ys}, where σ_f is the average of the yield and tensile strengths.)

In the connections tested at the Univ. of Michigan, acceleration of fatigue-crack-growth rates, which determines the transition from region II to region III was caused by the superposition of a ductile tear mechanism onto the mechanism of cyclic subcritical crack extension, which leaves fatigue striations on the fracture surface. Ductile tear occurs when the strain at the tip of the crack reaches a critical value. Thus, the fatigue-rate transition from region II to region III depends on $K_{\max}$ and on the stress

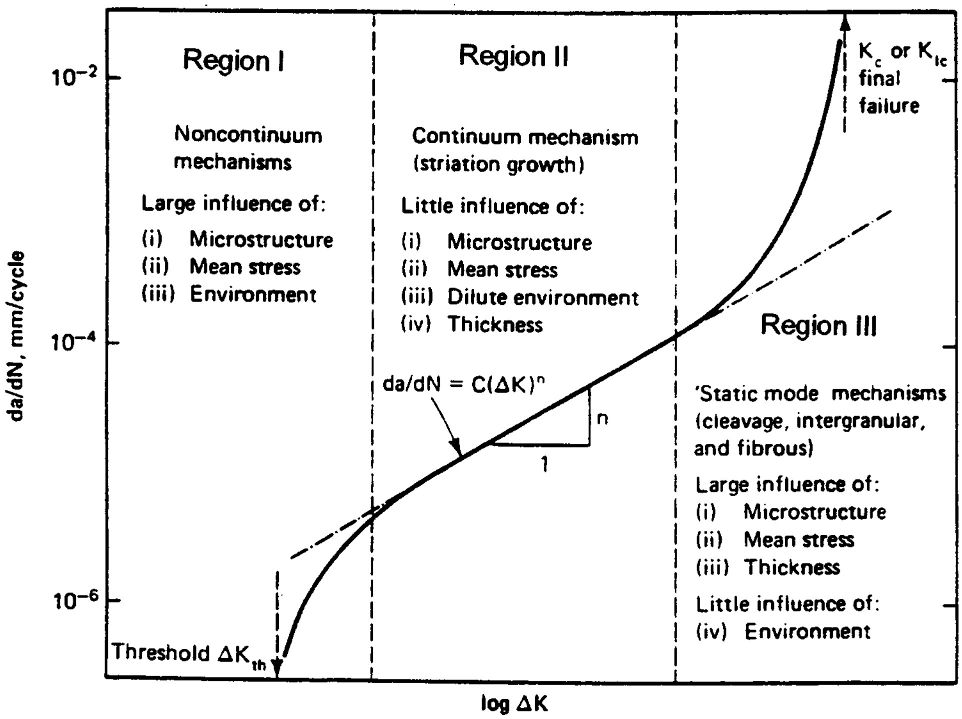

Fig. 3. Schematic representation of fatigue crack growth rate in steels

ratio, R. Most of the useful fatigue life is when the crack is in regions I and II. In region III, cracks extend by large increments with each load cycle.

Fatigue Critical Stress Intensity Factor

The first step in the development of fracture toughness for weld materials in seismic applications was to require the fracture toughness value to be higher than would be calculated from Eq. (2). In other words, the fracture toughness must be high enough to ensure that fatigue crack extension under seismic loads would take full advantage of the behavior in region II and would transition into the fast fatigue crack propagation region III without becoming an unstable fast-running crack.

The yield strength, σ_{ys}, and tensile strength, σ_u, of the E70TG-K2 weld metal used to fabricate the Univ. of Michigan moment frame connections were 76 and 90 ksi, respectively (Johnson 2000). For a stress ratio, R, equal to zero, the minimum fatigue-critical stress-intensity factor corresponding to the transition into region III, K_{C3}, is given by

$$K_{C3} = \sqrt{(E\sigma_{ys}\delta_T)} \quad (3)$$

where $K_{C3} = \Delta K_T = K_{max}$ for zero-to-tension loading, i.e., $R = 0$; E = Young's modulus $= 29\times 10^6$ psi; and $\delta_T = 1.6\times 10^{-3}$ in. therefore, $K_{C3} \cong 60$ ksi$\sqrt{\text{in}}$.

This K_{C3} value is conservative because it does not account for the elevation of the yield strength due to the triaxiality at the center one-third length of the beam-to-column weld or the minor effect of compressive stresses on fatigue crack propagation under the fully reversed simulated seismic loads. Therefore, the true fatigue-critical stress-intensity factor must be larger than 60 ksi$\sqrt{\text{in}}$.

Fracture Critical Stress Intensity Factor

Examination of the fatigue cracks that initiated from weld imperfections in the moment frame connections tested at the Univ. of Michigan (Barsom and Pellegrino 2000) suggested that an estimate of the critical crack size for the weld metal was either about 0.5 in. deep part-through crack or about a 1.5 in. through-thickness crack. These crack sizes in combination with assumed effective stresses were used to estimate the fracture-critical stress-intensity factor, K_c (i.e., fracture toughness) of the E70TG-K2 weld metal used to fabricate the Univ. of Michigan connections.

Because the moment frame weldments in the Univ. of Michigan tests were subjected to plastic deformation under the simulated seismic loads, the flow stress [Eq. (4)]

$$\sigma_{flow} = [\sigma_{yield} + \sigma_{tensile}]/2 = 83 \text{ ksi} \quad (4)$$

was used to calculate the fracture critical stress intensity factor, K_c, from the relationship [Eq. (5); Barsom and Rolfe (1999)]:

$$K_c = 1.12\sigma_{flow}\sqrt{\pi a_c} \quad (5)$$

for a part-through crack conservatively modeled as an edge crack (i.e., a part-through crack having infinite surface length) with $a_c = 0.5$ in., and

$$K_c = \sigma_{flow}\sqrt{\pi a_c} \quad (6)$$

for a through-thickness crack with $2a_c = 1.5$ in. Thus, the estimated fracture critical stress intensity factor, K_c, values for E70TG-K2 weld metal are 117 and 127 ksi$\sqrt{\text{in}}$., respectively.

At the time this methodology to estimate fracture toughness requirements for weld metal in seismic applications was being developed, only three room-temperature crack-tip-opening-displacement (CTOD) values were available. The three test specimens were from a single weldment made with E70TG-K2 filler metal. The weldment was part of the Univ. of Michigan full-size specimen test program and was fabricated in an identical manner as the full-size moment frame connection specimens. The CTOD tests were conducted at the Edison Welding Institute (Johnson 2000).

The three room temperature CTOD values for E70TG-K2 weld metal were 0.0019, 0.0043, and 0.0084 in. These values

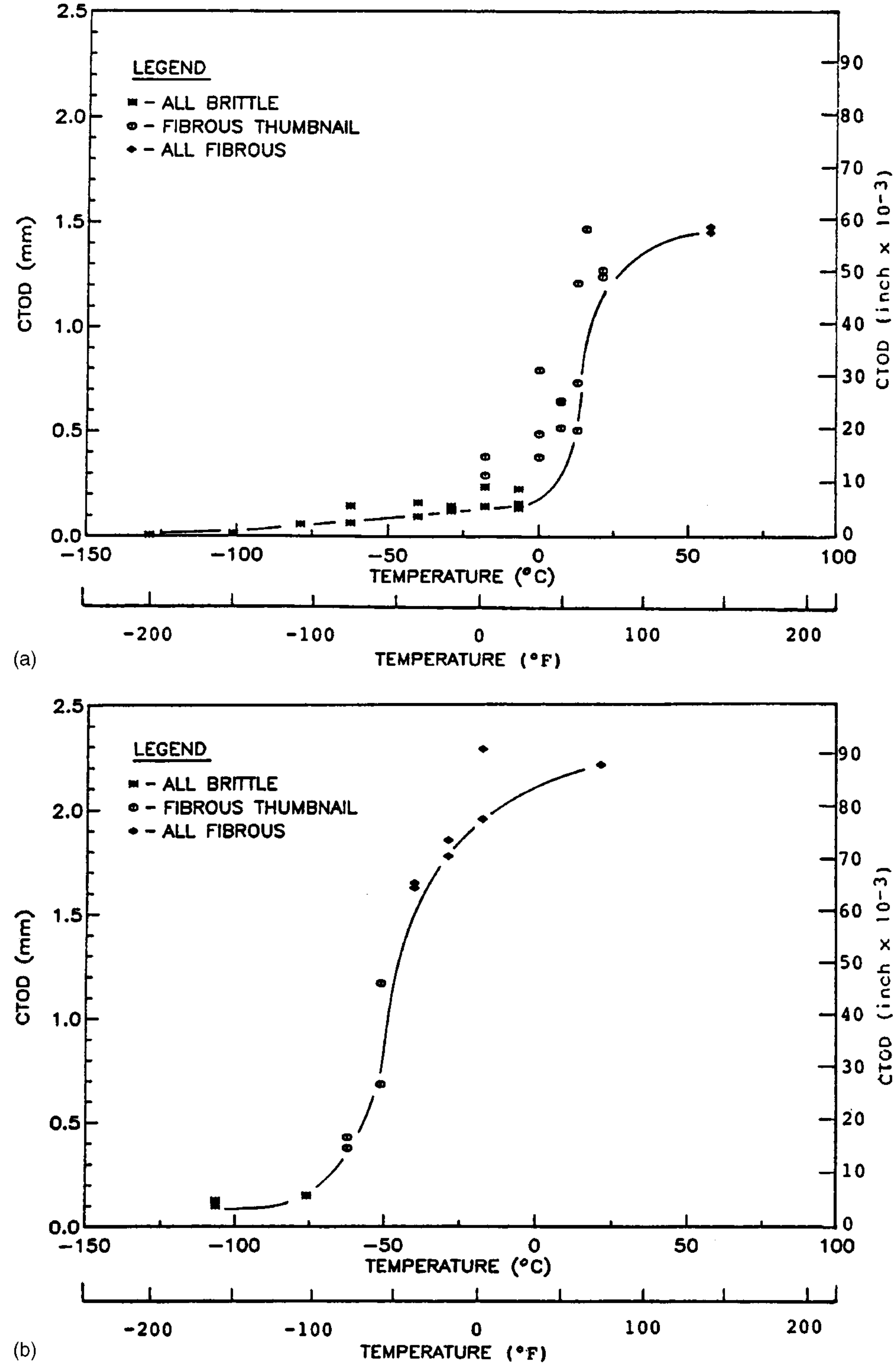

Fig. 4. Crack-tip-opening-displacement (CTOD) temperature transition curves for steels; (a) CTOD-temperature transition curve for A131 steel and (b) CTOD-temperature transition curve for A516 steel

were converted to fracture critical stress-intensity factors, K_c, by using the relationship [Eq. (7); Barsom and Rolfe (1999)]:

$$K_c = \sqrt{1.7\sigma_{\text{flow}} E \delta_c} \tag{7}$$

where $\sigma_{\text{flow}} = 83\times10^3$ psi; E = Young's modulus, psi; and δ_c = critical crack tip opening displacement, CTOD, in. [Note: Eqs. (2) and (7) are empirical correlations of K, σ, and δ for fatigue and fracture, respectively (Barsom and Rolfe 1999). Eq. (2) can be changed to include the 1.7 constant in Eq. (7). The value of $\delta_T = 1.6\times10^{-3}$ in., however, would have to be adjusted accordingly.]

The three CTOD values corresponded to K_c values of 88, 133, and 185 ksi$\sqrt{\text{in.}}$ Consequently, assuming that one or more of the specimens tested at the Univ. of Michigan contained weld metal having a $K_c = 88$ ksi$\sqrt{\text{in.}}$, one may conclude that this fracture toughness value resulted in a large ductile fatigue crack prior to

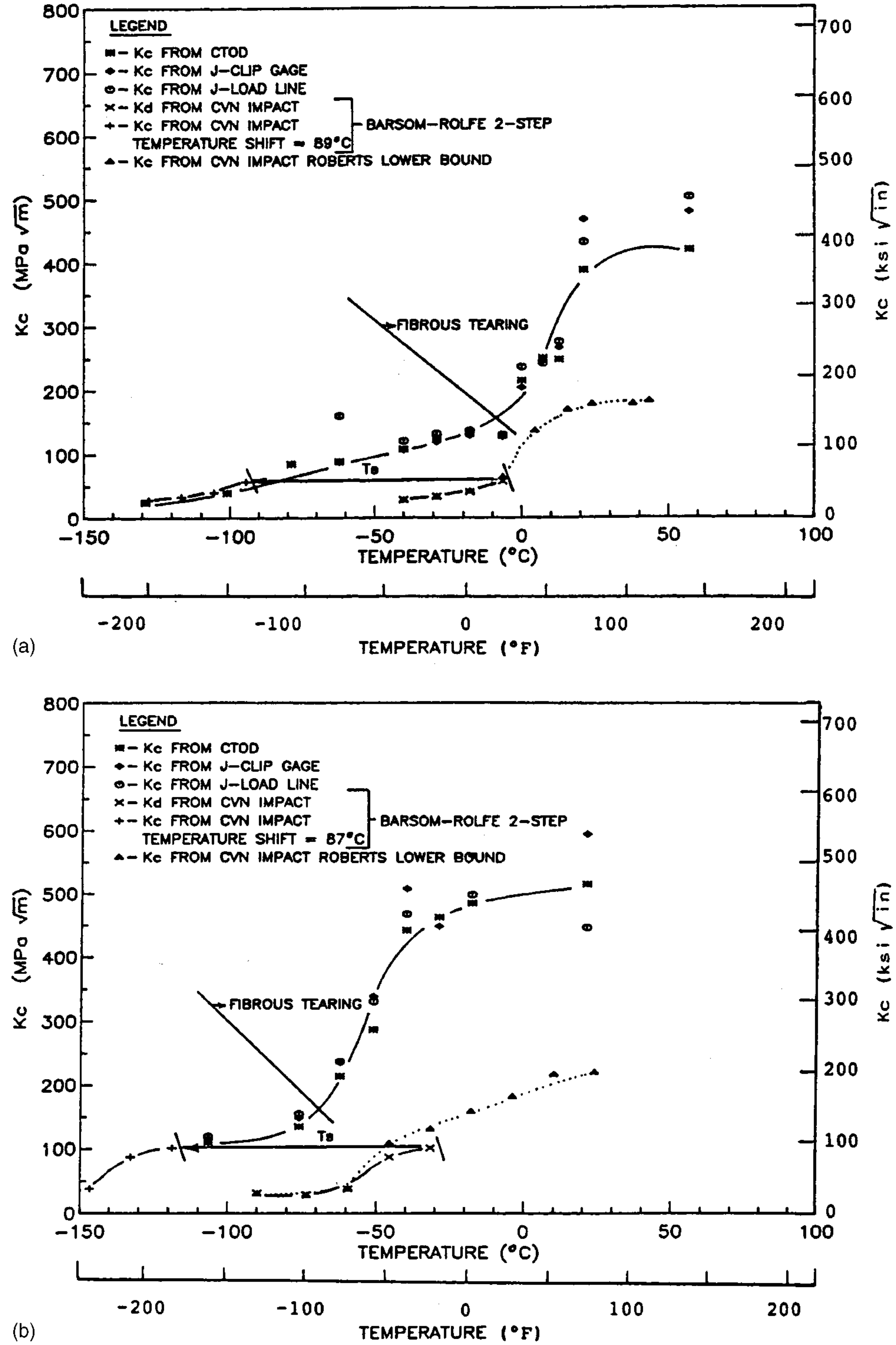

Fig. 5. K_c-CVN-CTOD-J correlations for steels; (a) K_c-CVN-CTOD-J correlations for A131 steel and (b) K_c-CVN-CTOD-J correlations for A516 steel

fracture (Barsom and Pellegrino 2000).

The preceding discussion indicates that the fracture toughness, K_c, of E70TG-K2 weld metal from full-scale test specimens and from CTOD tests of weldment ranged from 88 to 185 ksi$\sqrt{}$in. Also, ductile crack propagation preceded unstable crack extension in the welded moment frame connections tested at the Univ. of Michigan. Consequently, a minimum fracture toughness requirement of 90 ksi$\sqrt{}$in. for weldments subjected to seismic loads was established. Higher fracture toughness values would have negligible beneficial contribution to the performance of the connections. Further improvements in the performance of welded moment frame connections must be achieved by improvements in connection design, detailing, fabrication, and inspection.

Derivation of Equivalent Charpy V-Notch Impact Toughness

The minimum K_c requirement of 90 ksi$\sqrt{}$in. was used to derive an equivalent impact CVN foot pound value that can be used as a screening test for weld metal. A correlation between CTOD data and impact CVN toughness does not exist. Therefore, a procedure was developed based on the general behavior of CTOD test results as a function of temperature and by evaluating existing K_c–CVN correlations.

CTOD values of structural steels increase as the test temperature increases. Initially the increase is gradual, and then accelerates rapidly within a test temperature zone where significant stable ductile tearing prior to unstable crack extension becomes visible with the naked eye on the fracture surface of the CTOD specimens [Fig. 4; Barsom and Rolfe (1999)]. This rapid increase in fracture toughness would have a minor beneficial effect on the fracture behavior of welded moment frame connections subjected to the severe demands of cyclic seismic loads.

Having defined a K_c of 90 ksi$\sqrt{}$in. to be the desired minimum fracture toughness, an equivalent CVN impact energy absorption value had to be established. An evaluation of existing correlations suggested that the Roberts-Newton correlation [Eq. (8) Barsom and Rolfe (1999)] may be helpful. Extreme care should be exercised in the use of this correlation because it can produce erroneous results. This correlation is used here only because the K_c values calculated from the upper shelf impact CVN energy absorption appear to approximate the K_c value above which stable ductile (fibrous) tearing precedes unstable crack extension [Fig. 5; Barsom and Rolfe (1999)]:

$$K_c = 9.325(\text{CVN, ft-lb})^{0.63} \tag{8}$$

Thus, a K_c equal to 90 ksi$\sqrt{}$in. would be equivalent to an impact CVN upper-shelf value of about 37 ft-lb. A conservative value of 40 ft-lb was selected. Based on experiences with other engineering structures, this impact CVN requirement appears to be conservative. If and when a better correlation is developed, the required CVN toughness value could be revisited.

Proposed Charpy V-Notch Requirements

All component tests conducted in the SAC Project have been conducted at room temperature. Thus, the results of these tests are applicable to interior framed buildings. The minimum interior operating temperature for buildings, as expressed by several participants in the SAC Steel Project, is +50°F. Considering the difference in loading rate between seismic and CVN impact loads and the temperature increase of weldments under seismic loads, CVN requirements at 70°F should be adequate for use at +50°F.

The finite-element analysis and strain measurements by Fry (2000) demonstrate that the strain demands on the weld material are very high, even for the RBS specimens. These data show that the strain demand on the weld material is eight times the yield strain for an unreinforced post-Northridge connection and is five times the yield strain for an RBS connection. Consequently, the CVN requirements should be equally applicable to both connections.

The significance of the present 20 ft-lb at −20°F requirement for a moment frame connection exposed to 50°F and higher is not obvious. Although no data are available to investigate the significance, the 40 ft-lb at 70°F requirement may be used to justify relaxing the low temperature requirement to at least 20 ft-lb at 0°F.

In summary, based on the discussions presented in the preceding section, it is proposed that the impact requirement for filler metals used in the fabrication of seismically loaded rigid moment frame connections be 40 ft-lb at +70°F, and 20 ft-lb at 0°F for connections exposed to +50°F temperatures or higher. This CVN requirement should preclude weld-metal fracture toughness from being a contributing factor to the fracture of moment frame connections in seismic applications. Further improvements in the fracture performance of welded moment frame connections must be achieved by changes in design, detailing, fabrication, and inspection. Further research is needed to define the CVN requirements for connections exposed to temperatures below +50°F.

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