41

JRN

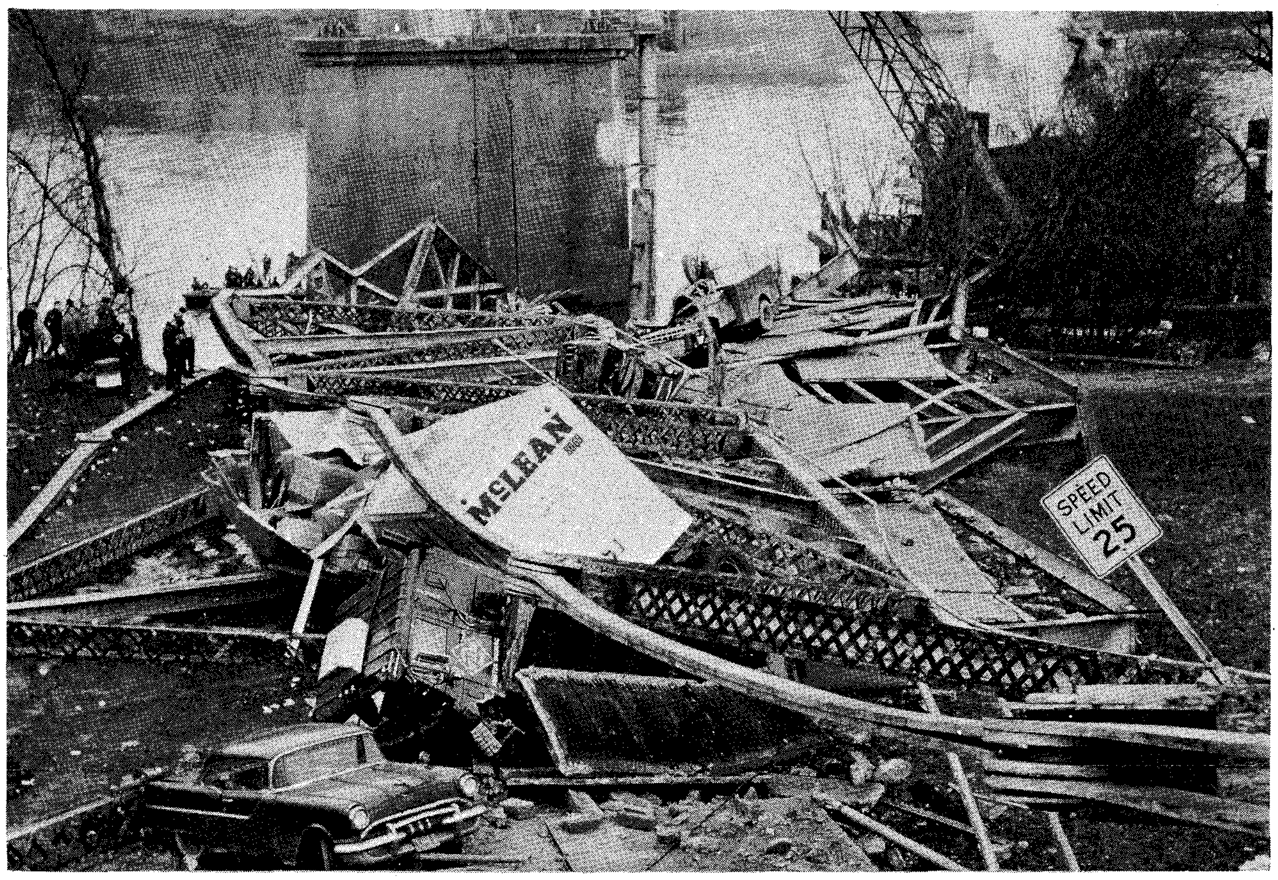
A PROPOSED
FRACTURE CONTROL PLAN FOR NEW BRIDGES
with
FRACTURE CRITICAL MEMBERS

Structural Engineering Series No. 5

June, 1978

Volume III

Prepared by the Bridge Division, Office of Engineering,
Federal Highway Administration, Washington, D.C. 20590



U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

FEDERAL HIGHWAY ADMINISTRATION

Structural Design Publications

The engineer who designs structures for highways is aware of the dearth of design information in the field of structural design. In order to satisfy this need for sound structural procedures, the Federal Highway Administration conducts and sponsors research, analyzes research results, and studies the practical aspects of design.

As design information is developed, it will be made available through the in-service Structural Engineering Series for limited distribution to highway agencies preparing construction plans. The contents of this series will vary, but basically, they will contain design information, including methods and procedures presented in a straightforward manner for ready use by highway engineers. Some material will be state-of-the-art, preliminary or tentative, subject to change upon further research, Criticisms and suggestions on material contained in the series are welcomed.

Requests for these documents and suggestions on the contents of any publications should be addressed to the Federal Highway Administration, Office of Engineering, Bridge Division, HNG-32, Washington, D.C. 20590.

COMMENTARY
on the
FHWA FRACTURE CONTROL PLAN

by
Carl E. Hartbower, P.E.
Welding Engineer

Volume III

Bridge Division
Office of Engineering
Federal Highway Administration
U.S. Department of Transportation
Washington, D.C. 20590

PREFACE

David W. Smith, a senior lecturer in the University of Dundee, Scotland and a consultant to Mott, Hay and Anderson, consulting engineers of London (he formerly was their chief bridge designer), a Fellow of the Institute of Civil Engineers a Fellow of the Welding Institute (UK), studied 143 bridge failures that occurred throughout the world between 1847 and 1975*. The failures were grouped into nine categories. Flood and foundation movement accounted for 70 of the failures. The second most frequent single cause of failure was brittle fracture (22 failures). In connection with the problem of brittle fracture, Professor Smith observed that:

> "Given good design and proper control of welding, this problem has now been beaten so far as it concerns the bridge engineer using Grades 43 or 60 steel in thickness up to 2 inches in temperate climates. Beyond these limits safe structures can still be built, but the bridge engineer needs special advice."

In conclusion, Professor Smith makes a plea for less complexity in the design codes.

> "For the general run of bridges, a short and simple code with which the designer can become thoroughly familiar is highly desirable and may prove to be a matter of life and death." (Emphasis added)

The Fracture Control Plan (FCP) deals with bridges containing fracture critical members so they can hardly be classed as a "general run of bridge". Brevity, nevertheless, is desirable in any code providing the code exercises the necessary controls. The FCP as written is based primarily on the American Welding Society's AWS D1.1 Structural Welding Code which contains 125 single-spaced pages in Sections 1 through 9 inclusive, including 19 single-spaced pages in the AASHTO revision of AWS D1.1. The FCP contains 140 double-spaced pages, superseding both of these specifications.

Those designing and fabricating bridge members should be very familiar with AWS D1.1 and, therefore, should find it easy to work with the FCP by a direct comparison between the provisions of AWS D1.1 and the amended provisions of the FCP. The California Department of Transportation "Standard Specifications for Welding Structural Steel, January 1978", and the New York State STEEL CONSTRUCTION MANUAL, which are used by those States in lieu of AWS D1.1, cannot easily be compared with each other or with AWS D1.1 because their paragraphing is not the same as AWS D1.1.

---

*PROCEEDINGS, Institute of Civil Engineers, Part 1, August 1976, pp. 367-382, London; also Civil Engineering - ASCE, November 1977, pp. 59-62.

INDEX

TABLES

FIGURES

APPENDIXES

COMMENTARY on the FRACTURE CONTROL PLAN

INTRODUCTION

In the last two decades several bridge girders have failed by brittle fracture; only two received widespread attention - the collapse of the Silver Bridge at Point Pleasant, West Virginia, and the near collapse of King's Bridge at Melbourne, Australia. The Silver Bridge did not involve welding; whereas, failure of the King's Bridge was the result of local stress elevations due to poor design details, limited weldability of the steel, and improper welding procedures. The fractured tension chord in the Silver Bridge was a non-redundant member; whereas, the failed span of the King's Bridge consisted of a four-girder system. The structural redundancy of the King's Bridge has been described as typical of that used in bridge design.* However, it is noteworthy that it was the concrete architectural sidewalls beneath the span that caught the bridge and prevented complete collapse when the four-girder system developed brittle fracture after little over one year of service. All four girders in the collapsed span contained welding-induced cracks at the ends of cover plates; one of the girders fractured to within a few inches of the top flange months before the bridge collapsed. Collapse occurred on July 11, 1962, when the three remaining girders fractured. The multiple defects in the King's Bridge were of an insidious type that seems to defy detection using conventional nondestructive examination methods. In the last few years there have been several "cracked" bridges presaging additional collapsed bridges.

On June 13, 1970, a brittle fracture occurred in one of three tension flanges of a large steel box girder under construction at Bryte Bend, California. A category "E" cross-bracing detail initiated a hot crack where it was welded to a tension flange.

On October 29, 1971, a brittle fracture occurred in a girder-truss joint of the Fremont Bridge, Portland, Oregon. The fracture initiated at a metallurgical defect produced during fabrication of the detail. Failure analysis in 1971 revealed numerous weld defects elsewhere in the bridge. Late last year, an inspection of the bridge revealed additional defects, one of which is two-feet long with an ultrasonic defect-severity rating of minus 8 db.

----

*Madison and Irwin, Journal Structural Division, Proceedings of ASCE, ST9, Paper No. 8377, September 1971, pp. 2,229-2,244.

On May 7, 1975, one of the 11-ft-deep main girders of the Lafayette Street Bridge over the Mississippi River at St. Paul, Minnesota was discovered to be fractured to within a few inches of the top flange. The fracture initiated from a defective gusset-to-stiffener weld.

On January 28, 1977, one of the 11-ft-deep main girders of the I-79 Bridge at Neville Island near Pittsburgh fractured. The fracture initiated in a defective electroslag-welded flange butt splice.

On January 4, 1978, three consecutive box-beam cap girders carrying Chicago Transit Authority commuter trains over the Rock Island RR tracks were found to be fractured. No details are available as yet, but this also could have been a disaster. The structure was finished in September, 1969, so it has seen less than 9 years of service.

In addition to the brittle fractures just described, many less dramatic but, nevertheless, potentially serious problems with cracking have occurred in the 1970's. For example, last summer the Silver Bridge at Point Pleasant, West Virginia was found to contain numerous cracks in flange butt splices. This is the bridge which replaces the Point Pleasant Bridge which collapsed in 1967 killing 46 persons. In the present Silver Bridge, the cracking involves two steels (A514 and A441), two fabricators and two steel suppliers. All of the joints were submerged-arc welded.

Another example, is the Yellow Mill Pond Bridge on the Connecticut Turnpike at Bridgeport. During the winter of 1970-71, the bridge was discovered to have a "cracked" girder. The crack parted the bottom flange and extended about 16 inches up the web. The fracture initiated from a cover-plate fillet-weld toe crack which in turn initiated fatigue crack growth. The bridge was built in 1958; the steel is A242. In May 1972, a study of problem (HPR 175-332) noted that "on the basis of current popular methods of fatigue analysis, which tend to neglect stress range below 3 ksi, fatigue failure of the beams tested (in the Yellow Mill Pond Bridge) would be a remote possibility for the near fracture". The recorded stresses were small (RMS stress range only about 1.3 ksi) but there is heavy truck traffic. In June, 1976, span 10 was sandblasted at the ends of the cover plates; a great number of cracks were identified at the ends of the cover plates, on both primary and secondary cover plates. Some of the cracks were as long as 2 inches and over 1/2-in. deep as determined by ultrasonic testing.

In November 1973, a large crack was discovered in a facia girder of the suspended span of the Quinnipiac River Bridge, near New Haven, Connecticut. The fracture entered the tension flange as a cleavage fracture but <u>arrested before penetrating the flange thickness</u>. Subsequent examination of the flange fracture-surface indicated extention of the part-thru crack by fatigue until it penetrated the full thickness, leading to discovery of the cracking.

Cause of cracking. The above examples are but a few of many similar situations that have occurred in the last decade. Why is there so much weld cracking? This has been and is continuing to be a subject of investigation. In the Bryte Bend Bridge, it was found that the weld-crack which initiated the brittle fracture was a hot crack; thus, the steel was seriously deficient in weldability as well as having extremely low toughness.

In the Fremont Bridge, there were workmanship problems, design deficiency and low-toughness steel. A critical member which served to join the arch rib to the bottom flange of the main box girder was improperly oriented with respect to rolling direction and had been locally overheated in preheating for welding, producing an embrittled surface. The steel had less than 10 ft-lb CVN impact at the lowest anticipated service temperature (the LAST) irrespective of specimen orientation.

In the Lafayette Street Bridge, the gusset-to-stiffener connection was intended by the designer to be a complete-penetration groove weld. But the connection as designed was only a few inches above the bottom flange, leaving no space for the welder to back gouge and weld from the bottom side; therefore, the fabricator used a backup bar and failed to get complete penetration. The ensuing fatigue-crack growth entered the girder web and triggered brittle fracture. The web steel which developed cleavage fracture gave a CVN-impact value of 30 ft-lb at +40°F and 58 ft-lb at +70°F, easily passing the present AASHTO toughness requirement.

In the electroslag-welded bridge at Neville Island near Pittsburgh, the defect that triggered brittle fracture in the flange butt splice appears to be a hot crack. This crack, approximately 3 inches in diameter at the time the web-to-flange fillet weld was made, should have been found by nondestructive inspection. To make matters worse, the toughness of the weld was very low - less than 15 ft-lb at 0°F (in service the bridge sees much lower temperatures than 0°F).

In the Yellow Mill Pond Bridge and the King's Bridge, fillet-weld toe cracking is the problem - and it is a serious problem because it is a type of cracking that is very hard to find, and it is a type of cracking that may be associated with a heat-affected-zone weldability problem. Moreover, when hydrogen is present, the cracking may be delayed. The tests that are most appropriate to appraise this type of cracking are not used by fabricators in this country (the British Controlled Thermal Severity (CTS) test and the cruciform test).

In the Quinnipiac River Bridge, the crack initiated from a defective (incomplete-penetration) butt-weld splice in a 3/8 x 4 1/2-inch longitudinal stiffener fillet welded to the face of the 7/16-inch-thick web plate. The butt weld in the stiffener cracked under bridge traffic which in turn initiated fatigue cracking in the web; the fatigue crack penetrated the 7/16-inch thickness before it ran catastrophically as a brittle fracture.[3]

---

(3) John W. Fisher, Fritz Engineering Lab. Report 385.3, December 1974.

CVN-impact testing of the web and flange revealed material which easily met the AASHTO specification for A36 steel.

AASHTO Test F	CVN-Impact (ft-lb)	
	Web	Flange
+40	20	35
+70	50	60

However, at $0^{o}F$ both web and flange developed less than 10 ft-lb CVN-impact; the fact that the fracture arrested in the flange suggests that the brittle fracture may have initiated at a temperature where there was an appreciable difference between the crack-arrest toughness of web and flange.

The service failures involving cracking and brittle fracture cited above together with FHWA management reviews of structural steel fabrication have revealed numerous and widely diversified deficiencies in design, steel, welding, inspection and erection.

Design has been found on occasion to involve: excessive welding, intersecting welds, unnecessary stiffeners (perpendicular to the applied stress), attachments welded in tension areas that would better have been bolted, vagaries in the drawings with respect to what is to be welded and where, specification of weld joint design without regard for the fabricator's capabilities, use of excessively thick flanges, and many other deficiencies.

Steel has been found on occasion to involve: serious deficiencies in static plane-strain (K_{Ic}) toughness at the service temperature, serious deficiencies in dynamic (K_{Id}) fracture toughness at the service temperature, nil-ductility transition (NDT) temperature at or above the service temperature, initiation of brittle fracture in cleavage in spite of acceptable Charpy V-notch impact values at the AASHTO test temperature, coarse-grain microstructure in spite of fine-grain melting practice, in A514/517 plate widely different microstructure at surface and mid-thickness (due to insufficient hardenability), a serious deficiency in weldability, etc.

Weld Fabrication has been found on occasion to involve: deviations from AWS/AASHTO welding specifications; little or no effective in-house quality control; welding inspectors that are poorly trained and/or inexperienced; prequalified weld procedures that are not available at the welding station or are incomplete; meters for setting amperage and voltage that are inaccurate, out of calibration or unreadable; no welding engineers with the academic training and/or experience to solve or even recognize welding problems that can lead to brittle fracture; repairs made without preheat and without the knowledge of quality control (and, therefore, are not inspected); weld-procedure-qualification test results that are not even an approximation of the production-weld properties*; contaminated SAW weld wire as produced and packaged by the wire supplier (chemical analysis revealed an unidentified long-chain hydrocarbon on the surface of the wire); an SAW wire involved in a cold-cracking problem with interstitial hydrogen in excess of 5 ppm; FCAW wire that was not a "low-hydrogen" electrode; CO_2 gas shielding blown away by strong drafts in the weld shop; gross piping porosity effectively halving the throat cross-sectional area of fillet welds and yet rarely breaking the weld surface and, therefore, easily missed by inspection; welds inadvertently highly alloyed by "active" SAW fluxes when arc voltage was too high; and may other deficiencies.

Inspection (quality control and quality assurance) has been found on occasion to involve: badly structured, loosely managed organizations that often are not even aware of serious welding problems on the shop floor; contracts for quality assurance inspection with payment based on the tonnage of steel fabricated; inspectors that are poorly trained and/or inexpérienced in the type of inspection required; radiographs of poor quality and sometimes not competently (or conscientiously) read; ultrasonic testing by personnel apparently incompetent to make the necessary observations and measurements; quality assurance that relies heavily on the integrity of the fabricator rather than following a rigid inspection program; nondestructive quality assurance inspection by contracted inspection agencies without supervision by the State**; magnetic-particle inspection with prods that cause arc-burn cracking in the surfaces being inspected; welds joining attachments to primary members that are considered "secondary" and, therefore, not inspected; undetected cracks in repair welds that trigger crack extension into the primary member; and many other discrepancies.

----

*The weld-procedure-qualification testing prior to fabrication of the fractured bridge girder at Neville Island, Pittsburgh, gave a Charpy V-notch impact value of 23.6 ft-lb at $0^{o}F$ (range 15.2 to 37.9 ft-lb); whereas in the as-welded bridge, the Charpy value was less than 15 ft-lb at $0^{o}F$. The weld-procedure-qualification test record for the tie girders of the tied arch at Neville Island showed a range of 56 to 85 ft-lb, with an average value of 73 ft-lb at $0^{o}F$; whereas, in the as-welded bridge, the Charpy impact exhibited no evidence of fibrous fracture and the average value was 11.5 ft-lb at $0^{o}F$ (with only 1 specimen meeting the AWS minimum requirement of 15 ft-lb at $0^{o}F$). Was the weld-procedure specification not followed by the fabricator or is the limitation of variables cited in AWS D1.1 Section 5.5 too loose?

**The welds in a large bridge were recently reradiographed and found to be defective (slugged, incomplete penetration, etc.); the welds previously had been radiographed and approved by a large well known independent testing agency. (Reference: "Structural Steel Management Review for (FHWA) Region 6.")

Erection has been found on occasion to involve a variety of temporary welds that may or may not have been made according to the code, including brackets for lifting girders, brackets for stiffening girders during handling, attachment welds superimposed on grcove-weld butt joints in tie girders, uninspected "temporary" welds left in place after erection, temporary welds removed but with no nondestructive inspection of the girder for residual cracking in the weld heat-affected zone, etc.

This brief overview of the problem points up a complex situation involving deficiencies in design, in steel, in fabrication, in quality control and quality assurance, and in erection. Perhaps the most serious overall observation is that until 1977 AASHTO gave no special consideration to the design, fabrication, inspection and/or erection of FCMs - and in 1977 the only change was to lower the stress range in fatigue. Much more needs to be done in the interest of public safety where bridges contain fracture critical members. The following fracture control plan corrects many of the important deficiencies in management and specification.

COMMENTARY

The following commentary on the Fracture Control Plan has been prepared to generate better understanding in application of the plan. Since the Fracture Control Plan is written in the form of a specification, it cannot present background material or discuss the writer's intent; it is the function of the Commentary to fill this need.

All references to numbered paragraphs, tables, and figures, unless otherwise indicated, refer to paragraphs, tables and figures in the Fracture Control Plan.

1.1 Scope. Fracture control, if it is to be meaningful, must be exercised in all phases of construction starting with design and material selection, in material specification, in fabrication, in quality control by the fabricator, in quality assurance by the owner or his agent, and in erection. Each phase is equally important in terms of fracture control; an error or omission at any stage can endanger the final structure. One faulty design detail, one improperly rolled plate with low toughness, one poorly made weld, one defect overlooked in nondestructive inspection, can initiate premature fatigue crack growth, which in turn can trigger catastrophic brittle fracture of a fracture critical member.

1.1.2 Fabrication qualification. Reference to "another suitable program" recognizes that a fabricator with facilities, equipment and personnel qualified, for example, to do nuclear work, say with an "N" stamp, should be at least as qualified for fabricating major steel bridges as a firm certified by AISC.

1.1.3 Welding inspector. It is the welding inspector who must assure that the AWS/AASHTO Structural Welding Code is adhered to in the fabrication of FCMs; it is the welding inspector who must assure that the qualified welding procedure is followed day after day, shift after shift within the "limitation of variables" as specified by AWS D1.1 Section 5.5. The Code is little more than window dressing if an experienced, competent, consciencious person is not on the job seeing that the Code is followed, and that high standards of workmanship are maintained at all times (e.g., even inside a box girder) when FCMs are being fabricated.

The intent here is rely heavily on personnel who have been tested to prove their competence, with the testing done by a disinterested organization to avoid any possibility of "conflict of interest" when the testing is done by the fabricating shop itself.

Reference to "or other suitable demonstration of competence" makes provision for persons who have had extensive relevant education or experience or are certified by another program (e.g., a class-III welding engineer in Japan).

Both fabricator and owner must have qualified welding inspectors because in this plan it is intended that quality assurance by the owner not be a passive operation, but an active inspection complementing and sometimes duplicating the inspection done in quality control by the fabricator.

1.1.4 Nondestructive inspection qualification. The same comments hold here as for "welding inspector". The welding inspector is responsible for welding procedure, workmanship, etc.; whereas the NDT technicians are responsible for finding defects produced by human error or faulty procedures. Recent experiences with slag, porosity and cracks that somehow were not found by nondestructive testing, suggests that for FCMs we must be assured of proper equipment and techniques at all stages of inspection, i.e., both in QC and QA.

1.2 Definitions.

1.2.1 Fracture critical members (FCMs). Some people are predicting that the more strigent requirements of this fracture control plan will be applied to all bridge members, fracture critical or not, and that some designers "to be safe" will indiscriminantly label members as fracture critical when they are in fact redundant. This obviously would not be good engineering because such practice would be wasteful of public funds and could result in concrete bridges where steel plate girders would be the better construction. AISC and FHWA will question the engineering judgment of any firm that misuses the definition of FCMs.

There are some bridge designs that clearly are fracture critical, such as the tie girders of a tied-arch bridge, or the girders of a simple-span two-girder bridge, or a steel cap girder. It has been suggested that a complete list of FCMs be appended to this fracture control plan, but in selecting such details there are complications that must be considered by the DESIGNER on a case-by-case basis; e.g., the extent of the lateral bracing system in a two-girder continuous system, etc. The responsibility for determining which, if any, steel bridge member components are "fracture critical" must rest with the designer.

1.2.2 Tension components. Bridges have developed brittle fracture as a result of cracking in a so-called secondary member welded to a main-load-carrying tension member. In the Lafayette Bridge, St. Paul, Minnesota (May 1975), a lateral-bracing gusset plate welded to both a transverse stiffener and to the web of an 11-foot-deep I-girder contained a defective weld at the gusset-to-stiffener connection; a crack developed, grew into the web and caused fracture of the girder. In the Quinnipiac Bridge, near New Haven, Connecticut (November 1973), a 3/8 x 4 1/2-inch longitudinal stiffener containing a defective butt-weld splice initiated fatigue cracking in the web, which in turn caused brittle fracture in the web. In the Bryte Bend Bridge, Sacramento, California (June 1970), a hot crack in the weld connecting a cross bracing to a tension flange initiated slow crack growth into the flange and then brittle fracture.

From the above in-service experiences, it is clear that any welds joining an attachment to a tension component of an FCM, whether the welds are parallel or perpendicular to the applied stress in the tension component, must be considered part of the tension component and, therefore, considered fracture critical. In the Bryte Bend Bridge, the weld joining the cross bracing to the tension flange was parallel to the principal stress but the crack that triggered brittle fracture was transverse to the applied stress. In the Lafayette Street Bridge, the defective weld that developed fatigue-crack growth was perpendicular to the stress in the girder and parallel to the stress in the gusset; when the crack grew into the web of the girder, it triggered brittle fracture. Thus, if the steel used in an attachment, welded to a tension component of an FCM, has low toughness or involves a low-toughness defective weld, a crack developing in the attachment or attachment weld may reach critical size, pop-in and run catastrophically into the fracture-critical tension component, or in fatigue could grow into the tension main-load-carrying tension member and then become a brittle fracture.

1.3 Design and Review.

1.3.1 In a survey of the fifty States under FHWA-RD-73-3, the Departments of Transportation were asked to identify those bridge members whose failure would result in total collapse of the bridge structure. The following replies are more or less representative:

NH: Truss and two-girder systems are susceptible to total collaspe.

OH: Truss chords and tension flanges of girders receiving their loads from floor beams.

PA: Either girder in a two-girder system, and main chord members of trusses.

TX: Simple-span floor beams carrying the entire reaction of a unit, or only two longitudinal girders supporting the deck.

A bridge designer* from California offered additional examples: the steel cap of a single- or two-column structure; a structural-steel C-type bent; the hanger plate in single and two-element girders; the bottom flange of an open multi-cell box girder, etc.

Bridge designers with FHWA have suggested some additional details that may be considered fracture critical:

(1) in a two-girder system, the girder within one span length of a roller expansion bearing, or in the end spans, or in a simple span, or in the spans adjacent to a hinge or a link;

(2) C- and T-type pier caps where stringers interlace the steel cap;

(3) anchors for the cables of cable-stayed girders;

(4) counterweight hangers of bascule bridges; and

(5) strap hangers and pins of two-girder birdges.

1.3.2 Some are asking for a definition of "welding engineer".** The obvious definition is a person with a bachelor of science degree from an accredited college or university with a major in welding engineering. Ohio State University has been graduating people with a BS degree in welding engineering for over a decade. Because there are so few graduating with the degree of Welding Engineer, many firms have people with the title "Welding Engineer" who do not have a degree in welding engineering but have a <u>combination</u> of experience and academic training in related fields such as metallurgical engineering, civil engineering, mechanical engineering, electrical engineering, chemical engineering and fracture mechanics. Because of the nature of welding, the emphasis should be on experience and/or training in materials science and metallurgical engineering.

---

*Roger Sunbury, private communication

**<u>One definition</u>: a professional, educated in welding and metallurgy, whose experience and technical training combined qualify him to recommend and approve weldment designs and procedures, and to solve unusual, complex and unprecedented problems in weld fabrication. This person can attain the necessary knowledge through formal training at a recognized college or university or combine professional experience with education to meet the same level of professional competance.

Welding inspectors are key people in fracture control because one of their principle responsibilities is to assure that the welding-procedure specifications are followed as specified in AWS D1.1 Section 5.5. If the welding-procedure specifications are not followed, the concept of a qualified welding procedure is meaningless and the cost of weld-procedure testing is largely a waste. This function of the welding inspector is so important, it is necessary that a welding engineer be available to supervise the welding inspector (usually) the welding inspector is a subprofessional person).

1.3.3 If the design firm is to verify that this fracture control plan has been followed in every detail, it will be necessary for the designer to have surveillance over each stage of construction, including material procurement, fabrication, quality control and erection. Thus, the designer will know if the steel is shipped to the fabricator without proper identification or with unacceptable mill test reports, or if welding consumables are shipped without proper identification or with damaged packaging, or if the fabricator doesn't hold to the "limitation of variables" of AWS D1.1, paragraph 5.5 or has problems with weld-procedure qualification testing, or if ultrasonic inspection is done by an unqualified or careless person, or there is a delayed-cracking problem, or if stay-in-place forms are tack welded to the edges of FCMs or lifting hardware is illegally welded to FCMs, etc. It is the intent of this paragraph to revert to a practice used in years past, requiring the designer to oversee all stages of bridge building from design to erection.

1.3.4 Any attachment welded to a tension component of a FCM must be nondestructively inspected if we are to minimize the possibility of slow crack growth into a main-load-carrying tension member. Groove-weld butt splices in back-up bars are a prime example often neglected in the past. Gusset plates welded to stiffeners which in turn are welded to the FCM provide another example.

Recent (1970s) reinspection of welded plate-girder bridges has revealed case after case of flaws that apparently escaped detection at the time of fabrication. Cracks discovered after the bridge is in service may be the result of delayed (cold) cracking occurring after the final inspection. Some bridges have been fabricated without a limit on the time dealy between completion of welding and final inspection; other bridges have been built with only 24 or 48 hours minimum delay. Research has shown that some steels will continue to undergo crack growth for 2 weeks or longer after completion of welding without external load (residual welding stresses only).* It is also possible that cracking found in bridges after they are in

---

*Hartbower, et al, ACOUSTIC EMISSION, ASTM STP505, American Society for Testing Materials, 1972, pp. 187-221.

service is the result of only fatigue; however, several bridges have been found with extensive cracking after less than 10 years of service. If the problem is strictly a matter of fatigue, then we have a very serious problem with bridge design. More likely, it is a combination of weld cracking undetected by quality control and fatigue-crack growth started (initiated) by the prior weld crack. Thus, there is a serious question about the adequacy of the present inspection procedures for quality control.

This fracture-control plan specifies that quality assurance (non-destructive inspection by the owner or his agent) will not be passive; i.e., quality assurance will consist of actual testing, sometimes complementing the test method used by the fabricator in quality control and sometimes duplicating the fabricator's test method.

1.5.1 Steel types. Steels have been developed for other industries which are both weldable and capable of high toughness at low service temperature. There can be a price advantage in specifying such steels because the penalties levied by the steel industry for high toughness may be less or nonexistant.

ASTM-A633-75 grade D, which falls within the chemical and physical requirements for ASTM A572 grade 50 has 0.20% carbon maximum and can be bought with a guaranteed nil-ductility transition (NDT) temperature of minus 60^{o}F and a Charpy V-notch impact test of 30 ft-lb at minus 30^{o}F in plate up to 2-inches thick when normalized.

ASTM A678-75 grade A, with a minimum yield strength of 50,000 psi can be procured to a guaranteed NDT temperature of minus 80^{o}F and a CVN-impact test of 70 ft-lb at minus 30^{o}F. The steel has 0.16% carbon maximum and is quenched and tempered.

A537-76 class 2 is a quenched-and-tempered carbon-manganese-silicon steel for fusion-welded pressure vessels and structures. The steel can be supplied up to 2 1/2-inches thick at 60-ksi-min. yield and over 2 1/2 to 4 inches at 55-ksi-min. yield. ASTM provides for supplementary requirements, including Charpy V-notch impact (S5) and drop-weight testing (S6). Studies for the Ship Structure Committee included A573, class 2 steel with an upper-shelf CVN impact energy of 140 ft-lb, a 20 ft-lb transition temperature of minus 150^{o}F, at a yield strength of 65 ksi.

A588-75 is well known to bridge designers, available in nine grades. For plate up to about 1 1/2-inch thick, the toughness requirements of the fracture control plan may be met by control rolling. However, there is an alternative which is important for those steel suppliers who feel it necessary to normalize to achieve the specified toughness levels. The alternative involves quenching and tempering, i.e., ASTM, A558-modified. At least one major bridge has been fabricated using quenched-and-tempered A558 steel; the Charpy test results were generally in excess of 150 ft-lb at +20^{O}F (the LAST). The yield strength was specified at 60-ksi minimum yield for Q&T plate. No problems were encountered with cracking or loss of toughness in the weld heat-affected zone of the Q&T A588 steel.

Provision for Q&T steels. It is recommended that the special provisions for FCMs do not preclude the use of quench-and-tempered (Q&T) steel, even if the steel is not usually supplied other than as-rolled or normalized. It is common knowledge among metallurigical engineers that quenching and tempering will enhance the toughness properties of many structural steels. In the prebid conference, the contractor should be urged to "shop around" for a steel supplier who is willing to supply steel in the Q&T condition.

International Standards Organization (ISO) plate steels. Several grades of plate steel have been evaluated recently* as to their Charpy V-notch impact properties. The steels were produced in Japan and data submitted by the Iron and Steel Institute of Japan to the Secretariat of TC17, SC10, LTP of ISO. Of the 14 grades of plate steel evaluated, three quenched-and-tempered grades were selected for purposes of demonstrating the properties achievable in ISO plate steels.

ISO No.	No. of SAMPLES	THICK mm	Average Composition				
			C	Mn	P	S	Other
PE460	46	12-48	0.13	1.25	.016	.007	.039V
PE500	54	15-75	0.12	1.18	.015	.005	.04V, .14Mo
PE690	28	19-55	0.12	0.87	.010	.005	.96Ni, .53CR .40Mo, .04V .0014B

The following table summarizes the mechanical properties of the steels:

ISO No.	FTY mean (SD)	FTU mean (SD)	CVN@ -40^{O}F (FT-LB) mean +1SD	mean	mean -1SD
PE460	81 (7)	94 (5)	180	120	60
PE500	80 (4)	94 (4)	210	160	105
PE690	114 (5)	121 (4)	170	130	90

where SD is "standard deviation", FTY is yield strength and FTU is ultimate strength. The chemistry, tensile properties and Charpy V-notch impact transition curves for these highly tough steels are

----

*John M. Hodge, "Analysis of Impact Property Data Submitted to TC17 by the Iron & Steel Institute of Japan", November 23, 1977. From an unpublished report of the Metal Properties Council, Inc., by permission.

reproduced in Figures C1.5.1a, b, c and d. One ISO composition (P43) containing 3 1/2% nickel, normalized, is included for comparison with the Q&T steels. Forty plates of the normalized steel were analyzed, with thicknesses up to 87 mm (3.87 in.). This particular ISO grade was previously evaluated, as supplied by several European countries. The mean 70 ft-lb Charpy V-notch impact transition temperature for the 59-ksi yield-strength European steel was minus 80°C (-112°F). The 2-sigma limit at minus 40°C (-40°F) for the Japanese steel in this grade was about 95 ft-lb. Thus, it appears that the toughness of the European-made steel would easily meet the requirements of this fracture control plan.

Clearly, steel of excellent toughness can be made. The super toughness of the Japanese steels is, presumably, the result of very low sulfur content and the generally low carbon content (overall mean carbon content 0.13%). It should be noted that the highest toughness required in this fracture control plan is 50 ft-lb in 90-ksi-yield-strength, 3-inch-thick plate and 55 ft-lb in high-strength, 2-inch-thick plate tested at minus 30°F (Alaska will require lower temperatures). Based on the tabulated data and figures for the ISO grades of plate steel, these requirements can easily be met by foreign steel.

The often repeated question: Who needs greater toughness than presently specified by AASHTO?, seems to ignore the research and development that went into linear elastic fracture mechanics (LEFM), the basic concepts of fracture control and years of service - failure experience. With sufficient toughness to assure general yielding (through-thickness yielding), there is a chance to find a crack by nondestructive inspection before the catastrophic brittle-fracture event occurs. Moreover, toughness is a factor in control of fatigue crack growth rate.

1.5.2 Plate edges. Edge defects can be a source of crack initiation both with and without welding on the edges. Laminations in plate edges are often exposed by flame cutting.*

1.5.3 Plate thickness. The limitation on plate thickness is dictated by the difficulties currently being experienced with weld cracking. The thicker the plate, the greater the number of weld passes and

---

*This provision has been extracted from the New York State STEEL CONSTRUCTION MANUAL as revised Aug. 1, 1974 & Feb. 2, 1976. The above notation will hereinafter be abbreviated simply as "N. Y. State".

FIGURE 10

PE 460 - 46 Plates - Mn V-Q+T 12-48 mm (.47-1.89 in)

Composition %

Element	Min	Max	Mean	SD
C	0.10	0.17	0.13	0.02
Mn	0.97	1.45	1.25	0.1
P	0.007	0.024	0.016	0.004
S	0.002	0.014	0.007	0.005
V	0.03	0.07	0.039	0.01
		Mechanical Properties - N/mm^2 (Ksi)		
YS	477(69)	682(84)	565(81)	50(7)
TS	571(83)	754(109)	651(94)	37(5)

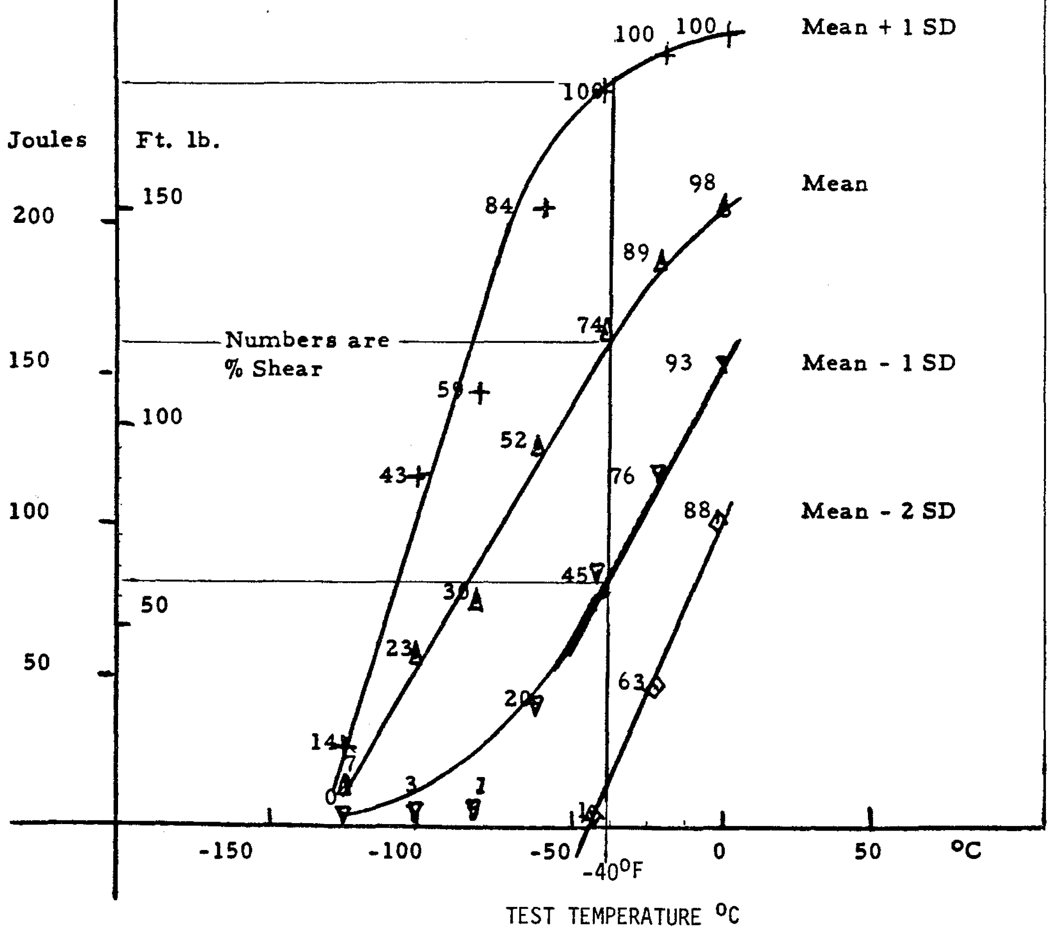


Figure C1.5.1.a - Analysis of Impact Property Data Submitted to TC17 by the Iron and Steel Institute of Japan (John M. Hodge).

FIGURE 11

PE 500 - 54 Plates - Mn Mo V-QT 15-75 mm (.6-3 in)

Composition %

Element	Min	Max	Mean	S.D.
C	0.10	0.17	0.12	0.02
Mn	0.86	1.44	1.18	0.15
P	0.006	0.028	0.015	0.005
S	0.002	0.018	0.005	0.003
V	0.03	0.07	0.04	0.01
Mo	0.09	0.25	0.14	0.04
Mechanical Properties N/mm^2 (Ksi)				
YS	497(72)	651(94)	552(80)	30(4)
TS	611(89)	749(108)	646(94)	29(4)

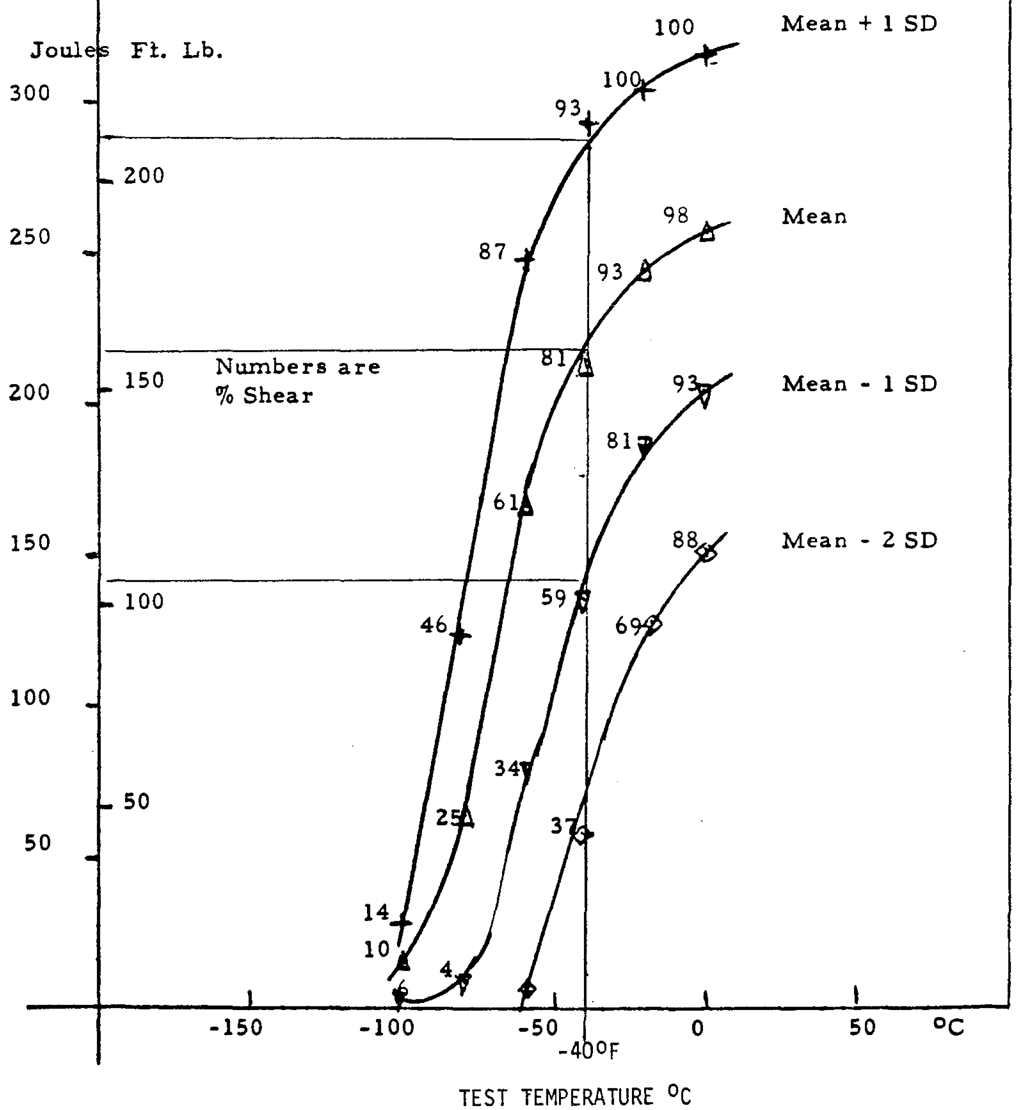


Figure C1.5.1.b - Analysis of Impact Property Data Submitted to TC 17 by the Iron and Steel Institute of Japan (John M. Hodge).

FIGURE 12

PE 690 - 28 Plates - Ni-Cr-Mo-V-B-QT-19-55 mm (3/4-2.2in)

Composition %

Element	Min	Max	Mean	S.D.
C	0.10	0.14	0.12	0.01
Mn	0.81	1.08	0.87	0.05
P	0.004	0.017	0.010	0.003
S	0.001	0.013	0.005	0.002
Ni	0.48	1.19	0.96	0.17
Cr	0.40	0.81	0.53	0.12
Mo	0.26	0.47	0.40	0.05
V	0.03	0.05	0.04	0.005
B	0.0008	0.0028	0.0014	0.0007
Mechanical Properties N/mm^2 (Ksi)				
YS	716(103)	862(125)	787(114)	34(5)
TS	788(114)	921(133)	837(121)	27(4)

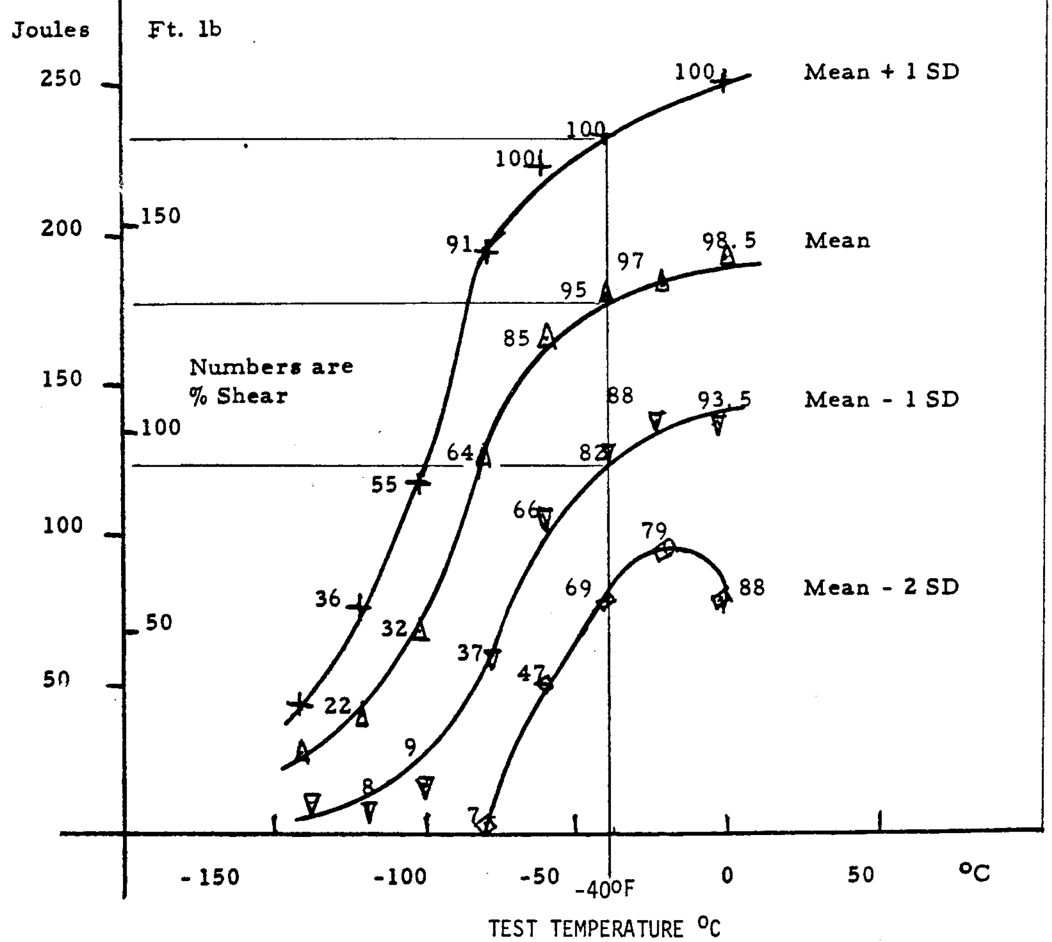


Figure C1.5.1.c - Analysis of Impact Property Data Submitted to TC17 by the Iron and Steel Institute of Japan (John M. Hodge).

FIGURE 13

P 43 - 40 Plates - 12-87 mm (.47-3.87 in.)

Composition %

Element	Min	Max	Mean	SD
C	0.05	0.12	0.10	0.01
Mn	0.54	0.65	0.59	0.03
P	0.003	0.013	0.0065	0.002
S	0.001	0.013	0.004	0.004
Ni	3.40	3.61	3.55	0.05
	Mechanical Properties - N/mm^2(Ksi)			
YS	352(51)	488(71)	409(59)	26.4(4)
TS	465(67)	598(87)	533(77)	25(4)

Heat Treatment - Normalized

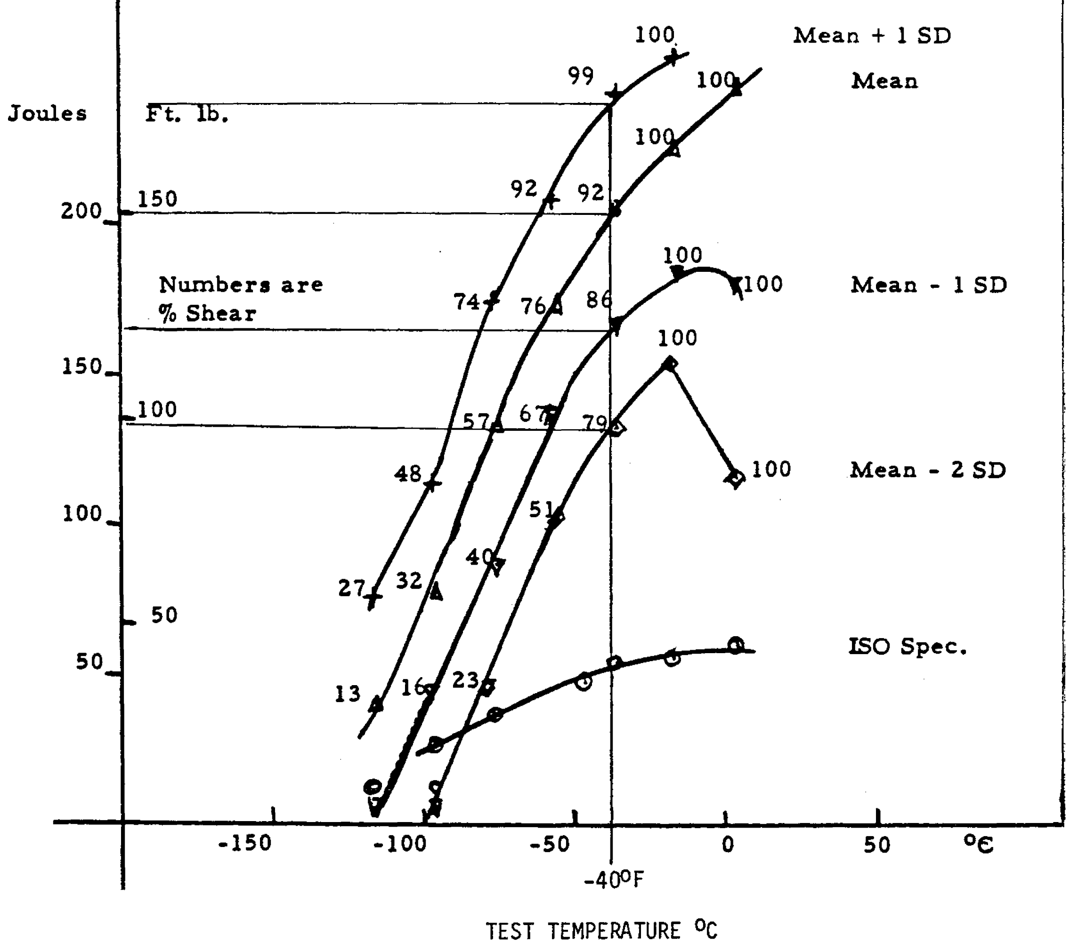


Figure C1.5.1.d - Analysis of Impact Property Data Submitted to TC17 by the Iron and Steel Institute of Japan (John M. Hodge).

greater the chance of weld defects. For example, with the submerged-arc-welding procedure used in two bridges, one for Nevada and one for California, for groove-weld butt splices in 1 7/8-inch-thick plate, there were forty (40) weld passes.

In high-strength Q&T steel with yield-strength of 100 ksi, the 2-inch thickness limitation is equivalent to 50-ksi yield-strength steel in a 4-inch thickness. Designers are encouraged to compensate for the 3-inch thickness limitation by use of quenched-and-tempered steel, where the yield strength does not exceed 90 ksi. When steel of the A588 and A514 chemistry is roller quenched and then tempered at a temperature high enough to give a yield strength in the range of 60 to 90 ksi, the toughness should be markedly improved over as-rolled A588 or high-strength A514. In a recent bridge built for California, A588 in the quenched-and-tempered (Q&T) condition was allowed; in 2-inch-thick plate, the Charpy V-notch impact was specified at 30 ft-lb at +20°F (the LAST). The lowest values recorded in the mill test reports* for the Q&T A588 were about 150 ft-lb.

1.5.4 Plate toughness. There are two approaches to fracture control based on fracture toughness concepts. One is based on the prevention of crack INITIATION and the other is based on the prevention of catastrophic crack PROPAGATION. The AASHTO toughness specification is based on the assumption that a CVN-impact test 70°F above the LAST can be used to estimate plane-strain fracture toughness (K_{Ic}) at the LAST. A 15 ft-lb or higher value obtained at +40°F, for example, is supposed to assure elastic-plastic behavior, i.e., a considerable resistance to crack INITIATION, at service temperatures down to minus 30°F. Unfortunately, the problem in welded structures is not with crack initiating (becoming unstable) in the plate material but with weld pop-in cracks which run out into the surrounding plate at high velocity (high strain rate). The crack resistance in such a case is measured in terms of dynamic fracture toughness not static (or intermediate rate-1 sec.) K_{Ic} testing. The correlation between CVN-impact test values and dymanic fracture toughness is well documented (see Figure C1.5.4a), but if the specification is to be realistic, the CVN-impact testing has to be done at the LAST.

not all right.

Fabrication problems, including hot and cold weld cracking, low-toughness weld metal, microcracking in the HAZ, arc strikes, improperly made fillets and tack welds and yield-point-magnitude residual welding stresses, appear to be an almost unavoidable source of crack initiation in bridge fabrication. Recent (1970s)

---

*The steel was Japanese.

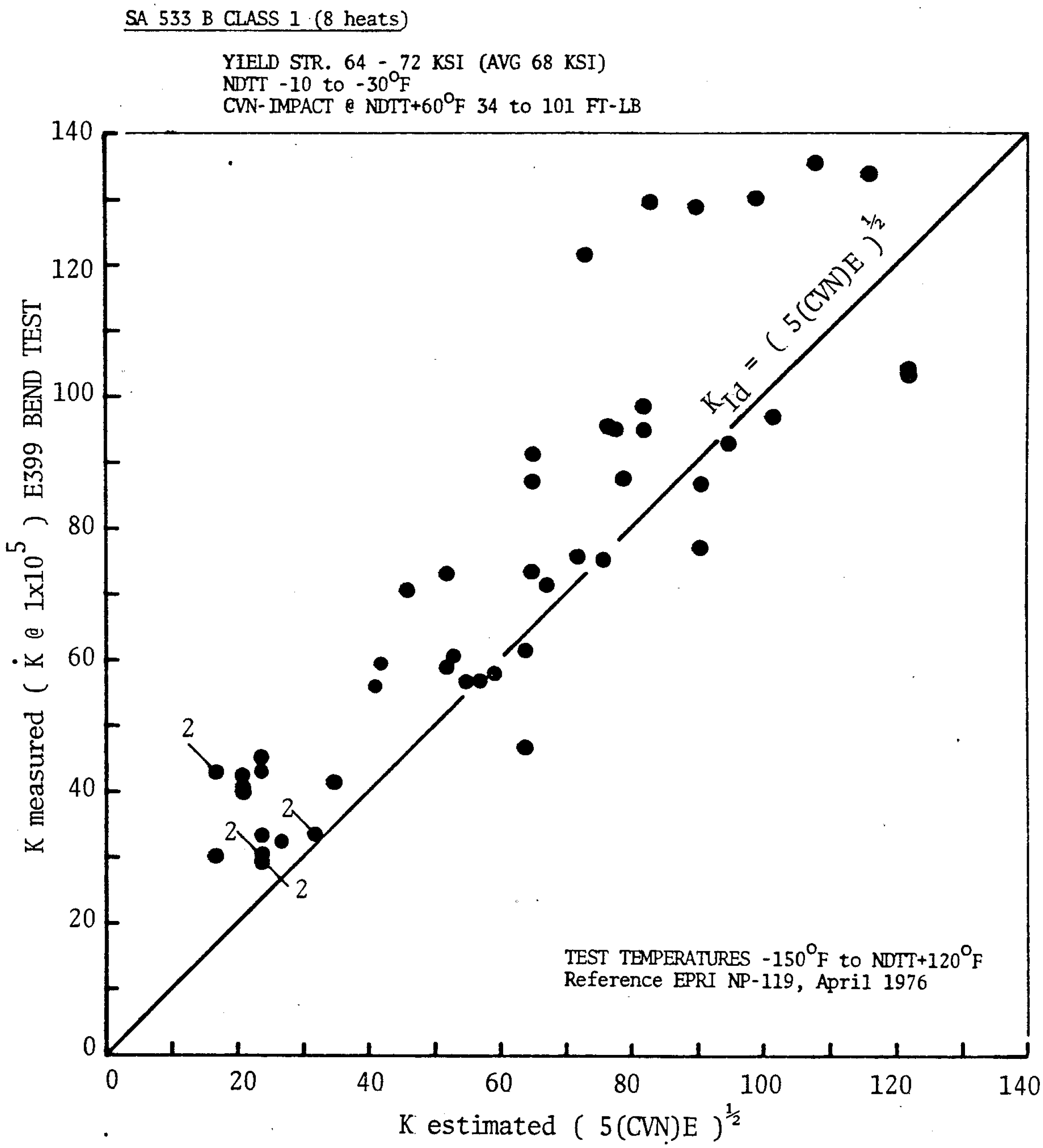


Figure C1.5.4.a - Relationship Between Charpy V-notch Impact and Dynamic K_{Id}.

inspections of bridges in service using radiography and/or ultrasonic testing makes this point incontrovertable. The current AASHTO toughness specification calls for Charpy impact testing but in concept is based on preventation of crack initiation.

Fracture control based on the prevention of catastrophic crack propagation required higher Charpy values than 15 ft-lb, requires testing at the lowest anticipated service temperature and requires taking into account both yield strength and thickness of the steel. The leak-before-burst concept proposed by Professor Irwin almost two decades ago for pressure vessels and later adopted by Rolfe, et at* as a through-thickness-yielding concept for off-shore platforms, provides a systematic, even if only approximate, method for taking yield strength and thickness into account in specifying an acceptable Charpy impact level. In contrast, the current AASHTO toughness specification allows some steels to be used up to 4-inches thick with only 15 ft-lb of Charpy V-notch impact. The requirements of paragraph 1.5.4 take thickness and yield strength into account based on the through-thickness-yielding concept. Testing is specified to be done at the LAST.

The Charpy V-notch impact rest is not an optimum test method as clearly shown by recent service-failure experience. The standard Charpy has too blunt a notch; this is corrected by use of the precrack Charpy impact test. Unfortunately the latter is not yet a standard ASTM test method; however, it is currently widely used by industries where its small size is an important advantage. The failure of the standard Charpy V-notch impact test in the Lafayette Street Bridge failure analysis is typical of those instances where the blunt 10-mil-radius V-notch caused the steel to be rated as tough when in fact it had low toughness in the presence of a natural crack. Plate from (1) the web of the fractured Fremont Bridge girder (Figure C1.5.4b), (2) a fractured sea-going barge (Figure C1.5.4C) and (3) a heat of replacement steel for the Bryte Bend Bridge repair (Figure C1.5.4d) provide additional examples of high Charpy V-notch impact values when in the presence of a fatigue crack the steel was demonstrated to be brittle. This deficiency in the standard Charpy V-notch impact test is not peculiar to type(s) of steel but rather to specify heats of steel which for reasons unknown are insensitive to the relatively blunt 10-mil radius of the standard V-notch and highly sensitive to a sharp fatigue crack. ?

The concept of through-thickness yielding based on Charpy testing has been in British Codes (BS153, BS449 and BS2573) for several years, as proposed by Professor A. A. Wells. When the precrack Charpy impact test method has been standarized as an ASTM test method, consideration should be given to substituting the precrack Charpy for the standard Charpy impact test. why? only your opinion

---

*Rolfe, Barsom and Gensamer, "Fracture-Toughness Requirements for Steels", Offshore Technology Conference, Houston, Texas, May 18-20, 1969, Paper OTC-1045.

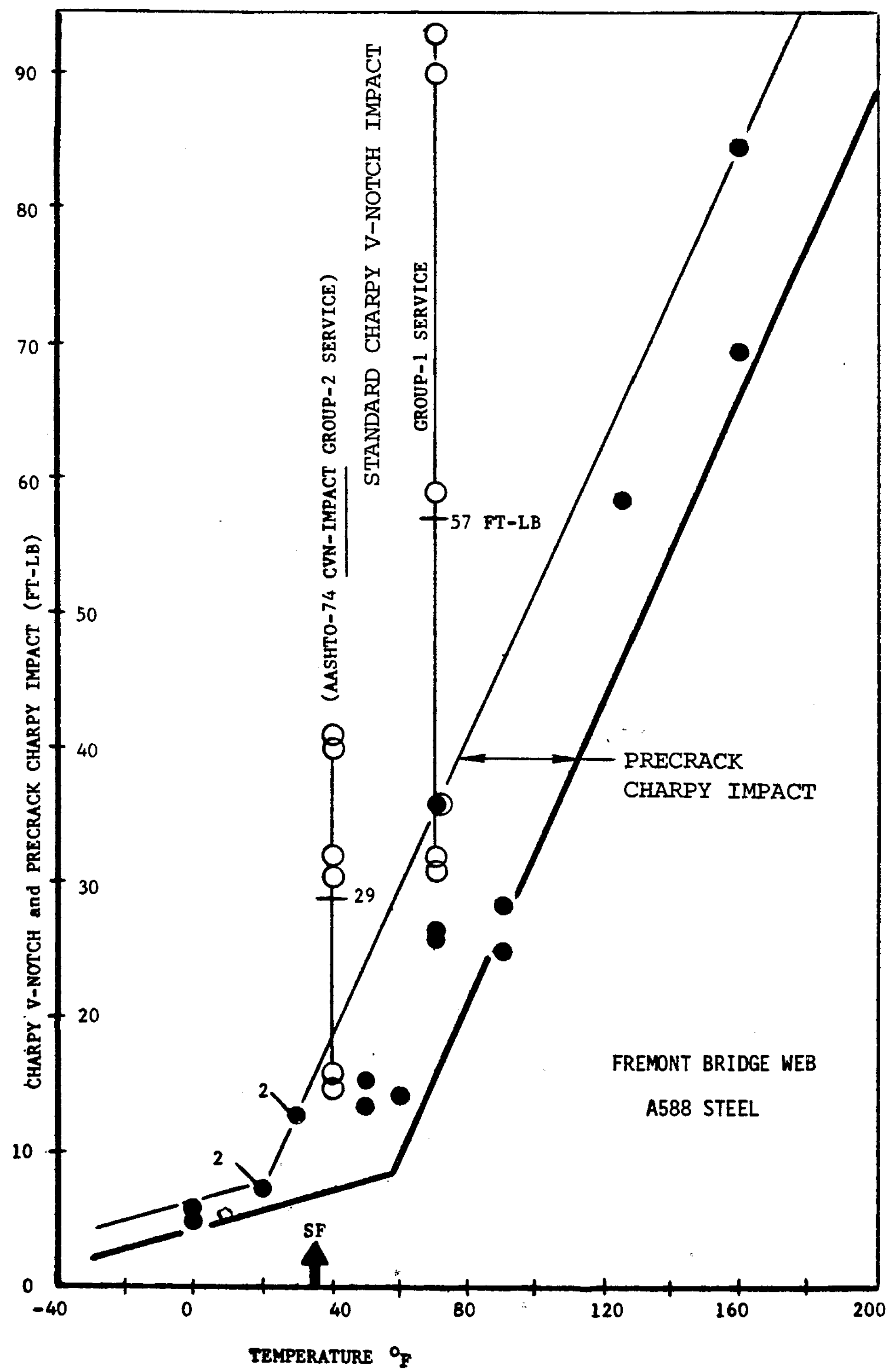


Figure C1.5.4.b - Charpy V-notch and Precrack Charpy Impact Data from the Fremont Bridge (Portland, Oregon).

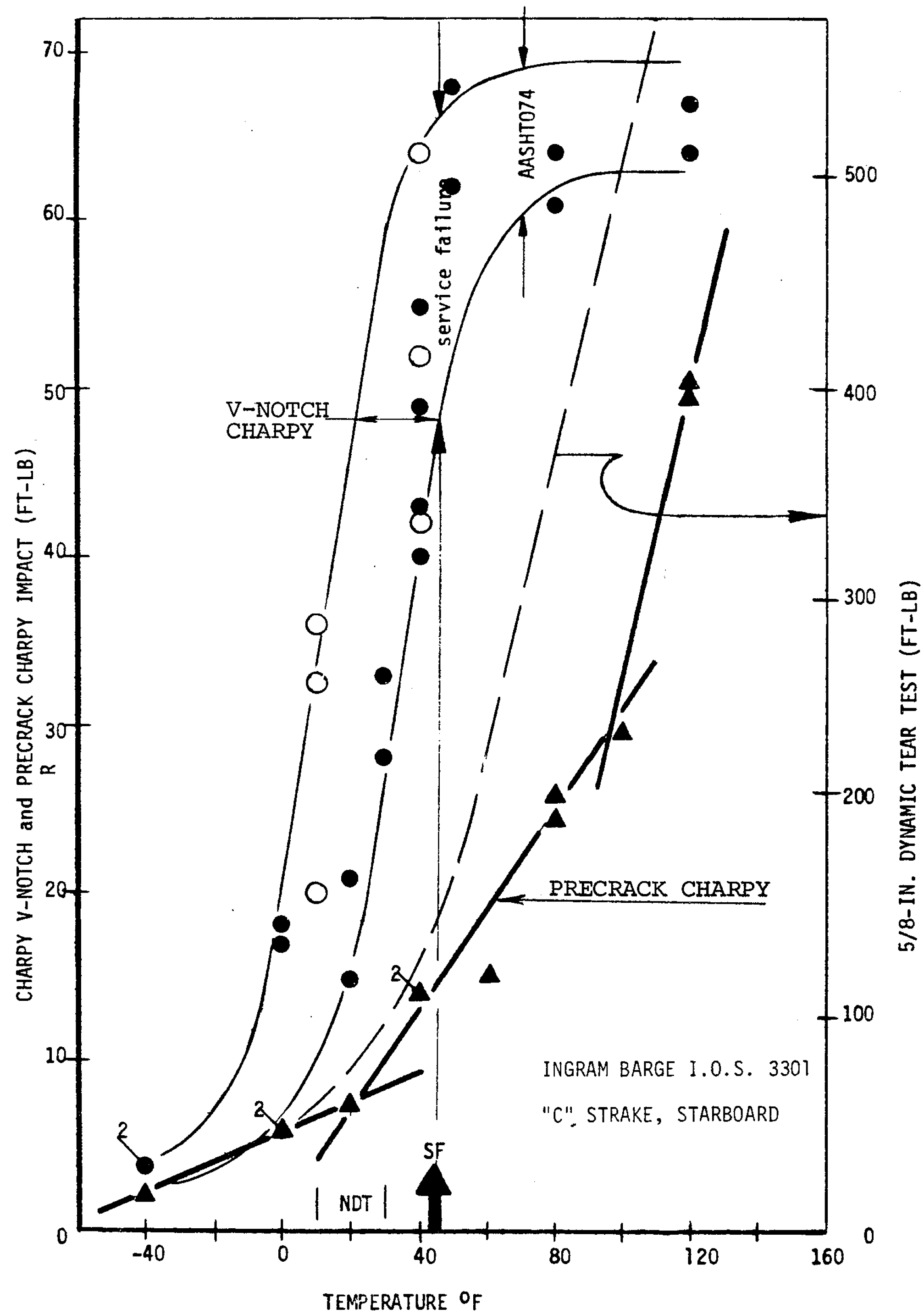


Figure C1.5.4.c - Charpy V-notch and Precrack Charpy Impact Data from a Fractured Ship.

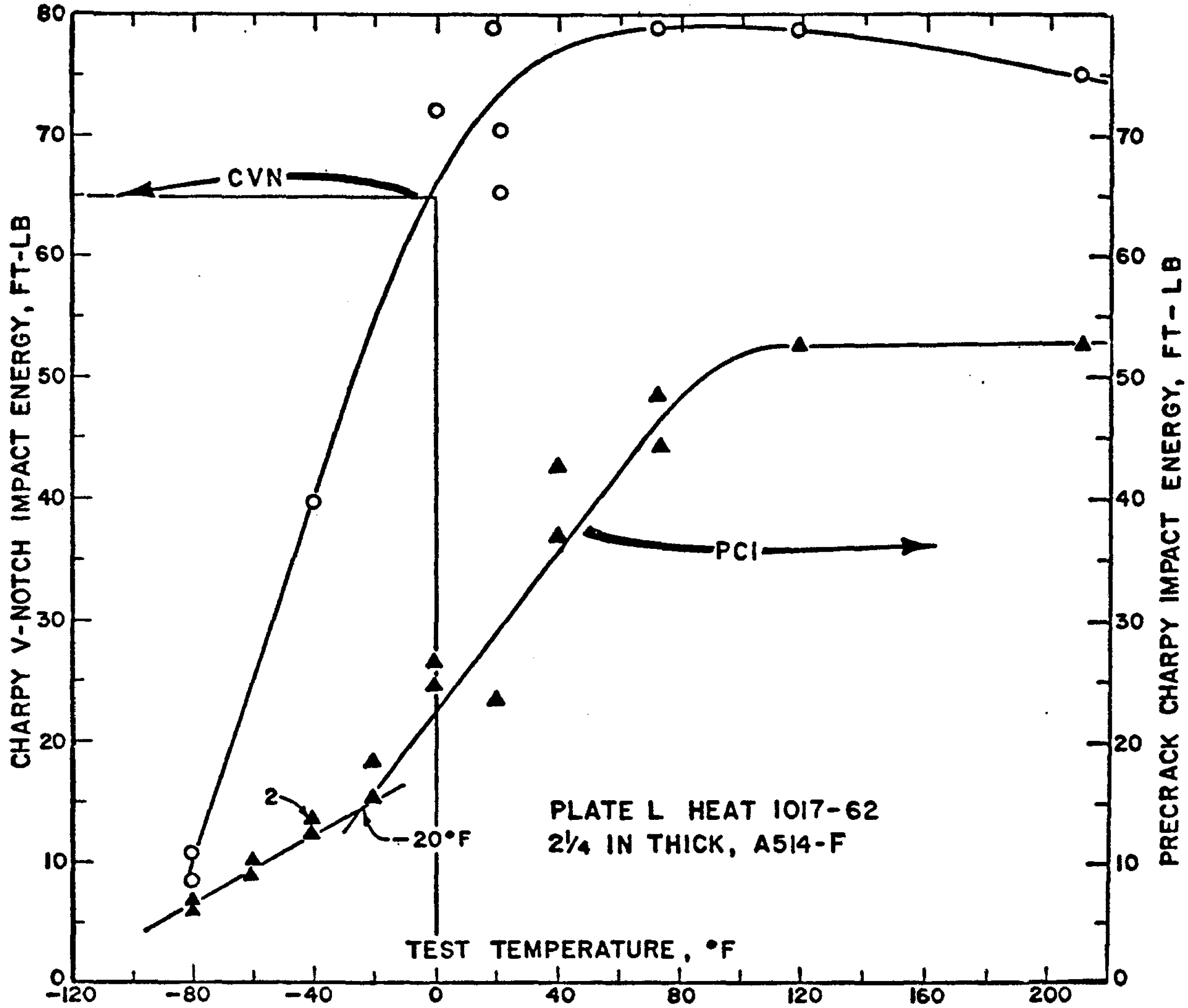


Figure C1.5.4.d - Charpy V-notch and Precrack Charpy Impact Data from Heat 97L168-06W2.

The upper-shelf CVN-K_{Ic} relationship as obtained by Barsom, Rolfe and Novak (ASTM STP466, 1970, pp. 281-302) for steels with yield strengths between 110 and 246 ksi has been found to be valid by Begley and Toolin for intermediate yield-strength Ni-Cr-Mo-V rotor steels (International Journal of Fracture, Vol. 9, 1973, pp. 243-253) and by Iwadate, et al for four 2 1/2 Cr-1 Mo steels with yields strengths of 59, 62, 63 and 85 ksi. ("Prediction of Fracture Toughness K_{Ic} of 2 1/4 Cr - 1 Mo Pressure Vessel Steels from Charpy V-Notch Test Results.") A theoretical explanation for the upper-shelf relationship has been given by Rice, Paris and Merkle ("Progress in Flaw Growth and Fracture Toughness Testing", ASTM STP536, 1973, pp. 231-245). Figure C1.5.4.e shows the remarkable relationship, now confirmed by a variety of steels of lower strength (Figure C1.5.4.f).

In Table 1.5.4 footnote a, the "upper-shelf" correlation is only valid when Charpy V-notch impact testing at the LAST provides data which are on or close to the upper shelf of the transition curve. The upper-shelf correlation as established by Rolfe and Novak

$$(K_{Ic}/FTY)^2 = 5\ (CVN-FTY/20)/FTY$$

when combined with the expression from Hahn and Rosenfield for through-thickness yielding

$$(K_{Ic}/KTY)^2/B = 1$$

results in the following relation for the minimum CVN-impact energy required to obtain through-thickness yielding before fracture

$$CVN = FTY(B+0.25)/5$$

Table 1.5.4 is based on this relationship between Charpy V-notch impact energy (CVN), yield strength (FTY), and thickness (B). When the CVN-impact test results are not on or close to the upper shelf (as demonstrated by 80 percent or more shear in the fracture surfaces), the transition-temperature correlation from Barsom should be used

$$K_{Ic}^2 = 5(CVN)E$$

When this equation is combined with the through-thickness yielding expression of Hahn and Rosenfield, the following equation is obtained

$$CVN = FTY^2\ B/5E$$

Table C1.5.4.a is based on this relationship, and is independent of fracture appearance. Unfortunately most heats of domestic steel will not meet the requirements of Table C1.5.4.a. why ~~[illegible]~~ [illegible]

The plots of CVN data from four grades of ISO plate steel presented in the commentary on 1.5.1 show that the 80 percent shear requirement of Table 1.5.4 can be met down to minus 40°F in many of the steels,

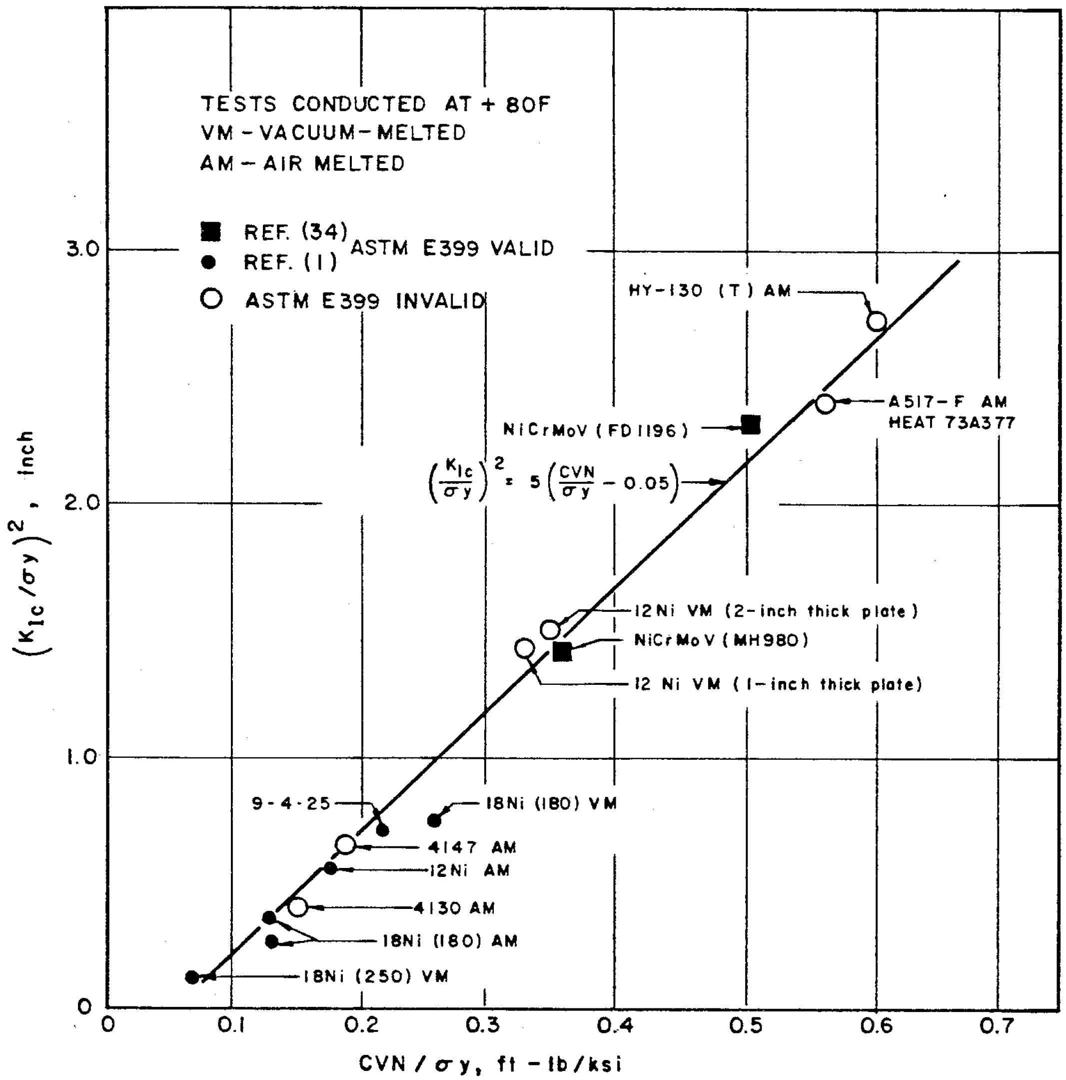


Figure C1.5.4.e - Relationship Between K_{Ic} and CVN in the Upper-Shelf Region (after Rolfe).

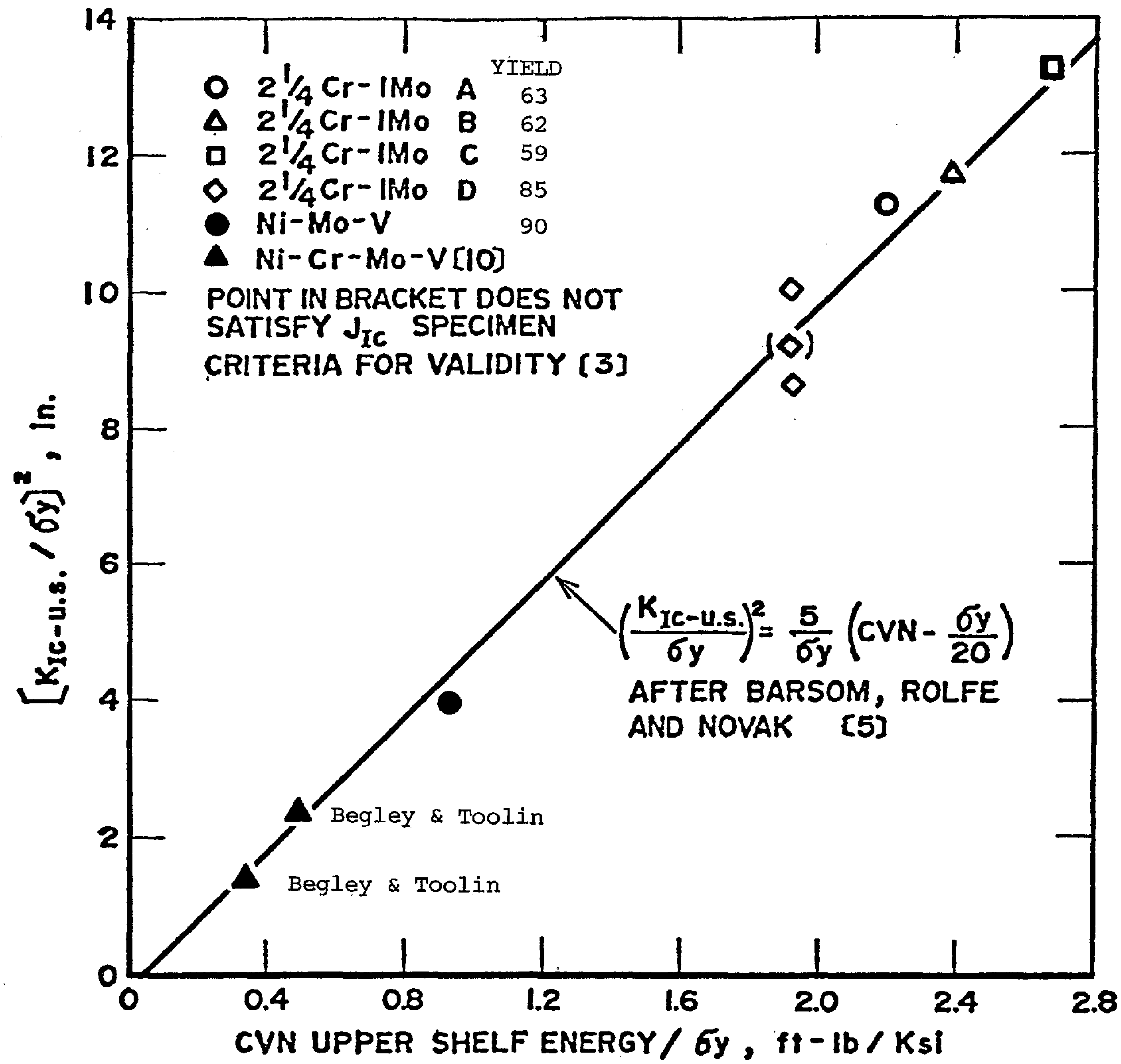


Figure C1.5.4.f - Relationship Between K_{Ic} and CVN in the Upper-Shelf Region (Iwadate, et al).

TABLE C1.5.4.a

BASE METAL[a]
CHARPY V-NOTCH IMPACT REQUIREMENT
for
FRACTURE-CRITICAL MEMBERS

YIELD[b] STRENGTH (KSI)	MINIMUM CVN-IMPACT[c] (FT-LB) AT THE LAST[d] for specified thickness ranges (in.)			
	up to 1 1/2	over 1 1/2 to 2	over 2 to 2 1/2	over 2 1/2 to 3
from 36 to 60	35	50	60	75
over 60 to 70	50	65	80	100
over 70 to 80	65	85	110	130
over 80 to 90	80	110	135	160
over 90 to 100	100	135	(e)	(e)
over 100 to 110	120	160	(e)	(e)
over 110 to 120	145	190	(e)	(e)

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.

and that the CVN-impact values of Table C1.5.4.a can be obtained down to minus 40°F in many of the steel plates analyzed. Because of the anticipated extreme difficulty in meeting the requirements of Table C1.5.4.4 using domestic steel, the transition-range CVN-K_{Ic} relationship was not used in the fracture control plan.

1.5.4.1 Lowest anticipated service temperature (the LAST). Over two decades of research and service experience indicate that the Charpy V-notch impact specimen tested at the service temperature provides a significant measurement which correlates with brittle-fracture experience in service. Furthermore, the static-to-dynamic transition-temperature shift upon which the AASHTO Materials Toughness Specification is based is unconservative on several counts; viz., (1) experience with brittle fracture in service has time and again correlated with CVN-impact testing at the temperature of the failure, (2) use of the shift is based on prevention of crack initiation and ignores the phenomenon of a pop-in crack which inherently involves high strain rate at the tip of the advancing crack, (3) the formula used in calculating the magnitude of the static-to-dynamic shift is unreliable - in some heats of steel the calculation estimates values that are too large and in others too small, and (4) the conservatism supposedly built into the AASHTO toughness specification in this regard is alleged to assure elastic-plastic behavior at a temperature 70°F below the temperature corresponding to the 15 ft-lb value (see Figure C1.5.4.1a). In the web of the Lafayette Street Bridge, brittle fracture initiating from fatigue cracking was cleavage and yet the Charpy value was more than twice the 15 ft-lb requirement.

Static/dynamic transition-temperature shift. An Office of Research (DOT) study of "Fracture Toughness of Bridge Steels - Phase II Report"* investigated several bridge steels. The research revealed instances where the predicted temperature shift based on

$$T_{shift} = 215 - 1.5\ FTY$$

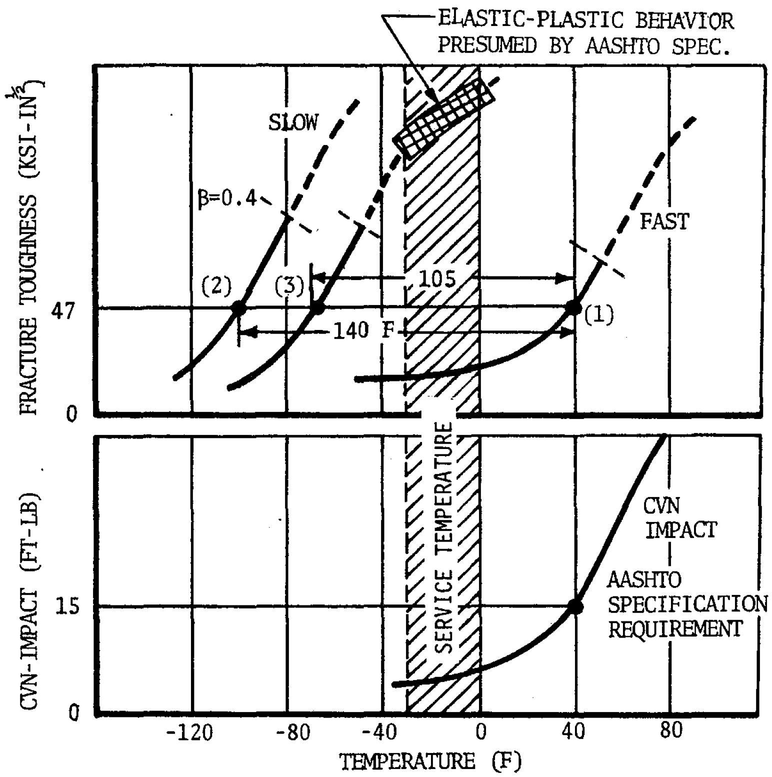
did not occur or was less than predicted. Consider, for example, the A441 data. Using standard Charpy V-notch specimens tested in slow bending and in impact

> "The shift was as expected for the 1/2" data. The 1" and 2" data exhibited a shift of about one-half than anticipated." (p. 227) (Emphasis added.)

Using precracked Charpy specimens tested in slow bending and impact, the situation was even less conservative.

---

*R. Roberts, G. R. Irwin, et al, Report No. FHWA-RD-74-59 under DOT-FH-11-7664, September 1974.



NOTES: (1) $K_{Id} = [5(CVN)E]^{\frac{1}{2}} = 47$ ksi-in.

(2) T(shift) = 215 - 1.5(FTY) = 140 F

(3) Intermediate Rate = 0.75 T(shift)

Figure C1.5.4.1.a - AASHTO-Toughness-Specification Concept of Static/Dynamic Temperature Shift

"The slow and fast precracked data showed zero shift for the three thicknesses. This was typical of results for all materials in the program. The slow and dynamic precracked data all showed definite temperature shifts but these shifts were less than Barsom's results." (p. 227) (Emphasis added.)

Table C1.5.4.1a summarizes the temperature-shift data obtained from standard V-notch and precrack Charpy testing of the several steels investigated in the FHWA research at Lehigh University.

Discrepancies in the temperature-shift calculation were not only in Charpy testing. When A441 E399-type bend specimens were tested under slow and rapid loading, the test results again indicated the temperature shift to be unpredictable.

"It is particularly interesting to note that for the A441 material the static and dynamic K_C results do show temperature shifts, which are not consistent with Barsom's prediction." (Emphasis added.)

Other steels tested with the E399-type bend specimens tended to show shifts that were consistent with Barsom's prediction (even when the Charpy data did not show a shift). However, this is not reassuring because much of the work done by Barsom in establishing the shift (ASTM STP466, March 1970) was based on standard V-notch and precrack Charpy testing.

The static/dynamic transition-temperature shift, which is the basis of the present AASHTO toughness specification test temperatures, is a phenomenon that has long been recognized by materials scientists. However, the varagies and complexities of the phenomenon are well documented. In 1970, a paper published on "Materials Sensitive to Slow Rates of Straining"* showed that some materials are actually embrittled by slow rates of loading. In the same book Barsom and Rolfe (pp. 281-302) presented data from Charpy slow-bend and impact tests and E399-type slow-bend tests showing what appeared to be a predictable shift in transition temperature based on yield strength.

More recent measurements of the shift between Charpy impact and slow-bend transition curves and K_{IC} values at the lowest anticipated service temperature (the LAST) confirm the data of the

---

*Hartbower, C. E., IMPACT TESTING OF METALS, ASTM STP466, American Society for Testing and Materials, 1970, pp. 115-147.

TABLE C1.5.4.1a

RATE-OF-LOADING TRANSITION-TEMPERATURE SHIFT

MAT'L	THICK. (in.)	T_{CVN}	T_B (°F)	T_P (°F)
1035	2	0	148	---
A242	1/2	200	134	---
A441	1	60	130	40
A441	2	65	130	40
A588B	2	85	121	135
A514M	1/2	110	24	0
A514P	1	200	52	0
A514M	2	95	55	105

T_{CVN}: Standard Charpy V-notch temperature shift at 15 ft-lb level between impact and slow bend (0.02 in./min. cross head speed).

T_B : Calculated shift based on 215 - 1.5 FTY where FTY is the room temperature yield strength.

T_P : Precrack Charpy temperature shift at 15 ft-lb level between impact and slow bend (0.02 in./min. cross-head speed).

Reference DOT-FH-11-7664 Contract Report No. FHWA-RD-74-59, Table 1 Data Summary, pp. 52-53, September 1974.

Lehigh study (FHWA-RD-74-59) showing that the phenomenon is not a reliable basis for a materials specification. The following example was selected from several similar examples because it is a case that is meticulously documented ty CALTRANS (Messrs. Nordlin and Kendrick). The steel is 3-inch-thick A36 with the following tensile properties:

	+70°F	+35°F
yield(ksi)	43.7	44.5
ultimate(ksi)	79.5	80.2
elong in 2"(%)	30.	-
RA(%)	62.	-

ASTM E399 compact tension tests were made at 32°F, and standard Charpy V-notch impact tests and precrack Charpy slow bend and impact tests were made over a range of temperature. The E399 compact-tension specimens had the dimensions B = 2 inches and W = 4 inches. Two loading rates were used, with duplicate tests at each rate. The test results are summarized in Table C1.5.4.1b.

From Figure C1.5.4.1b it will be seen that the shift in the precrack Charpy transition curves was no where near the shift predicted by the U. S. Steel formula. At 43.7 ksi yield strength, the shift should have been 150°F; whereas, the measured shift was only 50°F, even less than the allegedly conservative 70°F used by AASHTO.

From Table C1.5.4.1b note that there was no significant difference between the toughness measured at the two rates of loading. When toughness is estimated from the expression

$$K_{Ic}^2 = 5(CVN)E$$

using the CVN-impact value at +150°F (68 ft-lb in duplicate tests), the calculated toughness (101 ksi-in.$^{\frac{1}{2}}$) is much higher than the E399 measured values at +32°F. At 0°F (with a 150°F shift) the measured values would have been lower making the discrepancy even more pronounced.

AASHTO assumes a 70°F shift for purposes of CVN-impact testing. When toughness is calculated based on the CVN-impact value at 70°F (38 ft-lb), the calculated toughness is again unconservative in that 75 ksi-in.$^{\frac{1}{2}}$ is predicted when the measured values at 32°F are lower (and at 0°F, measured values would have been even lower). The shift concept appears even less conservative when it is noted that fracture in the 32°F compact tension tests INITIATED IN CLEAVAGE. The conservatism allegedly built into the AASHTO specification is supposed to assure elastic-plastic behavior at the LAST. (See Figure C1.5.4.1.a.)

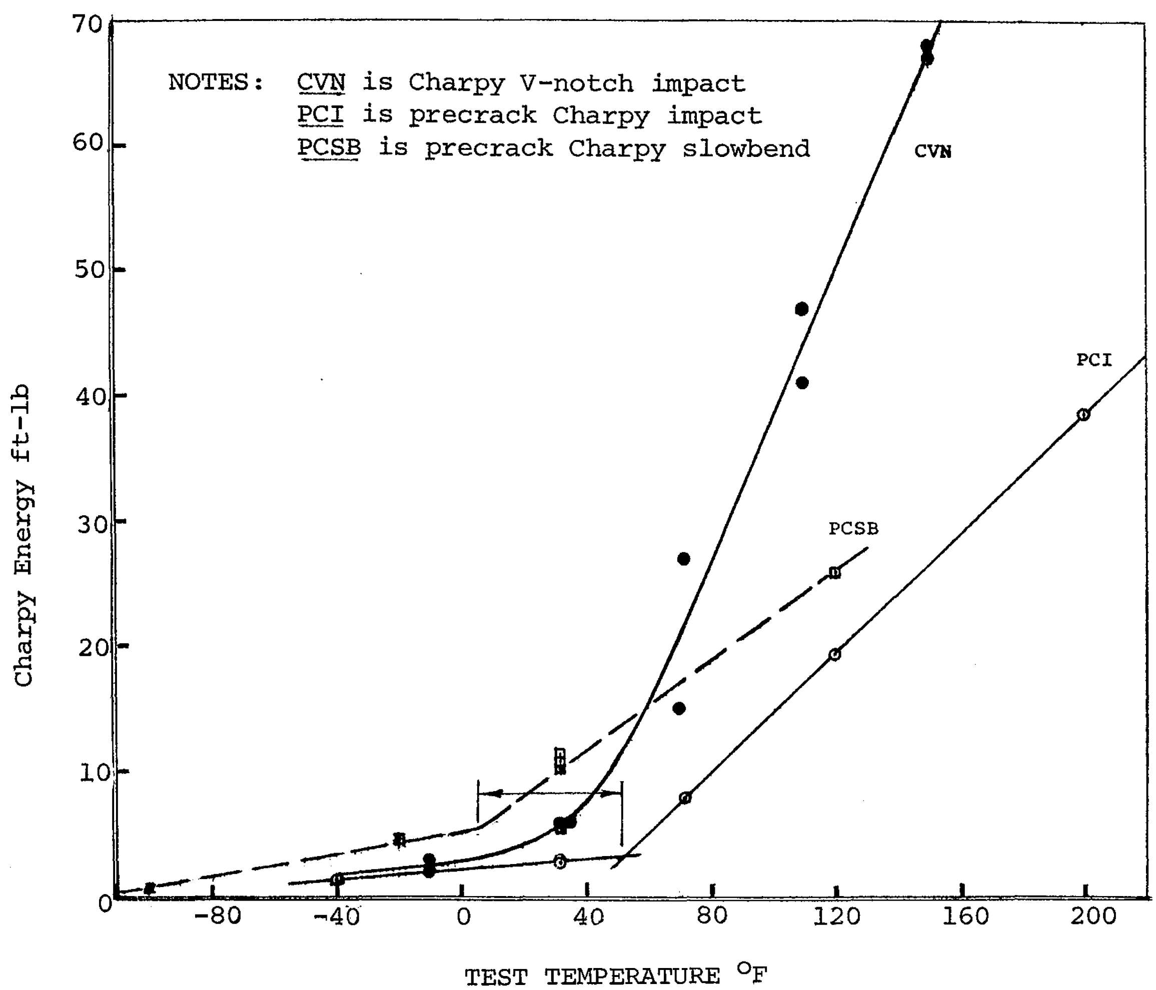


Figure C1.5.4.1.b - Use of Charpy Test Results to Determine Static/Dynamic Transition-Temperature Shift in A36 Steel Heat B (after Kendrick).

TABLE C1.5.4.1.b

E399 COMPACT-TENSION TEST RESULTS IN A36 STEEL

HEAT "B" TESTED AT +32°F

SPECIMEN No.	RATE (ksi-in.$^{1/2}$/sec)	$a_{avg.}$ (in.)	P_Q (kips)	P_{max} (kips)	K_Q ksi-in.$^{1/2}$	K_{max} ksi-in.$^{1/2}$	R_{sc}
B1	1.08	1.94	26.0	27.5	59.7	63.1	1.44
B2	1.11	1.98	22.5	28.3	53.1	66.8	1.55
B3	111.2	1.98	24.5	33.0	57.8	77.9	1.67
B4	110.5	1.97	20.8	20.8	48.8	48.8	1.04

It is highly interesting that the shift based on the precrack Charpy slow-bend (PCSB) and impact (PCI) transition curves can be used to predict the critical stress intensity K_{max} value of Table C1.5.4.1.b

$$\begin{aligned} \text{PCSB - PCI shift} &= 50^{\circ}F \\ 50 + 32^{\circ}F &= 80^{\circ}F \\ \text{CVN-impact @}80^{\circ}F &= 27 \text{ ft-lb} \\ [5(27)E]^{1/2} &= 64 \text{ ksi-in.}^{1/2} \\ K_{max} \text{ @}+32^{\circ}F &= 65 \text{ ksi-in.}^{1/2} \end{aligned}$$

Thus, the shift based on the precrack Charpy transition curves permits the calculation of K_{max} based on CVN-impact tests 50°F above the compact-tension test temperature. This confirms the AASHTO concept of a shift, but instead of 150°F as predicted from

$$T_{shift} = 215 - 1.5 \text{ FTY}$$

the shift was only 50°F. In some heats the shift has been found to be zero; i.e., there is no shift at all.

Service Failure Experience. World War II fractured Liberty ships and T-2 tankers provided a large amount of data which demonstrated a statistically significant correlation between the CVN-impact test and the in-service performance of World War II ship hulls WHEN TESTING was done at the TEMPERATURES AT WHICH THE FAILURE OCCURRED. And this observation holds in spite of the fact that a significant number of the ship failures occurred in still water; i.e., under static-load conditions (see Appendix A).

More recently, a study was made of service failures involving brittle fracture where Charpy V-notch (CVN) impact tests were made of the casualty plate over a range of test temperatures*. A comparison was made between CVN-impact values at the test temperature currently specified by AASHTO (70, 40 and/or 10°F) and the service temperature at which brittle fracture occurred. Case after case was found where, if the current AASHTO toughness specification had been in effect at the time, the steel would have been judged acceptable by AASHTO when the service experience showed the steel to be brittle. With few exceptions, CVN-impact tests at the service-failure temperature revealed low-toughness, generally less than 15 ft-lb.

The cases of brittle-fracture in service where CVN-impact test data were available on the casualty heat of steel included 11 bridge failures, 9 structural failures, 3 ship failures and 7 storage-tank and pressure-vessel failures. Seven of the thirty cases involved steel that would have been rejected by AASHTO; twenty-three (23) would have been found to be acceptable steel. Since 1974, when the above survey was made, there have been additional service failures where the AASHTO toughness specification proved to be unconservative:

(1) The Lafayette Street Bridge in St. Paul, Minnesota developed brittle (cleavage) fracture in the web and flange plates, starting from a defective gusset-to-stiffener connection (Winter 1974-75). The CVN-impact value at +40°F was over 30 ft-lb in the web steel; at the estimated service temperature the value was less than 10 ft-lb. Worse, fractography in other cracked areas in the bridge revealed** cleavage initiation when AASHTO is supposed to assure elastic-plastic behavior.

(2) A large crane for off-loading ships in Elizabeth, New Jersey, developed brittle fracture in the crane boom (December 1974). The CVN-impact value 70°F above the service temperature was 15 ft-lb; at the service temperature the value was less than 10 ft-lb. Again the brittle fracture involved cleavage initiation.

(3) Three consecutive rigid-frame bents of a railroad bridge developed brittle fracture (January 1978). The steel met the AASHTO toughness

---

*C. E. Hartbower, "The AASHTO Charpy V-Notch Impact Specification for Bridge Steels", (150 pages), June 1974.

**J. W. Fisher, A. W. Pense and R. Roberts "Investigation and Analysis of the Fractured Girder in Bridge No. 9800, T. H. No. 56 over the Mississippi River in St. Paul, Minnesota, October 1974, also Journal of Structural Division - ASCE, July 1977, pp. 1339-1357.

specification with values of 25 ft-lb and higher at +40°F. At +10°F, the fractured web plates gave values of less than 10 ft-lb.

In each case cited above, there were obvious explanations for why the initial cracking occurred, but some important questions are unanswered. What was the critical crack size (fracture toughness) in the steel at the service temperature? Was the critical crack size large enough so that there was a chance of finding the crack before it triggered brittle fracture? If the basic concept underlying the AASHTO toughness specification is valid (elastic-plastic behavior at the lowest anticipated service temperature) why did some of the service failures initiate in cleavage?

With this history of brittle fractures in service and the many unanswered questions in connection with the current AASHTO toughness specification, it is concluded that the present AASHTO CVN-impact specification will not assure the designer (and the public) of fracture-safe bridges. Both the temperature adjustment allowed by AASHTO and the specified energy levels are unconservative.

In specifying service temperature, the present AASHTO toughness specification does not cover the method of determining service temperature; thus, AASHTO does not specify whether the lowest service temperature will be based on weather-bureau records or someone's recollection of weather in recent years. For FCMs, such varagies cannot be tolerated. To correct this situation the FHWA contracted with the U. S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Data Center, Asheville, North Carolina, to produce isoline maps based on computer-banked data.

The first percentile minimum temperature of Figure 1.5.4.1 corresponds to a 1 percent chance of a lower daily minimum temperature occurring, or a 99 percent chance that the daily minimum will be no lower than the indicated value. Thus, this temperature can be equalled or exceeded once in 100 years on the average.

Figure C1.5.4.1.c for comparison, is an isoline map showing the lowest hourly temperatures recorded during the years 1950 to 1970 at about 340 locations throughout the United States. The circled numbers are the lowest temperatures ever recorded within each State based on records dating back to 1871. The maps were prepared for FHWA by Dr. D. M. Whiting, Meteorologist, National Climatic Center, Asheville, North Carolina. Tabulated minimum temperatures (of record) have been published in WEATHERWISE, December 1963 (see Figure C1.5.4.1.d).

EXTREME LOW HOURLY TEMPERATURES DURING THE WINTER SEASON WITH ABSOLUTE MINIMUMS FOR EACH STATE.

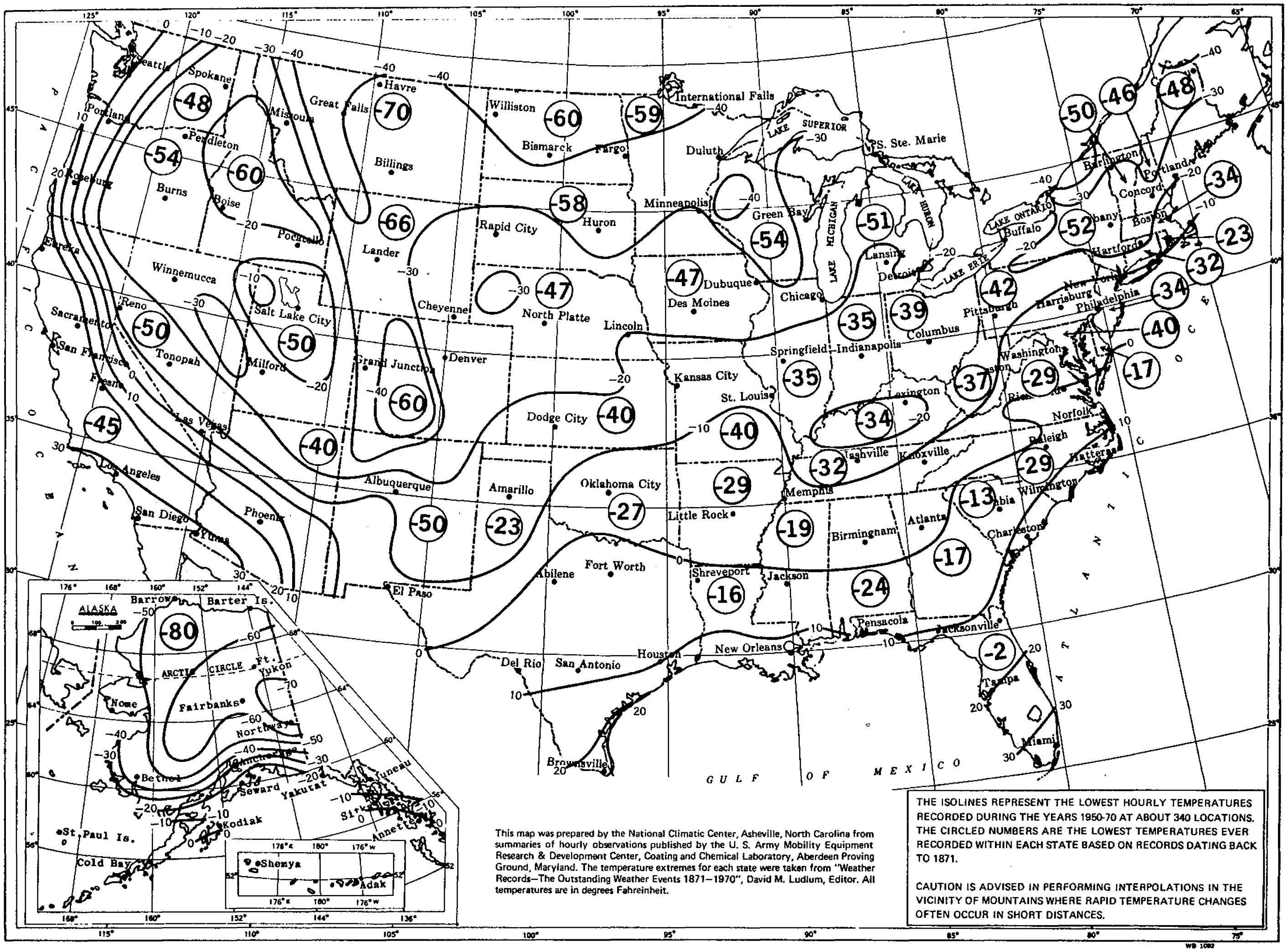


Figure C1.5.4.1.c - Isolines for Two-Decade Lowest-of-Record Temperatures.

TABLE III—ABSOLUTE MONTHLY MINIMUM TEMPERATURE FOR CITIES—°F—(Continued)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
LOUISIANA												
Alexandria 1893	4	3	21	32	39	50	59	55	40	27	19	10
Baton Rouge 1893	10	2	23	31	40	54	61	60	43	31	22	13
Lake Charles 1893–	12	3	21	32	40	51	60	58	45	32	20	16
New Orleans 1871–	13	7	28	38	50	58	66	63	54	40	29	18
Shreveport 1875–	−2	−5	15	31	42	53	60	54	43	29	18	10
MAINE												
Caribou 1939–	−32	−41	−20	2	19	30	40	34	23	14	−2	−24
Portland 1872–	−21	−39	−21	9	23	33	40	38	23	18	−6	−21
MARYLAND												
Baltimore 1871–	−6	−7	5	15	34	46	54	51	39	30	12	−3
Frederick 1942–	−10	−11	−1	20	28	41	42	43	28	22	11	−13
MASSACHUSETTS												
Blue Hill, Milton 1886–	−16	−25	−5	6	27	36	46	41	28	21	5	−19
Boston 1872–	−13	−18	−8	11	31	41	50	46	34	25	−2	−17
Nantucket 1887–	−4	−6	6	15	30	39	47	49	35	29	15	−3
Worcester 1901–	−19	−24	−6	9	27	35	41	38	27	19	3	−17
MICHIGAN												
Alpena 1873–	−28	−36	−27	−2	21	27	34	29	23	12	−6	−15
Detroit 1871–	−16	−20	−7	8	28	38	48	43	30	22	0	−24
Escanaba 1871–	−29	−32	−27	−1	20	29	41	34	24	10	−9	−20
Grand Rapids 1892–	−22	−24	−13	13	21	31	42	40	29	20	−10	−11
Lansing 1910–	−17	−25	−10	8	25	34	42	38	29	19	−5	−15
Marquette 1874–	−26	−27	−16	3	22	31	38	33	28	12	−9	−20
Muskegon 1897–	−21	−30	−11	9	22	34	40	40	28	19	−14	−4
Sault Ste. Marie 1889–	−32	−37	−27	−13	21	31	36	32	25	15	−12	−24
MINNESOTA												
Duluth 1874–	−41	−36	−28	−5	16	30	37	37	22	8	−29	−35
Int. Falls 1892–	−49	−46	−38	−14	8	27	31	20	20	7	−27	−41
Minneapolis 1891–	−34	−33	−32	2	22	34	44	40	26	10	−13	−27
Rochester 1887–	−42	−39	−31	6	21	31	40	32	22	−6	−24	−31
St. Cloud 1895	−42	−35	−32	2	18	33	41	34	18	6	−23	−32
MISSISSIPPI												
Jackson 1896–	−5	1	17	31	39	48	57	54	41	26	15	8
Meridian 1890–	−7	−6	19	29	38	45	55	49	39	24	16	4
Vicksburg 1873–	2	−1	17	31	43	52	59	54	41	31	20	9
MISSOURI												
Columbia 1890–	−23	−20	−9	14	28	42	45	40	26	19	−3	−23
Kansas City 1889–	−20	−22	−3	16	27	44	53	46	34	17	4	−13
St. Joseph 1910–	−24	−16	−11	13	30	43	50	44	32	11	4	−13
St. Louis 1871–	−22	−18	−5	21	32	44	51	50	32	21	3	−15
Springfield 1888–	−19	−29	−8	16	29	44	51	44	30	19	4	−11
MONTANA												
Billings 1894–	−39	−49	−34	−5	14	26	37	28	18	−11	−22	−38
Great Falls 1893–	−44	−49	−32	−10	15	31	35	34	10	−10	−25	−38
Havre 1880–	−57	−55	−41	−11	14	29	31	27	18	−16	−33	−50

Figure C1.5.4.1.d - Sample Table of Lowest-of-Record Temperatures for Cities of the USA.

If it is desired to place steels in categories corresponding to the "zones" of the current AASHTO toughness specification, this can be done providing the testing is done at a temperature no higher than the LAST. Thus, AASHTO "zone-1" steels would be tested at 0°F and thereby would be qualified for service at 0°F and above; "zone-2" steels would be tested at -30°F and thereby qualified for service between -30°F and -1°F, and higher; and "zone-3" steels would be tested at -60°F and thereby qualified for service between -60 and -31°F, and higher.

1.5.4.3 Nil-ductility transition (NDT) temperature. This is an ASTM (E208) standard method, listed as a Supplementary Requirement in several ASTM Standard Specifications (see 1.5.1). The 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. Furthermore, with the requirement that the LAST be no lower than 30°F above the NDT temperature, one can expect some degree of crack-arrest capability.

The use of NDT temperature as a supplementary acceptance criterion for steel is not new - it is in fact used by other industries and has been recommended by Professor Rolfe for use in the shipbuilding industry (reference "Fracture-Control Guidelines for Welded Steel Ship Hulls", SSC-244, November 1974):

> "....the primary material specification in an overall fracture-control plan for welded steel ship hulls is that all steels and weldments used in primary load-carrying plate members in the main stress regions of ships have a maximum NDT of 0°F as measured by ASTM Test Method E208-69."
>
> "Although necessary, this primary NDT requirement alone is not sufficient, since an additional toughness requirement is necessary to insure that the resistance to fracture of the steels and weldments whose NDT is 0°F (or lower) is actually satisfactory at +32°F."

The AASHTO specification calling for Charpy V-notch impact test values of 15 ft-lb 70°F above the LAST is tantamount to allowing service at NDT minus 70°F. This observation presumes the NDTT to occur at about 15 ft-lb; however, for fully killed and micro-alloyed steels, the Charpy impact value corresponding to the NDTT can be 25 ft-lb and higher which makes the AASHTO Specification even less conservative. "The AASHTO specification which allows the LAST to be 70°F below the NDT or lower is contradictory to other industries such as offshore platforms, ships, pipelines, etc., where the LAST is usually never less than 30°F <u>above</u> the NDT."*

<u>Effect of tramp elements.</u> In the ASTM A533-76 Standard Specification for PRESSURE VESSEL PLATES, ALLOY STEEL, QUENCHED AND TEMPERED MANGANESE-MOLYBDENUM AND MANGANESE-MOLYBDENUM-NICKEL, ther is provision for restricting residual (tramp) elements over what is considered normal for domestic steel; viz.,

ELEMENT	NORMAL CHEMISTRY MAXIMUM	COMMERCIAL LIMIT HEAT ANALYSIS	PRODUCT ANALYSIS
SULFUR	0.040	0.015	0.018
PHOSPHORUS	0.035	0.012	0.015

ASTM A588-75 allows even higher values for most grades: sulfur 0.05 maximum and phosphorus 0.04 maximum. In discussion of the role of residual elements, ASTM A533-76 points out that

> Reactor design requires review and control of residual elements that affect the material properties... Vanadium and sulfur can affect the upper energy shelf level. In the case of sulfur, control of this element or its morphology in the plate, or both, may offer alternative means of control.

The reference here to "upper energy shelf level" refers to Charpy V-notch impact testing. From Figure C1.5.1 note that the Japanese have the tramp elements down to levels of about 0.010 sulfur and 0.020 phosphorus. The excellent upper-shelf energy values and low transition temperatures in their steels make our domestic steels look very poor.

The limits presently allowed by ASTM are unreasonably high from the standpoint of the user. Actually no self-respecting metallargist would ever produce steel at the sulfur limits allowed by ASTM. Clearly it is time the limits were lowered, or the steel for FCMs special ordered to the commercial limits revealed in ASTM A533-76. The commercial limits cited above are not peculiar to the A533 chemistry; A588 or A514, for example, could just as well be ordered to a lower sulfur limit. U. S. Steel research has shown that as phosphorus content increases above 0.006 percent, stress-relief (temper) embrittlement occurs in some steel (see 4.1.1.2.3).

---

*Louis Raymond, "Toughness Requirements for Bridge Steel", ASM WESTEC Conference '78, Los Angeles, California, March 21, 1978.

2. DESIGN

2.1 Drawings. The requirement that all welds to be made in FCMs be shown on the drawings is an essential control. Too often "cosmetic" welds and "temporary" welds used during fabrication, in shipping and/or during erection are deposited without regard for the controls that are supposed to be exercised under AWS D1.1. Under the provisions of AWS D1.1, as amended, any weld made that is not shown on the shop drawings or erection plans could be cause for rejection of the member.

2.5 Partial-joint-penetration welds. The unfused land of a partial-penetration weld becomes a crack starter (initiator) and, likewise the toe and root of a fillet weld are prone to develop microcracking in the weld heat-affected zone and the root can initiate cracking in the center of the weld deposit, which in turn may initiate slow crack growth.

2.8 Allowable stresses.

2.8.2 Fatigue stress provisions (see AWS D1.1 paragraph 9.4). Fatigue (cyclic loading) is one of the most likely causes of slow crack growth. Many papers have been published pointing up the seriousness of this problem. Dr. Weck, former Director General of the British Welding Institute (April 1976, Metal Progress, pp. 26-43) and more recently a paper coauthored by several experts (April 1978, Civil Engineering - ASCE, pp. 70-73) explain the role of weld defects in initiating premature fatigue cracking. Professor John W. Fisher has authored on behalf of the American Institute of Steel Construction a Guide to 1974 AASHTO Fatigue Specifications and in 1977 a Bridge Fatigue Guide (Design and Details). But with all this, there is more that needs to be said.

Researchers in the aerospace industry, also facing fatigue problems, have shown that when the toughness is low, the stress ratio high and the stress intensity (mean stress) high, limiting the stress range is only a partial solution to the problem. A recent survey of the published literature on this subject led to the following observations*:

> Numerous references show conclusively that stress ratio (mean stress) and fracture toughness (critical crack size) can be important considerations in determining fatigue life. When the maximum applied stress in combination with the crack dimensions result in a maximum stress intensity approaching the critical stress intensity (K_{Ic} or K_c), there is an acceleration in the fatigue crack growth rate, and in low-toughness steels catastrophic crack propagation can occur at a low life cycle.

---

*C. E. Hartbower, "Effect of Toughness on the Fatigue Life of Fracture Critical Bridge Members", (43 pages), June 1976.

> The lowest level of toughness that can be tolerated in a bridge component from the standpoint of fatigue life depends upon the largest flaw that can escape detection in nondestructive inspection. If the flaw which escapes detection is large and the toughness is low, the life of the bridge component may be severely limited. To quantify the toughness requirement in terms of fatigue life, the Paris-Erdogen crack-growth-rate expression and the Forman-Kearney-Engle crack-growth-rate expression have been integrated by computer between the limits of a_o (the assumed initial crack size) and a_{cr} (the critical crack size based on the assumed fracture-toughness). The integration procedure demonstrated that with a flange edge-cracked to a depth of the order of 0.3 inch or deeper, the toughness levels presently allowed by the American Association of State Highway and Transportation Officials (AASHTO) can severely limit the life of a bridge member component.

Table C2.8.2 illustrates the magnitude of the problem when the largest undetected flaw is of the order of 1/2 inch and the toughness is of the order presently allowed by AASHTO.

TABLE C2.8.2

PREDICTED LIFE IN A 30-INCH WIDE FLANGE AS A FUNCTION OF EDGE CRACK DEPTH, FRACTURE TOUGHNESS AND STRESS RANGE

		MAXIMUM STRESS 39 KSI			
		STRESS RANGE 6 KSI		STRESS RANGE 12 KSI	
Initial Flaw Size	Fracture Toughness	Critical Crack	Cycles to Failure	Critical Crack	Cycles to Failure
0.10	55	0.503	1,117,403	0.503	234,905
0.30	55	0.503	333,554	0.503	70,121
0.50	55	0.503	3,881	0.503	816
0.10	70	0.812	1,406,941	0.812	295,776
0.30	70	0.812	623,109	0.812	130,992
0.50	70	0.812	293,436	0.812	61,687
0.10	100	1.617	1,789,429	1.617	376,190
0.30	100	1.617	1,005,633	1.617	211,406
0.50	100	1.617	675,960	1.617	142,101

The lower fracture toughness values selected for the fatigue-life estimations in the above Table correspond to the Charpy values specified in the present AASHTO Specification.

> "SUMMARY*
> 7. Conservative engineering estimates of K_{Ic} values in the toughness-transition region can be predicted by using the relationship
>
> $K_{Ic}^2/E = 5(CVN)$"

CVN	K_{Ic}
15	47
20	55
25	61
30	67
35	72
67	100

AWS D1.1-Rev. 2-77, Table 9.25.3 Ultrasonic Acceptance Criteria, <u>without</u> revision by AASHTO, specifies that:

> "Any discontinuity <u>longer than 3/4 inch (19 mm)</u> having a more serious rating (smaller number) than this (SMALL REFLECTOR) level shall be rejected.
>
> Any discontinuity <u>longer than 2 inches (51 mm)</u> having a more serious rating (smaller number) than this (MINOR REFLECTOR) level shall be rejected.
>
> Discontinuities which have a more serious rating than those of 'Minor Reflectors' and which have a <u>length greater than 3/4 inch (19 mm) and less than 2 inches (51 mm)</u> are permitted in the middle half of the weld thickness." (emphasis added)

Thus, AWS/AASHTO permit UT indications of length up to 3/4 inch when in the category of "Small Reflector" and up to 2 inches when in the category of "Minor Reflector". Unfortunately, the determination of length by UT techniques is at present not very quantitative and, furthermore, the severity rating (in dbs) is strongly dependent upon the relationship between the transducer angle and crack

---

*"AISI Project 168 TOUGHNESS CRITERIA FOR STRUCTURAL STEELS-Investigation of Toughness Criteria for Bridge Steels".

orientation. Furthermore, the inspector must exercise skill and diligence in determining the maximum reflected signal to assess whether a given discontinuity is in the "large", "small" or "minor" category.

With planar "discontinuities" permitted by AWS/AASHTO in the range of 3/4 to 2 inches long while allowing marginal toughness, it is not surprising that the fatigue-crack-growth rate is high in some bridges.

It is necessary to add at this point in the commentary that, in spite of its limitations, UT is the only method that we have to determine the "2D" dimensions of a discontinuity. Radiography may show a linear discontinuity of length "L"; ultrasonic testing may show the line to be in fact a planar defect of dimensions Lxd. From the standpoint of linear elastic fracture mechanics it is very important to have some idea of the depth of a crack.

2.10 Prohibited Types of Joints and Welds.

2.10.1.2 Elsewhere in AWS D1.1, as amended, the use of steel backing is prohibited for groove-welds perpendicular to the applied stress, unless the backing is removed, the weld back-gouged and welding completed from the backside of the joint. Also, the use of fillet welds to hold the backing in place are prohibited when the fillet welds are outside the groove-weld joint. The interface of a backup bar and the plate being joined is a source of microcracking and stress concentration that can be expected to initiate slow crack growth.

NOTE: By FHWA Notice N5040.23 of February 10, 1977, electroslag welding is prohibited for use in main structural tension members in Federal-aid projects as a result of an ES-weld fracture in a bridge near Pittsburgh, Pennsylvania.

3. WORKMANSHIP

3.1 General

3.1.6 Cosmetic welding is defined for FCMs as touch-up corrections of such workmanship deficiencies as weld undercut, underfill, surface porosity, and craters as seen by visual inspection after the weld has been allowed to cool below the specified preheat/interpass temperature.

An alert welder and/or the welding inspector should detect most workmanship deficiencies before the weld drops below the specified preheat/interpass temperature. Minor corrections at this time are but an extention of the welding operation. However, frequent corrections of this nature may be construed as a departure from the AWS D1.1 paragraph 5.5, as amended.

3.2.1.1 The requirement that no mill scale shall remain in the boundary of a groove weld is in addition to the requirement that all mill scale shall be removed from the surfaces on which flange-to-web welds are to be made by any of the approved welding processes.

3.7 Weld Repairs. (The provisions of section 3.7 have been extracted from the New York State STEEL CONSTRUCTION MANUAL as revised August 1, 1974 and February 2, 1976 with minor revisions as deemed necessary for FCMs.)*

Recent (1970's) experience with cracked bridges indicates that weld repairs are one of the principle causes of weld (bridge) cracking. The repair of a weld is almost always done under high restraint and too often done without strict compliance with the AWS/AASHTO Code. The ensuing cracking often results in the repair of a repair; sometimes the cracking of a repair is not detected. Visual inspection of a repair is never sufficient; because a repair is inherently made under adverse conditions (high restraint, etc.), repairs should receive extraordinary nondestructive inspection. When the crack necessitating the repair is not completely excavated, the repair weld is almost certain to be cracked. Complete records of repairs in FCMs are essential because the repaired area should be given special attention when inspections are made after the bridge is in service.

Recent experience with cracking indicates that plate-edge cracking and weld-end cracking are a frequent source of trouble. Moreover, fracture mechanics shows that an edge crack has a high stress-intensity factor. Also recent experience reinforces the concern over laminations that

---

*Warren Alexander, private communication.

intersect plate edges that are to be groove-weld butt spliced; a plate lamination can propagate in a butt splice as a weld-longitudinal crack. A weld repair of a lamination, if the lamination is not completely removed, may develop a crack as a result of micro-cracking at the weld-lamination interface.

Research has shown that delayed cracking can continue to occur several days after welding is completed with no load other than the inherent residual welding stress. Acoustic emission (AE) occurring during crack growth is a phenomenon that is well documented and now given extensive use in other industries as a means to not only detect the occurrence of cracking but also to locate the point at which the cracking is occurring. The technology is available (and has been for several years) to determine (1) if a given weld is suffering delayed cracking and (2) when the cracking stops. One of the earliest applications of AE in the study of weld cracking was published in the WELDING JOURNAL in 1966[1]; a more recent study was published by the American Society for Testing and Materials in 1972[2].

It is important to know when cracking has stopped, otherwise, inspection may be made at a time when the crack is too small to be detected, or too small to be rejectable, and later grow to a near-critical size.

3.7.2 This type of repair is innocuous if the same controls are exercised over procedure, consumables and inspection as used in the weld being repaired. Unfortunately, shop personnel are sometimes lax in this respect.

The requirement that deficiencies be detected while the joint is still under preheat/interpass temperature control forces the welding inspectors to visually inspect each weld immediately on completion of the last pass in any given joint.

3.7.3 Preapproval of the Contractor's procedures for the repair of defects which are noncritical and which may normally be expected to occur in the course of the work is intended to verify that the Contractor has an acceptable plan for such repair before the work is initiated.

3.7.7J Repair welds usually involve high restraints, making the situation extremely unforgiving of errors or omissions in procedure that introduce hydrogen. Postheat at 450°F should be highly efficient in outgassing the weld.

---

(1) Ken Notvest, The Welding Journal, Vol. 45(4), April 1966, pp. 173-178s.

(2) C. E. Hartbower, et al, ACOUSTIC EMISSION, ASTM STP505, 1972, pp. 187-221.

3.7.7M The time delay between welding and inspection can be shortened at the discretion of the ASNT Level-III engineer if acoustic-emission testing following repair welding shows that there is no delayed cracking or that cracking has stopped in less than 96 hours. In no case will the engineer approve less than 48 hours delay before final nondestructive examination.

3.8 Peening is an effective method of reducing residual tensile stresses in a weld. Moreover,, peening of the surface layer of a weld will induce compressive stresses that may be beneficial in terms of fatigue.

Normally the code prohibits peening of the surface layer of a weld because of the possible adverse effects of the cold working (mechanical and metallurgical). When groove welds are finished with CONVEX SURFACES exceeding the plate thickness (weld reinforcement), the peening operation may be safely performed on the convex surface layer and then ground flush (a requirement for all groove welds in FCMs).

As always, workmanship is an important consideration; the peening tool must not be allowed to strike the base metal or the fusion boundary between the weld and base metal.

3.9 Stress Relieving Welds. Stress relieving a weld joint serves more than one purpose: not only are the yield point-magnitude residual stresses reduced but also there is a tempering of any martensite formed in the heat-affected zone (HAZ); there generally is an increase in toughness of the weld and weld HAZ, and by virtue of the increase in toughness and the decrease in stress there is a marked inprovement (increase) in the critical crack size in the weld.

4. TECHNIQUES

4.1 General.

4.1.1.1 "Active" submerged-arc-welding fluxes. The use of high arc voltage with "active" (alloy bearing) fluxes can cause transverse weld cracking. The craking is associated with ultimate tensile strengths exceeding 100,000 ksi that occur due to high alloy recovery from the flux. The cracking can occur with any active flux (Linde or Lincoln). For example, Linde 350 and 231 fluxes are active; Linde 80, 124, 709-5 and 0091 fluxes are considered neutral, since multiple-pass welding or variations in voltage will not produce an increase in tensile strength or degradation of the weld-metal Charpy V-notch impact values. Suppliers of flux, with product liability in mind, are identifying their active fluxes in their sales literature by a footnote such as the following:

> CAUTION: Active fluxes should be limited to multipass welding in plate a maximum of 1-inch thick. More highly alloyed wires than Linde 80 or 81 should not be used. Voltages in multipass welds should be limited to a maximum of 35 or even lower if weld procedure tests indicate excessive hardness is encountered.

Lincoln 760, 761, 780 and 781 are active fluxes; Lincoln 860 is considered semi-neutral and 880 is neutral.

Recent experience with weld cracking in A36 girders illustrates the seriousness of the situation when the voltage is too high and the SAW is done with an active flux*:

5-Layer Test Pad Flux Linde 350

Wire	80	80	81	
Size	5/32	5/64	5/64	(inch)
Amp	550	360	360	(dcrp)
Volts	28	32	32	(volts)
Travel	16	16	16	(ipm)
Top bead	90	100	102	(RB hardness)
Tensile	89	113	120	(ksi, estimated)
Top bead	1.53% Mn	2.5% Mn	2.1% Mn	
	0.79% Si	1.4% Si	1.4% Si	(extrapolated)
Flux/wire	0.75	1.23	----	

----

*G. D. Uttrachi, private communication.

In contrast, with a neutral flux and an alloy wire, the wide variations in arc voltage (and attending flux/wire ratios) do not significantly affect the weld chemistry:*

Linde-80 Flux/Linde-551 Cr-Mo Wire
(400 amsp dcsp, 12ipm)

Volts	Flux/Wire	C	Mn	Si
37	1.29	0.09	0.95	0.90
40	1.52	0.10	0.97	0.97
43	1.76	0.10	0.97	0.99

Similar data can be obtained from Lincoln neutral fluxes.

Based on the above data, and the variability of voltage control from machine to machine, shop to shop and operator to operator (as in SAW using manual "squirt" welders), this fracture-control plan prohibits the use of active fluxes in fillet and groove welds involving more than three (3) passes.

Active fluxes should not be prohibited for all welding. Active fluxes may be much better for fillet welds under some circumstances, and for one- or two-layer groove welds, the alloy content cannot build up excessively. Therefore, fillet welds made in two layers (three passes) and butt welds in two layers maximum per weld side, will not cause difficulties under varying shop conditions.

4.1.1.2 Toughness of weld metal. This provision supersedes footnote 8 of AWS D1.1 Table 4.1.1 and footnote 2 of AWS D1.1 Table 4.1.4.

Both the AASHTO Standard Specification for Welding of Highway Bridges and the American Welding Society (AWS) Structural Welding Code D1.1 are incomplete and, in places ambiguous with regard to the controls exercised over the toughness of deposited weld metal. It is the weld deposit where there is the greatest probability of flaws. Cracks too small to be detected with conventional nondestructive inspection methods can become dangerous to the structure if there is slow crack growth and/or crack pop-in. With an acceptable level of fracture toughness

---

*G. D. Uttrachi, private communication.

in the deposited weld metal, there will be less chance of a pop-in crack with its inherently high strain rate, and less chance of premature Region-III fatigue crack growth. Therefore, it is imperative that the deposited weld metal have at least the toughness required of the base metal.

The codes are inadequate with regard to toughness because, as presently written, they mainly rely on Charpy impact testing done by the electrode manufacturers. The welding conditions used by a filler-wire producer in preparing test samples may be totally different from the welding procedure that is qualified for a bridge.

Appended to this commentary is a review of the AWS/AASHTO Code with respect to weld toughness, supporting the argument that the present Specifications are incomplete and sometimes ambiguous.

It is imperative in FCMs that the weld metal have a high resistance to the occurrence of a pop-in crack. The Charpy requirement for weld metal deposited with the qualified welding procedure (Table 4.1.1.2) is based on the through-thickness yielding criterion.

In light of several cases where the AASHTO static-dynamic transition temperature shift was not predictable in weld metal, the weld metal CVN-impact testing is to be done at the LAST and the K_{Ic} (K_{Id}) estimate based on the impact test value.

Provision is made for ASTM E399 fracture testing in lieu of Charpy V-notch impact testing weld metal. Workmanship defects in welds initiate cracking when the structure is under cyclic loading. Sometimes a weld is cracked <u>before</u> the first truck crosses the bridge. In either case, a crack is present (sooner or later) and is subject to fatigue-crack growth. When it reaches the critical crack size (a function of toughness and stress), the crack becomes unstable, there is pop-in and fracture. Plane-strain fracture testing under ASTM E399 provides a measure of the toughness, which in turn can be used to calculate the critical (unstable) crack size for a given stress. On the premise that cracks commonly <u>start</u> in a weld and propagate out into the surrounding base metal, resistance to crack <u>initiation</u> (the onset of unstable growth) is our prime concern in the weld, and resistance to crack <u>propagation</u> (dynamic fracture toughness and crack arrest) is our prime concern in base metal. Thus, it is appropriate in testing weld metal to use the ASTM E399 standard method.

In thick sections, the toughness required in 4.1.1.2 will be difficult to achieve unless suitable consumables are procured to AWS Specifications with toughness requirements. AWS electrode specifications A5.17-76 and A5.23-76, for example, provide consumables that will develop 20 ft-lb Charpy V-notch impact in certification testing at temperatures well below the LAST for bridges in the USA.

A5.17-76	F64-EXXX	20 ft-lb@	-40°F
	F74-EXXX	20 ft-lb@	-40°F
	F66-EXXX	20 ft-lb@	-60°F
	F76-EXXX	20 ft-lb@	-60°F
A5.23-76	F74-EXXX	20 ft-lb@	-40°F
	F76-EXXX	20 ft-lb@	-60°F
	F78-EXXX	20 ft-lb@	-80°F
	F710-EXXXX	20 ft-lb@	-100°F

The LAST for many of the Northern States (excluding Alaska) is minus 30°F; therefore, with suitable energy input, as determined in weld-procedure-qualification testing, a fabricator should be able to meet the requirements of 4.1.1.2.

Examples follow of the Charpy properties that consumable producers are willing to certify based on their testing using the AWS Specification weld joint and welding procedure.

Weld Yield Strength	Welding Process	CVN-Impact	Temperature
67 ksi	SAW	50 ft-lb	-100F
64 ksi	FCAW	40 ft-lb	- 40F
80 ksi	SAW	35 ft-lb	- 60F
78 ksi	FCAW	48 ft-lb	- 40F
102 ksi	SAW	35 ft-lb	- 60F
98 ksi	FCAW	55 ft-lb	- 40F

A recent investigation of an electrode-flux combination under AWS A5.23-76, viz., Lincoln 880/Lincore Ni_2 by the Michigan Department of State Highways and Transportation*, found the wire/flux combination to be very attractive for welding unpainted A588 structures, with the nickel content of the weld metal falling consistently within the midrange of the specified 2.0 to 2.9 percent and Charpy V-notch energies averaging <u>about 60 ft-lb at 0°F</u>. Two different fabricators were involved in

---

*J. D. Culp, private communication, February 1978.

the data collection, with plate thickness ranging up to 3 1/8 inches. Actual production welding procedures were used by both fabricators.

4.1.1.2.1 Weld-metal Charpy V-notch impact test specimens not meeting the 80 percent shear fracture "upper shelf" requirement of Table 4.1.1.2 would logically be subject to the requirement of Table C1.5.4.4; i.e.,

$$CVN \gtrsim FTY^2 \cdot B/5E$$

but this is impractical for deposited weld metal. A review of weld-procedure-qualification testing for the California Department of Transportation (CALTRANS)* in the period 1966 to 1976 revealed many acceptable weld tests based on the "upper-shelf" correlation (see Table C4.1.1.2.1). When near acceptable CVN-impact data were obtained at test temperatures below the LAST for continental USA, it is presumed that an appreciably higher test result would have been measured at the LAST. It is possible, however, that some of the reported values are upper-shelf values and, therefore, would not increase significantly at a higher test temperature.

From the production-weld-procedure CVN-impact test results of Table C4.1.1.2.1, note that the requirements of $CVN \gtrsim FTY^2 \cdot B/5E$ were only met with the procedures and consumables tested in the lighter thicknesses and lower strengths. The upper-shelf requirement, however, is easily met by many of weld-procedure-qualification tests for CALTRANS in the 1966-76 period.

ASTM E399-74 Standard Test Method for fracture toughness testing has provision for an alternative test result, the specimen strength ratio R_{sb} for bend testing and R_{sc} for compact tension testing (see E399-74 paragraphs 1.3, 9.1.6 and 9.1.7). This ratio has been found to be a useful comparative measure of the toughness of materials when the specimens tested are all of the same form and size, and when the size is sufficient to permit the maximum load to be determined by pronounced crack extension prior to plastic instability, even though not sufficient to meet the requirements for a valid K_{Ic} test. When only the specimen strength ratio is to be determined, the costly test record and associated instrumentation can be omitted because only the maximum load and the specimen dimensions, including average crack

---

*Eric Nordlin, private communication.

TABLE C4.1.1.2.1

WELD-PROCEDURE-QUALIFICATION TESTING for CALTRANS and New York State

	THICK (in.)	PROCESS/CONSUMABLES	CVN-IMPACT/°F	YEAR/FAB
A514/517	1/2	SMAW/E11016G	55@ -50	'76/XX
		SMAW/E11016G	45@ -50	'76/XX
		SMAW/E11016G	39@ -50	'76/XX
		SMAW/E11016G	39@ -50	'76/XX
		SMAW/E11018M	35@ -60	'69/C
		SMAW/E12018M	38@ -60	'69/C
		SAW/F110/MF38B	34@ -50	'76/XX
	3/4	SMAW/E12018M	61@ -60	'69/C
		SMAW/E11018M	91@ -60	'69/C
		SMAW/E12018M	42@ -60	'69/Y
	1	SMAW/E12018M	48@ -60	'69/C
		SMAW/E11016G	49@ -50	'76/XX
		SAW/120/709-5	47@ -60	'69/C
		SAW/L61/A1010x10S	36@ -60	'69/C
	1 3/16	SAW/L61/A1010X10S	25@ -60	'69/C
		SAW/L61/A1010X10S	24@ -60	'69/C
	1 5/16	SAW/120/709-5	43@ -60	'69/C
		SAW/120/0091	39@ -60	'69/C
	1 1/2	SAW/F110/MF38B	30@ -50	'76/XX
		SAW/L61/A1010X10S	21@ -60	'68/C
	2	SAW/120/709-5	36@ -60	'70/C
	2 1/4	SMAW/E12018M	54@ -60	'69/C
		SMAW/E12018M	59@ -60	'70/C
	2 1/2	SMAW/E12018M	36@ -60	'69/Y
A36	1/2	SMAW/E7018	61@ 0°F	'68/M
		SAW/L60/L860	62@ 0	'68/O
	3/4	SMAW/E7018	225@ -20	'66/A
		SMAW/E7018	254@ -20	'66/A
		SMAW/E7018	58@ 0	'70/B
		SAW/L61/L780	41@ 0	'69/Z
		SAW/L61/L860	69@ 0	'70/B
		SAW/L61/L860	77@ 0	'70/B

A36	1	FCAW/79	33@ -20	'71/AA
		FCAW/75	85@ -20	'70/B
		SAW/L61/L860	93@ 0	'67/H
		GMAW/A675	88@ 0	'68/CC
		GMAW/HB25B	97@ 0	'68/FF
		GMAW/HB25B	65@ 0	'68/FF
		GMAW/HB28C	58@ -20	'67/HH
	1 1/4	SAW/L61/L860	104@ 0	'68/A
		SAW/L60/L761	65@ 0	'70/B
		SAW/L61/L860	66@ 0	'69/M
	1 1/2	SAW/L61/L860	80@ 0	'67/H
		SAW/L61/L860	114@ 0	'67/H
	2	SAW/L61/L860	105@ 0	'68/A
	2 3/4	SAW/L61/L860	81@ 0	'67/H
	3	SAW/L60/L860	88@ 0	'68/M
A441	1/2	SAW/L61/L860	45@ 0	'69/B
	1	FCAW/75	105@ 0	'66/E
	1 1/2	SAW/36/50	83@ 0	'67/B
		FCAW/78	53@ 0	'69/T
	2 1/2	SAW/L61/L860	61@ 0	'70/E
		SAW/L61/L860	56@ 0	'68/E
		SAW/L61/L860	97@ 0	'68/E
		SAW/L61/L860	87@ 0	'67/E
		FCAW/75	90@ 0	'67/E
A242	5/8	SMAW/E7018	77@ 0	'67/D
	1 1/2	SMAW/E7018	110@ 0	'69/C
		SAW/L61/AXXX10S	48@ 0	'68/C
A588	1/2	SAW/E8018C3 type	43@ -40	'76/ZZ
	3/4	SAW/E8018C3 type	26@ -40	'76/ZZ
		SAW/E8018C3 type	33@ -40	'76/ZZ
	1	SMAW/E8018C3	73@ -40	'76/ZZ
		SMAW/E8018C3	80@ -40	'76/ZZ
		SMAW/E8018C3	144@ -40	'76/ZZ
		SAW/L60/L780	44@ 0	'66/W
		SAW/Ni 2/L880	45@ 10	'76/WW

A588	1 1/4	SAW/Ni2/L880	45@ 10	'76/WW
		SAW/Ni2/L880	60@ 10	'76/WW
	1 1/2	SAW/L61/7F13AXXX10	46@ 0	'71/X
	2	SAW/E8018C3 type	56@ -40	'76/ZZ
		SMAW/E8018C3	129@ -40	'76/ZZ
	2 1/8	SAW/E8018C3 type	55@ -40	'76/ZZ
	2 3/4	SMAW/E8018C3	70@ -40	'76/ZZ
	3 3/4	SMAW/E8018C3	73@ -40	'76/ZZ
		SAW/E8018C3 type	66@ -40	'76/ZZ
	1	SAW/L61/AXXX10	41@ 0	'69/NYS
		SAW/L60/780	51@ 0	'73/NYS
		SAW/L60/780	33.7@ 0	'74/NYS
		SAW/L61/AXXX10	43@ 0	'77/NYS
		SAW/L61/AXXX10	79@ 0	'75/NYS
		SAW/L61/860	34@ 0	'77/NYS
		SAW/L61/AXXX10	43@ 0	'77/NYS
		SAW/L61/AXXX10	82@ 0	'77/NYS
		SAW/81/80	36@ 0	'77/NYS
		SAW/81/350	32@ 0	'77/NYS
		FCAW/81/CO_2	32@ 0	'77/NYS
A36	1	SAW/815/860	39@ -20	'73/NYS
		SAW/L61/860	50@ 0	'75/NYS
		FCAW/71/CO_2	39@ 0	'66/NYS
		FCAW/T62/CO_2	39@ 0	'73/NYS
		FCAW/FC707/CO_2	49@ 0	77/NYS
A572	1	SAW/L61/860	32.3@ 0	'75/NYS
		FCAW/Super Cor/CO_2	33@ 0	'68/NYS
		FCAW/75/CO_2	134@ 0	'69/NYS
		FCAW/4/CO_2	34@ 0	'75/NYS
A441	1	SAW/L61/860	69@ 0	'69/NYS
		SAW/36/80	39@ 0	'71/NYS
		SAW/80/231	34@ 0	'71/NYS
		SAW/80/780	43@ 0	'71/NYS
		FCAW/78/CO_2	58@ 0	'67/NYS
		FCAW/111AC/CO_2	30@ 0	'70/NYS

length, are needed (Reference National Materials Advisory Board Report NMAB-328 "Rapid Inexpensive Tests for Determining Fracture Toughness").

Based on linear-elastic-fracture-mechanics concepts, K_{Ic} and net stress, FTN, are directly related for linear-elastic fractures. This relationship, expressed in terms of a linear slope m, is

$$K_{Ic}/FTY = m(FTN/FTY).$$

Values of m for various specimen types and a/W ratios are listed below (reference Server and Wullaert, "The Use of Small-Specimen Strength Ratio for Measuring Fracture Toughness", December 1977, submitted for publication in Engineering Fracture Mechanics):

Slope Parameter m Between K_{Ic} and FTN

	m values for specific a/W		
Specimen Type	0.45	0.50	0.55
Precrack Charpy	0.288	0.278	0.266
E399 1-in. bend	0.649	0.626	0.599
E399 1-in. compact	0.728	0.679	0.632

For purposes of this specification, the K_Q values for compliance with Table 4.1.1.2.1 will be determined with a 1-in. compact tension specimen and the expression:

$$K_{Ic}/FTY = 0.68\ R_{sc}$$

where the strength ratio, R_{sc}, is

$$R_{sc} = 2P_{max}\ (2W+a)/B(W-a)^2FTY$$

as described in E399-74 paragraph 9.1.7, or with a 1-inch E399 bend specimen and the expression:

$$K_{Ic}/FTY = 0.63\ R_{sb}$$

where the strength ratio, R_{sb}, is

$$R_{sb} = 6P_{max}\ W/B(W-a)^2FTY$$

4.1.1.2.3 Techniques are available for locally stress relieving a weld. A temperature of 1150°F ± 25°F held for one (1) hour per inch of thickness will usually suffice. The rules for Construction of Unfired Pressure Vessels, Section VIII of the ASME Code, 1966 Edition, paragraph UM-40 provide for local stress relief.

Vessels that are to contain lethal substances when fabricated of carbon or alloy steels are required to be postweld thermally stress relieved (paragraph UM-2) and vessels to operate at temperatures below minus 20°F must be stress relieved (paragraph USC-67). These examples where stress relieving is required gives some insight into the thinking of the ASME Boiler Code Committee with regard to the importance of thermal stress relief as assurance for safe performance of pressure vessels.

Stress relieving welds in bridge members is not common practice, and will be a costly operation demanding strong justification. There is in fact very strong justification for stress relief. Consider the following points:

(1) Stress relief properly applied not only will reduce the residual welding stresses but also will increase the toughness of both the weld and weld heat-affected zone in many steel chemistries. With a reduction of residual weld stresses and an increase in toughness, the most common mechanism producing subcritical crack growth, fatigue, may be less of a problem. On the other hand, with high residual stresses and marginal or low toughness, fatigue crack growth may be unpredictably rapid (Region-III accelerated cracking).

(2) Critical crack size as a consideration in fracture mechanics is basic to what fracture mechanics is all about. Given the toughness of plate or weld and the nominal stress, one can calculate the size of crack that will become unstable. Such calculations are presented in Table C4.1.1.2.3 for typical crack configurations. Two important observations are apparent; first, the toughness levels corresponding to the current (1974) and proposed (1978) AASHTO specifications are nonconservative at the very least and, secondly with near yield-point-magnitude residual welding stresses, the toughness in weld metal would have to be so high as to be totally impractical if the critical crack size is to be detectable by conventional nondestructive inspection methods, or large enough to provide an allowance for subcritical crack growth by whatever mechanism. With effective stress relief, the calculation of critical crack size would be based on the design allowable.

TABLE C4.1.1.2.3

SAMPLE OF CRITICAL-SIZE CALCULATIONS FOR FOUR CRACK CONFIGURATIONS

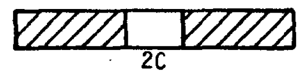
CRITICAL CRACK SIZE IN A588

ASSUME: YIELD PLATE 60 KSI
WELD 65 KSI
ALLOWABLE 27 KSI

EDGE CRACK $c = [KIc/1.12S(\pi)^{1/2}]^2$

CVN-IMPACT	KIc	CRACK PLATE	CRACK WELD
15	47	0.74	0.13
25	61	1.25	0.22
35	72	1.74	0.31

THRU-CRACK $c = [KIc/S(\pi)^{1/2}]^2$

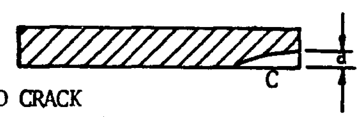


CVN-IMPACT	KIc	CRACK PLATE	CRACK WELD
15	47	0.93	0.17
25	61	1.57	0.28
35	72	2.18	0.39

CRITICAL CRACK SIZE IN A588

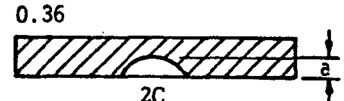
ASSUME: YIELD PLATE 60 KSI
WELD 65 KSI
ALLOWABLE 27 KSI

CORNER CRACK $a = [KIc(Q)^{1/2}/(1.12)^2S(\pi)^{1/2}]^2$



CVN-IMPACT	KIc	PLATE CRACK a=.1(2c)	PLATE CRACK 0.3(2c)	WELD CRACK .1(2c)	WELD CRACK 0.3(2c)
15	47	0.61	0.91	0.10	0.15
25	61	1.04	1.53	0.16	0.26
35	72	1.44	2.13	0.22	0.36

PART-THRU $a = [KIc(Q)^{1/2}/1.1S(\pi)^{1/2}]^2$



15	47	0.80	1.18	0.12	0.20
25	61	1.35	1.99	0.21	0.33
35	72	1.88	2.77	0.29	0.46

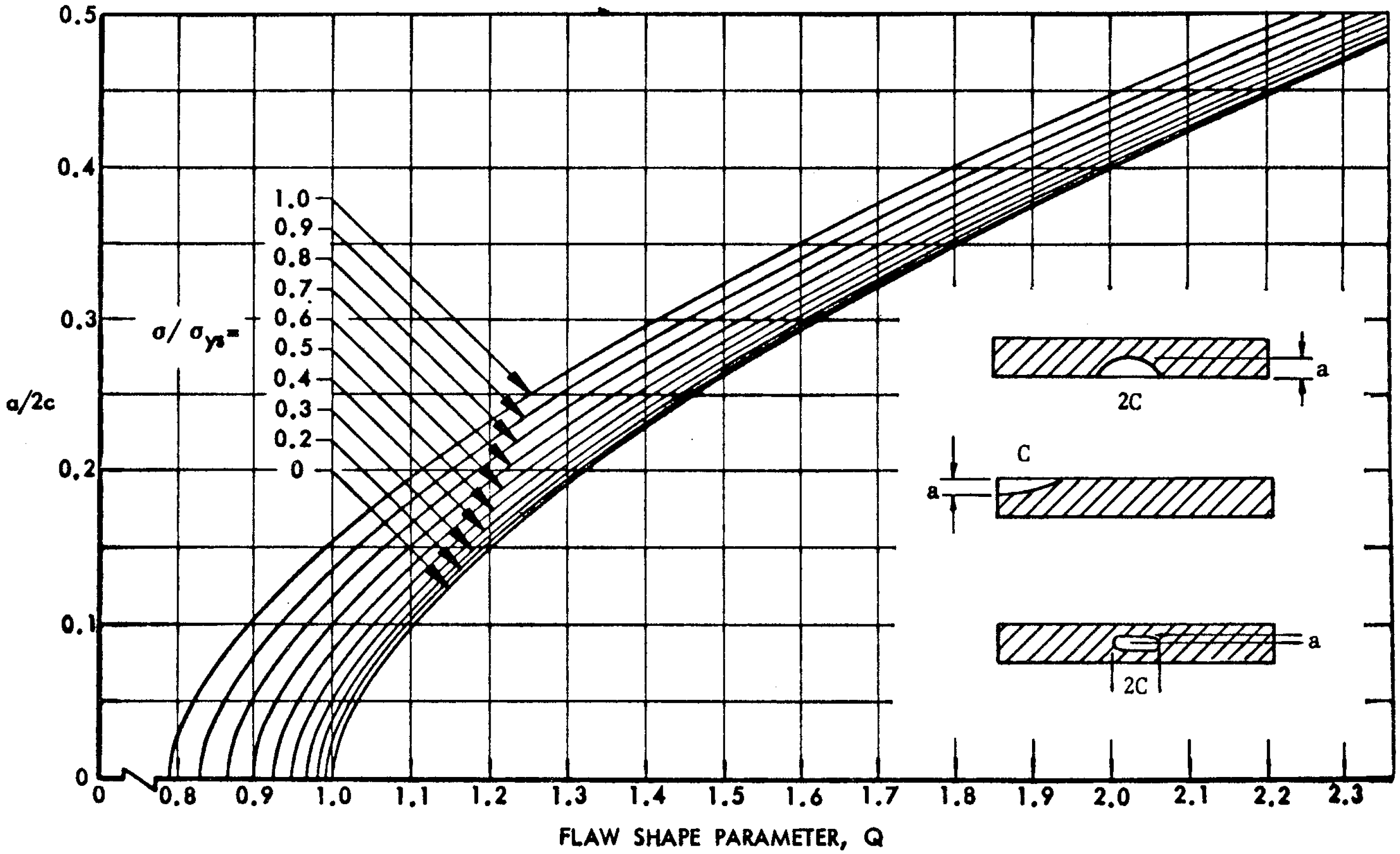


Figure 3 SHAPE PARAMETER CURVES FOR SURFACE AND INTERNAL FLAWS

Figure C4.1.1.2.3.a - Flaw Shape Parameter for Three Crack Configurations.

(3) The probability of a flaw is greatest in deposited weld metal. Whether porosity, slag, undercut, incomplete penetration or whatever, the tendency is for cracks to form under the action of residual weld stresses and/or cyclic loading. In the presence of hydrogen, the defect is often weld cracking at the outset, and insidious because classic hydrogen cracking is delayed, sometimes occurring for several days after welding is completed. With stress relief there will be an outgassing of hydrogen which should effectively eliminate this problem if the stress relieving is done soon after the welding operation.

Stress relieving welds in Q&T steel. If quenched-and-tempered steel is being joined, the stress-relief temperature must not exceed the temperature used in tempering the plate. A stress-relieving temperature about 50°F lower than the tempering temperature is desirable to avoid any danger of lowering the strength of the plate. However, for purposes of stress relieving, a temperature in the mean range of 1100°F to 1200°F is preferred because this is sufficiently high to insure a substantial reduction in residual stress in the weldment. Thus, if the weldment is to be effectively stress relieved, the tempering temperature should not be much below 1200°F.

In some steel compositions, stress relief can cause cracking. Stress-relief cracking is a problem that has been recognized for well over a decade (see J. D. Murray "Stress-Relief Cracking in Carbon and Low Alloy Steels", British Welding Journal, August 1967, pp. 447-457; and L. F. Porter, et al "A Study of Temper Embrittlement During Stress Relieving of 5Ni-Cr-Mo-V Steels", TEMPER EMBRITTLEMENT IN STEEL, ASTM STP407, American Society for Testing and Materials, 1968, pp. 20-45). Murray observed that molybdenum and vanadium which are potent elements in conferring creep strength (for high-temperature service) are problem alloys in promoting stress-relief cracking. Porter of the U. S. Steel Applied Research Laboratory found that in the super tough steel he was studying (HY130), manganese and molybdenum contributed most to the embrittlement; whereas, vanadium did not contribute to temper embrittlement. As phosphorus increased above 0.006 percent, embrittlement occurred; high oxygen content, on the other hand, lowered the toughness but did not contribute to temper embrittlement. The dramatic effect of manganese and the excellent toughness of the steel are shown in Fig. C4.1.1.2.3.b as reproduced from STP407. HY-130 steel and similar, lower strength, high-toughness steels were developed by the U. S. Steel Corporation in a multi-million-dollar R&D for the U.S. Navy.

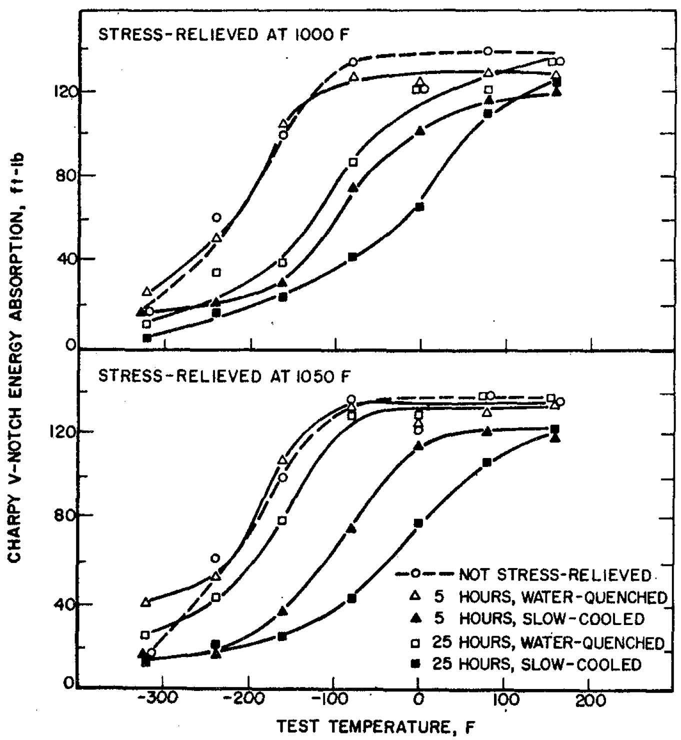


FIG. 12—*Effect of stress relieving treatments on the transition curves of standard-manganese 5Ni-Cr-Mo-V steel.*

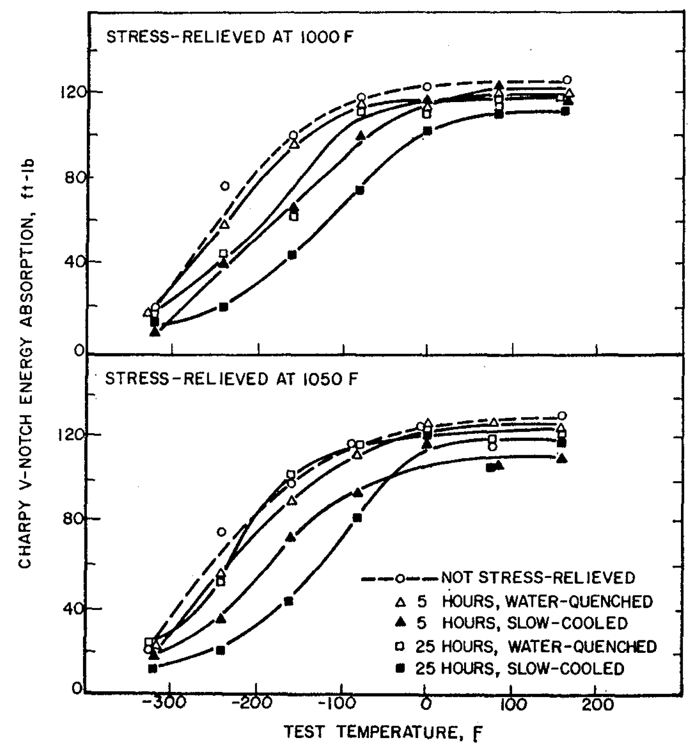


FIG. 13—*Effect of stress-relieving treatments on the transition curves of low-manganese 5Ni-Cr-Mo-V steel.*

Figure C4.1.1.2.3.b - Stress-Relief Embrittlement

From the above discussion it should be apparent that both the steel producer and welding-consumables producer should be consulted in procuring optimum compositions where stress relief may be used.

Generally speaking, with proper chemistry, stress relief will produce an increase in the toughness of weld metal. The selection of welding consumables should be discussed with the producers; some electrodes are intended for applications requiring stress-relieving treatment. The following are examples of such electrodes:

ELECTRODE	TEST TEMPERATURE	MINIMUM CVN-IMPACT
E8016-C2	-100°F	20 ft-lb
E8018-C2	-100°F	20 ft-lb
E9015-D1	- 60°F	20 ft-lb
E9018-D1	- 60°F	20 ft-lb
E10015-D2	- 60°F	20 ft-lb
E10016-D2	- 60°F	20 ft-lb
E10018-D2	- 60°F	20 ft-lb

4.1.1.5 The fabricator may wish to base his welding-procedure specifications on identical procedures being used (1) concurrently or (2) consectively in successive bridges, where the procedures were previously qualified by testing. If the engineer is satisfied by documented evidence of previous testing, he may waive the testing under 5.2 but in such case will require run-off-plate production-welding-procedure testing on a more frequent basis than otherwise required in 4.1.1.4.

4.2.2 Preheat/Interpass Temperature. One of the major problems in welding of steels has been the type of cracking generally known as hydrogen-induced cold cracking. Cold cracks cannot be tolerated in a bridge and they are often difficult to detect and expensive to repair. Therefore, it is essential for the fabricator to take precautions during welding to prevent their formation.

Over the last three decades much time and money has been spent in a search for a "weldability" test. As early as 1946, investigators at the Naval Research Laboratory reviewed what was then an already substantial literature on the subject.*

---

*G. G. Luther, C. E. Jackson and C. E. Hartbower, "A Review and Summary of Weldability Testing Carbon and Low-Alloy Steels", THE WELDING JOURNAL, Vol. 25 (7), p. 376-s, July 1946.

Research has continued over the years. The longitudinal-bead-weld underbead cracking test as described in Chapter 9 of the book "Weldability of Steels" Second Edition, by Dean Stout of Lehigh University and Dr. Dorr Doty of U. S. Steel Corporation, is a test for one type of cracking, hydrogen-induced cold cracking in the weld heat-affected zone. This test relies on the E6010 electrode as a source of hydrogen; use of a low-hydrogen electrode in such testing (as in the current AASHTO Welding Specification) can lead to only result - zero cracking.*

Many other tests can and are being used to determine cold-cracking susceptibility - the useful ones are all more expensive than the bead-on-plate test but also they are more informative. The controlled thermal severity (CTS) test, the cruciform test, the circular patch test, the Lehigh butt-weld test, the Naval Research Laboratory test, the Battelle test, and the Granjon implant test are a few of the many tests that have been researched. The salient features of many of these tests have been summarized by the Defense Metals Information Center, Battelle Memorial Institute.**

As an alternative to mechanical testing, there are obvious advantages in being able to compare heats based on a common analytical standard, and this has lead to the concept of carbon equivalent (C.E.). Much welding research has been done both in the USA and abroad to establish the "best" equation. Three examples follow:

Japan Welding Engineering Society -

$$CE = C + \frac{Mn}{6} + \frac{Si}{24} + \frac{Ni}{40} + \frac{Cr}{5} + \frac{Mo}{4} + \frac{V}{14}$$

International Institute of Welding -

$$CE = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15}$$

K. Winterton, Canadian Dept. of Mines & Technical Surveys -

$$CE = C + \frac{Mn}{6} + \frac{Ni}{20} + \frac{Cr}{10} + \frac{Cu}{40} - \frac{Mo}{50} - \frac{V}{10}$$

---

*As a recent example of this, see W. P. Benter, Jr., The Welding Journal, Vol. 53(8), p. 364-s, August 1974, Table 2.

**M. D. Randall, R. E. Monroe and P. J. Rieppel, "Methods of Evaluating Welded Joints", DMIC Report 165, December 28, 1961.

Winterton, in the WELDING JOURNAL (USA), June 1961, pp. 253-s to 258-s, discusses the above formula and several others. These equations are for one purpose, viz., providing a relationship between heat-affected-zone (HAZ) cracking and steel composition.

Since one of the factors affecting this type of cracking is hardenability, chemical composition has, in fact, been found useful as an indicator of HAZ crack susceptibility. The composition is expressed as a carbon equivalent. Critical cooling rate is a measure of the hardenability of the steel; many investigators have shown that critical cooling rate correlates very well with the steel composition expressed as a carbon equivalent.

The objective in using a carbon equivalent (CE) determination is (1) to set a limit on the CE that will be acceptable in steel used in FCMs or (2) to adjust the preheat/interpass/postheat treatment based on the CE level of a given heat of steel to minimize the weld HAZ cracking due to hydrogen. In 1969 (IIW Doc. No. IX-631-69), Ito and Bessyo showed that to avoid HAZ cracking one must take into consideration the restraint (or reaction stress) and diffusible hydrogen content in the weld metal, as well as the effect of chemistry as expressed in the conventional CE formulas. Based on data from about 200 steel types, widely ranging in chemistry, Ito and Bessyo showed that when the hydrogen content and restraint (e.g., plate thickness) was taken into account otherwise widely scattered data fell into a tight grouping. The Japanese research shows great promise for controlling weld HAZ cold cracking but some important questions remain requiring further welding research before the CE approach can be used in the fracture control plan.*

WELDABILITY is a term appearing in the AWS Structural Welding Code but provisions for its control are ill defined. Appended to this commentary is a summary of the AWS/AASHTO paragraphs referring to weldability; a review of these provisions reveals that the specifications as presently written are incomplete and in some places ambiguous. The AWS D1.1 Code gives no direction whatever as to how weldability is to be evaluated, and AASHTO specifies a weldability evaluation that is impractical. Until welding research resolves the questions on CE and/or provides a practical weldability test(s), it is recommended that more strigent preheat/interpass temperatures be used in the welding of FCMs.

For those who see the Table 4.2 preheat/interpass temperatures as being excessive, attention is directed to Lincoln Electric Bulletin

---

*B. A. Graveille, "The Principles of COLD CRACKING CONTROL in Welds", p. 132, published by Dominion Bridge Co, Ltd., P.O. Box 280, Station 'A', Montreal, Quebec, Canada HeC 2S7.

S213 on the selection and use of the LINCORE Low Alloy Submerged Arc Process. The following preheat/interpass temperatures are recommended in Bulletin S213:

	PLATE THICKNESS	
ELECTRODE	up to 1 1/2 in.	1 1/2 in. and up
M2	300°F	350°F
F1	250°F	350°F
B2	250°F	350°F
Ni2	200°F	350°F

Lincore low alloy tubular electrodes are used with Lincoln 880 flux which is a neutral flux (see 4.1.2.3 prohibiting "active" fluxes and 4.1.1.2 for recent toughness testing of the Lincoln 880/Ni2 Lincore flux-wire combination by Michigan DOT).

The preheat/interpass temperatures of Table 4.2 must be applied as a minimum requirement in all welding on FCMs. Temporary attachments are no exception. Even tack welds that are later to be incorporated in a groove weld are no exception. When a tack weld cracks or a tack weld generates a crack in the surrounding base metal, the first pass(es) of the groove weld may not entirely penetrate the cracked metal and, thus, leave an initiator for subsequent groove-weld cracking.

4.2.3 Postheating Repair Welds. Diffusion of hydrogen is greatly enchanced by a moderate increase in temperature; at temperatures below about 150°C (300°F) the diffusion rates tend to be widely variable and much lower than would be expected based on measurements above 150°C (see Granville loc. cit., p. 13, Figure 2.2).

Recent (1970's) experience with cracked bridge girders indicates that weld cracking may have occurred AFTER final inspection; i.e., cold cracking. The most likely cause of cold (delayed) cracking is hydrogen. Hydrogen is known to be introduced from several sources such as moisture in the flux or coating, condensation on the plate surface, grease (hydrocarbons), drawing compounds on the weld-wire surface, and hydrogen in solution in the wire. One effective method for reducing the hydrogen content of the weld is postheating. Time delays between weld passes while maintaining the interpass temperature, also may be effective.

Under 4.2.3 a 1-inch deep excavation in a 2 1/2-inch thick butt-splice groove weld requires a 3-hour postheat at 450°F. The preheat/interpass temperature must be sufficient to prevent cracking; for highly restrained welds such as weld repairs, temperature above the minimum specified in Table 4.2 as amended may be required as in 3.7.7G.

4.7 This provision prohibits the use of fillet welds along the edge(s) of the steel backing.

5. QUALIFICATION

5.1 Approved Procedures.

5.1.1 AWS D1.1 paragraph 5.1.1 which specifies that: "Welding procedures... may be deemed prequalified and may be exempt from tests...", is ambiguous in respect to when, if ever, a given welding-procedure specification is tested. Consider, for example, the case of an actual bridge. The "qualified" welding procedure used in fabrication of the bridge was tested several years earlier with excellent Charpy V-notch impact test results (approximately 70 ft-lb at $0^{o}F$). Apparently there was a change in personnel and/or equipment between the time of weld-procedure qualification and making the production welds in the second bridge because the Charpy values in the second bridge were an order of magnitude lower.

Some have interpreted the AWS code to mean that for certain welding procedures no testing need ever be done under the provisions of paragraph 5.1.1. This fracture control plan disallows the use of prequalified welding procedures in FCMs, except in SMAW with specified restrictions; furthermore, any change exceeding the Limitation of Variables specified in Section 5.5 of the AWS/AASHTO Code, as amended, shall be cause for requalification.

5.1.1.1 This paragraph of the fracture-control plan says that for FCMs, all welds must be tested at one time or another using the tests enumerated in 5.10, as amended, for FCMs; some welding procedures, as specified in 5.2.1, must be qualified by test for each job.

Weld categories other than those listed in 5.2.1 may be considered by the Engineer as prequalified based on evidence of previous weld-procedure-qualification testing. However, the weld categories listed in 5.2.1 will be subject to weld-procedure-qualification testing for each bridge to be fabricated, with the one exception under 5.1.1.3.

Incomplete-penetration fillet welds and groove-weld T- and corner-joints parallel to the applied stress generally will require weld-procedure-qualification testing as in 5.10; exception to such testing will be based on previous weld-procedure-qualification testing in a fabrication involving the same grade/type of steel (with all other variables subject to AWS D1.1 paragraph 5.5.2).

5.1.1.3 Sulfur and phosphorus are tramp elements in steel which should be kept as low as practicable. Low sulfur, we are told, may be one of the reasons Japanese steel tends to be super tough. Phosphorus over 0.006 percent is one of the elements which causes stress-relief embrittlement. Some steel producers use a "scrap-melting practice", i.e., their electric-furnance steel is almost entirely made from scrap metal. When the Bryte Bend Bridge developed brittle fracture, the chemistry of the steel was analyzed by mass spectrographic analysis and found to contain relatively large amounts of arsenic, antinomy, and tin, tramp elements which also produce temper embrittlement. At present, there is no control over tramp elements in ASTM specifications other than sulfur and phosphorus, and the ASTM specifications, in general, are much too loose in control of these elements.

ASTM A533-76 addressing the subject of residual elements points out that reactor design requires review and control of the residual elements that affect material properties. Sulfur and phosphorus are discussed. Sulfur, it says, can affect the Charpy upper-shelf energy level, and that control of this element or control of its morphology provide alternative means of controlling the upper-shelf energy level. According to ASTM A533, "the following table itemizes currently available commercial limits for these elements:"

ELEMENT	HEAT ANALYSIS, %	PRODUCT ANALYSIS, %
Sulfur	0.015	0.018
Phosphorus	0.012	0.015

Typically, ASTM allows 0.05 maximum sulfur and 0.04 maximum phosphorus.

Electrode and flux should be procured with the manufacturer's certification that the consumables will provide a specified level of Charpy impact down to at least minus 40 degrees.

5.2 Other Procedures.

5.2.1 Groove-weld T- and corner-joints and fillet welds perpendicular to the applied stress are singled out for mandatory procedure-qualification testing because a pop-in crack in the weld could run catastrophically down the length of the weld. When welds are parallel to the applied stress, a pop-in crack also could occur, but the stress state would trigger a transverse crack which would run into the base metal rather than down the length of the weld.

Base-metal considerations. In weld-procedure qualification testing, the requirement that testing be done with steel of the same grade/type as in the structure allows use of a different heat of steel. In general, the specification of a steel of the same type and grade as to be used in the bridge will result in steel from the same producer and, therefore, the same melting practice. However, sometimes a patented grade/type (chemistry) will be produced by more than one steel company under a licensing agreement (as in the case of A514/517 grade H). In such a case, under 5.2.1, there is no protection against differences in melting practice. This could be covered in the "Contract Special Provisions" by requiring steel of the same grade/ type and the same producer(s) supplying steel for the structure.

Melting-practice considerations. A517 grades F and H supplied for the Bryte Bend and Tuolumne bridges in California were made with a melting practice that produced inferior hardenability and toughness*. The steel producer added boron to the ladle before the addition of aluminum and vanadium; as a result the steel had low hardenability and, therefore, low quarter-point toughness. The melting practice was corrected after the time of the Bryte Bend brittle fracture by adding the boron after the protective agents. The point here is that there can be marked differences in melting practice from one producer to another.

When A517 grade F steel was produced for the Tuolumne Bridge in California, the melting practice did not call for the addition of titanium. Titanium is not required by ASTM specification but is important in terms of melting practice. Figures C5.2.1a, b, and c illustrate the differences in toughness as a function of melting practice. Figure C5.2.1a (plate A from the Tuolumne Bridge) was A517-F from steel producer "A" with the deliberate omission of titanium; C5.2.1b (plate J) was A517-F from steel producer "B" where titanium was inadvertantly omitted; and C5.2.1c (plate M) was A517-F from steel producer "B", with titanium added in melting the steel.

Heat-affected-zone considerations. The California Department of Transportation requires Charpy V-notch impact testing of the heat-affected zone (HAZ) in groove welds. However, present methods for locating the notch in the HAZ are arbitrary and may miss a zone of embrittled microstructure. The U.S. Coast Guard specifies tests of HAZ at arbitrary increments of distance from the weld fusion line (1, 3, 5, 7 mm). There is no question that HAZ should be tested, and better methods are available**, but there is no standard method at the present time.

5.2.2 Toughness of weld metal. Testing is specified for all groove welds whether they be longitudinal or transverse to the principal stress. A weld defect residing in a low-toughness weld metal together with near-yield-point-magnitude residual stresses can trigger a pop-in crack. If the weld is longitudinal to the principal stress, a pop-in crack may enter the heat-affected zone (HAZ) and

---

*C. E. Hartbower and R. D. Sunbury, "Variability of Fracture Toughness in A514/517 Plate", October 1975, DOT-FH-11-8250, 265 pages.

**See WELDING HANDBOOK, Section 1, Fifth Edition, "Fundamentals of Welding", American Welding Society, 1962, pp. 6.28-6.34.

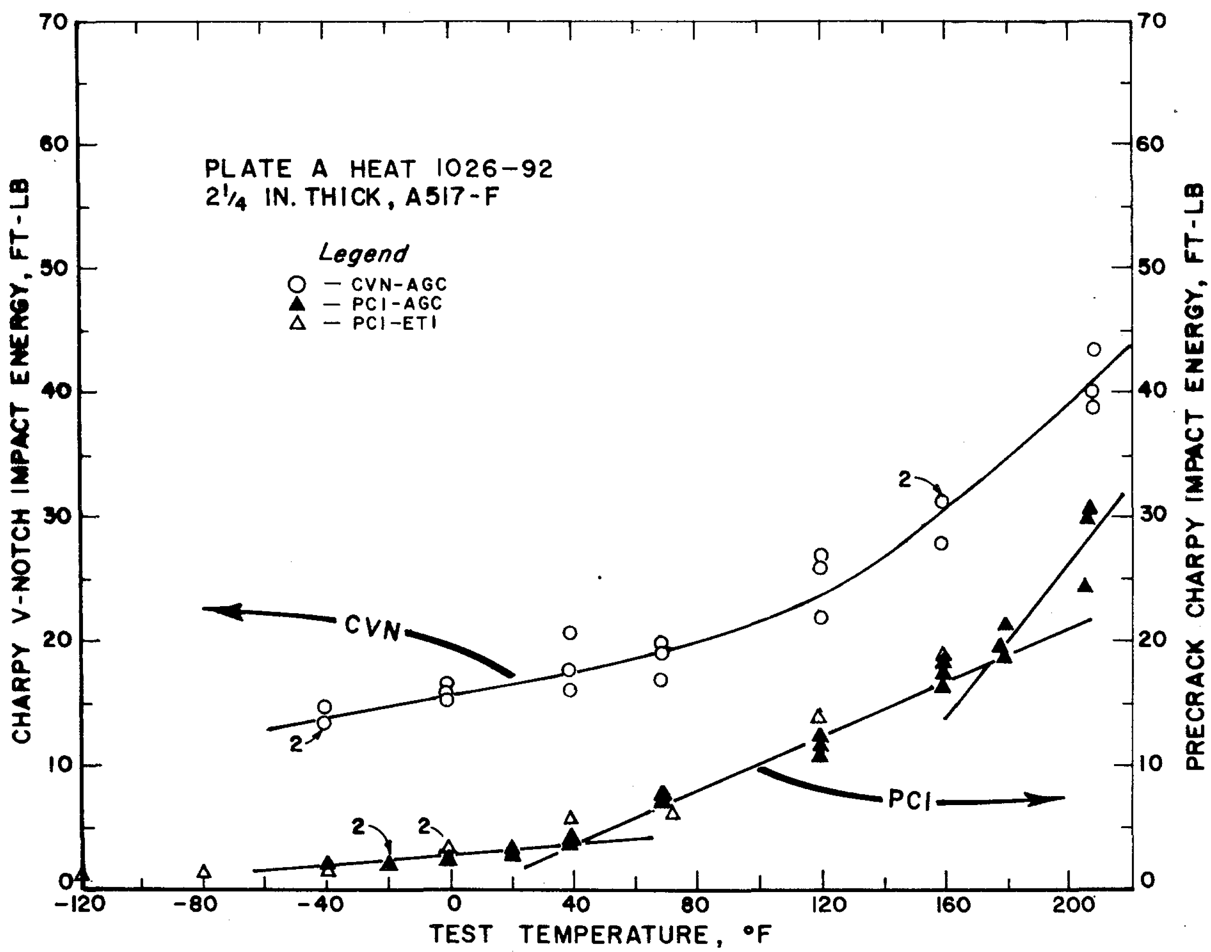


Figure C5.2.1.a - Charpy V-Notch and Precrack Charpy Impact Transition Curves for A517 Grade F Mill Heat B 9863-2C.

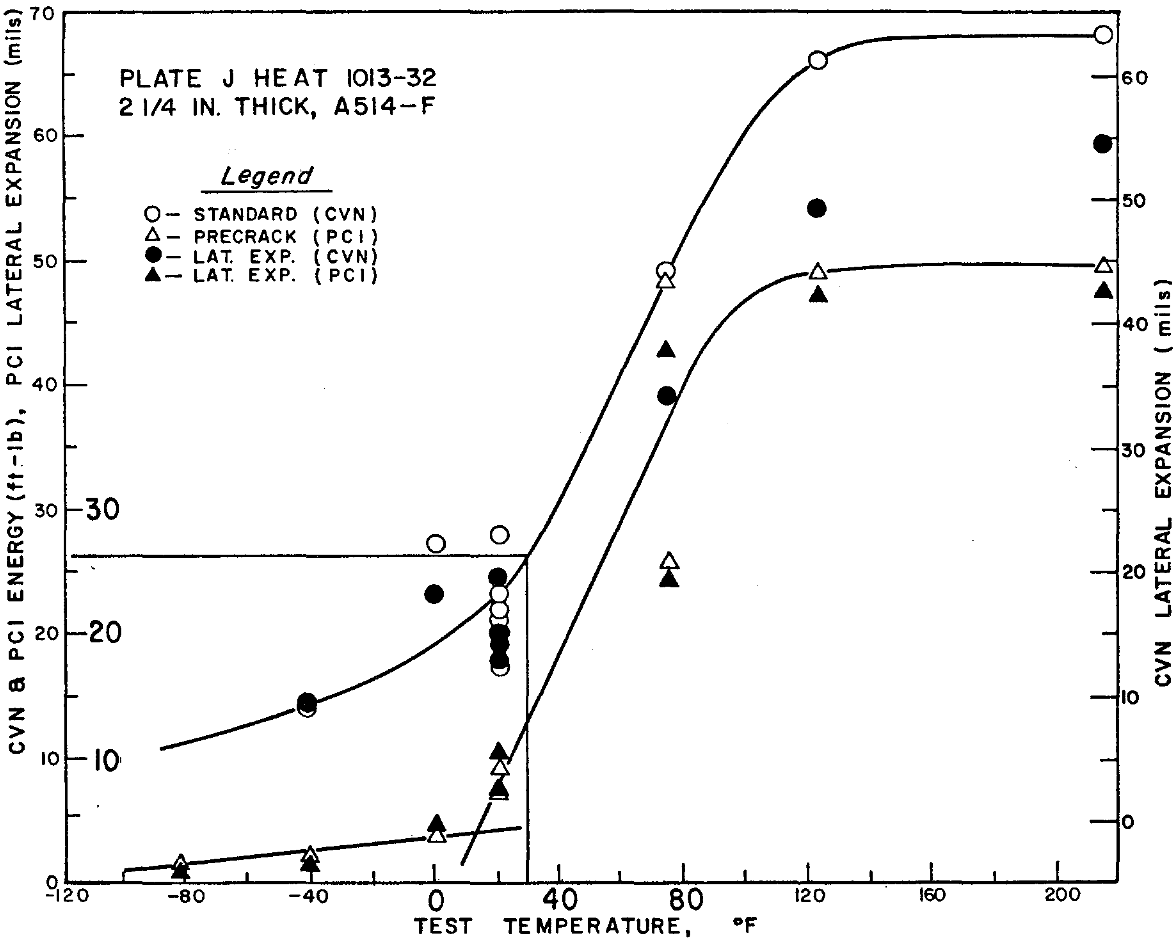


Figure C5.2.1.b - Charpy V-Notch and Precrack Charpy Impact Transition Curves for A514 Grade F Mill Heat 78L015-03W2.

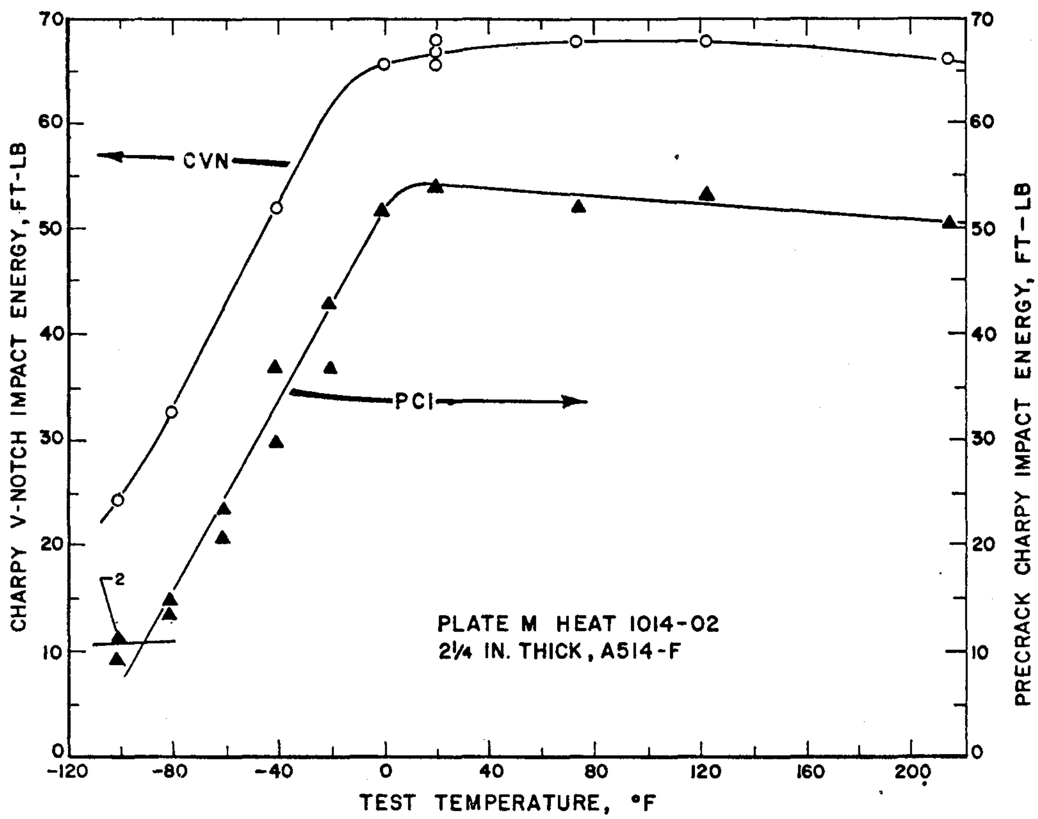


Figure C5.2.1.c - Charpy V-Notch and Precrack Charpy Impact Transition Curves for A514 Grade F Mill Heat 92L088-10W2.

base metal; if the weld is transverse to the principal stress, a pop-in crack may run catastrophically down the length of the weld. In either case, the result could be disaster in a FCM.

The chemistry of the weld metal will vary from pass to pass where there is fusion with the base metal; i.e., the amount of base metal melted by any given pass will affect the "dilution" of the weld-metal chemistry. When thickness permits, testing therefore is specified for multiple locations in the joint. When testing is specified "at both surfaces" this is intended to place the Charpy specimen as close as practicable to the face of the weld; i.e., the Charpy blank should be taken with one face of the blank in the plane of the as-rolled plate surface.

5.5 Limitation of Variables.

5.5.1.2 Here, as elsewhere in the fracture control plan, there is an assumption that the higher-strength steels, say over 90,000-psi yield strength, are a greater problem in fabrication than lower-strength steels. In general, when a steel is intended to provide high strength and high hardenability, the steel is more highly alloyed and will, therefore, affect the chemistry of the weld deposit by weld "dilution". Also, with higher strength steel and the attending higher design allowable, higher strength electrodes will be used. Thus, there are more strigent requirements for plate over 90-psi yield strength.

For plate up to 90-psi yield, qualification testing with plate 2-inches thick qualifies all plate of that grade/type in the range of 1 inch to 3 inches, inclusive (0.5 T to 1.5 T). For plate over 90-psi yield, qualification testing with plate 1-1/2 inches thick, qualifies plate of that grade/type in the range of 1 inch to 2 inches, inclusive, under the provisions of 5.5.1.2.

5.5.1.3 The chemistry of the consumables (wire, coatings, fluxes, etc.) used in welding have a marked effect on the mechanical properties of the deposited weld metal. The chemistry includes both the substitutional alloying elements and interstitial alloying elements, like hydrogen.

Books have been written on the behavior of hydrogen in steel. One of the adverse effects of hydrogen in weld metal is delayed cracking. Hydrogen comes from many sources, including the hydrogen in solution in the core wire of an electrode. Once in the weld metal, some of the hydrogen is combined with other elements, while the remaining hydrogen is free to diffuse through the metal. The diffusible hydrogen either collects at weld defects and causes slow crack growth, or slowly evolves from the weld (so-called out-gassing). The diffusible hydrogen, thus, is the bad actor in weld metal. Our neighbors in Canada require a test for diffusible hydrogen. (CSA Standard W487.)

The fracture control plan requires that the FCAW-electrode supplier certify the diffusible hydrogen content to be less than a specified level for his product (Table 4.18.1.1). This additional requirement is imposed on FCAW because there is no practical way to ascertain the moisture content of the flux core of the FCAW electrode.

When SAW flux and electrode are procured from a single supplier (which is sometimes not done), it would be appropriate to require a diffusible-hydrogen certification as for FCAW electrode. This then would certify a given heat/lot/batch of wire and flux as not exceeding the limits of Table 4.18.1.1.

5.6 Types of Tests and Purposes.

Many of the provisions of the fracture control plan from 5.6 through 5.13, inclusive, have been extracted from the State of California Department of Transportation Standard Specifications for Welding Structural Steel, January 1978, with minor revisions as deemed necessary for FCMs.

5.7 Base Metal and Its Preparation.

5.7.2 It should be noted that this provision requires qualification testing for each welding position (downhand, horizontal, vertical, overhead). The welding literature shows that the mechanical properties vary as a function of welding position; vertical welding, for example, will result in lower toughness than downhand. This requirement of qualifying each welding position is further covered in 5.8.1.

5.10 Test Specimens.

5.10.3 T-joints transverse to the applied stress must be groove welds under 5.2.1. However, attachments, stiffeners, diaphrams, etc., in this category should have fillet welds on either side of the groove weld to minimize stress concentrations. These fillets do not require weld-procedure qualification testing.

5.10.3.4 Fillet weld-procedure-qualification testing is important because large single-pass fillets tend to have low toughness in the coarse, columnar structure of the weld deposit, and also tend to develop microcracking in the HAZ at both the toe and root of the fillet weld. Because such cracking is minute and occurs at a geometrical discontinuity, it is difficult to detect with nondestructive inspection and is completely hidden at the root of a fillet weld. Magnetic-particle, dye-penetrant, eddy current and ultrasonic inspection are possible methods for detecting toe microcracking, but only ultrasonic inspection can be used for the detection of root microcracking. In either location, (toe or root), initially the cracking tends to be so fine and shallow that there is little chance of finding the cracking. When the welds (and, therefore, the microcracking at toe and root) are transverse to the applied stress, with high R values (ratio minimum/maximum stress), under traffic the cracks soon enlarge as fatigue cracks and, sooner, or later, may trigger brittle fracture.

CALTRANS requires weld-procedure-qualification testing of fillet welds by a test that reveals toe cracking (the keyhole T-bend test). The CALTRANS specification for fillet welds requires both the keyhole T-bend test for ductility and a fracture test for soundness. A toughness test is not required. NEW YORK STATE, on the other hand, requires the fillet-weld procedure to be used in depositing a groove weld. The advantage of the groove-weld for testing the fillet welding procedure is that it lends itself to Charpy V-notch impact testing; however, the tendency to toe cracking is not determined by this testing. Because toe and root fillet-weld cracking is suspected to initiate most of the fatigue cracking associated with fillet welds, the propensity to this type of cracking is an important consideration.

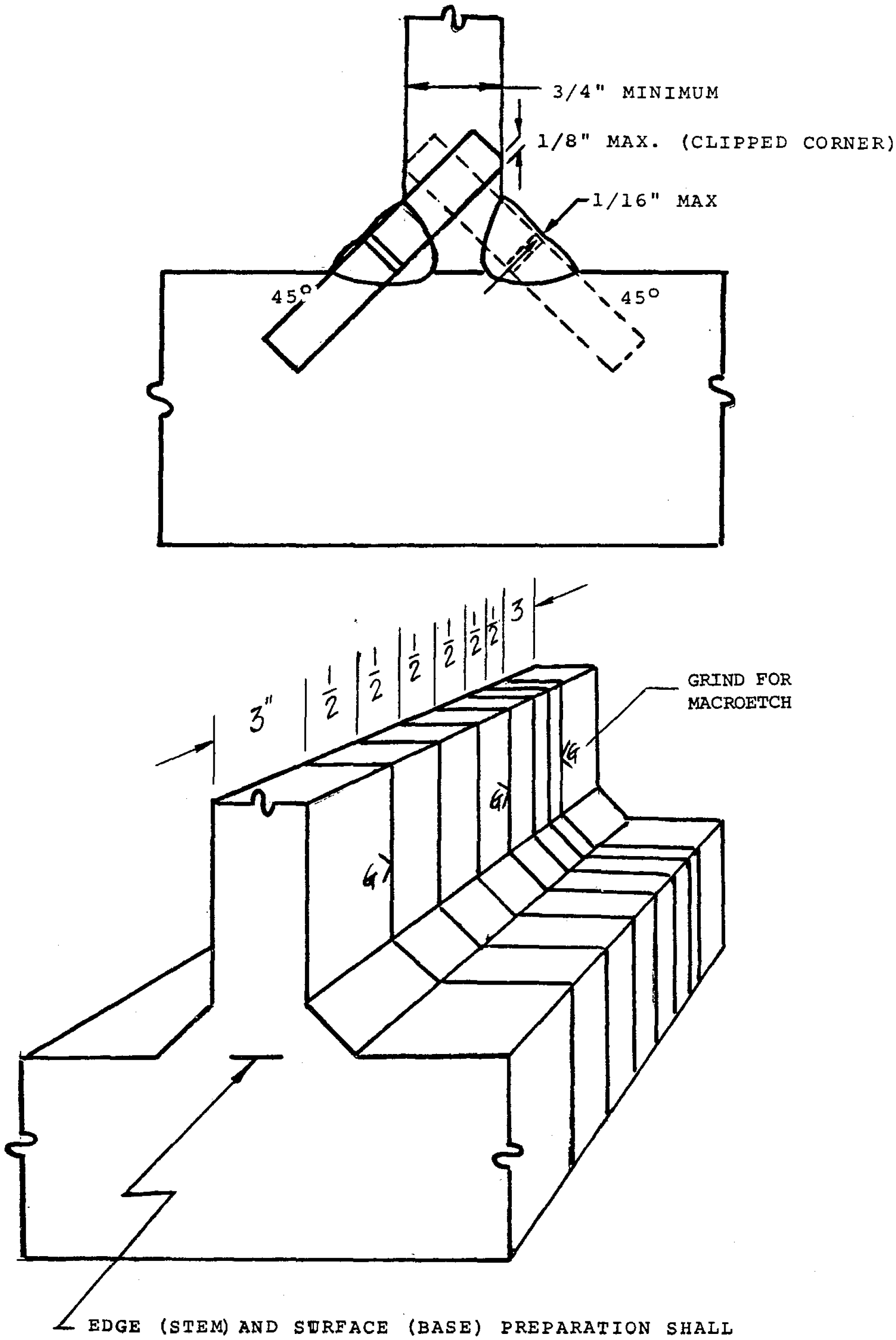
In box girders under tension (e.g., the tie girders of a tied arch) the corner joints can be fillets or incomplete-penetration groove welds. This involves less welding and, therefore, is less expensive and reduces the chance of lamellar tearing. Nevertheless, some designers, presumably with erection stresses in mind, require complete-penetration groove-weld corner joints in tie girders. When this is done, it is important to use joint designs that minimize the chance of lamellar tearing (ref. AISC - "Commentary on Highly Restrained Welded Connections", AISC Engineering Journal, third quarter/1973 and fourth quarter/1973, vol. 10 (4), pp. 112.).

In this fracture-control plan, due to the inherent notch effect associated with incomplete penetration and the inherent difficulties of inspecting such a joint, complete-penetration groove-weld butt-, corner-, and T-joints are specified for welds perpendicular to the applied stress for reasons previously stated but not without serious reservations about lamellar tearing and cost. In some designs there are many stiffeners transverse to the applied stress; thus, in deep box girders and in I-girders, the prohibition of incomplete-penetration fillet welds in tension is a costly and potentially troublesome requirement. Moreover, when stiffeners are opposite one another (on either side of the web of an I-girder, for example), there is an increased probability of lamellar tearing in the web plate. Unfortunately, at the present time there is no standard test for almellar tearing and, therefore, no requirement for lamellar tearing rests by the steel producer. Thus, the propensity of each heat of steel to lamellar tearing is unknown at the time of fabrication. Also, the toughness of incomplete-penetration fillet welds is unknown, and large single-pass fillet welds are likely to have low toughness and, therefore, a high fatigue crack growth rate both in the columnar weld metal (for root cracking) and in the HAZ (at the toe of the weld).

Incomplete-penetration fillet welds. To avoid the uncertainties of lamellar tearing with complete-penetration groove-weld T- and corner-joints (for welds perpendicular to the applied stress), incomplete-penetration fillet welds should be considered as an alternative. However, due to the inherent stress concentration associated with incomplete-penetration welds, when the welds are perpendicular to the applied stress, weld-procedure-qualification testing should include toughness testing. Figure C5.10.3.4.a shows one approach to Charpy testing fillet welds, but there are size limitations to this approach even when a subsize Charpy is used. As previously pointed out, New York State requires weld-procedure-qualification testing of fillet welds by making a groove weld using the fillet welding procedure. For purposes of this specification, in lieu of complete-penetration groove-weld corner- and T-joints for welds perpendicular to the applied stress, incomplete-penetration fillet welds may be used if the fillet-welding procedure is qualified by testing in accordance with 5.6.1.2 and Figure 5.10.3.4 which involves a 90-degree included angle butt joint to simulate the geometry inherent in fillet welding.

The temper-bead technique required in Figure 5.10.3.4 applies to the outer weld layer, i.e., in a very large fillet requiring more than three passes, the surface layer would be deposited in three passes, with the 3rd pass laid in place so as to not fuse to the base metal on either side of the weld bead. In preparing the weld-procedure-qualification test plate, a change in the sequence and/or positioning of weld passes where the weld-procedure qualification testing involves the temper-bead technique should be considered an essential variable under 5.5.2. The "position" of weld passes refers to the location of the respective passes relative to the two plates being joined; the "sequence" of passes refers to deposition of each pass, by the number, in the position called out in the welding-procedure specification. The concern here is holding to the sequence and positioning of passes necessary to the "temper-bead" technique".

The minimum toughness requirement for fillet welds in FCMs (5.12.1.7) may be a problem in single-pass fillets. The temper-bead technique consisting of three stringer beads in controlled sequence and placement should give the desired toughness if proper consumables are used. Also, fillet toe and root microcracking may be a problem in some heats of steel; again the temper-bead technique should minimize this problem by reducing the hardness in the heat-affected zone.



EDGE (STEM) AND SURFACE (BASE) PREPARATION SHALL BE THE SAME IN PRODUCTION WELDING AS IN PROCEDURE QUALIFICATION.

NOTES: THE SECTION SHOWN SHALL BE CUT FROM A WELDMENT BY EITHER SAWING OR FLAME CUTTING. THE SUB-SECTIONS FOR MACROETCH, HARDNESS AND CHARPY TESTING SHALL BE SAW CUT. THE MINIMUM DISTANCE FROM FILLET TO FLAME-CUT EDGE SHALL BE 3 INCHES.

TEST EVERY OTHER 1/2-IN. SECTION IN WELD "A" (3 SPECIMENS); LIKEWISE, FOR WELD "B". WELDS A AND B MAY REPRESENT DIFFERENT PROCEDURES (CONSUMABLES, HEAT INPUT, NUMBER OF PASSES, ETC.).

Figure C5.10.3.4.a - Weld-Procedure-Qualification Testing Fillet Welds with a Subsize Charpy Impact Specimen.

With regard to welding position, in the fabrication of large girders, where handling becomes a problem in the shop, there is a tendency to weld stiffeners, diaphrams, etc., in the vertical position. Unfortunately, welds are likely to have lower toughness when welded vertically upwards. Burdekin, et al point out that with vertical-up welding, a high heat input is virtually inevitable and there is also less effective shielding from the atomsphere, with a resultant loss of toughness (see Figure C5.10.3.4.b from IIW DOC. X-506-69. "Properties and Requirement for Weld Metal in Relation to Failure by Brittle Fracture" by Burdekin, Daws, Egan, Shackleton and Widgery of the British Welding Institute).

If the fabricator finds it necessary to make vertical fillets, where the welds are transverse to the applied stress in FCMs, the 90° bevel groove-weld fillet-weld procedure qualification plate also should be welded in the vertical position (see Figure 5.10.3.4).

5.11 Method of Testing.

5.11.2 The microetching procedure used here is that of CALTRANS. Other States use a saturated solution of ammonium persulphate. Obviously any etchant that clearly reveals the weld fusion zone and weld heat-affected zone will be acceptable for this test.

5.11.7.2 For the more sophisticated laboratory, the J-integral-test method is preferred for determining fracture toughness under elastic-plastic conditions. However, a less expensive, much less complicated test method is available as described by the National Materials Advisory Board in their report on "Rapid Inexpensive Tests for Determining Fracture Toughness (NMAB-328, National Academy of Sciences, Washington, D.C., 1976); the test method is based on the ratio of net-section stress at maximum load to the yield strength of the steel. Maximum load and the initial crack length are the only measurements required (the yield strength is obtained from the certified-mill-test reports).

Recently, Server and Wullaert in their report "The Use of Small-Specimen Strength Ratio for Measuring Fracture Toughness" (submitted for publication in Engineering Fracture Mechanics, December 1977) examined EPRI data from many heats of A533-B steel, where valid ASTM E399 K_{Ic} testing had been done, and compared these data with those based on specimen strength ratio, R_s.

From linear-elastic fracture mechanics concepts, K_{Ic} and net-section nominal stress, FTN, are directly related for linear-elastic fractures. The relationship, expressed in terms of

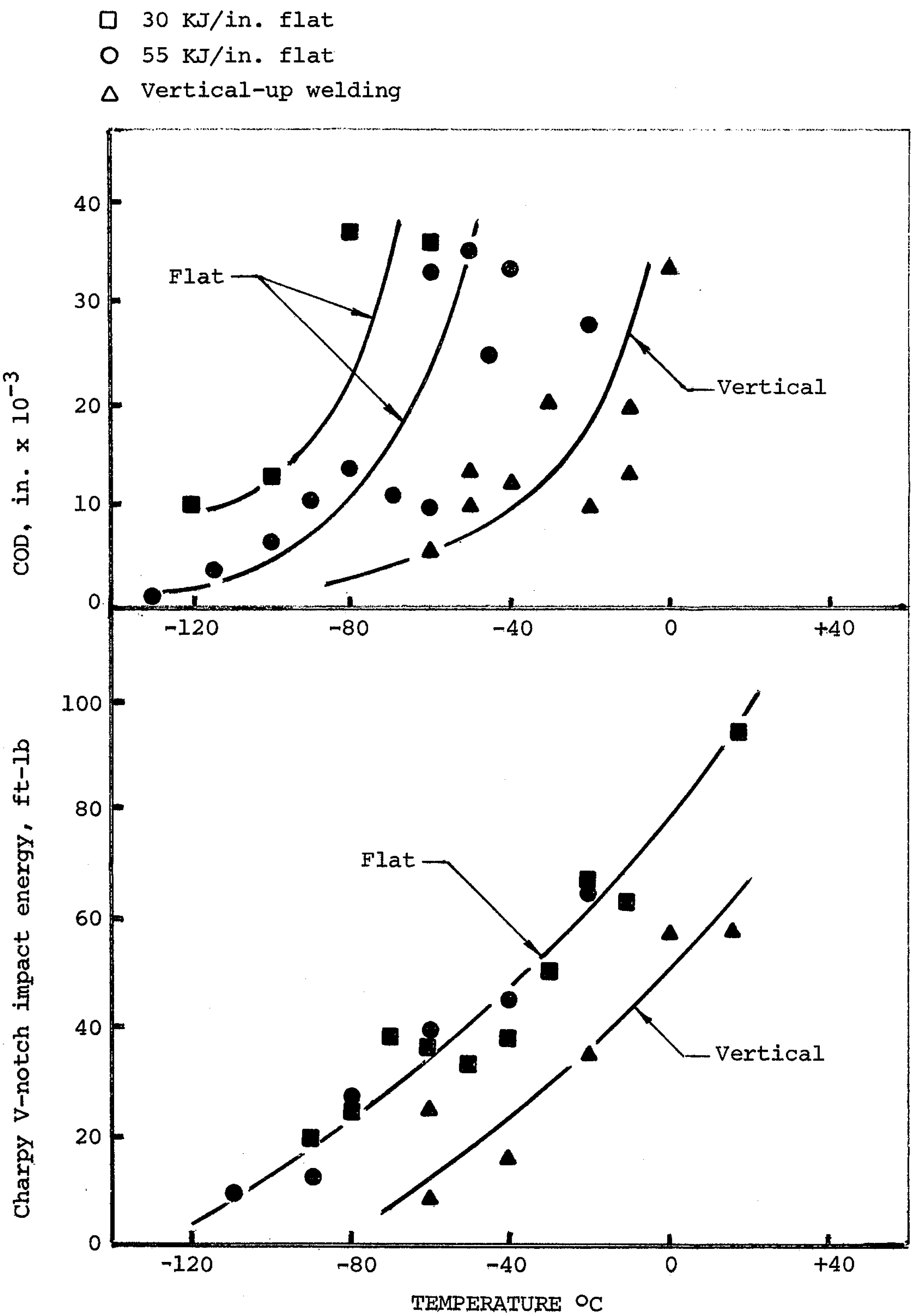


Figure C5.10.3.4.b - Effect of heat-input and welding position on fracture toughness (after Burdekin, et al).

slope, m, is as follows:

$$K_{Ic} = m \cdot FTN$$

or $$K_{Ic}/FTY = m(FTN/FTY)$$

where FTY is the yield strength of the steel as determined from the mill test report. Values of m as a function of a/W (crack depth to specimen width) for three specimen types were found by Server and Wullaert to be as follows:

SPECIMEN TYPE	m Values for Specified a/W values 0.45	0.50	0.55
1-in. compact tension	0.728	0.679	0.632
1-in. E399 bend	0.649	0.626	0.599
Precracked Charpy	0.288	0.278	0.266

The equations given for purposes of quality control in Table 4.1.1.2.1 are based on the above findings.

5.13 Recording and Certification of Procedure Qualification Test Results.

Weld-procedure qualification testing may be performed by a contracted inspection agency, testing laboratory, or the State in its own laboratory.

6. INSPECTION

PART A - GENERAL REQUIREMENTS

6.1 General.

6.1.1 The fracture control plan makes the fabricator responsible for determining the quality of his work.

6.1.2 The Welding Inspectors are NOT the same people who do the nondestructive examination of the welds; the welding inspectors may do much or eval all of the VISUAL inspection and make certain that the welding procedure specifications are followed within the limits set by AWS D1.1 paragraph 5.5.2, as well as other related welding quality-control responsibilities.

Both the QC and the QA Welding Inspectors must be qualified under the AWS Qualification and Certification Program.

6.1.3 The NDT Level-II Technicians performing RT and UT, whether in QC or QA, must be qualified under ASNT Recommended Practice SNT-TC-1A. The skills for UT are particularly critical. New York State Department of Transportation requires that anyone doing UT on their bridges be qualified by a written examination and performance test. The percentage passing the examination and performance test is shockingly low - fewer than 1 out of 5 are found to be qualified*.

The practical performance test consists of a weldment with an array of real flaws buried in a groove-weld butt splice. The person being examined has to find the flaws and estimate their size and position in the thickness. Such testing would seem reasonable - that's what the UT inspector is supposed to do when inspecting bridge girders. The fact that less than 20 percent are found to be qualified by the New York State test suggests that all States should require such a test - at least for FCMs.

6.2 Inspection of Materials.

6.2.1 If only the steel for main load carrying tension members is tested for toughness, it is imperative that the specified steel be properly placed in the girders during fabrication. The Welding Inspector is the logical person to see that this is done.

----

*Warren Alexander, private communication.

6.3 Weld Procedure Qualification.

6.3.1 One of the very important duties of the Welding Inspector is to supervise the preparation of the welding-procedure-qualification test plate. If the procedure qualification is to have any value, there must be assurance that the weld-procedure specification used in production is based on the welding procedure used in the qualification testing, within the limits of AWS D1.1 paragraph 5.5.2.

6.3.2 The welding-procedure qualification is so important that it is ess essential that the QA Welding Inspector certify to having witnessed the welding.

6.5 Inspection of Work and Records.

Footnote 27 of AWS D1.1 paragraph 6.1.1 clearly states that "The Inspector" acts for and in behalf of the Engineer, i.e., the owner. And then AWS D1.1 in paragraph after paragraph assigns responsibility for inspection to "The Inspector". Thus, AWS D1.1 sees inspection largely as the responsibility of the owner. The fracture control plan flatly rejects this concept, starting in 6.1.1, as amended, by distinguishing between Quality Control (QC) and Quality Assurance (QA), and assigning responsibility for QC to the fabricator and making QA the prerogative of the owner.

Thus, the "QC" Welding Inspector is a member of the fabricator's QC organization, or an inspection agency contracted by the fabricator.

AWS D1.1 paragraph 6.6.3 and 6.6.4 assigns responsibility to the contractor for visual examination and nondestructive examination, as required by contract, and assigns responsibility for the correction (repair) of defective welds. This is as it should be, but in paragraph 6.5 responsibility for inspection of the work is placed on "The Inspector", i.e., the owner. This appears to be contradictory, and is at least ambiguous.

The fracture-control plan assigns responsibility for the quality of the work to the fabricator. Under this plan, if defective welds or other unacceptable workmanship is found in the shop or on erection, there will be little or no question regarding liability. It is the prerogative of the owner to exercise QA and it is the responsibility of the fabricator ot exercise QC.

6.6 Obligations of the Contractor.

The provisions of the fracture control plan from 6.6 through 6.12, inclusive, have been extracted with minor modifications from the New York State Department of Transportation, Design and Construction Division, STEEL CONSTRUCTION MANUAL, as revised in 1977.

PART B

RADIOGRAPHIC TESTING

6.8.1 AWS D1.1 Section 6 - Part B makes no mention of ASTM Standard Methods E94 and E142. In the last decade there has been case after case of poor practice and a lack of control over the quality of RT. Clearly it is past time for the adoption of ASTM Standard Methods.

6.10.2 It is required in FCMs that butt welds be ground flush. If the fabricator wishes to determine the quality of his welds before grinding the welds flush, 6.10.2 specifies the limits of reinforcement at the time of RT. However, before UT is performed, it is required that all groove welds be ground flush.

6.10.11 On reinspection of groove welds, it is sometimes difficult to determine the exact location of a butt splice whether it be in the web or a flange. The center-punch marks required in the fracture control plan are located 12 inches away from the weld, with punch marks on each side of the weld, 3 inches in from the plate edge. THE WELD ITSELF IS NOT CENTER PUNCHED. The center-punch marks should be placed after edge preparation and before welding.

PART C

ULTRASONIC TESTING

6.13 General.

The provisions of the fracture control plan from 6.13 through 6.22, inclusive, have been extracted with minor modifications from the New York State Department of Transportation Design and Construction Division, STEEL CONSTRUCTION MANUAL of August 1, 1973 with addenda in August 1974 and February 1976.

6.14.2 Recent (1970s) experience with weld cracking in bridges in service indicates a high incidence of edge cracking at the ends of welds. Both RT and UT require extraordinary techniques for effective inspection at plate edges and at locations of intersecting welds; e.g., where the T-joints of transverse (to applied stress) stiffeners intersect web butt splices, or where corner joints of box girders intersect flange and web butt splices.

6.19.5.1 Figure 6.19.5.1 unlike AWS D1.1 Figure 6.22 shows no scanning pattern "E" which under AWS D1.1, paragraph 6.22.2.2 is used when the weld is NOT ground flush. IN FCMs all groove-welds must be ground flush before UT. When scanning pattern "C" is used in conjunction with scanning movement "A", this is tantamount to executing scanning pattern "E".

6.19.5.7 AWS D1.1 paragraph 6.19.5.7 specifically states that "Only those discontinuities which are rejectable need be recorded on the test report." The fracture control plan, paragraph 6.20.1, specifies that all indications with defect ratings within 5 db of being rejectable shall be recorded.

AWS D1.1 paragraph 9.25.3 (Ultrasonic Acceptance Criteria) allows discontinuities up to 3/4-inch long to be accepted when they fall in the category of "small" reflectors and allows discontinuities up to 2-inchs long to be accepted when they fall in the category of "minor" reflectors, and allows discontinuities in the middle half of the weld thickness to be accepted when they have a length greater than 3/4 inch and less than 2 inches even though they have a more serious (smaller number) than those of "minor" reflectors. The fracture control plan does NOT allow the latter. Moreover, the fracture control plan stresses the execution of the scanning patterns on a systemic, preplanned basis. If care is not taken to maximize the reflected signal by systematic manipulation of the transducer and a judicious selection of beam angle, planar defects in excess of 3/4 inch could be accepted.

California Department of Transportation in their Standard Specifications for Welding Structural Steel, January 1978, requires the use of an ultrasonic test record (Figures C6.19.5.7a, b) which calls for a record of the position of the ends of a discontinuity as well as the position of the transducer producing a maximized (most severe) defect rating. This is highly desirable from the standpoint of both making repairs (rejectable defects) and for future reference in determining possible crack growth (nonrejectable discontinuities). Note that CALTRANS requires reporting the findings from each side of the weld (Figure C6.19.5.7a) in searching for longitudinal defects and the findings from each direction of the weld (Figure C6.19.5.7b) in searching for transverse weld defects.

6.20.1 AWS D1.1 has no provision for recording nonrejectable indications. Several bridges reinspected after they were erected (sometimes new, sometimes after only a few years of service) have been found to contain weld cracks. It would have been extremely valuable to have a record of nonrejectable indications at the time of fabrication for reference today to see if there was slow crack growth. Moreover, in terms of future nondestructive examination of FCMs, it will be important to determine if there are any changes in the severity rating and/or dimensions of earlier nonrejectable indications.

CALTRANS in their UT DOCUMENTATION requirements specify that:

> "All defects, regardless of length, will be recorded if their evaluation severity is of a magnitude greater than that shown in Table 704.54 under Recordable Defect."

CALTRANS Table 704.54 specifies the "minimum defect requiring recording" as follows:

Plate Thickness (in.)			
1/2 to 1"	over 1 to 1 1/2"	over 1 1/2 to 2"	over 2 to 3"
+13 db	+11 db	+9 db	+6 db

New York State Department of Transportation specifies that:

> "All rejectable discontinuities shall be recorded in the test report. Other discontinuities shall be recorded if the defect rating is less than +6 db for buildings or +10 db for bridges.

Thus, both California and New York State are already requiring a record of nonrejectable indications for future reference.

Figure C6.19.5.7.a - CALTRANS Ultrasonic Test Record Sheet.

repair no.

ULTRASONIC TEST RECORD

CONTRACT NO. 03-052311 BRIDGE NO. 53-123 SCAN ORIG. ☒ R-1 2 3 4 5

MATERIAL A36 ☐ A441 ☐ A514 ☒ OTHER ____ THICKNESS 2¼ TO 2¼

SPLICE SHOP ☐ FIELD ☒ LONGITUDINAL – WELD ☐ SCAN ☒ TRANSVERSE – WELD ☒ SCAN ☐

WEB ☐ FLANGE TOP ☒ BOTTOM ☐ SCAN TOP ☒ BOTTOM ☐ GIRDER GA 21 SPAN 10 SPLICE W-2 ANGLE 45°

NORTH ↑☒ ←☐ LAMINATION yes ☐ no ☒ EAST ↑☐ →☐

scanning direction

4½" ← maximized location

extremities

℄ of weld

0" — 30"

3¼" 5¼"

path

Clear

magnitude → +2db V

½" south of ℄ → ½ ℄S

depth → ½" D ①

length → 2¼" L

flaw identification no.

SOUTH ↓☒ →☐ LAMINATION yes ☐ no ☒ WEST ↓☐ ←☐

4¼"

0" — 30"

3" 5"

+5db V

½ ℄S

½ D

2" L ①

Clear

MANUAL ☒ SIMIAUTOMATIC ☐ AUTOMATIC ☐ OTHER ____ SINGLE "V" ☐ DOUBLE "V" ☒ 50-50 ☒ 1/3-2/3 ☐

REMARKS: SURFACE CONDITION IS SMOOTH AND ACCEPTABLE

① REJECT

ULTRASONIC INSPECTOR John V Doe DATE 3-2-77

ULTRASONIC INSPECTOR R. J. Jones

Form HMR T-6043 (Orig 4/70)

COMPLETED ULTRASONIC TEST RECORD SHEET
(LONGITUDINAL SCAN)

Figure C6.19.5.7.b - CALTRANS Ultrasonic Test Record Sheet.

CONTRACT NO. 03-052311 BRIDGE NO. 53-123 SCAN ORIG ☒ R-1 2 3 4 5 ____

ULTRASONIC TEST RECORD

MATERIAL A36 ☐ A441 ☐ A514 ☒ OTHER ____ THICKNESS 2¼ TO 2¼

SPLICE SHOP ☐ FIELD ☒ LONGITUDINAL – WELD ☐ SCAN ☐ TRANSVERSE – WELD ☒ SCAN ☒

WEB ☐ FLANGE TOP ☒ BOTTOM ☐ SCAN TOP ☒ BOTTOM ☐ GIRDER GA 21 SPAN 10 SPLICE W-2 ANGLE 45°

NORTH ↑☐ ←☐ LAMINATION yes ☐ no ☐ EAST ↑☐ →☒

0" — 30"

9"

+5 db
¼"N → ½"S ℄
1"D
¾"L
2

22½"

+4 db
½"N → 1½"S ℄
½"D
2" L 3

SOUTH ↓☐ →☐ LAMINATION yes ☐ no ☐ WEST ↓☐ ←☒

0" — 30"

9"

+10 db
¼"N → ½"S ℄
1"D
¾"L
2

22¼"

+1 db
½"N → 1¾"S ℄
½"D
2¼"L 3

MANUAL ☒ SIMIAUTOMATIC ☐ AUTOMATIC ☐ OTHER ____ SINGLE "V" ☐ DOUBLE "V" ☒ 50-50 ☒ 1/3-2/3 ☐

REMARKS: 2 ACCEPT
3 REJECT

ULTRASONIC INSPECTOR John V. Doe DATE 3-2-77

ULTRASONIC INSPECTOR R. D. Jones

Form HMR T-6043 (Orig 4/70)

COMPLETED ULTRASONIC TEST RECORD SHEET
(TRANSVERSE SCAN)

APPENDIX A

Preface. Although some "write off" experience from merchant-ship failures as irrevelant to bridges, others contend that the experience is relevant and well worth studying. This Appendix is a brief overview of that experience starting with the Liberty Ship and T-2 Tanker disaster of World War II era and concludes with Lloyd's Register experience. The overall experience with brittle fractures in ships clearly indicates that Charpy V-notch impact testing must be done at the service temperature if the probability of brittle fracture is to be reduced to an acceptable level for FCMs.

THE STATISTICAL CORRELATION BETWEEN CVN-IMPACT and SHIP FRACTURE

Merchant ships are of welded construction, made of steel not much different from that used in many of our bridges. There is one notable difference but that is not reassuring - ship hull plating is usually of lesser thickness than the steel used in bridge girders. Research has shown, conclusively, that the tendency to develop brittle fracture increases with thickness.

Parker* has an excellent chapter on "Welded Ship Failures". The following paragraph is taken from his book because ironically it describes the first catastrophic ship failure, a failure remarkably similar to the last failure to occur - the Martha R. Ingram, a tank barge which fractured on January 10, 1972.

> "The first Liberty ships were placed in service near the end of 1941, and by January of 1943 the number in service had reached 500. By this time 10 fractures in hull structures that were serious enough to endanger the vessel had been reported. Most of these fractures had occurred during the winter of 1942-43. These failures were under study by the American Bureau of Shipping staff at the time of the spectacular failure of a T-2 tanker on the West Coast. On January 16, 1943, while lying quietly at her outfitting dock, she suddenly broke in two. The extent of the fracture in this vessel, occurring spontaneously with no apparent cause, was without precedent in shipbuilding history. The fracture extended across the deck just

---

*Earl R. Parker, BRITTLE BEHAVIOR OF ENGINEERING STRUCTURES, Wiley & Sons, Inc., New York, 1957, pp. 273-302.

> aft of the bridge and about midship; the break extended down both sides and around the bilges but did not cross the bottom shell plating. The fracture traversed all girders and plating, except the flat portion of the bottom shell plating, thus almost completely severing the ship... The calculated stress in the crown on the deck was only 9900 psi at the time of the failure."

The American Bureau of Shipping appointed a board of investigation. They found:

> "an accumulation of an abnormal amount of internal stress locked into the structure by the process used in construction together with an acute concentration of stress caused by defective welding at the starboard gunwale in a way of the abrupt ending of the bridge fashion plate, augmented by the hogging stress due to the ballasted condition."

In the failure of the Martha R. Ingram on January 10, 1972, almost thirty years later, the vessel was at mooring, broke almost completely in half, and one of the contributing factors was said to be her ballasted condition. The National Transportation Safety Board determined the probably cause to be:*

> "(1) the incapability of the steel, approved by the American Bureau of Shipping, to resist the INITIATION OF THE BRITTLE FRACTURE and (2) the failure of the 'special material' crack arrestors to STOP THE CRACK PROPAGATION. Contributing to the casualty were (1) lack of a requirement in the design procedures for a substantiation of the FRACTURE-SAFE limitation of the vessel, and (2) failure of the steel-grading system to provide information about the TRANSITION TEMPERATURE OF THE STEEL installed in the hull and to guarantee that supposedly higher grade steels selected for critical locations have greater resistance to brittle fracture than adjacent lower grade steels." (Emphasis added)

In both of these failures the loading was static; the ship was either dockside or at mooring. NO 70 DEGREE CORRECTION WAS NEEDED OR EVEN CONSIDERED in analyzing the CVN-impact test results from ship-failure

---

*"Structural Failure of the Tank Barge I.O.S. 3301 Involving the Motor Vessel Martha R. Ingram on January 10, 1972 Without Loss of Life." Report USCG/NTSB-Mar-74-1, December 28, 1973.

steels. As a matter of fact, in the Martha R. Ingram, the IMPACT test results indicated the steel to be tough; whereas, the cracked vessel proved the steel to be brittle at 45°F. Professor Parker pointed to other instances where fracture occurred in calm seas or at dock:

> "During the remainder of the 1942-43 winter, several more serious failures occurred. A large bulk freighter under construction on the Great Lakes suddenly fractured across the deck and part way down the sides. A Liberty ship being loaded at a pier suddenly fractured; the crack extended part way down one side. In March 1943 another T-2 tanker split in two at the entrance to New York Harbor; the sea was calm, and the calculated (still water) stress in the crown of the deck was only 12,200 psi. For the next several years there were frequent reports of fractures in welded ships."

It is a fact that most failures occurred during cold weather and in heavy seas. Nevertheless twenty-three (about 10 percent) of the Class-1 failures* occurred in calm water (10 Liberties, 9 T-2 tankers and 4 others). These ships were built in twelve different yards. The average temperature of such failures was about 15°F lower than for the heavy-weather casualties.

CVN-impact statistic from the National Bureau of Standards.

The Ship Structure Committee, in December 1967, published a report** entitled, "Twenty Years of Research Under the Ship Structure Committee". The results of laboratory tests were compared with ship performance. All testing was done or related to THE ACTUAL SERVICE TEMPERATURE AT WHICH SHIP FAILURES OCCURRED.

> "Figure 10 summarizes the correspondence of the ship fracture test data from the National Bureau of Standards and the crack-starter test data from the Naval Research Laboratory. The NBS data showed that the 'source' - plates (samples of plates at which whip fractures started) had Charpy V-notch energy values AT THE FRACTURE INITIATION TEMPERATURES of 11.4 ft-lb maximum and 7.4 ft-lb average. By

---

*Class 1 failures are fractures which weaken the main hull structure so that the vessel is lost or is in a dangerous condition.

**Captain S. R. Heller, Jr., USN, LCDR R. Nielsen, Jr., USCG, A. R. Lytle and John Vasta, SSC-182, December 1967.

> comparison, the NRL drop-weight test data were in complete agreement that the temperature corresponding to Charpy V-notch energy of 10 ft-lb was the critical temperature for fracture initiation of ABS World War II steels. Similarly, in the NRL explosion bulge test, steels having a range of Charpy V-notch energies of 13 to 27 ft-lb demonstrated 'T' ('through' or complete failure) and 'S' ('stop' or partial failure) conditions in the edge regions, respectively. This meant that, if a plate had an impact energy in excess of 27 ft-lb, propagation was prevented and, if it had an impact energy of less than 13 ft-lb, propagation was permitted. This was in excellent agreement with the NBS finding that 19 ft-lb was the highest Charpy V-notch energy for a 'through' plate. Thus, the NRL crack-starter tests correlated very well with service experience and bolstered the NBS findings regarding the significance of Charpy test data." (Emphasis added)

Thus, studies of scores of ship failures over the years provide a statistically significant correlation between the CVN-IMPACT test and the in-service performance of WWII ship hulls WHEN TESTING IS DONE AT THE SAME TEMPERATURE AS THE SERVICE FAILURE. And this observation holds in spite of the fact that a significant number of the ship failures occurred in still water; i.e., under static-loading conditions.

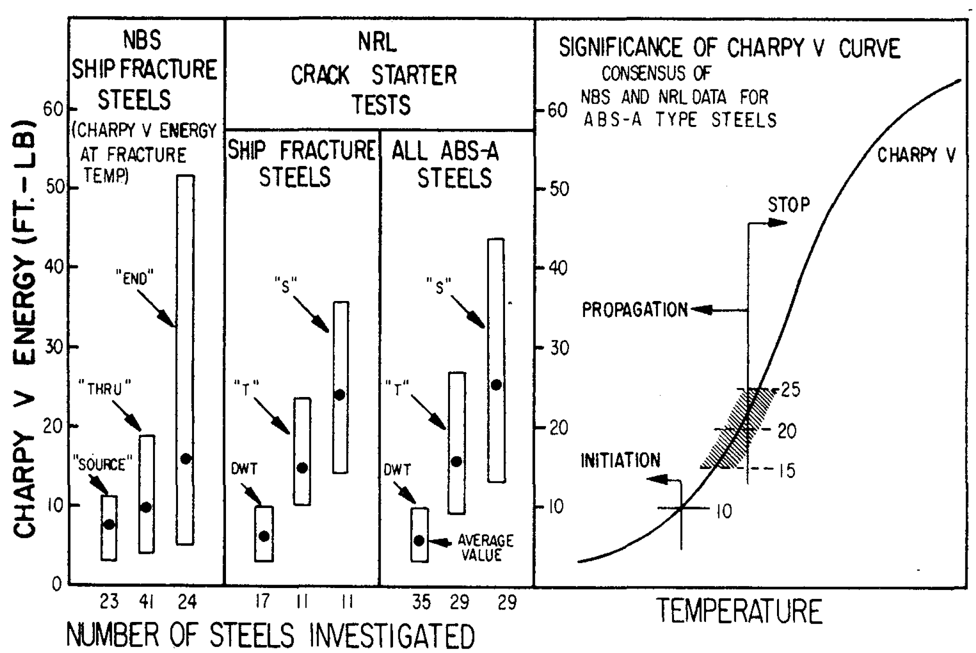


Fig. 10 Correspondence of NRL crack-starter test data to NBS ship fracture data

Lloyd's Register of Shipping's approach to the control of the incidence of brittle fracture in ship structures is an insurer's point of view and, therefore, perhaps close to the owner's and user's point of view (in contrast to the steel producer who must contend with problems of profit margin, production schedules, etc.).

In a report prepared by Lloyd's Register for the International Institute of Welding IIW)* based on data collected on brittle fracture in ships, the following findings emerged:

"(1) The number of failures has been small in comparison to the number of ships in service.

"(2) The age of a ship does seem to play some part in the incidence of fracture. The incidence of fracture decreases with the length of life of a ship, i.e., more fractures have taken place in, say, the first five years of a ship's life than subsequently.

"(3) Many of the service brittle fractures occurred at moderate or low nominal stress.

"(4) <u>Many of the fractures occurred without any impact loading, i.e., under essentially static conditions</u>.

"(5) Although fatigue cracks are known to be conductive to the initiation of brittle fracture, fatigue does not appear to be a necessary prerequisite. Many service fractures have occurred in circumstances which have precluded fatigue as a factor.

"(6) Residual stress alone does not appear to be a major cause of failure, but it is probable that residual stresses which are not relieved in service, in association with other factors, may initiate brittle failure.

"(7) Although many of the fractures have originated at or near welds, the crack path has generally been quite random in relation to the welding, and only in cases of gross defects in the welding has the fracture followed the welded seams.

----

*IIW Doc. X-515-69 by G. Buchanan, Deputy Chief Ship Surveyor, C. J. G. Jensen, Deputy Head of Research and Technical Advisory Services Department and Ir. R. J. C. Dobson, Senior Surveyor, Ship Development Section - all of Lloyd's Register of Shipping.

"(8) The temperature at the time of failure has had a strong influence on the extent of failure with fractures being more brittle, more frequent and more extensive at lower temperatures.

"(9) Most major brittle fractures in service have progressed at very high speeds." (Emphasis added)

Data from the report (tabulated on the next page) show that there is a decrease in the incidence of fracture in shell and upper-deck plating of the newer ships as compared with the older ships, whether measured by number of ships with fractures or by percentage of ships with fractures. The decrease in the incidence has been more marked in tankers than in dry-cargo ships and this in spite of the fact that the increase in use of "special steel" has been greater for dry-cargo ships.

"These figures reflect the more complicated nature of the dry-cargo ship's structure, and its greater number of potential crack initiation sources. In all groups there have been few occurrences of fracture in special."

In Lloyd's current fracture-control requirement, Grade D special steel must develop 35 ft-lb CVN-impact tested at 32°F and Grade E special steel must develop 45 ft-lb CVN-impact tested at 14°F.

"The temperature basis for the energy requirement was determined from a statistical study of a large number of ships which indicated that the lowest significant service temperature likely to be encountered lies around 0°C."

The following table summarizes the properties required in the "special steels":

	Special Steel	
	Grade D	Grade E
Deoxidation	any method except rimmed	fully killed fine-grain practice
McQuaid-Ehn Grain Size	--	5 or finer
Ladle Analysis		
Carbon	0.21 max	0.18 max
Manganese	0.60-1.40*	0.70-1.50*
Silicon	0.35 max	0.10-0.35
Sulfur	0.05 max	0.05 max
Phosphorus	0.05 max	0.05 max

*The sum of the carbon and 1/6 of the manganese shall not exceed 0.40 percent.

FRACTURES IN STRENGTH DECK AND SHELL PLATING

	Tankers		Dry Cargo	
	Old*	New+	Old*	New+
Number of ships at risk	889	1,362	2,240	3,039
Number of ships affected	53	29	88	89
Percentage of ships affected	5.96	2.13	3.93	2.86
Number of occasions	74	30	97	99
Number of ships with special steel	326	596	434	1,172
Percentage of ships fitted with special steel	37	44	19	39

*'Old' means ships built 1943-1957 inclusive

+'New' means ships built 1958-1967 inclusive

Years service counted to end of each period, i.e., 1957 and 1967 inclusive, or to exit from class, whichever is the shorter.

Tensile Strength		
kg/mm^2	40 to 50	41 to 50
(ksi)	(57 to 71)	(58 to 71)
Impact Test	$0^{o}C$ ($32^{o}F$)	$-10^{o}C$ ($14^{o}F$)
minimum energy	35 ft-lb	45 ft-lb

The basis for the 35 ft-lb CVN-impact energy requirement is the Hodgson-Boyd diagram (see the following page). The diagram was obtained from a plot of average CVN-impact energy values against percentage fibrous fracture appearance of specimens taken from a large number of casualties and TESTED AT THE CASUALTY TEMPERATURE.

Some readers may take comfort in the fact that they have no personal experience with brittle fracture in ships or bridges. Some readers may take comfort in the low percentage of brittle fractures. In Lloyd's statistic the percentage of ships involved in brittle fracture was:

3.9 to 5.9% in the "old" design

and

2.1 to 2.9% in the "new" design

To an engineer charged with the responsibility of fracture-safe design, 2 to 6 percent is not a small number. Another way to look at the situation - would you want even one brittle fracture if you were involved professionally? And more to the point, you or some member of your family might be on that one ship (or bridge) that fails. This is without mention of the great loss in time and money always associated with brittle fracture, whether it occurs during construction or in service.

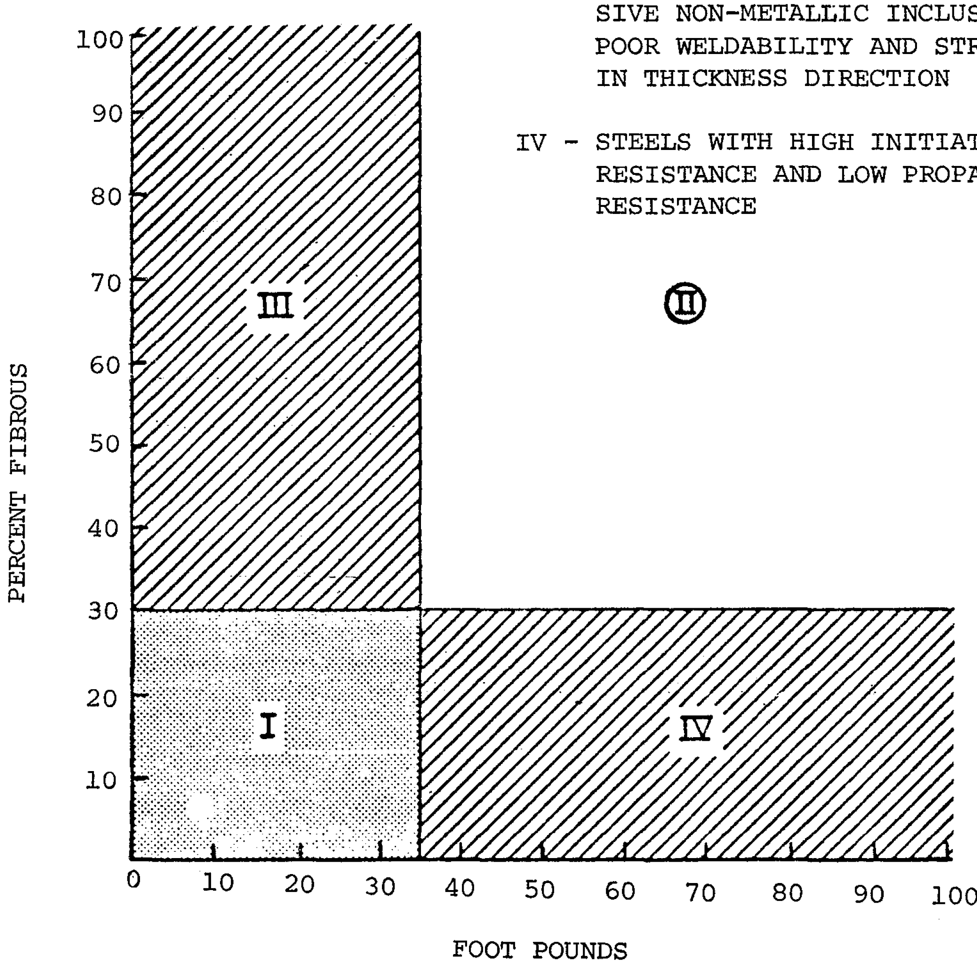
REGIONS

I - "CASUALTY" ZONE

II - "SUCCESS" ZONE

III - STEELS LIKELY TO SHOW EXCESSIVE NON-METALLIC INCLUSION, POOR WELDABILITY AND STRENGTH IN THICKNESS DIRECTION

IV - STEELS WITH HIGH INITIATION RESISTANCE AND LOW PROPAGATION RESISTANCE



HODGSON AND BOYD DIAGRAM

APPENDIX B

WELDABILITY in AWS/AASHTO

The following summarizes the paragraphs contained in the code which refer to "weldability". A review of these provisions reveals that the specifications as presently written are incomplete and in someplaces ambiguous. The AWS Structural Welding Code gives no direction whatsoever as to how weldability is to be evaluated, and AASHTO calls out a weldability evaluation that is impractical at best.

The AWS D1.1-75 Structural Welding Code.

9.2 Base Metal.

9.2.2 "When an ASTM A709 Grade structural steel is considered for use, ITS WELDABILITY SHALL BE ESTABLISHED BY THE STEEL PRODUCER and the procedure for welding it shall be established by qualification in accordance with the requirements of 5.2... with the following exception: if the grade to be supplied meets the chemical and mechanical properties of ASTM A36, A572 Gr. 50, A588 or A514, the applicable prequalified procedures of this code shall apply." (emphasis added)

Comment: What weldability testing does the committee have in mind?

9.2.3 "When an ASTM A242 or A618 Grade I, low-alloy steel is considfor use, ITS WELDABILITY SHALL BE INVESTIGATED BY THE ENGINEER, and..." (emphasis added)

Comment: What weldabilitv investigation does the committee have in mind? The engineer probably knows little if anything about weldability testing and we are requiring that he investigate the weldability of a steel. Nowhere in the code is there any direction provided to what constitutes a "weldability investigation". Because of this lack of direction from the AWS Structural Welding Committee, AASHTO in 1974 specified a WELDABILITY INVESTIGATION.

9.2.4 "When a steel other than one listed in 9.2.1 is approved under the provisions of the general bridge specifications and such steel is proposed for welded construction, THE WELDABILITY OF THE STEEL AND THE PROCEDURE FOR WELDING IT SHALL BE ESTABLISHED BY QUALIFICATION..." (emphasis added)

Comment: Here there is a subtle but important difference in wording as compared with paragraph 9.2.2. In 9.2.2 the weldability testing and the procedure qualification are clearly separate considerations. In 9.2.4 it states that "...,the weldability of the steel and the procedure for welding it shall be established by qualification in accordance with the requirements of 5.2..." THUS, FOR STEELS OUTSIDE 9.2.1, THE WELDABILITY IS ESTABLISHED BY PROCEDURE QUALIFICATION UNDER 5.2. Is this the intent of this committee?

9.2.4.1 "The responsibility for determining weldability, including the assumption of the additional testing costs involved, is assigned to the party who either specifies a material other than listed in 9.2.1 or who proposes the use of a substitute material not listed in 9.2.1. The fabricator shall have the responsibility of establishing the welding procedure by qualification."

Comment: Is this consistent with 9.2.4 which states that both the weldability and the qualification of procedure shall be accomplished by 5.2? It appears from 9.2.4.1 that two separate and distinctly different efforts are intended: (1) weldability testing and (2) procedure qualification. Mentioning the additional costs involved in weldability testing implies some specific plan for "determining the weldability".

9.2.5 "Extension bars, run-off plates and backing used in welding shall conform to the following requirements:

(1) When used in welding with an approved steel listed in 9.2.1, it may be any of the steels listed in 9.2.1."

Comment: Among the steels listed in 9.2.1 are A514 and A588 would this committee allow the use of either of these steels as backing for A36 and A441? The weldability for A588 and A514 is not the same as that of A36 or A441 and, therefore, a different level of preheat is required for the steels with greater alloy content. Isn't it conceivable that cracking could occur in the backing and propagate into the parent weld? In this connection, shouldn't there be some provision in the code to control the material used as a spacer in joints such as B-U3a, TC-U5a and B-U3a-S? The chemistry of root passes could be adversely affected if free-machining stock or a piece of rebar were used (as far-out examples of what could happen).

<u>The AASHTO-74 Standard Specification for Welding of Structural Steel.</u>

9.2 <u>Base Metal</u>.

9.2.15 "Specifications for high-yield strength, quenched-and-tempered alloy steel plate, SUITABLE FOR WELDING, AASHTO M244 (ASTM A514)."

<u>Comment</u>: The steel producers are prone to consider all A514/517 steel "suitable for welding". The particular heat of A517-H steel that initiated fracture in the Bryte Bend Bridge (Sacramento, California) was found to be highly susceptible to hot cracking.* A hot crack initiated slow crack growth which in turn triggered brittle fracture.

9.2.2 "When an AASHTO M161 (ASTM A242) type of low-alloy structural steel is considered for use, it shall be made the subject of <u>a special investigation by the Engineer as to weldability</u> in accordance with Part IIA of Section 5 of this code..." (Emphasis added)

Comment: ASTM A242 Type 1 steel is allowed 0.15 phosphorus. Liquation cracking is principally affected by the carbon, sulfur, <u>phosphorus</u> and manganese, and in heats made with a scrap-melting process, arsenic, antimony, tin and boron also can be a problem. The reason for the special attention given to A242 probably stems from this type of cracking and unfortunately, liquation cracking is not detected by the longitudinal bead-weld underbead-cracking test. Use of Type 1 A242 should be prohibited in bridges or there should be a requirement for a hot-cracking test such as the VARESTRAINT test (see Figure 8).

9.2.3 "When a structural steel other than those listed in 9.2.1 is approved under the provisions of the general bridge specifications and such steel is proposed for welded construction, <u>a weldability investigation of the steel shall be made in accordance with Part IIA of Section 5 of this Code</u>. When such steel has been approved for welding, and also, when ASTM A514/A517 steel is to be welded, the welding procedure shall be submitted to the Engineer for approval and the procedure qualified by tests as prescribed in Part II of Section 5 of this Code prior to the start of production welding. The Engineer, at his discretion, may accept evidence of previous qualification of the procedure to be used."

---

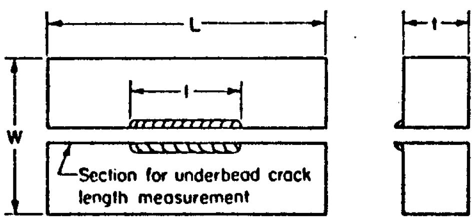
*Professor Warren Savage, private communication.

Comment: Reference to Part IIA of Section 5 of the AASHTO Code refers to the bead-on-plate weldability test. In connection with A514/517 steel the Specification calls for procedure qualification "by tests as prescribed in Part II of Section 5 of this Code prior to start of production welding". There is no way that one can derive a qualified procedure from a go-no-go bead-on-plate weldability test. The intent here is to require procedure qualification of A514/517 steel based on Section 5 part B of the AWS Code as well as to require weldability evaluation. The latter should be mandatory; i.e., not to be waived at the Engineer's discretion.

General comment: Many tests can and are being used by other industries and in other countries to determine the susceptibility of a steel to cold cracking; unfortunately, the useful ones are all more expensive than the bead-on-plate test but also they are more informative. The salient features of some of these tests have been summarized by the Defense Metals Information Center, Battelle Memorial Institute (reference M. D. Randall, R. E. Monroe, and P. J. Rieppel, "Methods of Evaluating Welded Joints", DMIC Report 165, December 28, 1961). The following Figures from the above report describe some of the weldability tests.

SPECIMEN 80

UNDERBEAD-CRACKING SPECIMEN - LONGITUDINAL WELD



Dimensions, inches			
l	L	W	t
3	6	3	Plate t
1-1/4	3	2	Plate t

Smaller dimensions used in early research on underbead cracking. Bead usually deposited at room temperature but subzero as well as preheat temperature used.

Material Evaluated - Steels.

Purpose of Test - Crack susceptibility and, notably, underbead cracking tendencies.

Number of Specimens Tested - Usually 3.

Nondestructive Inspection (in order of decreasing use) - Visual, magnetic, penetrant, and X-ray.

Important Variables - Specimen geometry, weld quality, welding conditions, weld inspection, temperature of specimen when welded, and data analysis.

Data Obtained - Extent and type of cracking.

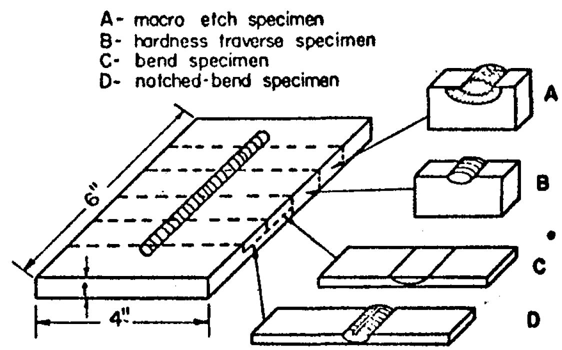
Specifications - None.

Reference - Stout, R. D., and Doty, W. D., Weldability of Steels, Welding Research Council, 1953, 225-229.

Remarks - Specimen developed to study underbead cracking. Length of cracking expressed as percentage of bead length. Although extent of cracking varies with individual specimens, average of ten tests reproducible within 10 per cent; average of 5 tests reproducible within 20 per cent.

SPECIMEN 79

BEAD-ON-PLATE WELDABILITY SPECIMEN



Dimensions often changed to vary cooling rate.

Materials Evaluated - All.

Purpose of Test - Crack susceptibility; crack propagation; weld and HAZ hardness, bend ductility; study effects of welding conditions such as heat input, preheat, etc.; occasionally used for quality control.

Number of Specimens Tested - Usually a single specimen.

Nondestructive Inspection (in order of decreasing use) - Visual, X-ray, penetrant, and magnetic particle (where possible).

Important Variables - Specimen geometry (only as it affects weld cooling rate), weld quality, weld conditions, weld inspection, and data analysis.

Data Obtained - Hardness, soundness, microstructure, and elongation (in the case of bend specimens).

Specifications - None.

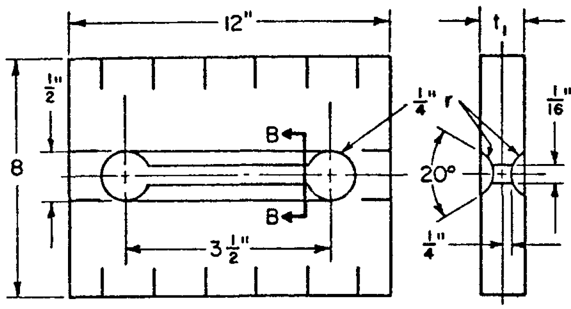
References - Henry, D. H., and Claussen, G. E., Welding Metallurgy, 2nd Edition, revised by Linnert, G. E., American Welding Society, pp 481-483.

Remarks - Simple but extremely useful test.

FIGURE 1

SPECIMEN 59

CRACK-SUSCEPTIBILITY SPECIMEN – LEHIGH



Dimensions, inches									
l_1	L	W	X	W_2	W_3	t_1	t_2	ϕ[a]	R
3-1/2	12	8	Var.	1/2	1/16	<1	1/4	20°	1/4
5-1/2	12	8	Var.	1/2	1/16	>1	1/4	20°	1/4

Dimension X usually varied in 1/2-inch increments. Temperature of base plate at time of depositing test weld varied depending on condition studied, i.e., preheat, ambient temperature, etc.

Materials Evaluated - All (used only for steels in survey).

Purpose of Test - Crack susceptibility; developmental.

Number of Specimens Tested - Usually 2 (see test procedure below)

Nondestructive Inspection (in order of decreasing use) - Visual, penetrant, magnetic, and radiographic.

Important Variables - Specimen geometry, weld quality, weld inspection, and data analysis.

Data Obtained - Degree of restraint necessary to cause cracking.

Specifications - None.

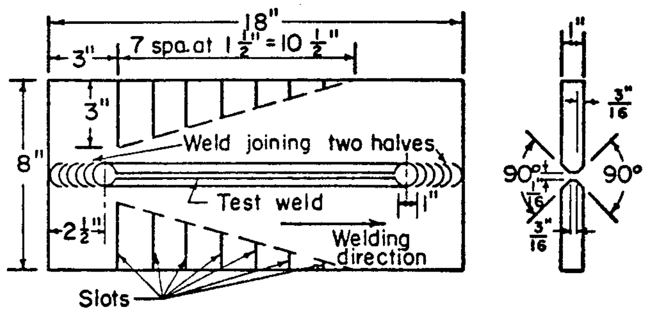
References - Stout, R. D., and Doty, W. D., Weldability of Steels, Welding Research Council, 1953, pp 232-234. Borland, J. C., "Cracking Tests for Assessing Weldability", British Welding Journal, 7 (10), 623 (1960).

Remarks - This test was designed to obtain, quantitatively, the degree of restraint necessary to cause weld-metal cracking during cooling. It is a go - no go test and requires several specimens to estimate cracking susceptibility. Variables that can be studied include base plate, filler metal, preheat, postheat, heat input, and effects of multipass welding. Generally, the first specimen welded will be under full restraint (no saw cuts). If this specimen cracks, another specimen is welded that contains less restraint (saw cuts). Sufficient specimens are welded until a restraint level is reached at which no further cracking occurs.

FIGURE 2

SPECIMEN 82 – EXTRA

CRACK-SUSCEPTIBILITY SPECIMEN – BATTELLE



Test weld deposited in groove in direction shown; welded with or without preheat in air.

Materials Evaluated - Plain-carbon, low-alloy high-strength, and ultrahigh strength steels.

Purpose of Test - Developmental, weldability, and crack susceptibility.

Number of Specimens Tested - Usually 3.

Nondestructive Inspection (in order of decreasing use) - Visual, radiographic, and penetrant.

Important Variables - Specimen geometry, weld inspection, crack-length determination, and data analysis.

Data Obtained - Unslotted width of specimen at point where hot cracks initiate; this is considered the cracking index of the test weld.

Specifications - None.

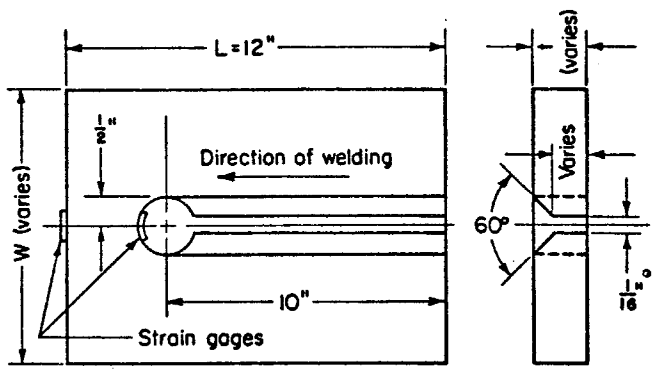
References - Mishler, H. W., Monroe, R. E., and Rieppel, P. J., "Determination of the Causes of Weld Metal Cracking in High Strength Steels and the Development of Heat-Treatable Low-Alloy Steel Filler Wires for Use With the Inert-Gas-Shielded Arc Welding Process", WADC Technical Report 59-531 (August 1959).

Remarks - Considered a good, semiquantitative test for evaluating relative cracking susceptibility; had advantage of limited testing to determine a cracking index as compared with other go - no go tests.

FIGURE 3

SPECIMEN 65

CRACK-SUSCEPTIBILITY SPECIMEN – U. S. NAVAL RESEARCH LABORATORY



Restraint varied by varying L, W, and t; normally L held constant: example - for t = 1/2 inch, W varied from 6 to 10 inches.

Materials Evaluated - Although only used for low-alloy, high-strength steels by surveyed organizations, probably could be used for all materials.

Purpose of Test - Developmental, weldability, and crack susceptibility.

Number of Specimens Tested - Usually 3.

Nondestructive Inspection - Visual and radiographic.

Important Variables - Specimen geometry, weld quality, welding conditions, weld inspection, instrumentation, measurements, and data analysis.

Data Obtained - Extent and type of cracking.

Specifications - None.

References - White, S. S., Moffatt, W. G., and Adams, C.M., Jr., "Dynamic Measurements of Stress Associated With Weld Cracking", Welding Journal, 37 (4), p 185s (1958). Borland, J. C., "Cracking Tests for Assessing Weldability", British Welding Journal, 7 (10), 623 (1960).

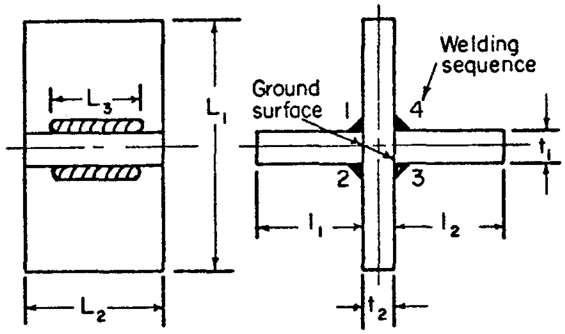
Test Procedure - Weld deposited in groove beginning at edge of plate and proceeding toward hole; strains from two gages recorded continuously during welding and as weld cools to room temperature; gross cracking indicated from strain measurements.

Remarks - A semiquantitative test to determine if and when cracking occurs; cracking usually occurs along weld and occasionally in HAZ; simultaneous monitoring of strain, time, and temperature of weld can provide useful information regarding crack formation, particularly delayed cracking (hydrogen induced).

FIGURE 4

SPECIMEN 68

CRACK-SUSCEPTIBILITY SPECIMEN - CRUCIFORM



Dimensions, inches						
l_1	l_2	L_1	L_2	t_1	t_2	L_3
6	6	12	12	Var.	Var.	6
4	4	8	12	1/4-2	1/4-2	6
3-t	3-t	3	3	0.04	0.04	3

Dimensions varied greatly; the first set of dimensions are preferred; the other sets of dimensions are given to show the range of dimension variation reported; tests usually made at room temperature in air.

Materials Evaluated - All (but used mostly for steels).

Purpose of Test - Developmental, crack susceptibility, and weldability.

Number of Specimens Tested - Usually 1, varying from 1 to 5.

Nondestructive Inspection (in order of decreasing use) - Visual, penetrant, radiographic, magnetic (where possible), and ultrasonic.

Important Variables - Specimen geometry, weld quality, weld inspection, specimen positioning, measurements, and data analysis.

Data Obtained - Extent of cracking.

Specifications - "Procedure: Armor Plate Crack Sensitivity Test", Research Group Ordnance Advisory Committee on Welding of Armor.

References - Weiss, S., Ramsey, J. N., and Udin, H., "Evaluation of Weld-Cracking Tests on Armor Steel", Welding Journal, 35 (7), 348s (1956). Poteat, L. E., and Warner, W. L., "The Cruciform Test of Plate-Cracking Susceptibility", ibid., 39 (2), 70s (1960). Borland, J. C., "Cracking Tests for Assessing Weldability", British Welding Journal, 7 (10), 623 (1960).

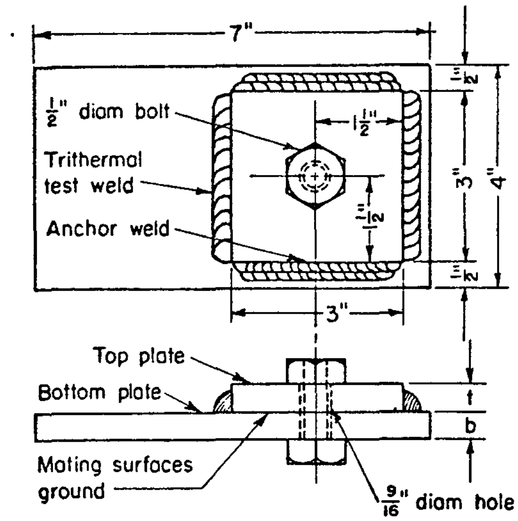
Test Procedure - Fillet welds are deposited in sequence shown; temperature at start of each fillet maintained constant (either room temperature or predetermined preheat temperature); after welding, specimen is aged at room temperature for 48 hours, then stress relieved at 1150 F for 2 hours (heating and cooling from 1150 F at the rate of 100 F per hour); specimen is inspected for cracking and cross sections are cut for metallographic examination.

Remarks - Considerable uncertainty regarding usefulness of specimen; Poteat and Warner (see references) concluded that the test was more sensitive to testing conditions (temperature, fit-up, plate size, etc.) than it is to differences in crack susceptibility of the base metal used; they concluded that the cruciform test in its present form, under test conditions in a production shop, was unsuitable as a crack-susceptibility test for armor steel; its usefulness for other materials appears equally uncertain.

FIGURE 5

SPECIMEN 81 - EXTRA

CRACK-SUSCEPTIBILITY SPECIMEN - CONTROLLED THERMAL SEVERITY (CTS)



Dimensions t and b varied to change thermal severity; the thermal severity number is obtained from the following formula:

TSN = 4(t+b) for bithermal welds

TSN = 4(t+2b) for trithermal welds.

After specimen assembled and anchor welds deposited, assembly allowed to cool to room temperature for depositing bithermal test weld; similarly, specimen cooled to room temperature before trithermal weld deposited; severity of cracking determined by measurements of crack length on metallographic sections.

Materials Evaluated - Plain-carbon and low-alloy, high-strength steels.

Purpose of Test - Developmental, weldability, and crack susceptibility.

Number of Specimens Tested - Usually 3.

Nondestructive Inspection - Visual, penetrant, and magnetic.

Important Variables - Specimen geometry, weld quality, crack detection, and data analysis.

Data Obtained - Extent of cracking.

Specifications - None.

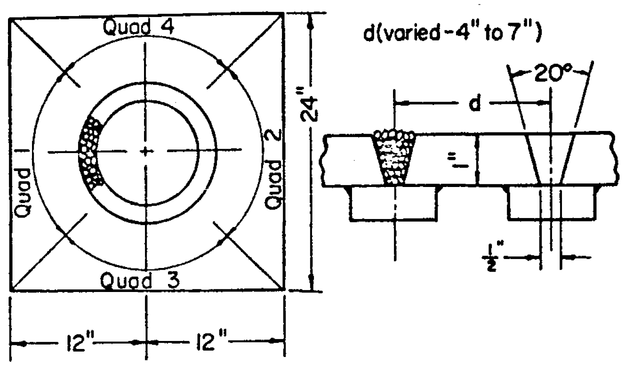
References - Cottrell, C. L. M., "Controlled Thermal Severity Cracking Test Simulates Practical Welded Joints", Welding Journal, 33 (6), 257s (1953). Borland, J. C., "Cracking Tests for Assessing Weldability", British Welding Journal, 7 (10), 623 (1960).

Remarks - Test based on assumption that extent of hard-zone cracking is mainly dependent on cooling rate at about 572 F (300 C), as measured in HAZ adjacent to fusion line; when a critical rate of cooling for a given electrode-steel combination is exceeded, cracking is supposed to occur irrespective of external restraint applied; test evaluates effects of weld and HAZ cooling rates and not external restraint.

FIGURE 6

SPECIMEN 64

CRACK-SUSCEPTIBILITY SPECIMEN – U. S. NAVY CIRCULAR PATCH



Dimensions shown are MIL-E-986C; "d" varies with coated electrode diameter, increasing from 4 to 7 inches as electrode diameter increases from 5/32 to 5/16 inch; for Grade HT steel, preheat and interpass temperature is 0 F; for Grade STS, it is 75 F (+25 F tolerance).

Materials Evaluated - Low-alloy high-strength steels.

Purpose of Test - Developmental, crack susceptibility, and weldability.

Number of Specimens Tested - Surveyed organizations used 2 specimens but specifications require 1 per each size of electrode.

Nondestructive Inspection - Visual, radiographic, and penetrant.

Important Variables - Specimen geometry, weld quality, weld inspection, and test-temperature control.

Data Obtained - Extent of cracking; if welded specimen is radiographically sound, microsections cut from each quadrant for examination for cracks, and weld-metal hardness.

Specifications - MIL-E-22200 and MIL-E-986 C.

References - Wooding, W. H., "Welding Air-Hardening Alloy Steels", Welding Journal, 29 (11), 552s (1950). Stout, R. D., and Doty, W. D., Weldability of Steels, Welding Research Council, 1953, p 236. Borland, J. C., "Cracking Tests for Assessing Weldability", British Welding Journal, 7 (10), 623 (1960).

Test Procedure - Each quadrant completely welded in numerical sequence shown in sketch; the number of layers using a "2-1/2-diameter split-weave" buildup sequence depends on electrode diameter; completed weld allowed to stand 24 hours; if no cracks visible, a concentric disk 2 inches larger in diameter than weld is flame cut from plate, surface machined on each side, and radiographed; if weld is sound, then sections for metallographic examination and hardness tests are cut from each quadrant.

Remarks - An acceptability test for electrode qualification; considered a severe test for hot and cold cracking.

FIGURE 7

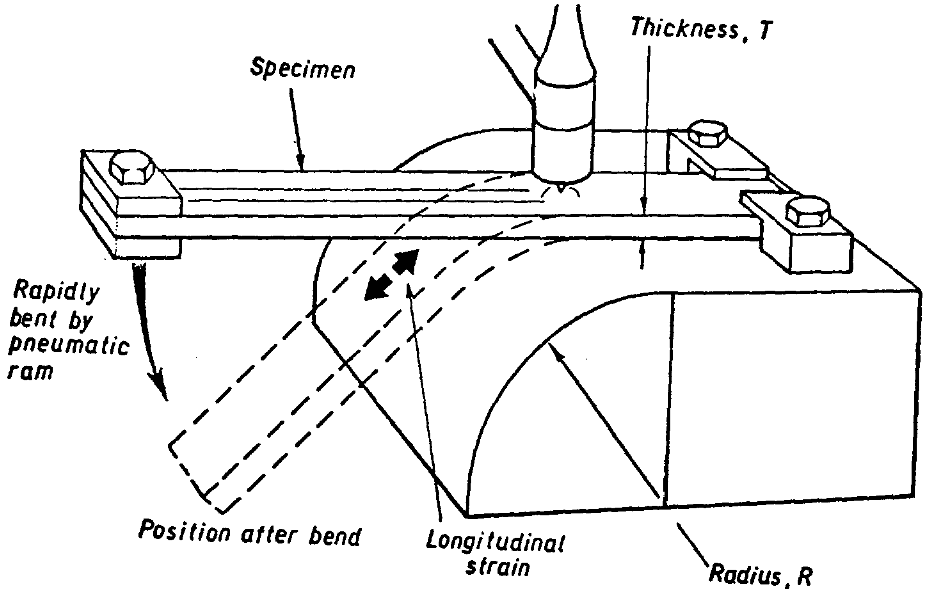


Fig.1 Diagram of Varestraint test

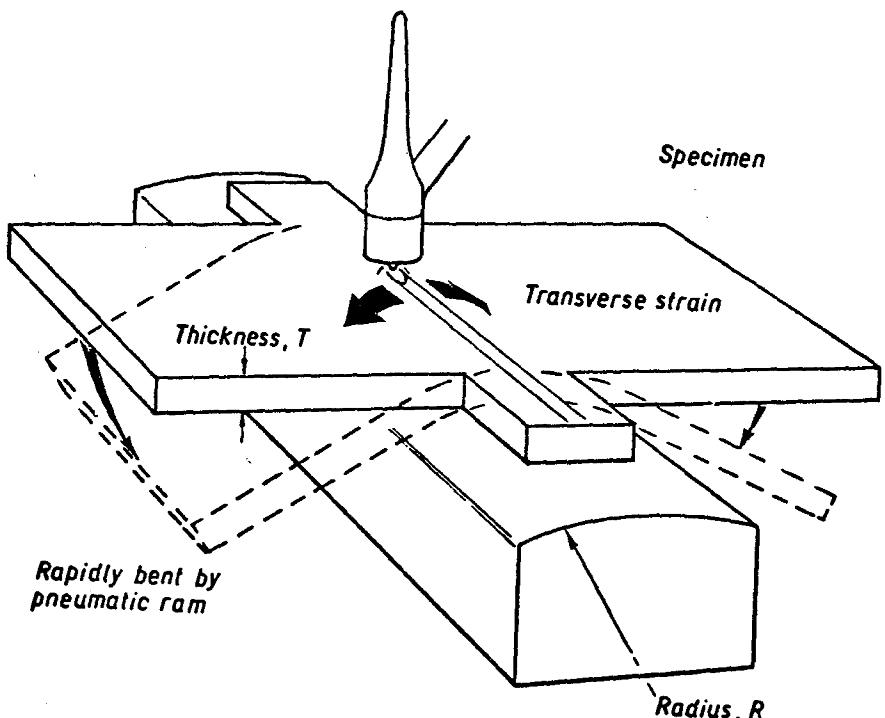


Fig.2 Diagram of Transvarestraint test

FIGURE 8

APPENDIX C

TOUGHNESS of WELD METAL

BACKGROUND. Both the AASHTO Standard Specification for Welding of Highway Bridges and the American Welding Society Structural Welding Code are incomplete and, in places, ambiguous with regard to the controls exercised over the toughness of deposited weld metal. It is in the weld deposit where there is the greatest probability of flaws. Cracks too small to be detected with conventional nondestructive inspection methods can become dangerous to the structure if there is slow crack growth and/or crack pop-in. With an acceptable level of fracture toughness in the deposited weld metal, there will be less chance of a pop-in crack with its inherently high strain rate, and less chance of premature Region-III fatigue crack growth.

The codes are inadequate in this regard because, as presently written, they mainly rely on Charpy impact testing done by the electrode manufacturers. The welding conditions used by a filler-wire producer in preparing test samples may be totally different from the welding procedure that is qualified for a given bridge member.

The following summarizes the paragraphs contained in the AWS Structural Welding Code D1.1-75 and the Amendments thereto contained in the AASHTO Standard Specifications for Welding Structural Steel Highway Bridges which call out Charpy V-notch impact testing as a means to assure toughness in deposited weld metal. A review of these provisions reveals that the specifications as presently written are incomplete and in some cases ambiguous.

The AWS STRUCTURAL WELDING CODE (AWS D1.1-75) - specifies Charpy V-notch impact testing of deposited weld metal in the following sections and paragraphs:

4.1 Filler Metal Requirements - Table 4.1.4 Footnote 2. This table is for weld metals deposited by specific shielded-metal-arc electrodes and for weld metal deposited by the subarc, gas-metal-arc and flux-cored-arc processes where the weld is deposited in A242 or A588 steel for exposed (bare) applications.

Comment: All butt welds in main loading-carrying tension members should be required to have a minimum Charpy V-notch impact strength at least equal to that currently required in the base metal.

4.1 Filler Metal Requirements - Table 4.1.1 Footnote 8. This table is greatly expanded over what appeared in AWS D1.1-Rev. 2-74. All the prequalified filler-metal base-metal combinations are included. Footnote 8 specifies that "deposited weld metal shall have a minimum impact strength of 20 ft-lb at $0^{o}F$ when Charpy V-notch specimens are used". Footnote 8 is applied to only selected filler-metal base-metal combinations where the yield strength of the filler metal and/or the base metal is 60 ksi or higher when welded with subarc, gas-metal-arc and flux-cored-arc processes. The shielded-metal-arc process was exempted from impact testing regardless of yield strength.

Comment: All weld-metal base-metal combinations in main load-carrying tension members should be required to have a minimum impact strength at least equal to that presently specified by AASHTO for the base metal.

4.16 Gas-Metal-Arc and Flux-Cored-Arc Welding. A Charpy V-notch impact value of 20 ft-lb at $0^{o}F$ is specified for these particular processes and is restricted to weld metal with yield strength in excess of 60,000 psi.

Comment: The AWS Code D1.1-Rev. 2-74 had a footnote to this requirement:

> "This value shall govern unless base metal requirements are more restrictive in which case the latter shall govern."

The words "this value" were somewhat ambiguous, but presumably it meant 20 ft-lb at $0^{o}F$. Here we have recognition of the fact that the weld-metal requirement should be equal to the base-metal toughness requirement. In AWS Code D1.1-75, the footnote simply states:

> "Base metal requirements, if more restrictive, shall be met."

Again we have recognition that the weld metal should be equal to the base metal - but why only for these particular welding processes and these particular strength levels?

4.19 Electroslag and Electrogas Welding, Qualification of Process, Procedures and Joint Details. The code specifies that when required by contract drawings or specifications, impact tests shall be included in the welding procedure qualification of material 3 in. or less in thickness. The specified minimum average value is 15 ft-lb at $0^{o}F$.

Comment: The basis for dropping the minimum energy value from 20 to 15 ft-lb is questionable. Also restricting the toughness requirement to plate 3 in. or less in thickness is questionable.

THE AASHTO STANDARD SPECIFICATIONS FOR WELDING OF STRUCTURAL STEEL HIGHWAY BRIDGES - elaborates on the Charpy V-notch impact requirement in the following paragraphs:

4.1 Filler Metal Requirements.

Paragraph 4.1.2 The present paragraph 4.1.2 shall be deleted and shall be replaced with a modified paragraph as follows:

4.1.2 The electrode-flux combination, or grade of weld metal for complete joint penetration or partial joint penetration groove welds and for fillet welds may be of lower strength than required for complete joint penetration butt welds provided the weld metal on main members meets the stress requirements of 9.3 and the filler metal classification used has specified impact properties of 20 ft-lb at $0^{\circ}F$ or below.

Comment: The impact-strength requirement deals only with vendor certification of the consumables; this provides no information on the interaction of the welding consumables and the qualified welding procedure.

Table 4.1.1 Footnote (6). Electrodes of other classifications in the specification will be acceptable for use if the manufacturer's Certificate of Test shows that the mechanical properties (tensile strength, yield strength, and elongation) and the impact properties meet the requirements of the classification, or classifications, listed.

Comment: Again this pertains to vendor certification of the consumables; this provides no information on the interaction of the welding consumables and the qualified welding procedure to be used in fabrication of the bridge.

4.19 Electroslag and Electrogas Welding - Qualification of Process, Procedure and Joint Details.

Paragraph 4.19.3 The present paragraph 4.19.3 shall be deleted and shall be replaced with a modified paragraph as follows:

4.19.3 The welding procedure qualification shall include impact tests. The impact tests, requirements, and procedure shall be in accordance with the provisions of Appendix C.

Paragraph 4.20 The present paragraph 4.20 shall be deleted and shall be replaced with a modified paragraph as follows:

4.20 Prior to use, the Contractor shall demonstrate by the tests prescribed in Part II of Section 5, and in Appendix C, that each combination of shielding and filler metal will produce welds having the following mechanical properties, supplemented by the impact strength requirements of Appendix C, when welded in accordance with the Procedure Specification. The Engineer, at his discretion, may accept evidence of record of a combination that has been satisfactorily tested in lieu of...

Comment: This makes Charpy testing mandatory for electroslag and electrogas welding and recognizes the fact that the impact testing should be made of a weldment prepared using the qualified procedure. The "welding procedure specification" is understood to be the qualified joint design and the qualified procedure. Without a user test of the qualified joint and procedure, there is no assurance of toughness in the deposited weld metal. The electrode specifications, referenced in AWS D1.1-75, call out specific welding conditions for the test weldment; this provides no information on the interaction of the welding consumables and the qualified welding procedure in terms of weld-metal toughness.

NOTE: By FHWA Notice N5040.23 of February 10, 1977, electroslag welding is prohibited for sue in main structural tension members in Federal-aid projects as a result of an ES-weld fracture in a bridge near Pittsburgh, Pennsylvania.

☆U.S. GOVERNMENT PRINTING OFFICE: 1978-261-264/201

Publications listed below are not available from the Government Printing Office. These publications are available in limited number to State highway agencies and other public agencies from the Federal Highway Administration. Requests for these documents and suggestions on the contents of any publications should be addressed to the Federal Highway Administration Office of Engineering, Bridge Division, HNG-30, Washington, D.C. 20590.

Structural Engineering Series

SES No. 1 - ICE LOADS ON BRIDGE PIERS, January, 1976
F. J. Watts and Walter Podolny, Jr.

SES No. 2 - ARCH BRIDGES, September, 1977
D. A. Nettleton and J. S. Torkelson

SES No. 3 - ATLANTA DISCUSSION OF PROPOSED FHWA FRACTURE CONTROL PLAN, March, 1978

SES No. 4 - CABLE-STAYED BRIDGES, June, 1978