Brittle Fracture: Another View
Brittle Fracture:
Another View

- Definition of brittle fracture
- Significance of brittle fracture
- Factors affecting brittle fracture
- Case studies involving brittle fracture
- Designing to prevent brittle fracture
COMMENTARY GLOSSARY

**Brittle fracture.**

Abrupt cleavage with little or no prior ductile deformation.
Brittle fracture in metals is characterized by a rapid rate of crack propagation, with no gross deformation and very little micro-deformations. The tendency for brittle fracture is increased with decreasing temperature, increasing strain rate, and triaxial stress conditions (usually produced by a notch).
Brittle fracture is a type of failure in structural materials that usually occurs without prior plastic deformation and at extremely high speeds (as fast as 7000 ft/s [210 m/s] in steels). The fracture is usually characterized by a flat cleavage fracture surface...and at average stress levels below those of general yielding.
It is well known that a metal may be ductile under one set of conditions and brittle under another.

Ductility and brittleness, then are properties that must be considered as referring to some particular set of testing or service conditions.
Most structural materials exhibit considerable strain (deformation) before reaching the tensile or ultimate strength, $\sigma_{\text{tens}}$....In contrast, brittle materials exhibit almost no deformation before fracture....However, under conditions of low temperature, rapid loading and/or high constraint...even ductile materials may not exhibit any deformation before fracture.
Most structural materials exhibit considerable strain (deformation) before reaching the tensile or ultimate strength, $\sigma_{\text{tens}}$. In contrast, brittle materials exhibit almost no deformation before fracture. However, under conditions of low temperature, rapid loading and/or high constraint... even ductile materials may not exhibit any deformation before fracture.
...the science of fracture mechanics can be used to describe quantitatively the tradeoffs among stress, material fracture toughness, and flaw size so that the designer can determine the importance of each during the design process.
Basic Fracture Mechanics

Material fracture toughness ($K_{IC}$)

\[ K_{IC} \geq 1.12 \sigma (\pi a)^{0.5} \]

Stress ($\sigma$)

Flaw size ($a$)
If $K_{IC}$ is high enough, $\sigma$ can be $> F_u$. Net section will eventually control.
If $K_{IC}$ is high enough, $a$ can be $> b/2$.

$K_{IC} \geq 1.12 \sigma (\pi a)^{0.5}$

Net section will eventually control.
If \( a = 0 \), \( \sigma \) can be infinite, even if \( K_{IC} \) is low.

\[
K_{IC} \geq 1.12 \sigma (\pi a)^{0.5}
\]

Gross section will eventually control.
Brittle Fracture:
Another View

• Definition of brittle fracture
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Brittle fracture is to be avoided at all cost, because it occurs without warning and usually produces disastrous consequences.
Because it is very difficult to fabricate large welded structures without introducing some type of notch, flaw, or stress concentration, the design engineer must be aware of the effect of notches and constraint on material behavior.
Thus, in addition to the material properties such as yield strength, modulus of elasticity, and tensile strength, there is another very important material property, namely notch toughness that may be related to the behavior of a structure. Notch toughness is defined as the ability of a material to absorb energy in the presence of a sharp notch, often when subjected to an impact load.
Brittle Fracture:
Another View

- Definition of brittle fracture
- Significance of brittle fracture
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- Designing to prevent brittle fracture
The Unholy Trinity

- Triaxial Stress
- Low Temperature
- High Strain Rate
Commentary A3.1a

For especially demanding service conditions such as structures exposed to low temperatures, particularly those with impact loading, the specification of steels with superior notch toughness may be warranted.
A triaxial state-of-stress can also result from uniaxial loading when notches or geometric discontinuities are present. A triaxial state-of-stress will cause the yield stress of the material to increase above its nominal value, resulting in brittle fracture by cleavage, rather than ductile shear deformations.
Brittle fracture in metals is characterized by a rapid rate of crack propagation, with no gross deformation and very little micro-deformations. The tendency for brittle fracture is increased with decreasing temperature, increasing strain rate, and triaxial stress conditions (usually produced by a notch).
Most structural materials exhibit considerable strain (deformation) before reaching the tensile or ultimate strength, $\sigma_{\text{tens}}$. In contrast, brittle materials exhibit almost no deformation before fracture. However, under conditions of low temperature, rapid loading and/or high constraint...even ductile materials may not exhibit any deformation before fracture.
The Unholy Trinity

Triaxial Stress

Low Temperature

High Strain Rate
7-11 NOTCH EFFECTS

…However, the chief effect of the notch is not in introducing a stress concentration but in producing a triaxial state of stress at the notch.
In summary, a notch increases the tendency for brittle fracture in four important ways:

• By producing high local stresses
• By introducing a triaxial tensile state of stress
• By producing high local strain hardening and cracking
• By producing a local magnification to the strain rate
Two things:

- \( A_{\text{net}} < A_{\text{gross}} \)
- Stress is not uniform
In summary, a notch increases the tendency for brittle fracture in four important ways:

- By producing high local stresses
- By introducing a triaxial tensile state of stress
- By producing high local strain hardening and cracking
- By producing a local magnification to the strain rate
Thin Plate
Plane-stress

Thin Plate

\( \sigma_x = + \)

\( \sigma_y = + \)

\( \sigma_z = 0 \)
Thick Plate

\[ \sigma_x, \sigma_z, \sigma_y \]
**Plane-strain**

**Thick Plate**

\[ \sigma_x = + \]
\[ \sigma_y = + \]
\[ \sigma_z = + \]
Shear

Tensile
Smooth specimen
(no notches)
Smooth specimen
(no notches)
Smooth specimen  
(no notches)
Shear strength $\tau_{x-y}, \tau_{x-z}$

$\sigma_y, \sigma_z$

$F_y, F_u$

$\sigma_x$
Shear strength $\tau_{x-y}, \tau_{x-z}$
Shear strength $\tau_{x-y}, \tau_{x-z}$

$\sigma_y, \sigma_z$

$F_y, F_u$

$\sigma_x$
Plane-stress

Thin Plate

\[ \sigma_x = F_y \]
\[ \sigma_y = F_y / 2 \]
\[ \sigma_z = 0 \]
**Plane-stress**

**Thin plate**
Plane-stress
Thin plate
Plane-strain

Thick Plate

\[ \sigma_x = F_y \]
\[ \sigma_y = F_y/2 \]
\[ \sigma_z = 3F_y/4 \]
Plane-strain
Thick plate
Plane-strain
Thick plate
In summary, a notch increases the tendency for brittle fracture in four important ways:

- By producing high local stresses
- By introducing a triaxial tensile state of stress
- By producing high local strain hardening and cracking
- By producing a local magnification to the strain rate
Crack tip plastic zone

- inelastic zone
- yielded zone
- strain-hardened zone
The ability of a component to plastically deform in the vicinity of a crack tip is the saving grace of countless engineering structures.
Crack tip plastic zone

• inelastic zone
• yielded zone
• strain-hardened zone
In summary, a notch increases the tendency for brittle fracture in four important ways:

- By producing high local stresses
- By introducing a triaxial tensile state of stress
- By producing high local strain hardening and cracking
- By producing a local magnification to the strain rate
Stress
Strain
\[ \dot{\varepsilon}_p = 10 \times \dot{\varepsilon}_e \]

\[ \dot{\varepsilon}_e = 1 \]

Strain Rate

Numerical values are illustrative only.
The Unholy Trinity

- Triaxial Stress
- Low Temperature
- High Strain Rate
Temperature $t_{\text{max}}$ and $\sigma_{\text{max}}$,

- Normal Stress for Fracture
- Shear Stress for Flow

Adapted from Gensamer
\[ \tau_{\text{max}} / \sigma_{\text{max}} = 1/2 \]

- Fracture
- Plastic, ductile
- Elastic

Temperature

- \( \sigma_{\text{max}} \) and \( \tau_{\text{max}} \)
\( \frac{\tau_{\text{max}}}{\sigma_{\text{max}}} = \frac{5}{6} \)

Temperature

Elastic

Plastic, ductile

Fracture

\( \sigma_{\text{max}} \) and \( \tau_{\text{max}} \)
Body Centered Cubic (BCC)

Atomic Packing
Body Centered Cubic (BCC)
Body Centered Cubic (BCC)
Body Centered Cubic (BCC)
Body Centered Cubic (BCC)

Shear Planes
Body Centered Cubic (BCC)

Cleavage Plane
Body Centered Cubic (BCC)

Cleavage Plane
Body Centered Cubic (BCC)

Cleavage Plane
Body Centered Cubic (BCC)

Cleavage Plane
Body Centered Cubic (BCC)

Shear Plane

Cleavage Plane
Adapted from Gensamer

Temperature

Normal Stress for Fracture
Shear Stress for Flow

\( \sigma_{\text{max}} \) and \( \tau_{\text{max}} \)
The Unholy Trinity

- Triaxial Stress
- Low Temperature
- High Strain Rate
FIG. 7 INFLUENCE OF RATE OF STRAIN ON TENSILE PROPERTIES OF MILD STEEL AT ROOM TEMPERATURE

From Manjoine
Average strain rate, sec$^{-1}$

- $10^{-5}$
- $10^{-4}$
- $10^{-3}$
- $10^{-2}$
- $10^{-1}$
- 1
- 10
- $10^2$
- $10^3$

$F_y$ and $F_u$, ksi

$F_y/F_u$, %

Ratio $F_y/F_u$

$F_u$

$F_y$

Adapted from Manjoine
Adapted from Manjoine
Adapted from Manjoine
Average strain rate, sec$^{-1}$

F$y$ and F$u$, ksi

Ratio F$y$/F$u$

Adapted from Manjoine
The Unholy Trinity

Triaxial Stress

- Low Temperature
- High Strain Rate
Smooth “bar”

Multi-directional loading
Smooth “bar”
Multi-directional loading

\[ \tau_{x-y}, \tau_{x-z}, \tau_{y-z} \]
Smooth “bar”
Multi-directional loading

\[ \tau_{x-y}, \tau_{x-z}, \tau_{y-z} \]
Smooth “bar”

Multi-directional loading

\( \tau_{x-y}, \tau_{x-z}, \tau_{y-z} \)
Smooth “bar”

Also, residual stresses
Plane-strain

Thick plate, with notch

$\tau_{\text{critical}}$

$\sigma_z$

$\sigma_y$

$\sigma_x$

$F_y$

$F_u$
The Unholy Trinity

- Triaxial Stress
- Low Temperature
- High Strain Rate
For this example, $\tau_{\text{critical}} = 0.5F_y$

Smooth warm specimen
For this example, $\tau_{\text{critical}} = 0.5F_y$

$\tau_{x-y}, \tau_{x-z}$

$\sigma_y, \sigma_z$

$F_y, F_u, \sigma_x$

Smooth warm specimen
For low temperature, let $F_y = 0.90 F_u$

$\tau_{x-y}, \tau_{x-z}$

$\sigma_y, \sigma_z$

$\sigma_x$

$F_y, F_u$

Smooth cold specimen
For low temperature, let $F_y = 0.90F_u$.
The Unholy Trinity

Triaxial Stress

Low Temperature

High Strain Rate
Fracture Toughness, $K_C$, ksi-in$^{0.5}$

Temperature ($^\circ$F)

-300 -250 -200 -150 -100 -50 0 50

A572 Gr 50 Steel (50 ksi, 345 MPa)

- Slow-bend load ($\dot{\varepsilon} = 10^{-5}$ sec$^{-1}$)
- Intermediate strain-rate load ($\dot{\varepsilon} = 10^{-3}$ sec$^{-1}$)
- Dynamic load ($\dot{\varepsilon} = 10$ sec$^{-1}$)

Adapted from Barsom & Rolfe
Fracture Toughness, $K_C$, ksi-in$^{0.5}$

Temperature ($^\circ$F) -300 -250 -200 -150 -100 -50 0 +50

A572 Gr 50 Steel (50 ksi, 345 MPa)

- Slow-bend load ($\dot{\varepsilon} \approx 10^{-5}$ sec$^{-1}$)
- Intermediate strain-rate load ($\dot{\varepsilon} \approx 10^{-3}$ sec$^{-1}$)
- Dynamic load ($\dot{\varepsilon} \approx 10$ sec$^{-1}$)

Adapted from Barsom & Rolfe
A572 Gr 50 Steel (50 ksi, 345 MPa)

- Slow-bend load ($\dot{\varepsilon} \approx 10^{-5}$ sec$^{-1}$)
- Intermediate strain-rate load ($\dot{\varepsilon} \approx 10^{-3}$ sec$^{-1}$)
- Dynamic load ($\dot{\varepsilon} \approx 10$ sec$^{-1}$)

Fracture Toughness, $K_C$, ksi-in$^{0.5}$

*Adapted from Barsom & Rolfe*
The changes produced by the introduction of a notch have important consequences in the fracture process. For example, the presence of a notch will increase appreciably the ductile/brittle transition temperature of a steel.
For this example, $\tau_{\text{critical}} = 0.5F_y$

Smooth, slowly strained specimen
Smooth, rapidly strained specimen

For $\varepsilon$ = high, let $F_y = 0.90F_u$
Smooth, rapidly strained specimen

For $\varepsilon = \text{high}$, let $F_y = 0.90F_u$.

$\tau_{x-y}$

$\tau_{\text{critical}}$

$\sigma_x, \sigma_y, \sigma_z$

$F_y, F_u$
The Unholy Trinity

- Triaxial Stress
  - Loading, notches
  - Plus, effect on material
- Low Temperature
- High Strain Rate
  - Effect on material
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Another View

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Brittle Studies Involving Brittle Fracture:

- Case 1: Liberty Ships
- Case 2: Silver Bridge
- Case 3: Ingram Barge
- Case 4: Hoan Bridge
FINAL REPORT OF A BOARD OF INVESTIGATION

Convened by Order of

THE SECRETARY OF THE NAVY

To Inquire Into

THE DESIGN
AND METHODS OF CONSTRUCTION
OF WELDED STEEL MERCHANT
VESSELS

15 JULY 1946
Convened by Order of
THE SECRETARY OF THE NAVY

To Inquire Into

THE DESIGN AND METHODS OF CONSTRUCTION OF WELDED STEEL MERCHANT VESSELS

15 JULY 1946
The Design and Methods of Construction of Welded Steel Merchant Vessels
Early in the war, welded merchant vessels experienced difficulties in the form of fractures which could not be explained. The fractures, in many cases, manifested themselves with explosive suddenness and exhibited a quality of brittleness which was not ordinarily associated with the behavior of a normally ductile materials such as ship steel.
<table>
<thead>
<tr>
<th>Description</th>
<th>Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total number of ships</td>
<td>4,696</td>
</tr>
<tr>
<td>Total number of these ships reporting no casualties</td>
<td>3,724</td>
</tr>
<tr>
<td>Total number of these ships which sustained casualties</td>
<td>970</td>
</tr>
<tr>
<td>Total number of casualties</td>
<td>1,442</td>
</tr>
<tr>
<td>Total number of fractures</td>
<td>4,720</td>
</tr>
<tr>
<td>Total cases of serious casualties (Class 1)</td>
<td>127</td>
</tr>
<tr>
<td>Total ships sustaining a complete fracture of strength deck</td>
<td>24</td>
</tr>
<tr>
<td>Total ships sustaining a complete fracture of the bottom</td>
<td>1</td>
</tr>
</tbody>
</table>
Eight vessels have been lost, as follows:

<table>
<thead>
<tr>
<th>Name</th>
<th>Date</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thomas Hooker</td>
<td>5 Mar 1943</td>
<td>Abandoned</td>
</tr>
<tr>
<td>J.L.M. Curry</td>
<td>7 Mar 1943</td>
<td>Abandoned</td>
</tr>
<tr>
<td>John P. Gaines</td>
<td>24 Nov 1943</td>
<td>Broke in two, abandoned</td>
</tr>
<tr>
<td>Joseph Smith</td>
<td>9 Jan 1944</td>
<td>Abandoned</td>
</tr>
<tr>
<td>Samuel Dexter</td>
<td>21 Jan 1944</td>
<td>Abandoned</td>
</tr>
<tr>
<td>Joel R. Poinsett</td>
<td>4 Mar 1944</td>
<td>Broke in two; stern portion salvaged</td>
</tr>
<tr>
<td>Sackett’s Harbor</td>
<td>1 Mar 1946</td>
<td>Broke in two; stern portion salvaged</td>
</tr>
<tr>
<td>Fort Sumter</td>
<td>10 May 1946</td>
<td>Broke in two; both portions scuttled</td>
</tr>
<tr>
<td>Four other ships broke in two but were not lost</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Schenectady</td>
<td>15 Jan 1943</td>
<td></td>
</tr>
<tr>
<td>Esso Manhattan</td>
<td>29 Mar 1943</td>
<td></td>
</tr>
<tr>
<td>Valeri Chkalov</td>
<td>11 Dec 1943</td>
<td></td>
</tr>
<tr>
<td>Donbass III</td>
<td>17 Feb 1946</td>
<td></td>
</tr>
</tbody>
</table>
The Design and Methods of Construction of Welded Steel Merchant Vessels

The fracture started at the juncture of the fashion plate at the aft starboard corner of the bridge superstructure and the sheer strake.

Without warning and with no report which was heard for at least a mile, the deck and sides of the vessel fractured just aft of the bridge superstructure. The fracture extended almost horizontally to the turn of the bilge port and starboard. The deck sides and longitudinal bulkheads and bottom girders fractured. Only the bottom plating held. The vessel Submerged and the lower portion rose so that no water entered the hull. The bow and stern settled into the silt of the river bottom. Russing taken around the vessel eliminated the alleged possibility of the vessel having grounded amidsthips to a drag in water level.

Vessels required and put in service.
**The Design and Methods of Construction of Welded Steel Merchant Vessels**

### REPORT OF STRUCTURAL FAILURE OF INSPECTED VESSEL

**UNITED STATES COAST GUARD**

**NAME:** SCHENECTADY

**BUILDERS:** Kaiser Co., Inc., Portland, Oregon

**OWNER:** War Shipping Administration

**TYPE:** Tank Vessel

**HULL NO.:** 1

**DATE COMPLETED:** 31 Dec., '43

**M/C DESIGN:** T2-SE-Al

---

### DESCRIPTION OF VESSEL

#### EXTENT OF WELDING

<table>
<thead>
<tr>
<th>Hull all welded</th>
<th>SIDE SHELL SEAMS</th>
<th>DECK SEAMS</th>
<th>SIDE SHELL BUTTS</th>
<th>BOTTOM SEAMS</th>
<th>INNER BOTTOM SEAMS</th>
<th>DECK BUTTS</th>
<th>FRAMES TO SIDE SHELL</th>
<th>BOTTOM BUTTS</th>
<th>INNER BOTTOM BUTTS</th>
<th>BEAMS TO DECK</th>
<th>BULKHEADS</th>
<th>FLOORS TO SHELL</th>
<th>FLOORS TO INNER BOTTOM</th>
<th>DECK TO SHELL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>-</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

---

### CIRCUMSTANCES SURROUNDING FAILURE

**DATE OF FAILURE:** 16 Jan., 1943

**TIME:** 2230 PWT

**SHIP'S LOCATION:** Tied up at fitting out pier, Swan Island

**DRAFT FWD.**

**DRAFT AFT.**
The fracture started at the juncture of the fashion plate at the aft starboard corner of the bridge superstructure and the sheer strake.

GENERAL HISTORY AND DESCRIPTION OF FAILURE, INCLUDING KNOWN CONTRIBUTORY FACTORS:
Without warning and with a report which was heard for at least a mile, the deck and sides of the vessel fractured just aft of the bridge superstructure. The fracture extended almost instantaneously to the turn of the bilge port and starboard. The deck side shell, longitudinal bulkheads and bottom girders fractured. Only the bottom plating held. The vessel jack-knifed and the center portion rose so that no water entered the hull. The bow and stern settled into the silt of the river bottom. Sounding taken around the vessel eliminated the alleged possibility of the vessel having grounded amidships to a drop in water level. Bending moment in still water = 184,000 Ft. x Tons Hog amidships. Stress in crown of deck = 9900 Lbs./in.² Tension.

DISPOSITION OF VESSEL
(Repaired, lost, etc.)
Vessel repaired and put in service.

Figure 14.
Without warning and with a report which was heard for at least a mile, the deck, and sides of the vessel fractured just aft of the bridge superstructure.

Stress in crown of deck = 9900 Lbs./in. Tension.

Vessel repaired and put in service.
I. Conclusions

The Board concludes that:

(a) The fractures in welded ships were caused by notches and by steel which was notch sensitive at operating temperatures. When an adverse combination of these occur the ship may be unable to resist the bending moments of normal service.
The majority of the fractures in the Liberty ships started at **square hatch corners** or **square cutouts** at the **top of the sheer strake**. Design changes involved **rounding and strengthening** of the hatch corners, **removing square cutouts** in the sheer strake, and **adding riveted crack arresters** in various locations led to **immediate reductions** in the incidence of failures.
Control of Steel Construction to Avoid Brittle Failure
1980

Fracture control considerations for steel bridges, March 1980

J. M. Barsom
J. W. Fisher
K. H. Frank
G. R. Irwin
Although steel quality later was found to be an important factor in these failures, the immediate solution to the problem was achieved by design changes and better quality fabrication. It was not until the 1950’s that changes in material toughness were made.
Brittle Studies Involving Brittle Fracture:

- Case 1: Liberty Ships
- Case 2: Silver Bridge
- Case 3: Ingram Barge
- Case 4: Hoan Bridge
Silver Bridge Summary

• Opened to traffic May 1928

• Collapsed December 1967

• Eyebar suspension bridge

• 30 °F at time of collapse
• Total length: 1756 feet
• Main span: 700 feet
• River width: 1240 feet
Question
It is well known that a metal may be ductile under one set of conditions and brittle under another. Ductility and brittleness, then are properties that must be considered as referring to some particular set of testing or service conditions.
○ Battelle Mem. Inst., Chain bent post LO-UO, N (solid curve)

□ Nat'l. Bur. Stds., Chain bent post, longitudinal, LO-UO, (dotted curve)

△ U.S. Steel Lab., Chain bent post, longitudinal, L58-U58N (dashed curve)
HIGHWAY ACCIDENT REPORT
National Transportation Safety Board
EVENTS BEFORE BRIDGE COLLAPSE

At about 4:35 p.m. on December 15, 1967, two witnesses saw objects on the roadway of the bridge just east of the Ohio tower. The first person, who was a machinist, identified the object he saw as a large nut that he believed had the shank of a bolt in the nut in a position near the curb of the eastbound lane. He identified the nut as similar to the 1-1/4-inch nuts used on the bridge to secure the pin retainers on the eyebar joints. The other witness stated she saw an object resembling an automobile hubcap on the north side of the roadway. She was unable to state the object was a pin retainer. Both witnesses were in moving automobiles and did not stop. Their observations were therefore of very brief duration.
At about 4:35 p.m. on December 15, 1967, two witnesses saw objects on the roadway of the bridge just east of the Ohio tower.

The first person, who was a machinist, identified the object he saw as a large nut...similar to the 1-1/4-inch nuts used on the bridge to secure the pin retainers on the eyebar joints.

roadway. She was unable to state the object was a pin retainer.

Both witnesses were in moving automobiles and did not stop. Their observations were therefore of very brief duration.
The other witness stated she saw an object resembling an automobile hubcap on the north side of the roadway. She was unable to state the object was a pin retainer.
J. A. Bennett¹ and Harold Mindlin²

Metallurgical Aspects of the Failure of the Point Pleasant Bridge


ABSTRACT: Examination of the fractured eyebar which caused the collapse of the bridge led to the conclusion that a stress-corrosion crack had penetrated to a depth of ¾ in. during the 40 years that the bridge was in service. This flaw was sufficient to initiate fracture across the remainder of the 16 in.² area of the lower limb of the eye due to the high local stress and the low fracture toughness of the steel.

KEY WORDS: corrosion, stress corrosion, cracking (fracturing), fractures (materials), mechanical properties, microstructure, tensile properties, fatigue (materials), stress corrosion tests, humidity, toughness

be considered under three principal categories;

1. Examination of the fractures in eyebar 330 and the metallographic investigation of the material close to the initial fracture.

2. Evaluation of the mechanical properties of the eyebar material including fracture toughness and resistance to crack propagation under fatigue and steady load conditions.

3. Electron microprobe and other studies of the surfaces of freshly opened cracks in the eyes. As some of this work has previously been reported, only a brief account of the results will be given here.
4. The fracture resulted from a combination of factors; in the absence of any of these it probably would not have occurred. These are; a) the high hardness of the steel which rendered it susceptible to stress-corrosion cracking; b) the close spacing of the components in the joint which made it impossible to apply paint to the most highly stressed region of the eye, yet provided a crevice in this region where water could collect; c) the high design load in the eyebar chain, which resulted in a local stress at the inside of the eye greater than the yield strength of the steel; d) the low fracture toughness of the steel which permitted the initiation of complete fracture from the slowly propagating stress-corrosion crack when it had reached a depth of only 0.12 in.
The fracture resulted from a combination of factors; in the absence of any of these it probably would not have occurred. These are:

4. The fracture resulted from a combination of factors; in the absence of any of these it probably would not have occurred. These are:

- High stress at the inside of the eye greater than the yield strength of the steel;
- The low fracture toughness of the steel which permitted the initiation of complete fracture from the slowly propagating stress-corrosion crack when it had reached a depth of only 0.12 in.
a) The high hardness of the steel which rendered it susceptible to stress-corrosion cracking.
b) The close spacing of the components in the joint which made it impossible to apply paint to the most highly stressed region of the eye, yet provided a crevice in this region where water could collect.
c) the high design load in the eyebar chain, which resulted in a local stress at the inside of the eye greater than the yield strength of the steel.
d) The low fracture toughness of the steel which permitted the initiation of complete fracture from the slowly propagating stress-corrosion crack when it had reached a depth of only 0.12 in. [3 mm].
THE SILVER BRIDGE DISASTER

On December 15, 1967, about one mile downstream from this historic marker, a national tragedy occurred. Forty-six interstate travelers lost their lives when the Silver Bridge collapsed into the Ohio River during five o’clock rush hour traffic. The 2,235 foot two-way vehicular bridge connected Point Pleasant, West Virginia and Kanawha, Ohio via U.S. Route 35. The West Virginia Ohio River Bridge Company built the structure in 1928 for $1.2 million. The bridge, unique in its engineering conception, was the first of its design in America and the second in the world. Instead of woven-wire cable, the bridge was suspended on heat-treated eye-bar chains. It was named the “Silver Bridge” because it was the first in the world to be painted with aluminum paint. In 1969, two years later, its replacement, the Silver Memorial Bridge, was dedicated.

GALLIA COUNTY HISTORICAL SOCIETY
O. O. McIntyre PARK DISTRICT
AND
THE OHIO HISTORICAL SOCIETY

1992

8-27
SILVER BRIDGE COLLAPSE

SILVER BRIDGE COLLAPSE AND CREATION OF NATIONAL BRIDGE INSPECTION STANDARDS (NBIS) – POINT PLEASANT, WEST VIRGINIA

ON DECEMBER 15, 1967 AT 4:58 PM, THE 39-YEAR-OLD SILVER BRIDGE SUDDENLY COLLAPSED INTO THE OHIO RIVER DURING HEAVY RUSH HOUR AND HOLIDAY SEASON TRAFFIC. FORTY-SIX LIVES WERE TRAGICALLY LOST. THE CAUSE OF THE COLLAPSE WAS A SINGLE HAIRLINE CRACK IN A STEEL EYEBAR IN THE NORTHERN SUSPENSION CHAIN. IN RESPONSE TO THIS CATASTROPHE, CONGRESS ESTABLISHED NATIONAL BRIDGE INSPECTION STANDARDS. THESE STANDARDS CREATED A RIGOROUS NATIONWIDE BRIDGE SAFETY INSPECTION PROGRAM TO DETECT UNSAFE STRUCTURAL CONDITIONS, PREVENT FUTURE TRAGEDIES, AND SAVE COUNTLESS LIVES.

Dedication Date: December 15, 2019
The cause of the collapse was single hairline crack in a steel eyebart in the northern suspension chain.
In response to this catastrophe, Congress established National Bridge Inspection Standards.
Brittle Studies Involving Brittle Fracture:

- Case 1: Liberty Ships
- Case 2: Silver Bridge
- Case 3: Ingram Barge
- Case 4: Hoan Bridge
Martha R. Ingram Barge

- January 10, 1972
- 584 foot [178 m]
- Air temperature 45 °F [7 °C]
- In service for 9 months
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• Unusual loading: 2.5X design load, 24 ksi [165 MPa]
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• 584 foot [178 m]
• Air temperature 45 °F [7 °C]
• In service for 9 months
• Unusual loading: 2.5X design load, 24 ksi [165 MPa]
• 55 ft-lbs [74 J] at service temperature
• No pre-existing flaws were observed
...the primary cause of failure was established to be an unusually high loading stress caused by improper ballasting at a highly constrained welded detail.
Thus, heavily constrained structures, such as the Ingram Barge, can fail under severe loads even though the inherent notch toughness and ductility may be very good. In contrast, well-designed simple structures can operate successfully at temperatures where their notch toughness may be very low. Thus, constraint and loading are the key factors in prevention of brittle fracture.
No amount of inspection would have solved this problem.

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Brittle Studies Involving Brittle Fracture:

- Case 1: Liberty Ships
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- Case 3: Ingram Barge
- Case 4: Hoan Bridge
• Built 1972, Opened 1977
• “The Bridge to Nowhere”
• Tied Arch
• Total length: 1.9 miles (3058 m)
• Longest span: 607 feet (185 m)
• December 2000: major fracture discovered
Transverse Connection Plate

Fracture Initiation Site in the Web Gap Area

Gusset Plate Cutout
Memorandum

U.S. Department of Transportation
Federal Highway Administration

Subject: **ACTION:** Hoan Bridge Failure Investigation

From: James D. Cooper
Director, Bridge Technology

To: Directors of Field Services
Division Administrators
Federal Lands Highway Division Engineers

Date: July 10, 2001

Reply to: HIBT-10
Attn of:

This memorandum presents the latest findings from the forensic investigation into the cause of failure of the Hoan Bridge in Milwaukee, Wisconsin. In a memorandum dated February 1, I reported observations of the bridge's structure and identified stresses that have been factors in similar events.
“...the primary cause of failure of the Hoan Bridge is the joint detail used to connect the lateral bracing system to the main girder webs.”

The team concluded that the primary cause of failure of the Hoan Bridge is the joint detail used to connect the lateral bracing system to the main girder webs. Some specific details of the joint created a condition that reduced the fracture resistance and made it vulnerable to premature failure. Research is indicating that this vulnerability is not an inherent problem with this class of joint, but that it is related to the specific details used in the Hoan Bridge.

“Some specific details of the joint created a condition that reduced the fracture resistance and make it vulnerable to premature failure.”
“There was no evidence of fatigue cracking prior to fracture initiation. This indicates that there was not observable damage prior to the sudden fracture.”

- There was no evidence of fatigue cracking prior to fracture initiation. This indicates that there was no observable damage prior to the sudden fracture. Even the most rigorous fracture critical inspection would not have provided warning of the impending fracture.

- The web material properties met modern standards for A36 steel. Toughness met the 2001 AASHTO requirements for zone 2, fracture critical use.

- The flange material properties met modern properties for A588 steel. Toughness met the 2001 AASHTO requirements for zone 2, non-fracture critical use.
“Toughness met the 2001 AASHTO requirements for zone 2....” (note: FCM for the A36, non-fracture critical for A588.)

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- The flange material properties met modern properties for A588 steel. Toughness met the 2001 AASHTO requirements for zone 2, non-fracture critical use.
A narrow gap between the gusset plate and the transverse connection/stiffener plate created a local triaxial constraint condition and increased the stiffness in the web gap region at the fracture initiation site. This constraint prevented yielding and redistribution of the local stress concentrations occurring in this region. As a result, the local stress state in the web gap was forced well beyond the yield strength of the material. Under triaxial constraint, the apparent fracture toughness of the material is reduced and brittle fracture can occur under service conditions where ductile behavior is normally expected.
• Joint Details

The primary cause of fracture initiation was determined to be the geometry and fabrication tolerance of the joint where the lateral bracing frames into the web. The joint was detailed with a narrow web gap that caused a local high constraint, increased stiffness, and reduced the apparent fracture resistance. As ideally detailed, the joint has only 1/8 in. separating the welds on the two plates. The fabrication tolerance resulted in reduced gaps as well as intersecting welds in many locations throughout the structure. Stress analysis showed that the intersecting welds increased the rigidity of the joint and made the constraint problem worse. This non-ductile behavior in the joint caused by a triaxial constraint and state of stress has never been documented before as being a potential problem in bridge detailing. This is the first time this problem is being reported.

Additionally, the “K” pattern in the lower lateral brace system introduces an axial force in the girder to satisfy equilibrium in the joint area. A stress analysis showed that this increased the live load stress range at the outside ends of the shelf plate, but that there was little effect in the gap area.
Joint Details

The primary cause of fracture initiation was determined to be the geometry and fabrication tolerances of the joint, the lateral barriers for cracking through the web. This non-ductile behavior in the joint caused by a triaxial constraint and state of stress has never been documented before as being a potential problem in bridge detailing. This is the first time this problem is being reported.

in the girder to satisfy equilibrium in the joint area. A stress analysis showed that this increased the live load stress range at the outside ends of the shelf plate, but that there was little effect in the gap area.
Evaluation of Steel Bridge Details for Susceptibility of Constraint-Induced Fracture

Publication No. FHWA-HIF-21-046
September 2021
## Evaluation of Steel Bridge Details for Susceptibility of Constraint-Induced Fracture

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<td>Domenic Coletti, P.E., Brandon Chavel, P.E., PhD., Anthony Ream, P.E., Caroline Bennett, P.E., PhD., Rob Connor, P.E., PhD., Karl Frank, P.E., PhD., Michael Grubb, P.E., Finn Hubbard, P.E., Ronnie Medlock, P.E., Duane Miller, Sc.D., P.E., Frank Russo, P.E., PhD.</td>
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<td>301 Grant Street, Suite 1700</td>
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<td>Pittsburgh, PA 15219-1408</td>
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16. Abstract
This report explains how to evaluate steel bridge details for susceptibility to constraint-induced fracture. The report begins with a review of fundamental principles of ductile behavior of steel structures and the effects of constraint and stress triaxiality. A brief history of constraint-induced fractures of steel bridges in the United States and a review of published research, policies, and practices is also provided. The report then presents a possible method for evaluating a steel detail for the presence of the three conditions associated with elevated susceptibility to constraint-induced fracture: high tensile stresses (including residual stress effects), a high degree of constraint, and planar discontinuities approximately perpendicular to the primary flow of tensile stresses. Next, a series of commonly used steel bridge details are evaluated to illustrate the procedure and to provide a baseline library of evaluations. Redesign, inspection, retrofit, and repair options for problematic details are briefly discussed. The report also presents general design details and construction considerations and possible future research topics.
This report explains how to evaluate steel bridge details for susceptibility to constraint-induced fracture. The report begins with a review of fundamental principles of ductile behavior of steel structures and the effects of constraint and stress triaxiality.
The findings in this report are:

- Steel bridge details featuring intersecting welds are not necessarily at elevated susceptibility to CIF.

- Three conditions typically contribute to elevated susceptibility of steel bridge details to CIF: a high net tensile stress, a high degree of constraint, and a planar discontinuity approximately perpendicular to the primary flow of tensile stress.

- Evaluating details with respect to criteria rooted in a technical understanding of CIF can help bridge owners identify details that are candidates for redesign and retrofit.

- Retrofitting and redesigning details with intersecting welds without proper understanding of CIF can lead owners to undertake design and/or retrofit strategies that may result in poorer, not better, performance.
Three conditions typically contribute to elevated susceptibility of steel bridge details to CIF: a high net tensile stress, a high degree of constraint, and a planar discontinuity approximately perpendicular to the primary flow of tensile stress.
CHAPTER 3 - STRESS TRIAXIALITY, CONSTRAINT, AND SUSCEPTIBILITY TO CIF

3.1 FUNDAMENTAL PRINCIPLES OF DUCTILE BEHAVIOR OF STEEL STRUCTURES AND THE EFFECTS OF CONSTRAINT AND STRESS TRIAXIALITY

While it has often been said that steel is an inherently ductile material, that ductile nature can be compromised if a structure is detailed in manner that inhibits the typical stress-strain behavior of the material. Clarification of this concept is instructive in understanding the nature and causes of CIF.
While it has often been said that steel is an inherently ductile material, that ductile nature can be compromised if a structure is detailed in a manner that inhibits the typical stress-strain behavior of the material.
Connor and Lloyd (2017) describe three conditions that contribute to the susceptibility of a detail to CIF:

1. “There must be an elevated level of tensile residual stress locked into the local area. While the dominating contribution is residual stresses from welding, other factors contribute to a lesser degree, such as dead load and erection stress. As is well documented, residual stresses due to welding can easily reach the yield strength of the base metal.

2. “The joint must be highly constrained, resulting in a three-dimensional state of stress that prevents plastic flow, as would [otherwise] occur in a simple uniaxial stress state.

3. “Localized area of stress concentration that intensifies dead load and live load stress level.”
Maintenance Actions to Address Fatigue Cracking in Steel Bridges Structures

PROPOSED GUIDELINES AND COMMENTARY

Connor and Lloyd
March, 2017
There are three contributing elements to constraint-induced fracture, characteristic of all CIF-prone
details, which when any one of the elements is missing, the likelihood of constraint-induced fracture drops
dramatically. Figure 7.3 illustrates these elements, conceptually showing that the risk of CIF exists at the
intersection of the three elements.

1. There needs to be a localized area of stress concentration that intensifies the dead and live
load stress level. The presence of defects within the weld, as well as certain geometry of the
connection can both act as discontinuities that interrupt stress flow and cause concentrations.
2. The joint must be highly constrained, resulting in a three dimensional state of stress that
prevents plastic flow, as would occur in a simple uniaxial stress state.
3. There must be an elevated level of tensile residual stresses locked into the local area. While
the dominating contributor are residual stresses from welding, other factors contribute to a
lesser degree, such as dead load and erection stress. As is well documented, residual stresses due to welding can easily reach the yield strength of the base metal.
There are three contributing elements to CIF... dramatically. Figure 7.3 illustrates these elements, conceptually showing that the risk of CIF exists at the concentrations.

1. There needs to be a localized area of stress concentration...

2. The joint must be highly constrained...

3. There must be an elevated level of tensile residual stresses locked into the local area. While residual stress can be measured explicitly, the determination of local areas is a more challenging aspect.
Maintenance Actions to Address Fatigue Cracking in Steel Bridges Structures

Figure 7.3 Defining characteristics of CIF details
Subsequent assessments of the Hoan Bridge fracture, studies of similar fractures in other bridges, and other related research and investigations, largely supported this conclusion (e.g., Fisher et al. 2001; Wright et al., 2003). The cause of the Hoan Bridge fracture was CIF originating in details with high-stress triaxiality, which resulted from:

- a high level of constraint, provided by the various attachments locally constraining the ability of the web to yield;
- high levels of tensile stress associated with residual stresses induced by welding of the various attachments to the web; and
- crack-like geometry, specifically where the so-called “web gap” (a constraint-relief gap) between the lateral bracing connection plate (the “gusset plate” in Figure 19) and the cross-frame connection plate (the “transverse connection plate” in Figure 19) was very narrow.

The steel was found to exhibit reasonable toughness with no evidence of fatigue cracking prior to the CIF event.
The cause of the Hoan Bridge fracture as CIF originating in details with high-stress triaxiality, which resulted from:

- A high level of constraint…
- High levels of tensile stress associated with residual stress induced by welding…
- A crack-like geometry…

The steel was found to exhibit reasonable toughness with no evidence of fatigue cracking prior to the CIF event.
At the same time, the non-binding Reference Manual for FHWA/NHI *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual* (Russo et al., 2016), provides a suggestion to use a wider constraint-relief gap, and directly quotes language from the same article of the previous 7th Edition of the AASHTO BDS, which is different from Article 6.6.1.2.4 of the AASHTO BDS, 8th Edition (23 CFR 625.4(d)(1)(v)):

To the extent practical, welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture. Welds that are parallel to the primary stress but interrupted by intersecting members shall be detailed to allow a minimum gap of 1 inch between weld toes.
To the extent practical, welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture.
Case Study Lessons

Liberty Ships

- Notches are bad
- Square corners are bad
- Notch sensitive steel is bad
- Good design is important
- Good fabrication is important
- Notch tough steel is helpful
Case Study Lessons

Silver Bridge

• High hardness, subject to SCC, is bad
• High stresses are bad
• Initial fabrication discontinuities are bad
• Cyclic loading can extend initial discontinuities
• Low fracture toughness is bad
• Non-redundant designs can fail catastrophically
Case Study Lessons

Ingram Barge

- Overloading of barges is bad
- Highly constrained details are bad
- Constraint can induce fracture with no pre-existing cracks
- Good notch toughness does not preclude fracture in highly constrained details
Case Study Lessons

Hoan Bridge

- Highly constrained details are bad
- Constraint can induce fracture with no pre-existing cracks
- Good notch toughness does not preclude fracture in highly constrained details
<table>
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<th>Detailing/Constraint</th>
<th>Notches/Cracks</th>
<th>Loading</th>
<th>Material Toughness</th>
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Commentary A3.1a

“Good workmanship and good design details incorporating joint geometry that avoids severe stress concentrations are generally the most effective means of providing fracture-resistant construction.”
“Fracture mechanics has shown that because of the interaction among materials, design, fabrication, and loading, brittle fractures cannot be eliminated in structures merely by using materials with improved notch toughness. The designer still has the fundamental responsibility for the overall safety and reliable of his or her structure.”
To the extent practical, welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture.
It is not possible to simply quantify mathematically the degree of restraint offered by the surrounding steel, but an intuitive feel can be developed.
Flange Splice

3 in. (75 mm) thick, 10 in. (250 mm) wide,
Two 40 foot (13 m) lengths
Flange Splice

3 in. (75 mm) thick, 10 in. (250 mm) wide,
Two 40 foot (13 m) lengths
Wide Flange Splice

W14 X 730
5 in. (125 mm) thick flange, 3 in. (75 mm thick web)
High Constraint
and a bad sequential practice
Brittle Fracture: Another View

- Definition of brittle fracture
- Significance of brittle fracture
- Factors affecting brittle fracture
- Case studies involving brittle fracture
- Designing to prevent brittle fracture
A Holistic Approach to Improving Fracture Resistance in Cold Temperature Applications
Principle 1: Reduce Stress

1.1 Reduce the loads/forces.
1.2 Increase the resisting area/section.
1.3 Provide easy paths for stress flow though the member.
1.4 Provide gradual changes in stiffness and section.

\[ K_C > \sigma \sqrt{\pi a} \]
Principle 1: Reduce Stress

1.5 Eliminate the number and severity of localized stress concentrations.
1.6 Locate welded joints at points of low stress when possible.
1.7 Avoid the introduction of secondary stresses.
1.8 Avoid the introduction of triaxial constraint.
Principle 1: Reduce Stress

1. When applicable, consider proof loading.
1.10 Consider thermal stress relief.
1.11 Provide “contouring” fillet welds at T and corner joints.
1.12 Provide a minimum radius at copes and re-entrant corners.
Principle 2: Reduce Flaw Size

2.1 Select materials with good weldability.
2.2 Provide ample access for welding and inspection.
2.3 Carefully inspect incoming steel.
2.4 Visually inspect cut surfaces.
2.5 Control the quality of cut surfaces.
$K_C > \sigma \sqrt{\pi a}$

Principle 2: Reduce Flaw Size

2.6 Drill holes versus punching them, or ream punched holes.
2.7 Take measures to eliminate all forms of fabrication-related weld cracking.
2.8 Use weld tabs on groove welds, where practical, and remove them after welding.
2.9 Control tack welding
Principle 2: Reduce Flaw Size

2.10 Require continuous steel backing (where backing is needed and when left in place).
2.11 Remove steel backing, as applicable.
2.12 Consider roots of fillets and PJP groove welds in cruciform joints.
2.13 Inspect welds for surface breaking flaws.
2.14 Inspect welds for internal flaws.
Principle 3: Increase Material Toughness

3.1 Specify materials with known toughness.
3.2 Realize that steel is not purely isotropic.
3.3 Recognize areas of potential low toughness in steel members.
3.4 Increase the temperature shift.
3.5 Properly establish the operating temperature of the steel structure or weldment.
3.6 Develop a limit for low temperature operation.

\[ K_C > \sigma \sqrt{\pi a} \]
Principle 4: Increase Fatigue Life

4.1 Reduce the stress range.
4.2 Use improved fatigue details.
4.3 Limit the life of the weldment.
4.4 Use fatigue life enhancement techniques.
4.5 Recognize the role of steel strength in fatigue of weldments.

\[ K_C > \sigma \sqrt{\pi a} \]
Principle 5: Additional Considerations

5.1 Consider the effects of corrosion.
5.2 Develop and implement a realistic maintenance program.
5.3 Develop a realistic in-service inspection program.

\[ K_C > \sigma \sqrt{\pi a} \]
\[ K_C > \sigma \sqrt{\pi a} \]

**Principle 5: Additional Considerations**

5.4 Consider the use of structural redundancy.
5.5 Recognize there are no secondary members in welded construction.
5.6 Carefully select the appropriate strength level for the steel.
43 Ideas For Increased Fracture Resistance

Principle 1: Reduce Stress (12)
Principle 2: Reduce Flaw Size (14)
Principle 3: Increase Material Toughness (6)
Principle 4: Increase Fatigue Life (5)
Principle 5: Additional Considerations (6)

1 Involves Specification of Higher Material Toughness
Brittle Fracture: Another View

- Definition of brittle fracture
- Significance of brittle fracture
- Factors affecting brittle fracture
- Case studies involving brittle fracture
- Designing to prevent brittle fracture
Brittle Fracture: Another View