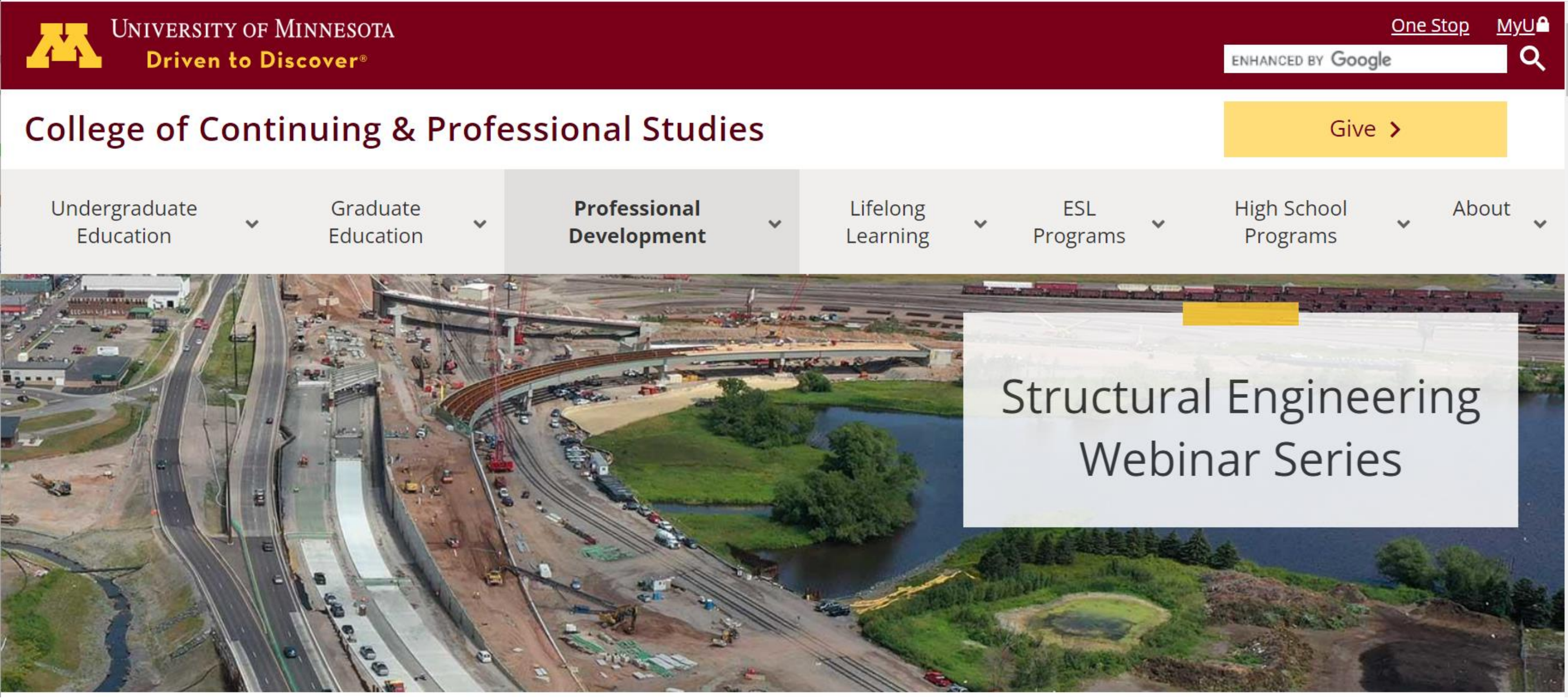


LISTEN to the STEEL

Brittle Fracture: Another View

LINCOLN® ELECTRIC

Brittle Fracture:

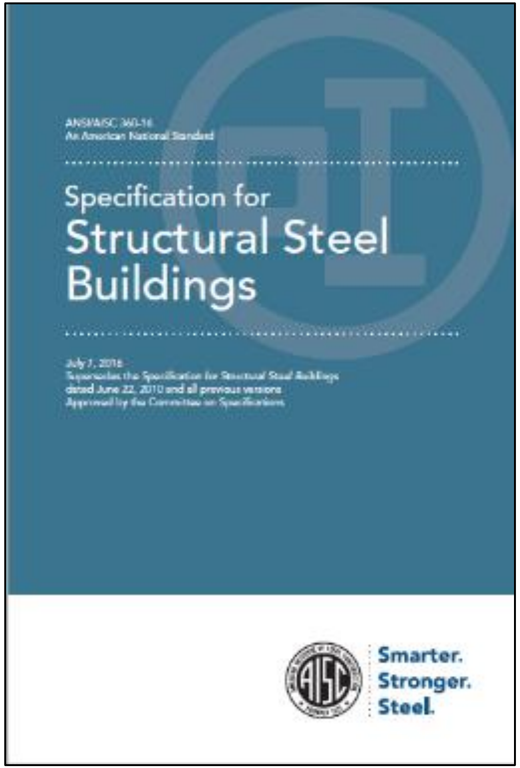
Another View

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

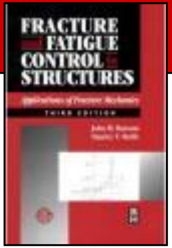
COMMENTARY GLOSSARY

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).

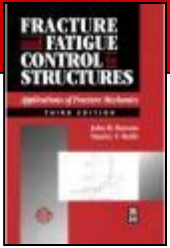


Brittle fracture is a type of failure in structural materials that usually occurs without prior plastic deformation and at extremely high speeds (as fast as 7000 ft/s [210 m/s] in steels). The fracture is usually characterized by a flat cleavage fracture surface…and at average stress levels below those of general yielding.

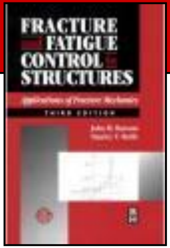
Gensamer: STRENGTH OF METALS UNDER COMBINED STRESS

It is well known that a metal may be ductile under one set of conditions and brittle under another.

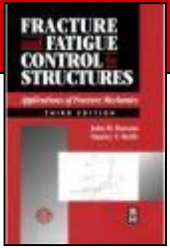
Ductility and brittleness, then are properties that must be considered as referring to some particular set of testing or service conditions.



Most structural materials exhibit considerable strain (deformation) before reaching the tensile or ultimate strength, σ_{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 *like steel* exhibit considerable strain (deformation) before reaching the tensile or ultimate strength, σ_{tens}….In contrast, brittle materials *like cast iron* exhibit almost no deformation before fracture….However, under conditions of low temperature, rapid loading and/or high constraint…even ductile materials *like steel* 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.

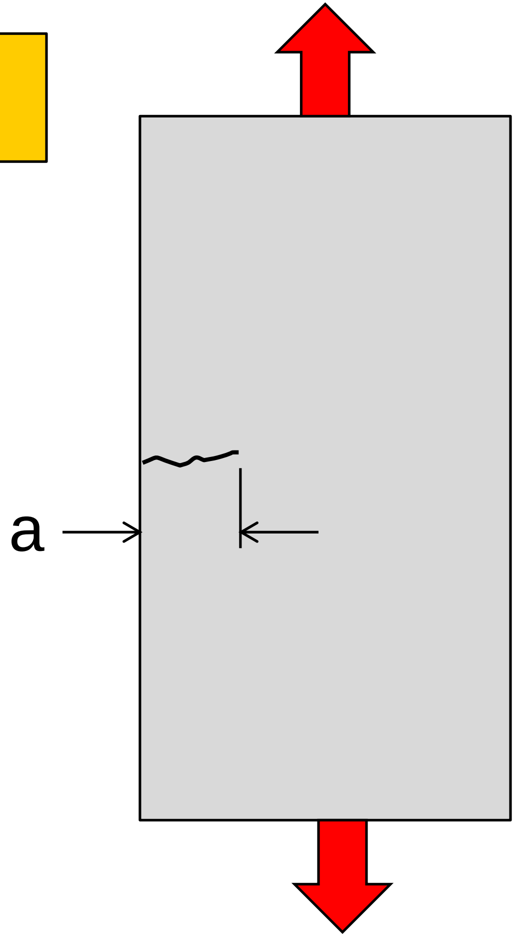
Basic Fracture Mechanics

Material fracture toughness (K_{IC})

Stress (σ)

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

Flaw size (a)

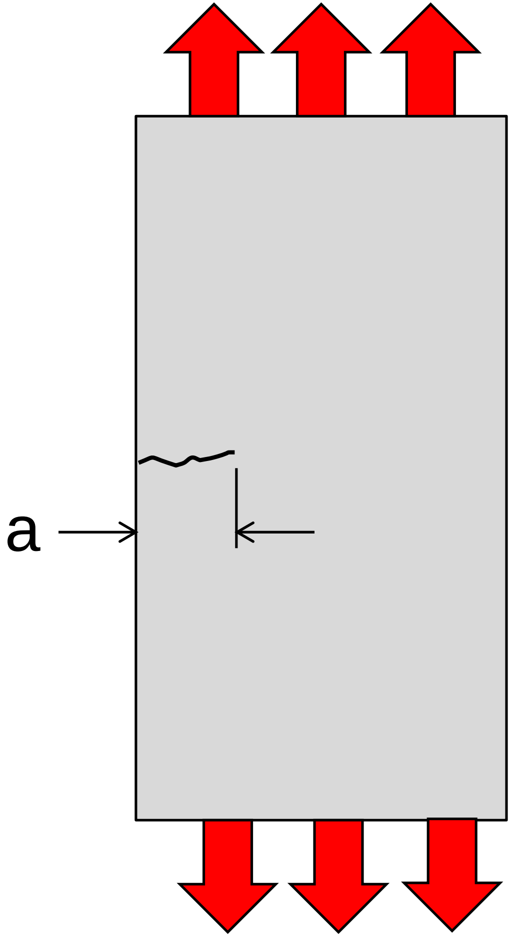


Basic Fracture Mechanics

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

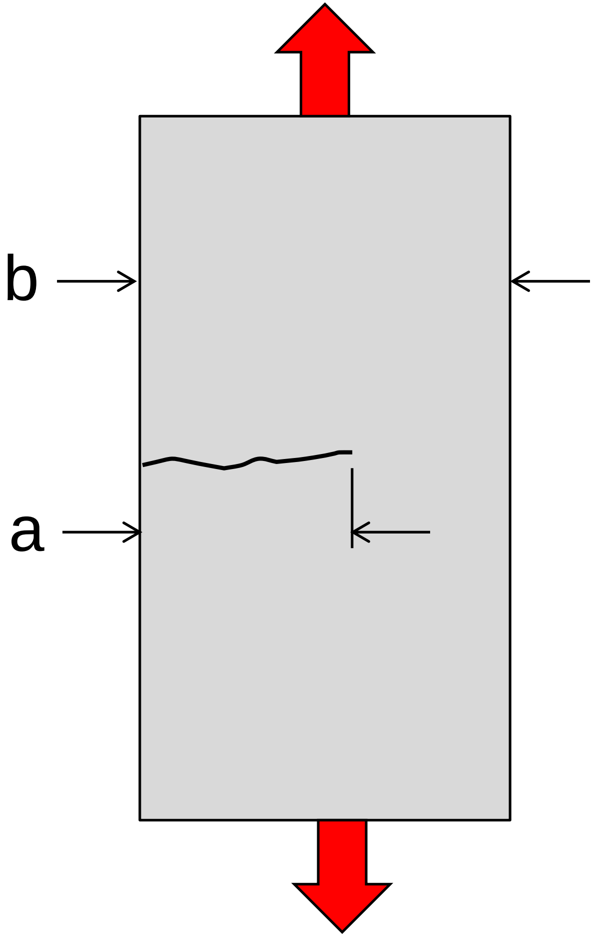
If K_{IC} is high enough,
σ can be > F_u.

Net section will
eventually control.



Basic Fracture Mechanics

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



If K_{IC} is high enough,
a can be > b/2.

Net section will
eventually control.

Basic Fracture Mechanics

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

If $a = 0$, σ can be infinite, even if K_{IC} is low.

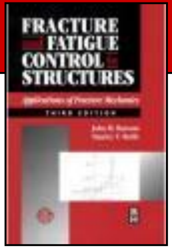
Gross section will eventually control.

Brittle Fracture:

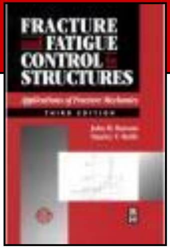
Another View

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

Brittle fracture 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

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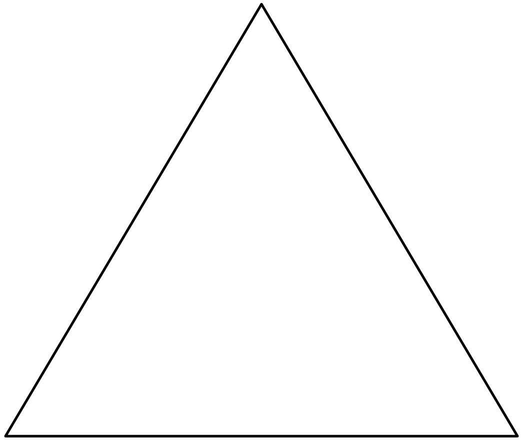
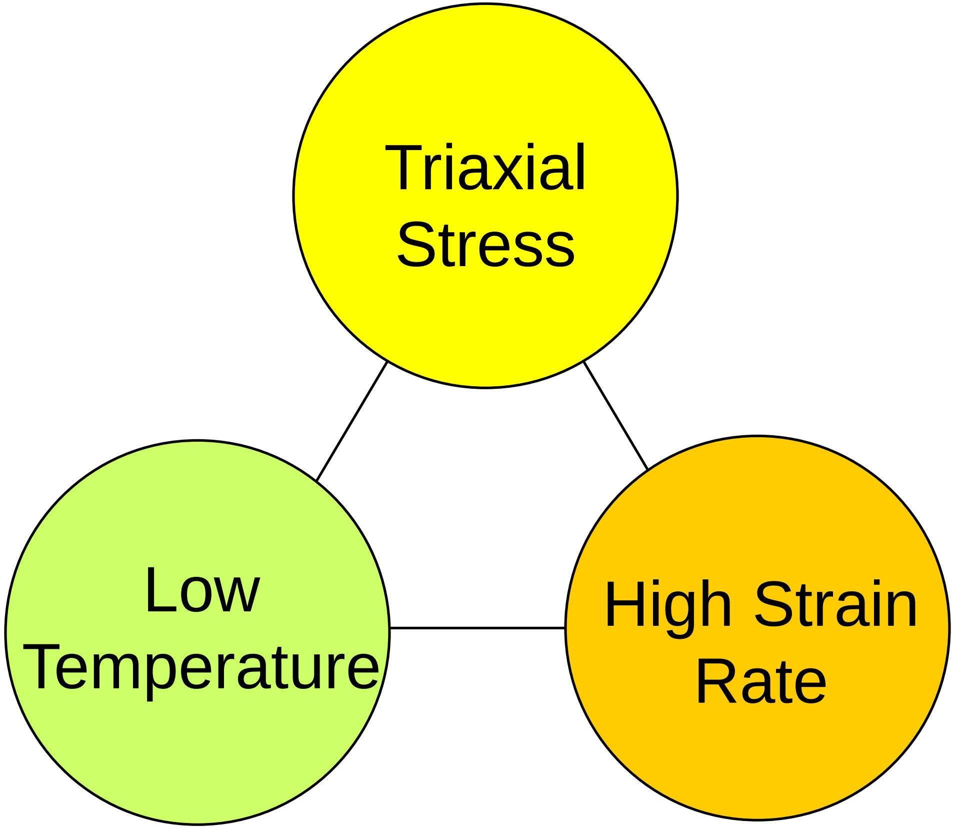
The Holy Trinity

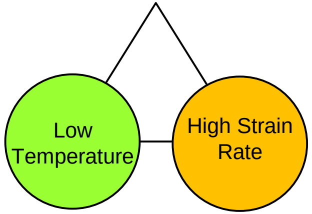


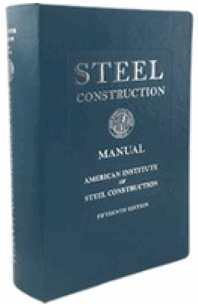
The Unholy Trinity



Commentary A3.1a

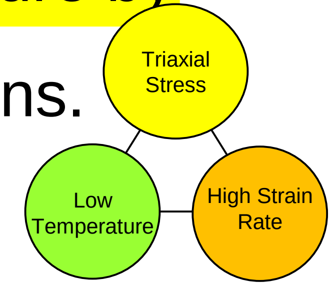
For especially demanding service conditions such as structures exposed to low temperatures, particularly those with impact loading, the specification of steels with superior notch toughness may be warranted.





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 it nominal value, resulting in brittle fracture by cleavage, rather than ductile shear deformations.

page 2-38



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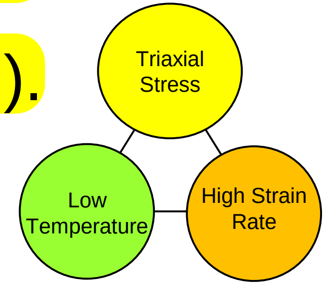

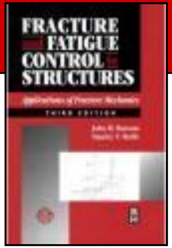

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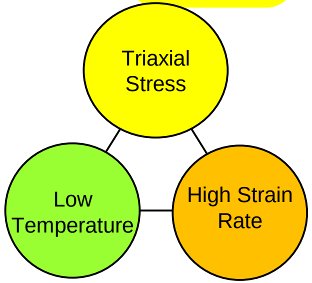
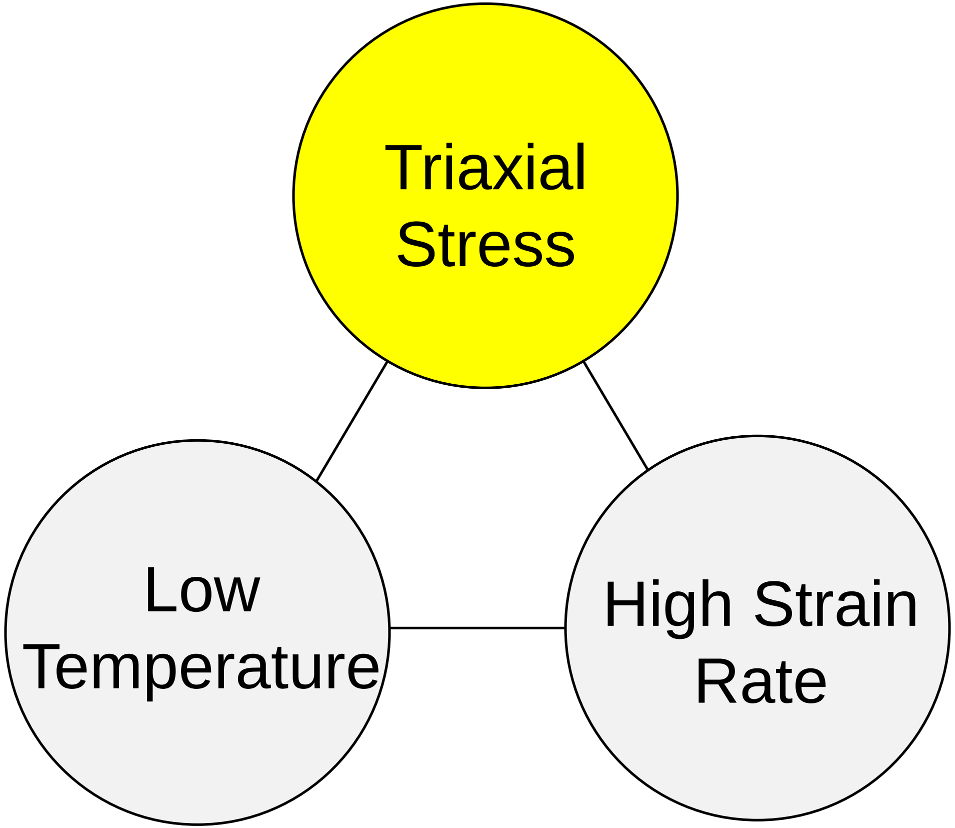
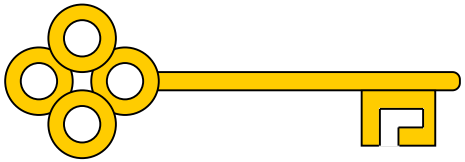


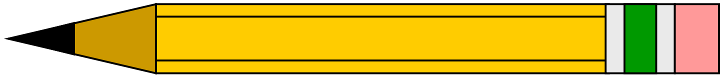
The Unholy Trinity



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.



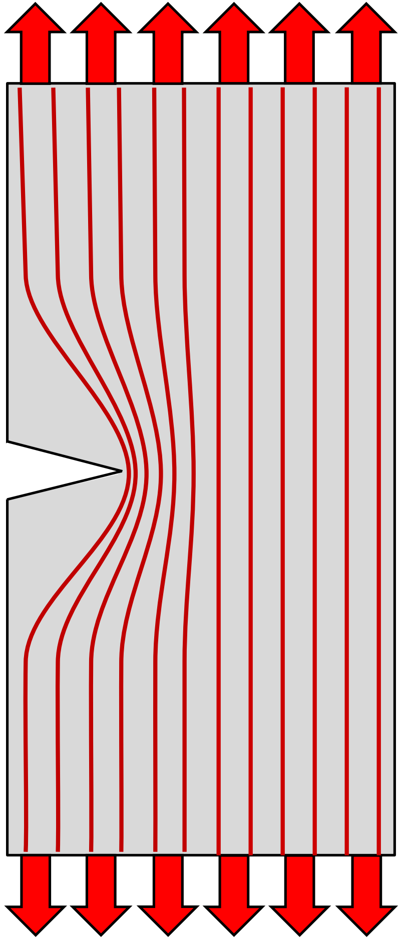


In summary, a notch increases the tendency for brittle fracture in four important ways:

- 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:

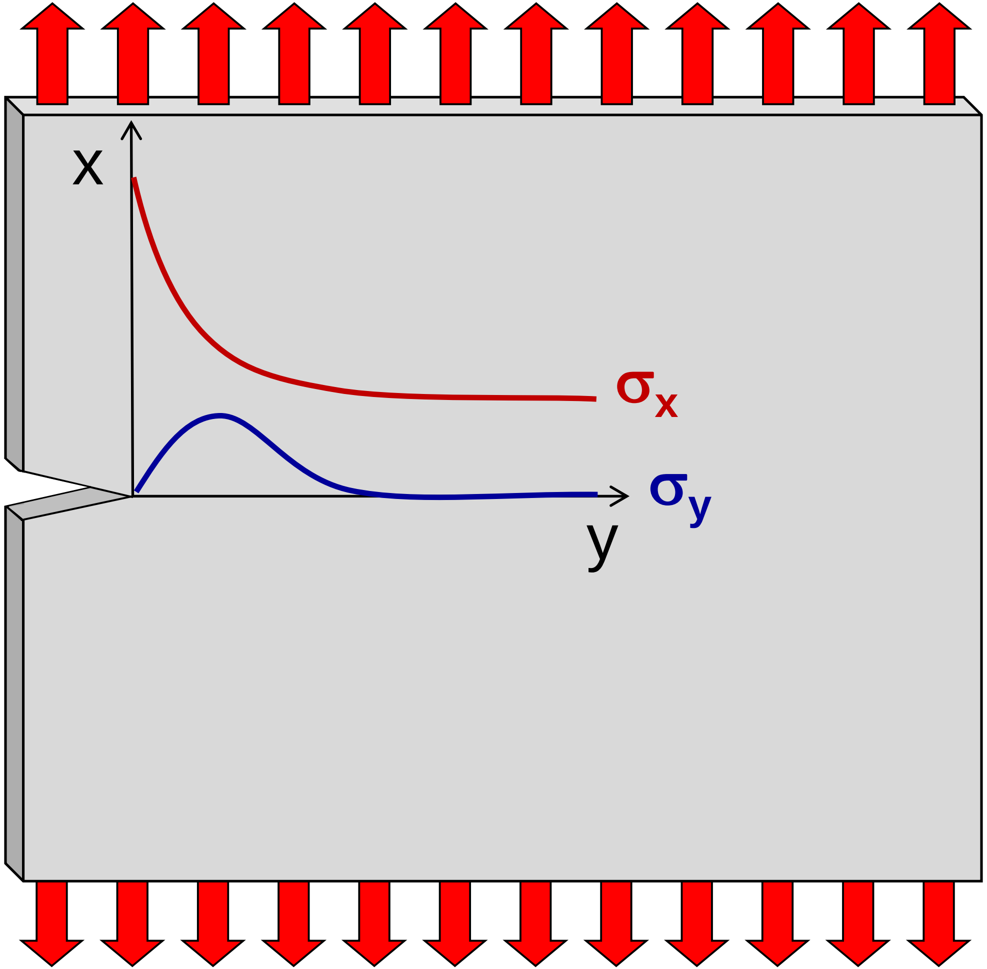
- $A_{net} < A_{gross}$
- Stress is not uniform



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



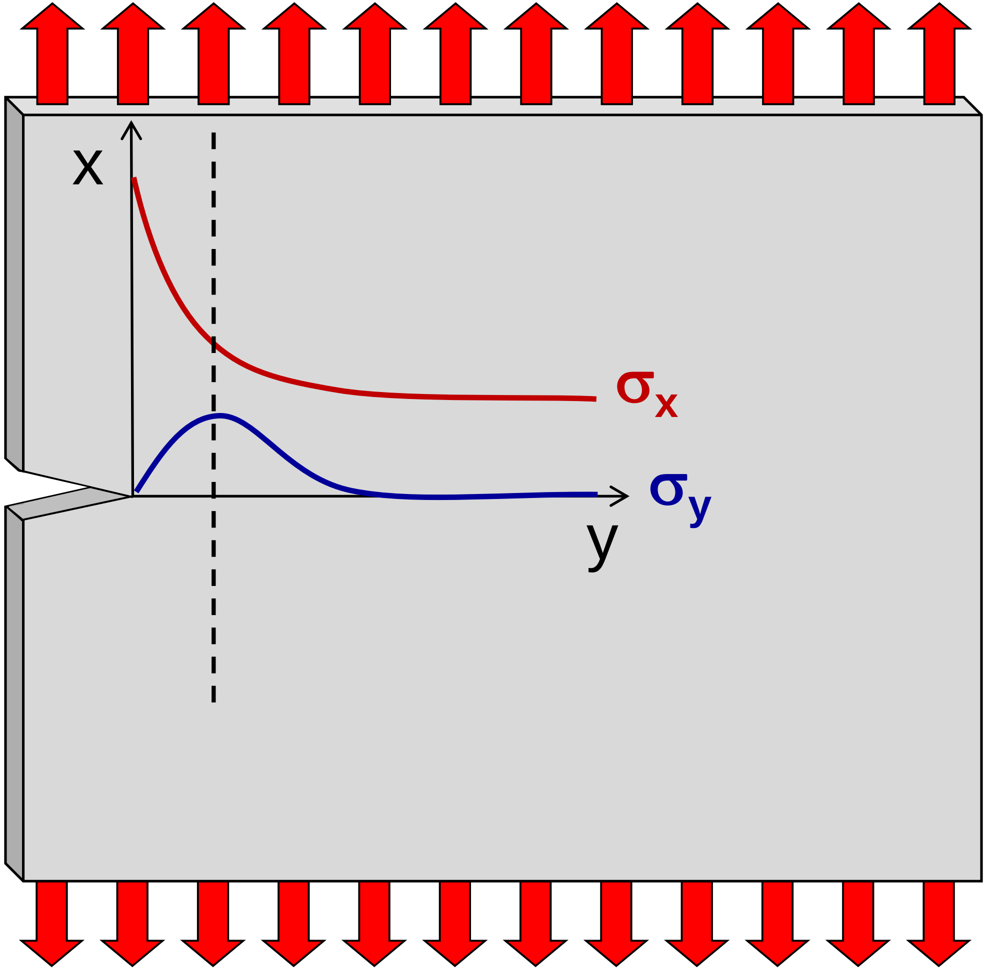
Plane-stress

Thin Plate

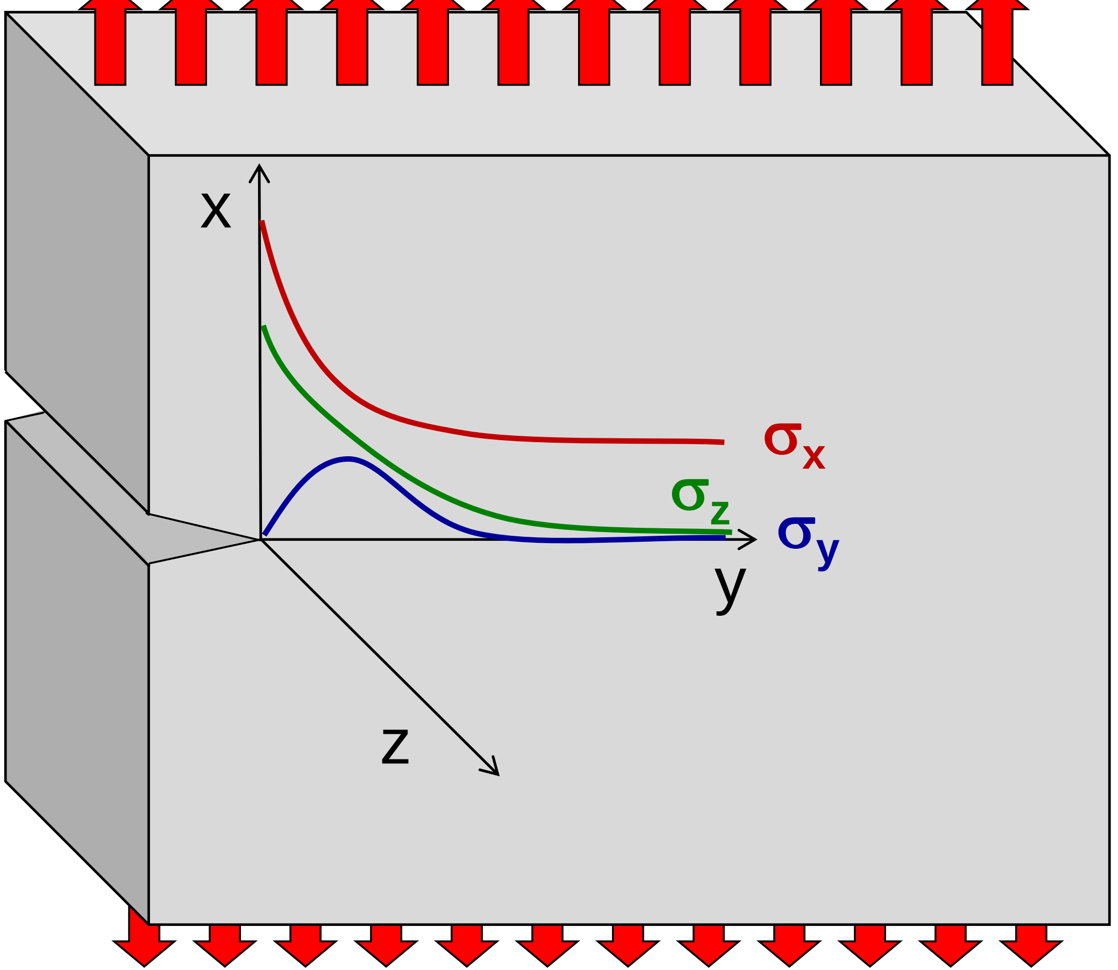
$\sigma_x = +$

$\sigma_y = +$

$\sigma_z = 0$



Thick Plate



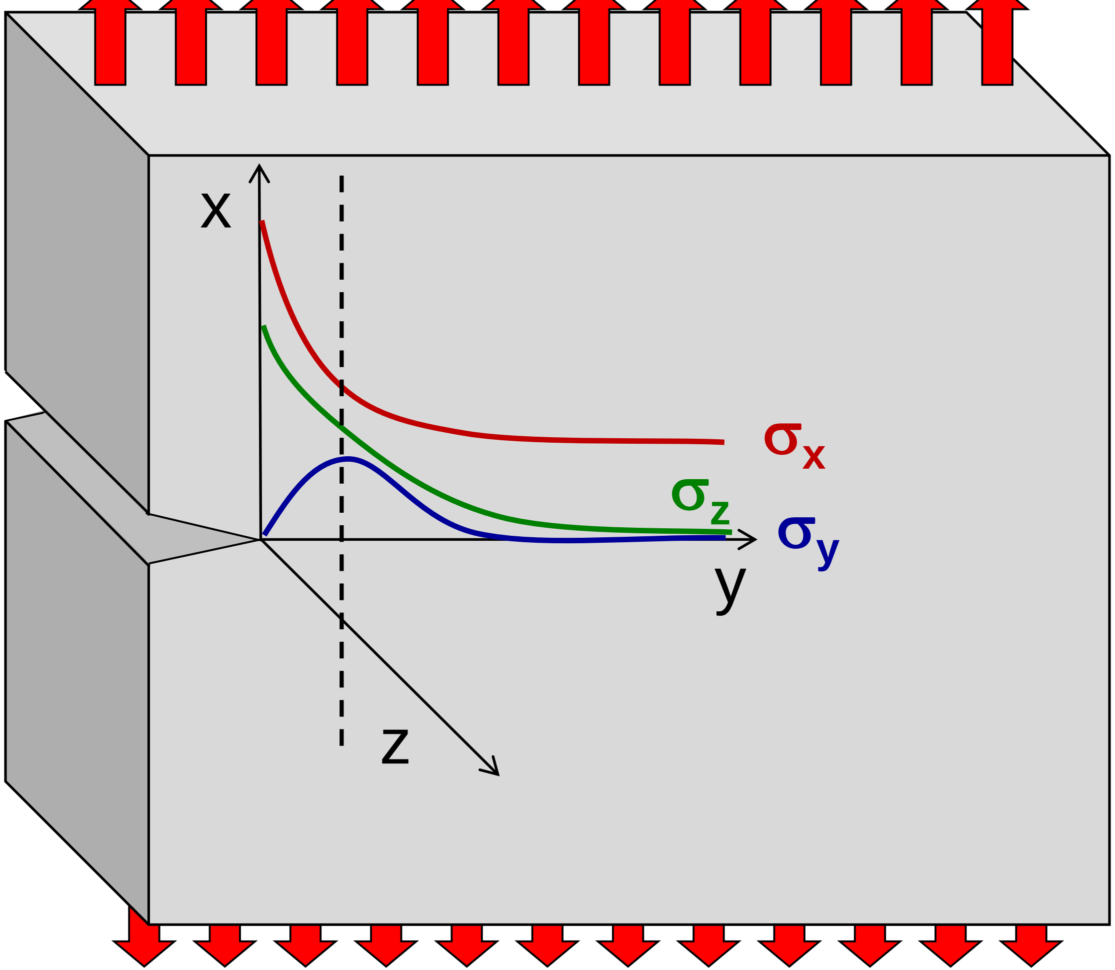
Plane-strain

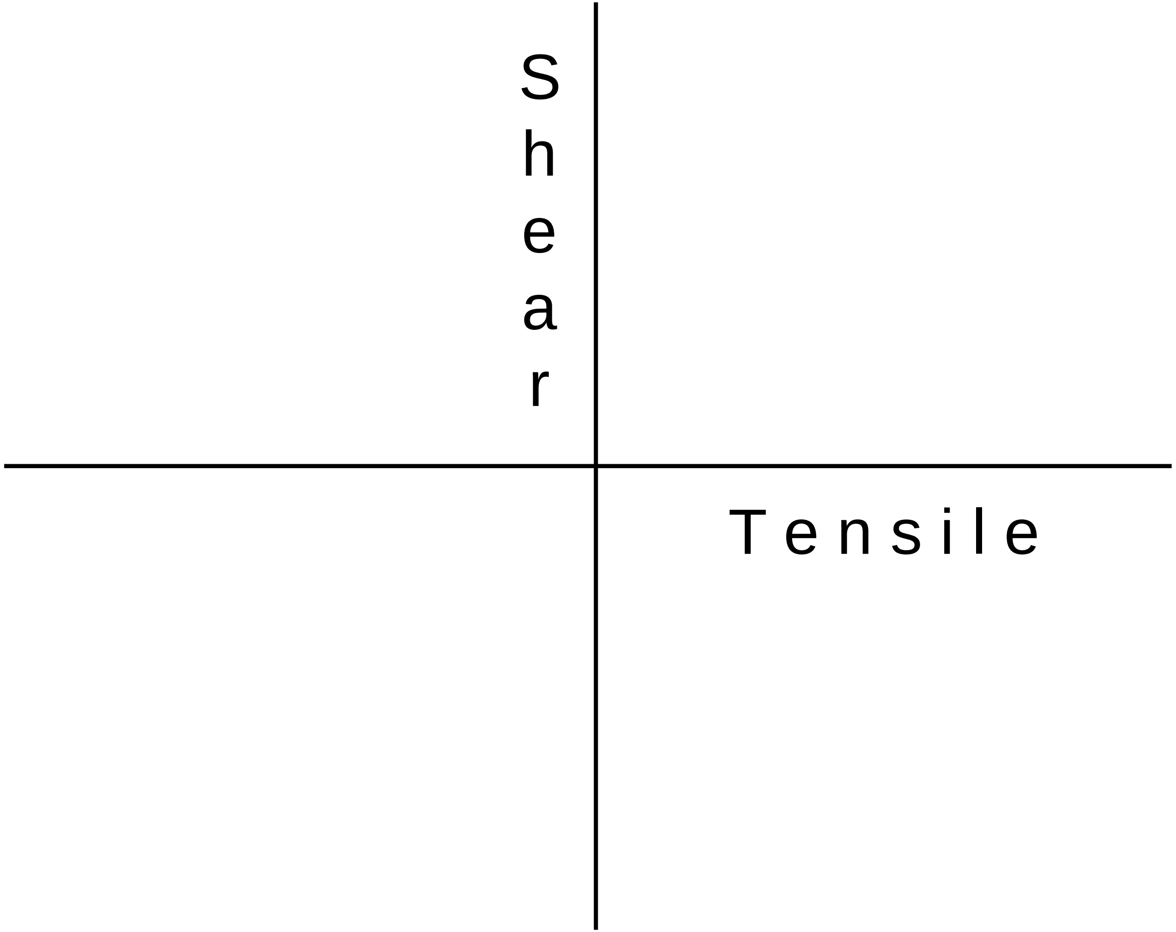
Thick Plate

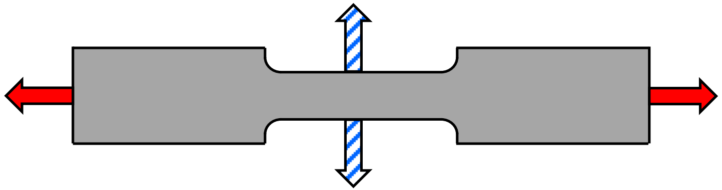
$\sigma_x = +$

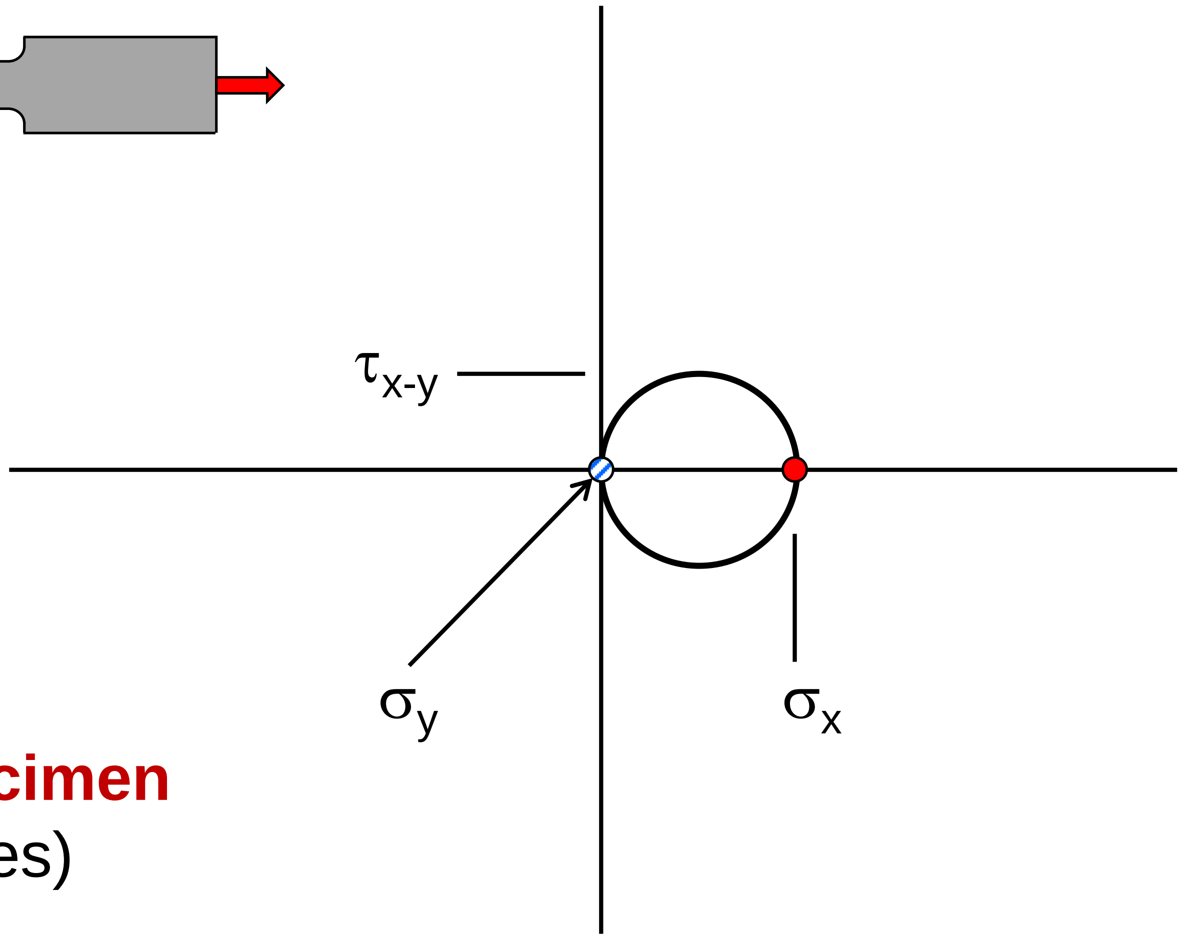
$\sigma_y = +$

$\sigma_z = +$

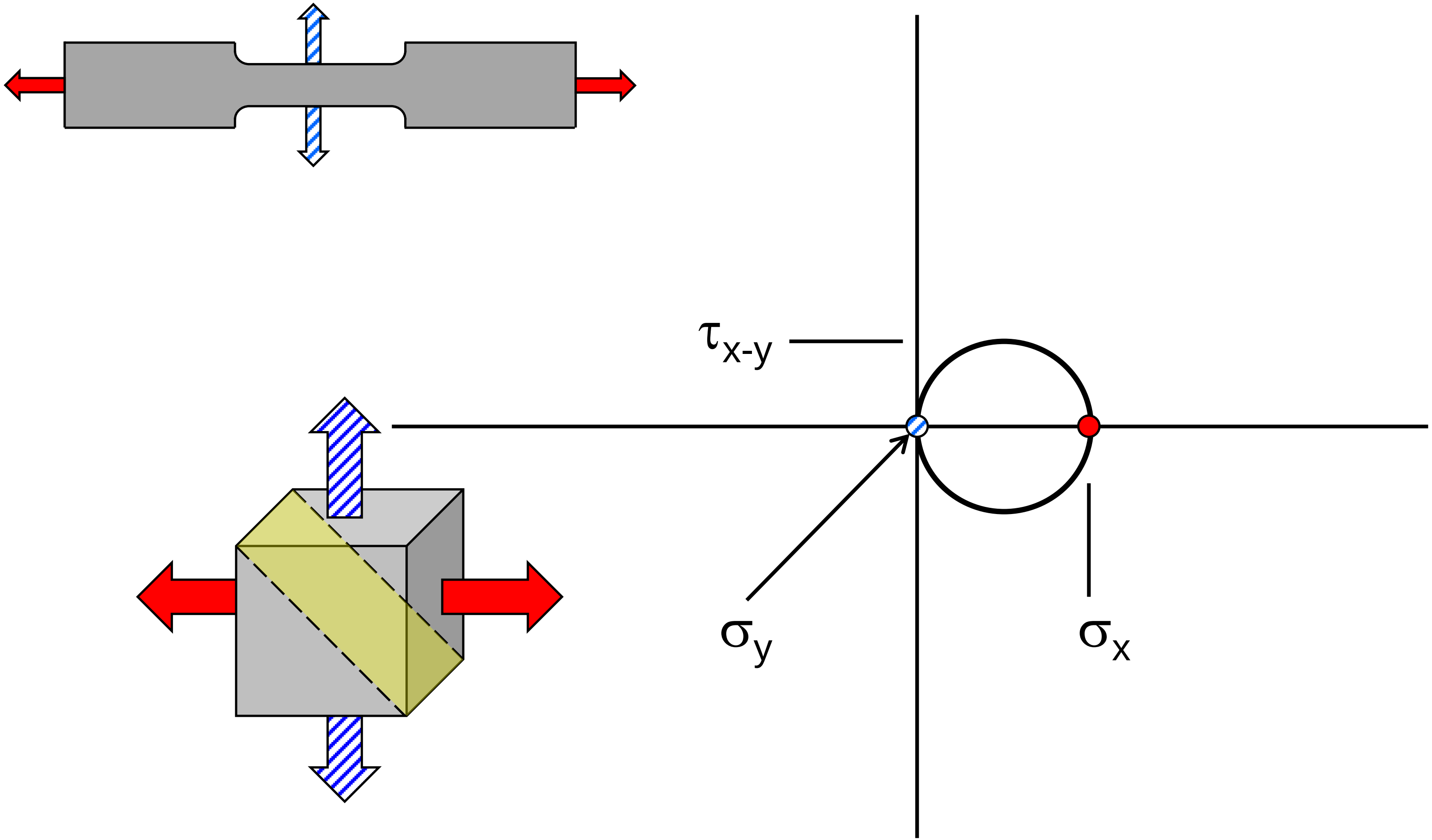



Shear
Tensile

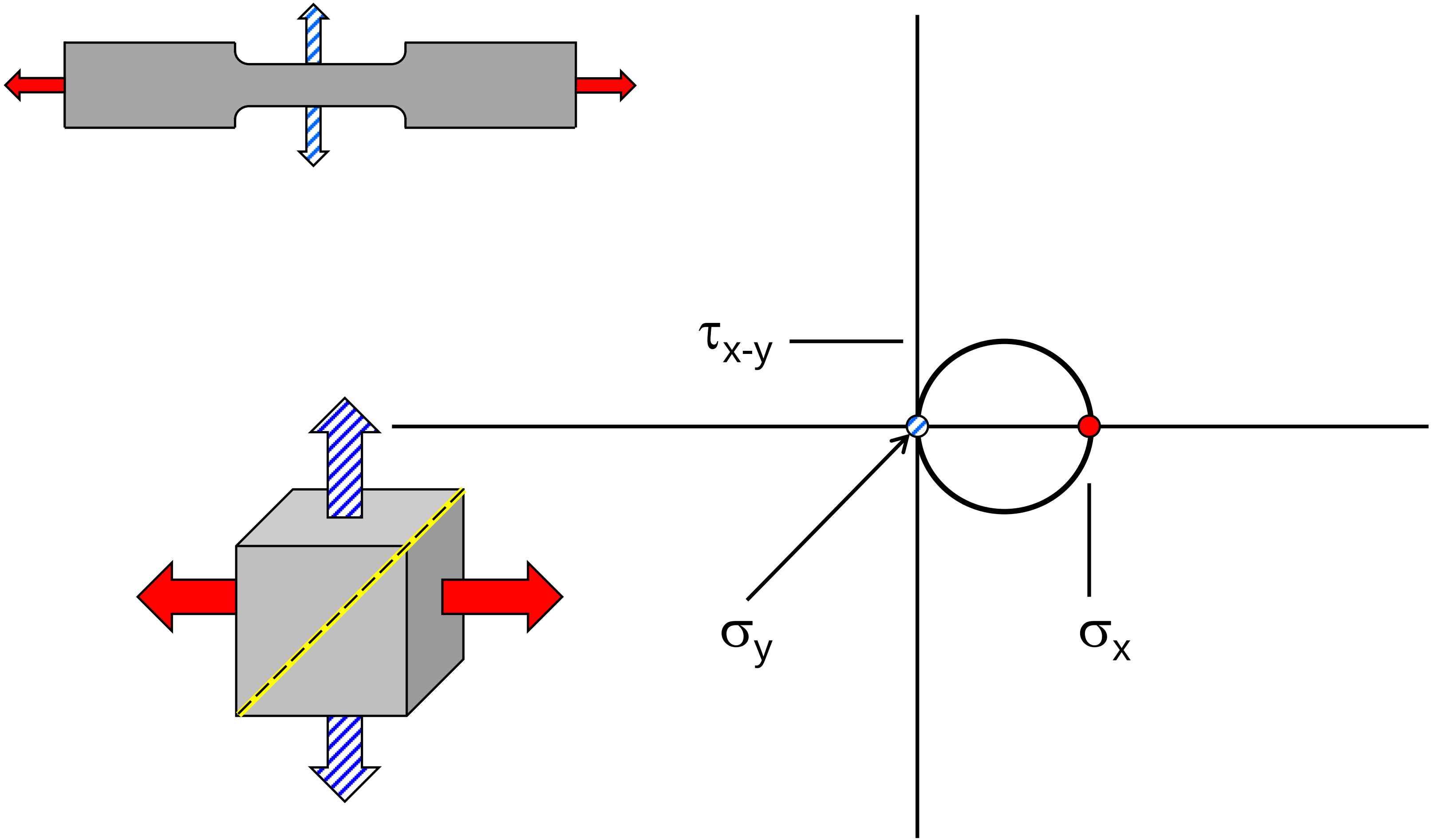




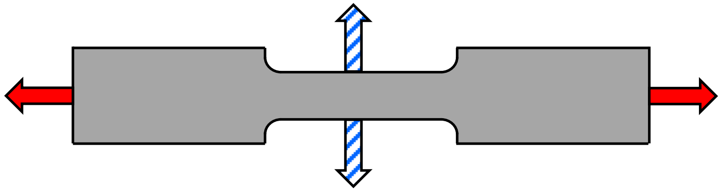
Smooth specimen
(no notches)

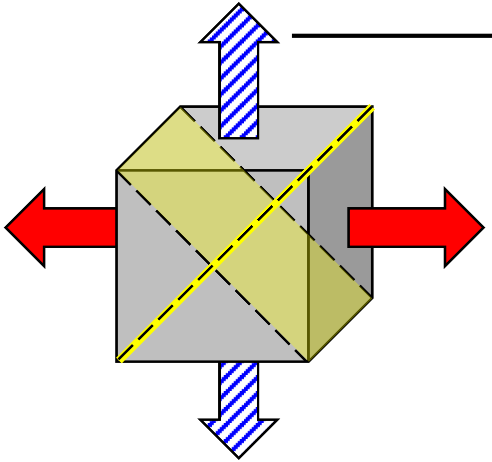


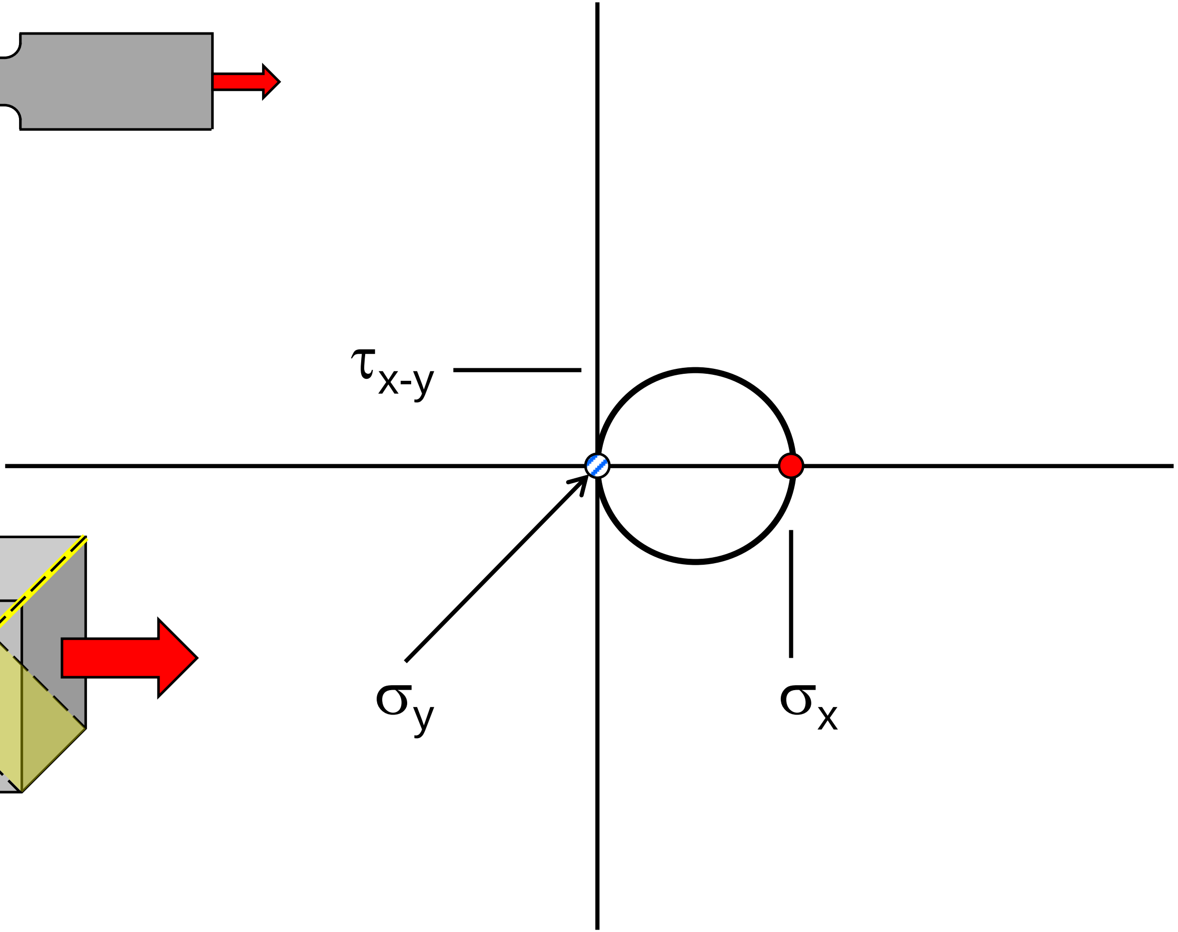
τx-y
σy
σx



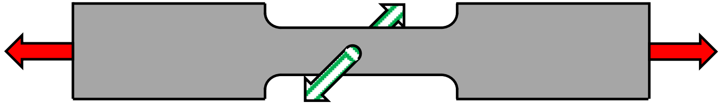
τ_{x-y}
σ_y
σ_x

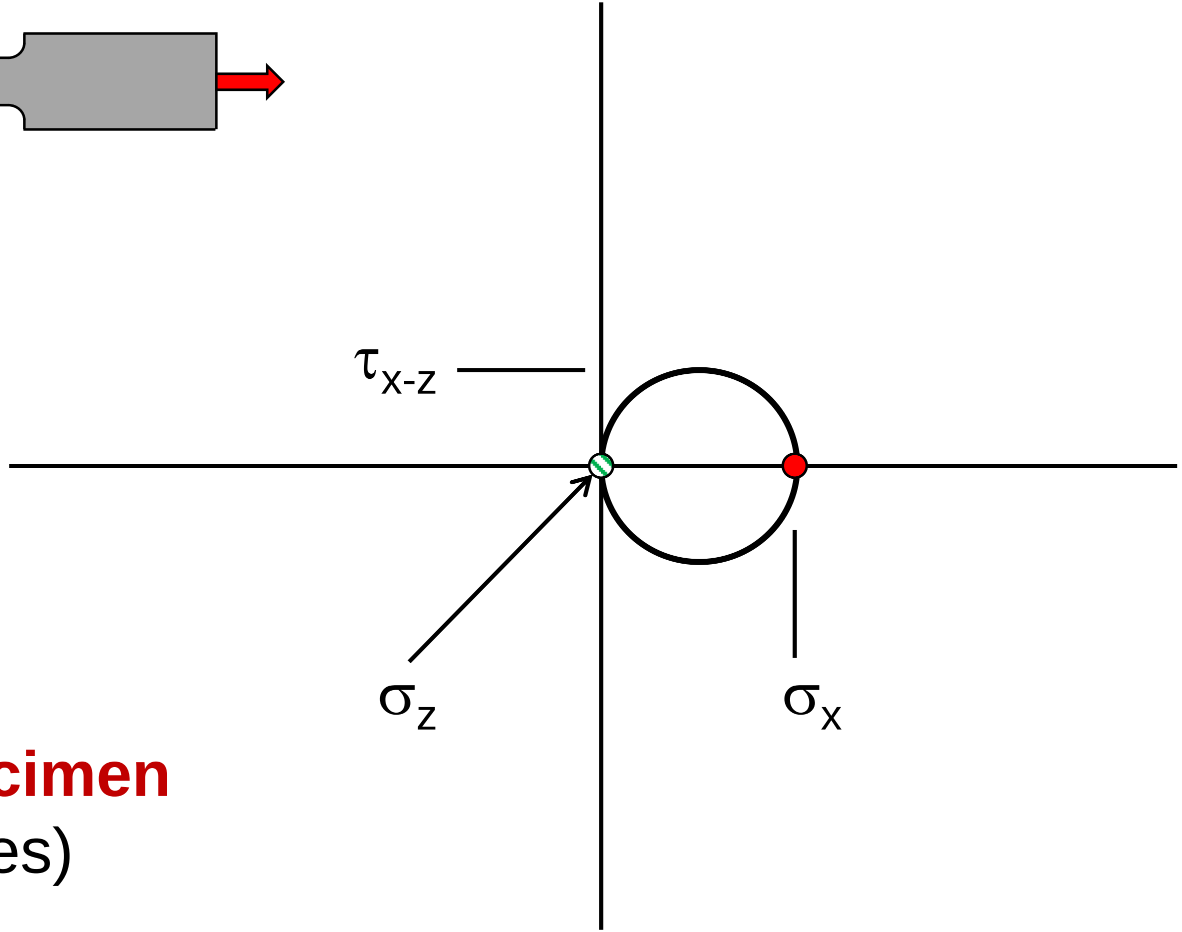




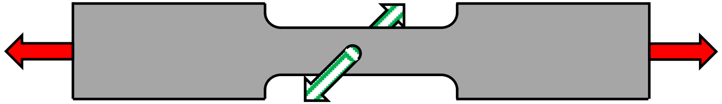


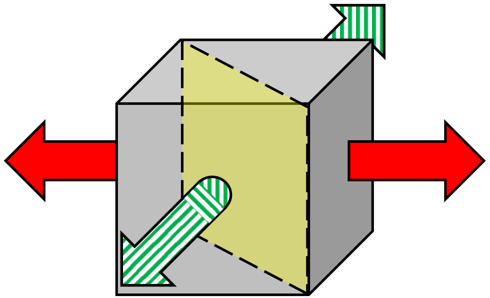
$\tau_{x\text{-}y}$
σ_y
σ_x

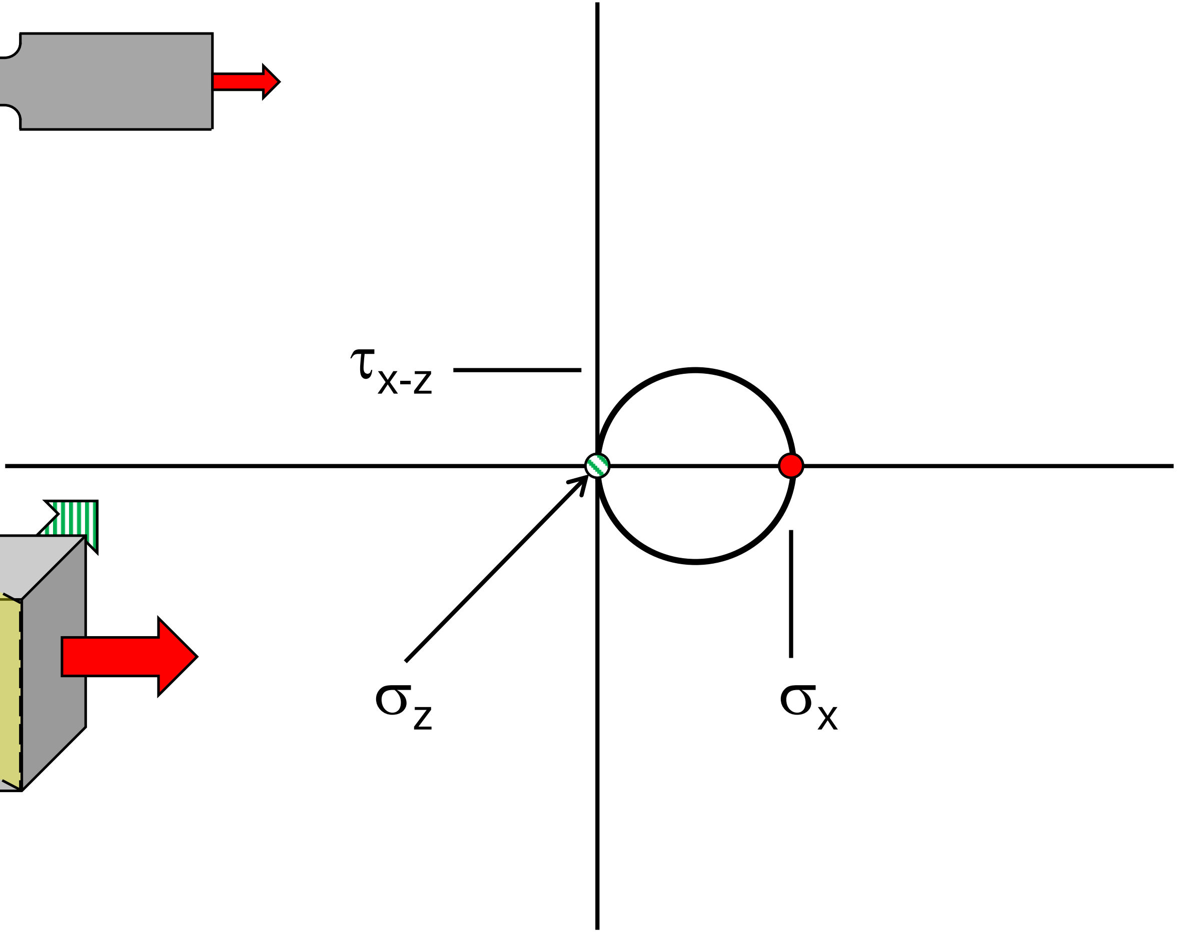


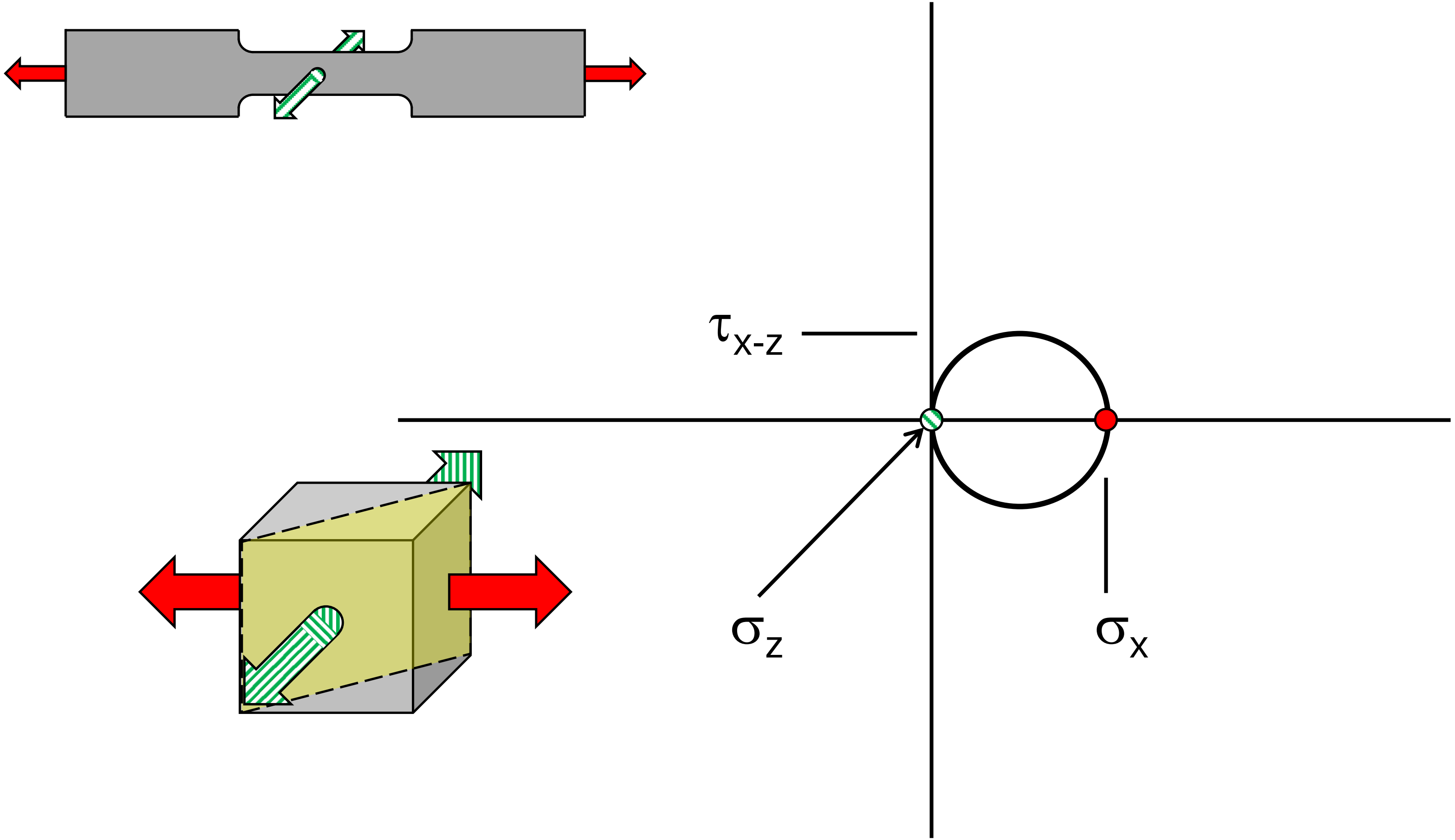


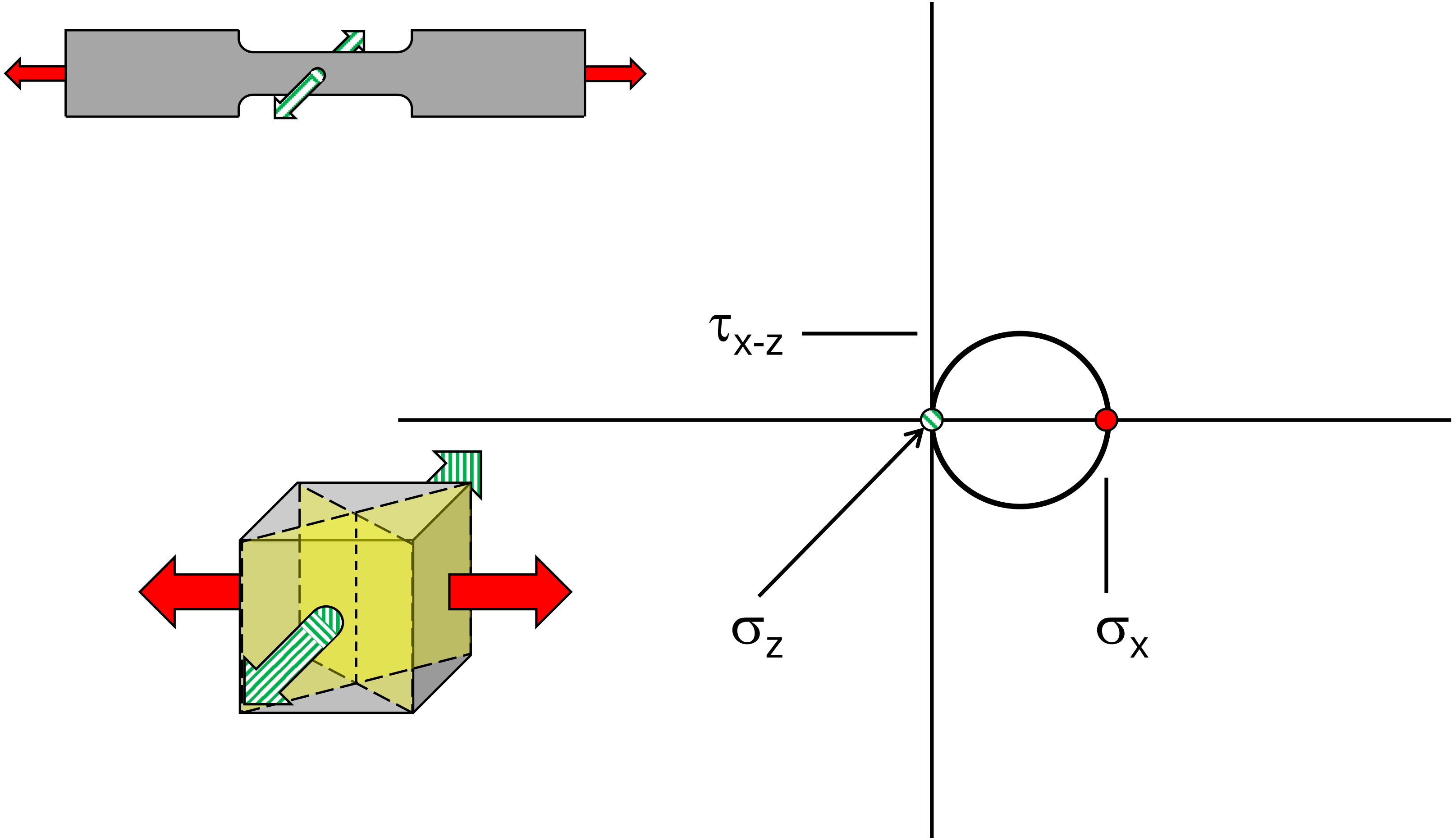
Smooth specimen
(no notches)



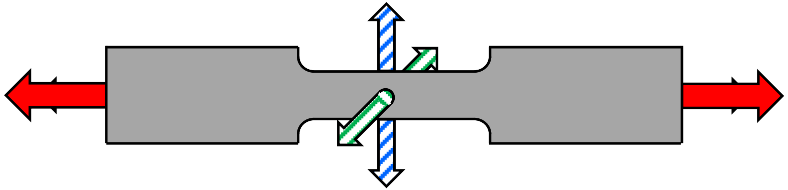


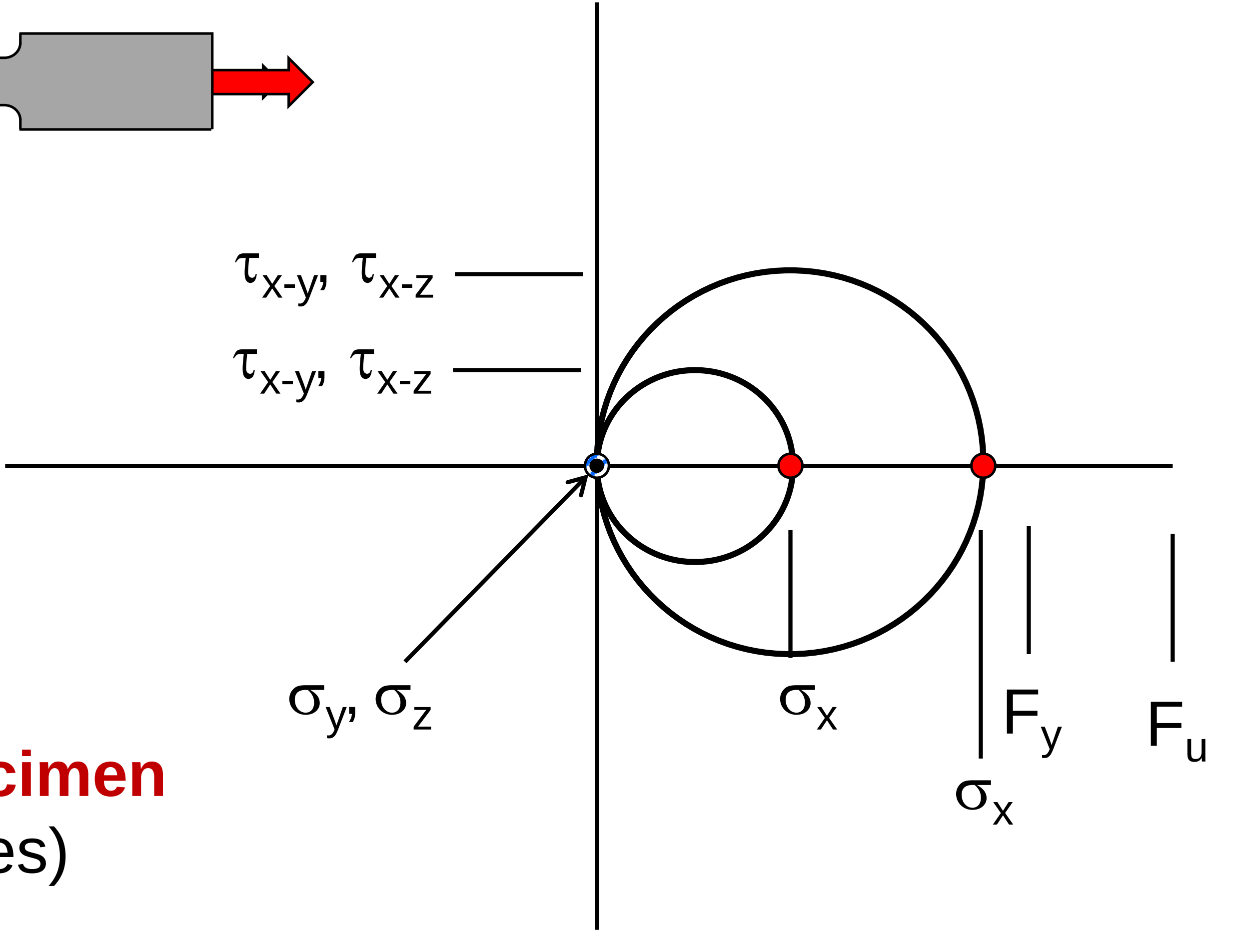

$\tau_{x\text{-}z}$
σ_z
σ_x


$\tau_{x\text{-}z}$
σ_z
σ_x



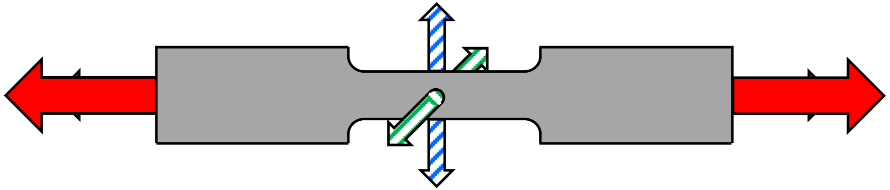
τx-z
σz
σx

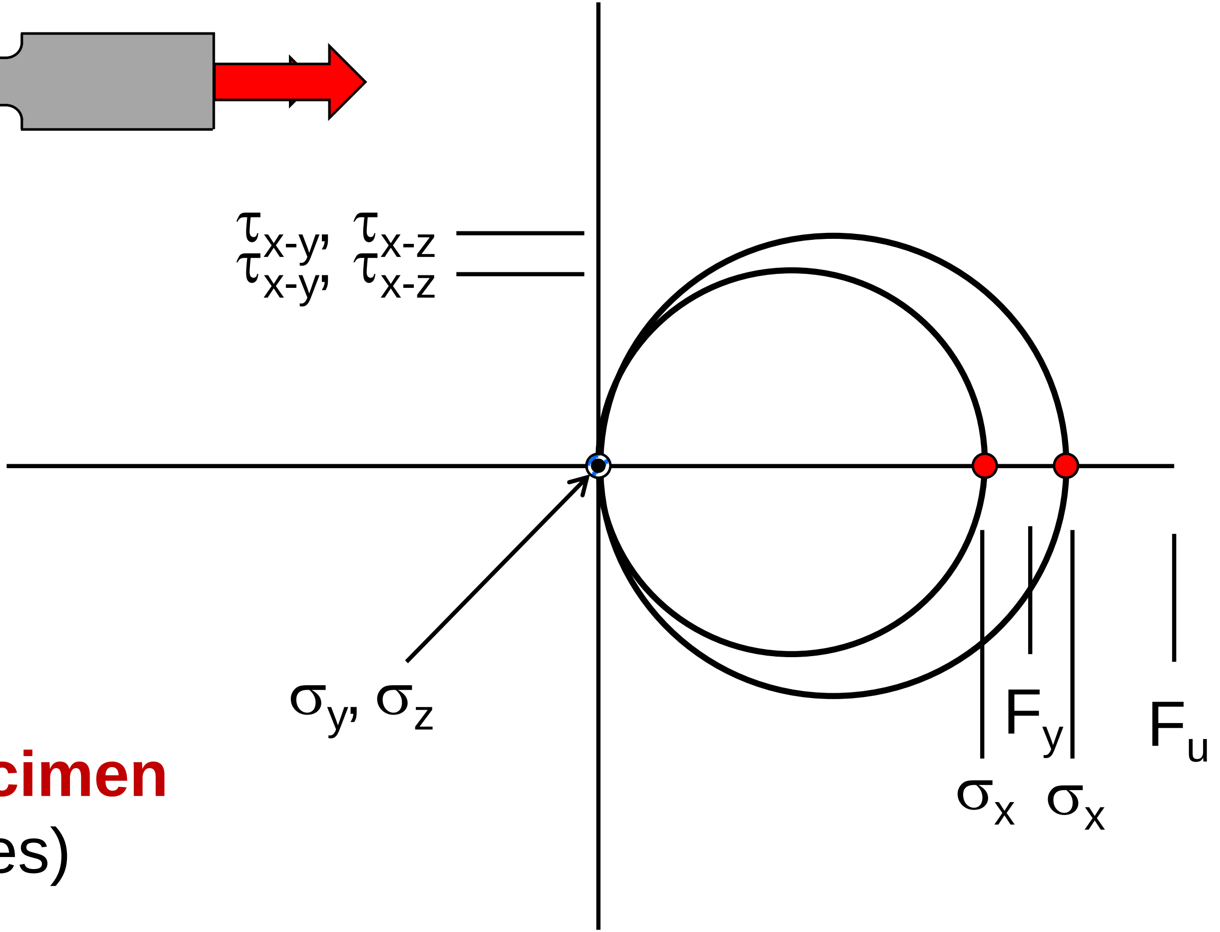




Smooth specimen

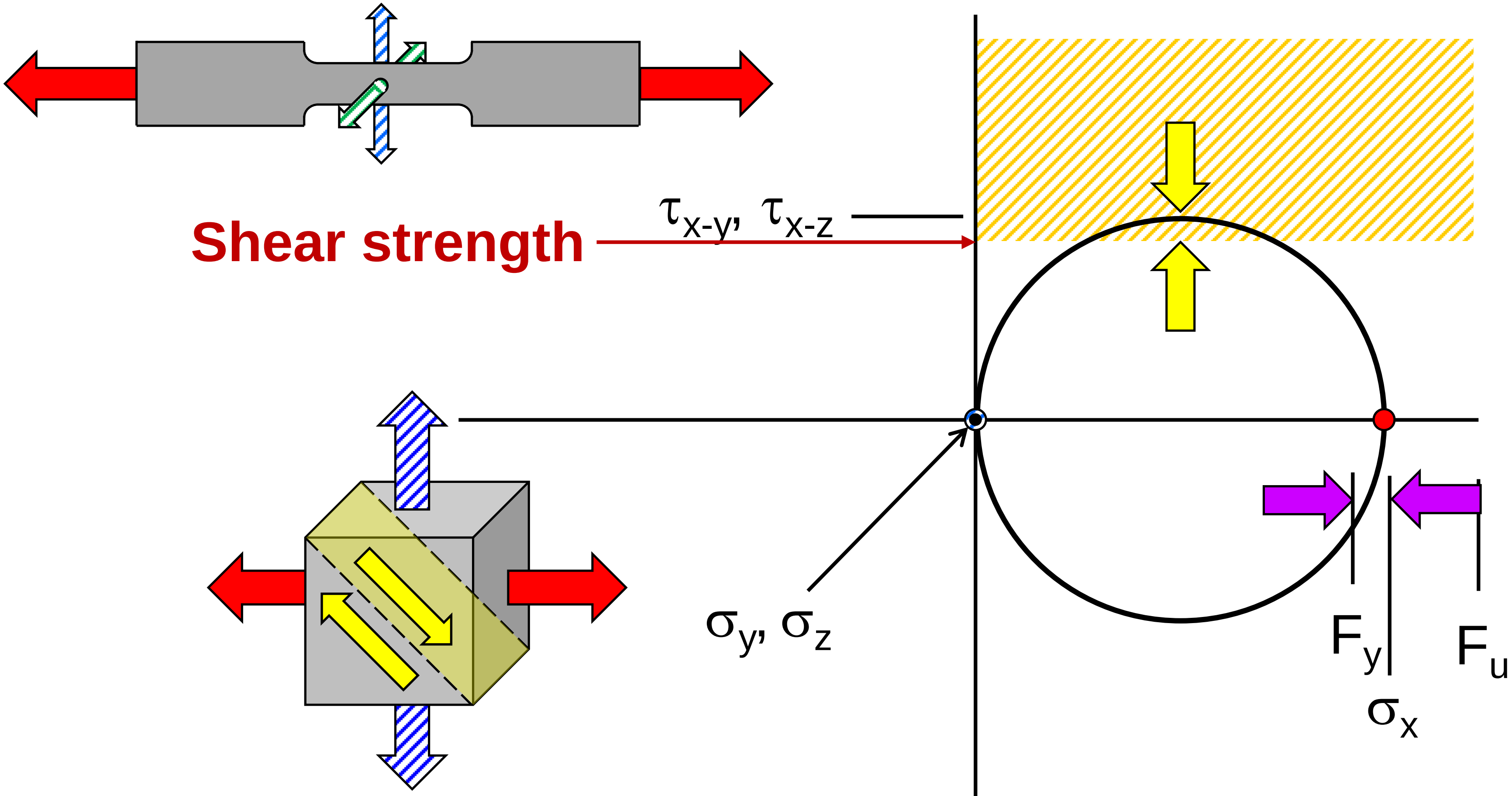
(no notches)

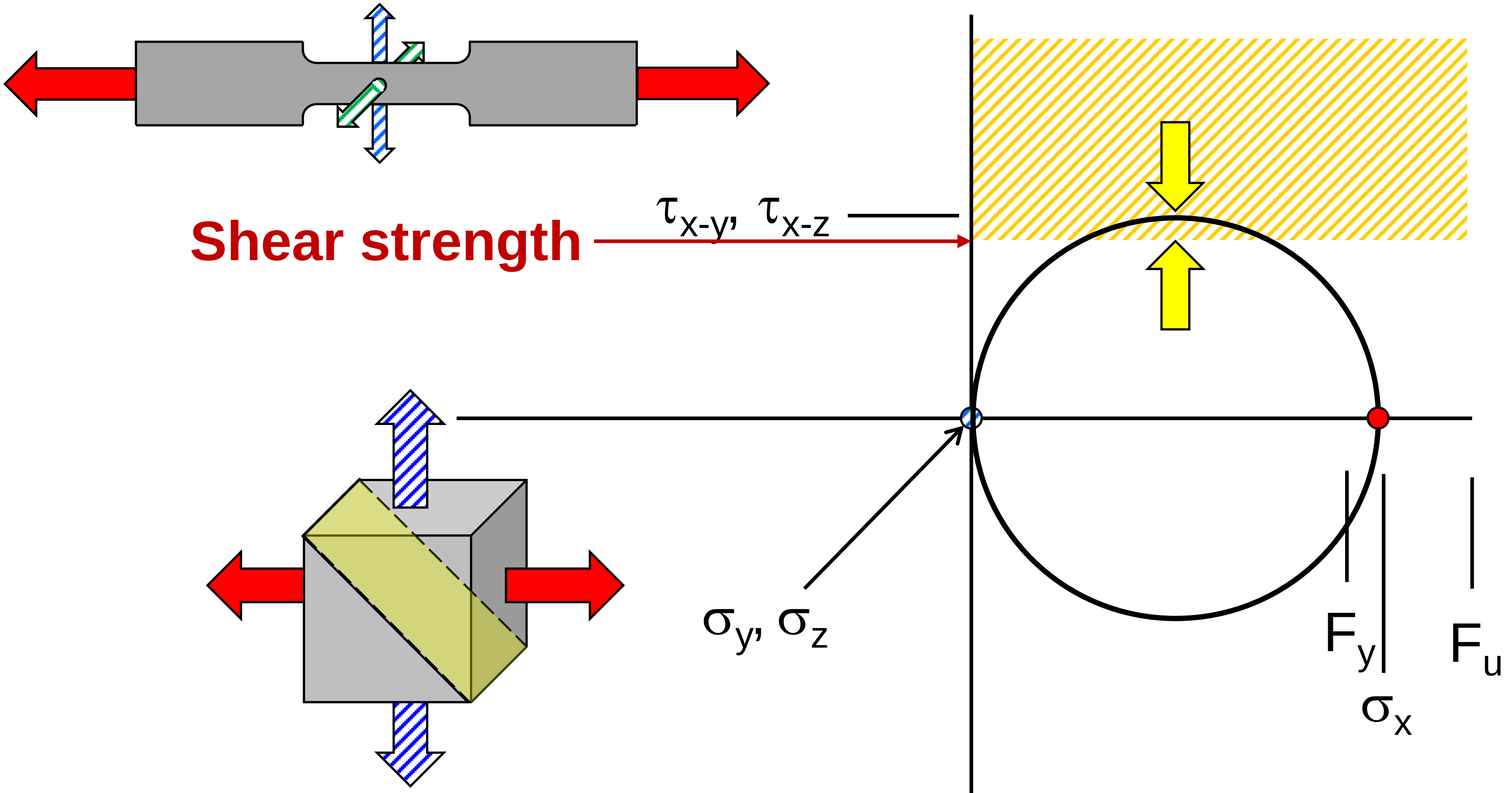




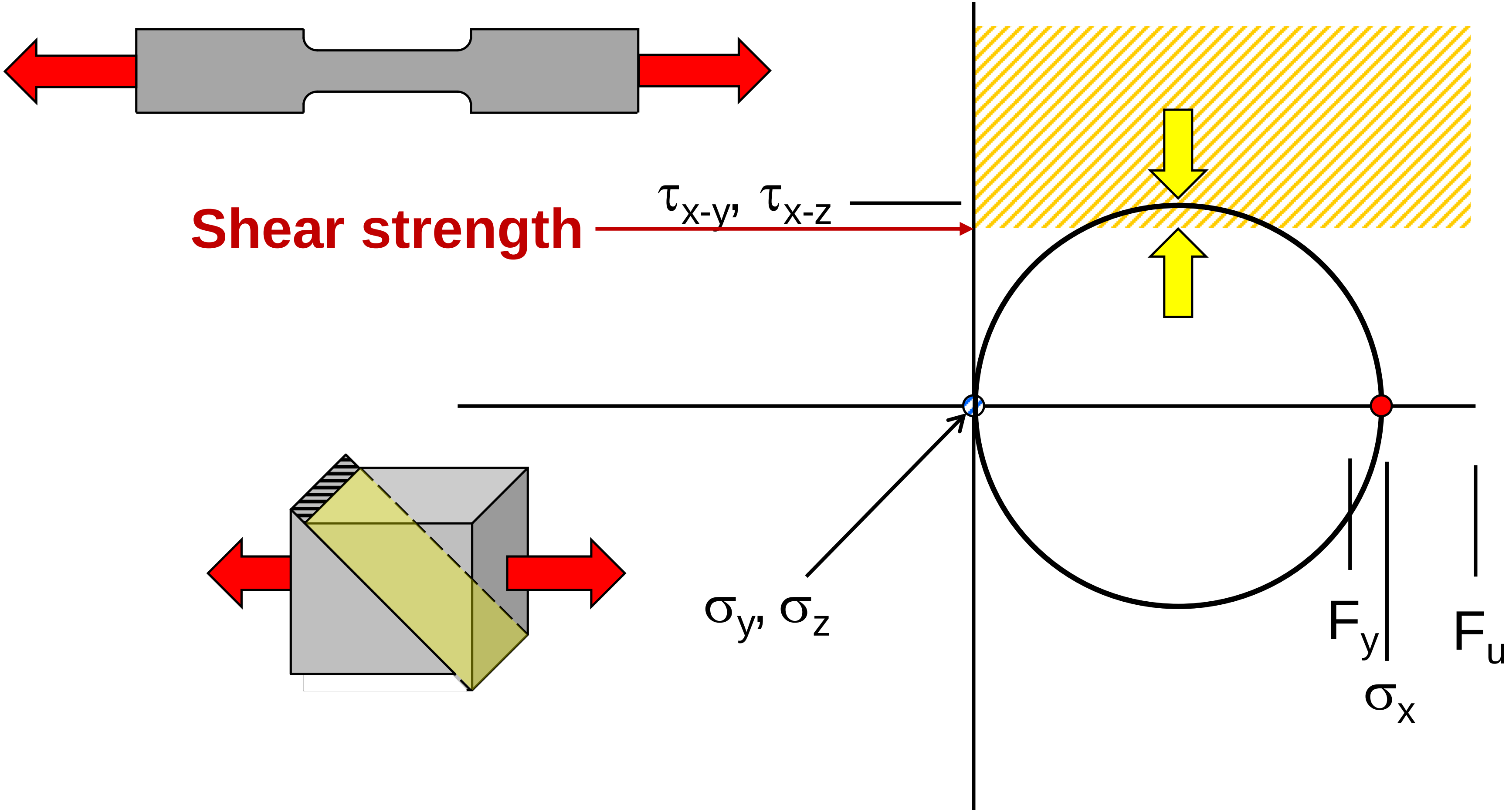
Smooth specimen

(no notches)


τx-y, τx-z
Shear strength
σy, σz
Fy
Fu
σx



Shear strength
τx-y, τx-z
σy, σz
Fy
Fu
σx



Shear strength
τx-y, τx-z
σy, σz
Fy
Fu
σx

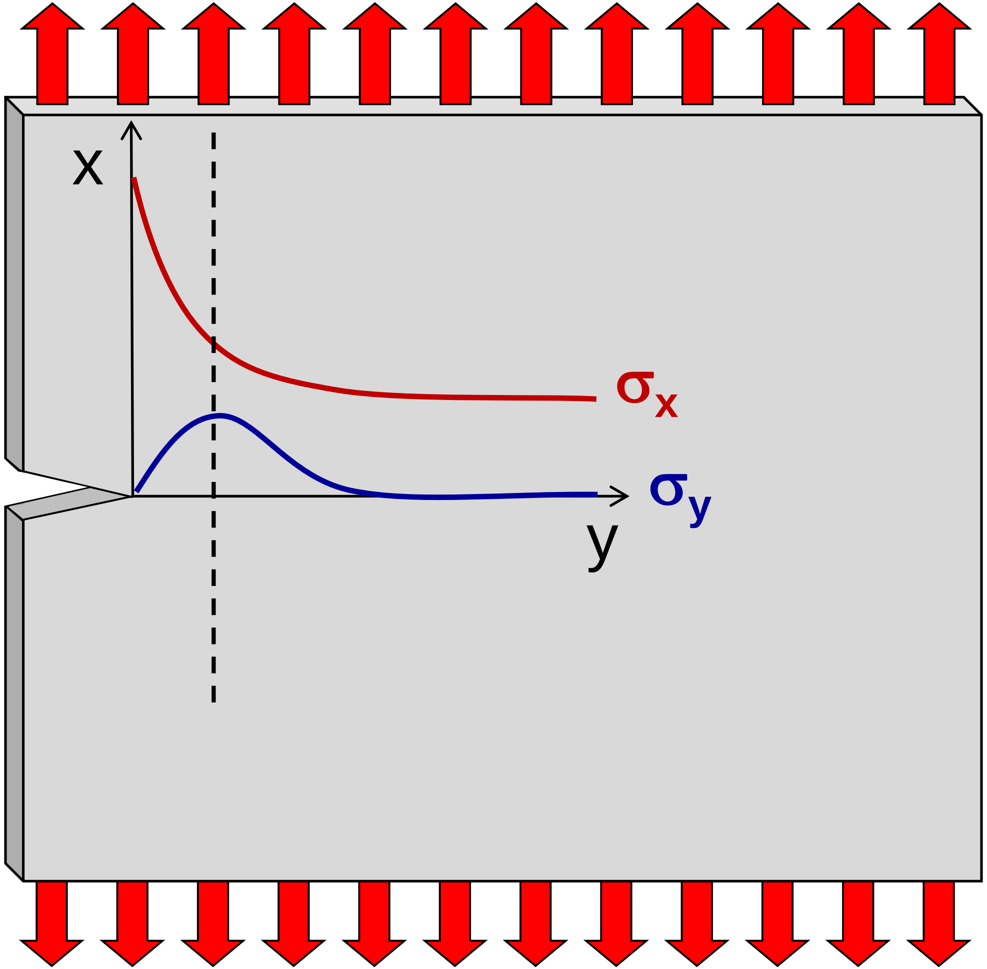
Plane-stress

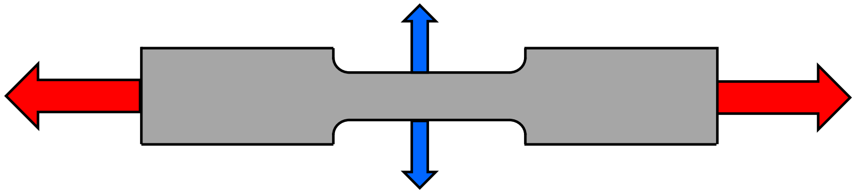
Thin Plate

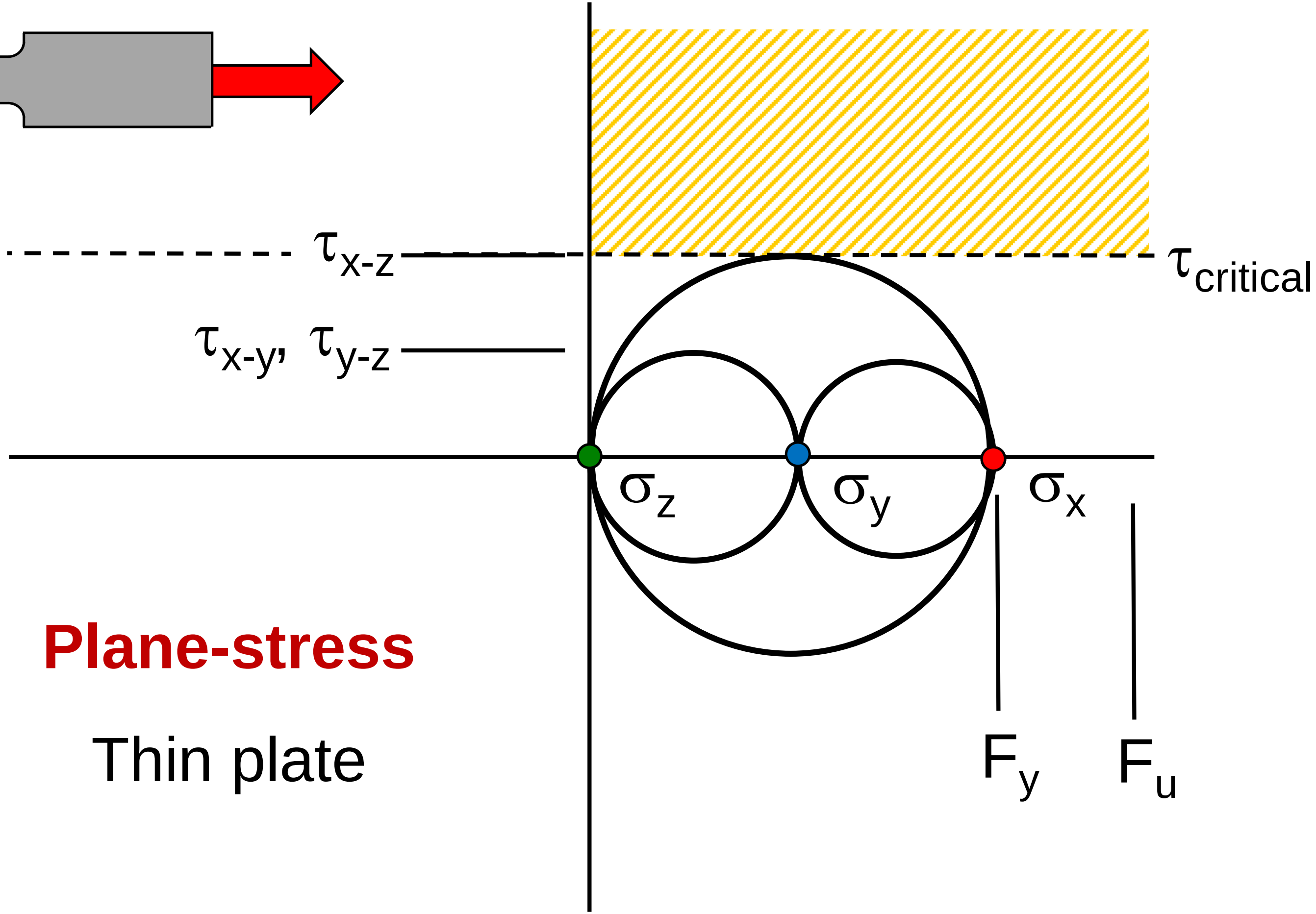
$\sigma_x = F_y$

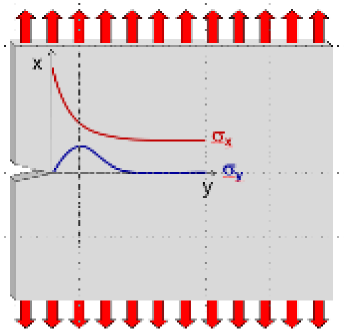
$\sigma_y = F_y/2$

$\sigma_z = 0$



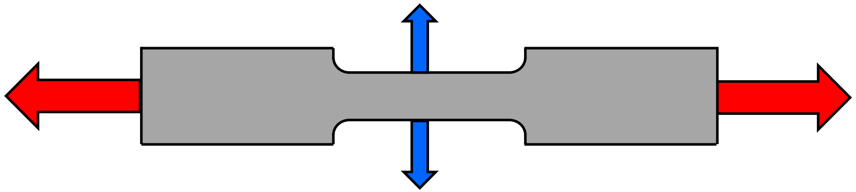


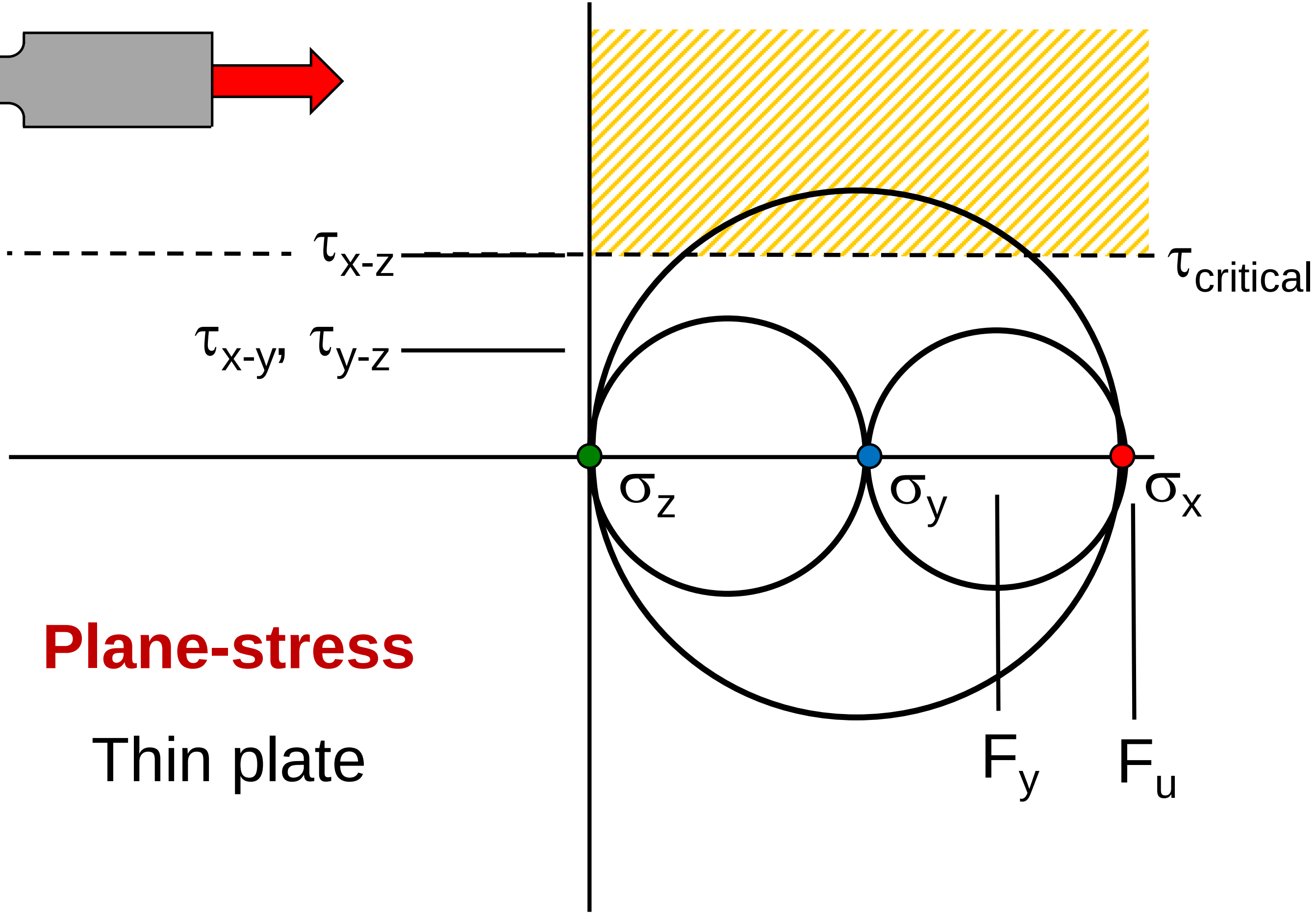


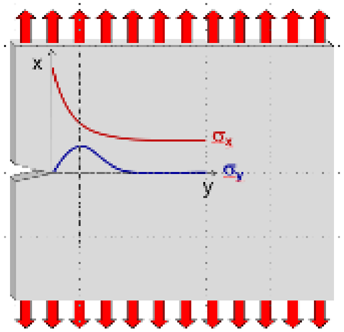


Plane-stress

Thin plate







Plane-stress

Thin plate

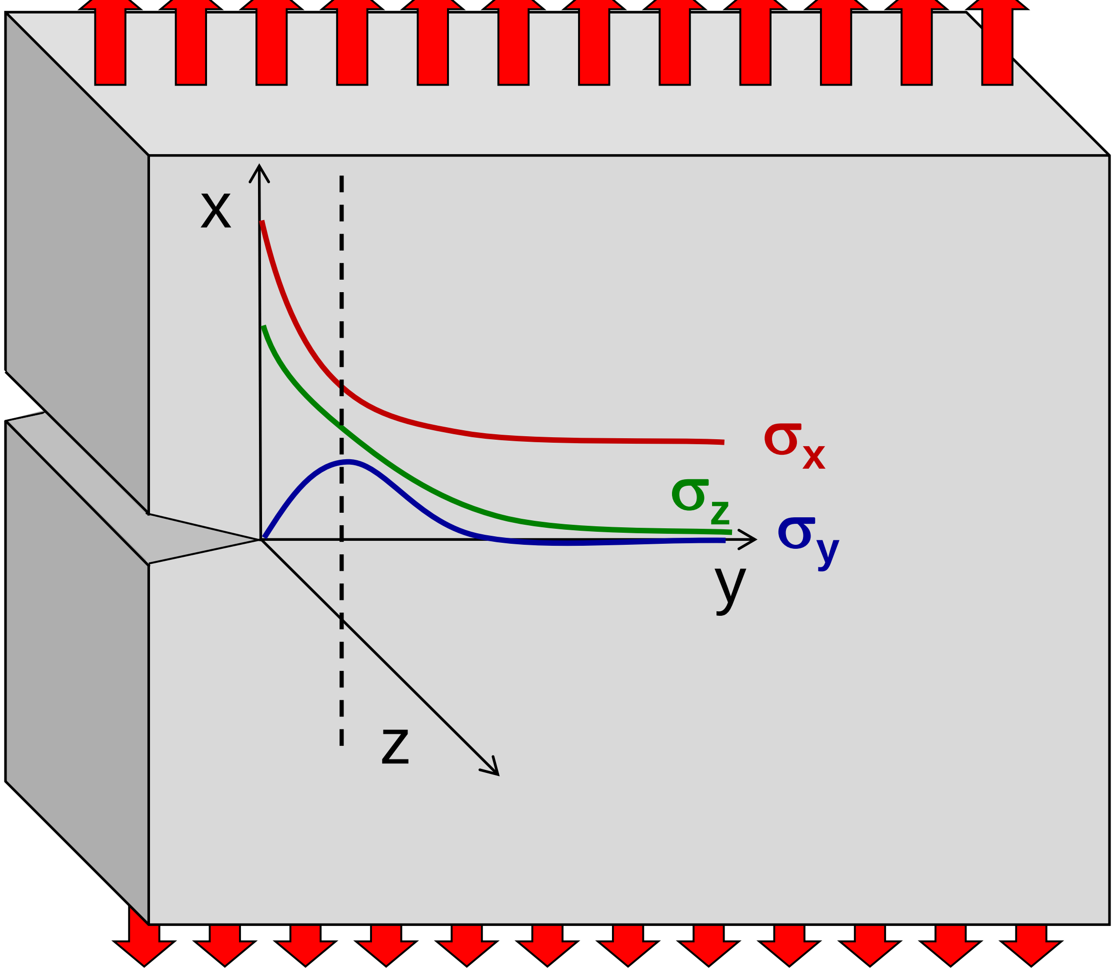
Plane-strain

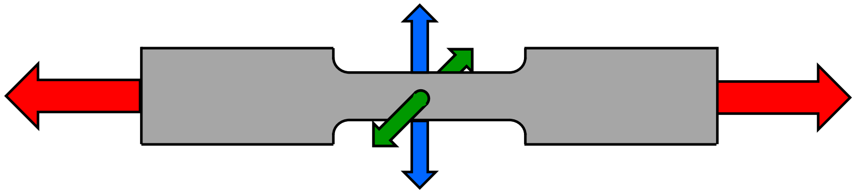
Thick Plate

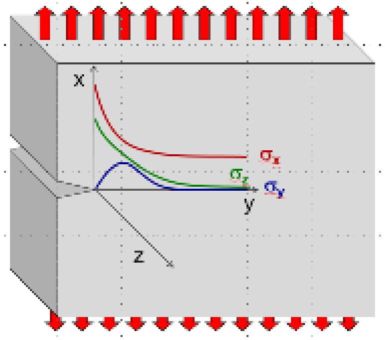
$\sigma_x = F_y$

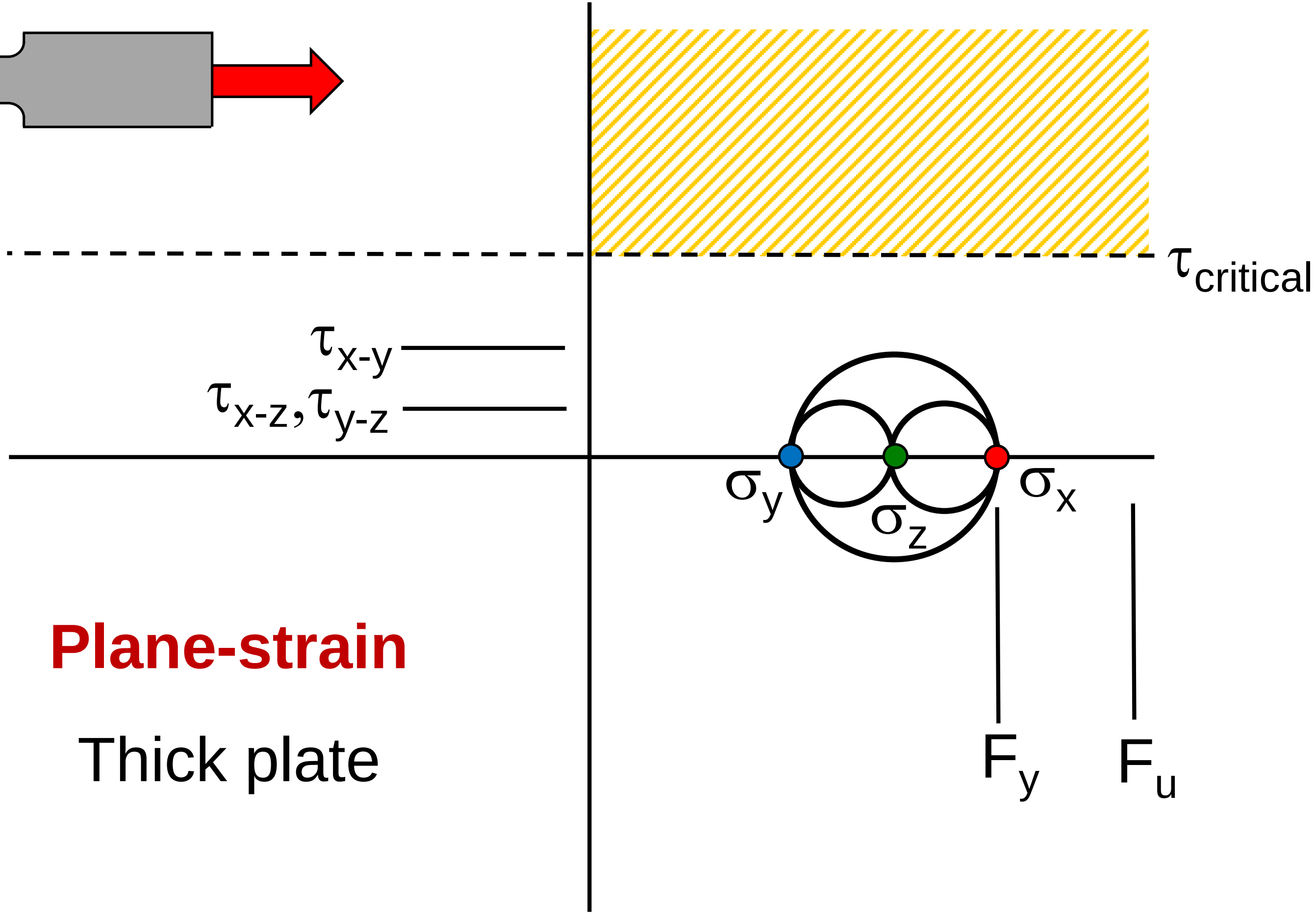
$\sigma_y = F_y/2$

$\sigma_z = 3F_y/4$



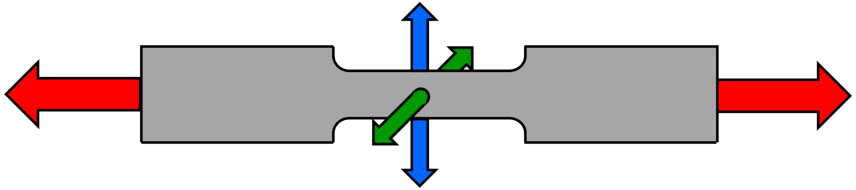


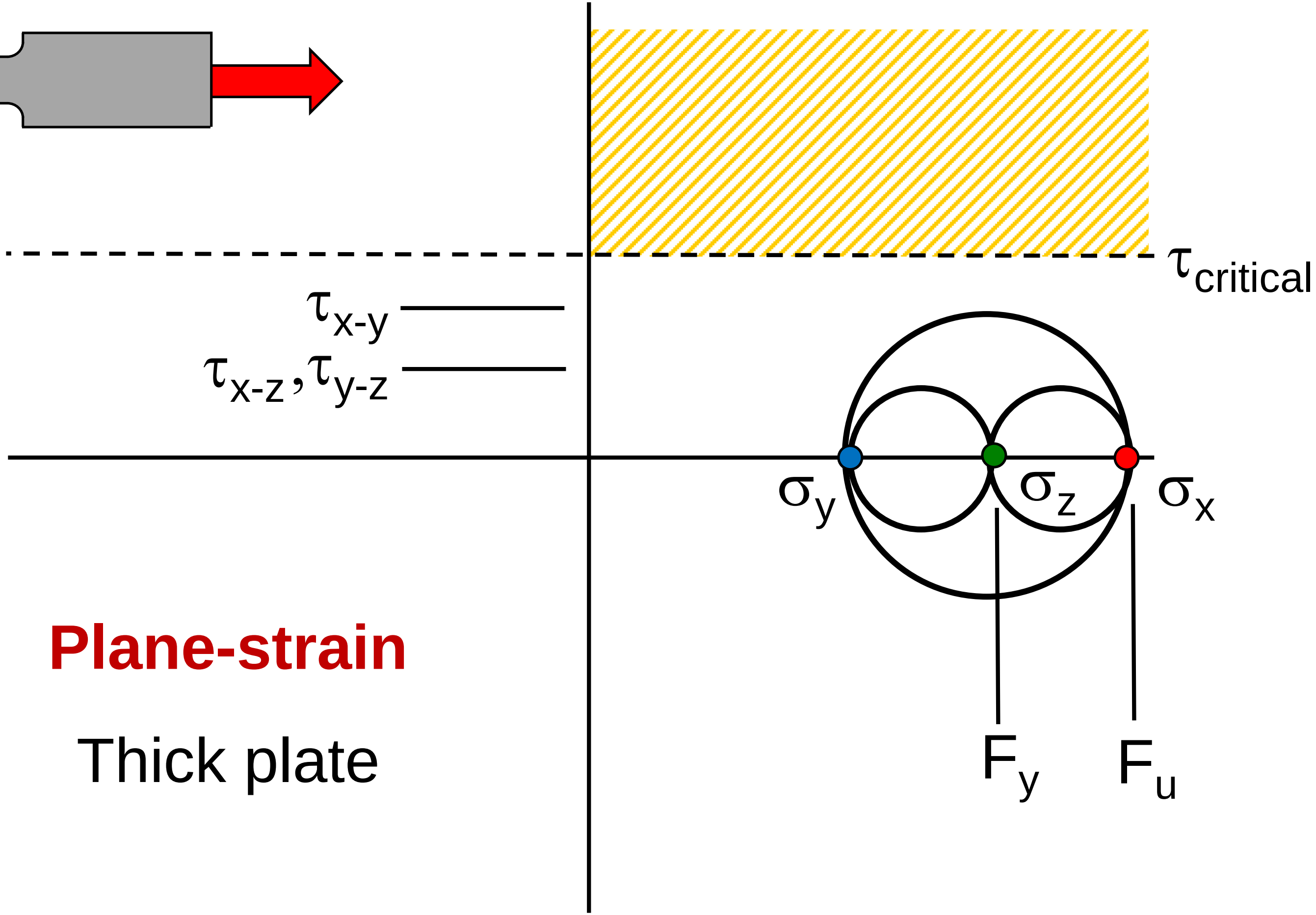


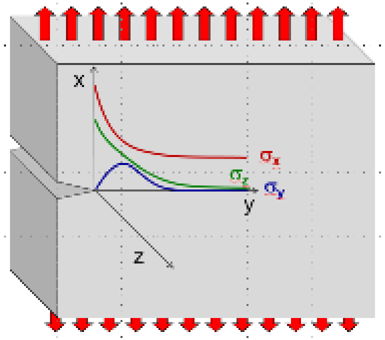


Plane-strain

Thick plate







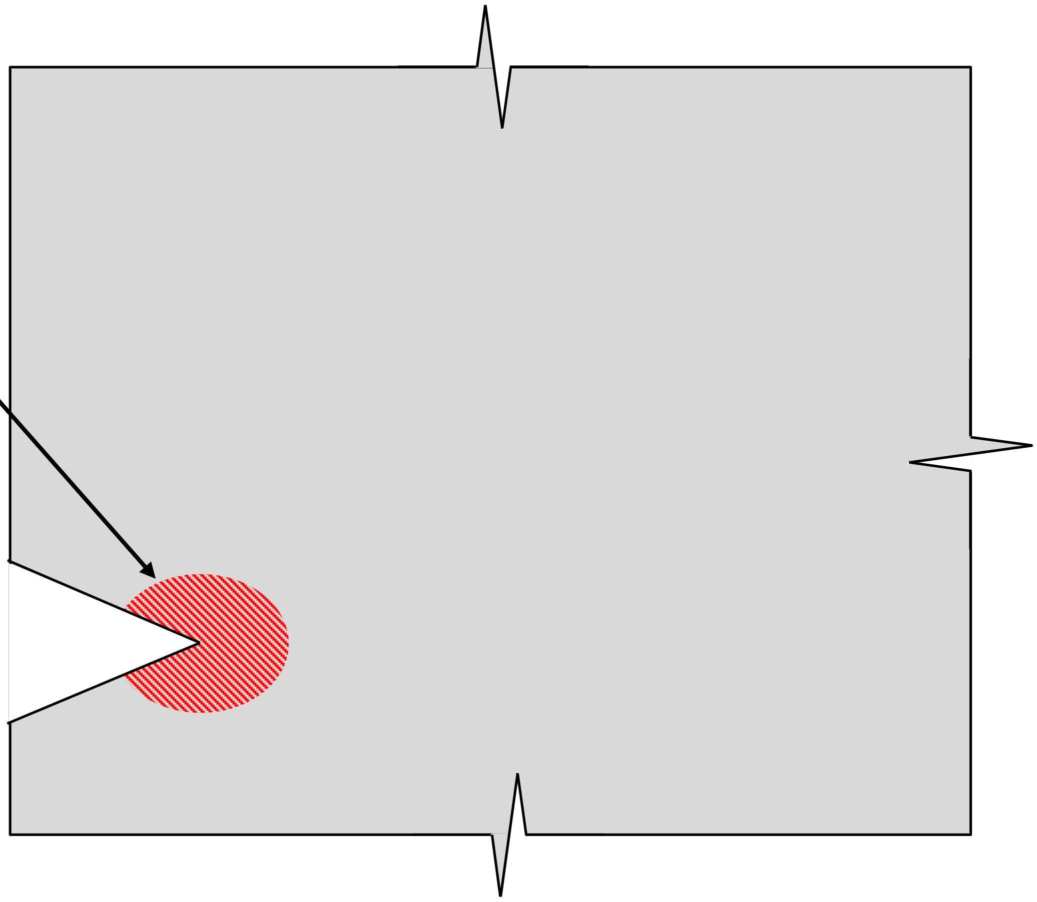
Plane-strain

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

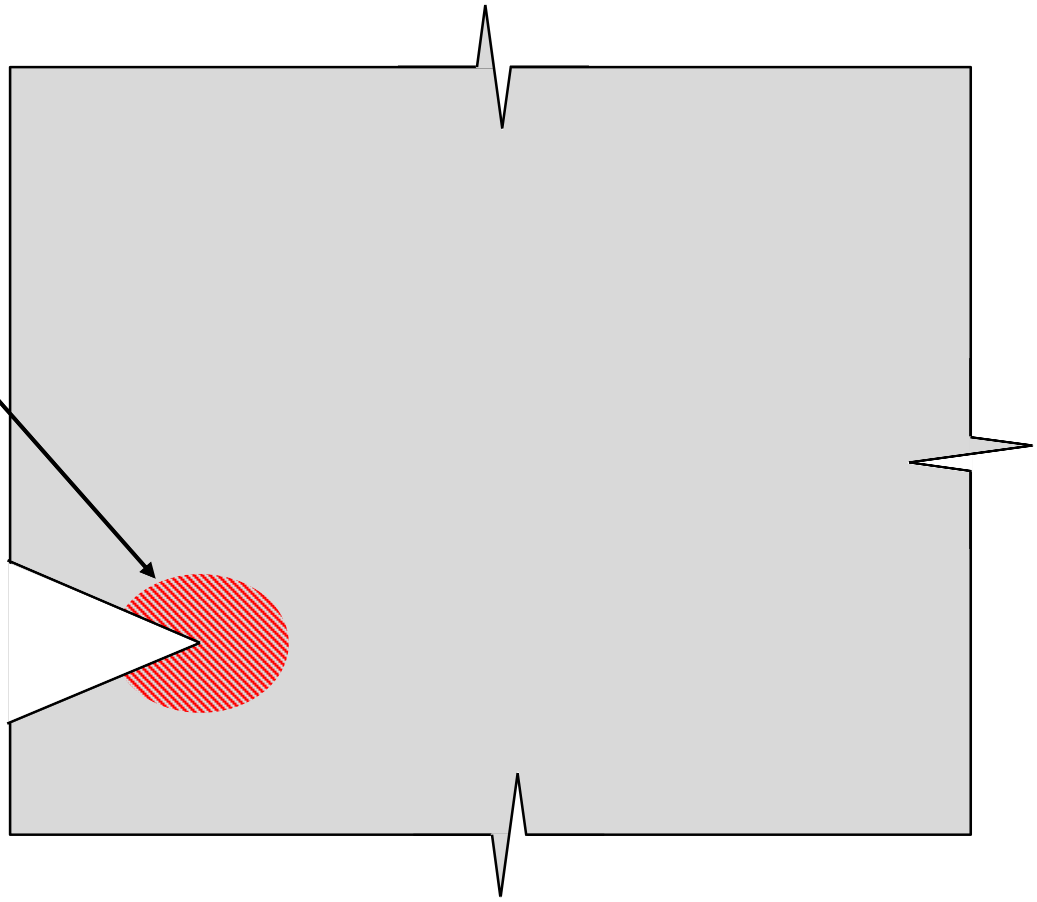


- inelastic zone
- yielded zone
- strain-hardened zone

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
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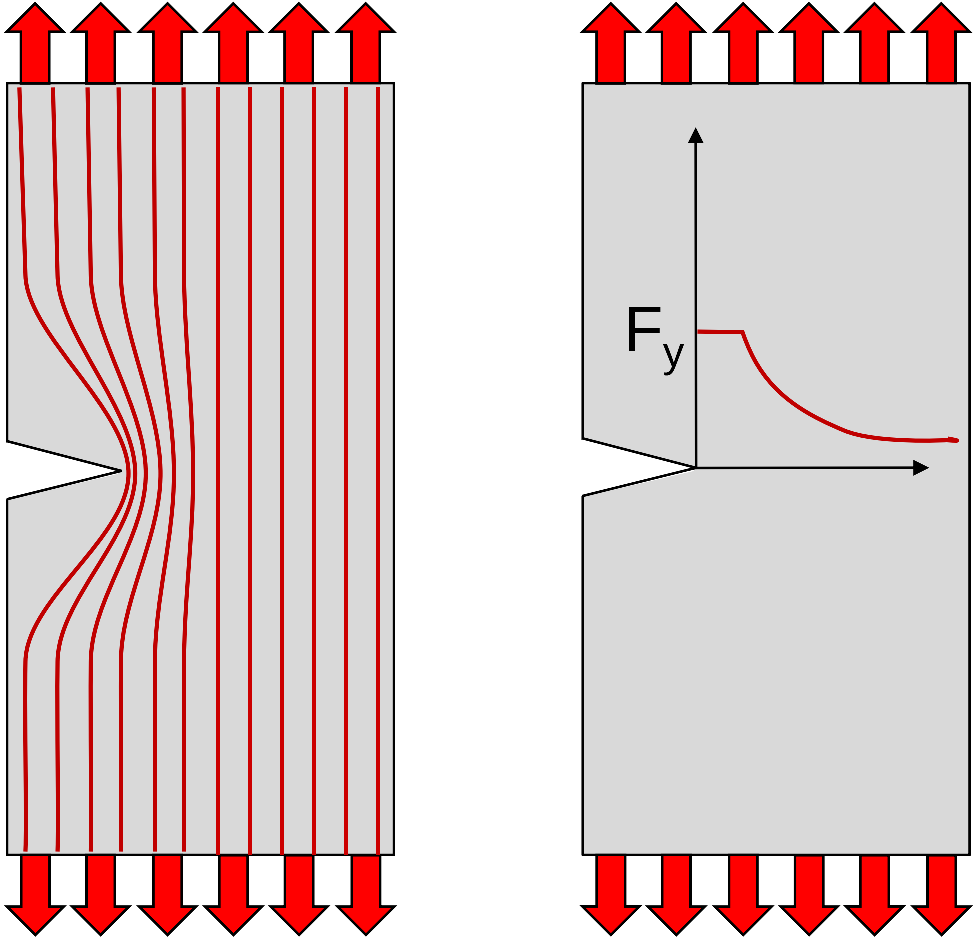
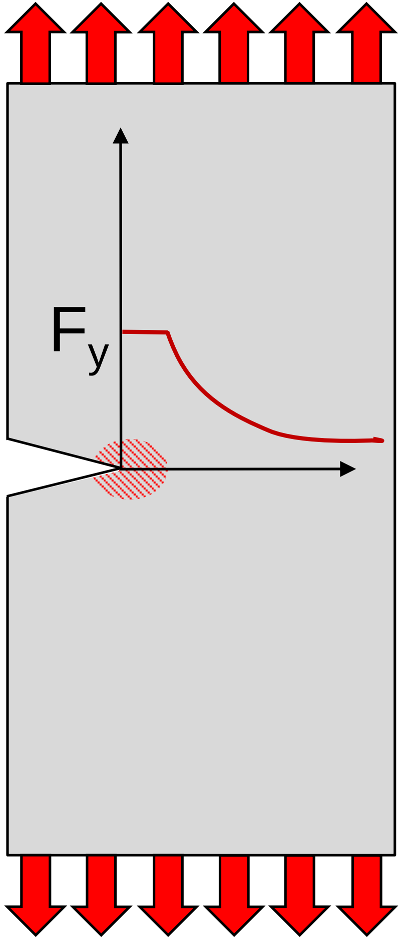
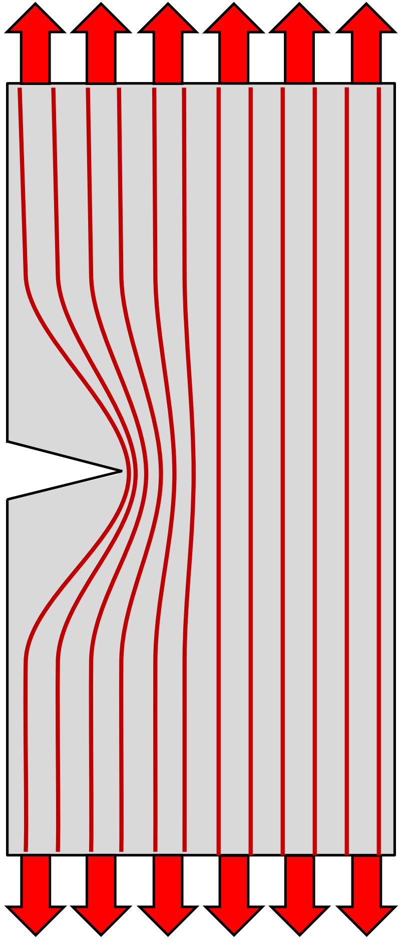
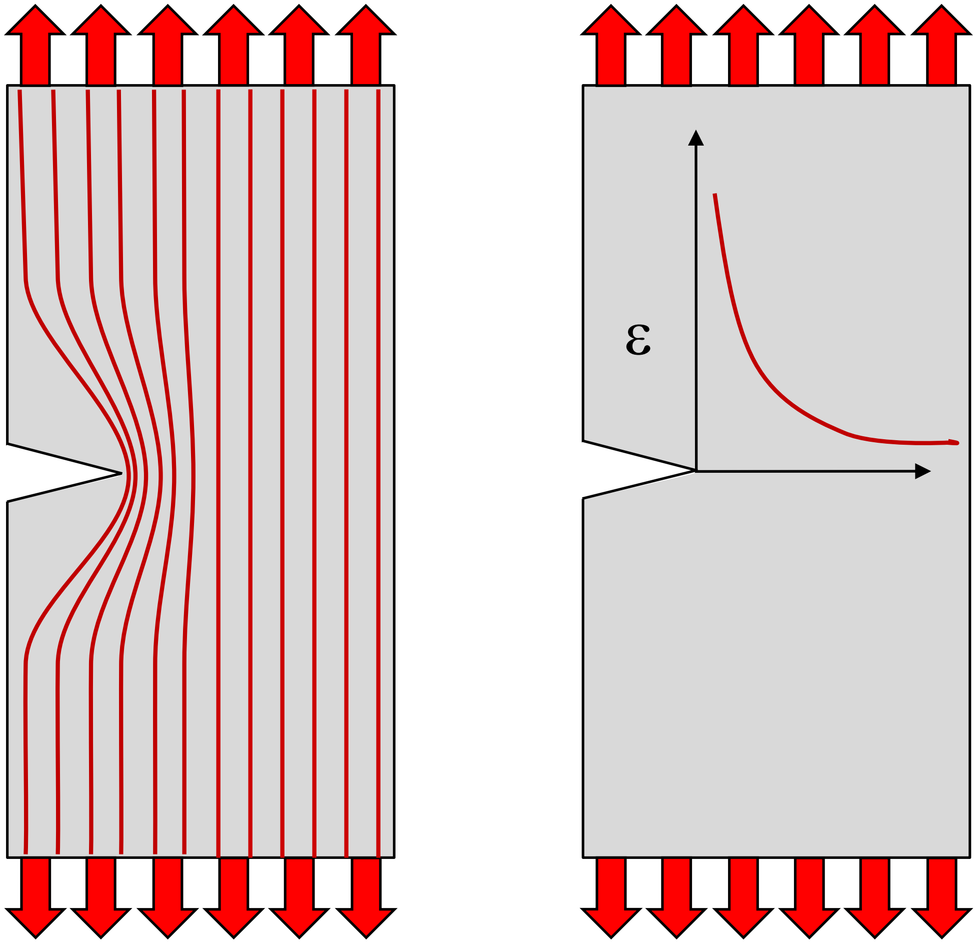


Stress

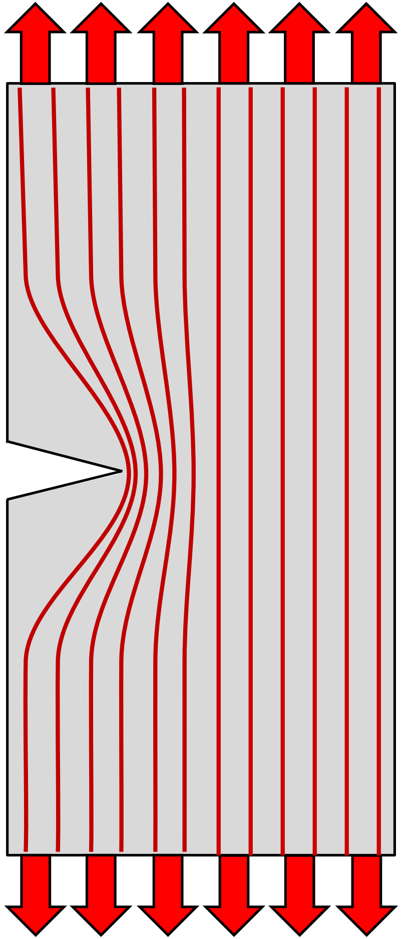


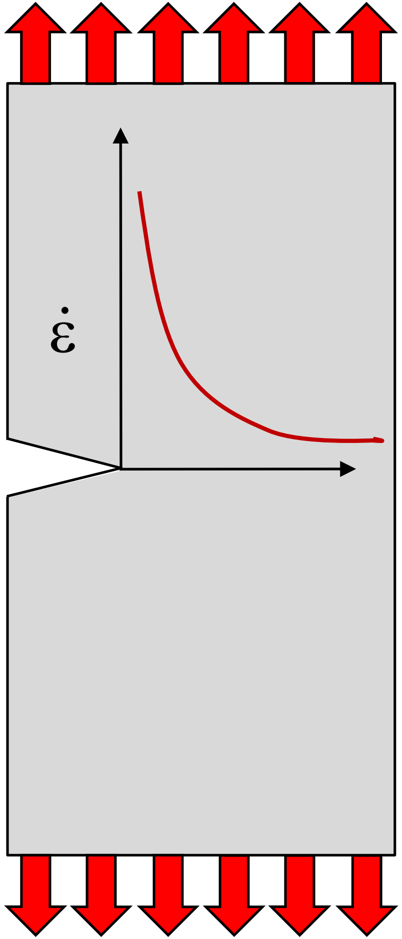


Stress



Strain





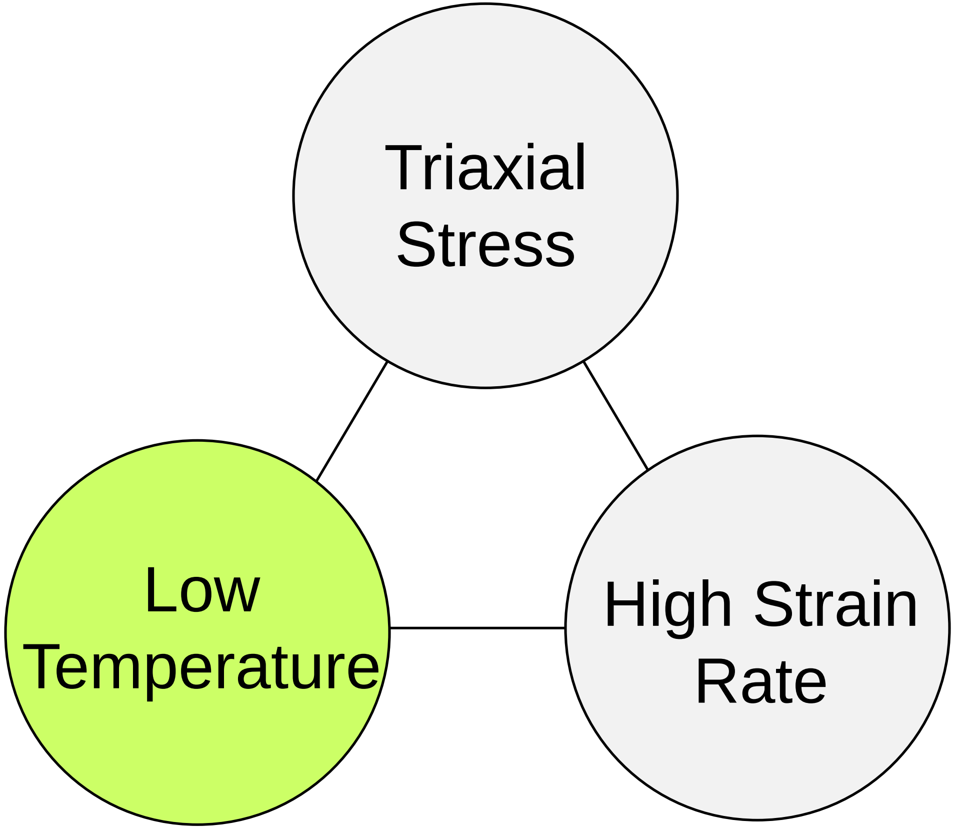
Strain Rate

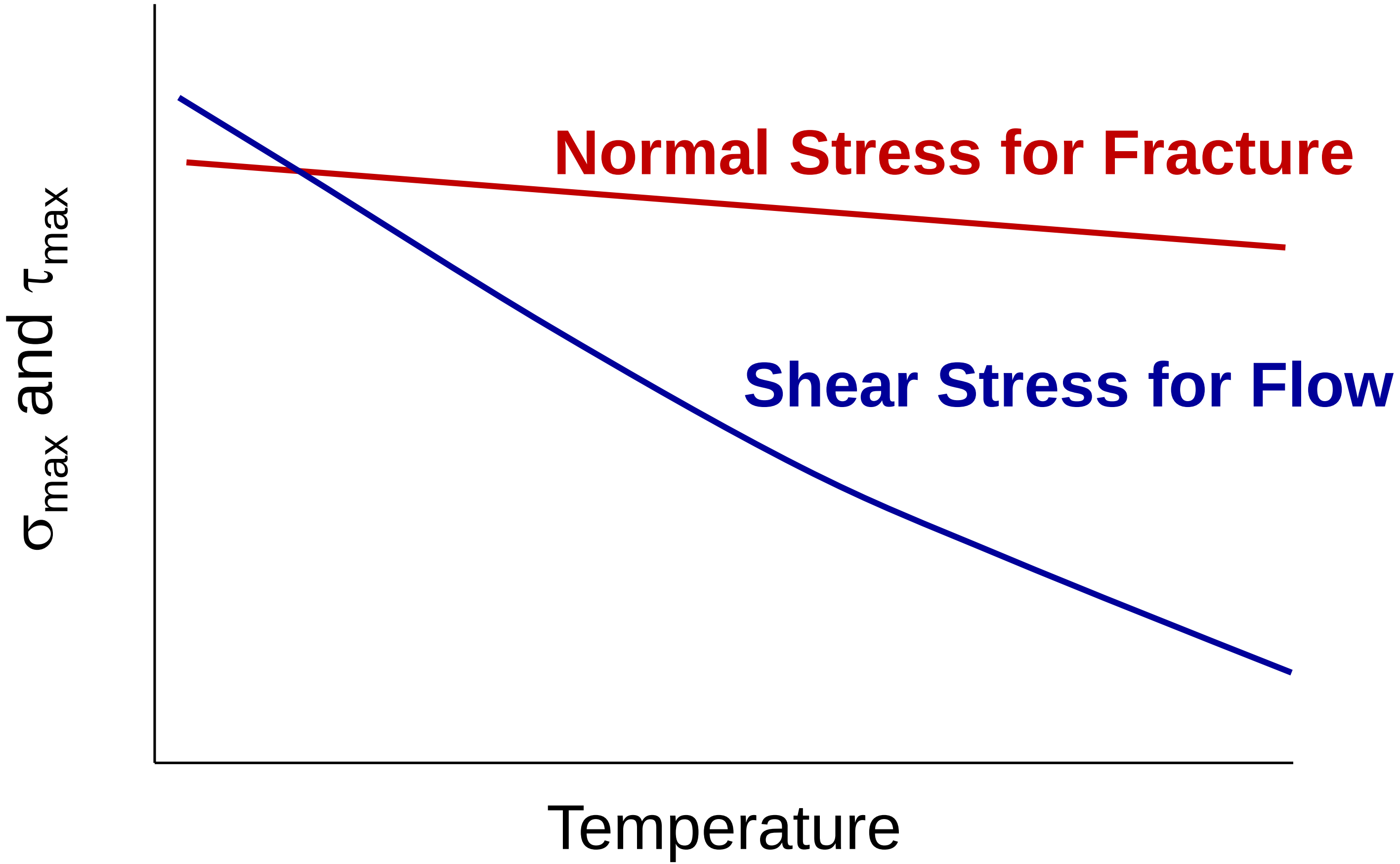
$\dot{\varepsilon}_p = 10x\ \dot{\varepsilon}_e$

$\dot{\varepsilon}_e = 1$

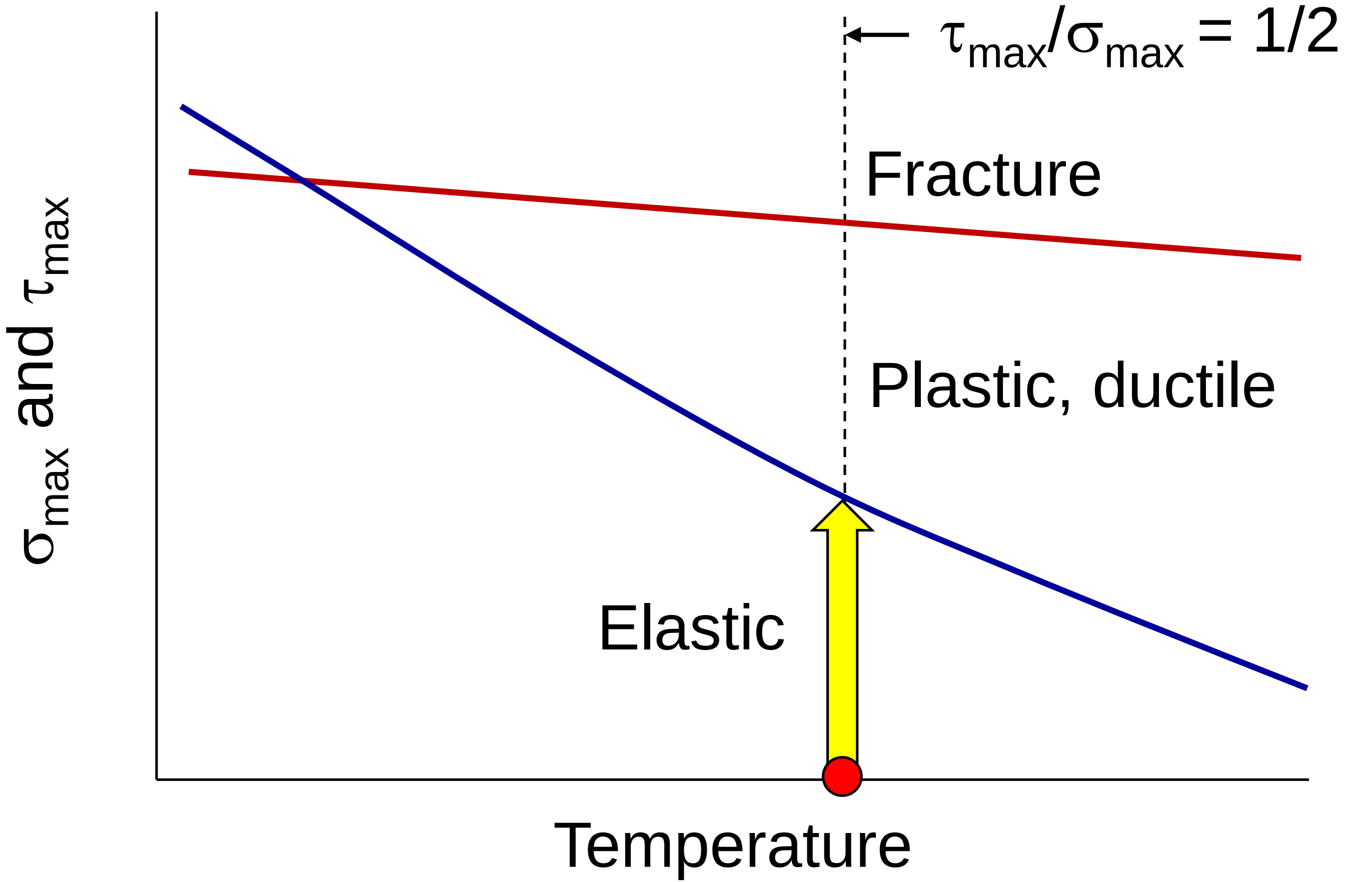
Numerical values are illustrative only

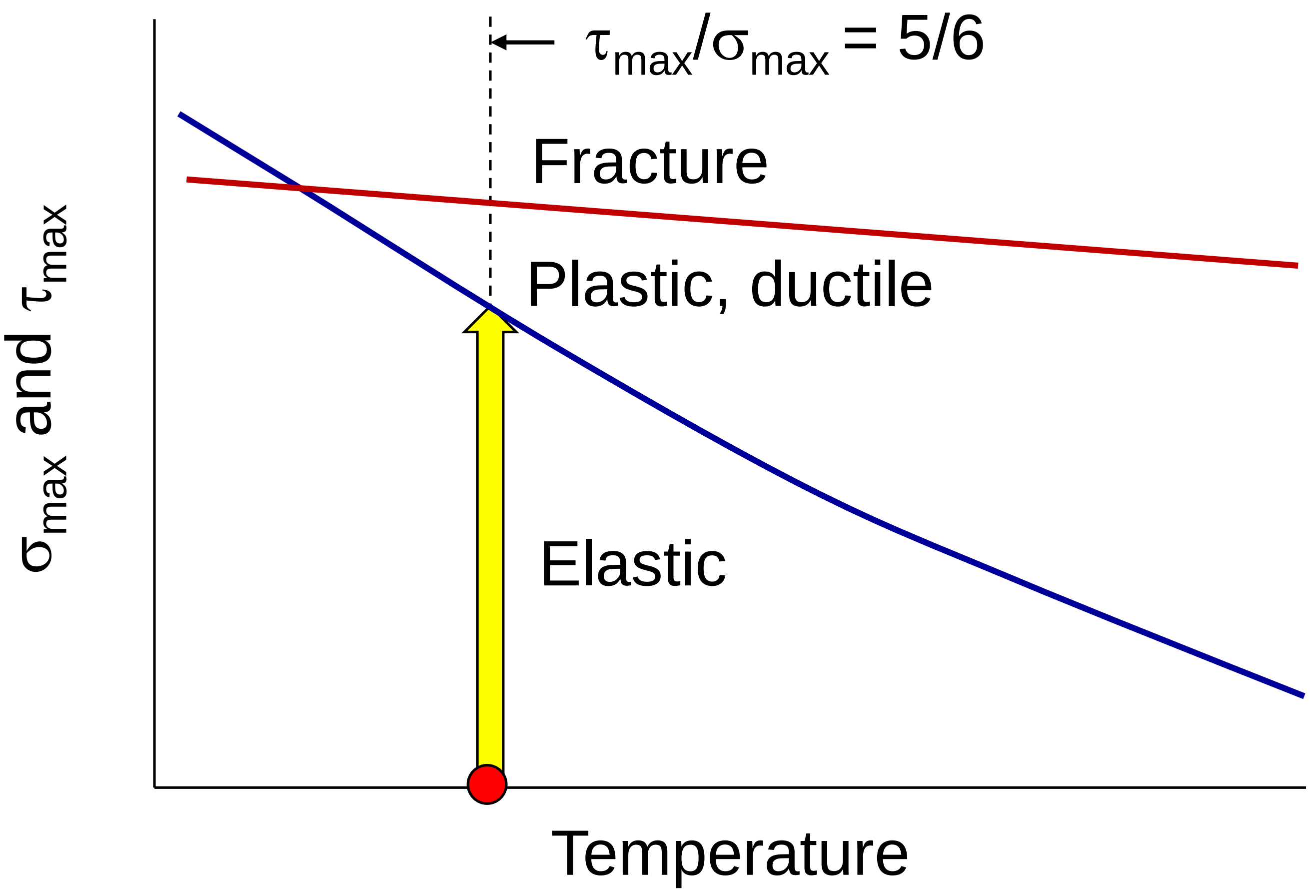
The Unholy Trinity



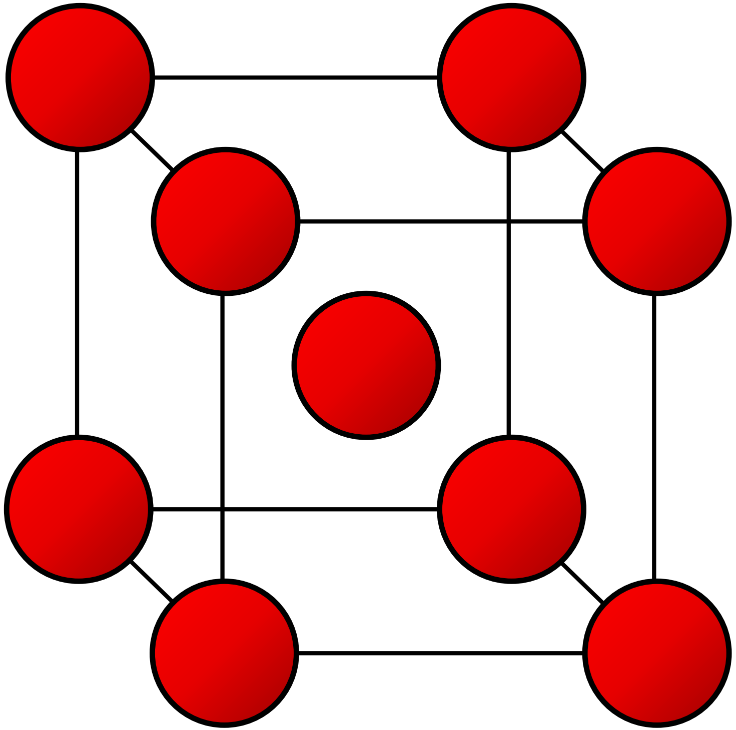


Adapted from Gensamer


$\tau_{max}/\sigma_{max} = 1/2$
Fracture
Plastic, ductile
Elastic
σ_{max} and τ_{max}
Temperature

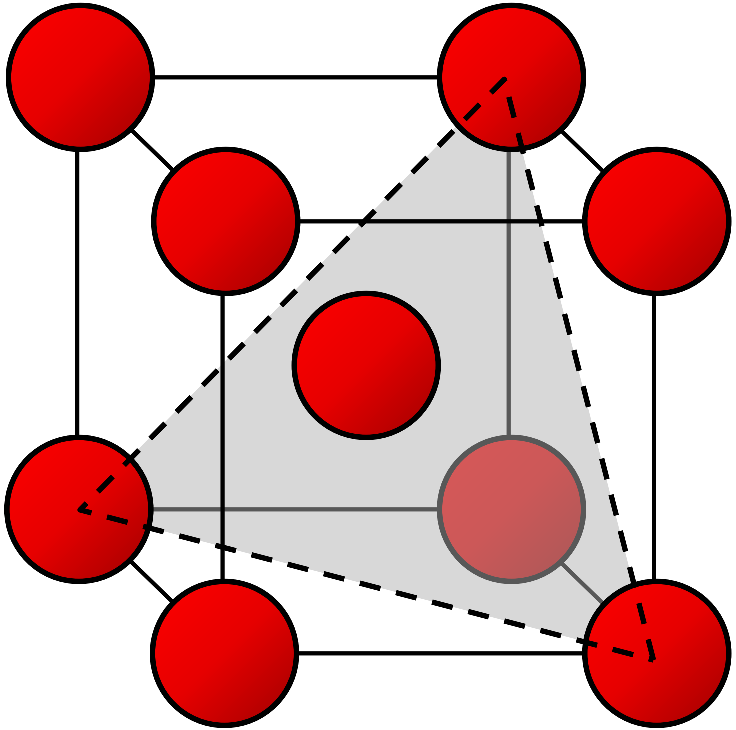

τmax/σmax = 5/6
Fracture
Plastic, ductile
Elastic
σmax and τmax
Temperature

Body Centered Cubic (BCC)

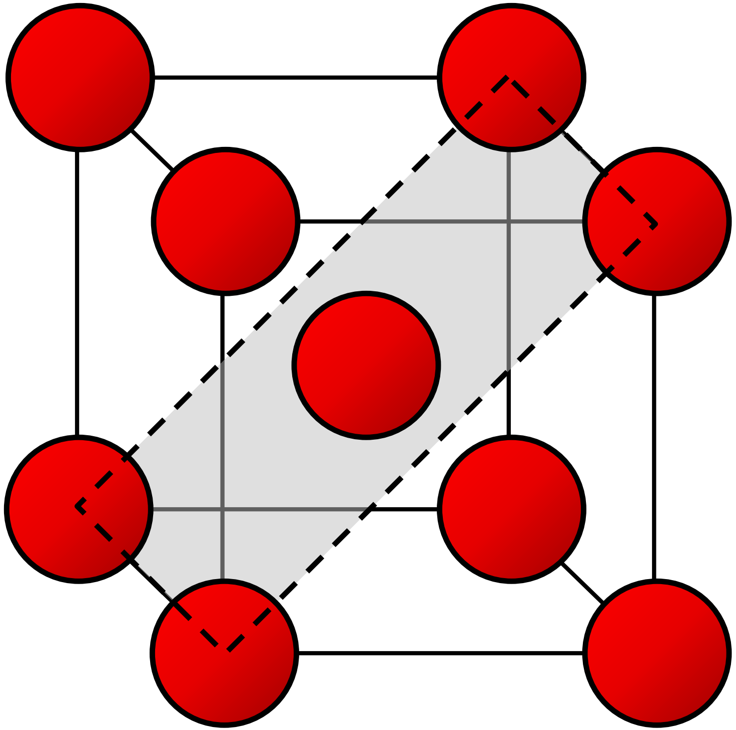


Atomic Packing

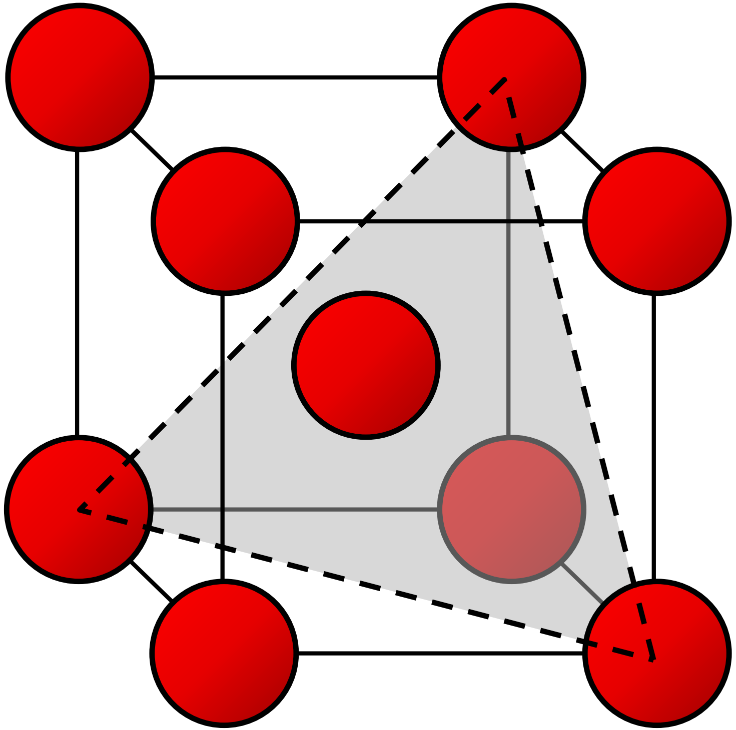
Body Centered Cubic (BCC)

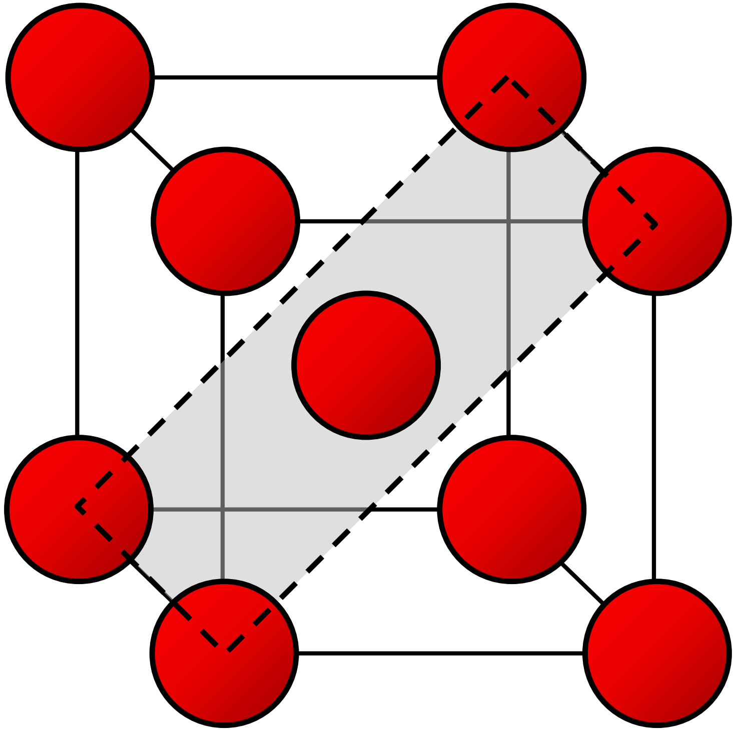


Body Centered Cubic (BCC)

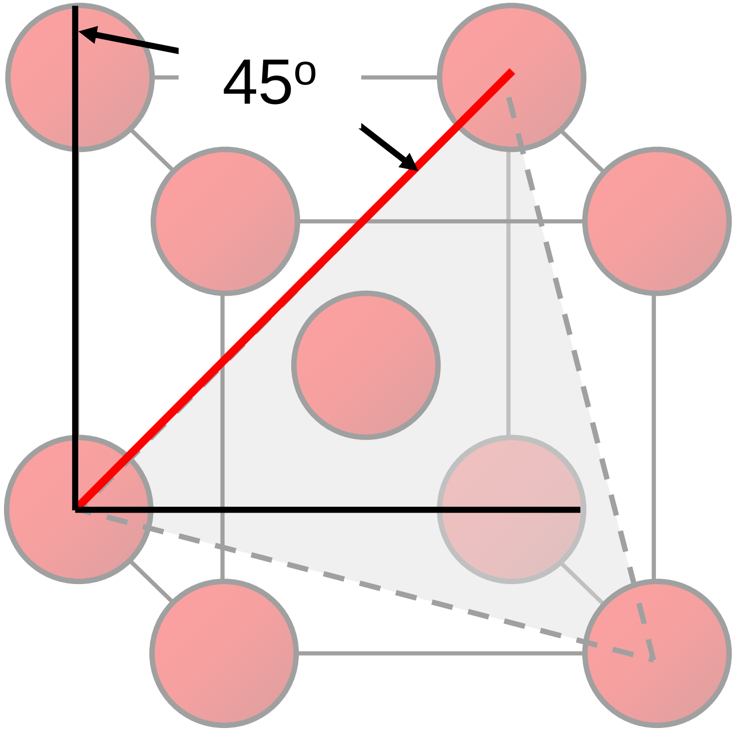


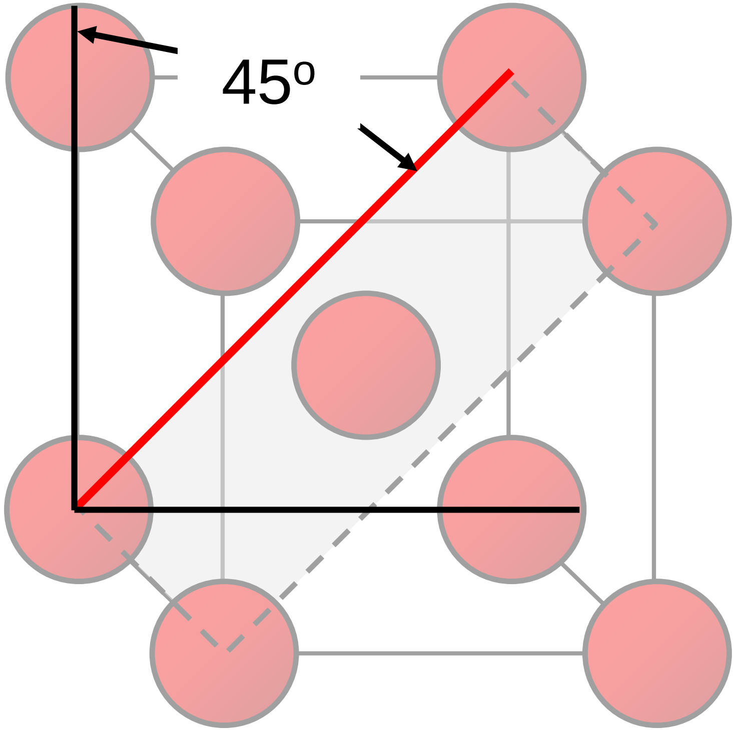
Body Centered Cubic (BCC)





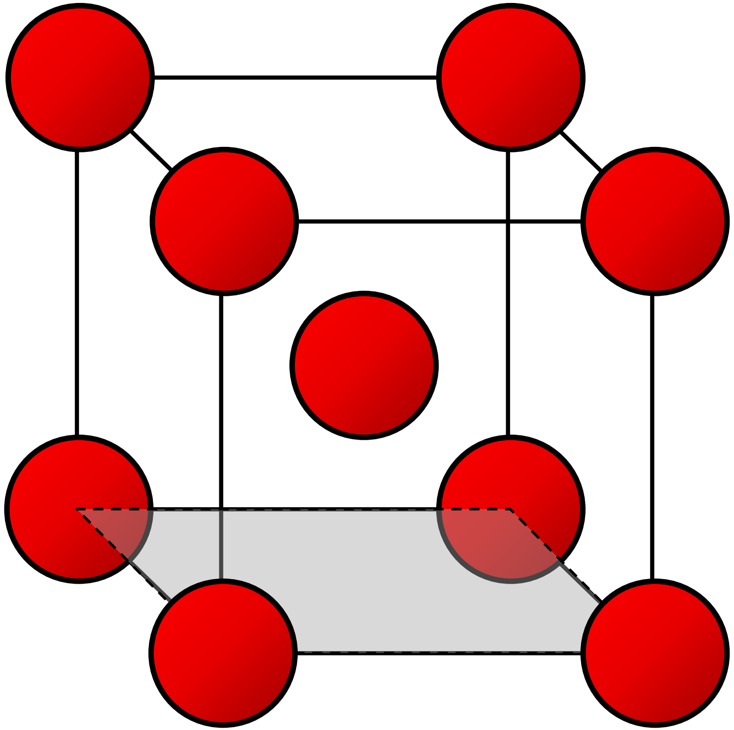
Body Centered Cubic (BCC)





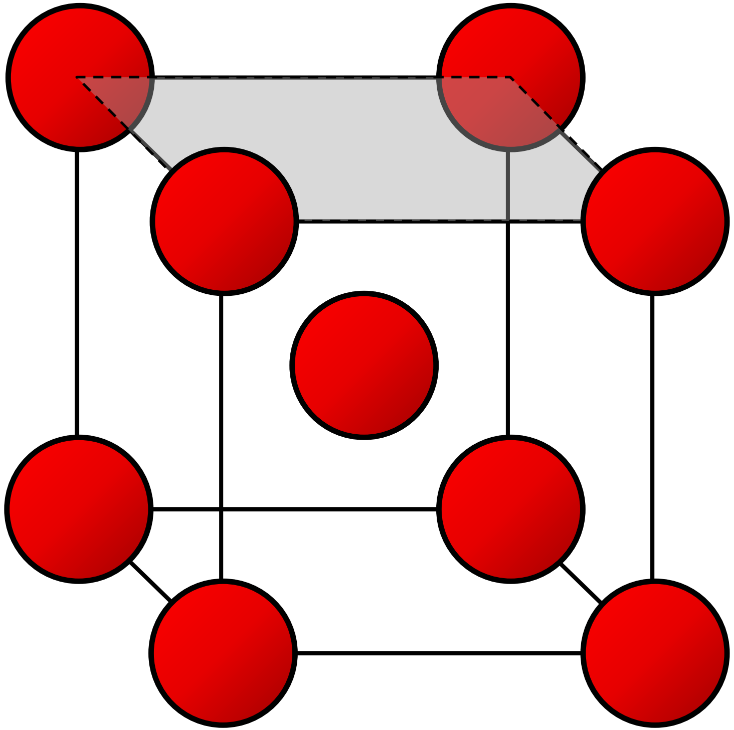
Shear Planes

Body Centered Cubic (BCC)



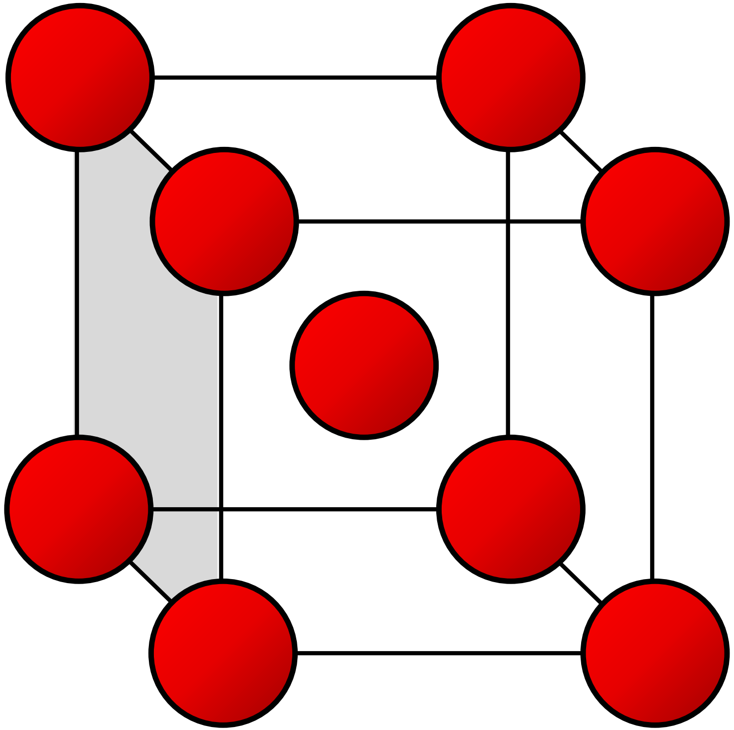
Cleavage Plane

Body Centered Cubic (BCC)



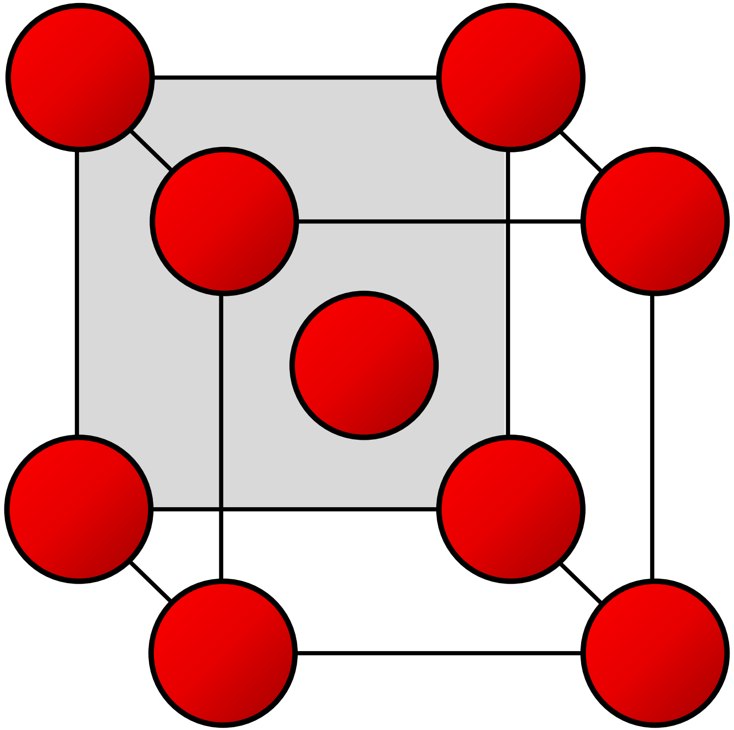
Cleavage Plane

Body Centered Cubic (BCC)



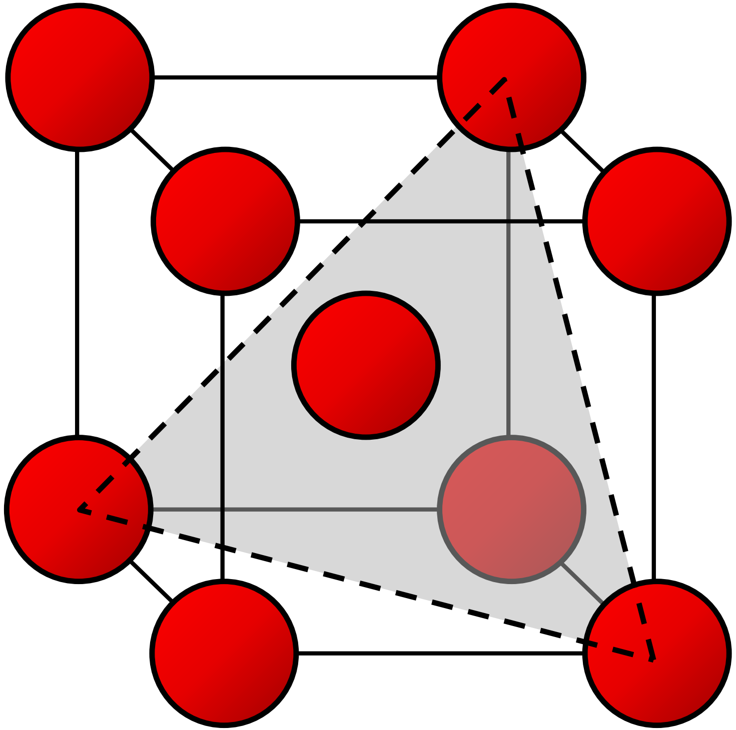
Cleavage Plane

Body Centered Cubic (BCC)

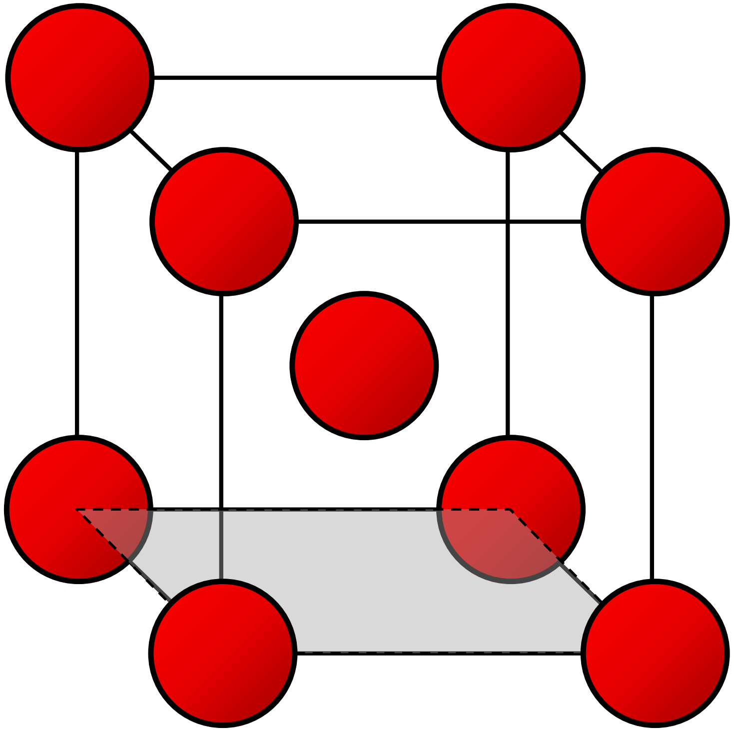


Cleavage Plane

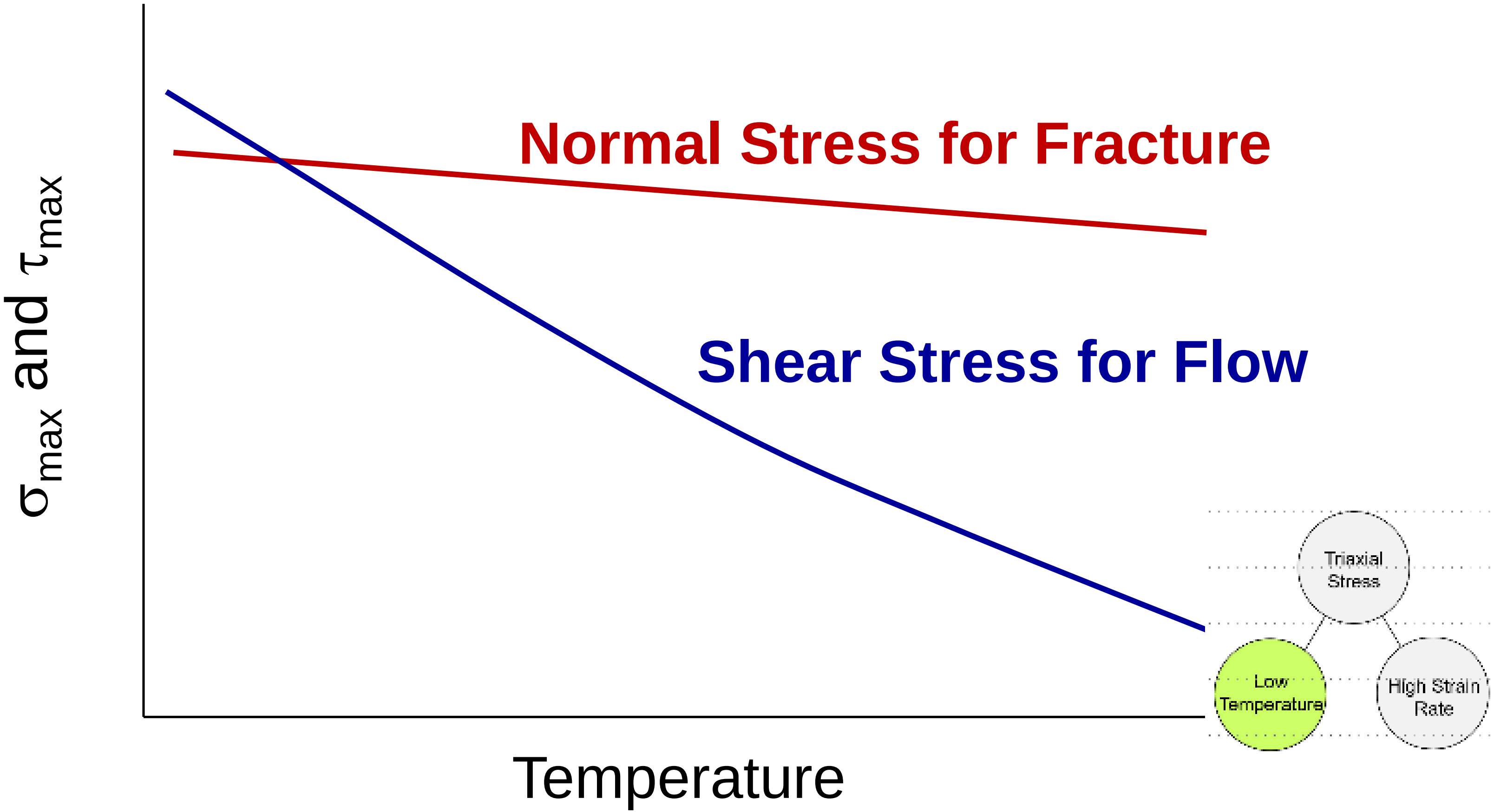
Body Centered Cubic (BCC)



Shear Plane

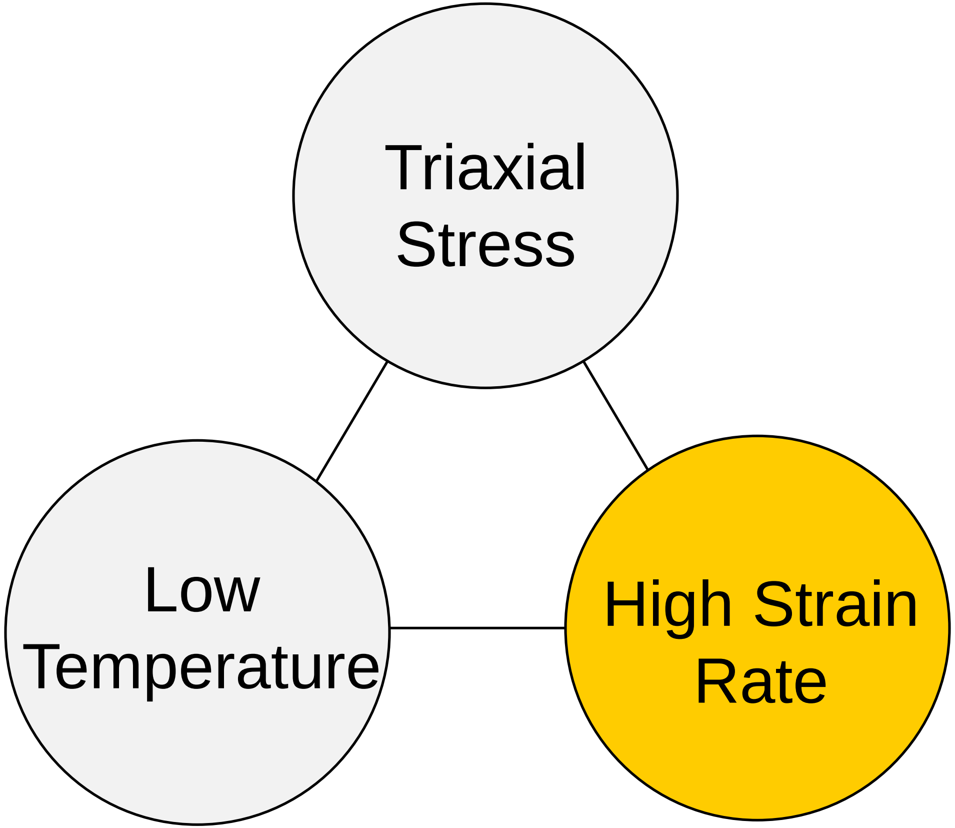


Cleavage Plane



Adapted from Gensamer

The Unholy Trinity



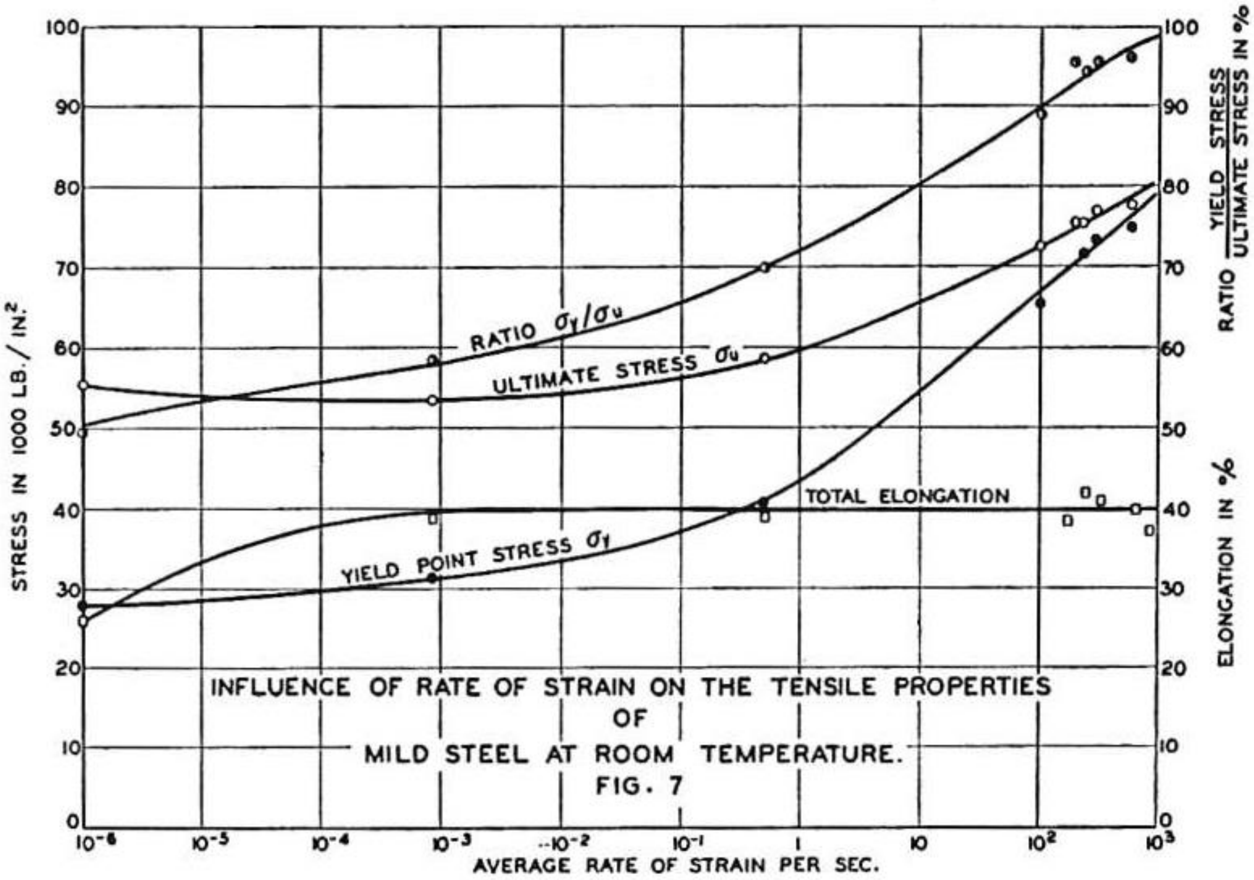
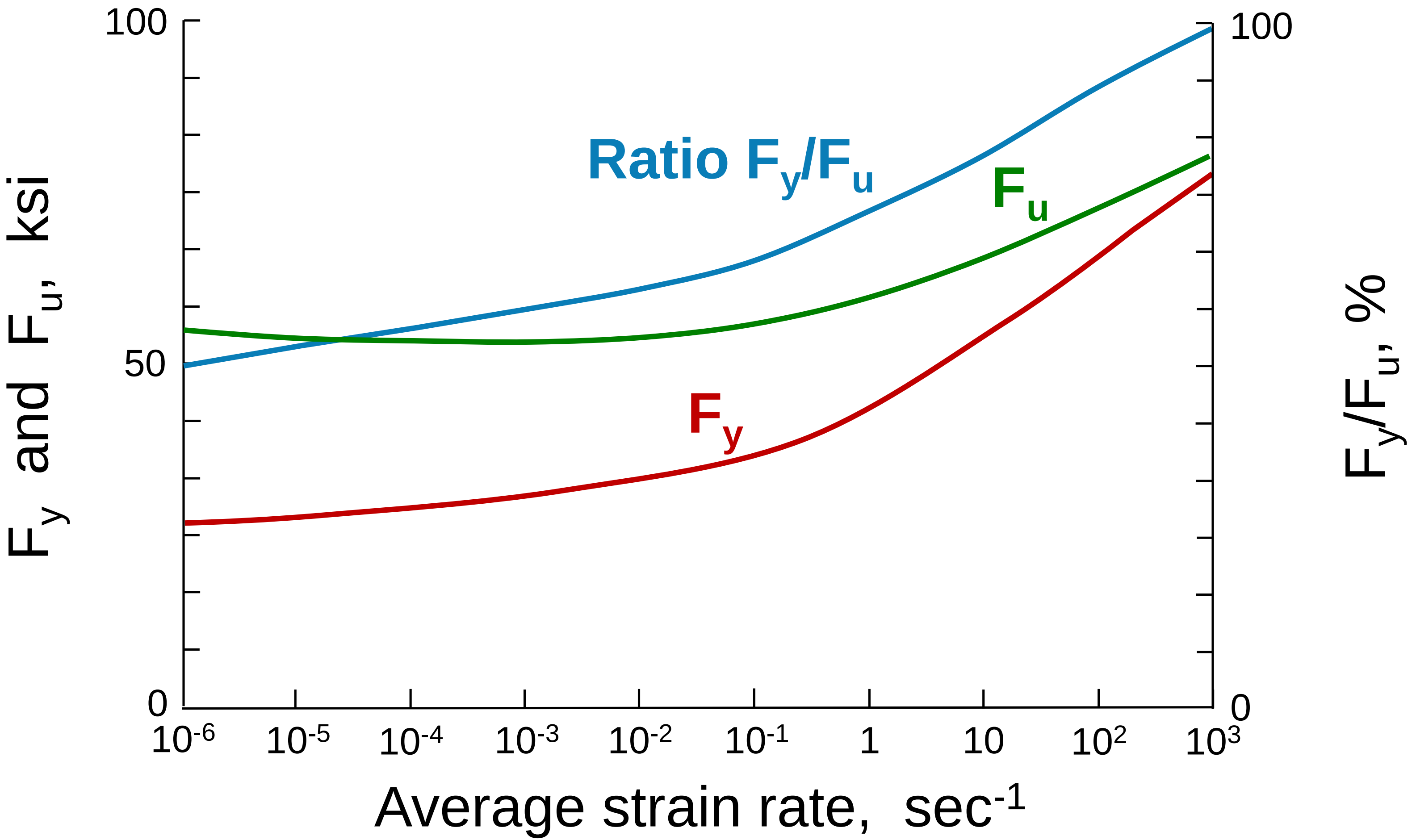
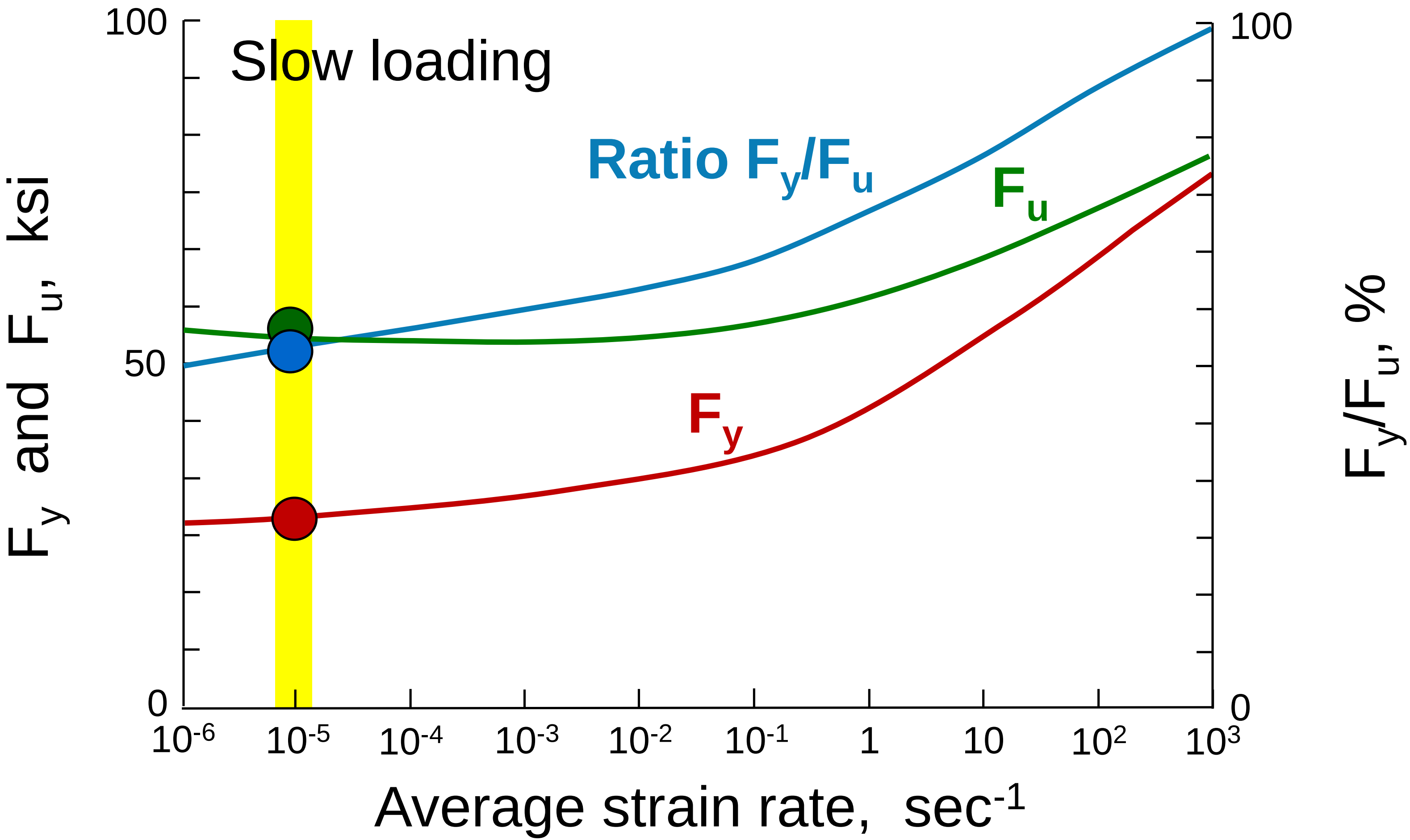


Fig. 7 Influence of Rate of Strain on Tensile Properties of Mild Steel at Room Temperature

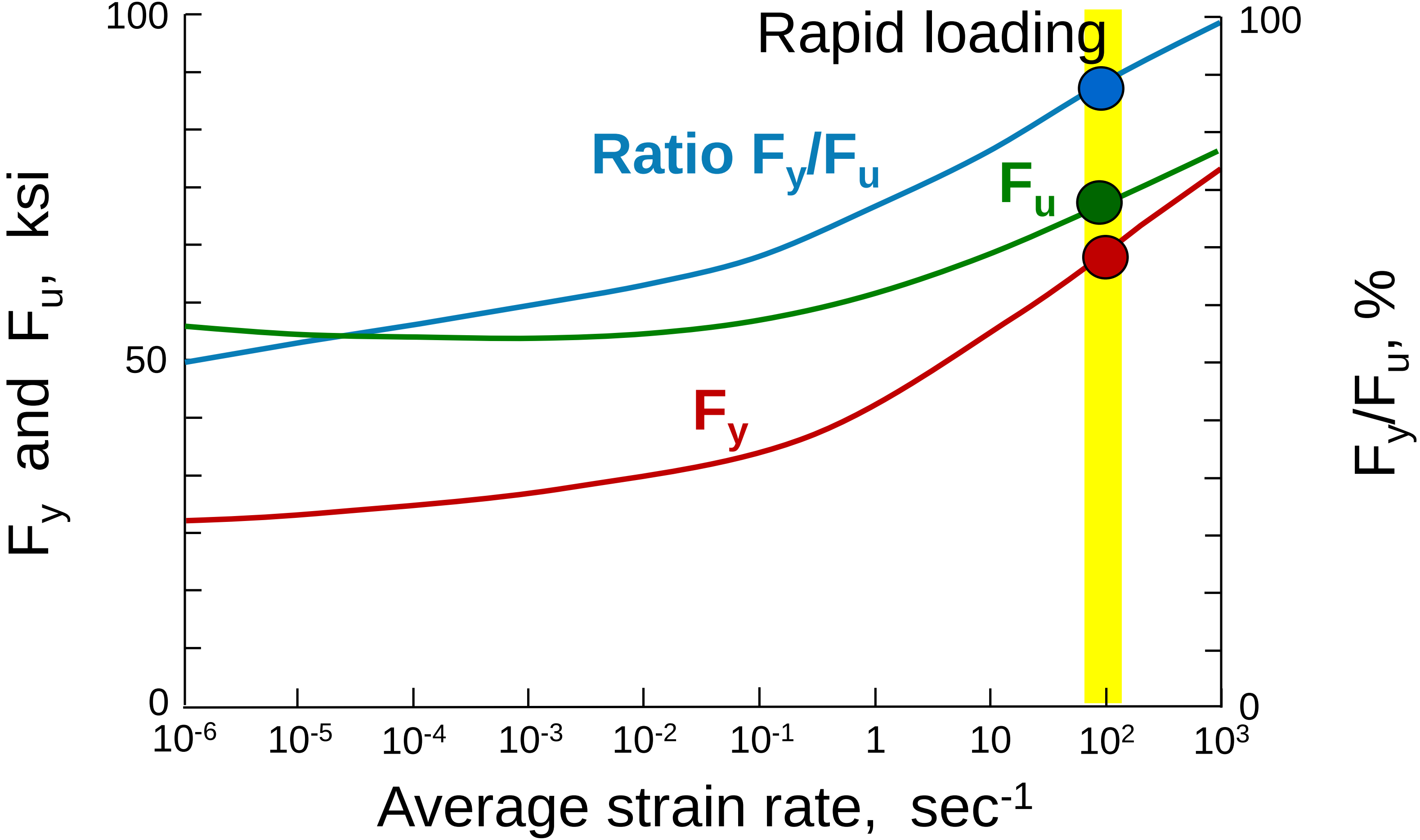
From Manjoine



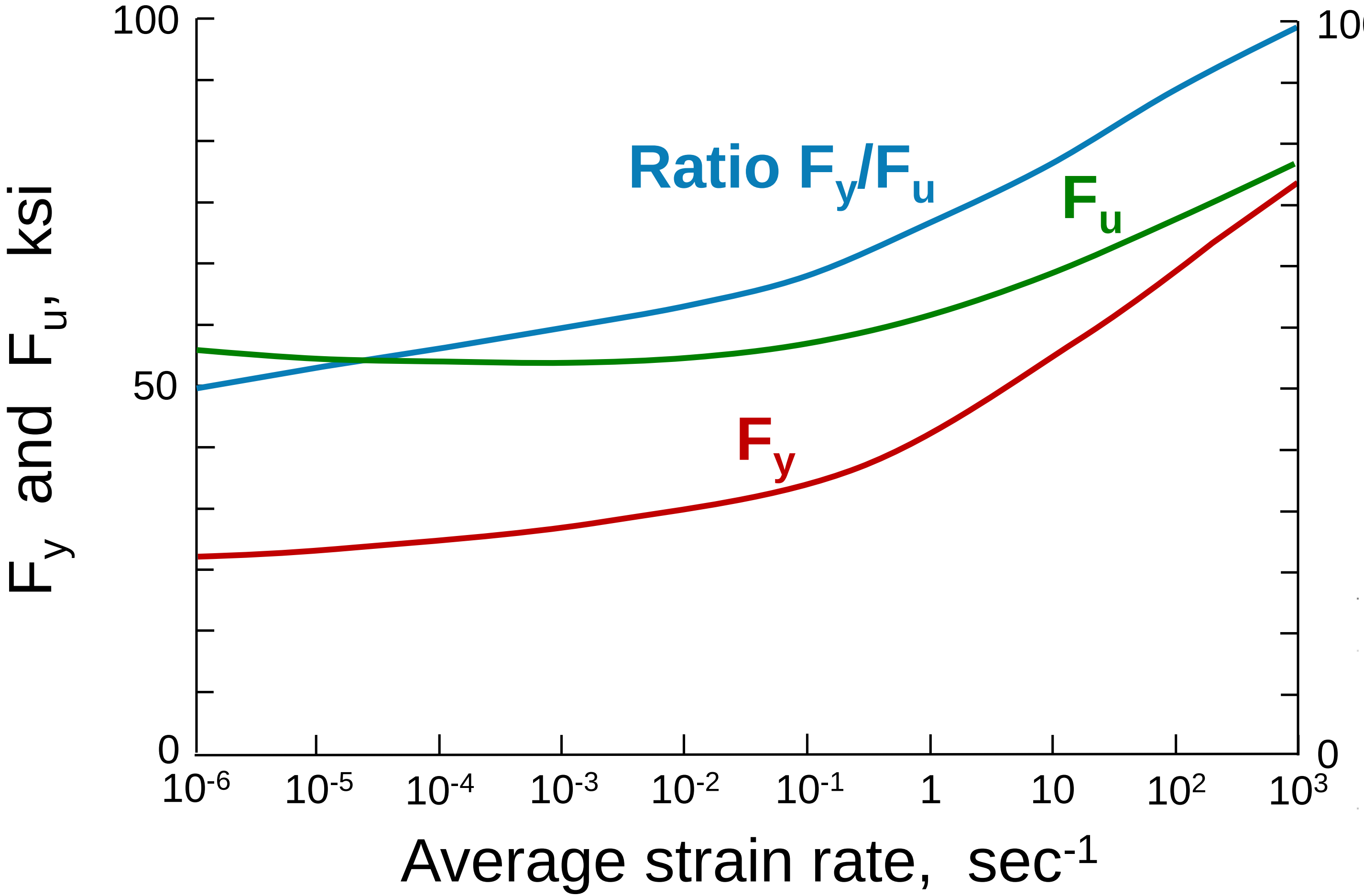
Adapted from Manjoine

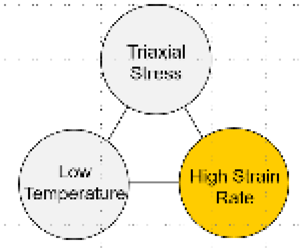


Adapted from Manjoine



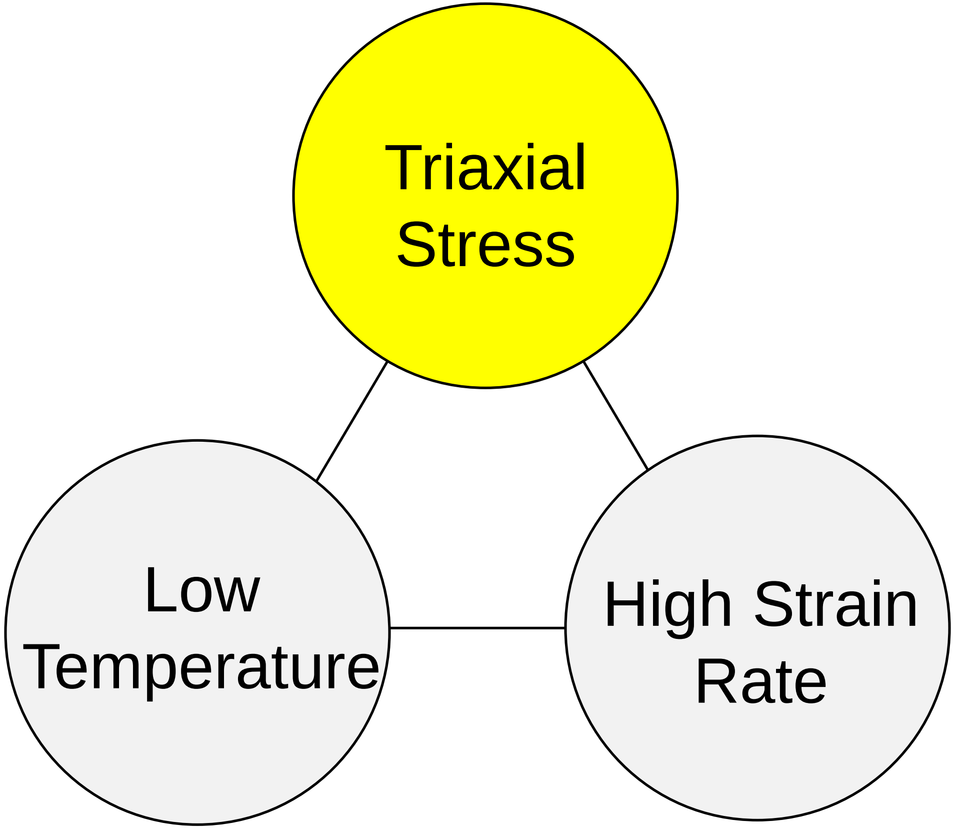
Adapted from Manjoine

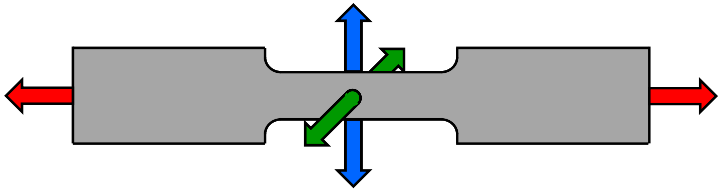


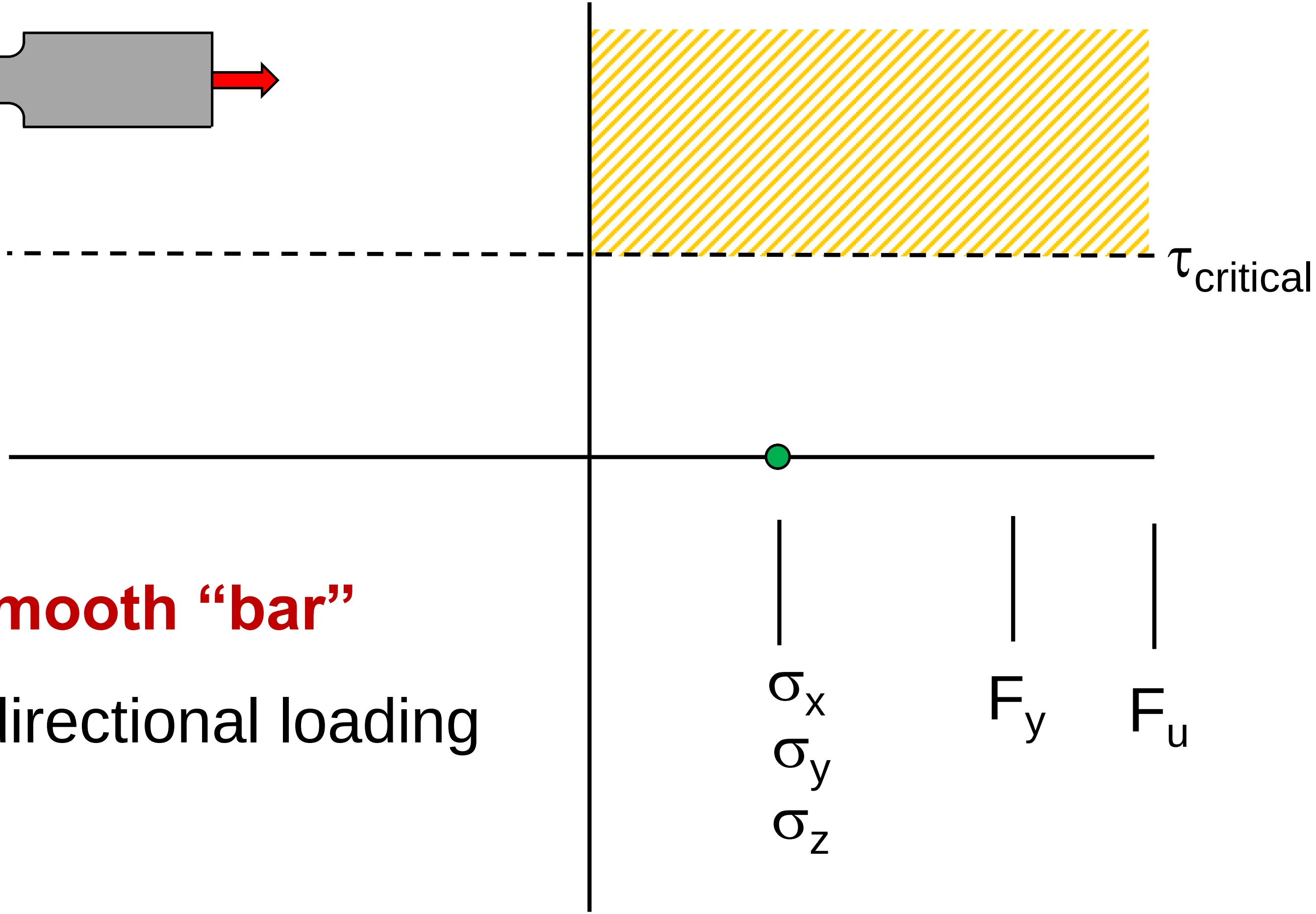


Adapted from Manjoine

The Unholy Trinity

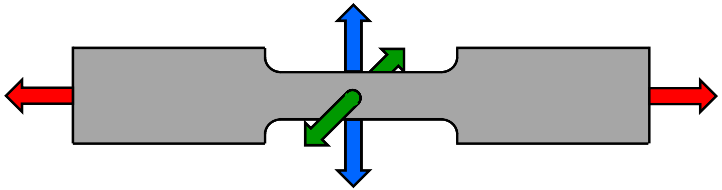


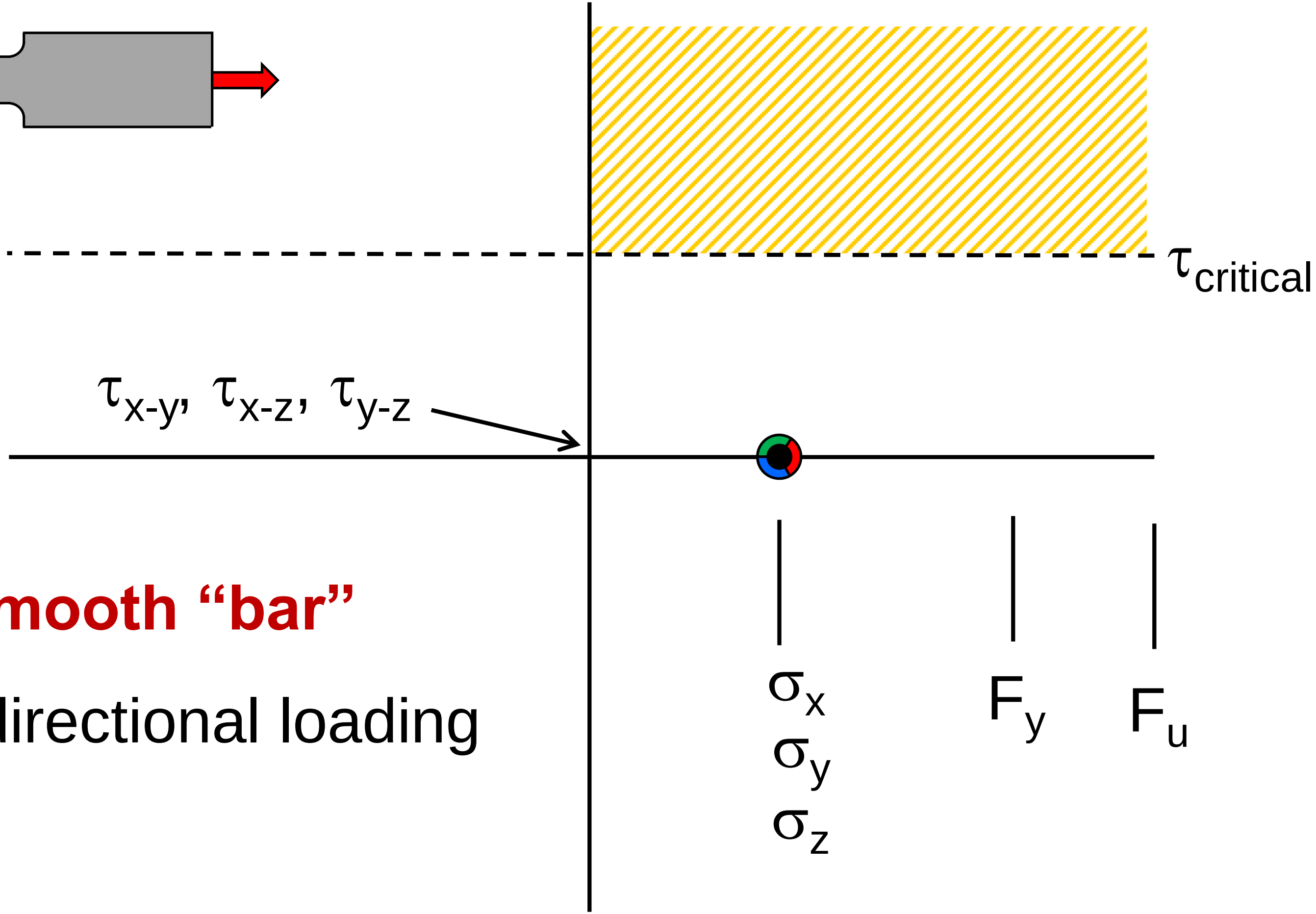




Smooth “bar”

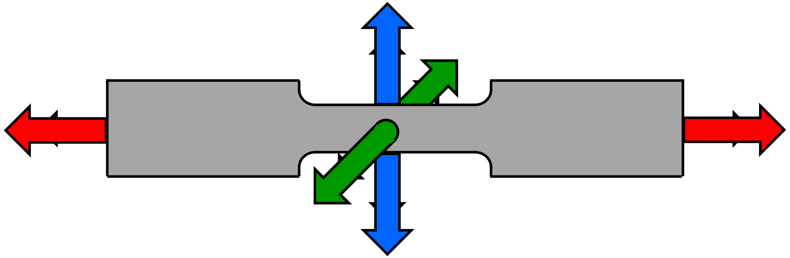
Multi-directional loading

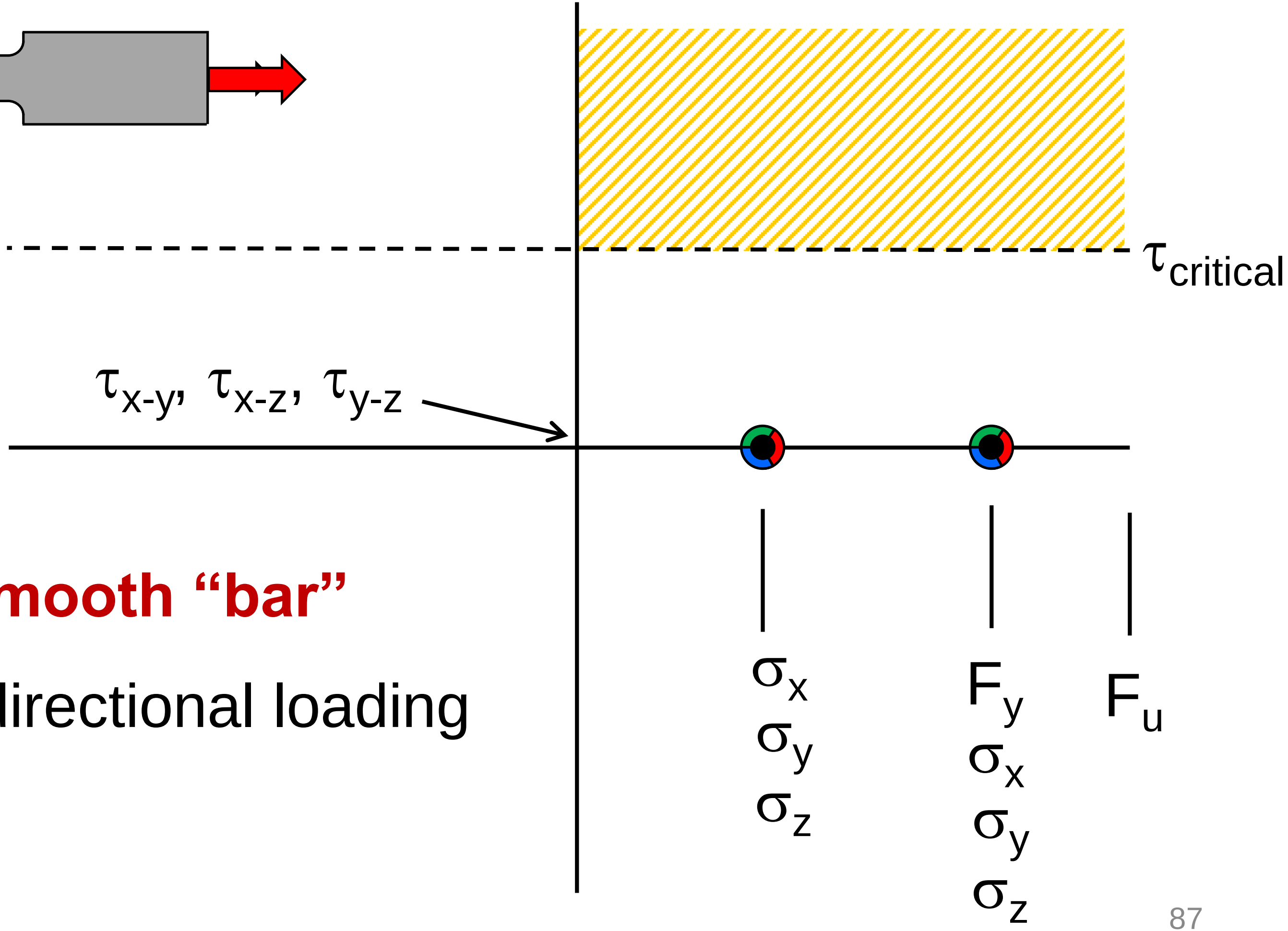




Smooth “bar”

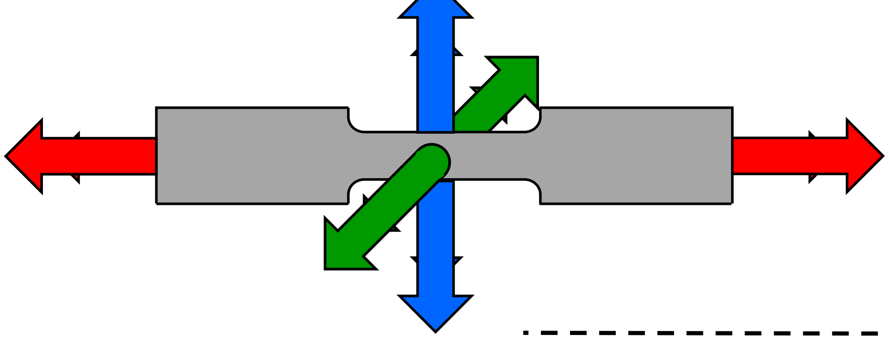
Multi-directional loading





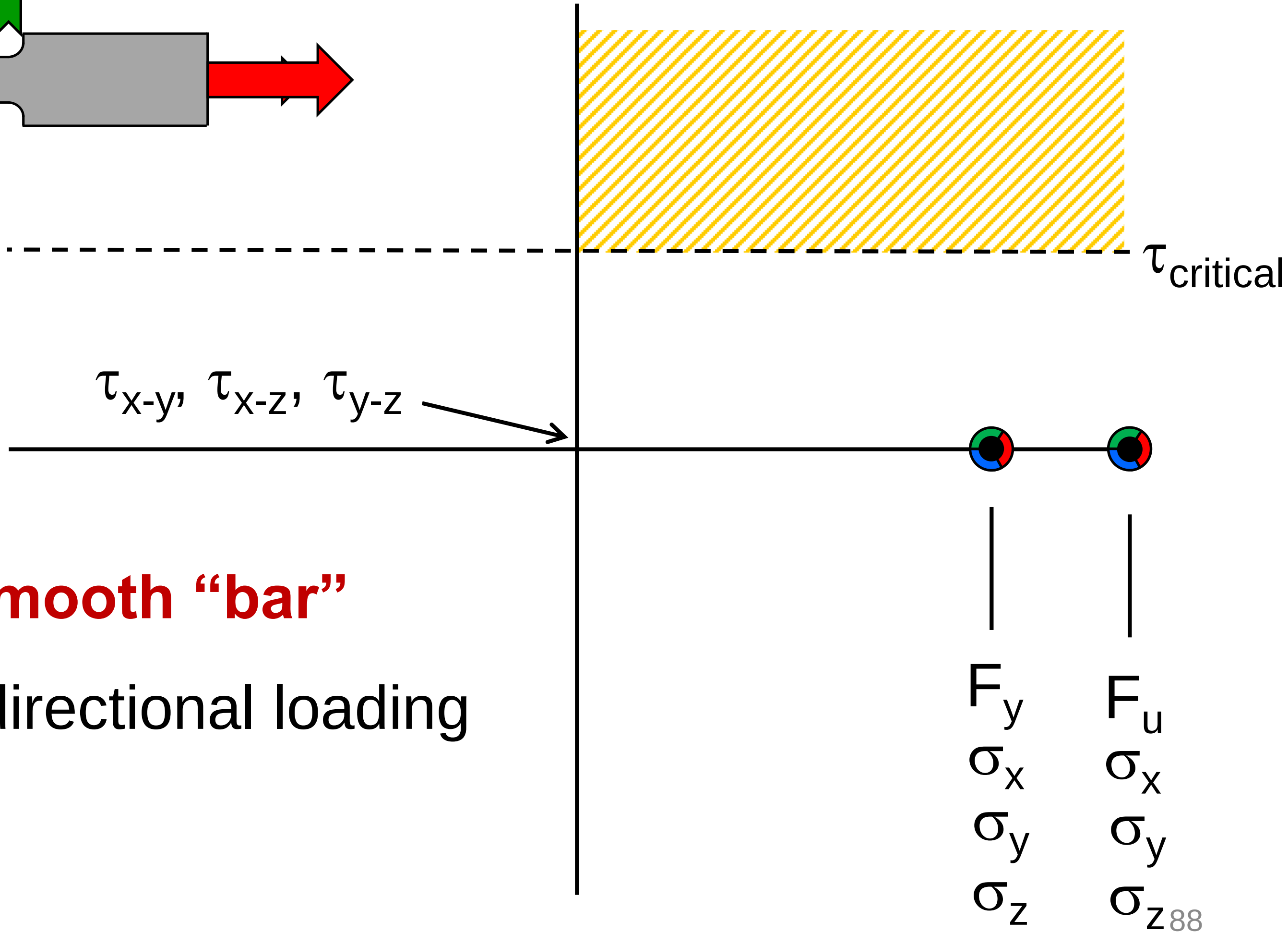
Smooth "bar"

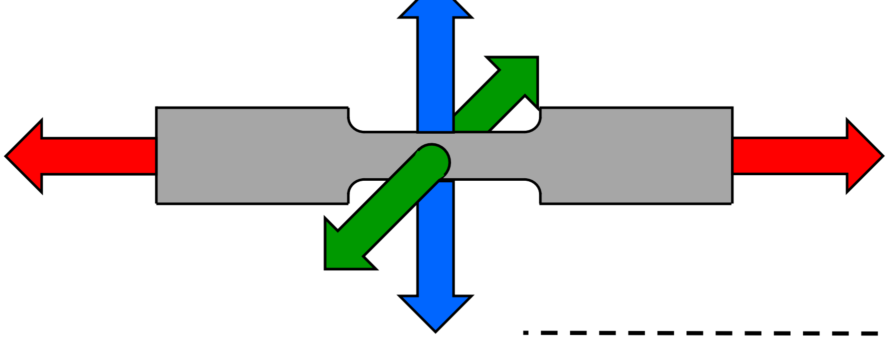
Multi-directional loading

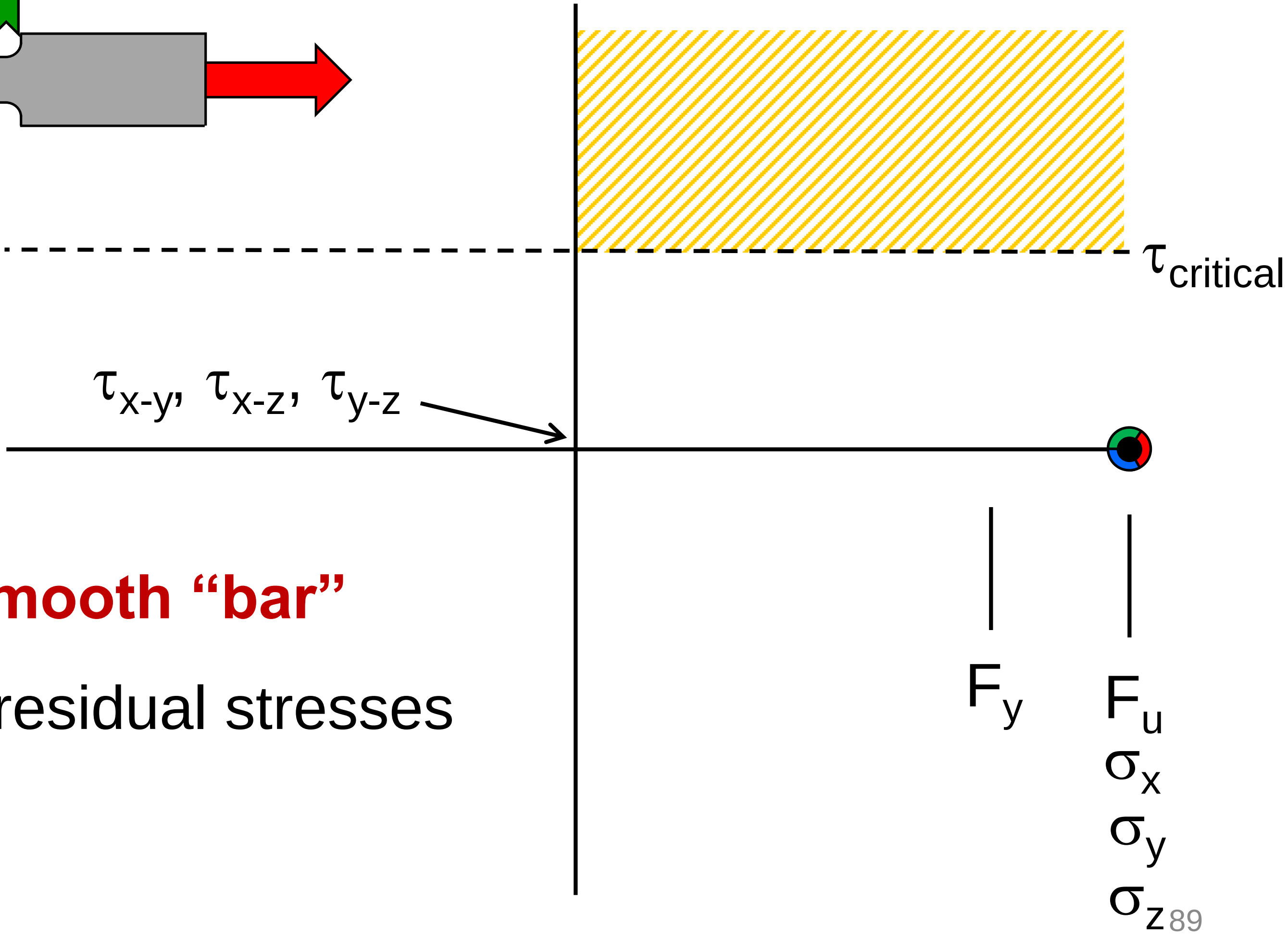


Smooth “bar”

Multi-directional loading

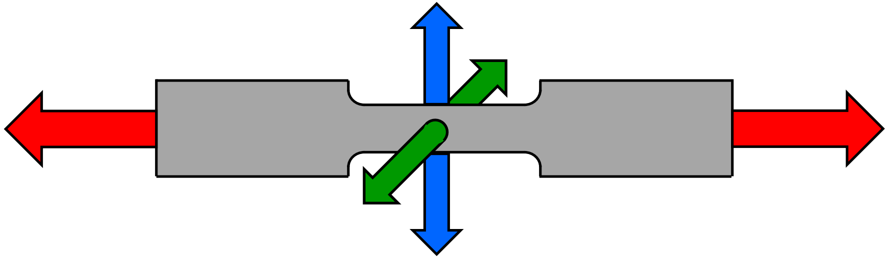


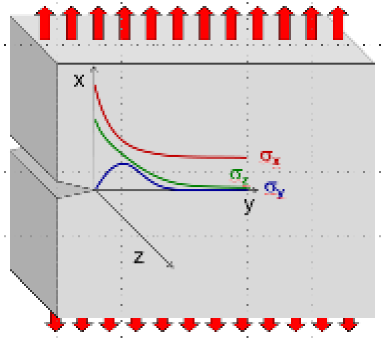




Smooth "bar"

Also, residual stresses

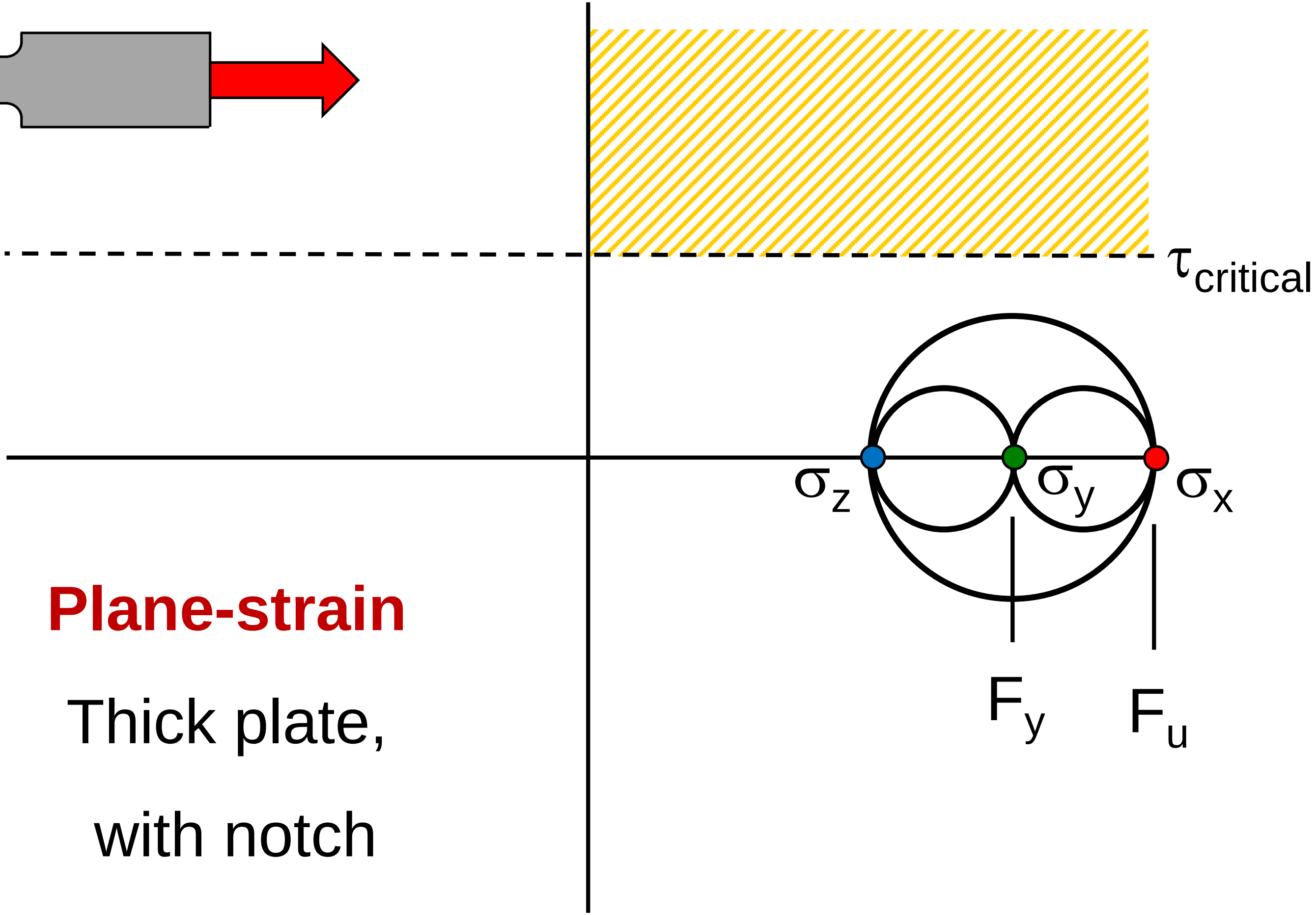




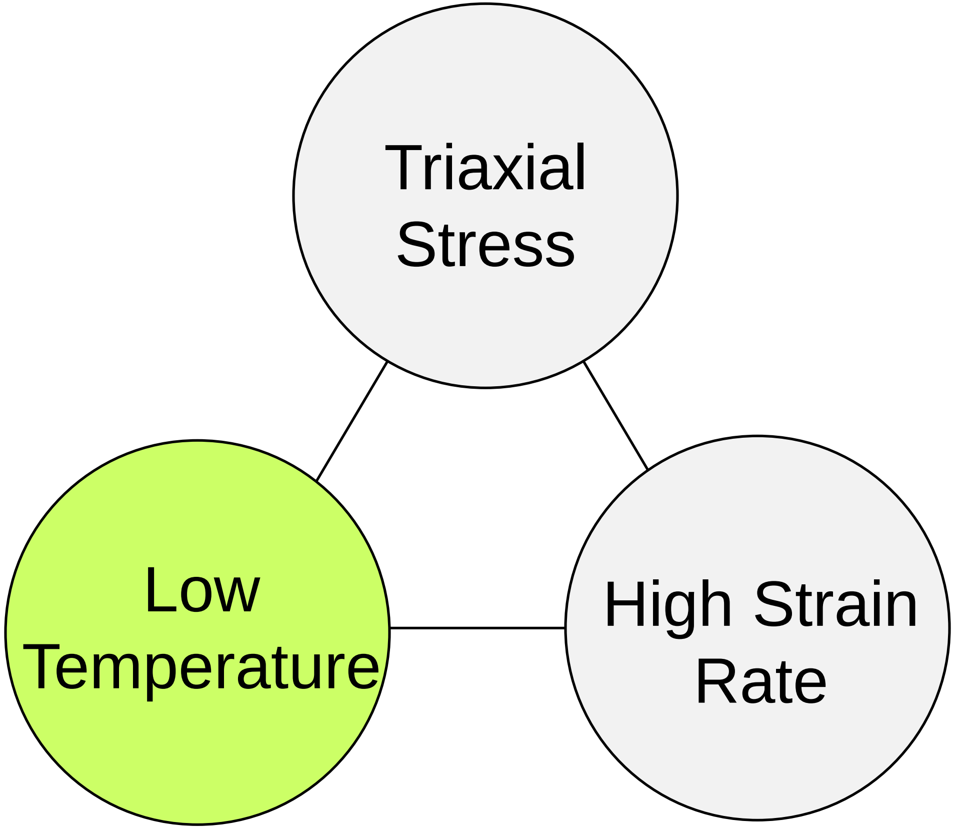
Plane-strain

Thick plate,

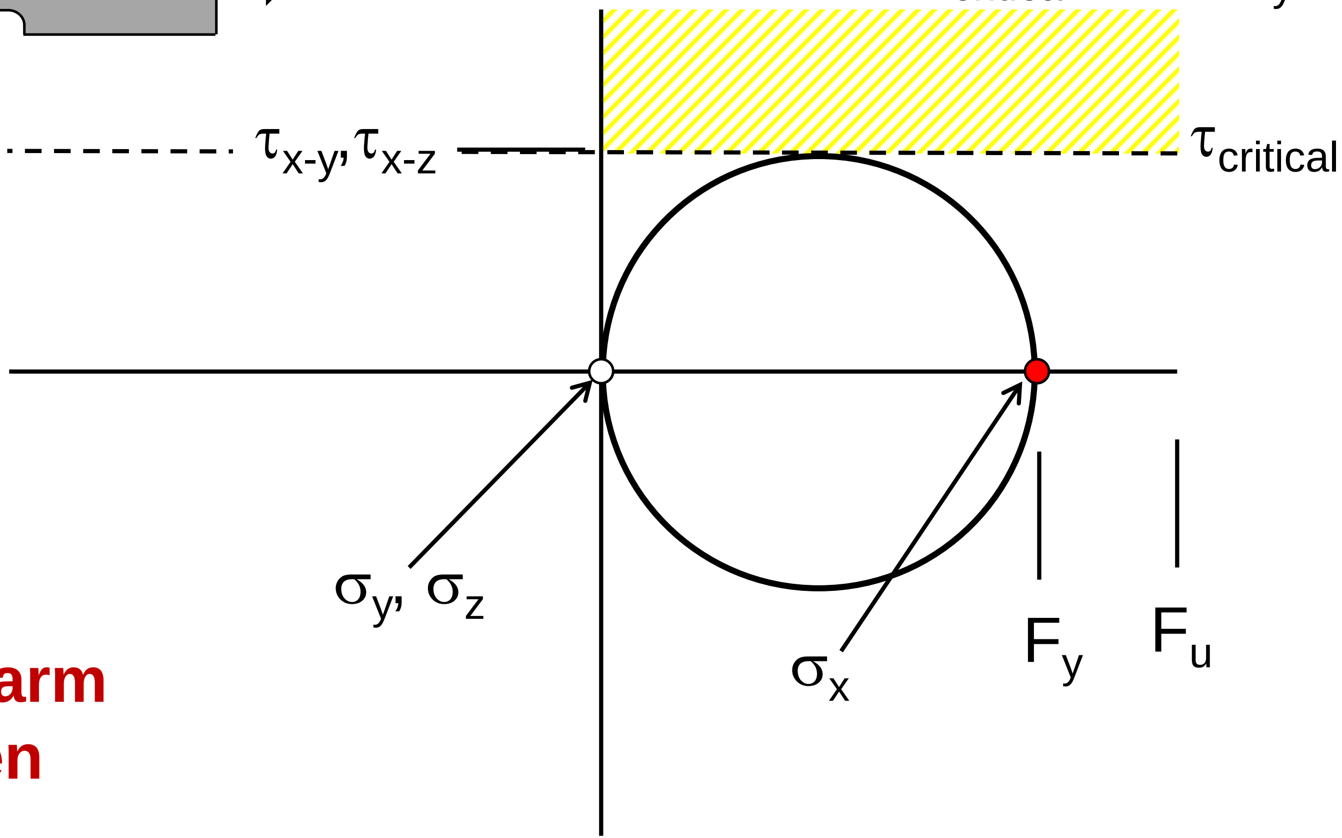
with notch



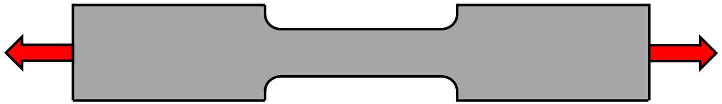
The Unholy Trinity



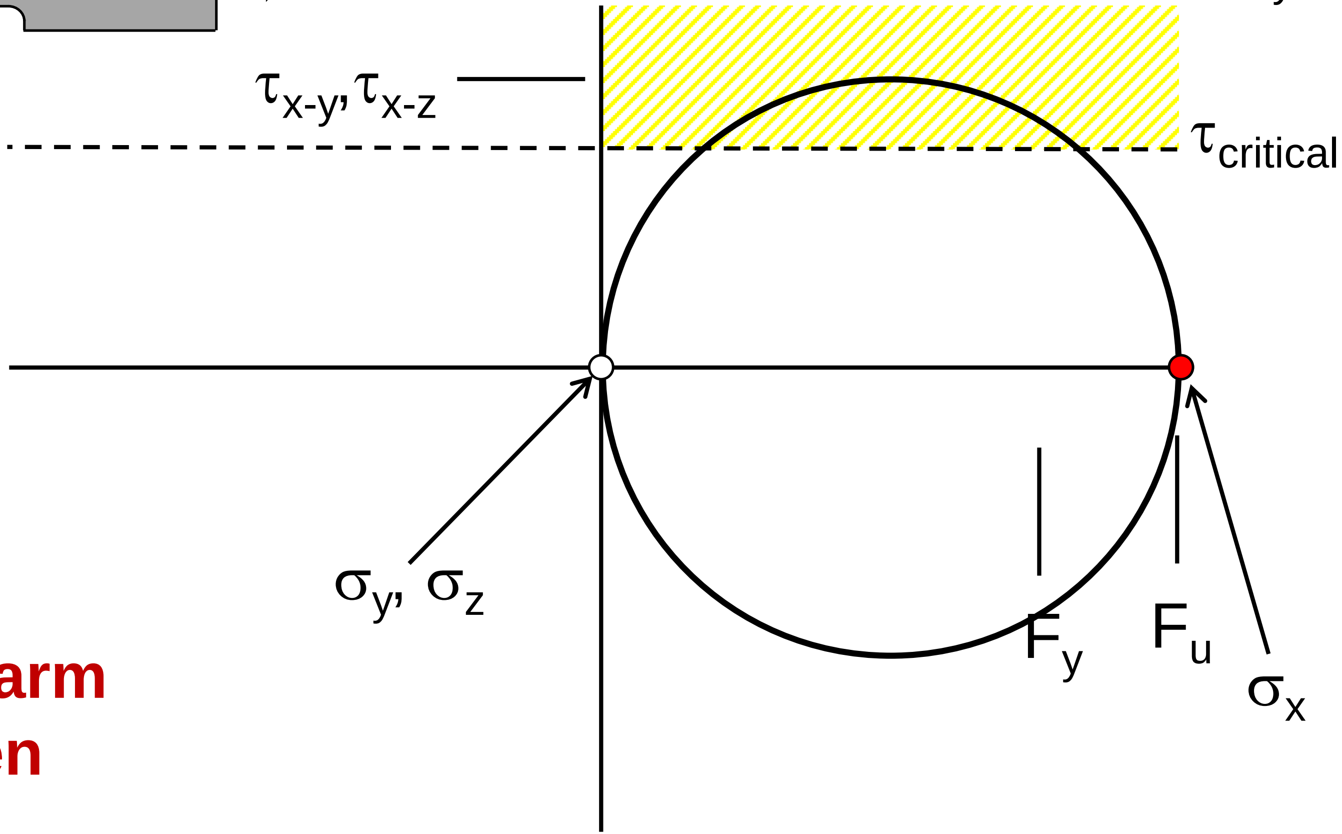
For this example, $\tau_{critical} = 0.5F_y$



Smooth warm specimen

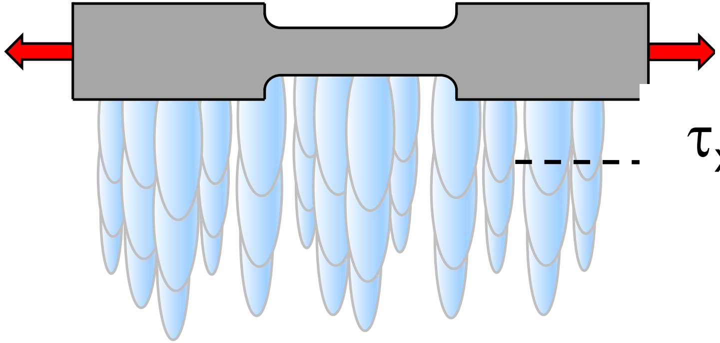


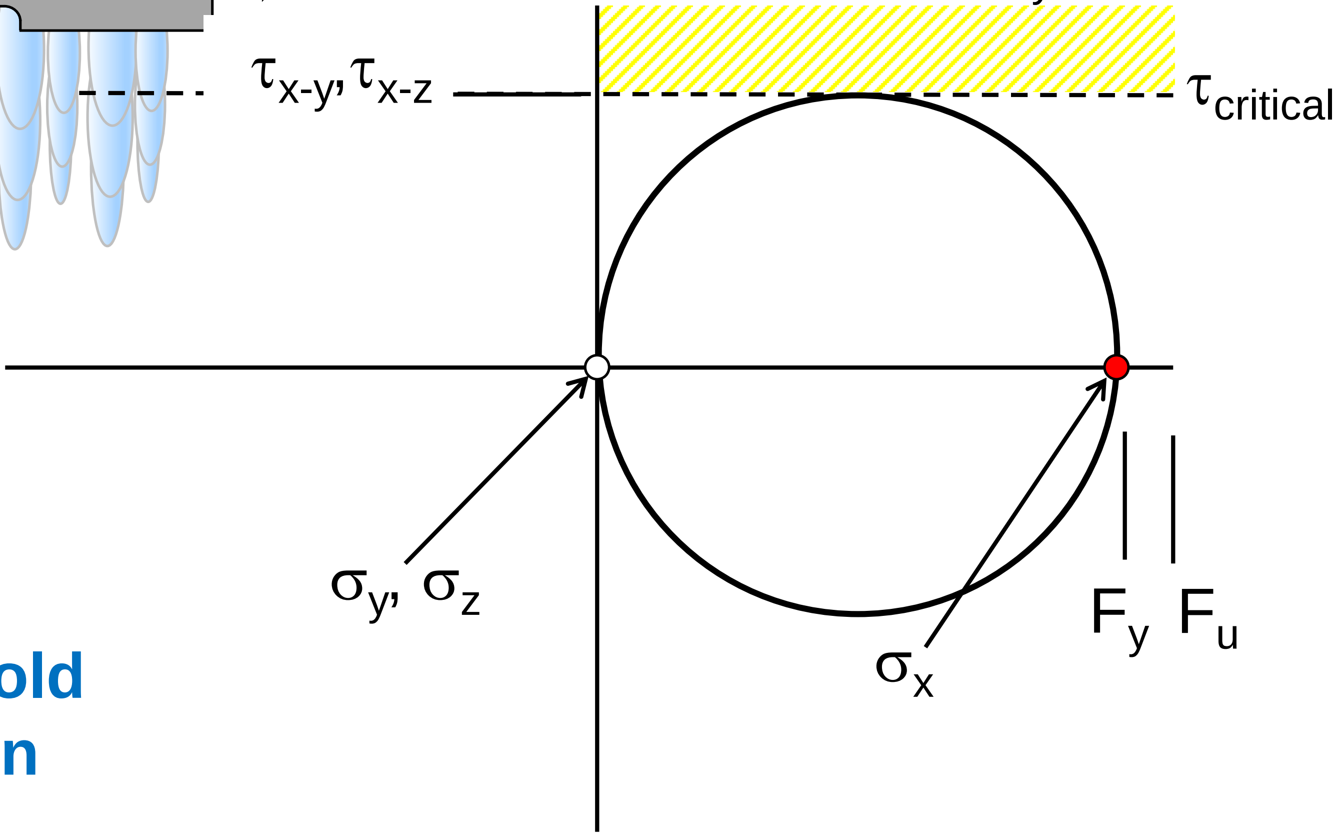
For this example, $\tau_{critical} = 0.5F_y$



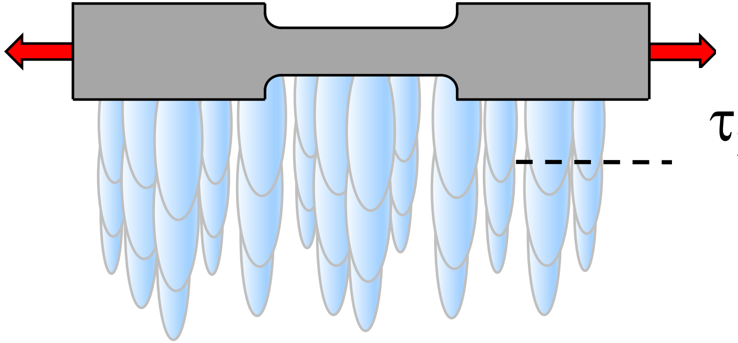
Smooth warm specimen

For low temperature, let $F_y = 0.90F_u$

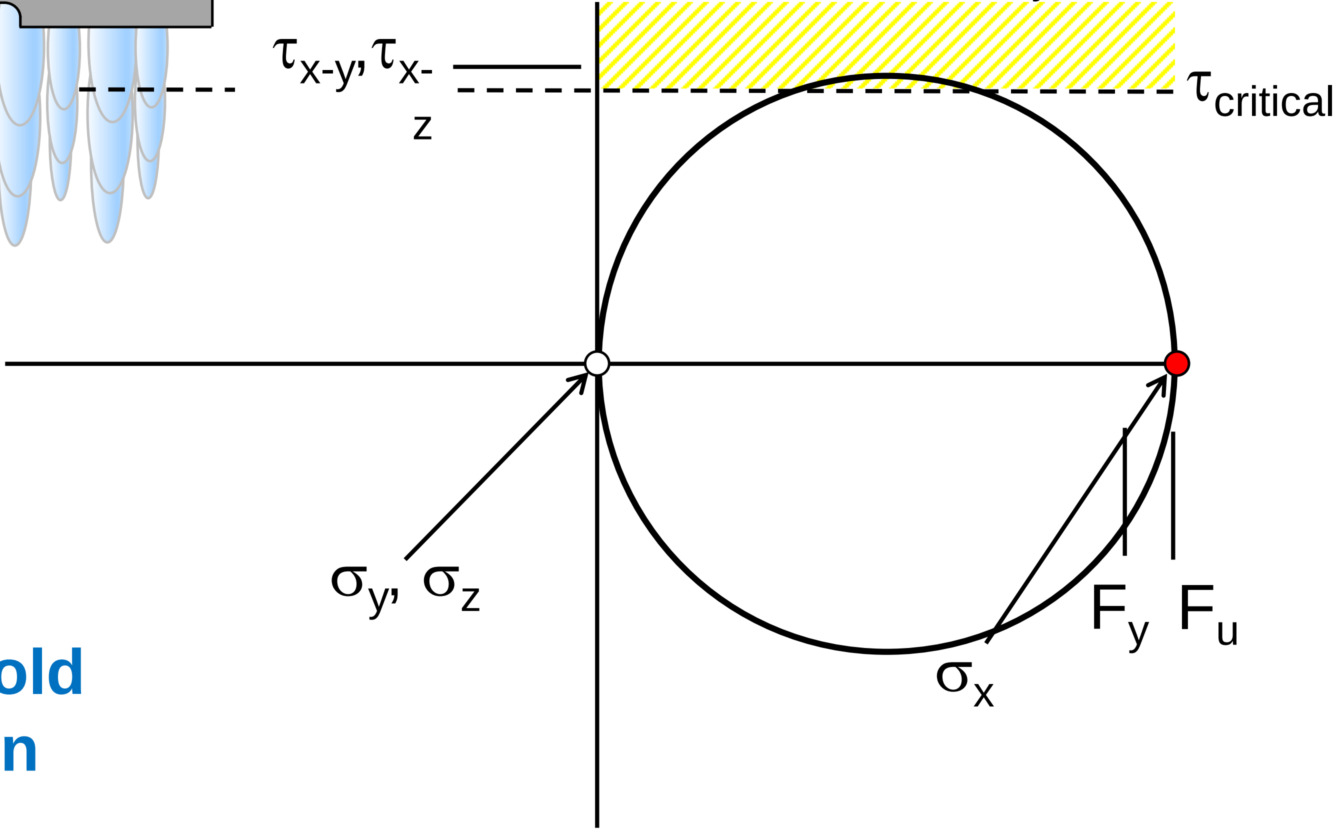




Smooth cold specimen

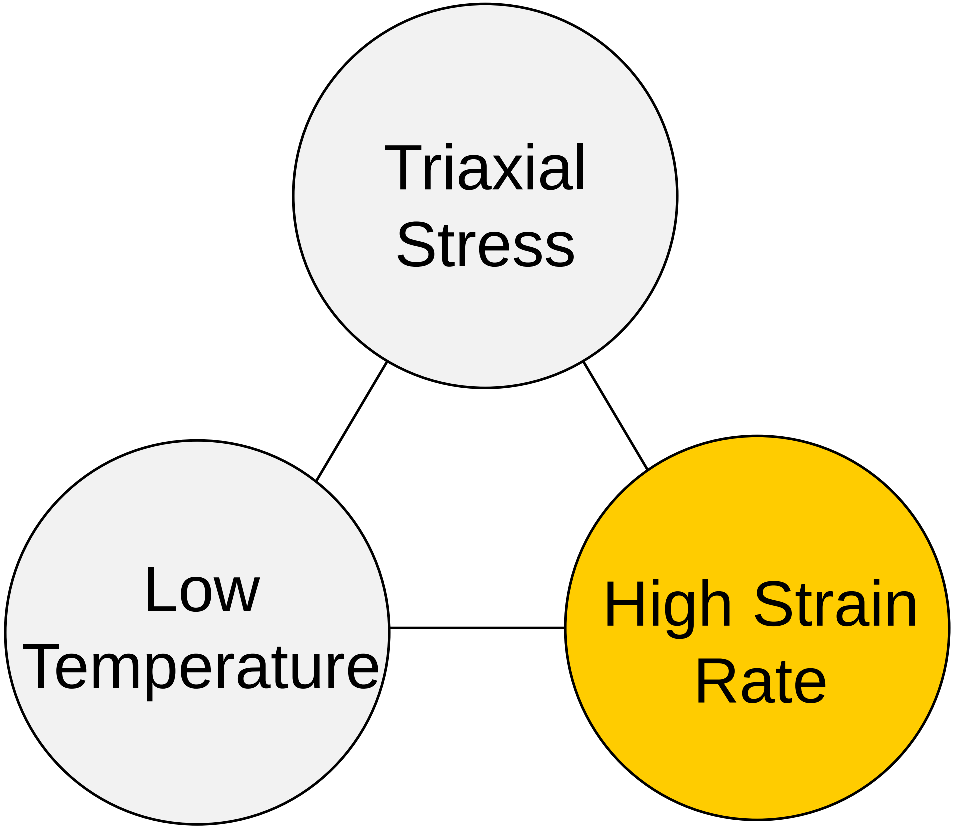


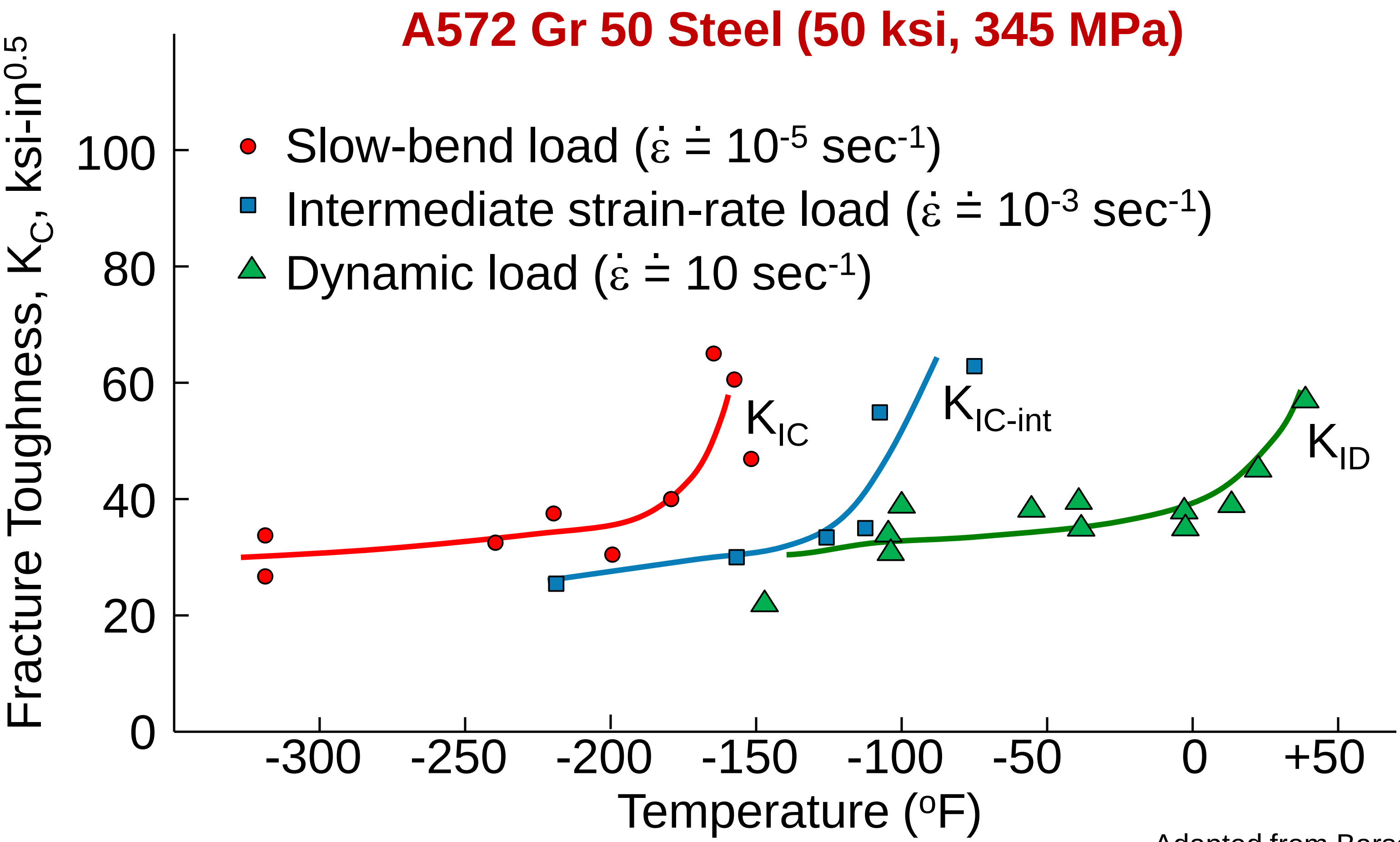
For low temperature, let $F_y = 0.90F_u$



Smooth cold specimen

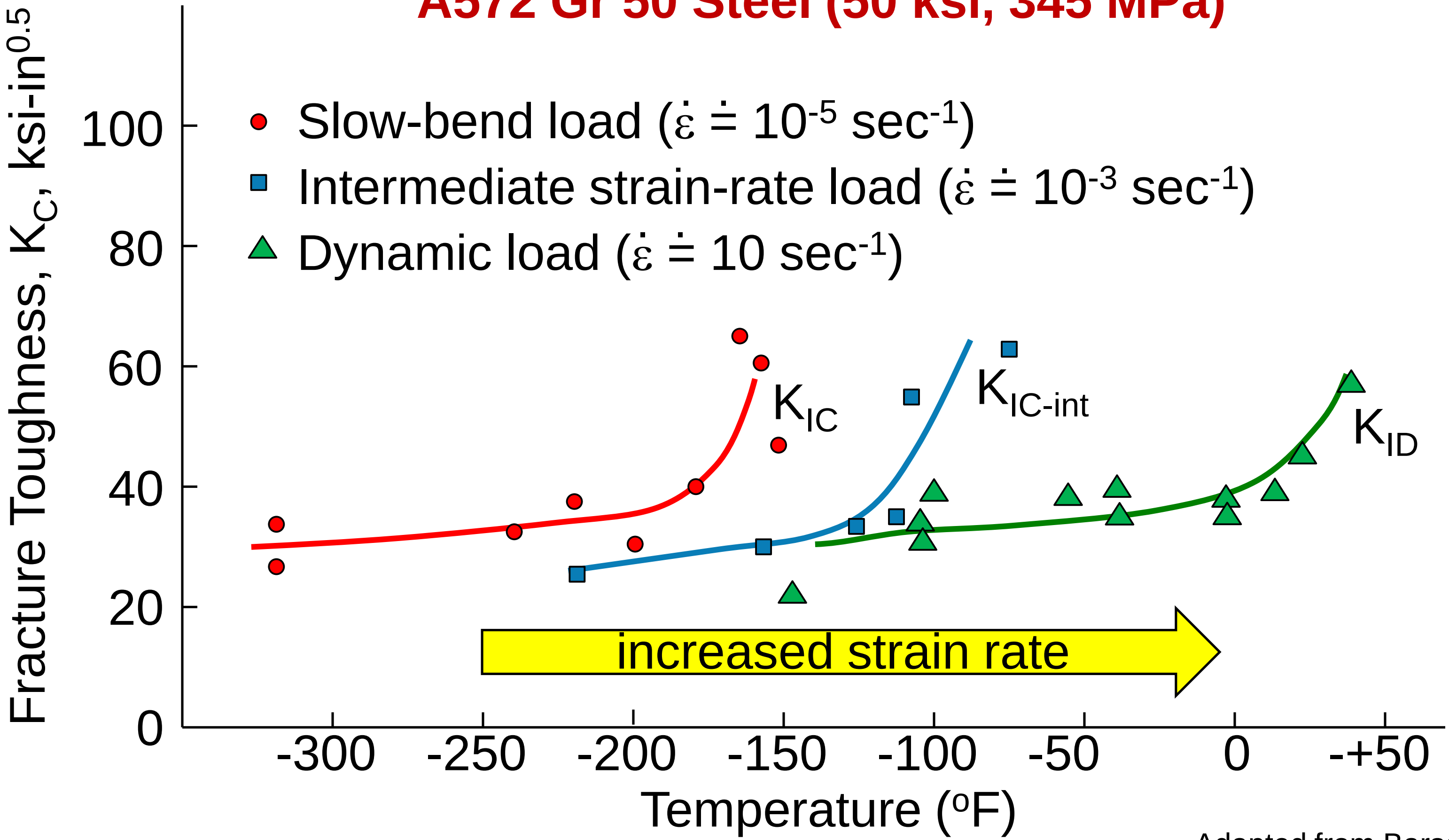
The Unholy Trinity



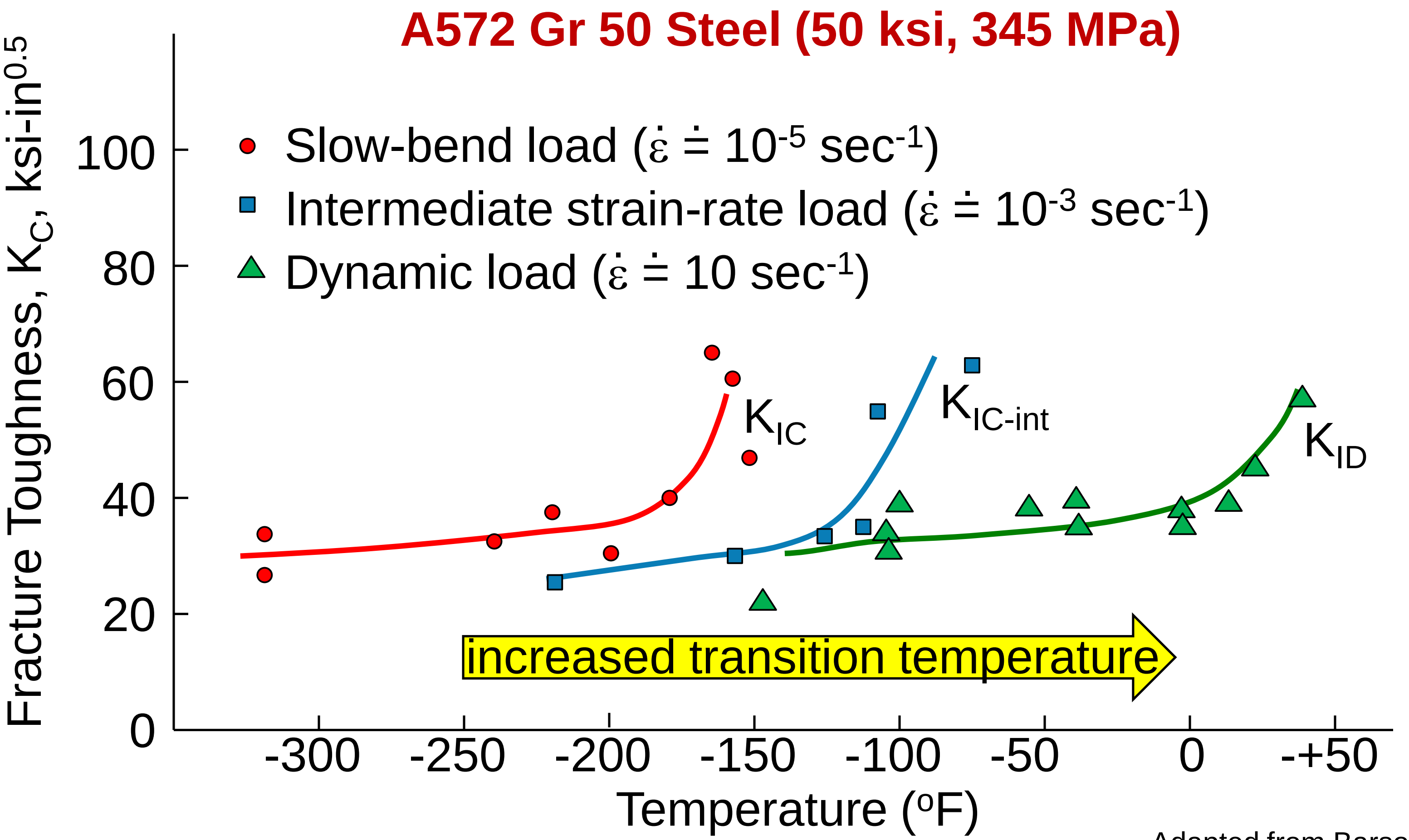


Adapted from Barsom & Rolfe

A572 Gr 50 Steel (50 ksi, 345 MPa)



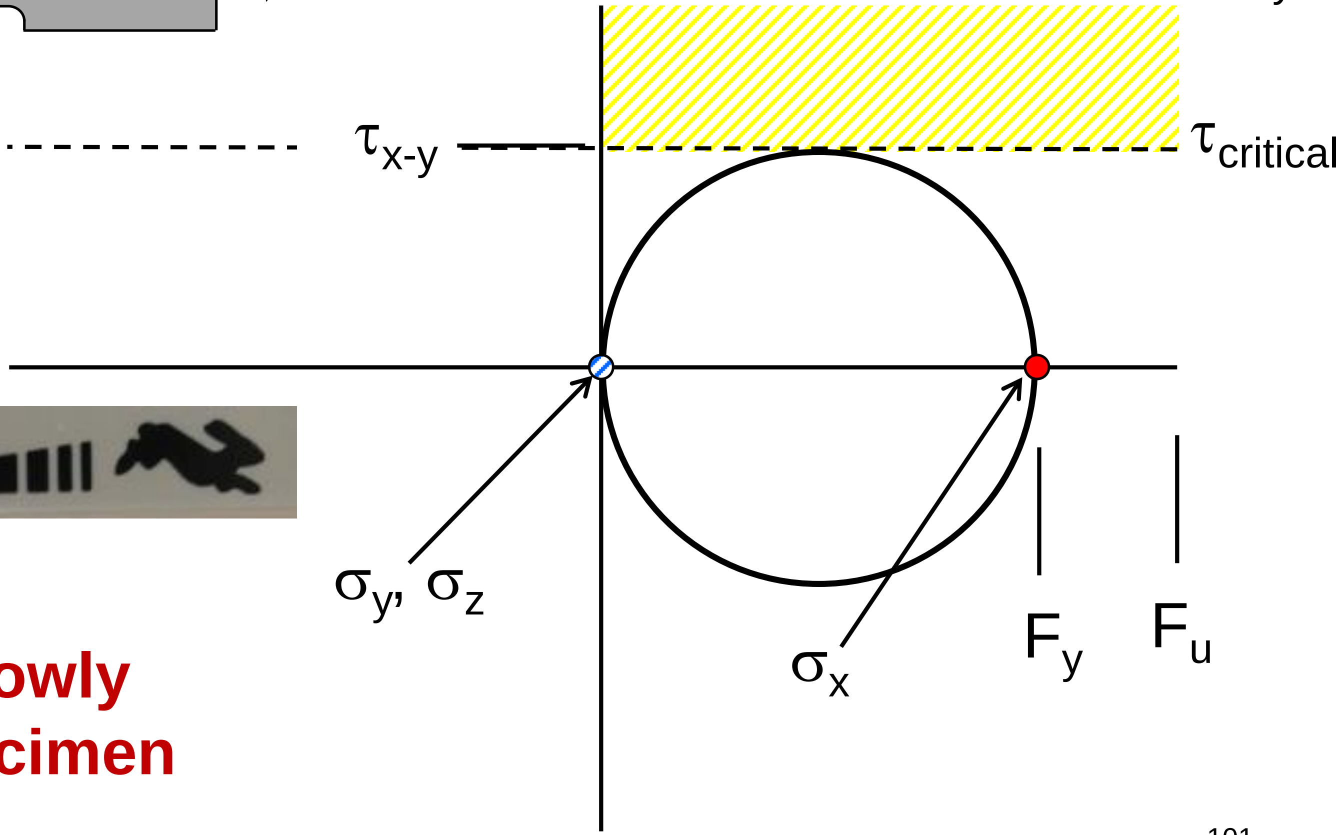
Adapted from Barsom & Rolfe



Adapted from Barsom & Rolfe

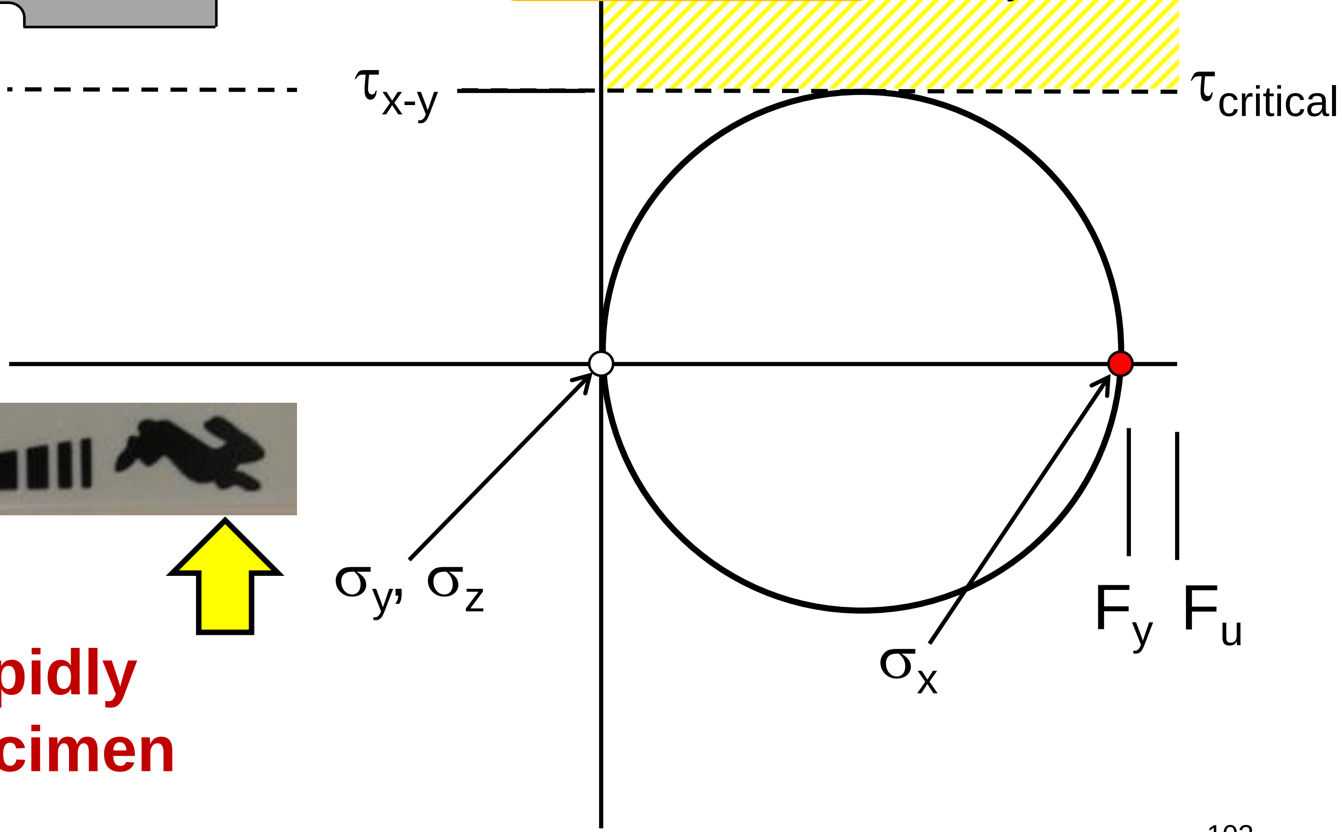
The changes produced by the introduction of a notch have important consequences in the fracture process. For example, the presence of a notch will increase appreciably the ductile/brittle transition temperature of a steel.

For this example, $\tau_{critical} = 0.5F_y$

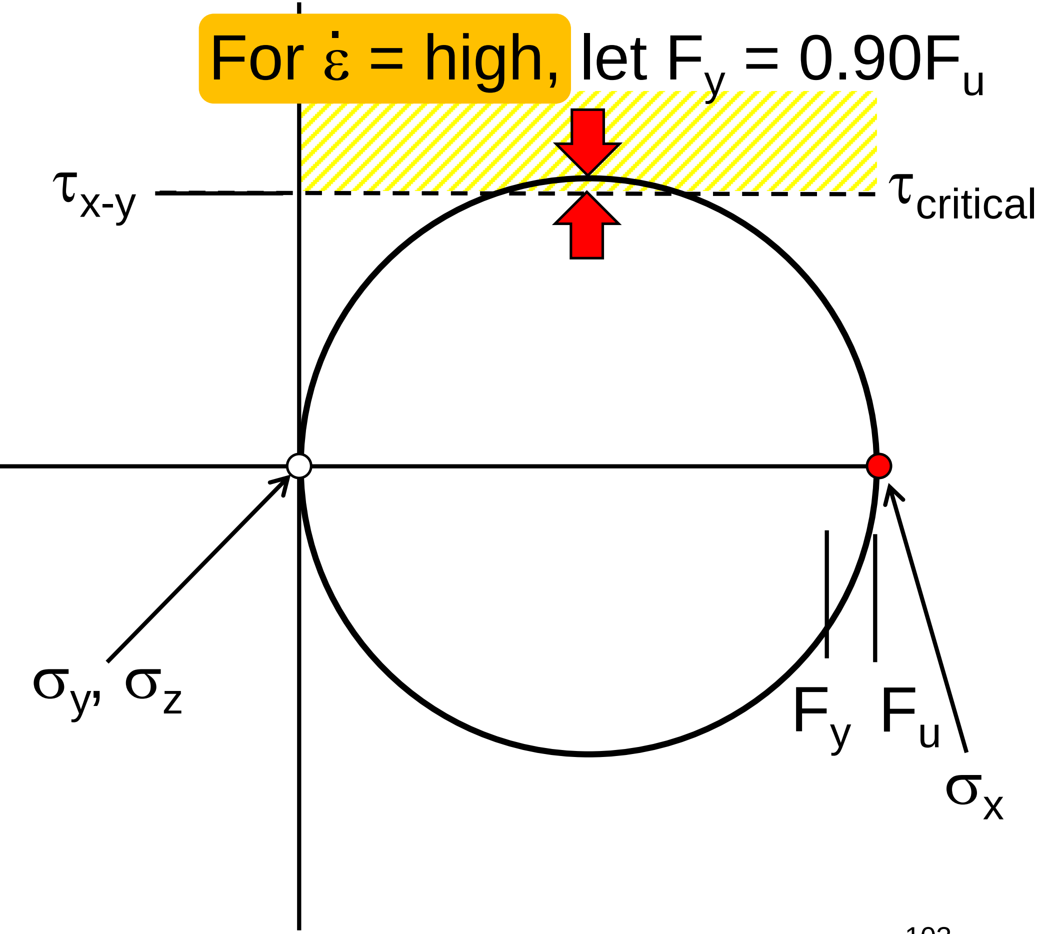


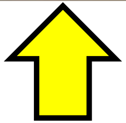
Smooth, slowly strained specimen

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



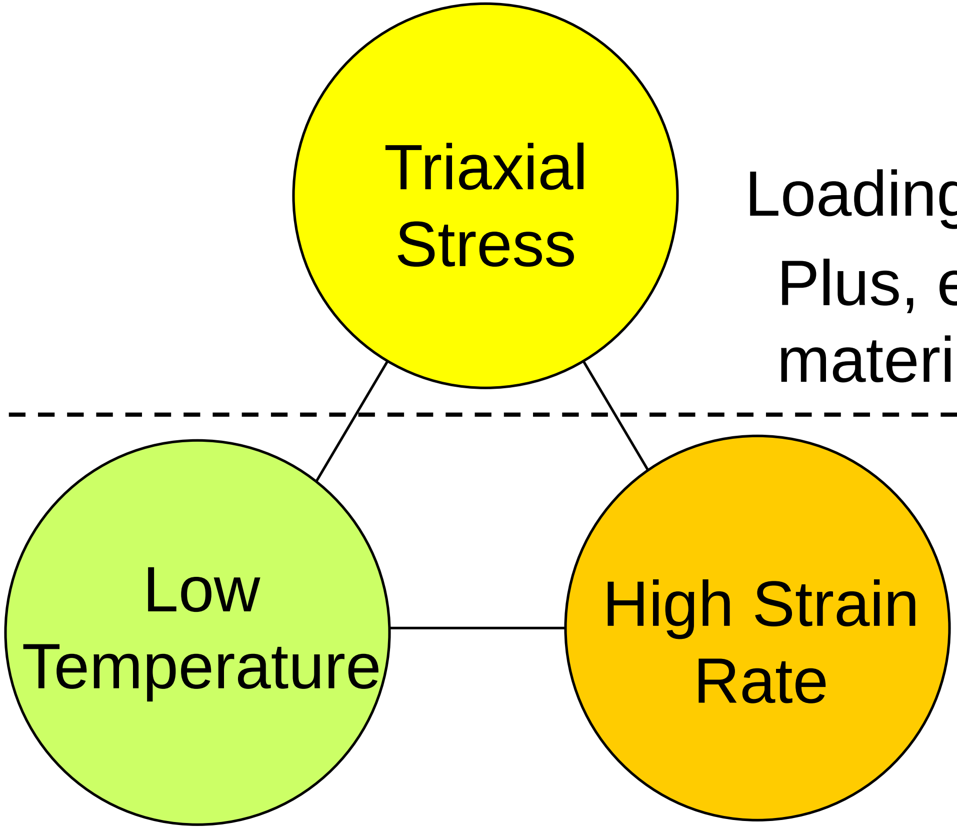
Smooth, rapidly strained specimen





Smooth, rapidly strained specimen

The Unholy Trinity



Loading, notches

Plus, effect on material

Effect on material

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 Studies Involving Brittle Fracture:

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


SS SCHENECTADY

The Design and Methods of Construction of Welded Steel Merchant Vessels

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

GOVERNMENT PRINTING OFFICE
WASHINGTON: 1947

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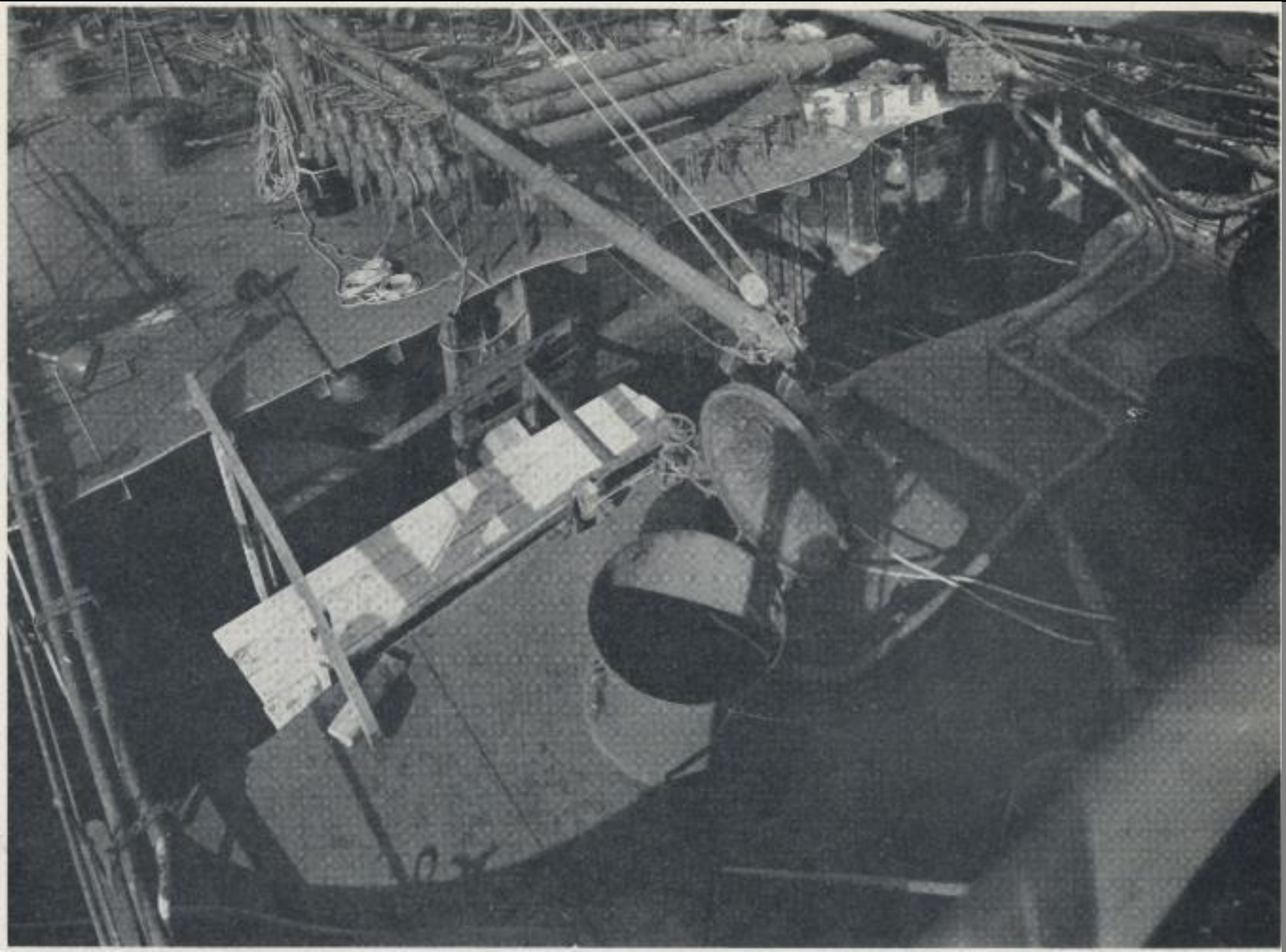


Early in the war, welded merchant vessels experienced difficulties in the form of fractures which could not be explained. The fractures, in many cases, manifested themselves with explosive suddenness and exhibited a quality of brittleness which was not ordinarily associated with the behavior of a normally ductile materials such as ship steel.

Total number of ships	4,696
Total number of these ships reporting no casualties	3,724
Total number of these ships which sustained casualties	970
Total number of casualties	1,442
Total number of fractures	4,720
Total cases of serious casualties (Class 1)	127
Total ships sustaining a complete fracture of strength deck	24
Total ships sustaining a complete fracture of the bottom	1

Eight vessels have been lost, as follows:		
Name	Date	Remarks
Thomas Hooker	5 Mar 1943	Abandoned
J.L.M. Curry	7 Mar 1943	Abandoned
John P. Gaines	24 Nov 1943	Broke in two, abandoned
Joseph Smith	9 Jan 1944	Abandoned
Samuel Dexter	21 Jan 1944	Abandoned
Joel R. Poinsett	4 Mar 1944	Broke in two; stern portion salvaged
Sackett's Harbor	1 Mar 1946	Broke in two; stern portion salvaged
Fort Sumter	10 May 1946	Broke in two; both portions scuttled

Four other ships broke in two but were not lost	
Schenectady	15 Jan 1943
Esso Manhattan	29 Mar 1943
Valeri Chkalov	11 Dec 1943
Donbass III	17 Feb 1946

REPORT OF STRUCTURAL FAILURE OF INSPECTED VESSEL
UNITED STATES COAST GUARD

This report includes all available information up to:
1 Apr., 1944

DESCRIPTION OF VESSEL

SCHENECTADY	242620	Tank Vessel	T2-SE-A1
Kaiser Co., Inc., Portland, Oregon		1	31 Dec., '42
War Shipping Administration		Deconhill Shipping Company	

EXTENT OF WELDING

Hull all welded
No inner bottom

Yes						Yes	
Yes		Yes		-		Yes	
Yes		Yes		-		Yes	
Yes		Yes		-		Yes	

CIRCUMSTANCES SURROUNDING FAILURE

16 Jan., 1943	2250 PM	Tied up at fitting out pier, Swan Island	
0	-	8'-4"	17'-0"
Still Water	Clear	No waves	
Light	West wind	26° F	40°F

DESCRIPTION OF FAILURE

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 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.

Broke in two

DISPOSITION OF VESSEL

Vessel repaired and put in service.

Figure 14.

23

The Design and Methods of Construction of Welded Steel Merchant Vessels

REPORT OF STRUCTURAL FAILURE OF INSPECTED VESSEL
UNITED STATES COAST GUARD
NAVCG-2757

This report includes all available information up to: 1 Apr., 1944 *(Date)*

DESCRIPTION OF VESSEL

NAME	OFFICIAL NO.	TYPE *(Dry Cargo, Passenger, etc.)*	M.C. DESIGN
SCHENECTADY	242620	Tank Vessel	T2-SE-A1

BUILDER	BUILDER'S HULL NO.	DATE COMPLETED
Kaiser Co., Inc., Portland, Oregon	1	31 Dec., '43

OWNER	OPERATOR
War Shipping Administration	Deconhill Shipping Company

EXTENT OF WELDING

Hull all welded
No inner bottom

Yes	SIDE SHELL SEAMS					Yes	DECK SEAMS
Yes	SIDE SHELL BUTTS	Yes	BOTTOM SEAMS	-	INNER BOTTOM SEAMS	Yes	DECK BUTTS
Yes	FRAMES TO SIDE SHELL	Yes	BOTTOM BUTTS	-	INNER BOTTOM BUTTS	Yes	BEAMS TO DECK
Yes	BULKHEADS	Yes	FLOORS TO SHELL	-	FLOORS TO INNER BOTTOM	Yes	DECK TO SHELL

CIRCUMSTANCES SURROUNDING FAILURE
(Attach all available details of ship's loading)

DATE OF FAILURE	TIME	SHIP'S LOCATION
16 Jan., 1943	2230 PWT	Tied up at fitting out pier, Swan Island

SHIP'S SPEED | COURSE | DRAFT FWD. | DRAFT AFT

(Include sketch of fracture showing starting point and relative location of decks and other structural features)

APPARENT STARTING POINT

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.

CLASSIFICATION OF FAILURE
Broke in two

DISPOSITION OF VESSEL
(Repaired, lost, etc.)

Vessel repaired and put in service.

SIGNED *(Name and Title)*	DISTRICT

(P)

Figure 14.

(Include sketch of fracture showing starting point and relative location of decks and other structural features)

APPARENT STARTING POINT

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 bilge port and starboard. The deck side shell, longitudinal bulkheads and bottom girders fractured. Only the bottom plating held. The vessel jackknifed and the center portion rose so that no water entered the hull.

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

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.

Vessel repaired and put in service.

repaired, lost, etc.)

Vessel repaired and put in service.

SIGNED (Name and Title)	DISTRICT

(P) 29

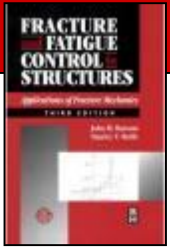
701292—47—3

Figure 14.

I. Conclusions

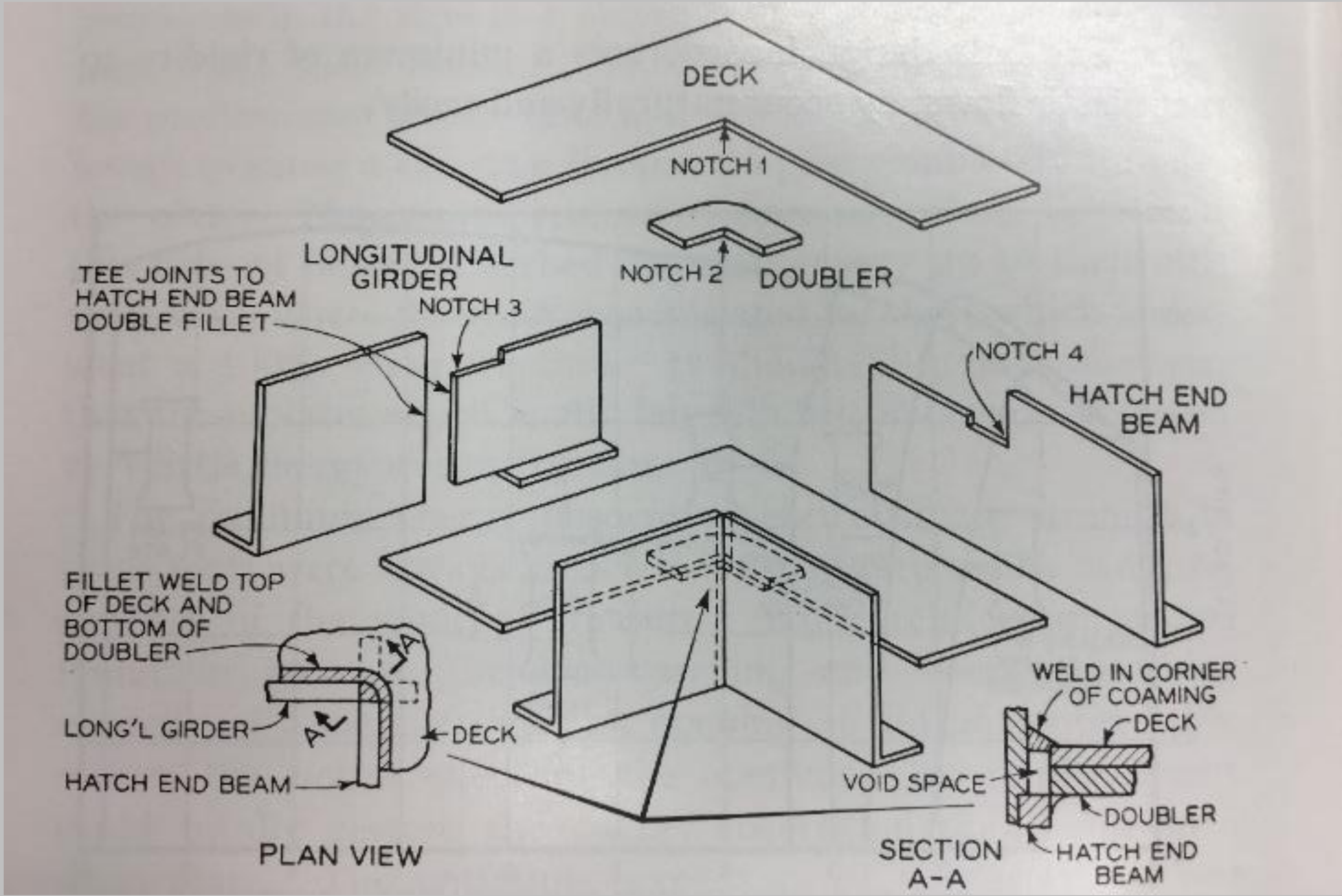
The Board concludes that:

(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.

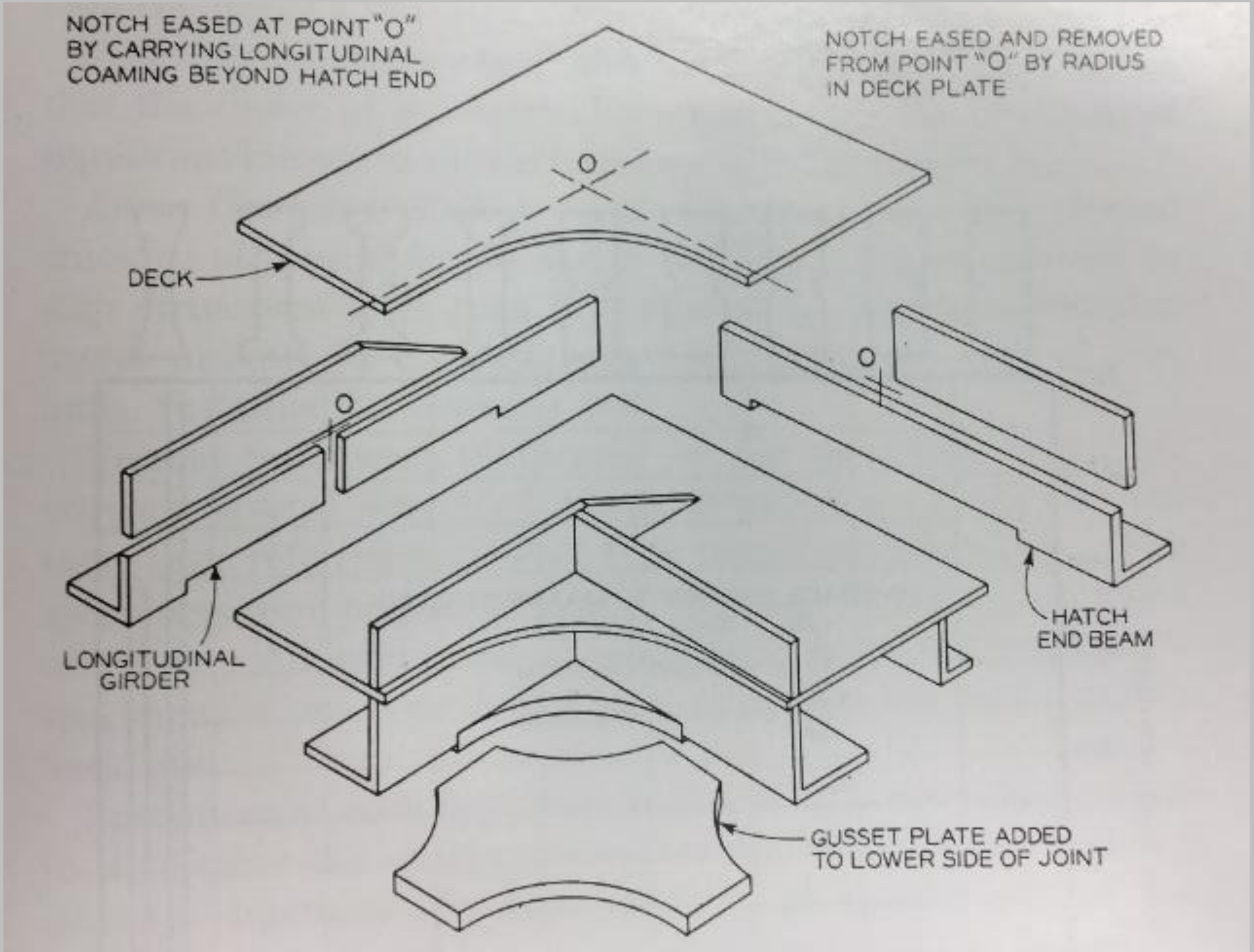


The majority of the fractures in the Liberty ships started at square hatch corners or square cutouts at the top of the sheer strake. Design changes involved rounding and strengthening of the hatch corners, removing square cutouts in the sheer strake, and adding riveted crack arresters in various locations led to immediate reductions in the incidence of failures.

Control of Steel Construction to Avoid Brittle Failure



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

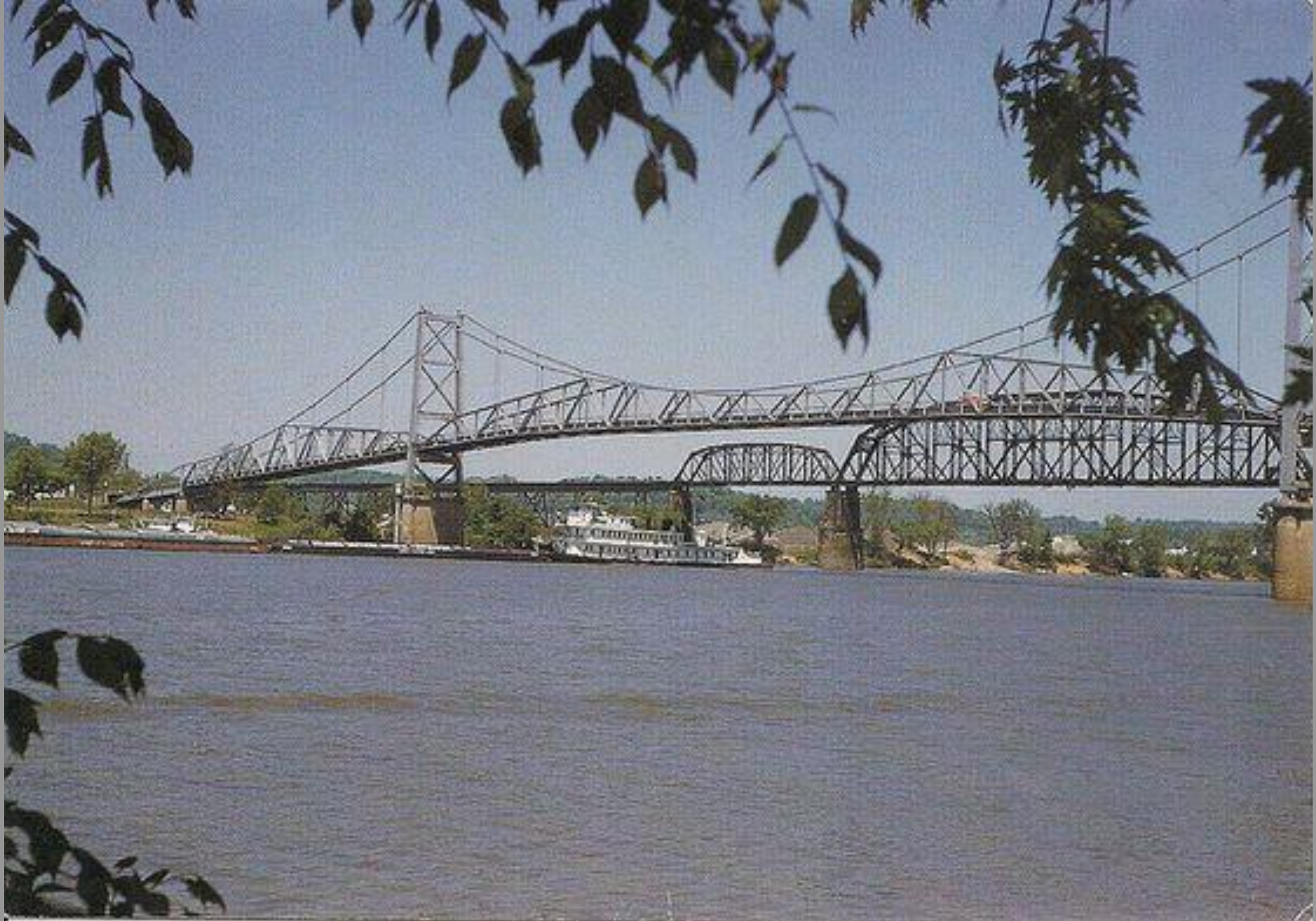
G. R. Irwin

Fracture control considerations for bridge steels

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.

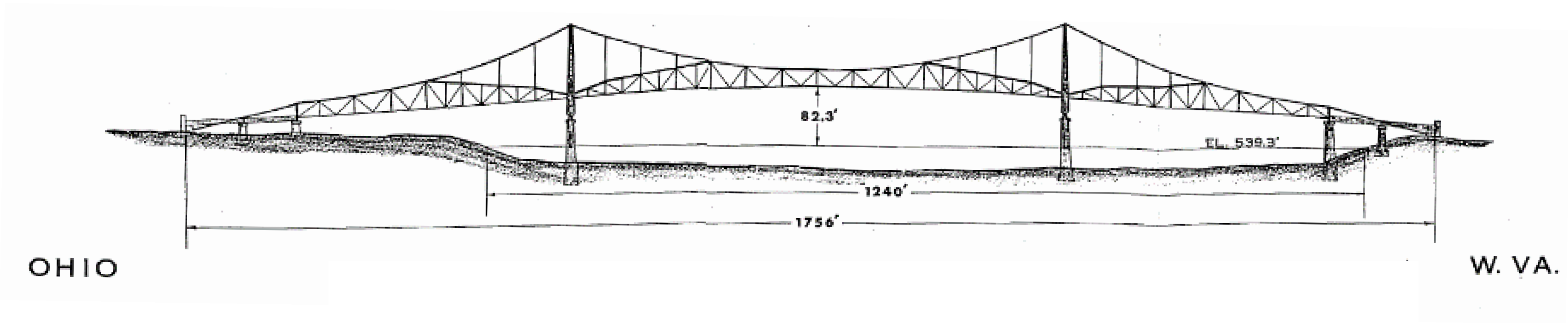
Brittle Studies Involving Brittle Fracture:

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

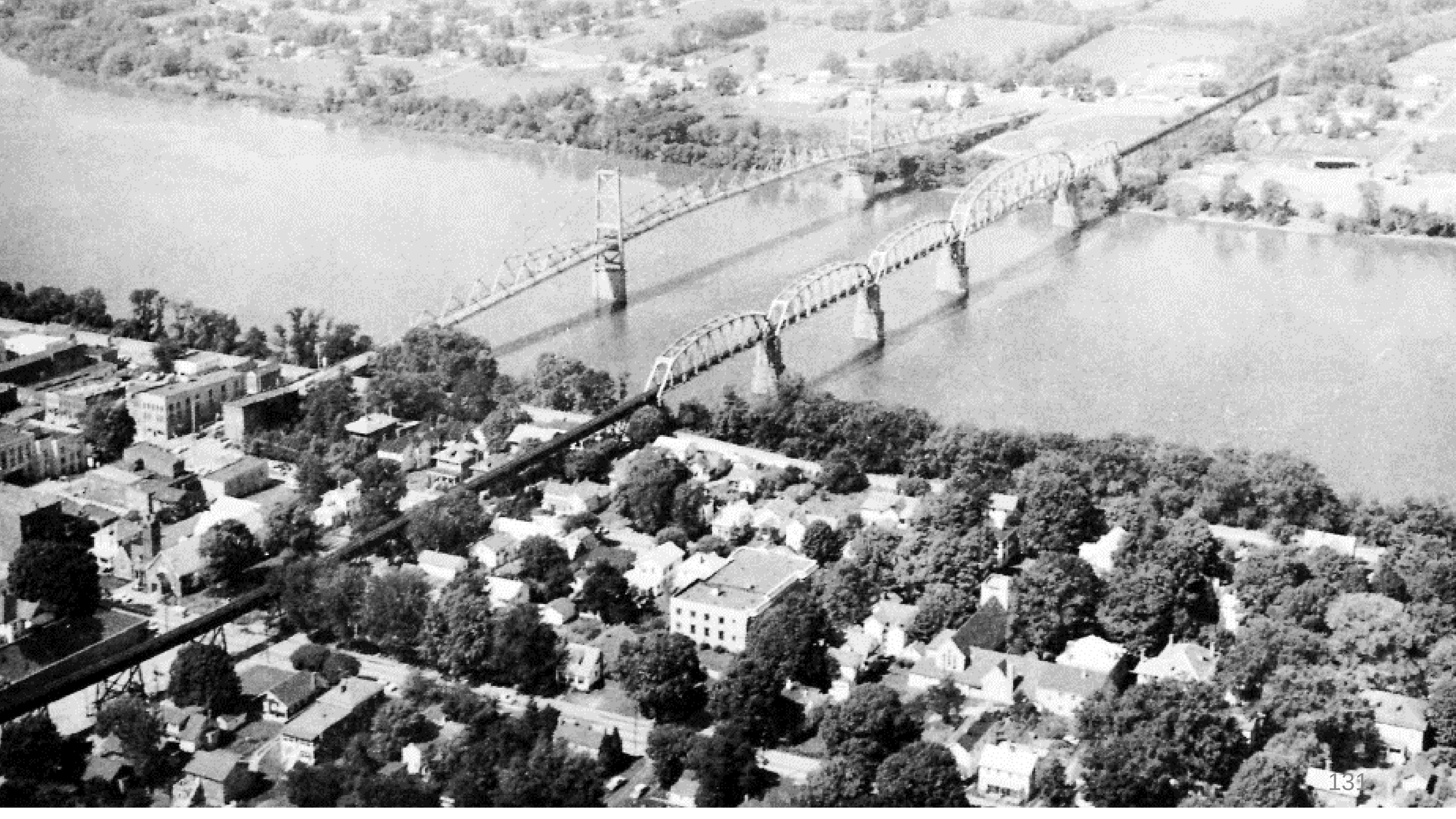


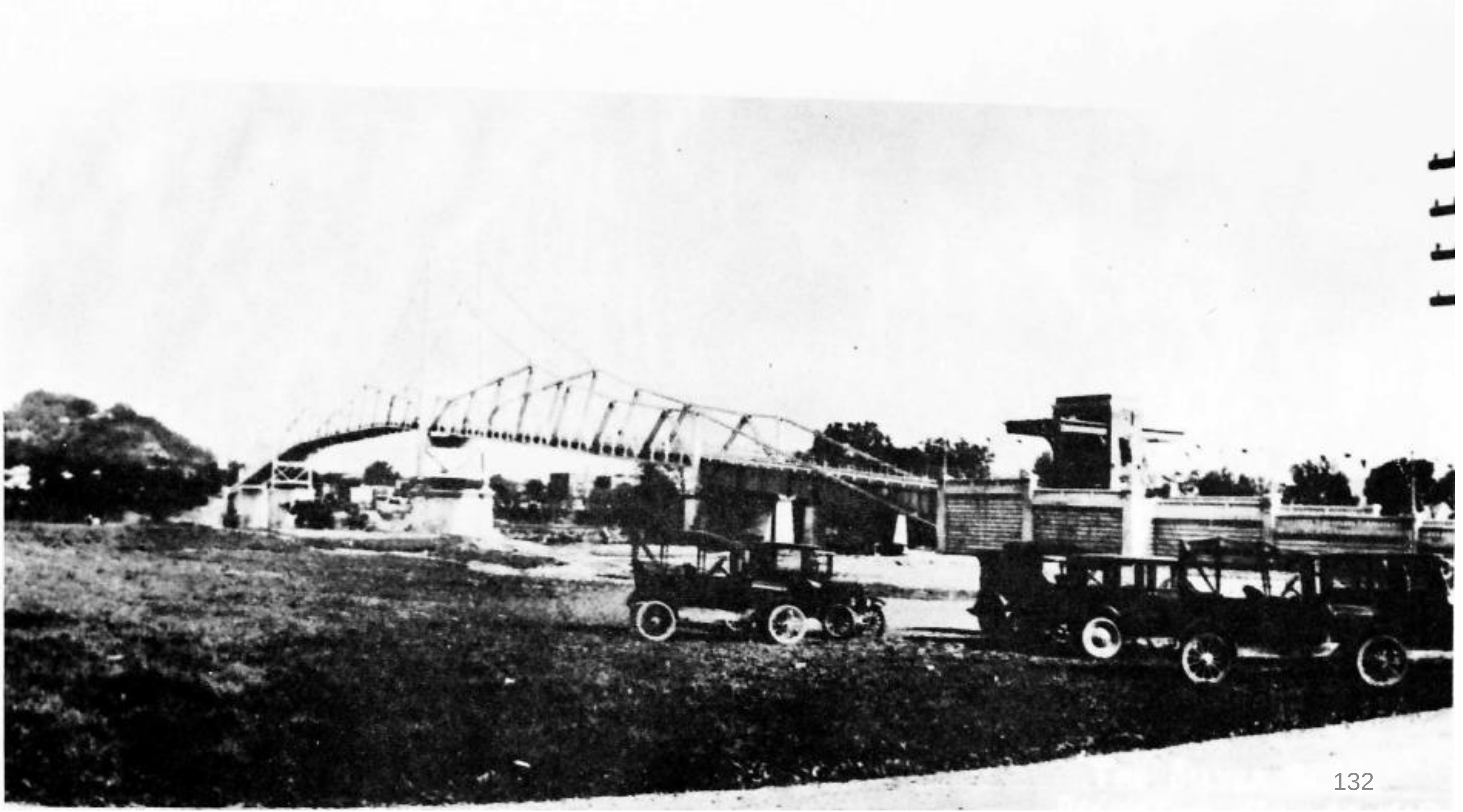
Silver Bridge Summary

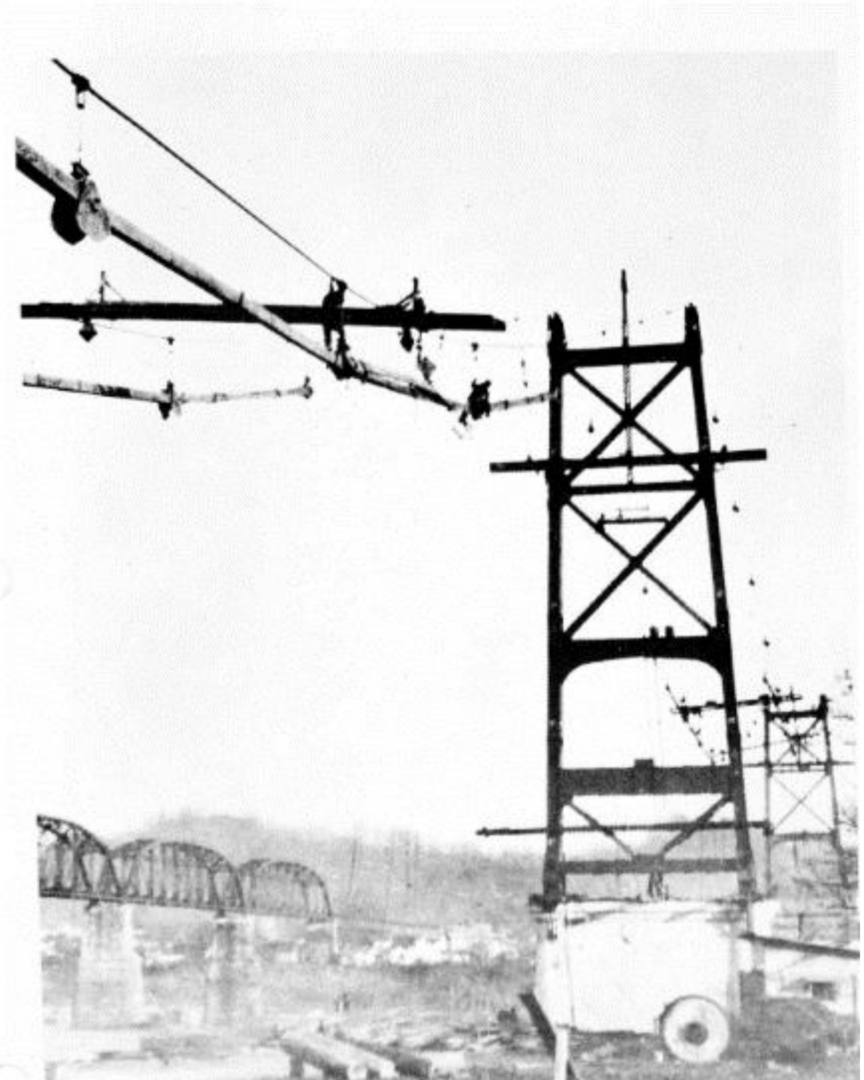
- Opened to traffic May 1928
- Collapsed December 1967
- Eyebar suspension bridge
- 30 ^{o}F at time of collapse

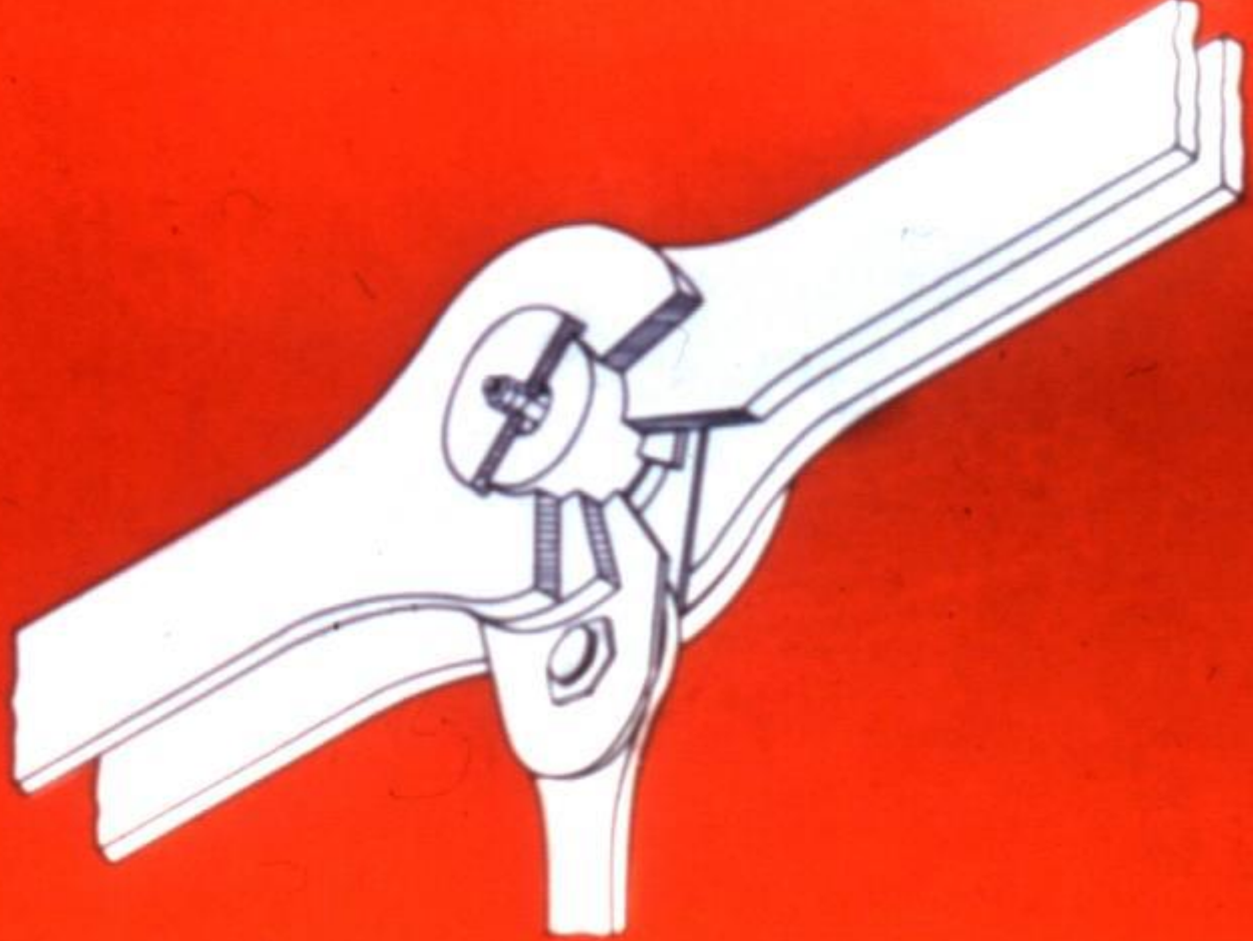


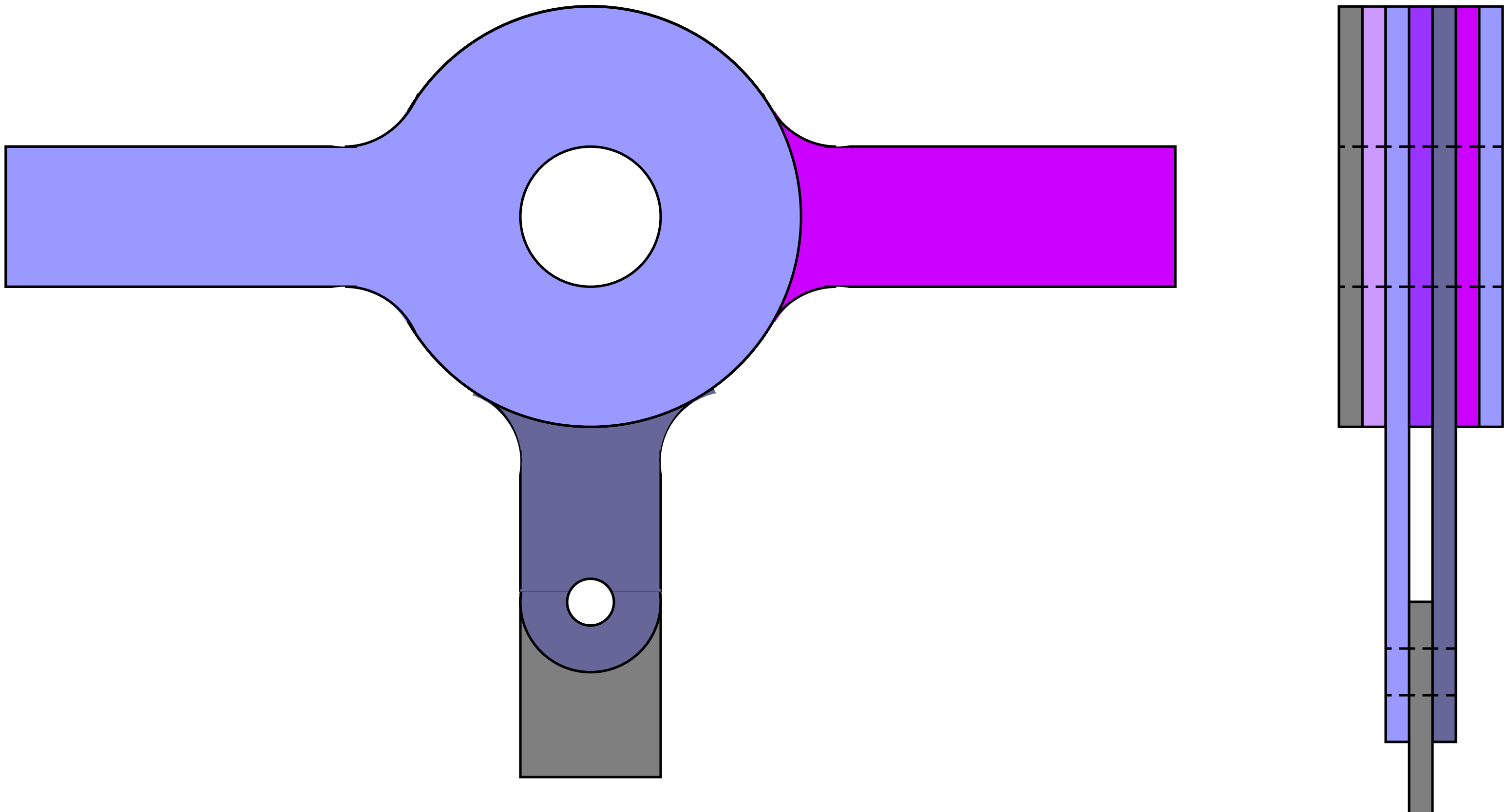
- Total length: 1756 feet
- Main span: 700 feet
- River width: 1240 feet

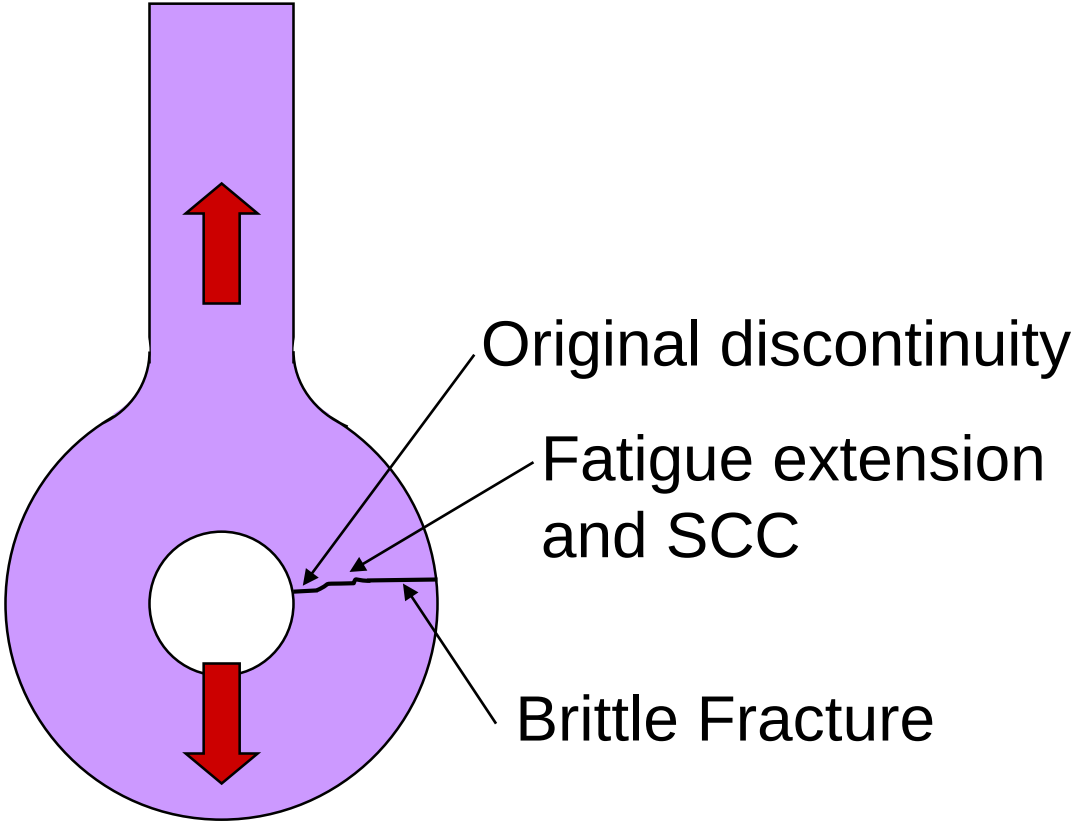


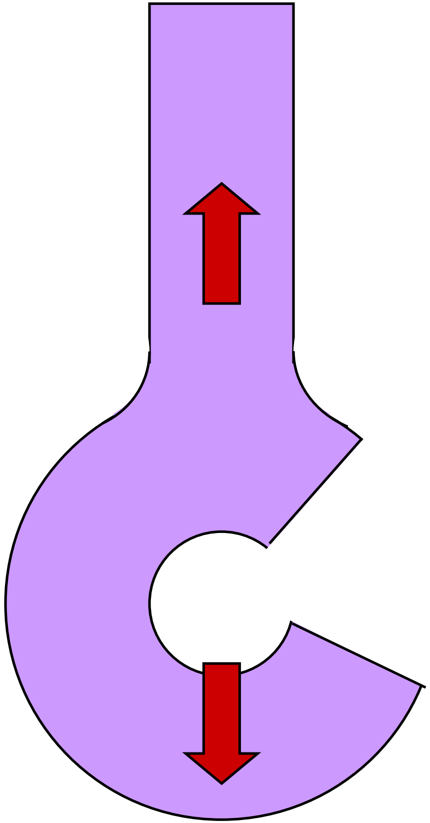


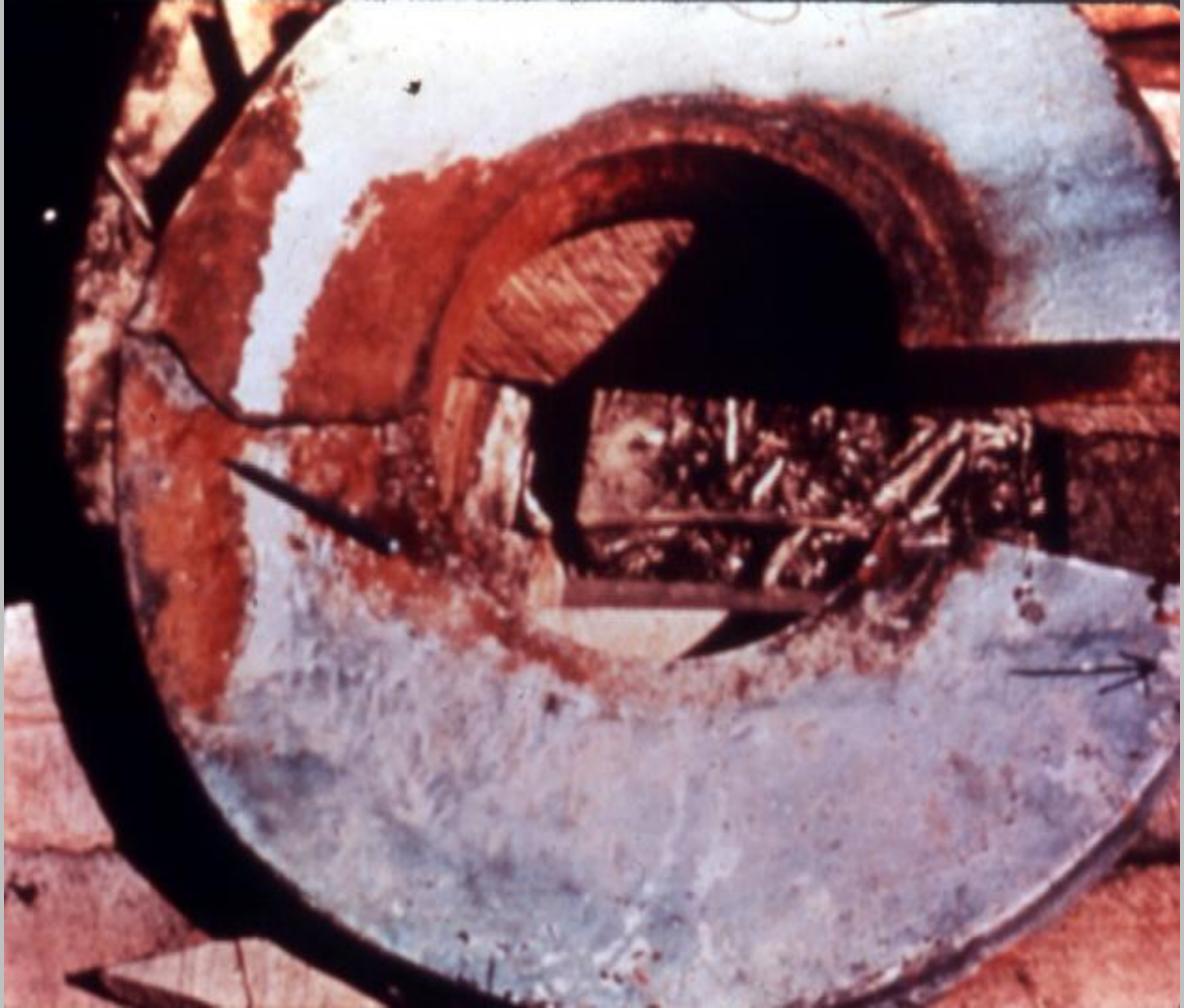


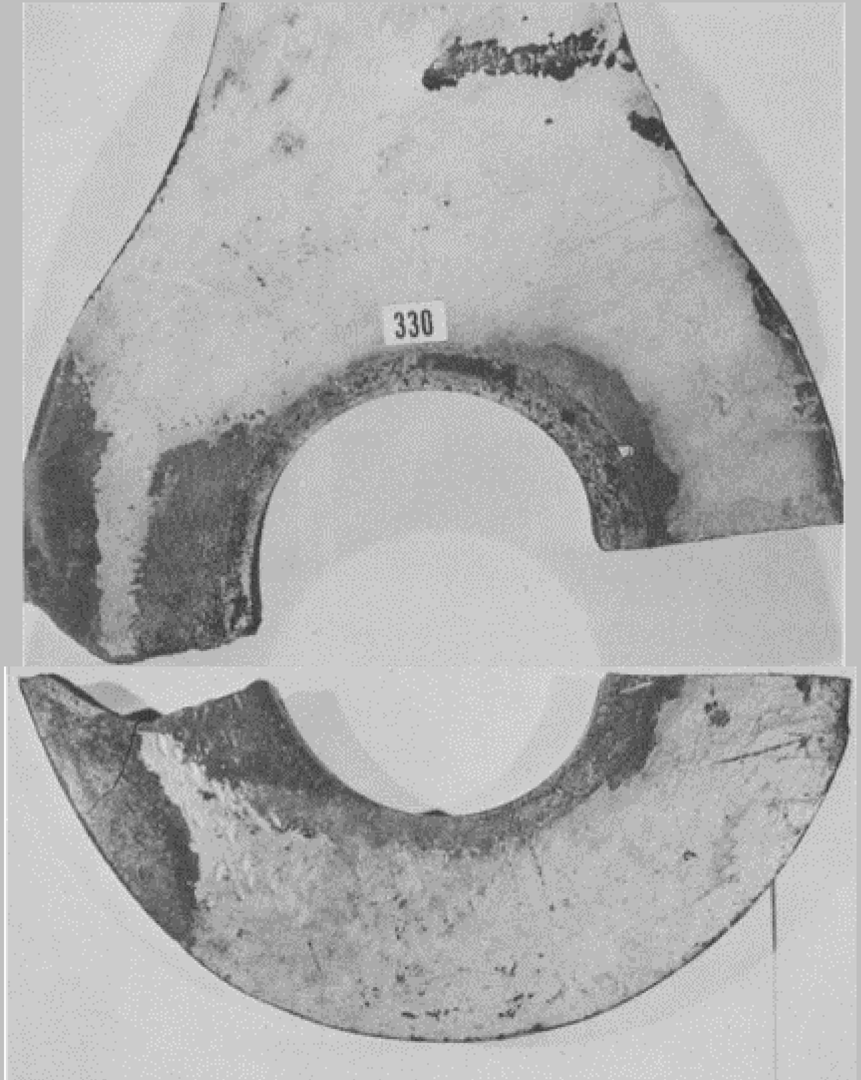





Original discontinuity
Fatigue extension and SCC
Brittle Fracture



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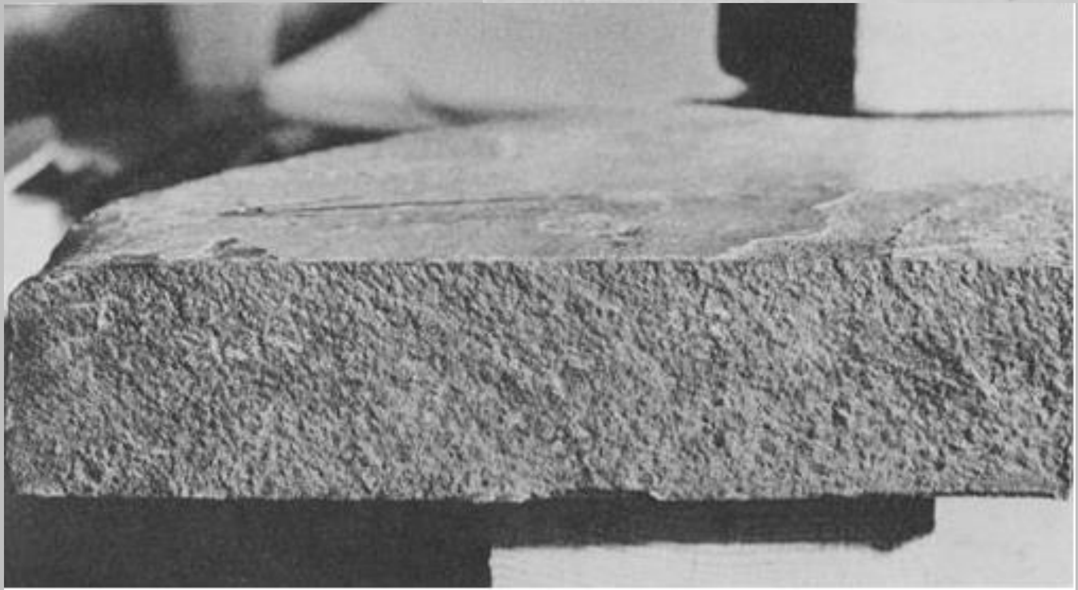
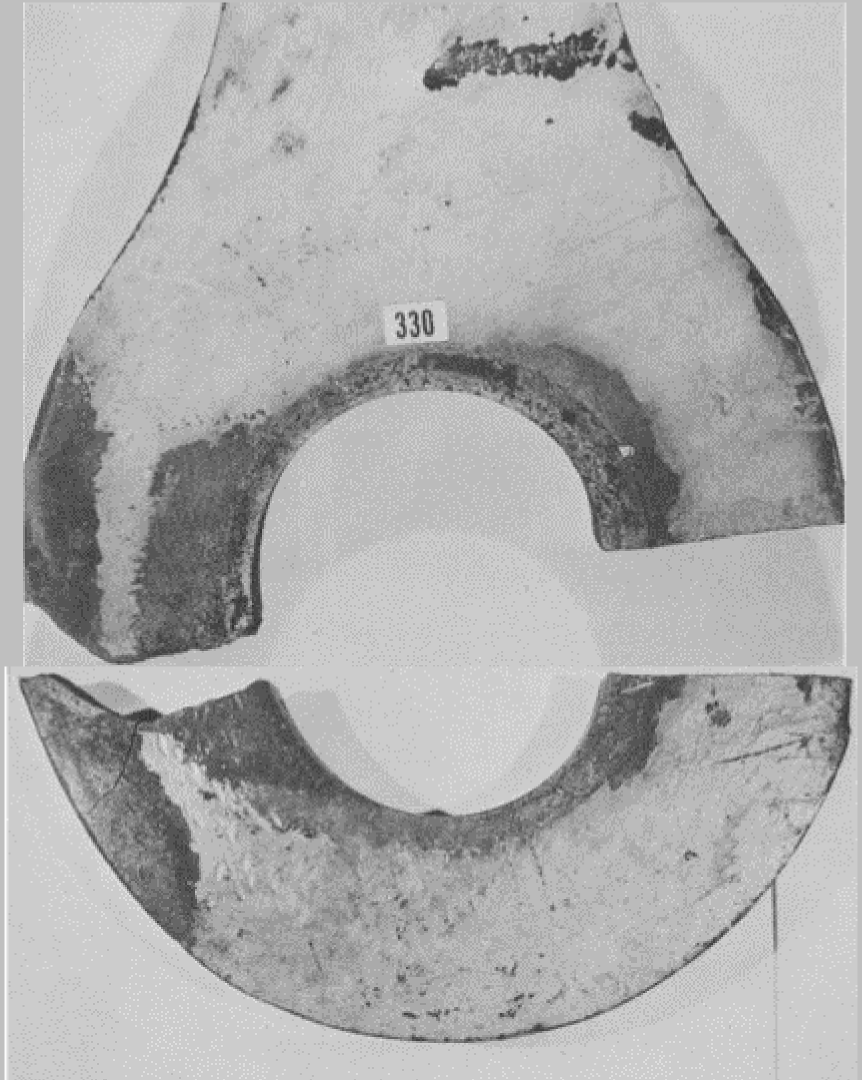
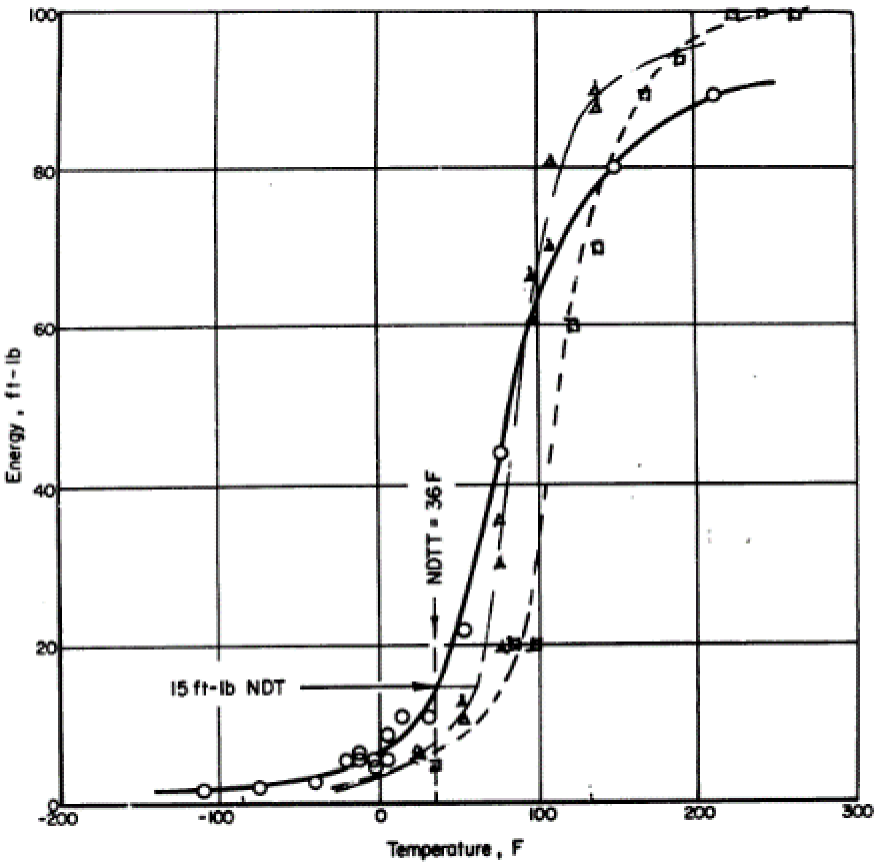


Question



Gensamer: STRENGTH OF METALS UNDER COMBINED STRESS

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.



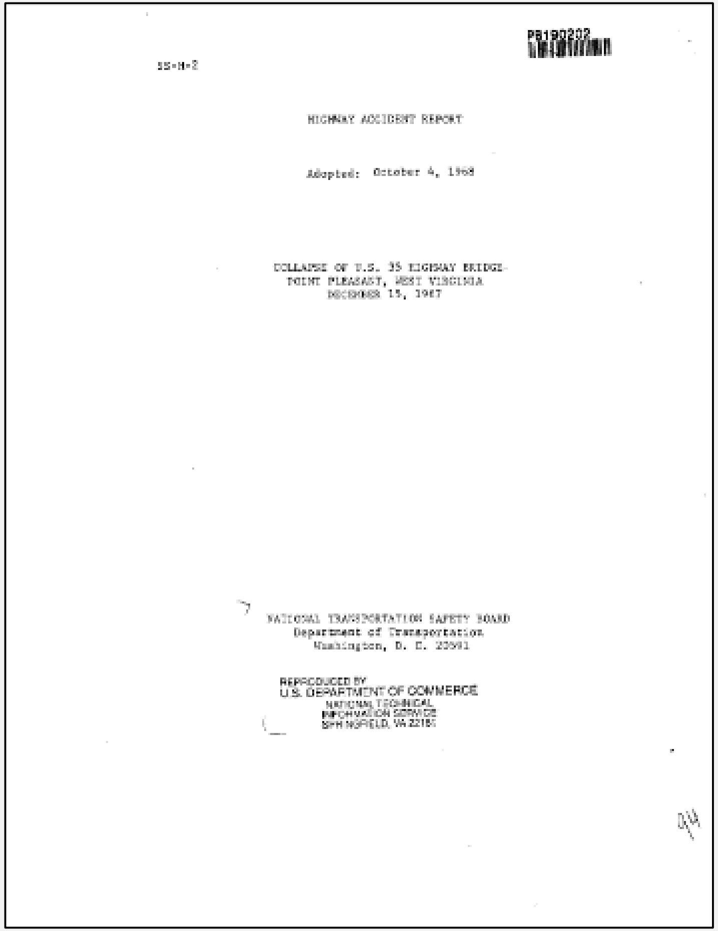
○ Battelle Mem. Inst., Chain bent post LO-UO, N (solid curve)

□ 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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SS-H-2

HIGHWAY ACCIDENT REPORT

Adopted: October 4, 1968

COLLAPSE OF U.S. 35 HIGHWAY BRIDGE
POINT PLEASANT, WEST VIRGINIA
DECEMBER 15, 1967

NATIONAL TRANSPORTATION SAFETY BOARD
Department of Transportation
Washington, D. C. 20591

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.

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J. A. Bennett[1] *and Harold Mindlin*[2]

Metallurgical Aspects of the Failure of the Point Pleasant Bridge

REFERENCE: Bennett, J. A. and Mindlin, Harold, **"Metallurgical Aspects of the Failure of the Point Pleasant Bridge,"** *Journal of Testing and Evaluation*, JTEVA, Vol. 1, No. 2, March 1973, pp. 152–161.

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 ⅛ 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.

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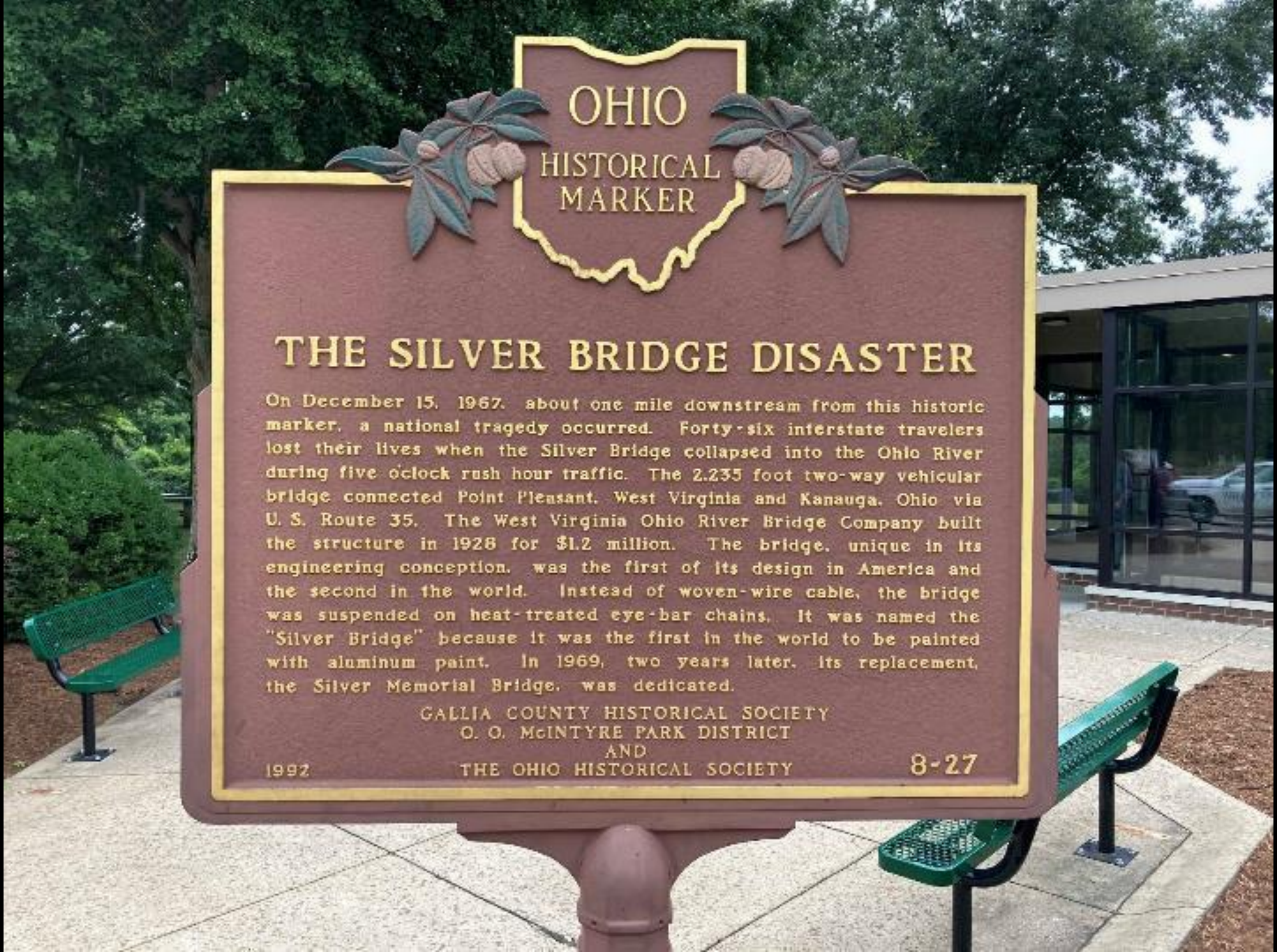
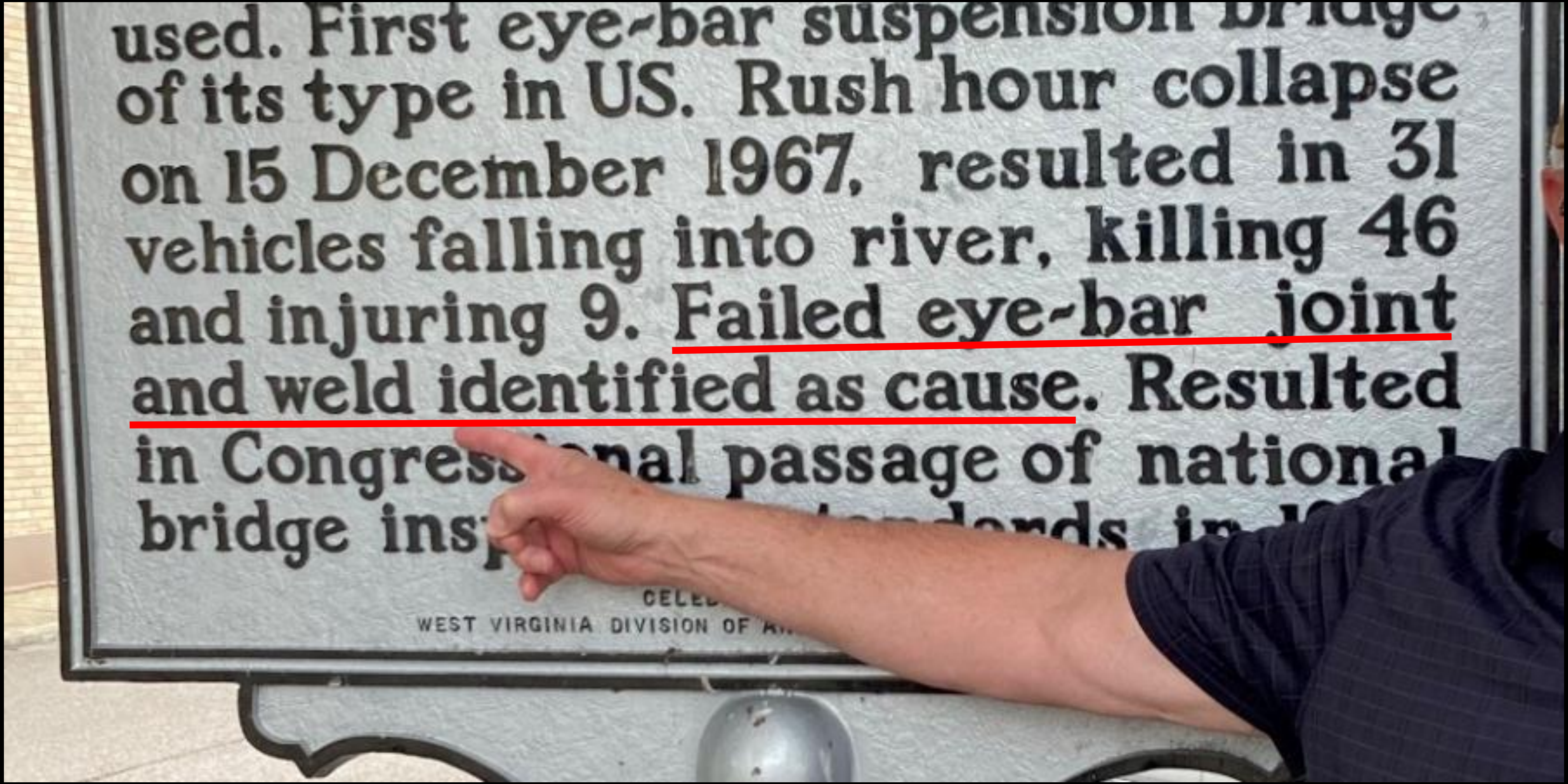

OHIO
HISTORICAL
MARKER
THE SILVER BRIDGE DISASTER
On December 15, 1967, about one mile downstream from this historic marker, a national tragedy occurred. Forty-six interstate travelers lost their lives when the Silver Bridge collapsed into the Ohio River during five o'clock rush hour traffic. The 2,235 foot two-way vehicular bridge connected Point Pleasant, West Virginia and Kanauga, 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


STATE OF WEST VIRGINIA
SILVER BRIDGE COLLAPSE
Constructed in 1928, connected Point Pleasant and Kanauga, OH. Name credited to aluminum colored paint used. First eye-bar suspension bridge of its type in US. Rush hour collapse on 15 December 1967, resulted in 31 vehicles falling into river, killing 46 and injuring 9. Failed eye-bar joint and weld identified as cause. Resulted in Congress
nal passage of national
bridge insp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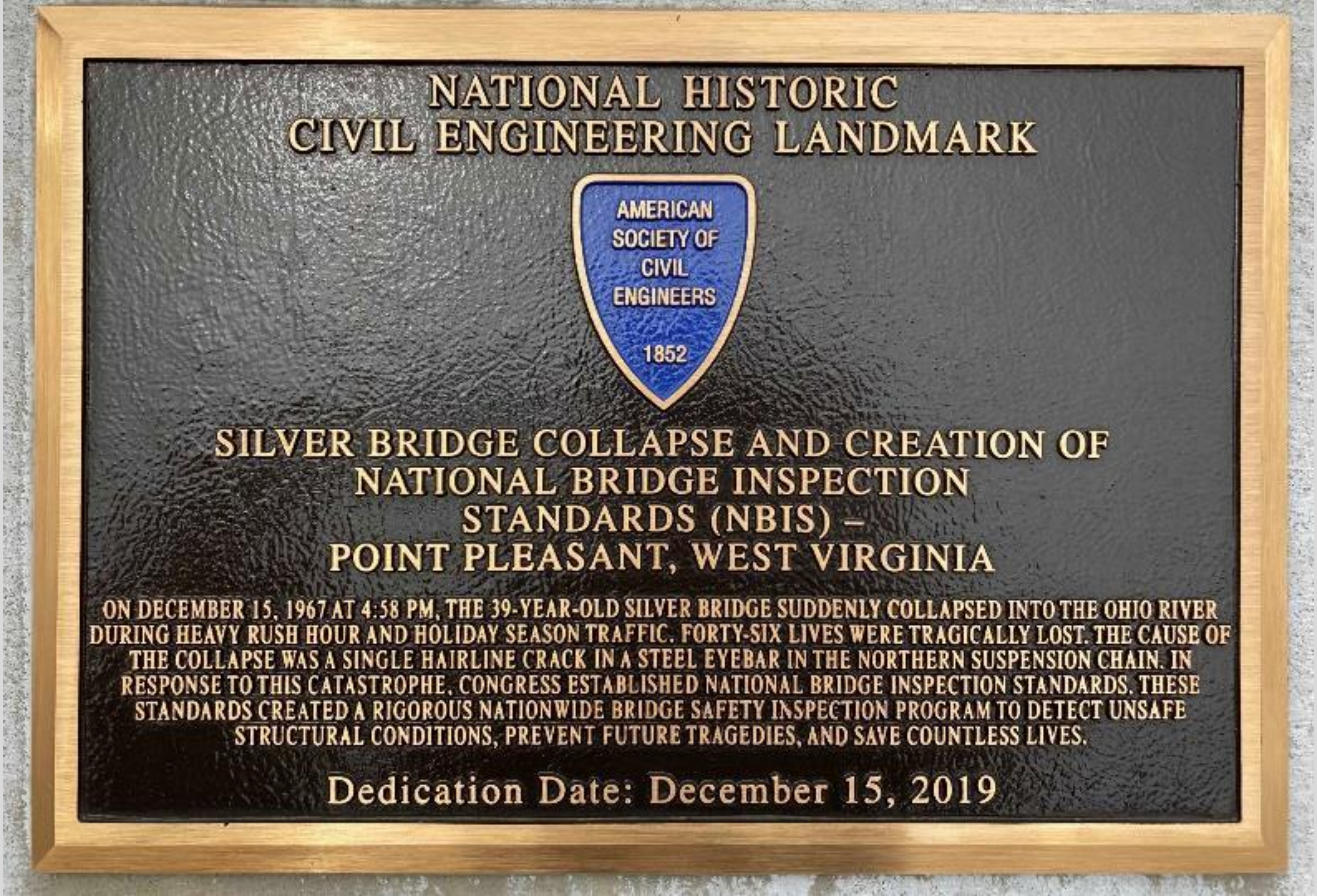
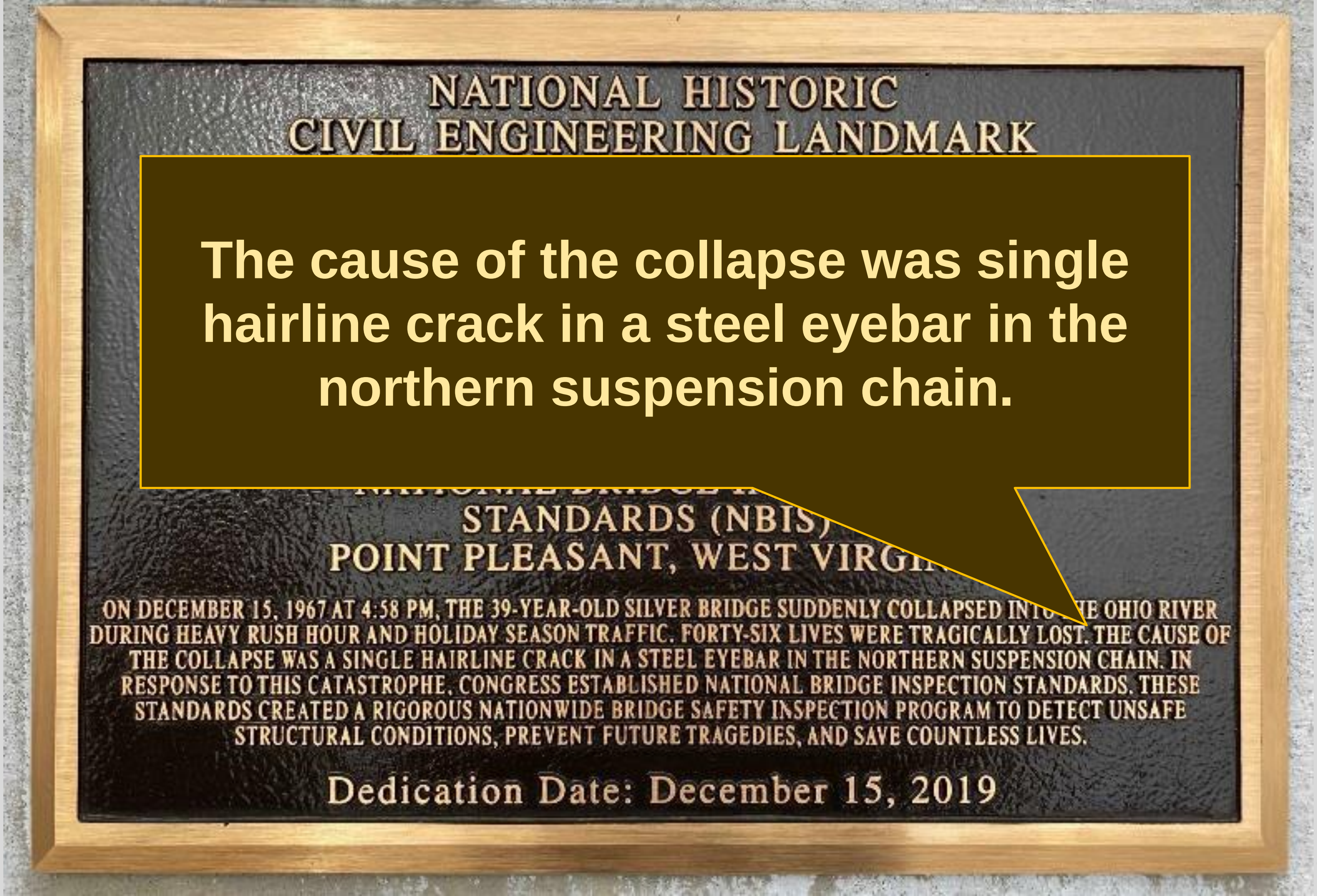
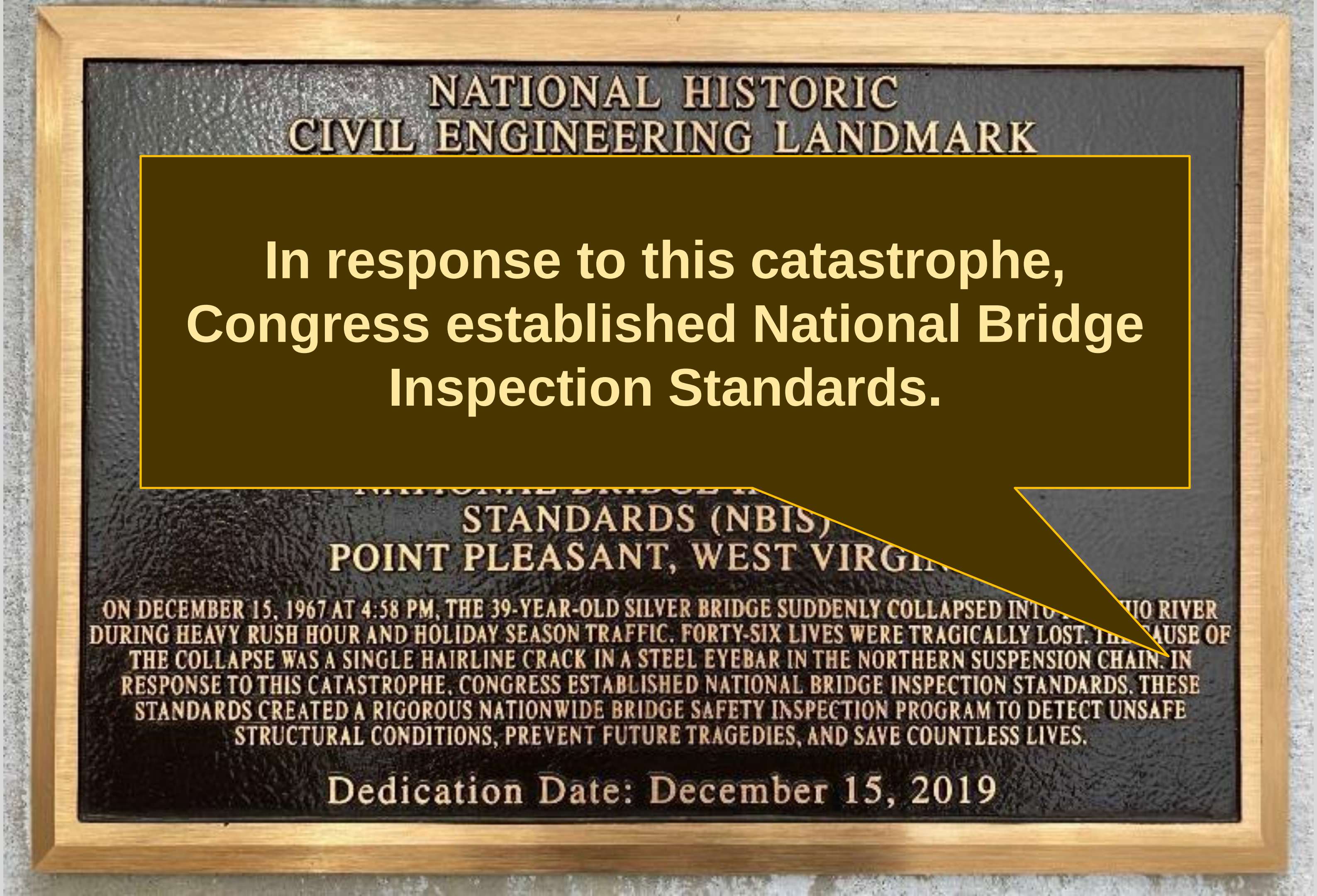
WEST VIRGINIA DIVISION OF


NATIONAL HISTORIC
CIVIL ENGINEERING LANDMARK
AMERICAN SOCIETY OF CIVIL ENGINEERS
1852
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 eyebar in the northern suspension chain.



In response to this catastrophe, Congress established National Bridge Inspection Standards.

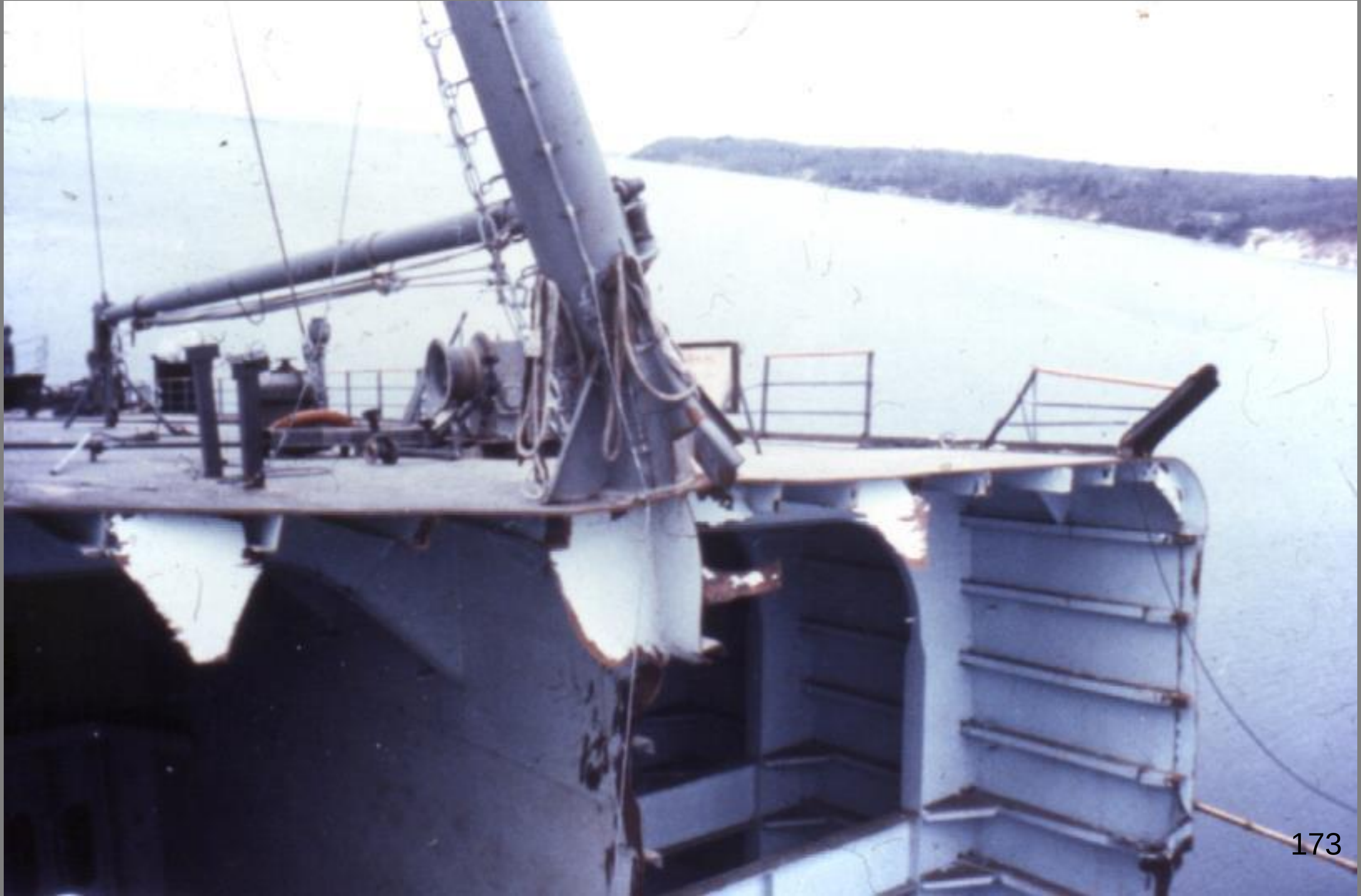
Brittle Studies Involving Brittle Fracture:

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


INGRAM
MARTHA R.INGRAM

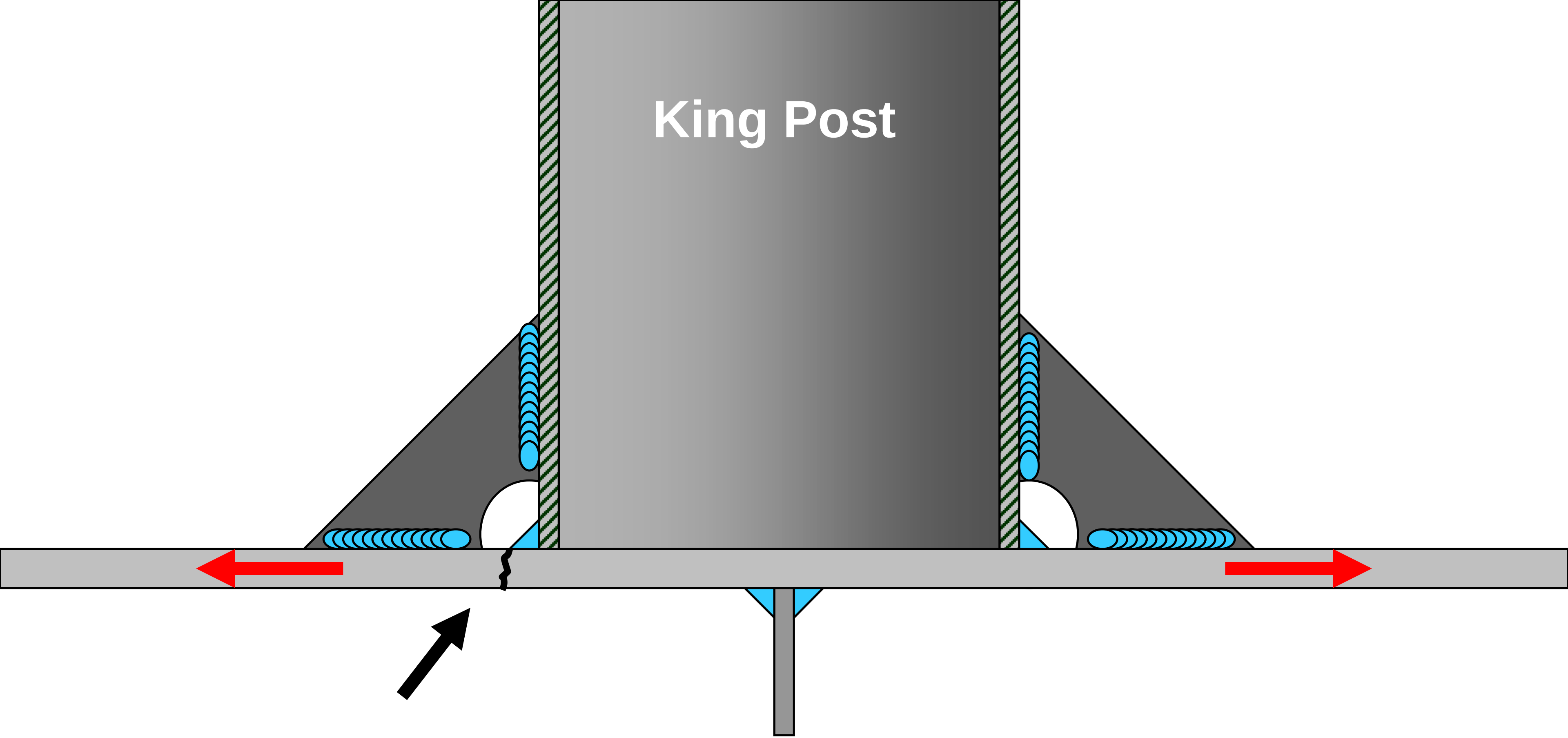
Martha R. Ingram Barge

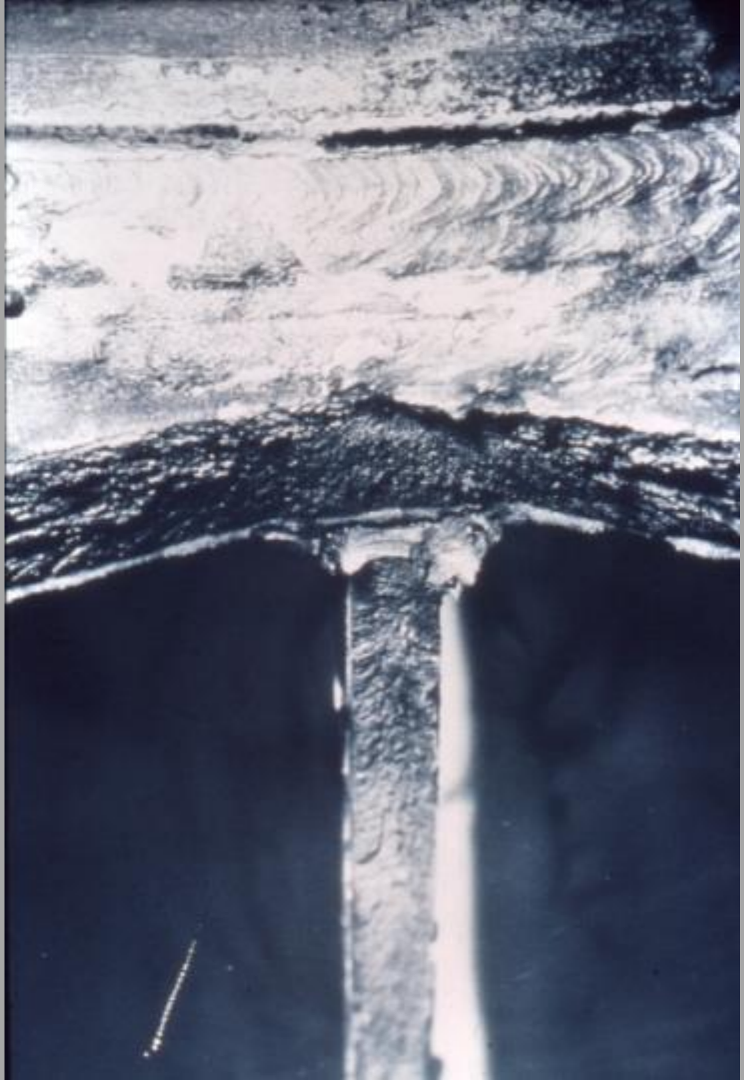
- January 10, 1972
- 584 foot [178 m]
- Air temperature 45 ^{o}F [7 ^{o}C]
- In service for 9 months

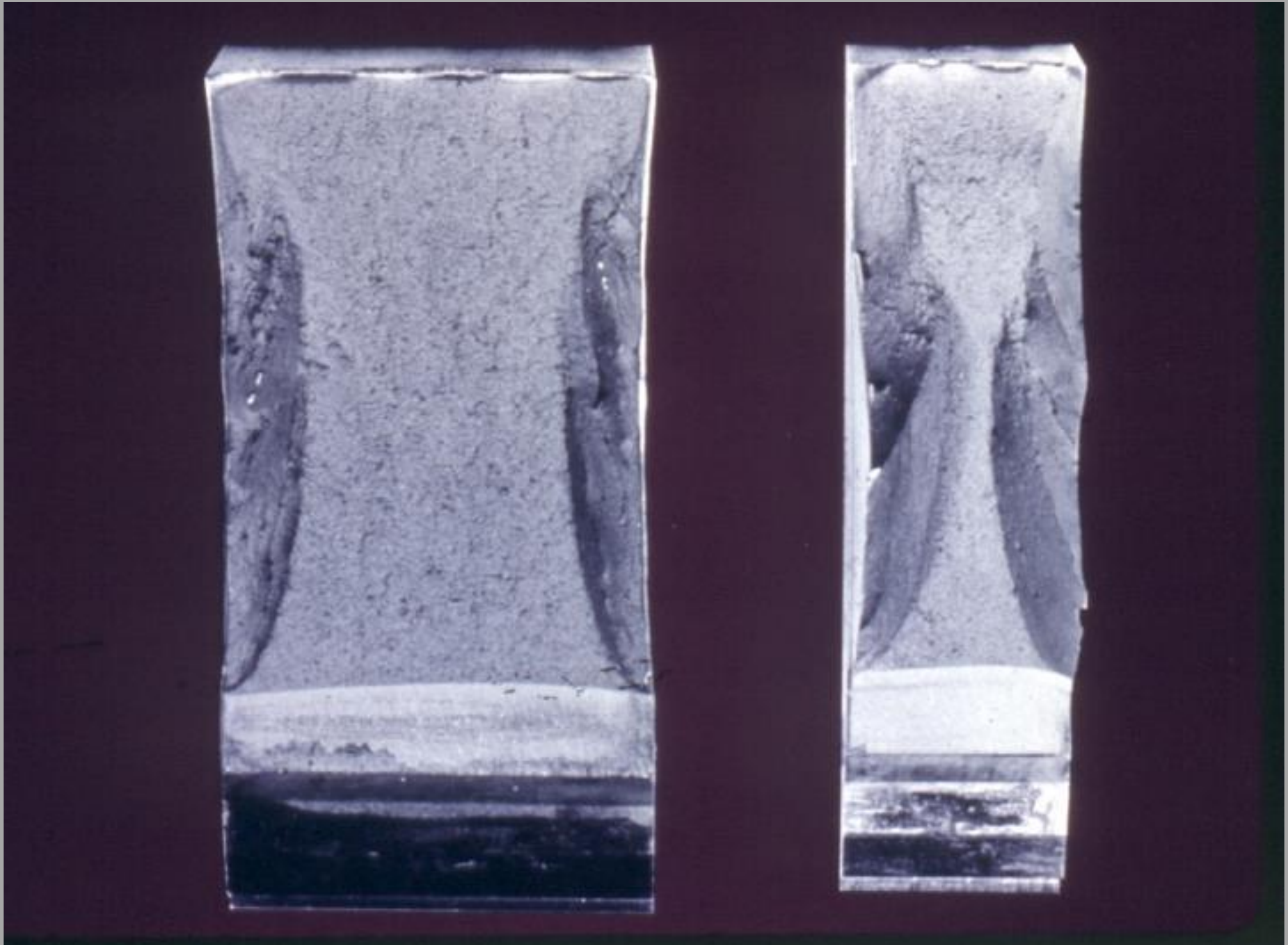


Martha R. Ingram Barge

- January 10, 1972
- 584 foot [178 m]
- Air temperature 45 ^{o}F [7 ^{o}C]
- In service for 9 months
- Unusual loading: 2.5X design load, 24 ksi [165 MPa]


King Post





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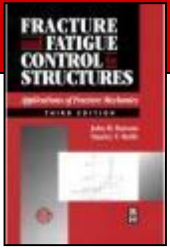
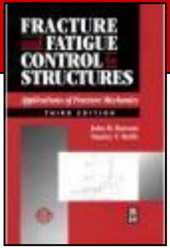
- 584 foot [178 m]
- Air temperature 45 ^{o}F [7 ^{o}C]
- In service for 9 months
- Unusual loading: 2.5X design load, 24 ksi [165 MPa]
- 55 ft-lbs [74 J] at service temperature
- No pre-existing flaws were observed



…the primary cause of failure was established to be an unusually high loading stress caused by improper ballasting at a highly constrained welded detail.

Barsom/Rolfe: FRACTURE AND FATIGUE CONTROL IN STRUCTURES



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.

Martha R. Ingram Barge

No amount of inspection would have solved this problem.

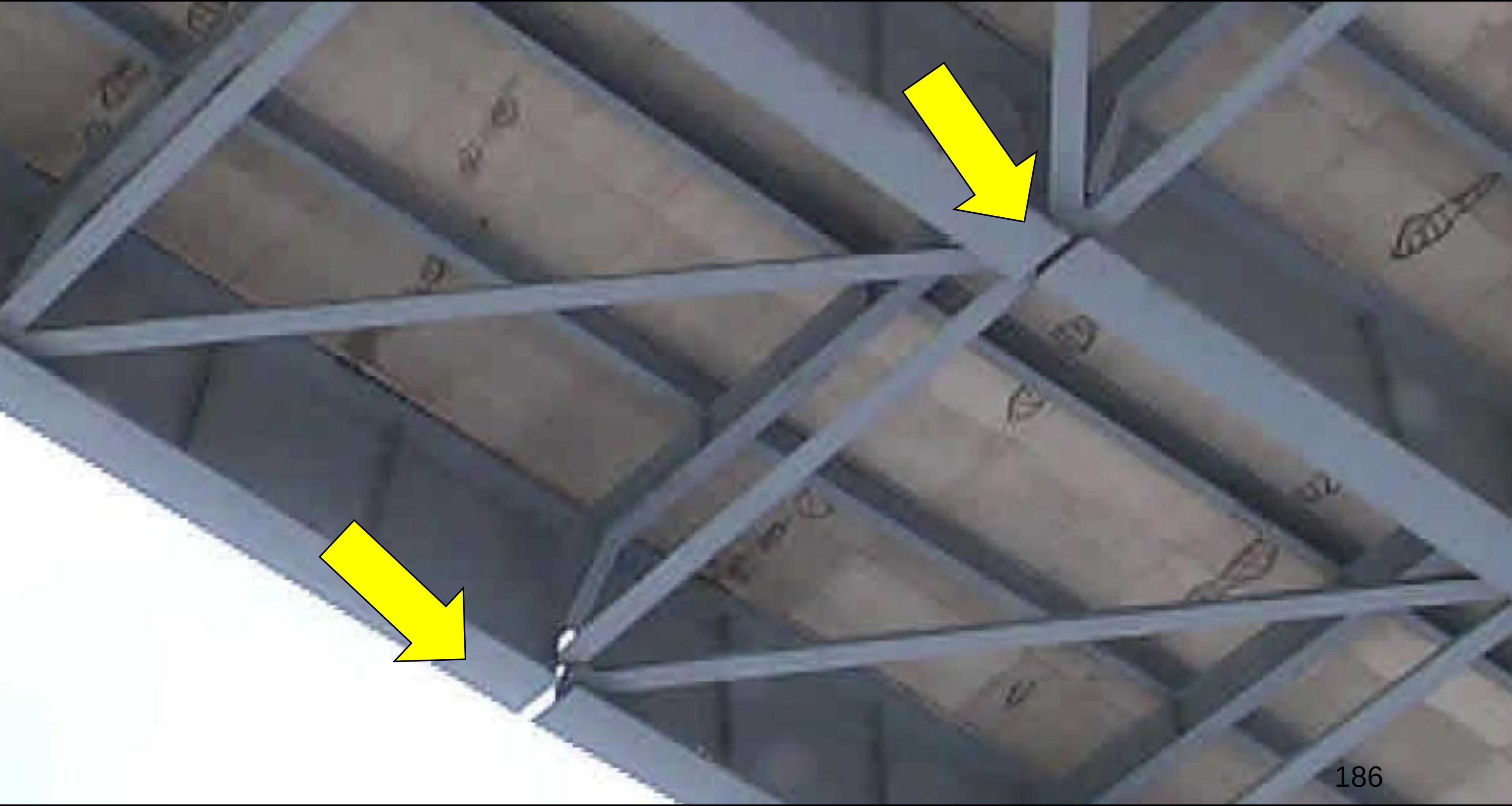
- January 10, 1972
- 584 foot [178 m]
- Air temperature 45 ^{o}F [7 ^{o}C]
- In service for 9 months
- Unusual loading: 2.5X design load, 24 ksi [165 MPa]
- 55 ft-lbs [74 J] at service temperature
- No pre-existing flaws were observed

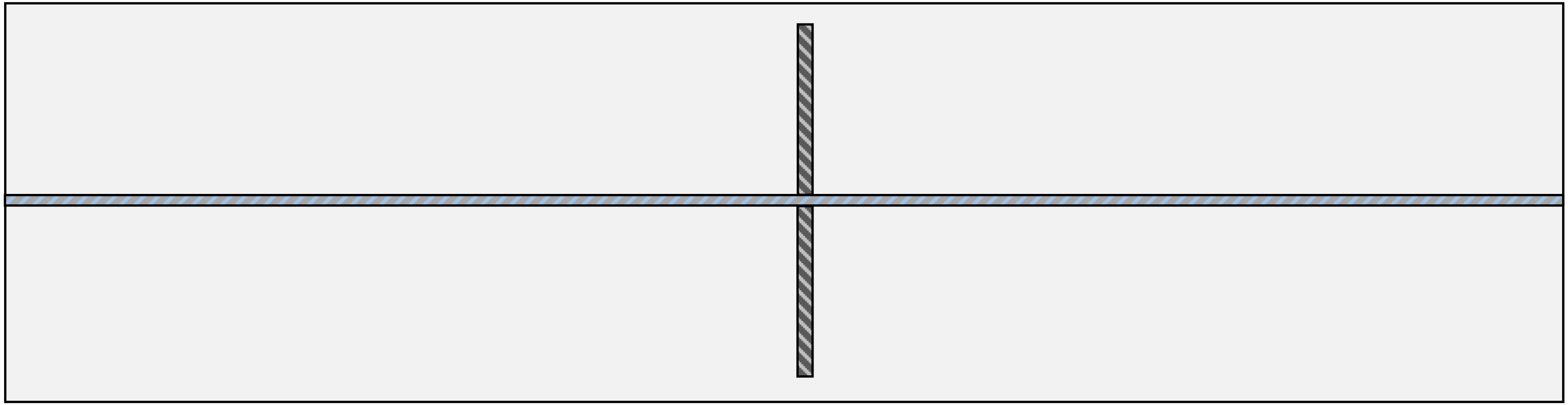
Brittle Studies Involving Brittle Fracture:

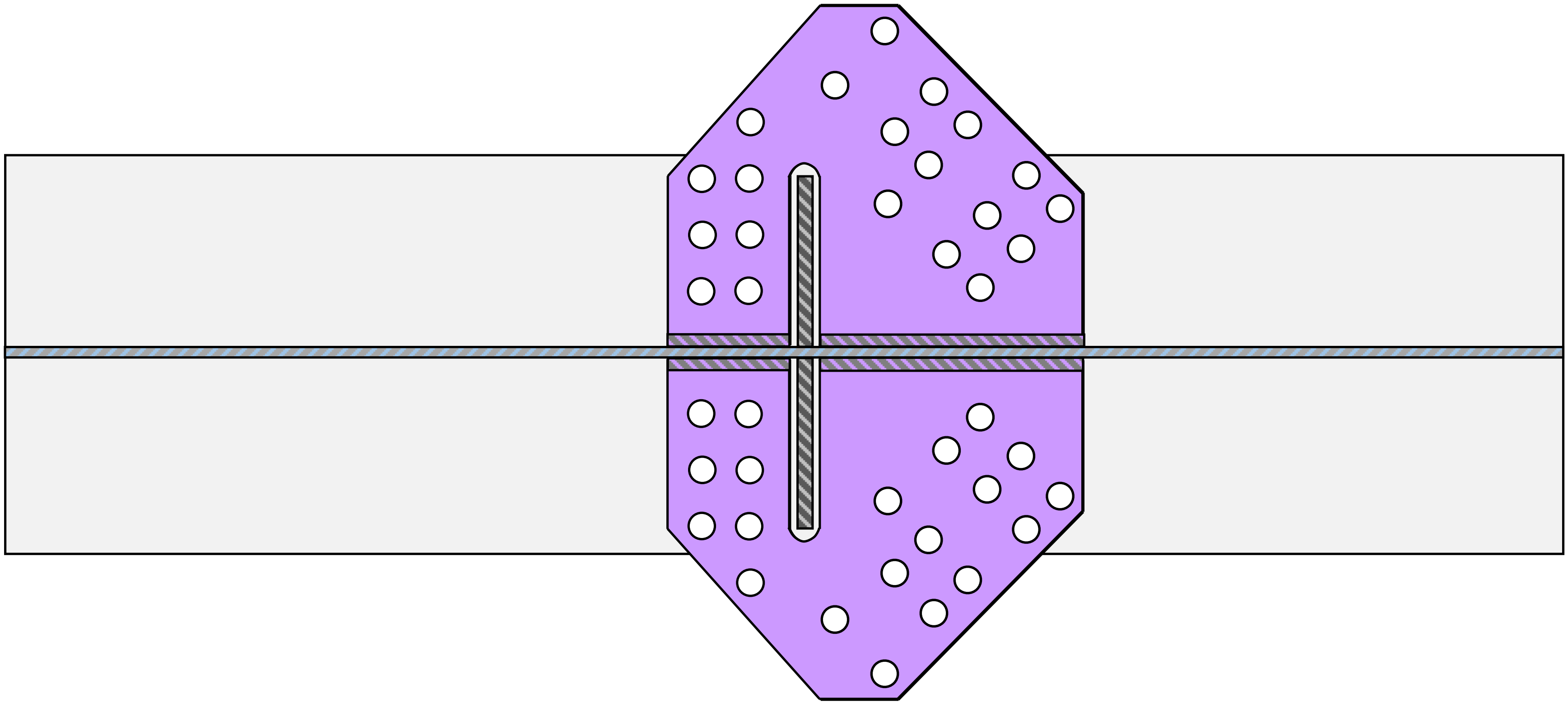
- Case 1: Liberty Ships
- Case 2: Silver Bridge
- Case 3: Ingram Barge
- Case 4: Hoan Bridge

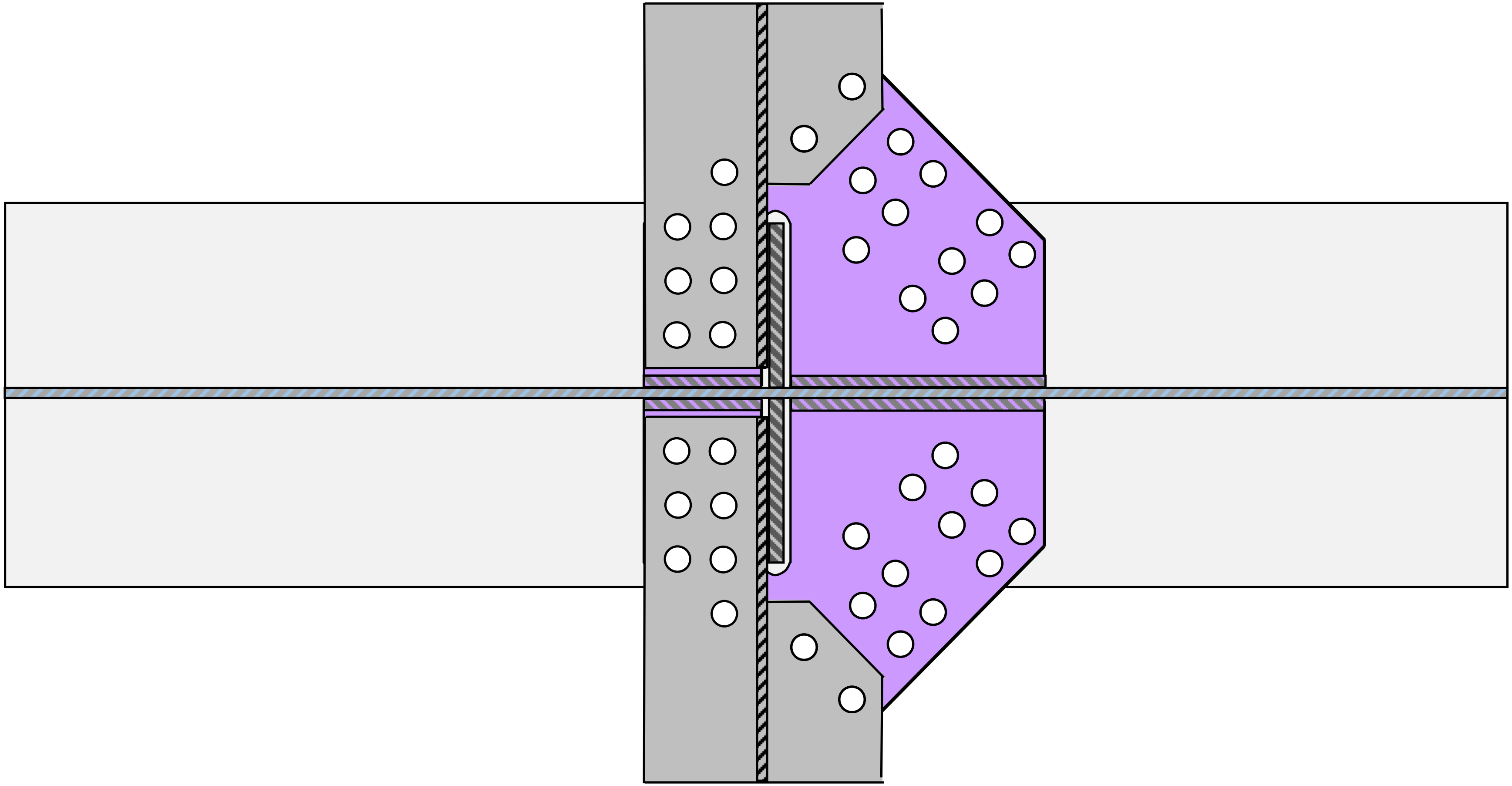
Hoan Bridge

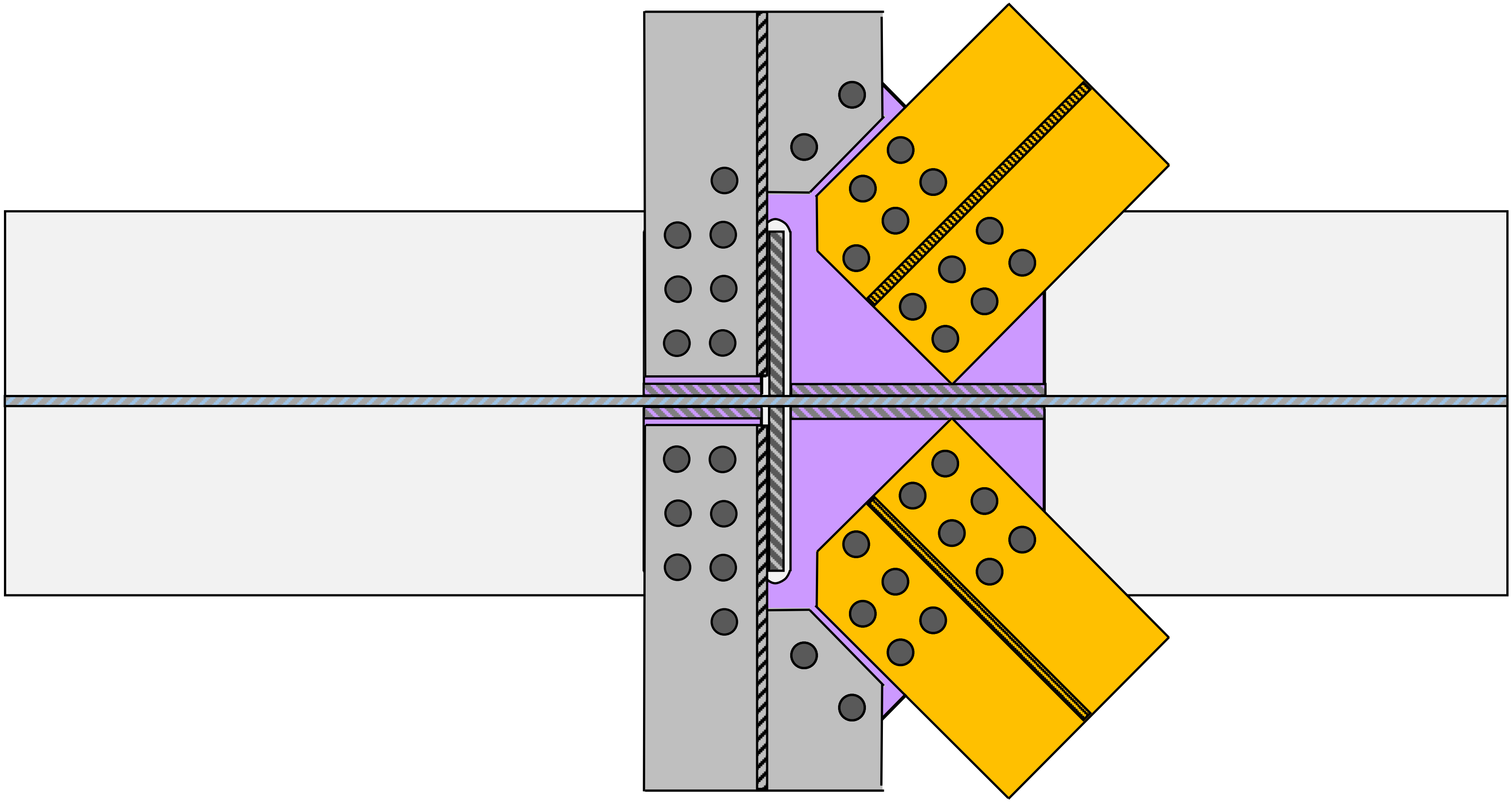
- Built 1972, Opened 1977
- “The Bridge to Nowhere”
- Tied Arch
- Total length: 1.9 miles (3058 m)
- Longest span: 607 feet (185 m)
- December 2000: major fracture discovered

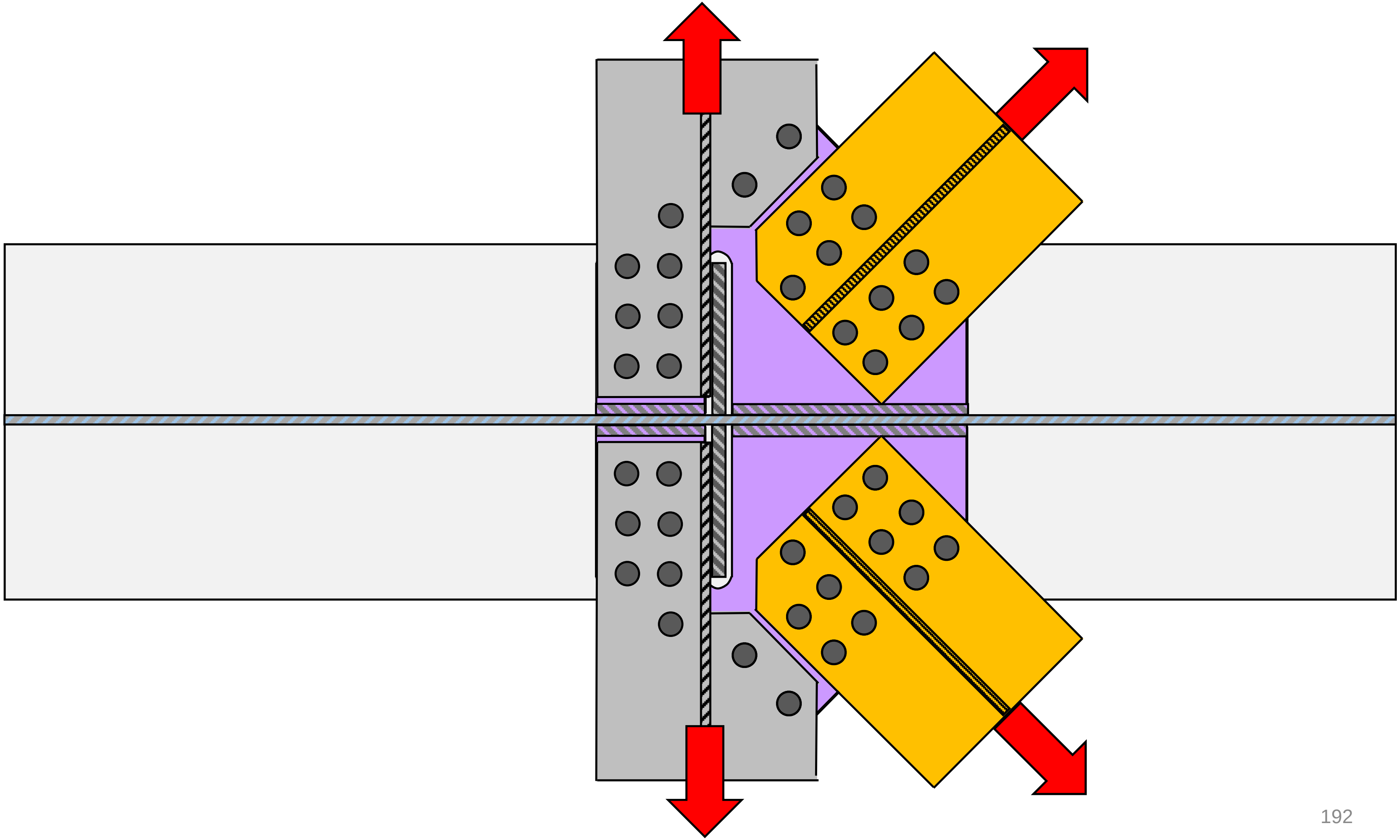


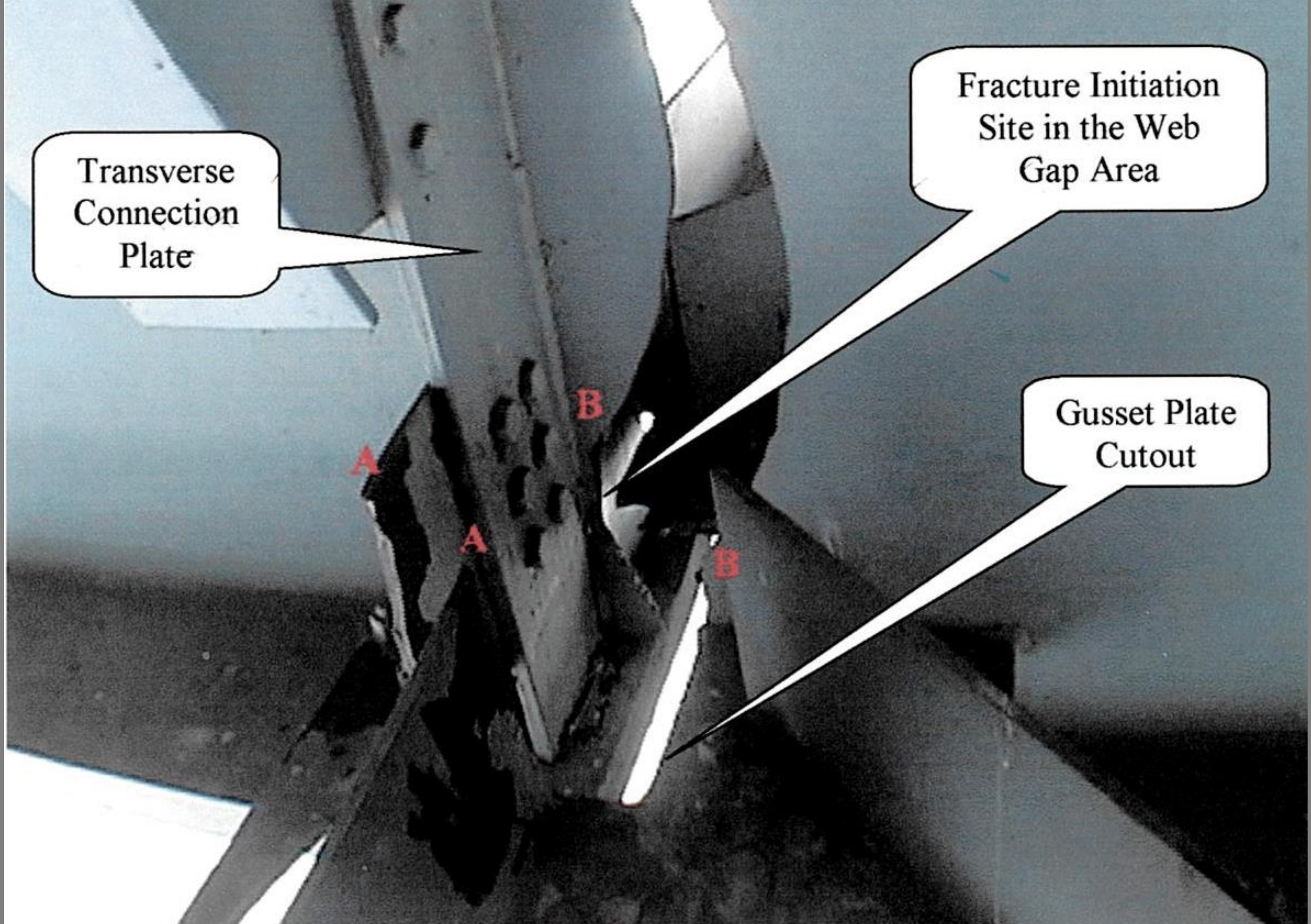











Transverse Connection Plate
Fracture Initiation Site in the Web Gap Area
B
A
A
B
Gusset Plate Cutout

U.S. Department
of Transportation
Federal Highway Administration

Memorandum

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

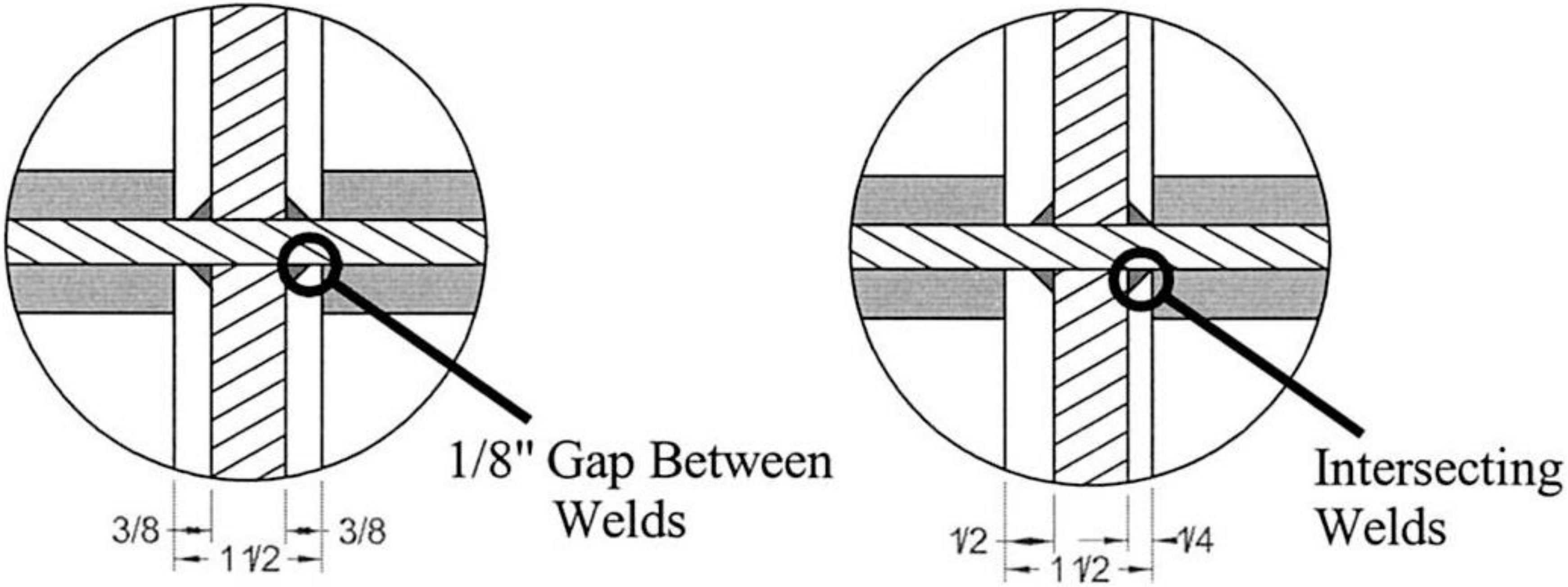
Date: July 10, 2001

From: James D. Cooper
Director, Bridge Technology

Reply to Attn of: HIBT-10

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

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



A

B

“…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.)

- 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.

"A narrow gap between the gusset plate and the transverse connection/stiffener plate created a local triaxial constraint conditions and increased the stiffness in the web gap region at the fracture initiation site.

- 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**

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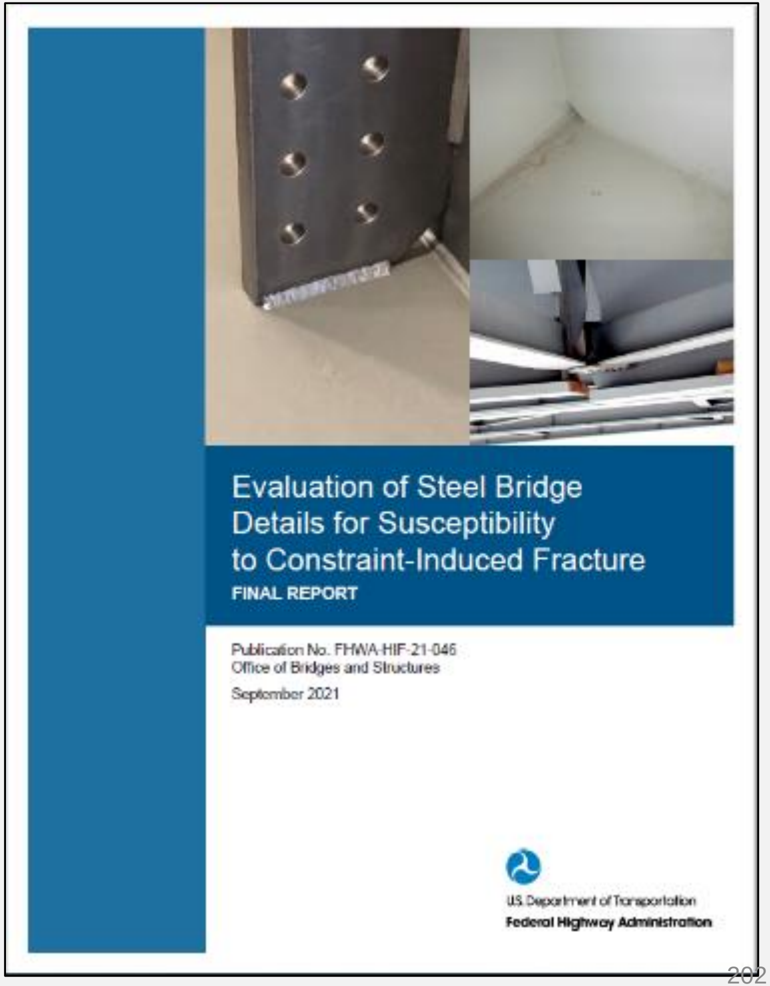
"This non-ductile behavior in the joint caused by a triaxial constraint and state of stress has never been documented before as being a potential problem in bridge detailing. This is the first time this problem is being reported."

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



TECHNICAL REPORT DOCUMENTATION PAGE

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		6. Performing Organization Code:
7. Author(s) 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.		8. Performing Organization Report No.
9. Performing Organization Name and Address HDR Engineering, Inc. 301 Grant Street, Suite 1700 Pittsburgh, PA 15219-1408		10. Work Unit No.
		11. Contract or Grant No. DTFH61-14-D-00049

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 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.

Evaluation of Steel Bridge Details for Susceptibility of Constraint-Induced Fracture

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.

CHAPTER 3 - STRESS TRIAXIALITY, CONSTRAINT, AND SUSCEPTIBILITY TO

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.

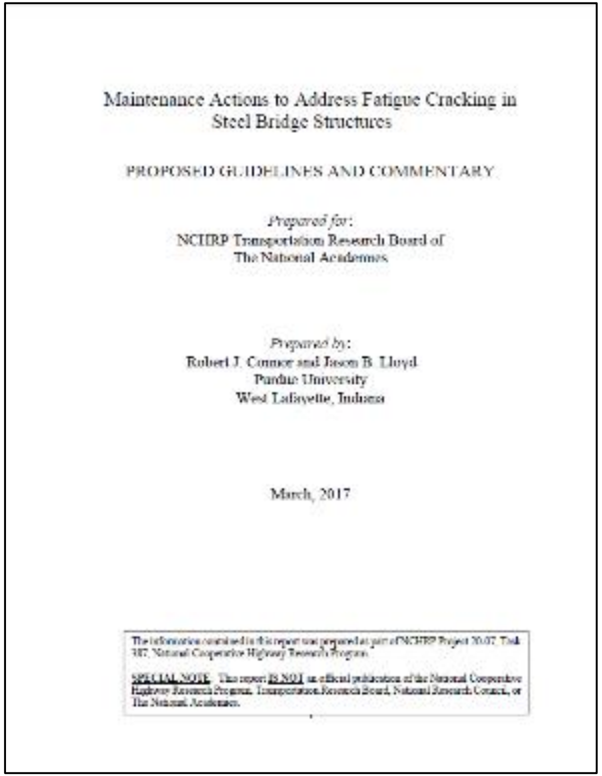
Connor and Lloyd (2017) describe three conditions that contribute to the susceptibility of a detail to CIF:

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



Maintenance Actions to Address Fatigue Cracking in Steel Bridge Structures

PROPOSED GUIDELINES AND COMMENTARY

Prepared for:
NCHRP Transportation Research Board of
The National Academies

Prepared by:
Robert J. Connor and Jason B. Lloyd
Purdue University
West Lafayette, Indiana

March, 2017

The information contained in this report was prepared as part of NCHRP Project 20-07, Task 387, National Cooperative Highway Research Program.

SPECIAL NOTE: This report IS NOT an official publication of the National Cooperative Highway Research Program, Transportation Research Board, National Research Council, or The National Academies.

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

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

concentrations.

2. The joint must be highly constrained…

3. There must be an elevated level of tensile residual stresses locked into the local area. While

3. There must be an elevated level of tensile residual stress.

Maintenance Actions to Address Fatigue Cracking in Steel Bridges Structures

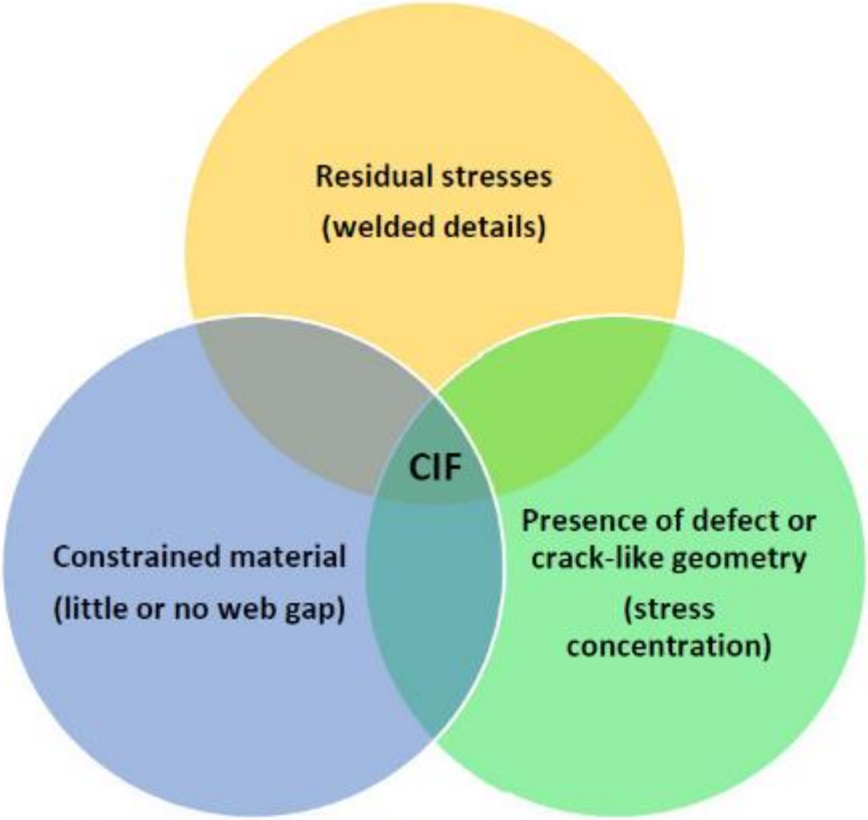


Figure 7.3 Defining characteristics of CIF details

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 stressed induced by welding…

• crack-like geometry, specifically where the so-called "web gap" (a constraint-relief gap)

- 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.

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

> 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

Case Study Lessons

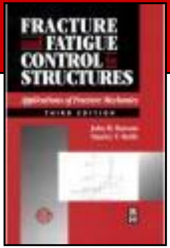
Case Study	Detailing/ Constraint	Notches/ Cracks	Loading	Material Toughness
Liberty Ships	√	√		√
Silver Bridge		√	√	√
Ingram Barge	√		√	
Hoan Bridge	√	√		

AISC 360-16 Specifications for Structural Steel Buildings

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."

Barsom/Rolfe: FRACTURE AND FATIGUE CONTROL IN STRUCTURES



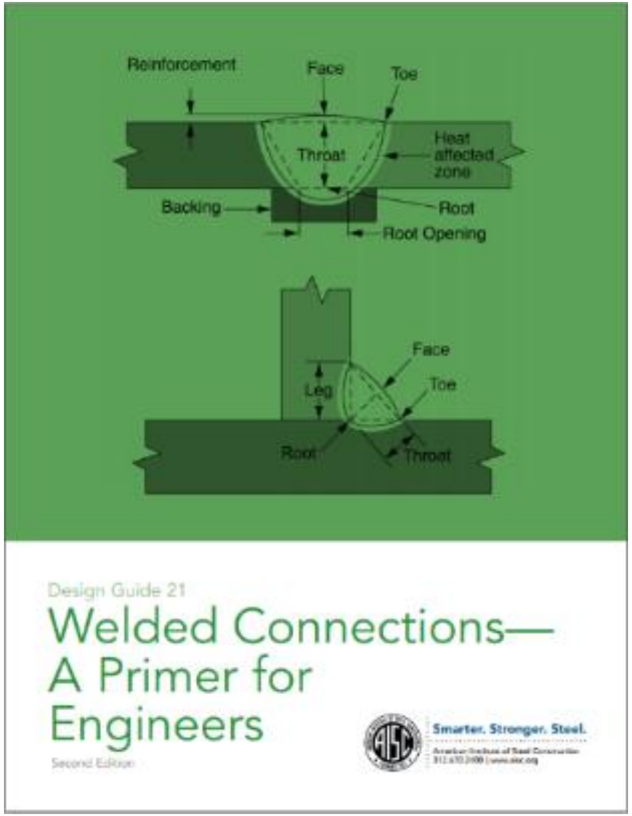
“Fracture mechanics has shown that because of the *interaction* among *materials, design, fabrication,* and *loading,* brittle fractures cannot be eliminated in structures merely by using materials with improved notch toughness. The designer still has the fundamental responsibility for the overall safety 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.

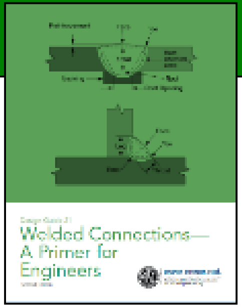
AISC Design Guide 21, 2nd Edition

Welded Connections–

A Primer for Engineers

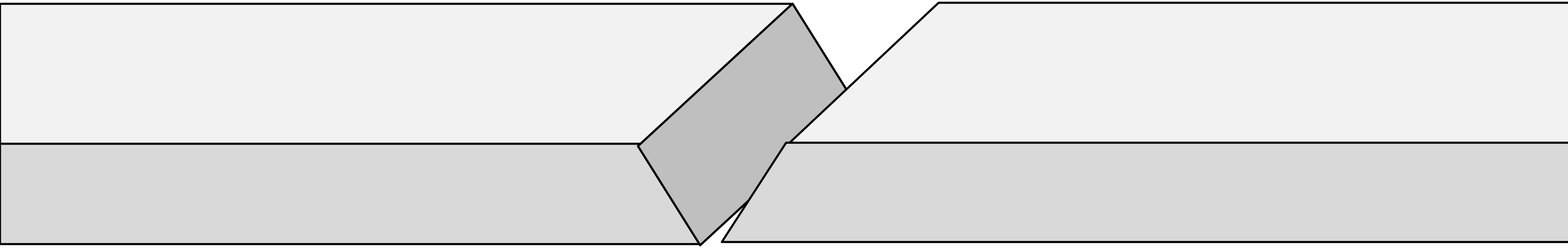


DESIGN GUIDE 21



It is not possible to simply quantify mathematically the degree of restraint offered by the surrounding steel, but an intuitive feel can be developed.

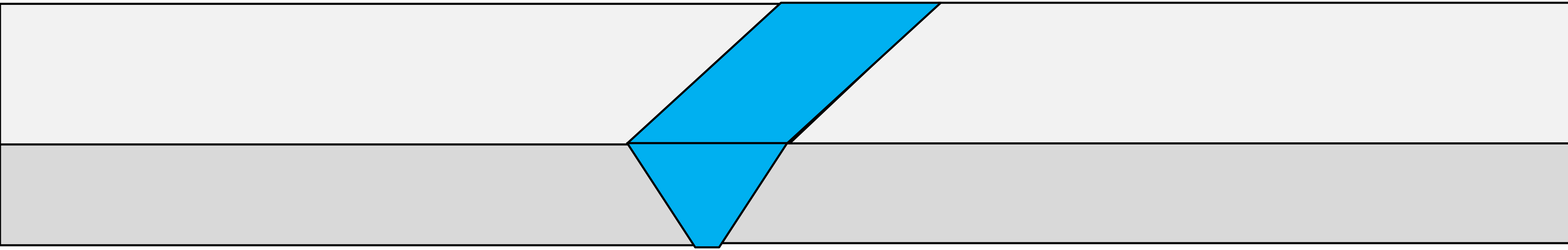
Flange Splice



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

Flange Splice

LOW CONSTRAINT

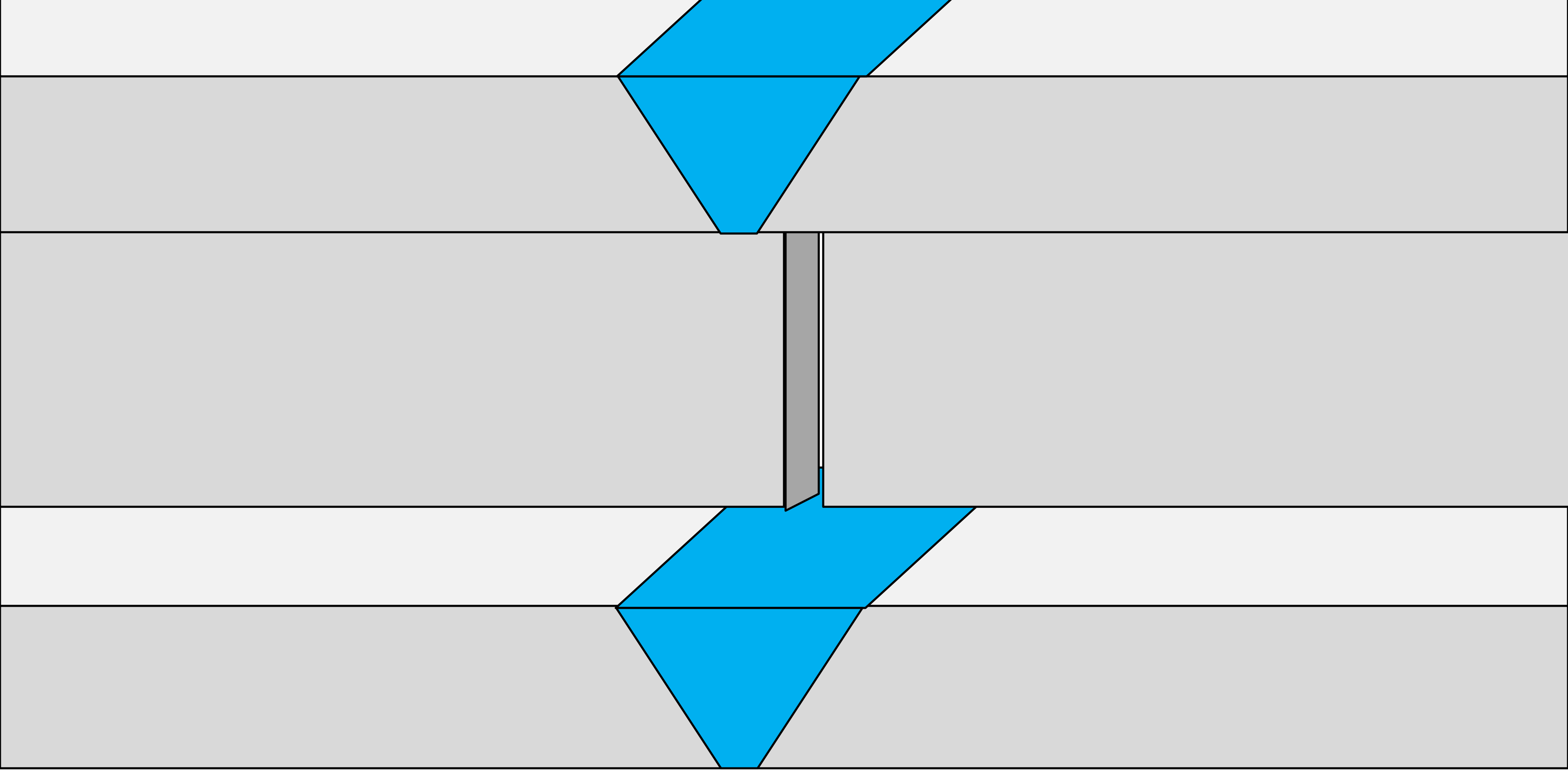


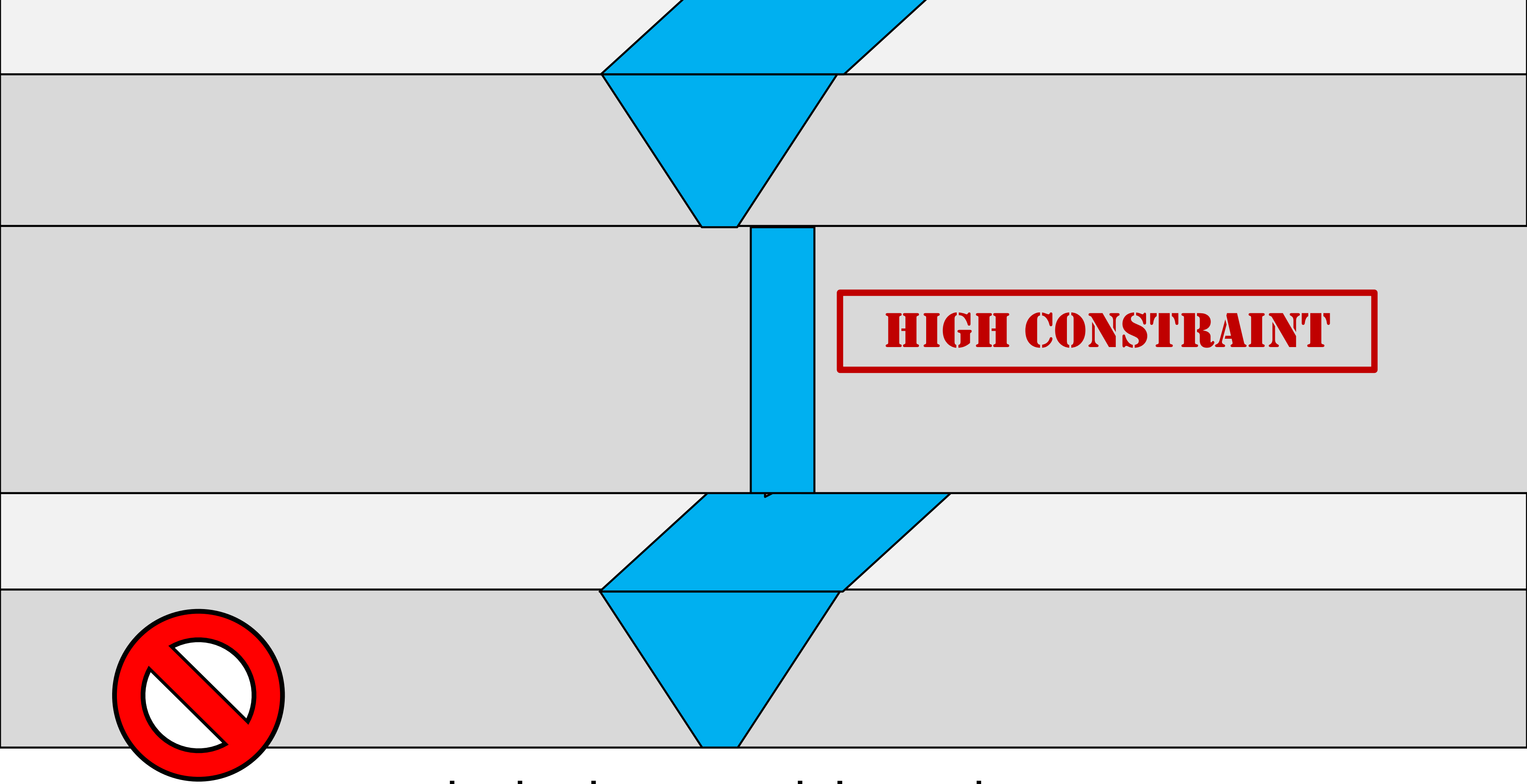
3 in. (75 mm) thick, 10 in. (250 mm) wide,
Two 40 foot (13 m) lengths

Wide Flange Splice

W14 X 730

5 in. (125 mm) thick flange, 3 in. (75 mm thick web)





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

Improving Fracture Resistance in Cold Temperature Applications

$$K_C > \sigma\sqrt{\pi a}$$

12 sub-principles

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.

Improving Fracture Resistance in Cold Temperature Applications

$$K_C > \sigma\sqrt{\pi a}$$

Principle 1: Reduce Stress

1. 9 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.

$$K_C > \sigma\sqrt{\pi a}$$

14 sub-principles

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.

Improving Fracture Resistance in Cold Temperature Applications

$$K_C > \sigma\sqrt{\pi a}$$

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

$$K_C > \sigma\sqrt{\pi a}$$

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.

$$K_c > \sigma\sqrt{\pi a}$$

6 sub-principles

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.

$$K_C > \sigma\sqrt{\pi a}$$

5 sub-principles

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.

$$K_C > \sigma\sqrt{\pi a}$$

6 Principles

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.

$$K_C > \sigma\sqrt{\pi a}$$

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.

Improving Fracture Resistance in Cold Temperature Applications

$$K_C > \sigma\sqrt{\pi a}$$

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
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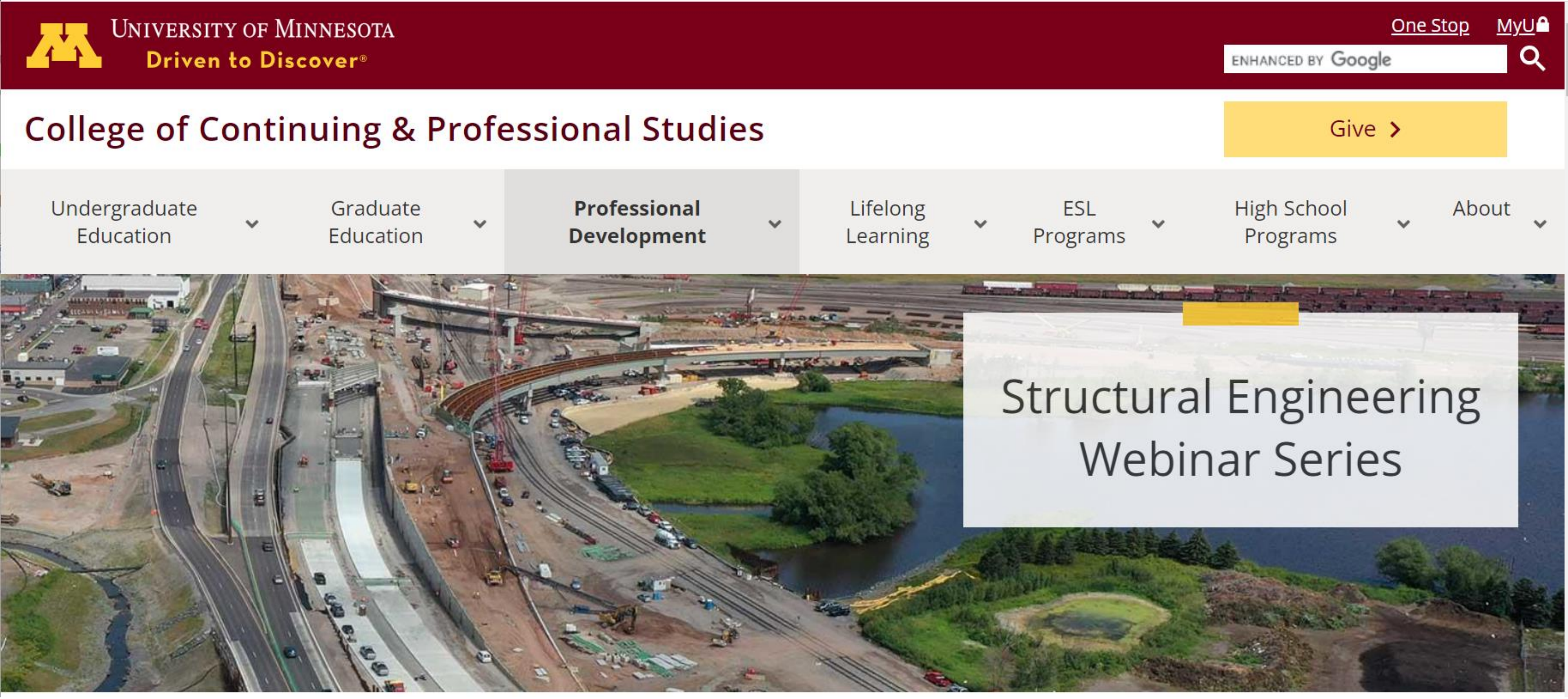

Brittle Fracture: Another View