

Design of Seismic-Resistant Steel Building Structures

2. Moment Resisting Frames

Prepared by:
Michael D. Engelhardt
University of Texas at Austin

with the support of the
American Institute of Steel Construction.

Version 1 - March 2007



Design of Seismic-Resistant Steel Building Structures

- 1 - Introduction and Basic Principles**
- 2 - Moment Resisting Frames**
- 3 - Concentrically Braced Frames**
- 4 - Eccentrically Braced Frames**
- 5 - Buckling Restrained Braced Frames**
- 6 - Special Plate Shear Walls**

2 - Moment Resisting Frames

- **Definition and Basic Behavior of Moment Resisting Frames**
- **Beam-to-Column Connections: Before and After Northridge**
- **Panel-Zone Behavior**
- **AISC Seismic Provisions for Special Moment Frames**

Moment Resisting Frames

- **Definition and Basic Behavior of Moment Resisting Frames**

- **Beam-to-Column Connections: Before and After Northridge**
- **Panel-Zone Behavior**
- **AISC Seismic Provisions for Special Moment Frames**

MOMENT RESISTING FRAME (MRF)

Beams and columns with moment resisting connections; resist lateral forces by flexure and shear in beams and columns

Develop ductility by:

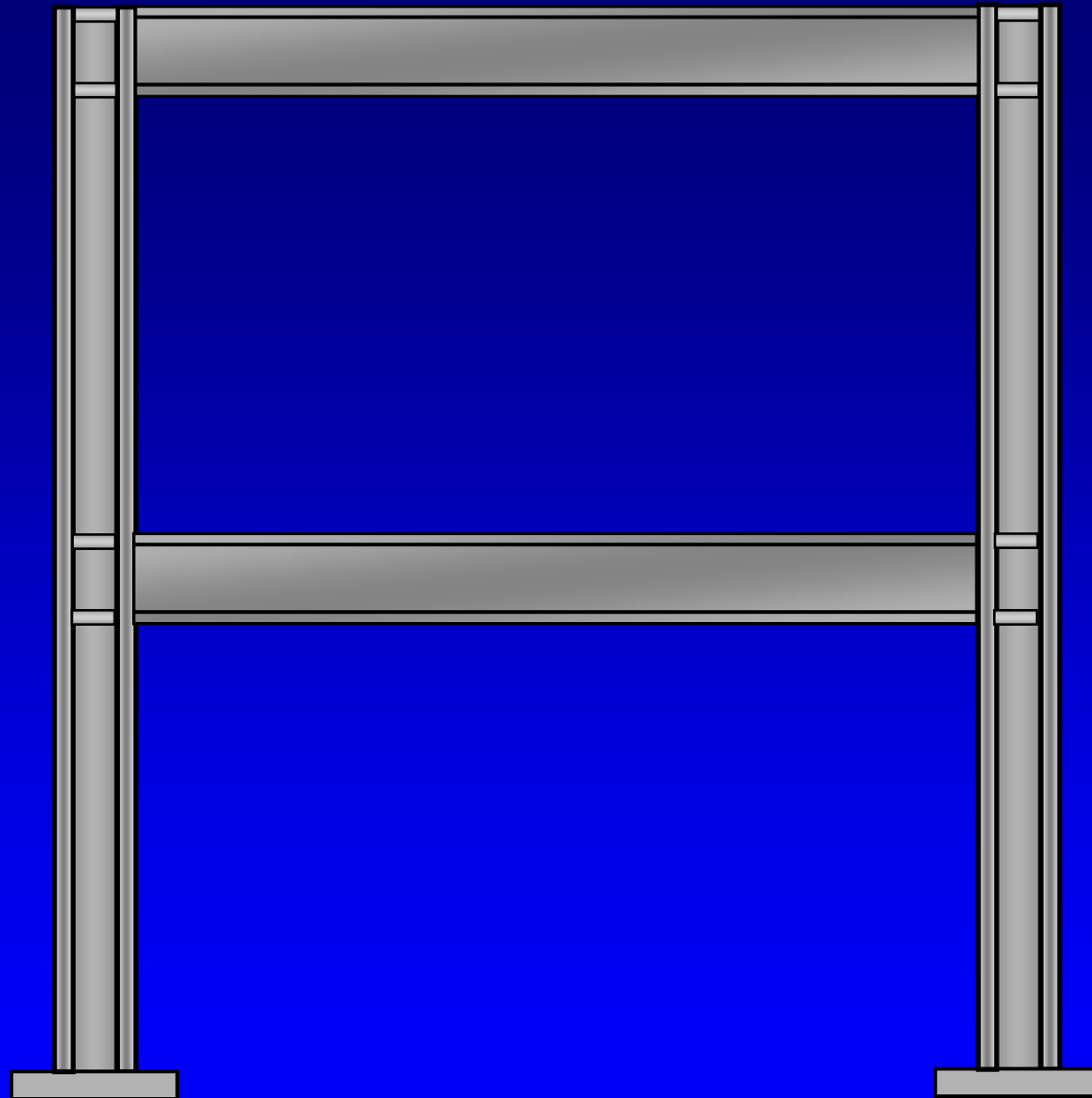
- flexural yielding of beams
- shear yielding of column panel zones
- flexural yielding of columns

Advantages

- Architectural Versatility
- High Ductility and Safety

Disadvantages

- Low Elastic Stiffness



Moment Resisting Frame



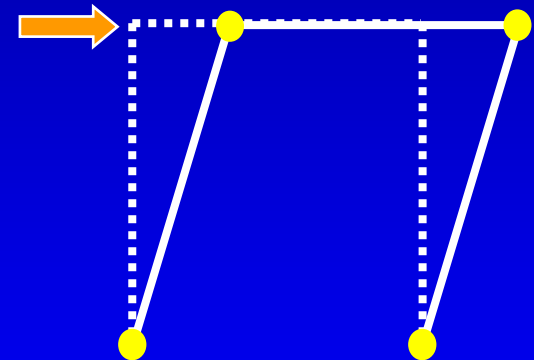


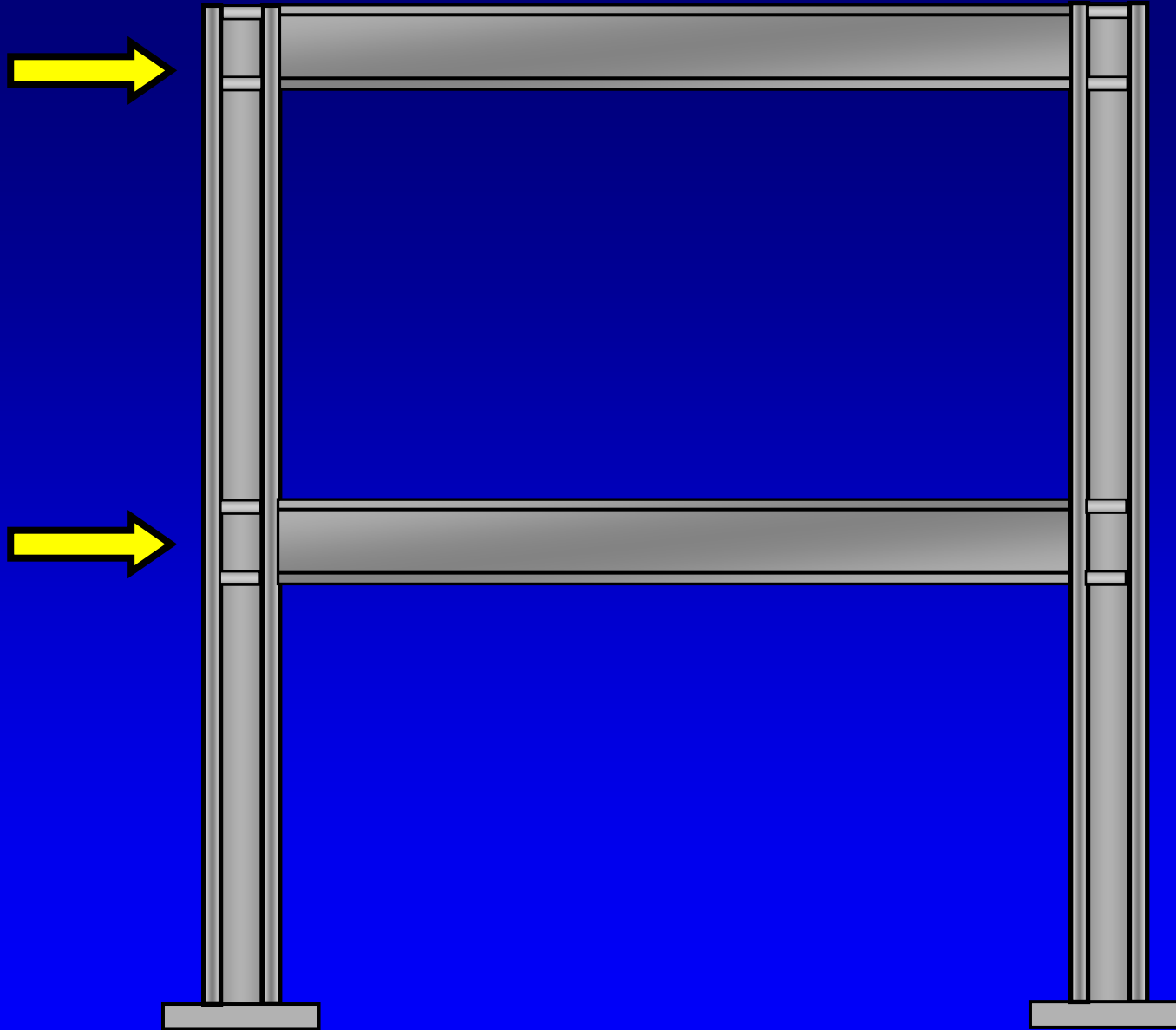


Achieving Ductile Behavior:

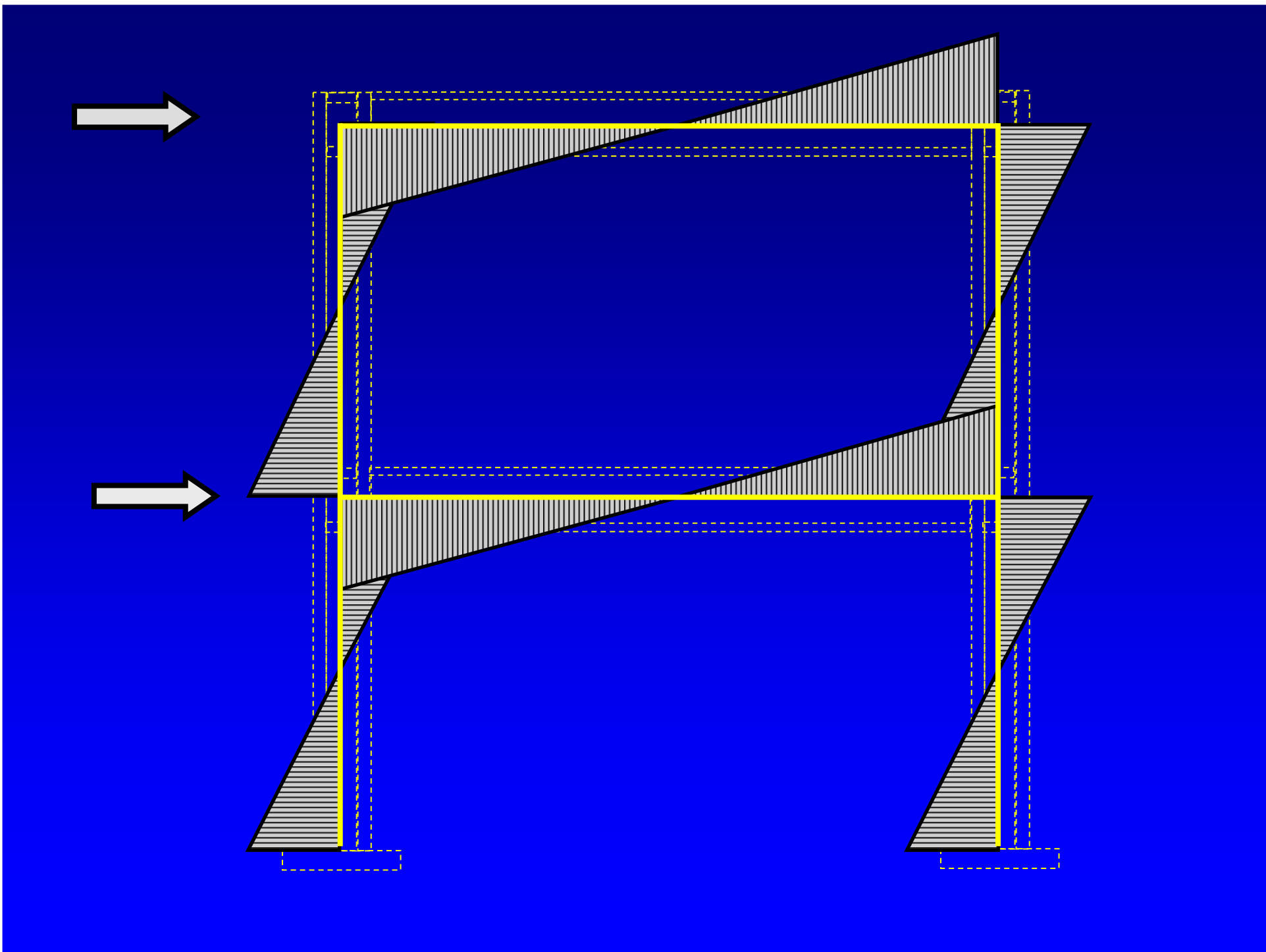
Understand and Control Inelastic Behavior:

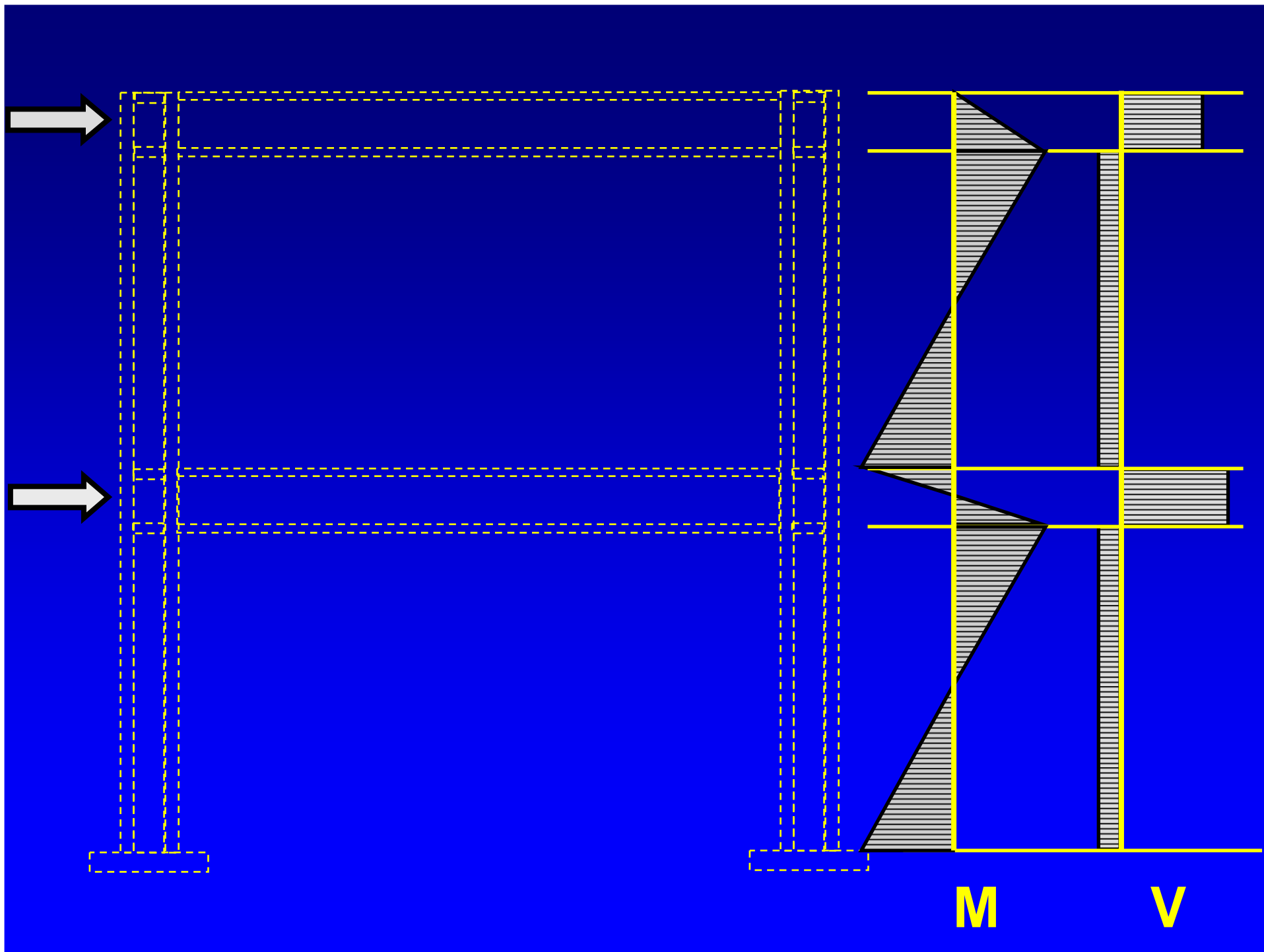
- Choose frame elements ("fuses") that will yield in an earthquake, i.e, choose plastic hinge locations.
- Detail plastic hinge regions to sustain large inelastic rotations prior to the onset of fracture or instability.
- Design all other frame elements to be stronger than the plastic hinge regions.



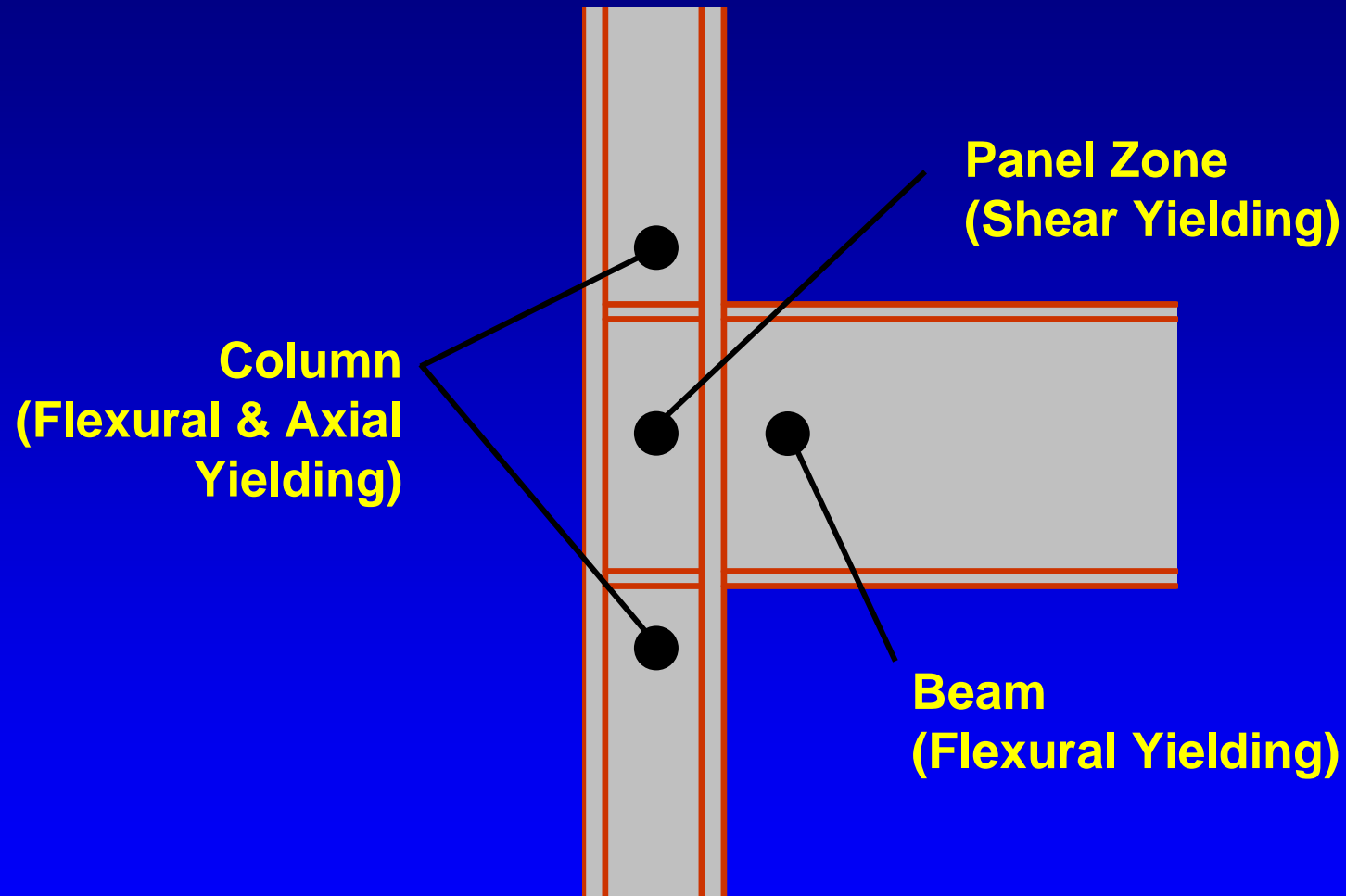


Behavior of an MRF Under Lateral Load:
Internal Forces and Possible Plastic Hinge Locations

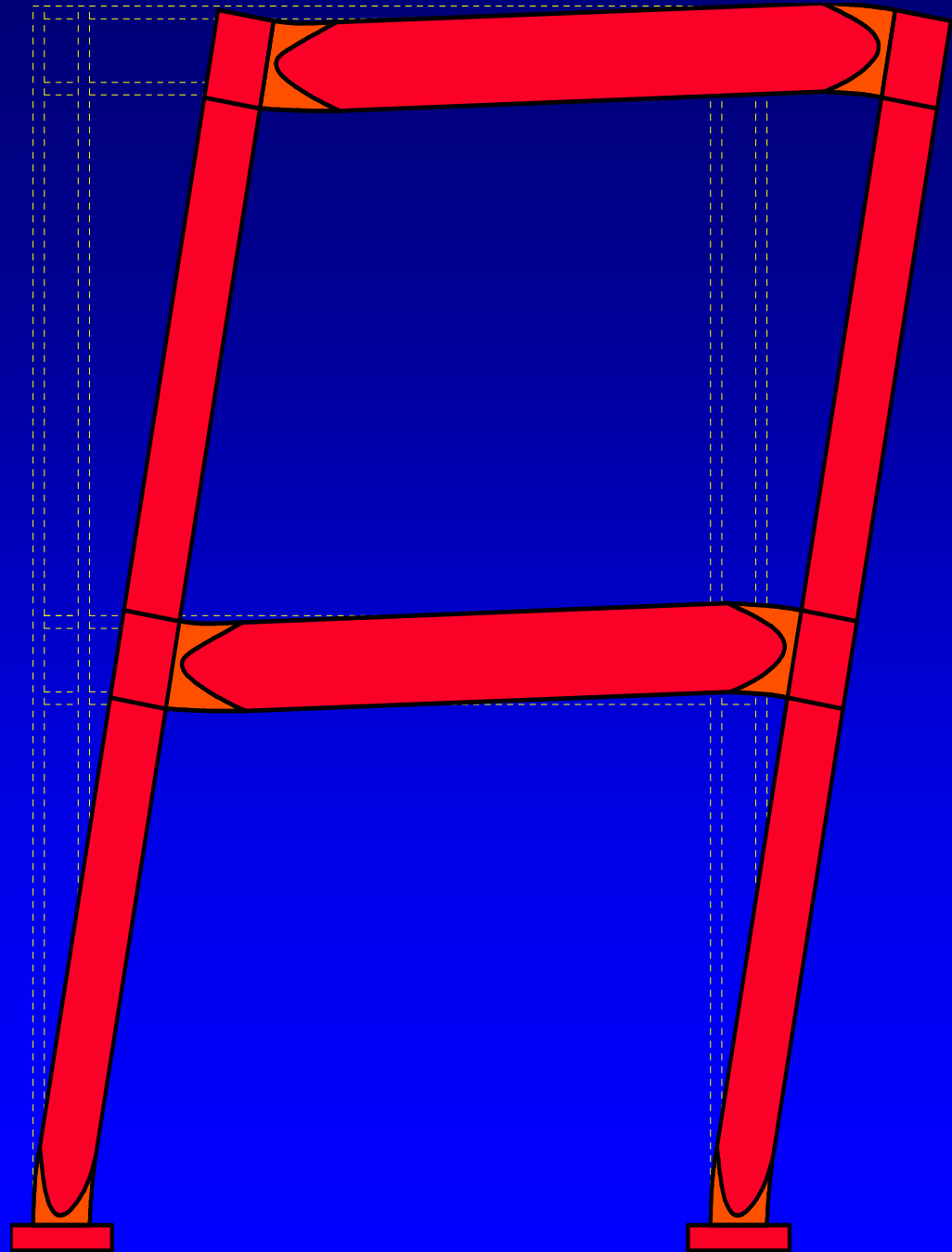




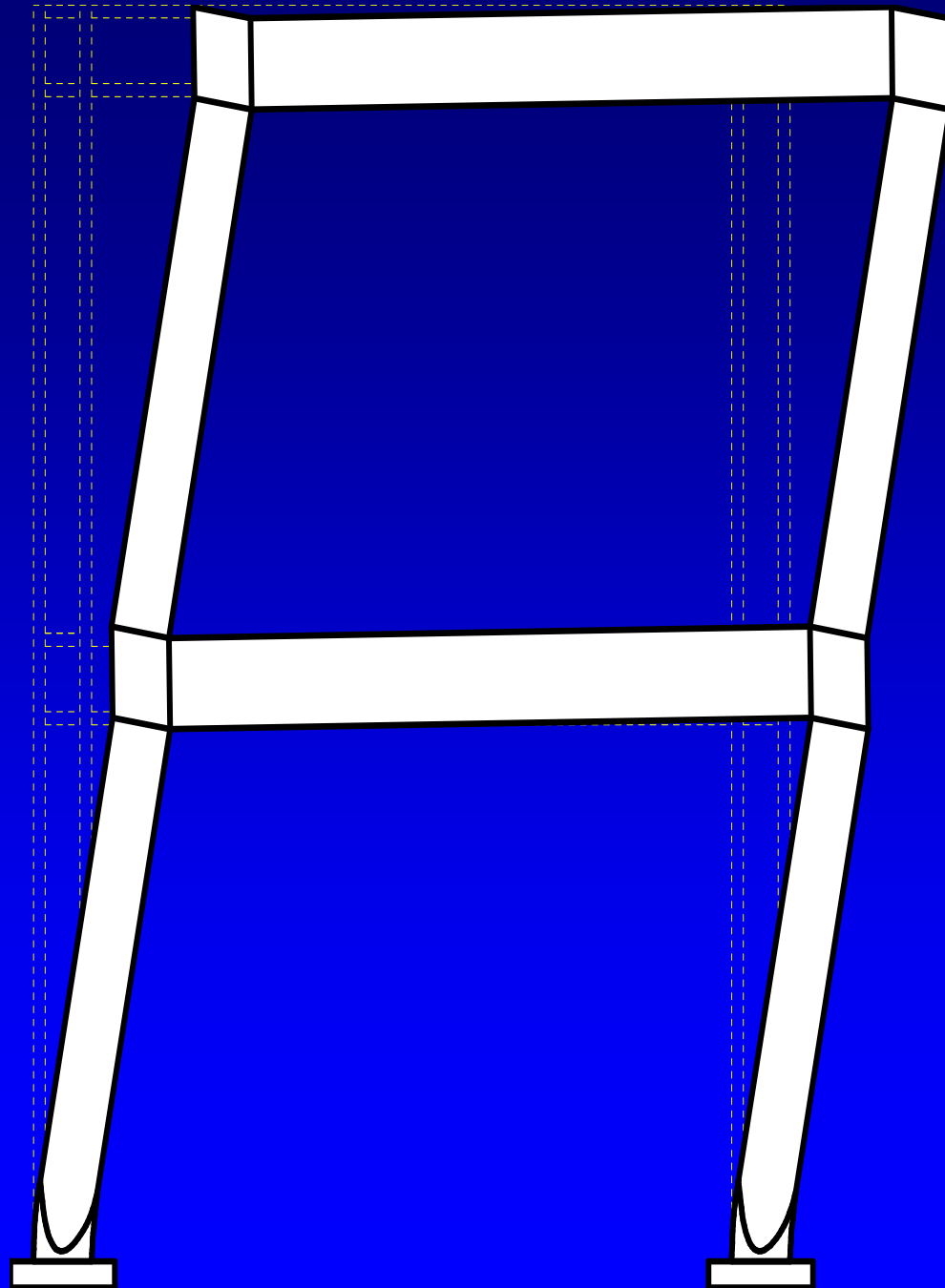
Possible Plastic Hinge Locations



Plastic Hinges In Beams

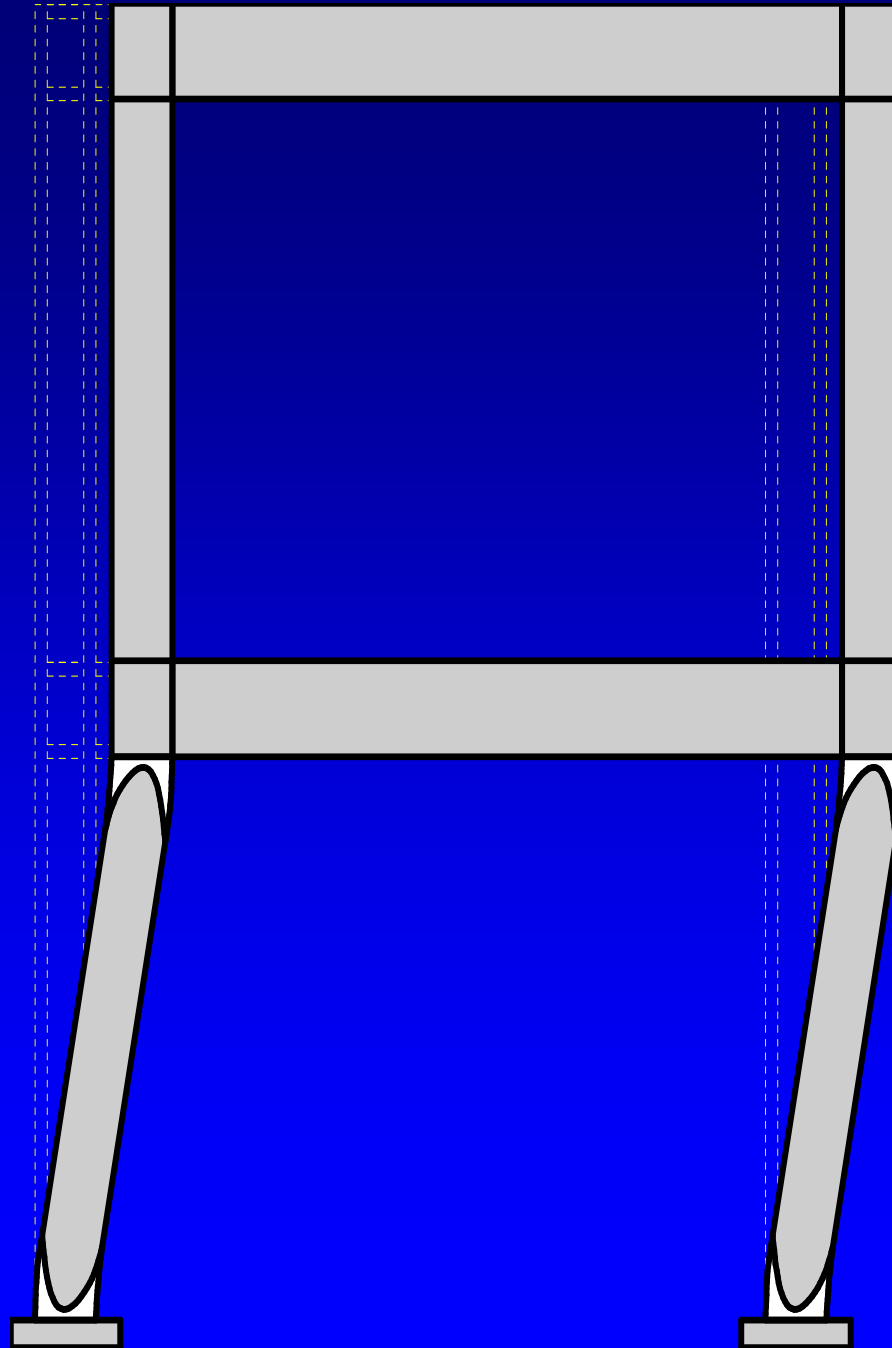


Plastic Hinges In Column Panel Zones



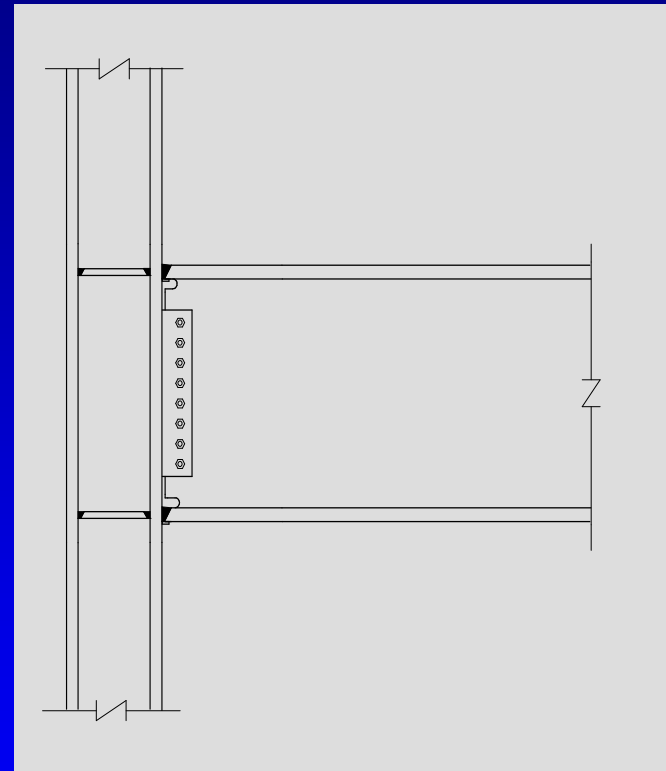
Plastic Hinges In Columns:

Potential for Soft
Story Collapse



Critical Detailing Area for Moment Resisting Frames: Beam-to-Column Connections

Design Requirement:
Frame must develop large ductility
without failure of beam-to-column
connection.



Moment Resisting Frames

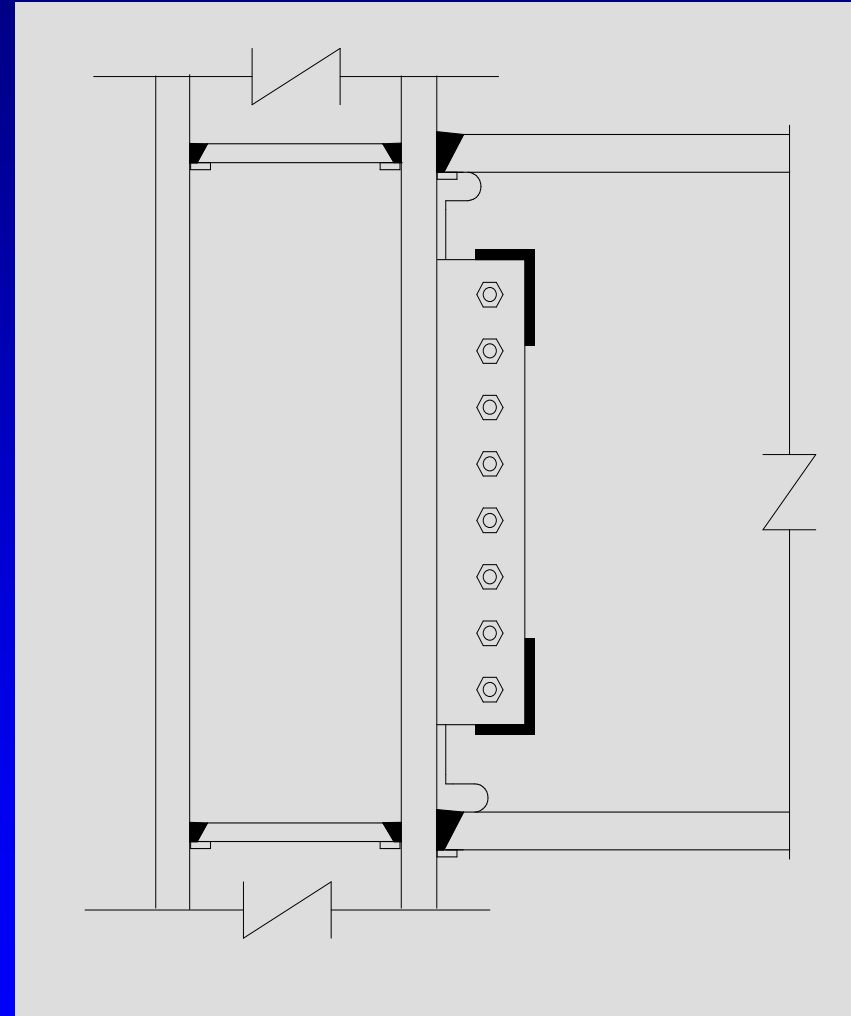
- Definition and Basic Behavior of Moment Resisting Frames

- Beam-to-Column Connections: Before and After Northridge

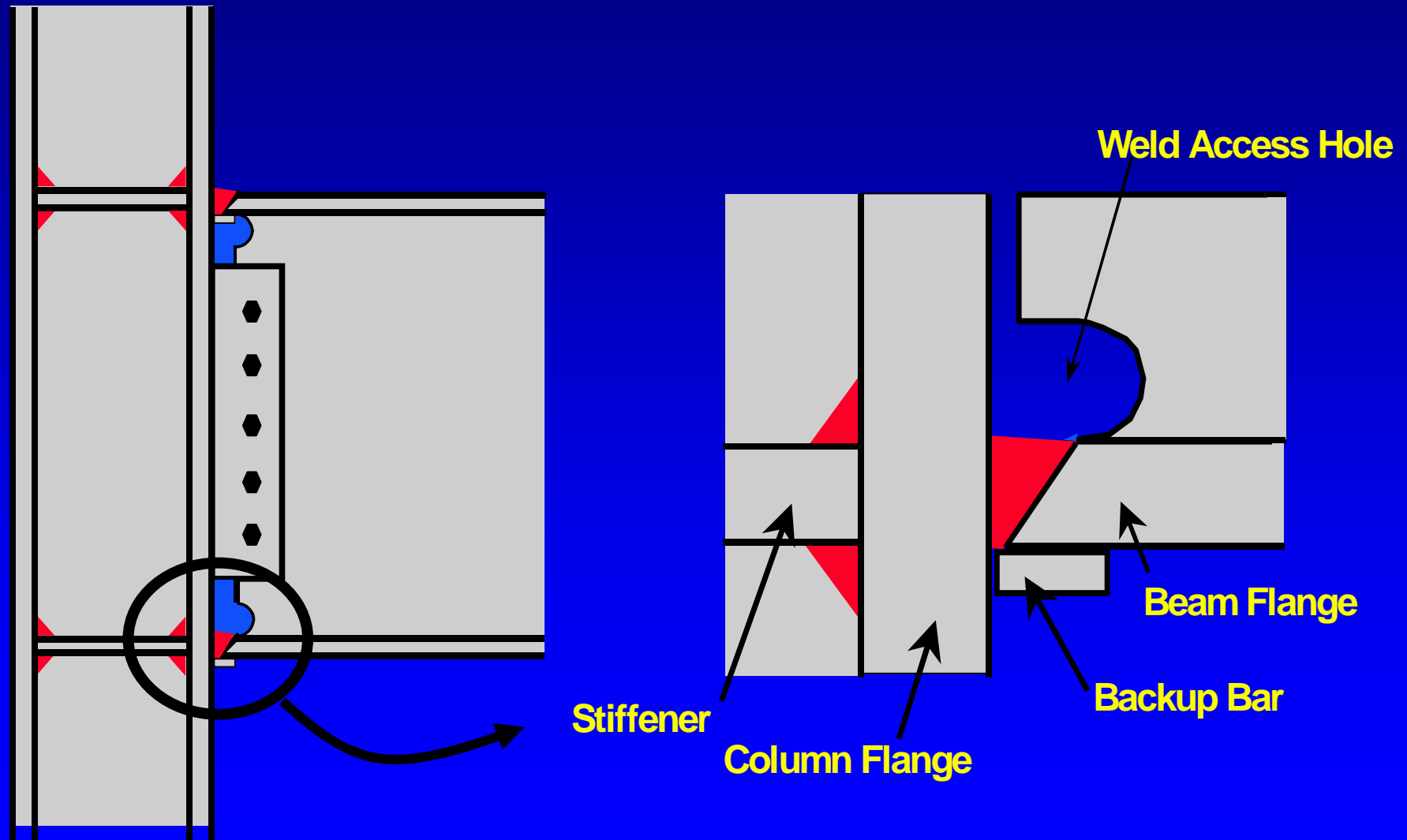
- Panel-Zone Behavior
- AISC Seismic Provisions for Special Moment Frames

Moment Connection Design Practice Prior to 1994 Northridge Earthquake:

Welded flange-bolted web moment connection widely used from early 1970's to 1994



Pre-Northridge Welded Flange – Bolted Web Moment Connection

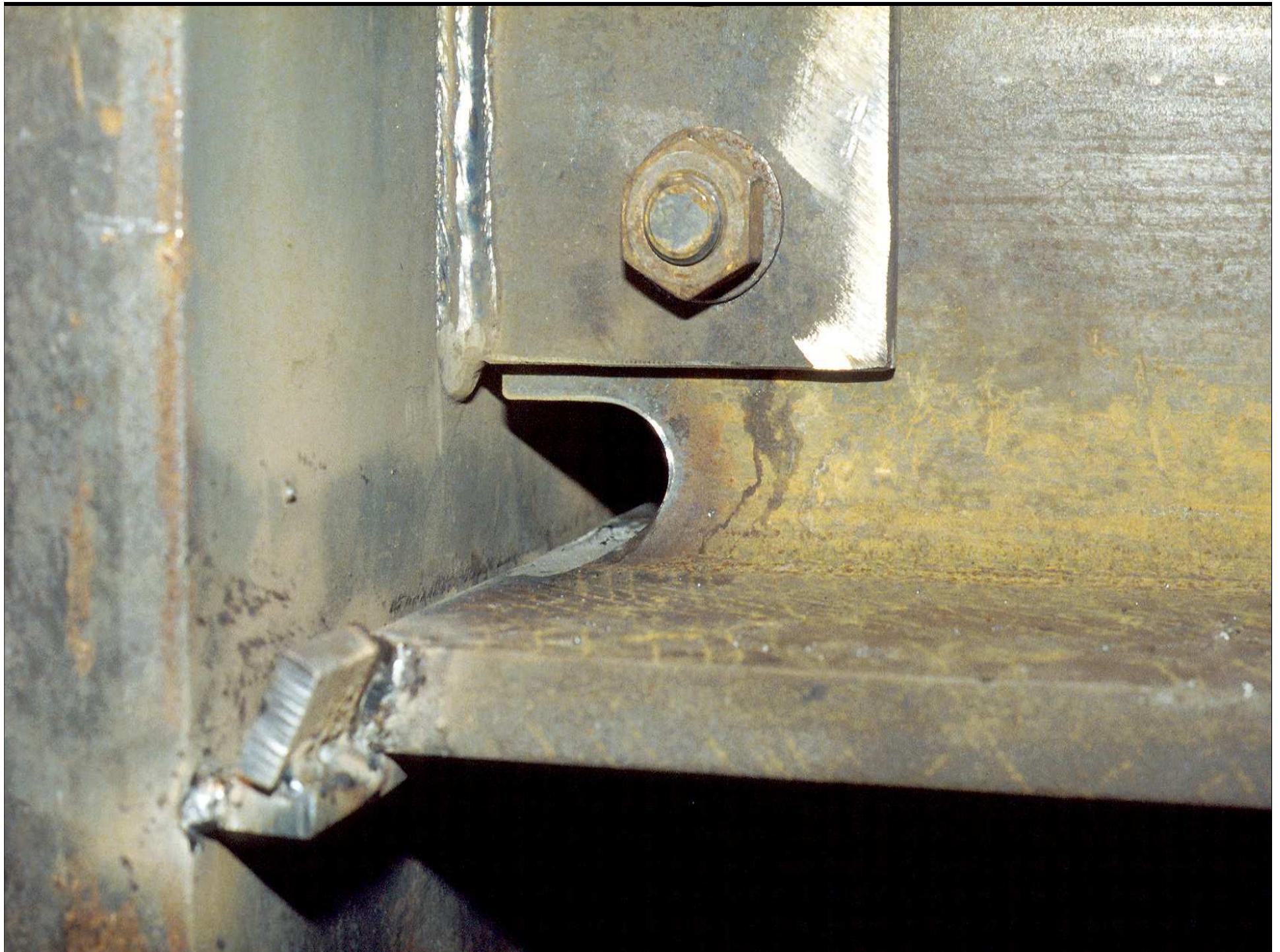


















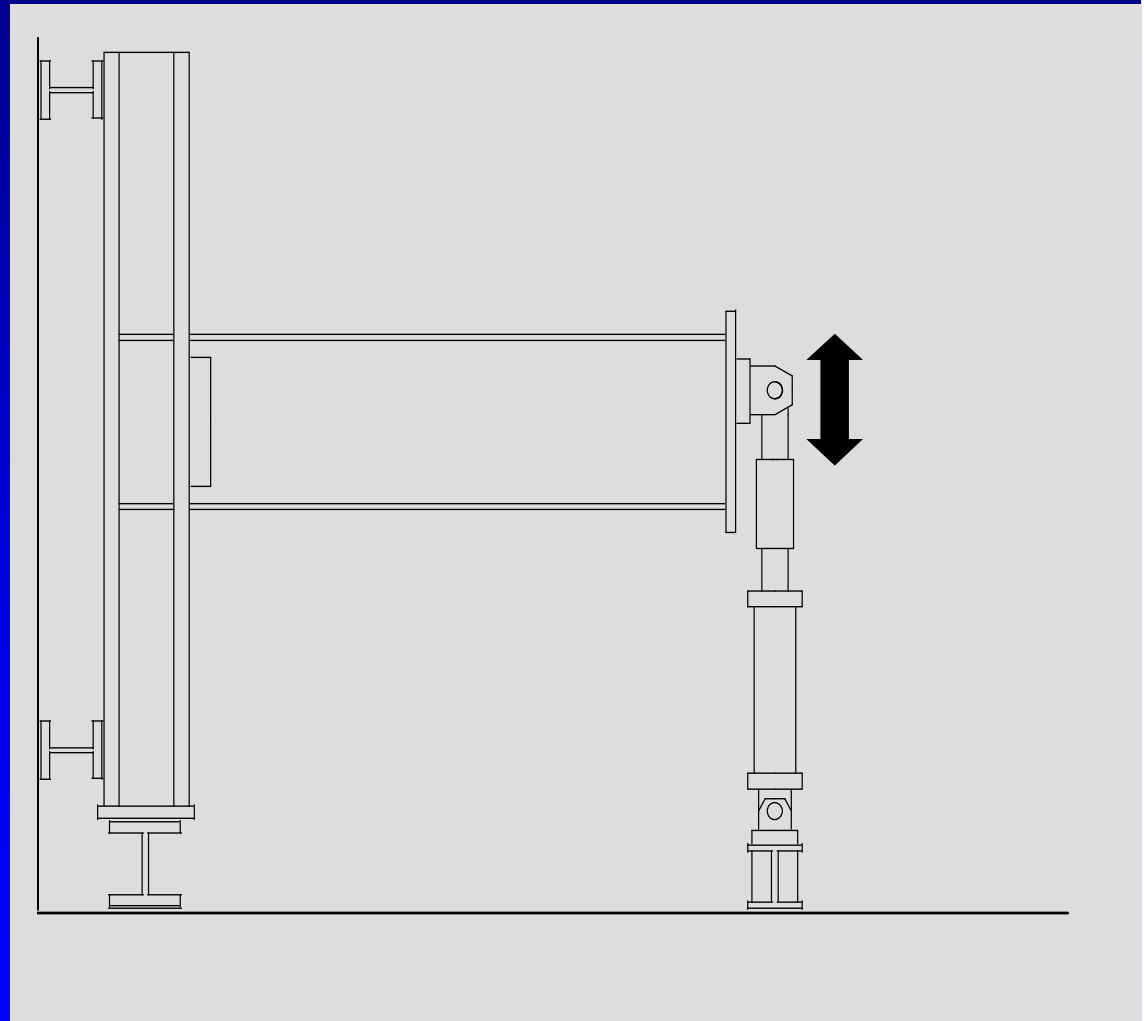






Experimental Data on “Pre-Northridge” Moment Connection

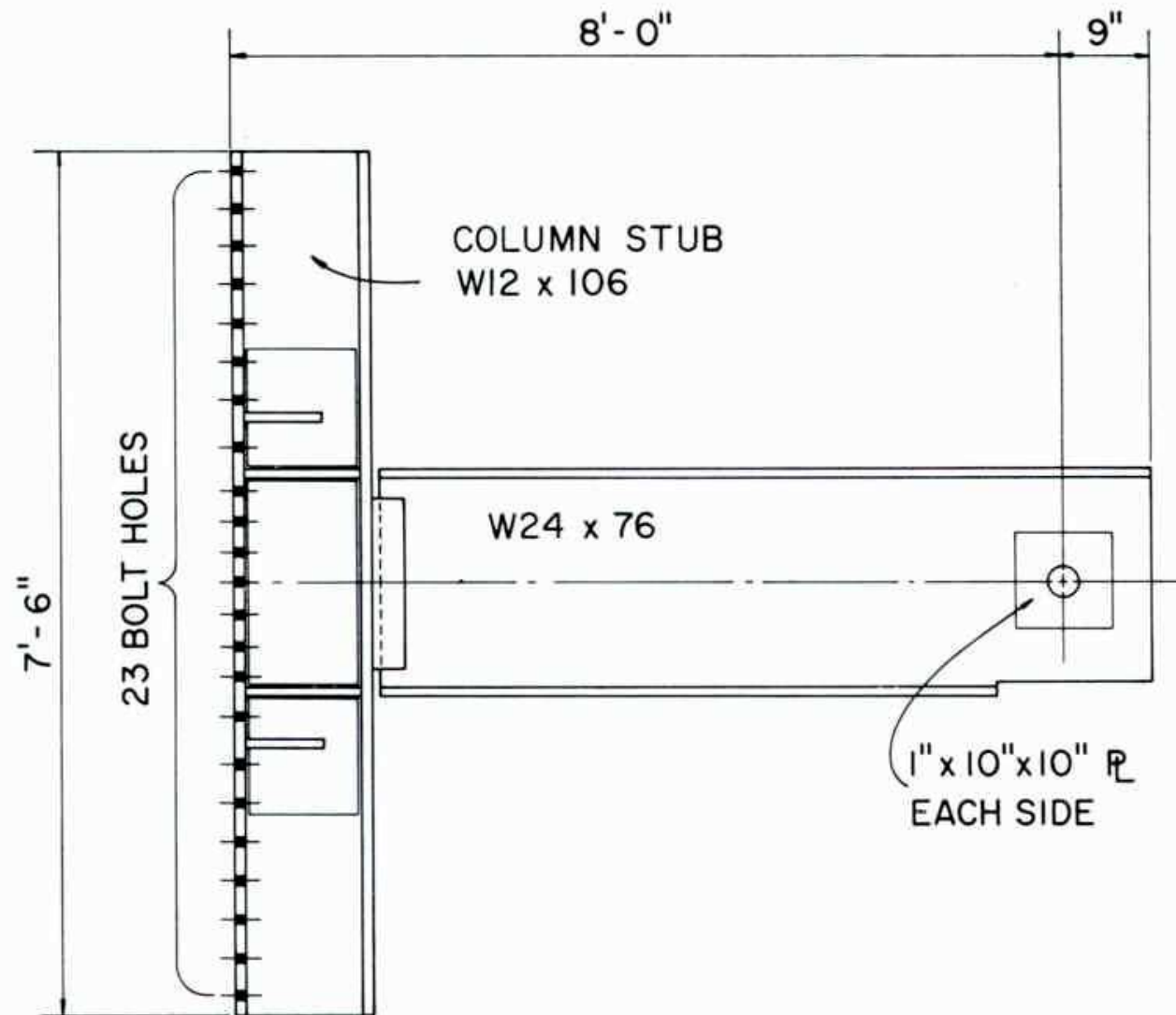
Typical Experimental
Setup:



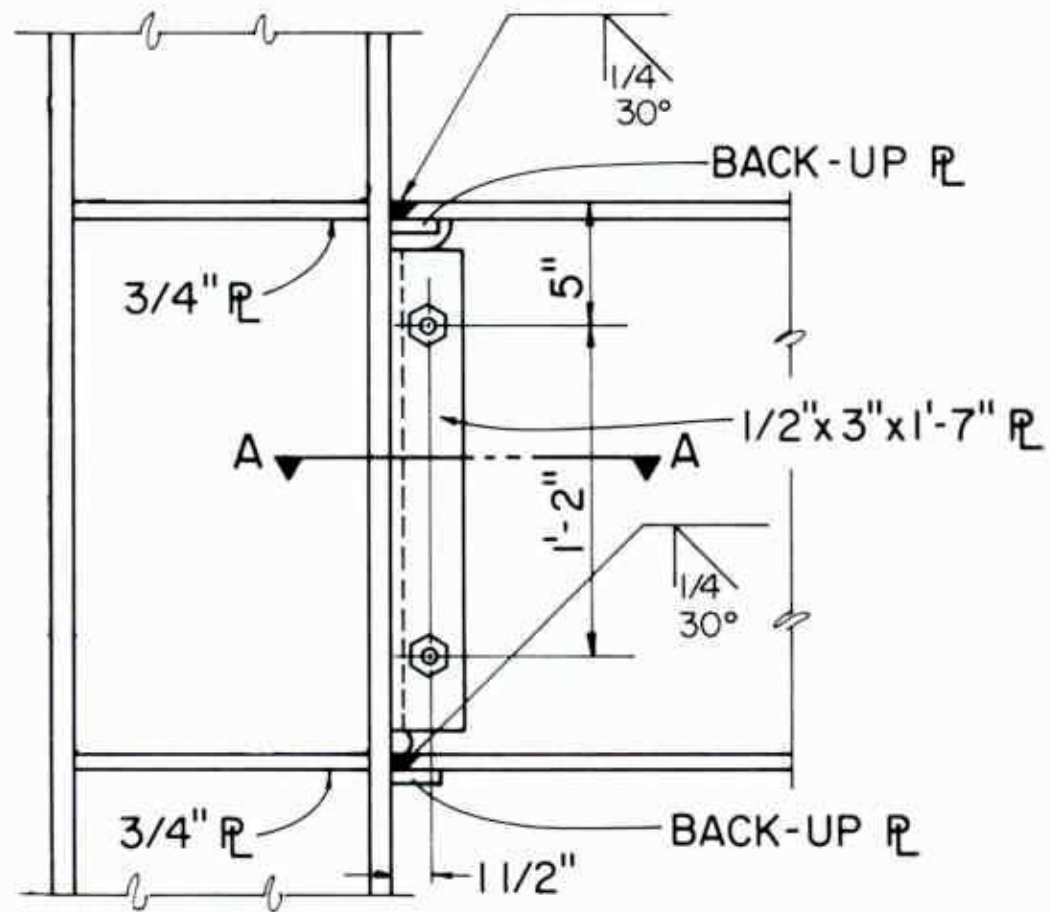


Initial Tests on Large Scale Specimens:

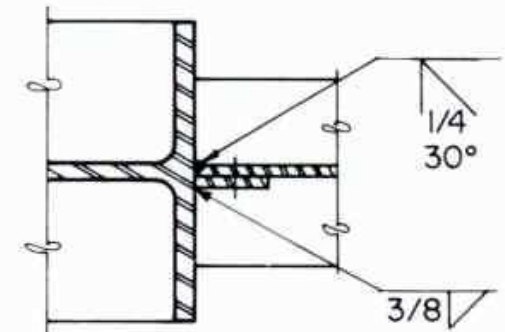
- **Tests conducted at UC Berkeley ~1970**
- **Tests on W18x50 and W24x76 beams**
- **Tests compared all-welded connections with welded flange-bolted web connections**



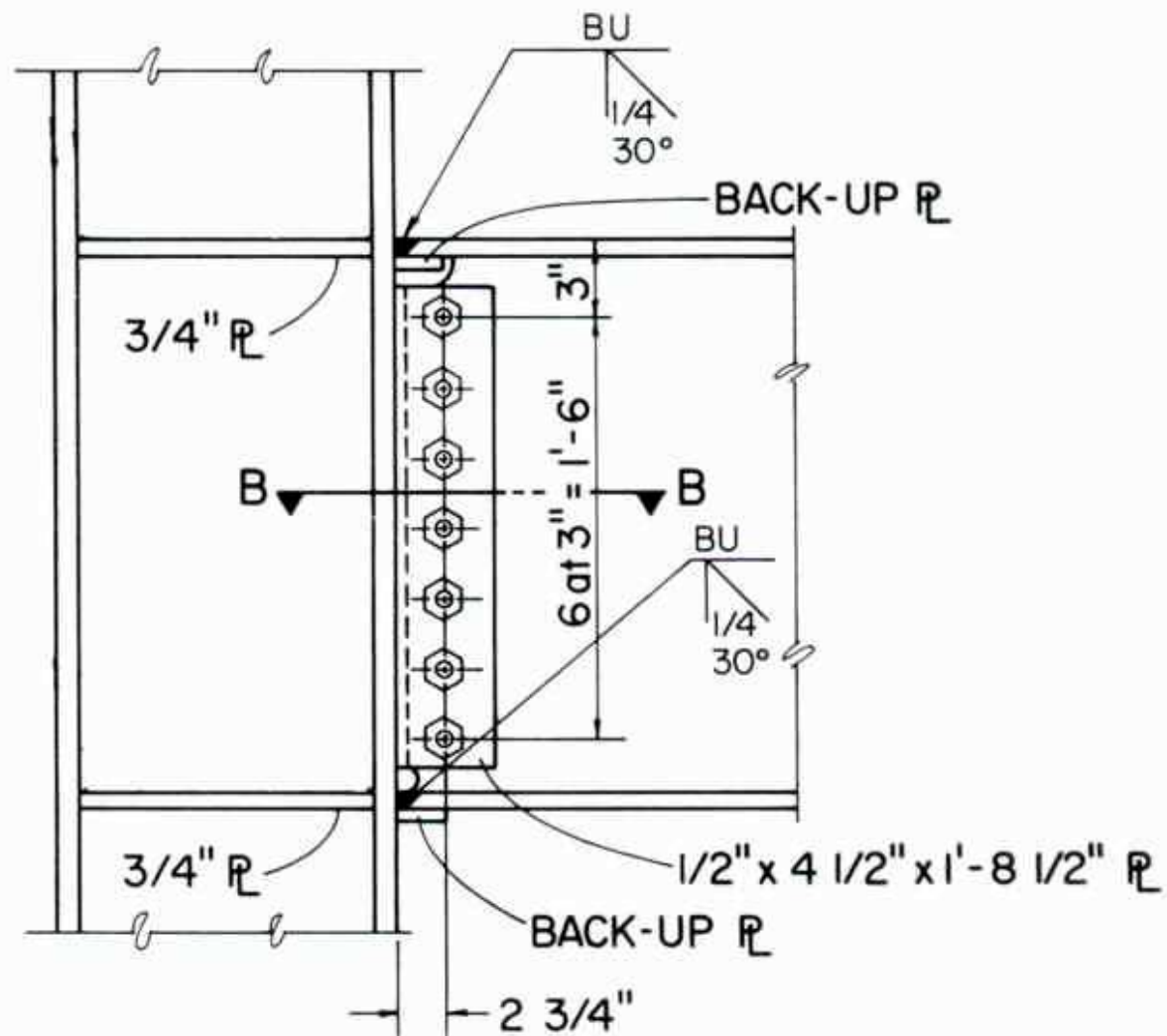
TYPICAL W24 x 76 SPECIMEN



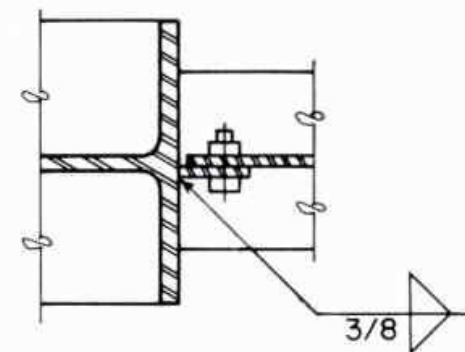
W24 x 76 WELDED
JOINT DETAIL



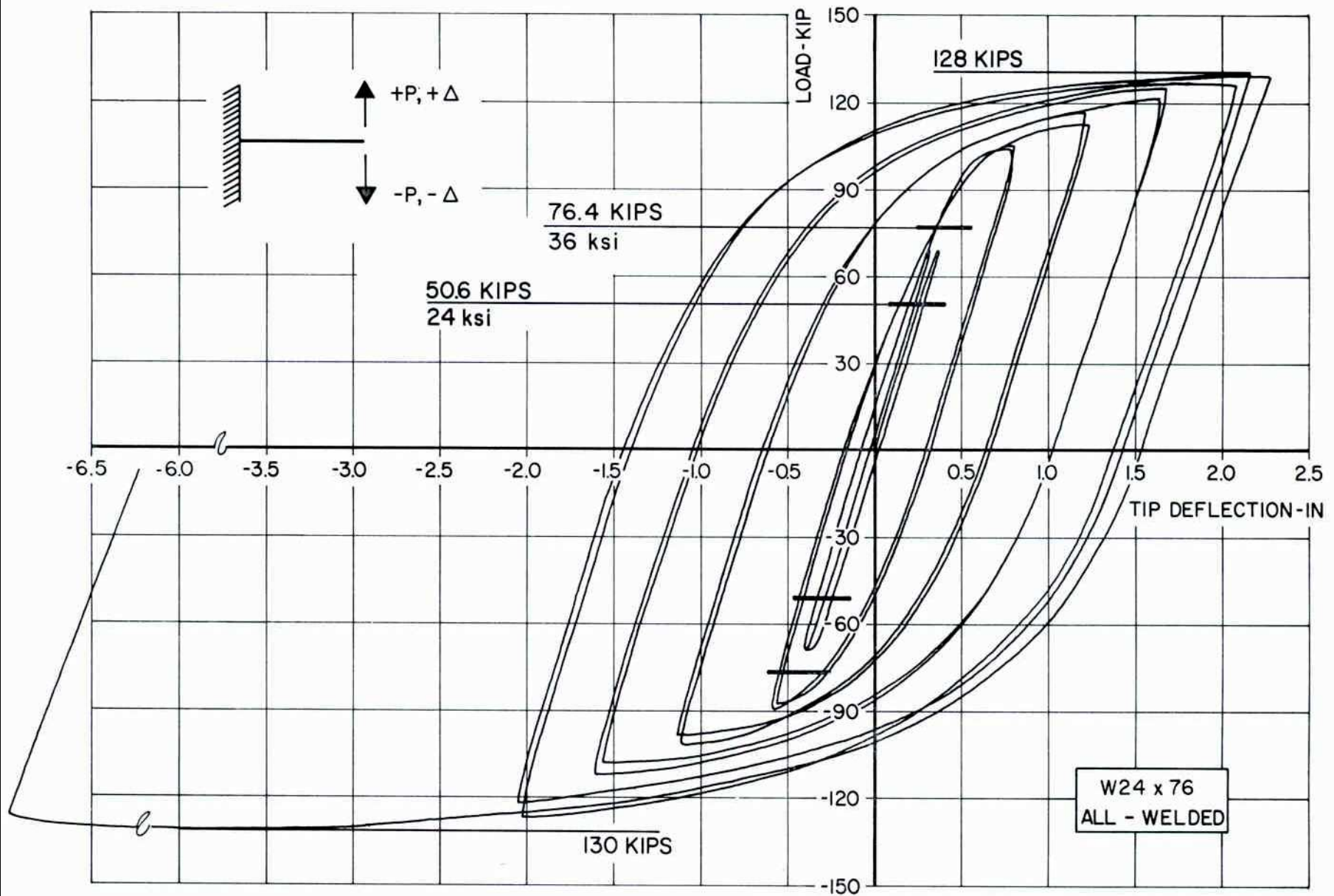
SECTION A-A



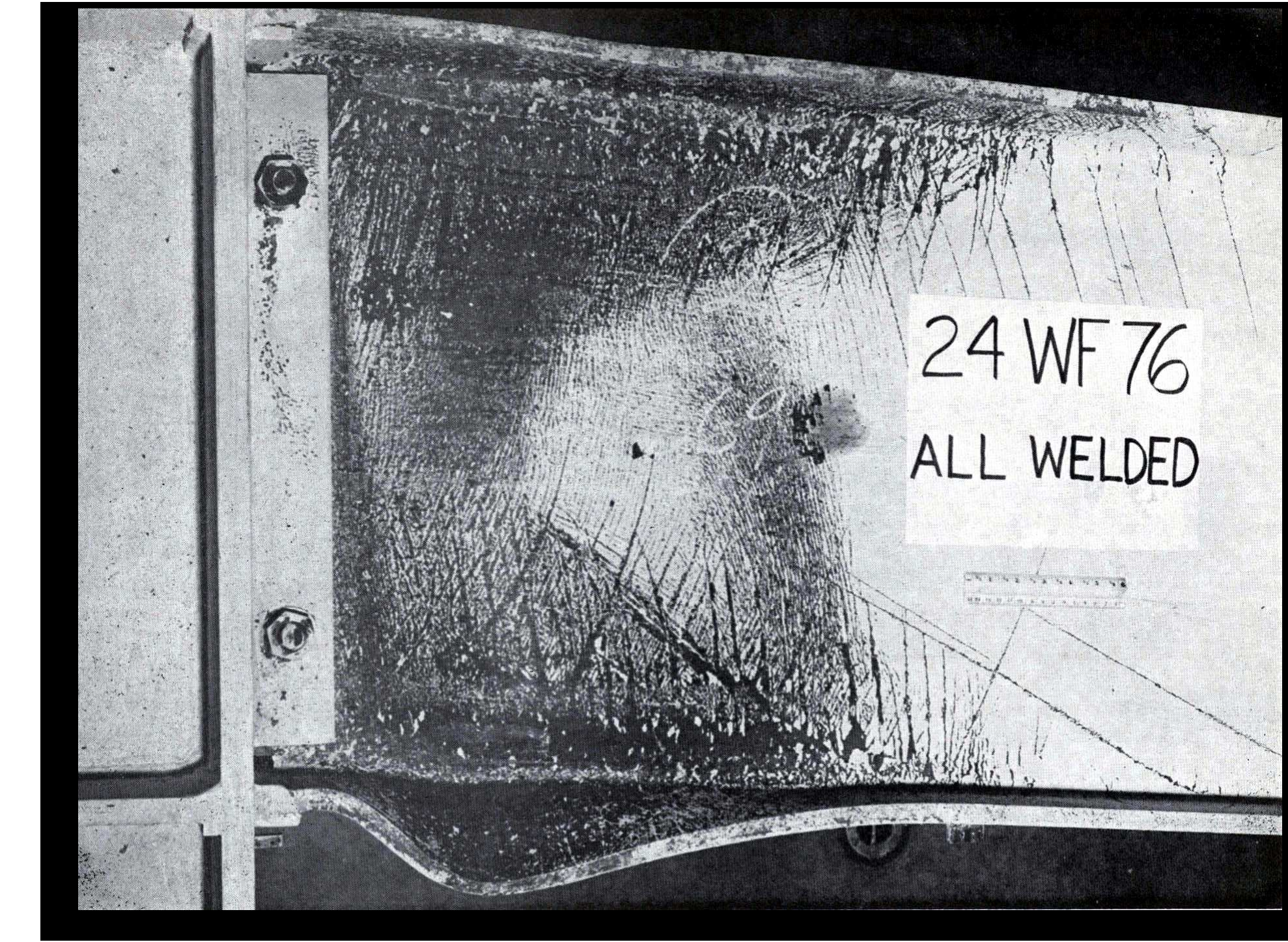
W24 x 76 BOLTED
JOINT DETAIL



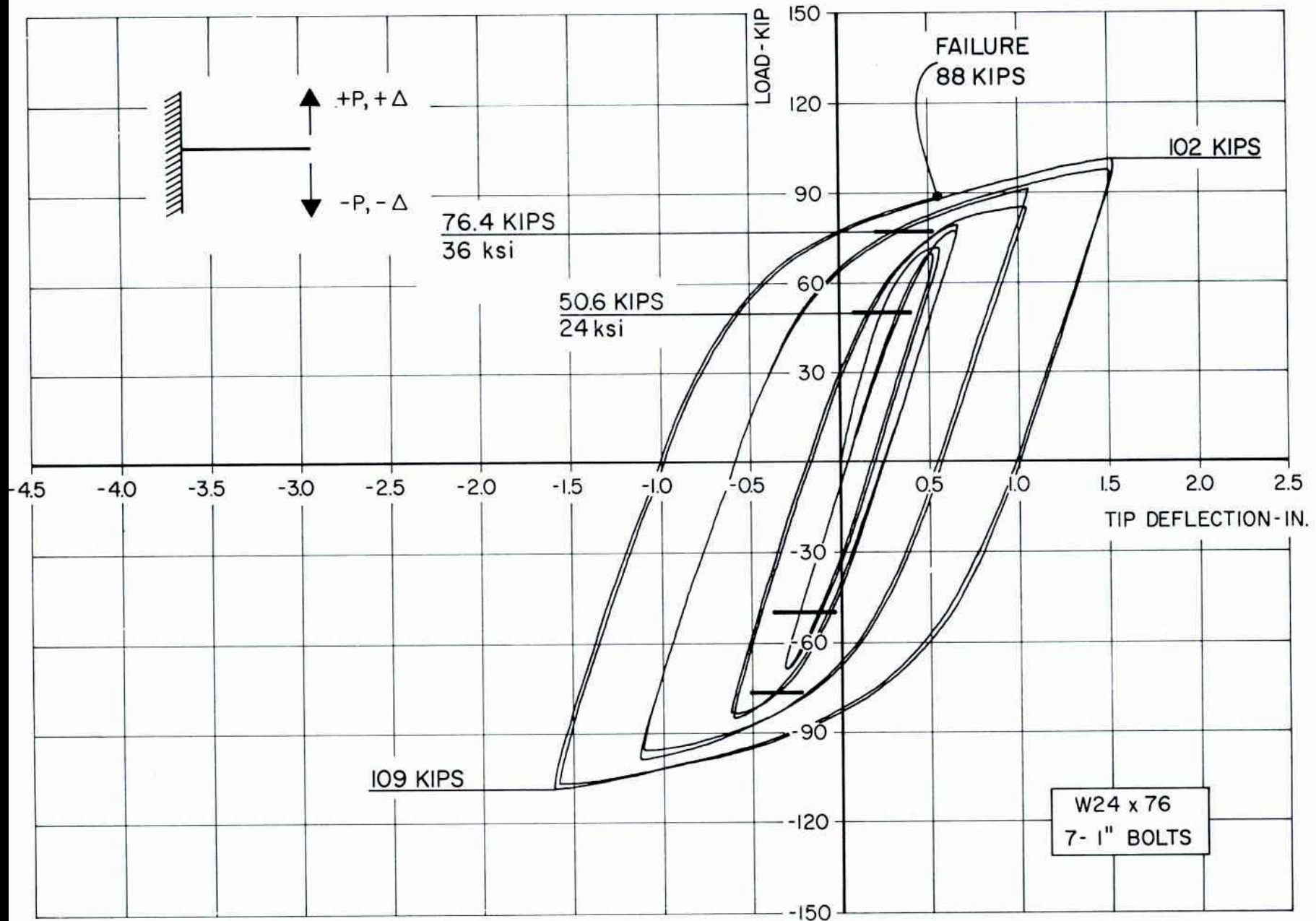
SECTION B-B



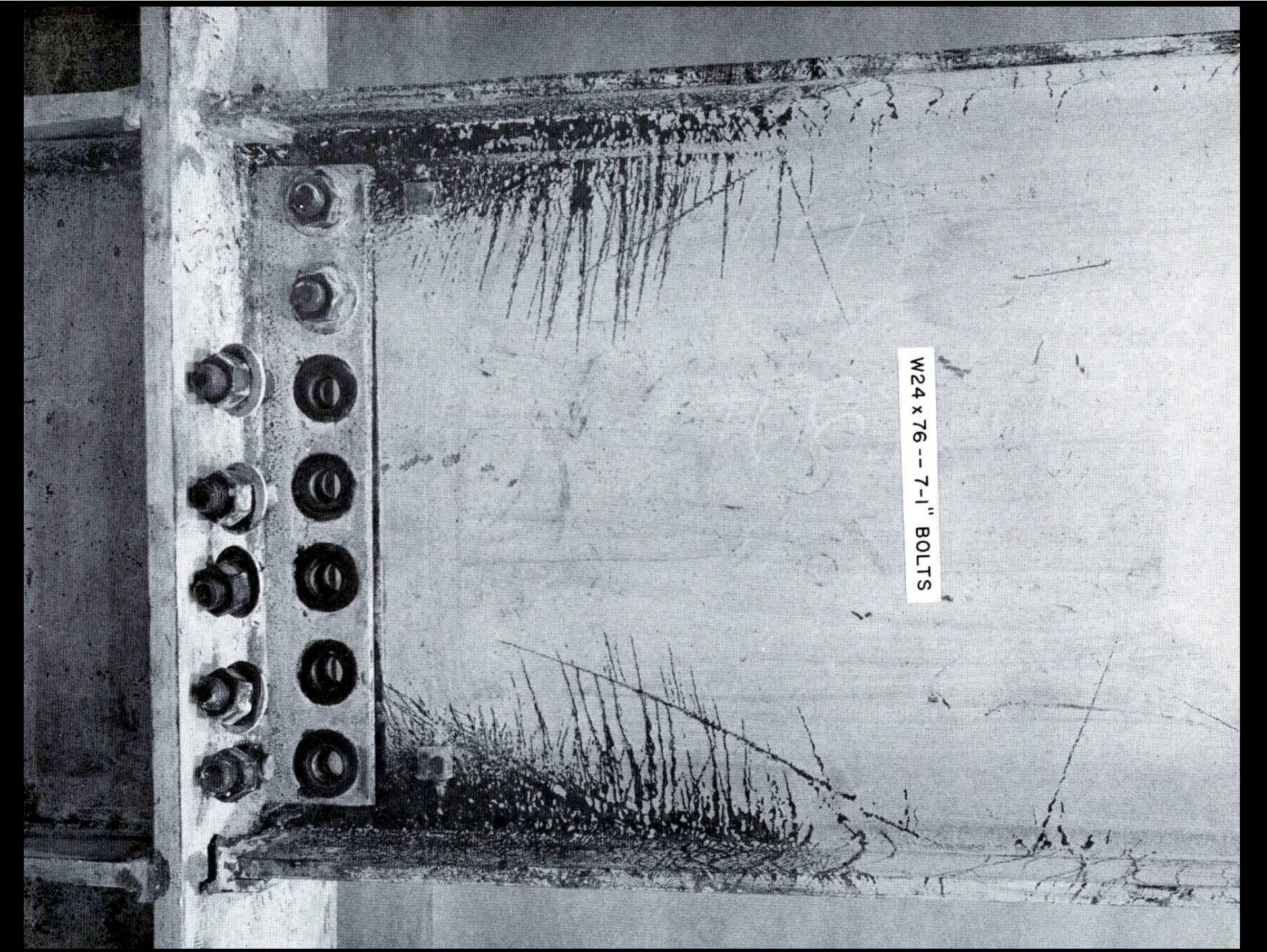
All-Welded Detail



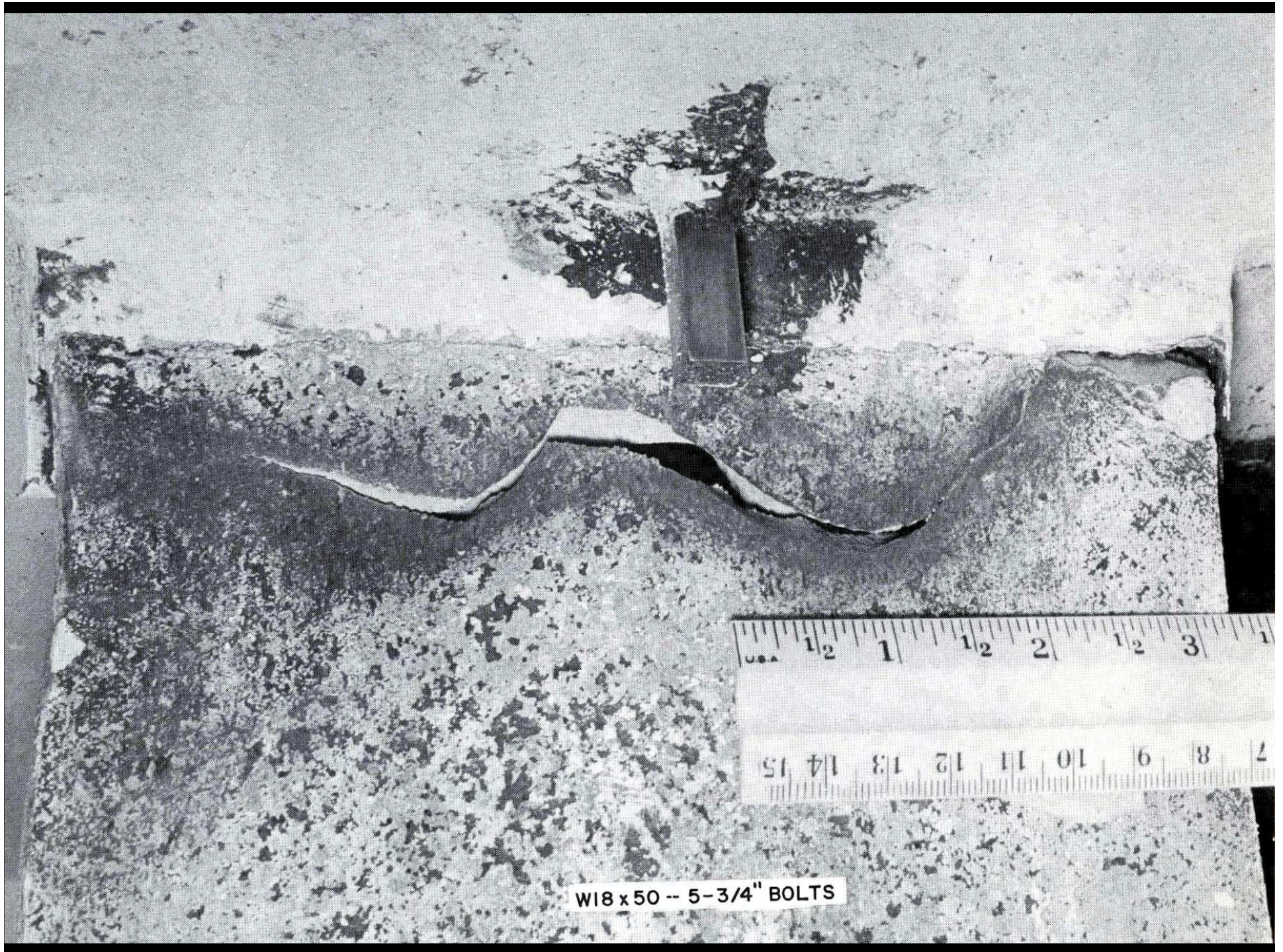
24 WF 76
ALL WELDED



Welded Flange – Bolted Web Detail

A black and white photograph showing a close-up of a steel beam-to-column connection. A vertical steel plate, likely a stiffener or part of the column web, is bolted to the side of a horizontal steel beam. The plate has two rows of bolts: an outer row of 7 bolts and an inner row of 6 bolts. The beam's top and bottom flanges are visible, showing some surface wear and paint. A white rectangular label with black text is positioned vertically on the right side of the image.

W24 x 76 -- 7-1" BOLTS



Observations from Initial UC Berkeley Tests:

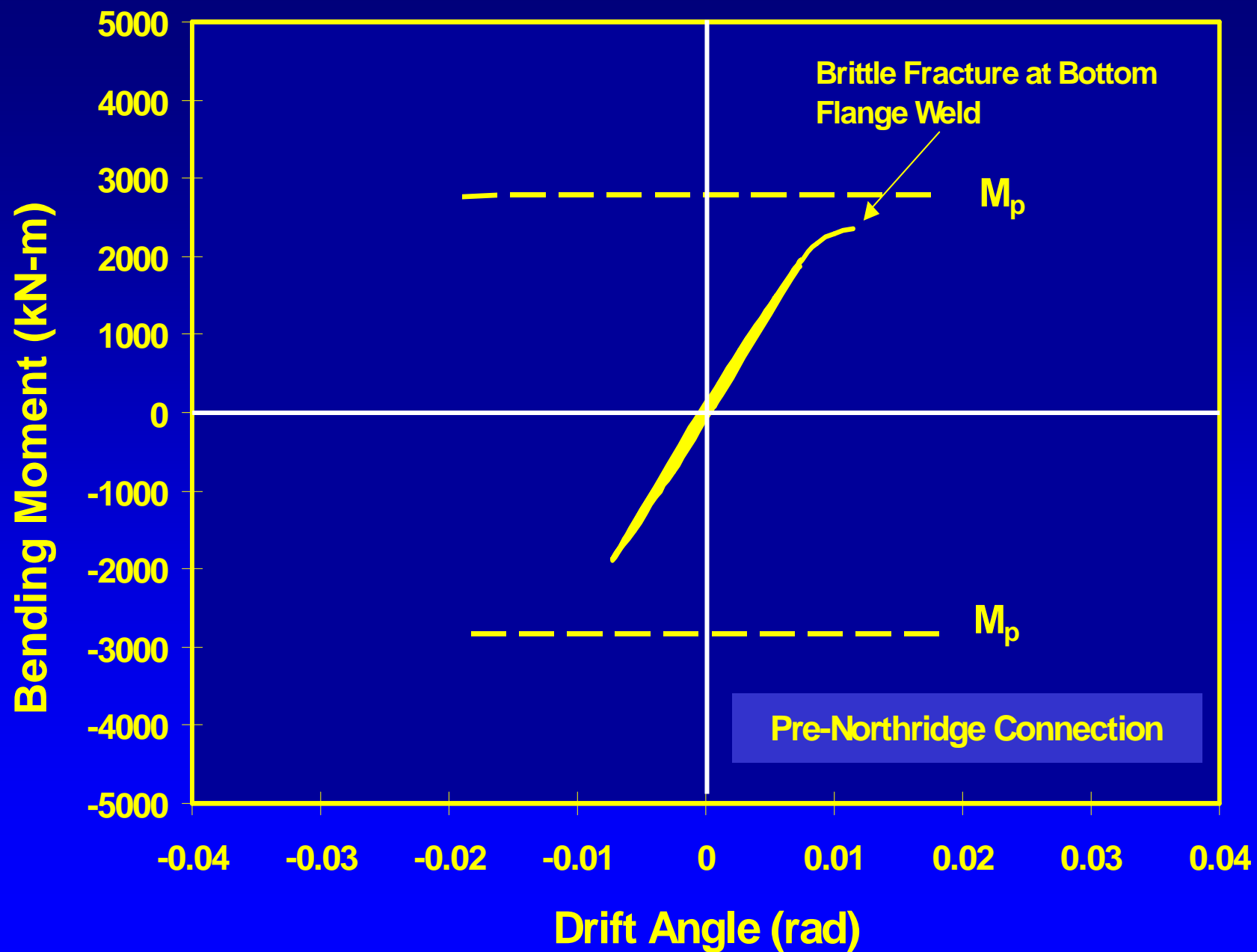
- Large ductility developed by all-welded connections.
- Welded flange-bolted web connections developed less ductility, but were viewed as still acceptable.

Subsequent Test Programs:

- **Welded flange-bolted web connections showed highly variable performance.**
- **Typical failure modes: fracture at or near beam flange groove welds.**
- **A large number of laboratory tested connections did not develop adequate ductility in the beam prior to connection failure.**







Summary of Testing Prior to Northridge Earthquake

- **Welded flange – bolted web connection showed highly variable performance**
- **Many connections failed in laboratory with little or no ductility**

1994 Northridge Earthquake

**Widespread failure of
welded flange - bolted
web moment
connections**



1994 Northridge Earthquake

- January 17, 1994
- Magnitude = 6.8
- Epicenter at Northridge - San Fernando Valley
(Los Angeles area)
- Fatalities: 58
- Estimated Damage Cost: \$20 Billion

Northridge - Ground Accelerations

- Sylmar: 0.91g H 0.60g V
- Sherman Oaks: 0.46g H 0.18g V
- Granada Hills: 0.62g H 0.40g V
- Santa Monica: 0.93g H 0.25g V
- North Hollywood: 0.33g H 0.15g V







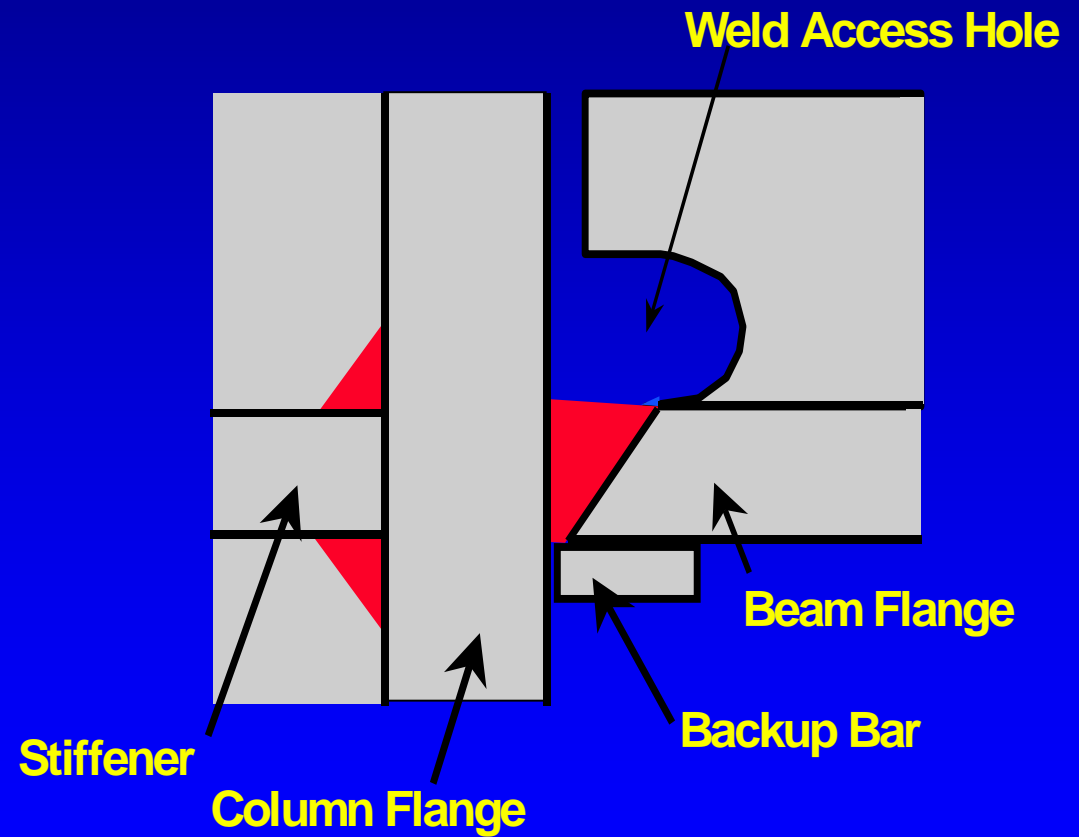
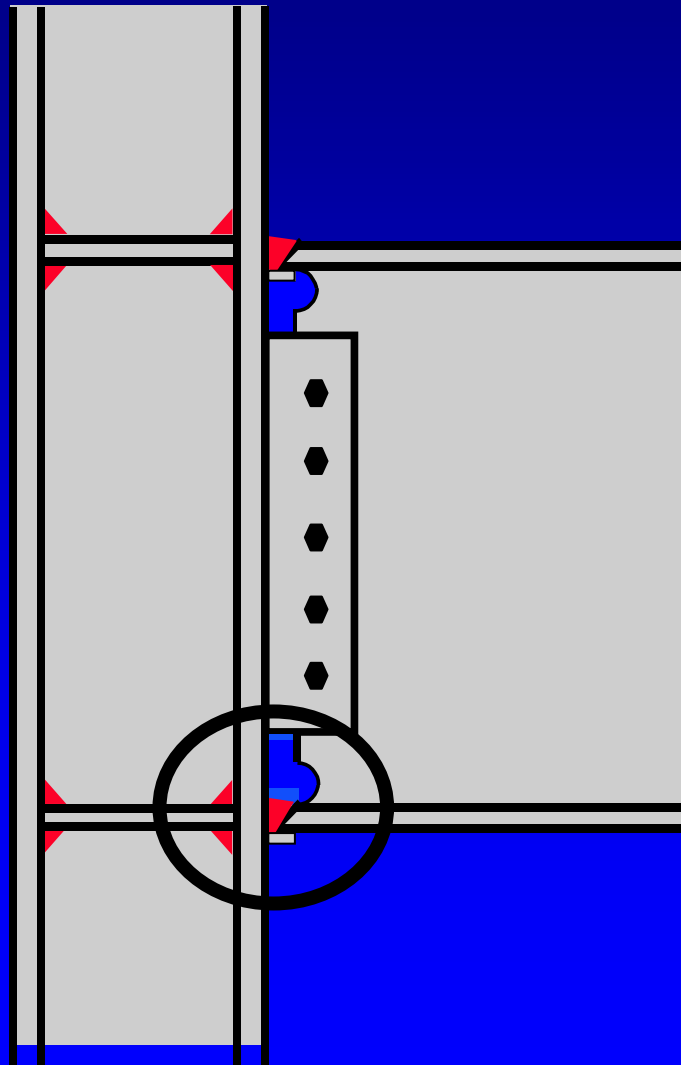
Damage to Steel Buildings in the Northridge Earthquake

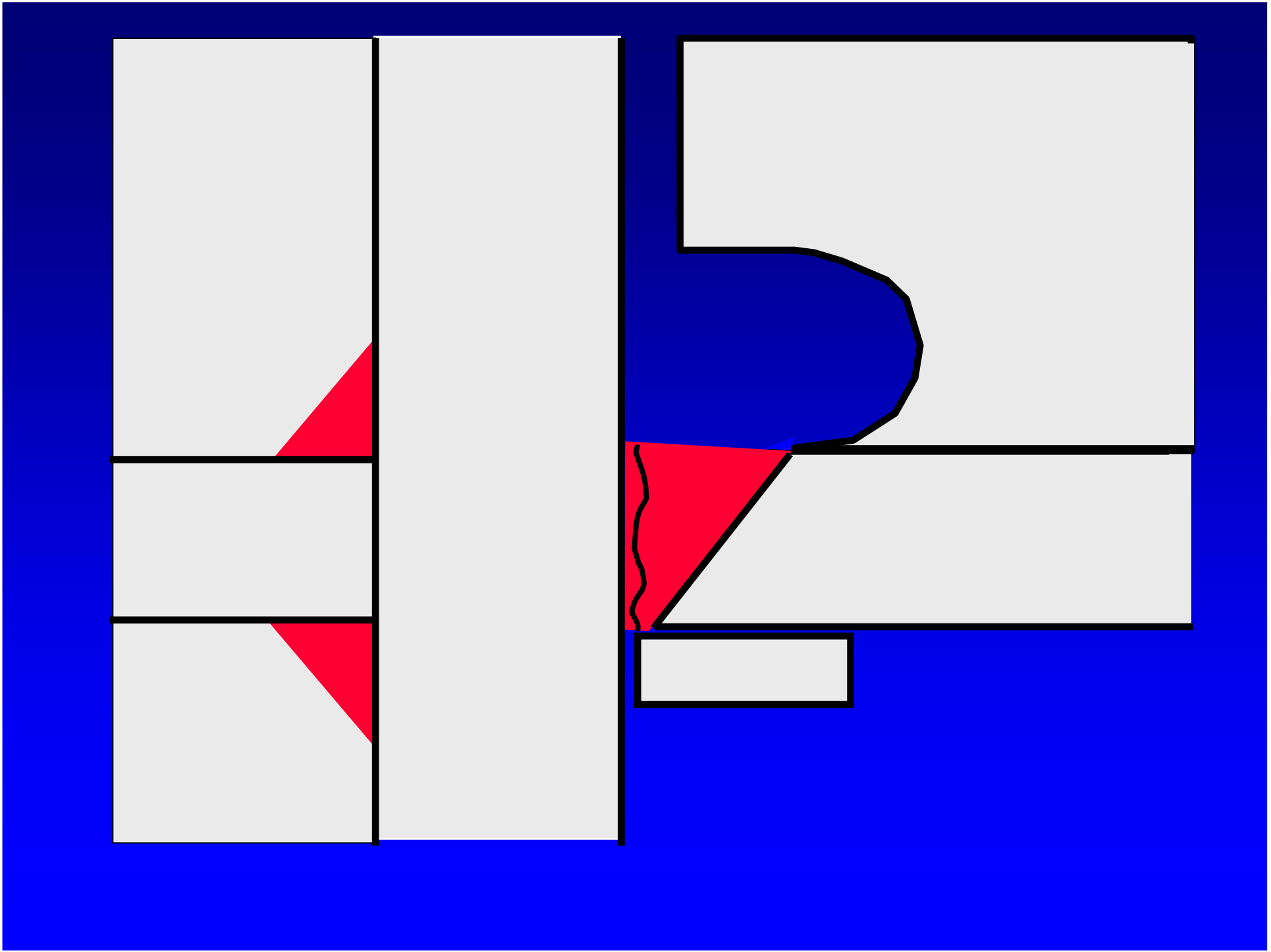
- Large number of modern steel buildings sustained severe damage at beam-to-column connections.
- Primary Damage: Fracture in and around beam flange groove welds
- Damage was largely unexpected by engineering profession

Damage Observations:

**Steel Moment
Connections**

Pre-Northridge Welded Flange – Bolted Web Moment Connection

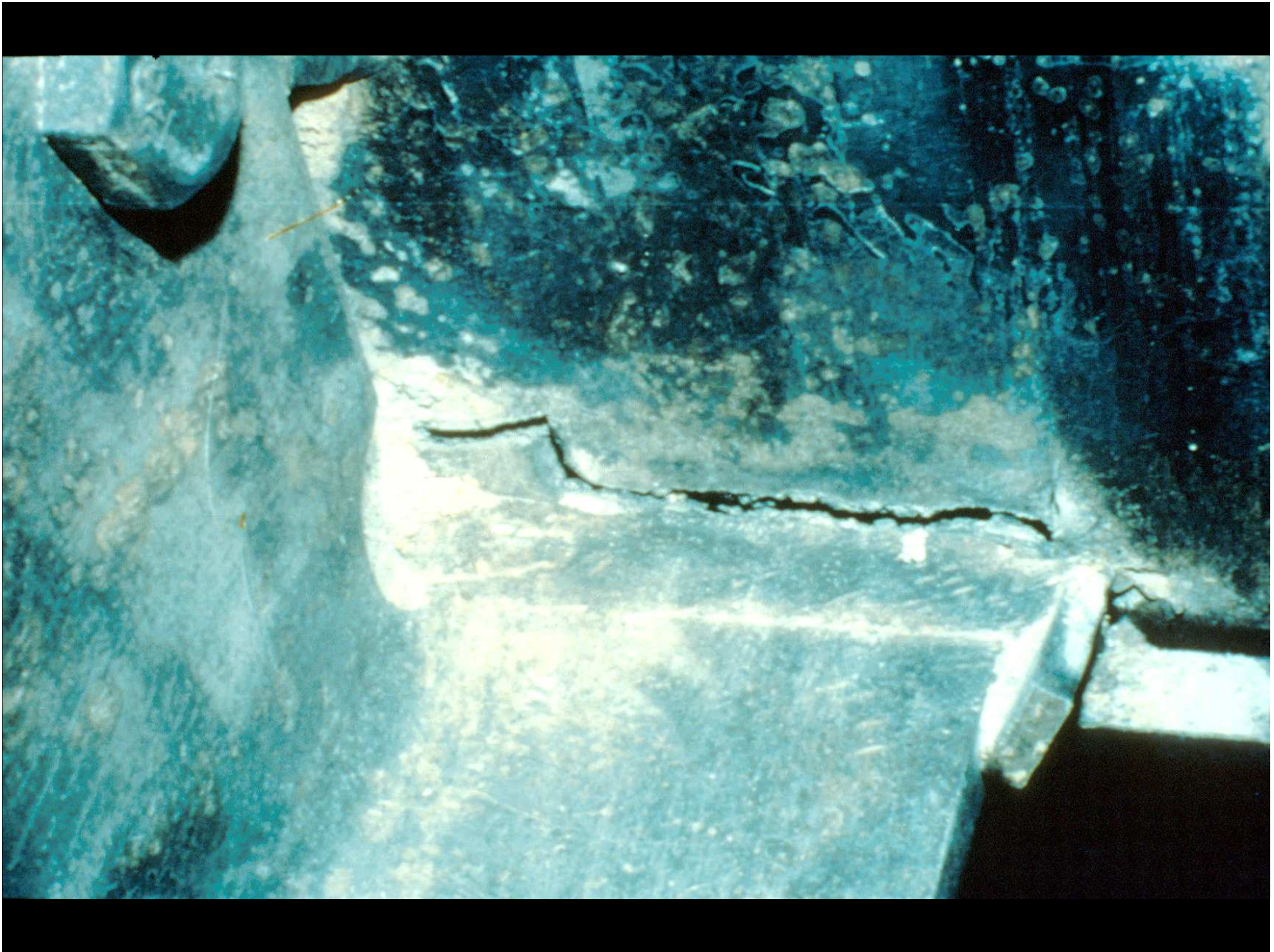


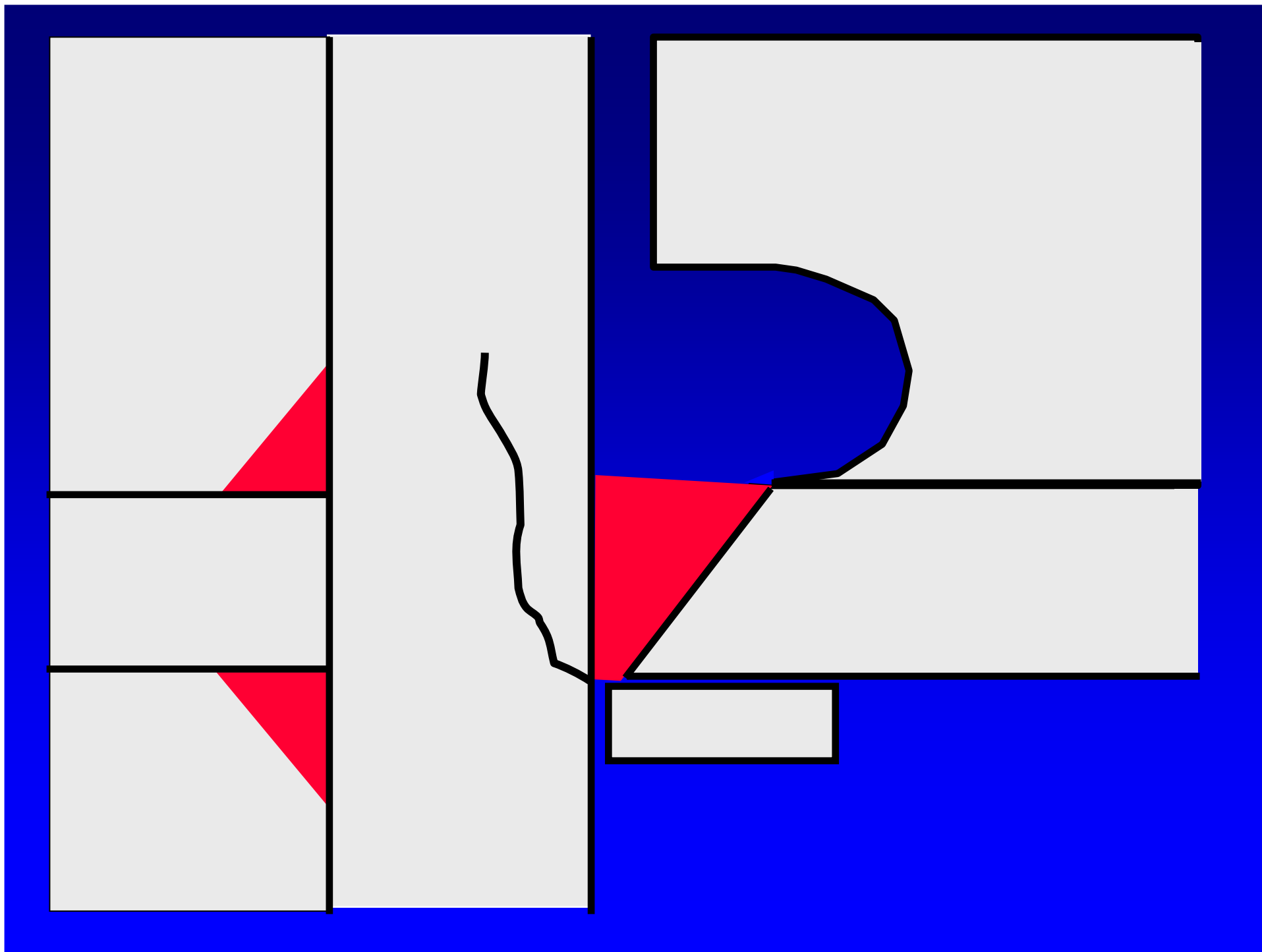


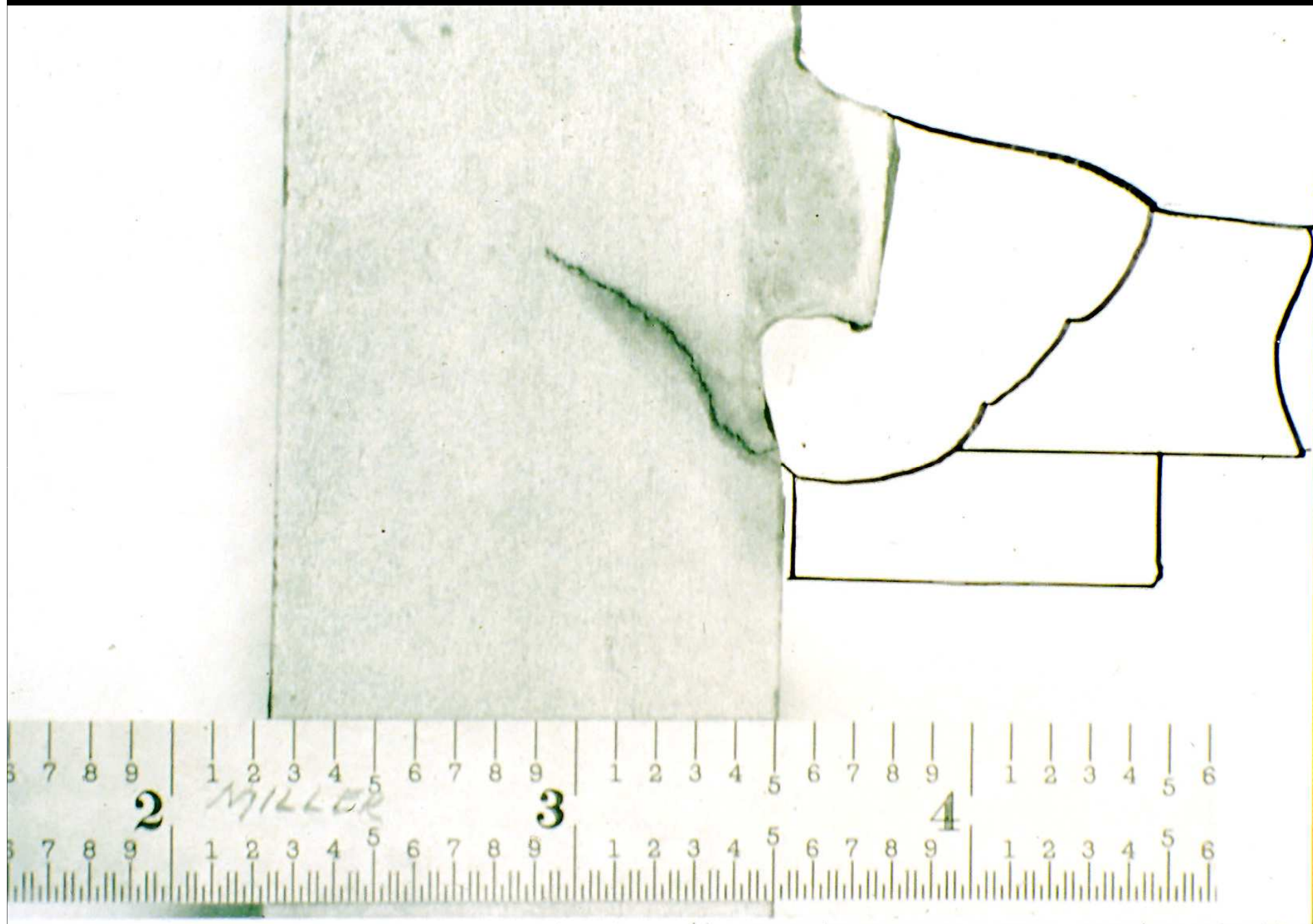


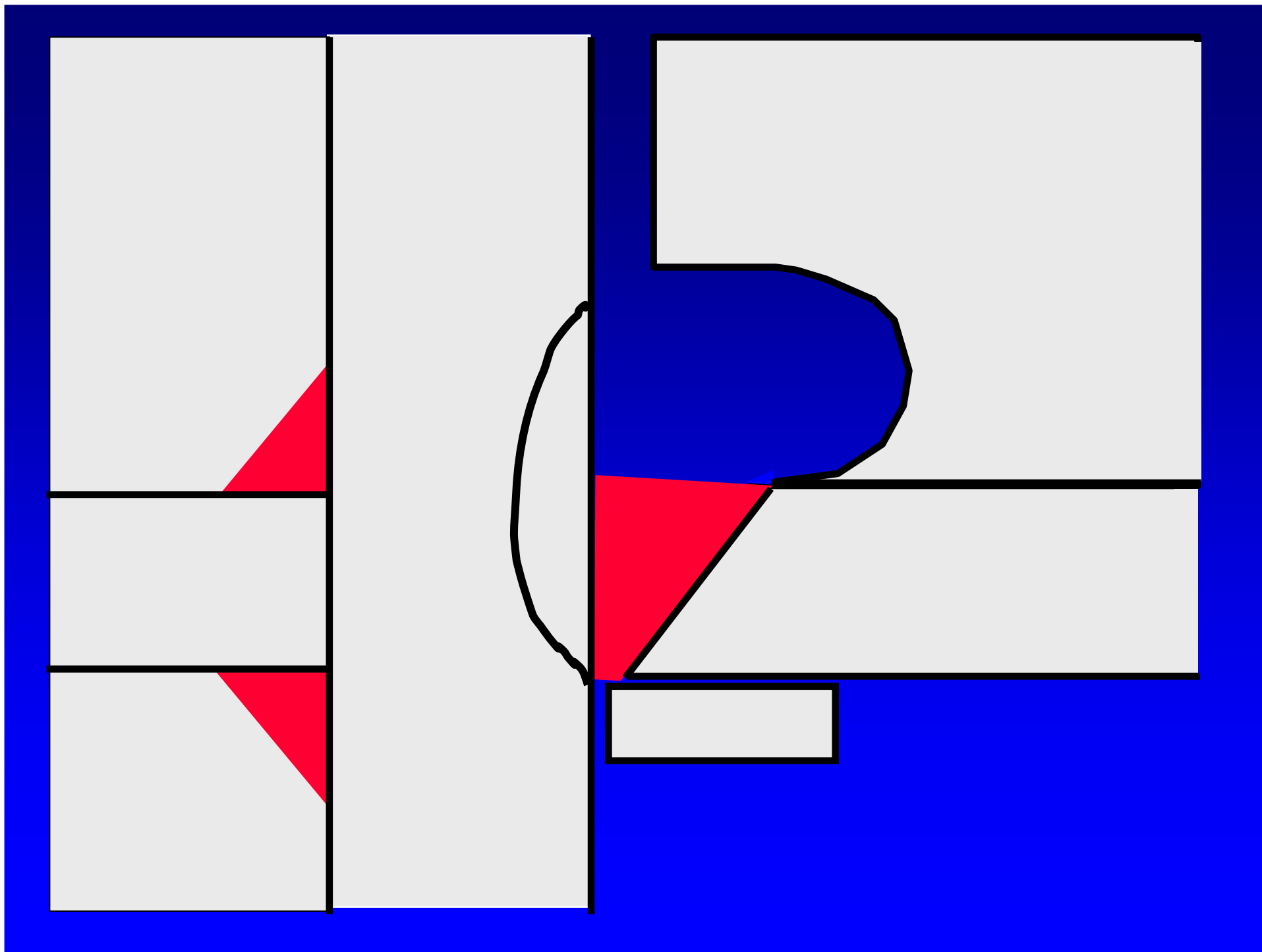










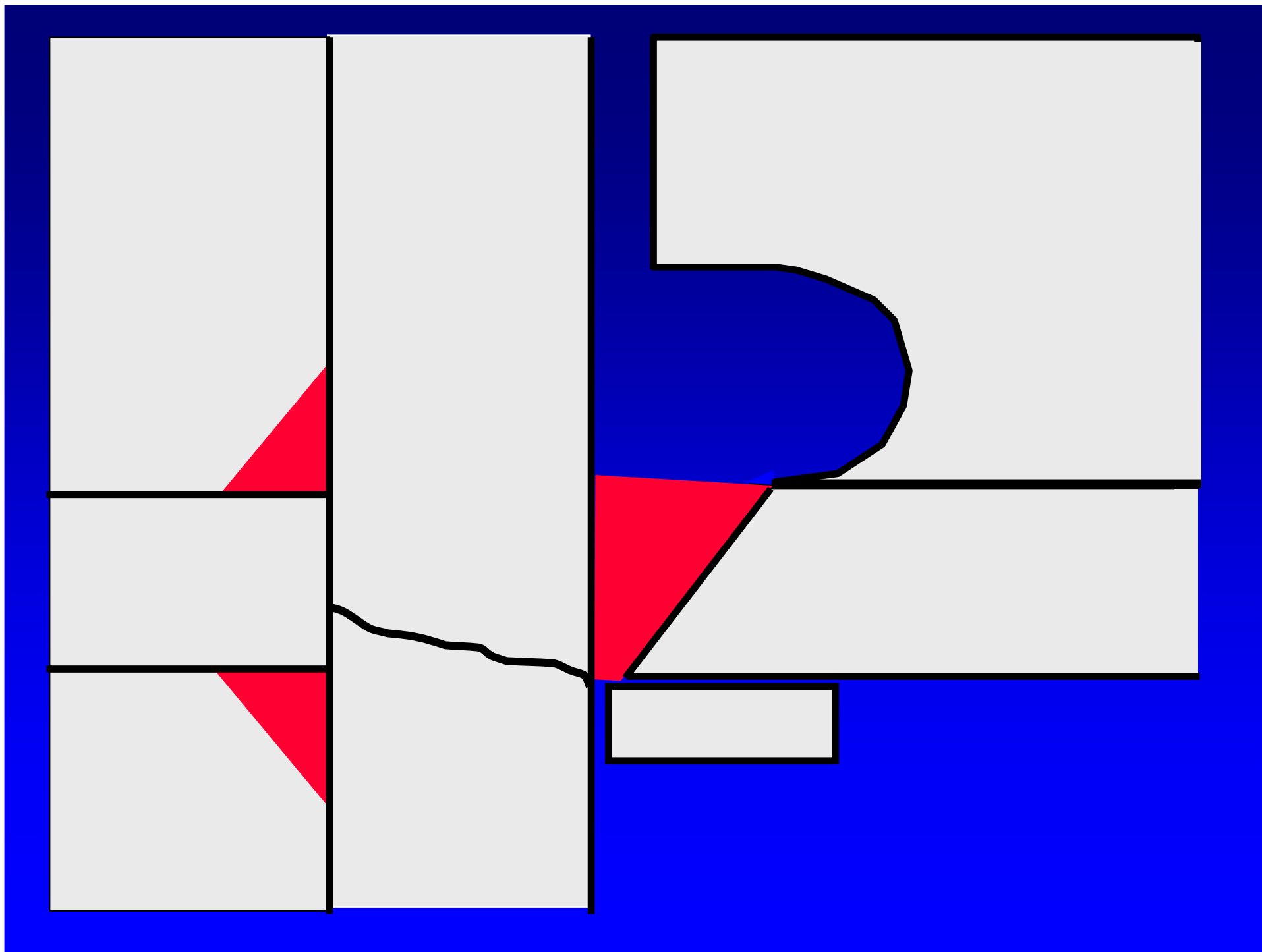


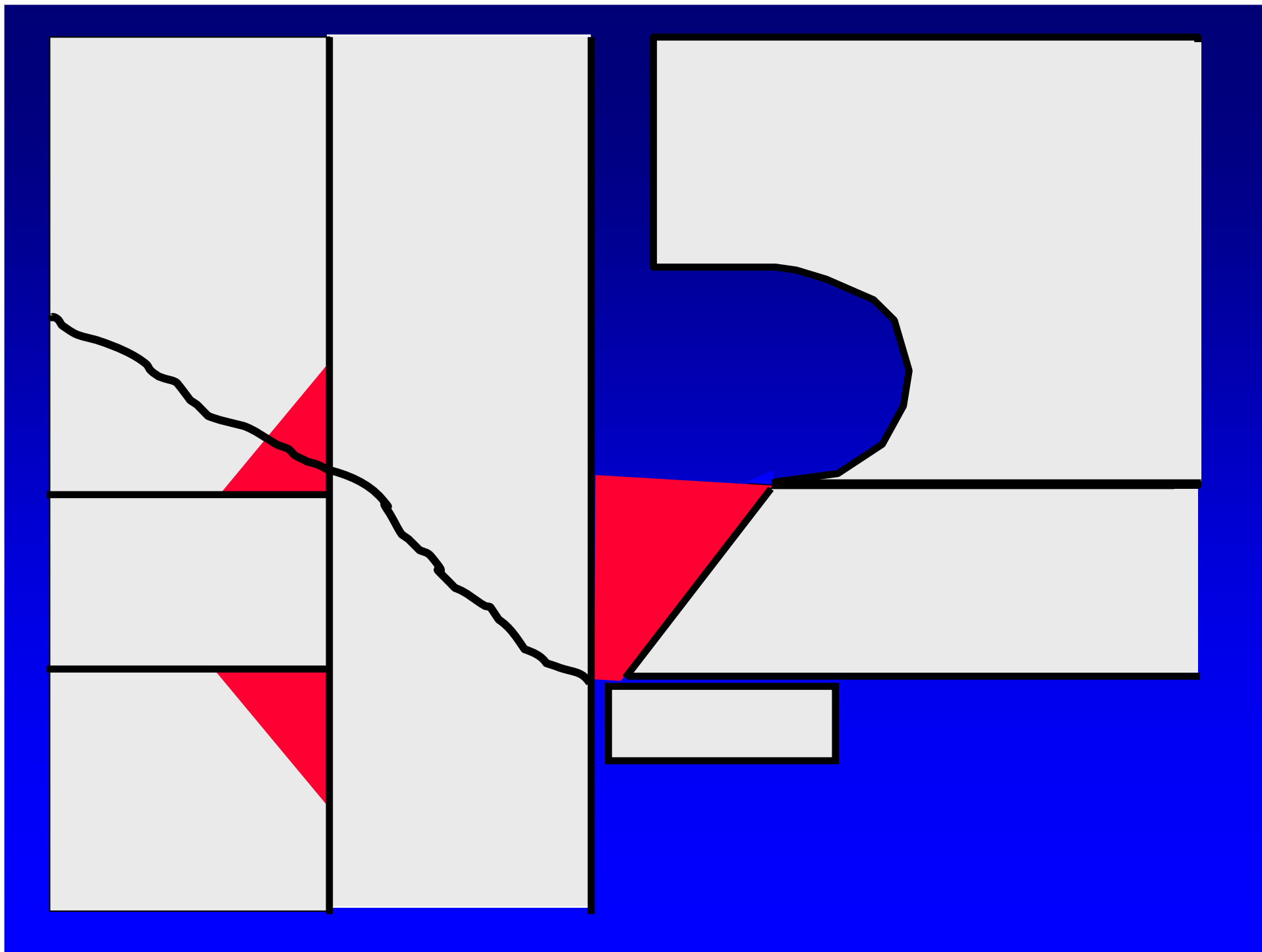








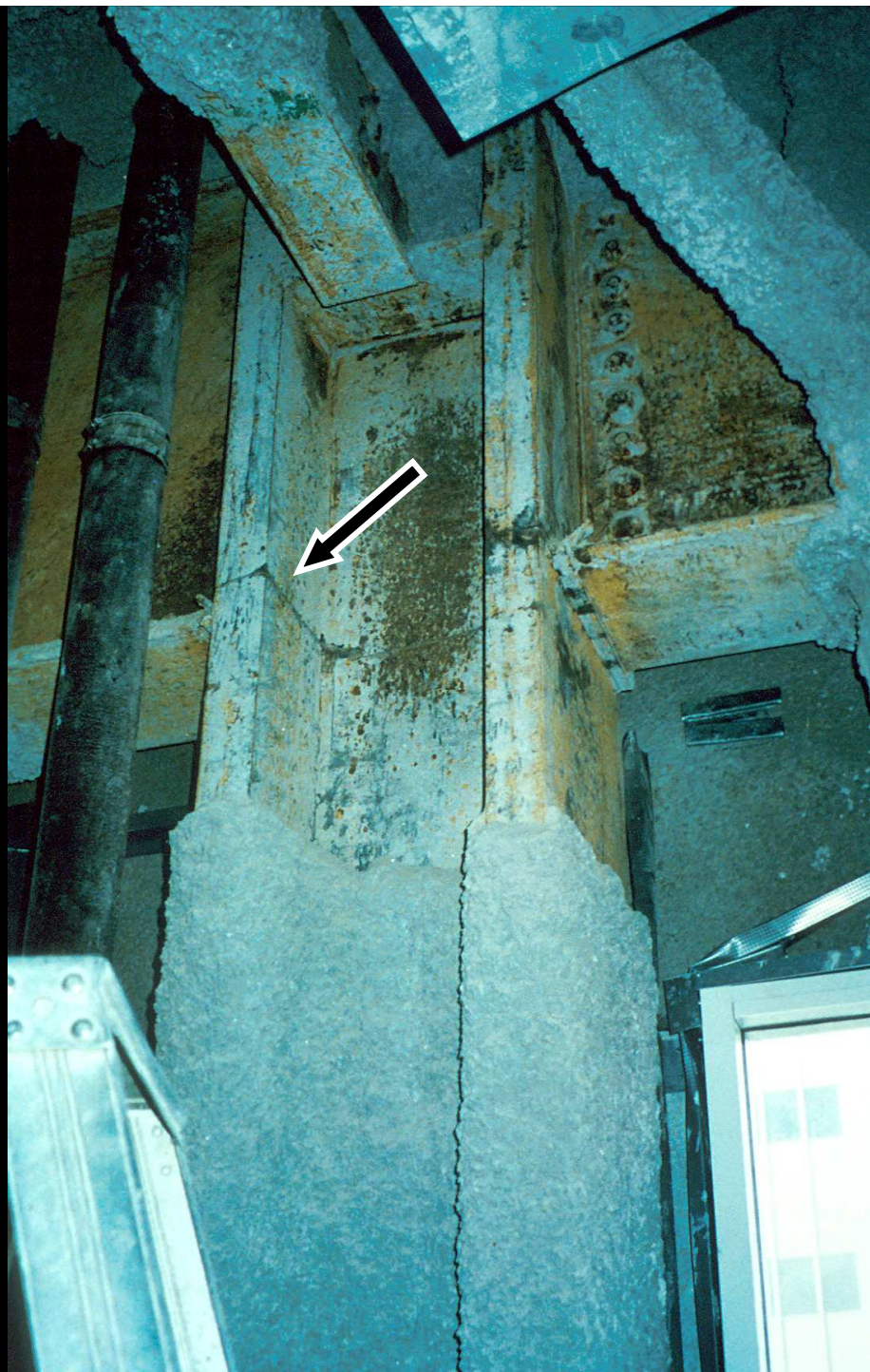












Damage Observations

- A large number of steel moment frame buildings suffered connection damage
- No steel moment frame buildings collapsed
- Typical Damage:
 - fracture of groove weld
 - “divot” fracture within column flange
 - fracture across column flange and web

Observations from Studies of Fractured Connections

- Many connections failed by brittle fracture with little or no ductility
- Brittle fractures typically initiated in beam flange groove welds

Response to Northridge Moment Connection Damage

- Nearly immediate elimination of welded flange - bolted web connection from US building codes and design practice
- Intensive research and testing efforts to understand causes of damage and to develop improved connections
 - AISC, NIST, NSF, etc.
 - SAC Program (FEMA)

Causes of Moment Connection Damage in Northridge

- **Welding**
- **Connection Design**
- **Materials**

Causes of Northridge Moment Connection Damage:

Welding Factors

- Low Fracture Toughness of Weld Metal
- Poor Quality
- Effect of Backing Bars and Weld Tabs

Weld Metal Toughness

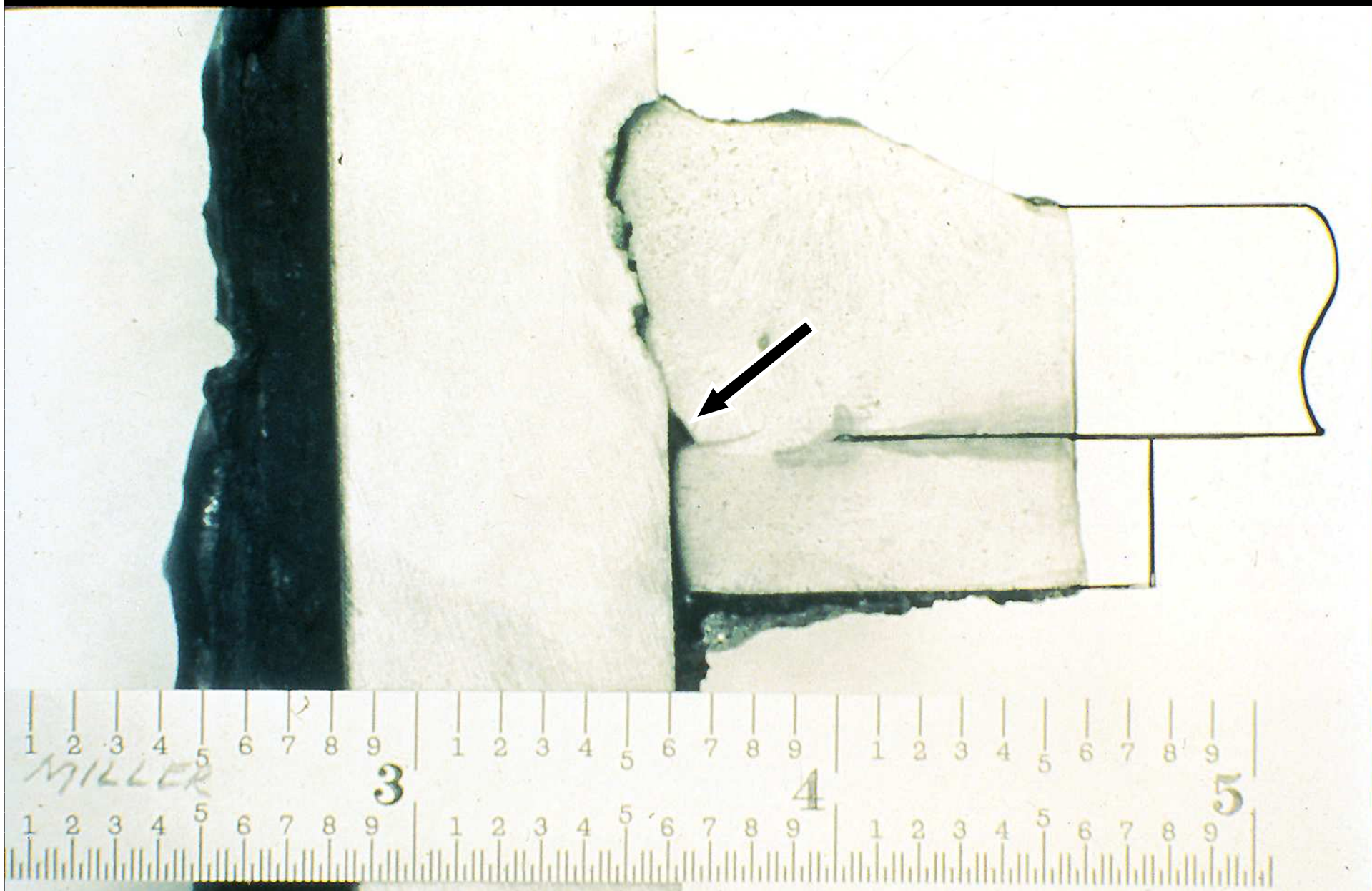
- Most common Pre-Northridge welding electrode (E70T-4) had very low fracture toughness.

Typical Charpy V-Notch: < 5 ft.-lbs at 70°F
(7 J at 21°C)



Welding Quality

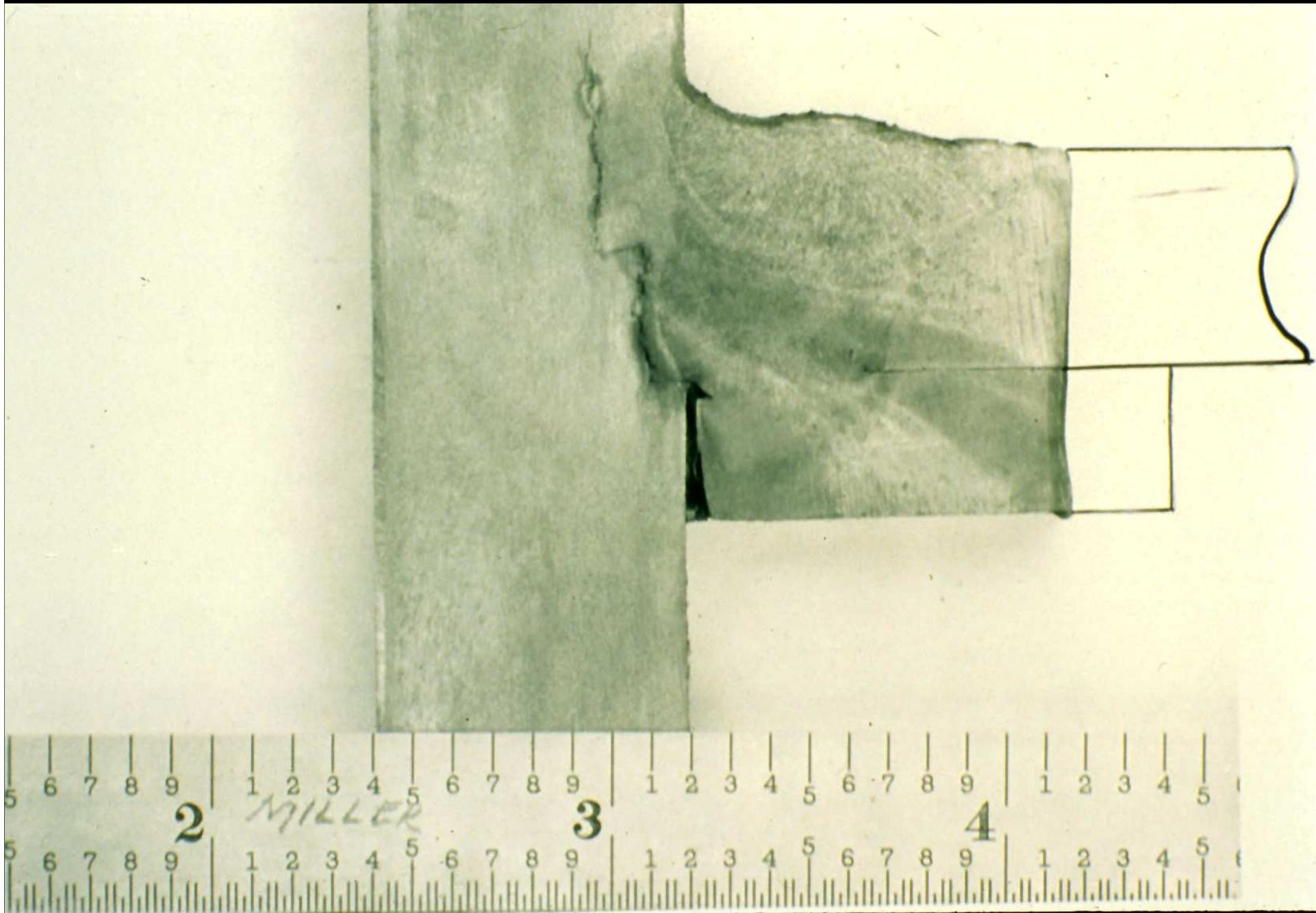
- Many failed connections showed evidence of poor weld quality
- Many fractures initiated at root defects in bottom flange weld, in vicinity of weld access hole





Weld Backing Bars and Weld Tabs

- Backing Bars:
 - Can create notch effect
 - Increases difficulty of inspection
- Weld Tabs:
 - Weld runoff regions at weld tabs contain numerous discontinuities that can potentially initiate fracture





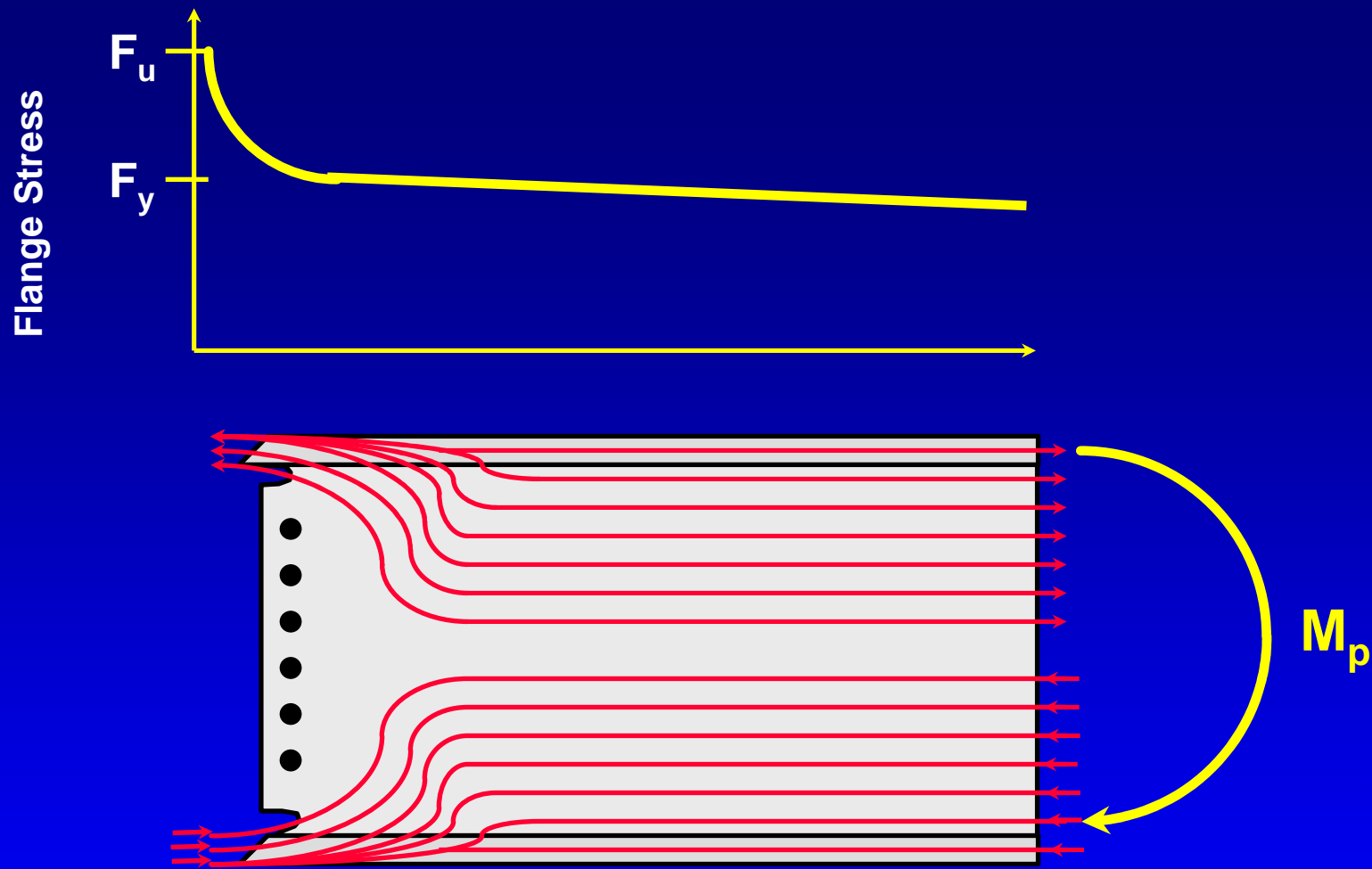


Causes of Northridge Moment Connection Damage:

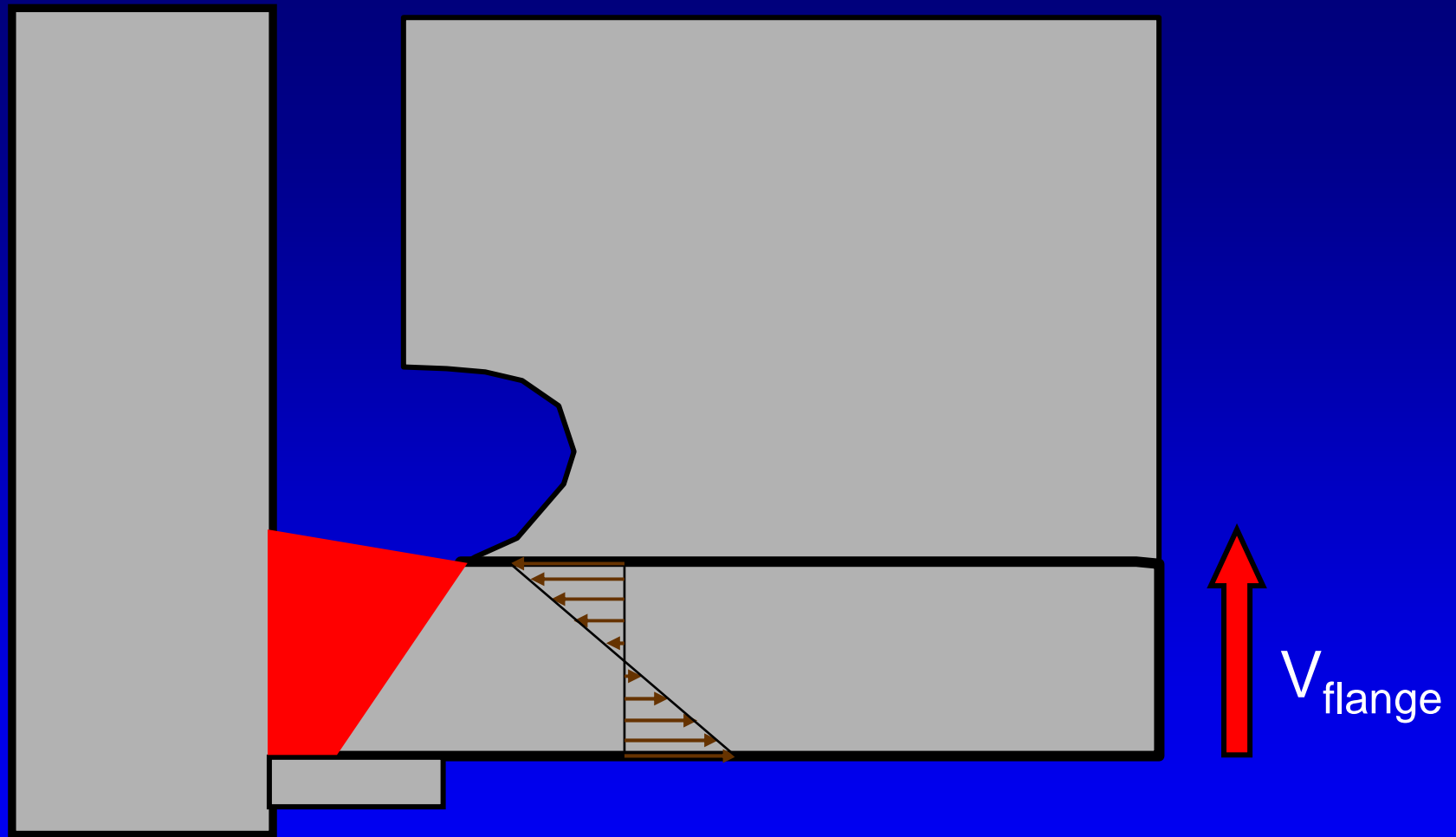
Design Factors:

Stress/Strain Too High at Beam Flange Groove Weld

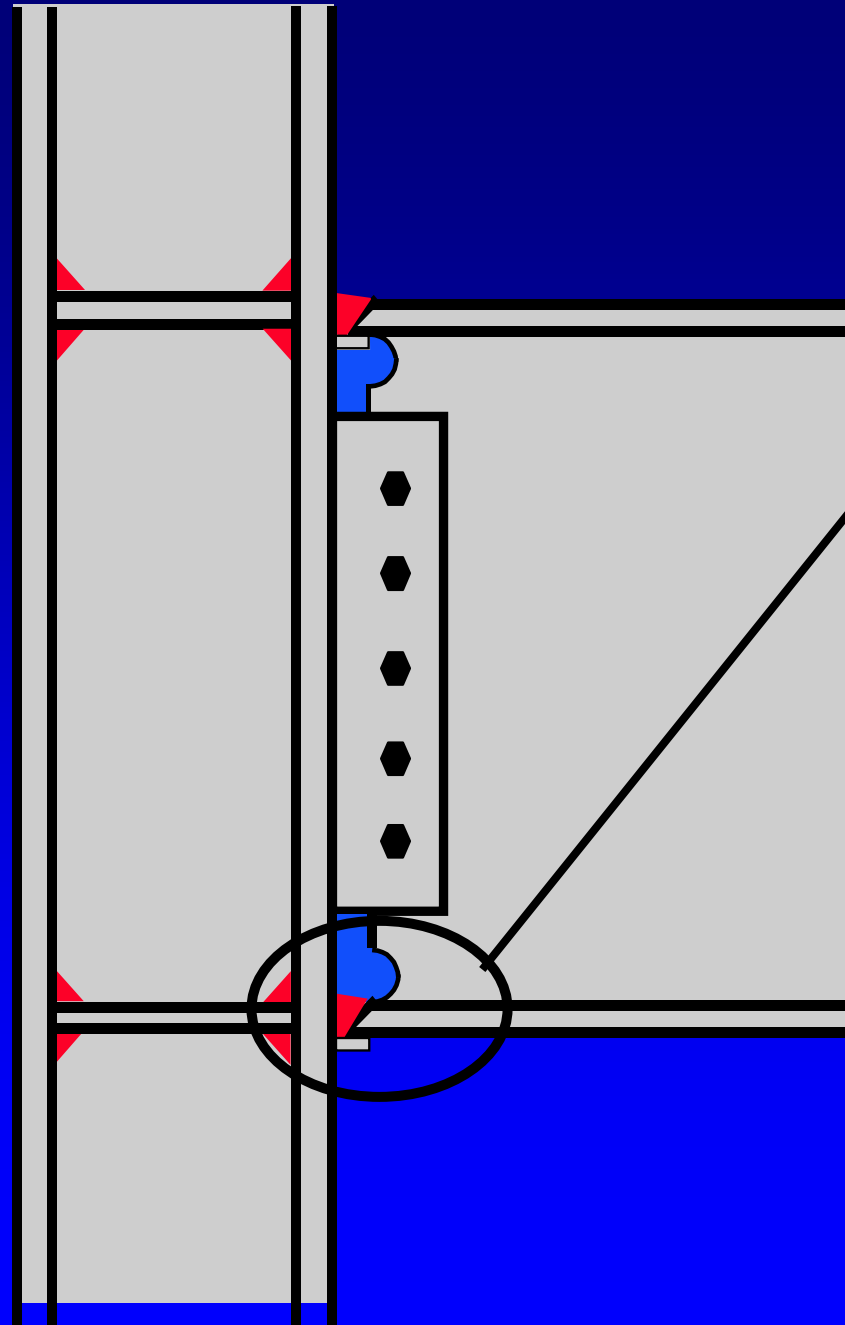
- Inadequate Participation of Beam Web Connection in Transferring Moment and Shear
- Effect of Weld Access Hole
- Effect of Column Flange Bending
- Other Factors



**Increase in Flange Stress Due to
Inadequate Moment Transfer Through Web Connection**

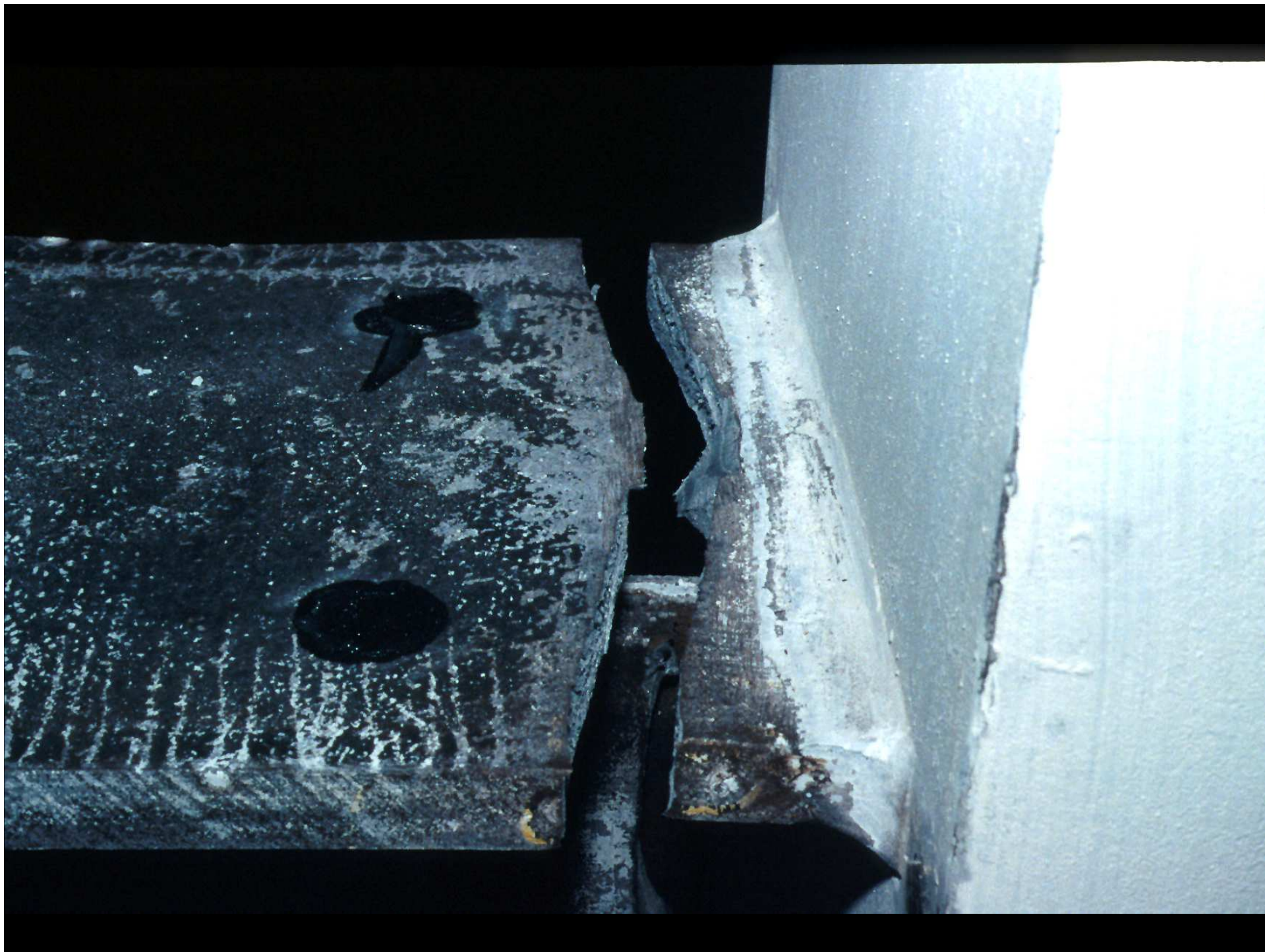


Increase in Flange Stress Due to Shear in Flange



Stress Concentrations:

- Weld access hole
- Shear in flange
- Inadequate flexural participation of web connection



Causes of Moment Connection Damage in Northridge:

Material Factors (Structural Steel)

- Actual yield stress of A36 beams often significantly higher than minimum specified

Strategies for Improved Performance of Moment Connections

- **Welding**
- **Materials**
- **Connection Design and Detailing**

Strategies for Improved Performance of Moment Connections:

WELDING

- Required minimum toughness for weld metal:
 - Required CVN for all welds in SLRS:
20 ft.-lbs at 0° F
 - Required CVN for *Demand Critical* welds:
20 ft.-lbs at -20° F and 40 ft.-lbs at 70° F

Strategies for Improved Performance of Moment Connections:

WELDING

- Improved practices for backing bars and weld tabs

Typical improved practice:

- Remove bottom flange backing bar
 - Seal weld top flange backing bar
 - Remove weld tabs at top and bottom flange welds
- Greater emphasis on quality and quality control (AISC Seismic Provisions - Appendix Q and W)









Strategies for Improved Performance of Moment Connections:

Materials (Structural Steel)

- Introduction of “expected yield stress” into design codes

$$\text{Expected Yield Stress} = R_y F_y$$

F_y = minimum specified yield strength

R_y = 1.5 for ASTM A36

= 1.1 for A572 Gr. 50 and A992

(See AISC Seismic Provisions - Section 6 for other values of R_y)

Strategies for Improved Performance of Moment Connections:

Materials (Structural Steel)

- Introduction of ASTM A992 steel for wide flange shapes

ASTM A992

Minimum $F_y = 50$ ksi

Maximum $F_y = 65$ ksi

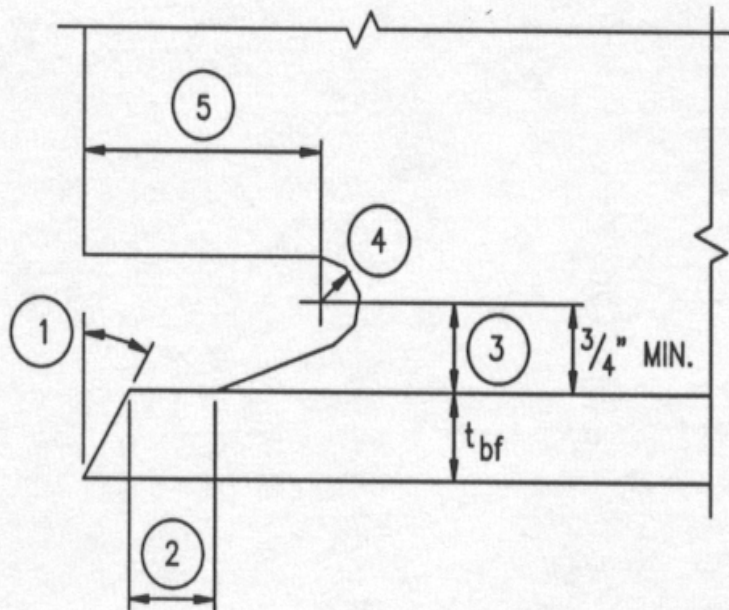
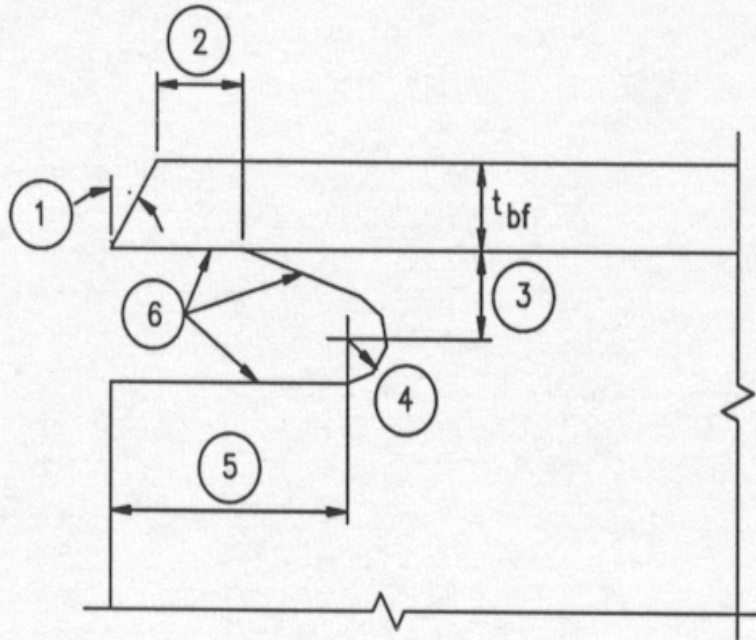
Minimum $F_u = 65$ ksi

Maximum $F_y / F_u = 0.85$

Strategies for Improved Performance of Moment Connections:

Connection Design

- Improved Weld Access Hole Geometry



Improved Weld Access Hole

See Figure 11-1 in the 2005 AISC *Seismic Provisions* for dimensions and finish requirements



Strategies for Improved Performance of Moment Connections:

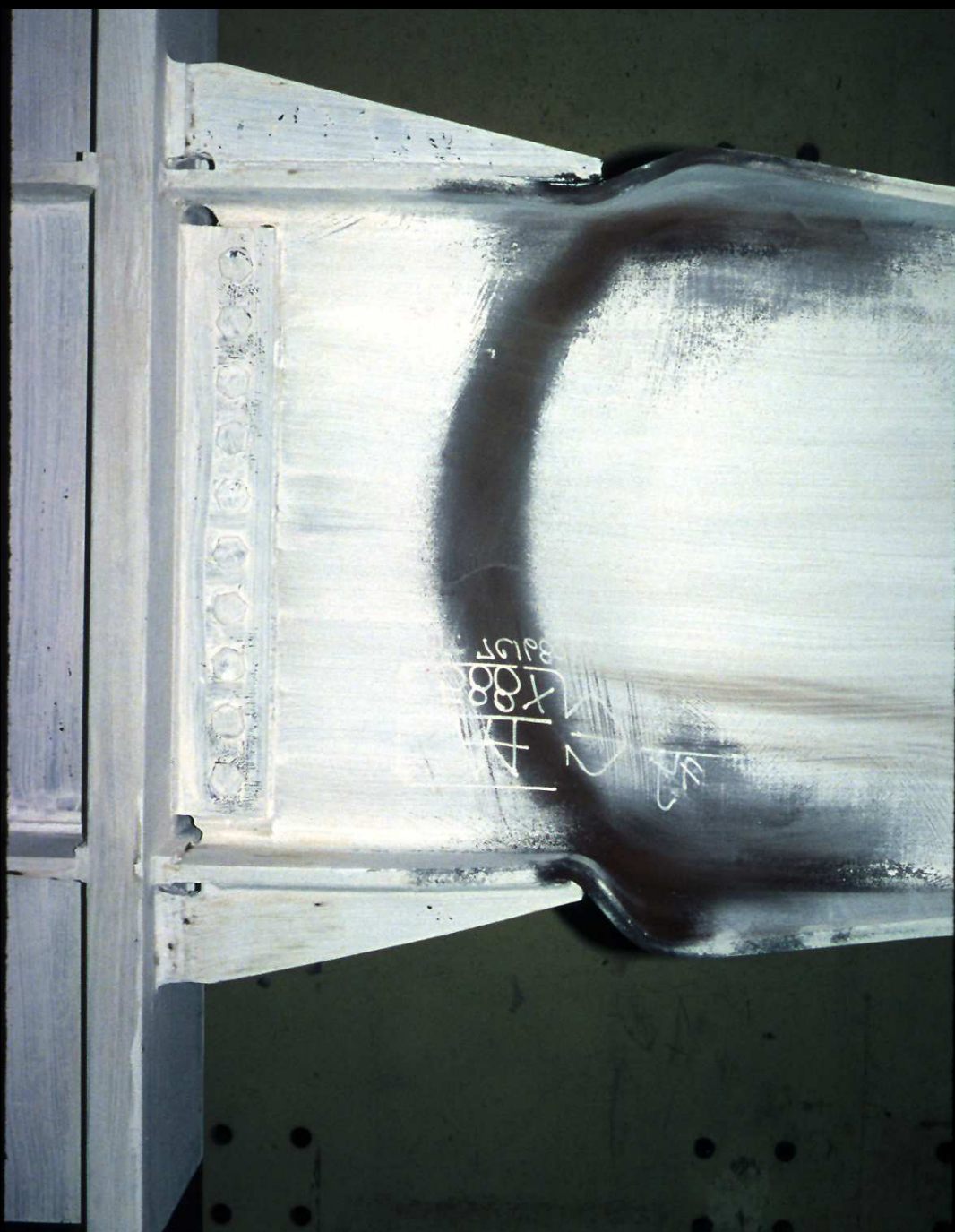
Connection Design

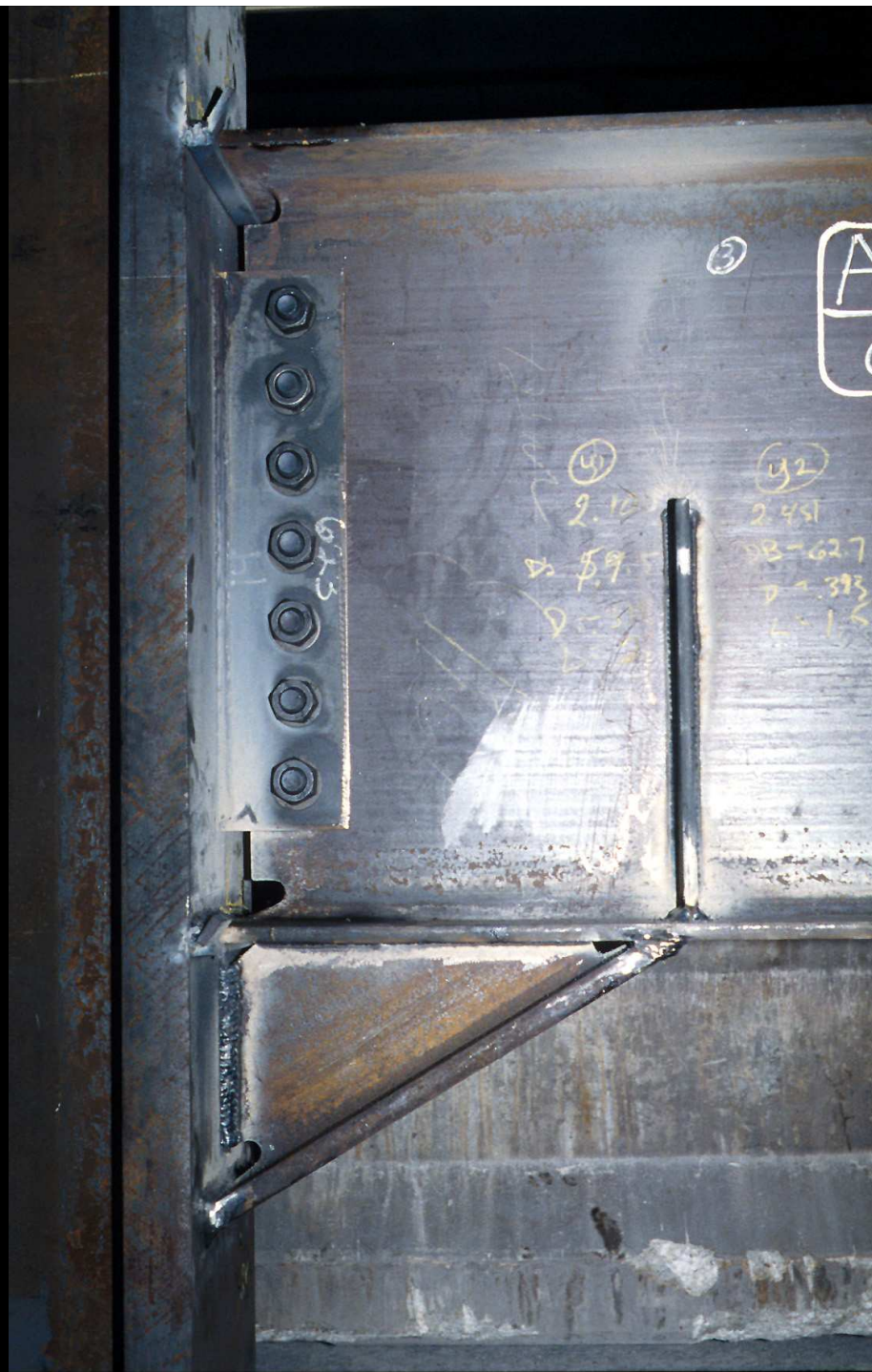
- Development of Improved Connection Designs and Design Procedures
 - Reinforced Connections
 - Proprietary Connections
 - Reduced Beam Section (Dogbone) Connections
 - Other SAC Investigated Connections







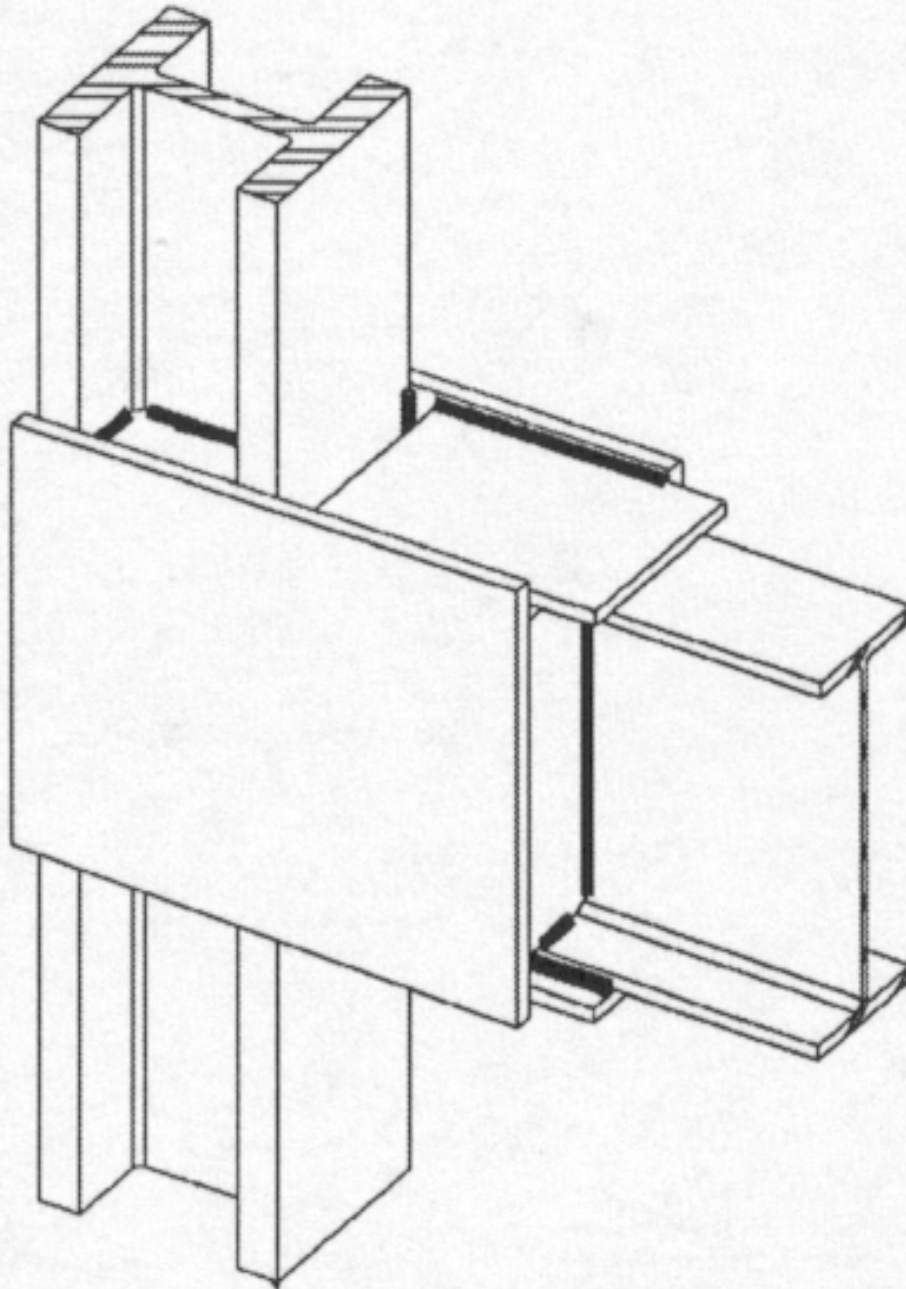




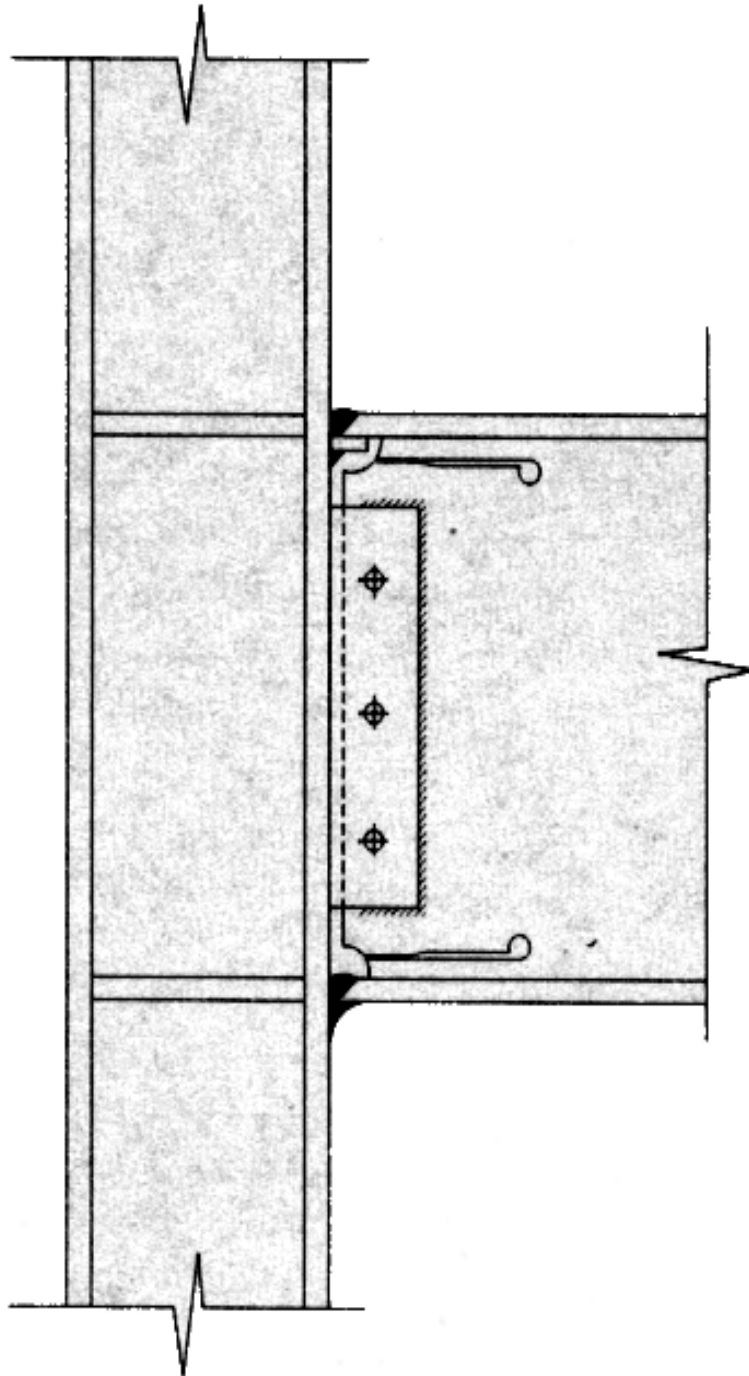




Proprietary Connections



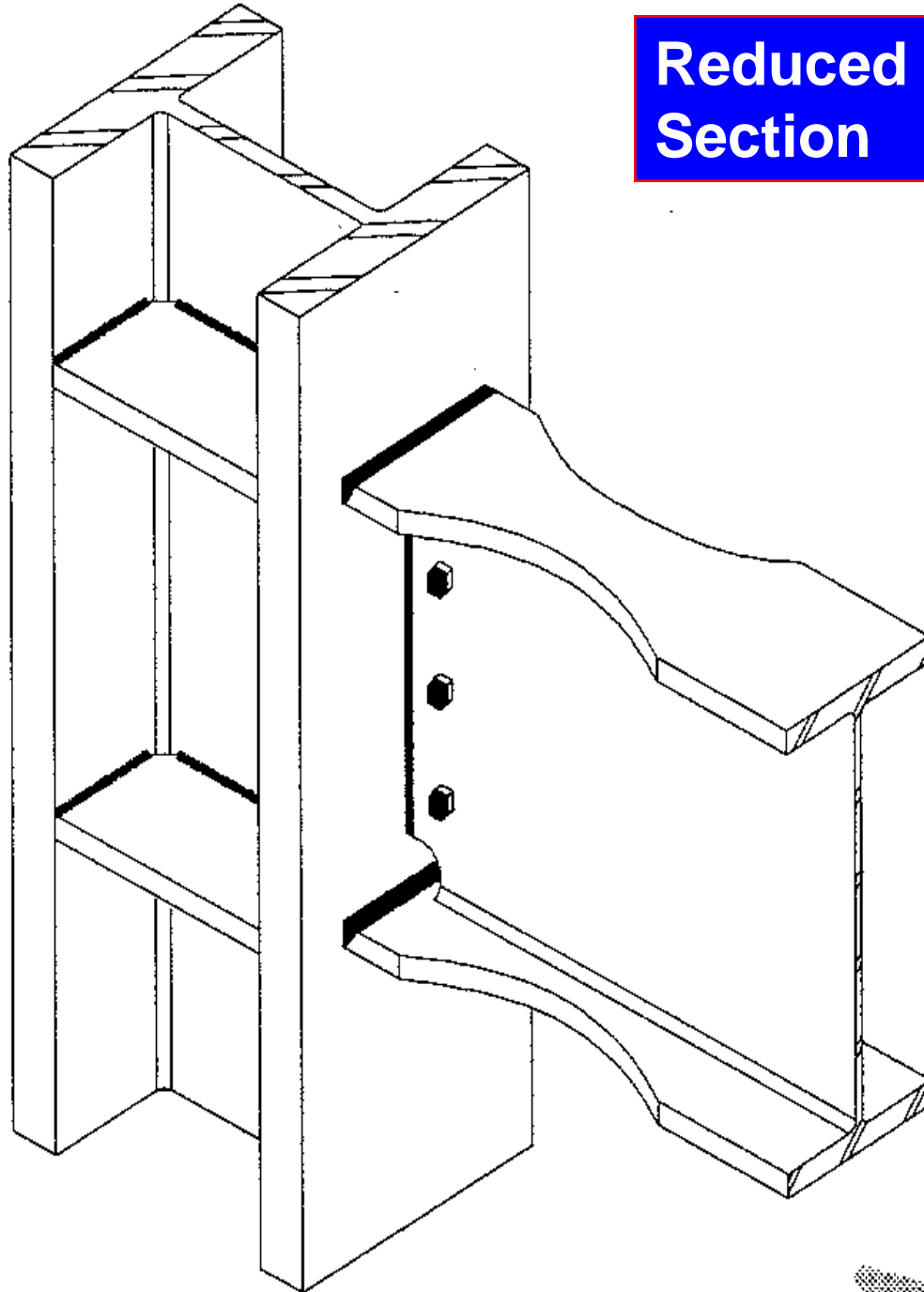
SIDE PLATE CONNECTION



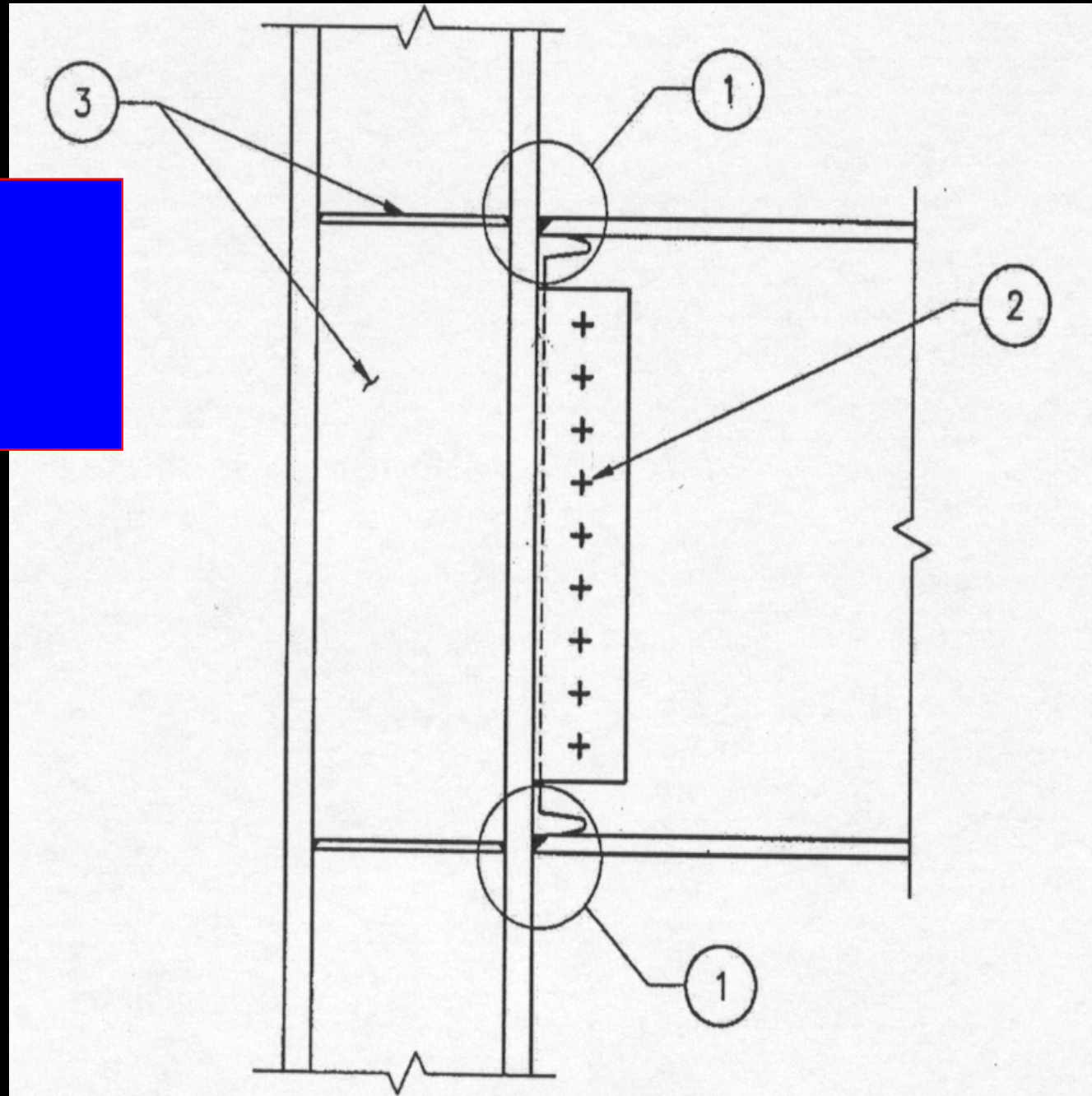
**SLOTTED WEB
CONNECTION**

Connections Investigated Through SAC-FEMA Research Program

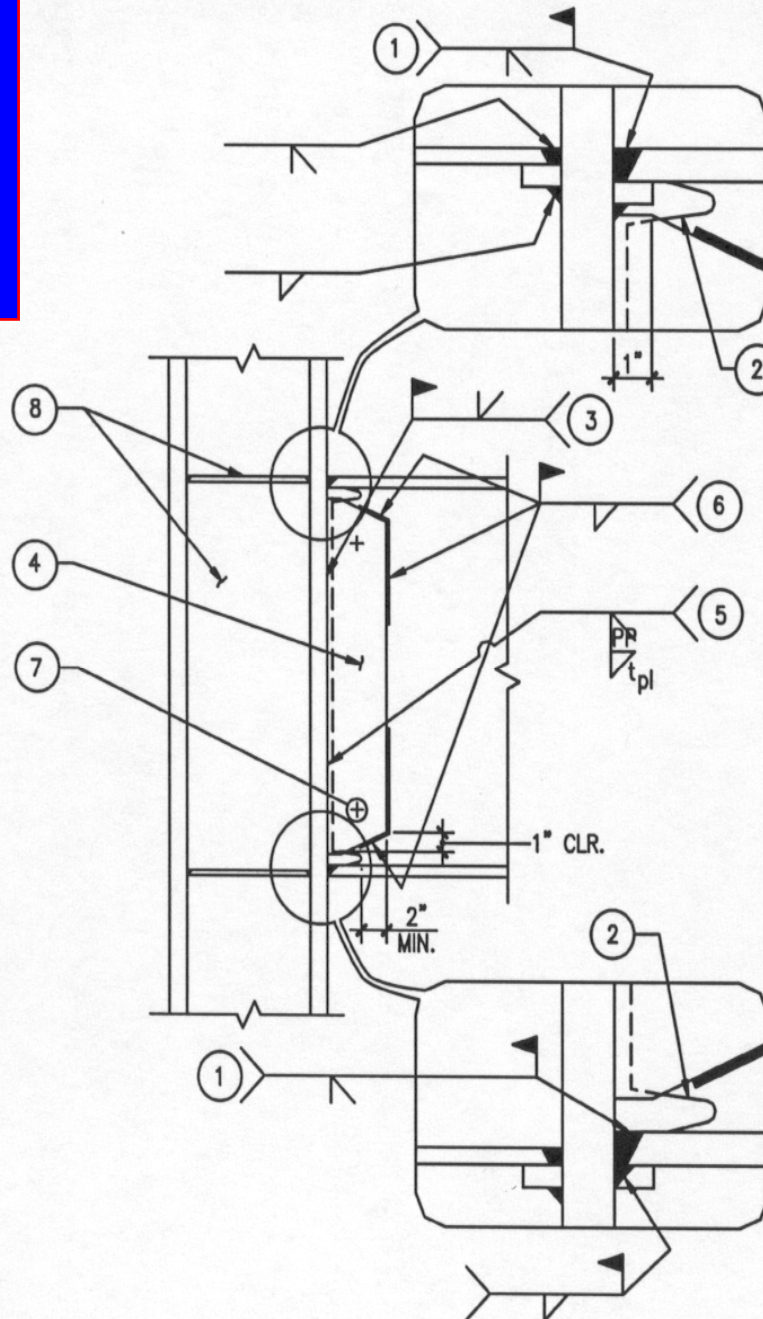
Reduced Beam Section



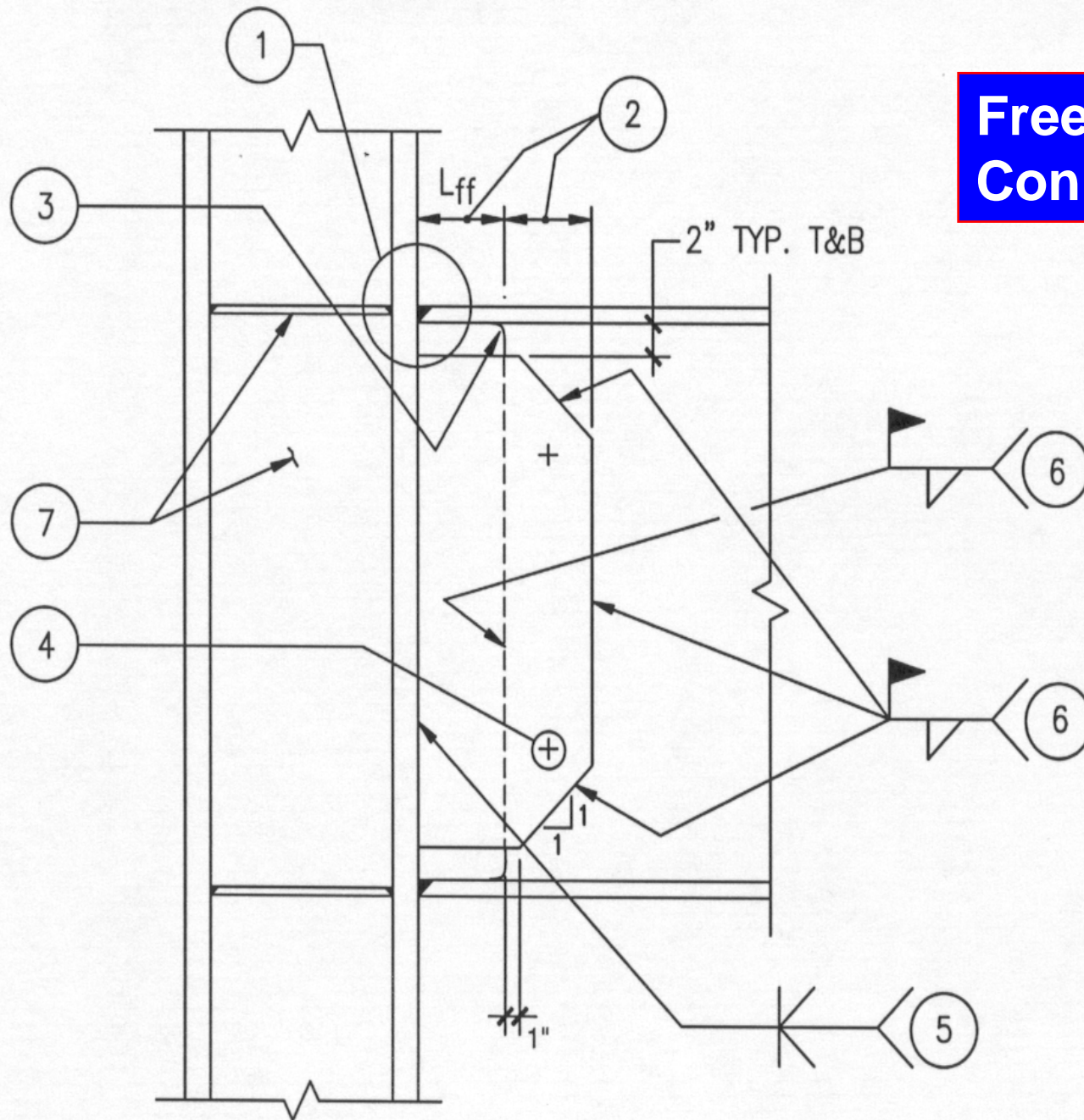
**Welded
Unreinforced
Flange - Bolted
Web**



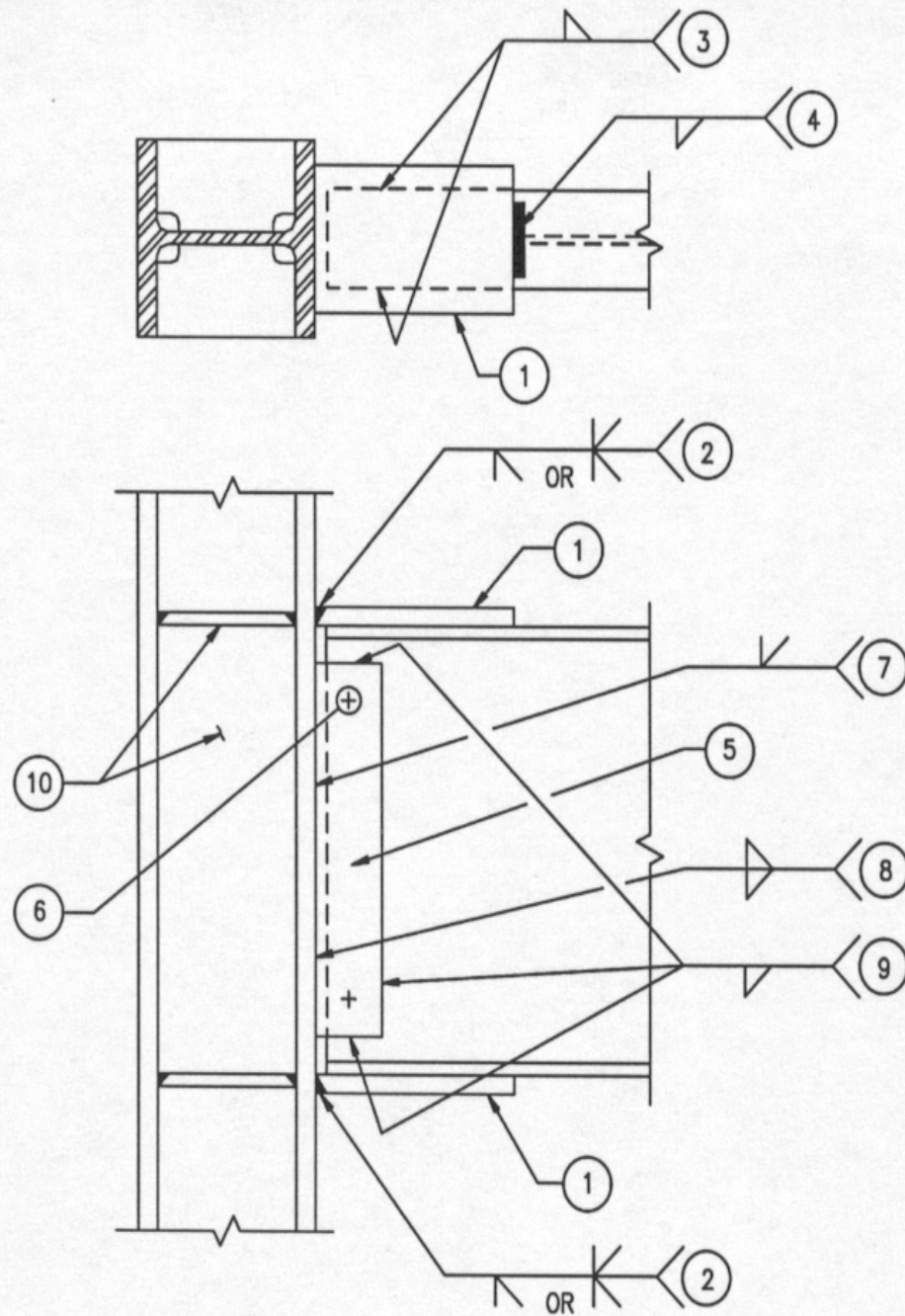
Welded Unreinforced Flange - Welded Web



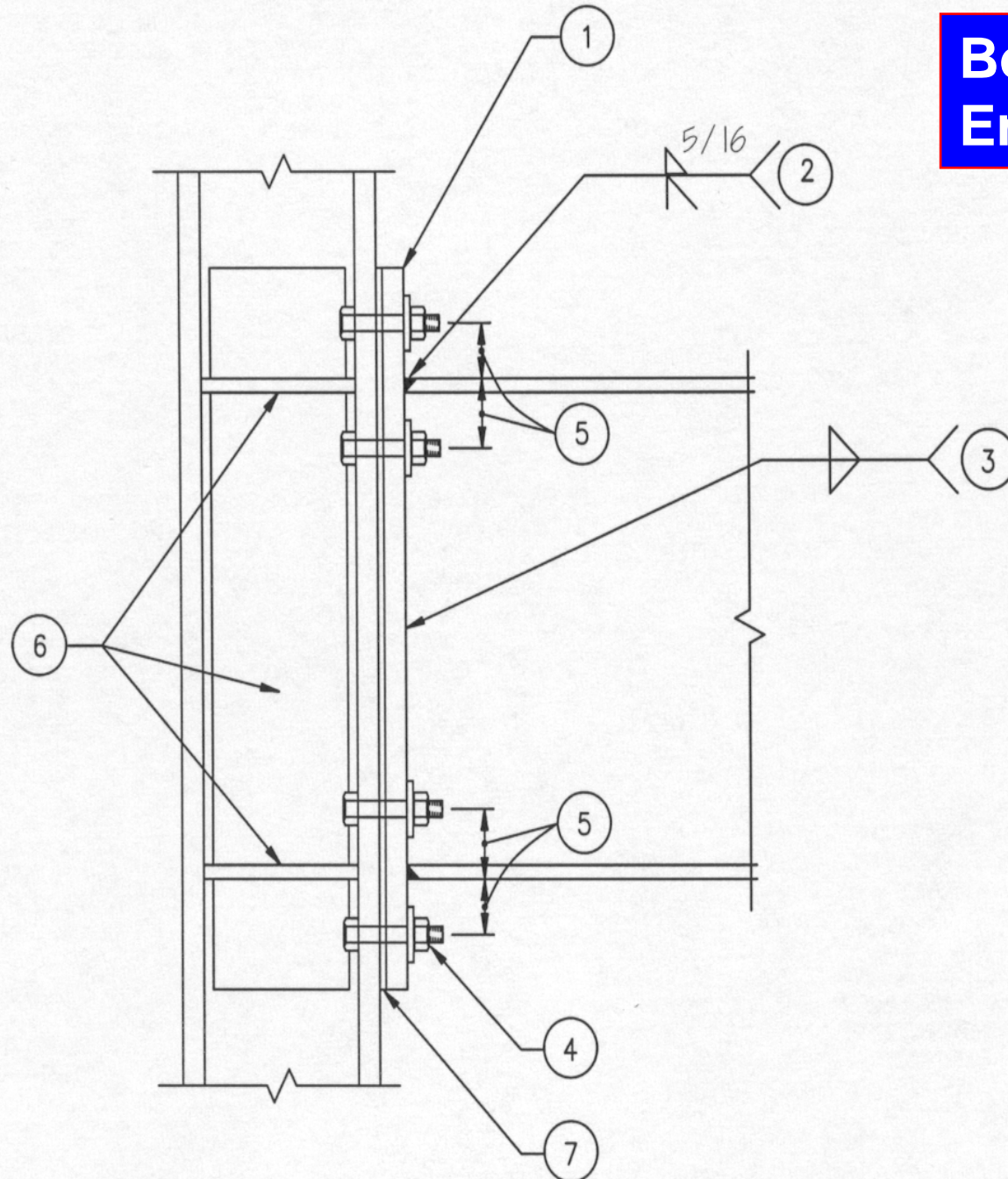
Free Flange Connection



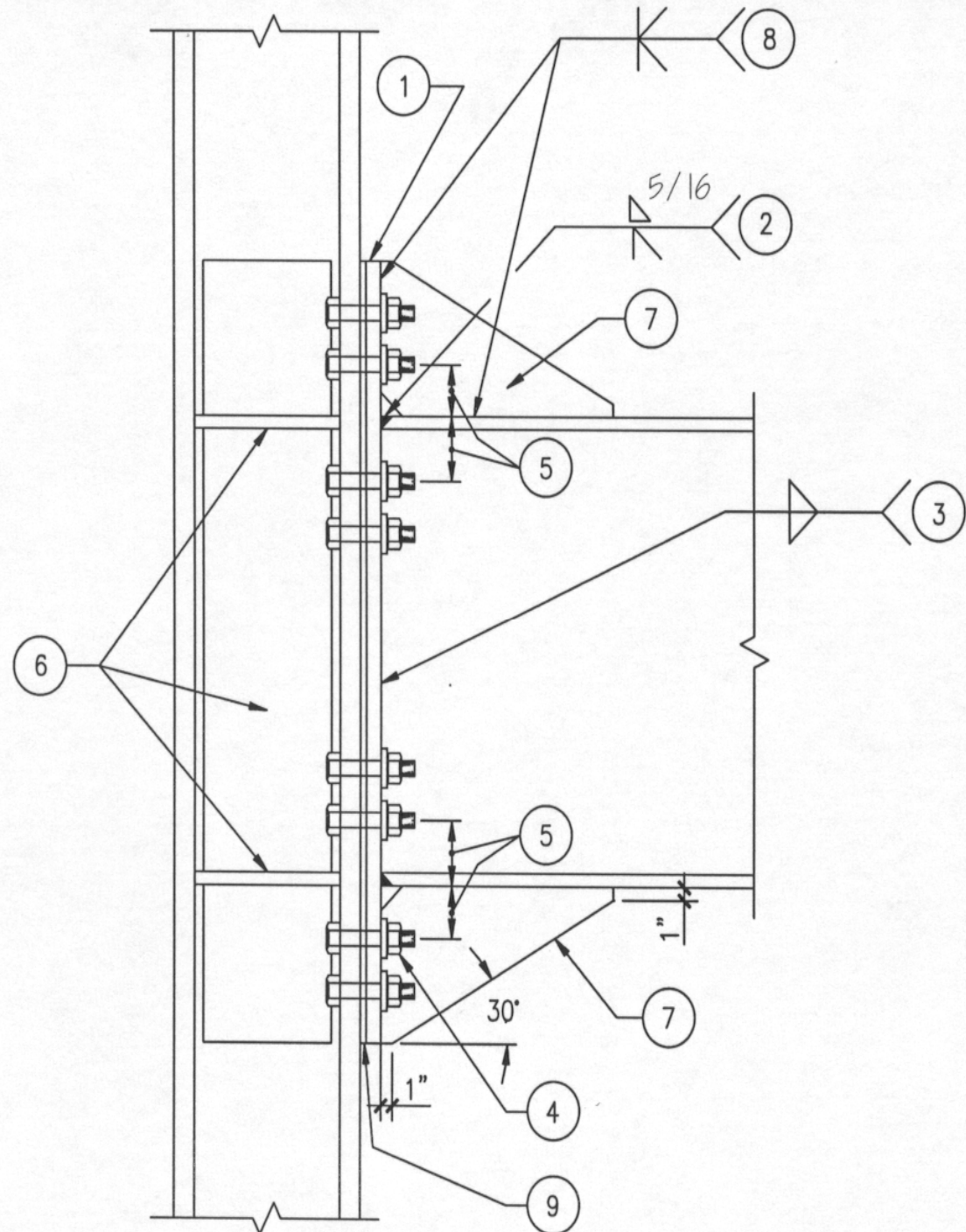
Welded Flange Plate Connection



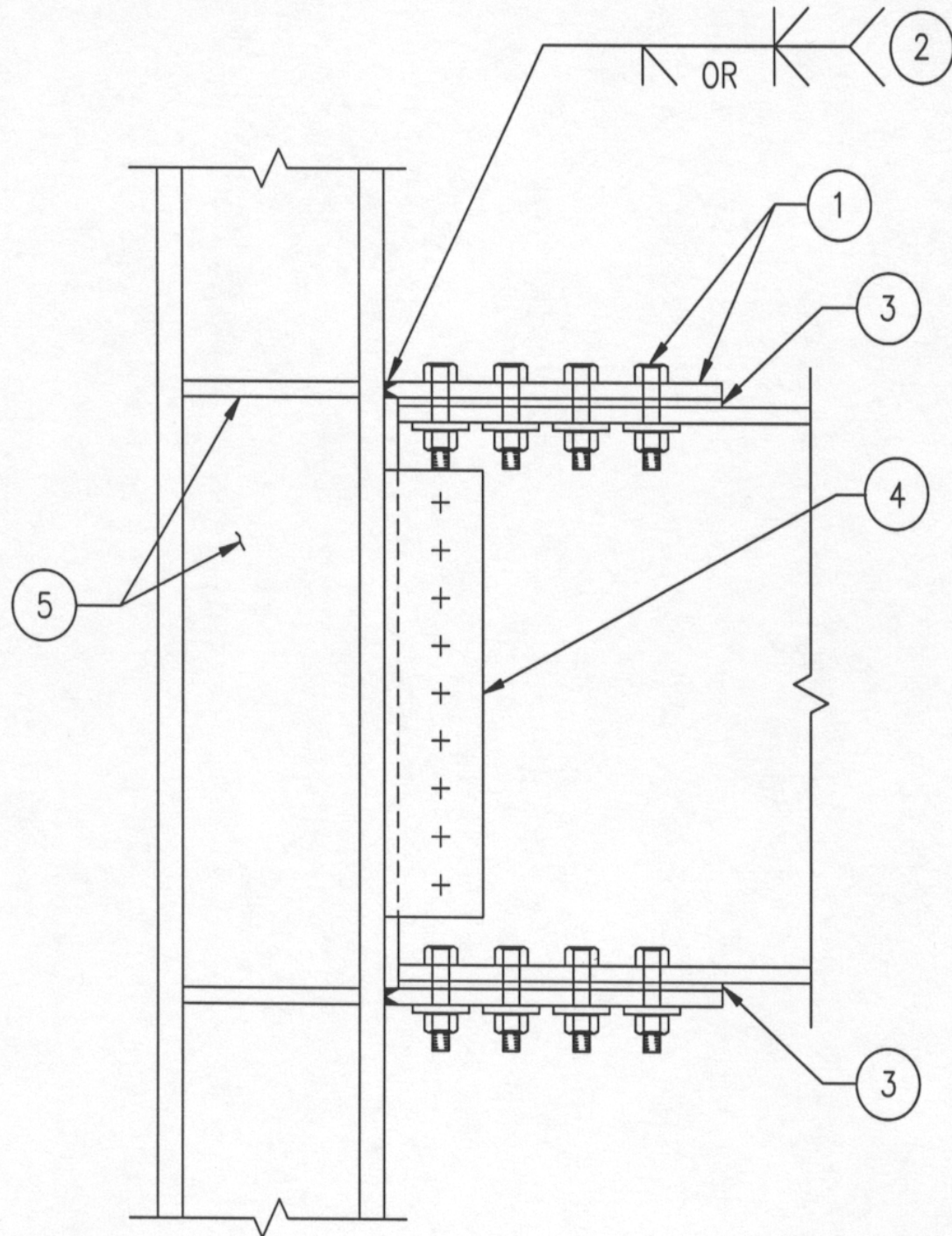
Bolted Unstiffened End Plate



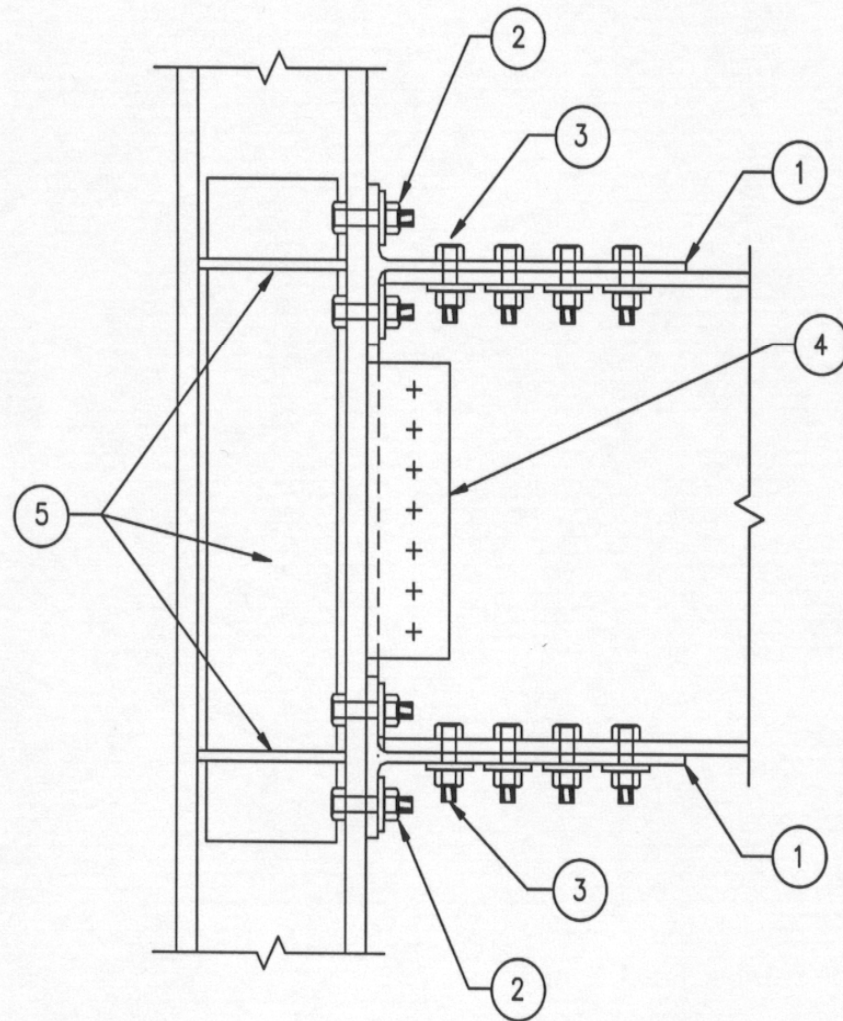
Bolted Stiffened End Plate



Bolted Flange Plate



Double Split Tee

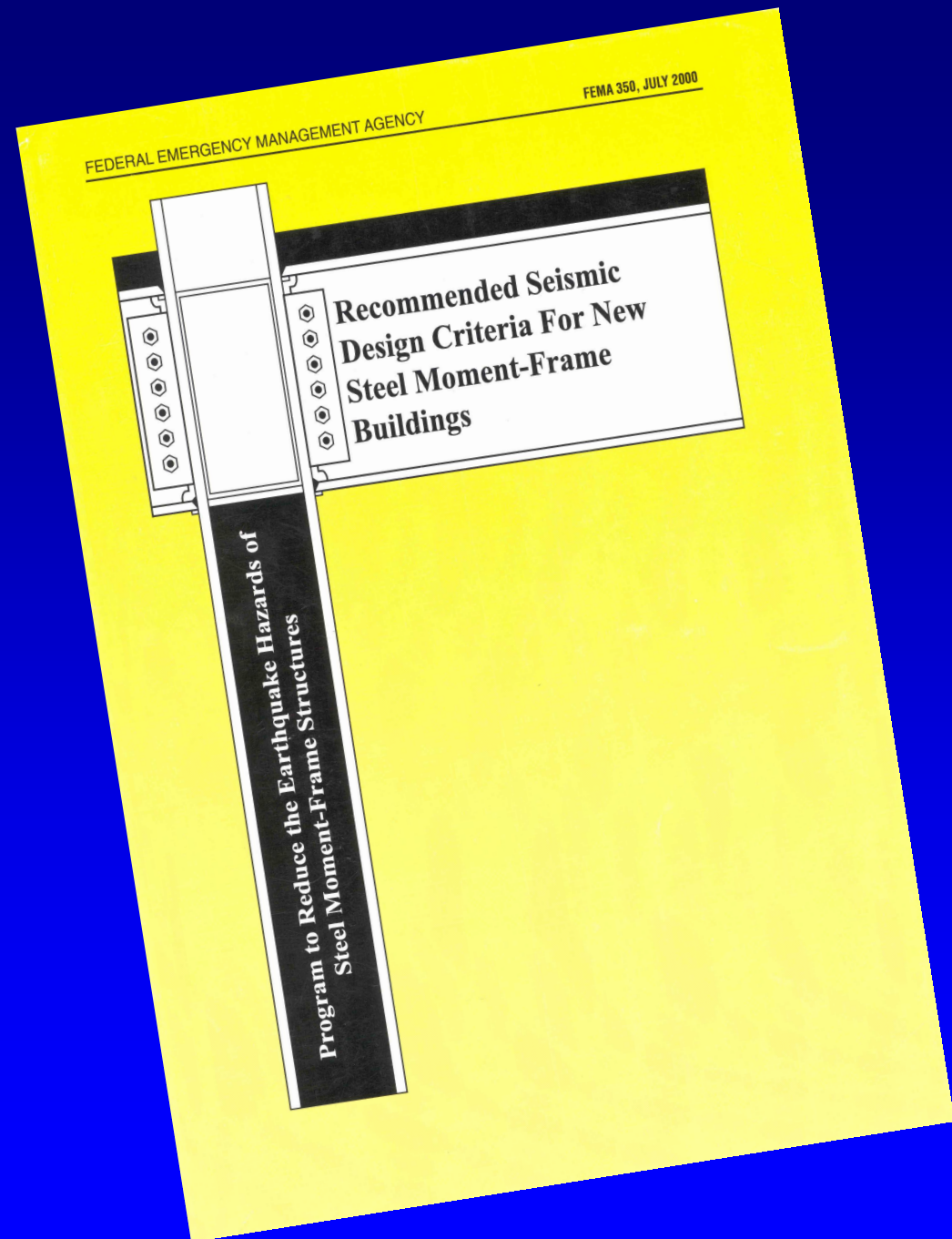


Results of SAC-FEMA Research Program

Recommended Seismic Design Criteria for Steel Moment Frames

- FEMA 350
Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings
- FEMA 351
Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings
- FEMA 352
Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings
- FEMA 353
Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications

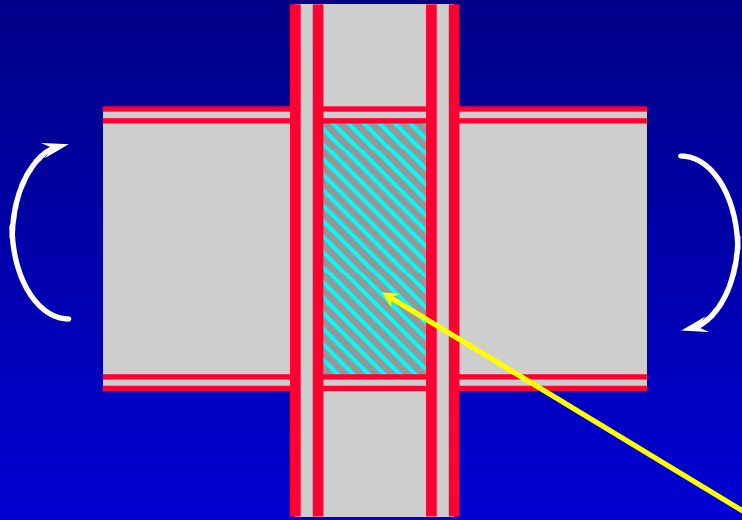
FEMA 350



Moment Resisting Frames

- Definition and Basic Behavior of Moment Resisting Frames
- Beam-to-Column Connections: Before and After Northridge
- **Panel-Zone Behavior**
- AISC Seismic Provisions for Special Moment Frames

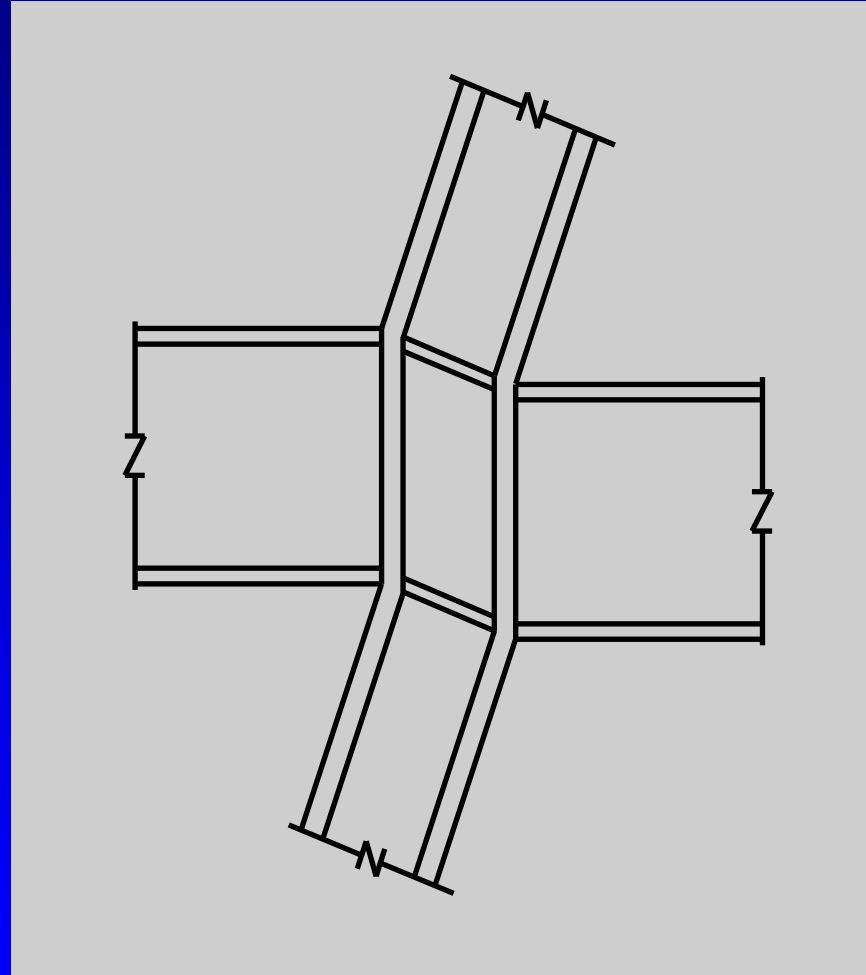
Column Panel Zone



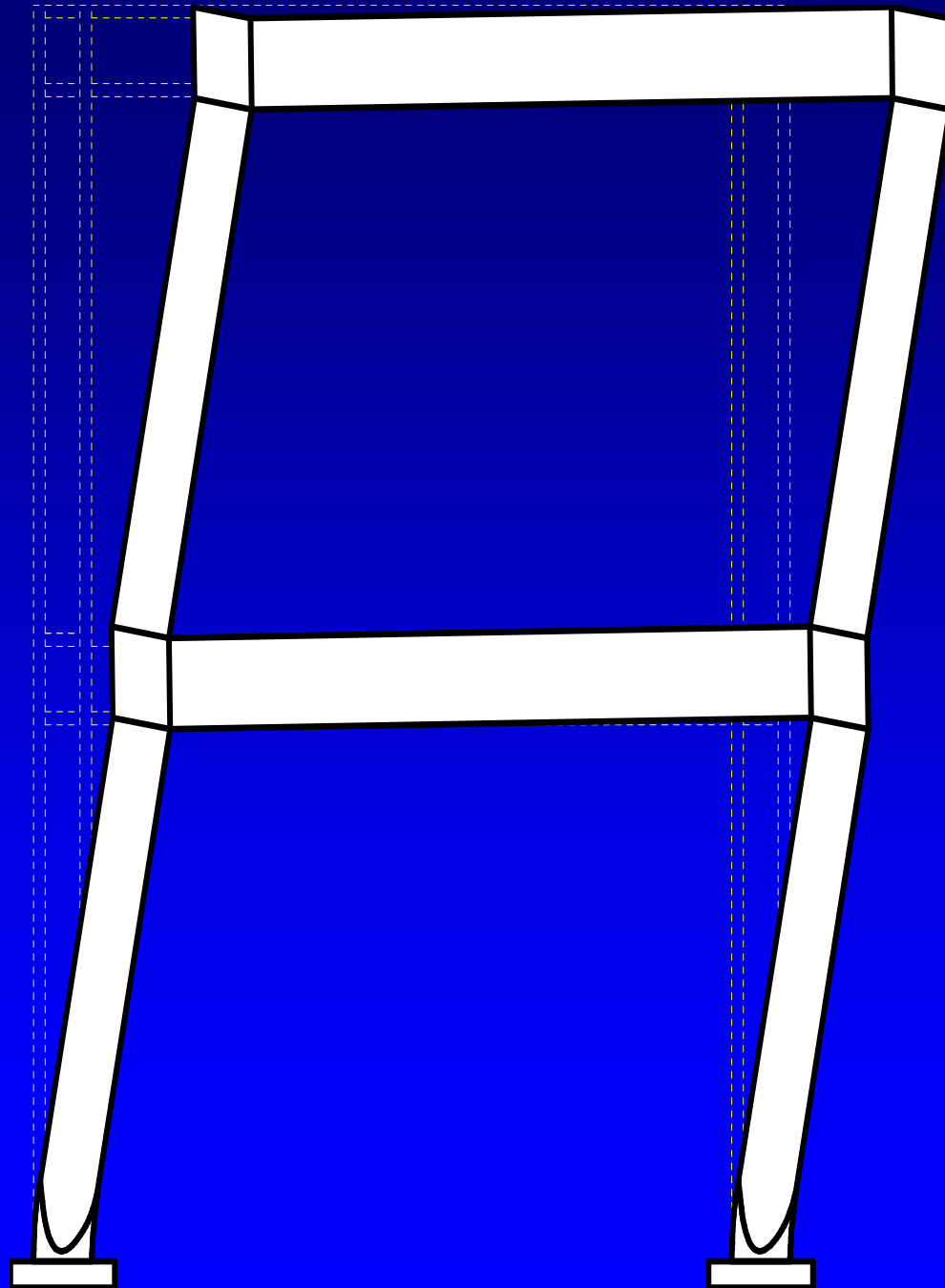
Column Panel Zone:

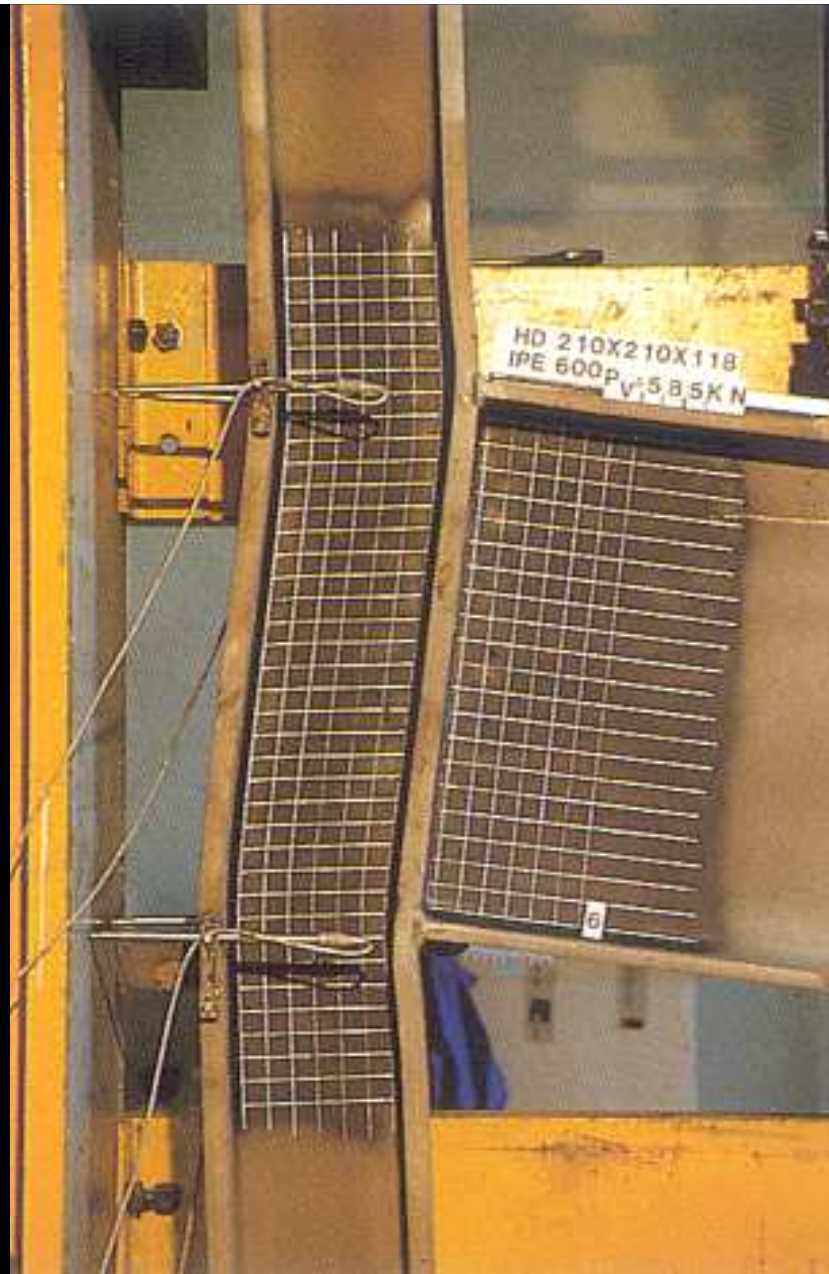
- subject to high shear
- shear yielding and large shear deformations possible (forms “shear hinge”)
- provides alternate yielding mechanism in a steel moment frame

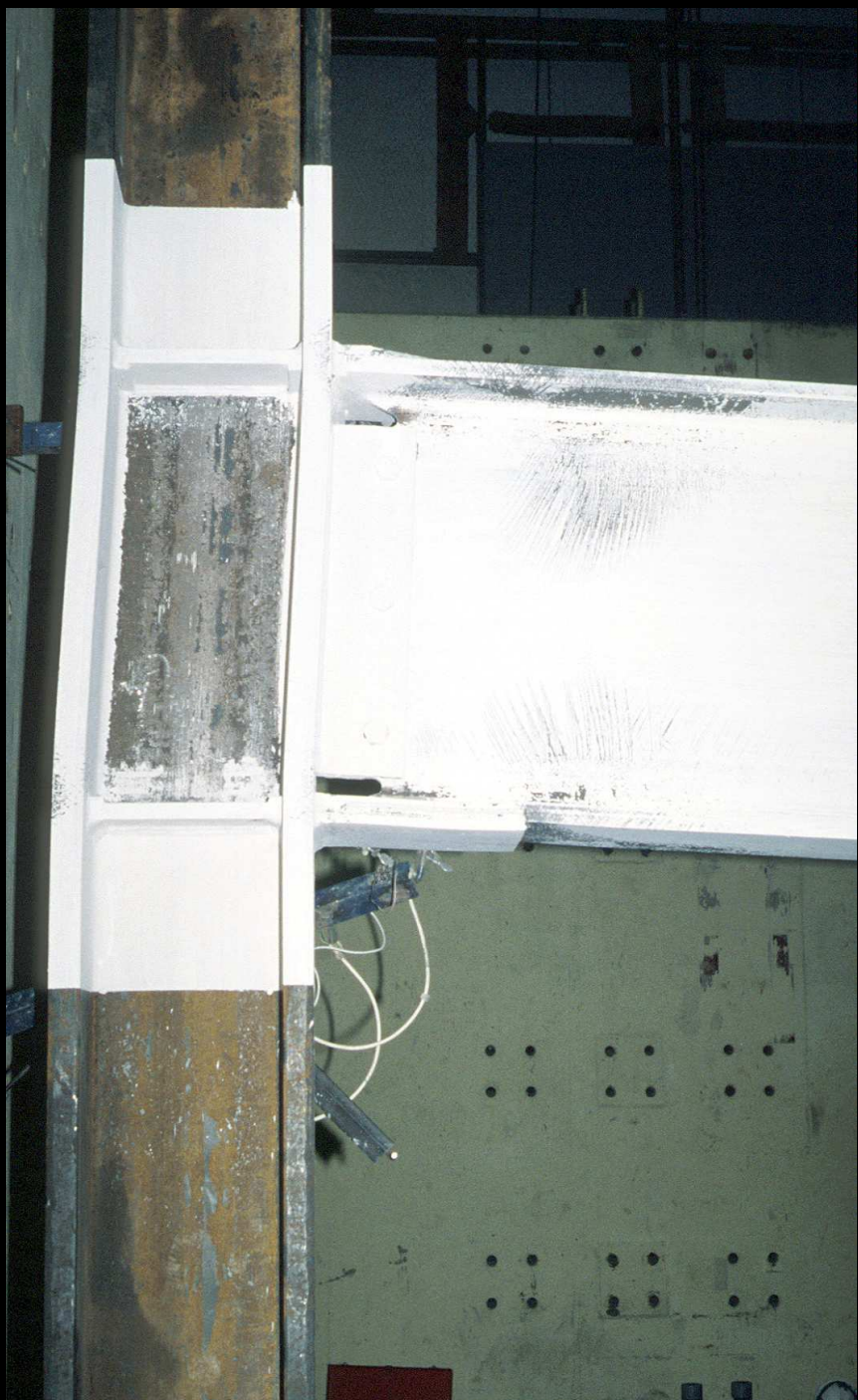
**Joint deformation
due to panel zone
shear yielding**

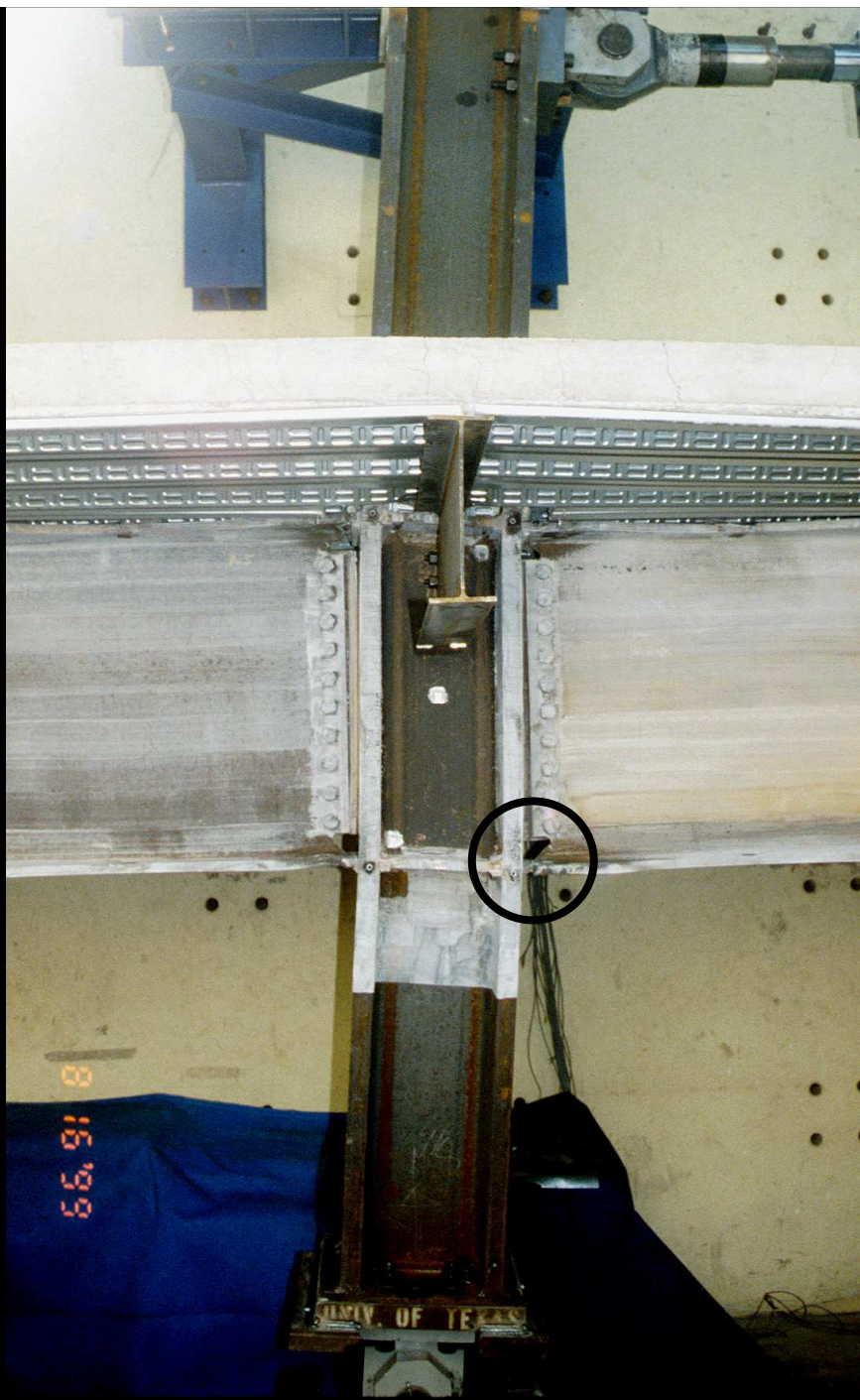


Plastic Shear Hinges In Column Panel Zones

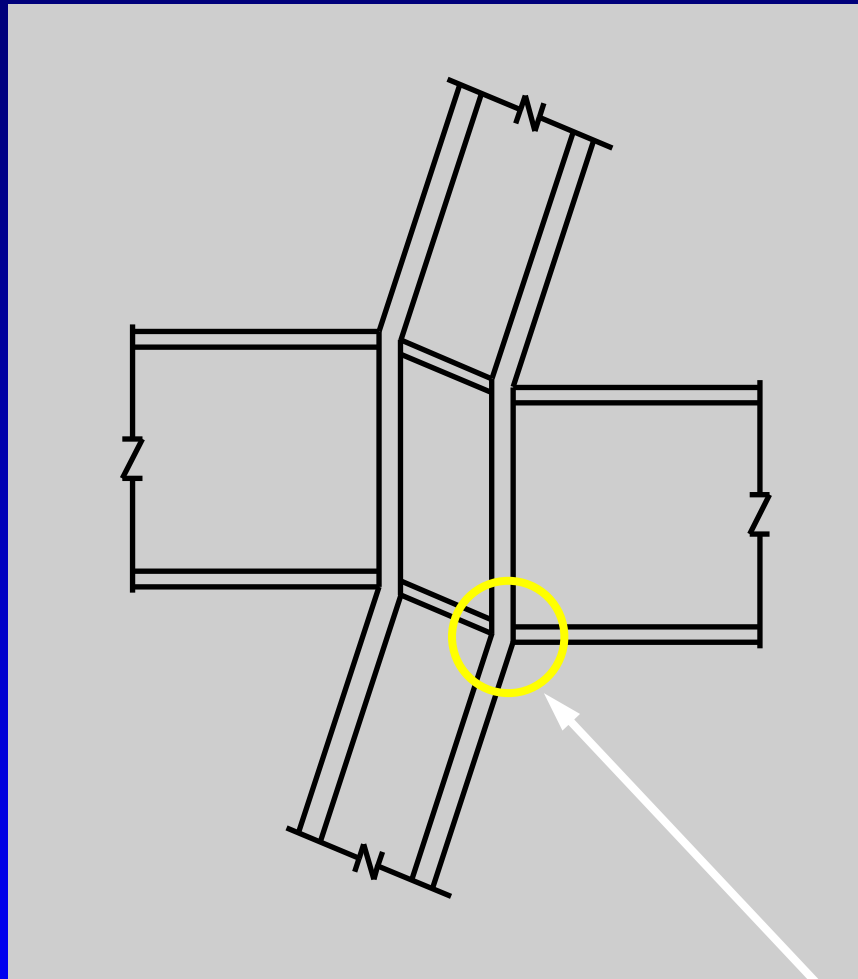




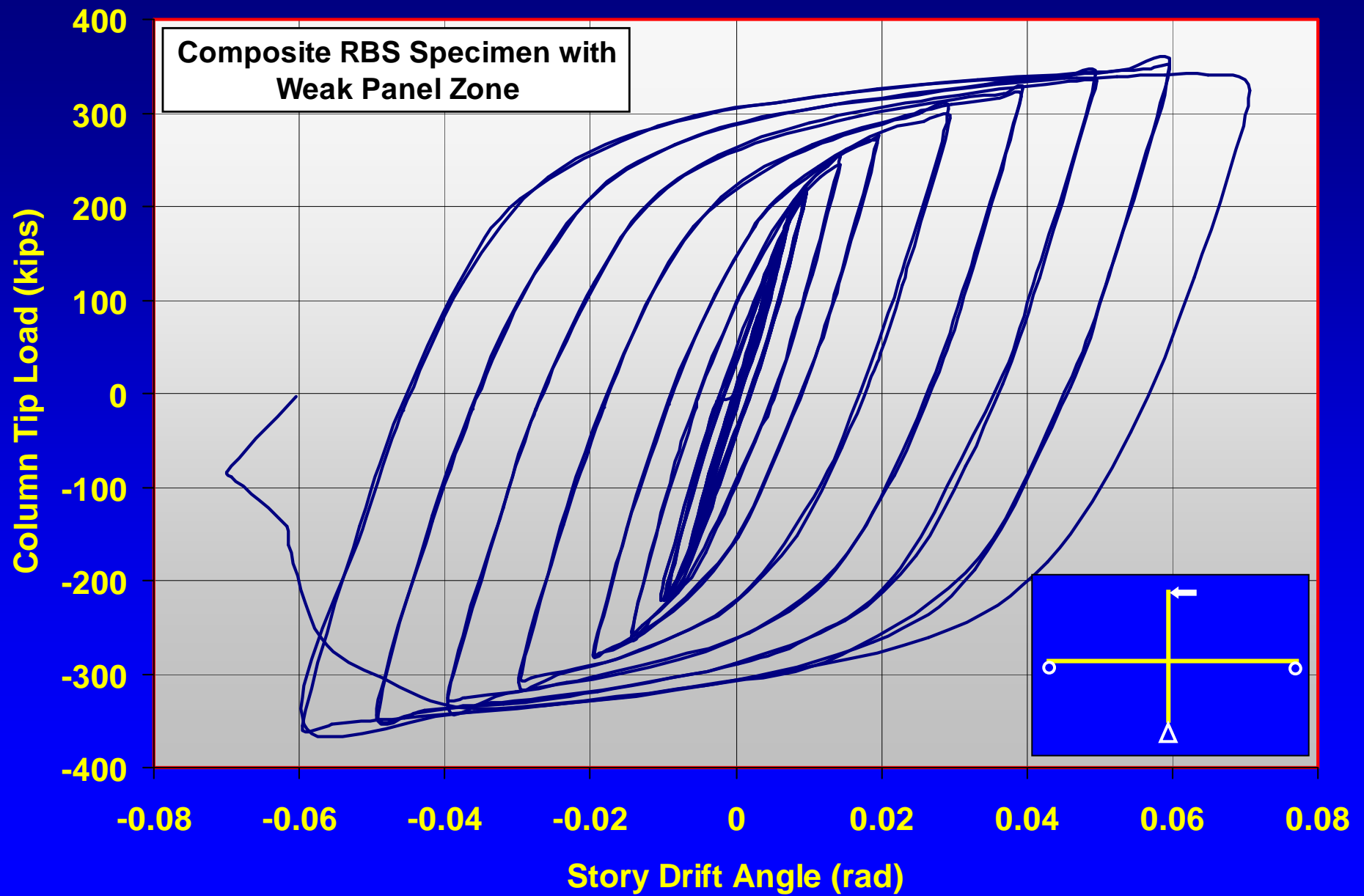


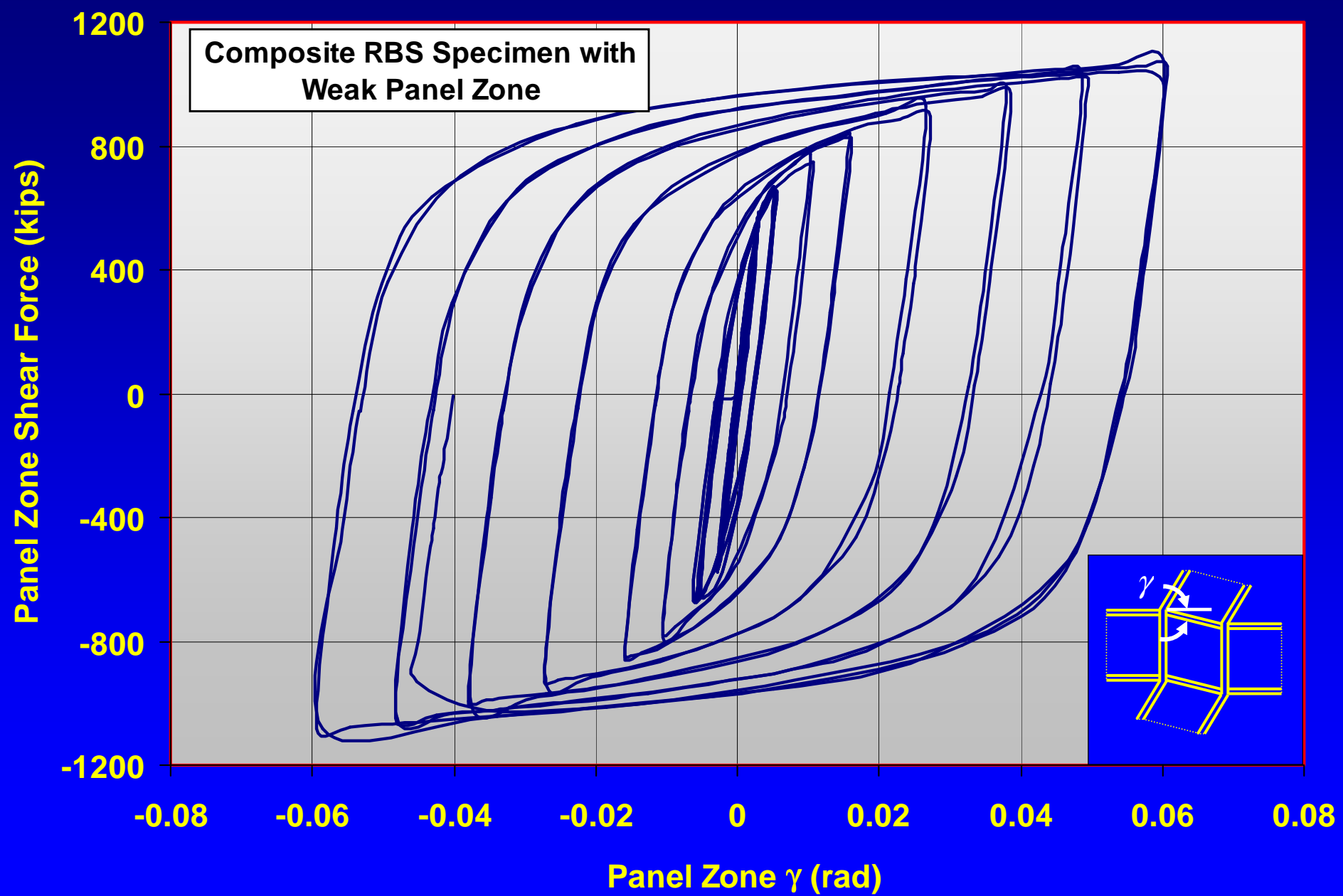






"kink" at corners of
panel zone





Observations on Panel Zone Behavior

- Very high ductility is possible.
- Localized deformations (“kinking”) at corners of panel zone may increase likelihood of fracture in vicinity of beam flange groove welds.
- Building code provisions have varied greatly on panel zone design.
- Current AISC Seismic Provisions permits limited yielding in panel zone.
- Further research needed to better define acceptable level of panel zone yielding

Moment Resisting Frames

- Definition and Basic Behavior of Moment Resisting Frames
- Beam-to-Column Connections: Before and After Northridge
- Panel-Zone Behavior

- AISC Seismic Provisions for Special Moment Frames

2005 AISC Seismic Provisions

Section 9 Special Moment Frames (SMF)

Section 10 Intermediate Moment Frames (IMF)

Section 11 Ordinary Moment Frames (OMF)

Section 9

Special Moment Frames (SMF)

- 9.1 Scope**
- 9.2 Beam-to-Column Joints and Connections**
- 9.3 Panel Zone of Beam-to-Column Connections**
- 9.4 Beam and Column Limitations**
- 9.5 Continuity Plates**
- 9.6 Column-Beam Moment Ratio**
- 9.7 Lateral Bracing of at Beam-to-Column Connections**
- 9.8 Lateral Bracing of Beams**
- 9.9 Column Splices**

AISC Seismic Provisions - SMF

9.1 Scope

Special moment frames (SMF) are expected to withstand **significant inelastic deformations** when subjected to the forces resulting from the motions of the design earthquake.

AISC Seismic Provisions - SMF

9.2 Beam-to-Column Connections

9.2a Requirements

9.2b Conformance Demonstration

9.2c Welds

9.2d Protected Zones

AISC Seismic Provisions - SMF - Beam-to-Column Connections

9.2a Requirements

Beam-to-column connections shall satisfy the following three requirements:

1. The connection shall be capable of sustaining an interstory drift angle of at least 0.04 radians.
2. The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80 M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

9.2a Requirements

Beam-to-column connections shall satisfy the following three requirements (cont):

3. The required shear strength of the connection shall be determined using the following quantity for the earthquake load effect E :

$$E = 2 [1.1 R_y M_p] / L_h \quad (9-1)$$

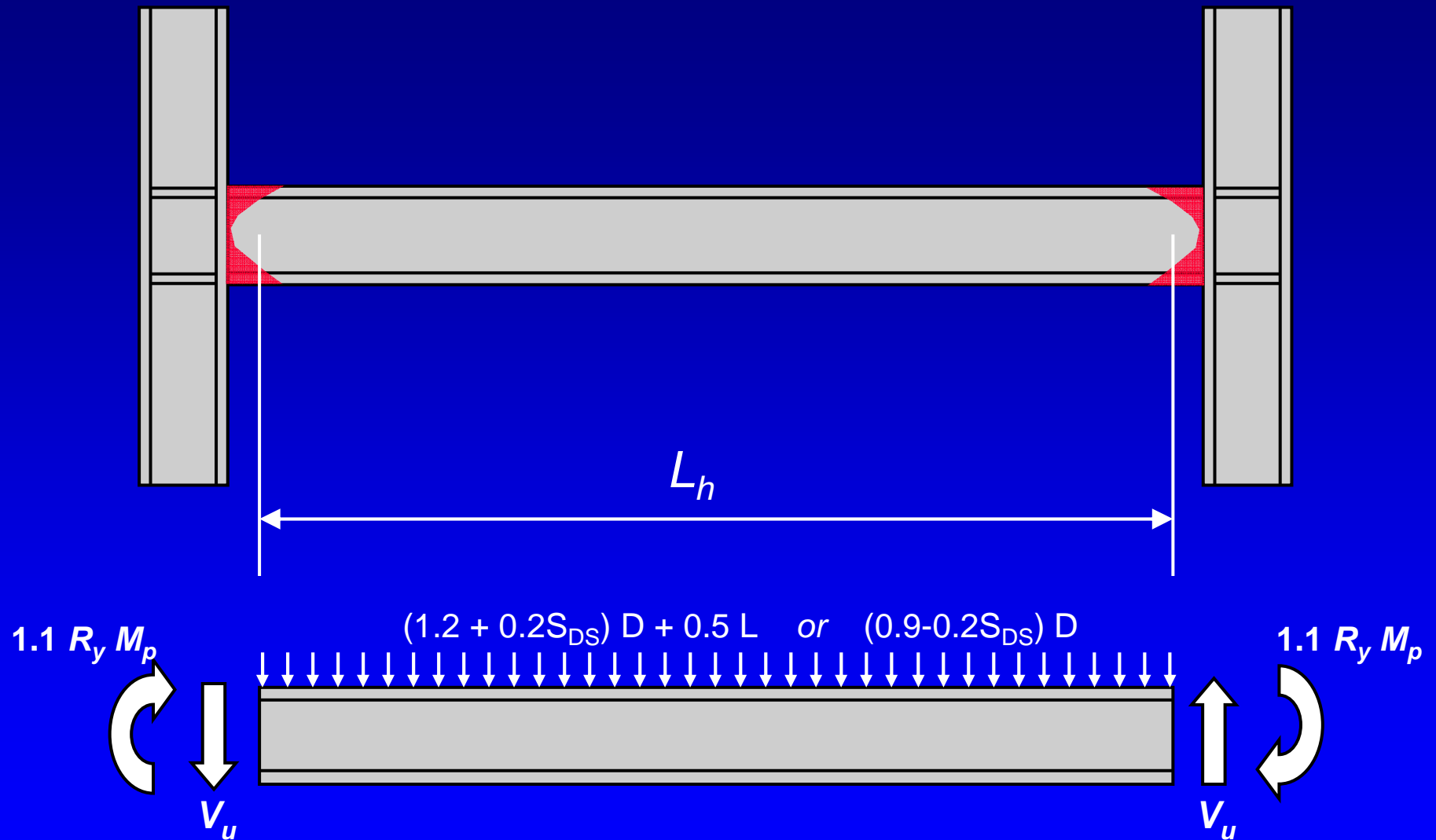
where:

R_y = ratio of the expected yield strength to the minimum specified yield strength

M_p = nominal plastic flexural strength

L_h = distance between plastic hinge locations

Required Shear Strength of Beam-to-Column Connection



$$V_u = 2 [1.1 R_y M_p] / L_h + V_{gravity}$$

AISC Seismic Provisions - SMF - Beam-to-Column Connections

9.2b Conformance Demonstration

Demonstrate conformance with requirements of Sect. 9.2a by one of the following methods:

I. **Conduct qualifying cyclic tests in accordance with Appendix S.**

Tests conducted specifically for the project, with test specimens that are representative of project conditions.

or

Tests reported in the literature (research literature or other documented test programs), where the test specimens are representative of project conditions.

9.2b Conformance Demonstration

Demonstrate conformance with requirements of Sect. 9.2a by one of the following methods (cont):

II. Use connections *prequalified* for SMF in accordance with Appendix P

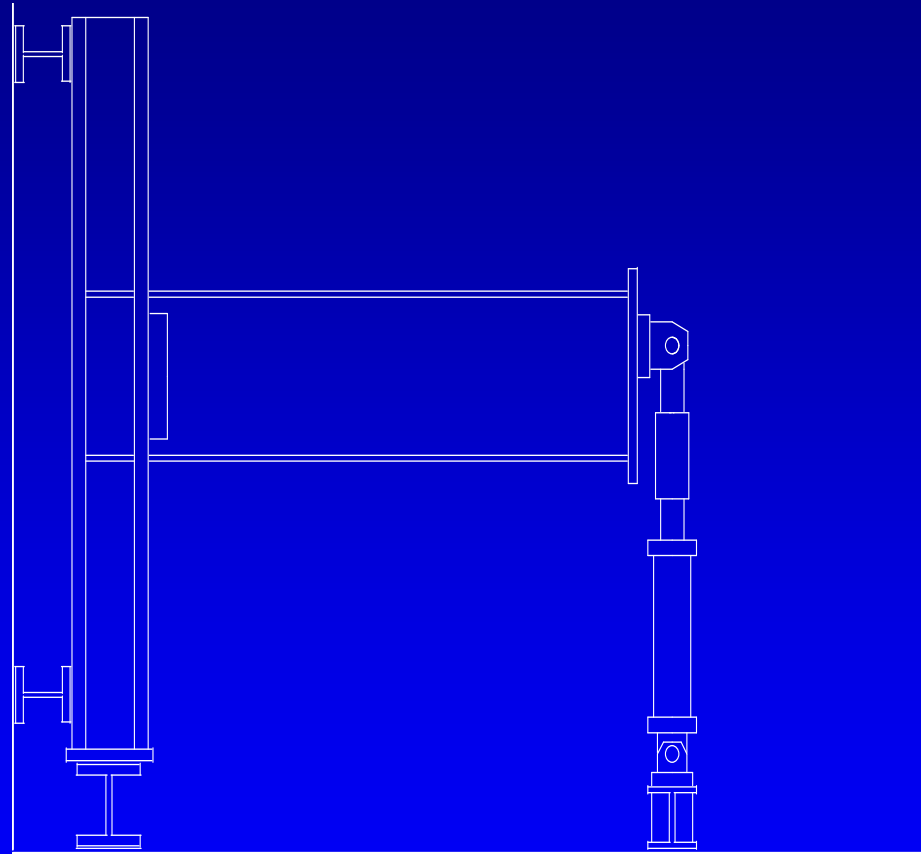
Use connections prequalified by the AISC Connection Prequalification Review Panel (CPRP) and documented in Standard ANSI/AISC 358 - "*Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*"

or

Use connection prequalified by an alternative review panel that is approved by the Authority Having Jurisdiction.

9.2b Conformance Demonstration - by Testing

**Test connection
in accordance
with Appendix S**



Appendix S

Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections

Testing Requirements:

- Test specimens should be representative of prototype (Prototype = actual building)
- Beams and columns in test specimens must be nearly full-scale representation of prototype members:
 - depth of test beam $\geq 0.90 \times$ depth of prototype beam
 - wt. per ft. of test beam $\geq 0.75 \times$ wt. per ft. of prototype beam
 - depth of test column $\geq 0.90 \times$ depth of prototype column
- Sources of inelastic deformation (beam, panel zone, connection plates, etc) in the test specimen must similar to prototype.

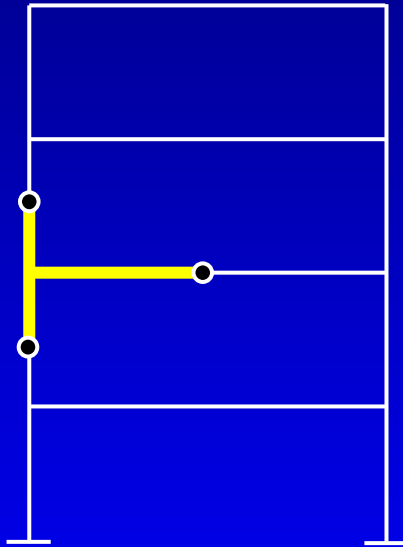
Appendix S

Testing Requirements (cont):

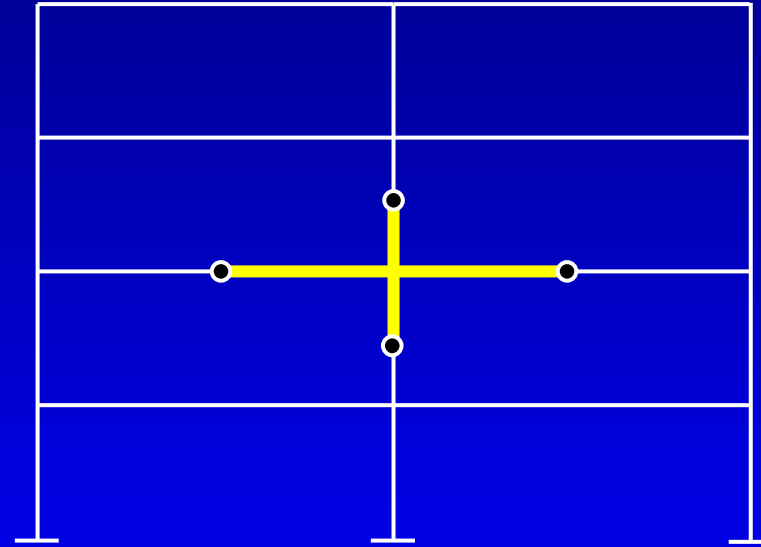
- Lateral bracing in test specimen should be similar to prototype.
- Connection configuration used for test specimen must match prototype.
- Welding processes, procedures, electrodes, etc. used for test specimen must be representative of prototype.

See Appendix S for other requirements.

Typical Test Subassemblages

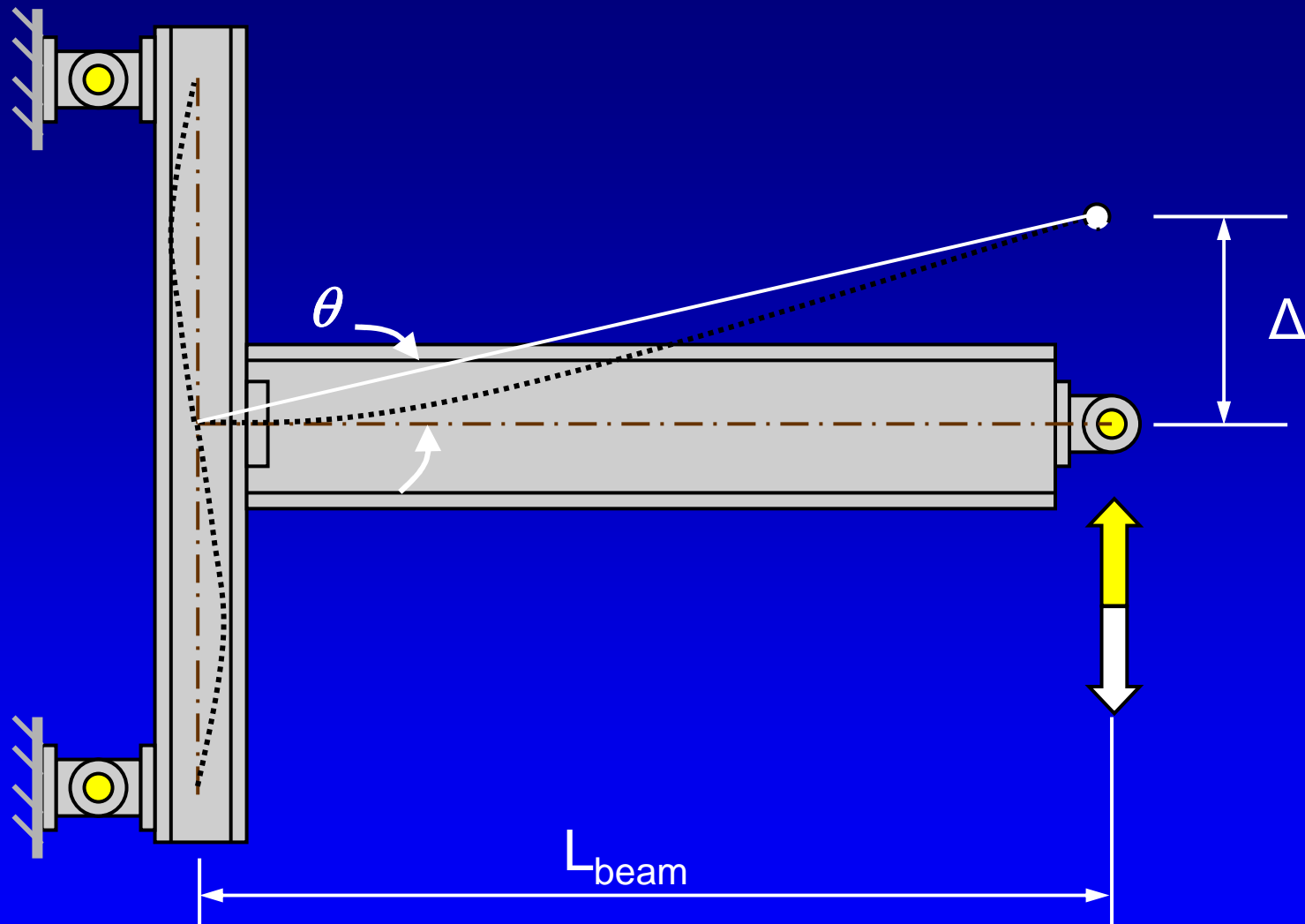


Exterior Subassemblage



Interior Subassemblage

Typical Exterior Subassembly

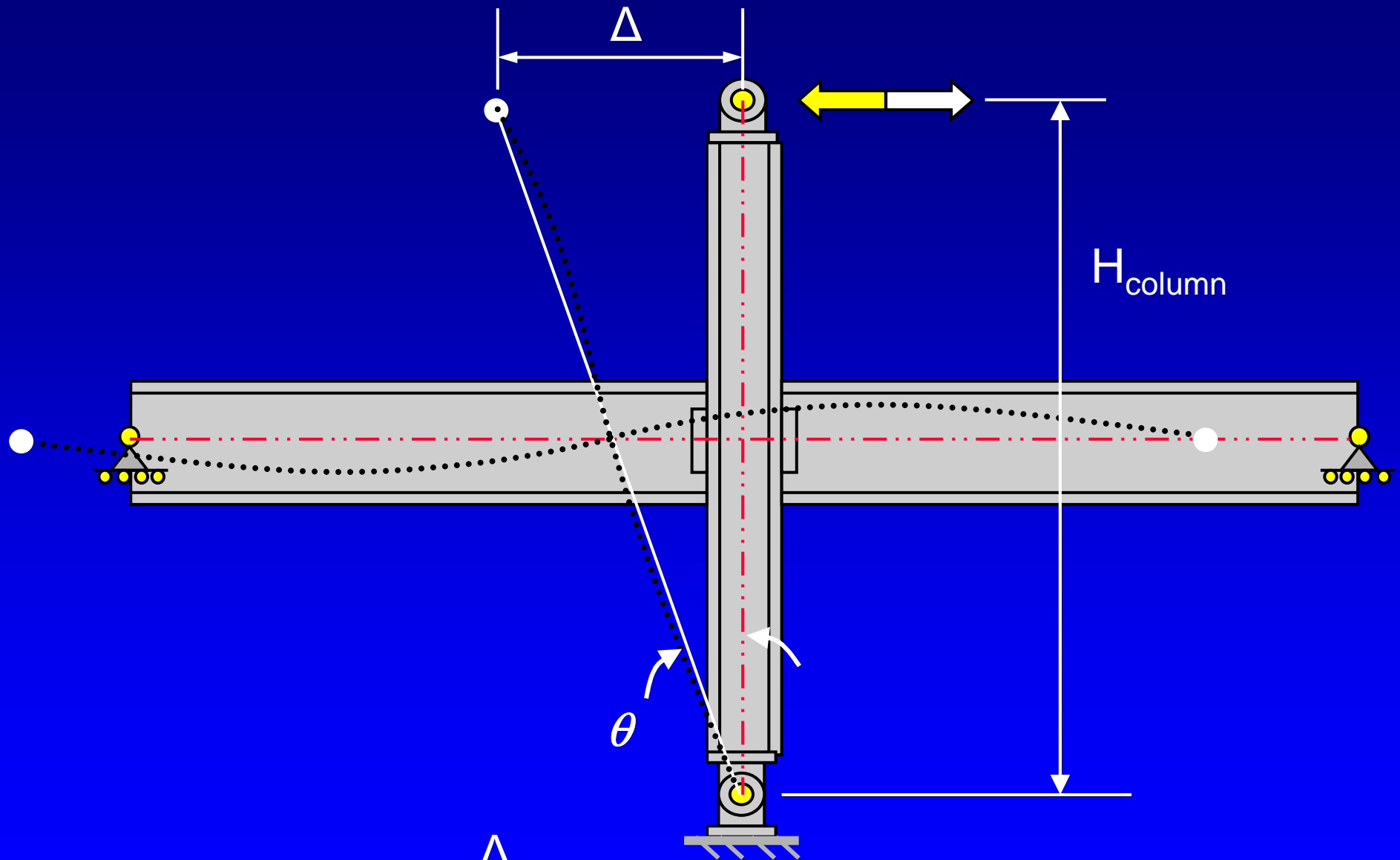


$$\text{Interstory Drift Angle } \theta = \frac{\Delta}{L_{\text{beam}}}$$

Typical Exterior Subassembly

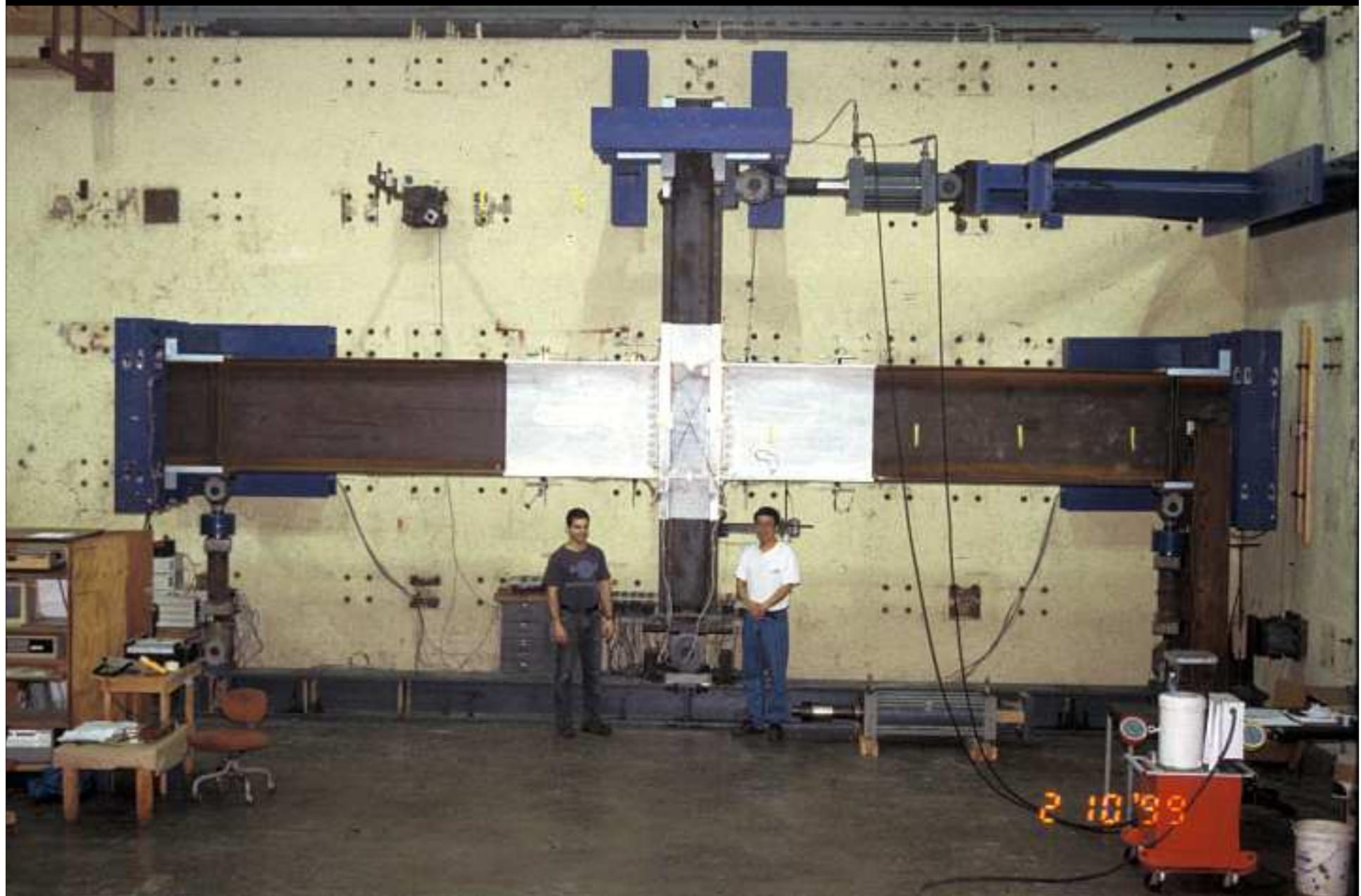


Typical Interior Subassembly



Interstory Drift Angle $\theta = \frac{\Delta}{H_{\text{column}}}$

Typical Interior Subassemblage



Typical Interior Subassemblage (with concrete floor slab)



Appendix S

Testing Requirements - Loading History

Apply the following loading history:

6 cycles at $\theta = \pm 0.00375$ rad.

6 cycles at $\theta = \pm 0.005$ rad.

6 cycles at $\theta = \pm 0.0075$ rad.

4 cycles at $\theta = \pm 0.01$ rad.

2 cycles at $\theta = \pm 0.015$ rad.

2 cycles at $\theta = \pm 0.02$ rad.

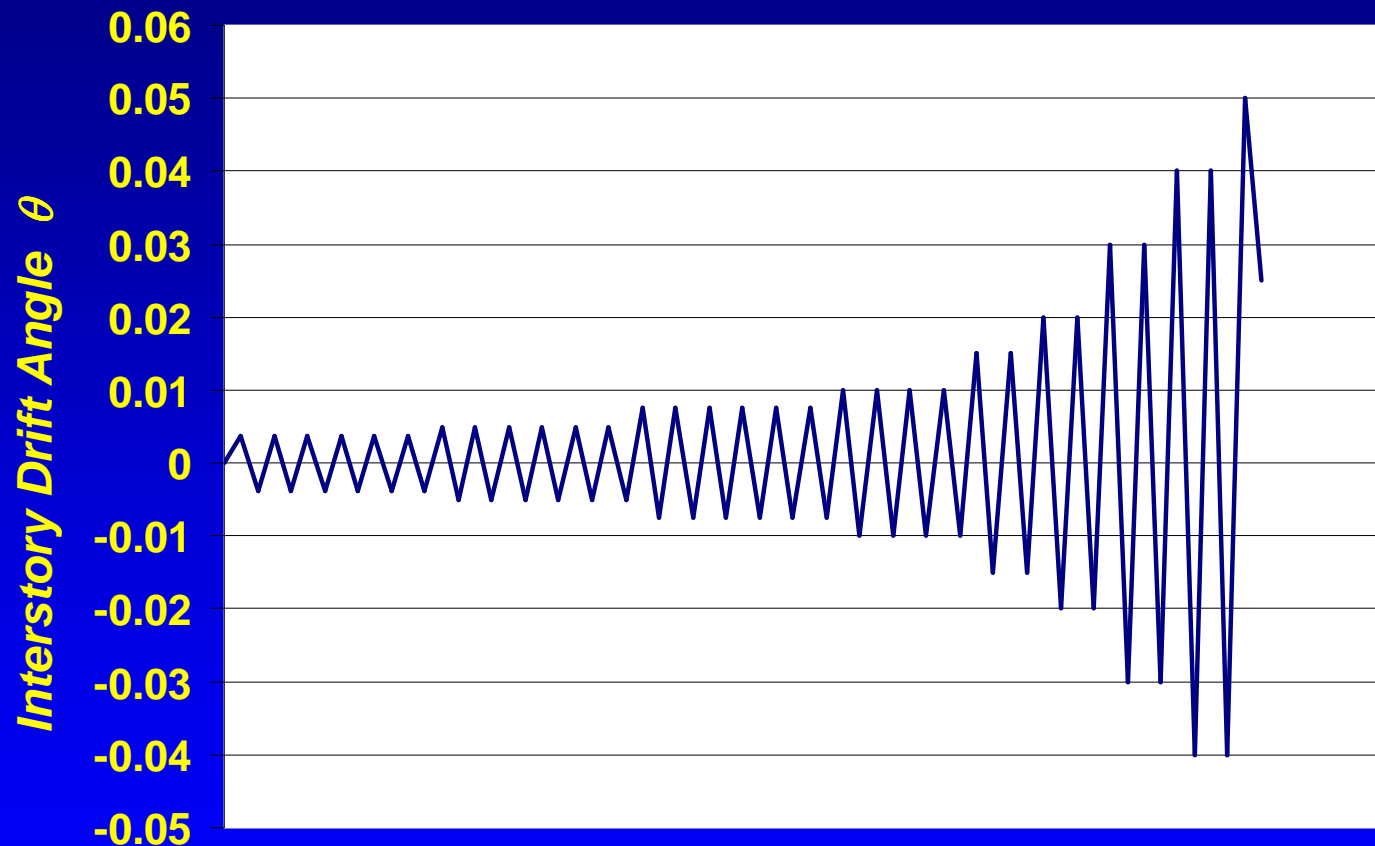
2 cycles at $\theta = \pm 0.03$ rad.

2 cycles at $\theta = \pm 0.04$ rad.

continue at increments of ± 0.01 rad, with two cycles of loading at each step

Appendix S

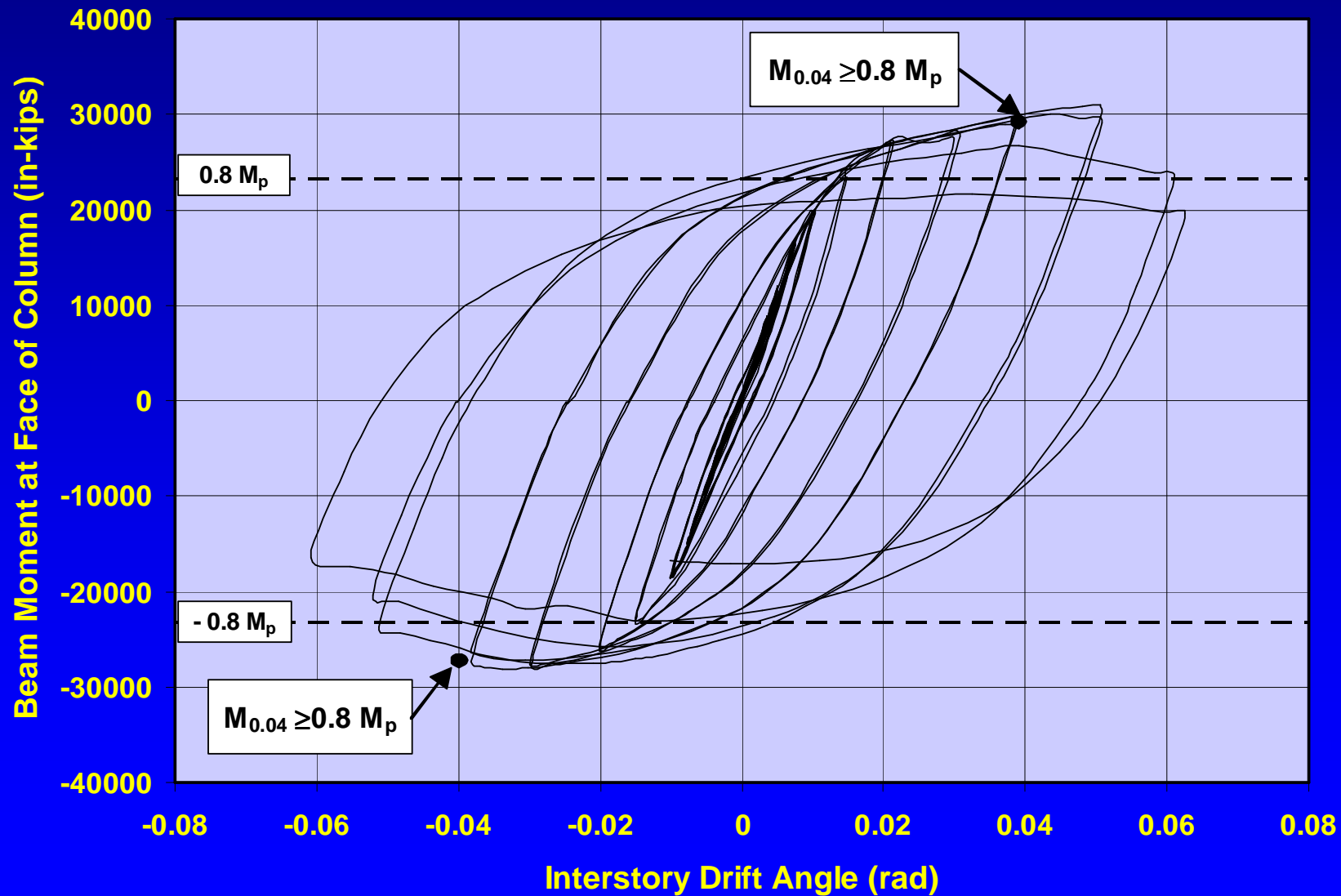
Testing Requirements - Loading History



Acceptance Criteria for SMF Beam-to-Column Connections:

After completing at least one loading cycle at ± 0.04 radian, the measured flexural resistance of the connection, measured at the face of the column, must be at least $0.80 M_p$ of the connected beam

Example of Successful Conformance Demonstration Test per Appendix S:



9.2b Conformance Demonstration

.....by use of Prequalified Connection

A Prequalified connection is one that has undergone sufficient

testing (per Appendix S)

analysis

evaluation and review

**so that a high level of confidence exists that the connection can
fulfill the performance requirements specified in Section 9.2a for
Special Moment Frame Connections**

9.2b Conformance Demonstration **by use of Prequalified Connection**

Requirements for Prequalification of Connections:

***Appendix P - Prequalification of Beam-to-Column
and Link-to-Column Connections***

Authority to Prequalify of Connections:

AISC Connection Prequalification Review Panel (CPRP)

Information on Prequalified Connections:

***Standard ANSI/AISC 358 - "Prequalified Connections for
Special and Intermediate Steel Moment Frames for Seismic
Applications"***

ANSI/AISC 358-05
An American National Standard

Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

December 13, 2005

Approved by the AISC Connection Prequalification Review Panel
and issued by the AISC Board of Directors



AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

ANSI/AISC 358 - "*Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*"

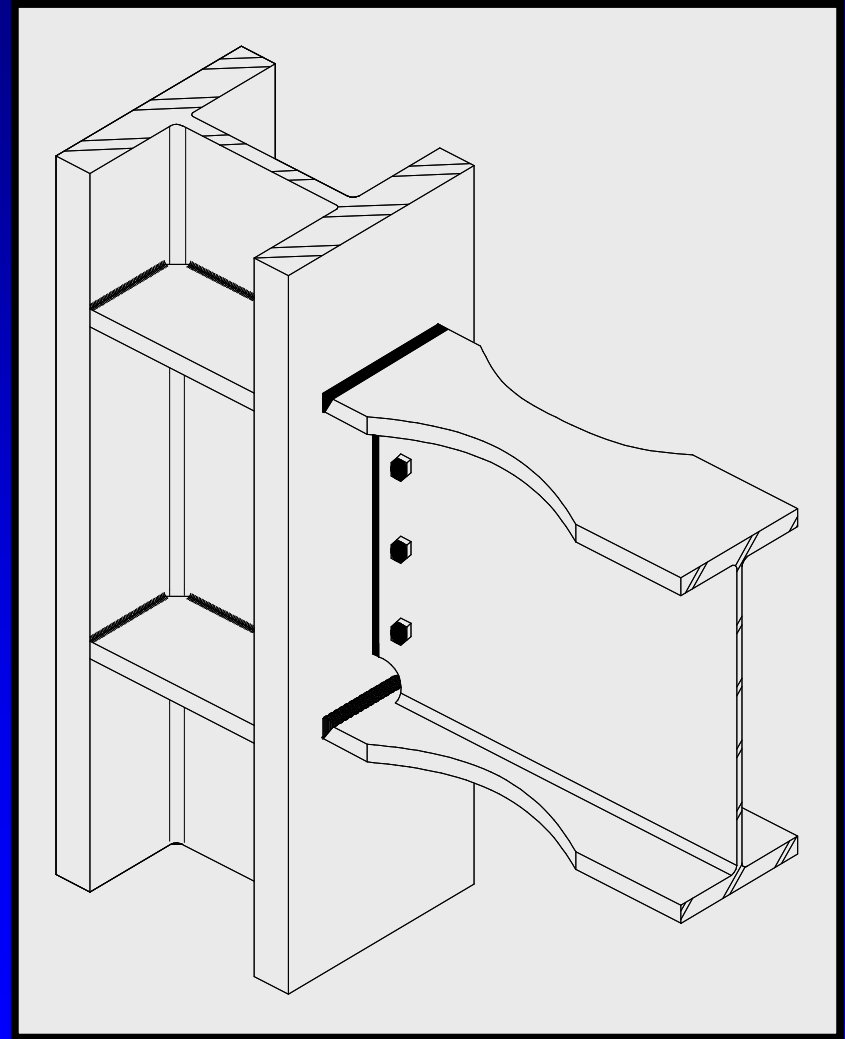
Connections Prequalified in ANSI/AISC 358 (1st Ed - 2005)

- **Reduced Beam Section (RBS) Connection**
- **Bolted Unstiffened and Stiffened Extended End-Plate Connection**

Reduced Beam Section (RBS) Moment Connection

RBS Concept:

- Trim Beam Flanges Near Connection
- Reduce Moment at Connection
- Force Plastic Hinge Away from Connection



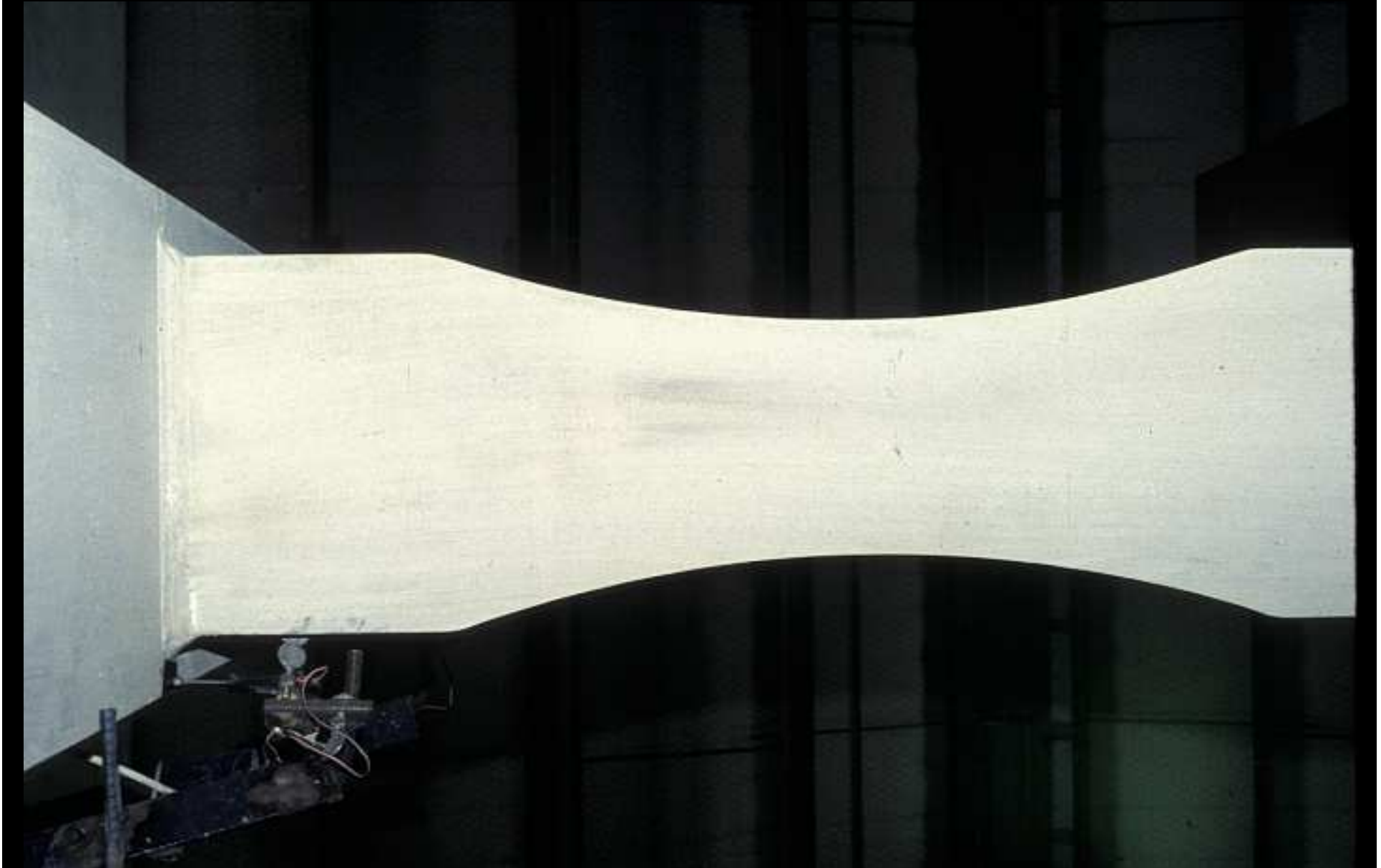
Example of laboratory performance of an RBS connection:



Whitewashed connection prior to testing:



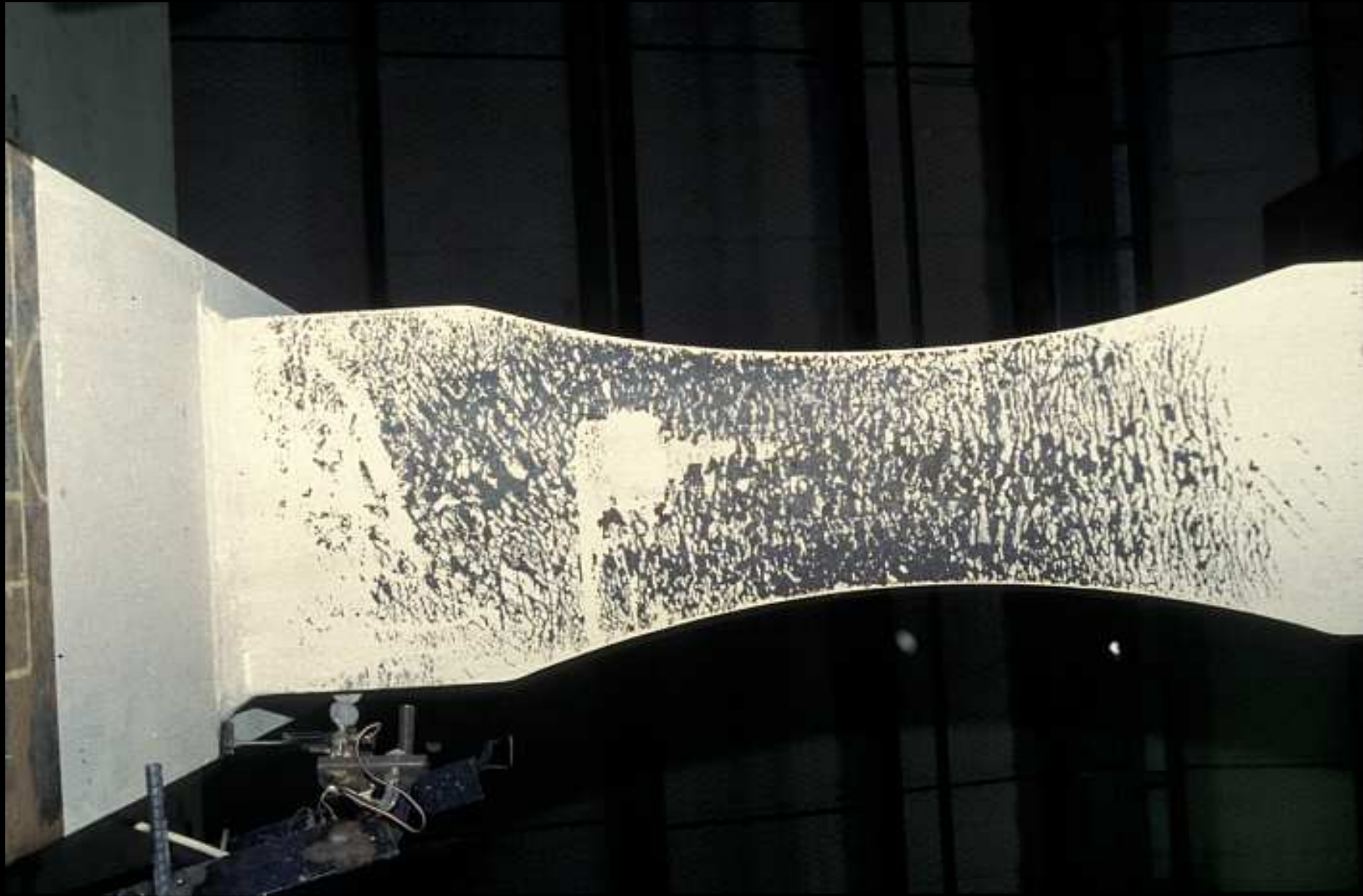
Whitewashed connection prior to testing:



Connection at $\theta \cong 0.02$ radian.....



Connection at $\theta \cong 0.02$ radian.....

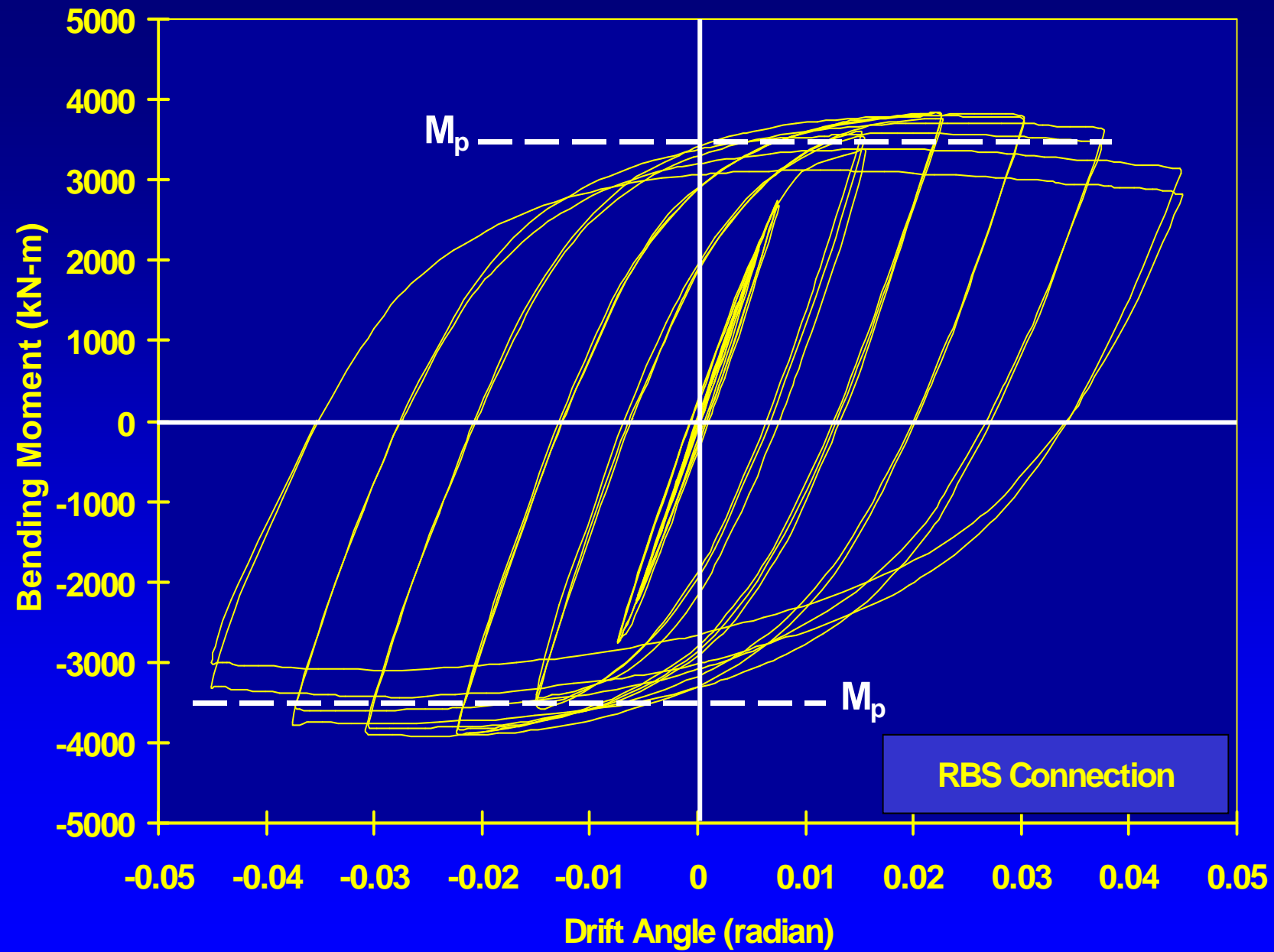


Connection at $\theta \cong 0.03$ radian.....



Connection at $\theta \cong 0.04$ radian.....





ANSI/AISC 358:

Prequalification Requirements for RBS in SMF

- Beam depth: up to W36
- Beam weight: up to 300 lb/ft
- Column depth: up to W36 for wide-flange
up to 24-inches for box columns
- Beam connected to column flange
(connections to column web not prequalified)
- RBS shape: circular
- RBS dimensions: per specified design procedure

ANSI/AISC 358:

Prequalification Requirements for RBS in SMF

cont.....

- Beam flange welds:
- CJP groove welds
 - Treat welds as *Demand Critical*
 - Remove bottom flange backing and provide reinforcing fillet weld
 - Leave top flange backing in-place; fillet weld backing to column flange
 - Remove weld tabs at top and bottom flanges

Beam web to column connection:

- Use fully welded web connection (CJP weld between beam web and column flange)

See ANSI/AISC 358 for additional requirements (continuity plates, beam lateral bracing, RBS cut finish req'ts., etc.)

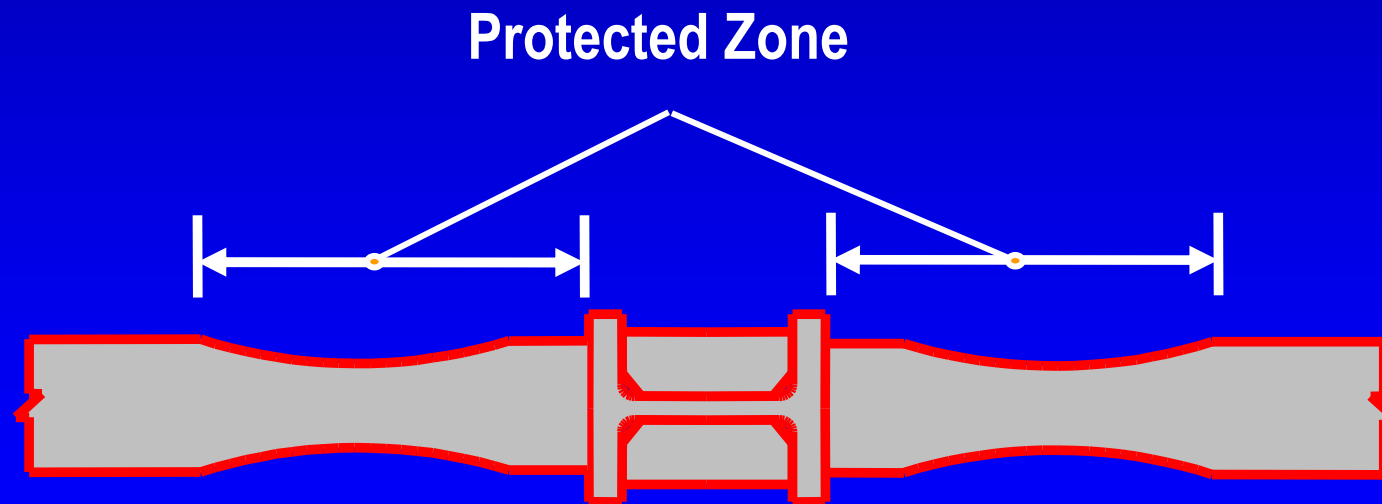
RBS with welded web
connection:



ANSI/AISC 358:

Prequalification Requirements for RBS in SMF

cont.....



Lateral brace at center of RBS - violates Protected Zone



Examples of RBS Connections.....



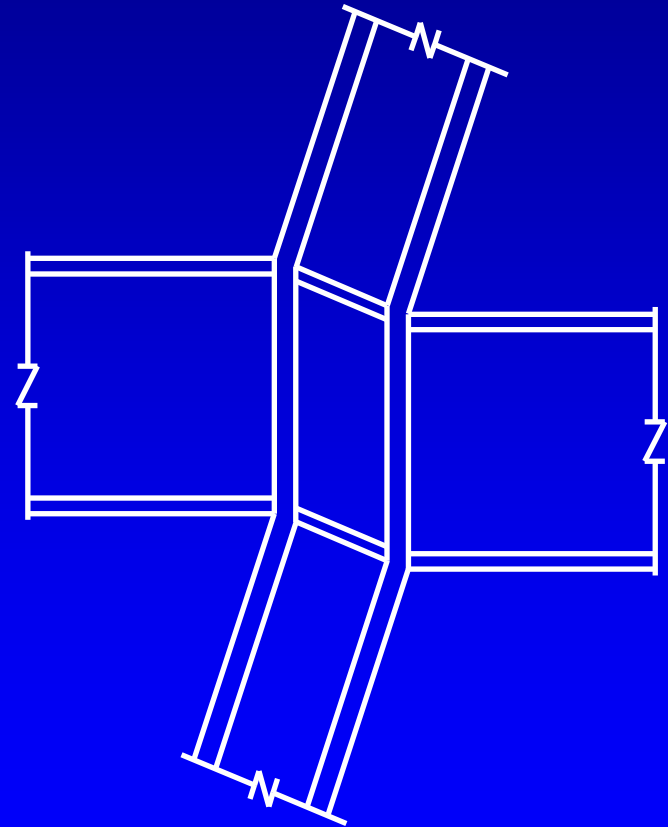
AISC Seismic Provisions - SMF

9.3 Panel Zone of Beam-to-Column Connections

9.3a Shear Strength

9.3b Panel Zone Thickness

9.3c Panel Zone Doubler Plates



AISC Seismic Provisions - SMF - Panel Zone Requirements

9.3a Shear Strength

The minimum required shear strength, R_u , of the panel zone shall be taken as the shear generated in the panel zone when plastic hinges form in the beams.

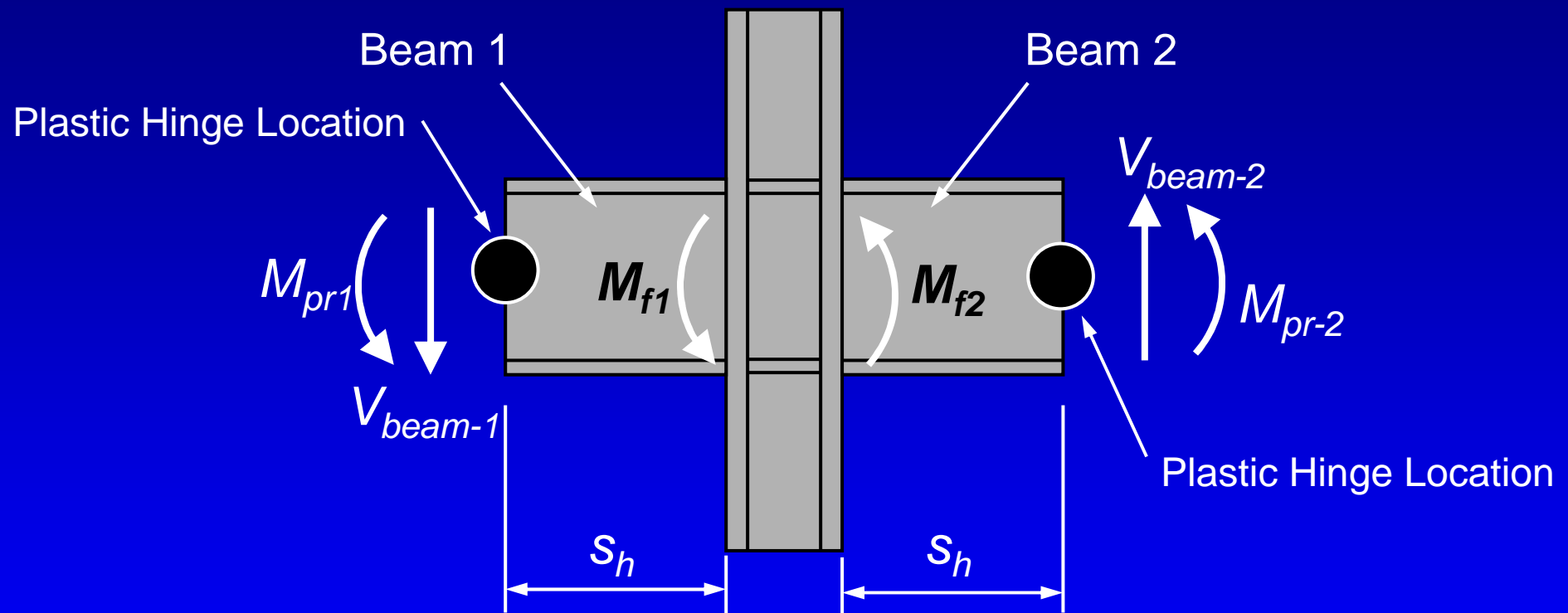
To compute panel zone shear.....

Determine moment at beam plastic hinge locations
($1.1 R_y M_p$ or as specified in ANSI/AISC 358)

Project moment at plastic hinge locations to the face
of the column (based on beam moment gradient)

Compute panel zone shear force.

Panel Zone Shear Strength (cont)

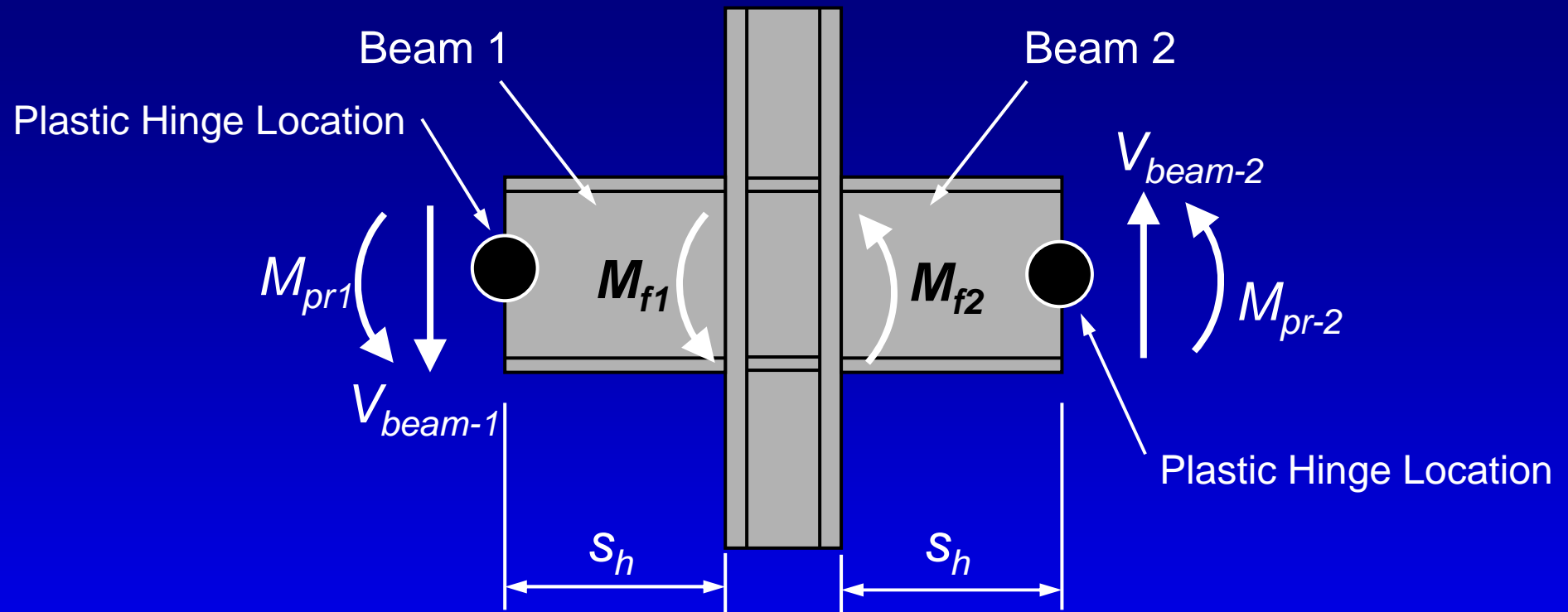


M_{pr} = expected moment at plastic hinge = $1.1 R_y M_p$ or as specified in ANSI/AISC 358

V_{beam} = beam shear (see Section 9.2a - beam required shear strength)

s_h = distance from face of column to beam plastic hinge location (specified in ANSI/AISC 358)

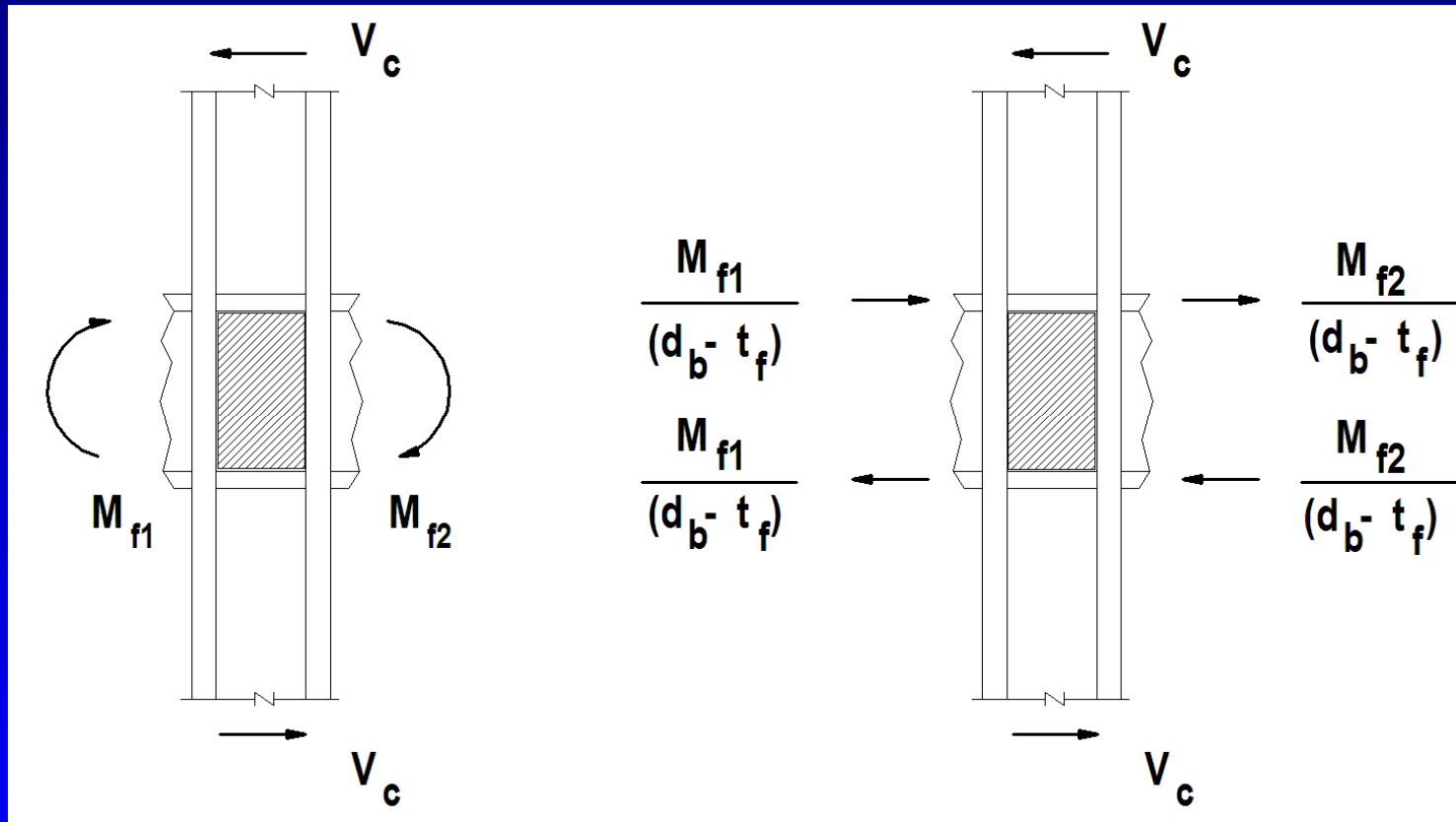
Panel Zone Shear Strength (cont)



M_f = moment at column face

$$M_f = M_{pr} + V_{beam} \times s_h$$

Panel Zone Shear Strength (cont)

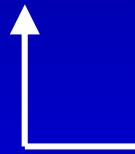


Panel Zone Required Shear Strength = $R_u = \frac{\sum M_f}{(d_b - t_f)} - V_c$

Panel Zone Shear Strength (cont)

Panel Zone Design Requirement:

$$R_u \leq \phi_v R_v \quad \text{where } \phi_v = 1.0$$



R_v = nominal shear strength, based on a limit state of shear yielding, as computed per Section J10.6 of the *AISC Specification*

Panel Zone Shear Strength (cont)

To compute nominal shear strength, R_v , of panel zone:

When $P_u \leq 0.75 P_y$ in column:

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_p} \right] \quad (\text{AISC Spec EQ J10-11})$$

Where:

d_c	=	column depth
d_b	=	beam depth
b_{cf}	=	column flange width
t_{cf}	=	column flange thickness
F_y	=	minimum specified yield stress of column web
t_p	=	thickness of column web including doubler plate

Panel Zone Shear Strength (cont)

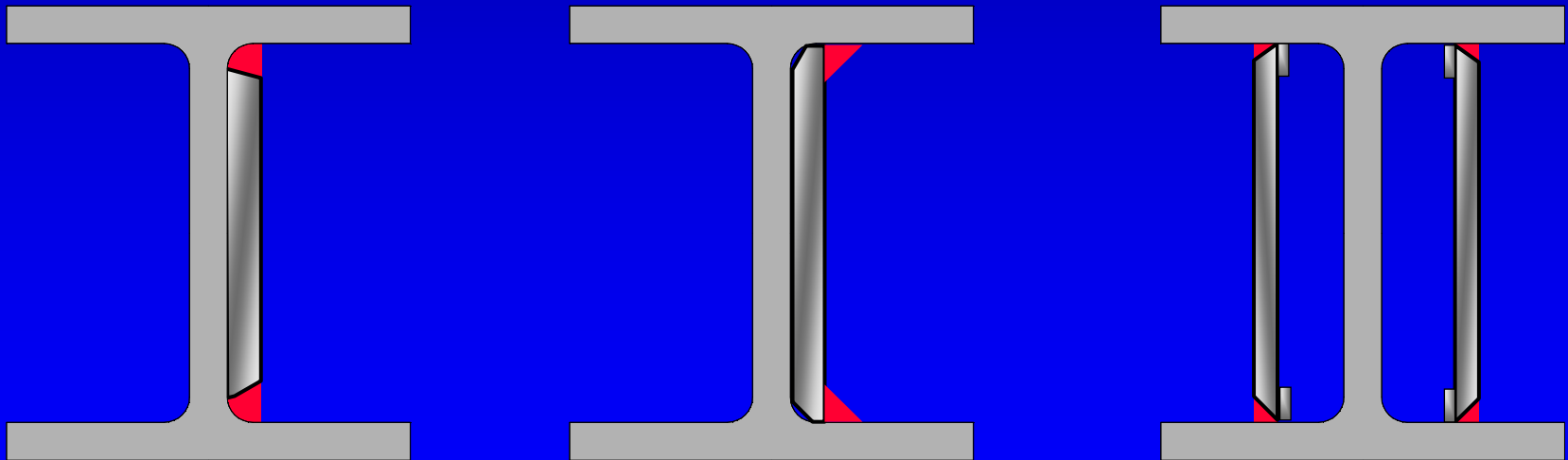
To compute nominal shear strength, R_v , of panel zone:

When $P_u > 0.75 P_y$ in column (not recommended):

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_p} \right] \left[1.9 - \frac{1.2P_u}{P_y} \right] \quad (\text{AISC Spec EQ J10-12})$$

If shear strength of panel zone is inadequate:

- Choose column section with larger web area
- Weld doubler plates to column



Options for Web Doubler Plates

AISC Seismic Provisions - SMF

9.4 Beam and Column Limitations

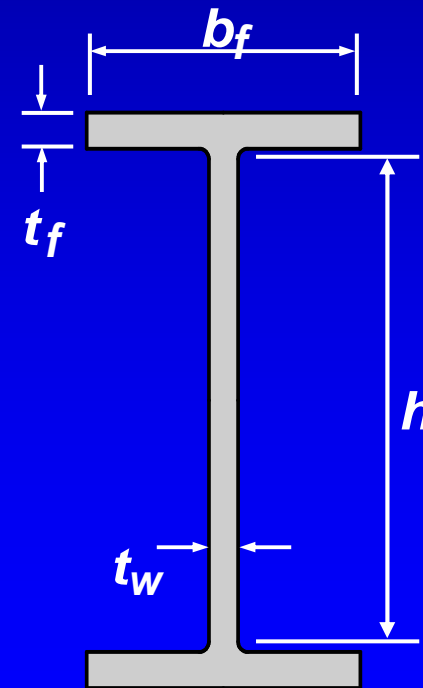
Beam and column sections must satisfy the width-thickness limitations given in Table I-8-1

Beam Flanges

$$\frac{b_f}{2t_f} \leq 0.30 \sqrt{\frac{E_s}{F_y}}$$

Beam Web

$$\frac{h}{t_w} \leq 2.45 \sqrt{\frac{E_s}{F_y}}$$



9.4 Beam and Column Limitations

Column Flanges

$$\frac{b_f}{2t_f} \leq 0.30 \sqrt{\frac{E_s}{F_y}}$$

Column Web

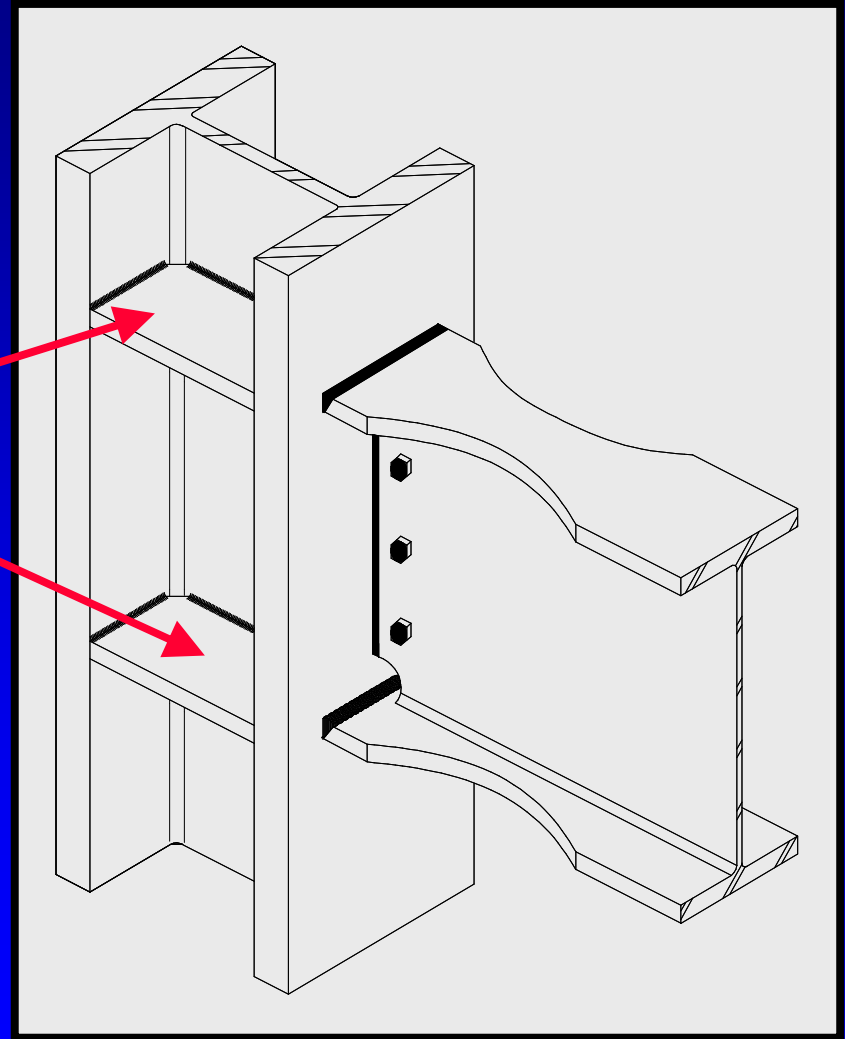
$$\frac{P_u}{\phi P_y} \leq 0.125 \quad \frac{h}{t_w} \leq 3.14 \sqrt{\frac{E_s}{F_y}} \left[1 + 1.54 \frac{P_u}{\phi P_y} \right]$$

$$\frac{P_u}{\phi P_y} > 0.125 \quad \frac{h}{t_w} \leq 1.12 \sqrt{\frac{E_s}{F_y}} \left[2.33 - \frac{P_u}{\phi P_y} \right] > 1.49 \sqrt{\frac{E_s}{F_y}}$$

Note: Column flange and web slenderness limits can be taken as λ_p in AISC Specification Table B4.1, if the ratio for Eq. 9-3 is greater than 2.0

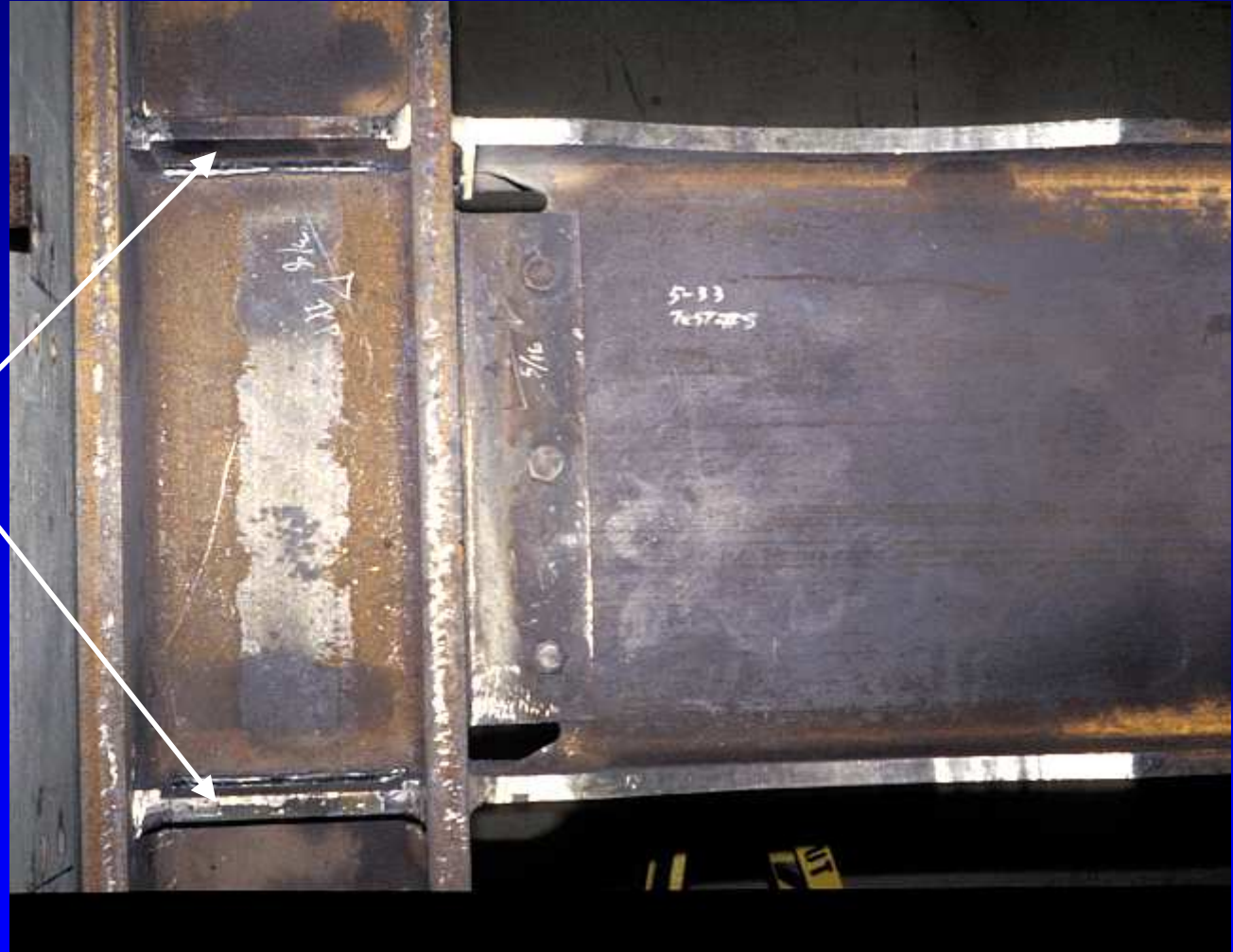
AISC Seismic Provisions - SMF
9.5 Continuity Plates

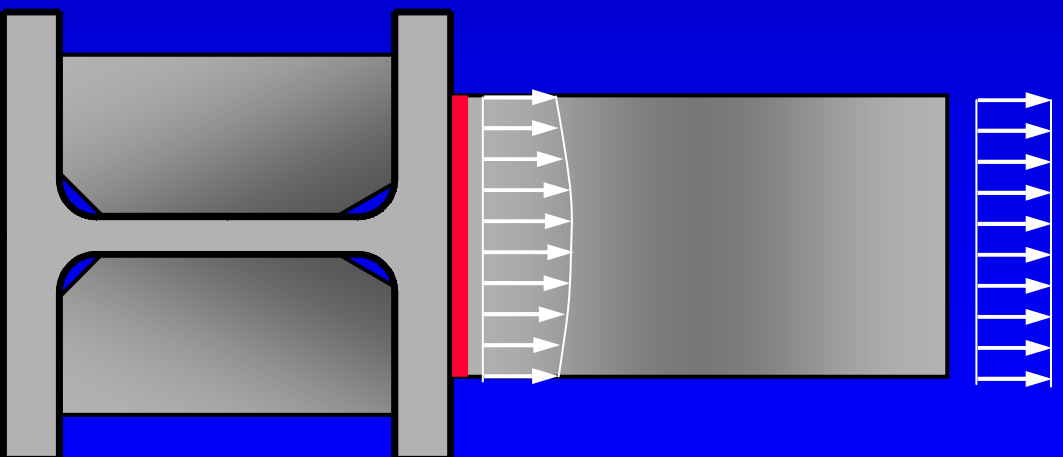
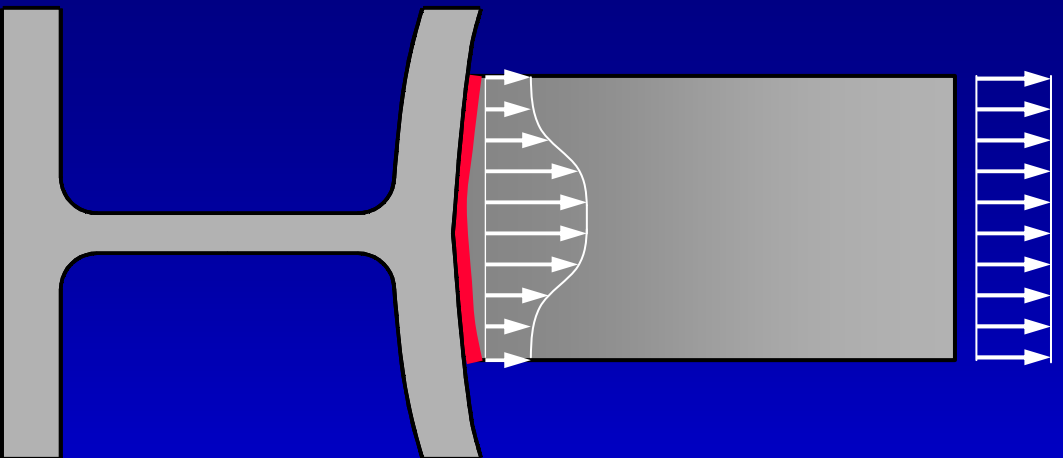
Continuity Plates



9.5 Continuity Plates

Continuity Plates





AISC Seismic Provisions - SMF

9.5 Continuity Plates

Continuity plates shall be consistent with the requirements of a prequalified connection as specified in **ANSI/AISC 358** (*Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*)

or

As determined in a program of qualification testing in accordance with Appendix S

ANSI/AISC 358 - Continuity Plate Requirements

For Wide-Flange Columns:

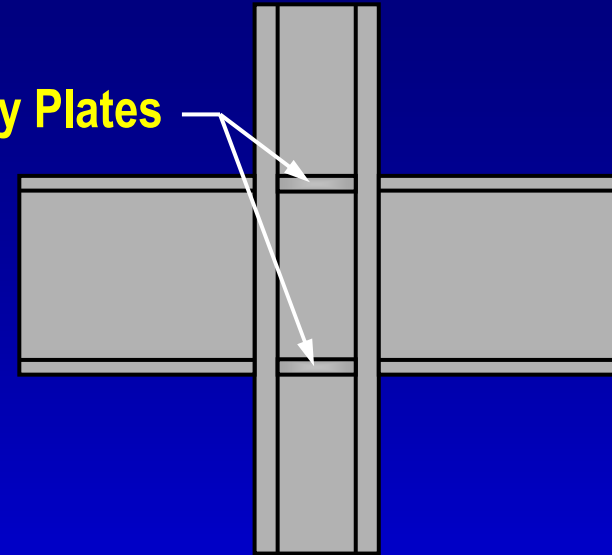
Continuity plates are required, unless:

$$t_{cf} \geq 0.4 \sqrt{1.8 b_{bf} t_{bf} \frac{R_{yb} F_{yb}}{R_{yc} F_{yc}}}$$

and

$$t_{cf} \geq \frac{b_{bf}}{6}$$

Continuity Plates



t_{cf} = column flange thickness

b_{bf} = beam flange width

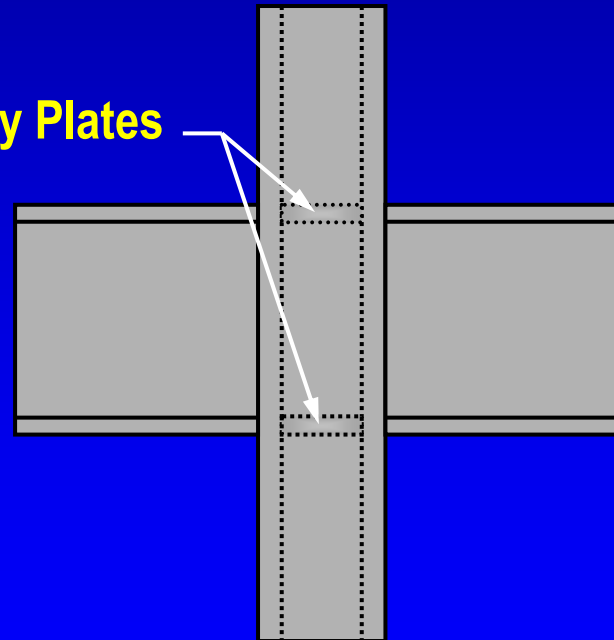
t_{bf} = beam flange thickness

ANSI/AISC 358 - Continuity Plate Requirements

For Box Columns:

Continuity plates must be provided.

Continuity Plates



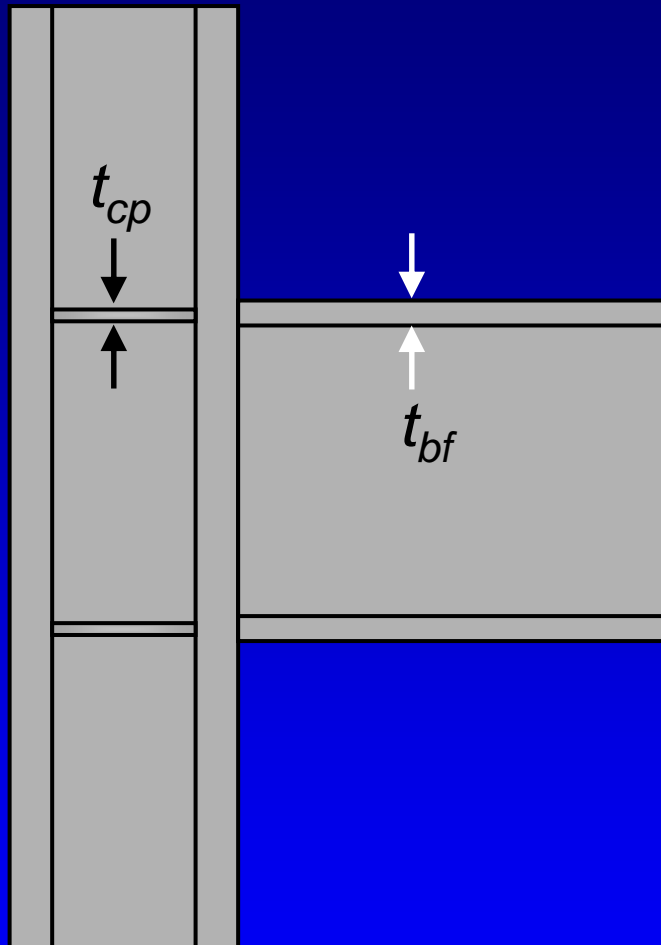
ANSI/AISC 358 - Continuity Plate Requirements

Required thickness of continuity plates

- a) For one-sided (exterior) connections, continuity plate thickness shall be at least one-half of the thickness of the beam flange.
- b) For two-sided (interior) connections, continuity plate thickness shall be at least equal to the thicker of the two beam flanges on either side of the column

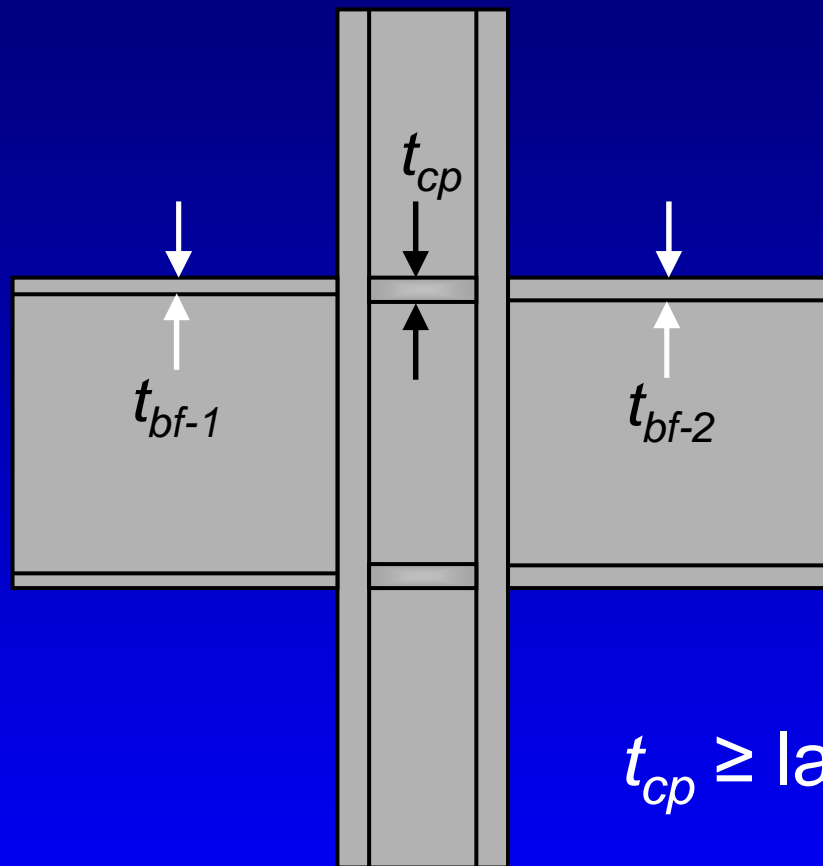
For other design, detailing and welding requirements for continuity plates - See ANSI/AISC 358

ANSI/AISC 358 - Continuity Plate Requirements



$$t_{cp} \geq 1/2 \times t_{bf}$$

ANSI/AISC 358 - Continuity Plate Requirements



$$t_{cp} \geq \text{larger of } (t_{bf-1} \text{ and } t_{bf-2})$$

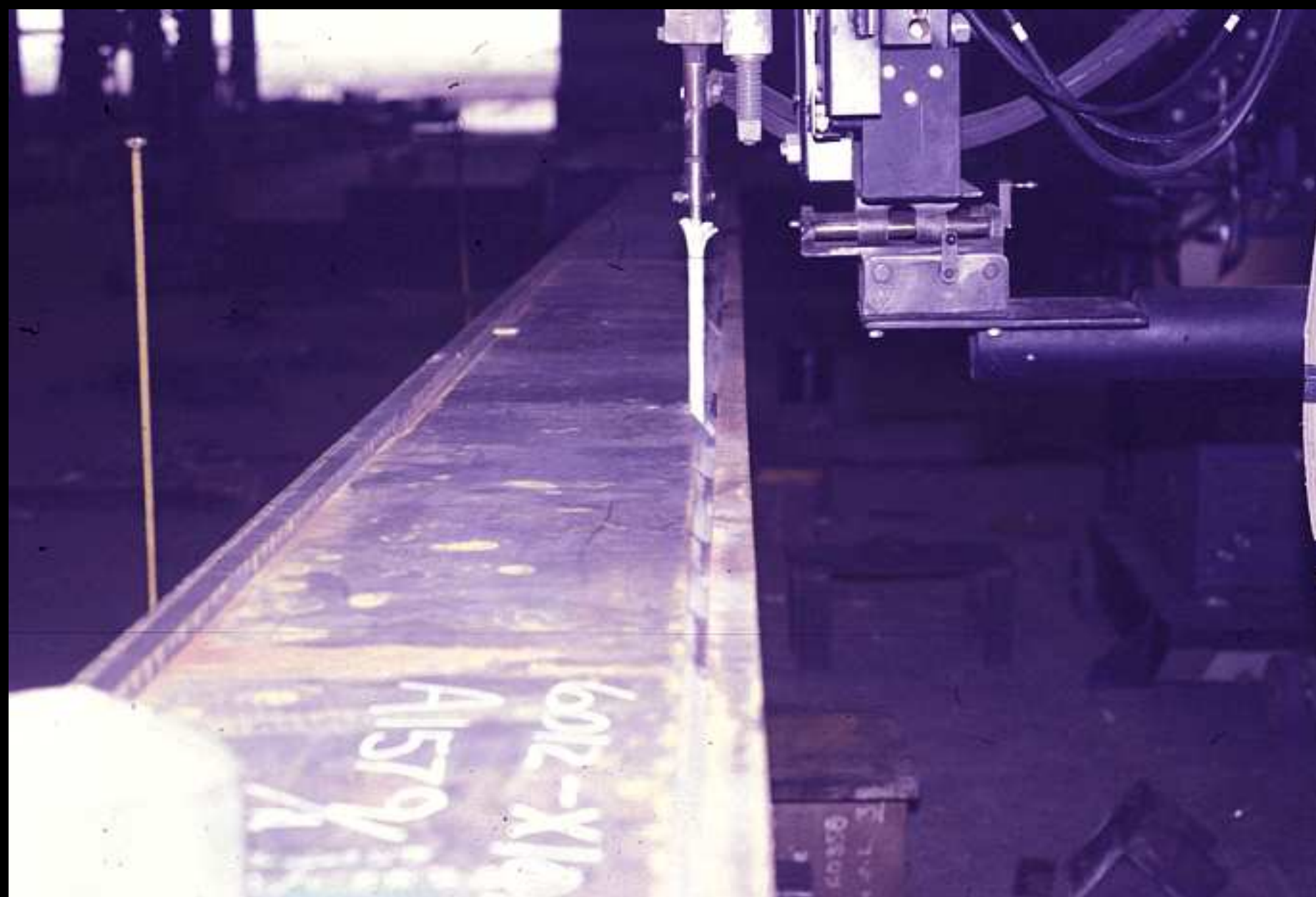












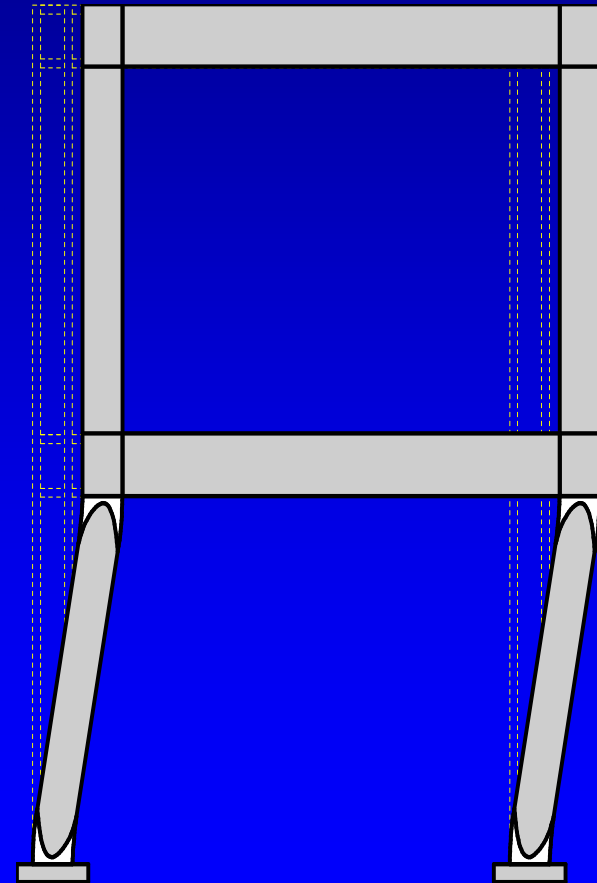
AISC Seismic Provisions - SMF

9.6 Column-Beam Moment Ratio

Section 9.6 requires **strong column - weak girder** design for SMF (with a few exceptions)

Purpose of strong column -
weak girder requirement:

Prevent Soft Story Collapse



AISC Seismic Provisions - SMF

9.6 Column-Beam Moment Ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0$$

Eqn. (9-3)

9.6 Column-Beam Moment Ratio

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0$$

$\sum M_{pc}^*$ = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines.

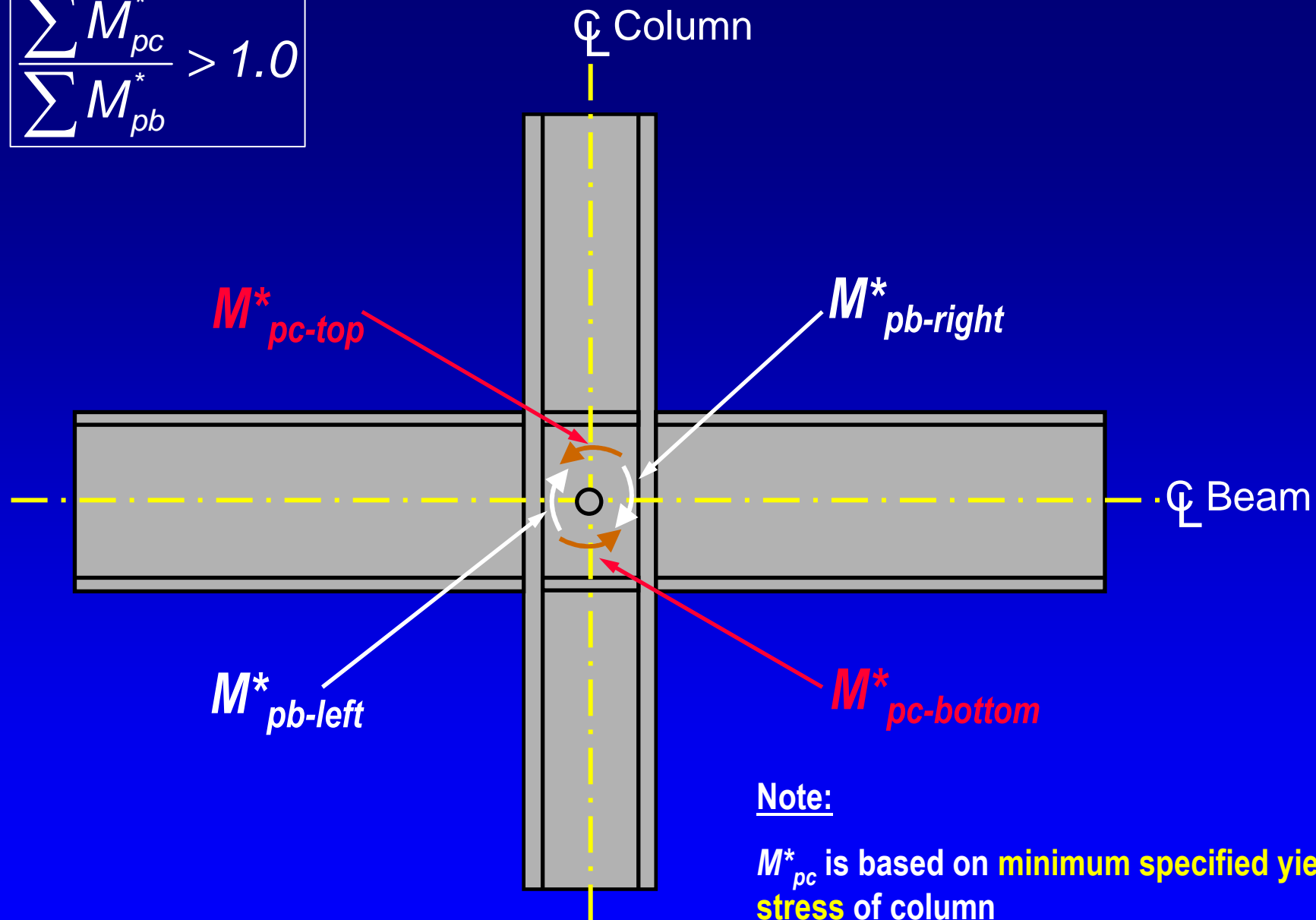
$\sum M_{pc}^*$ is determined by summing the projections of the **nominal flexural strengths of the columns** above and below the joint to the beam centerline with a reduction for the axial force in the column.

It is permitted to take $\sum M_{pc}^* = \sum Z_c (F_{yc} - P_{uc}/A_g)$

$\sum M_{pb}^*$ = the sum of the moments in the beams at the intersection of the beam and column centerlines.

$\sum M_{pb}^*$ is determined by summing the projections of the **expected flexural strengths of the beams** at the plastic hinge locations to the column centerline.

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0$$

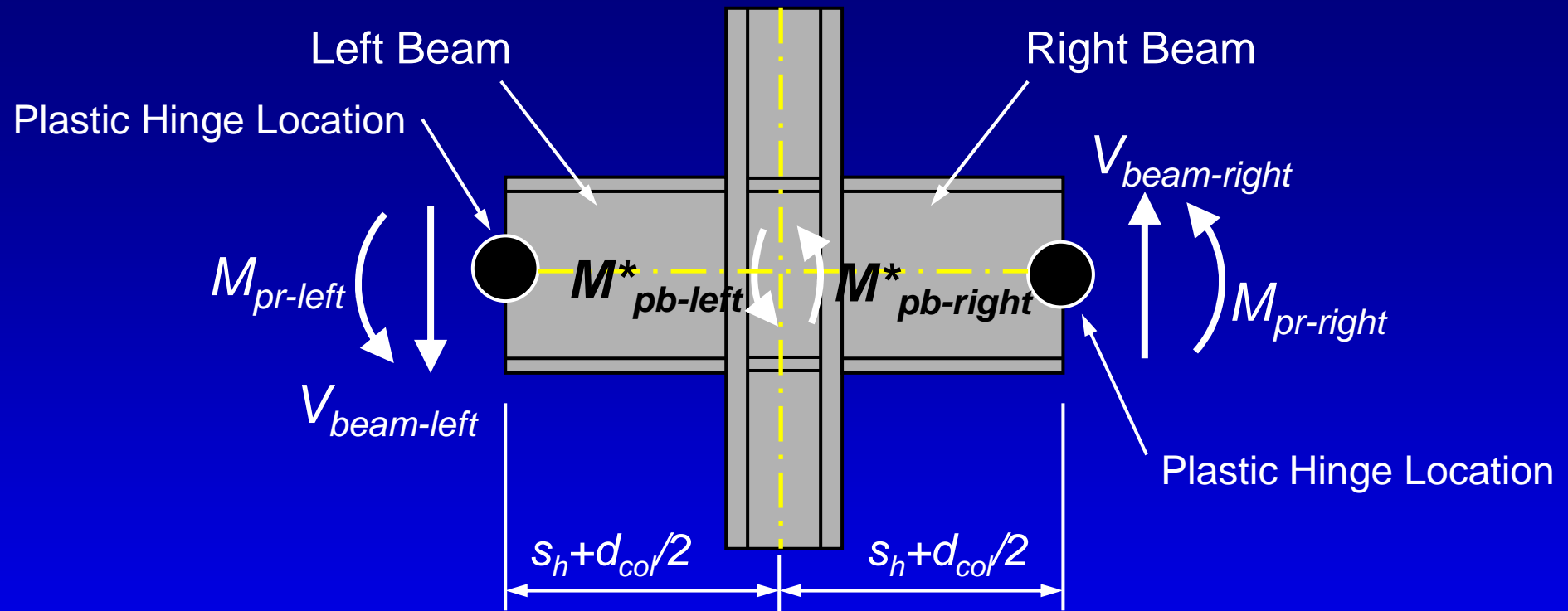


Note:

M_{pc}^* is based on **minimum specified yield stress** of column

M_{pb}^* is based on **expected yield stress** of beam and includes allowance for strain hardening

Computing M_{pb}^*



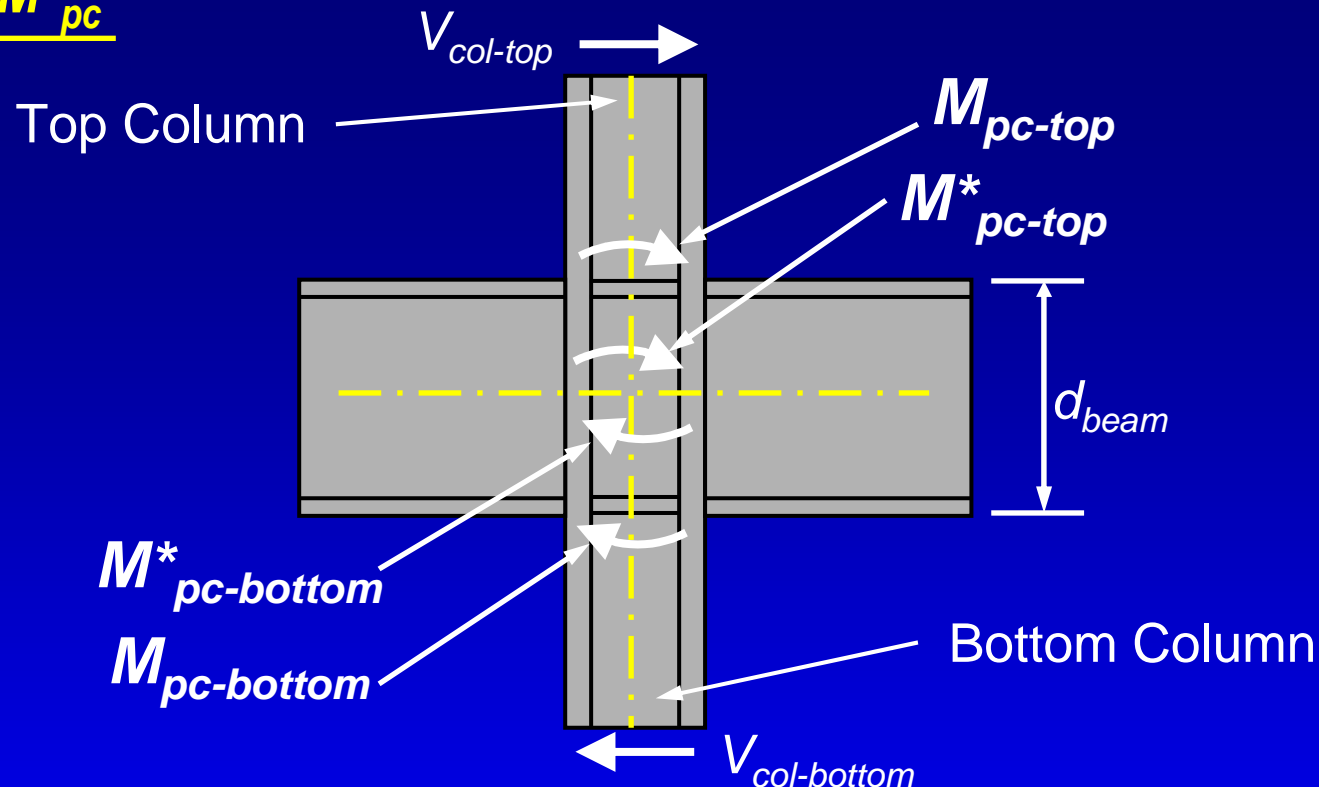
M_{pr} = expected moment at plastic hinge = $1.1 R_y M_p$ or as specified in ANSI/AISC 358

V_{beam} = beam shear (see Section 9.2a - beam required shear strength)

s_h = distance from face of column to beam plastic hinge location (specified in ANSI/AISC 358)

$$M_{pb}^* = M_{pr} + V_{beam} (s_h + d_{col}/2)$$

Computing M_{pc}^*



M_{pc} = nominal plastic moment capacity of column, reduced for presence of axial force; can take $M_{pc} = Z_c (F_{yc} - P_{uc} / A_g)$ [or use more exact moment-axial force interaction equations for a fully plastic cross-section]

V_{col} = column shear - compute from statics, based on assumed location of column inflection points (usually midheight of column)

$$M_{pc}^* = M_{pc} + V_{col} (d_{beam} / 2)$$

AISC Seismic Provisions - SMF

9.8 Lateral Bracing of Beams

Must provide adequate lateral bracing of beams in SMF so that severe strength degradation due to **lateral torsional buckling is delayed** until sufficient ductility is achieved

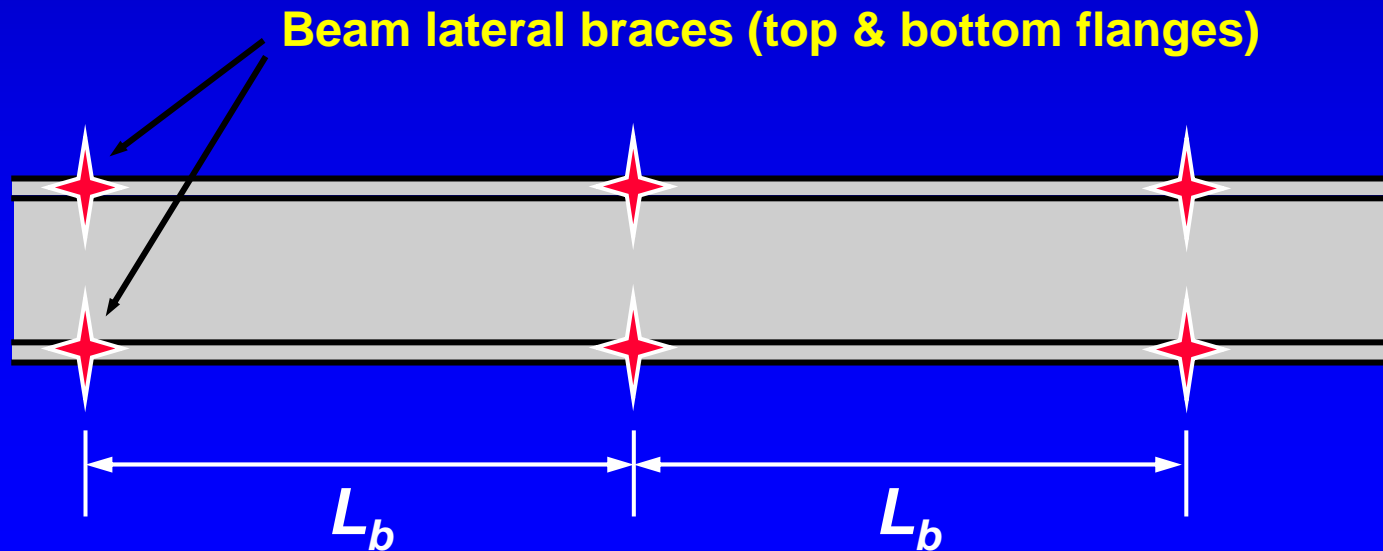
(Sufficient ductility = interstory drift angle of at least 0.04 rad is achieved under Appendix S loading protocol)

Lateral Torsional Buckling

Lateral torsional
buckling controlled by: $\frac{L_b}{r_y}$

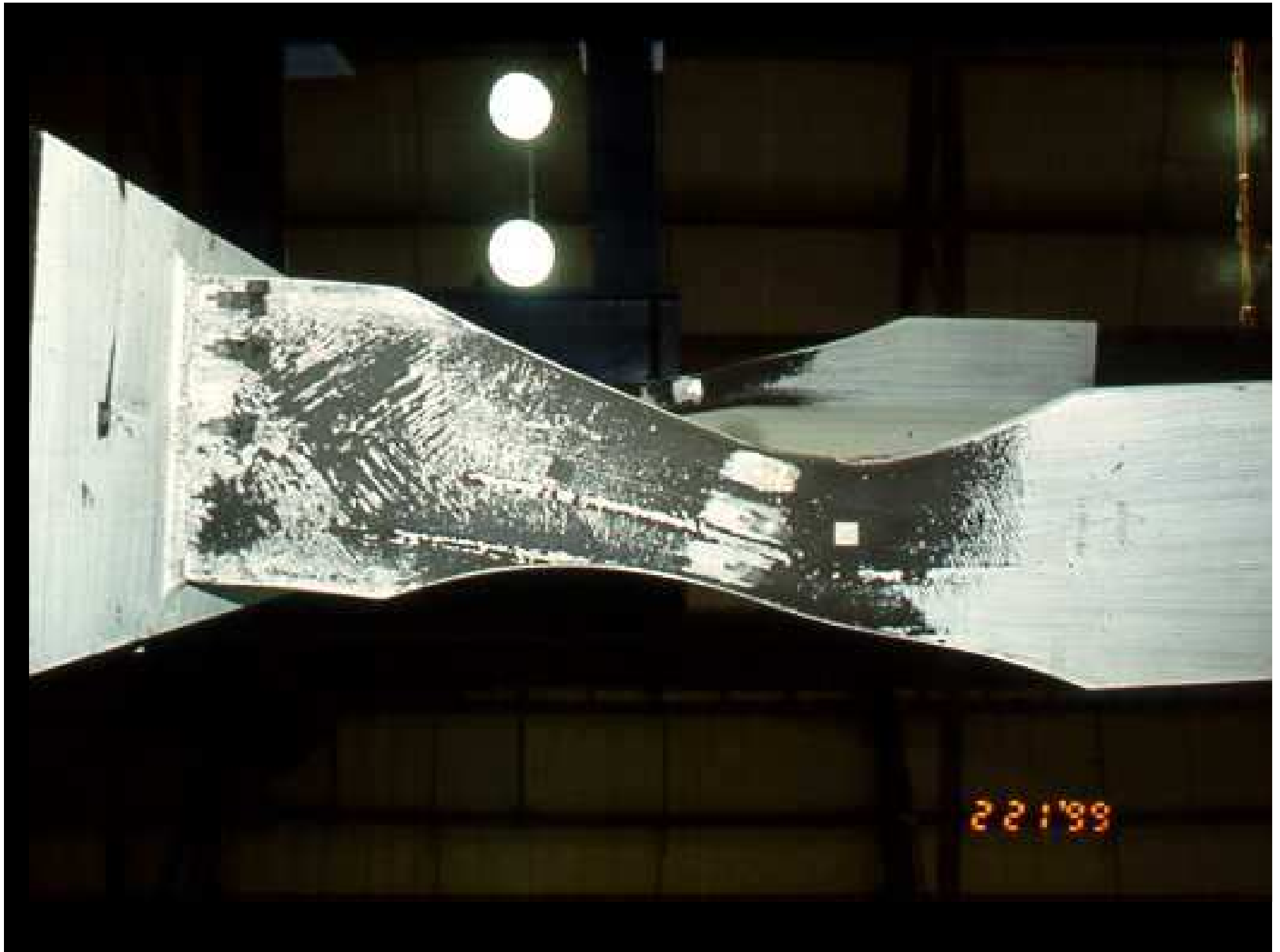
L_b = distance between beam lateral braces

r_y = weak axis radius of gyration

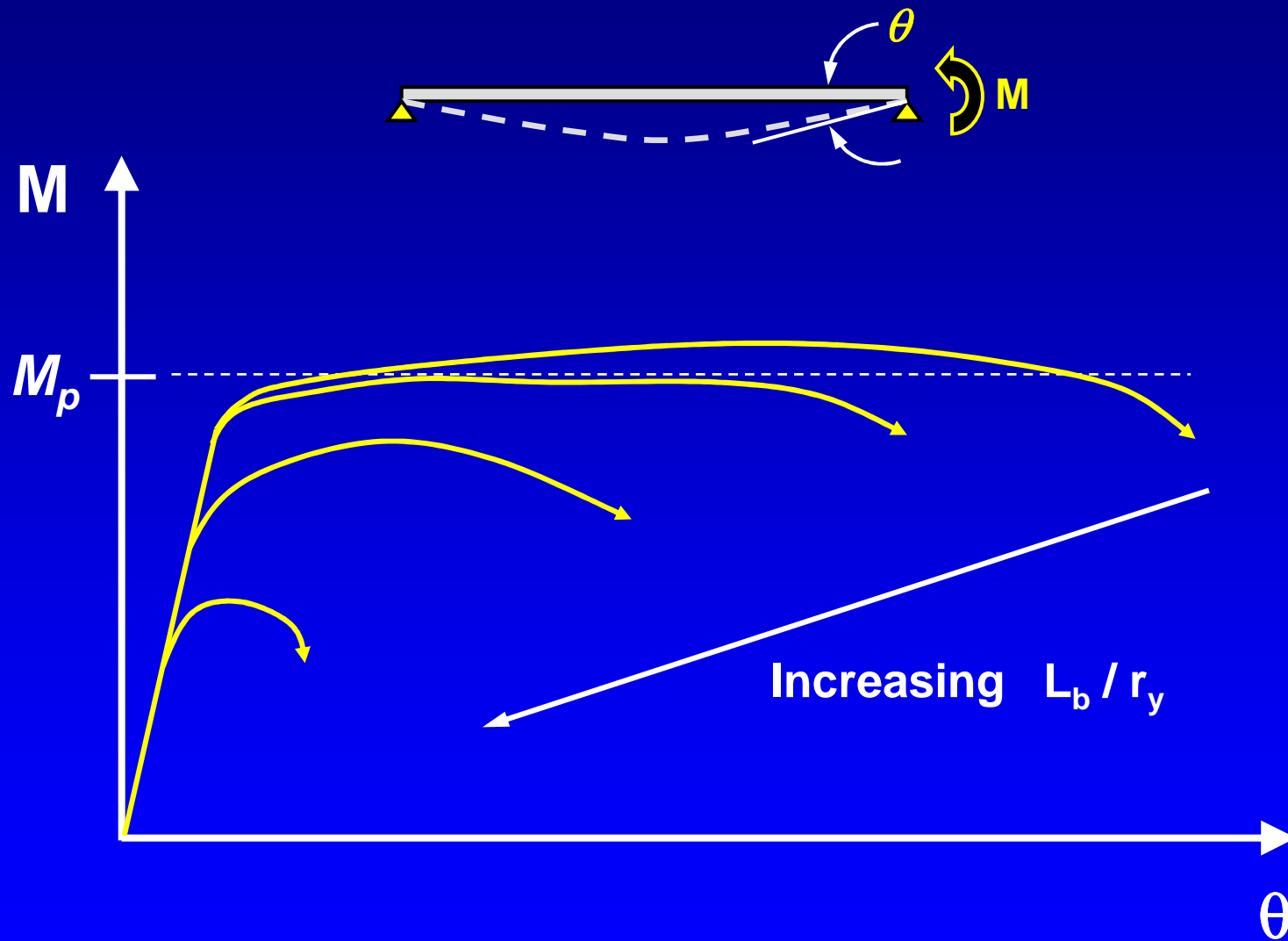








Effect of Lateral Torsional Buckling on Flexural Strength and Ductility:



AISC Seismic Provisions - SMF

9.8 Lateral Bracing of Beams

Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = 0.086 r_y E / F_y$

$$L_b \leq 0.086 \left(\frac{E}{F_y} \right) r_y \quad (= 50 r_y \text{ for } F_y = 50 \text{ ksi})$$

Note:

**For typical SMF beam: $r_y \cong 2$ to 2.5 inches.
and $L_b \cong 100$ to 125 inches (approx. 8 to 10 ft)**





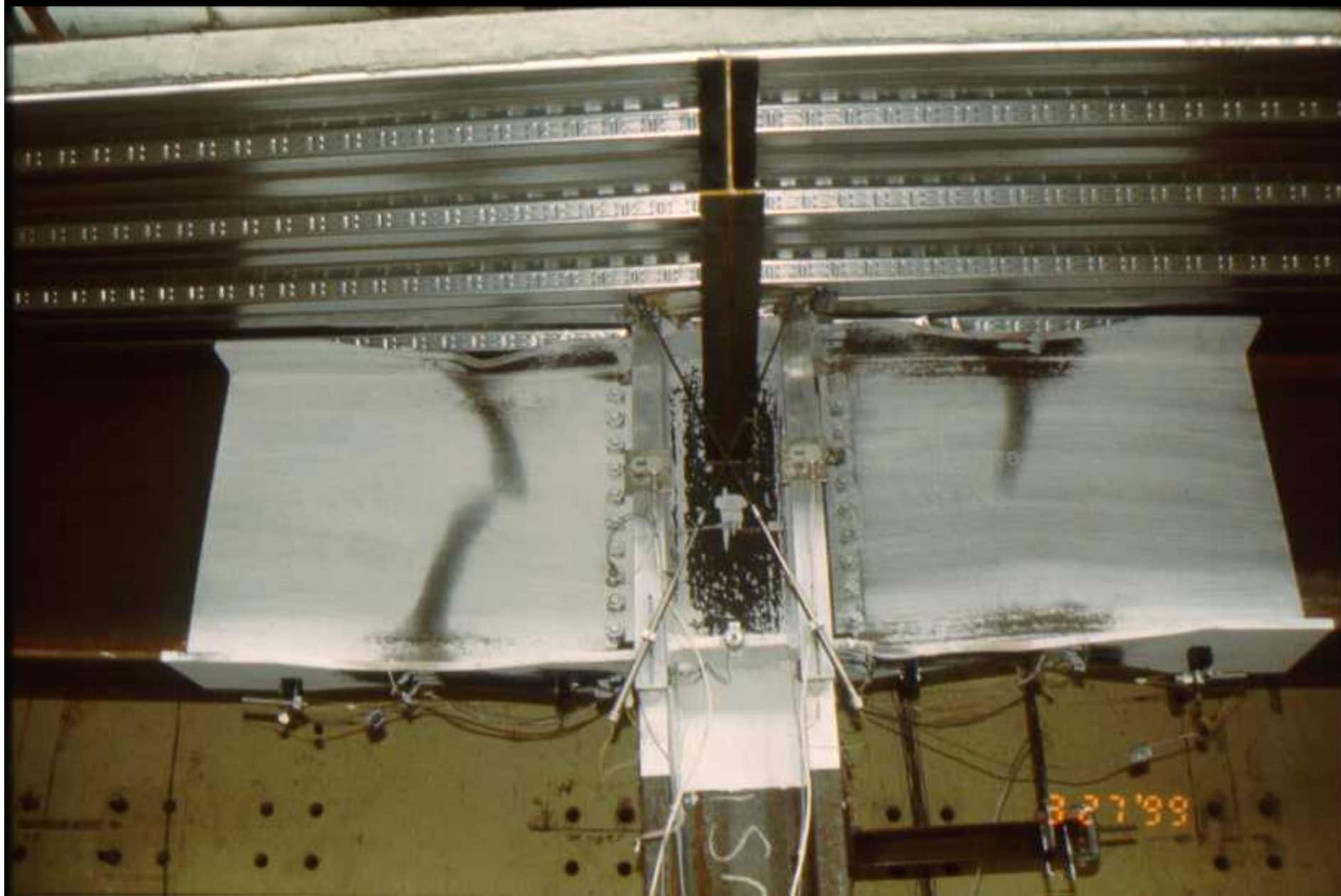
AISC Seismic Provisions - SMF

9.8 Lateral Bracing of Beams

In addition to lateral braces provided as a maximum spacing of $L_b = 0.086 r_y E / F_y$:

Lateral braces shall be placed near concentrated forces, changes in cross-section and other locations where analysis indicates that a plastic hinge will form.

The placement of lateral braces shall be consistent with that specified in ANSI/AISC 358 for a *Prequalified Connection*, or as otherwise determined by qualification testing.





ANSI/AISC 358 - Lateral Bracing Requirements for the RBS

For beams with an RBS connection:

When a composite concrete floor slab is present, no additional lateral bracing is required at the RBS.

When a composite concrete floor slab is not present, provide an additional lateral brace at the RBS. Attach brace just outside of the RBS cut, at the end farthest from the column face.



Section 9

Special Moment Frames (SMF)

- 9.1 Scope**
- 9.2 Beam-to-Column Joints and Connections**
- 9.3 Panel Zone of Beam-to-Column Connections**
- 9.4 Beam and Column Limitations**
- 9.5 Continuity Plates**
- 9.6 Column-Beam Moment Ratio**
- 9.7 Lateral Bracing of at Beam-to-Column Connections**
- 9.8 Lateral Bracing of Beams**
- 9.9 Column Splices**