Revisiting W1 Indications

Duane K. Miller, Sc.D, P.E.

LINCOLN® ELECTRIC

NORTHRIDGE 20
SYMPOSIUM

MISSION:
IMPOSSIBLE

Revisiting W1 Indications

**How W1s mislead many in the aftermath of Northridge…
…and continue to do so today.**

Revisiting W1 Indications

W1s: What Are They?

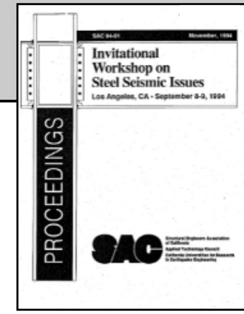
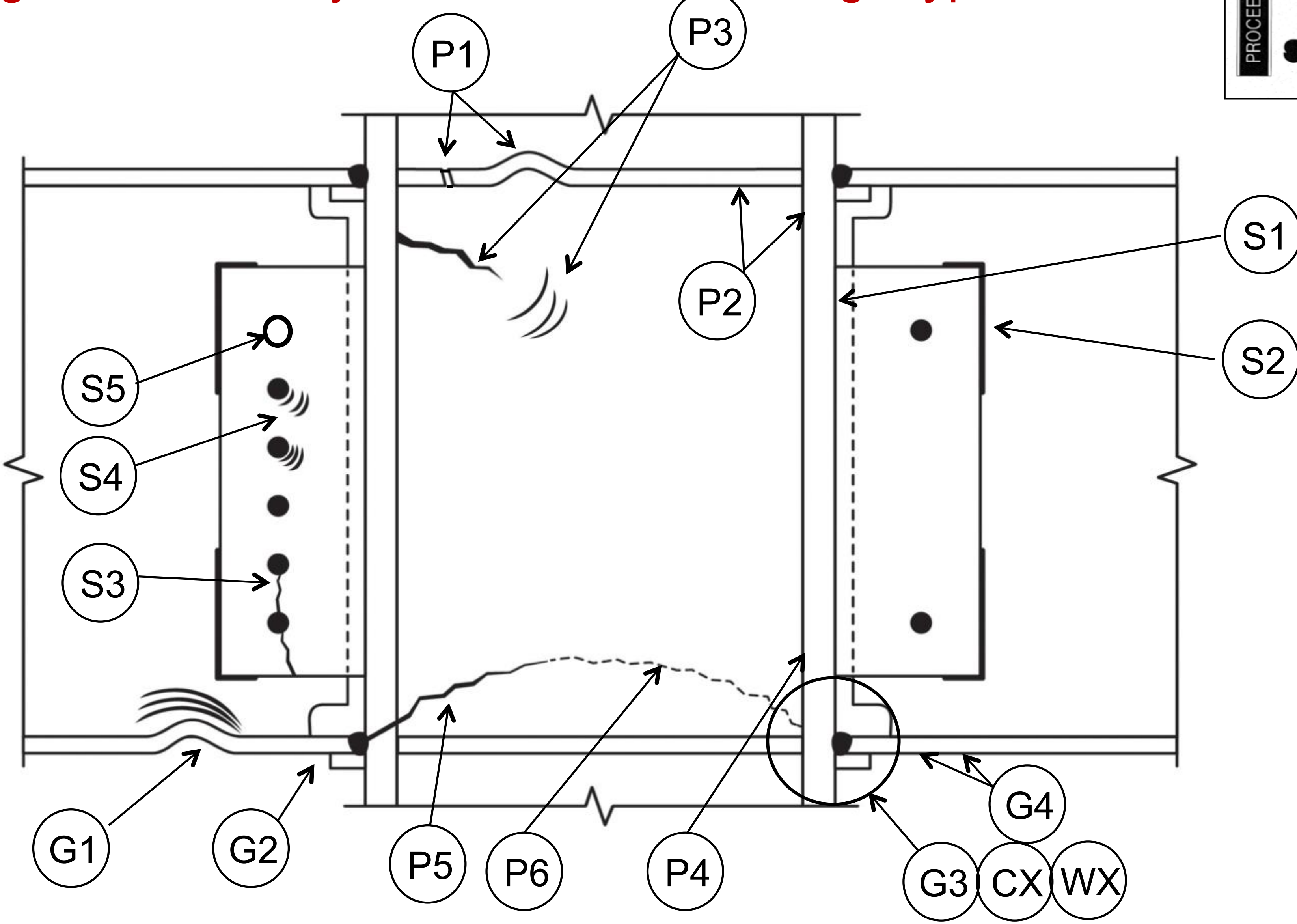


Figure 2b. Survey form Section V damage types.



94-01 SAC Invitational Workshop on Steel Seismic Issues

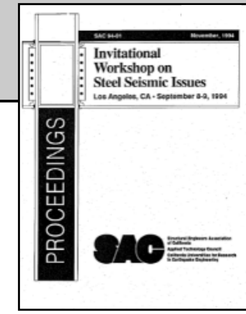
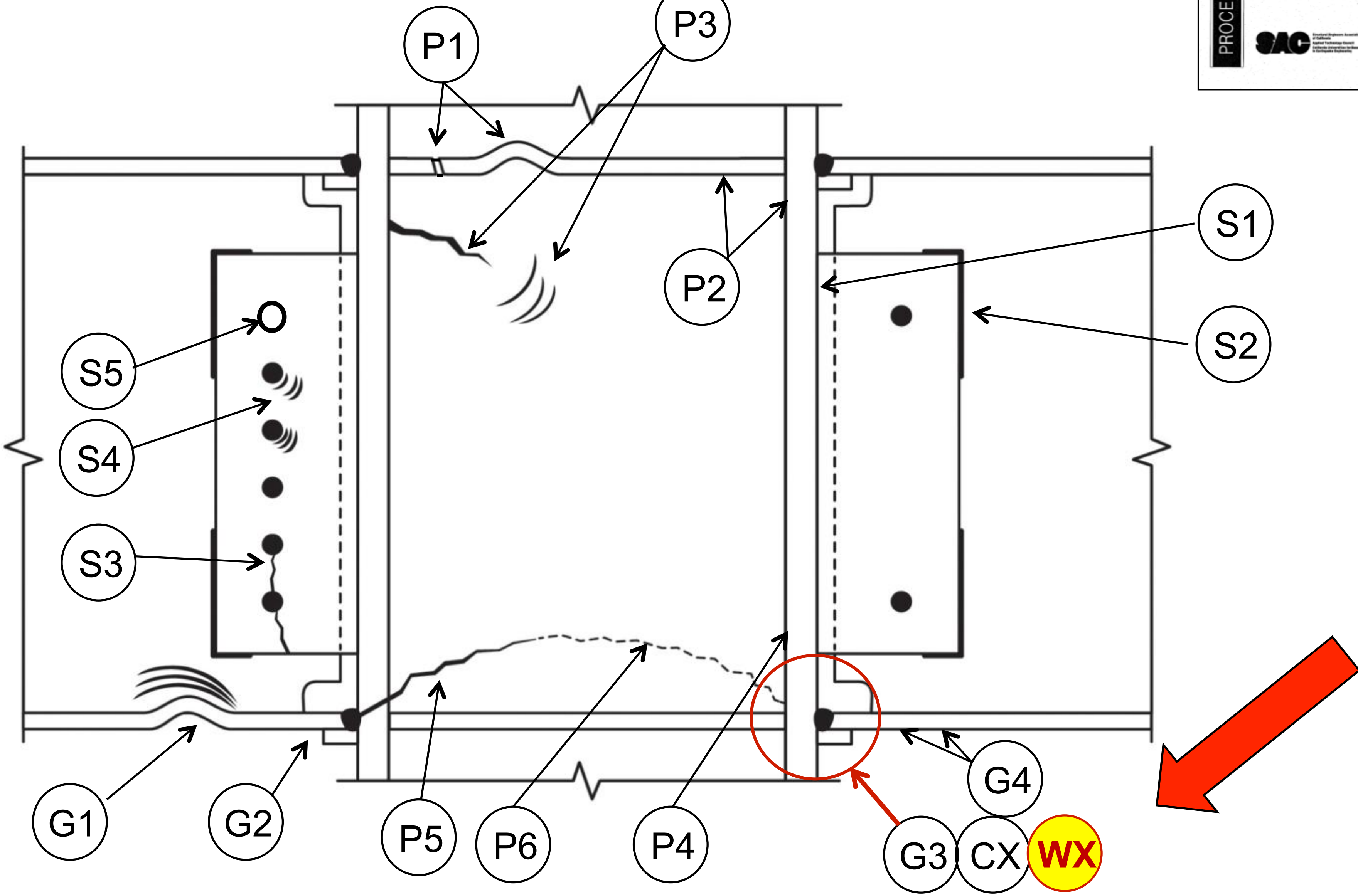


Figure 2b. Survey form Section V damage types.



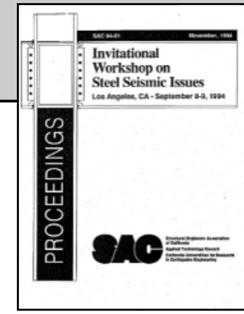
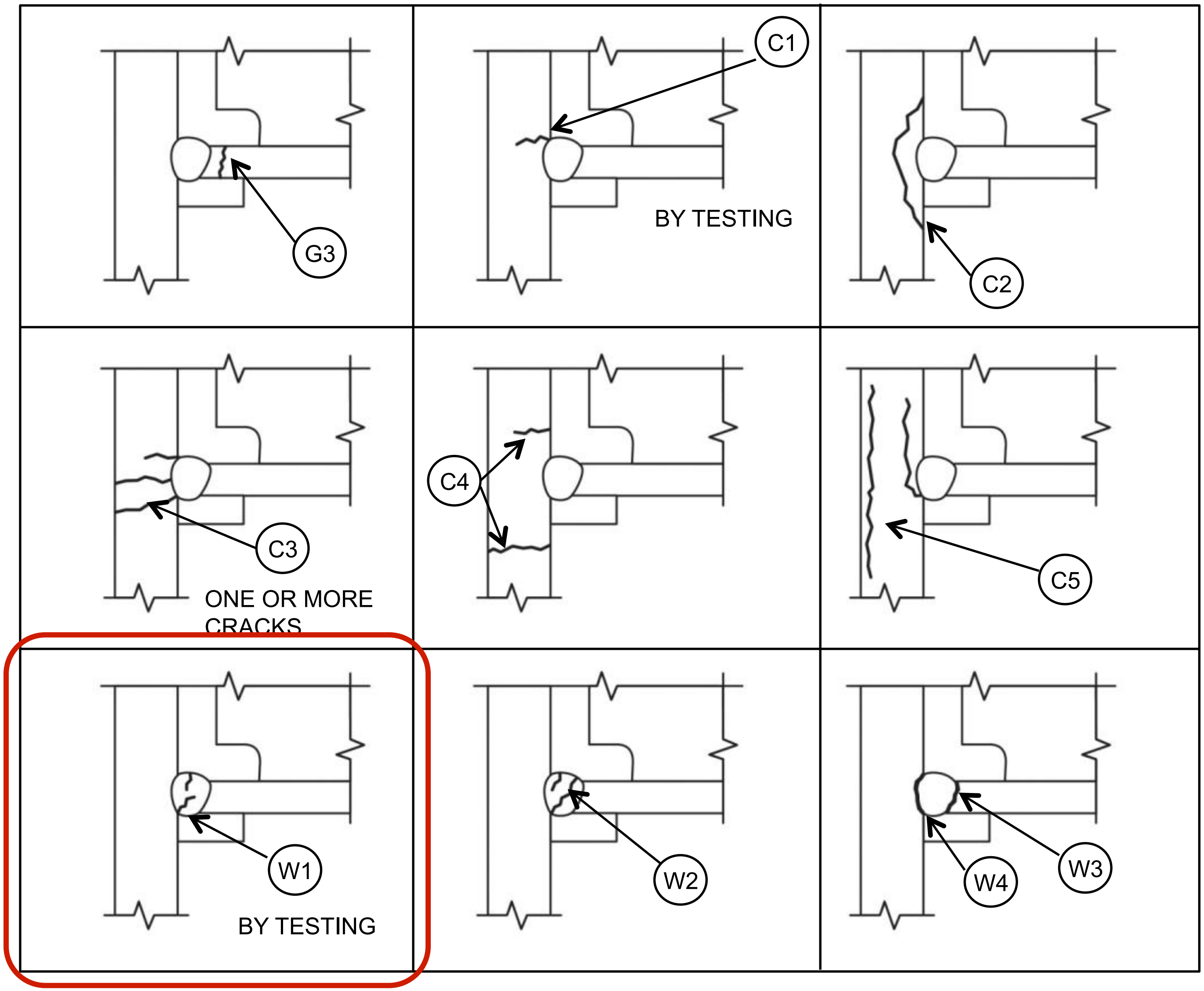


Figure 2c. Form Section V damage types, continued.



94-01 SAC Invitational Workshop on Steel Seismic Issues

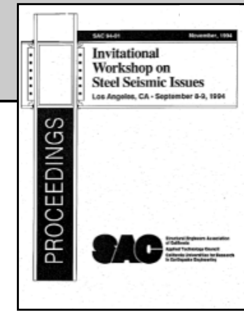
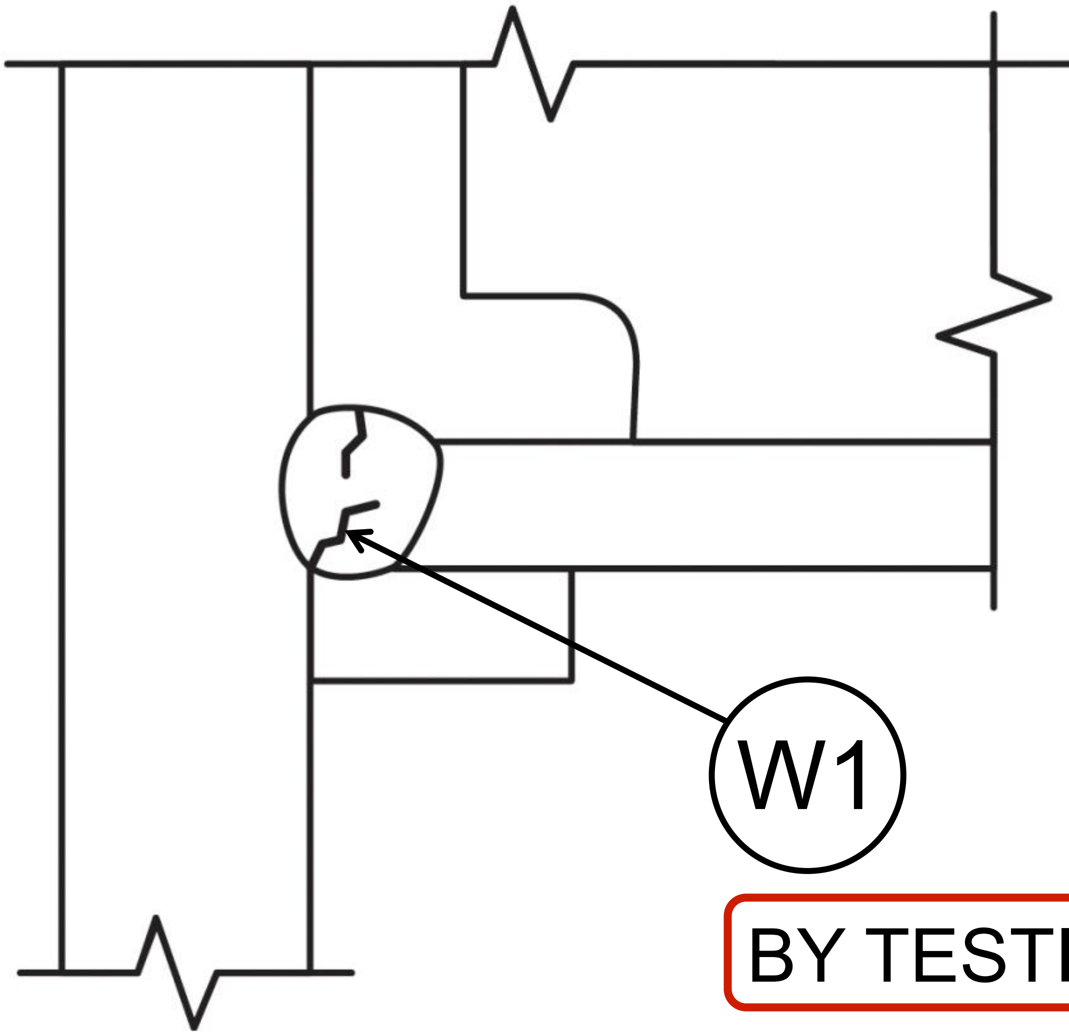


Figure 2c. Form Section V damage types, continued.



BY TESTING

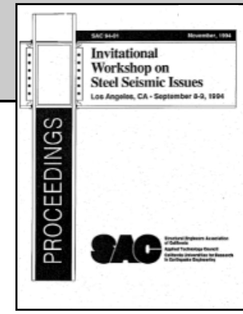


Figure 2a. Survey form Section V damage description page.

SURVEY OF STEEL MRF BUILDINGS DAMAGED BY THE NORTHRIDGE EARTHQUAKE, JANUARY, 1994

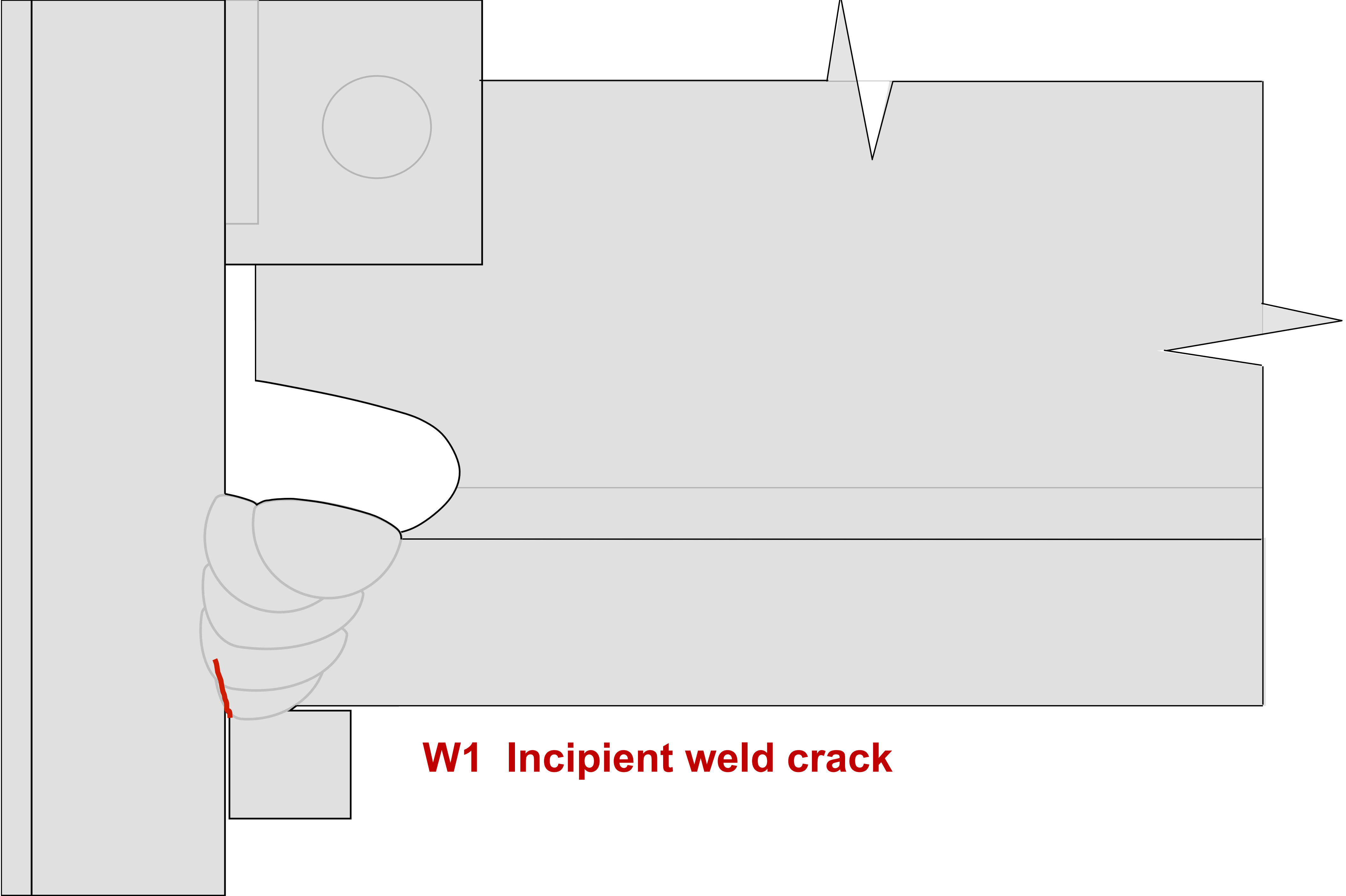
Building: ______
Engineer: ______
Firm: ______
Date: ______ Page: ______

FLOOR ______ FRAME ______

Flange Weld Damage	W1	Incipient weld crack
	W2	Full or partial crack through weld metal
	W3	Fracture at girder interface
	W4	Fracture at column interface

	C5	Lamellar flange crack	5518X	5519X
Flange Weld Damage	W1	Incipient weld crack	5520X	5521X
	W2	Full or partial crack through weld metal	5522X	5523X
	W3	Fracture at girder interface	5524X	5525X
	W4	Fracture at column interface	5526X	5527X
Shear Connection Damage	S1	Weld crack at column (welded web only)	5528X	5529X
	S2	Weld crack at shear tab	5530X	5531X
	S3	Crack in girder web or shear plate through bolt holes	5532X	
	S4	Plastic deformation of web or plate at bolt holes	5533X	
	S5	Loose, damaged, or missing bolts	5534X	
Panel Zone Damage	P1	Damage to continuity plate	5535X	5536X
	P2	Crack in continuity plate weld	5537X	5538X
	P3	Damage to doubler plate	5539X	
	P4	Crack in doubler plate weld	5540X	
	P5	Partial depth crack in column web (extension of C3)	5541X	5542X
	P6	Full (or near full) depth crack in column web	5543X	5544X

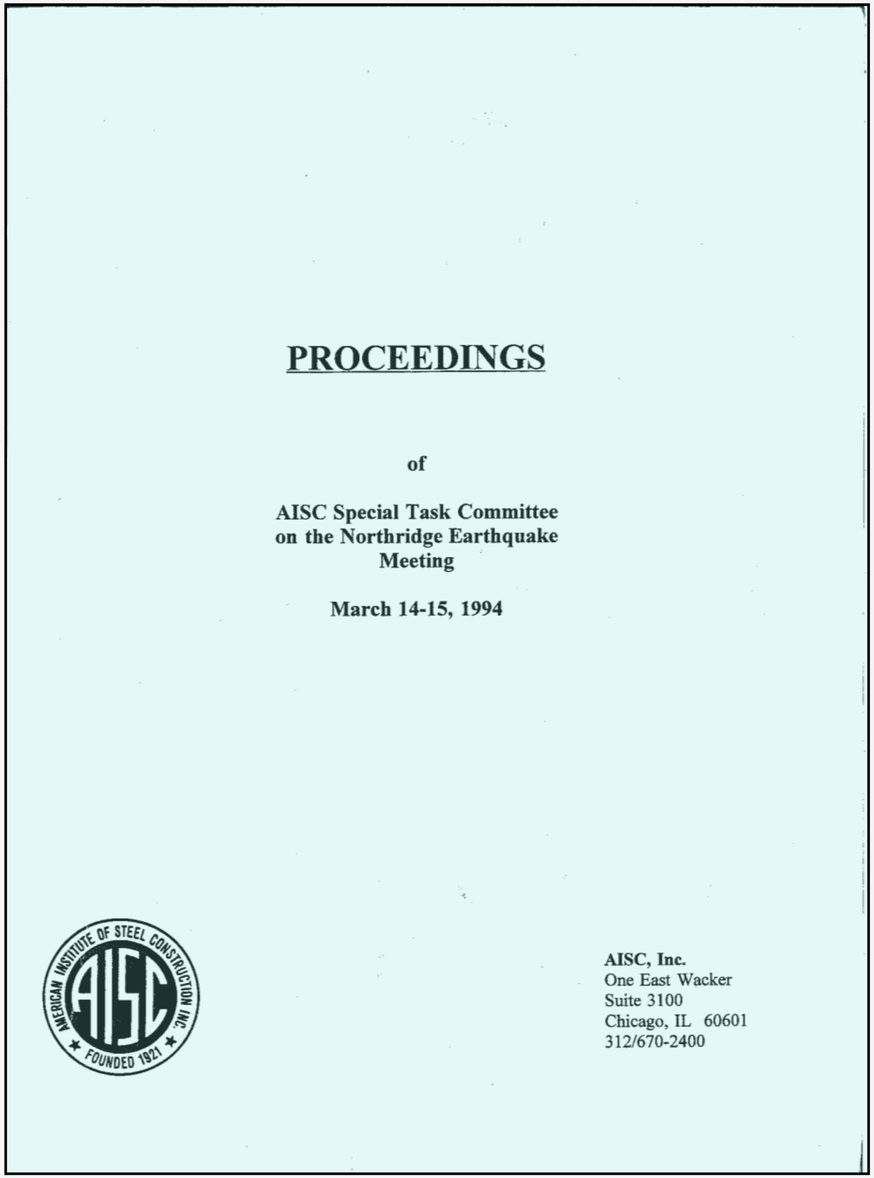
Provide additional descriptions of MRF joint damage as appropriate: ______ 5545X


W1 Incipient weld crack

PROCEEDINGS

of
AISC Special Task Committee
on the Northridge Earthquake
Meeting

March 14-15, 1994



PROCEEDINGS

of

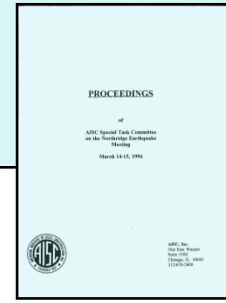
AISC Special Task Committee
on the Northridge Earthquake
Meeting

March 14-15, 1994

AMERICAN INSTITUTE OF STEEL CONSTRUCTION INC. AISC FOUNDED 1921

AISC, Inc.
One East Wacker
Suite 3100
Chicago, IL 60601
312/670-2400

PROCEEDINGS of AISC Special Task Committee on the Northridge Earthquake Meeting


PROCEEDINGS

of

AISC Special Task Committee
on the Northridge Earthquake
Meeting

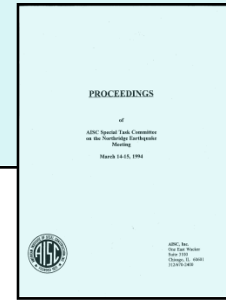
March 14-15, 1994

AISC, Inc.
One East Wacker
Suite 3100
Chicago, IL 60601
312/670-2400

DAMAGE TO DUCTILE STEEL FRAMES IN THE NORTHRIDGE EARTHQUAKE

Examination of a few damaged welds reveals that only half of the bottom flange has cracked. In addition, some welds appear to have been cracked prior to the earthquake. These cracks have been identified through the presence of rust in the weld crack.

PROCEEDINGS of AISC Special Task Committee on the Northridge Earthquake Meeting


PROCEEDINGS

of

AISC Special Task Committee
on the Northridge Earthquake
Meeting

March 14-15, 1994

AISC, Inc.
One East Wacker
Suite 3100
Chicago, IL 60601
312/670-2400

SUMMARY REPORT

Appropriate action must be taken to improve the observed performance, especially wherein a high 60-80% connection failure rate occurred in some moment frames.

Revisiting W1 Indications

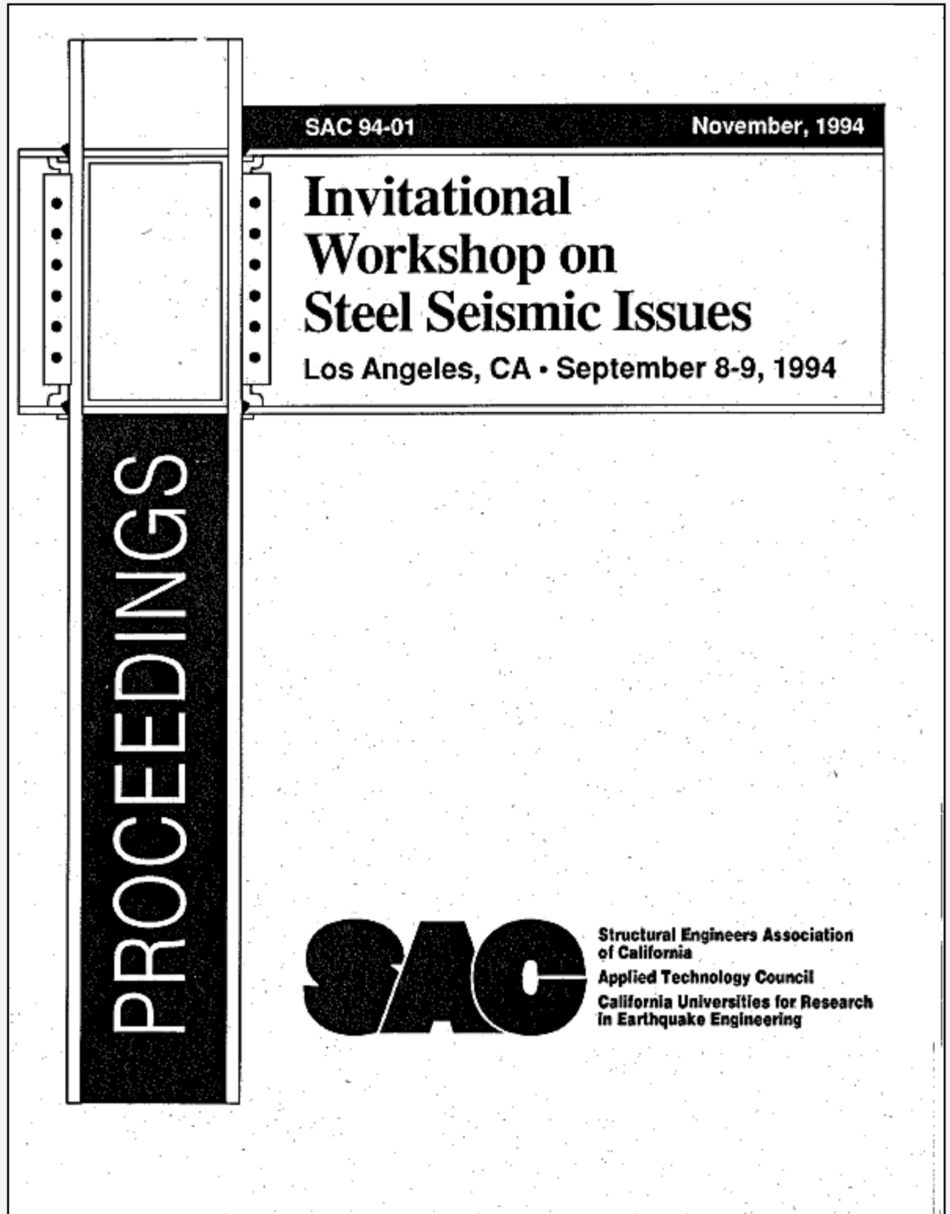
W1s: What Caused Them?
(early thoughts)

Invitational Workshop on Steel Seismic Issues

Los Angeles, CA

September 8-9, 1994

SAC 94-01


SAC 94-01 November, 1994

Invitational Workshop on Steel Seismic Issues

Los Angeles, CA • September 8-9, 1994

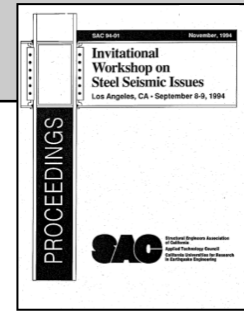
PROCEEDINGS

SAC

Structural Engineers Association of California

Applied Technology Council

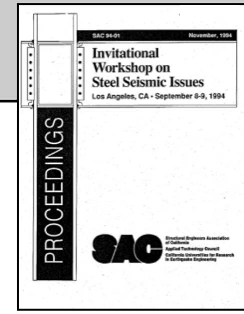
California Universities for Research in Earthquake Engineering



WELDING AND MECHANIAL PROPERTIES OF WELDS

Preheat, Cooling Rates and Postheat

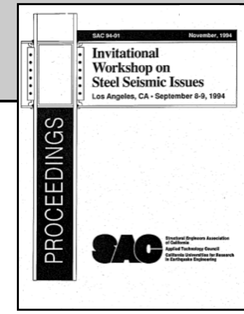
It was noted that evidence of pre-existing cracks, especially in the root of the welds, had been detected in many of the damaged SMRF connections. This could have easily been the result of inadequate preheat.



WELDING AND MECHANIAL PROPERTIES OF WELDS

Preheat, Cooling Rates and Postheat

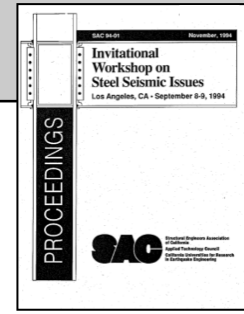
It was acknowledged that structural steel erectors do not closely adhere to good preheat practices, and that increased monitoring to ensure minimum uniform preheats are properly applied is imperative.



WELDING AND MECHANIAL PROPERTIES OF WELDS

Preheat, Cooling Rates and Postheat

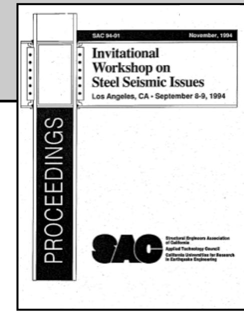
It was suggested that hardness in the Heat Affected Zone (HAZ) could have played a significant role in the failure mechanism of weld in the Northridge earthquake. High hardness could have reduced toughness, increased a hydrogen embrittlement problem, and reduce ductility of the HAZ.



WELDING AND MECHANIAL PROPERTIES OF WELDS

Preheat, Cooling Rates and Postheat

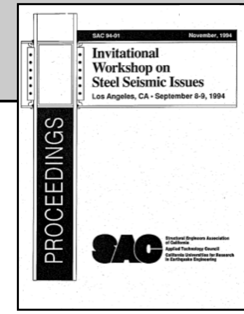
For repair work, it was agreed that an increase of 100 degrees Fahrenheit above the minimum required preheat temperature required by AWS D1.1 for a given material and thickness should be adopted as an inexpensive way to mitigate the initiation of cracking during the repair of damaged SMRF connections.



WELDING AND MECHANIAL PROPERTIES OF WELDS

Preheat, Cooling Rates and Postheat

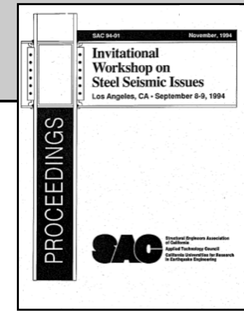
Slow cooling with insulating blankets was considered to be worthwhile to diffuse hydrogen.



WELDING AND MECHANIAL PROPERTIES OF WELDS

Preheat, Cooling Rates and Postheat

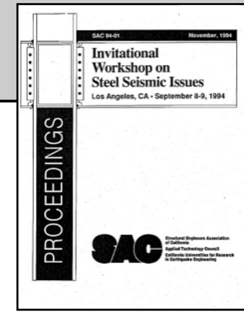
The use of Dehydrogenation Heat Treat (DHT) for thicker joints was considered to be worthwhile, and a recommendation was made to use 450 degrees Fahrenheit for one hour per inch of thickness, when the weld joint exceeds 1”.



WELDING AND MECHANIAL PROPERTIES OF WELDS

Welding Electrodes

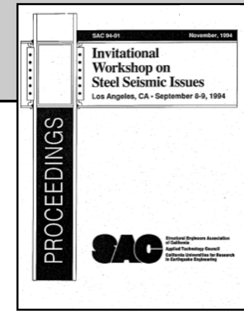
It was agreed that the SMAW E7018 low hydrogen electrode was the most reliable and exhibited the best properties under field conditions.



SURVEY OF DAMAGE—Preliminary Report

Background

As of September 1994, eight months after the earthquake, the estimate has grown to over 100 damaged MRF buildings….



THE SAC STEEL PROGRAM

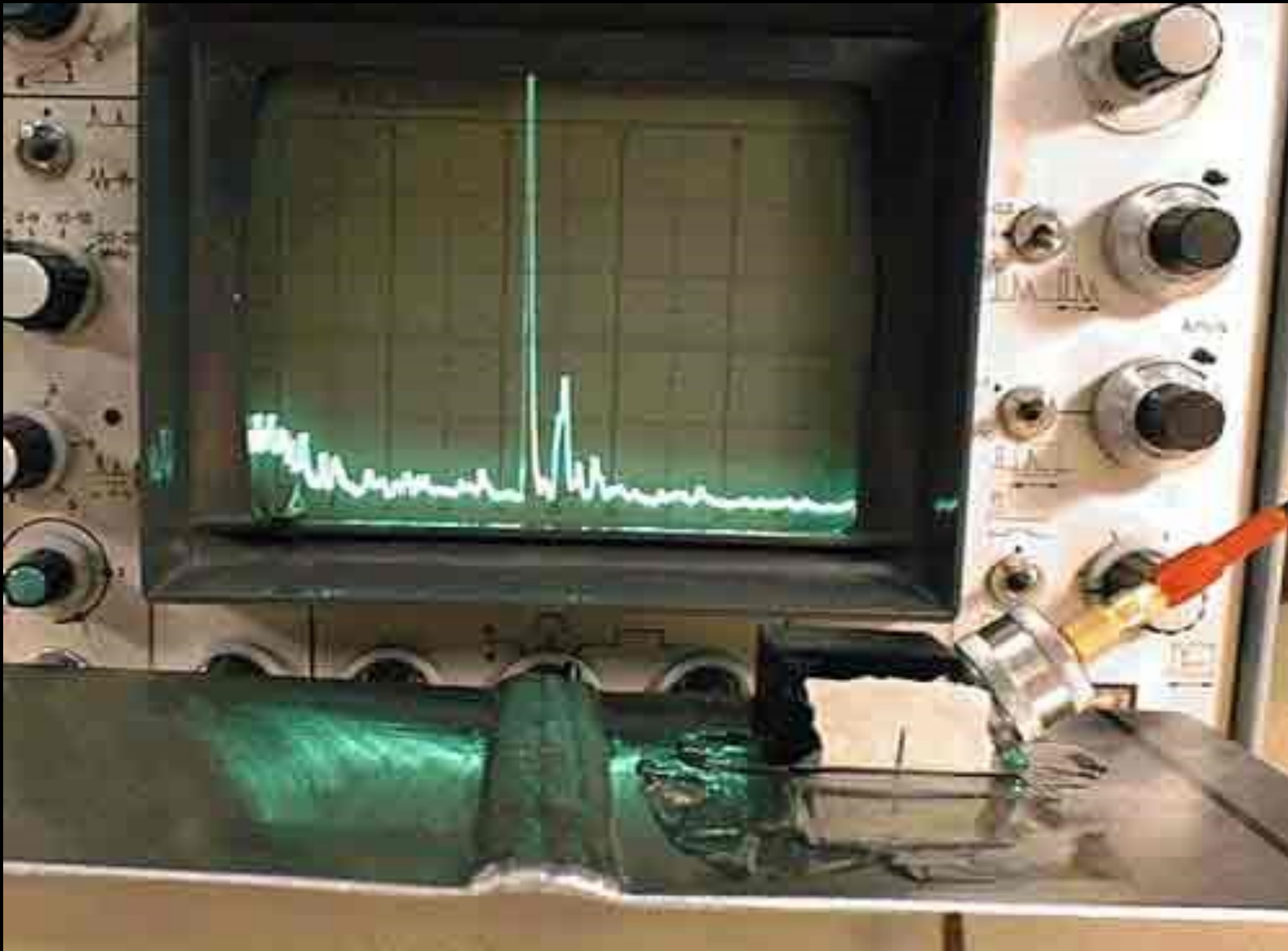
The Problem

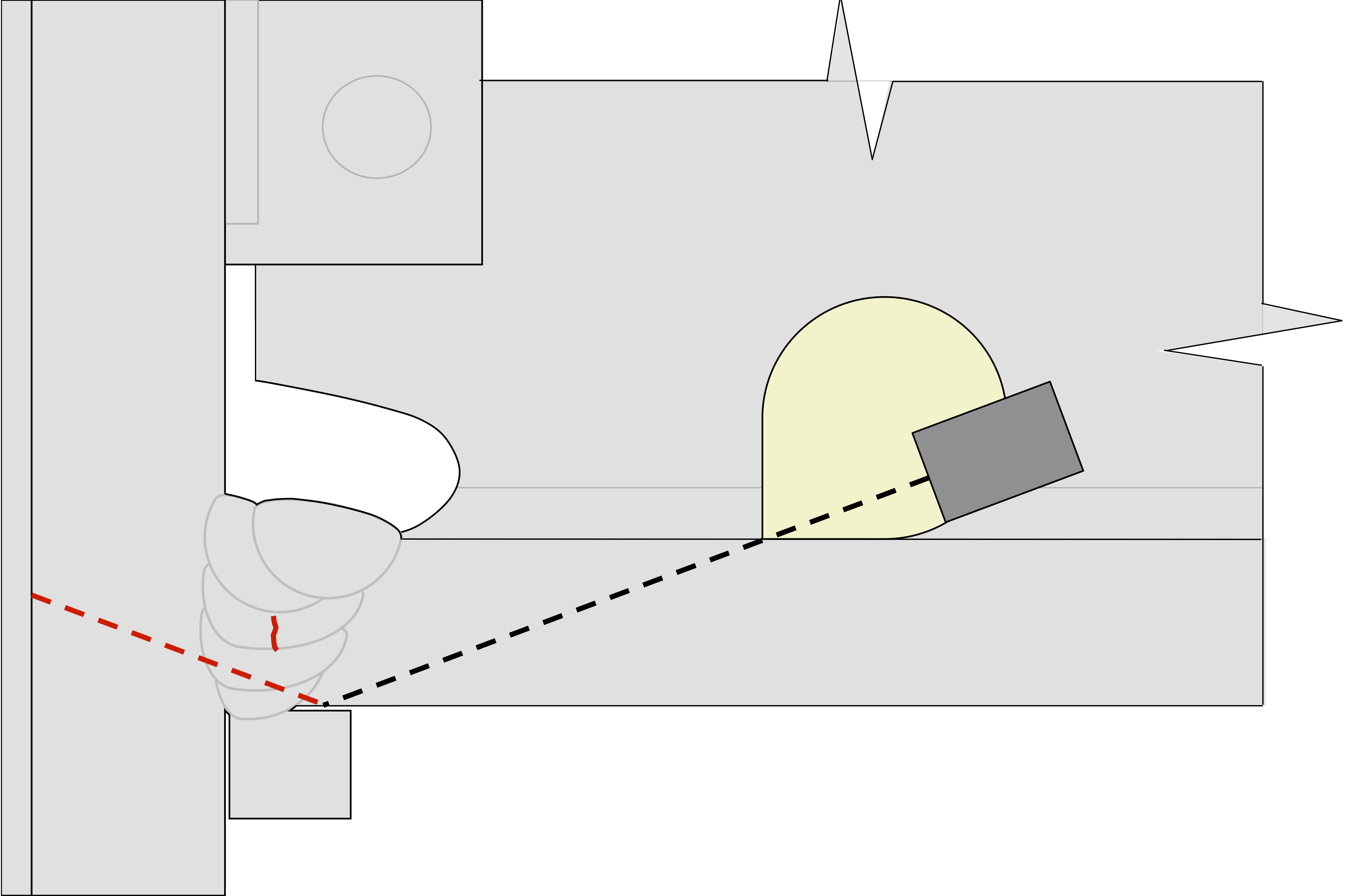
Among the many issues discussed…there are **six main problems** most often put forth:

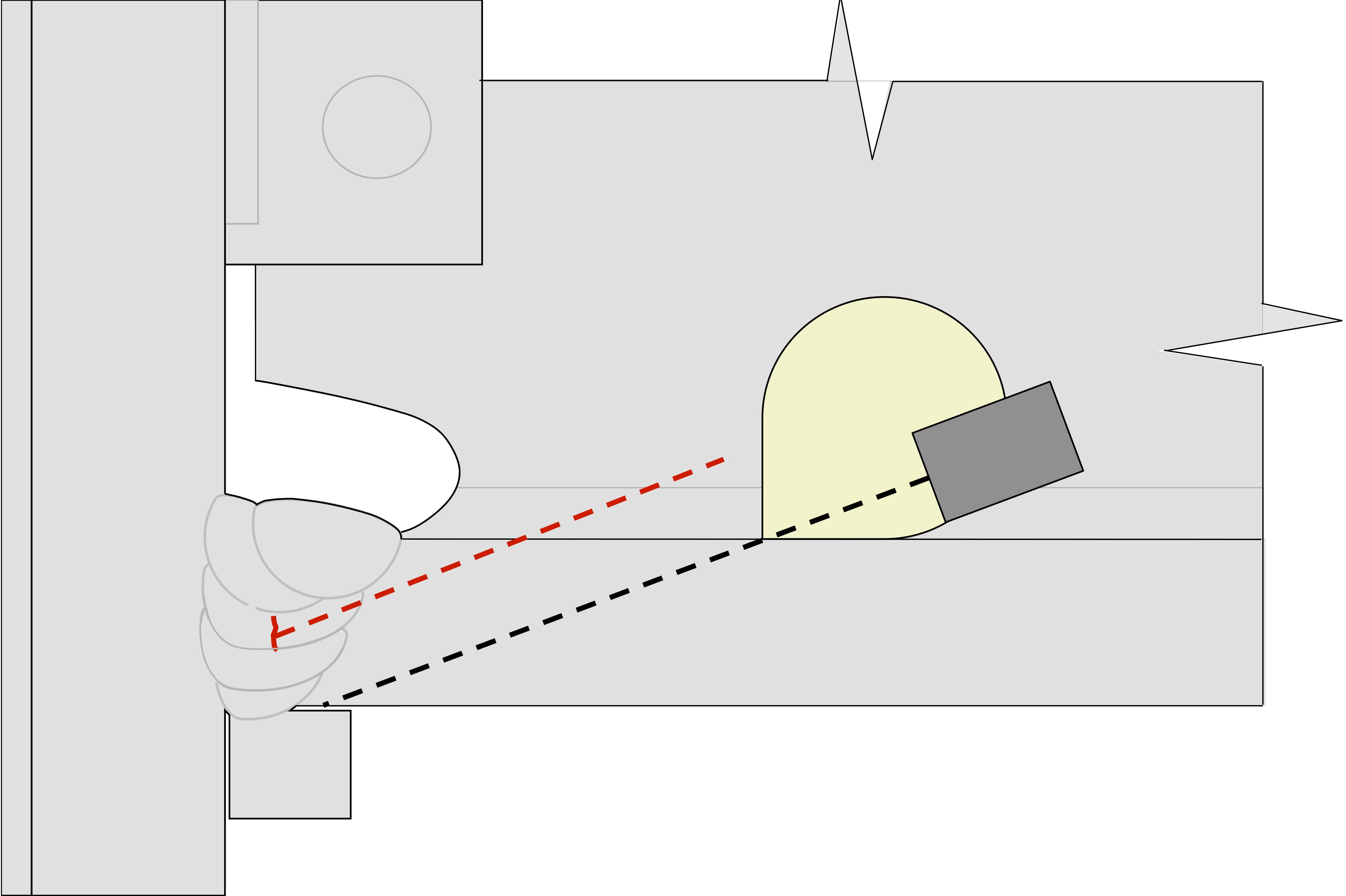
- Inadequately executed welds
- Pre-existent cracks in the weldments
- Residual stresses in the joint resulting from the welding and construction process
- Use of inappropriate weld material, preparation, process and heat treatment
- Through-thickness tension failure of the column flanges
- Fundamental problems with the joint configuration.

Revisiting W1 Indications

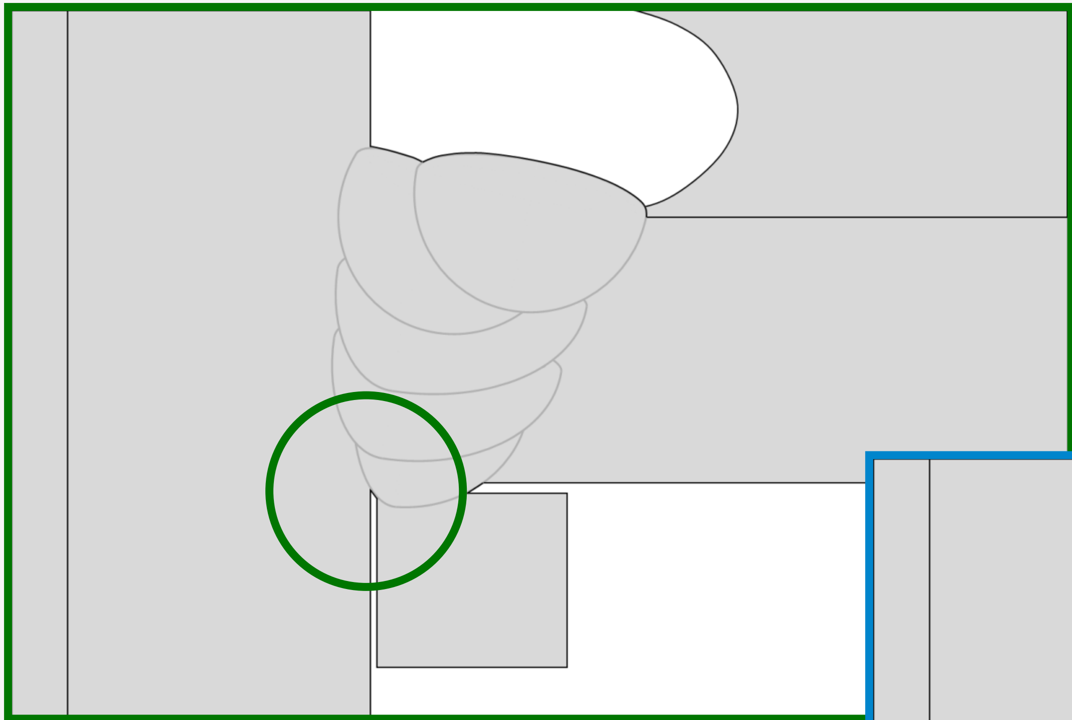
W1s: How Were They Detected?



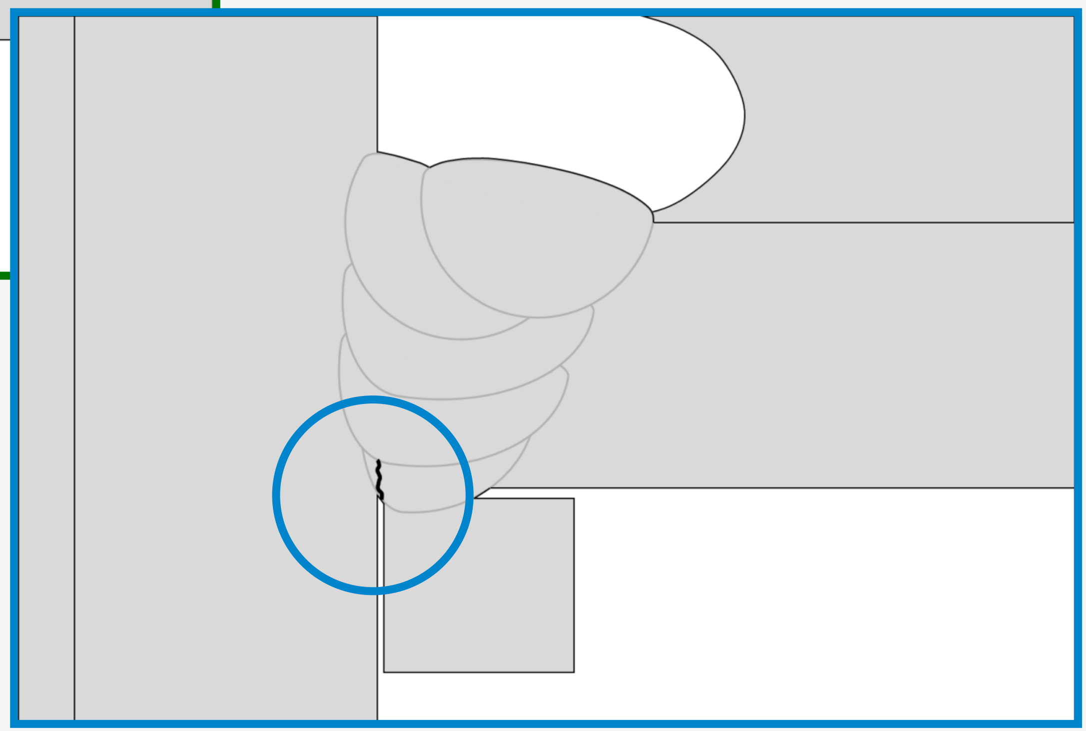




Difficult to distinguish with UT between an acceptable weld with backing left in place and a weld with a root crack

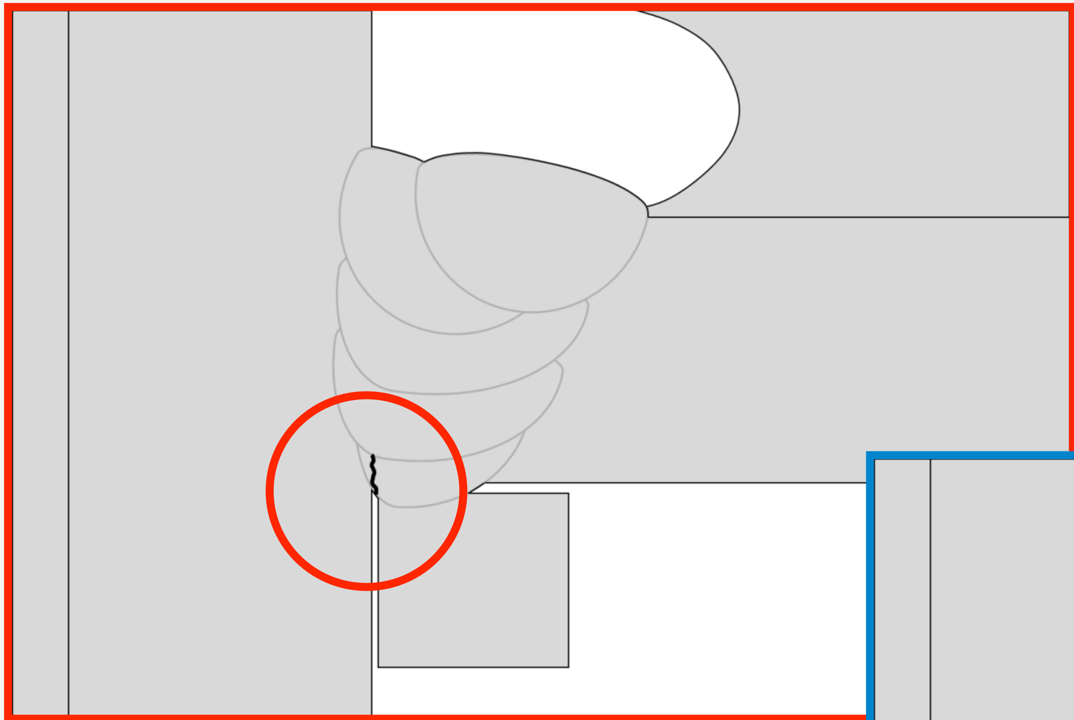


Acceptable

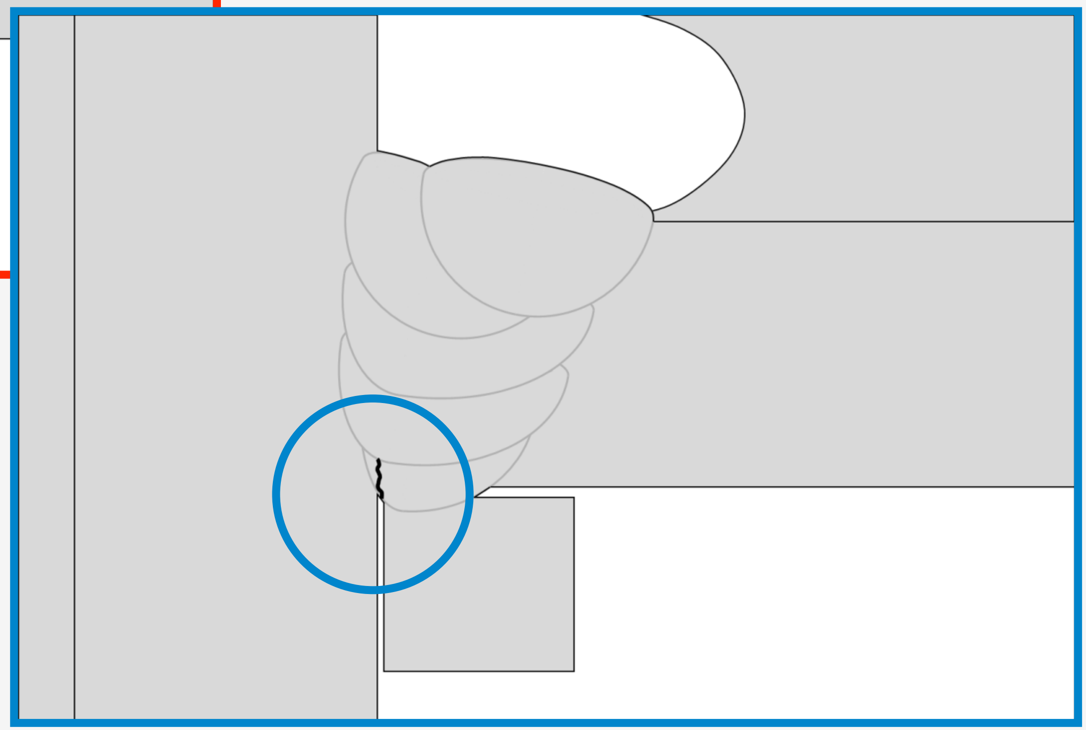


Crack in root

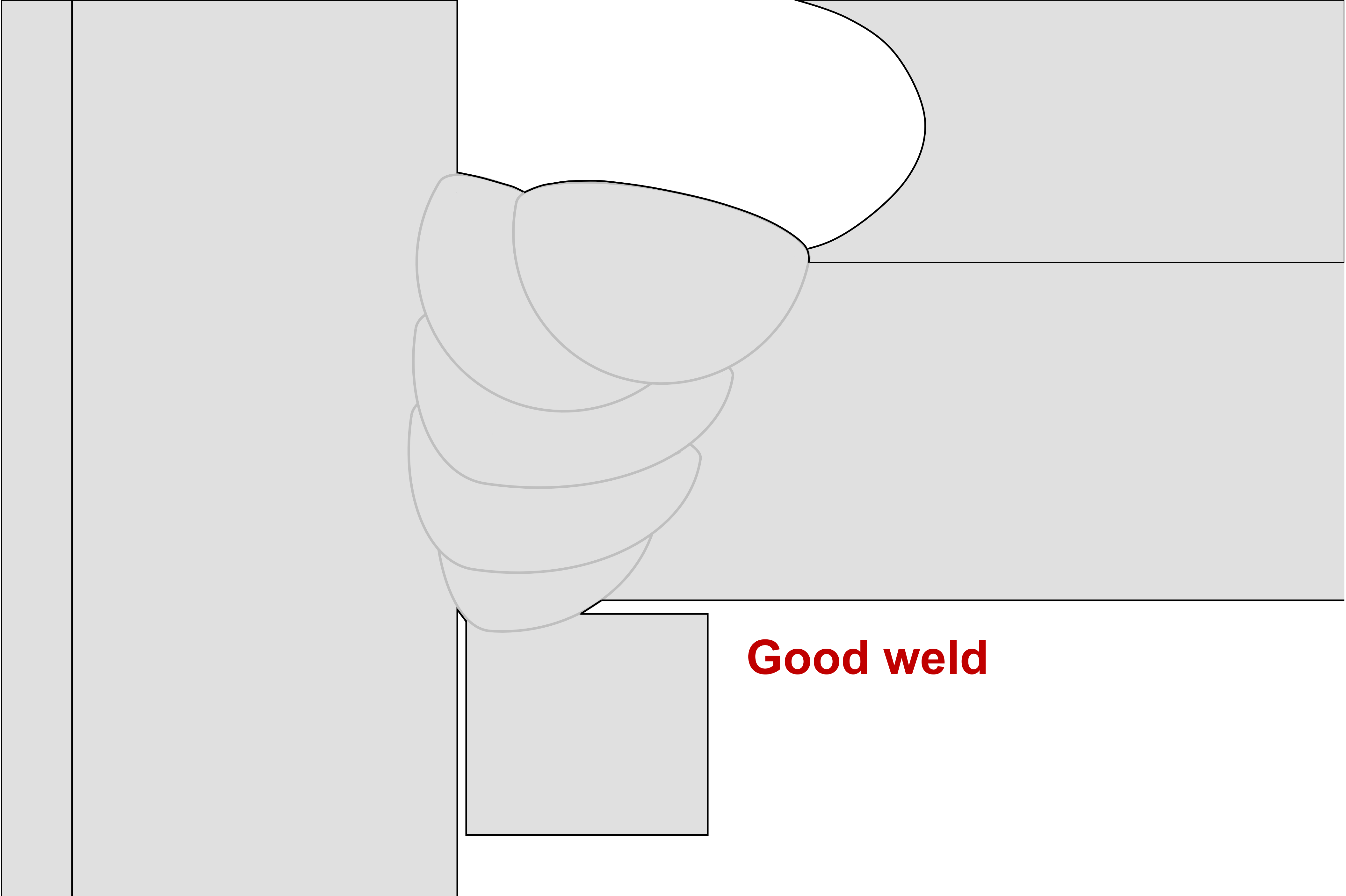
Impossible to distinguish with UT whether a crack is pre-existing, or due to earthquake

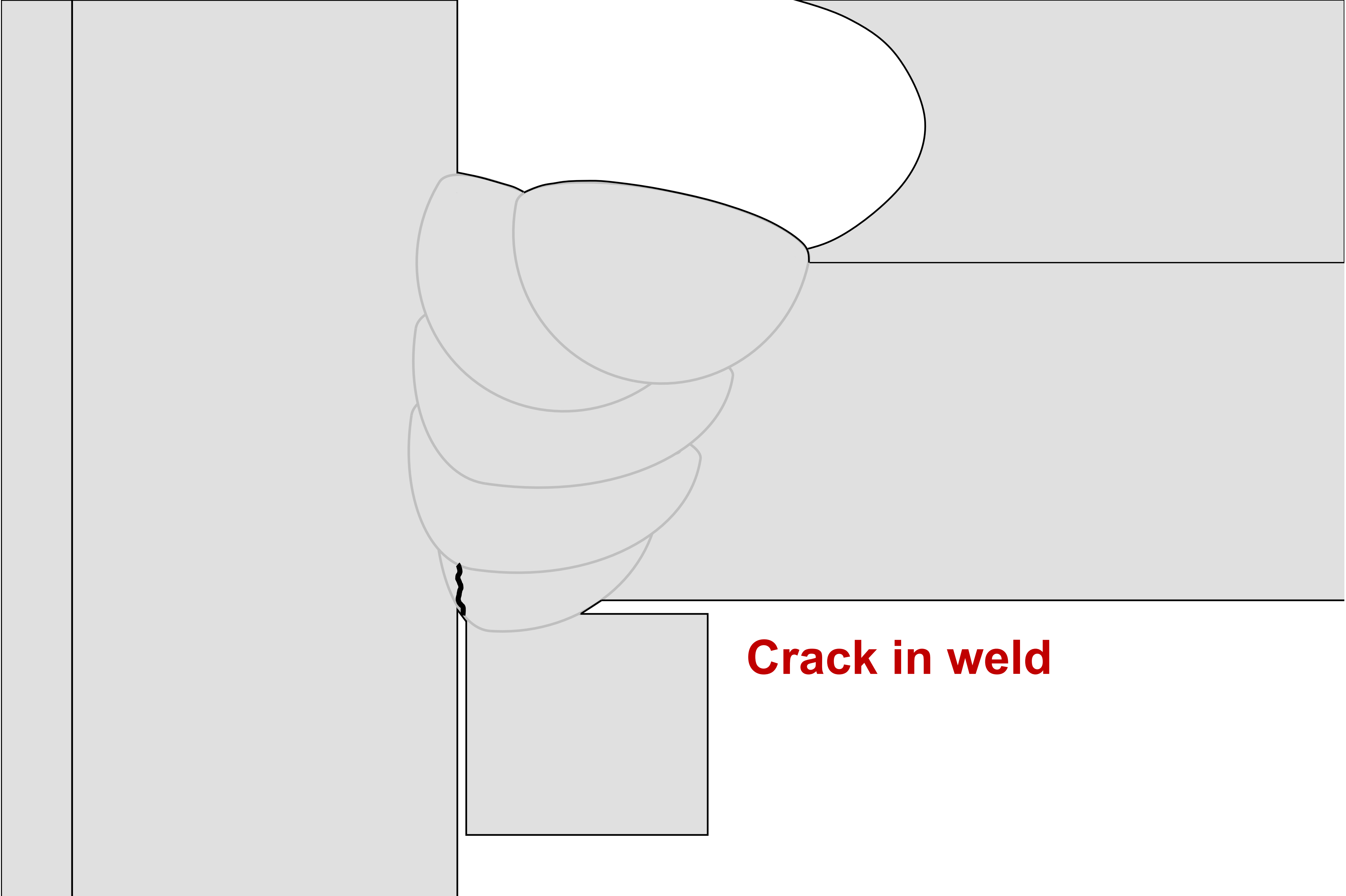


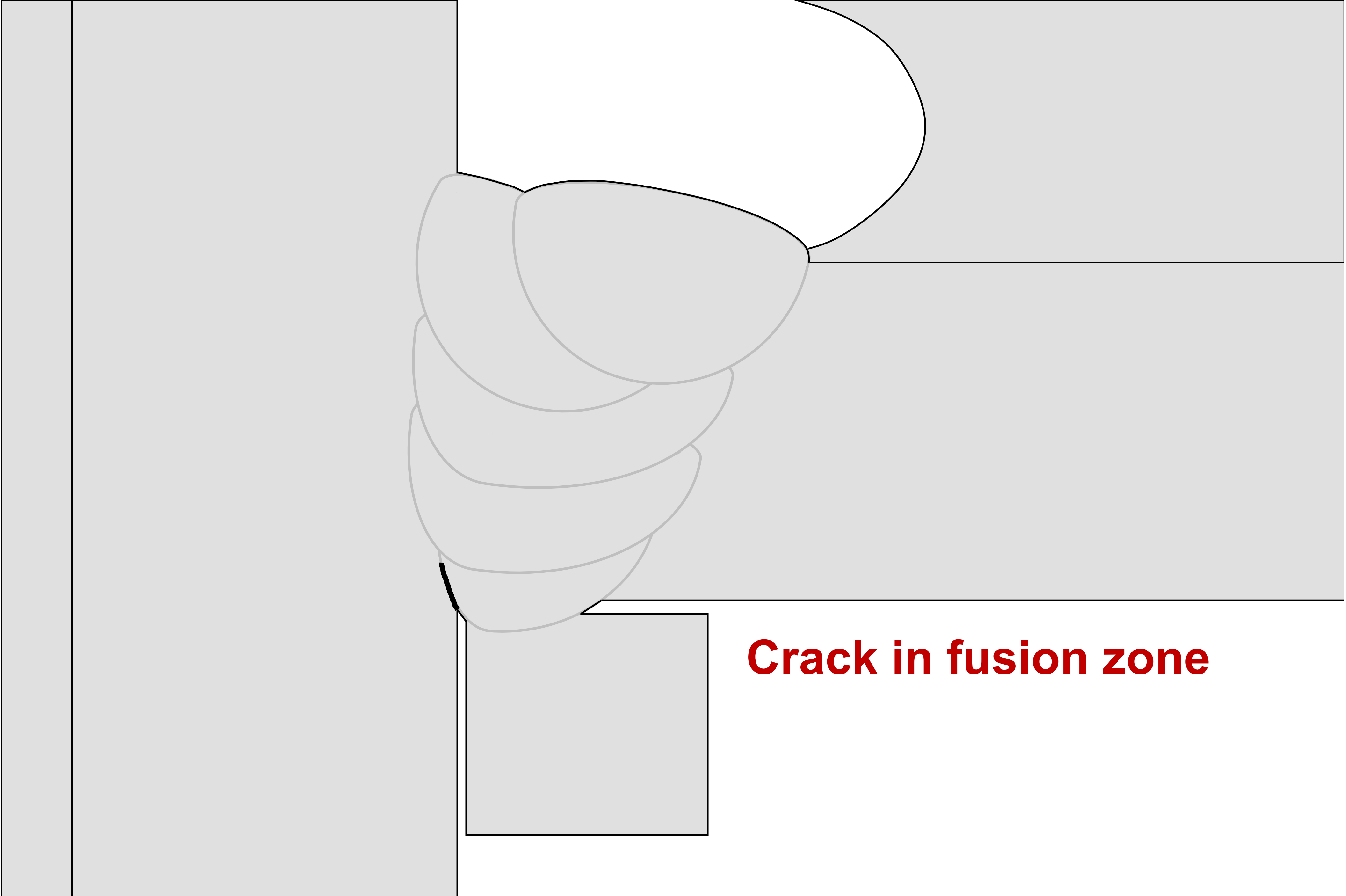
Pre-existing crack

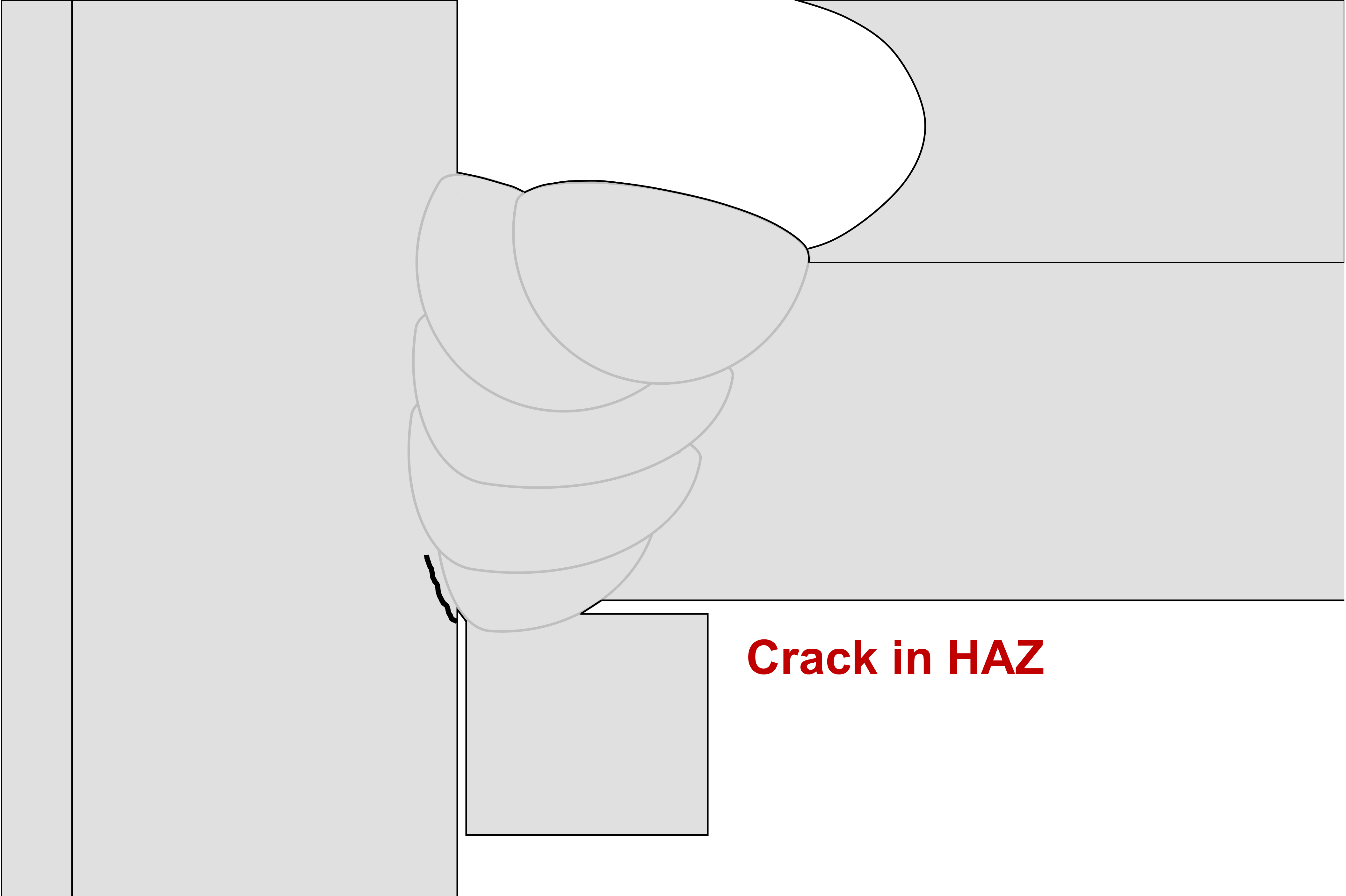


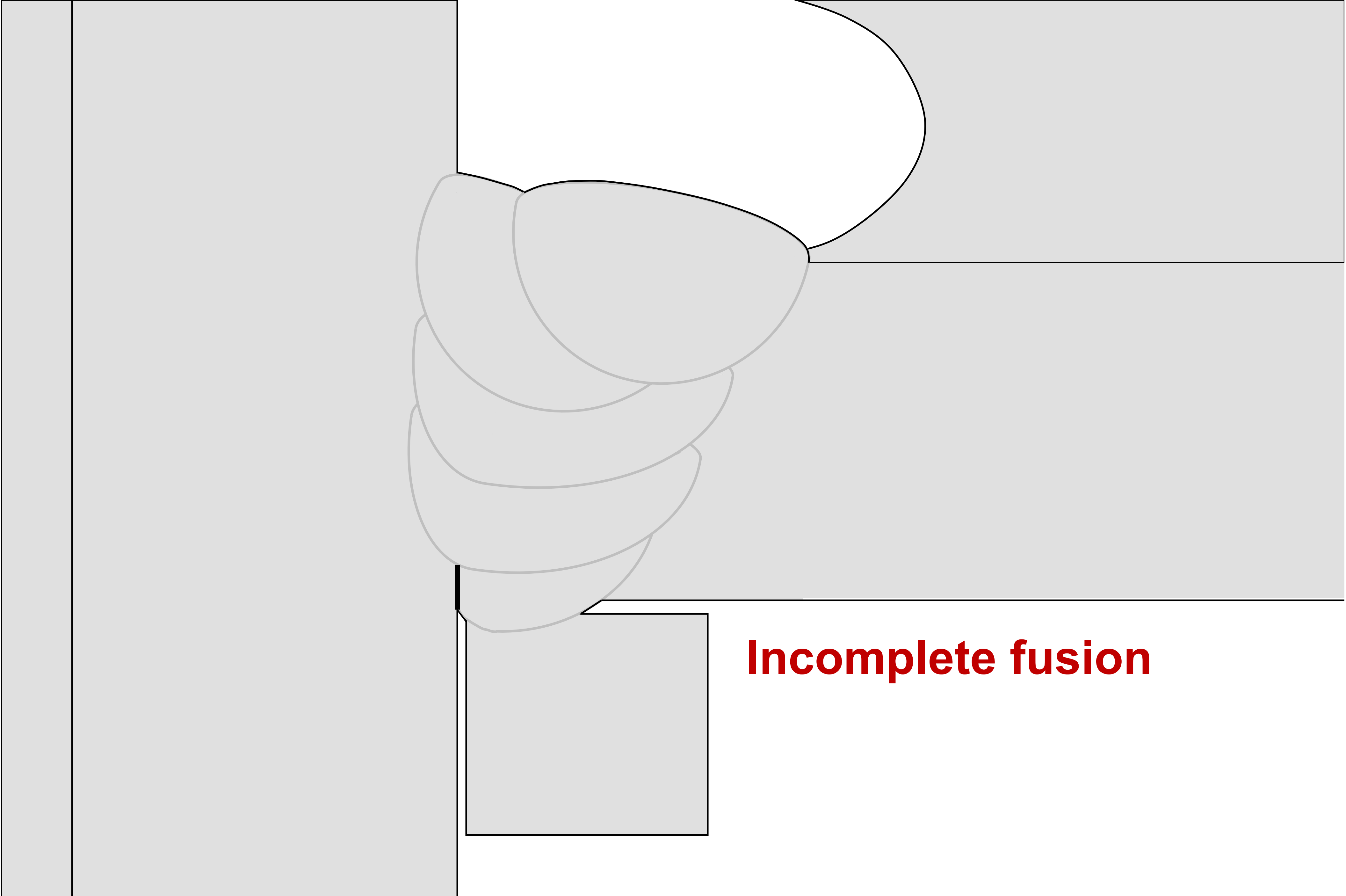
Earthquake crack

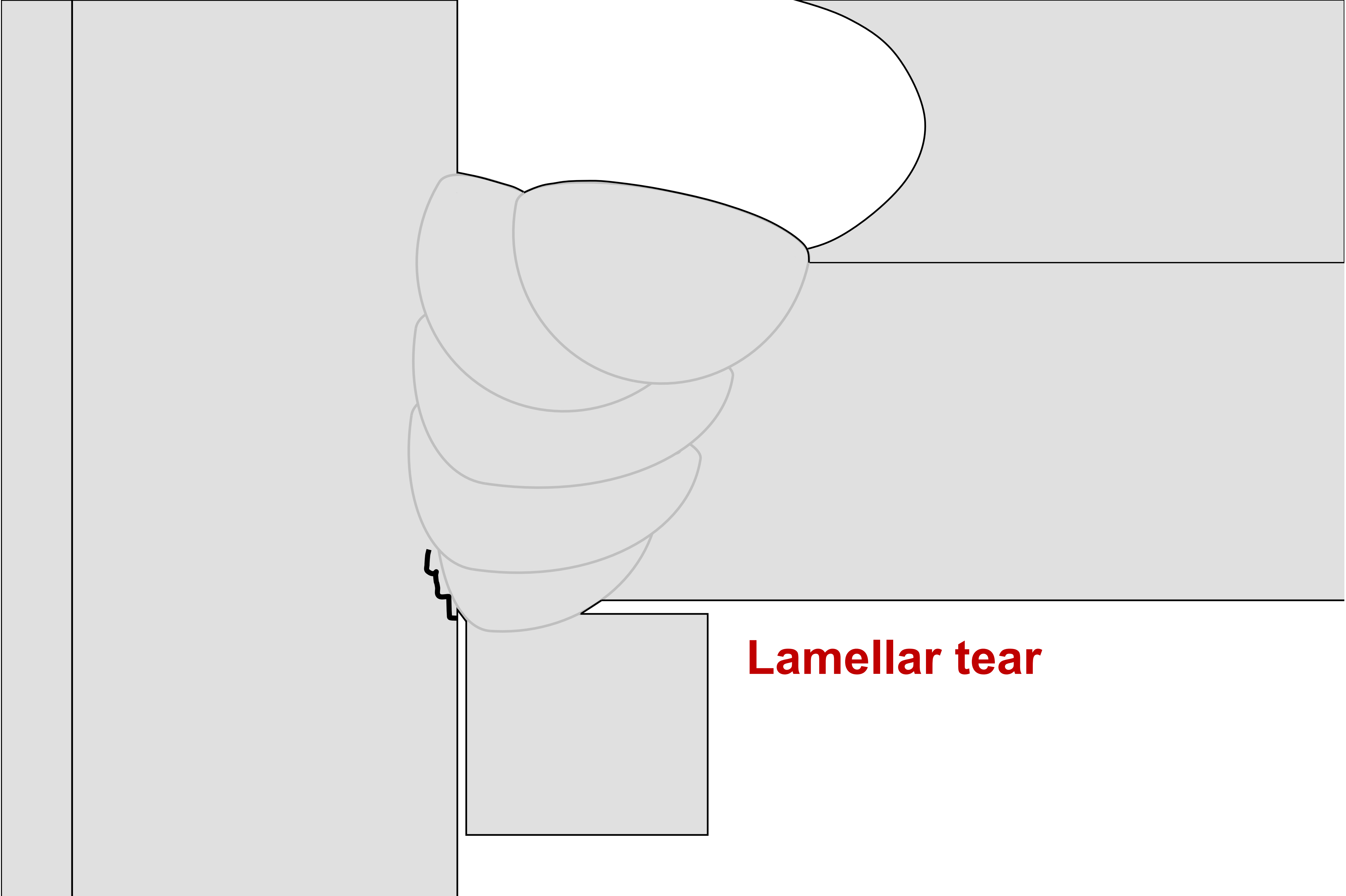

Good weld

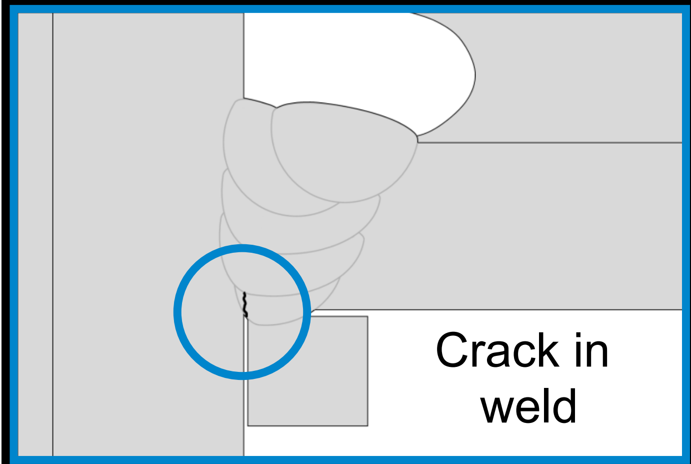

Crack in weld

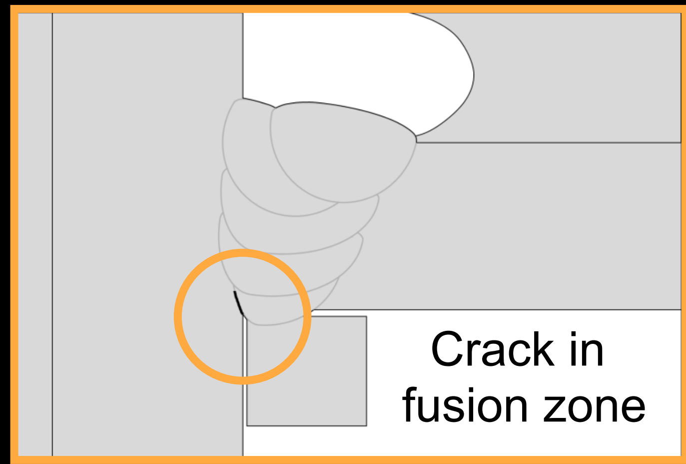

Crack in fusion zone

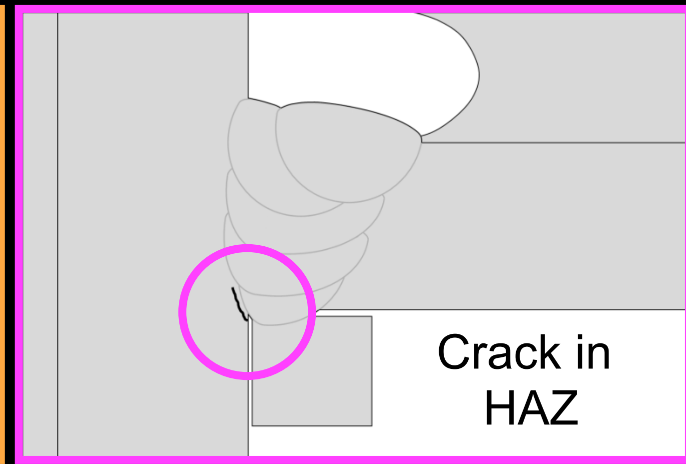

Crack in HAZ


Incomplete fusion

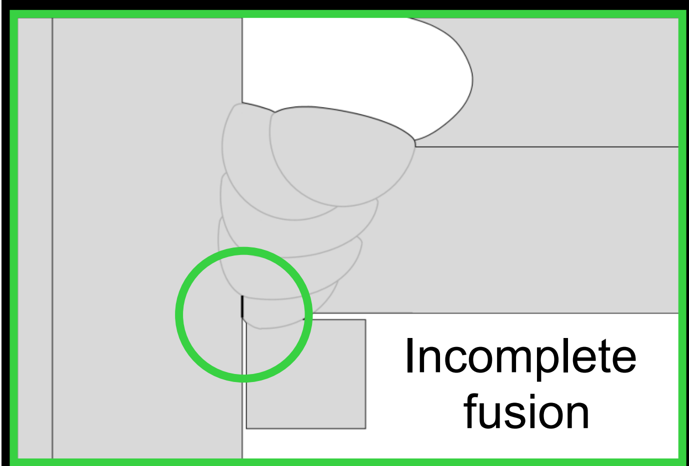

Lamellar tear

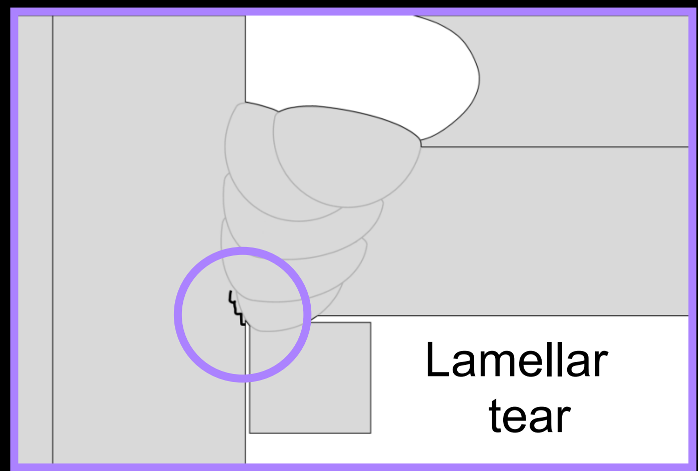


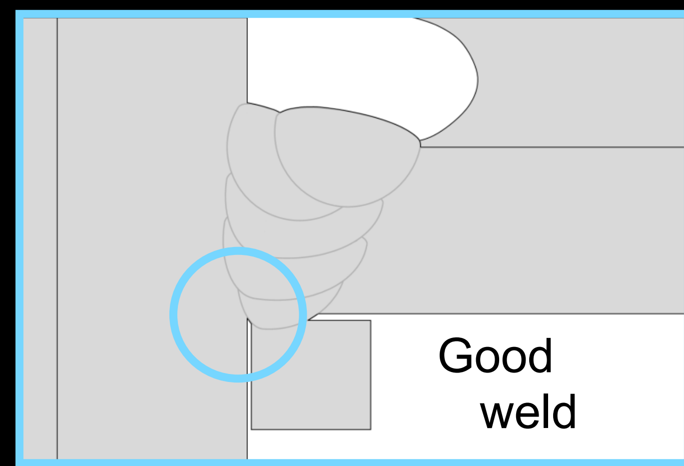


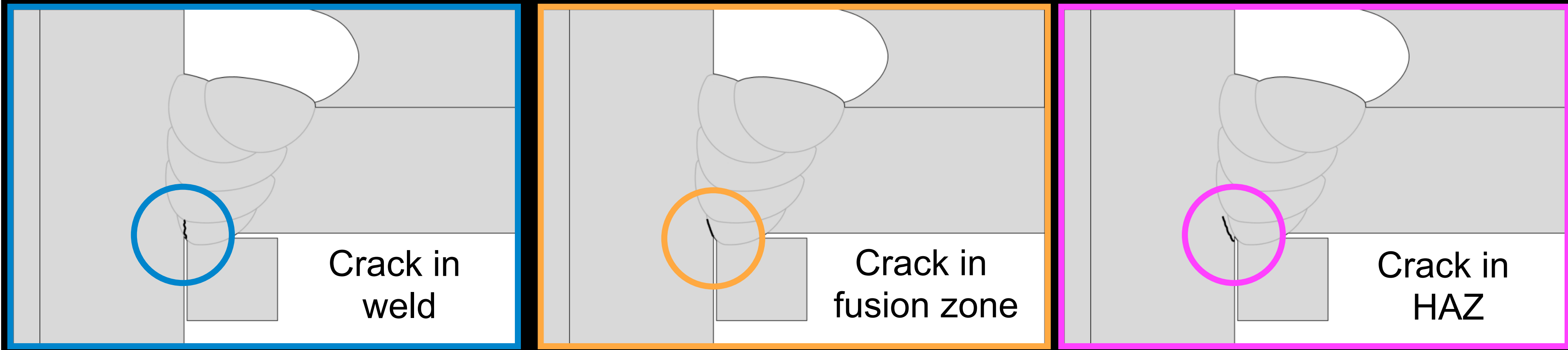


All could be identified as W1 in inspection reports

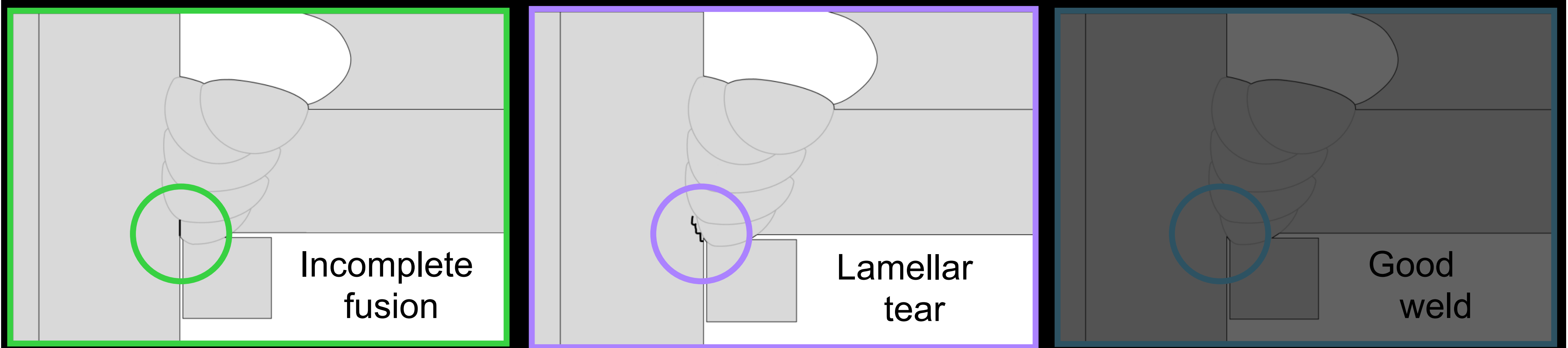








The solutions to these problems are very different



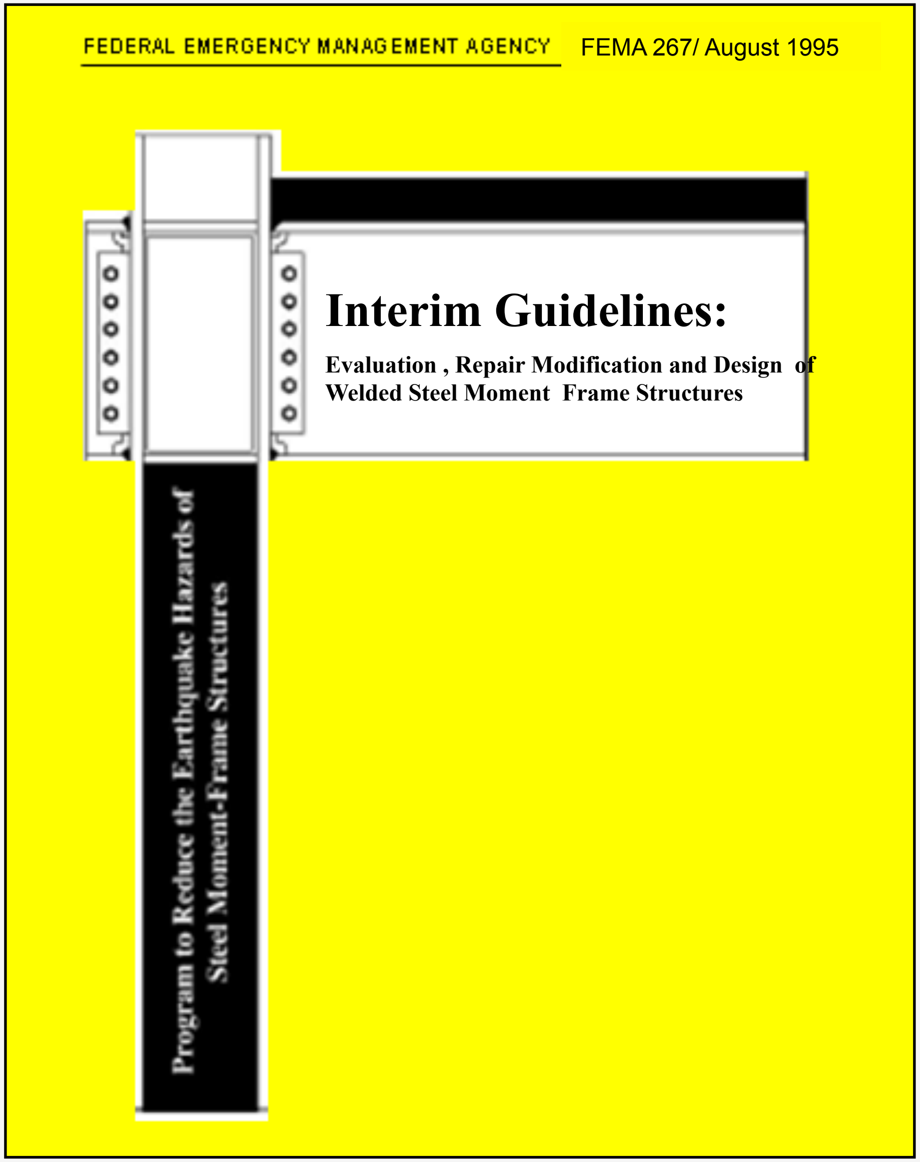
Revisiting W1 Indications

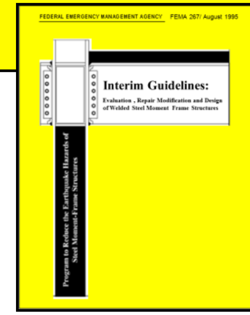
W1s: Did They Cause the Northridge Fractures?

FEMA 267

Interim Guidelines:
Evaluation , Repair,
Modification and Design of
Welded Steel Moment Frame
Structures

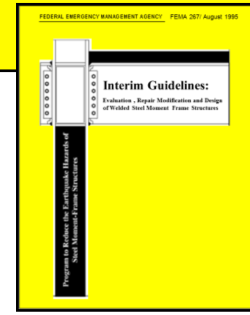
August 1995





INTRODUCTION
Background

Investigators initially identified a number of factors which may have contributed to the initiation of fractures at the weld root including: notch effects created by the backing bar…substandard welding…and potentially, pre-earthquake fractures resulting from initial shrinkage of the highly restrained weld during cool-down.

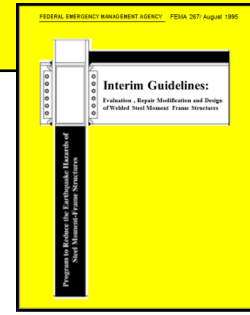


INTRODUCTION

Background

Such problems could be minimized in future construction, with the application of appropriate welding procedures and more careful exercise of quality control during the construction process. However, it is now known that these were not the only cause of the fractures which occurred.

FEMA 267: Interim Guidelines

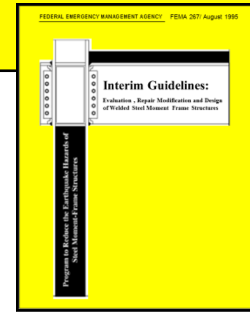


DAMAGE CLASSIFICATION

Weld Damage, Defects and Discontinuities

Type W1 is the single most commonly reported non-conforming condition, representing more than 80 per cent of the total damage reported.

FEMA 267: Interim Guidelines



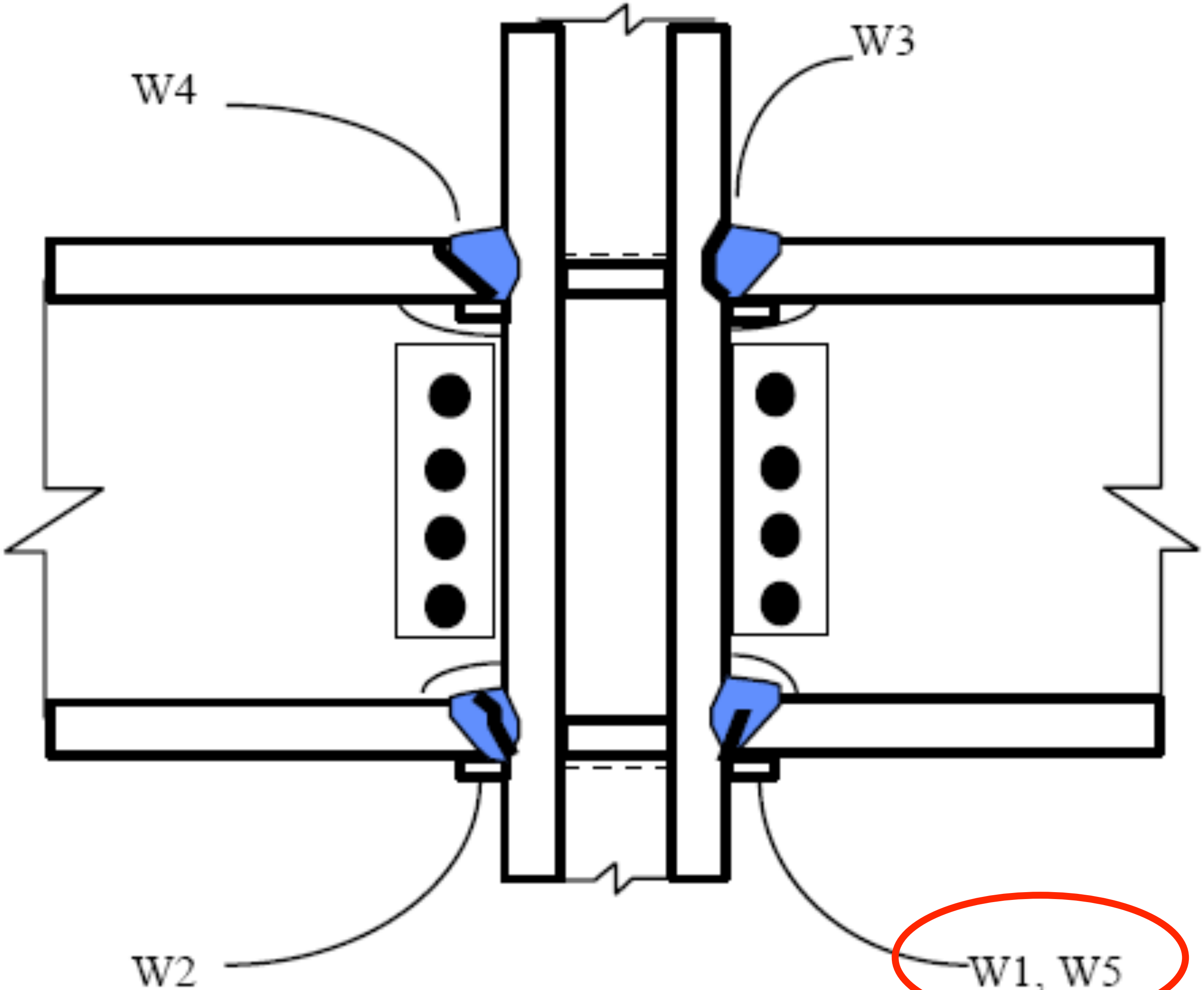


Figure 3-4 Types of Weld Damage

FEMA 267: Interim Guidelines

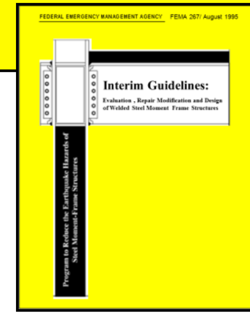
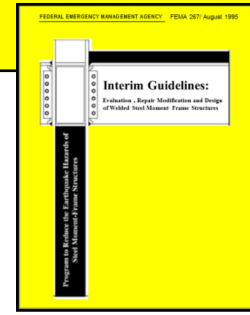


Table 3-4 Types of Weld Damage, Defects and Discontinuities

Type	Description
W1	Weld root indications
W1a	Incipient indications – depth , 3/16” or $t_f/4$; width < $b_f/4$
W1b	Root indications larger than that for W1a
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface
W5	UT detectable indication – non-rejectable

FEMA 267: Interim Guidelines

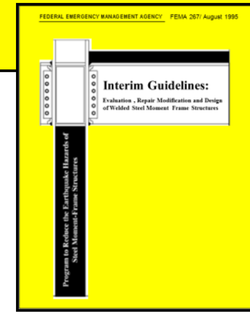


DAMAGE CLASSIFICATION

Weld Damage, Defects and Discontinuities

Some engineers believe that type W1a indications are not earthquake damage at all, but rather, previously undetected defects from the original construction process. A W1b indication is one that exceeds these limits but is not clearly characterized by one of the other types. It is more likely that W1b indications are the result of the earthquake than the construction process.

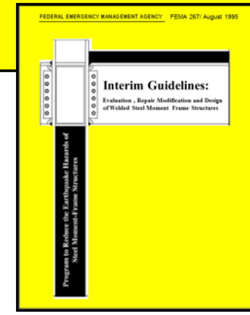
FEMA 267: Interim Guidelines



INTRODUCTION

Background

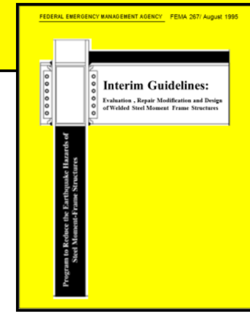
Some engineers, with knowledge of fracture mechanics, have suggested if materials with adequate toughness are used, and welding procedures are carefully specified and followed, adequate reliability can be obtained from the traditional connection details.



INTRODUCTION

Background

Others believe that the conditions of high tri-axial restraint present in the beam flange to column flange joint (Blodgett—1995) would further prevent ductile behavior of these joints regardless of the procedure used to make the welds. Further they point to the important influence of the relative yield and tensile strength of beam and column materials, and other variables that can affect connection behavior.



INTRODUCTION

Background

To date, there has not been sufficient research conducted to resolve this issue.

Revisiting W1 Indications

W1s: What Did The SAC Investigations Learn About Them?

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

CONCLUSIONS

CONCLUSIONS

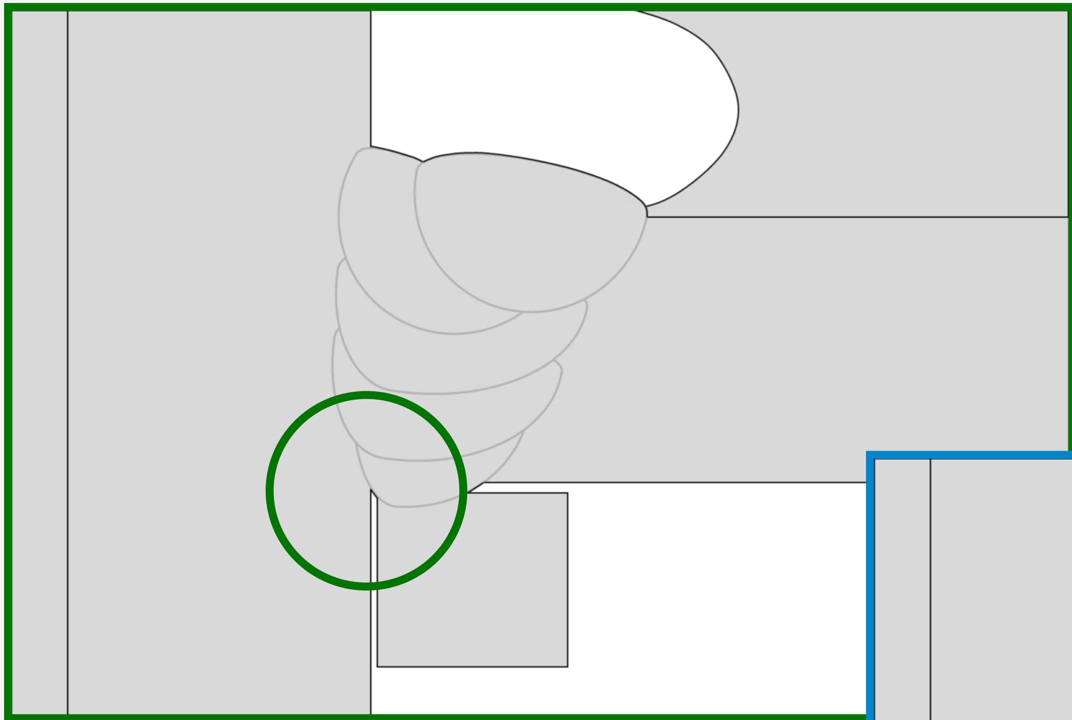
1. W1's are a result of poor welding and inspection practices during construction, not a result of earthquake ground motions.

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

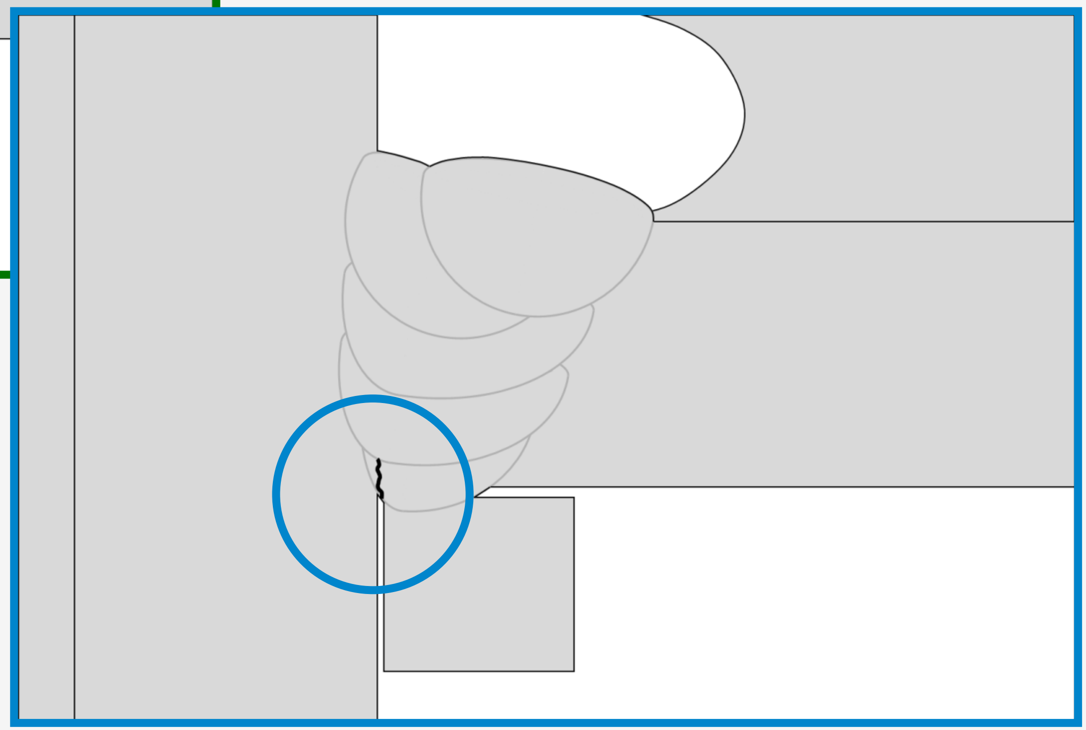
CONCLUSIONS

2. Ultrasonic inspection as normally employed by testing laboratory personnel is not a reliable inspection technique for identifying defects in the roots of welded full penetration "T" joints with backing.

Difficult to distinguish with UT between an acceptable weld with backing left in place and a weld with a root crack

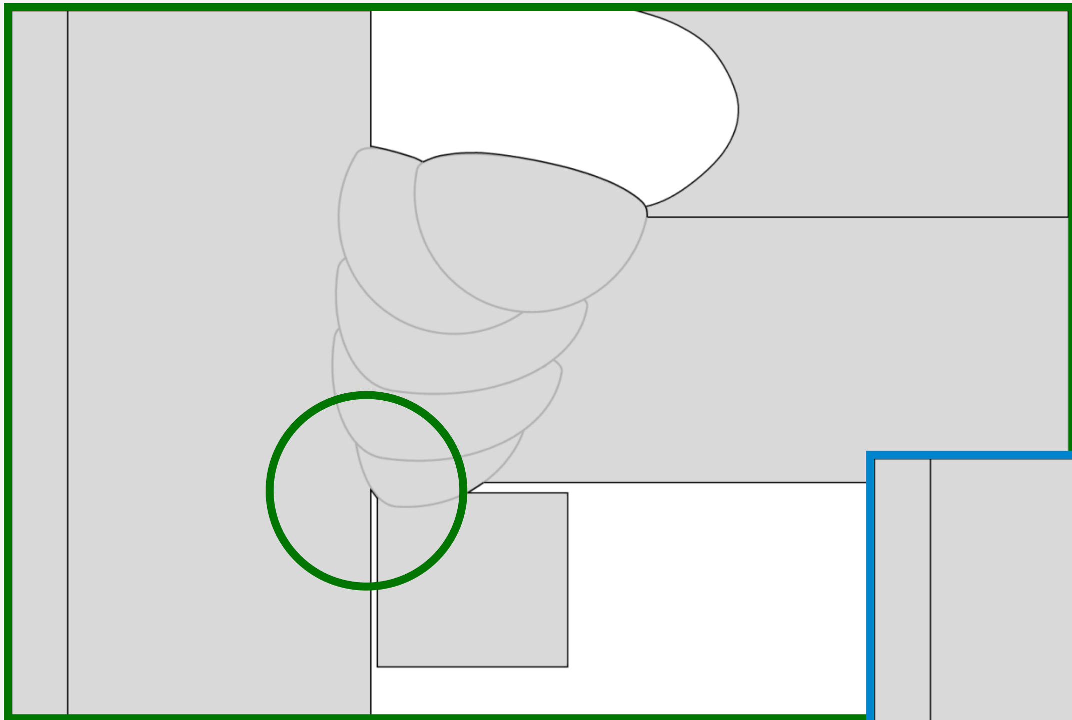


Acceptable

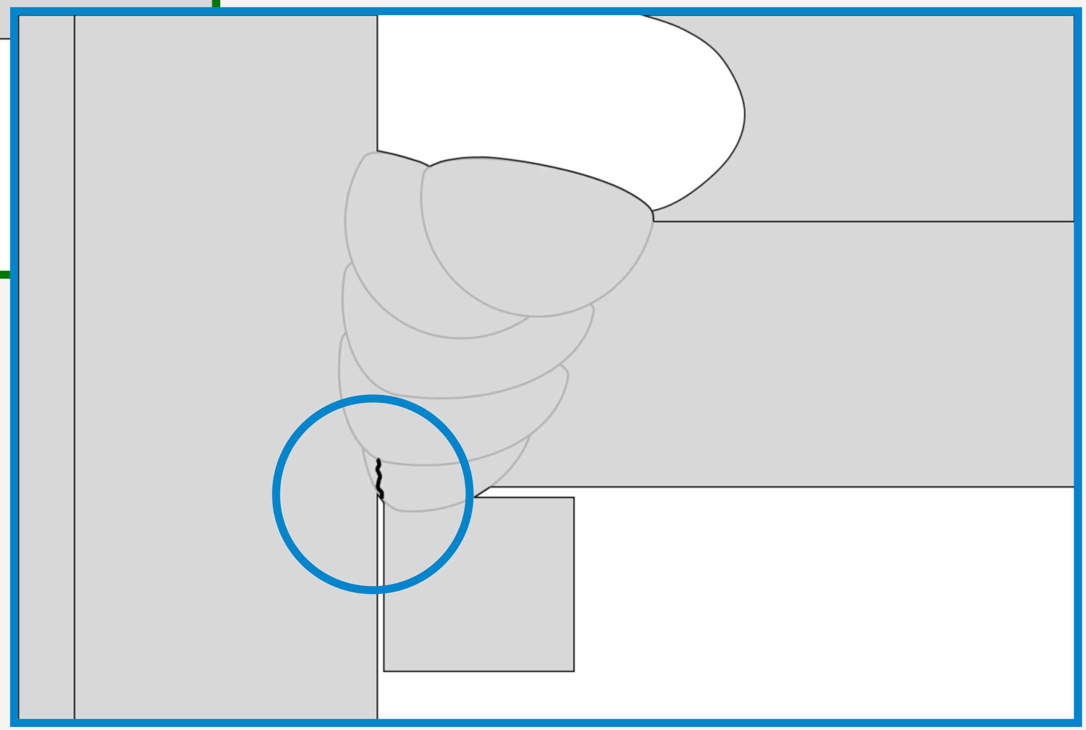


Crack in root

“Ultrasonic Inspection…is **not a reliable inspection technique** for identifying defects in the roots of welded full penetration “T” joints with backing.”



Acceptable



Crack in root

Steel Project

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

CONCLUSIONS

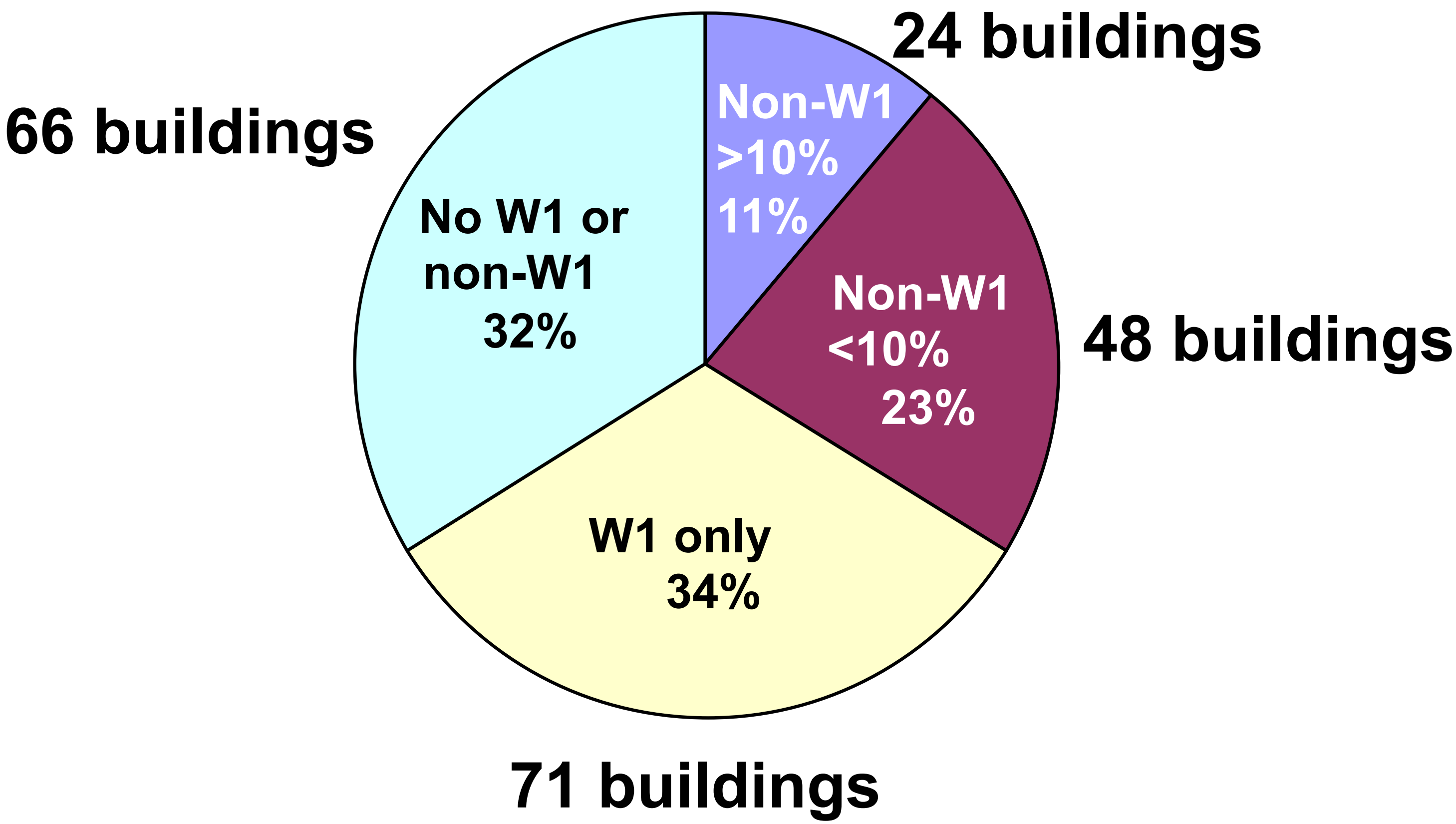
3. The extent of earthquake damage to WSMF buildings is substantially less than has previously been reported.

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

However, assorted anecdotal evidence suggested that W1's might not be earthquake related at all. For example, a number of samples of W1b's trepanned from welded connections and examined in the laboratory were determined to contain only areas of nonfusion and slag, without any crack extension or other potentially earthquake-related conditions.

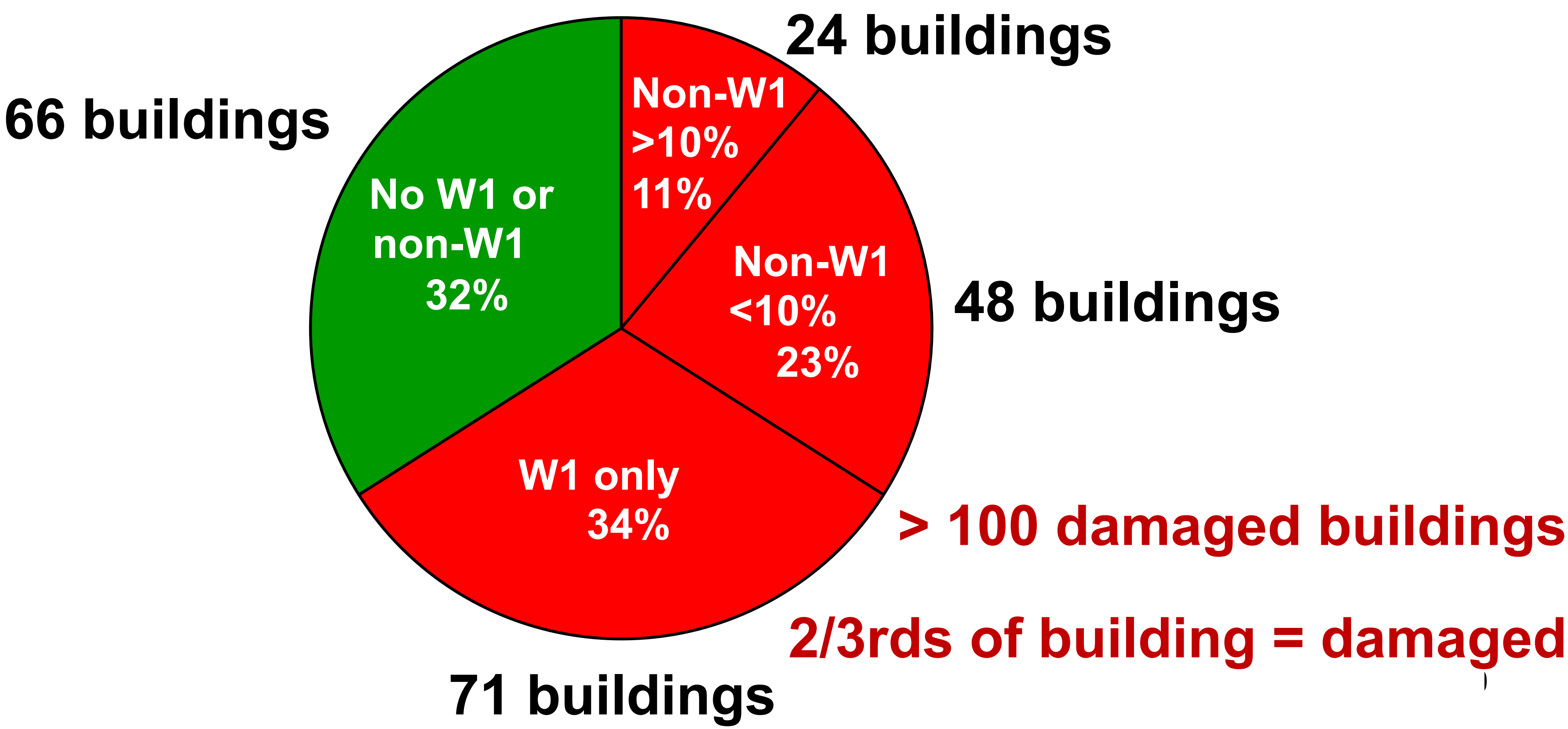
Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

Figure 6 Distribution of W1's and non-W1's in City of Los Angeles Inventory



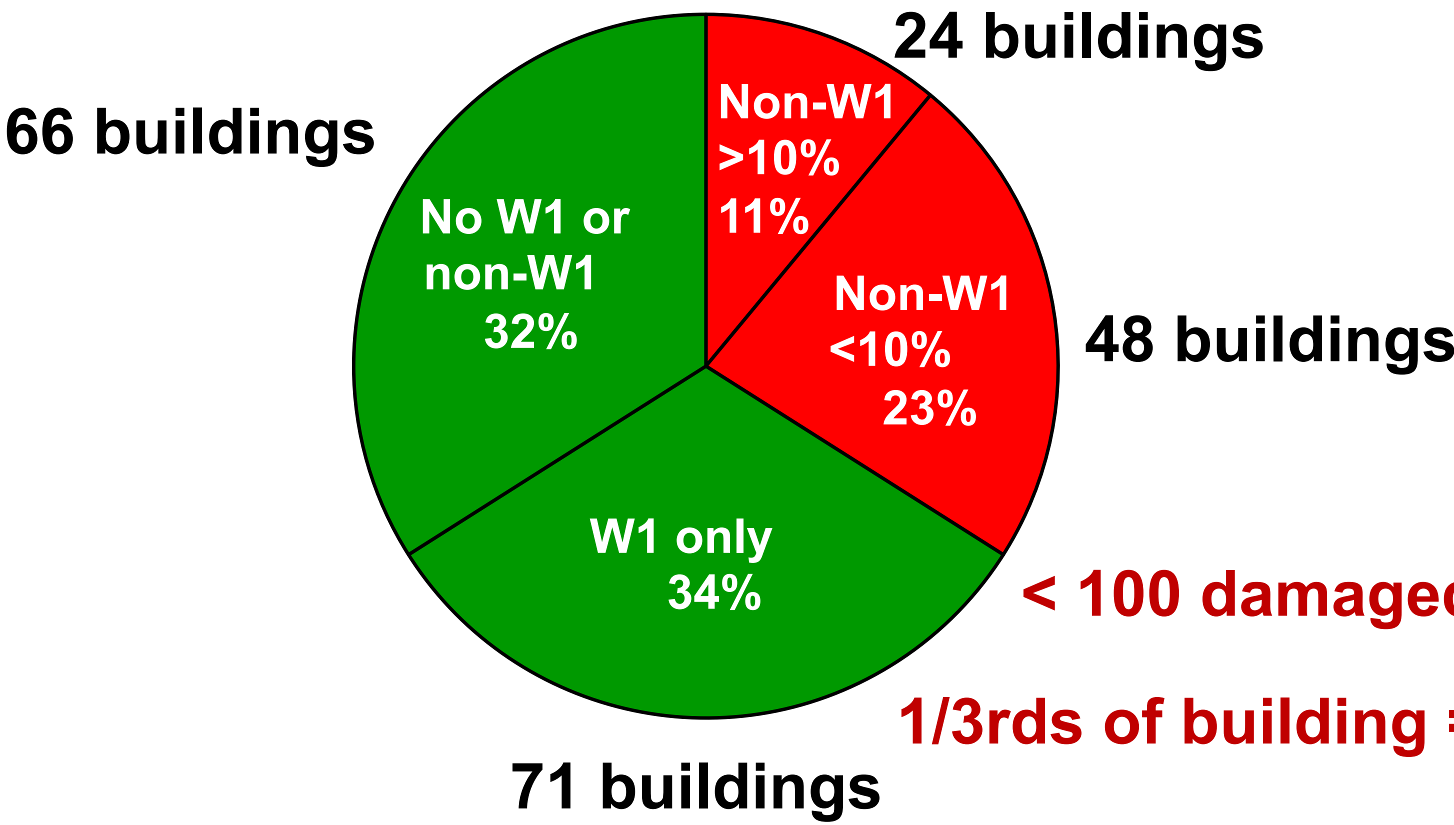
Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

Figure 6 Distribution of W1's and non-W1's in City of Los Angeles Inventory



Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

Figure 6 Distribution of W1's and non-W1's in City of Los Angeles Inventory

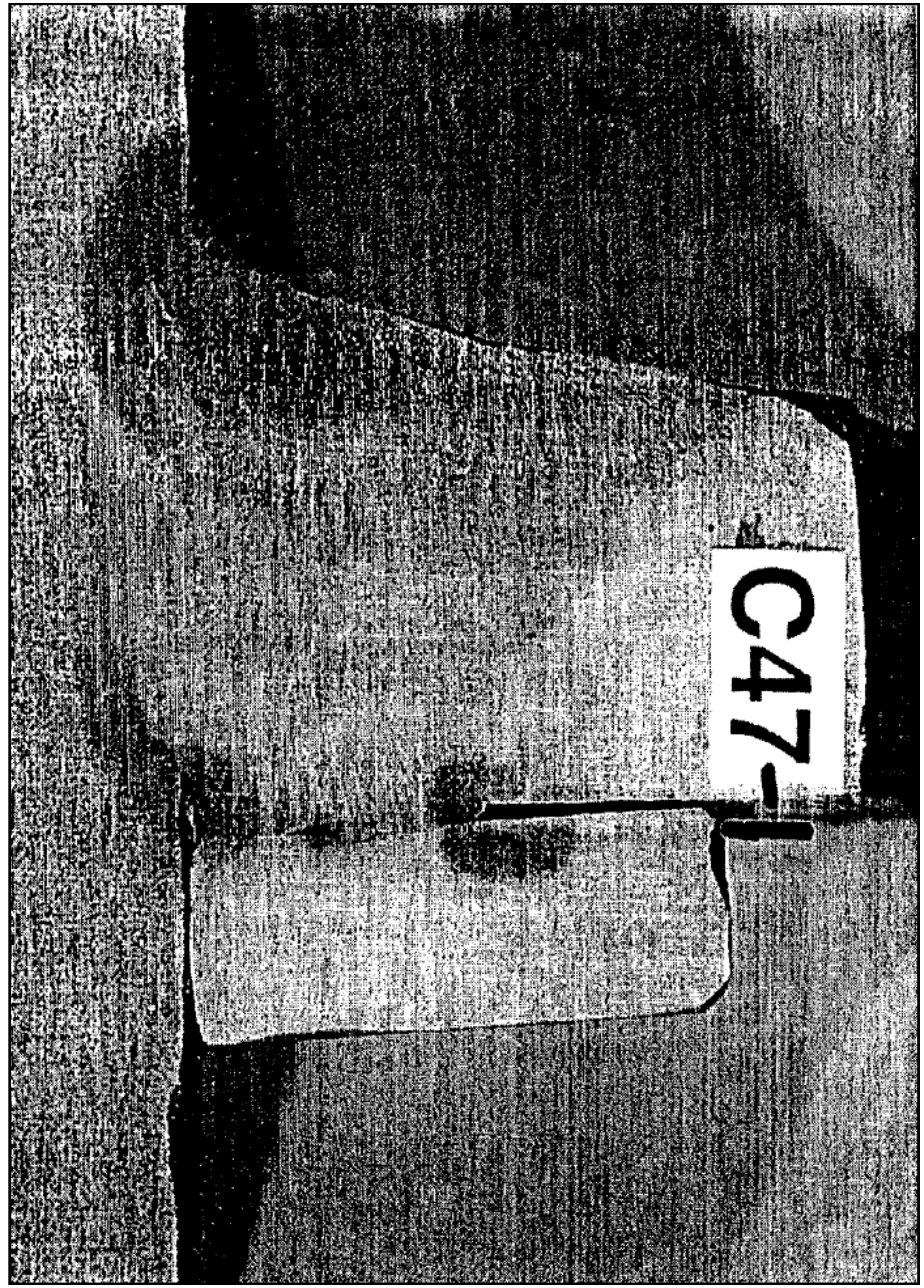


< 100 damaged buildings

1/3rds of building = damaged

From SAC/BD-99/10

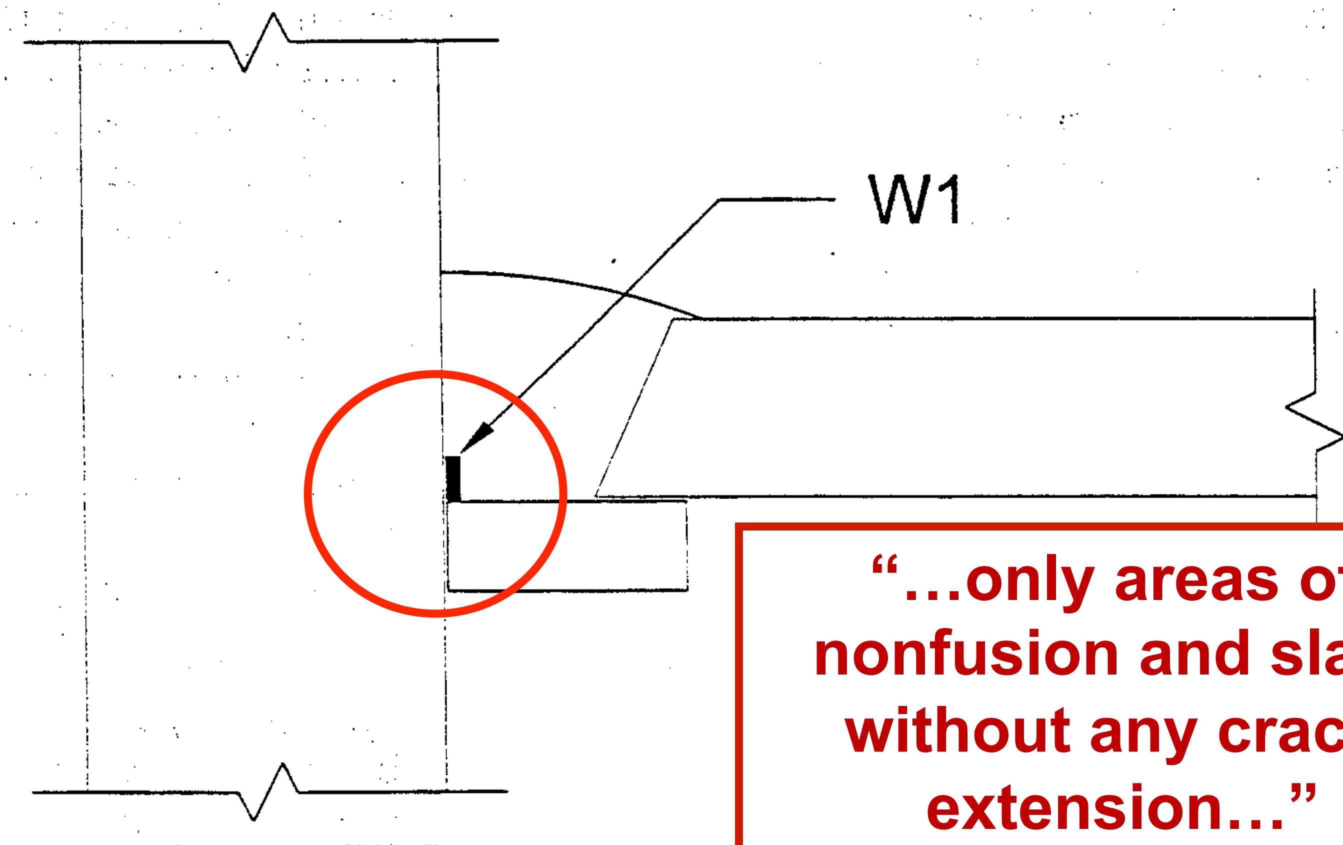
Figure 1. Polished and Etched Weld Sample with W1



SAC Steel Project

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

Figure 2. Location most often defined for W1's by ultrasonics



"…only areas of nonfusion and slag, without any crack extension…"

SAC Steel Project

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

Figure 7. Cumulative Occurrence of Non-W1's in City of Los Angeles Inventory

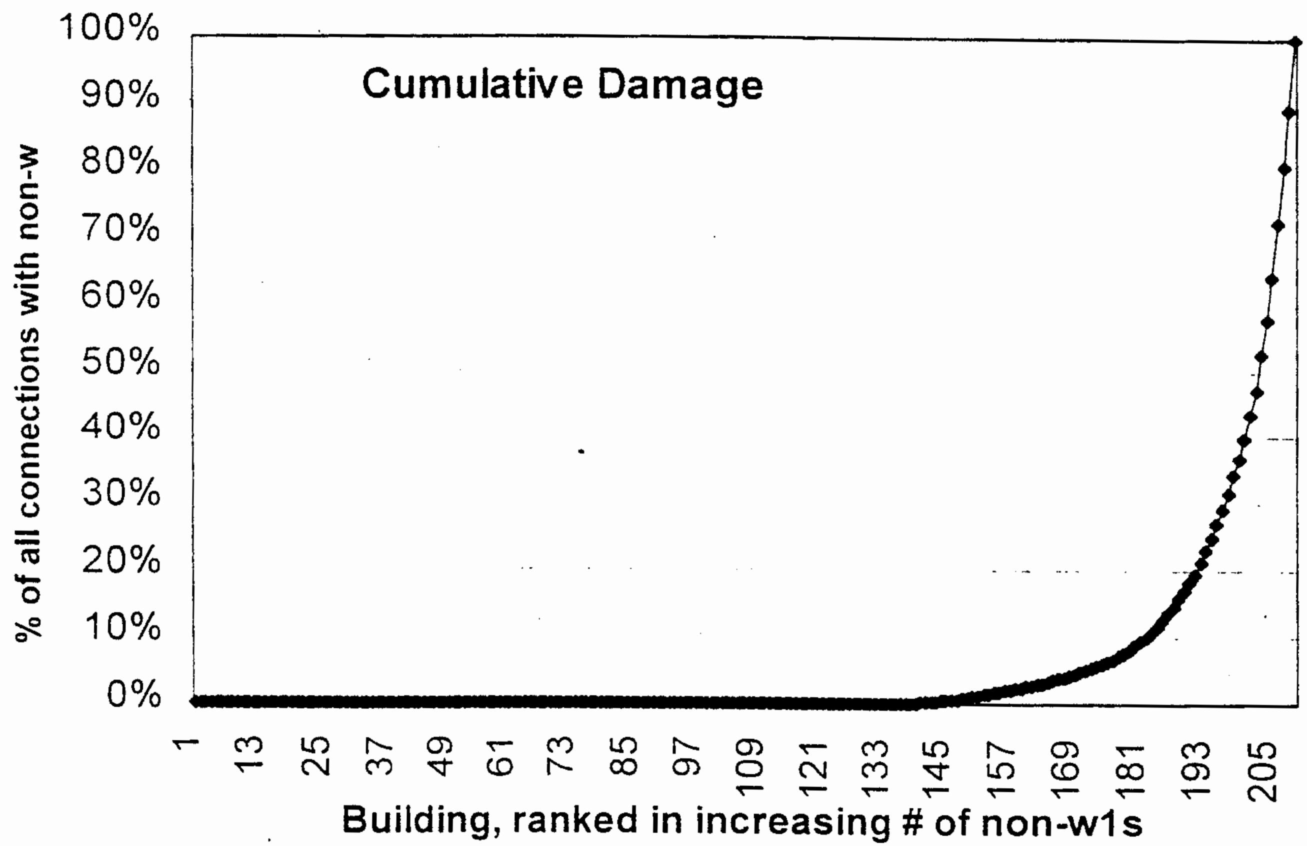


Figure 7. Cumulative Occurrence of Non-W1's in City of Los Angeles Inventory

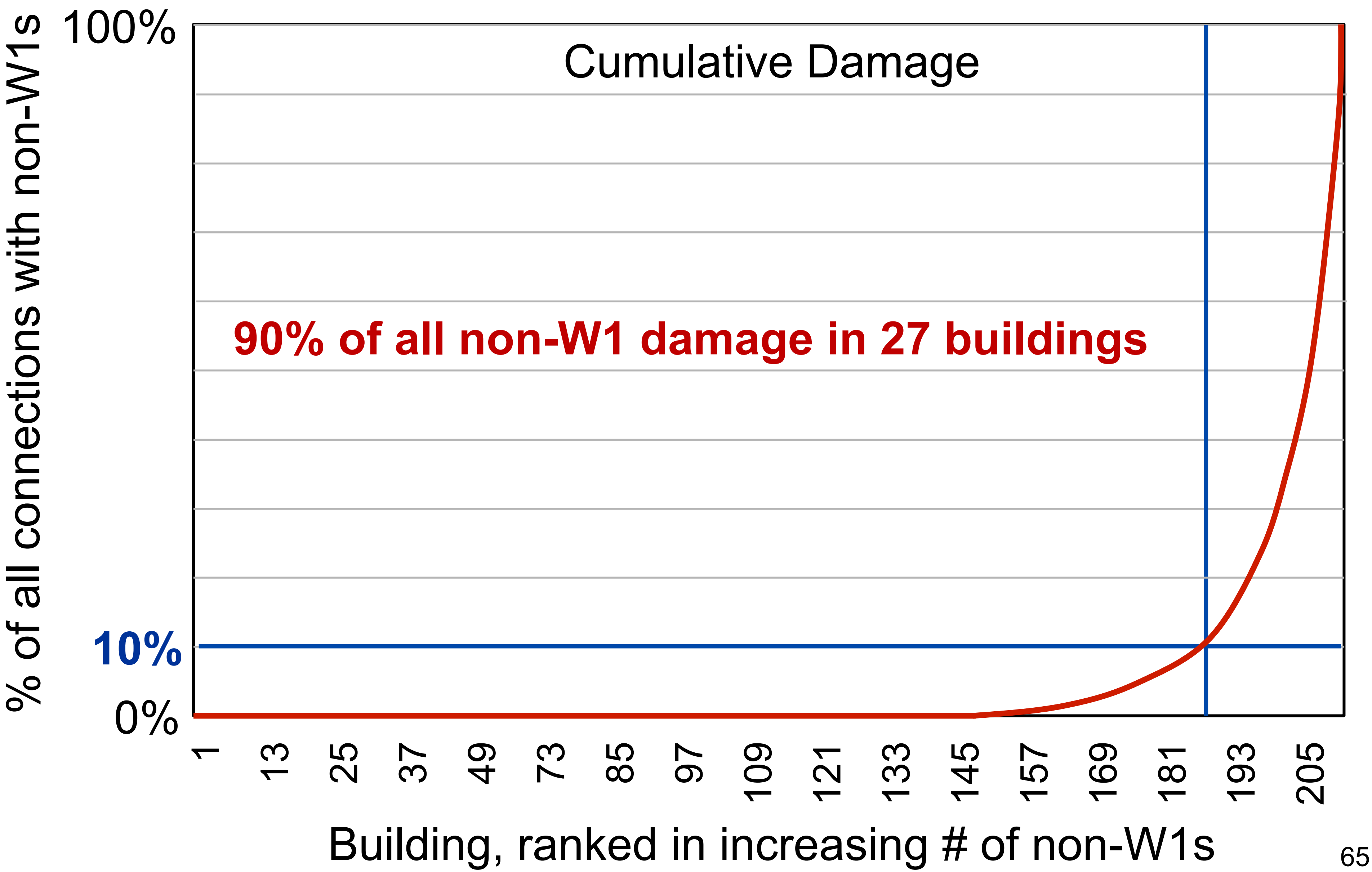


Figure 7. Cumulative Occurrence of Non-W1's in City of Los Angeles Inventory

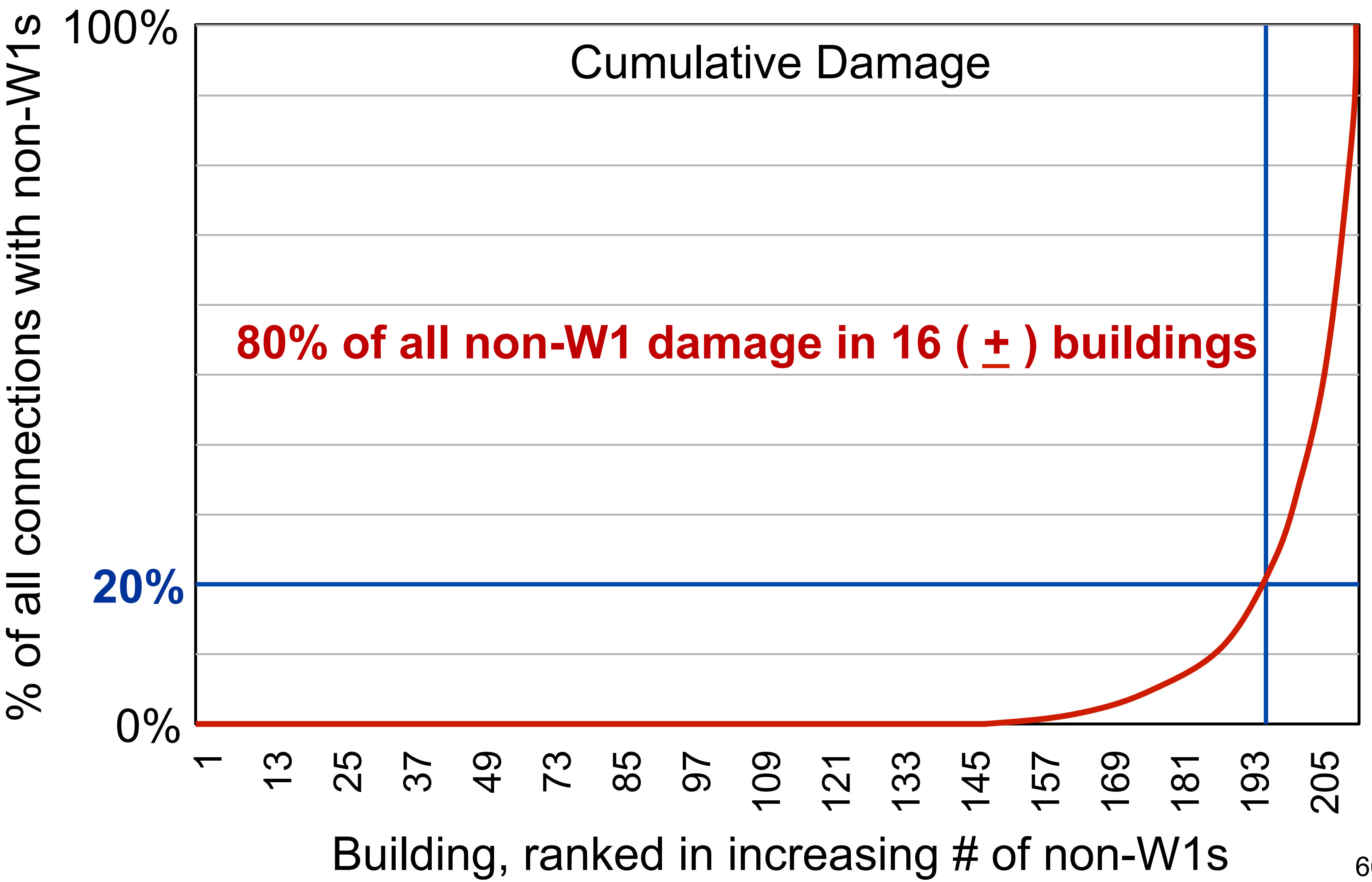


Figure 7. Cumulative Occurrence of Non-W1's in City of Los Angeles Inventory

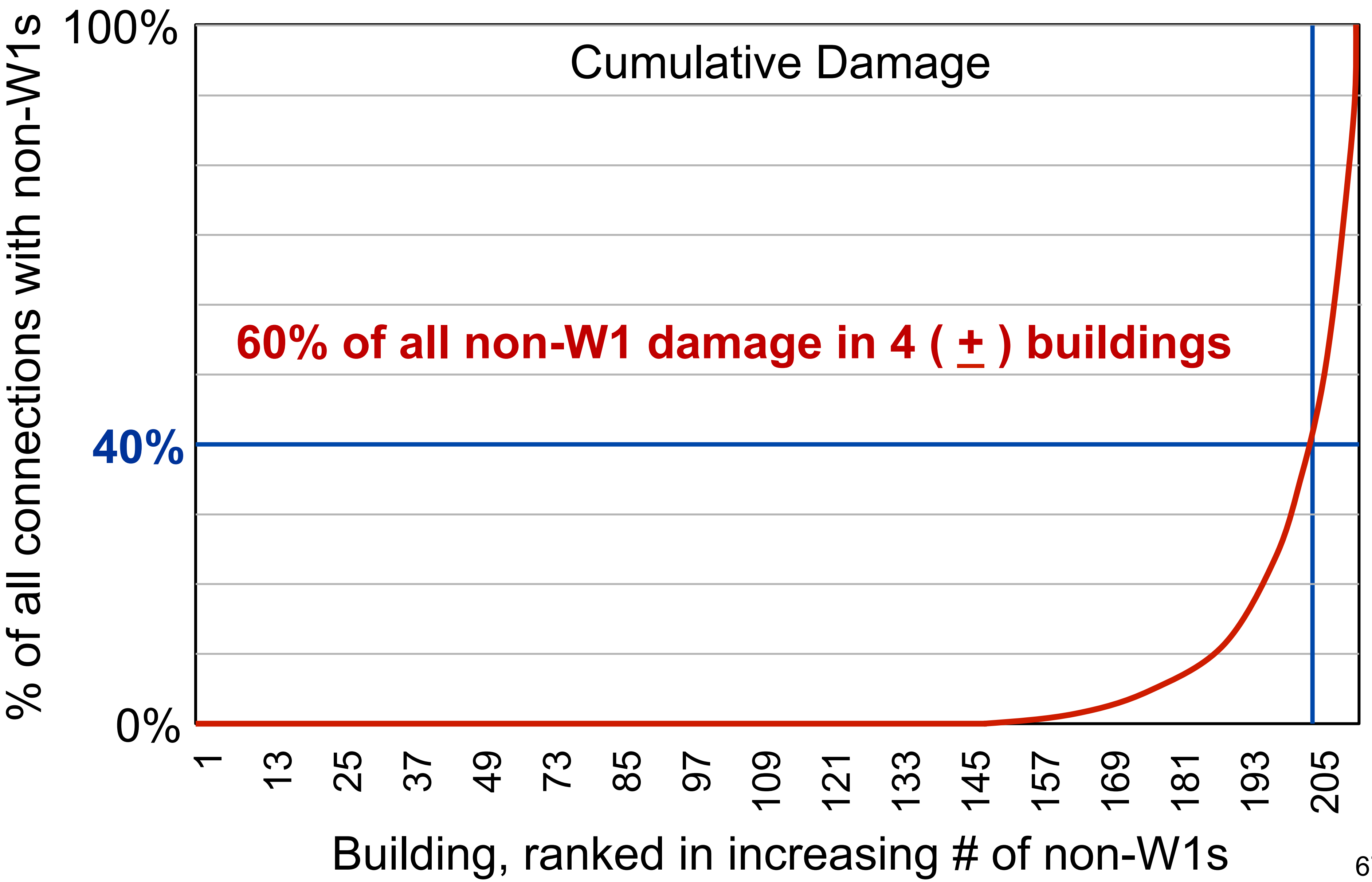
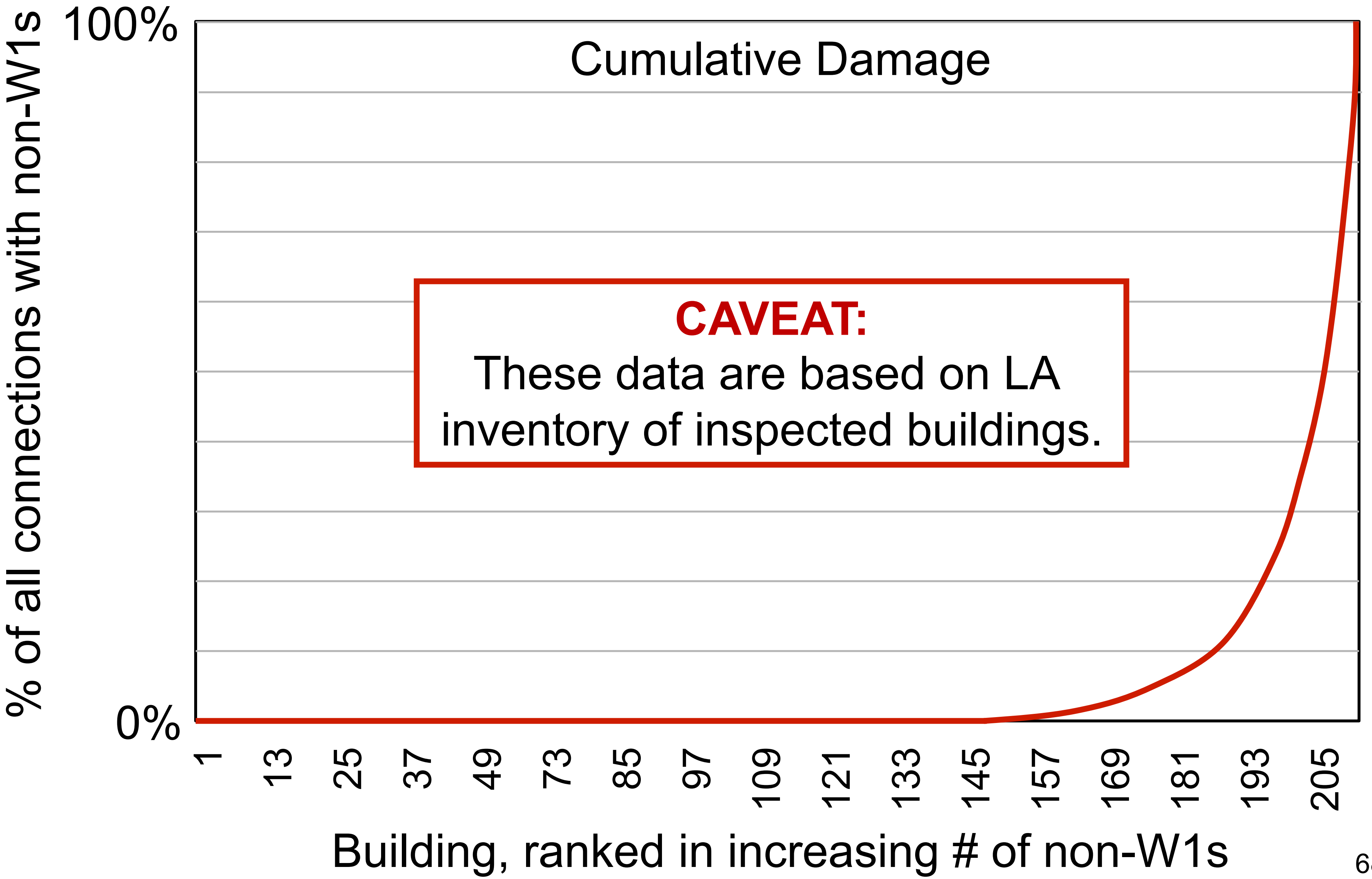


Figure 7. Cumulative Occurrence of Non-W1's in City of Los Angeles Inventory



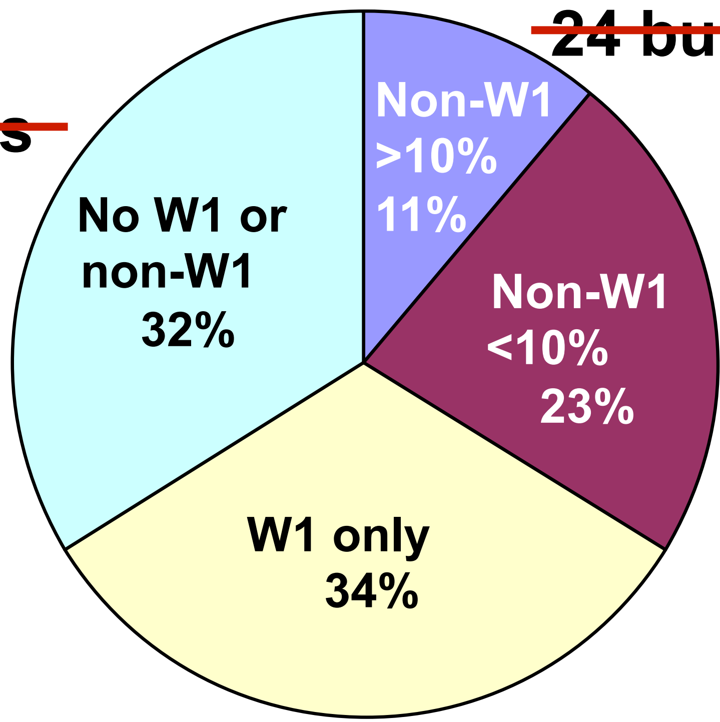
Report No. SAC/BD-99/10 Paret

SAC Steel Project

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

Figure 6 Distribution of W1's and non-W1's ~~in City of Los Angeles Inventory~~

~~66 buildings~~ ~~24 buildings~~



~~48 buildings~~

~~71 buildings~~

CAVEAT:
These data are based on LA inventory of inspected buildings.

General **ratios** might apply (1/3rd-1/3rd-1/3rd)

Building **numbers** do not apply outside data base

Clarifying the Extent of Northridge-Induced Weld Fracturing; Examining the Related Issue of UT Reliability

Therefore, in the general region of strongest ground shaking only 24 buildings (11% of the total sample) were found to have more than 10% of their connections damaged by the earthquake…..the scope of the "welded moment frame problem"—previously characterized as having results in many scores of severely damaged buildings—appears to be greatly reduced."

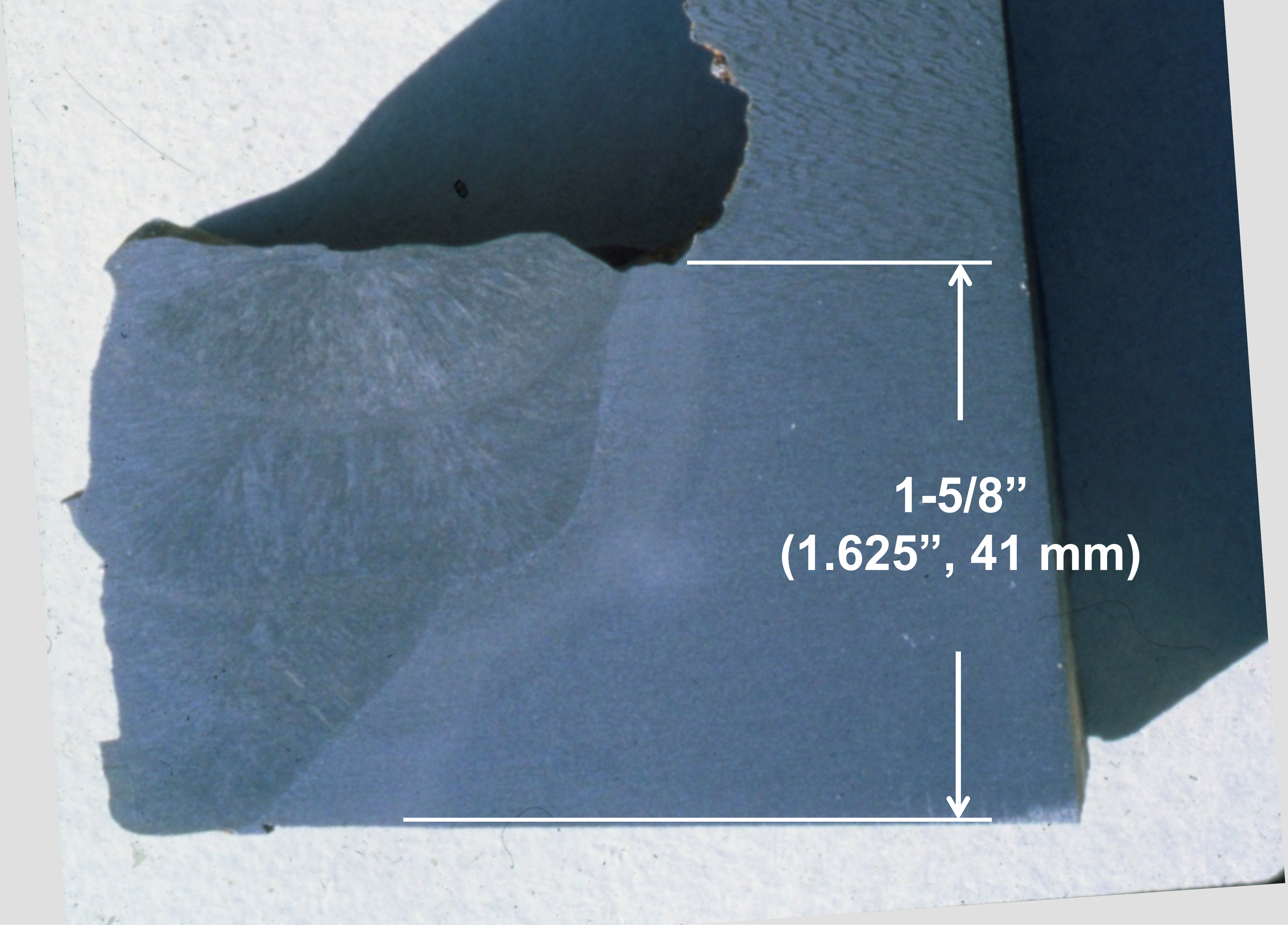
Revisiting W1 Indications

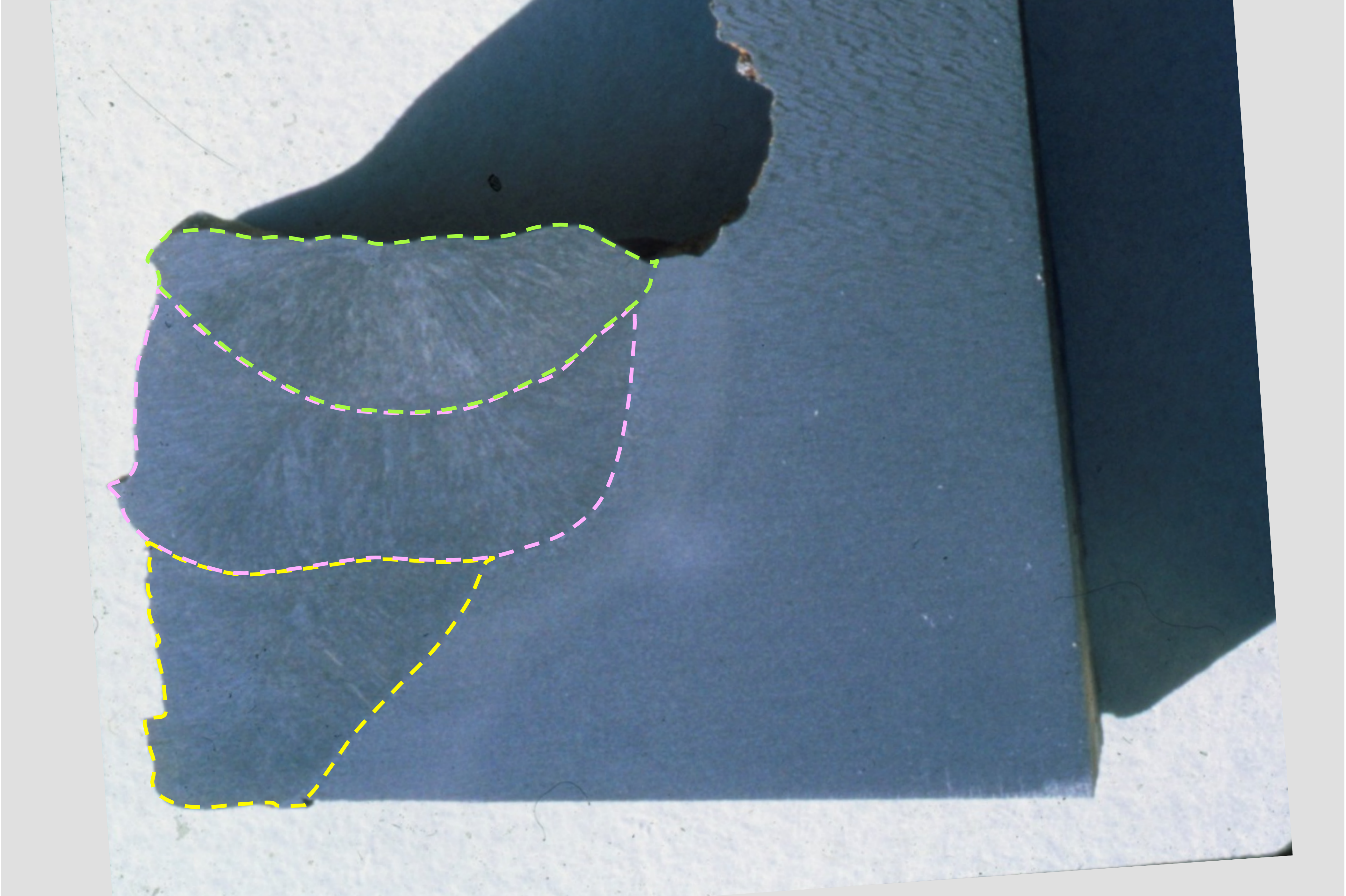
W1s: What Was Done To Eliminate Them?

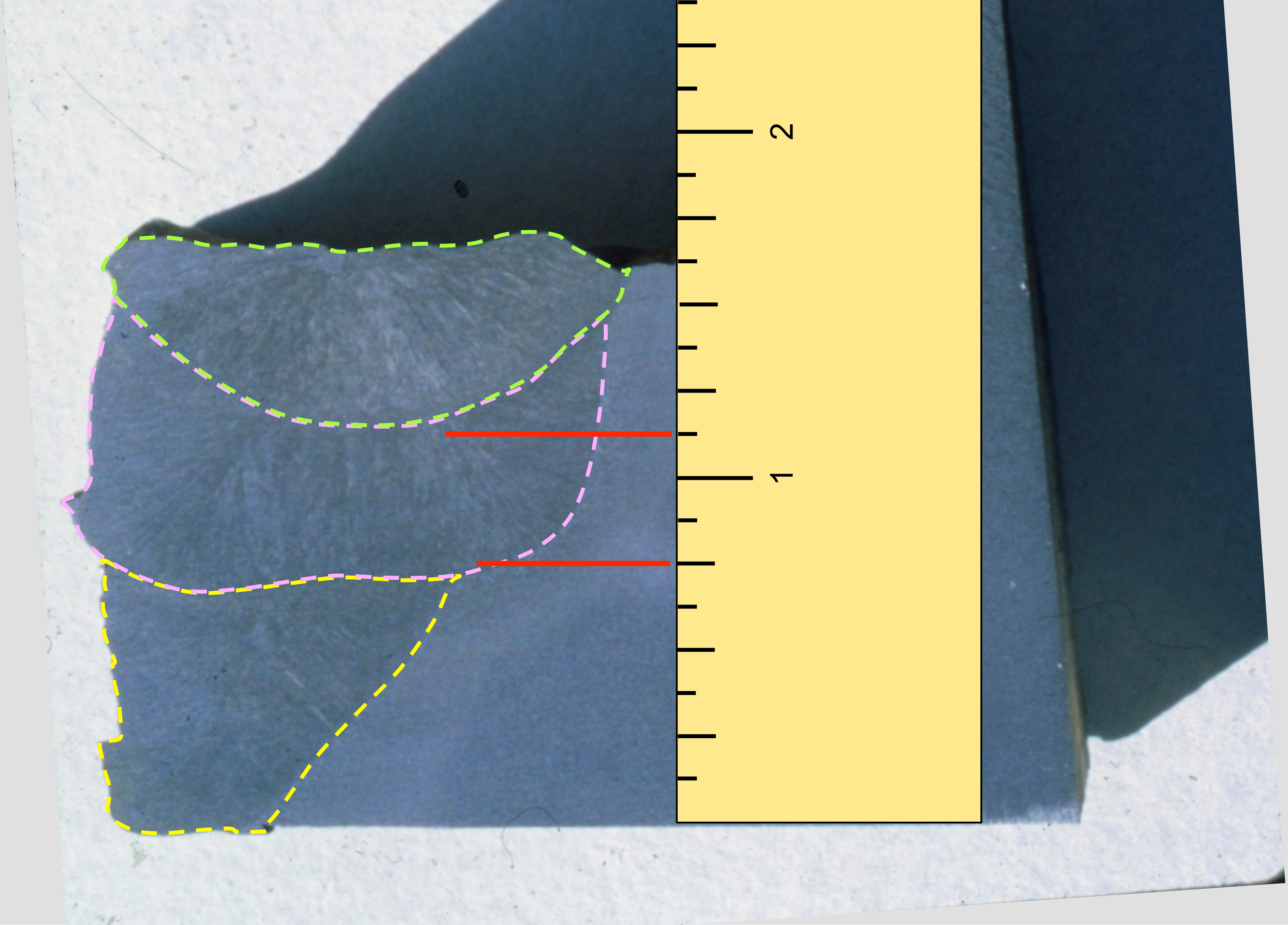
Revisiting W1 Indications

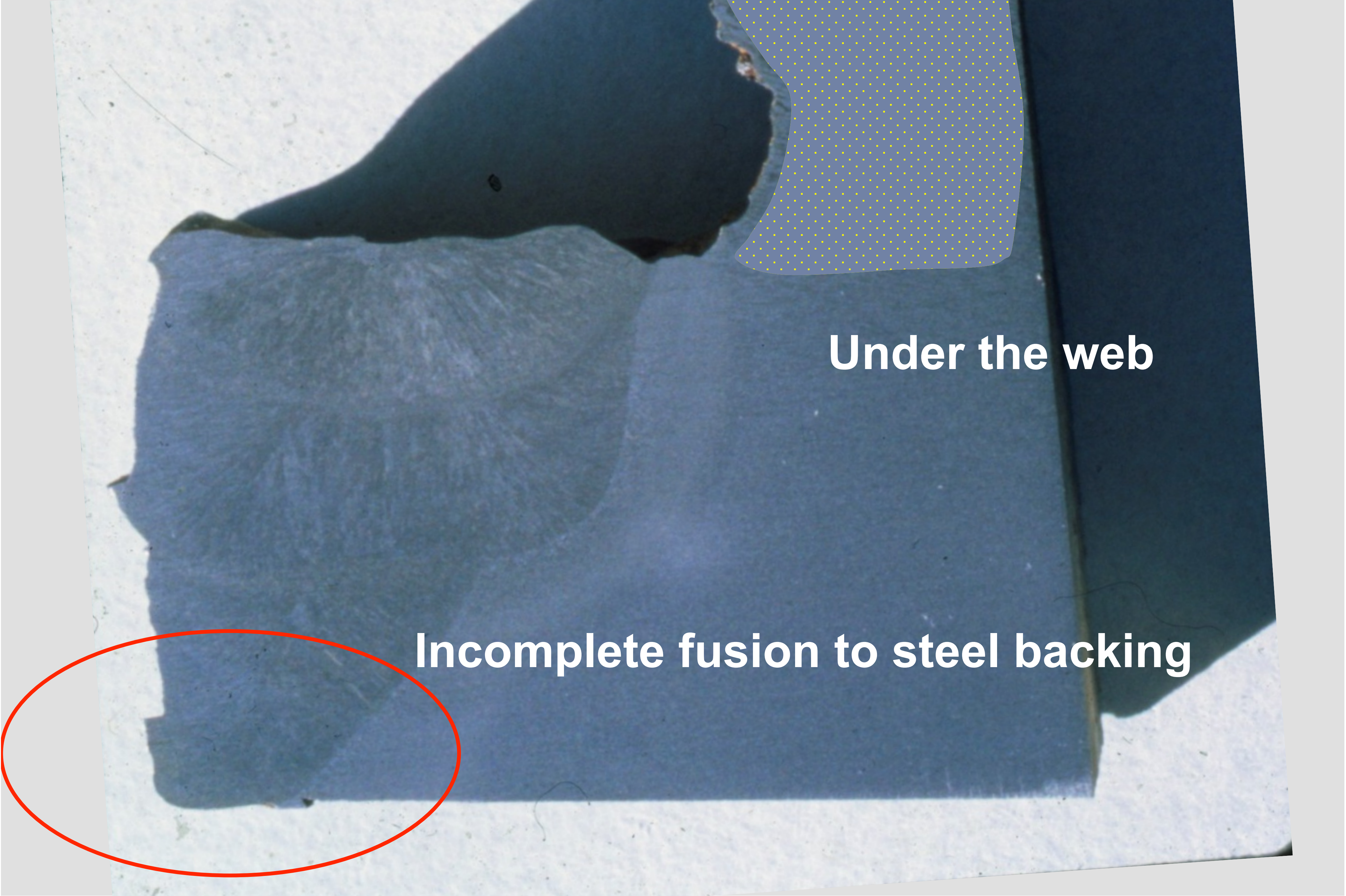
W1s: What Was Done To Enable Better Detection of Them?

Likely a W4


1-5/8”
(1.625”, 41 mm)




2
1


Under the web
Incomplete fusion to steel backing


1/4”
[6 mm]

AWS D1.1-94

Structural Welding Code--
Steel

ANSI/AWS D1.1-94
An American National Standard

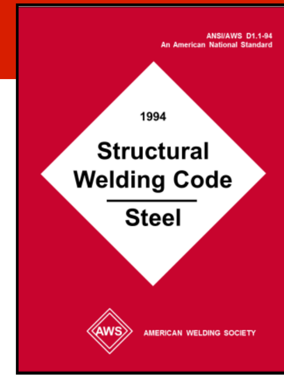
1994

Structural Welding Code

Steel

AMERICAN WELDING SOCIETY

AWS D1.1-94 Structural Welding Code--Steel



4.14 Procedures for Gas Metal Arc and Flux Cored Arc Welding with Single Electrode

4.14.1.5 Flux Cored Arc Welding

The thickness of the weld layers in groove welds, except root and surface layers, shall not exceed 1/4 in (6 mm).

AWS D1.1-96

Structural Welding Code--
Steel

AWS D1.1-96 Structural Welding Code--Steel

Table 3.7

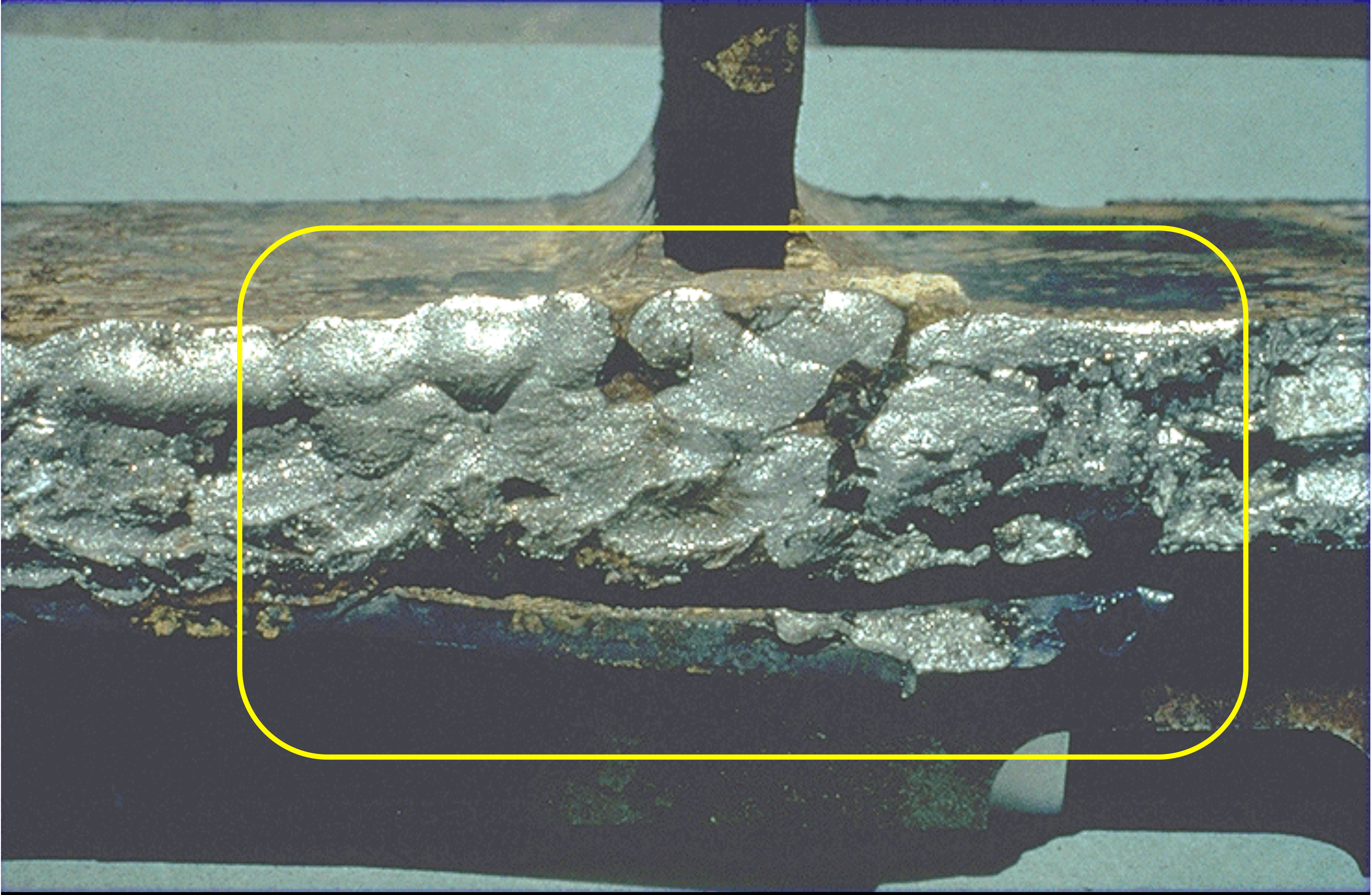
Prequalified WPS Requirements

Table 3.7
Prequalified WPS Requirements[f] (see 3.7)

Variable	Position	Weld Type	SMAW	SAW[d] Single	SAW[d] Parallel	SAW[d] Multiple	GMAW/ FCAW[g]
Maximum Root Pass Thickness[d]	Flat	All	3/8 in [10 mm]	Unlimited			3/8 in [10 mm]
	Horizontal		5/16 in [8 mm]				5/16 in [8 mm]
	Vertical		1/2 in [12 mm]				1/2 in [12 mm]
	Overhead		5/16 in [8 mm]				5/16 in [8 mm]

In 2006, for prequalified WPSs, the maximum root pass thickness is now 3/8 in [10 mm]

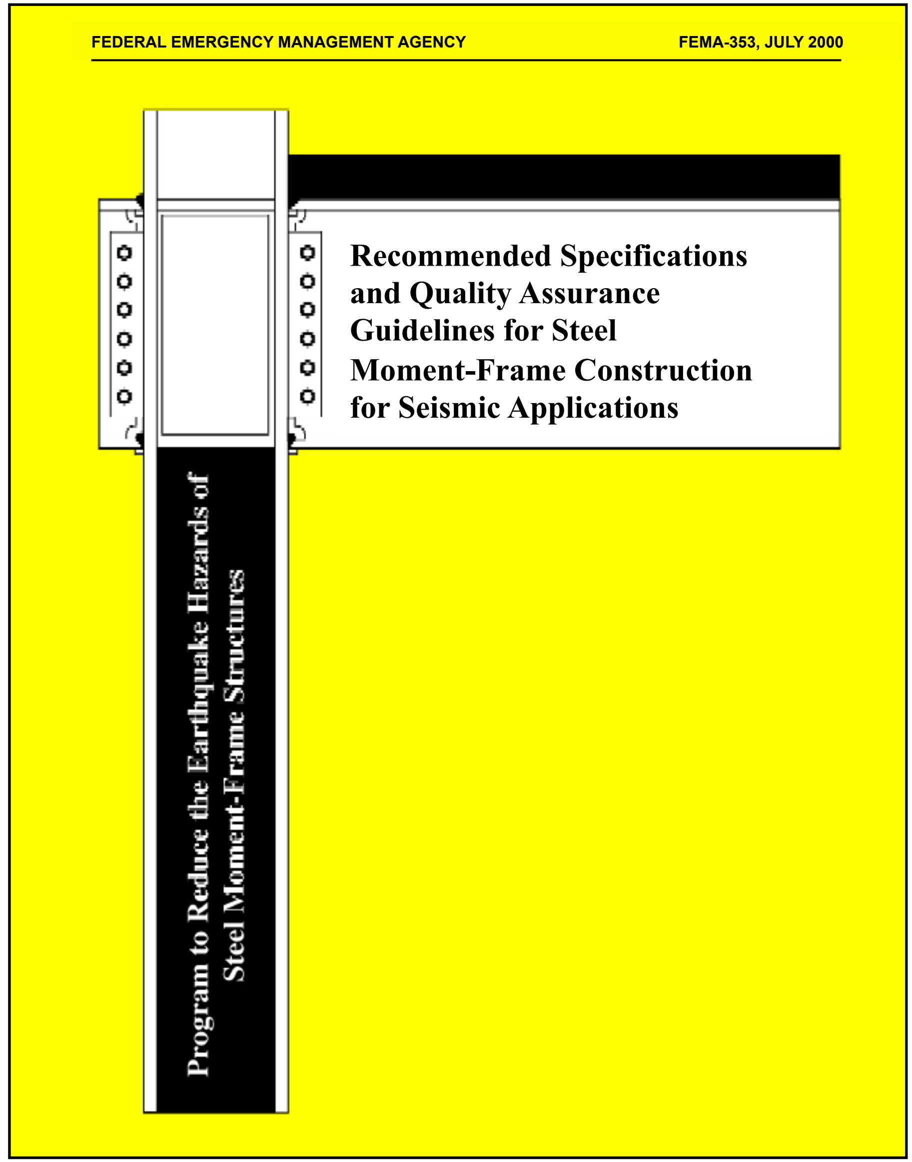

B-1
RD FLR.

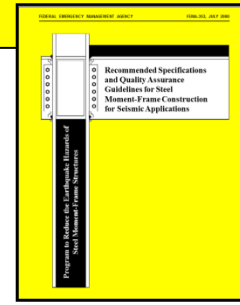


FEMA 353

Recommended Specification and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications

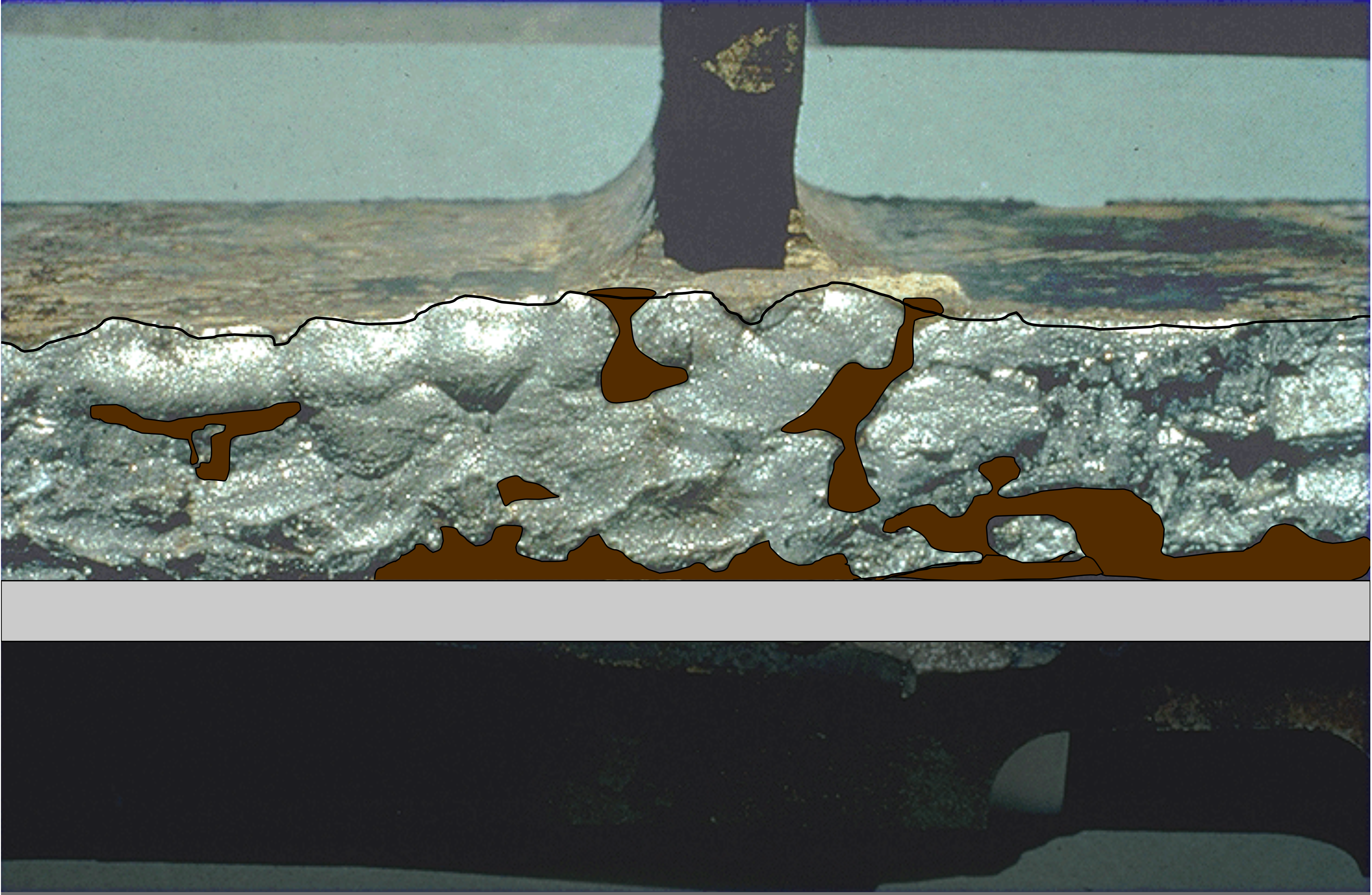
July 2000

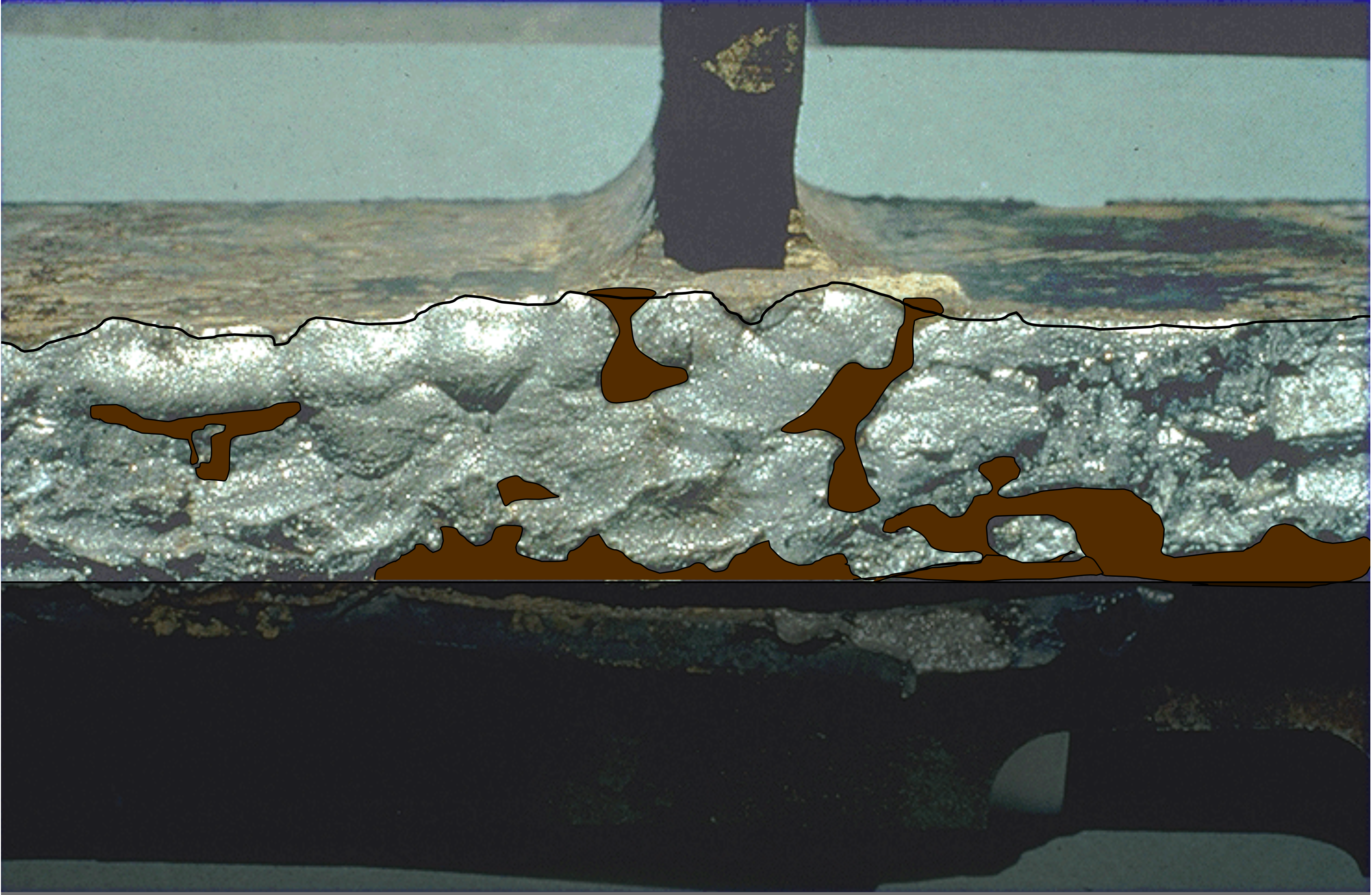


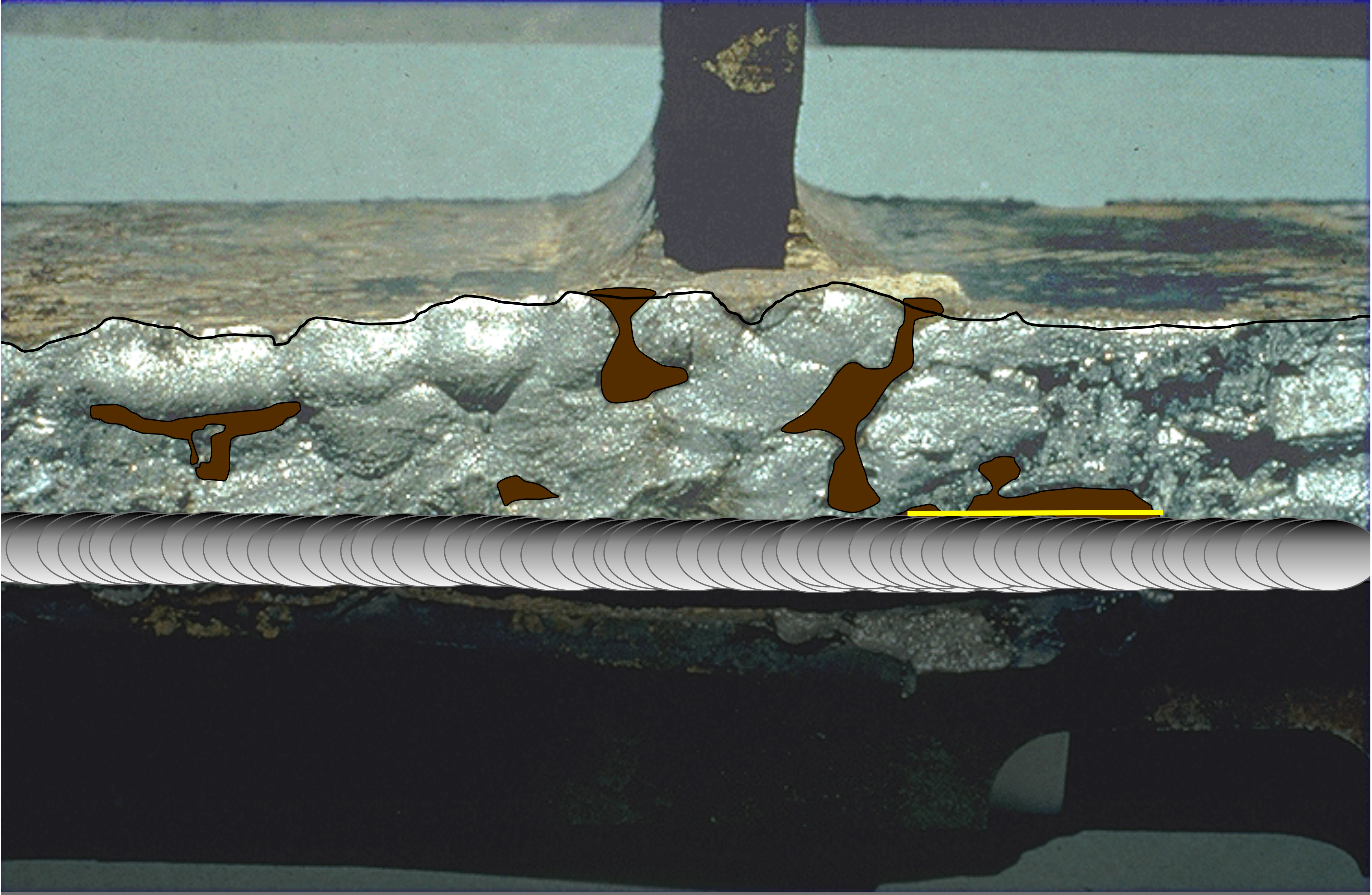


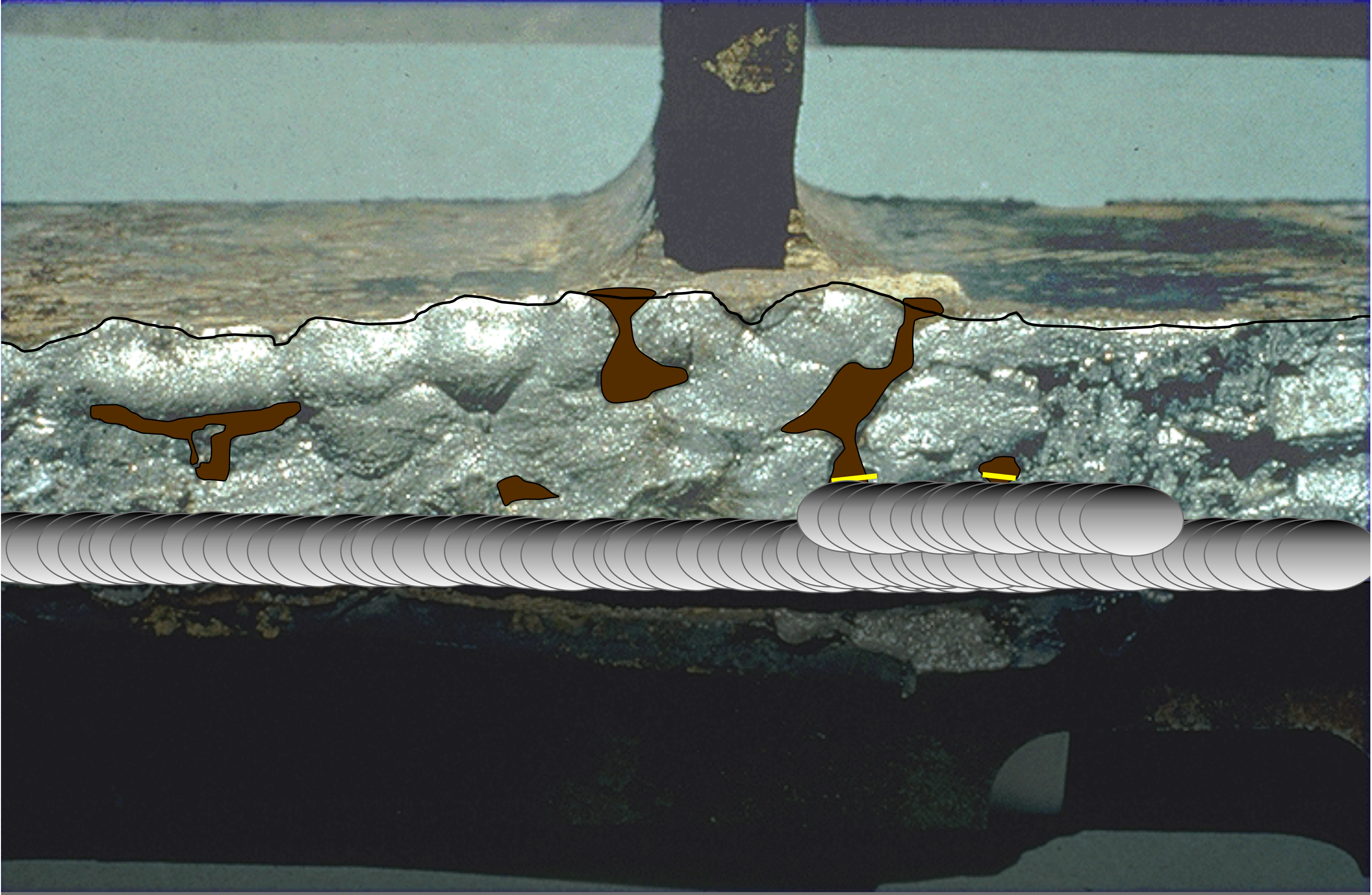
4.1.2 Moment Connection Joints Requiring Removal of Backing Bars

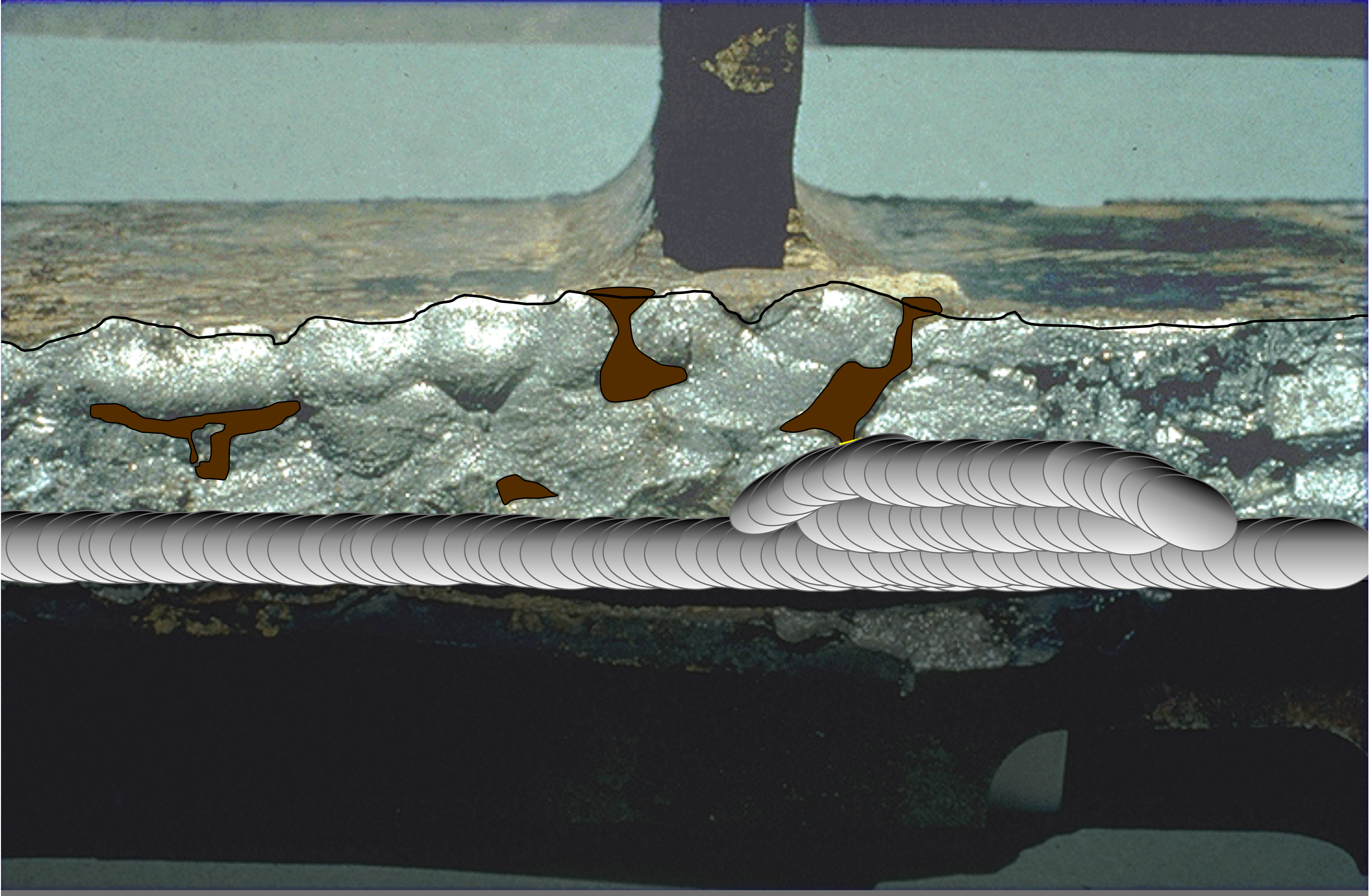
Backing bars shall be removed from the joint when required on the design drawings. Following removal of backing, the root pass shall be backgouged to sound metal, and backwelded.

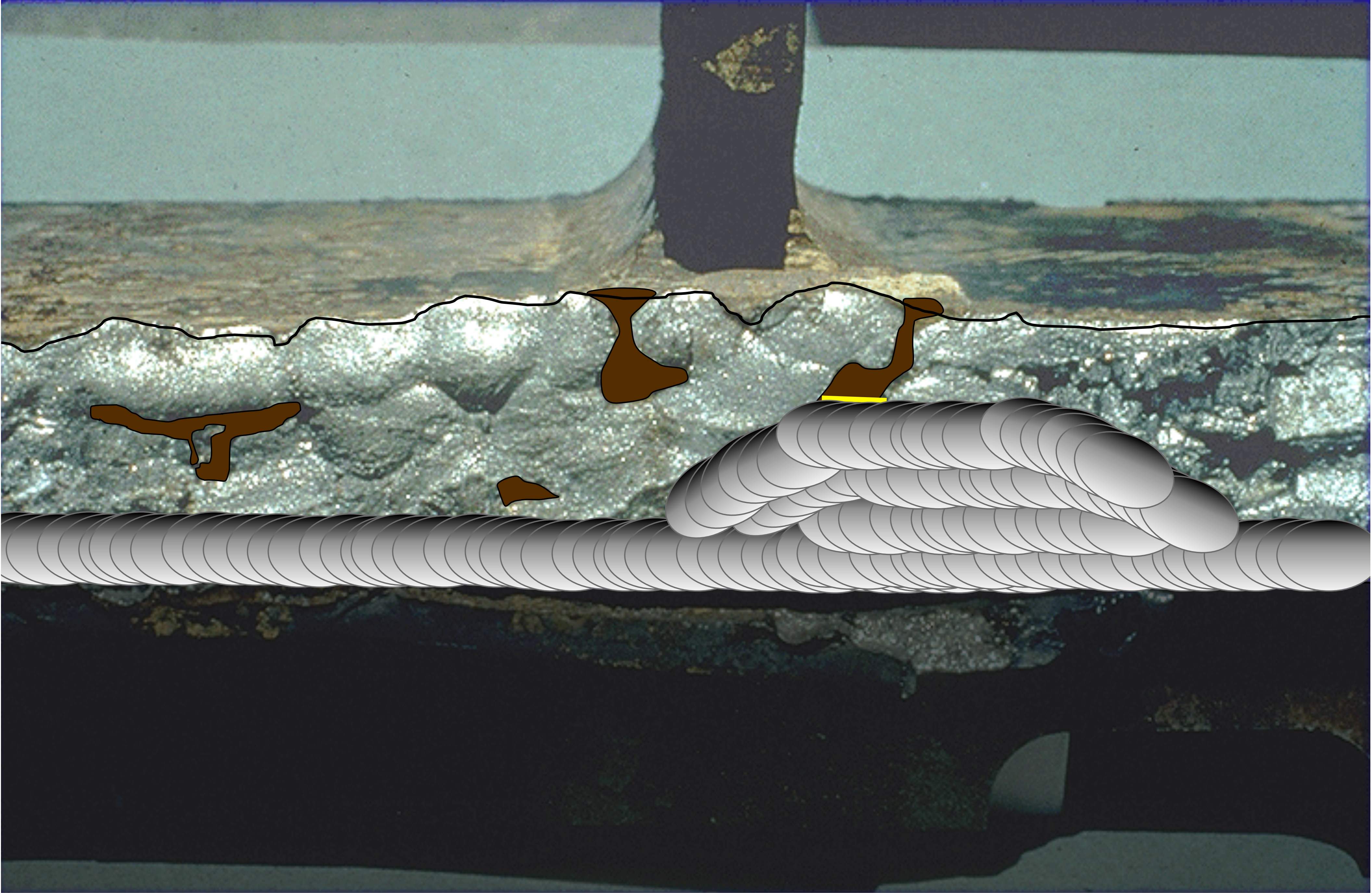


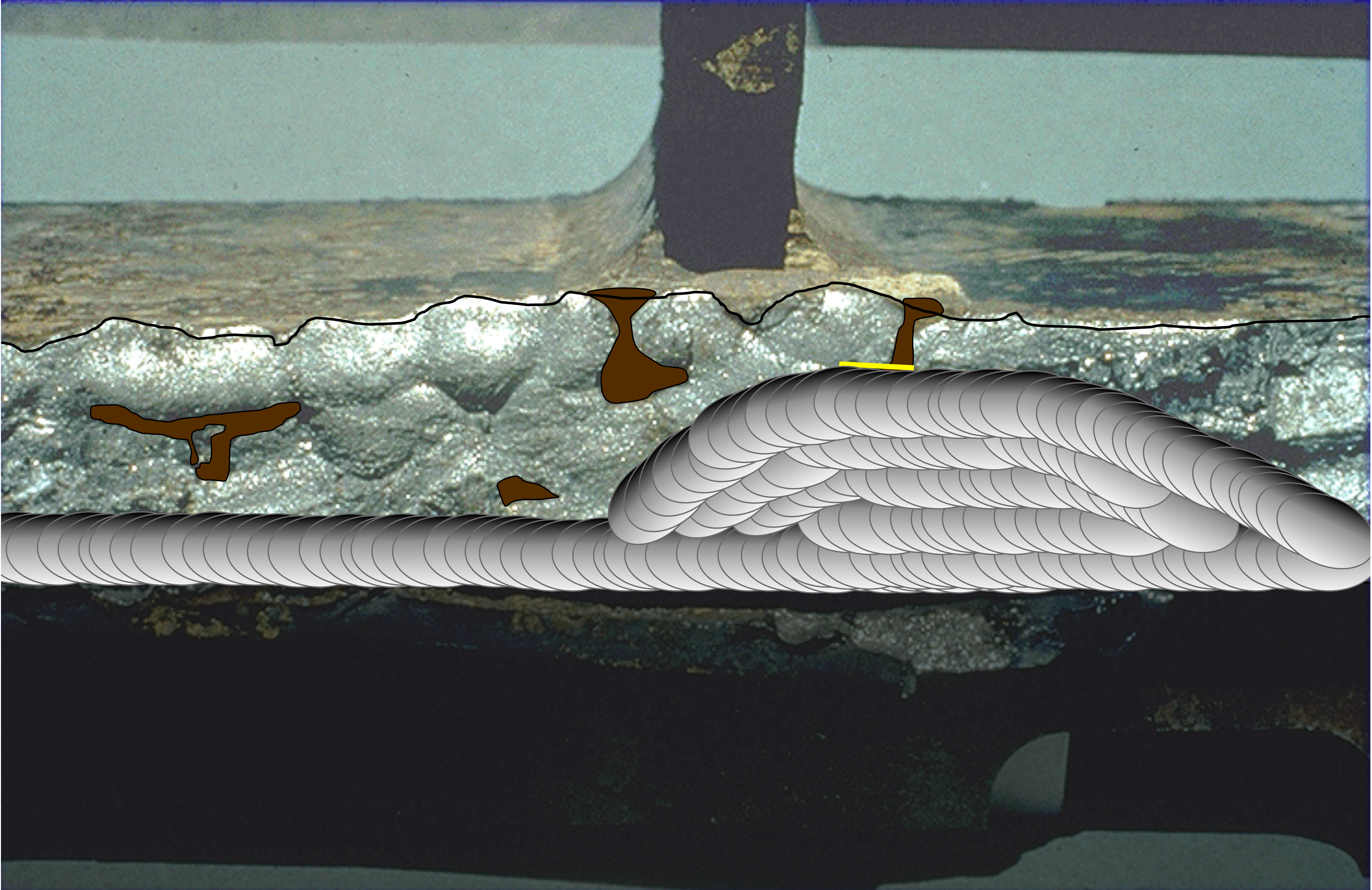


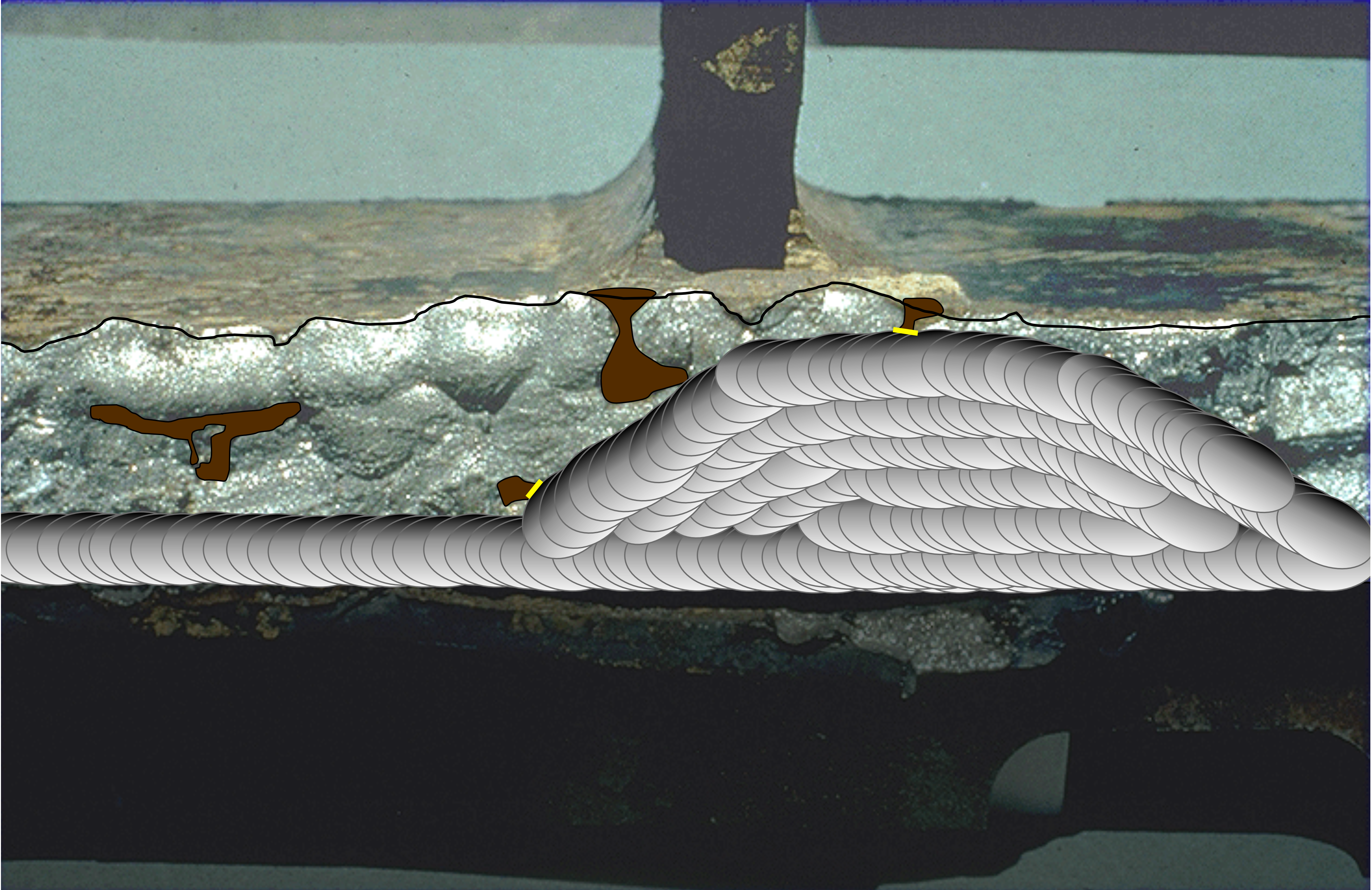


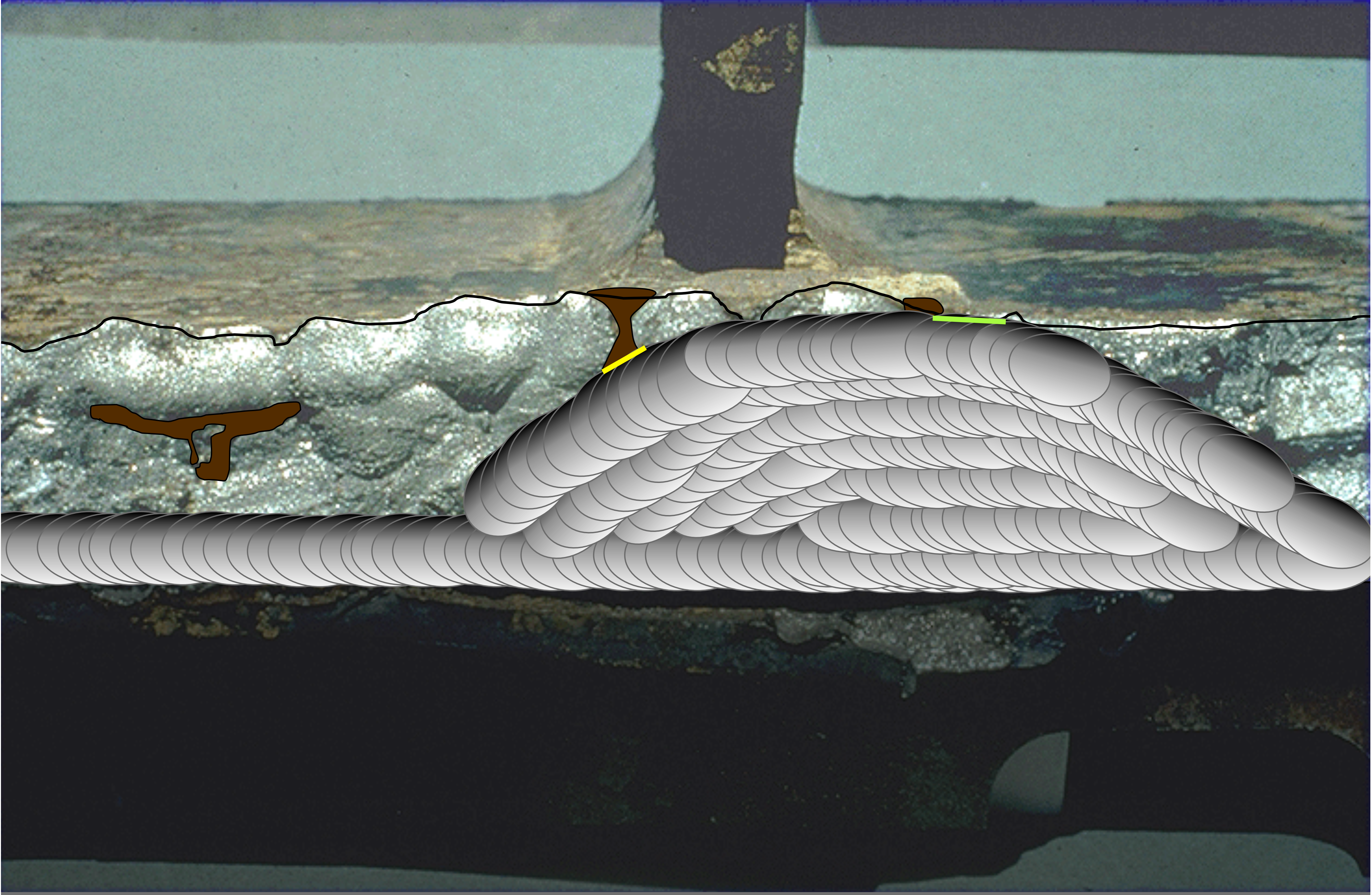


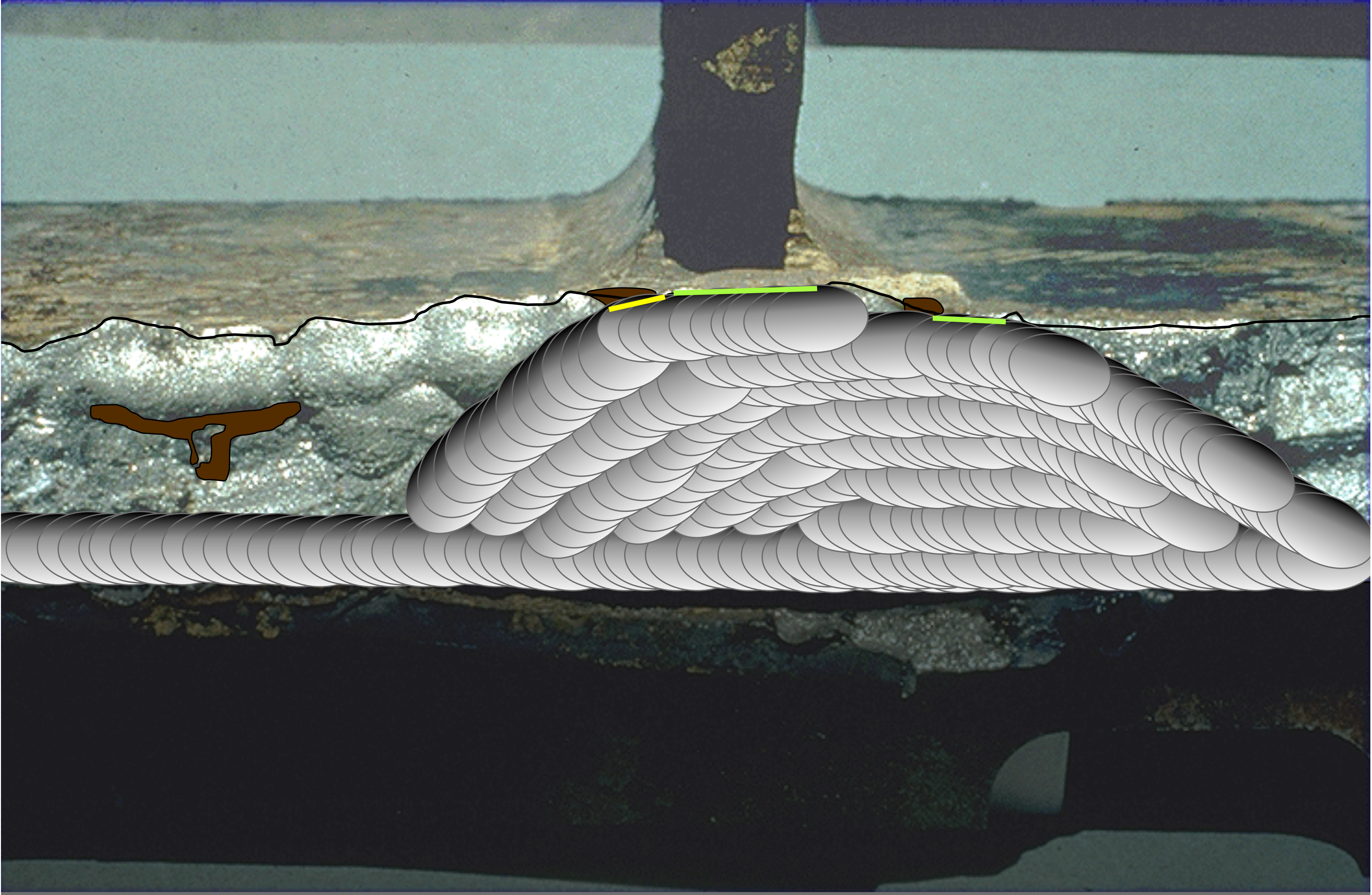


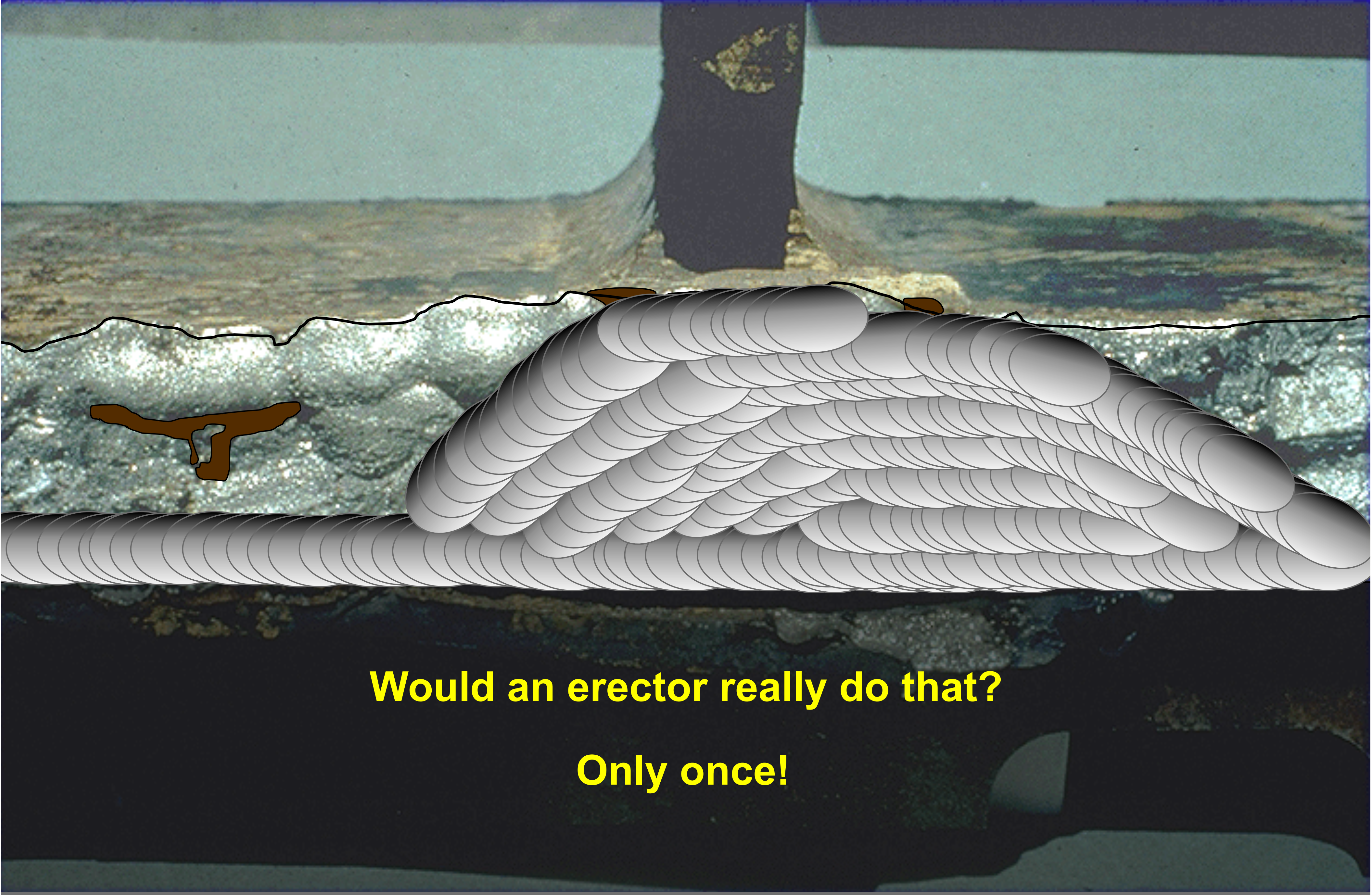


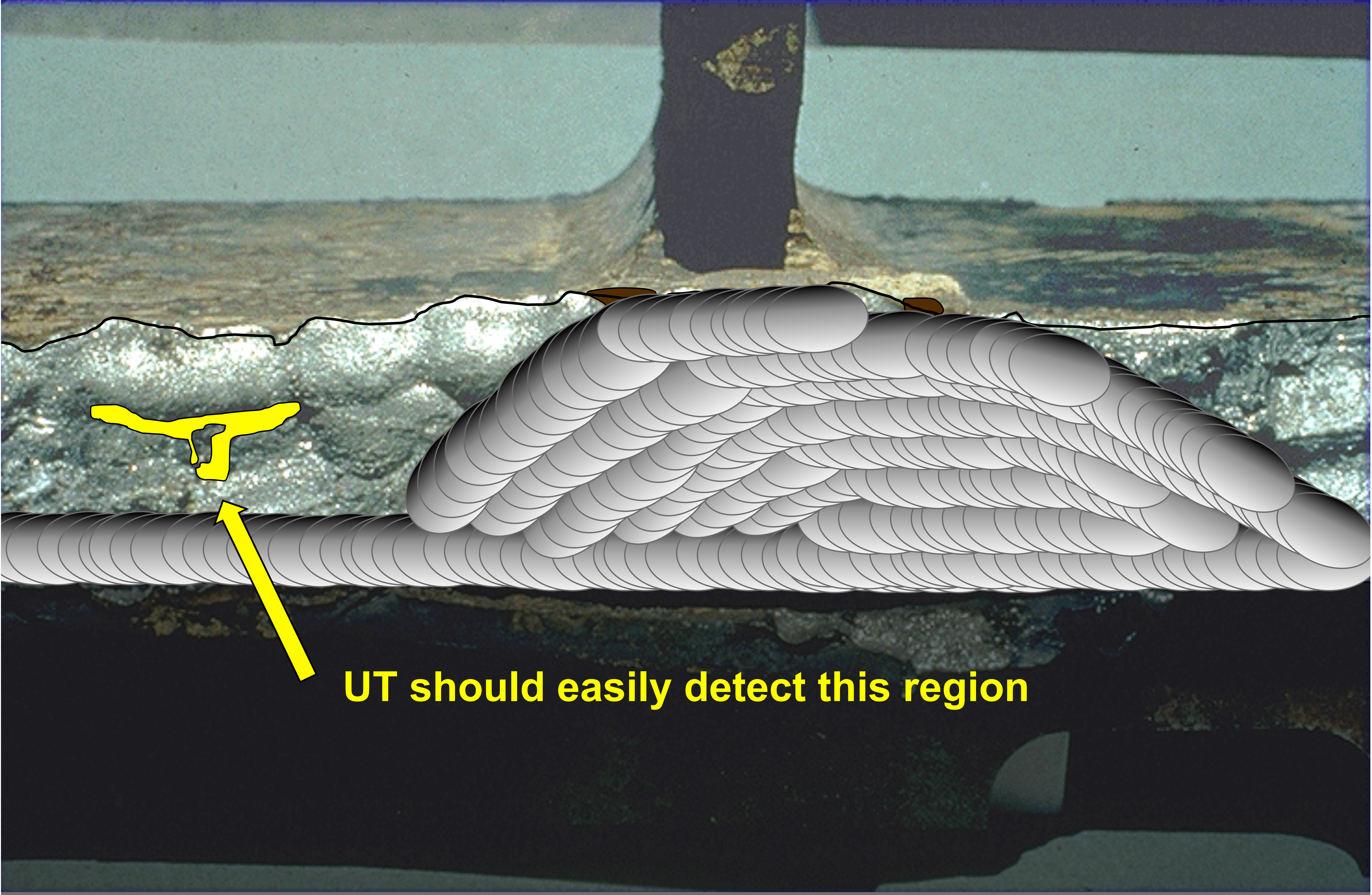


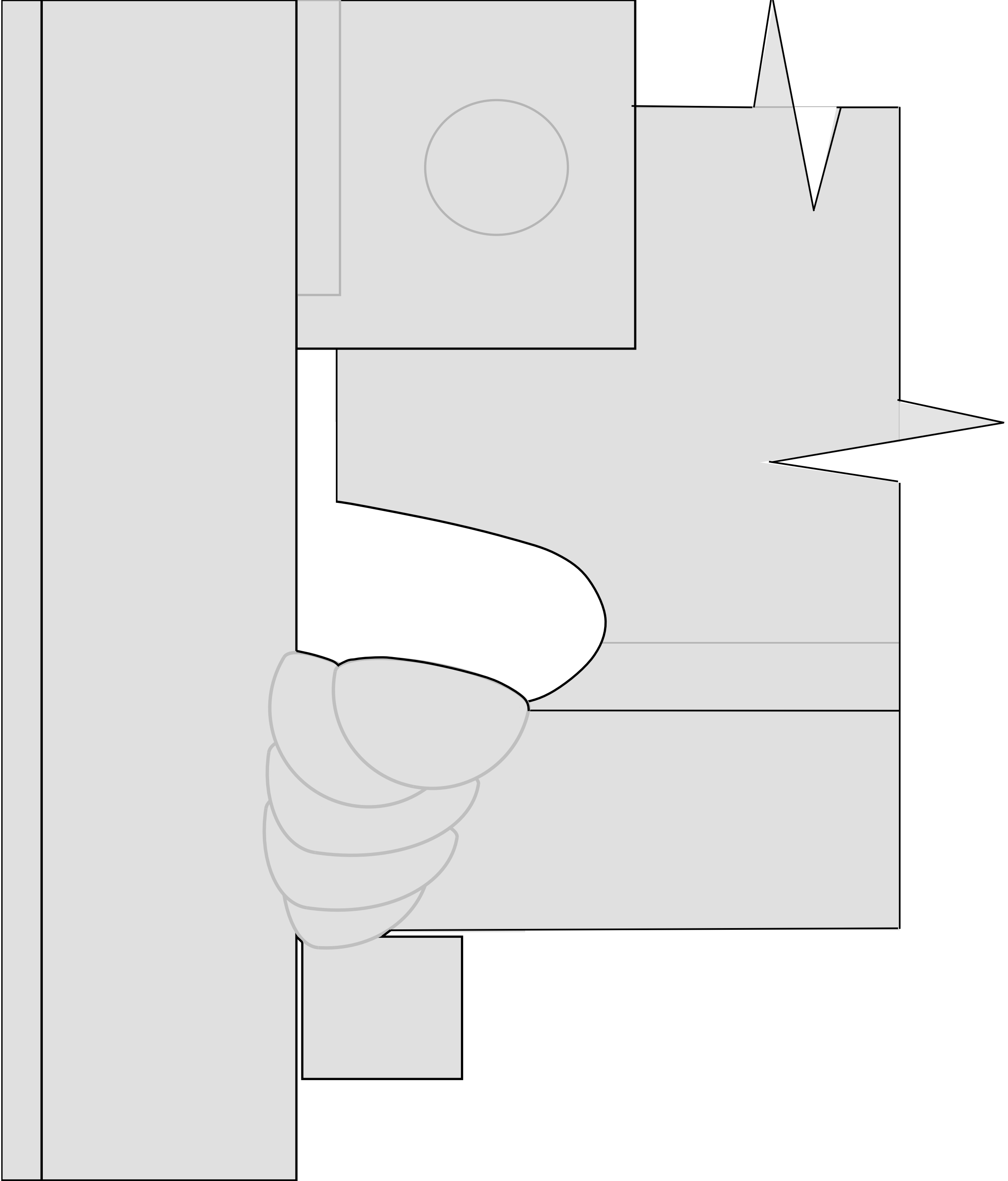




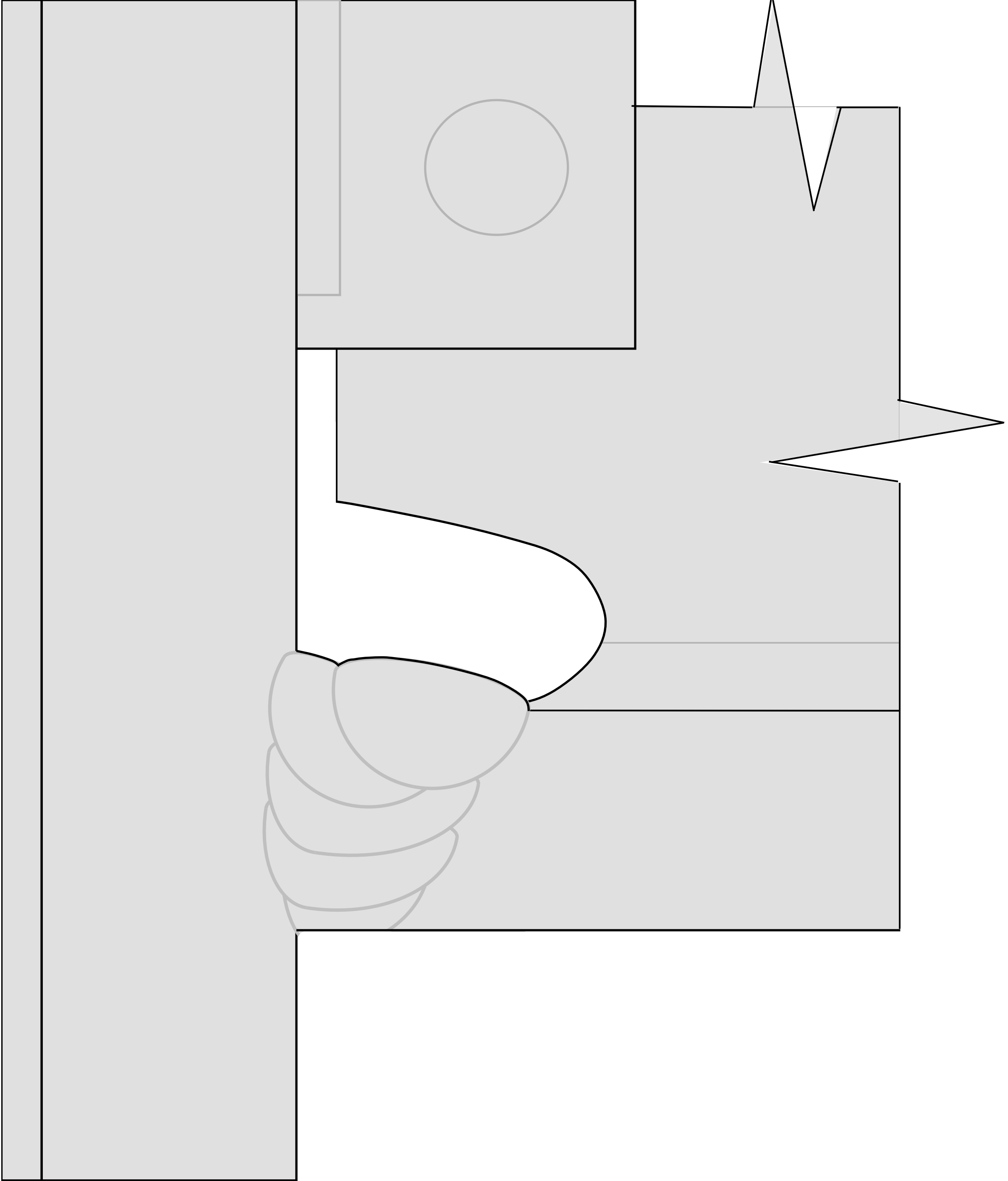



Would an erector really do that?
Only once!


UT should easily detect this region

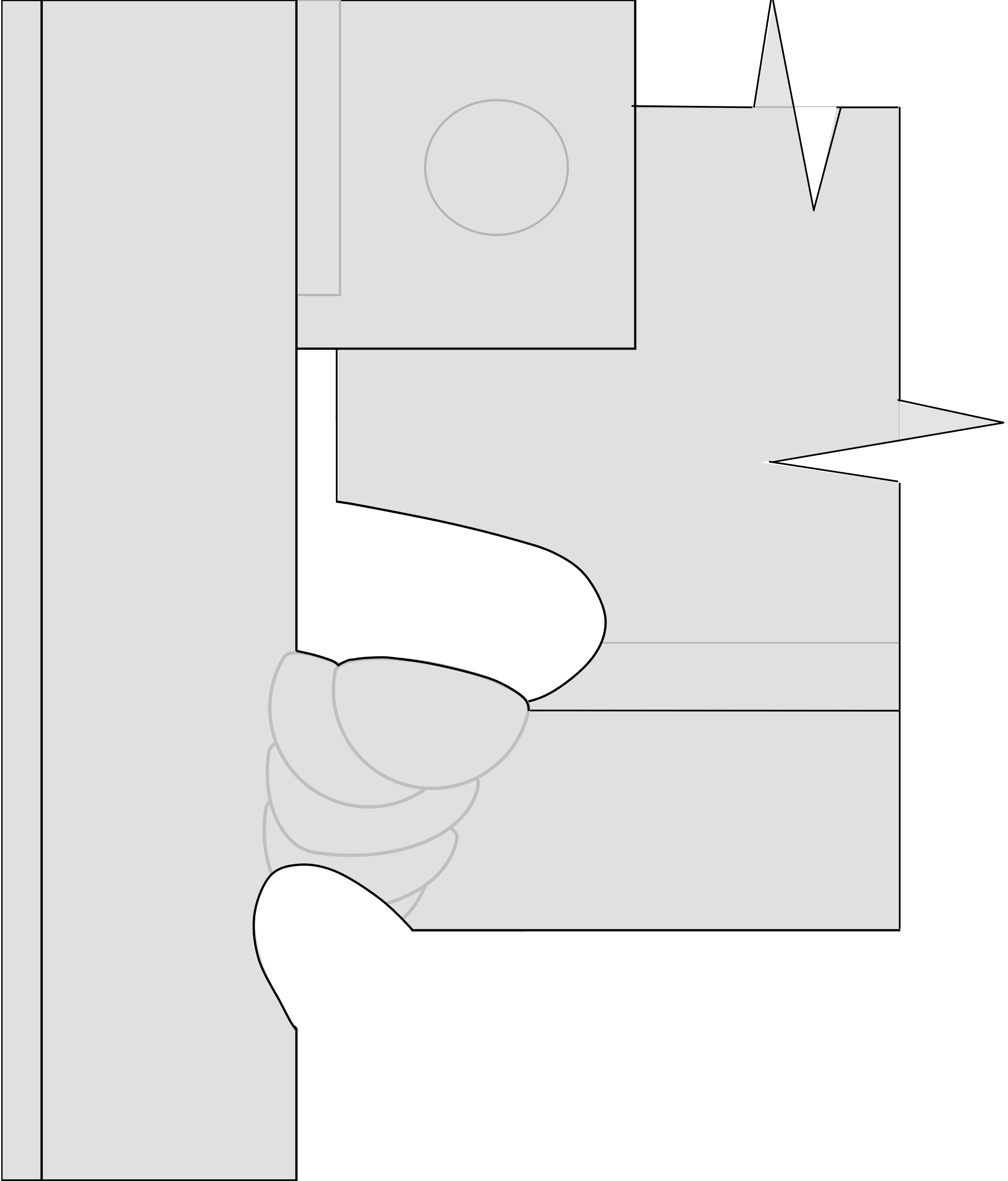


Benefits of Backing Removal



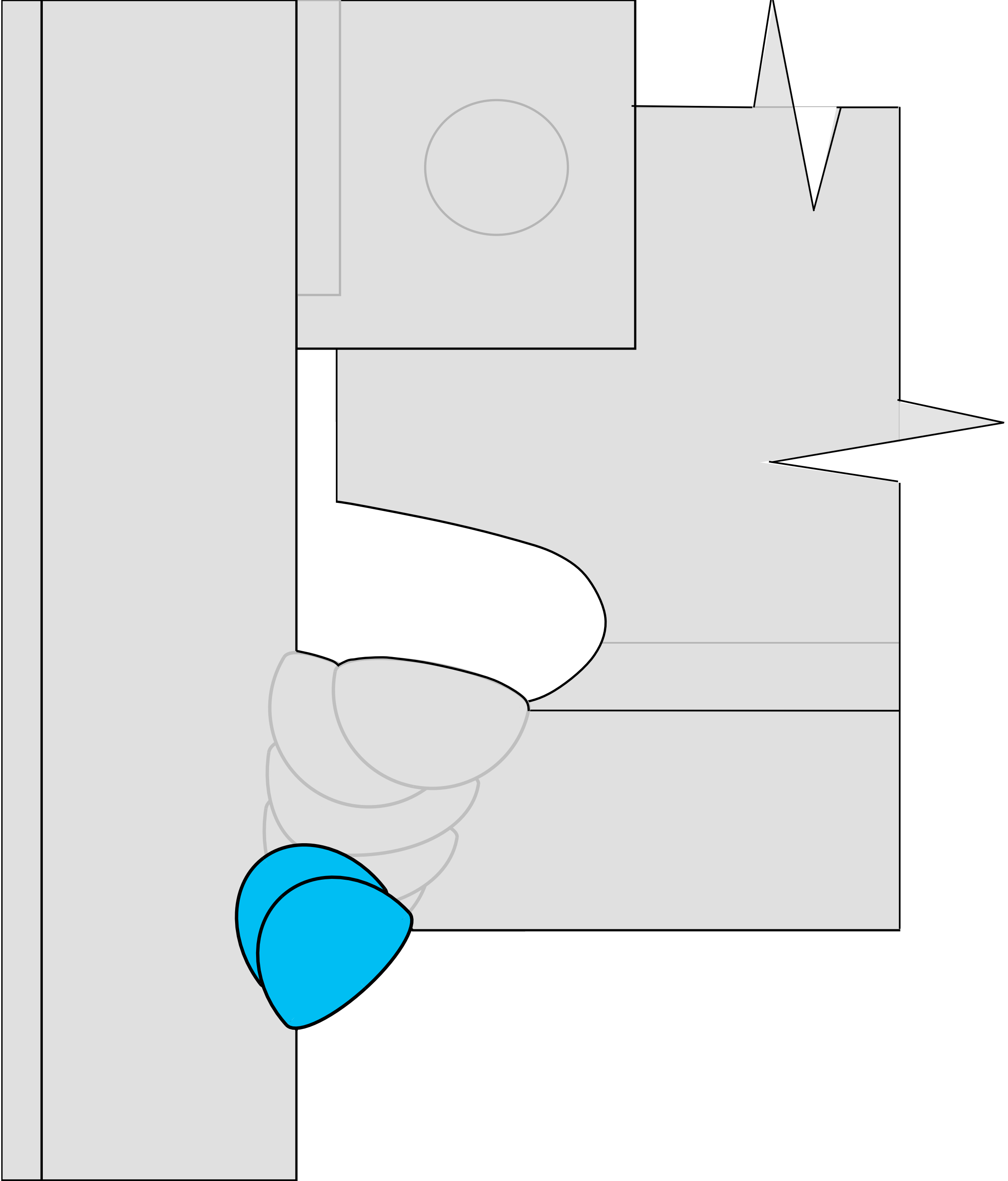
Benefits of Backing Removal

- Eliminates notch created by backing



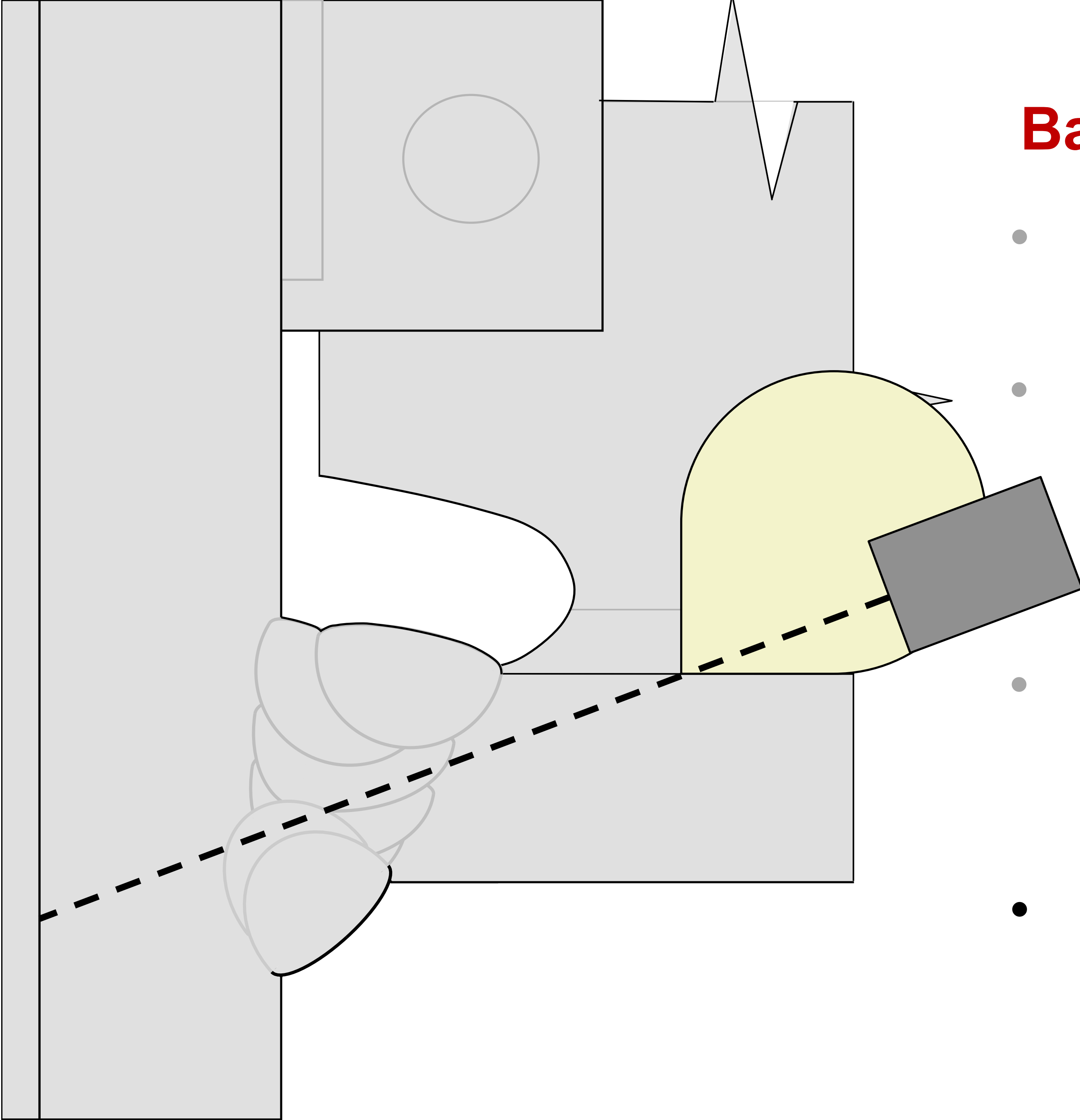
Benefits of Backing Removal

- Eliminates notch created by backing
- Eliminates root discontinuities (cracks, incomplete fusion, slag)



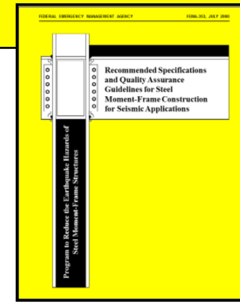
Benefits of Backing Removal

- Eliminates notch created by backing
- Eliminates root discontinuities (cracks, incomplete fusion, slag)
- Contouring fillet "softens" the 90^o intersection



Benefits of Backing Removal

- Eliminates notch created by backing
- Eliminates root discontinuities (cracks, incomplete fusion, slag)
- Contouring fillet “softens” the 90° intersection
- UT inspection results are easier to interpret



4.8 Welding Sequence for Moment Connection of Bottom Beam Flange

When welding the bottom flange of the column flange of welded moment-resisting connections, the following sequence shall be followed:

1. When welding from side A (one side of the beam), the root pass shall begin beyond the center of the joint on Side B, reaching past the beam web (or web plate, for FF connections) through the weld access hold (or opening, for FF connections). After the arc is

AWS D1.8:2009

Seismic Welding Supplement

AWS D1.8:2009 Seismic Welding Supplement

6.7 Removal of Backing and Weld Root Treatment

When fusible (steel) backing is required to be removed, removal shall be by air carbon arc cutting (CAC-A), plasma arc gouging (PAC-G), grinding , chipping , or thermal cutting. The process shall be controlled to minimize errant gouging. After backing removal (both for steel and nonfusible backing), the weld root shall be backgouged to sound metal. Backgouged joints shall be filled with weld metal as necessary, to achieve at

6.14 Bottom Flange Welding Sequence

Complete joint penetration groove welds of beam bottom flanges to column flanges, or to continuity plates, using weld access holes shall be sequenced as follows:

(1) As far as is practicable, starts and stops shall not be directly under the beam web

(2) Each layer shall be complete across the full width

Revisiting W1 Indications

Changes to the Northridge Record

Changes to the Northridge Record

1994		2014
Problem: W1 incipient cracks	→	**Problem:** Incomplete fusion
Solutions: preheat, slow cooling, postheat, peening, welding sequence, process change	→	**Solutions:** root pass thickness; bottom flange welding sequence; backgouging

Changes to the Northridge Record

1994

Assumption:
UT is effective at detecting weld root problems

→

2014

New Practice:

Remove backing

Backgouge to sound metal

Apply contouring fillet

UT with backing removed

Changes to the Northridge Record

1994

Problem Description:
Hundreds of damaged buildings

60-80% damaged connections

2/3rd of inspected buildings were damaged

→

2014

Problem Description:
Widespread original workmanship and inspection issues

Concentrated earthquake damage

1/3rd of inspected buildings were damaged

Recommendations

- Use welded steel SMRFs with confidence
- Use AISC Seismic Specs
- Use AISC CPRP Connections
- Avoid special welding-related job requirements except in special situations

Revisiting W1 Indications