A FITNESS-FOR-PURPOSE EVALUATION OF FRACTURE CRITICAL ELECTRO-SLAG WELDS

FINAL REPORT

A FITNESS-FOR-PURPOSE EVALUATION OF FRACTURE CRITICAL ELECTRO-SLAG WELDS

Final Report

by

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16. A fitness-for-purpose evaluation was performed on the electro-slag flange welds of the West Fremont bridge approach superstructures, per the request of FHWA. This evaluation required gathering knowledge of the material properties, fabrication defects and service loads pertaining to the weldments in question. The serviceability of the weldments was assessed based on the fatigue and fracture performance. It was concluded that the welds have sufficient toughness and fatigue resistance to remain in service with no retrofitting.

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APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
	NOTE: Volumes greater than 1000 L shall be shown in m^3.			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

ACKNOWLEDGEMENTS

The author would like to thank Mr. Robert Hodel, PE, of MEI-Charlton Inc, who was instrumental in overseeing the structural, NDE and materials testing. His attention to detail and accountability are greatly appreciated. Special thanks are warranted to the technical review committee, which was composed of the following members: John Barsom, Stanley Rolfe, David McQuiad, Warren Alexander and Jon Edwards. It is a great honor and privilege to have engineers of this caliber taking time out of their busy schedules to consider this work.

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1.0 INTRODUCTION

1.1 PURPOSE AND OBJECTIVES

This report summarizes the electro-slag weld (ESW) fracture assessment in four of the six bridge superstructures comprising the west approach to the Fremont Bridge, which crosses the Willamette River in Portland, Oregon. In this report, the test area will be referred to as the West Fremont approach complex. This study was undertaken at the request of the Federal Highway Administration (FHWA) (notice numbers N5040.23 dated 2/16/1977, N5040.29 dated 2/23/1977 and HNG-30 dated 6/21/1978) to verify the safety and serviceability of these structures. The objectives were to quantify the fracture resistance of the welds and assess their suitability for continued service. Based on the evaluation, guidance was developed to either leave the superstructures in service as fabricated or to retrofit them, and recommend future inspection criteria.

1.2 BACKGROUND

A similar evaluation was conducted by Lovejoy, and was published in 2002. The evaluation provided an identical assessment on Oregon's other bridge superstructure containing ESW, on the Willamette River crossing of I-205 at West Linn, Oregon. As with the West Linn Bridge, the West Fremont approach structures were initially examined in 1978 with limited non-destructive testing (NDT) and materials testing. A more rigorous examination was begun in 2006 and completed in 2008 as will be described later in this report.

MEI-Charlton Inc. of Portland, Oregon was contracted by the Oregon Department of to perform the following tasks for this evaluation:

1) Weld identification and location
2) In-service structural load testing
3) Material core removal
4) Materials testing
5) Non-destructive testing of in-service weldments.

The detailed description and results summary of each of these tasks is provided in the contractor report (*MEI-Charlton Inc. 2007*).

1.3 STRUCTURE DESCRIPTION

The West Fremont approach complex connects US30 and I-405 to the Fremont Bridge, which extends over the Willamette River in Portland, Oregon. As shown in Figure 1.1, the approach complex is comprised of six bridge superstructures, which were fabricated in 1970. Four of the six superstructures contain ESW in fracture critical applications. The other two structures do not have fracture critical girders and are believed to employ only submerged arc welds (SAW) for the complete joint penetration (CJP) flange welds.

Figure 1.1: West Fremont approach complex.

1.3.1 Description of bridge superstructures under evaluation

The following section summarizes the four bridge superstructures that were evaluated as part of this study. The structures are defined by the bridge identification number, the orientation (line) and unit numbers.

Bridge #09268A (SW Line, Units 1, 2 and 3) connects I-405 NB to US30 WB. The superstructure is composed of three units. Unit 1 has three spans which are continuous at the interior supports (Piers 6 and 7) and are simply supported at the ends (Figure 1.2). The superstructure is composed of two trapezoidal steel box girders connected by rolled steel wide flange (WF) sections crossbeams and a reinforced concrete deck continuously composite with the box girder and crossbeam top flanges (Figure 1.3).

Units 2 and 3 have similar configurations and are connected with an expansion joint near Pier 3 (Figure 1.4). The structural details of Units 2 and 3 are shown in Figures 1.5 and 1.6 respectively. All girder butt welds were fabricated with ESW with the exception of two top flange welds, which appear to have been made with SAW.

Bridge #09268B (WS Line, Units 1, 2 and 3) connects US30 EB to I-405 SB. The superstructure is composed of three units. Unit 1 has two spans which are continuous at the interior support (Pier 2) and are simply supported at the ends (Figure 1.7). The superstructure is composed of two trapezoidal steel box girders connected by rolled steel WF sections for crossbeams and a reinforced concrete deck, which is composite with the box girder and crossbeam top flanges (Figure 1.8 and 1.9).

Units 2 and 3 have similar configurations with the exception of the three spans on Unit 3 (Figure 1.10). The structural details of Unit 3 are shown in Figure 1.11. All girder butt welds were fabricated with ESW.

Bridge # 09268E (WE Line, Unit 1) connects US30 EB to the west side of the Fremont Bridge (Bridge # 02529). It is the lower deck of a two deck structure as seen in Figure 1.12. The structure has four spans, which are continuous at the interior supports (Piers 3, 4 and 5) and are simply supported at the ends. Construction is similar to the previous bridges with the structural details shown in Figure 1.13. According to the MEI-Charlton Inc. contractor report, 33 of the 81 flange butt welds that were located were found to be made with SAW and the remaining were made with ESW (*2007*).

Bridge # 09268W (EW Line, Unit 1) connects the west side of the Fremont Bridge (Bridge # 02529) to US30 WB. It is the upper deck of a two deck structure as seen in Figure 1.14. The structure has four spans, which are continuous at the interior supports (Piers 3, 4 and 5) and are simply supported at the ends. Construction is similar to the previous bridges with the structural details shown in Figure 1.15. According to the MEI-Charlton Inc. contractor report, four of the 86 flange butt welds that located were found to be made with SAW and the remaining were made with ESW (*2007*).

1.4 FABRICATION REQUIREMENTS

The steel box girders were fabricated under the requirements of AWS D2.0-69 (Welded Highway and Railway Bridges). For weld procedure qualification, Section E102 (Impact Properties) of the specification was modified by raising the minimum weld metal average Charpy V-Notch (CVN) energy at 0 ^{o}F from 15 to 20 ft-lbf and also requiring impact testing in the heat affected zone (HAZ). The HAZ was tested at quarter depth and was required to have the same minimum impact energy as the ASTM A 588 base metal (15 ft-lbf at 30 ^{o}F). All tension and stress reversal welds were inspected with magnetic particle testing (MT), ultrasonic testing (UT) and radiographic testing (RT) in accordance with Appendices C and B of this report. Final acceptance was based on MT and RT only.

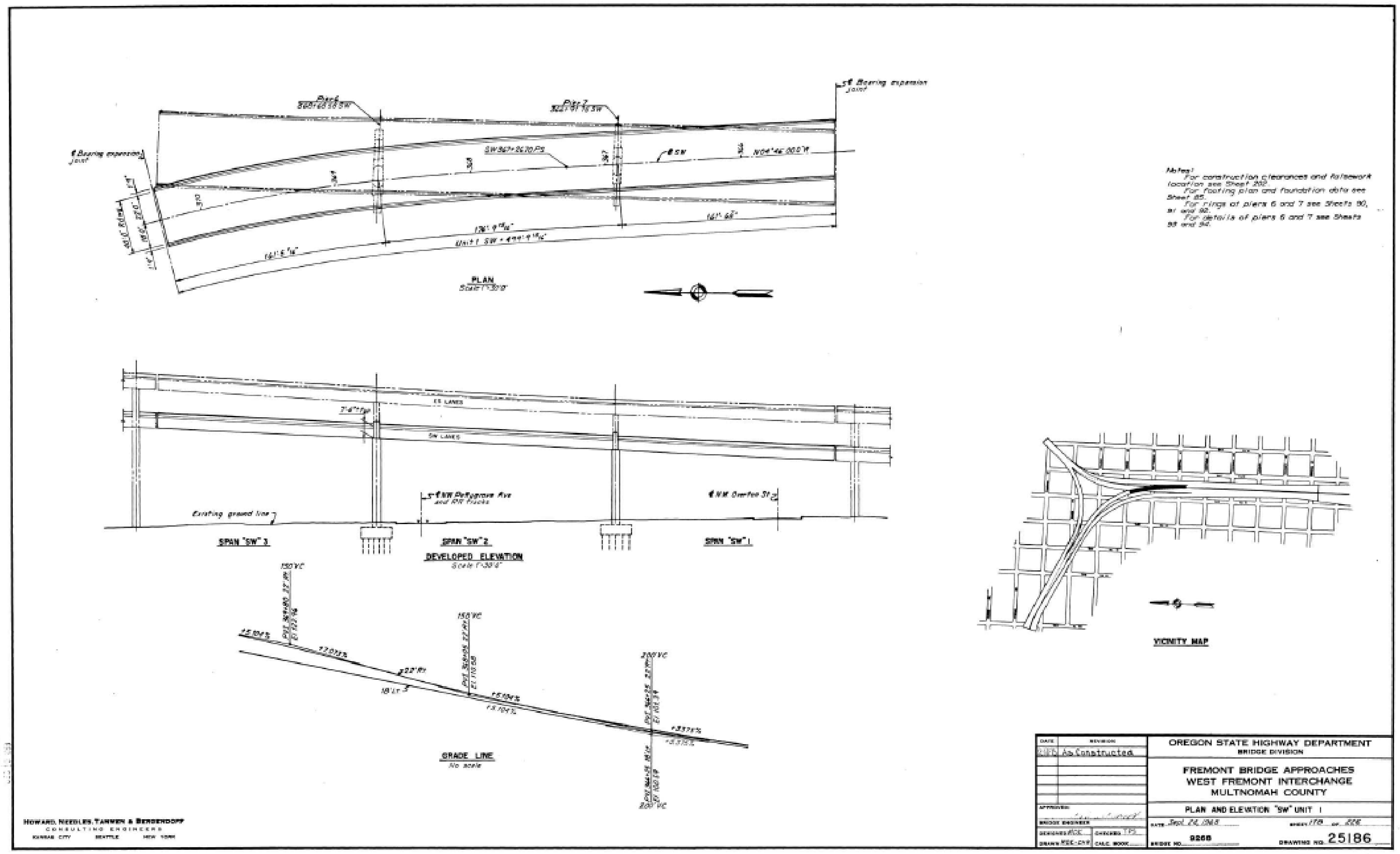


Figure 1.2: Plan and elevation views of SW Unit 1 Bridge superstructure 09268A.

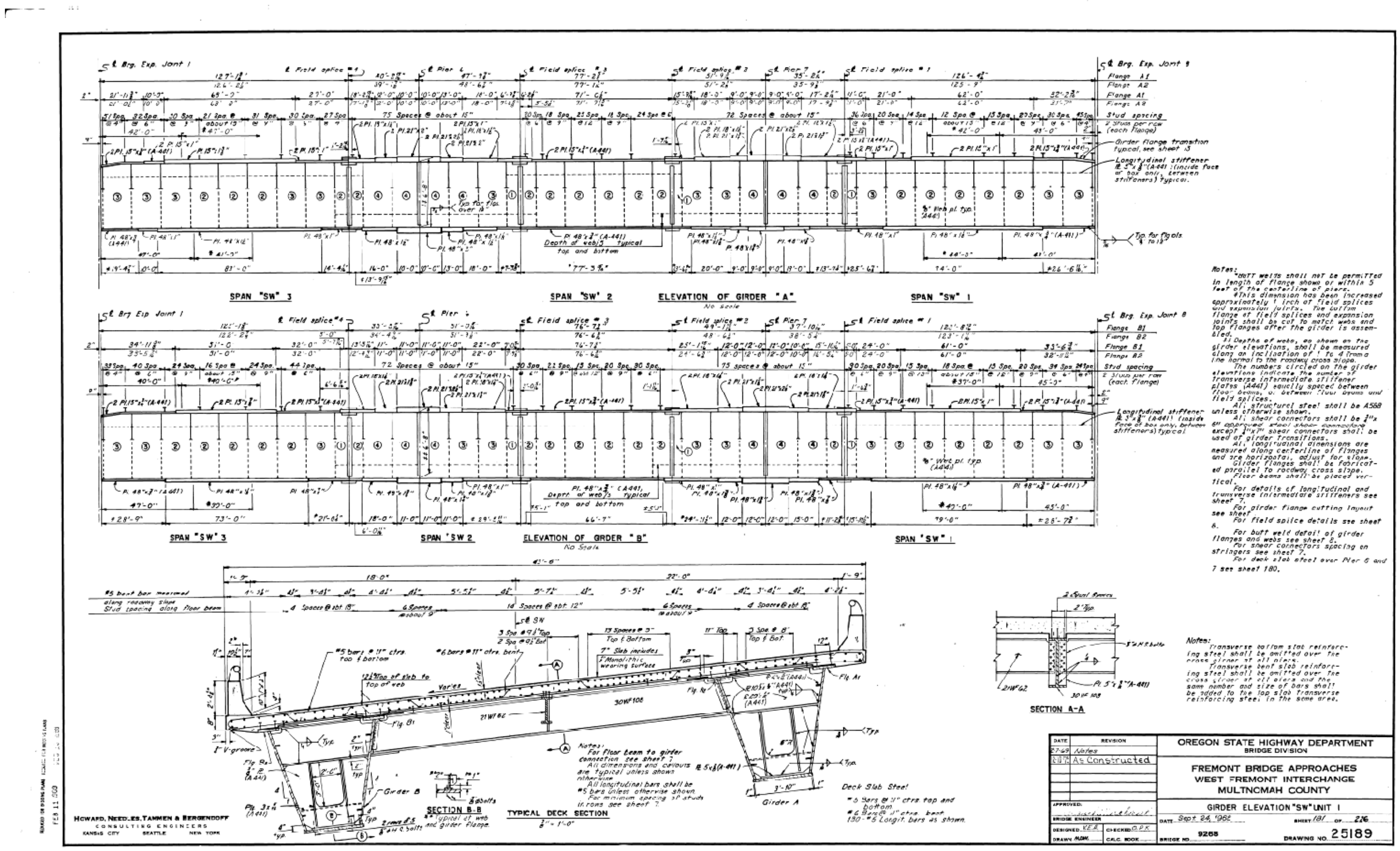


Figure 1.3: Structural details of SW Unit 1 Bridge superstructure 09268A.

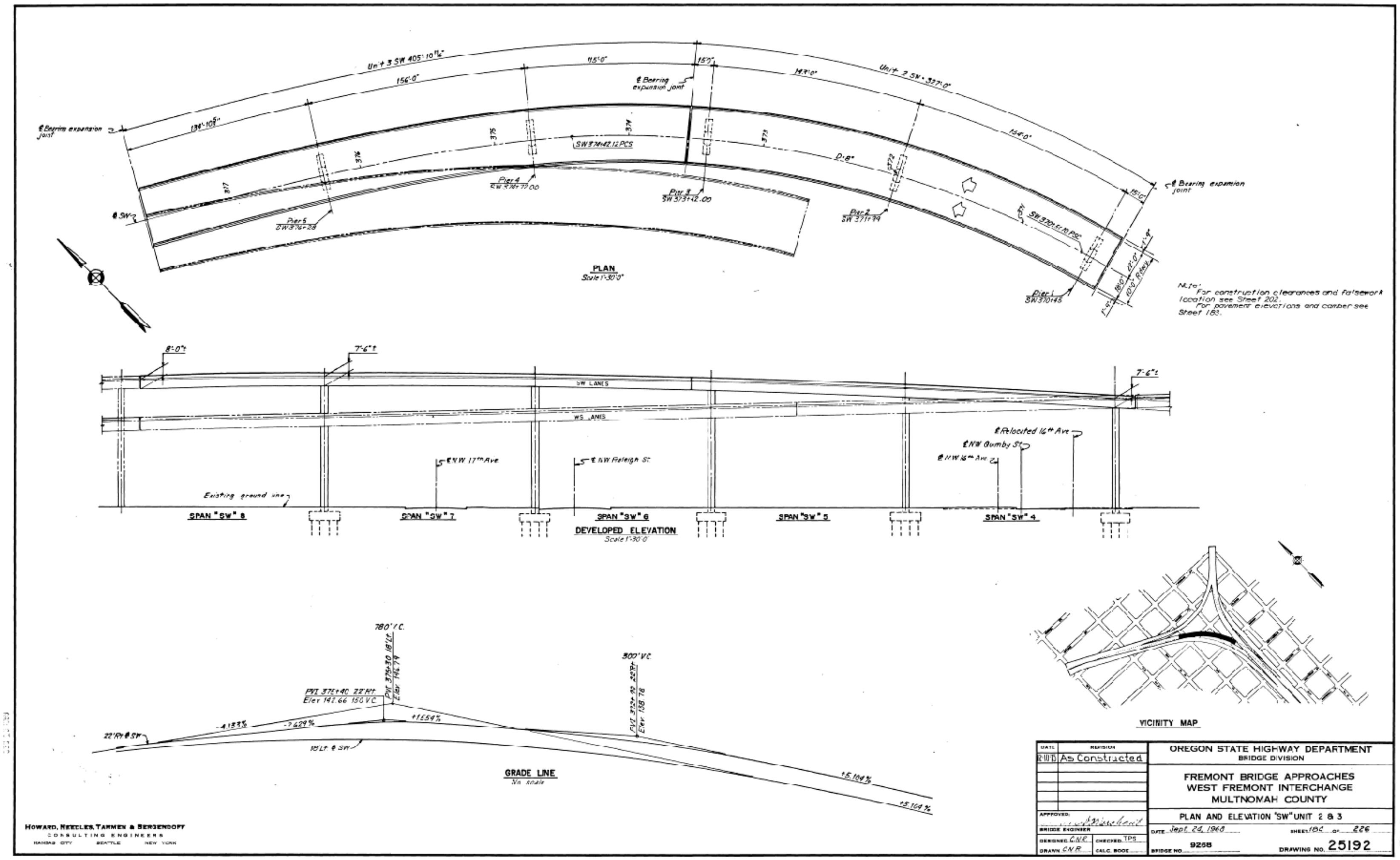


Figure 1.4: Plan and elevation views of SW Unit 2 and 3 Bridge superstructure 09268A.

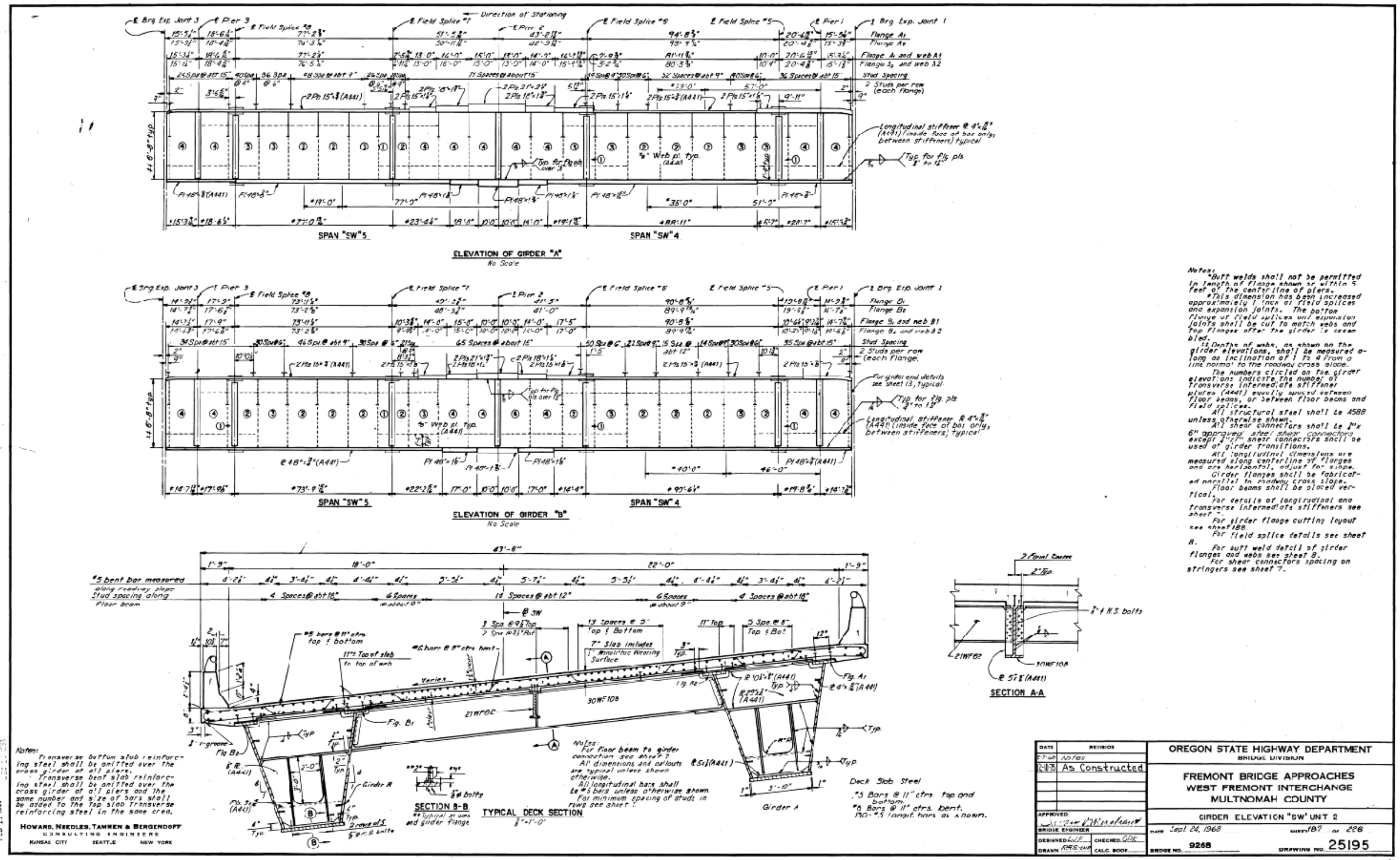


Figure 1.5 Structural details of SW Unit 2 Bridge superstructure 09268A.

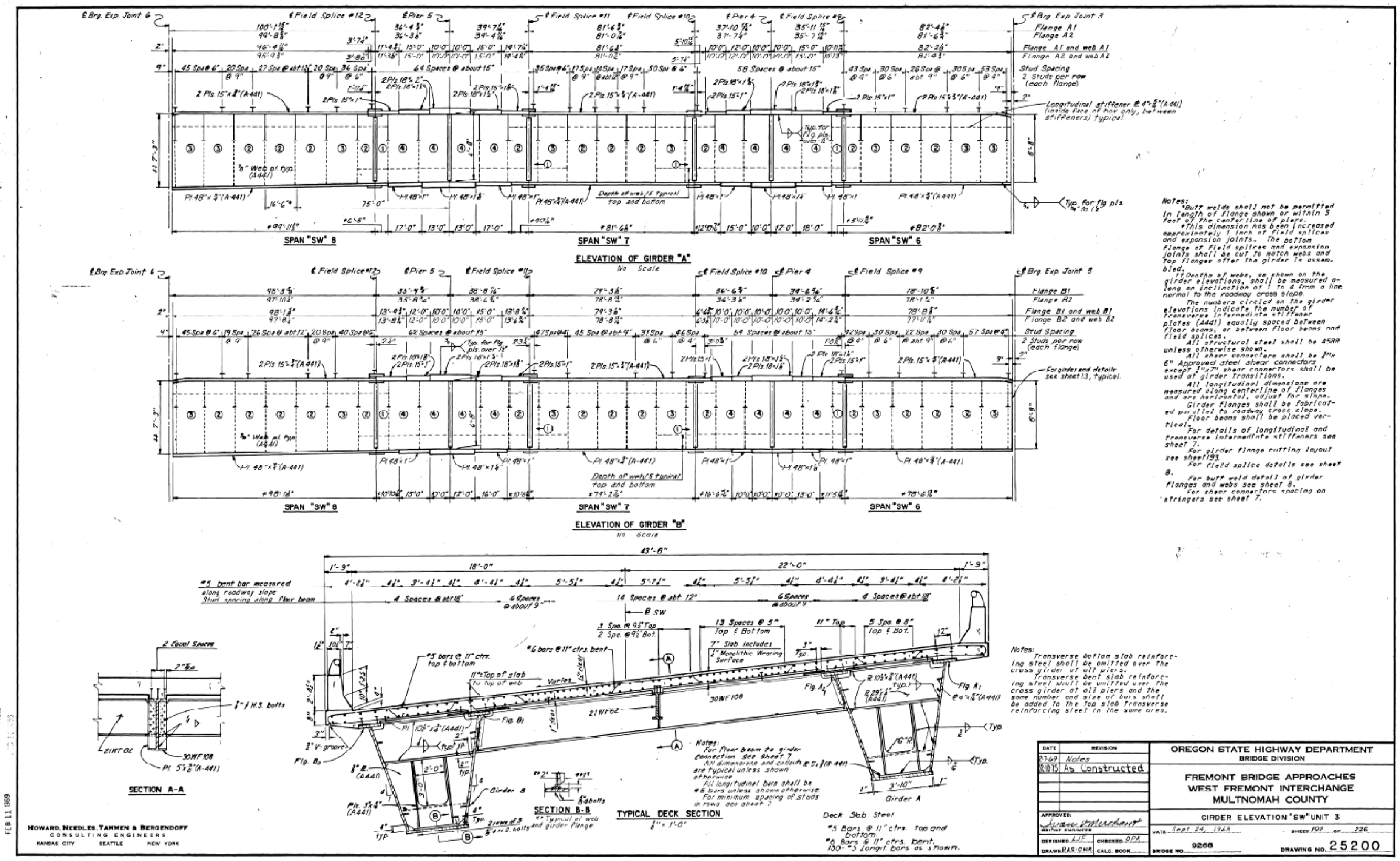


Figure 1.6: Structural details of SW Unit 3 Bridge superstructure 09268A.

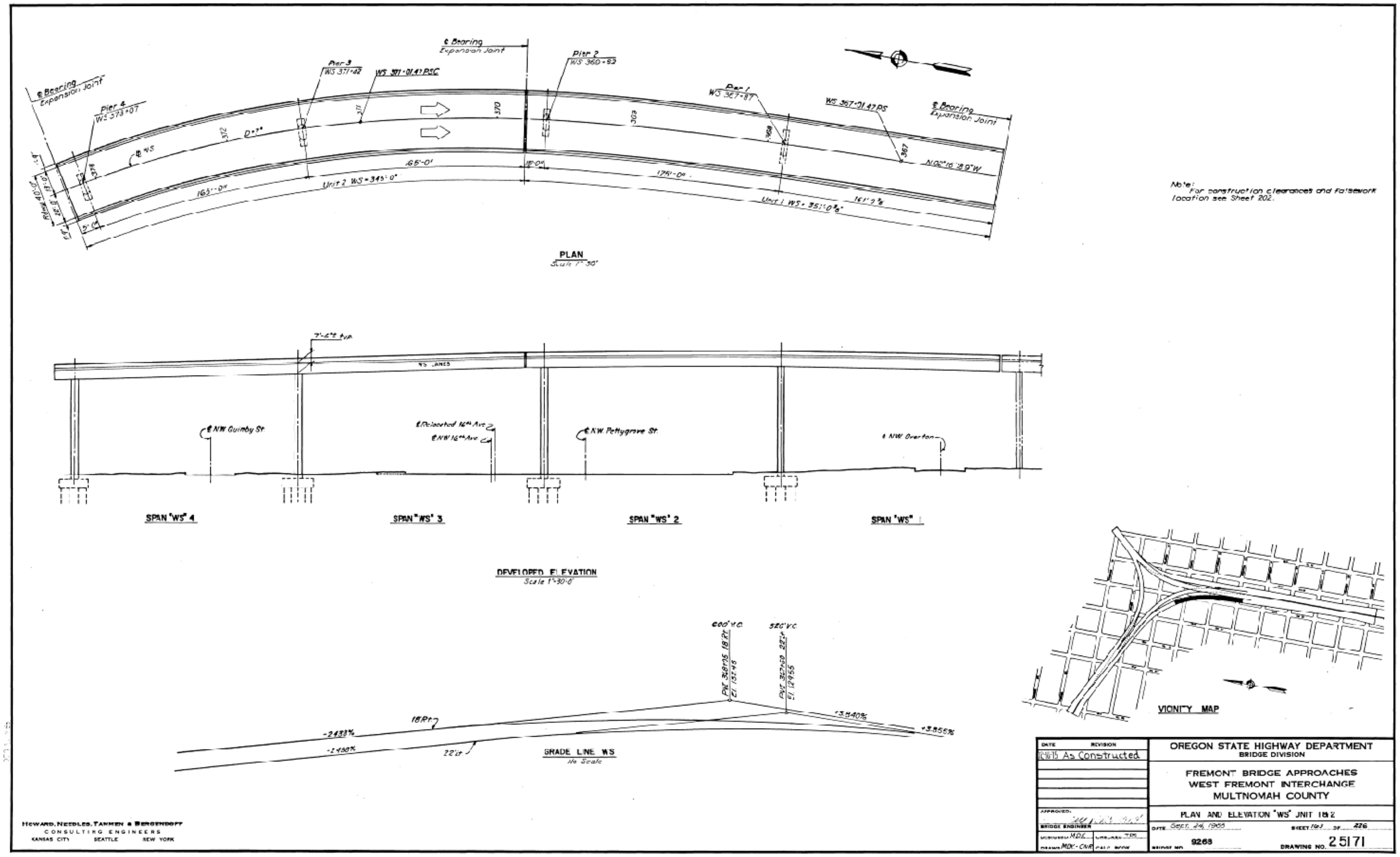


Figure 1.7: Plan and elevation views of WS Units 1 and 2 Bridge superstructure 09268B.

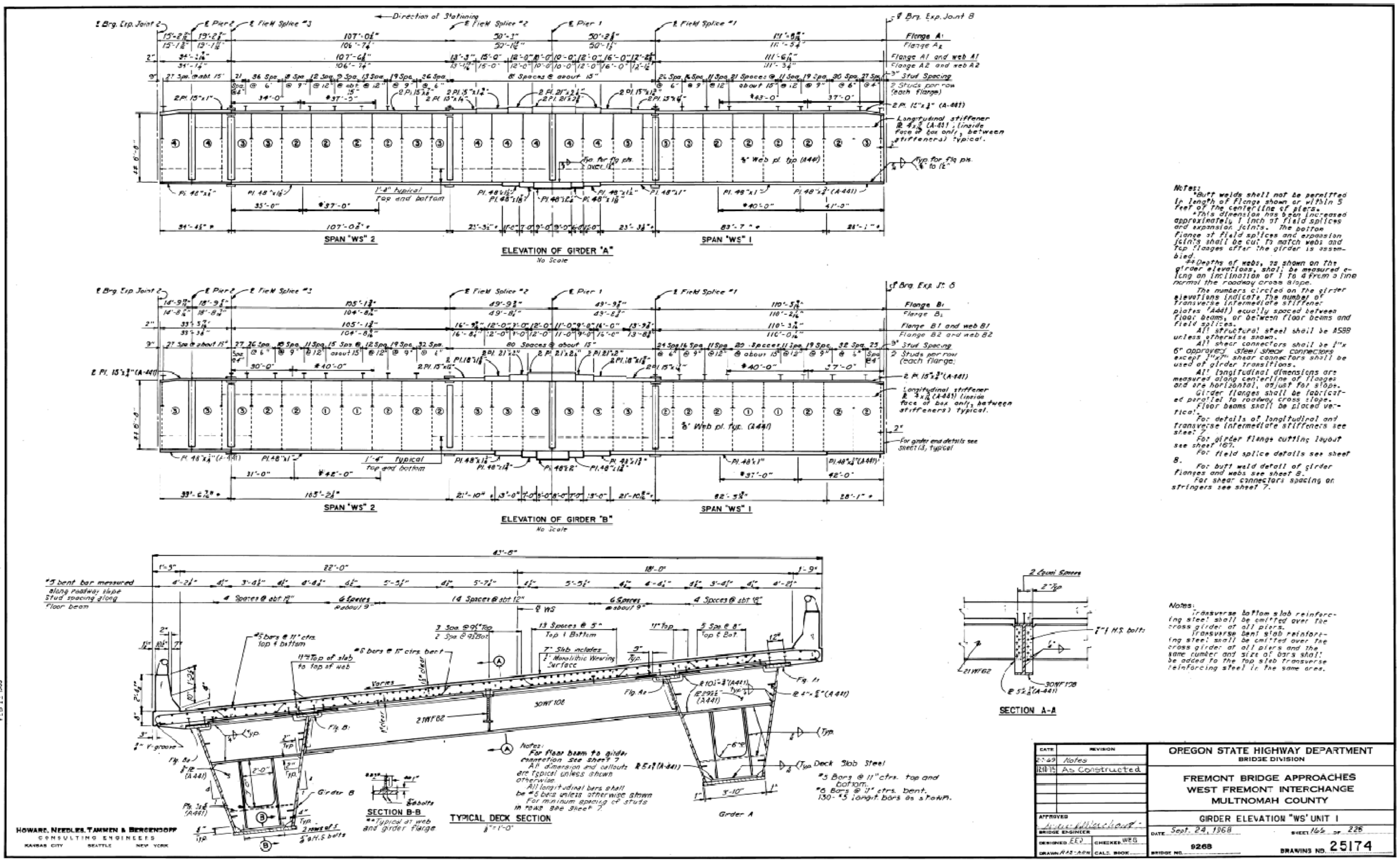


Figure 1.8: Structural details of WS Unit 1 Bridge superstructure 09268B.

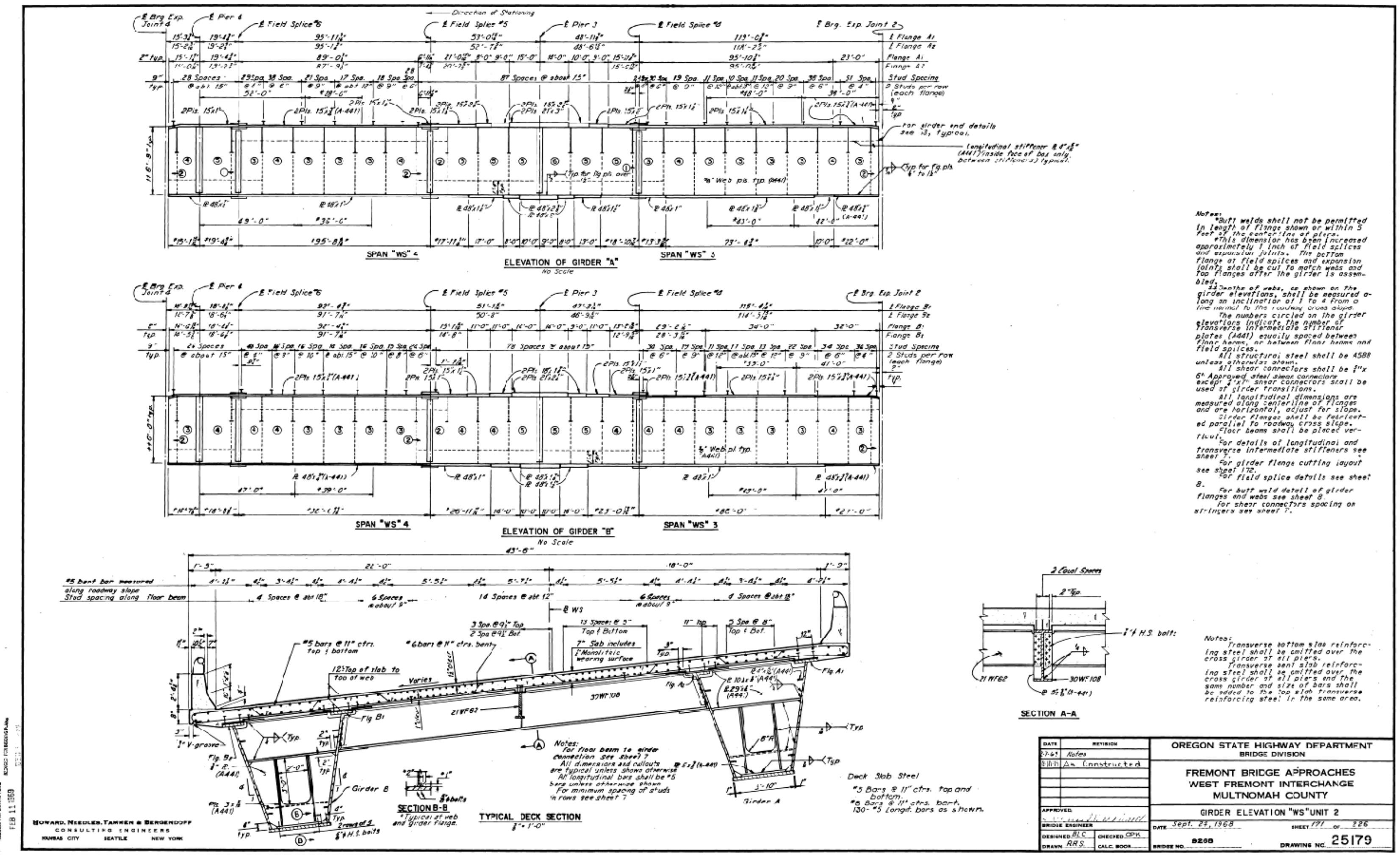


Figure 1.9: Structural details of WS Unit 2 Bridge superstructure 09268B.

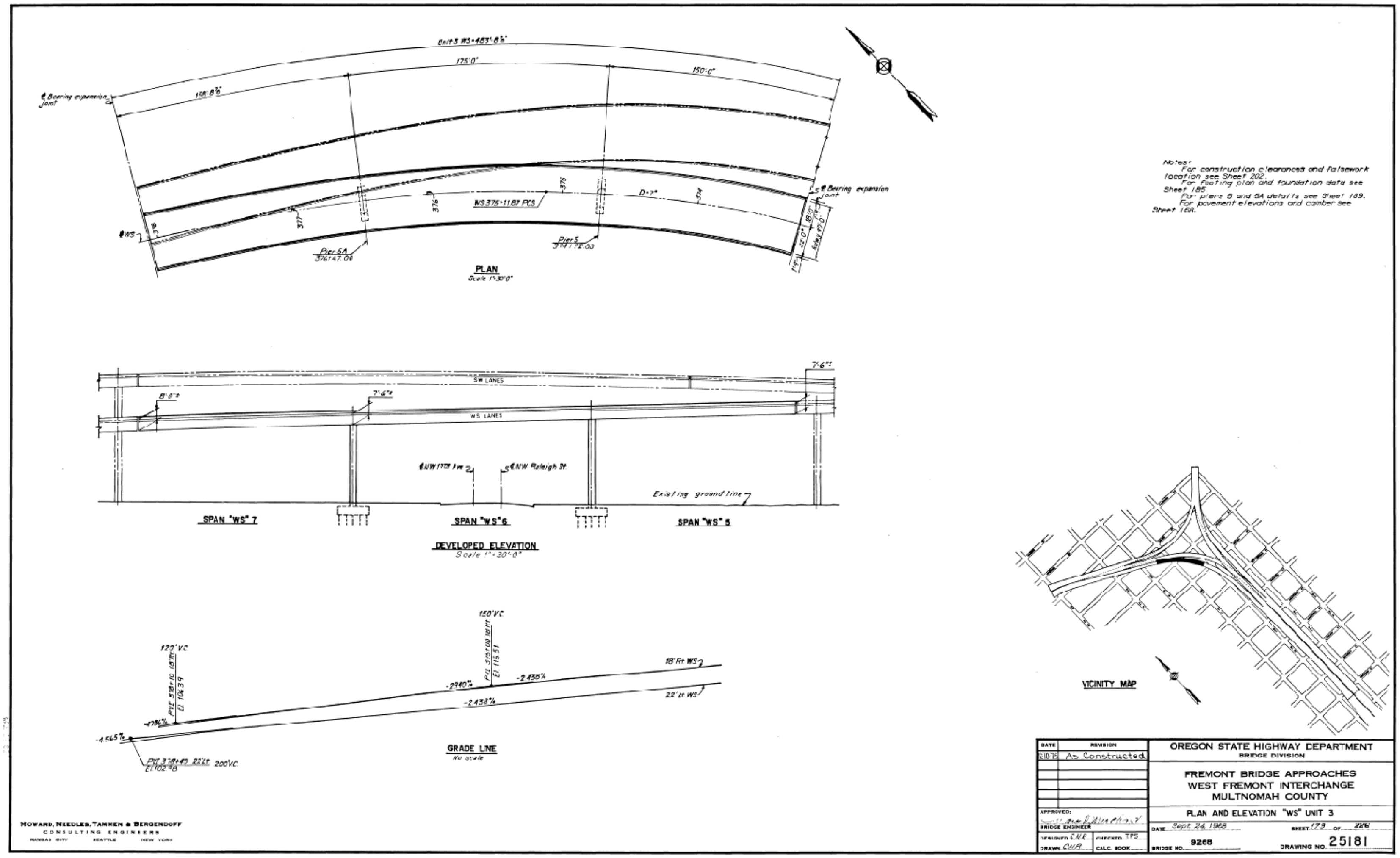


Figure 1.10: Plan and elevation views of WS Unit 3 Bridge superstructure 09268B.

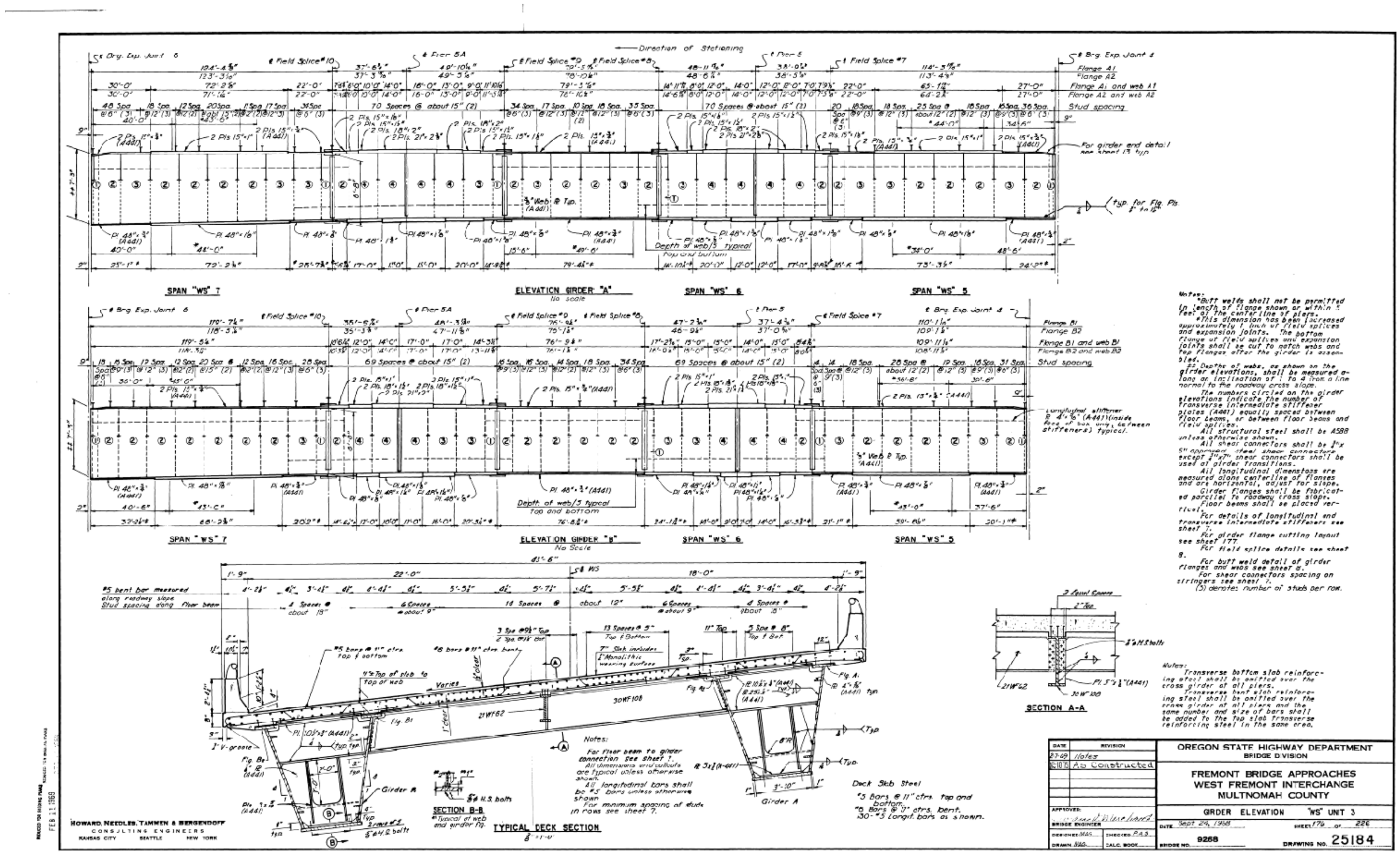


Figure 1.11: Structural details of WS Unit 3 Bridge superstructure 09268B.

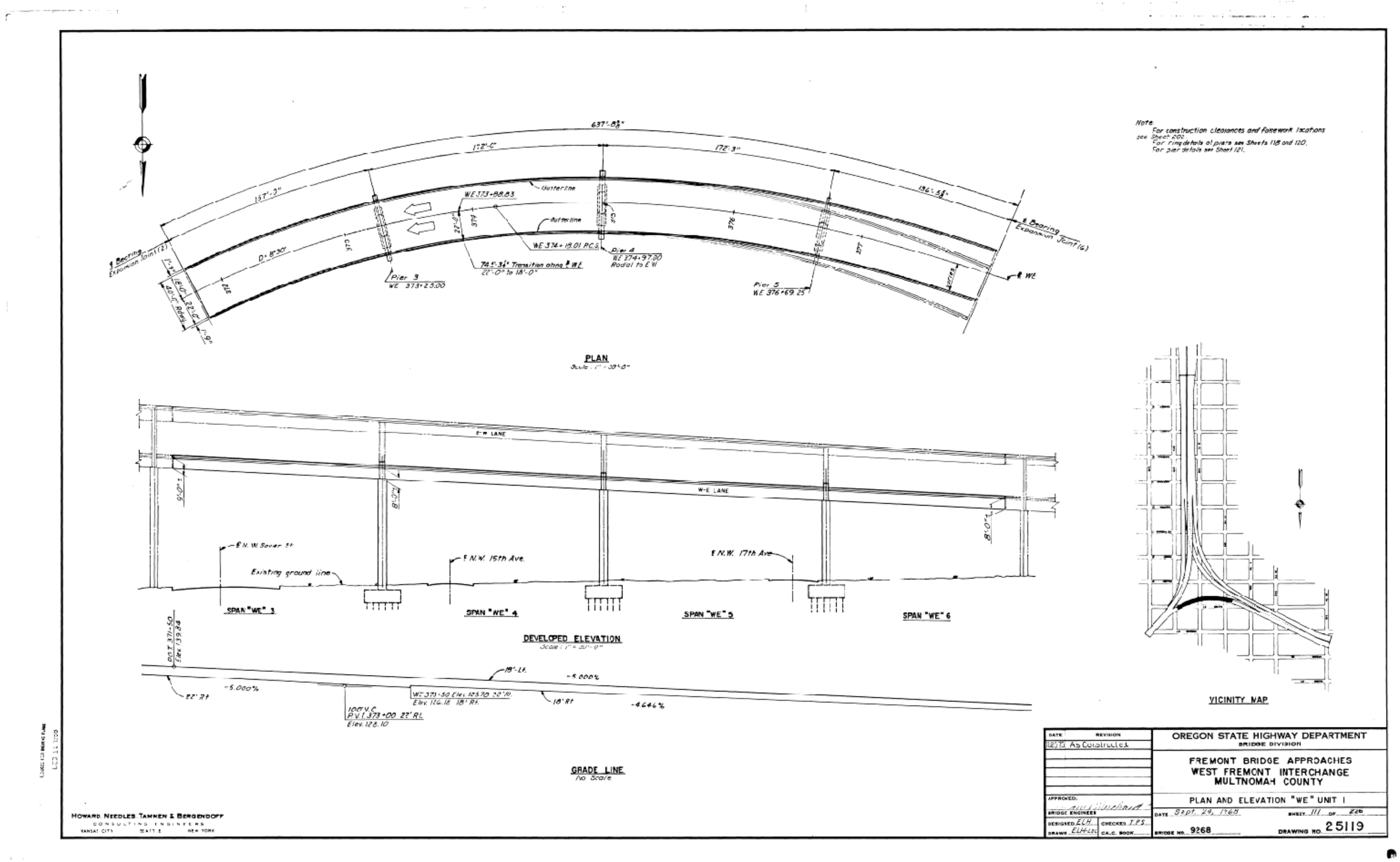


Figure 1.12: Plan and elevation views of WE Unit 1 Bridge superstructure 09268E.

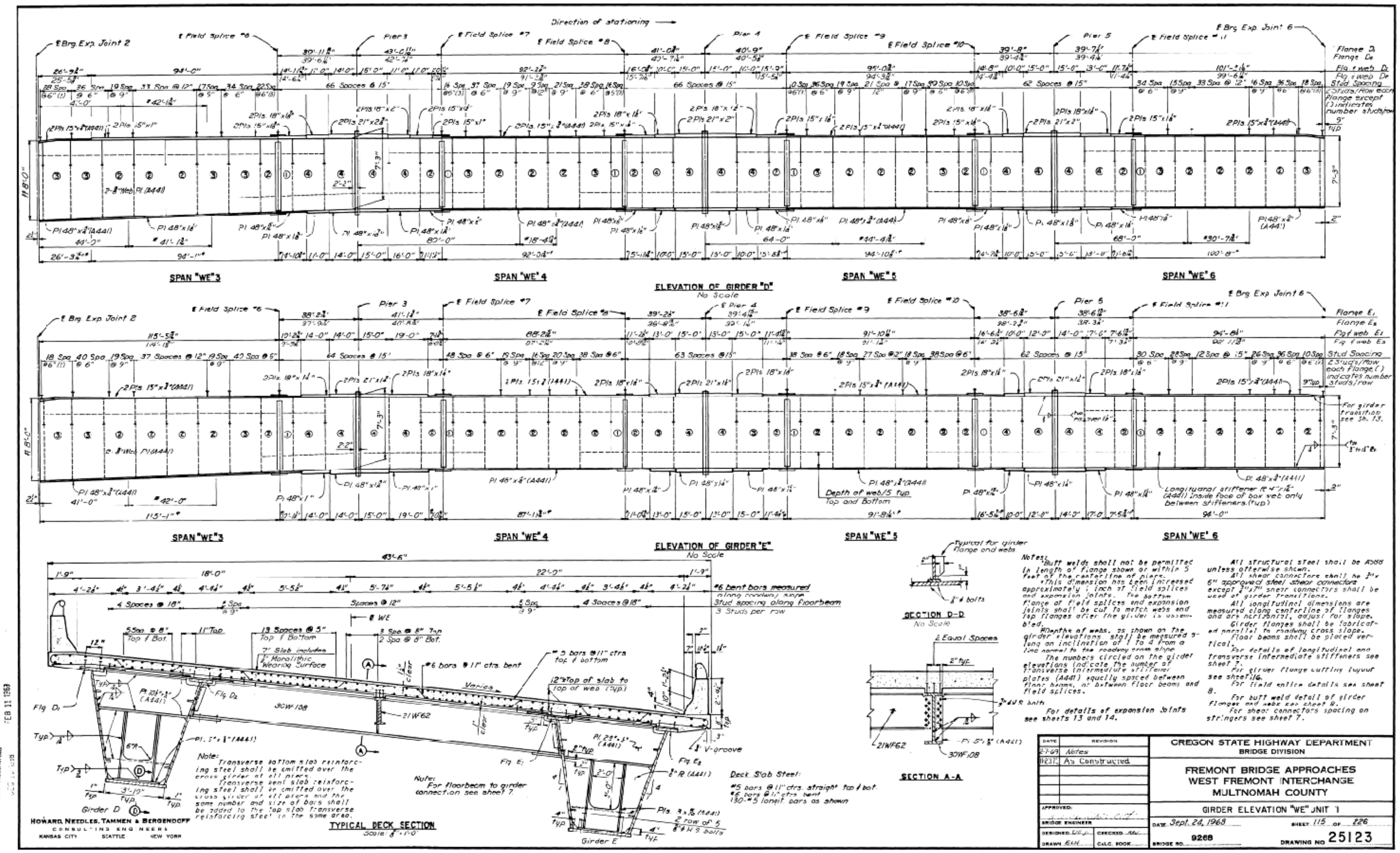


Figure 1.13: Structural details of WE Unit 1 Bridge superstructure 09268E.

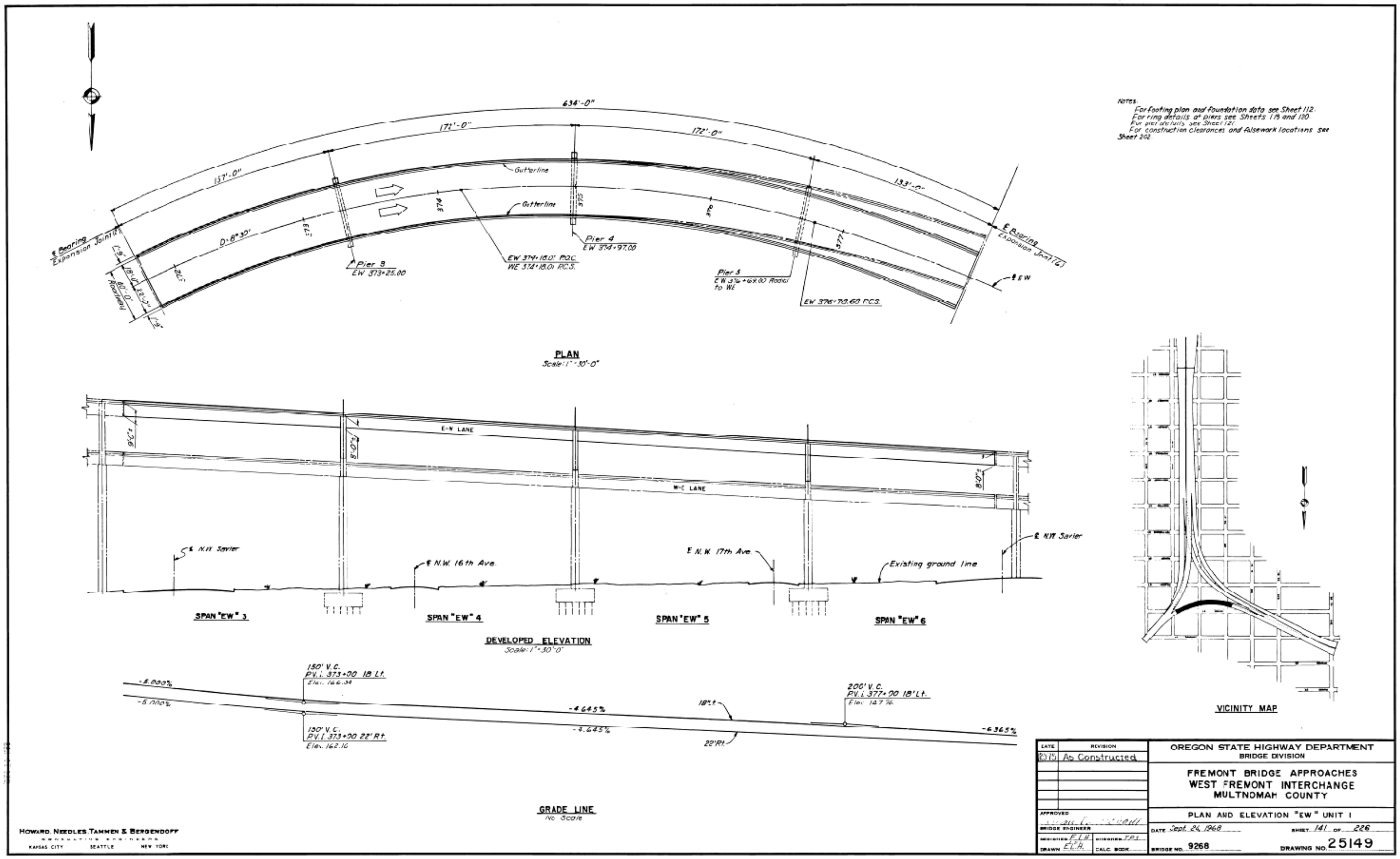


Figure 1.14: Plan and elevation views of EW Unit 1 Bridge superstructure 09268W.

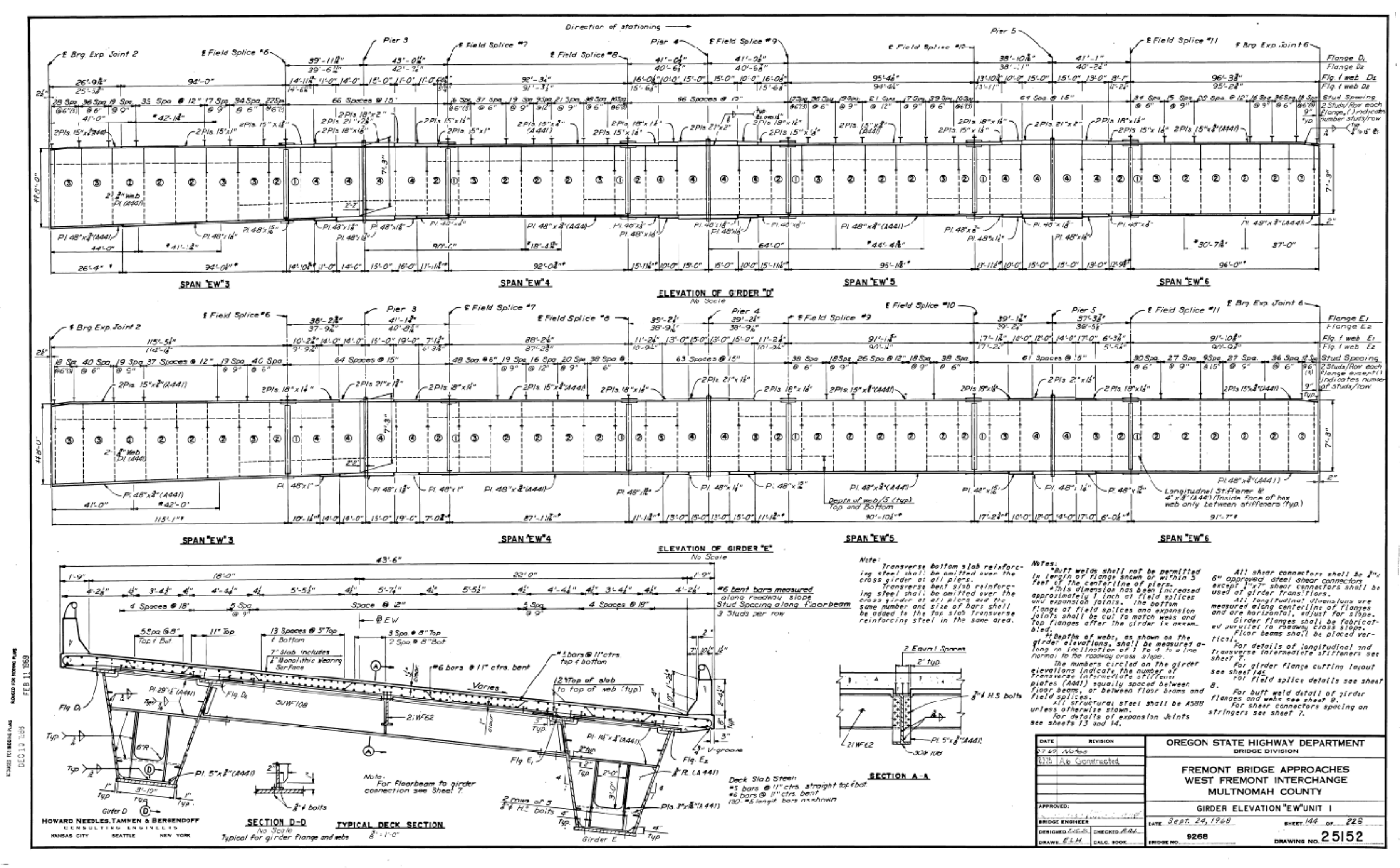


Figure 1.15: Structural details of EW Unit 1 Bridge superstructure 09268W

2.0 WELD IDENTIFICATION AND LOCATION

2.1 DESCRIPTION OF WORK

MEI-Charlton Inc. was tasked with identifying and locating all flange butt welds in the four bridge structures under evaluation. Using ODOT design drawings as a map, all constructed complete joint penetration (CJP) welds in both the upper and lower flanges of the trapezoidal box girders were identified and examined to determine if the electro-slag weld (ESW) process was employed. The weld location and adjacent flange thicknesses were measured and then compared with the design drawings. MEI-Charlton Inc. published a detailed report of this work in 2007.

2.2 WELD IDENTIFICATION

Welds were located using visual testing (VT), augmented with directional light to highlight the surface profile. This method proved very effective and efficient for weld identification. Many of the welds were not in the exact locations shown on the design drawings. ESW has a very unique surface profile and can usually be easily differentiated from submerged arc welds (SAW) or shielded metal arc welds (SMAW). Some of the top flange welds were obscured by the sheet metal forming the underside of the deck. Thus, even though great care was taken in identification, some top flange welds were not identified.

The flange thickness on each side of the weld was measured using ultrasonic testing (UT). The interior width of each flange was also measured and recorded. Each weld was assigned an identification code and labeled as shown in the following example.

Example weld identification code: SW6-A-T2-4

In the example shown above, the code represents the following:

- 'SW6' refers to the Line and span. The first two letters refer to the line number shown in the design drawings, in this case the Southwest or SW line. The number in the third space identifies the span number, as shown on the design drawings.
- 'A' refers to the Girder. Girders are designated as shown in the design drawings, either A or B in the case of the SW and WS lines, or D or E in the case of the EW and WE lines.
- 'T2' refers to the Flanges. Flanges are designated as B for bottom flanges, and T1 or T2 for top flanges, where 1 and 2 are as shown on the design drawings.
- '4' refers to the Weld number. The final digit identifies the sequential number of the weld in the flange.

Thus the example code designates the fourth weld in the sixth span of the SW line in the A girder and is located in the T2 upper flange.

2.3 WELD TYPE AND FLANGE THICKNESS

All flange welds were identified and located including both tension and compression areas. As noted in the MEI-Charlton Inc. contractor report, many of the welds were not in the locations shown by the design drawings (*2007*). In addition, the design drawings indicate thickness transitions at many weld locations, but thicknesses sometimes remained constant, with a thickness greater than or equal to the larger specified. The range of flange thicknesses and the number of welds for each bridge are summarized in Table 2.1.

Table 2.1 Summary of flange thickness range and weld type.

Bridge Structure #	Line	Flange	t_{max} (inch)	t_{min} (inch)	# of welds	Type* (# - note)
09268A	SW	Top	2.55	0.78	130	128-1 2-2
		Bottom	2.20	0.80	47	47-1
09268B	WS	Top	3.08	0.78	118	118-1
		Bottom	2.47	0.80	52	52-1
09268E	WE	Top	2.50	0.78	55	33-1, 22-3
		Bottom	1.95	0.78	26	15-1, 11-3
09268W	EW	Top	2.43	0.78	58	54-1, 4-3
		bottom	1.98	0.80	28	28-1

*note 1 ESW confirmed, note 2 SAW confirmed, note 3 ESW with SMAW repair or SAW.

2.4 FRACTURE CRITICAL FLANGE WELDS

The moment inflection points for the continuous spans were estimated at 0.25*l*, where *l* is the span length. A total of 261 flange butt welds were identified in locations of net tension or stress reversal. Of those 261 welds, 81 were located in a bottom flange and 180 in top flanges. ODOT has historically used a strict, literal interpretation of fracture critical (i.e., tension or stress reversal components) with no load path redundancy. The bottom flange welds in tension or stress reversal applications are clearly fracture critical. The top flange welds are in situations that offer both internal and structural redundancy. The potential benefit of these redundancies has not been formally analyzed to date. A list of all fracture critical welds can be found in Appendix A.

3.0 LOAD ANALYSIS

The function of the load analysis is to characterize the fracture potential and crack growth driving forces. For this, the maximum stress is required to calculate critical crack size, and the stress range from cyclic loading is needed to calculate potential fatigue crack growth. The maximum stress in the flange welds is a combination of dead load, live and wind load, thermally induced load, possible seismically induced load, and residual stresses from the fabrication process. The dead and live loads are reasonably easy to calculate and vary over the length of the girders in a smooth or gradual manner. Thermally induced load is similar. Fabrication and construction induced stresses, especially the high residuals and steep spatial gradients associated with ESW, are more difficult to quantify.

This analysis considers the loads and stresses in the box girder flange plates. Design load stresses are not presented in this study, but were included in the first study performed on the West Linn Bridge (*Lovejoy 2002*). In lieu of performing a detailed analysis to quantify the dead load, live load, thermal and seismic stresses, researchers assumed that the maximum combined stresses for expected dead, live and thermal sources do not exceed 55% of the base metal's 50 ksi yield stress, or 27.5 ksi. This was supported by the detailed analyses performed by Lovejoy (*2002*). The two bridge structures were designed and fabricated under the same codes at nearly the same time.

The in-service live and thermal stress responses are quantified experimentally in this project. Since no formal analysis is presented on the design loads, the experimental work is much more detailed and rigorous when compared to the experimental work performed by Lovejoy (*2002*).

3.1 DESIGN LOADS

The original design drawings specify that the West Freemont superstructures were designed using the AASHO Allowable Stress Design (ASD) with a design load of HS20-44 and military loading. The maximum allowable stress on the flange butt welds is 27.5 ksi including dead load, live load and thermal loads. To date, load rating using the AASHTO Load Resistance Factor Rating (LRFR) code has not been performed on these structures. *When such an analysis is performed, using the actual flange plate thicknesses measured in these structures, as opposed to the design flange thicknesses, will yield higher load rating factors.*

Each bridge superstructure that was investigated had multiple spans that were continuous over interior supports and were free to rotate at end supports. Some of the structures had interior expansion joints on interior spans. Another key feature affecting the flange stresses was the composite concrete deck, which was rigidly attached to the top flanges.

3.2 LIVE AND THERMAL STRESS MEASUREMENTS

One interior span on each of the four subject bridges was instrumented with uniaxial resistance strain gages. The strain gages were attached to the top and bottom flanges measuring longitudinal (bending and axial) strain. One gage was attached to each top flange and two gages attached to each bottom flange as shown in Figure 3.1. This arrangement was used to capture bending at both the horizontal and vertical axes of the girder, axial and torsional strains of the box girder.

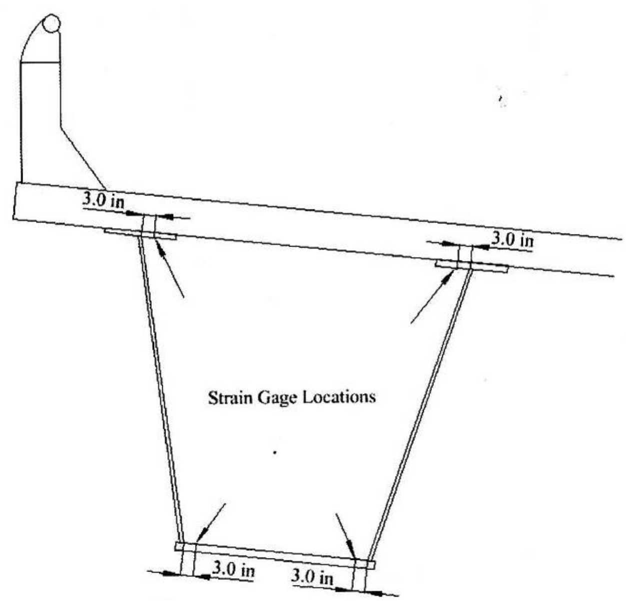


Figure 3.1: Cross section of box girder with strain gage locations shown.

For each span tested, both girders were instrumented at two different locations along the span. Both positive and negative moment sections of the girders were instrumented with strain gages. The specific locations for each bridge are shown in Figures 3.2 through 3.5.

MEI-Charlton Inc. was contracted to install the instruments and collect the data. Details of the installation and testing can be found in the MEI-Charlton Inc. report (*2007*). Data were collected at each section, recording both girders and both sections simultaneously and continuously for seven days. Currently these bridges have a maximum Average Daily Traffic (ADT) count of 112,300 vehicles per day, with 6.7 % truck traffic or 7,524 trucks per day. Seven days of data at this level of ADT produces a very good estimate of the loading spectrum.

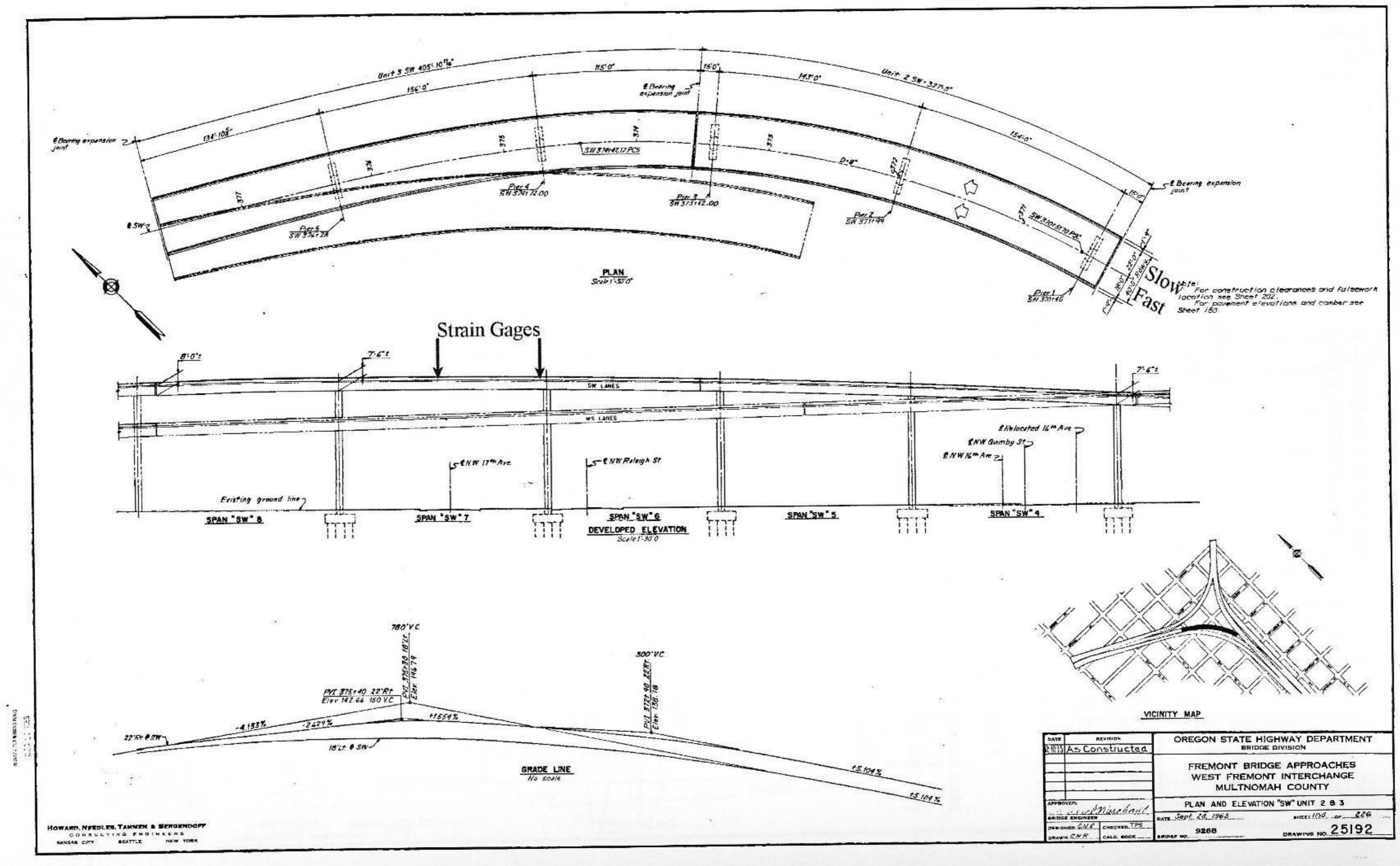


Figure 3.2: Instrumented sections for Bridge superstructure 09268A (SW line).

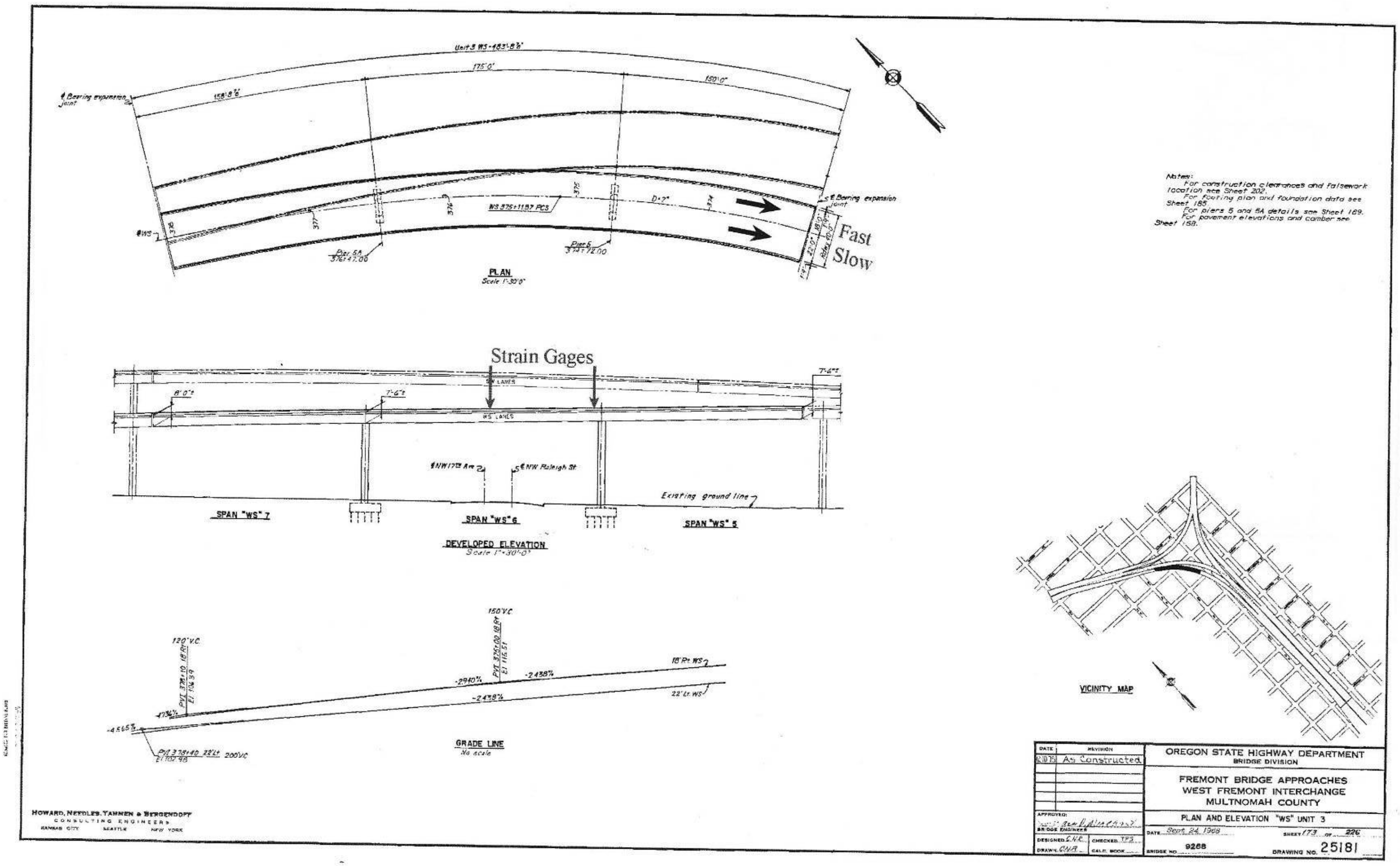


Figure 3.3: Instrumented sections for Bridge superstructure 09268B (WS line).

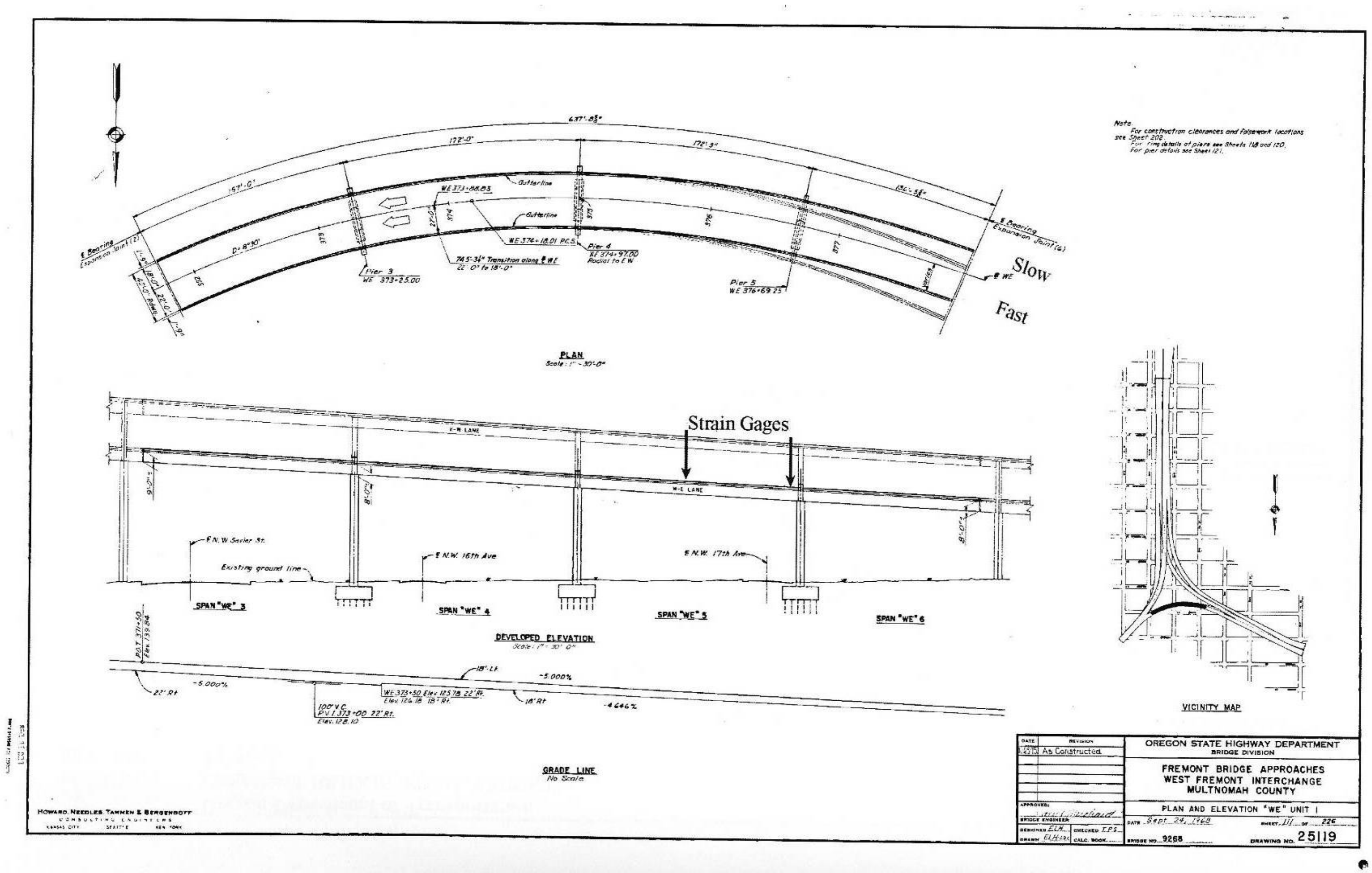


Figure 3.4: Instrumented sections for Bridge superstructure 09268E (WE line).

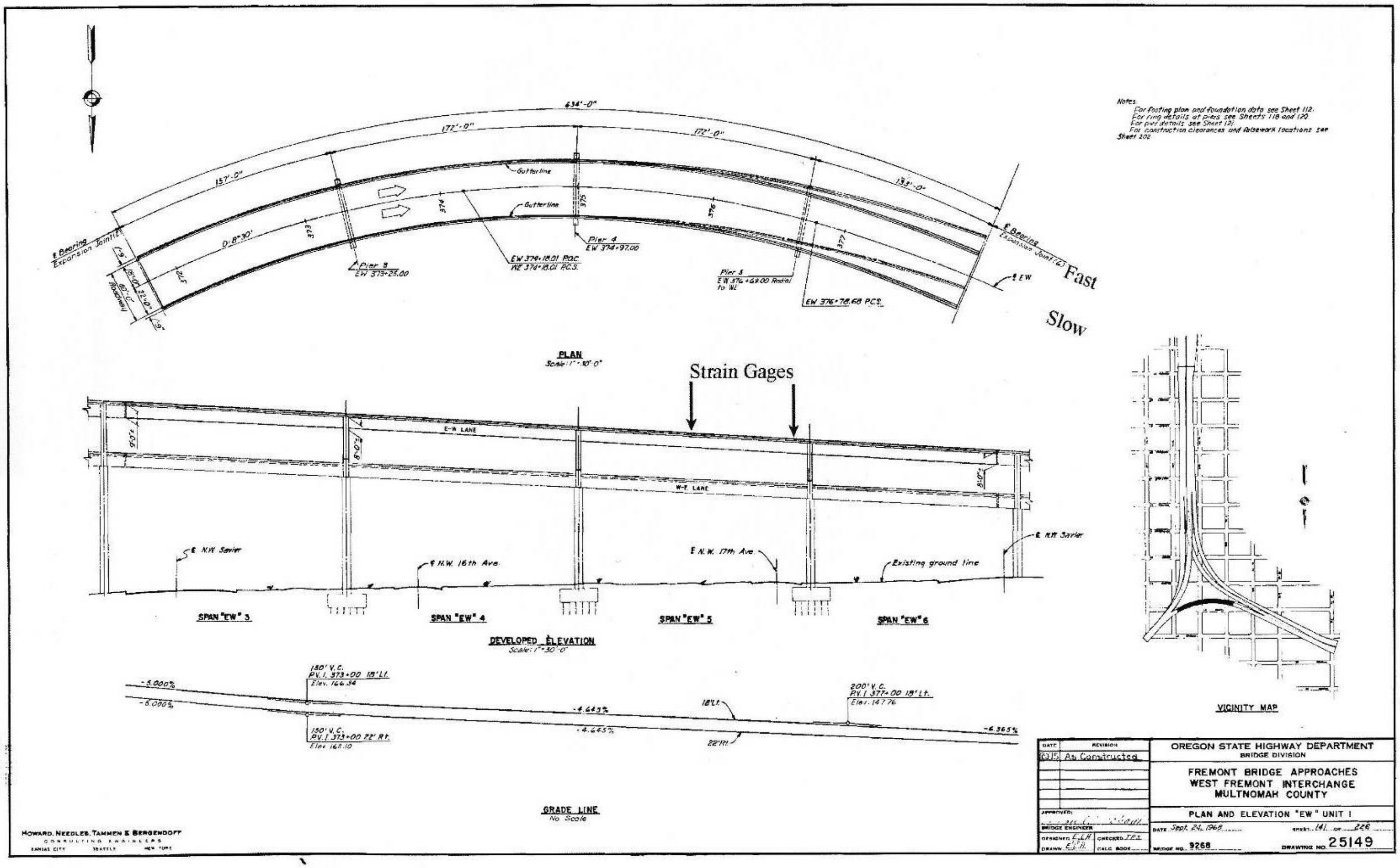


Figure 3.5: Instrumented sections for Bridge superstructure 09268W (EW line).

3.2.1 Results of Field Testing

Data were reduced and analyzed in the following four different formats:

1. peak-valley strain history;
2. rainflow, using a 2 microstrain (58 psi) hysteresis;
3. transient time history of peak strain events; and
4. root-mean-cube stress range, using a 10 microstrain (290 psi) hysteresis.

The peak-valley strain history presentation took the full transient time history and removed data where there was little to no change in strain. This is particularly useful for highway bridge loading that has cyclic loading with randomly occurring components in the temporal domain. Typical results for the top and bottom flanges at both positive and negative moment sections are shown in Figures 3.6 through 3.9 respectively.

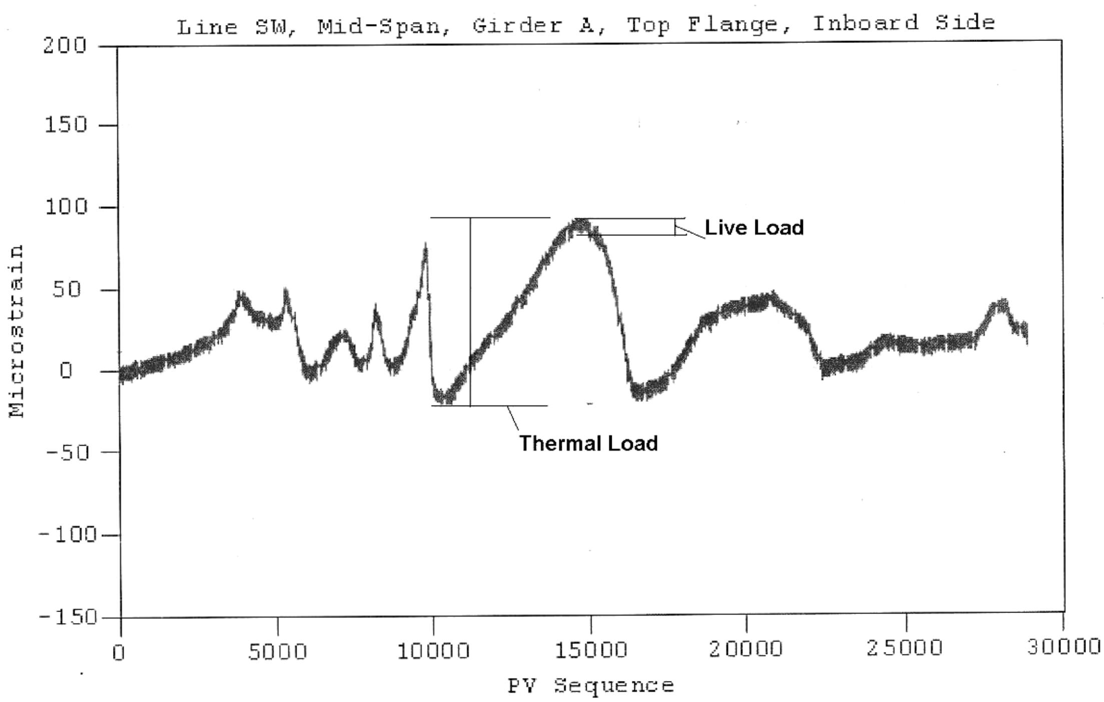


Figure 3.6: Typical peak-valley strain data for a top flange in positive moment section.

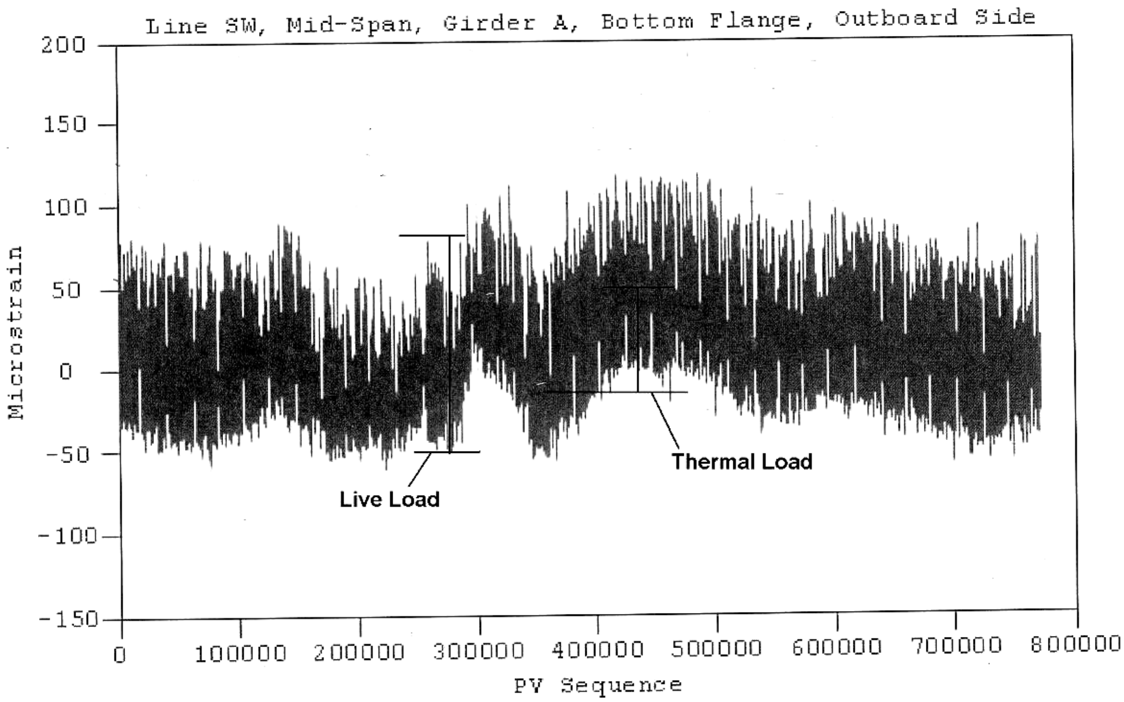


Figure 3.7: Typical peak-valley strain data for a bottom flange in positive moment section.

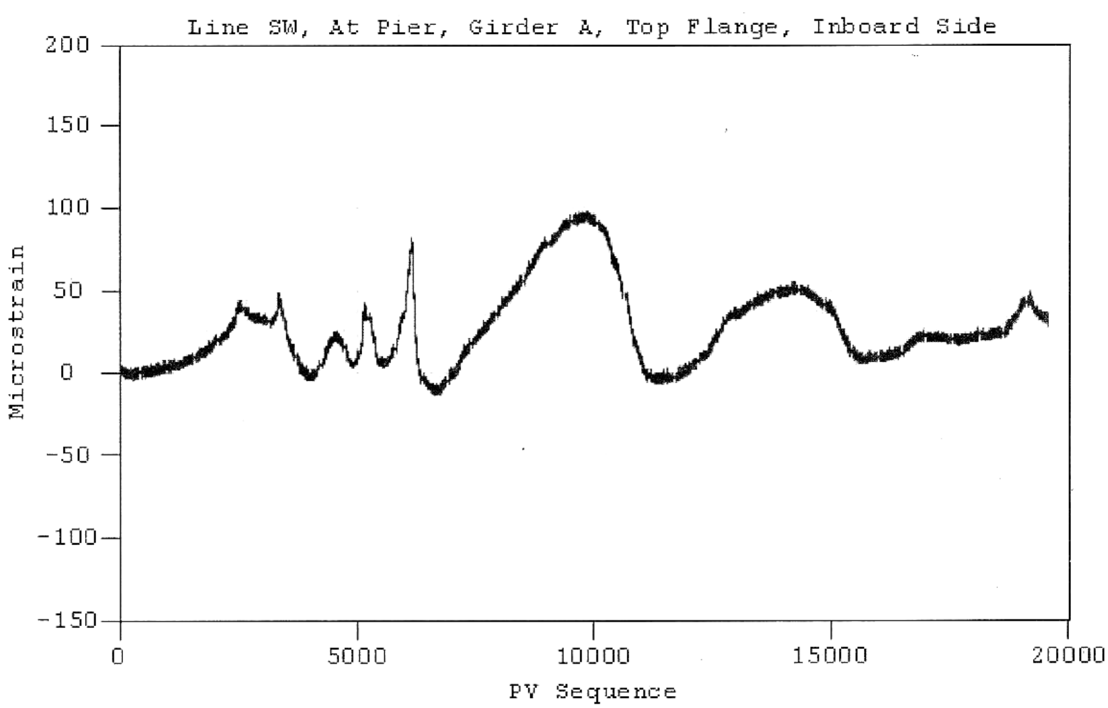


Figure 3.8: Typical peak-valley strain data for a top flange in negative moment section.

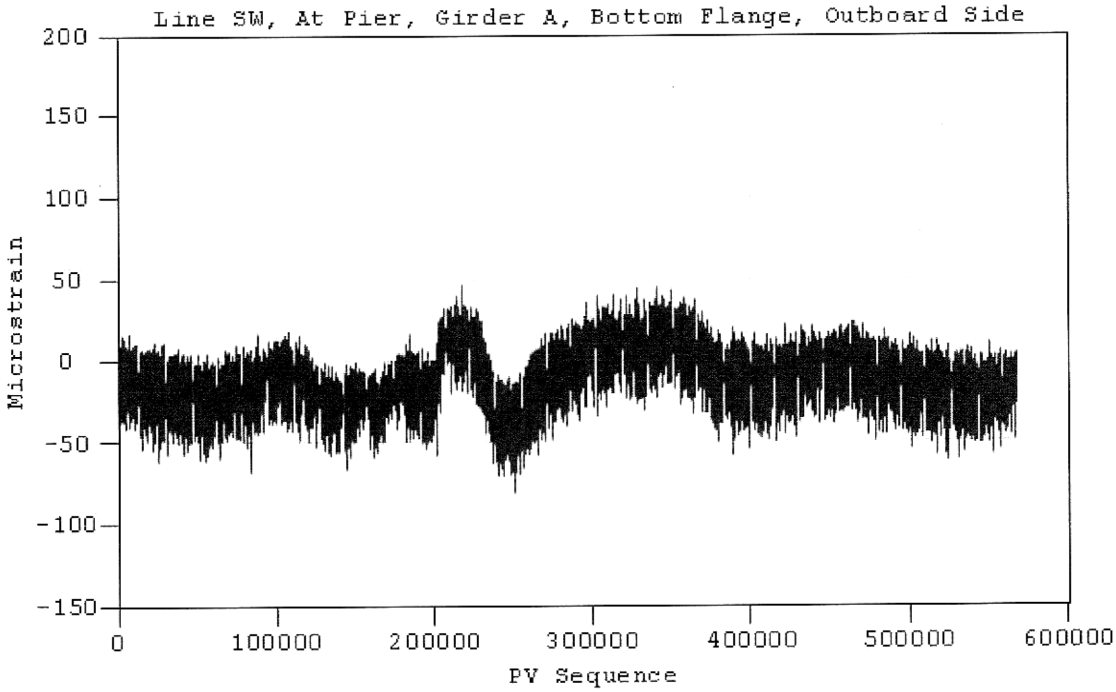


Figure 3.9: Typical peak-valley strain data for a bottom flange in negative moment section.

The rainflow presentation of data was useful for quantifying the loading and stress spectrum with respect to the frequency of occurrence. The root-mean-cube effective stress range was calculated and used for the fatigue analysis. Figures 3.10 through 3.13 show typical rainflow results for the top and bottom flanges at both positive and negative moment sections.

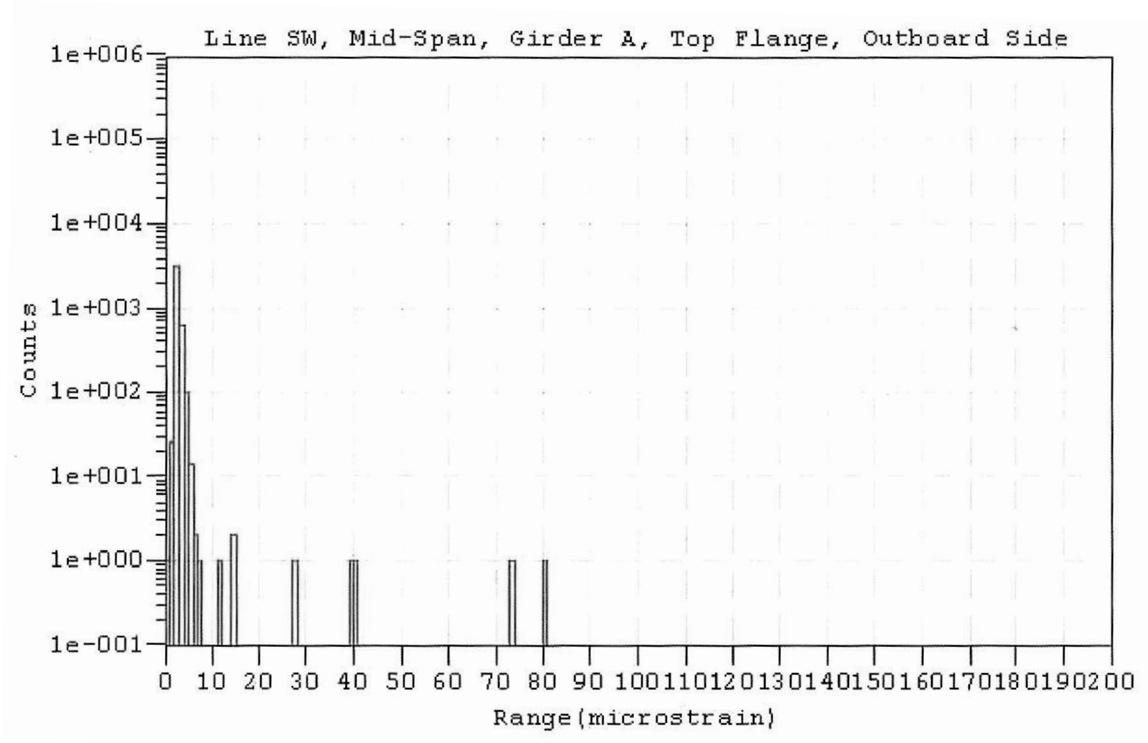


Figure 3.10: Typical rainflow strain data for a top flange in positive moment section.

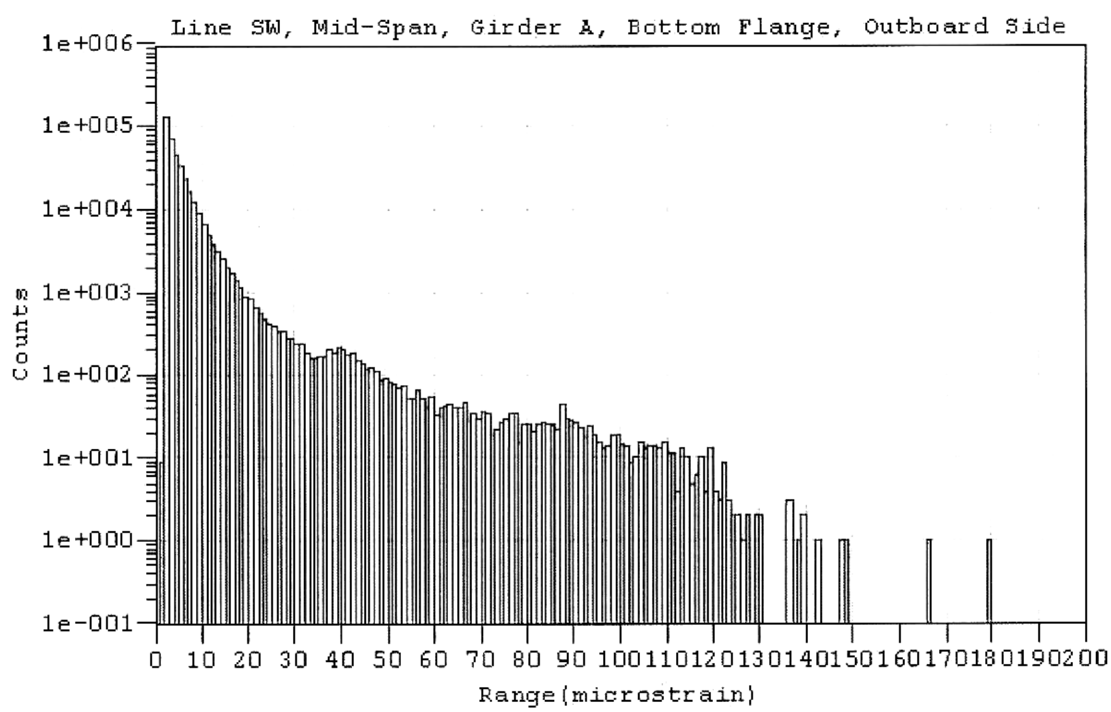


Figure 3.11: Typical rainflow strain data for a bottom flange in positive moment section.

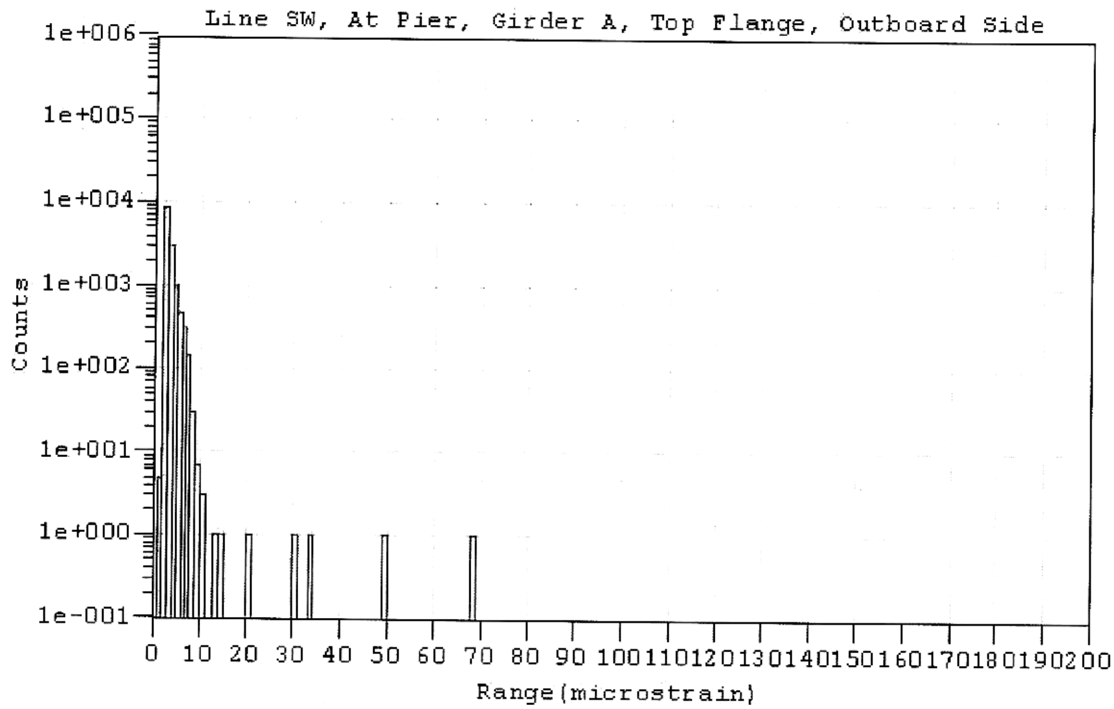


Figure 3.12: Typical rainflow strain data for a top flange in negative moment section.

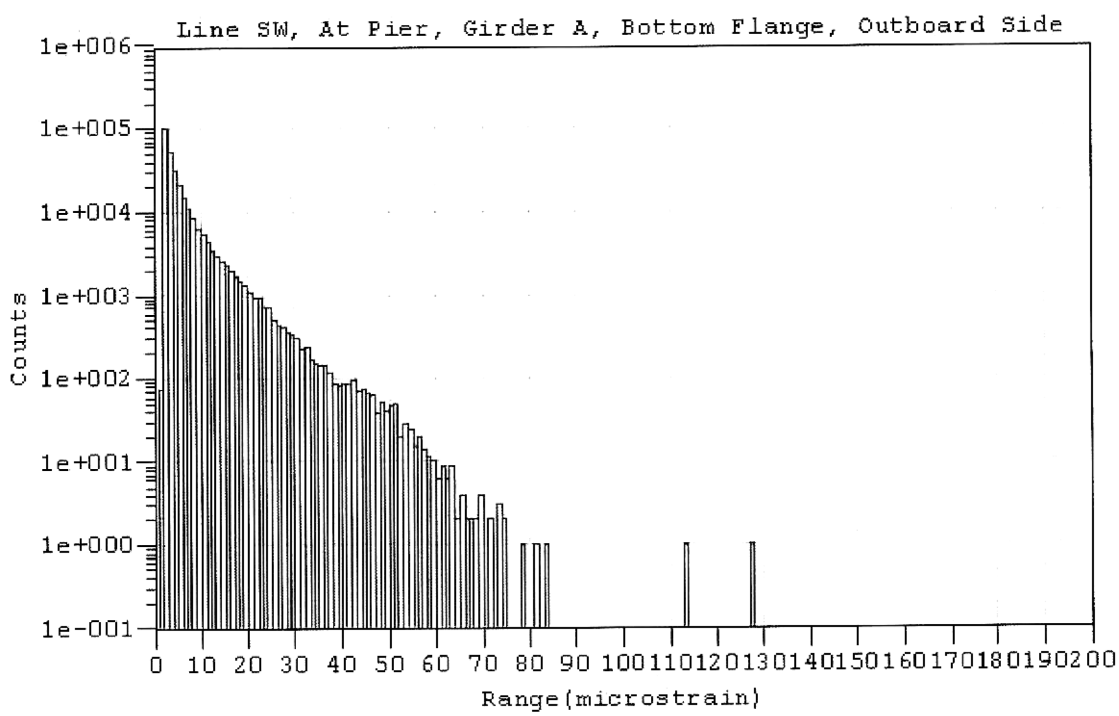


Figure 3.13: Typical rainflow stain data for a bottom flange in negative moment section.

Transient time histories were recorded during peak live load strain events to capture the detailed response of the flanges under heavy loads. This form of presentation, along with the peak-valley presentation, was useful for validating the results of the rainflow presentation as well. Figures 3.14 and 3.15 show typical examples of the transient responses at both positive and negative moment sections.

The root-mean-cube (rmc) effective stress range was calculated for each gage location over the seven days of data acquisition, using a 10 microstrain (290 psi) hysteresis in order to reduce the diluting affect passenger cars have on the truck induced live load and thermal strains. A summary of results is shown in Table 3.1.

Table 3.1 Summary of fatigue stress for each bridge.

Bridge #	Line	Flange	Max. thermal stress range (ksi)	Max. live load stress range (ksi)	RMC* effective stress range (ksi)
9268A	SW	Top	3.05	0.61	1.97
		Bottom	1.89	2.18	0.96
9268B	WS	Top	2.90	0.87	0.81
		Bottom	1.60	5.08	1.28
9268E	WE	Top	2.90	1.07	0.64
		Bottom	1.60	5.22	0.96
9268W	EW	Top	2.90	0.58	1.33
		bottom	1.74	2.90	0.87

* 7 days of continuous data with a 290 psi hysteresis.

3.3 RESIDUAL STRESSES FROM WELDING

As discussed in the West Linn Bridge study (*Lovejoy 2002*), the literature review and experimental results obtained from in-service welds and weld cores, concluded that ESW has a unique residual stress field compared to multi-pass welding processes such as SAW and SMAW. The stress field is relatively simple and potentially favorable with respect to fatigue performance. In cross section, the weld's outer periphery, near the heat affected zone (HAZ), sustains a compressive residual stress close to the weld metal yield strength. The inner core is in triaxial tension and also near yield of the weld metal. No residual stress measurements were taken under this particular study. Even though the base metals of the West Linn (ASTM A36) and West Fremont bridges (ASTM A588) differ, the welding procedures and filler metals are essentially identical. Residual stresses in the subject bridges are therefore presumed similar in nature and relation to yield, as are those in the West Linn Bridge.

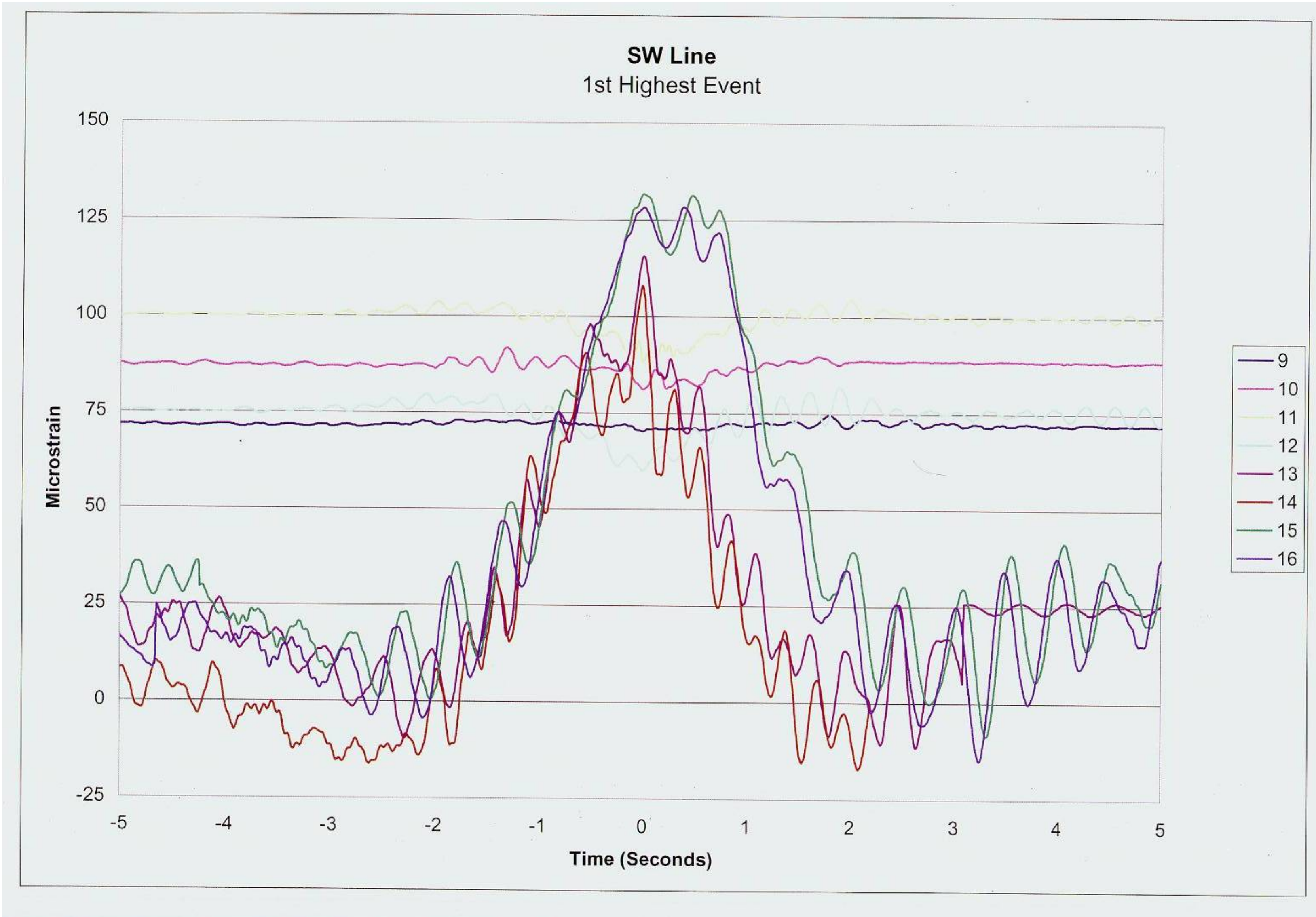


Figure 3.14: Typical strain time history for a large live load event in a positive moment section. Data 9 thru 12 are top flange gages and 13 thru 16 are bottom flange gages.

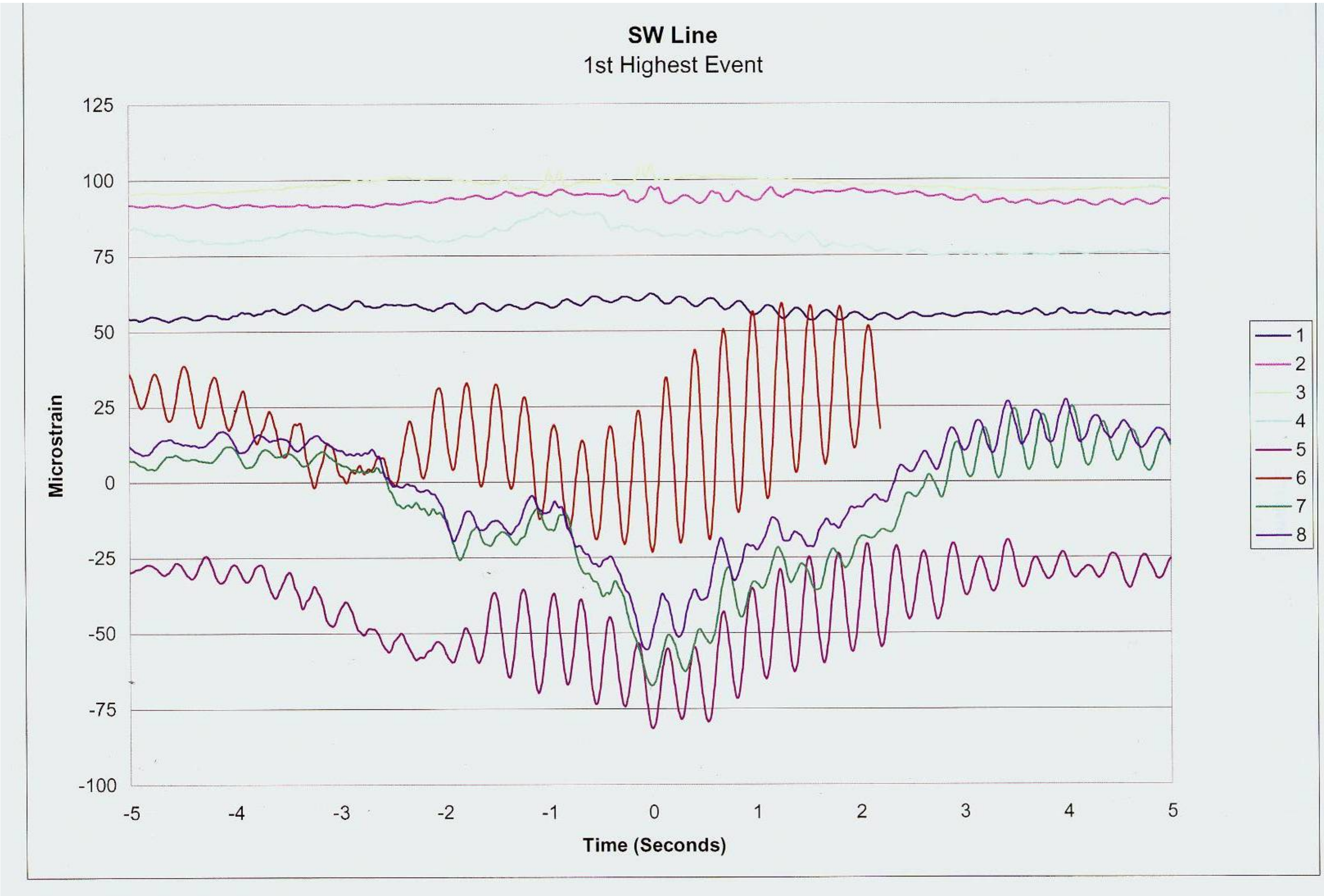


Figure 3.15: Typical strain time history for a large live load event in a negative moment section. Data 1 thru 4 are top flange gages and 5 thru 8 are bottom flange gages

4.0 MATERIALS TESTING

4.1 PURPOSE OF MATERIALS TESTING

To accurately assess the fitness-for–purpose of the electro-sag weld (ESW) in box girder flanges, the weld, base metal and heat affected zone (HAZ), need to have pertinent properties quantified. The primary characteristics investigated were the fracture toughness and potential fatigue crack growth. Other properties such as hardness, tensile strength, Charpy–V notch (CVN) toughness, chemistry and microstructure were investigated as supportive data. The fracture toughness was affected by the thickness, temperature and load rate, so it is not an intrinsic material property. This testing program was tailored to the specific environment, thickness and loading requirements of the bridge.

4.2 MATERIALS TESTING STRATEGY

Standard gap ESW in the flange thicknesses of the West Linn and West Fremont approach structures has a failure mode that varies between ductile and brittle, depending on the temperature, thickness and metallurgical region where crack-like defects exist. As discussed by Lovejoy (*2002*), selecting the appropriate materials testing program is paramount to a successful and valid fitness-for-purpose evaluation. In almost all cases, at a lowest anticipated service temperature (LAST) of 0 ^{o}F, the fracture properties are the limiting case as opposed to general yielding. When the material is near the mean service temperature of approximately 50 ^{o}F, general yielding is more likely to govern. Thus, the fracture testing program must include the brittle to ductile transition region.

Similar to the West Linn, the materials testing program for this project was divided into two phases. Phase 1 cores were used primarily for CVN impact testing, plus examination of the weld morphology, hardness, chemical and tensile testing. Cores were positioned to test all-weld metal, heat affected zone (HAZ) and base metals. The CVN specimens were tested over a range of temperatures to quantify the transition region and estimate fracture toughness.

Phase 2 used the results of Phase 1 to tailor the fracture toughness testing. As in the West Linn study, a modified form of ASTM E 1820 J-Integral was selected to quantify the fracture toughness and ASTM E 647 was selected to quantify the crack growth characteristics. Details of reducing the J-integral test results to in-service critical stress intensity, K_C , are discussed in Section 6.3.

4.3 CORE REMOVAL

Twelve, four inch diameter through-thickness cores were removed from bottom flanges in negative moment areas of Bridge superstructures 09268A and B. A summary of the core locations and test phases is shown in Table 4.1.

Table 4.1: Core removal locations

Core #	Weld ID	Metallurgical position
Phase 1		
1	SW2-A-B-4	Weld metal
2	SW3-A-B-1	Weld metal
3	SW3-B-B-1	HAZ
4	SW4-A-B-3	HAZ
5	SW5-A-B-1	HAZ
Phase 2		
6	WS4-A-B-2	HAZ
7	WS4-A-B-1	Weld metal
8	WS3-A-B-5	Weld metal
9	WS3-A-B-4	HAZ
10	(WS1-A-B-X)*	Base metal
11	WS1-A-B-5	HAZ
12	WS1-A-B-4	Weld metal

* base metal core taken 2 feet from core #11.

At each core location the coating was removed from the top flange face and the surface was acid etched to precisely locate the weld and HAZ. Ultrasonic Testing (UT) was performed at each proposed core location to verify defect free material. The cores were removed and structural compression plugs were installed in each hole to replace the material. Details of the core removal can be found in the MEI-Charlton Inc. report (*2007*). Figure 4.1 shows the UT prior to core removal. Figure 4.2 shows an etched ESW prior to core removal (including probable SMAW shop repairs to undercut near the fusion boundary). Figure 4.3 shows the core removal. Figure 4.4 shows the core plug and its installation. Figure 4.5 shows the Phase 1 cores after removal. Figure 4.6 shows the Phase 2 cores after removal.

Figure 4.1: Ultrasonic testing of ESW prior to core removal.

Figure 4.2: Etched ESW showing fusion lines prior to core removal

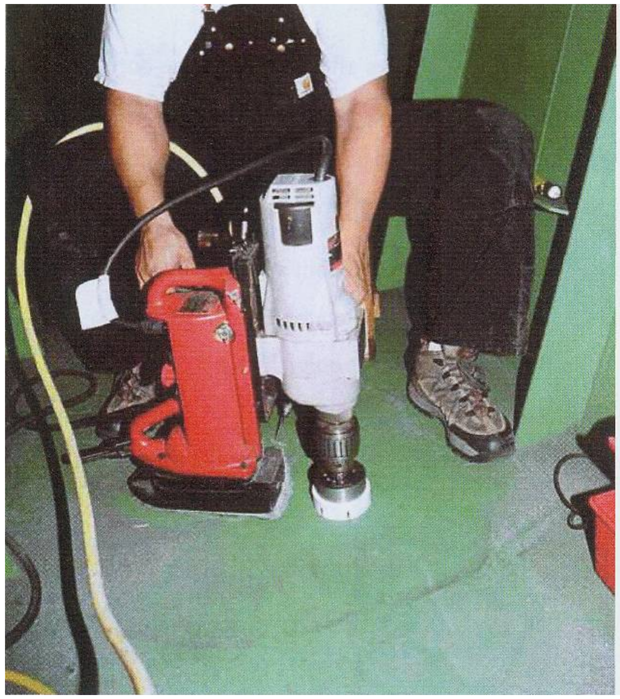


Figure 4.3: Core removal.

Figure 4.4: Core plug and core plug installation after core removal.

Figure 4.5: Phase 1 testing material cores.

Phase 2 Cores, Nos. 7 through 12

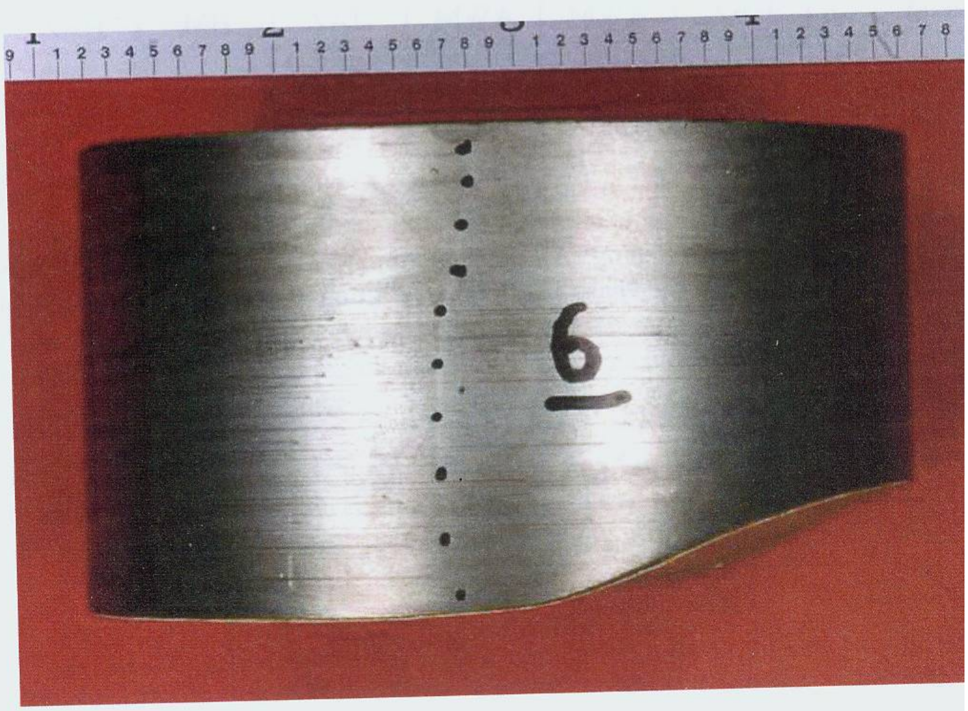


Figure 4.6: Phase 2 testing material cores.

4.4 PHASE 1 MATERIAL TESTING

The following tasks were performed on the Phase 1 core samples:

- Macro etch the cores per ASTM E340-00e1 and identify the location/profile of the base metal, weld metal and HAZ
- Micro polish and etch the samples and perform metallurgical characterization of the base metal, weld metal and HAZ
- Conduct CVN testing of the weld and HAZ per ASTM E23-04 over a range of temperatures to produce impact energy vs. temperature transition curves to identify the upper and lower energy shelves
- Perform chemical analysis of the base and weld metal and evaluate conformance with base metal specifications
- Calculate the Carbon Equivalent (CE) of the base and weld metal based on chemical analysis

4.4.1 Weld Profiles

All cores were taken from welds that included a thickness transition. ESW, which were made with the standard gap method, typically exhibited a barreling of the fusion line as viewed from a side section. The thicker plate side of the welds displayed concave barreling and the thinner side welds displayed slightly convex curvature. A typical example is shown in Figure 4.5.

4.4.2 Metallurgical Characterization

Base metal: The base metal consisted of equiaxed grains of ferrite and pearlite in a banded structure typical of hot-rolled, low carbon steel plates.

HAZ: The HAZ exhibited a transition from the banded structure, with a breakdown of the pearlite due to recrystallization. Closer to the weld metal the HAZ exhibited course prior austenitic grains, with a grain size of 2.5 to 3.0 per ASTM E 112, with grain boundary ferrite.

Weld metal: The structure in the weld metal consisted of large prior austenitic grains with blocky, lath-like grain boundary ferrite, with grain size of 0.5 to 1.0 per ASTM E 112. Finer Widmanstatten ferrite was present within the large grains.

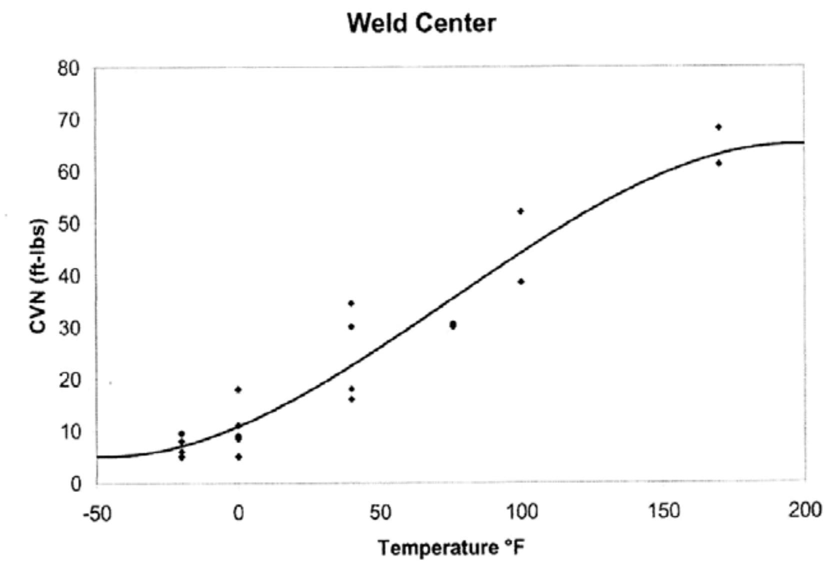
No significant non-metallic inclusions were observed. Photomicrographs are presented in MEI-Charlton Inc. report (*2007*)

4.4.3 Base Metal Chemistry

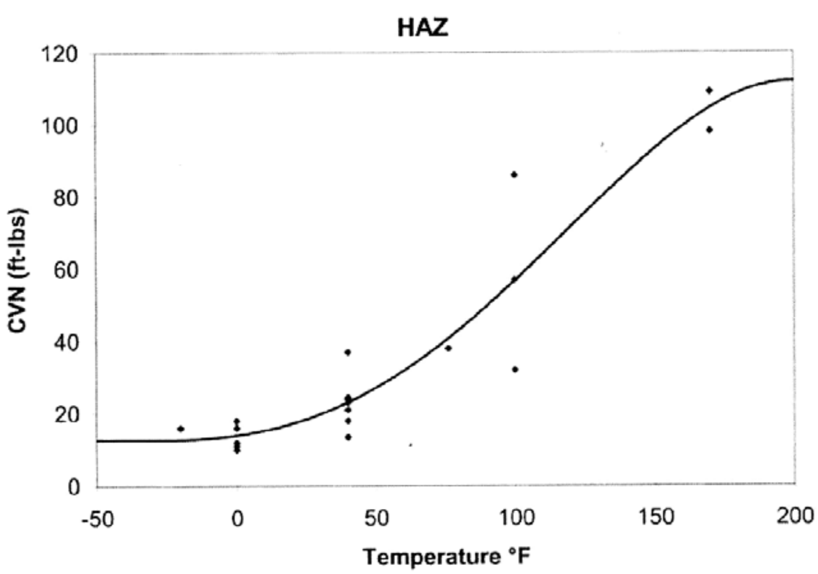
Four samples of base metal were tested for chemical composition and calculation of the CE. All four samples were found to be well within the ASTM A 588 Grade B specifications. The CE ranged from 0.44 to 0.56.

4.4.4 Impact Testing

Ten CVN test bars were machined from each of the five Phase 1 material cores, yielding a total of 50 test bars that were tested from -20 to 170 ^{o}F. The results of the impact testing can be seen in Figure 4.7.



Charpy V-Notch Transition Curve, Weld Metal



Charpy V-Notch Transition Curve, HAZ

Figure 4.7: Results of Phase 1 impact testing of ESW cores.

The data labeled “center” was extracted from mid-thickness or T/2 of the core. The data labeled ‘surface” was extracted at the quarter point or T/4. As discussed in the MEI-Charlton Inc. report (*2007*), the HAZ samples were mounted, polished and etched after testing to confirm the location of the notch tip relative to the microstructure. The notches were found to be located 1 to 2 mm off of the intended HAZ microstructure, so the test results are more indicative of the impact energy of the weld metal near the HAZ. The minimum recommended impact energy required for qualifying standard gap ESW of 15 ft-lbf is shown on the plotted data. The all-weld metal was substandard at 0^{o}F, but the HAZ passes the recommendations at 40 ^{o}F.

The impact test data were used to estimate the fracture toughness of the weld metal and HAZ using the Barsom-Rolfe two stage correlation between Charpy V-notch (CVN) impact test data and plane strain fracture toughness K_{IC} data for bridge steels at intermediate strain rates (*Barsom and Rolfe 1987*). Figure 4.8 shows the results of this correlation for both the West Fremont (current study) and West Linn (similar previous study) Phase 1 testing. The data shown are averages from each set at each temperature. This correlation predicts that the weld metal and HAZ will have a fracture toughness between 80 and 95 ksi-in$^{1/2}$ at the LAST of 0^{o}F.

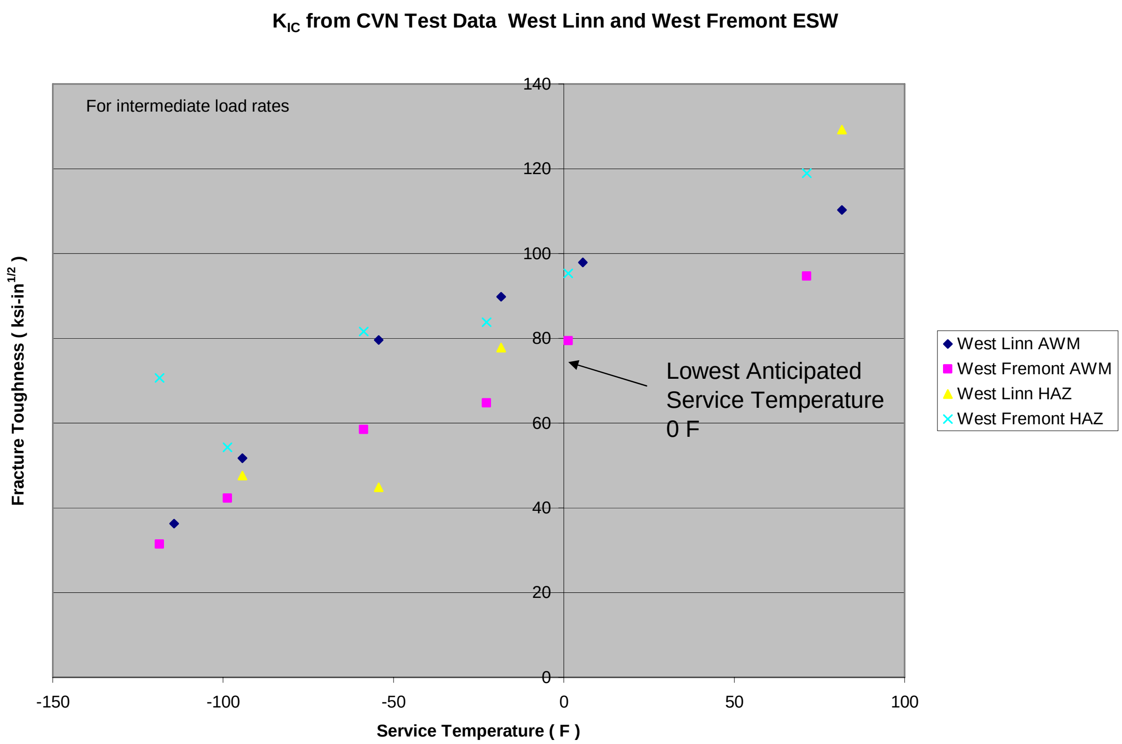


Figure 4.8: Estimated fracture toughness from impact data using the Barsom-Rolfe two stage correlation to adjust for intermediate loading rates.

4.5 PHASE 2 MATERIAL TESTING

The following tasks were performed on the Phase 2 material cores:

- Test three weld metal specimens, three HAZ specimens and one base metal specimen in accordance with ASTM E 1820 using compact tension specimen configuration at 0^{o}F and a target loading rate of zero to P_{max} in 1 to 3 seconds to establish the fracture toughness
- Test three weld metal specimens and one base metal specimen in accordance with ASTM E 647 to establish the fatigue crack growth rates and fatigue thresholds at ambient temperature of 70 ^{o}F.
- Test three weld metal and three base metal specimens in accordance with ASTM E8 to establish tensile properties at room temperature.

4.5.1 Fracture Toughness Testing

The test specimens were machined into compact tension specimens with the geometry shown in Figure 4.9 and 4.10. The specific specimen measurements are summarized in Table 4.2.

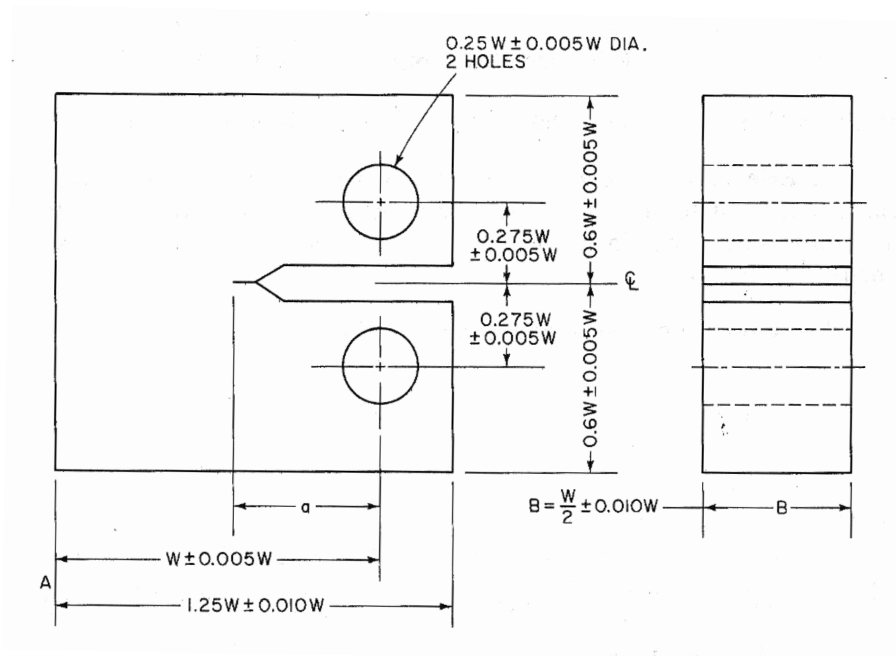


Figure 4.9: Compact tension specimen geometry used for JR curve and crack growth testing.

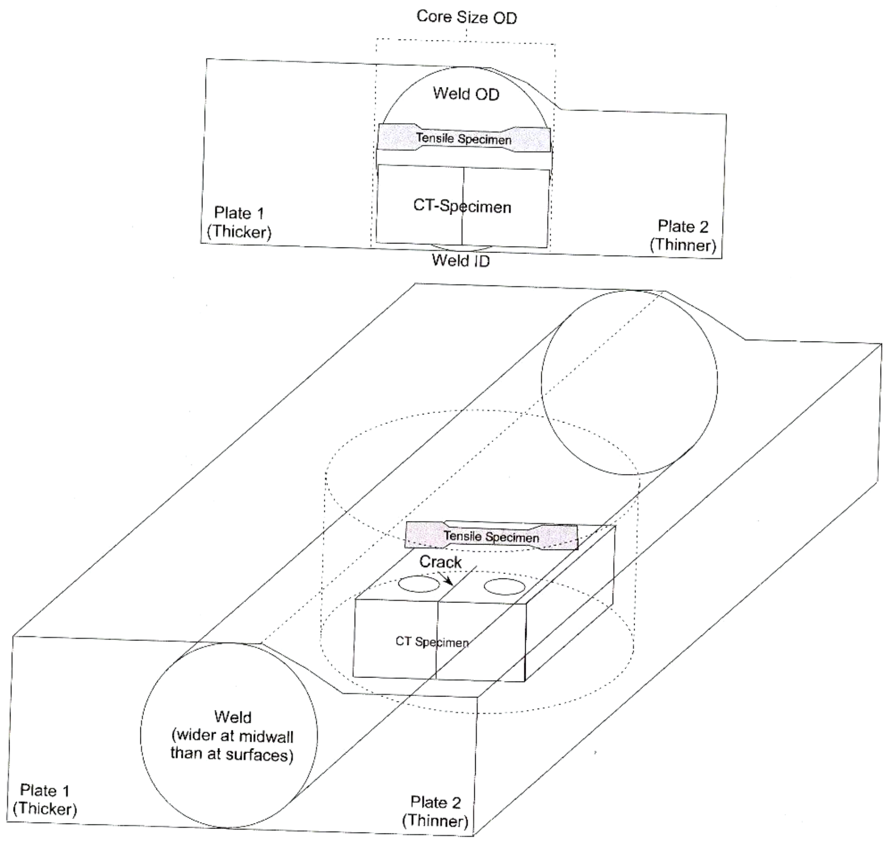


Figure 4.10: Cutting plane for Phase 2 material cores.

Table 4.2: Fracture toughness testing specimen geometry summary

	10-WS1	7-WS4AB1	8-WS3AB5	12-WS1AB4	6-WS4AB2	9-WS3AB4	11-WS1AB5
Sample Condition	Base Metal	AWM	AWM	AWM	HAZ	HAZ	HAZ
Specimen Width (W), inch	1.9921	1.9873	1.9875	1.9905	1.9960	1.9978	1.9963
Crack Length (a), before fatigue	1.1548	1.2360	1.1720	1.1753	N/A	N/A	N/A
Crack Length a(o), after pre-cracking	1.3730	1.3990	1.3540	1.3960	1.3990	1.4000	1.4010
Specimen Thickness B, inch	0.9940	0.9934	0.9948	0.9923	0.9980	0.9984	0.9968
Specimen Net Thickness (B_N), inch	0.800	0.800	0.800	0.800	0.800	0.800	0.800
Physical Crack Size (a(p)), inch	1.3499	1.3365	1.3465	1.3659	1.3928	1.4017	1.3921
Remaining Ligament b(o), inch	0.6422	0.6508	0.6410	0.6246	0.6032	0.5961	0.6042
Yield Strength (YS), ksi	56.0	55.6	55.6	55.6	55.6	55.6	55.6
Ultimate Tensile Strength (UTS), ksi	84.3	82.6	82.6	82.6	82.6	82.6	82.6
Flow Strength (FS), ksi	70.1	69.1	69.1	69.1	69.1	69.1	69.1
Elastic Modulus (E), ksi	28,000	27,817	27,817	27,817	27,817	27,817	27,817
Poisson's Ratio (ν)	0.3	0.3	0.3	0.3	0.3	0.3	0.3

(N/A) = Not applicable

4.5.1.1 Base Metal Results

The single base metal test specimen produced a well behaved J-R curve, as shown in Figure 4.11, yielding a J(Q) value of 723 lbf/in (126.6 kJ/m^2).

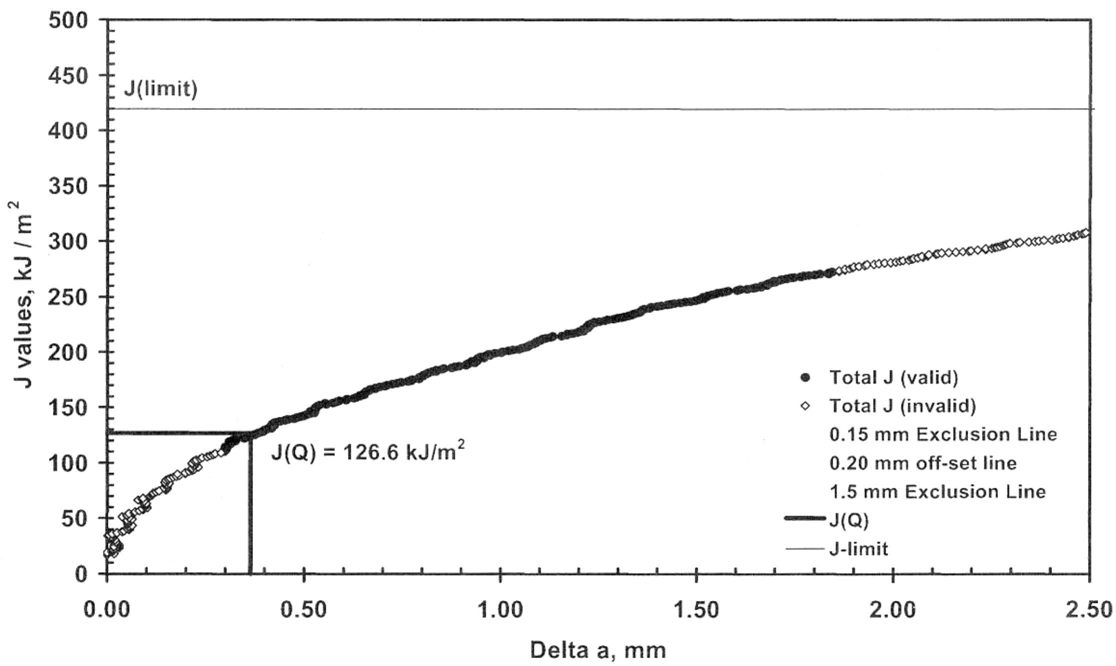


Figure 4.11: Base metal JR curve at 0 ^{o}F.

4.5.1.2 HAZ Results

The following paragraph was quoted directly from MEI-Charlton Inc. (*2007*) with the exception of figure numbers, which were modified to fit this report.

> The data for the HAZ specimens revealed a steeper slope and had J(Q) values higher than those for the base metal specimen. Specimen 9-WS3AB4 and specimen 11-WS1AB5 failed suddenly within the area of J-R valid values, as seen from the sudden increase in crack length at 976 and 1937 lbf/in (171 and 339 kJ/m^2), respectively. See Figures 4.12 and 4.13. Specimen 6-WS4AB2 did not fail below the J-limit value for valid test data to be reached, and therefore that value 2236 lbf/in (391.6 kJ/m^2) was reported as J(Q). See Figure 4.14.

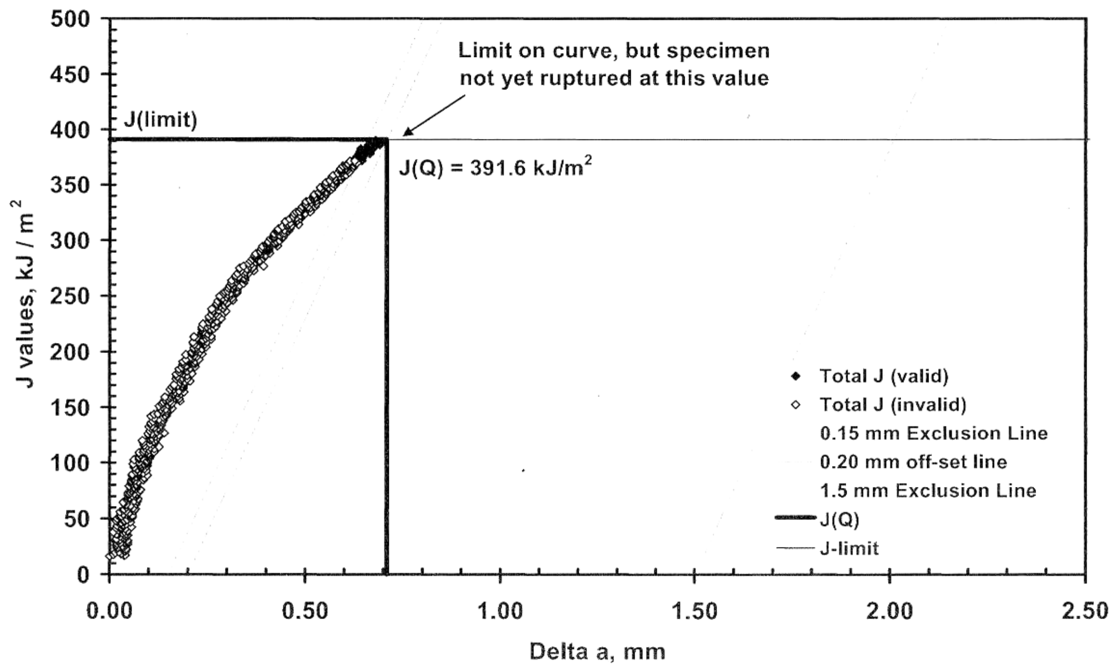


Figure 4.12: HAZ JR curve for specimen 6-WS4AB2 at 0 °F.

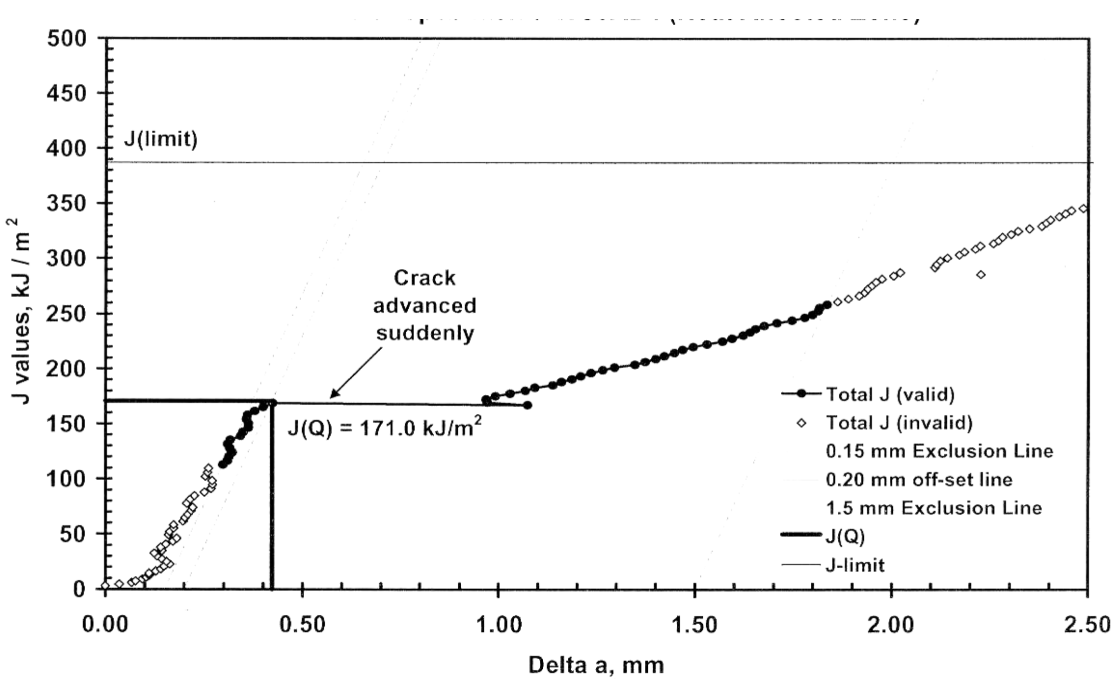


Figure 4.13: HAZ JR curve for specimen 9-WS3AB4 at 0 °F.

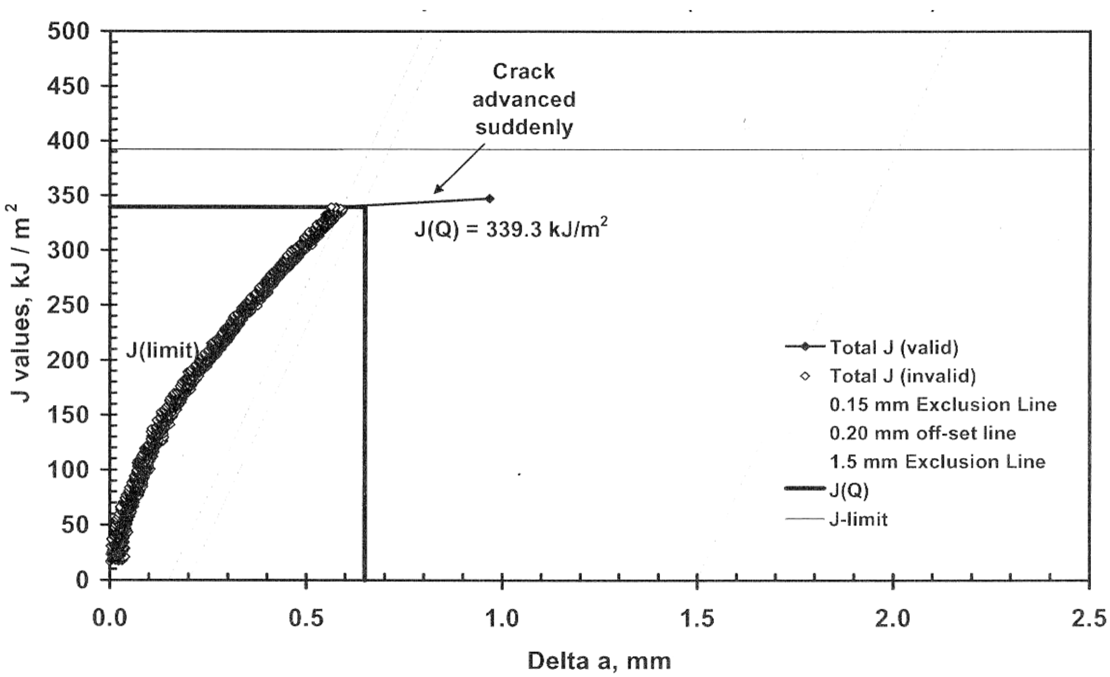


Figure 4.14: HAZ JR curve for specimen 11-WS1AB5 at 0 ^{o}F.

4.5.1.3 Weld Metal Results

The following paragraph was also quoted directly from MEI-Charlton Inc. (*2007*) with the exception of figure numbers, which were modified to fit this report.

> The data for the AWM specimens was difficult to interpret. The EPD signals during the first phase of load increase showed an erroneous crack increase, followed by a decrease to the approximate initial value for the specimens. This is visible as a “nose” in the JR-curves; see figures 4.15, 4.16 and 4.17. This phenomenon occurred in the normally elastic deformation range, where the crack is not expected to grow, but the plastic zone size around the crack tip can increase. By ignoring the nose, and proceeding with the standard analysis of the remainder of the data, a J(Q) value for the specimens could be determined, nonetheless. The three specimens (Specimen 7-WS4AB1, 8-WS3AB5 and 12-WS1AB4) behaved approximately the same, with values of 369,332 and 397 lbf/in (64.7,58.2 and 69.6 kJ/m^2), respectively.

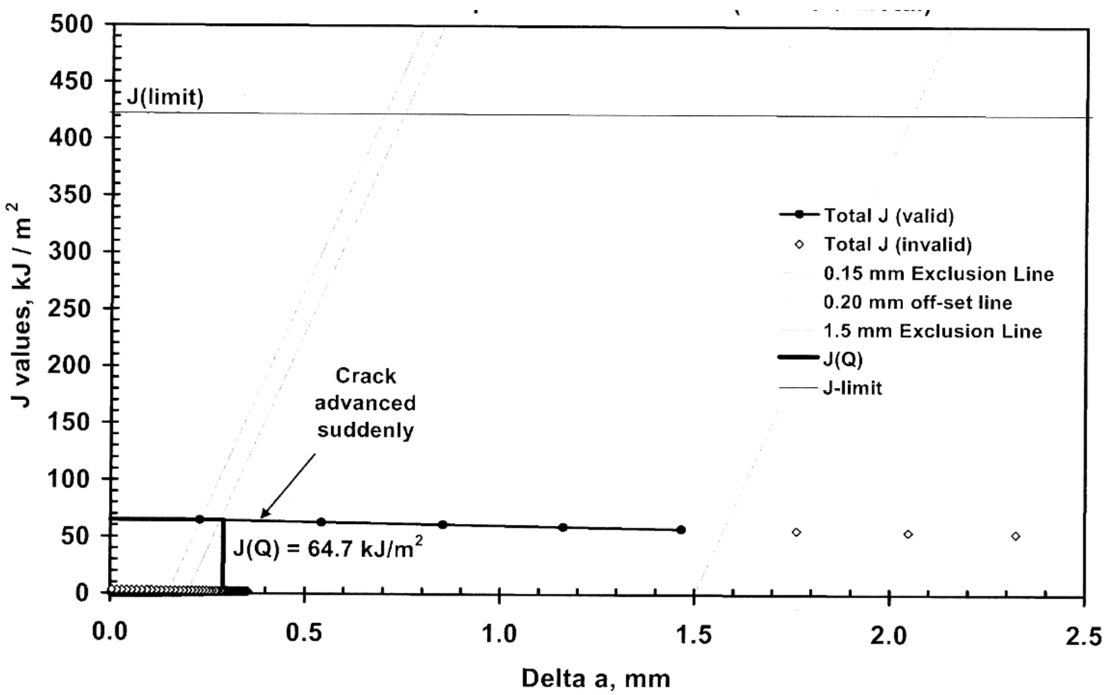


Figure 4.15: Weld metal JR curve for specimen 7-WS4AB1 at 0 °F.

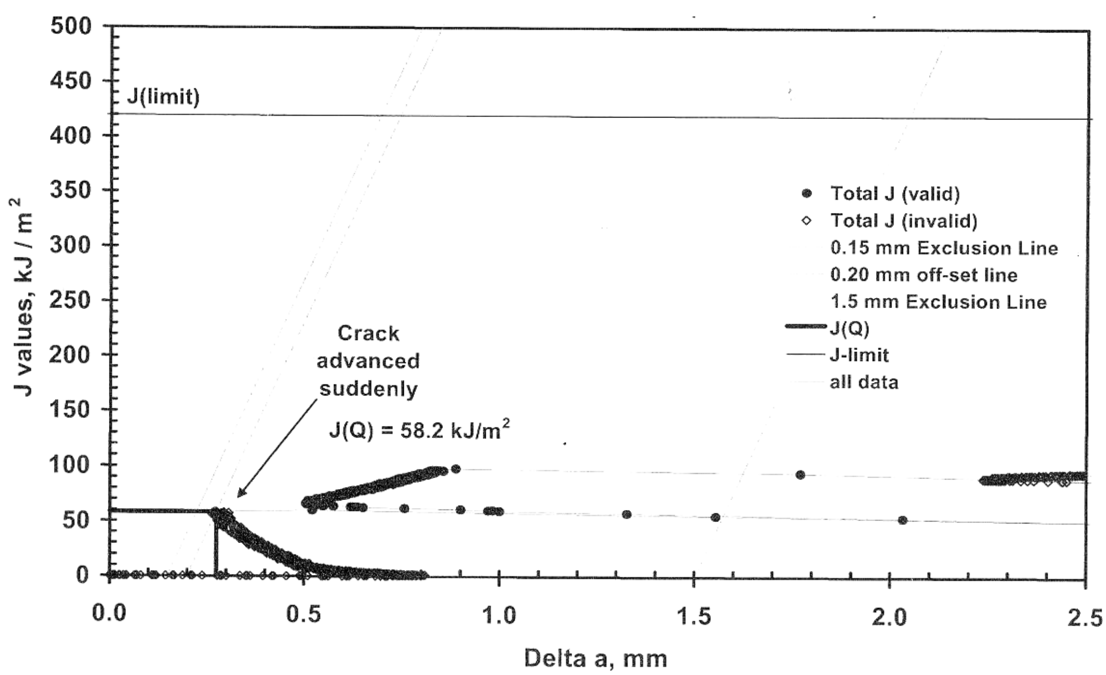


Figure 4.16: Weld metal JR curve for specimen 8-WS3AB5 at 0 °F.

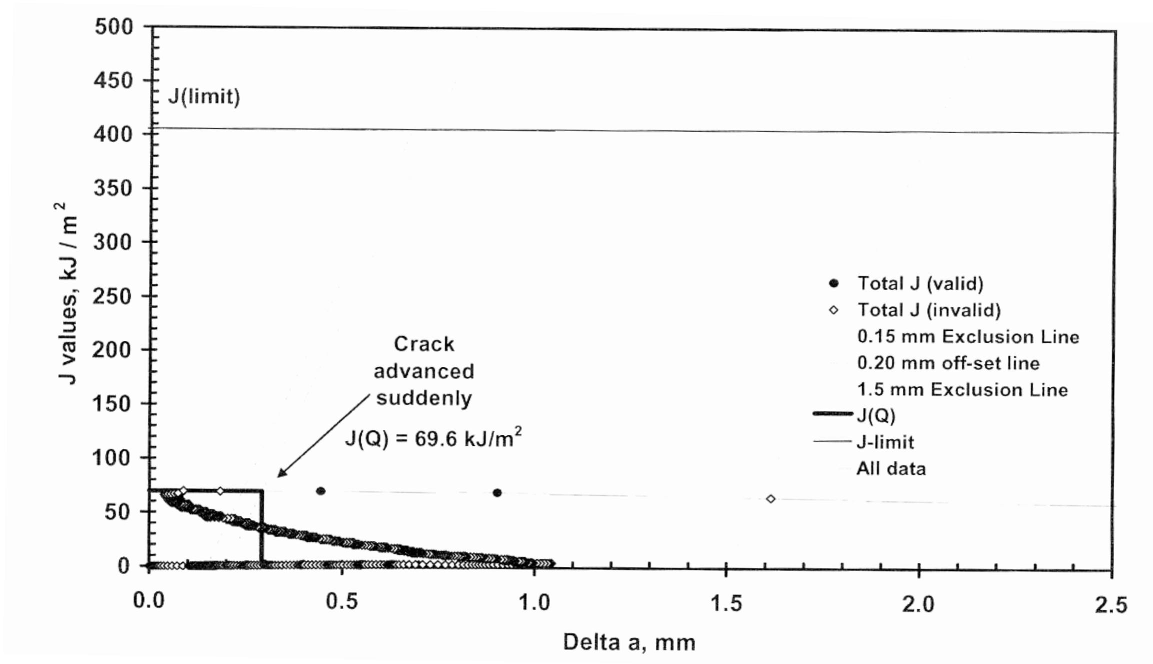


Figure 4.17: Weld metal JR curve for specimen 12-WS1AB4 at 0 °F.

Crack propagation of the all-weld metal (AWM) specimens at zero degrees F is by cleavage, which entails very little plastic deformation or stable crack growth. For these tests, performed in accordance with ASTM E1820, the Electrical Potential Displacement (EPD) method of monitoring crack growth was used as opposed to the alternative of using a clip type displacement transducer. Based on the author's experience, the EPD method has disadvantages compared to the clip gage when cleavage is encountered. But the results of the crack growth during fracture testing of such specimens is of academic interest only since the critical J values are calculated using the initial crack length, which is physically measured after the specimen has failed. In other words, there is no J-R plot for these test specimens because there is no J-R behavior. There is only J_{crit} behavior with the deformation at the crack tip very much elastic with only small (negligible) plasticity. J_{crit} can be calculated from the data using the initial crack length and maximum load, P_{max}, from the load displacement curve, which is the case for the three all weld metal test specimens (Appendix C).

Note that the selection of ASTM E1820 for characterizing the fracture behavior of the base, HAZ and AWM ,was very purposeful. The base and portions of the HAZ exhibit J-R behavior and the AWM does not so only the J_Q value is reported. With the specimen geometry and material behavior seen, J_Q is a reasonable estimate of J_{crit}.

4.5.2 Crack Growth Testing Results

One base metal and three weld metal specimens were tested for crack growth rate characterization and the fatigue threshold stress intensity. Figure 4.18 shows the test results. The base metal showed the lowest fatigue threshold at 7.13 ksi-in^{½} with the weld metal ranging from 8.45 to 9.58 ksi-in^{½}.

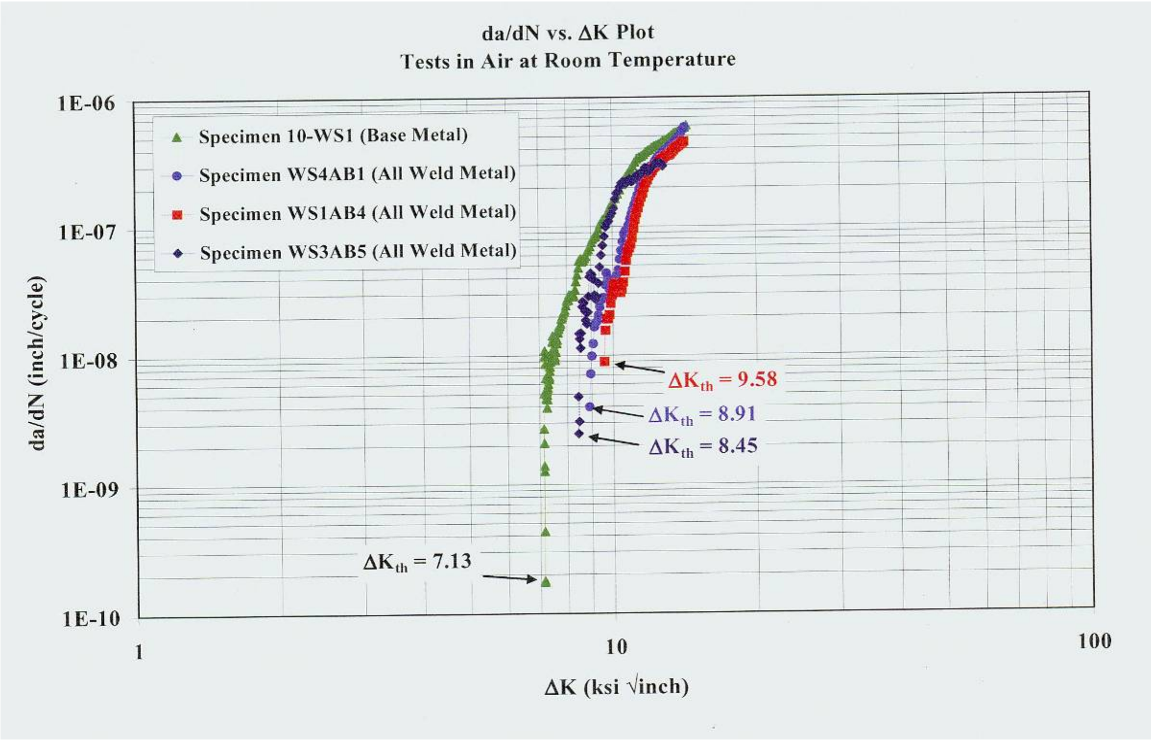


Figure 4.18: Crack growth rate testing results.

Coefficients for the crack growth rates are shown in Table 4.3. Fatigue threshold results, from both the West Linn and West Fremont studies, are shown together in Figure 4.19.

Table 4.3: Summary of fracture toughness and crack growth rates

Sample ID	Region	da/dN (in./cycle)		ΔK$_{th}$		J(Q) (**)	
		Near Threshold	Higher ΔK	(MPa√m)	(ksi√inch)	(kJ/m^2)	lb$_f$/in.
10-WS1	Base Metal	1.45E-15 × ΔK$^{8.02}$	9.10E-10 × ΔK$^{2.44}$	7.84	7.13	126.6	723
6-WS4AB2	HAZ	–	–	-	–	391.6	2,236
9-WS3AB4	HAZ	–	–	-	–	171.0	976
11-WS1AB5	HAZ	–	–	-	–	339.3	1,937
7-WS4AB1	AWM	4.20E-19 × ΔK$^{11.0}$	1.25E-11 × ΔK$^{4.04}$	9.80	8.91	64.7	369
8-WS3AB5	AWM	5.74E-21 × ΔK$^{13.3}$	1.99E-09 × ΔK$^{1.99}$	9.30	8.45	58.2	332
12-WS1AB4	AWM	1.97E-21 × ΔK$^{13.1}$	7.94E-11 × ΔK$^{3.26}$	10.54	9.58	69.6	397

(**) 1 kJ/m^2 = 5.710148 lb$_f$/in.

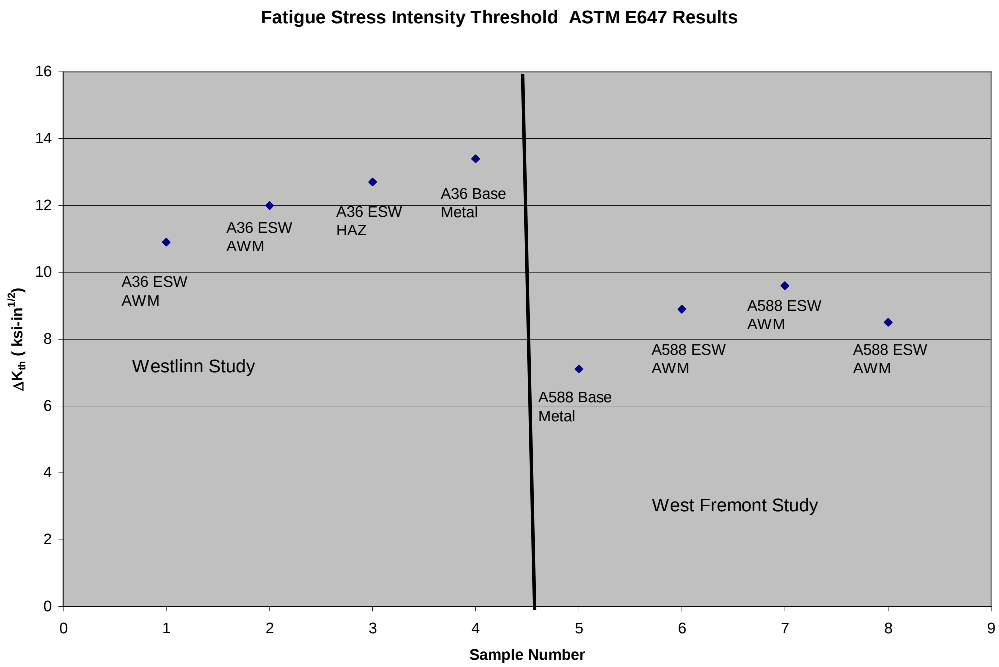


Figure 4.19: Crack growth threshold results for both the West Linn and West Fremont ESW studies.

4.5.3 Tensile Testing Results

The weld metal and base metal were found to have very similar tensile properties as shown in Table 4.4. Both metal types fall within specification.

Table 4.4: Weld and base metal tensile test results

	ASTM A588	Core No.						Average	
		7	8	12	1-Oct	2-Oct	3-Oct		
		Weld	Weld	Weld	Base metal			Weld metal	Base metal
Ultimate Strength, ksi	70.0 min.	84.6	77.6	85.5	84.6	84.4	83.8	82.6	84.3
Yield Strength, ksi	50.0 min.	57.6	50.5	58.8	55.4	55.8	56.8	55.6	56.0
Flow Strength (YS+UTS)/2 , ksi		71.1	64.1	72.2	70.0	70.1	70.3	69.1	70.1
Elongation, %	19.0 min.	22.0	27.0	24.5	27.5	29.0	29.0	24.5	28.5
Reduction of area, %		61.5	66.5	59.5	72.5	72.5	72.5	62.5	72.5
E-Modulus, ksi								27,817	28,000

4.6 ESTIMATING THE IN-SERVICE FACTURE TOUGHNESS

Anderson and Dodds (*1991*) presented a method to estimate the critical stress intensity of full thickness, in-service ESW. They based it upon J integral test data from the brittle to ductile transition region, using the small scale yielding criteria on sub thickness test specimens. In summary, this method scales the results of sub thickness test specimens to estimate the results for specimens of sufficient thickness to produce small scale yield criteria at the crack tip. If such criteria are met, then a single parameter approach to failure by fracture can be reasonably employed. The scaling parameters are the specimen thickness, b, or in this case net thickness b_n, the crack length ratio a/W, the measured critical J value and the flow stress of the weld metal. Using these data and the finite element analysis (FEA) results presented in Anderson and Dodds (*1991*), the measured critical J values J(Q) can be converted to the expected critical J values for specimens thick enough to enforce small scale yielding at the crack tip. In-service weldments would be expected to produce this value at the LAST.

This small scale yielding toughness is identified in the report as J_{ssy}. Because it is based on a small plastic zone at the crack tip, the crack propagation can be assumed to be stress dominated, as opposed to strain dominated found in more ductile or tough material like the base metal. With this understanding it is appropriate to convert J_{ssy} to an equivalent critical stress intensity, K_C, using the relationship of $K_C = SQRT(E*J_{ssy}/(1-\nu^2))$, which is appropriate for plane strain conditions at the crack tip, where E is Young's Modulus and ν is Poisson's ratio.

Table 4.5 summarizes the conversion of the sub thickness fracture test specimen J results to full thickness K_C results. It can be seen from the J(Q)/J_{ssy} ratio for the weld metal is near unity indicating that the sub thickness test specimens require very little correction to predict in-service thickness fracture toughness. The weld metal shows the lowest toughness with the critical stress intensity ranging from 97 to 104 ksi-in^{½} . Figure 4.20 shows the test results from both the West Linn and West Fremont corrected test data.

Table 4.5: Summary of test result conversion to small scale yielding facture toughness estimates at a LAST of 0°F.

Metallurgical region	Specimen ID#	Test results J(Q) (lbf/in)	Scaling parameter $a\sigma_{flow}$/J(Q)	J(Q)/J_{ssy}	J_{ssy} (lbf/in)	K_C (from J_{ssy}) (ksi-in ^{½})
Base metal	10-WS1	723	131	1.20	603	135
HAZ	6-WS4AB2	2236	43	1.65	1355	203
HAZ	9-WS3AB4	977	99	1.25	782	154
HAZ	11-WS1AB5	1937	50	1.57	1230	193
Weld metal	7-WS4AB1	369	262	1.08	343	102
Weld metal	8-WS3AB5	332	282	1.06	312	97
Weld metal	12-WS1AB4	397	243	1.11	356	104

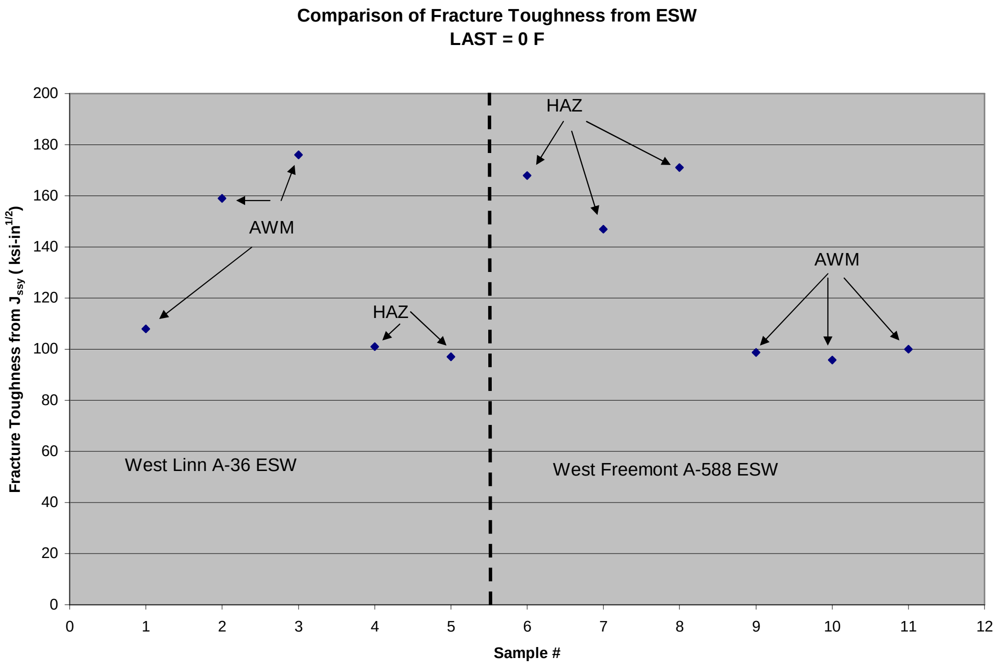


Figure 4.20: Estimated fracture toughness from J-integral testing using small scale yielding adjustments for test specimen thickness.

5.0 NON-DESTRUCTIVE TESTING OF WELDMENTS

As discussed in Section 1.3, all tension and stress reversal flange welds were required to pass both magnetic particle testing (MT) and radiographic testing (RT) during fabrication. Records indicated that all welds, including compression welds, were examined with ultrasonic testing (UT) prior to RT. The RT acceptance criteria are defined in Appendix B of American Welding Society, Specification D2.0-69. Based on the author's knowledge of ODOT staff and procedures during the 1969 thru 1972, when girders were fabricated, all accepted flange welds most likely met the contract requirements.

In 2006, the box girders were given a full NBIS (National Bridge Inspection Standard) fracture critical visual inspection with no defects reported in the fracture critical flange welds (*ODOT 2006*). In addition to the full visual inspection, selected flange welds also received UT and RT in accordance with Section 6 of the AASHTO-AWS D1.5-2004 "Bridge Welding Code". Twenty bottom and 20 top flange ESW were inspected, all in tension or stress reversal regions of the girders. Only 15 of the bottom flanges received the addition RT inspection after UT, due to the top flange being composite with the deck and traffic restrictions. The inspection locations were evenly divided in the four bridge structures under study (*MEI-Charlton Inc. 2007*). Table 5.1 and 5.2 summarize the locations and types of inspections performed.

Table 5.1: Summary of NDT for Bridge Superstructures 09268A and 09268B.

Bridge	Line	Span	Top Flange (UT)			Bottom Flange (UT & RT)		
			Weld ID	Inside	Outside	Weld ID	UT	RT
9268A	SW	2	SW2-A-T1-8	NRI	NRI			
			SW2-B-T1-4	NRI	NRI			
			SW2-B-T1-5	NRI	NRI			
		4	SW4-A-T1-5	NRI	NRI			
			SW4-A-T2-5	NRI	NRI			
		3				SW3-A-B-2	NRI	NRI
						SW3-B-B-2	NRI	NRI
		4				SW4-A-B-2	NRI	NRI
						SW4-B-B-2	note [8]	NRI
		5				SW5-B-B-2	NRI	NRI
9268B	WS	1	WS1-A-T2-4	NRI	NRI			
		3	WS3-A-T1-5	NRI	NRI			
			WS3-B-T1-4	NRI	NRI			
			WS3-B-T2-4	NRI	note [9]			
		4	WS4-A-T2- 2	NRI	NRI			
		1				WS1-A-B-2	NRI	NRI
		3				WS3-A-B-2	note [10]	NRI
						WS3-B-B-2	note [11]	NRI
		5				WS5-A-B-1	NRI	NRI
						WS5-B-B-1	NRI	NRI

Table 5.2: Summary of NDT for Bridge Superstructures 09268E and 09268W.

Bridge	Line	Span	Top Flange (UT)			Bottom Flange (UT & RT)		
			Weld ID	Inside	Outside	Weld ID	UT	RT
9268	WE	3	WE3-D-T2-3	NRI	NRI			
			WE3-E-T1-3	NRI	NRI			
		4	WE4-D-T2-1	NRI	NRI			
			WE4-E-T1-1	NRI	NRI			
		6	WE6-D-T2-1	NRI	NRI			
		3				WE3-D-B-1	note [12]	NRI
						WE3-E-B-2	NRI	NRI
		4				WE4-D-B-2	NRI	NRI
						WE4-E-B-2	NRI	NRI
		6				WE6-D-B-2	NRI	NRI
9268W	EW	3	EW3-D-T1-3	NRI	not insp			
		5	EW5-D-T1-1	NRI	not insp			
			EW5-D-T2-1	NRI	not insp			
			EW5-E-T2-4	NRI	not insp			
		6	EW6-E-T2-1	NRI	not insp			
		3				EW3-D-B-3	NRI	not insp
						EW3-E-B-2	NRI	not insp
		5				EW5-D-B-2	NRI	not insp
						EW5-E-B-2	note [13]	not insp
		6				EW6-D-B-2	NRI	not insp

5.1 ULTRASONIC INSPECTION

To avoid extensive repainting, lead debris containment and traffic disruptions, ODOT requested the coating system be left on welds subjected to UT and/or RT. The UT procedures were adjusted to compensate for the estimated attenuation caused by the coating system. The following paragraph is from MEI-Charlton Inc. (*2007*):

> Of the 20 bottom flange welds and 20 top flange welds (six top flange welds, inside the box girder only) inspected ultrasonically, five welds -all on the bottom flanges- had relevant indications per the reporting requirements of AWS D1.5. Of these, the indications on three of the welds were reportable but not rejectable per the requirements of AWS D1.5. The indications on the other two welds were rejectable per the requirements of AWS D1.5.

Table 5.3 characterizes the flaw severity for Ultrasonic inspection. Figures 5.1 and 5.2 show the UT inspection form for the two welds with rejectable UT indications. Note that the indication locations are within 1/16” to 1/32 ” of the bottom surface of the bottom flange. A more detailed discussion of the UT indications summarized in the *MEI-Charlton report* (*2007*).

Table 5.3: Summary of UT flaw severity per AWS D1.5.

Flaw Severity Class	Acceptance/Rejection Criteria ·
A	Any indication in this category shall be rejected (regardless of length).
B	Any indication in this category having a length greater than 3/4 inch shall be rejected
C	Any indication in this category having a length greater than 2 inches in the middle half or 3/4 inch length in the top or bottom quarter of the weld thickness shall be rejected
D	Any indication in this category shall be accepted regardless of length or location in the weld

NPI

NDE Professionals, Inc., 7505 NE Ambassador Place, Suite N
Portland, OR 97220
Office (503) 287.5255 Fax (503) 287.5992

ULTRASONIC TEST REPORT

CUSTOMER: **MEI Charlton, Inc.** DATE : **10/13/06**

DESCRIPTION **Electorslag Welds** JOB#:

PART SN: **405 Bridge Portland Oregon – Freemont Bridge**

Technique

SPEC: **AWS D1.5 - 2002**
ACCEPTANCE STANDARD: **Tension Criteria (Fracture Critical)**
PROCEDURE: **TIP-UT-103.0**
TECHNIQUE: **Contact Table 6.2 from Face A**
EQUIPMENT: **Panametrics Epoch IV**
SERIAL #: **011362910** CAL. DUE: **10/30/06**
CAL. BLOCK: **IIW**
COUPLANT: **UT-X**
TRANSDUCER **Panametrics**
ANGLE/ FREQ.: **0°,60°,70°**
MATERIAL: **Carbon Steel**
MATERIAL THICKNESS: **0.80" to 2.85"**
JOINT TYPE: **BUTT**
WELD IDENTIFICATION: **SEE BELOW**

RESULTS

Actual thickness 0.83" to 0.83"

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	From X	From Y	Discontinuity. Evaluation	Remarks
SW2 BT14														No Indications/Inner flange
SW2 BT15														No Indications/Inner Flange
SW3 BB2														No Indications
SW4 BB2	1	70	A	59	41.5	3	14.5	2-1/4"	2.429"	.831"	+1/4"	1-3/8"	Class D	Acceptable
SW5 BB2														No Indications
WE3 DB1	1	70	A	60.2	41.5	5	13.7	1"	3.348"	1.145"	+1/2"	1-3/4"	Class D	Acceptable
EW5 EB2	1	70	A	52.7	41.5	2	9.2	3-1/2"	2.185"	.747"	+3/8"	0	Class B	Rejectable
	2	70	A	52.4	41.5	3	7.9	3"	4.496"	.806"	0	43"	Class A	Rejectable
EW5 DT11														No Indications/Inner Flange
EW3 DT13														No Indications/Inner Flange
WE6 DB2														No Indications
WE4 DB2														No Indications

Remarks: All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ dB was determined by procedure in AWS Ref: D1.1-2004- para. K6.1 .

UT Level II Technician: ______ **Date:** 10/27/06

Figure 5.1: UT inspection report for weld EW5EB2. (It was later determined the 3 dB attenuation factor for indication 2 was incorrectly reported and should be 7 dB. SCL 11/20/08)

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Portland, OR 97220
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ULTRASONIC TEST REPORT

CUSTOMER: **MEI Charlton,Inc.** DATE : **10/12/06**

DESCRIPTION: **Electorslag Welds** JOB#:

PART SN: **405 Bridge Portland Oregon – Freemont Bridge**

Technique

SPEC:	**AWS D1.5 - 2002**	ACCEPTANCE STANDARD:	**Tension Criteria (Fracture Critical)**
PROCEDURE:	**TIP-UT-103.0**	TECHNIQUE:	**Contact Table 6.2 from Face A**
EQUIPMENT:	**Panametrics Epoch IV**	SERIAL #:	**011362910** CAL. DUE: **10/30/06**
CAL. BLOCK:	**IIW**	COUPLANT:	**ULTEX**
TRANSDUCER	**Panametrics**	ANGLE/ FREQ.:	**0°,60°,70°**
MATERIAL:	**Carbon Steel**	MATERIAL THICKNESS:	**0.80" to 2.85"**
JOINT TYPE:	**BUTT**	WELD IDENTIFICATION:	**SEE BELOW**

RESULTS

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	from X	From Y	Discontinuity. Evaluation	Remarks
WS3 AB2	1	70	A	65.9	45.1	7	13.8	¾"	4.297"	1.413	0	12-7/8"	Class D	Acceptable
	2	70	A	62.3	45.1	7	10.2	5/8"	4.262"	1.402	0	2-1/4"	Class C	Acceptable
	3	70	A	62.1	45.1	7	10	1-1/4"	4.285"	1.409	0	4"	Class C	Rejectable
WS3 BB2	1	70	A	60.1	45.1	4	11	1/8"	3.131"	.970	0	4-3/4"	Class D	Acceptable
WS1 AB2														No Indications
WS5 AB1														No Indications
WS5 BB1														No Indications
WS1 AT24														No indications inner flange
WS3 BT14														No indications inner flange
WS3 BT24														No indications inner flange
WS4 AT22														No indications inner flange

Remarks: All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ dB was determined by procedure in AWS Ref: D1.1-2004- para. K6.1 .

Level II Technician: **Date:** 10/27/06

Figure 5.2: UT inspection report for weld WS3AB2.

5.2 RADIOGRAPHIC INSPECTION

The following paragraph is from MEI-Charlton Inc. (*2007*):

> Of the 15 bottom flange welds inspected radiographically, none had relevant or reportable indications. It is important to note that these 15 welds included the three welds with reportable-but-not-rejectable UT indications and one of the two welds with rejectable UT indications. That is, no reportable radiographic indications were found in any weld with UT indications which were also radiographed (amounting to 4 of the 5 welds with UT indications. Thus, the ultrasonic indications in these welds can likely be attributed to either paint-related coupling issues or non-significant discontinuities in the ultrasonic properties, rather then actual flaws in the material.

6.0 FITNESS FOR PURPOSE EVALUATION

6.1 OVERVIEW OF EVALUATION

In 1977, as a result of an earlier bridge girder failure, Federal Highway Administration (FHWA) issued Notice N5040.23 directing state highway departments to examine in detail ESW in fracture critical bridge members. A detailed fracture control plan was developed by Carl Hartbower for assessing the safety of these structures and distributed by FHWA. A portion of this plan was executed in 1978 and 1979. The recommendation of this plan is contained in Section 8.

In a nutshell, the West Fremont approach structures contain fracture critical weldments that typically show substandard toughness or crack tolerance, when compared to newer bridges designed and fabricated after the development and execution of the FHWA Fracture Control Plan (FCP). The FCP is based on Clause 12 of the AASHTO/AWS D1.5 Bridge Welding Code, and imposes toughness requirements on bridge steels and welds. These requirements, if followed, almost guarantee ductile failure modes in all critical steel bridge members. Clouding this issue is the fact that standard gap ESW has a very course grain structure that can lead to UT indications, but that are not identified by RT.

The purpose of this evaluation was to consider:

- possible crack-like defects in the welds,
- the weld metal's ability to tolerate cracks,
- whether service loading stresses may cause cracks to grow and
- develop a fracture control plan to insure public safety and structure serviceability with reasonable inspection costs.

6.2 STANDARD GAP ESW ACCEPTANCE CRITERIA

In addition to the fracture control plan developed by Carl Hartbower and distributed to State DOT's via the FHWA, the Transportation Research Board (TRB) published further guidelines for making such assessments with the NCHRP Report 201, "Acceptance Criteria for Electroslag Weldments in Bridges" (*Benter and Shilling 1979*). The acceptance criteria are summarized below:

- The minimum requirements of the American Welding Society AWS D1.1 Structural Welding Code should be met.
- For tension members in bridge applications, mid-thickness weld metal Charpy V-notch (CVN) impact energy should average at least 15 ft-lbf at 0 ^{o}F with no single value lower than 10 ft-lbf and the heat affected zone (HAZ) at quarter thickness

should provide average impact energy of at least 15 ft-lbf at 40 ^{o}F, as required for the base metal.

During the fabrication of these structures all tension and stress reversal welds passed AWS D2.0-69 RT requirements. Contract specifications required that the weld metal qualification test produce an average CVN energy of 20 ft-lbf at 0 ^{o}F, as opposed to15 ft-lbs required by AWS D2.0-69 and NCHRP 201. The HAZ was actually tested at 30 ^{o}F instead of the 40 ^{o}F specified in NCHRP 201. The welding procedure qualification records for the ESW used to fabricate these structures are shown in Figures 6.1 and 6.2, which show that both electro-slag welding procedures met contract requirements. Based on the contract documents, these bridges meet and exceed the acceptance criteria presented in NCHRP 201. However, as discussed in Section 4.4.4 and shown in Figure 6.3, the CVN specimens from Phase 1 test cores failed the weld metal specifications at 0 ^{o}F.

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REPORT OF IMPACT TESTS

PROJECT: 4901

DATE January 16, 1970

REPORTED TO: Canron Ltd., 145 West 1st Ave., Vancouver, B.C.

FILE 480-129-C.4

P/O 4140

Report No. 10/70

TYPE OF TEST: Charpy Impact

SPECIMEN SIZE: Standard 10x10 mm

TYPE OF SPECIMEN: V-Notch TESTED @ 0°F & +30°F TEMP.

SPECIFICATION: AWS 1-67

MATERIAL: ASTM A588(C) Electroslag (2-3/4" Plate)

SPECIMEN PREPARED BY:

TEST RESULTS

weld metal 0°F Specimen Mk.	No.	Impact Value in ft. lbs.	heat-affected zone +30°F Specimen Mk.	No.	Impact Value in ft. lbs.	Specimen Mk.	No.	Impact Value in ft. lbs.
2V	1	40	2Y	1	15.5			
	2	35		2	25.5			
	3	43.5		3	21.5			
	4	42		4	15			
	5	33		5	21			

20 ft. lbs. reqd. 15 ft. lbs. reqd.

RECEIVED JAN 22 1970 BRIDGE DIVISION

REMARKS:

TESTED BY: Howard Grisack

WITNESSED BY: Dwayne Peterson - Pittsburg Testing
Frank Graham - Canron

COAST ELDRIDGE

Brian T. Shankey, Supervisor
Physical Testing Services

PER

Figure 6.1: Impact test results for ESW Procedure Qualification Record (PQR) for thick plates.

FORM NO.

COAST ELDRIDGE

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REPORT OF IMPACT TESTS

PROJECT: 4901

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145 West 1st Ave.,
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DATE January 16, 1970

FILE 480-129-C.4

P/O ~~xxx~~ Report No. 8/70
~~P.O. 4140~~

TYPE OF TEST: Charpy Impact

SPECIMEN SIZE: 10x10 mm Standard

TYPE OF SPECIMEN: V-Notch TESTED @ 0°F & +30°F TEMP.

SPECIFICATION: AWS 1-67

MATERIAL: ASTM A588(C) Electroslag (1½" Plate)

SPECIMEN PREPARED BY: Client

TEST RESULTS

weld metal 0°F SPECIMEN MK.	NO.	IMPACT VALUE IN FT. LBS.	heat affected zone +30°F SPECIMEN MK.	NO.	IMPACT VALUE IN FT. LBS.	SPECIMEN MK.	NO.	IMPACT VALUE IN FT. LBS.
V	1	27	Y	1	28			
	2	49		2	21			
	3	22		3	18.5			
	4	18		4	22.5			
	5	25		5	27.5			
Spec.:		[illegible]0 ft.lbs.			15 ft.lbs.			

RECEIVED
JAN 28 1970
BRIDGE DIVISION

REMARKS:

TESTED BY: H. Grisack

WITNESSED BY: Dwayne Peterson - Pittsburg Testing
Frank Graham - Canron

/nb

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PER Brian T. Shankey, Supervisor
Physical Testing Services

Figure 6.2: Impact test results for ESW PQR for thin plates.

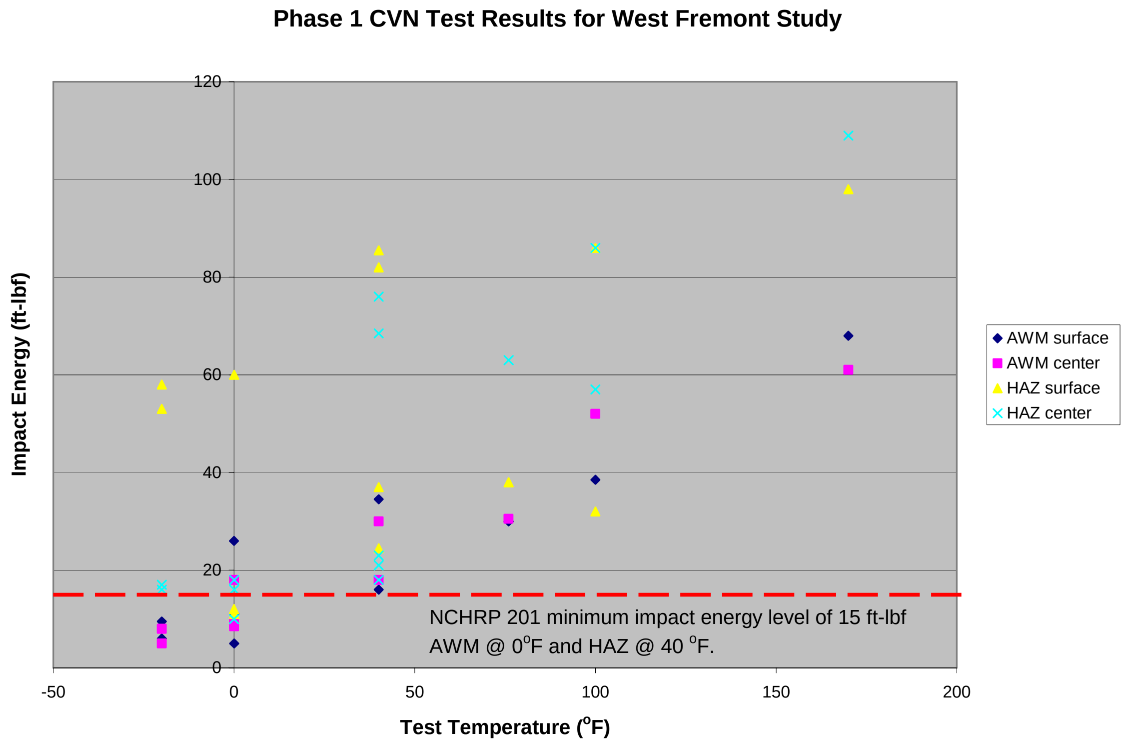


Figure 6.3: Impact test results from Phase 1 testing on West Fremont study.

6.3 FAILURE ASSESSMENT

The main concern about standard gap ESW is its low toughness compared to other common processes. Poor toughness can result in rapid and unstable crack growth, as in the liberty ship failures of the 1940's. High toughness welds can tolerate routine fabrication defects, stress concentrations caused by poor detailing, or distortion induced cracking under service conditions. With sufficient material fracture resistance, large cracks could exist in a member, but it would still fail by ductile yielding of the remaining section without brittle behavior.

Low toughness materials are less tolerant of these deficiencies and under the right conditions, can fail in a brittle and rapid manner at a load far below general yielding.

For this analysis a Failure Assessment Diagram (FAD) is appropriate. This approach looks at both ends of the failure mode spectrum and the interaction of both ductile and brittle failure modes. The FAD used in this analysis is based on a modified form of the strip yield approximation, first proposed by Dugdale (*1960*) and Barenblatt (*1962*). The two-criterion FAD approach was first proposed by Dowling and Townely (*1975*) and Harrison, et al. (*1976*). A failure envelop is defined in terms of the critical stress and fracture ratios S_r and K_r respectively, both of which are defined in Section 6.3.1. For materials with high toughness, failure in tension occurs when the tensile stress reaches the flow stress. If cracks are present they are accounted for by a reduction in the net section, occurring when S_r equals 1.0. For brittle materials, failure occurs when the combination of stress and flaw size produce a critical stress intensity at the crack resulting in fracture when K_r equals 1.0. For intermediate cases, gross yielding and facture interact, and both S_r and K_r are less then 1.0 at the point of failure. Points inside the envelope are predicted to not fail in-service, but points on or outside the failure envelop risk some combination of ductile and/or brittle failure.

6.3.1 Ductile Failure Mode

The ductile failure mode for the box girder flanges could yield the gross section, if no critical defects are present. In this failure mode, the critical stress for the flange plates, σ_c, can be considered to be the flow stress of the base metal. Once this stress level has been reached, the plate cannot carry additional load. For both the A588 base metal and electro-slag weld metal, this flow stress is 70 ksi, as determined by tension testing. For a ductile failure with gross section yielding, the base and weld metal will have similar capacities. The maximum in-service stress on the flange is caused by the sum of the dead, thermal, live and wind loads. As discussed earlier in the report, based on the original design, routine stress levels are assumed to be less than or equal to 55% of the base metal minimum yield strength or 27.5 ksi. Thus, for a completely ductile failure of the flange plates, the worst case stress ratio (S_r) is 0.39, where S_r is the ratio of the maximum predicted stress to the ductile failure stress Equation 6-1.

$$\mathbf{S_r = maximum\ service\ stress\ /\ ductile\ failure\ stress\ = 0.39} \qquad \mathbf{(6\text{-}1)}$$

6.3.2 Brittle Failure Mode

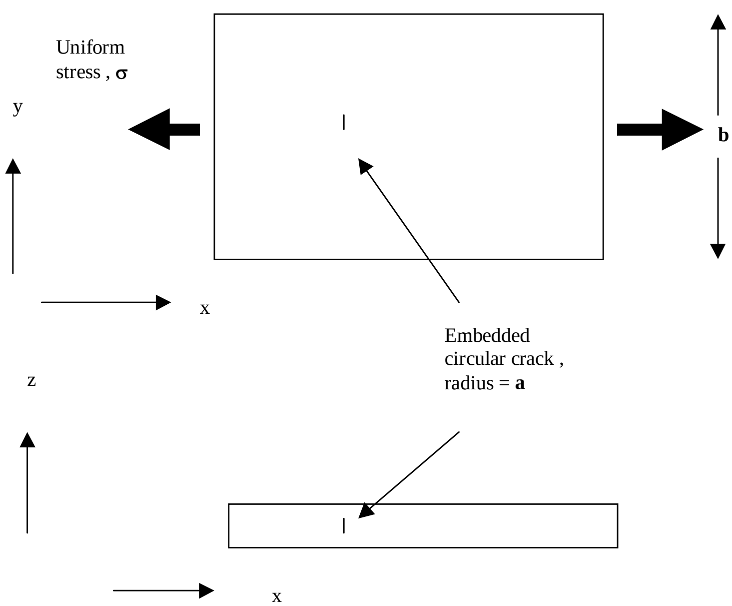
Brittle failure of a flange would probably entail a critical-sized flaw within an ESW, since they have inferior crack tolerance compared to the base metal. Lovejoy (*2002*) outlined the response of the weld metal to a crack or other severe defect as being best represented using the Linear Elastic Fracture Mechanics (LEFM approach, which attributes crack growth, both stable and unstable, to the single parameter term (K), the stress intensity. The term K represents a function of applied stress around the crack tip and the size and shape of the crack. In a spatially constant stress field, as the crack gets bigger the stress intensity raises until it reaches a critical level and then unstable crack growth occurs, which generally causes complete failure in a brittle material.

The critical K level for brittle fracture is often a function of plate thickness; the crack tip's surrounding stress field, strain rate and temperature. It is not an intrinsic material property like the elastic modulus (E) or Poisson's ratio (ν). Three and one half inch diameter cores were centered on various metallurgical regions (base metal, HAZ and the ESW) in order to prepare specimens (see Figure 4.10) that would best predict flange toughness at the lowest anticipated service temperature and expected strain rates. The lowest toughness found in weld metal with a critical stress intensity was K_{Jssy}, of 97 ksi-in $^{1/2}$, as determined from brittle failure modes in J-Integral testing (J-controlled cleavage).

To quantify the maximum stress intensity in service conditions, two parameters must be evaluated: the flaw characterization, including size, shape and location and the maximum stress at the flaw. As in the ductile failure mode analysis above, the maximum expected far field stress from a global view of the structures is 27.5 ksi. As discussed in Section 3.3 the weld residual stresses are probably much higher, but are neglected for this scenario. As described in Section 5.1, the rejectable indications identified are not thought to be crack-like (i.e. having an infinitesimally sharp edge), but statistically, crack-like flaws may exist in these structures. The maximum crack-like flaw size escaping Non-destructive Testing (NDT) is estimated, by the author, at less than a ½ inch in length. Based on the ability to perform surface inspection of the welds (VT and MT), and the compressive stresses on and near the surface of these ESW, the most likely critical flaw location to escape detection is in the core of the weld.

Assuming a circular or penny shaped crack of **radius a**, in a plate of finite (**width b)**, the maximum stress intensity (K), can be expressed as shown in Figure 6.4. With a ½ inch diameter crack and a far field stress of 27.5 ksi, K = 15.8 ksi-in ^{½}. The brittle failure stress intensity, K_{Jssy}, is 97 ksi-in^{½} and the stress intensity ratio, K_r, is equal to K/ K_{Jssy} (Equation 6-2).

$$K_r = K(\text{ service stress intensity }) / K_{Jssy} (\text{ critical stress intensity }) = 15.8 \text{ ksi-in}^{1/2} / 97 \text{ ksi-in}^{1/2} = 0.16 \quad \textbf{(6-2)}$$



$$K_I = 1.13 \ \sigma \ f(a/b) \sqrt{a}$$

Figure 6.4: Stress Intensity definition for an embedded circular crack in a finite width plate.

6.3.3 Example of the probable maximum service condition

$K_r = .16$ and $S_r = 0.39$ are the probable maximum service condition. In case some welds exhibit even lower toughness than predicted by K_{Jssy} (K-controlled cleavage), and K_Q is a better representation of the actual toughness then the K ratio would be about twice as large or $K_r = 0.32$

6.3.4 Example representing the extreme service condition

$K_r = .32$ and $S_r = 0.39$ represent the extreme service condition. Figure 6.5 shows the failure assessment diagram (FAD) for the strip yield model of a through thickness crack as developed by Dowling and Townley (*1975*), and Harrison et al. (*1976*). The vertical axis defines brittle failure modes and the horizontal axis defines ductile failure modes. The plot shows the locus of predicted points of failure. All points inside the line are predicted not to fail and those outside the line are expected to fail.

The flanges probably operate in the region shown by the smaller triangle. This is a realistic estimate of maximum service conditions based on a ½ inch diameter crack escaping NDT, the maximum design loads occurring, and the lowest toughness measured in the welds. The second triangle area is for the same flaw and loads, but with extremely brittle behavior in the welds with the toughness being near 50 ksi-in $^{1/2}$. None of the test data indicate a toughness this low, but only 6 toughness tests were used to quantify 261 fracture critical welds. It is possible for the toughness of some welds to be this low near the lowest anticipated service temperature (LAST).

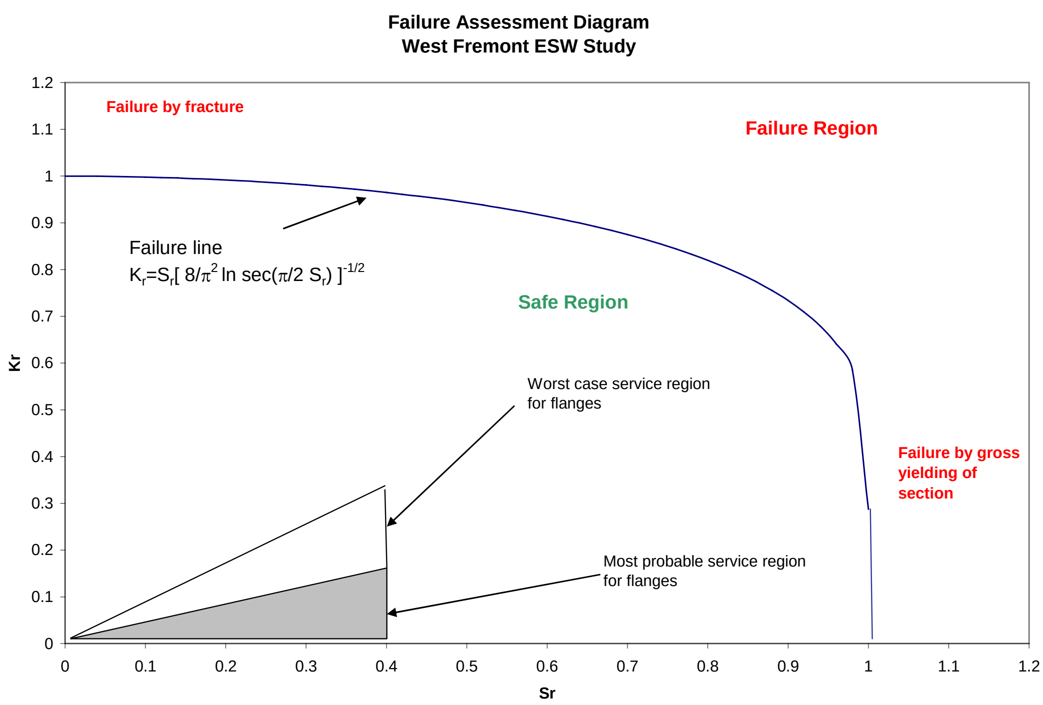


Figure 6.5: Failure Assessment Diagram (FAD) for the flange plates on the West Fremont approach structures. Both the most likely and worst case operating regimes are shown schematically.

6.3.4.1 Estimated Critical Crack Size

To estimate the critical crack size considering only LEFM, a full thickness through crack could be assumed in a lower flange. If an overload event causes the crack to increase in width, reducing the net section in the flange, the net section stress rises. The stress intensity for such a crack is shown in Figure 6.6. At the minimum estimated weld metal fracture toughness of 97 ksi-in$^{1/2}$, the crack would need a 6 inch width in the 46 inch wide flange to produce 31.6 ksi on net section and become critical at the maximum design stress of 27.5 ksi on the full section. Presuming a maximum ½ inch size crack-like defect that might escape NDT, the safety factor against brittle fracture is 3.9, as depicted in the figure.

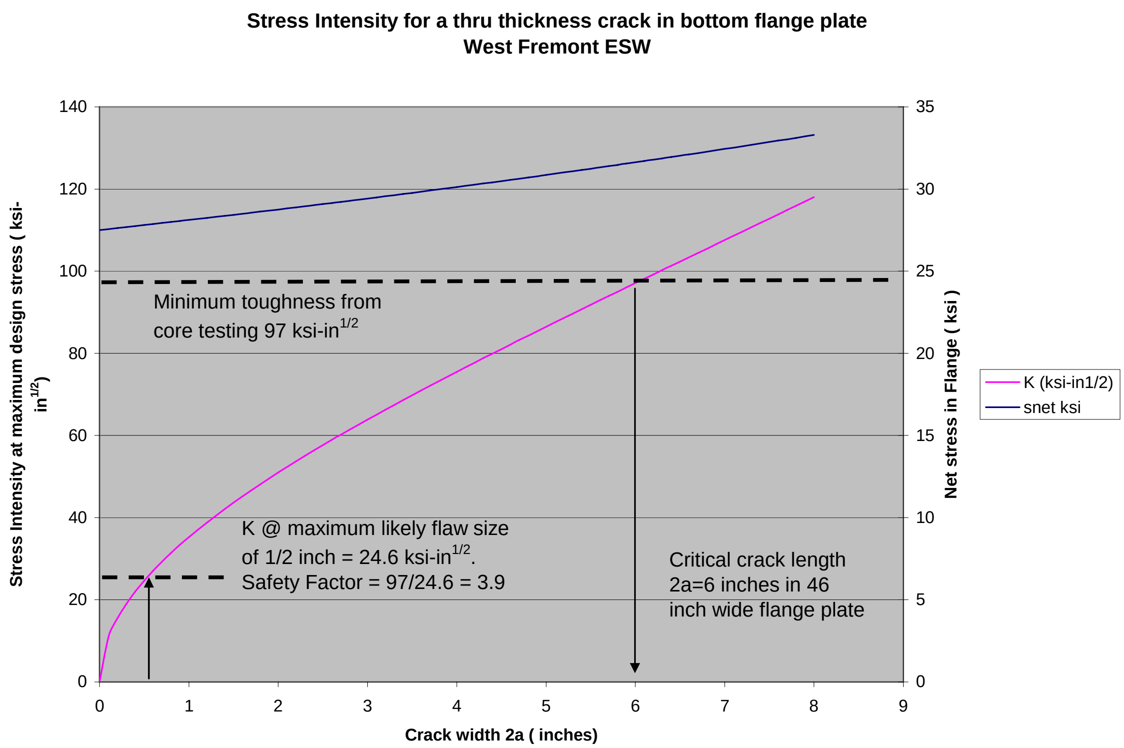


Figure 6.6: Stress intensity from a through crack in bottom flange plate.

6.3.5 Defects located in High Residual Stress Zones

Another consideration is the potential effect of ESW residual stresses on the defect. The failure scenario in 6.3.2 mentioned the sub-critical defect being affected by extreme service loads and residual stresses, but neglected the residual stresses for the calculation of critical crack size. The load analysis in Section 3.3 indicated that the outer shell or fusion zone of the standard gap ESW was subjected to large compressive residual stresses and inferred that the weld core was subject to large magnitude, triaxial tensile stresses.

These weld regions have steep spatial, multi-axis stress gradients typically not addressed in structural designs, but assumed to develop at least the same strength as the base metal. Experience has generally supported this assumption. However, given the lower toughness of the weld metal, the course grain structure, which could present crack initiation sites and the high triaxiality of the stress, which decreases ductility, the potential for a subcritical flaw in the weld core growing into a critical length crack must be addressed.

Standard gap ESW has several different crystalline grain structures depending on the welding parameters and cooling rates (*Culp 1979*). These typically have long columnar grains in the middle of the weld metal. Assuming adjacent boundaries of some larger grains provide an embedded elliptical discontinuity 1.5 inches along the weld axis and 0.25 inches high, perpendicular to the plate surface (2a=0.25", 2c=1.5",σ/σ_{ys}=1.0,Q=0.85 using conventional LEFM nomenclature), the calculated approaches 34 ksi-in ^{½} if near yield weld metal residual stresses are assumed. With a critical stress intensity of 97 to 175 ksi-in ^{½}, even such high residual stress would not be expected to propagate the assumed discontinuity into a crack.

6.4 FATIGUE CRACK GROWTH

Fatigue crack growth may progress through three regimes of behavior: initiation, stable growth and unstable growth. Initiation is the process of transforming a flaw into an actively growing fatigue crack. The flaw may be very small or relatively large, and planar or rounded: size and geometry each affect stress concentration and the likelihood of initiating the next stage of crack growth. In welded structures the initiation portion of the fatigue life is often neglected under the conservative assumption that crack-like flaws may have escaped NDT. Once a crack is established in a structural component, it will grow in a stable manner provided that the stress intensity range (ΔK) at the crack tip(s) is greater than a threshold value (ΔK_{th}) and that the incremental growth of the crack sufficiently relieves stress so the material toughness can arrest its progress. The combined dead and live load stresses on fracture critical welds are intermittently or continuously tensile and the residual stresses are assumed to be tensile at the core of the weld. Thus, if the applied tensile stress and flaw susceptibility are sufficient, a small defect could propagate into a fully active fatigue crack.

The NDT performed on this structure will very likely detect discontinuities less than 0.5 inch in dimension. The maximum tensile stress range due to applied loads was found to be less than 2 ksi at any ESW location. The stress intensity range from this combination of flaw size and stress range is 1.1 ksi-in^{½} or less. The lowest crack growth threshold measured from the cores was 8.45 ksi-in^{½}. Thus, the largest flaw expected to be missed by NDT would not be expected to grow or propagate due to the anticipated live load stress range.

From another perspective, based on the measured ΔK_{th}, thru-thickness crack up to 5 inches long in the bottom flange would be expected to remain dormant under current live loads. Thus, developing and propagating an active fatigue crack within a flange ESW is very unlikely. Proper NDT can assure this with a very high probability of success.

6.5 S-N APPROACH

Looking at ESW from a design perspective, the fatigue life is typically characterized by the number of stress cycles at a particular stress amplitude. Prior research (*Shilling and Kippstein 1981*) has shown that standard gap ESW can be characterized as an AASHTO fatigue Category B at constant thickness butt joints and Category C for thickness transition butt joints. Category C also applies to the studs welded to the top flanges for composite action. The constant amplitude fatigue limit (CAFL) for more than 2 million stress cycles is 16 ksi for Categories B and 9 ksi for Category C.

Using the fatigue life procedures suggested by Yen, et al. (*1990*) the measured effective stress range should be compared with the CAFL. The largest effective stress range measured at the 64 strain gauge locations was 2.0 ksi. The maximum stress cycle measured had a range of 5.8 ksi. Both of these parameters are substantially below the CAFL and infinite fatigue life is therefore predicted under the current loading.

7.0 SUMMARY AND CONCLUSIONS OF STUDY

7.1 SUMMARY

A fitness-for-purpose evaluation was performed on the flange butt welds on the West Fremont approach bridges as per FHWA notices N5040.23 and 5040.29, dated 2/16/77 and 2/23/77, respectively, as well as Memorandum HNG-30, dated 6/21/78. This requires defining the material properties, the flaw or defect population from fabrication and service load stresses on the weldments. With these data the serviceability of the weldments can be assessed and repair or inspection requirements can be developed to assure public safety.

Service Loads –To date, a formal load rating or seismic analysis has not been performed for these four bridge structures. The maximum design stress for the box girder flange plates is less then or equal to 55% of the minimum yield stress of the base metal or 27.5 ksi. Structural load testing concluded that the maximum thermal stress range is approximately 3.0 ksi, the maximum single event live load is less than 6 ksi and the root-mean-cube (rmc) effective stress range is less then 2.0 ksi.

Fabrication Stresses – Residual stresses from the welding process were shown by previous studies (*Lovejoy 2002*) to have a core of triaxial tension and a compressive outer shell surrounding the weld region. The magnitude of these stresses was near the yield strength of the weld metal.

Material Properties – The tensile, toughness and crack growth threshold properties of the base, HAZ and weld metals were quantified. The lowest measured fracture toughness as defined by the small scale yielding criteria applied to ASTM E 1820 test data was found to be 97 ksi-in^{½} in the weld metal at 0ºF. The minimum fatigue crack growth threshold measured, as defined by ASTM E 647, was 8.4 ksi-in^{½} in the weld metal and 7.1 ksi-in$^{1/2}$ in the base metal.

Non-destructive Testing – Based on the original fabrication inspection documents and UT / RT / VT in 2006, the most likely size of crack-like weld defects to escape inspection is less than ¼ inch in maximum dimension. The largest flaw anticipated to escape detection is ½ inch in minimum dimension. Two of the 20 fracture critical ESW inspected contained UT rejectable defects based on the AASHTO/AWS D1.5-2002 Bridge Welding Code. One of these welds with UT rejectable defect indications was inspected with RT and no defects were identified. Based on radiographic testing and detailed analysis of the UT results, these defect indications were not considered to be crack-like.

Failure Analysis – Based on the quantified data above, a failure assessment was made considering two modes of failure – brittle and ductile. Based on the strip yield theory of combined failure modes, the ESW were found to be very unlikely to fail under even the most extreme combination of service loads and relatively low material crack tolerance. Critical crack dimensions were on the order of four inches for an assumed through-thickness crack in a finite width plate. Given the maximum crack size to escape NDT, the fatigue crack growth threshold and service loads, fatigue crack growth under live load was not anticipated for the service life of this structure. From a traditional Stress Life (S-N) approach, infinite fatigue life was predicted based on comparison with fatigue testing of similar ESW weldments.

7.2 CONCLUSIONS

A number of conclusions resulted from the study. The conclusions are summarized as follows:

- The ESW metal had equal strength yet slightly less ductility than the base metal. The toughness was substantially lower than the base metal, yet adequate to sustain anticipated service conditions.
- The fatigue crack growth threshold indicated that fatigue crack growth was very unlikely under service loads.
- The fabrication and inspection quality of these welds were very good. A crack-like flaw larger than ½ inch is unlikely to exist in any of the ESW, they are probably less than ¼ inch.
- The bottom flange welds experienced higher fatigue stress ranges than the upper flange welds.
- From a conventional S-N approach the ESW are expected to have an infinite fatigue life.
- Retrofitting of the ESW is not necessary nor is it recommended.
- A rigorous yet reasonable NDT program can insure public safety.

8.0 FRACTURE CONTROL PROGRAM FOR THE BOX GIRDERS

8.1 OVERVIEW

Between 1977 and 2007, ODOT carried out the fracture control program for the fracture critical ESW in the box girders of the West Fremont Approach Bridge at Portland, Oregon. This plan characterized the fracture potential of these welds by quantifying the structural loading, material toughness and weld defects present. The results of this study concluded that the ESW were fit for continued service without any modifications or repairs. Under the National Bridge Inspection Standards (NBIS) act of 1967, the box girders are required to be visually inspected by a licensed lead bridge inspector every two years. Details of fracture critical inspections can be found in (*ODOT 2006*).

8.2 FRACTURE CONTROL PLAN FOR REMAINING LIFE OF STRUCTURE

In addition to the biennial NBIS inspections, the box girders shall receive the inspection described below at the specified period.

Using the fracture critical weld identification tables found in Appendix A of this report all fracture critical flange butt welds shall receive a hands on visual inspection meeting the requirements of the NBIS for Fracture Critical Inspections. Both the interior and exterior surfaces of the welds shall be fully examined with the exception of the portion of the top flanges that are covered by the concrete deck.

Any crack-like defects discovered by VT shall be followed up with MT and UT. If UT discloses rejectable defects in the bottom flanges then RT will be employed to resolve the UT defects. UT shall be performed by an ASNT level 2 inspector.

This special fracture critical inspection shall be performed cyclically, with a period of time between inspections not exceeding 6 years (every 3rd biannual inspection).

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APPENDIX A: FRACTURE CRITICAL FLANGE WELD IDENTIFICATION

Welds that are highlighted are considered to be fracture critical by the Agency.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 1 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
SW1-A-T1-1	41	1.00	7	1.00	7
SW1-A-T1-2	84	1.00	7	1.00	8 1/2
SW1-A-T1-3	142	1.50	8 1/2	1.75	10 1/2
SW1-A-T1-4	153	1.77	10 1/2	2.15	10 1/2
Span 1 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
SW1-A-T2-1	41	1.00	7	1.00	7
SW1-A-T2-2	85	1.00	7	1.00	8 1/2
SW1-A-T2-3	142	1.50	8 1/2	1.75	10 1/2
SW1-A-T2-4	153	1.77	10 1/2	2.15	10 1/2
Span 1 Girder A Flange B (distance measured from bulkhead of expansion joint No. 8)					
SW1-A-B-1	41	1.15		1.15	
SW1-A-B-2	86	1.18		1.18	
SW1-A-B-3	152	1.43		1.75	
Span 1 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
SW1-B-T1-1	39	1.05	7	1.05	7
SW1-B-T1-2	82	1.05	7	1.07	7
SW1-B-T1-3	117	1.05	7	1.27	9
SW1-B-T1-4	140	1.27	9	1.68	10 1/2
SW1-B-T1-5	148	1.7	10 1/2	2.17	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 1 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
SW1-B-T2-1	39	1.05	7	1.05	7
SW1-B-T2-2	82	1.05	7	1.07	7
SW1-B-T2-3	117	1.05	7	1.27	9
SW1-B-T2-4	140	1.27	9	1.68	10 1/2
SW1-B-T2-5	148	1.68	10 1/2	2.13	10 1/2
Span 1 Girder B Flange B (distance measured from bulkhead of expansion joint No. 8)					
SW1-B-B-1	43	1.17		1.17	
SW1-B-B-2	84	1.17		1.17	
SW1-B-B-3	148	1.45		1.78	
Span 2 Girder A Flange T1 (distance measured from centerline of pier No. 7)					
SW2-A-T1-1	9	2.18	NR	1.77	NR
SW2-A-T1-2	18	1.80	NR	1.52	NR
SW2-A-T1-3	36	1.55	NR	1.03	NR
SW2-A-T1-4	58	1.03	NR	1.53	NR
SW2-A-T1-5	90	1.55	NR	1.55	NR
SW2-A-T1-6	123	1.50	NR	1.0	NR
SW2-A-T1-7	155	1.55	NR	2.05	NR
SW2-A-T1-8	167	2.05	NR	2.55	NR

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 2 Girder A Flange T2 (distance measured from centerline of pier No. 7)					
SW2-A-T2-1	9	2.13	NR	1.80	NR
SW2-A-T2-2	18	1.78	NR	1.55	NR
SW2-A-T2-3	36	1.55	NR	1.05	NR
SW2-A-T2-4	58	1.05	NR	1.53	NR
SW2-A-T2-5	90	1.55	NR	1.53	NR
SW2-A-T2-6	123	1.50	NR	1.03	NR
SW2-A-T2-7	156	1.55	NR	2.05	NR
SW2-A-T2-8	168	2.05	NR	2.52	NR
Span 2 Girder A Flange B (distance measured from centerline of pier No. 7)					
SW2-A-B-1	9	1.78		1.42	
SW2-A-B-2	18	1.43		1.20	
SW2-A-B-3	155	1.12		1.55	
SW2-A-B-4	167	1.55		2.05	
Span 2 Girder B Flange T1 (distance measured from centerline of pier No. 7)					
SW2-B-T1-1	12	2.18	10	1.68	10
SW2-B-T1-2	24	1.70	9.5	1.25	8
SW2-B-T1-3	87	0.80	7	0.80	7
SW2-B-T1-4	155	1.25	8 1/2	1.65	10
SW2-B-T1-5	166	1.68	10	2.15	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 2 Girder B Flange T2 (distance measured from centerline of pier No. 7)					
SW2-B-T2-1	12	2.18	10	1.68	10
SW2-B-T2-2	24	1.68	9 1/2	1.28	8
SW2-B-T2-3	87	0.80	7	0.80	7
SW2-B-T2-4	155	1.25	8 1/2	1.65	10
SW2-B-T2-5	166	1.68	10	2.15	10
Span 2 Girder B Flange B (distance measured from centerline of pier No. 7)					
SW2-B-B-1	12	1.78		1.43	
SW2-B-B-2	24	1.45		1.05	
SW2-B-B-3	54	1.03		1.58	
SW2-B-B-4	121	0.83		1.01	
SW2-B-B-5	155	1.05		1.43	
SW2-B-B-6	166	1.45		1.77	
Span 3 Girder A Flange T1 (distance measured from centerline of pier No. 6)					
SW3-A-T1-1	10	2.52	NR	2.05	NR
SW3-A-T1-2	22	2.05	NR	1.55	NR
SW3-A-T1-3	78	1.40	NR	1.40	NR
SW3-A-T1-4[1]	125	1.40	NR	1.40	NR
SW3-A-T1-5	136	1.40	NR	1.02	NR

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder A Flange T2 (distance measured from centerline of pier No. 6)					
SW3-A-T1-1	11	2.52	NR	2.05	NR
SW3-A-T1-2	23	2.05	NR	1.55	NR
SW3-A-T1-3	78	1.50	NR	1.40	NR
SW3-A-T1-4	125	1.40	NR	1.40	NR
SW3-A-T1-5	136	1.40	NR	1.03	NR
Span 3 Girder A Flange B (distance measured from centerline of pier No. 6)					
SW3-A-B-1	10	2.05		1.52	
SW3-A-B-2	78	1.52		1.57	
Span 3 Girder B Flange T1 (distance measured from centerline of pier No. 6)					
SW3-B-T1-1	11	2.17	10 1/2	1.70	10 1/2
SW3-B-T1-2	46	1.25	8 1/2	0.92	7
SW3-B-T1-3	78	0.90	7	0.90	7
SW3-B-T1-4	118	0.92	7	0.90	7
Span 3 Girder B Flange T2 (distance measured from centerline of pier No. 6)					
SW3-B-T2-1	12	2.20	10 1/2	1.70	10 1/2
SW3-B-T2-2	46	1.25	8 1/2	0.92	7
SW3-B-T2-3	78	0.92	7	0.92	7
SW3-B-T2-4	118	0.92	7	0.92	7
Span 3 Girder B Flange B (distance measured from centerline of pier No. 6)					
SW3-B-B-1	11	1.80		1.50	
SW3-B-B-2	81	1.20		1.20	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 1)					
SW4-A-T1-1	31	1.20	7	0.83	7
SW4-A-T1-2	81	0.83	7	0.80	7
SW4-A-T1-3	112	0.80	7	1.33	7
SW4-A-T1-4	131	1.33	7	1.83	9
SW4-A-T1-5	146	1.85	9	2.30	10 1/2
Span 4 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 1)					
SW4-A-T2-1	31	1.20	7	0.85	7
SW4-A-T2-2	81	0.83	7	0.80	7
SW4-A-T2-3	112	0.80	7	1.35	7
SW4-A-T2-4	131	1.30	7	1.83	9
SW4-A-T2-5	146	1.85	9	2.30	10 1/2
Span 4 Girder A Flange B (distance measured from bulkhead of expansion joint No. 1)					
SW4-A-B-1	51	1.03		1.03	
SW4-A-B-2	86	1.05		1.05	
SW4-A-B-3	148	1.45		1.85	
Span 4 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 1)					
SW4-B-T1-1	66	0.85	7	0.85	7
SW4-B-T1-2	128	1.20	7	1.55	9
SW4-B-T1-3	142	1.55	9	1.83	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 1)					
SW4-B-T2-1	66	0.83	7	0.83	7
SW4-B-T2-2	128	1.20	7	1.55	9
SW4-B-T2-3	142	1.57	9	1.85	10
Span 4 Girder B Flange B (distance measured from bulkhead of expansion joint No. 1)					
SW4-B-B-1	47	0.83		0.83	
SW4-B-B-2	87	0.83		0.83	
SW4-B-B-3	142	1.18		1.45	
Span 5 Girder A Flange T1 (distance measured from centerline of pier No. 2)					
SW5-A-T1-1	16	2.32	10 1/2	1.85	9
SW5-A-T1-2	31	1.83	9	1.32	7 1/2
SW5-A-T1-3[2]	90	0.83	7 1/2	0.83	7 1/2
Span 5 Girder A Flange T2 (distance measured from centerline of pier No. 2)					
SW5-A-T2-1	16	2.32	10 1/2	1.85	9
SW5-A-T2-2	31	1.83	9	1.32	7 1/2
SW5-A-T2-3[2]	90	0.83	7 1/2	0.83	7 1/2
Span 5 Girder A Flange B (distance measured from centerline of pier No. 2)					
SW5-A-B-1	10	1.83		1.43	
SW5-A-B-2	28	1.43		0.93	
SW5-A-B-3[1]	32	0.95		0.95	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder B Flange T1 (distance measured from centerline of pier No. 2)					
SW5-B-T1-1	11	1.85	10	1.55	9
SW5-B-T1-2	25	1.55	9	1.2	7
SW5-B-T1-3	89	0.83	7	0.83	7
Span 5 Girder B Flange T2 (distance measured from centerline of pier No. 2)					
SW5-B-T2-1	11	1.83	10	1.55	9
SW5-B-T2-2	25	1.58	9	1.2	7
SW5-B-T2-3	89	0.83	7	0.83	7
Span 5 Girder B Flange B (distance measured from centerline of pier No. 2)					
SW5-B-B-1	11	1.43		1.20	
SW5-B-B-2	83	0.83		0.85	
Span 6 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 3)					
SW6-A-T1-1	40	0.80	7	0.80	7
SW6-A-T1-2	92	1.45	9	1.45	9
SW6-A-T1-3	107	1.45	9	1.80	9
Span 6 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 3)					
SW6-A-T2-1	40	0.83	7	0.83	7
SW6-A-T2-2	92	1.40	9	1.40	9
SW6-A-T2-3	107	1.45	9	1.83	9
Span 6 Girder A Flange B (distance measured from bulkhead of expansion joint No. 3)					
SW6-A-B-1	39	0.83		0.80	
SW6-A-B-2	105	1.05		1.30	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 6 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 3)					
SW6-B-T1-1	39	0.80	7	0.80	7
SW6-B-T1-2	102	1.20	8 1/2	1.50	8 1/2
Span 6 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 3)					
SW6-B-T2-1	39	0.80	7	0.80	7
SW6-B-T2-2	102	1.20	8 1/2	1.50	8 1/2
Span 6 Girder B Flange B (distance measured from bulkhead of expansion joint No. 3)					
SW6-B-B-1	39	0.85		0.85	
SW6-B-B-2	102	1.05		1.20	
Span 7 Girder A Flange T1 (distance measured from centerline of pier No. 4)					
SW7-A-T1-1	11	1.83	9	1.45	9
SW7-A-T1-2	23	1.45	9	1.08	7
SW7-A-T1-3	51	0.83	7	0.83	7
SW7-A-T1-4	79	0.83	8 1/2	0.83	8 1/2
SW7-A-T1-5	150	1.55	8 1/2	2.10	8 1/2
Span 7 Girder A Flange T2 (distance measured from centerline of pier No. 4)					
SW7-A-T2-1	11	1.83	9	1.48	9
SW7-A-T2-2	23	1.45	9	1.08	9
SW7-A-T2-3	51	0.83	7	0.83	7
SW7-A-T2-4	79	0.83	8 1/2	0.83	8 1/2
SW7-A-T2-5	150	1.53	8 1/2	2.10	8 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld Thickness (inch)	Flange Before Weld Width[3] (inch)	Flange After Weld Thickness (inch)	Flange After Weld Width[3] (inch)
Span 7 Girder A Flange B (distance measured from centerline of pier No. 4)					
SW7-A-B-1	10	1.28		1.08	
SW7-A-B-2	79	0.83		0.85	
SW7-A-B-3	146	1.08		1.50	
Span 7 Girder B Flange T1 (distance measured from centerline of pier No. 4)					
SW7-B-T1-1	10	1.55	8 1/2	1.20	8 1/2
SW7-B-T1-2	20	1.20	8 1/2	1.05	7 1/2
SW7-B-T1-3	76	0.83	7 1/2	0.85	7 1/2
SW7-B-T1-4	145	1.43	9	1.83	9
Span 7 Girder B Flange T2 (distance measured from centerline of pier No. 4)					
SW7-B-T2-1	10	1.55	8 1/2	1.20	8 1/2
SW7-B-T2-2	20	1.20	8 1/2	1.03	7 1/2
SW7-B-T2-3	76	0.83	7 1/2	0.85	7 1/2
SW7-B-T2-4	145	1.40	9	1.83	9
Span 7 Girder B Flange B (distance measured from centerline of pier No. 4)					
SW7-B-B-1	10	1.18		1.08	
SW7-B-B-2	76	0.85		0.85	
SW7-B-B-3	143	1.03		1.27	
Span 8 Girder A Flange T1 (distance measured from centerline of pier No. 5)					
SW8-A-T1-1	10	2.10	8 1/2	1.55	8 1/2
SW8-A-T1-2	41	1.08	7	0.78	7
SW8-A-T1-3	93	0.80	7	0.80	7

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table I. Bridge 9268AWelds (Line "SW")

Weld No.	Distance (feet)	Flange Before Weld Thickness (inch)	Flange Before Weld Width[3] (inch)	Flange After Weld Thickness (inch)	Flange After Weld Width[3] (inch)
Span 8 Girder A Flange T2 (distance measured from centerline of pier No. 5)					
SW8-A-T2-1	10	2.10	8 1/2	1.53	8 1/2
SW8-A-T2-2	41	1.08	7	0.78	7
SW8-A-T2-3	93	0.80	7	0.80	7
Span 8 Girder A Flange B (distance measured from centerline of pier No. 5)					
SW8-A-B-1	13	1.45		1.08	
SW8-A-B-2	93	0.80		0.80	
Span 8 Girder B Flange T1 (distance measured from centerline of pier No. 5)					
SW8-B-T1-1	10	1.83	9	1.43	9
SW8-B-T1-2	54	0.80	7 1/2	0.80	7 1/2
SW8-B-T1-3	95	0.78	7 1/2	0.80	7 1/2
Span 8 Girder B Flange T2 (distance measured from centerline of pier No. 5)					
SW8-B-T2-1	10	1.85	9	1.45	9
SW8-B-T2-2	55	0.80	7 1/2	0.80	7 1/2
SW8-B-T2-3	95	0.80	7 1/2	0.83	7 1/2
Span 8 Girder B Flange B (distance measured from centerline of pier No. 5)					
SW8-B-B-1	10	1.30		1.05	
SW8-B-B-2	54	0.80		0.80	
SW8-B-B-3	95	0.83		0.83	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 1 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
WS1-A-T1-1	36	0.83	7	0.80	7
WS1-A-T1-2	79	0.80	7	0.85	7
WS1-A-T1-3	138	1.85	7	2.36	10
WS1-A-T1-4	151	2.36	10	2.85	10
Span 1 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
WS1-A-T2-1	36	0.83	7	0.80	7
WS1-A-T2-2	79	0.80	7	0.83	7
WS1-A-T2-3	138	1.85	7	2.36	10
WS1-A-T2-4	151	2.32	10	2.85	10
Span 1 Girder A Flange B (distance measured from bulkhead of expansion joint No. 8)					
WS1-A-B-1	40	1.08		1.05	
WS1-A-B-2	80	1.10		1.10	
WS1-A-B-3	133	1.05		1.55	
WS1-A-B-4	146	1.60		2.00	
WS1-A-B-5	152	1.98		2.30	
Span 1 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
WS1-B-T1-1	34	0.83	7	0.80	7
WS1-B-T1-2	78	0.80	7	0.85	7
WS1-B-T1-3	125	1.70	8 1/2	1.70	8 1/2
WS1-B-T1-4	139	1.70	8 1/2	2.03	10
WS1-B-T1-5	148	2.05	10	2.58	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 1 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
WS1-B-T2-1	34	0.83	7	0.80	7
WS1-B-T2-2	78	0.80	7	0.85	7
WS1-B-T2-3	125	1.70	8 1/2	1.70	8 1/2
WS1-B-T2-4	139	1.70	8 1/2	2.05	10
WS1-B-T2-5	148	2.05	10	2.58	10
Span 1 Girder B Flange B (distance measured from bulkhead of expansion joint No. 8)					
WS1-B-B-1	38	1.08		1.08	
WS1-B-B-2	78	1.05		1.08	
WS1-B-B-3	144	1.45		1.85	
WS1-B-B-4	151	1.88		2.08	
Span 2 Girder A Flange T1 (distance measured from centerline of pier No. 1)					
WS2-A-T1-1	10	2.83	10	2.36	10
WS2-A-T1-2	22	2.33	10	1.85	7
WS2-A-T1-3	123	0.92	7	0.90	7
Span 2 Girder A Flange T2 (distance measured from centerline of pier No. 1					
WS2-A-T2-1	10	2.86	10	2.36	10
WS2-A-T2-2	22	2.36	10	1.87	7
WS2-A-T2-3	123	0.92	7	0.90	7

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

		Flange Before Weld		Flange After Weld	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 2 Girder A Flange B (distance measured from centerline of pier No. 1					
WS2-A-B-1	9	2.90		1.98	
WS2-A-B-2	17	1.98		1.56	
WS2-A-B-3	26	1.55		1.29	
WS2-A-B-4	85	1.56		1.20	
WS2-A-B-5	122	1.20		1.20	
Span 2 Girder B Flange T1 (distance measured from centerline of pier No. 1					
WS2-B-T1-1	12	2.58	10	2.03	10
WS2-B-T1-2	21	2.03	10	1.70	9
WS2-B-T1-3	84	0.80	7	0.83	7
WS2-B-T1-4	125	0.83	7	0.83	7
Span 2 Girder B Flange T2 (distance measured from centerline of pier No. 1					
WS2-B-T2-1	12	2.58	10	2.06	10
WS2-B-T2-2	21	2.02	10	1.70	9
WS2-B-T2-3	84	0.80	7	0.80	7
WS2-B-T2-4	125	0.83	7	0.83	7
Span 2 Girder B Flange B (distance measured from centerline of pier No. 1					
WS2-B-B-1	8	2.08		1.92	
WS2-B-B-2	15	1.92		1.45	
WS2-B-B-3	84	1.08		1.10	
WS2-B-B-4	125	1.05		1.07	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WS3-A-T1-1	22	0.80	7	1.30	7
WS3-A-T1-2	37	1.30	7	1.32	7
WS3-A-T1-3	86	1.32	7	1.32	7
WS3-A-T1-4	133	1.55	7	2.05	7
WS3-A-T1-5	142	2.05	7	2.55	9 1/2
WS3-A-T1-6	153	2.58	9 1/2	3.08	10
Span 3 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WS3-A-T2-1	22	0.80	7	1.30	7
WS3-A-T2-2	37	1.30	7	1.32	7
WS3-A-T2-3	86	1.30	7	1.30	7
WS3-A-T2-4	133	1.55	7	2.05	7
WS3-A-T2-5	142	2.05	7	2.55	9 1/2
WS3-A-T2-6	153	2.58	9 1/2	3.08	10
Span 3 Girder A Flange B (distance measured from bulkhead of expansion joint No. 2)					
WS3-A-B-1	31	1.20		1.45	
WS3-A-B-2	84	1.48		1.48	
WS3-A-B-3[4]	136	1.08		1.55	
WS3-A-B-4	149	1.55		2.10	
WS3-A-B-5	158	2.08		2.47	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WS3-B-T1-1	31	0.80	7	0.93	7
WS3-B-T1-2	39	0.93	7	0.95	7
WS3-B-T1-3	85	0.93	7	0.83	7
WS3-B-T1-4	137	1.55	7	1.93	9 1/2
WS3-B-T1-5	148	1.95	9 1/2	2.30	10
Span 3 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WS3-B-T2-1	31	0.80	7	0.93	7
WS3-B-T2-2	39	0.93	7	0.95	7
WS3-B-T2-3	85	0.93	7	0.83	7
WS3-B-T2-4	137	1.55	7	1.93	9 1/2
WS3-B-T2-5	148	1.95	9 1/2	2.30	10
Span 3 Girder B Flange B (distance measured from bulkhead of expansion joint No. 2)					
WS3-B-B-1	39	1.05		1.08	
WS3-B-B-2	85	1.05		1.08	
WS3-B-B-3	137	1.05		1.55	
WS3-B-B-4	151	1.58		1.95	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder A Flange T1 (distance measured from centerline of pier No. 3)					
WS4-A-T1-1	14	3.05	10	2.58	9
WS4-A-T1-2	23	2.52	9	2.22	7
WS4-A-T1-3	32	2.22	7	1.65	7
WS4-A-T1-4	59	1.28	7	0.83	7
WS4-A-T1-5	87	0.80	7	0.80	7
WS4-A-T1-6	115	0.83	7	0.80	7
Span 4 Girder A Flange T2 (distance measured from centerline of pier No. 3)					
WS4-A-T2-1	14	3.05	10	2.58	9
WS4-A-T2-2	23	2.52	9	2.22	7
WS4-A-T2-3	32	2.22	7	1.65	7
WS4-A-T2-4	59	1.28	7	0.83	7
WS4-A-T2-5	87	0.80	7	0.80	7
WS4-A-T2-6	115	0.80	7	0.80	7
Span 4 Girder A Flange B (distance measured from centerline of pier No. 3)					
WS4-A-B-1	10	2.47		2.08	
WS4-A-B-2	17	2.08		1.55	
WS4-A-B-3	34	1.55		1.05	
WS4-A-B-4	82	1.08		1.05	
WS4-A-B-5	118	1.10		1.10	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder B Flange T1 (distance measured from centerline of pier No. 3)					
WS4-B-T1-1	14	2.30	10	1.95	8 1/2
WS4-B-T1-2	25	1.90	8 1/2	1.53	7
WS4-B-T1-3	75	0.80	7	0.80	7
WS4-B-T1-4	114	0.80	7	0.78	7
Span 4 Girder B Flange T2 (distance measured from centerline of pier No. 3)					
WS4-B-T2-1	14	2.30	10	1.95	8 1/2
WS4-B-T2-2	25	1.95	8 1/2	1.55	7
WS4-B-T2-3	75	0.80	7	0.80	7
WS4-B-T2-4	114	0.80	7	0.78	7
Span 4 Girder B Flange B (distance measured from centerline of pier No. 3)					
WS4-B-B-1	10	1.93		1.58	
WS4-B-B-2	24	1.58		1.10	
WS4-B-B-3	75	0.80		0.85	
WS4-B-B-4	114	0.85		0.80	
Span 5 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 4)					
WS5-A-T1-1	33	1.07	7	1.05	7
WS5-A-T1-2	78	1.05	7	1.05	7
WS5-A-T1-3	128	1.55	7	2.08	8
WS5-A-T1-4	140	2.05	8	2.42	9

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 4)					
WS5-A-T2-1	33	1.08	7	1.05	7
WS5-A-T2-2	78	1.05	7	1.05	7
WS5-A-T2-3	128	1.55	7	2.05	8
WS5-A-T2-4	140	2.05	8	2.42	9
Span 5 Girder A Flange B (distance measured from bulkhead of expansion joint No. 4)					
WS5-A-B-1	47	1.20		1.20	
WS5-A-B-2	140	1.43		1.90	
Span 5 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 4)					
WS5-B-T1-1	38	0.80	7	0.80	7
WS5-B-T1-2	76	0.80	7	0.80	7
WS5-B-T1-3	132	1.45	8 1/2	1.85	10
Span 5 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 4)					
WS5-B-T2-1	38	0.83	7	0.80	7
WS5-B-T2-2	76	0.80	7	0.83	7
WS5-B-T2-3	132	1.45	8 1/2	1.80	10
Span 5 Girder B Flange B (distance measured from bulkhead of expansion joint No. 4)					
WS5-B-B-1	36	0.93		0.93	
WS5-B-B-2	139	1.33		1.55	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 6 Girder A Flange T1 (distance measured from centerline of pier No. 5)					
WS6-A-T1-1	13	2.40	10	2.08	8 1/2
WS6-A-T1-2	27	2.05	8 1/2	1.55	7 1/2
WS6-A-T1-3	89	0.83	7 1/2	0.83	7 1/2
WS6-A-T1-4	150	1.60	7 1/2	2.08	9
WS6-A-T1-5	162	2.10	9	2.43	10
Span 6 Girder A Flange T2 (distance measured from centerline of pier No. 5)					
WS6-A-T2-1	13	2.40	10	2.05	8 1/2
WS6-A-T2-2	27	2.08	8 1/2	1.55	7 1/2
WS6-A-T2-3	89	0.83	7 1/2	0.83	7 1/2
WS6-A-T2-4	150	1.55	7 1/2	2.08	9
WS6-A-T2-5	162	2.10	9	2.43	10
Span 6 Girder A Flange B (distance measured from centerline of pier No. 5)					
WS6-A-B-1	13	1.93		1.50	
WS6-A-B-2	63	0.85		0.85	
WS6-A-B-3	113	0.85		0.85	
Span 6 Girder B Flange T1 (distance measured from centerline of pier No. 5)					
WS6-B-T1-1	15	1.83	10	1.43	8 1/2
WS6-B-T1-2	86	0.83	7	0.80	7
WS6-B-T1-3	155	1.55	8 1/2	2.02	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 6 Girder B Flange T2 (distance measured from centerline of pier No. 5)					
WS6-B-T2-1	15	1.83	10	1.43	8 1/2
WS6-B-T2-2	86	0.83	7	0.80	7
WS6-B-T2-3	155	1.55	8 1/2	2.02	10
Span 6 Girder B Flange B (distance measured from centerline of pier No. 5)					
WS6-B-B-1	9	1.55		1.33	
WS6-B-B-2	86	0.83		0.80	
WS6-B-B-3	160	1.32		1.70	
Span 7 Girder A Flange T1 (distance measured from centerline of pier No. 5A)					
WS7-A-T1-1	13	2.40	10	2.05	8
WS7-A-T1-2	24	2.08	8	1.55	8
WS7-A-T1-3	78	1.03	7	1.05	7
WS7-A-T1-4	122	1.03	7	1.03	7
Span 7 Girder A Flange T2 (distance measured from centerline of pier No. 5A)					
WS7-A-T2-1	13	2.43	10	2.05	8
WS7-A-T2-2	24	2.10	8	1.55	8
WS7-A-T2-3	78	1.03	7	1.05	7
WS7-A-T2-4	122	1.05	7	1.05	7
Span 7 Girder A Flange B (distance measured from centerline of pier No. 5A)					
WS7-A-B-1	13	1.93		1.45	
WS7-A-B-2	77	1.18		1.18	
WS7-A-B-3	122	1.18		1.18	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table II. Bridge 9268B Welds (Line "WS")					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 7 Girder B Flange T1 (distance measured from centerline of pier No. 5A)					
WS7-B-T1-1	14	2.05	10	1.55	9
WS7-B-T1-2	72	0.83	7	0.83	7
WS7-B-T1-3	120	0.80	7	0.80	7
Span 7 Girder B Flange T2 (distance measured from centerline of pier No. 5A)					
WS7-B-T2-1	14	2.02	10	1.55	9
WS7-B-T2-2	72	0.80	7	0.83	7
WS7-B-T2-3	120	0.80	7	0.80	7
Span 7 Girder B Flange B (distance measured from centerline of pier No. 5A)					
WS7-B-B-1	10	1.68		1.35	
WS7-B-B-2	73	1.03		1.03	
WS7-B-B-3	116	1.08		1.05	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table III. Bridge 9268E Welds (Line "WE")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder D Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-T1-1[5]	39	1.05	7	1.05	7
WE3-D-T1-2[5]	84	1.05	7	1.07	7
WE3-D-T1-3	145	1.95	9	2.47	10
Span 3 Girder D Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-T2-1[5]	39	1.03	7	1.05	7
WE3-D-T2-2	84	1.05	7	1.03	7
WE3-D-T2-3	145	1.97	9	2.50	10
Span 3 Girder D Flange B (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-B-1	41	1.20		1.13	
WE3-D-B-2	86	1.50		1.20	
WE3-D-B-3[5]	145	1.45		1.95	
Span 3 Girder E Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-T1-1	45	0.80	7 1/2	0.78	7 1/2
WE3-E-T1-2	91	0.80	7 1/2	0.80	7 1/2
WE3-E-T1-3	138	1.27	9	1.83	10
Span 3 Girder E Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-T2-1	45	0.78	7 1/2	0.78	7 1/2
WE3-E-T2-2	91	0.80	7 1/2	0.78	7 1/2
WE3-E-T2-3	138	1.23	9	1.80	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table III. Bridge 9268E Welds (Line "WE")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder E Flange B (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-B-1	38	0.85		0.83	
WE3-E-B-2	83	0.83		0.85	
WE3-E-B-3[5]	138	1.05		1.45	
Span 4 Girder D Flange T1 (distance measured from centerline of pier No. 3)					
WE4-D-T1-1	15	2.45	10	2.05	9
WE4-D-T1-2	26	2.10	9	1.57	7 1/2
WE4-D-T1-3[5]	79	0.80	7 1/2	0.83	7 1/2
WE4-D-T1-4[5]	116	0.83	7 1/2	0.80	7 1/2
WE4-D-T1-5	160	1.58	8	2.08	8
Span 4 Girder D Flange T2 (distance measured from centerline of pier No. 3)					
WE4-D-T2-1	15	2.47	10	2.07	9
WE4-D-T2-2	26	2.15	9	1.57	7 1/2
WE4-D-T2-3[5]	89	0.80	7 1/2	0.80	7 1/2
WE4-D-T2-4	160	1.60	8	2.10	8
Span 4 Girder D Flange B (distance measured from centerline of pier No. 3)					
WE4-D-B-1[5]	15	1.93		1.45	
WE4-D-B-2	76	0.78		0.83	
WE4-D-B-3	101	0.83		0.80	
WE4-D-B-4[5]	160	1.20		1.68	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table III. Bridge 9268E Welds (Line "WE")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder D Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-T1-1[5]	39	1.05	7	1.05	7
WE3-D-T1-2[5]	84	1.05	7	1.07	7
WE3-D-T1-3	145	1.95	9	2.47	10
Span 3 Girder D Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-T2-1[5]	39	1.03	7	1.05	7
WE3-D-T2-2	84	1.05	7	1.03	7
WE3-D-T2-3	145	1.97	9	2.50	10
Span 3 Girder D Flange B (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-B-1	41	1.20		1.13	
WE3-D-B-2	86	1.50		1.20	
WE3-D-B-3[5]	145	1.45		1.95	
Span 3 Girder E Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-T1-1	45	0.80	7 1/2	0.78	7 1/2
WE3-E-T1-2	91	0.80	7 1/2	0.80	7 1/2
WE3-E-T1-3	138	1.27	9	1.83	10
Span 3 Girder E Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-T2-1	45	0.78	7 1/2	0.78	7 1/2
WE3-E-T2-2	91	0.80	7 1/2	0.78	7 1/2
WE3-E-T2-3	138	1.23	9	1.80	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table III. Bridge 9268E Welds (Line "WE")						
		Flange Before Weld		**Flange After Weld**		
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 3 Girder E Flange B (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-B-1	38	0.85		0.83	
WE3-E-B-2	83	0.83		0.85	
WE3-E-B-3[5]	138	1.05		1.45	
Span 4 Girder D Flange T1 (distance measured from centerline of pier No. 3)					
WE4-D-T1-1	15	2.45	10	2.05	9
WE4-D-T1-2	26	2.10	9	1.57	7 1/2
WE4-D-T1-3[5]	79	0.80	7 1/2	0.83	7 1/2
WE4-D-T1-4[5]	116	0.83	7 1/2	0.80	7 1/2
WE4-D-T1-5	160	1.58	8	2.08	8
Span 4 Girder D Flange T2 (distance measured from centerline of pier No. 3)					
WE4-D-T2-1	15	2.47	10	2.07	9
WE4-D-T2-2	26	2.15	9	1.57	7 1/2
WE4-D-T2-3[5]	89	0.80	7 1/2	0.80	7 1/2
WE4-D-T2-4	160	1.60	8	2.10	8
Span 4 Girder D Flange B (distance measured from centerline of pier No. 3)					
WE4-D-B-1[5]	15	1.93		1.45	
WE4-D-B-2	76	0.78		0.83	
WE4-D-B-3	101	0.83		0.80	
WE4-D-B-4[5]	160	1.20		1.68	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table III. Bridge 9268E Welds (Line "WE")					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 4 Girder E Flange T1 (distance measured from centerline of pier No. 3)					
WE4-E-T1-1	15	1.80	10	1.30	10
WE4-E-T1-2	86	0.80	7	0.80	7
WE4-E-T1-3	153	1.20	9	1.63	10 1/2
Span 4 Girder E Flange T2 (distance measured from centerline of pier No. 3)					
WE4-E-T2-1	15	1.80	10	1.30	10
WE4-E-T2-2	86	0.80	7	0.80	7
WE4-E-T2-3	153	1.20	9	1.60	10 1/2
Span 4 Girder E Flange B (distance measured from centerline of pier No. 3)					
WE4-E-B-1	52	0.80		0.80	
WE4-E-B-2	86	0.80		0.80	
WE4-E-B-3[5]	153	1.08		1.30	
Span 5 Girder D Flange T1 (distance measured from centerline of pier No. 4)					
WE5-D-T1-1	15	2.10	10	1.58	8 1/2
WE5-D-T1-2[5]	72	0.80	7	0.80	7
WE5-D-T1-3[5]	104	0.83	7	0.83	7
WE5-D-T1-4	160	1.63	9	2.08	10 1/2
Span 5 Girder D Flange T2 (distance measured from centerline of pier No. 4)					
WE5-D-T2-1	15	2.08	10	1.58	8 1/2
WE5-D-T2-2[5]	72	0.80	7	0.80	7
WE5-D-T2-3[5]	104	0.80	7	0.83	7
WE5-D-T2-4	160	1.58	9	2.08	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table III. Bridge 9268E Welds (Line "WE")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder D Flange B (distance measured from centerline of pier No. 4)					
WE5-D-B-1[5]	15	1.77		1.20	
WE5-D-B-2	75	0.83		0.85	
WE5-D-B-3	110	0.80		0.83	
WE5-D-B-4[5]	160	1.20		1.70	
Span 5 Girder E Flange T1 (distance measured from centerline of pier No. 4)					
WE5-E-T1-1	13	1.60	10 1/2	1.08	9
WE5-E-T1-2[5]	76	0.78	7	0.78	7
WE5-E-T1-3[5]	113	0.80	7	0.80	7
WE5-E-T1-4	157	1.20	8 1/2	1.58	10 1/2
Span 5 Girder E Flange T2 (distance measured from centerline of pier No. 4)					
WE5-E-T2-1	13	1.60	10 1/2	1.10	9
WE5-E-T2-2[5]	76	0.80	7	0.80	7
WE5-E-T2-3[5]	113	0.80	7	0.80	7
WE5-E-T2-4	157	1.20	8 1/2	1.60	10 1/2
Span 5 Girder E Flange B (distance measured from centerline of pier No. 4)					
WE5-E-B-1[5]	13	1.38		1.08	
WE5-E-B-2	79	0.80		0.80	
WE5-E-B-3	119	0.80		0.80	
WE5-E-B-4[5]	157	1.08		1.28	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table III. Bridge 9268E Welds (Line "WE")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 6 Girder D Flange T1 (distance measured from centerline of pier No. 5)					
WE6-D-T1-1	15	2.07	10 1/2	1.58	9
WE6-D-T1-2[5]	85	0.80	7 1/2	0.80	7 1/2
WE6-D-T1-3[5]	129	0.80	7 1/2	0.80	7 1/2
Span 6 Girder D Flange T2 (distance measured from centerline of pier No. 5)					
WE6-D-T2-1	15	2.10	10 1/2	1.58	9
WE6-D-T2-2[5]	85	0.80	7 1/2	0.80	7 1/2
WE6-D-T2-3[5]	129	0.80	7 1/2	0.80	7 1/2
Span 6 Girder D Flange B (distance measured from centerline of pier No. 5)					
WE6-D-B-1[5]	15	1.73		1.20	
WE6-D-B-2	65	0.83		0.85	
WE6-D-B-3	106	0.83		0.83	
Span 6 Girder E Flange T1 (distance measured from centerline of pier No. 5)					
WE6-E-T1-1	13	1.55	10	1.18	8 1/2
WE6-E-T1-2[5]	69	0.80	7	0.83	7
WE6-E-T1-3[5]	101	0.83	7	0.83	7
Span 6 Girder E Flange T2 (distance measured from centerline of pier No. 5)					
WE6-E-T2-1	13	1.58	10	1.18	8 1/2
WE6-E-T2-2[5]	69	0.83	7	0.80	7
WE6-E-T2-3[5]	101	0.80	7	0.83	7
Span 6 Girder E Flange B (distance measured from centerline of pier No. 5)					
WE6-E-B-1[5]	13	1.38		1.05	
WE6-E-B-2	77	0.85		0.85	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table IV. Bridge 9268W Welds (Line "EW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder D Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
EW3-D-T1-1	36	0.80	7	1.05	7
EW3-D-T1-2	84	1.03	7	1.03	7
EW3-D-T1-3	145	1.90	8 1/2	2.43	10
Span 3 Girder D Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
EW3-D-T2-1	36	0.83	7	1.05	7
EW3-D-T2-2[5]	84	1.03	7	1.05	7
EW3-D-T2-3	145	1.90	8 1/2	2.43	10
Span 3 Girder D Flange B (distance measured from bulkhead of expansion joint No. 2)					
EW3-D-B-1	24	0.80		1.15	
EW3-D-B-2	43	1.12		1.12	
EW3-D-B-3	85	1.17		1.17	
EW3-D-B-4	145	1.48		1.98	
Span 3 Girder E Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
EW3-E-T1-1	38	0.78	7 1/2	0.80	7 1/2
EW3-E-T1-2	78	0.80	7 1/2	0.80	7 1/2
EW3-E-T1-3	138	1.30	9	1.78	10 1/2
Span 3 Girder E Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
EW3-E-T2-1	38	0.80	7 1/2	0.80	7 1/2
EW3-E-T2-2	78	0.80	7 1/2	0.80	7 1/2
EW3-E-T2-3	138	1.28	9	1.80	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table IV. Bridge 9268W Welds (Line "EW")

Weld No.	Distance (feet)	Flange Before Weld Thickness (inch)	Flange Before Weld Width[3] (inch)	Flange After Weld Thickness (inch)	Flange After Weld Width[3] (inch)
Span 3 Girder E Flange B (distance measured from bulkhead of expansion joint No. 2)					
EW3-E-B-1	39	0.80		0.83	
EW3-E-B-2	82	0.80		0.80	
EW3-E-B-3	138	1.07		1.45	
Span 4 Girder D Flange T1 (distance measured from centerline of pier No. 3)					
EW4-D-T1-1	15	2.40	10	2.05	8
EW4-D-T1-2	26	2.03	8	1.55	7
EW4-D-T1-3	83	0.80	7	0.80	7
EW4-D-T1-4	122	0.80	7	0.80	7
EW4-D-T1-5	161	1.55	8 1/2	2.05	10
Span 4 Girder D Flange T2 (distance measured from centerline of pier No. 3)					
EW4-D-T2-1	15	2.42	10	2.05	8
EW4-D-T2-2	26	2.03	8	1.55	7
EW4-D-T2-3	83	0.80	7	0.80	7
EW4-D-T2-4	122	0.80	7	0.80	7
EW4-D-T2-5	161	1.55	8 1/2	2.05	10
Span 4 Girder D Flange B (distance measured from centerline of pier No. 3)					
EW4-D-B-1	15	1.95		1.45	
EW4-D-B-2	78	0.85		0.80	
EW4-D-B-3	99	0.80		0.80	
EW4-D-B-4	161	1.17		1.68	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table IV. Bridge 9268W Welds (Line "EW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder E Flange T1 (distance measured from centerline of pier No. 3)					
EW4-E-T1-1	15	1.82	10 1/2	1.32	8 1/2
EW4-E-T1-2	81	0.80	7	0.78	7
EW4-E-T1-3	121	0.80	7	0.80	7
EW4-E-T1-4	153	1.20	9	1.55	10
Span 4 Girder E Flange T2 (distance measured from centerline of pier No. 3)					
EW4-E-T2- 1	15	1.83	10 1/2	1.30	8 1/2
EW4-E-T2- 2	81	0.80	7	0.83	7
EW4-E-T2- 3	121	0.78	7	0.80	7
EW4-E-T2- 4	153	1.20	9	1.55	10
Span 4 Girder E Flange B (distance measured from centerline of pier No. 3)					
EW4-E-B-1	15	1.43		1.05	
EW4-E-B-2	81	0.85		0.80	
EW4-E-B-3	121	0.80		0.80	
EW4-E-B-4	153	1.07		1.28	
Span 5 Girder D Flange T1 (distance measured from centerline of pier No. 4)					
EW5-D-T1-1	15	2.02	10 1/2	1.55	8 1/2
EW5-D-T1-2[5]	81	0.80	7	0.80	7
EW5-D-T1-3	121	0.80	7	0.80	7
EW5-D-T1-4	160	1.55	8 1/2	2.05	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table IV. Bridge 9268W Welds (Line "EW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder D Flange T2 (distance measured from centerline of pier No. 4)					
EW5-D-T2-1	15	2.05	10 1/2	1.55	8 1/2
EW5-D-T2-2[5]	81	0.80	7	0.80	7
EW5-D-T2-3[5]	121	0.83	7	0.80	7
EW5-D-T2-4	160	1.55	8 1/2	2.05	10 1/2
Span 5 Girder D Flange B (distance measured from centerline of pier No. 4)					
EW5-D-B-1	15	1.68		1.20	
EW5-D-B-2	109	0.80		0.80	
EW5-D-B-3	161	1.20		1.65	
Span 5 Girder E Flange T1 (distance measured from centerline of pier No. 4)					
EW5-E-T1-1[6]	13	1.55	10	1.18	8 1/2
EW5-E-T1-2	78	0.78	7	0.80	7
EW5-E-T1-3	118	0.80	7	0.80	7
EW5-E-T1-4	157	1.18	8 1/2	1.55	10 1/2
Span 5 Girder E Flange T2 (distance measured from centerline of pier No. 4)					
EW5-E-T2-1[6]	13	1.53	10	1.18	8 1/2
EW5-E-T2-2	78	0.78	7	0.78	7
EW5-E-T2-3	118	0.78	7	0.80	7
EW5-E-T2-4	157	1.18	8 1/2	1.55	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table IV. Bridge 9268W Welds (Line "EW")

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder E Flange B (distance measured from centerline of pier No. 4)					
EW5-E-B-1[6]	13	1.28		1.05	
EW5-E-B-2	78	0.83		0.83	
EW5-E-B-3	118	0.83		0.80	
EW5-E-B-4	157	1.03		1.30	
Span 6 Girder D Flange T1 (distance measured from centerline of pier No. 5)					
EW6-D-T1-1	15	2.02	10 1/2	1.55	9
EW6-D-T1-2	81	0.80	7	0.83	7
EW6-D-T1-3	120	0.80	7	0.80	7
Span 6 Girder D Flange T2 (distance measured from centerline of pier No. 5)					
EW6-D-T2-1	15	2.02	10 1/2	1.55	9
EW6-D-T2-2	81	0.83	7	0.83	7
EW6-D-T2-3	120	0.80	7	0.83	7
Span 6 Girder D Flange B (distance measured from centerline of pier No. 5)					
EW6-D-B-1	15	1.65		1.21	
EW6-D-B-2	81	0.80		0.80	
EW6-D-B-3	99	0.83		0.83	
Span 6 Girder E Flange T1 (distance measured from centerline of pier No. 5)					
EW6-E-T1-1	14	1.53	10	1.18	9
EW6-E-T1-2	77	0.83	7	0.83	7
EW6-E-T1-3	116	0.80	7	0.80	7

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table IV. Bridge 9268W Welds (Line "EW")					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 6 Girder E Flange T2 (distance measured from centerline of pier No. 5)					
EW6-E-T2-1	14	1.60	10	1.18	9
EW6-E-T2-2	77	0.80	7	0.80	7
EW6-E-T2-3	116	0.78	7	0.80	7
Span 6 Girder E Flange B (distance measured from centerline of pier No. 5)					
EW6-E-B-1	14	1.30		1.03	
EW6-E-B-2	77	0.80		0.83	
EW6-E-B-3	116	0.83		0.83	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

APPENDIX B: WPS AND PQR REPORTS FROM ORIGINAL CONSTRUCTION DOCUMENTS

8/1
7214

February 26, 1970

Canron Limited
Fremont Project Office
22 East 5th Avenue
Vancouver 10, B.C.

Attention: I. M. Nitkin

Gentlemen:

Re: West Fremont Interchange
Contract No. 7214
Welding Procedures

By letter of January 20 you sent us a revision of your Procedure Specification 5.0 for approval. This material has been reviewed and we find your Specifications 5.0, pages 1 and 2, 5.01 and 5.02 are satisfactory and are hereby approved.

It is noted that no impact values are included with the above letter of January 20 for 1" thick A441 steel. Our letter to you on October 9, 1969, for the approval of electroslag welding required impact values. This material should be supplied at the earliest possible time. Other than the impact values for A441 Steel the material is satisfactory to the State.

My apologies to you for being so slow in reviewing this procedure, however, illness has taken its toll in the intervening time since we received your January 20 letter.

Yours very truly,

Ivan D. Merchant
Bridge Engineer

IDM:jf
cc: Pittsburgh Testing Laboratory
Attn: Don Scott
Pittsburgh Testing Laboratory
Attn: Ken Petterson

NOTED
IVAN D. MERCHANT

CANRON LIMITED
WESTERN BRIDGE DIVISION
145 WEST FIRST AVENUE, VANCOUVER 10, B.C. — TEL. 874-2311

Reply to: FREMONT PROJECT OFFICE
22 East Fifth Avenue
Vancouver 10, B.C.
874-2311

January 20, 1970

Mr. Ivan D. Merchant
Oregon State Highway Dept.
Highway Building
Salem, Oregon 97310
U S A

Dear Sir:

Enclosed are copies of the electroslag procedure qualification test results for your file, and also a revision of Procedure Specification 5.0 for your approval.

Welding variables shown on the original submission were as provided by the equipment supplier but did not give the required impact results. This situation has now been corrected.

Wire feed speed and amperage are dependent variables and we have chosen to specify the former which can be more precisely determined.

Yours very truly,

CANRON LIMITED
WESTERN BRIDGE DIVISION

F. A. Graham

F. A. GRAHAM, P.Eng.
Welding Engineer

Encl:

cc: I.M. Nitkin
K. Peterson (encl.)

RECEIVED
JAN 22 1970
BRIDGE DIVISION

NOTED
D. O. Christensen

COAST ELDRIDGE
ENGINEERS & CHEMISTS LTD.
125 EAST 4TH AVE., VANCOUVER 10, B.C.
TELEPHONE: TRINITY 6-4111
CABLE ADDRESS "ELDRICO"

REPORT OF IMPACT TESTS

PROJECT: 4901

DATE: January 16, 1970

REPORTED TO: Canron Ltd.,
145 West 1st Ave.,
Vancouver, B.C.

FILE: 480-129-C.4

P/O: 4140

Report No. 10/70

TYPE OF TEST: Charpy Impact

SPECIMEN SIZE: Standard 10x10 mm

TYPE OF SPECIMEN: V-Notch TESTED @ 0° F & +30° TEMP.

SPECIFICATION: AWS 1-67

MATERIAL: ASTM A588(C) Electroslag (2-3/4" Plate)

SPECIMEN PREPARED BY:

TEST RESULTS

weld metal 0° F SPECIMEN MK.	No.	IMPACT VALUE IN FT. LBS.	heat-affected zone +30° F SPECIMEN MK.	No.	IMPACT VALUE IN FT. LBS.	SPECIMEN MK.	No.	IMPACT VALUE IN FT. LBS.
2V	1	40	2Y	1	15.5			
	2	35		2	25.5			
	3	43.5		3	21.5			
	4	42		4	15			
	5	33		5	21			

20 ft. lbs. reqd. 15 ft. lbs. reqd.

RECEIVED
JAN 22 1970
BRIDGE DIVISION

REMARKS:

TESTED BY: Howard Grisack

WITNESSED BY: Dwayne Peterson - Pittsburg Testing
Frank Graham - Canron

COAST ELDRIDGE

Brian T. Shankey, Supervisor
Physical Testing Services
PER [signature]

WARNOCK HERSEY INTERNATIONAL LTD. PROFESSIONAL SERVICES DIVISION
125 EAST 4TH AVE. – VANCOUVER 10, B.C. TELEPHONE: 876-4111 TELEX: 04-50353

PHYSICAL TEST REPORT

FILE No.: 480-129-C.4

MATERIAL TESTED: 2-3/4" Plate

SPECIFICATION:

DATE: January 16, 1970

PROJECT: 4901

REPORT No.: 7/70

REPORTED TO: Canron Ltd.,
145 West 1st Ave.,
Vancouver, B.C.

- P A G E 2 -

ORDER No.: 4140

DESCRIPTION OF SPECIMEN	SPECIMEN IDENTIFICATION MK	DIAM. DIM.	AREA	YIELD STRENGTH – P.S.I. @ 0.2 % OFFSET (50,000 min)	TENSILE STRENGTH – P.S.I. (70,000 – 90,000)	% ELONG IN 2" (22% min)	TYPE OF FRACTURE	REDUCTION OF AREA	BEND TEST	WEIGHT LBS./LIN. FT.
All Weld Tensile Specimen		0.501	0.1971	53,780	82,700	31.2	1/2 cup	73.7		
Reduced Section Tensile Specimen		1.003 x 1.490	1.494		85,010		Failure in weld zone			
"		1.006 x 1.490	1.499		82,390		"	"	"	
Standard Side Bend Specimen		Satisfactory to AWS Suppl. 1-67								
"		Satisfactory to AWS Suppl. 1-67								
"		Satisfactory to AWS Suppl. 1-67								
"		Satisfactory to AWS Suppl. 1-67								

SPECIFICATION REQUIREMENTS

GRADE — MIN. MIN. MIN. MIN.

SAMPLED BY: Client AT: Canron DATE: Jan. 15/70

REMARKS:

Witnessed by: Dwayne Peterson - Pittsburg Testing
Frank Graham - Canron

TECHNICIAN:

WARNOCK HERSEY INTERNATIONAL LTD.

Brian T. Shankey, Supervisor
PER Physical Testing Services

FOI ! NO.

COAST ELDRIDGE

ENGINEERS & CHEMISTS LTD.

125 EAST 4TH AVE., VANCOUVER 10, B.C. TELEPHONE: TRINITY 6-4111 CABLE ADDRESS "ELDRICO"

REPORT OF IMPACT TESTS

PROJECT: 4901

DATE January 16, 1970

REPORTED TO: Canron Limited, 145 West 1st Ave., Vancouver, B.C.

FILE 480-129-C.4

P/O xxx Report No. 8/70

~~P.O.~~ ~~4140~~

TYPE OF TEST: Charpy Impact

SPECIMEN SIZE: 10x10 mm Standard

TYPE OF SPECIMEN: V-Notch TESTED @ 0°F & +30°F TEMP.

SPECIFICATION: AWS 1-67

MATERIAL: ASTM A588(C) Electroslag ($1\frac{1}{2}$" Plate)

SPECIMEN PREPARED BY: Client

TEST RESULTS

weld metal 0°F SPECIMEN MK.	NO.	IMPACT VALUE IN FT. LBS.	heat-affected zone +30°F SPECIMEN MK.	NO.	IMPACT VALUE IN FT. LBS.	SPECIMEN MK.	NO.	IMPACT VALUE IN FT. LBS.
V	1	27	Y	1	28			
	2	49		2	21			
	3	22		3	18.5			
	4	18		4	22.5			
	5	25		5	27.5			
Spec.:		[illegible]0 ft.lbs.			15 ft.lbs.			

RECEIVED JAN 28 1970 BRIDGE DIVISION

REMARKS:

TESTED BY: H. Grisack

WITNESSED BY: Dwayne Peterson - Pittsburg Testing
Frank Graham - Canron

COAST ELDRIDGE

PER Brian T. Shankey, Supervisor
Physical Testing Services

/nb

WARNOCK HERSEY INTERNATIONAL LTD. PROFESSIONAL SERVICES DIVISION

125 EAST 4TH AVE. – VANCOUVER 10, B.C. TELEPHONE: 876-4111 TELEX: 04-50353

PHYSICAL TEST REPORT

FILE No.: 480-129-C.4

MATERIAL TESTED: 1½" Plate ASTM A588 (C) Electroslag

SPECIFICATION:

DATE: January 16, 1970

PROJECT: 4901

REPORT No.: 7/70

REPORTED TO: Canron Limited, 145 West 1st Ave., Vancouver, B.C.

ORDER No.: 4140

DESCRIPTION OF SPECIMEN	SPECIMEN IDENTIFICATION MK	DIAM. DIM.	AREA	YIELD STRENGTH — P.S.I. @ 0.2 % OFFSET (50,000 min)	TENSILE STRENGTH — P.S.I. (70,000–95,000)	% ELONG IN 2" (22% min)	TYPE OF FRACTURE	REDUCTION OF AREA	BEND TEST	WEIGHT LBS./LIN. FT.
All Weld Tensile Specimen		0.502	0.1979	54,070	78,830	29.8	3/4 cup	69.8		
Reduced Section Tensile Specimen		0.993 x 1.490	1.480		83,450		Failure in weld zone			
" "		1.003 x 1.482	1.486		84,790		"	"	"	
Standard Side Bend Specimen		SATISFACTORY TO AWS Suppl. 1-67								
"		SATISFACTORY TO AWS Suppl. 1-67								
"		SATISFACTORY TO AWS Suppl. 1-67								
"		SATISFACTORY TO AWS Suppl. 1-67								

SPECIFICATION REQUIREMENTS

GRADE MIN. MIN. MIN. MIN.

SAMPLED BY Client AT Canron DATE: Jan. 15/70

REMARKS: Witnessed by: Dwayne Peterson – Pittsburg Testing
Frank Graham – Canron

TECHNICIAN:

WARNOCK HERSEY INTERNATIONAL LTD.

PER Brian T. Shankey, Supervisor
Physical Testing Services

Specification No. 5.0

Date: 28/7/69

PROCEDURE SPECIFICATION FOR CONSUMABLE GUIDE ELECTROSLAG WELDING

1. SCOPE

This specification in conjunction with the General Specification 1.0 covers electroslag welding, utilizing a single electrode and consumable guide tube, on flange components shop-fabricated for the West Fremont Interchange.

2. BASE METAL:

The base metals that may be welded under this specification are as outlined in Section 2 of Specification No. 1.0.

3. FILLER METAL & FLUX:

(a) The electrode shall conform to specification A.S.T.M. 559-65T, classification E70S-3 (Hobart Type 25P).

(b) The consumable guide shall conform to specification A.I.S.I. 1018 and shall be 1/2" O.D. x 1/8" I.D.

(c) The molten weld pool shall be shielded with Hobart PF 201 electroslag flux.

4. POSITION:

Welding shall be done only within the range of ± 10° to the vertical position.

5. MOLDING SHOES:

Water cooled copper shoes shall be used at each side of the gap opening to retain the molten weld pool and provide uniform weld profile with minimum weld reinforcement.

6. PREHEAT:

Base materials shall be preheated to 250°F prior to welding.

7. PREPARATION OF BASE MATERIAL:

(a) The edges or surfaces of parts to be joined by welding shall be squarely cut by flame cutting or machining.

(b) The requirements of Section 4 of the General Specification 1.0. also to apply.

8. NATURE OF THE ELECTRIC CURRENT:

Direct current, reverse polarity (electrode positive).

9. WELDING TECHNIQUE:

(a) The welding shall be done in a single pass. A starting sump and run-off tab shall be used for each joint and shall be of sufficient length to ensure complete fusion.

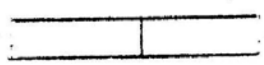
(b) Welding of a joint shall be continuous from start to finish.

10. The following Welding Procedure data sheet forms a part of this specification.

WELDING PROCEDURE No. 5.01

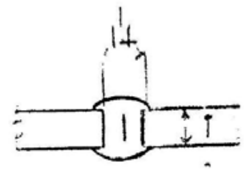
Welding Process: ELECTROSLAG	Welding Position: VERTICAL

Symbol



Joint Detail

FOR COMPRESSION FLG'S.



Welding Sequence

△2

Thickness "T" or Fillet size "L"	Side	Layer No.	Pass No.	Electrode Type	Electrode Dia.	WIRE FEED (I.P.M)	Arc Voltage		
△1 ¾			1	E70S-3	3/32	100	[illegible]		
△ 1			1			110			
1¼			1			120			
1½			1			130	[illegible]		
1¾			1			140			
2			1			150			
2¼			1			160			
2⅜			1			165			
2½			1			170			
2¾			1			18[illegible]	[illegible]		
3			1			[illegible]			

△1 30 OCT/69 - REV AS INDICATED

△2 19 JAN/70 - do do

Notes

- OSCILLATE 1½" & OVER
- FOR JOINING UNEQUAL THICKNESSES SELECT WIRE FEED SPEED FOR THINNER PLATE & VOLTAGE SET FOR THICKER PLATE.
- USE 1¼" ROOT GAP FOR ALL THICKNESSES

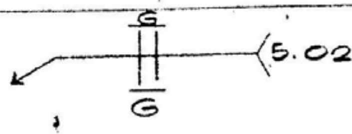
Appr Date

Eng.

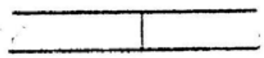
Wks

WELDING PROCEDURE - No. 5.02

Welding Process: ELECTROSLAG	Welding Position: VERTICAL



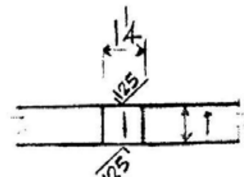
Symbol



Joint Detail

FOR TENSION FLG'S

NOTE: FINAL GRINDING IN LONGITUDINAL DIRECTION OF ℄.



Welding Sequence

△2

Thickness "T" or Fillet size "L"	Side	Layer No.	Pass No.	Electrode Type	Electrode Dia.	WIRE FEED (I.P.M)	Arc Voltage		
△1 3/4			1	E70S-3	3/32	120	[illegible] 39		
△2 1			1	│	│	[illegible]	│		
1¼			1	│	│	[illegible]	│		
1½			1	│	│	[illegible]	[illegible]		
1¾			1	│	│	[illegible]	│		
2			1	│	│	150	│		
2¼			1	│	│	160	│		
2⅜			1	│	│	165	│		
2½			1	│	│	[illegible]	│		
2¾			1	│	│	[illegible]	[illegible] 40		
3			1	│	│	[illegible]	│		

△1 - 30 OCT/69 - REV. AS INDICATED

△2 - 19 JAN/70 - do do

Notes

- OSCILLATE 1/2" & OVER
- FOR JOINTS [illegible] UNEQUAL THICKNESSES SELECT WIRE FEED [illegible] FOR THINNER PLATE & VOLTAGE FOR THICKER PLATE
- USE 1¼ ROOT GAP FOR ALL THICKNESSES.

[illegible] Date

Wks.

Specification No. 5.0

Date: 28/7/69

PROCEDURE SPECIFICATION FOR CONSUMABLE GUIDE ELECTROSLAG WELDING

1. SCOPE

This specification in conjunction with the General Specification 1.0 covers electroslag welding, utilizing a single electrode and consumable guide tube, on flange components shop-fabricated for the West Fremont Interchange.

2. BASE METAL:

The base metals that may be welded under this specification are as outlined in Section 2 of Specification No. 1.0.

3. FILLER METAL & FLUX:

(a) The electrode shall conform to specification A.S.T.M. 559-65T, classification E70S-3 (Hobart Type 25P).

(b) The consumable guide shall conform to specification A.I.S.I. 1018 and shall be 1/2" O.D. x 1/8" I.D.

(c) The molten weld pool shall be shielded with Hobart PF 201 electroslag flux.

4. POSITION:

Welding shall be done only within the range of ± 10° to the vertical position.

5. MOLDING SHOES:

Water cooled copper shoes shall be used at each side of the gap opening to retain the molten weld pool and provide uniform weld profile with minimum weld reinforcement.

6. PREHEAT:

Base materials shall be preheated to 250°F prior to welding.

7. PREPARATION OF BASE MATERIAL:

(a) The edges or surfaces of parts to be joined by welding shall be squarely cut by flame cutting or machining.

(b) The requirements of Section 4 of the General Specification 1.0. also to apply.

8. NATURE OF THE ELECTRIC CURRENT:

Direct current, reverse polarity (electrode positive).

9. WELDING TECHNIQUE:

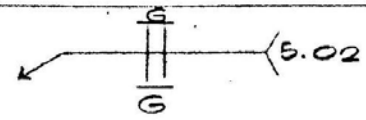
(a) The welding shall be done in a single pass. A starting sump and run-off tab shall be used for each joint and shall be of sufficient length to ensure complete fusion.

(b) Welding of a joint shall be continuous from start to finish.

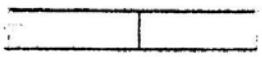
10. The following Welding Procedure data sheet forms a part of this specification.

WELDING PROCEDURE - No. 5.04 (0 2

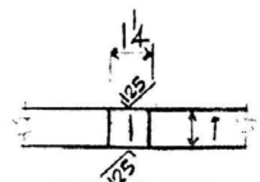
Welding Process: ELECTROSLAG	Welding Position: VERTICAL



Symbol



NOTE: FINAL GRINDING IN LONGITUDINAL DIRECTION OF ℞.



Joint Detail

FOR TENSION FLG'S

Welding Sequence

Thickness "T" or Fillet size "L"	Side	Layer No.	Pass No.	Electrode Type	Electrode Dia.	WIRE FEED (I.P.M)	Arc Voltage		
△1 ¾			1	E70S-3	3/32	[illegible]	[illegible] 59		
△2 1			1			[illegible]			
1¼			1			[illegible]			
1½			1			[illegible]	[illegible] 59		
1¾			1			[illegible]			
2			1			[illegible]			
2¼			1			[illegible]			
2⅜			1			165			
2½			1			[illegible]			
2¾			1			[illegible]	[illegible] 40		
3			1			[illegible]			

△1 - 30 OCT/67 - REV. AS INDICATED

△2 - 19 JAN/70 - do do

Notes

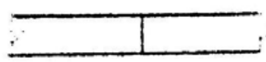
- OSCILLATE ½" & OVER
- FOR JOINING UNEQUAL THICKNESSES SELECT WIRE FEED SPEED FOR THINNER PLATE & VOLTAGE FOR THICKER PLATE
- USE 1¼ ROOT GAP FOR ALL THICKNESSES

	Appr.	Date
Eng.		
Wks.		

WELDING PROCEDURE - No. 5.01

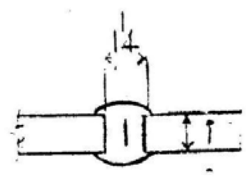
Welding Process:	Welding Position:
ELECTROSLAG	VERTICAL

Symbol



Joint Detail

FOR COMPRESSION FLG'S



Welding Sequence

△2

	Thickness "T" or Fillet size "L"	Side	Layer No.	Pass No.	Electrode Type	Electrode Dia.	WIRE FEED (I.P.M)	Arc Voltage		
△1	¾			1	E70S-3	3/32	100	38/39		
△2	1			1			110			
	1¼			1			120			
	1½			1			130	[illegible]		
	1¾			1			140			
	2			1			150			
	2¼			1			160			
	2⅜			1			165			
	2½			1			170			
	2¾			1			180	[illegible]		
	3			1			190			

△1 30 OCT/69 - REV AS INDICATED

△2 19 JAN/70 - do do

Notes

- OSCILLATE 1½" & OVER.
- FOR JOINING UNEQUAL THICKNESSES SELECT WIRE FEED SPEED FOR THINNER PLATE & VOLTAGE FOR THICKER PLATE.
- USE 1¼" ROOT GAP FOR ALL THICKNESSES

	Appr.	Date
Eng.		
Wks.		

APPENDIX C:
STRUCTURAL LOAD TESTING AND MATERIALS EVALUATION OF WEST FREMONT APPROACHES FINAL REPORT

MEI-CHARLTON
A BISHANTI + BHRAGUVANSHI ENTERPRIZE
7220 N Lombard Street
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STRUCTURAL LOAD TESTING AND MATERIALS EVALUATION OF WEST FREMONT APPROACHES

FINAL REPORT

Client:
Steven C. Lovejoy, P.E.
Senior Mechanical Engineer
Oregon Department of Transportation
355 Capitol Street NE, Room 301
Salem, OR 97301

Submitted by:
Robert E. Hodel, PE
Principal Engineer
MEI-Charlton, Inc
7220 N Lombard Street
Portland, OR 97203

REGISTERED PROFESSIONAL
ENGINEER
12,244
Robert E Hodel
OREGON
July 15, 1983
ROBERT E. HODEL

expires 6/30/2007

21 May 2007

TABLE OF CONTENTS

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TO:	Oregon Department of Transportation Attention: Steven C. Lovejoy, PE 355 Capitol Street NE, Room 301 Salem, OR 97301-3871	CLIENT NO.: REFERENCE NO.: DATE:	PSK 23179 WOC 5 7112003 21 May 2007

SUBJECT: STRUCTURAL LOAD TESTING AND MATERIALS EVALUATION OF WEST FREMONT APPROACHES; FINAL REPORT

Background/Overview

The approach structures for the west side of the Fremont Bridge in Portland, Oregon were fabricated using the Electroslag Welding (ESW) process of joining girder flange plates. Girders fabricated using ESW welds have been found to exhibit brittle failure modes in some instances. As a result, the Federal Highway Administration (FHWA) issued a moratorium on using such welding processes and required all states with bridges fabricated by such methods to determine if they are serviceable, as described in FWHA Notice N5040.23 and N5040.29.

The State of Oregon has two such structures, the I-205 West Linn Bridge and the West Fremont Bridge Approaches. The West Linn Bridge has already been investigated by the Oregon Department of Transportation (ODOT) and determined to be fit for service. Currently, ODOT is in process of evaluating the fitness-for-purpose of the West Fremont Bridge Approaches.

MEI-Charlton, Inc. (MEI-C) was retained by (ODOT) to collect the physical evidence needed by the Agency to perform the fitness-for-purpose analysis. This assignment was defined in Work Order Contract (WOC) No. 5, and was part of MEI-C's Nondestructive, Materials, and Performance Testing of Highway Bridges Contract, PSK 23179. WOC No. 5 contains the following five tasks:

- Task 1. Weld Identification
- Task 2. Strain Gaging
- Task 3. Core Removal from Bottom Flange Plates; Phases 1 and 2
- Task 4. Material Testing; Phases 1 and 2
- Task 5. Nondestructive Testing of In-Service Weldments

Interim or draft review reports were submitted previously to ODOT on Tasks 1, 2, 3, 4 (Phase 1 only), and 5. This final report includes all the work done in WOC #5 and incorporates comments or clarifications resulting from reviews of the earlier, interim/draft reports.

Robert E. Hodel, PE
Project Director

REH:sas
6 hard copies, 1 digital copy on CD

TASK 1
WELD IDENTIFICATION

Scope of Work

Figure 1 (provided by ODOT) shows the west side approaches to the Fremont Bridge. Task 1 of WOC 5 was to locate, identify, and mark all visibly identifiable flange butt welds in the girders of the following bridges:

- Bridge 09268A (“SW” Line), Units 1, 2, and 3
- Bridge 09268B (“WS” Line), Units 1, 2, and 3
- Bridge 09268E (“WE” Line), Unit 1
- Bridge 09268W (“EW” Line), Unit 1

These bridges (colored blue, green, orange, and pink in the photograph) each have two girders and are the only bridges included in the study because the other bridges (brown, red, and yellow in the photograph) each have three girders and thus, the welds are not considered by ODOT to be “fracture critical”.

The basis for locating the welds was visual inspection, with reference to the design drawings of the structures. The shop drawings, which would have identified the locations of all the welds, were not available for reference.

The inspection revealed that some of the welds shown in the design drawings were not present. In particular, MEIC engineers found that at some locations, thicker flange plates had been used than what the design drawings called for; thus, at some locations where the design drawings specified a transition from a thinner plate to a thicker plate, no weld was present because the material on both sides of the specified transition was already at the thicker value.

Visual inspection was a viable means of identifying the welds at many locations, where the weld crown had either not been ground at all or had only been partially ground and was still visible. At these locations, the welds were readily visible, either with just normal, incident lighting or with shallow-angle lighting (which enhances any slight surface contours). However, at some locations, the welds had been ground very smooth, to the extent that we were unable to visually find the welds even though we knew where they were. In particular, this condition was evident on the top flanges at some locations, where we could see a thickness transition along the edge of the flange, and thus knew there was a weld at that location, but we could not identify the weld visually on the surface, even with shallow-angle lighting.

Although etching and/or ultrasonic inspection would have been viable means of identifying the welds, such techniques are much more time consuming than visual inspection and were not deemed practical from a cost standpoint. Thus, they were not specified in the WOC.

Figure 1. Fremont Bridge, West Side Approaches

Fremont Bridge West Side Approaches Girders

Figure 2 shows the typical appearance of the girders on the west side approaches, as viewed from inside a girder. The girders are composed of a single, full width bottom flange and two, narrower top flanges. The top and bottom flanges are joined to the webs with fillet welds.

According to the design drawings, the webs are welded to the bottom flange plates about one inch from the edge of the flange plate. In contrast, the webs are welded to the top flanges at about the middle of the flange plates; thus, while nearly the full width of the bottom flanges is visible from inside the girder, only about half the width of the top flange is visible from inside.

Figure 2 (D71055-19) Typical Appearance Inside of Girders

Bottom Flange Welds

Figures 3 and 4 show typical bottom flange welds observed during our inspection. At some locations, the welds had not been ground at all or were only partially ground and thus, were readily evident with incident lighting (Figure 3). At other locations, the welds had been ground somewhat smooth, but were still relatively easy to find with shallow-angle lighting (Figure 4).

Figure 3 (D71055-08) Bottom Flange Weld

Figure 4 (71056-02) Bottom Flange Weld; As Ground During Construction

Top Flange Welds

Figures 5 and 6 show examples of top flange welds observed during MEIC's inspection. At some locations, the edges of the top flanges were visible, and thus, we could visually inspect the flanges for thickness transitions as a means of identifying weld locations (Figure 6). Additionally, because the flanges had apparently been cut by a thermal process and had relatively rough edges, we could also inspect for the presence of welds by looking for areas where the edges exhibited a change in roughness, thereby suggesting the presence of a weld.

However, at many locations, the edges of the top flanges were covered by the sheet metal forming the underside of the deck (Figure 5), and at these locations, because we could not see the edges, we could not utilize them for our weld search. This was of concern, because, while at some locations, the weld crowns on the top flanges were readily visible, at other locations they had been ground very smooth, to the extent that the welds were not visible, even with shallow-angle lighting and event though we knew precisely where they were located based on our inspection of the flange edges.

Figure 6 shows a location where the flange changed thickness, indicating the presence of a weld joint. At this location, the weld had been ground smooth and was covered by a thick layer of paint, obscuring any visual evidence of the weld on the plate surface. Had the edge of the flange not been visible at this location, we would not have been able to find this weld by visual inspection.

The fact that we found several top flange welds via our "edge" inspection, where the weld crowns were not visually evident, raises concerns about how many similar welds may exist in the bridges. That is, given that we were able to find a few top flange welds only because the edges of the flanges were visible, it is reasonable to assume that other flange welds may be present that we were unable to find because the flange edges were not visible (either because they were obscured by decking sheet metal in the case of the top flanges, or because they were on the bottom flanges, where the edges are not visible from inside the girders.)

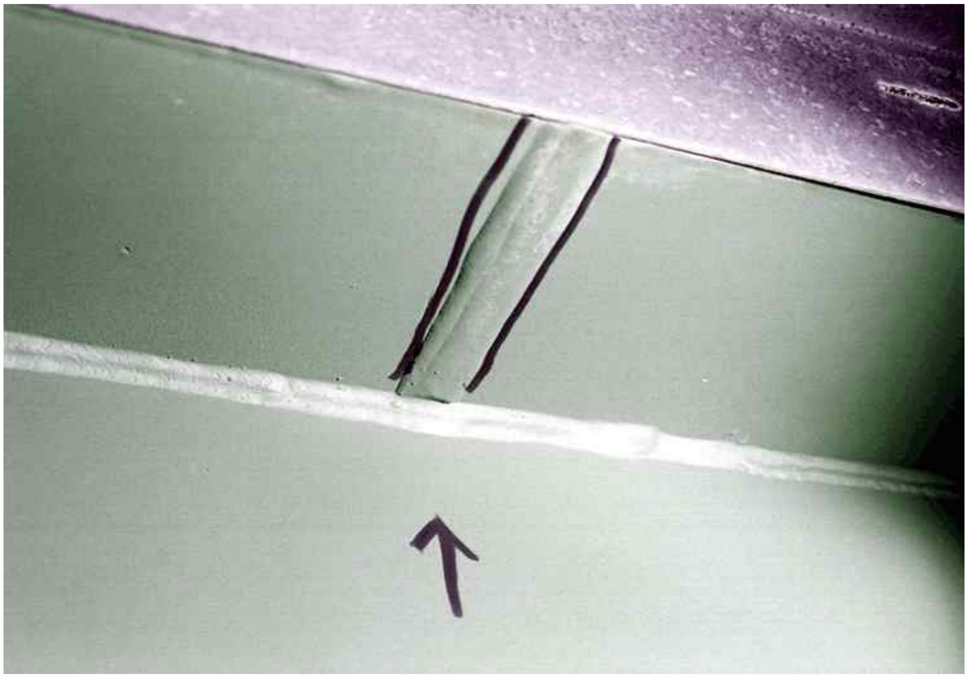


Figure 5 (71056-04) Top Flange Weld; Weld Crown Visible

Figure 6 (71056-12) Top Flange Weld Thickness Transition; Weld Crown Ground Smooth; Weld Not Visible

Weld Identifications

Tables 1 through 4 show the locations of the welds MEIC engineers identified during the inspection. The tables show the weld number and distance to the weld from either the centerline of the pier or, in the case of an expansion joint, from the start of the Unit. (A "unit" goes from one expansion joint to the next.) The welds and distances were numbered/measured in the same direction as the spans were numbered, as shown in the design drawings.

To allow the welds to be easily identified in the future, MEIC engineers marked them (on the inside the girders) with a black, indelible felt-tip pen per the following notation:

- Line and Span: designated by the bridge notation shown on the design drawings, either "SW", "WS", "EW", or "WE", followed by a digit 1 through 8 to indicate the span number.
- Girders: designated as shown in the design drawings, either "A" or "B" in the case of Lines "SW" and "WS", or "D" or "E" in the case of Lines "EW" and "WE".
- Flanges: identified as either "B" for bottom flanges, or "T1" or "T2" for top flanges, where the designations "1" and "2" for the top flanges are as shown in the design drawings.

So, for example, a designation of SW6-A-T2-4 would indicate the fourth weld in Line "SW" (Bridge No. 09268A), Span No. 6, Girder A, Top Flange T2.

The design drawings showing plan and elevation views of the spans and elevation views of the girders for each unit follow the tables. Reference to these drawings can be used to identify the welds listed in the tables.

Table 1. Bridge 9268AWelds (Line SW)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 1 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
SW1-A-T1-1	41	1.00	7	1.00	7
SW1-A-T1-2	84	1.00	7	1.00	8 1/2
SW1-A-T1-3	142	1.50	8 1/2	1.75	10 1/2
SW1-A-T1-4	153	1.77	10 1/2	2.15	10 1/2
Span 1 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
SW1-A-T2-1	41	1.00	7	1.00	7
SW1-A-T2-2	85	1.00	7	1.00	8 1/2
SW1-A-T2-3	142	1.50	8 1/2	1.75	10 1/2
SW1-A-T2-4	153	1.77	10 1/2	2.15	10 1/2
Span 1 Girder A Flange B (distance measured from bulkhead of expansion joint No. 8)					
SW1-A-B-1	41	1.15		1.15	
SW1-A-B-2	86	1.18		1.18	
SW1-A-B-3	152	1.43		1.75	
Span 1 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
SW1-B-T1-1	39	1.05	7	1.05	7
SW1-B-T1-2	82	1.05	7	1.07	7
SW1-B-T1-3	117	1.05	7	1.27	9
SW1-B-T1-4	140	1.27	9	1.68	10 1/2
SW1-B-T1-5	148	1.7	10 1/2	2.17	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 1 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
SW1-B-T2-1	39	1.05	7	1.05	7
SW1-B-T2-2	82	1.05	7	1.07	7
SW1-B-T2-3	117	1.05	7	1.27	9
SW1-B-T2-4	140	1.27	9	1.68	10 1/2
SW1-B-T2-5	148	1.68	10 1/2	2.13	10 1/2
Span 1 Girder B Flange B (distance measured from bulkhead of expansion joint No. 8)					
SW1-B-B-1	43	1.17		1.17	
SW1-B-B-2	84	1.17		1.17	
SW1-B-B-3	148	1.45		1.78	
Span 2 Girder A Flange T1 (distance measured from centerline of pier No. 7)					
SW2-A-T1-1	9	2.18	NR	1.77	NR
SW2-A-T1-2	18	1.80	NR	1.52	NR
SW2-A-T1-3	36	1.55	NR	1.03	NR
SW2-A-T1-4	58	1.03	NR	1.53	NR
SW2-A-T1-5	90	1.55	NR	1.55	NR
SW2-A-T1-6	123	1.50	NR	1.0	NR
SW2-A-T1-7	155	1.55	NR	2.05	NR
SW2-A-T1-8	167	2.05	NR	2.55	NR

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 2 Girder A Flange T2 (distance measured from centerline of pier No. 7)					
SW2-A-T2-1	9	2.13	NR	1.80	NR
SW2-A-T2-2	18	1.78	NR	1.55	NR
SW2-A-T2-3	36	1.55	NR	1.05	NR
SW2-A-T2-4	58	1.05	NR	1.53	NR
SW2-A-T2-5	90	1.55	NR	1.53	NR
SW2-A-T2-6	123	1.50	NR	1.03	NR
SW2-A-T2-7	156	1.55	NR	2.05	NR
SW2-A-T2-8	168	2.05	NR	2.52	NR
Span 2 Girder A Flange B (distance measured from centerline of pier No. 7)					
SW2-A-B-1	9	1.78		1.42	
SW2-A-B-2	18	1.43		1.20	
SW2-A-B-3	155	1.12		1.55	
SW2-A-B-4	167	1.55		2.05	
Span 2 Girder B Flange T1 (distance measured from centerline of pier No. 7)					
SW2-B-T1-1	12	2.18	10	1.68	10
SW2-B-T1-2	24	1.70	9.5	1.25	8
SW2-B-T1-3	87	0.80	7	0.80	7
SW2-B-T1-4	155	1.25	8 1/2	1.65	10
SW2-B-T1-5	166	1.68	10	2.15	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 2 Girder B Flange T2 (distance measured from centerline of pier No. 7)					
SW2-B-T2-1	12	2.18	10	1.68	10
SW2-B-T2-2	24	1.68	9 1/2	1.28	8
SW2-B-T2-3	87	0.80	7	0.80	7
SW2-B-T2-4	155	1.25	8 1/2	1.65	10
SW2-B-T2-5	166	1.68	10	2.15	10
Span 2 Girder B Flange B (distance measured from centerline of pier No. 7)					
SW2-B-B-1	12	1.78		1.43	
SW2-B-B-2	24	1.45		1.05	
SW2-B-B-3	54	1.03		1.58	
SW2-B-B-4	121	0.83		1.01	
SW2-B-B-5	155	1.05		1.43	
SW2-B-B-6	166	1.45		1.77	
Span 3 Girder A Flange T1 (distance measured from centerline of pier No. 6)					
SW3-A-T1-1	10	2.52	NR	2.05	NR
SW3-A-T1-2	22	2.05	NR	1.55	NR
SW3-A-T1-3	78	1.40	NR	1.40	NR
SW3-A-T1-4[1]	125	1.40	NR	1.40	NR
SW3-A-T1-5	136	1.40	NR	1.02	NR

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder A Flange T2 (distance measured from centerline of pier No. 6)					
SW3-A-T1-1	11	2.52	NR	2.05	NR
SW3-A-T1-2	23	2.05	NR	1.55	NR
SW3-A-T1-3	78	1.50	NR	1.40	NR
SW3-A-T1-4	125	1.40	NR	1.40	NR
SW3-A-T1-5	136	1.40	NR	1.03	NR
Span 3 Girder A Flange B (distance measured from centerline of pier No. 6)					
SW3-A-B-1	10	2.05		1.52	
SW3-A-B-2	78	1.52		1.57	
Span 3 Girder B Flange T1 (distance measured from centerline of pier No. 6)					
SW3-B-T1-1	11	2.17	10 1/2	1.70	10 1/2
SW3-B-T1-2	46	1.25	8 1/2	0.92	7
SW3-B-T1-3	78	0.90	7	0.90	7
SW3-B-T1-4	118	0.92	7	0.90	7
Span 3 Girder B Flange T2 (distance measured from centerline of pier No. 6)					
SW3-B-T2-1	12	2.20	10 1/2	1.70	10 1/2
SW3-B-T2-2	46	1.25	8 1/2	0.92	7
SW3-B-T2-3	78	0.92	7	0.92	7
SW3-B-T2-4	118	0.92	7	0.92	7
Span 3 Girder B Flange B (distance measured from centerline of pier No. 6)					
SW3-B-B-1	11	1.80		1.50	
SW3-B-B-2	81	1.20		1.20	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 4 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 1)					
SW4-A-T1-1	31	1.20	7	0.83	7
SW4-A-T1-2	81	0.83	7	0.80	7
SW4-A-T1-3	112	0.80	7	1.33	7
SW4-A-T1-4	131	1.33	7	1.83	9
SW4-A-T1-5	146	1.85	9	2.30	10 1/2
Span 4 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 1)					
SW4-A-T2-1	31	1.20	7	0.85	7
SW4-A-T2-2	81	0.83	7	0.80	7
SW4-A-T2-3	112	0.80	7	1.35	7
SW4-A-T2-4	131	1.30	7	1.83	9
SW4-A-T2-5	146	1.85	9	2.30	10 1/2
Span 4 Girder A Flange B (distance measured from bulkhead of expansion joint No. 1)					
SW4-A-B-1	51	1.03		1.03	
SW4-A-B-2	86	1.05		1.05	
SW4-A-B-3	148	1.45		1.85	
Span 4 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 1)					
SW4-B-T1-1	66	0.85	7	0.85	7
SW4-B-T1-2	128	1.20	7	1.55	9
SW4-B-T1-3	142	1.55	9	1.83	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 4 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 1)					
SW4-B-T2-1	66	0.83	7	0.83	7
SW4-B-T2-2	128	1.20	7	1.55	9
SW4-B-T2-3	142	1.57	9	1.85	10
Span 4 Girder B Flange B (distance measured from bulkhead of expansion joint No. 1)					
SW4-B-B-1	47	0.83		0.83	
SW4-B-B-2	87	0.83		0.83	
SW4-B-B-3	142	1.18		1.45	
Span 5 Girder A Flange T1 (distance measured from centerline of pier No. 2)					
SW5-A-T1-1	16	2.32	10 1/2	1.85	9
SW5-A-T1-2	31	1.83	9	1.32	7 1/2
SW5-A-T1-3[2]	90	0.83	7 1/2	0.83	7 1/2
Span 5 Girder A Flange T2 (distance measured from centerline of pier No. 2)					
SW5-A-T2-1	16	2.32	10 1/2	1.85	9
SW5-A-T2-2	31	1.83	9	1.32	7 1/2
SW5-A-T2-3[2]	90	0.83	7 1/2	0.83	7 1/2
Span 5 Girder A Flange B (distance measured from centerline of pier No. 2)					
SW5-A-B-1	10	1.83		1.43	
SW5-A-B-2	28	1.43		0.93	
SW5-A-B-3[1]	32	0.95		0.95	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 5 Girder B Flange T1 (distance measured from centerline of pier No. 2)					
SW5-B-T1-1	11	1.85	10	1.55	9
SW5-B-T1-2	25	1.55	9	1.2	7
SW5-B-T1-3	89	0.83	7	0.83	7
Span 5 Girder B Flange T2 (distance measured from centerline of pier No. 2)					
SW5-B-T2-1	11	1.83	10	1.55	9
SW5-B-T2-2	25	1.58	9	1.2	7
SW5-B-T2-3	89	0.83	7	0.83	7
Span 5 Girder B Flange B (distance measured from centerline of pier No. 2)					
SW5-B-B-1	11	1.43		1.20	
SW5-B-B-2	83	0.83		0.85	
Span 6 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 3)					
SW6-A-T1-1	40	0.80	7	0.80	7
SW6-A-T1-2	92	1.45	9	1.45	9
SW6-A-T1-3	107	1.45	9	1.80	9
Span 6 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 3)					
SW6-A-T2-1	40	0.83	7	0.83	7
SW6-A-T2-2	92	1.40	9	1.40	9
SW6-A-T2-3	107	1.45	9	1.83	9
Span 6 Girder A Flange B (distance measured from bulkhead of expansion joint No. 3)					
SW6-A-B-1	39	0.83		0.80	
SW6-A-B-2	105	1.05		1.30	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 6 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 3)					
SW6-B-T1-1	39	0.80	7	0.80	7
SW6-B-T1-2	102	1.20	8 1/2	1.50	8 1/2
Span 6 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 3)					
SW6-B-T2-1	39	0.80	7	0.80	7
SW6-B-T2-2	102	1.20	8 1/2	1.50	8 1/2
Span 6 Girder B Flange B (distance measured from bulkhead of expansion joint No. 3)					
SW6-B-B-1	39	0.85		0.85	
SW6-B-B-2	102	1.05		1.20	
Span 7 Girder A Flange T1 (distance measured from centerline of pier No. 4)					
SW7-A-T1-1	11	1.83	9	1.45	9
SW7-A-T1-2	23	1.45	9	1.08	7
SW7-A-T1-3	51	0.83	7	0.83	7
SW7-A-T1-4	79	0.83	8 1/2	0.83	8 1/2
SW7-A-T1-5	150	1.55	8 1/2	2.10	8 1/2
Span 7 Girder A Flange T2 (distance measured from centerline of pier No. 4)					
SW7-A-T2-1	11	1.83	9	1.48	9
SW7-A-T2-2	23	1.45	9	1.08	9
SW7-A-T2-3	51	0.83	7	0.83	7
SW7-A-T2-4	79	0.83	8 1/2	0.83	8 1/2
SW7-A-T2-5	150	1.53	8 1/2	2.10	8 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)					
Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 7 Girder A Flange B (distance measured from centerline of pier No. 4)					
SW7-A-B-1	10	1.28		1.08	
SW7-A-B-2	79	0.83		0.85	
SW7-A-B-3	146	1.08		1.50	
Span 7 Girder B Flange T1 (distance measured from centerline of pier No. 4)					
SW7-B-T1-1	10	1.55	8 1/2	1.20	8 1/2
SW7-B-T1-2	20	1.20	8 1/2	1.05	7 1/2
SW7-B-T1-3	76	0.83	7 1/2	0.85	7 1/2
SW7-B-T1-4	145	1.43	9	1.83	9
Span 7 Girder B Flange T2 (distance measured from centerline of pier No. 4)					
SW7-B-T2-1	10	1.55	8 1/2	1.20	8 1/2
SW7-B-T2-2	20	1.20	8 1/2	1.03	7 1/2
SW7-B-T2-3	76	0.83	7 1/2	0.85	7 1/2
SW7-B-T2-4	145	1.40	9	1.83	9
Span 7 Girder B Flange B (distance measured from centerline of pier No. 4)					
SW7-B-B-1	10	1.18		1.08	
SW7-B-B-2	76	0.85		0.85	
SW7-B-B-3	143	1.03		1.27	
Span 8 Girder A Flange T1 (distance measured from centerline of pier No. 5)					
SW8-A-T1-1	10	2.10	8 1/2	1.55	8 1/2
SW8-A-T1-2	41	1.08	7	0.78	7
SW8-A-T1-3	93	0.80	7	0.80	7

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 1. Bridge 9268AWelds (Line SW)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 8 Girder A Flange T2 (distance measured from centerline of pier No. 5)					
SW8-A-T2-1	10	2.10	8 1/2	1.53	8 1/2
SW8-A-T2-2	41	1.08	7	0.78	7
SW8-A-T2-3	93	0.80	7	0.80	7
Span 8 Girder A Flange B (distance measured from centerline of pier No. 5)					
SW8-A-B-1	13	1.45		1.08	
SW8-A-B-2	93	0.80		0.80	
Span 8 Girder B Flange T1 (distance measured from centerline of pier No. 5)					
SW8-B-T1-1	10	1.83	9	1.43	9
SW8-B-T1-2	54	0.80	7 1/2	0.80	7 1/2
SW8-B-T1-3	95	0.78	7 1/2	0.80	7 1/2
Span 8 Girder B Flange T2 (distance measured from centerline of pier No. 5)					
SW8-B-T2-1	10	1.85	9	1.45	9
SW8-B-T2-2	55	0.80	7 1/2	0.80	7 1/2
SW8-B-T2-3	95	0.80	7 1/2	0.83	7 1/2
Span 8 Girder B Flange B (distance measured from centerline of pier No. 5)					
SW8-B-B-1	10	1.30		1.05	
SW8-B-B-2	54	0.80		0.80	
SW8-B-B-3	95	0.83		0.83	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 1 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
WS1-A-T1-1	36	0.83	7	0.80	7
WS1-A-T1-2	79	0.80	7	0.85	7
WS1-A-T1-3	138	1.85	7	2.36	10
WS1-A-T1-4	151	2.36	10	2.85	10
Span 1 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
WS1-A-T2-1	36	0.83	7	0.80	7
WS1-A-T2-2	79	0.80	7	0.83	7
WS1-A-T2-3	138	1.85	7	2.36	10
WS1-A-T2-4	151	2.32	10	2.85	10
Span 1 Girder A Flange B (distance measured from bulkhead of expansion joint No. 8)					
WS1-A-B-1	40	1.08		1.05	
WS1-A-B-2	80	1.10		1.10	
WS1-A-B-3	133	1.05		1.55	
WS1-A-B-4	146	1.60		2.00	
WS1-A-B-5	152	1.98		2.30	
Span 1 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 8)					
WS1-B-T1-1	34	0.83	7	0.80	7
WS1-B-T1-2	78	0.80	7	0.85	7
WS1-B-T1-3	125	1.70	8 1/2	1.70	8 1/2
WS1-B-T1-4	139	1.70	8 1/2	2.03	10
WS1-B-T1-5	148	2.05	10	2.58	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 1 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 8)					
WS1-B-T2-1	34	0.83	7	0.80	7
WS1-B-T2-2	78	0.80	7	0.85	7
WS1-B-T2-3	125	1.70	8 1/2	1.70	8 1/2
WS1-B-T2-4	139	1.70	8 1/2	2.05	10
WS1-B-T2-5	148	2.05	10	2.58	10
Span 1 Girder B Flange B (distance measured from bulkhead of expansion joint No. 8)					
WS1-B-B- 1	38	1.08		1.08	
WS1-B-B-2	78	1.05		1.08	
WS1-B-B-3	144	1.45		1.85	
WS1-B-B-4	151	1.88		2.08	
Span 2 Girder A Flange T1 (distance measured from centerline of pier No. 1)					
WS2-A-T1-1	10	2.83	10	2.36	10
WS2-A-T1-2	22	2.33	10	1.85	7
WS2-A-T1-3	123	0.92	7	0.90	7
Span 2 Girder A Flange T2 (distance measured from centerline of pier No. 1					
WS2-A-T2-1	10	2.86	10	2.36	10
WS2-A-T2-2	22	2.36	10	1.87	7
WS2-A-T2-3	123	0.92	7	0.90	7

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 2 Girder A Flange B (distance measured from centerline of pier No. 1					
WS2-A-B-1	9	1.90		1.98	
WS2-A-B-2	17	1.98		1.56	
WS2-A-B-3	26	1.55		1.29	
WS2-A-B-4	85	1.56		1.20	
WS2-A-B-5	122	1.20		1.20	
Span 2 Girder B Flange T1 (distance measured from centerline of pier No. 1					
WS2-B-T1-1	12	2.58	10	2.03	10
WS2-B-T1-2	21	2.03	10	1.70	9
WS2-B-T1-3	84	0.80	7	0.83	7
WS2-B-T1-4	125	0.83	7	0.83	7
Span 2 Girder B Flange T2 (distance measured from centerline of pier No. 1					
WS2-B-T2-1	12	2.58	10	2.06	10
WS2-B-T2-2	21	2.02	10	1.70	9
WS2-B-T2-3	84	0.80	7	0.80	7
WS2-B-T2-4	125	0.83	7	0.83	7
Span 2 Girder B Flange B (distance measured from centerline of pier No. 1					
WS2-B-B-1	8	2.08		1.92	
WS2-B-B-2	15	1.92		1.45	
WS2-B-B-3	84	1.08		1.10	
WS2-B-B-4	125	1.05		1.07	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 3 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WS3-A-T1-1	22	0.80	7	1.30	7
WS3-A-T1-2	37	1.30	7	1.32	7
WS3-A-T1-3	86	1.32	7	1.32	7
WS3-A-T1-4	133	1.55	7	2.05	7
WS3-A-T1-5	142	2.05	7	2.55	9 1/2
WS3-A-T1-6	153	2.58	9 1/2	3.08	10
Span 3 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WS3-A-T2-1	22	0.80	7	1.30	7
WS3-A-T2-2	37	1.30	7	1.32	7
WS3-A-T2-3	86	1.30	7	1.30	7
WS3-A-T2-4	133	1.55	7	2.05	7
WS3-A-T2-5	142	2.05	7	2.55	9 1/2
WS3-A-T2-6	153	2.58	9 1/2	3.08	10
Span 3 Girder A Flange B (distance measured from bulkhead of expansion joint No. 2)					
WS3-A-B-1	31	1.20		1.45	
WS3-A-B-2	84	1.48		1.48	
WS3-A-B-3[4]	136	1.08		1.55	
WS3-A-B-4	149	1.55		2.10	
WS3-A-B-5	158	2.08		2.47	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 3 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WS3-B-T1-1	31	0.80	7	0.93	7
WS3-B-T1-2	39	0.93	7	0.95	7
WS3-B-T1-3	85	0.93	7	0.83	7
WS3-B-T1-4	137	1.55	7	1.93	9 1/2
WS3-B-T1-5	148	1.95	9 1/2	2.30	10
Span 3 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WS3-B-T2-1	31	0.80	7	0.93	7
WS3-B-T2-2	39	0.93	7	0.95	7
WS3-B-T2-3	85	0.93	7	0.83	7
WS3-B-T2-4	137	1.55	7	1.93	9 1/2
WS3-B-T2-5	148	1.95	9 1/2	2.30	10
Span 3 Girder B Flange B (distance measured from bulkhead of expansion joint No. 2)					
WS3-B-B-1	39	1.05		1.08	
WS3-B-B-2	85	1.05		1.08	
WS3-B-B-3	137	1.05		1.55	
WS3-B-B-4	151	1.58		1.95	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder A Flange T1 (distance measured from centerline of pier No. 3)					
WS4-A-T1-1	14	3.05	10	2.58	9
WS4-A-T1-2	23	2.52	9	2.22	7
WS4-A-T1-3	32	2.22	7	1.65	7
WS4-A-T1-4	59	1.28	7	0.83	7
WS4-A-T1-5	87	0.80	7	0.80	7
WS4-A-T1-6	115	0.83	7	0.80	7
Span 4 Girder A Flange T2 (distance measured from centerline of pier No. 3)					
WS4-A-T2-1	14	3.05	10	2.58	9
WS4-A-T2-2	23	2.52	9	2.22	7
WS4-A-T2-3	32	2.22	7	1.65	7
WS4-A-T2-4	59	1.28	7	0.83	7
WS4-A-T2-5	87	0.80	7	0.80	7
WS4-A-T2-6	115	0.80	7	0.80	7
Span 4 Girder A Flange B (distance measured from centerline of pier No. 3)					
WS4-A-B-1	10	2.47		2.08	
WS4-A-B-2	17	2.08		1.55	
WS4-A-B-3	34	1.55		1.05	
WS4-A-B-4	82	1.08		1.05	
WS4-A-B-5	118	1.10		1.10	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder B Flange T1 (distance measured from centerline of pier No. 3)					
WS4-B-T1-1	14	2.30	10	1.95	8 1/2
WS4-B-T1-2	25	1.90	8 1/2	1.53	7
WS4-B-T1-3	75	0.80	7	0.80	7
WS4-B-T1-4	114	0.80	7	0.78	7
Span 4 Girder B Flange T2 (distance measured from centerline of pier No. 3)					
WS4-B-T2-1	14	2.30	10	1.95	8 1/2
WS4-B-T2-2	25	1.95	8 1/2	1.55	7
WS4-B-T2-3	75	0.80	7	0.80	7
WS4-B-T2-4	114	0.80	7	0.78	7
Span 4 Girder B Flange B (distance measured from centerline of pier No. 3)					
WS4-B-B-1	10	1.93		1.58	
WS4-B-B-2	24	1.58		1.10	
WS4-B-B-3	75	0.80		0.85	
WS4-B-B-4	114	0.85		0.80	
Span 5 Girder A Flange T1 (distance measured from bulkhead of expansion joint No. 4)					
WS5-A-T1-1	33	1.07	7	1.05	7
WS5-A-T1-2	78	1.05	7	1.05	7
WS5-A-T1-3	128	1.55	7	2.08	8
WS5-A-T1-4	140	2.05	8	2.42	9

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder A Flange T2 (distance measured from bulkhead of expansion joint No. 4)					
WS5-A-T2-1	33	1.08	7	1.05	7
WS5-A-T2-2	78	1.05	7	1.05	7
WS5-A-T2-3	128	1.55	7	2.05	8
WS5-A-T2-4	140	2.05	8	2.42	9
Span 5 Girder A Flange B (distance measured from bulkhead of expansion joint No. 4)					
WS5-A-B-1	47	1.20		1.20	
WS5-A-B-2	140	1.43		1.90	
Span 5 Girder B Flange T1 (distance measured from bulkhead of expansion joint No. 4)					
WS5-B-T1-1	38	0.80	7	0.80	7
WS5-B-T1-2	76	0.80	7	0.80	7
WS5-B-T1-3	132	1.45	8 1/2	1.85	10
Span 5 Girder B Flange T2 (distance measured from bulkhead of expansion joint No. 4)					
WS5-B-T2-1	38	0.83	7	0.80	7
WS5-B-T2-2	76	0.80	7	0.83	7
WS5-B-T2-3	132	1.45	8 1/2	1.80	10
Span 5 Girder B Flange B (distance measured from bulkhead of expansion joint No. 4)					
WS5-B-B-1	36	0.93		0.93	
WS5-B-B-2	139	1.33		1.55	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 6 Girder A Flange T1 (distance measured from centerline of pier No. 5)					
WS6-A-T1-1	13	2.40	10	2.08	8 1/2
WS6-A-T1-2	27	2.05	8 1/2	1.55	7 1/2
WS6-A-T1-3	89	0.83	7 1/2	0.83	7 1/2
WS6-A-T1-4	150	1.60	7 1/2	2.08	9
WS6-A-T1-5	162	2.10	9	2.43	10
Span 6 Girder A Flange T2 (distance measured from centerline of pier No. 5)					
WS6-A-T2-1	13	2.40	10	2.05	8 1/2
WS6-A-T2-2	27	2.08	8 1/2	1.55	7 1/2
WS6-A-T2-3	89	0.83	7 1/2	0.83	7 1/2
WS6-A-T2-4	150	1.55	7 1/2	2.08	9
WS6-A-T2-5	162	2.10	9	2.43	10
Span 6 Girder A Flange B (distance measured from centerline of pier No. 5)					
WS6-A-B-1	13	1.93		1.50	
WS6-A-B-2	63	0.85		0.85	
WS6-A-B-3	113	0.85		0.85	
Span 6 Girder B Flange T1 (distance measured from centerline of pier No. 5)					
WS6-B-T1-1	15	1.83	10	1.43	8 1/2
WS6-B-T1-2	86	0.83	7	0.80	7
WS6-B-T1-3	155	1.55	8 1/2	2.02	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 6 Girder B Flange T2 (distance measured from centerline of pier No. 5)					
WS6-B-T2-1	15	1.83	10	1.43	8 1/2
WS6-B-T2-2	86	0.83	7	0.80	7
WS6-B-T2-3	155	1.55	8 1/2	2.02	10
Span 6 Girder B Flange B (distance measured from centerline of pier No. 5)					
WS6-B-B-1	9	1.55		1.33	
WS6-B-B-2	86	0.83		0.80	
WS6-B-B-3	160	1.32		1.70	
Span 7 Girder A Flange T1 (distance measured from centerline of pier No. 5A)					
WS7-A-T1-1	13	2.40	10	2.05	8
WS7-A-T1-2	24	2.08	8	1.55	8
WS7-A-T1-3	78	1.03	7	1.05	7
WS7-A-T1-4	122	1.03	7	1.03	7
Span 7 Girder A Flange T2 (distance measured from centerline of pier No. 5A)					
WS7-A-T2-1	13	2.43	10	2.05	8
WS7-A-T2-2	24	2.10	8	1.55	8
WS7-A-T2-3	78	1.03	7	1.05	7
WS7-A-T2-4	122	1.05	7	1.05	7
Span 7 Girder A Flange B (distance measured from centerline of pier No. 5A)					
WS7-A-B-1	13	1.93		1.45	
WS7-A-B-2	77	1.18		1.18	
WS7-A-B-3	122	1.18		1.18	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 2. Bridge 9268B Welds (Line WS)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 7 Girder B Flange T1 (distance measured from centerline of pier No. 5A)					
WS7-B-T1-1	14	2.05	10	1.55	9
WS7-B-T1-2	72	0.83	7	0.83	7
WS7-B-T1-3	120	0.80	7	0.80	7
Span 7 Girder B Flange T2 (distance measured from centerline of pier No. 5A)					
WS7-B-T2-1	14	2.02	10	1.55	9
WS7-B-T2-2	72	0.80	7	0.83	7
WS7-B-T2-3	120	0.80	7	0.80	7
Span 7 Girder B Flange B (distance measured from centerline of pier No. 5A)					
WS7-B-B-1	10	1.68		1.35	
WS7-B-B-2	73	1.03		1.03	
WS7-B-B-3	116	1.08		1.05	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 3. Bridge 9268E Welds (Line WE)					
Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder D Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-T1-1[5]	39	1.05	7	1.05	7
WE3-D-T1-2[5]	84	1.05	7	1.07	7
WE3-D-T1-3	145	1.95	9	2.47	10
Span 3 Girder D Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-T2-1[5]	39	1.03	7	1.05	7
WE3-D-T2-2	84	1.05	7	1.03	7
WE3-D-T2-3	145	1.97	9	2.50	10
Span 3 Girder D Flange B (distance measured from bulkhead of expansion joint No. 2)					
WE3-D-B-1	41	1.20		1.13	
WE3-D-B-2	86	1.50		1.20	
WE3-D-B-3[5]	145	1.45		1.95	
Span 3 Girder E Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-T1-1	45	0.80	7 1/2	0.78	7 1/2
WE3-E-T1-2	91	0.80	7 1/2	0.80	7 1/2
WE3-E-T1-3	138	1.27	9	1.83	10
Span 3 Girder E Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-T2-1	45	0.78	7 1/2	0.78	7 1/2
WE3-E-T2-2	91	0.80	7 1/2	0.78	7 1/2
WE3-E-T2-3	138	1.23	9	1.80	10

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 3. Bridge 9268E Welds (Line WE)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder E Flange B (distance measured from bulkhead of expansion joint No. 2)					
WE3-E-B-1	38	0.85		0.83	
WE3-E-B-2	83	0.83		0.85	
WE3-E-B-3[5]	138	1.05		1.45	
Span 4 Girder D Flange T1 (distance measured from centerline of pier No. 3)					
WE4-D-T1-1	15	2.45	10	2.05	9
WE4-D-T1-2	26	2.10	9	1.57	7 1/2
WE4-D-T1-3[5]	79	0.80	7 1/2	0.83	7 1/2
WE4-D-T1-4[5]	116	0.83	7 1/2	0.80	7 1/2
WE4-D-T1-5	160	1.58	8	2.08	8
Span 4 Girder D Flange T2 (distance measured from centerline of pier No. 3)					
WE4-D-T2-1	15	2.47	10	2.07	9
WE4-D-T2-2	26	2.15	9	1.57	7 1/2
WE4-D-T2-3[5]	89	0.80	7 1/2	0.80	7 1/2
WE4-D-T2-4	160	1.60	8	2.10	8
Span 4 Girder D Flange B (distance measured from centerline of pier No. 3)					
WE4-D-B-1[5]	15	1.93		1.45	
WE4-D-B-2	76	0.78		0.83	
WE4-D-B-3	101	0.83		0.80	
WE4-D-B-4[5]	160	1.20		1.68	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 3. Bridge 9268E Welds (Line WE)					
Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder E Flange T1 (distance measured from centerline of pier No. 3)					
WE4-E-T1-1	15	1.80	10	1.30	10
WE4-E-T1-2	86	0.80	7	0.80	7
WE4-E-T1-3	153	1.20	9	1.63	10 1/2
Span 4 Girder E Flange T2 (distance measured from centerline of pier No. 3)					
WE4-E-T2-1	15	1.80	10	1.30	10
WE4-E-T2-2	86	0.80	7	0.80	7
WE4-E-T2-3	153	1.20	9	1.60	10 1/2
Span 4 Girder E Flange B (distance measured from centerline of pier No. 3)					
WE4-E-B-1	52	0.80		0.80	
WE4-E-B-2	86	0.80		0.80	
WE4-E-B-3[5]	153	1.08		1.30	
Span 5 Girder D Flange T1 (distance measured from centerline of pier No. 4)					
WE5-D-T1-1	15	2.10	10	1.58	8 1/2
WE5-D-T1-2[5]	72	0.80	7	0.80	7
WE5-D-T1-3[5]	104	0.83	7	0.83	7
WE5-D-T1-4	160	1.63	9	2.08	10 1/2
Span 5 Girder D Flange T2 (distance measured from centerline of pier No. 4)					
WE5-D-T2-1	15	2.08	10	1.58	8 1/2
WE5-D-T2-2[5]	72	0.80	7	0.80	7
WE5-D-T2-3[5]	104	0.80	7	0.83	7
WE5-D-T2-4	160	1.58	9	2.08	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 3. Bridge 9268E Welds (Line WE)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder D Flange B (distance measured from centerline of pier No. 4)					
WE5-D-B-1[5]	15	1.77		1.20	
WE5-D-B-2	75	0.83		0.85	
WE5-D-B-3	110	0.80		0.83	
WE5-D-B-4[5]	160	1.20		1.70	
Span 5 Girder E Flange T1 (distance measured from centerline of pier No. 4)					
WE5-E-T1-1	13	1.60	10 1/2	1.08	9
WE5-E-T1-2[5]	76	0.78	7	0.78	7
WE5-E-T1-3[5]	113	0.80	7	0.80	7
WE5-E-T1-4	157	1.20	8 1/2	1.58	10 1/2
Span 5 Girder E Flange T2 (distance measured from centerline of pier No. 4)					
WE5-E-T2-1	13	1.60	10 1/2	1.10	9
WE5-E-T2-2[5]	76	0.80	7	0.80	7
WE5-E-T2-3[5]	113	0.80	7	0.80	7
WE5-E-T2-4	157	1.20	8 1/2	1.60	10 1/2
Span 5 Girder E Flange B (distance measured from centerline of pier No. 4)					
WE5-E-B-1[5]	13	1.38		1.08	
WE5-E-B-2	79	0.80		0.80	
WE5-E-B-3	119	0.80		0.80	
WE5-E-B-4[5]	157	1.08		1.28	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 3. Bridge 9268E Welds (Line WE)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 6 Girder D Flange T1 (distance measured from centerline of pier No. 5)					
WE6-D-T1-1	15	2.07	10 1/2	1.58	9
WE6-D-T1-2[5]	85	0.80	7 1/2	0.80	7 1/2
WE6-D-T1-3[5]	129	0.80	7 1/2	0.80	7 1/2
Span 6 Girder D Flange T2 (distance measured from centerline of pier No. 5)					
WE6-D-T2-1	15	2.10	10 1/2	1.58	9
WE6-D-T2-2[5]	85	0.80	7 1/2	0.80	7 1/2
WE6-D-T2-3[5]	129	0.80	7 1/2	0.80	7 1/2
Span 6 Girder D Flange B (distance measured from centerline of pier No. 5)					
WE6-D-B-1[5]	15	1.73		1.20	
WE6-D-B-2	65	0.83		0.85	
WE6-D-B-3	106	0.83		0.83	
Span 6 Girder E Flange T1 (distance measured from centerline of pier No. 5)					
WE6-E-T1-1	13	1.55	10	1.18	8 1/2
WE6-E-T1-2[5]	69	0.80	7	0.83	7
WE6-E-T1-3[5]	101	0.83	7	0.83	7
Span 6 Girder E Flange T2 (distance measured from centerline of pier No. 5)					
WE6-E-T2-1	13	1.58	10	1.18	8 1/2
WE6-E-T2-2[5]	69	0.83	7	0.80	7
WE6-E-T2-3[5]	101	0.80	7	0.83	7
Span 6 Girder E Flange B (distance measured from centerline of pier No. 5)					
WE6-E-B-1[5]	13	1.38		1.05	
WE6-E-B-2	77	0.85		0.85	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 4. Bridge 9268W Welds (Line EW)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder D Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
EW3-D-T1-1	36	0.80	7	1.05	7
EW3-D-T1-2	84	1.03	7	1.03	7
EW3-D-T1-3	145	1.90	8 1/2	2.43	10
Span 3 Girder D Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
EW3-D-T2-1	36	0.83	7	1.05	7
EW3-D-T2-2[5]	84	1.03	7	1.05	7
EW3-D-T2-3	145	1.90	8 1/2	2.43	10
Span 3 Girder D Flange B (distance measured from bulkhead of expansion joint No. 2)					
EW3-D-B-1	24	0.80		1.15	
EW3-D-B-2	43	1.12		1.12	
EW3-D-B-3	85	1.17		1.17	
EW3-D-B-4	145	1.48		1.98	
Span 3 Girder E Flange T1 (distance measured from bulkhead of expansion joint No. 2)					
EW3-E-T1-1	38	0.78	7 1/2	0.80	7 1/2
EW3-E-T1-2	78	0.80	7 1/2	0.80	7 1/2
EW3-E-T1-3	138	1.30	9	1.78	10 1/2
Span 3 Girder E Flange T2 (distance measured from bulkhead of expansion joint No. 2)					
EW3-E-T2-1	38	0.80	7 1/2	0.80	7 1/2
EW3-E-T2-2	78	0.80	7 1/2	0.80	7 1/2
EW3-E-T2-3	138	1.28	9	1.80	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 4. Bridge 9268W Welds (Line EW)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 3 Girder E Flange B (distance measured from bulkhead of expansion joint No. 2)					
EW3-E-B-1	39	0.80		0.83	
EW3-E-B-2	82	0.80		0.80	
EW3-E-B-3	138	1.07		1.45	
Span 4 Girder D Flange T1 (distance measured from centerline of pier No. 3)					
EW4-D-T1-1	15	2.40	10	2.05	8
EW4-D-T1-2	26	2.03	8	1.55	7
EW4-D-T1-3	83	0.80	7	0.80	7
EW4-D-T1-4	122	0.80	7	0.80	7
EW4-D-T1-5	161	1.55	8 1/2	2.05	10
Span 4 Girder D Flange T2 (distance measured from centerline of pier No. 3)					
EW4-D-T2-1	15	2.42	10	2.05	8
EW4-D-T2-2	26	2.03	8	1.55	7
EW4-D-T2-3	83	0.80	7	0.80	7
EW4-D-T2-4	122	0.80	7	0.80	7
EW4-D-T2-5	161	1.55	8 1/2	2.05	10
Span 4 Girder D Flange B (distance measured from centerline of pier No. 3)					
EW4-D-B-1	15	1.95		1.45	
EW4-D-B-2	78	0.85		0.80	
EW4-D-B-3	99	0.80		0.80	
EW4-D-B-4	161	1.17		1.68	

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 4. Bridge 9268W Welds (Line EW)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 4 Girder E Flange T1 (distance measured from centerline of pier No. 3)					
EW4-E-T1-1	15	1.82	10 1/2	1.32	8 1/2
EW4-E-T1-2	81	0.80	7	0.78	7
EW4-E-T1-3	121	0.80	7	0.80	7
EW4-E-T1-4	153	1.20	9	1.55	10
Span 4 Girder E Flange T2 (distance measured from centerline of pier No. 3)					
EW4-E-T2- 1	15	1.83	10 1/2	1.30	8 1/2
EW4-E-T2- 2	81	0.80	7	0.83	7
EW4-E-T2- 3	121	0.78	7	0.80	7
EW4-E-T2- 4	153	1.20	9	1.55	10
Span 4 Girder E Flange B (distance measured from centerline of pier No. 3)					
EW4-E-B-1	15	1.43		1.05	
EW4-E-B-2	81	0.85		0.80	
EW4-E-B-3	121	0.80		0.80	
EW4-E-B-4	153	1.07		1.28	
Span 5 Girder D Flange T1 (distance measured from centerline of pier No. 4)					
EW5-D-T1-1	15	2.02	10 1/2	1.55	8 1/2
EW5-D-T1-2[5]	81	0.80	7	0.80	7
EW5-D-T1-3	121	0.80	7	0.80	7
EW5-D-T1-4	160	1.55	8 1/2	2.05	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 4. Bridge 9268W Welds (Line EW)					
		Flange Before Weld		Flange After Weld	
Weld No.	Distance (feet)	Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder D Flange T2 (distance measured from centerline of pier No. 4)					
EW5-D-T2-1	15	2.05	10 1/2	1.55	8 1/2
EW5-D-T2-2[5]	81	0.80	7	0.80	7
EW5-D-T2-3[5]	121	0.83	7	0.80	7
EW5-D-T2-4	160	1.55	8 1/2	2.05	10 1/2
Span 5 Girder D Flange B (distance measured from centerline of pier No. 4)					
EW5-D-B-1	15	1.68		1.20	
EW5-D-B-2	109	0.80		0.80	
EW5-D-B-3	161	1.20		1.65	
Span 5 Girder E Flange T1 (distance measured from centerline of pier No. 4)					
EW5-E-T1-1[6]	13	1.55	10	1.18	8 1/2
EW5-E-T1-2	78	0.78	7	0.80	7
EW5-E-T1-3	118	0.80	7	0.80	7
EW5-E-T1-4	157	1.18	8 1/2	1.55	10 1/2
Span 5 Girder E Flange T2 (distance measured from centerline of pier No. 4)					
EW5-E-T2-1[6]	13	1.53	10	1.18	8 1/2
EW5-E-T2-2	78	0.78	7	0.78	7
EW5-E-T2-3	118	0.78	7	0.80	7
EW5-E-T2-4	157	1.18	8 1/2	1.55	10 1/2

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 4. Bridge 9268W Welds (Line EW)

Weld No.	Distance (feet)	Flange Before Weld		Flange After Weld	
		Thickness (inch)	Width[3] (inch)	Thickness (inch)	Width[3] (inch)
Span 5 Girder E Flange B (distance measured from centerline of pier No. 4)					
EW5-E-B-1[6]	13	1.28		1.05	
EW5-E-B-2	78	0.83		0.83	
EW5-E-B-3	118	0.83		0.80	
EW5-E-B-4	157	1.03		1.30	
Span 6 Girder D Flange T1 (distance measured from centerline of pier No. 5)					
EW6-D-T1-1	15	2.02	10 1/2	1.55	9
EW6-D-T1-2	81	0.80	7	0.83	7
EW6-D-T1-3	120	0.80	7	0.80	7
Span 6 Girder D Flange T2 (distance measured from centerline of pier No. 5)					
EW6-D-T2-1	15	2.02	10 1/2	1.55	9
EW6-D-T2-2	81	0.83	7	0.83	7
EW6-D-T2-3	120	0.80	7	0.83	7
Span 6 Girder D Flange B (distance measured from centerline of pier No. 5)					
EW6-D-B-1	15	1.65		1.21	
EW6-D-B-2	81	0.80		0.80	
EW6-D-B-3	99	0.83		0.83	
Span 6 Girder E Flange T1 (distance measured from centerline of pier No. 5)					
EW6-E-T1-1	14	1.53	10	1.18	9
EW6-E-T1-2	77	0.83	7	0.83	7
EW6-E-T1-3	116	0.80	7	0.80	7

Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on <u>inside</u> of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

Table 4. Bridge 9268W Welds (Line EW)					
		Flange Before Weld		**Flange After Weld**	
Weld No.	**Distance (feet)**	**Thickness (inch)**	**Width[3] (inch)**	**Thickness (inch)**	**Width[3] (inch)**
Span 6 Girder E Flange T2 (distance measured from centerline of pier No. 5)					
EW6-E-T2-1	14	1.60	10	1.18	9
EW6-E-T2-2	77	0.80	7	0.80	7
EW6-E-T2-3	116	0.78	7	0.80	7
Span 6 Girder E Flange B (distance measured from centerline of pier No. 5)					
EW6-E-B-1	14	1.30		1.03	
EW6-E-B-2	77	0.80		0.83	
EW6-E-B-3	116	0.83		0.83	

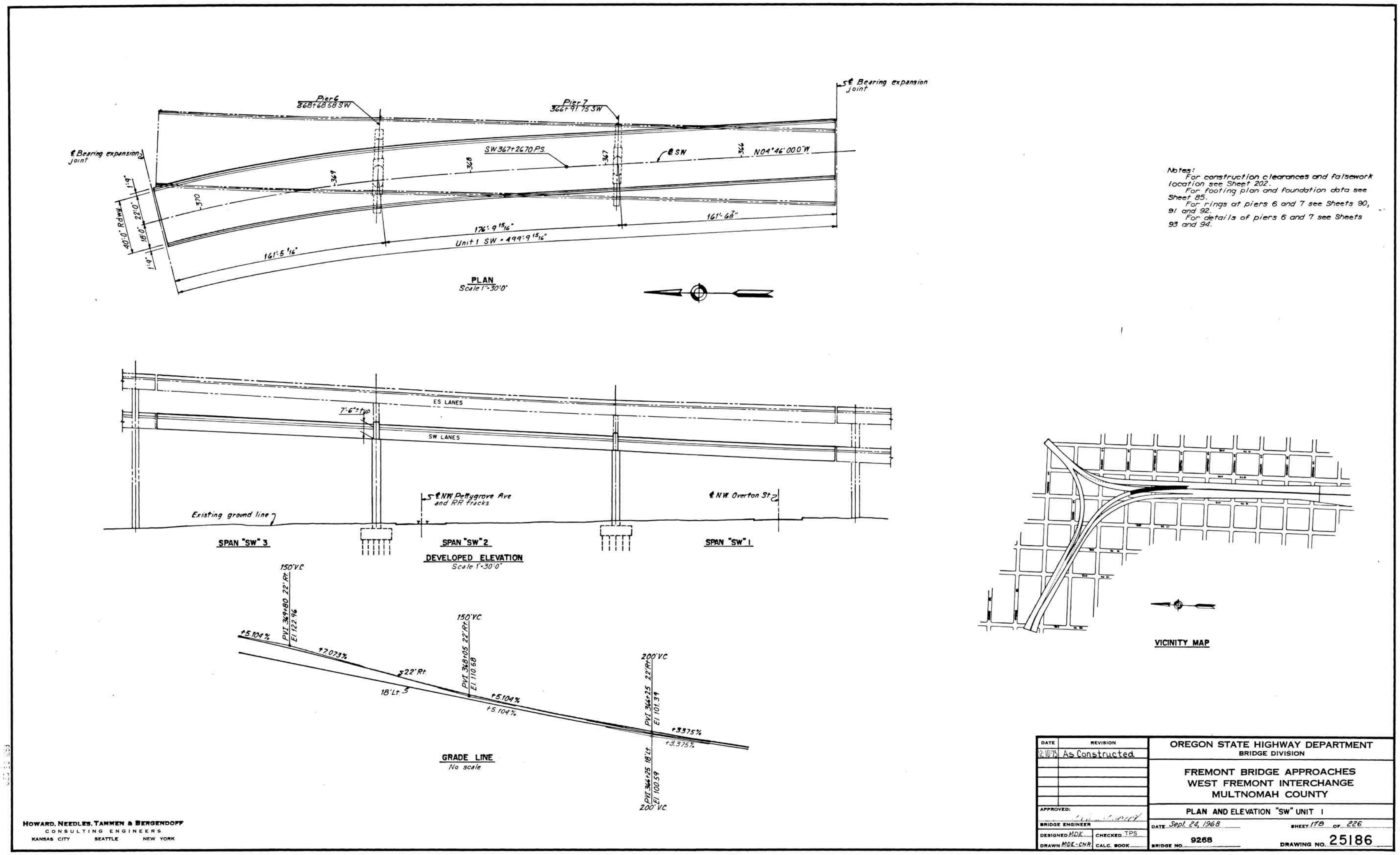
Notes: [1]Weld crown difficult to distinguish; presence of weld not confirmed at this location. [2]Weld did not appear to be an Electroslag Weld (ESW). [3]Top flange width on inside of girder only. [4]Weld located under gusset. [5]Weld crown appearance suggested the weld had either been repaired by some process other than ESW, or the weld was not an ESW. [6]Gussets located 1.5 ft on either side of weld. NR = Flange width not recorded.

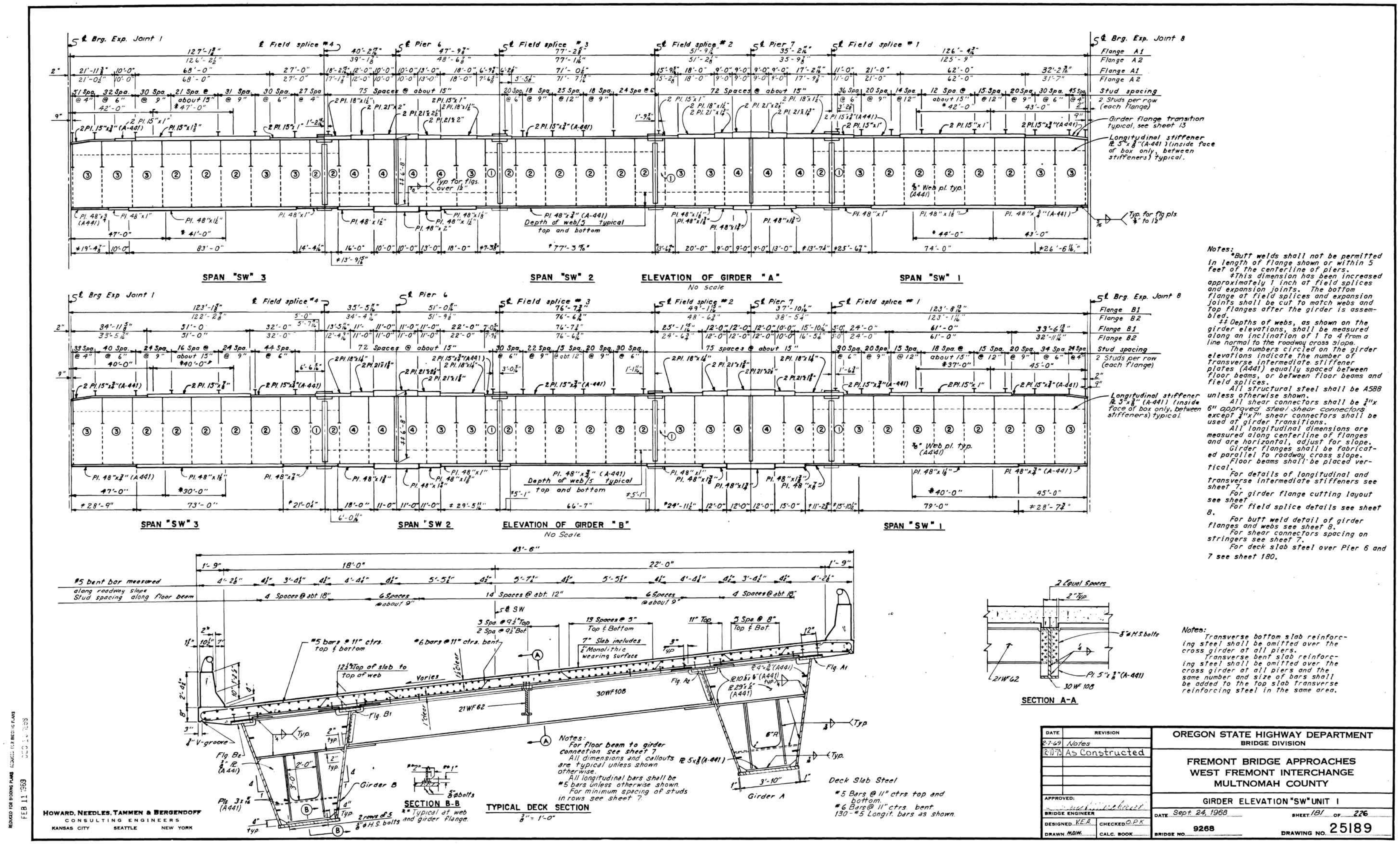
Table 5. Bridge Drawings

Oregon State Highway Department
Bridge Division

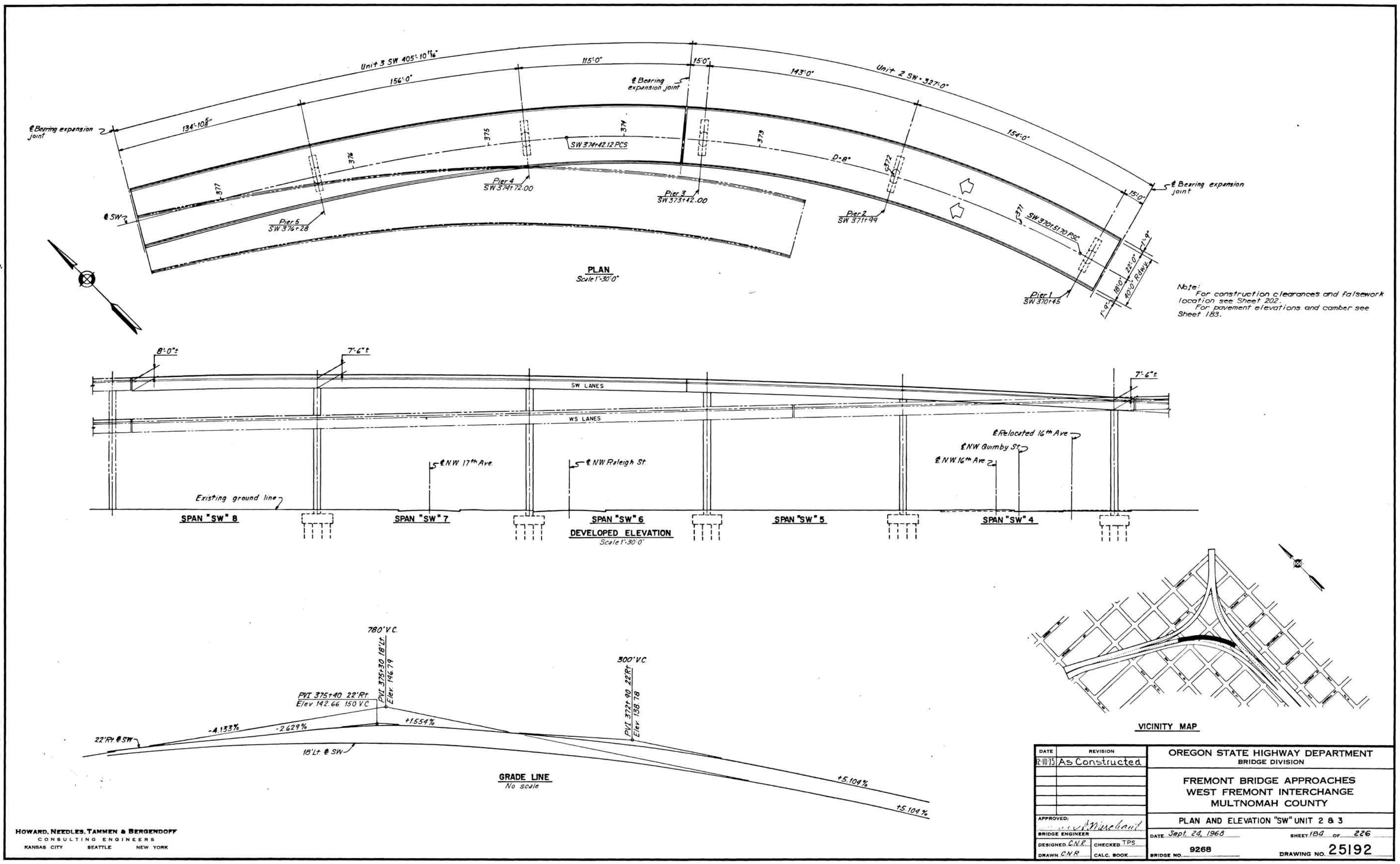
Fremont Bridge Approaches
West Fremont Interchange
Multnomah County

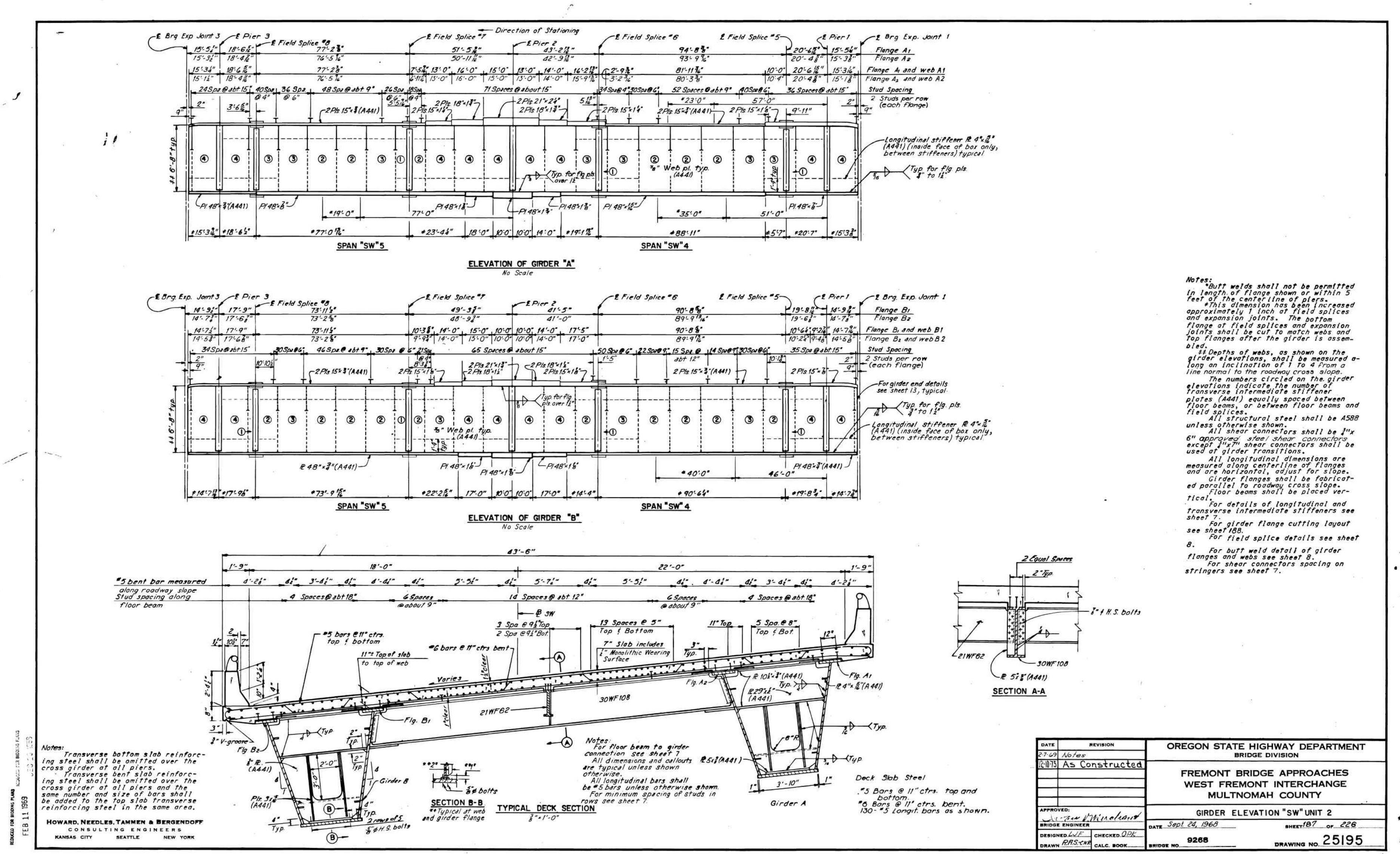
Drawing Title	Sheet No.	Drawing No.	Page No.
Plan and Elevation "SW" Unit 1	178	25186	43
Girder Elevation "SW" Unit 1	181	25189	44
Plan and Elevation "SW" Unit 2 & 3	184	25192	45
Girder Elevation "SW" Unit 2	187	25195	46
Girder elevation "SW" Unit 3	192	25200	47
Plan and Elevation "WS" Unit 1 & 2	163	25171	48
Girder Elevation "WS" Unit 1	166	25174	49
Girder Elevation"WS" Unit 2	171	25179	50
Plan and Elevation "WS" Unit 3	173	25181	51
Girder Elevation "WS" Unit 3	176	25184	52
Plan and Elevation "WE" Unit 1	111	25119	53
Girder Elevation "WE" Unit 1	115	25123	54
Plan and Elevation "EW" Unit 1	141	25149	55
Girder Elevation "EW" Unit 1	144	25152	56

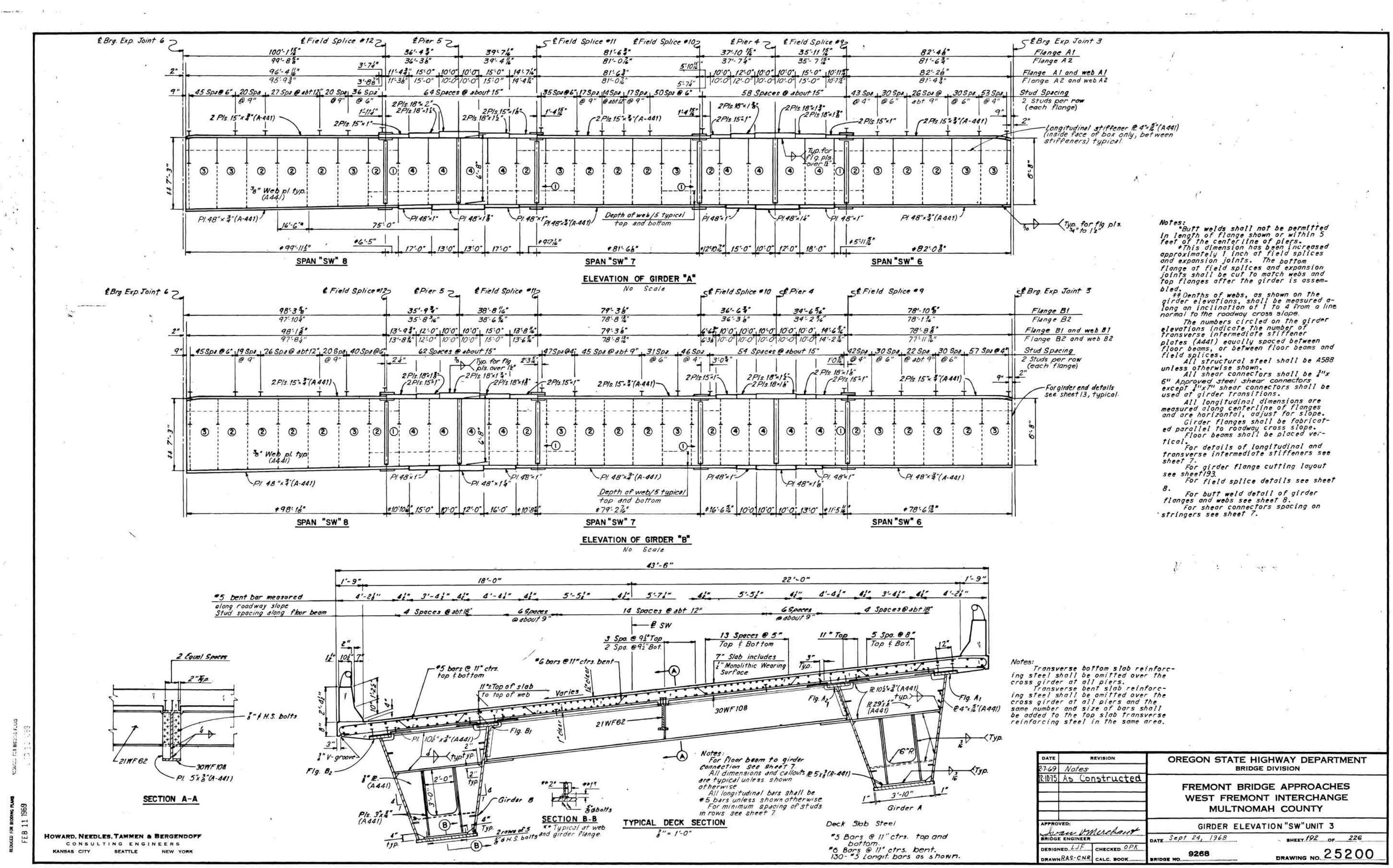

PLAN
Scale 1"=30'0"
DEVELOPED ELEVATION
Scale 1"=30'0"
GRADE LINE
No scale
VICINITY MAP
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
PLAN AND ELEVATION "SW" UNIT 1
DATE Sept. 24, 1968
SHEET 178 OF 226
BRIDGE NO. 9268
DRAWING NO. 25186

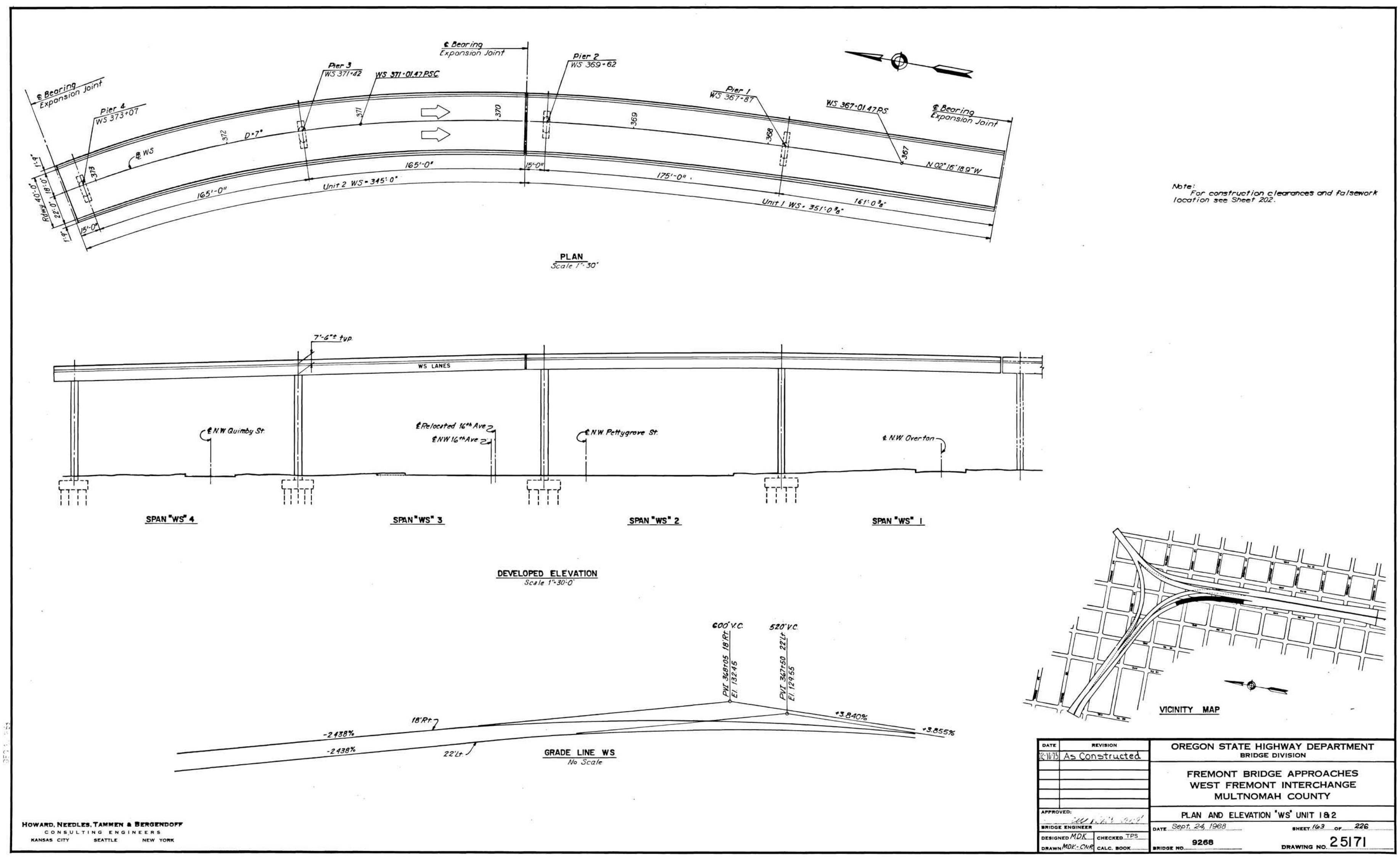


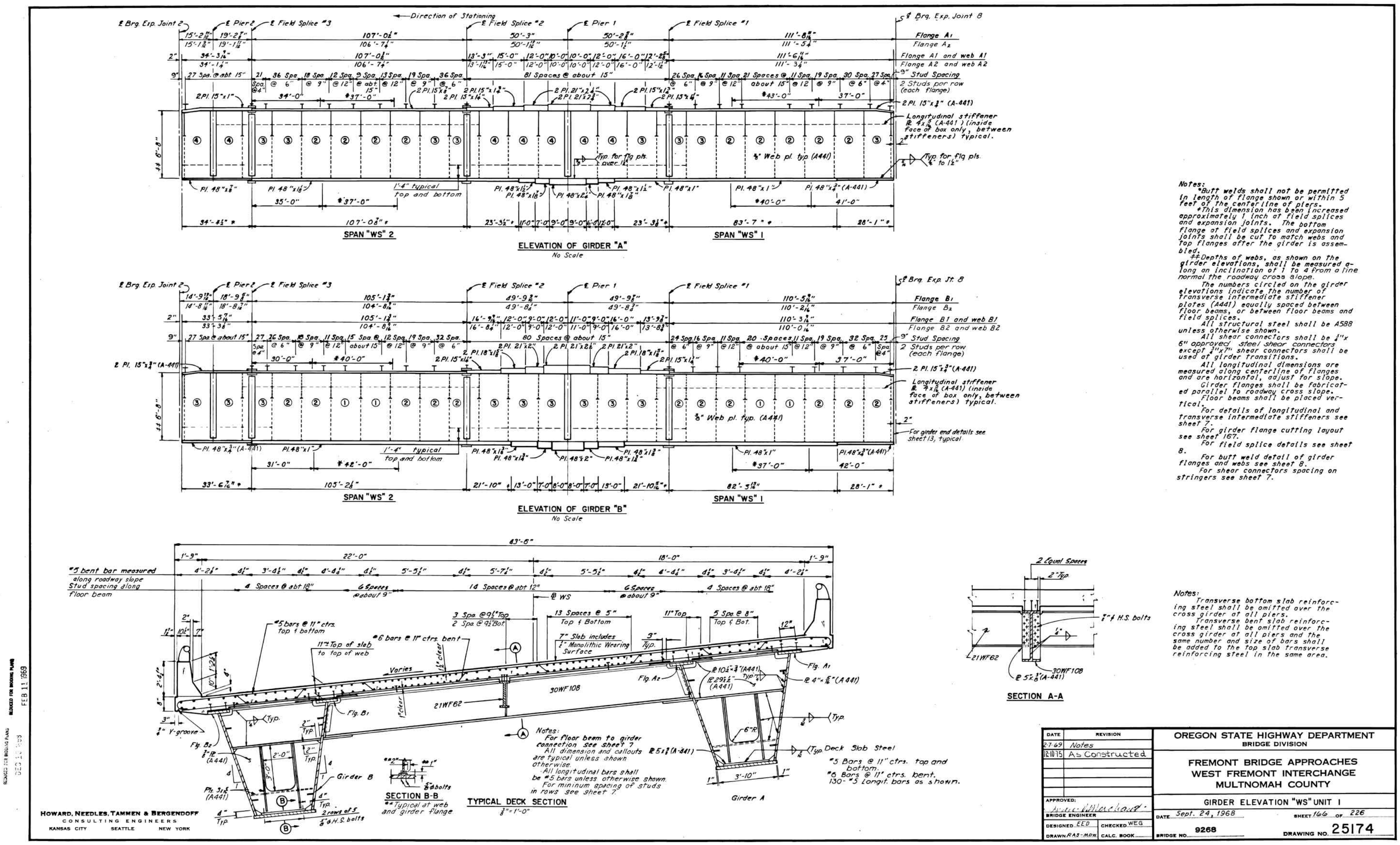
SPAN "SW" 3
SPAN "SW" 2
ELEVATION OF GIRDER "A"
No Scale
SPAN "SW" 1
SPAN "SW" 3
SPAN "SW" 2
ELEVATION OF GIRDER "B"
No Scale
SPAN "SW" 1
TYPICAL DECK SECTION
SECTION B-B
SECTION A-A
Girder B
Girder A
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION "SW" UNIT 1
As Constructed
DATE Sept 24, 1968
SHEET 181 OF 226
BRIDGE NO. 9268
DRAWING NO. 25189

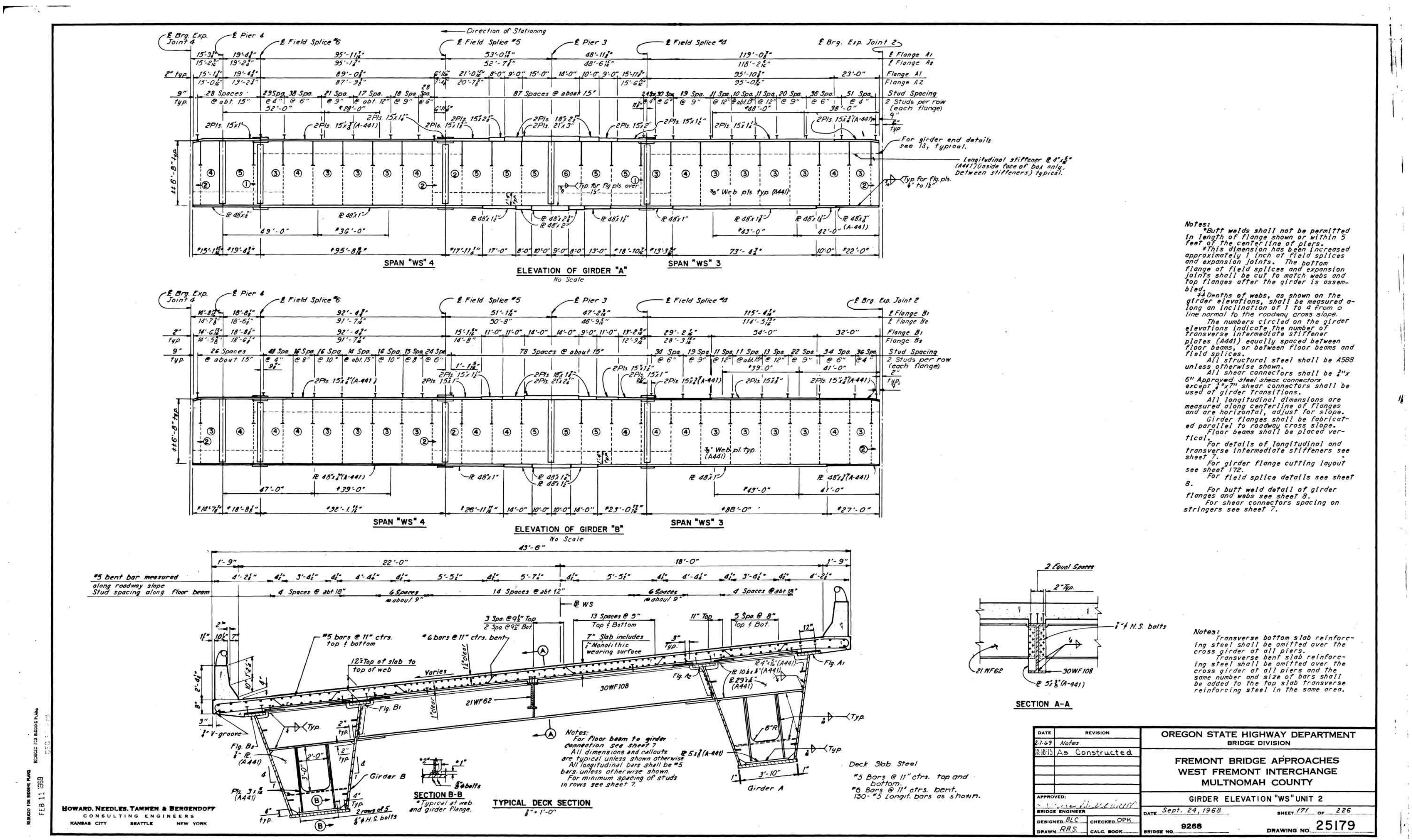

PLAN
Scale 1"=30'0"
DEVELOPED ELEVATION
Scale 1"=30'0"
GRADE LINE
No scale
VICINITY MAP
SPAN "SW" 8
SPAN "SW" 7
SPAN "SW" 6
SPAN "SW" 5
SPAN "SW" 4
SW LANES
WS LANES
Existing ground line
Note:
For construction clearances and falsework location see Sheet 202.
For pavement elevations and camber see Sheet 183.
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
PLAN AND ELEVATION "SW" UNIT 2 & 3
As Constructed
DATE Sept. 24, 1968
SHEET 184 OF 226
BRIDGE NO. 9268
DRAWING NO. 25192

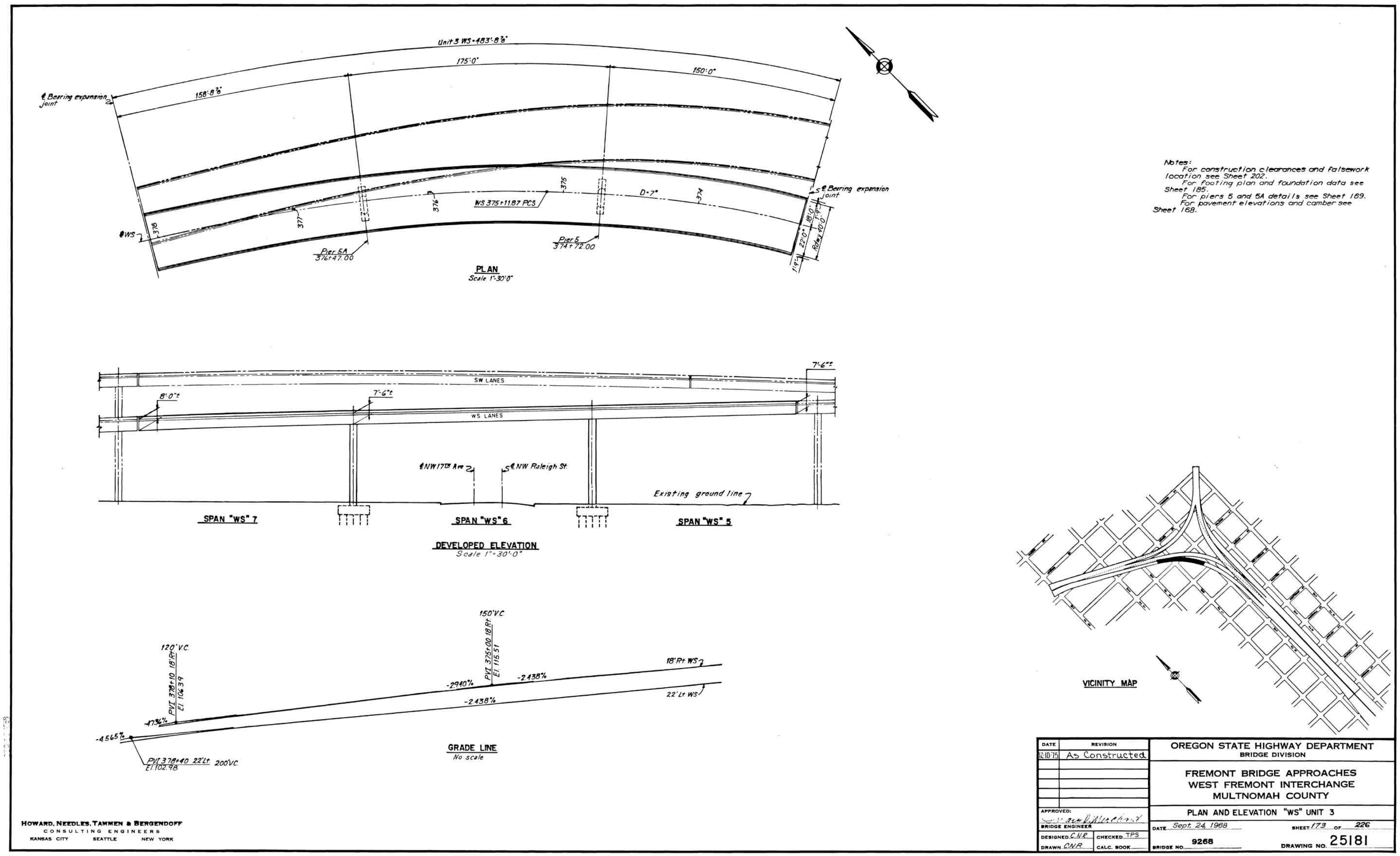

ELEVATION OF GIRDER "A"
No Scale
SPAN "SW" 5
SPAN "SW" 4
ELEVATION OF GIRDER "B"
No Scale
TYPICAL DECK SECTION
SECTION A-A
SECTION B-B
Notes:
*Butt welds shall not be permitted in length of flange shown or within 5 feet of the centerline of piers.
*This dimension has been increased approximately 1 inch at field splices and expansion joints. The bottom flange at field splices and expansion joints shall be cut to match webs and top flanges after the girder is assembled.
‡‡ Depths of webs, as shown on the girder elevations, shall be measured along an inclination of 1 to 4 from a line normal to the roadway cross slope.
The numbers circled on the girder elevations indicate the number of transverse intermediate stiffener plates (A441) equally spaced between floor beams, or between floor beams and field splices.
All structural steel shall be A588 unless otherwise shown.
All shear connectors shall be ¾"x 6" approved steel shear connectors except ¾"x7" shear connectors shall be used at girder transitions.
All longitudinal dimensions are measured along centerline of flanges and are horizontal, adjust for slope.
Girder flanges shall be fabricated parallel to roadway cross slope.
Floor beams shall be placed vertical.
For details of longitudinal and transverse intermediate stiffeners see sheet 7.
For girder flange cutting layout see sheet 188.
For field splice details see sheet 8.
For butt weld detail of girder flanges and webs see sheet 8.
For shear connectors spacing on stringers see sheet 7.
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION "SW" UNIT 2
DATE Sept. 24, 1968
SHEET 187 OF 226
BRIDGE NO. 9268
DRAWING NO. 25195
2-7-69 Notes
12-10-75 As Constructed
APPROVED: BRIDGE ENGINEER
DESIGNED LJF
CHECKED OPK
DRAWN RAS-CNR
CALC. BOOK

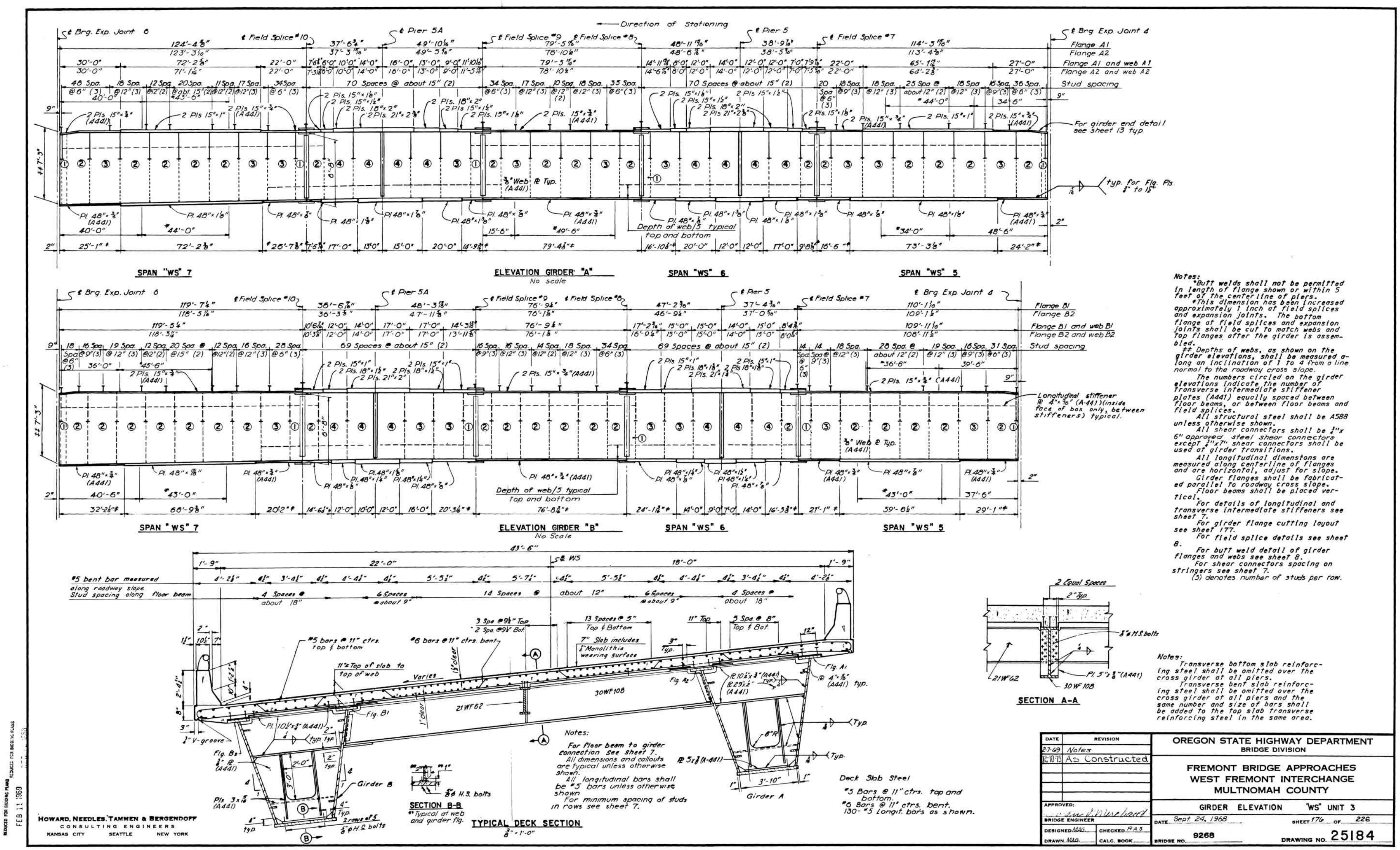

ELEVATION OF GIRDER "A"
No Scale
SPAN "SW" 8
SPAN "SW" 7
SPAN "SW" 6
ELEVATION OF GIRDER "B"
No Scale
SECTION A-A
SECTION B-B
TYPICAL DECK SECTION
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION "SW" UNIT 3
DATE Sept 24, 1968
SHEET 192 OF 226
BRIDGE NO. 9268
DRAWING NO. 25200
As Constructed
FEB 11 1969

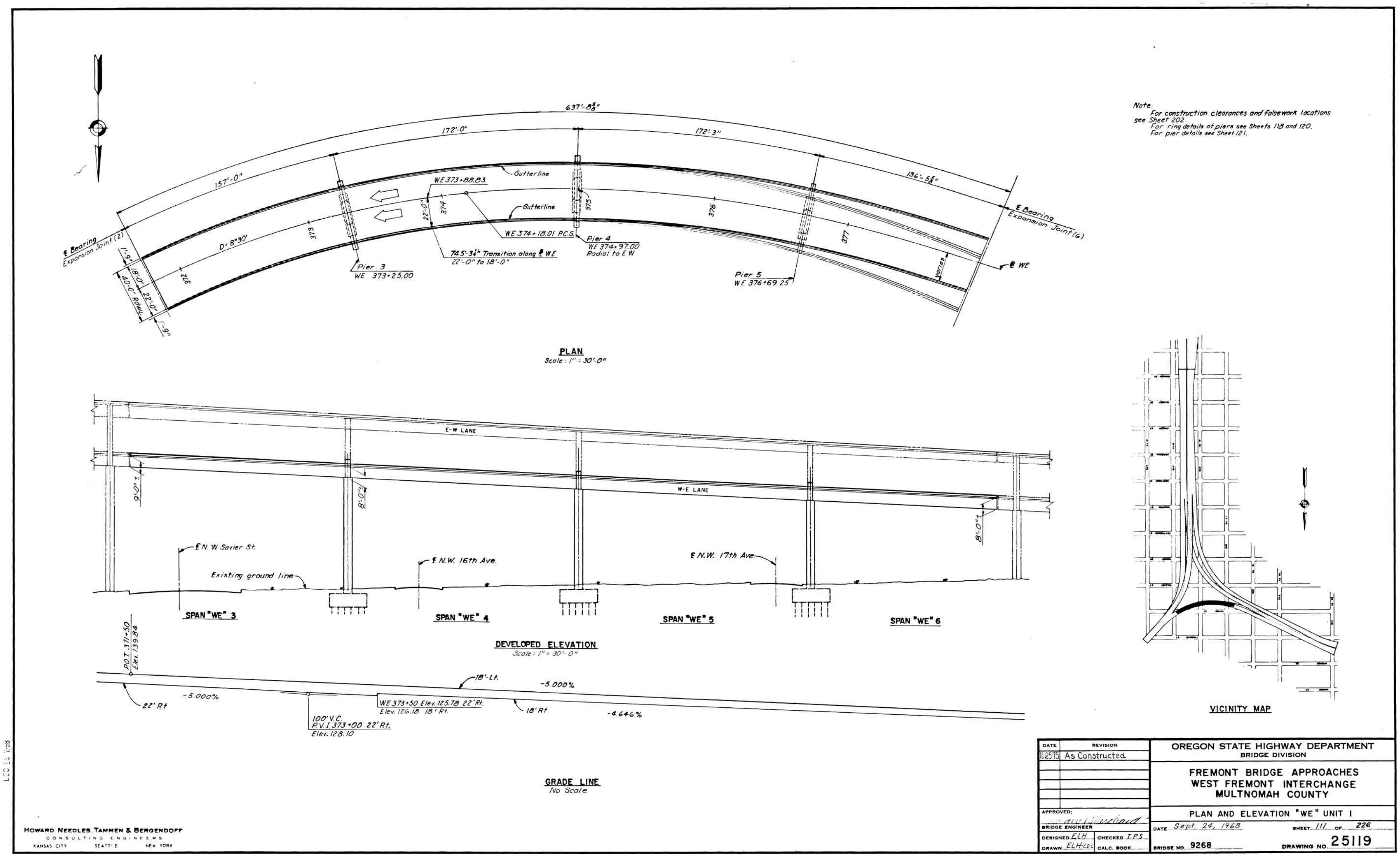

℄ Bearing Expansion Joint
Pier 4 WS 373+07
Pier 3 WS 371+42
WS 371+01.47 PSC
Pier 2 WS 369+62
Pier 1 WS 367+87
WS 367+01.47 PS
D=7°
165'-0"
Unit 2 WS=345'-0"
175'-0"
Unit 1 WS=351'-0⅜"
161'-0⅜"
N 02°16'18.9" W
PLAN
Scale 1"=30'
Note: For construction clearances and falsework location see Sheet 202.
7'-6"± typ.
WS LANES
℄ N.W. Quimby St.
℄ Relocated 16th Ave
℄ NW 16th Ave
℄ N.W. Pettygrove St.
℄ N.W. Overton
SPAN "WS" 4
SPAN "WS" 3
SPAN "WS" 2
SPAN "WS" 1
DEVELOPED ELEVATION
Scale 1"=30'-0
600' V.C.
520' V.C.
PVI 368+05 18'Rt El. 132.45
PVI 367+50 22'Lt El. 129.55
-2.438%
+3.840%
+3.855%
18'Rt
22'Lt
GRADE LINE WS
No Scale
VICINITY MAP
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
As Constructed
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
PLAN AND ELEVATION "WS" UNIT 1 & 2
DATE Sept. 24, 1968
SHEET 163 OF 226
DESIGNED MDK CHECKED TPS
DRAWN MDK-CNK
BRIDGE NO. 9268
DRAWING NO. 25171

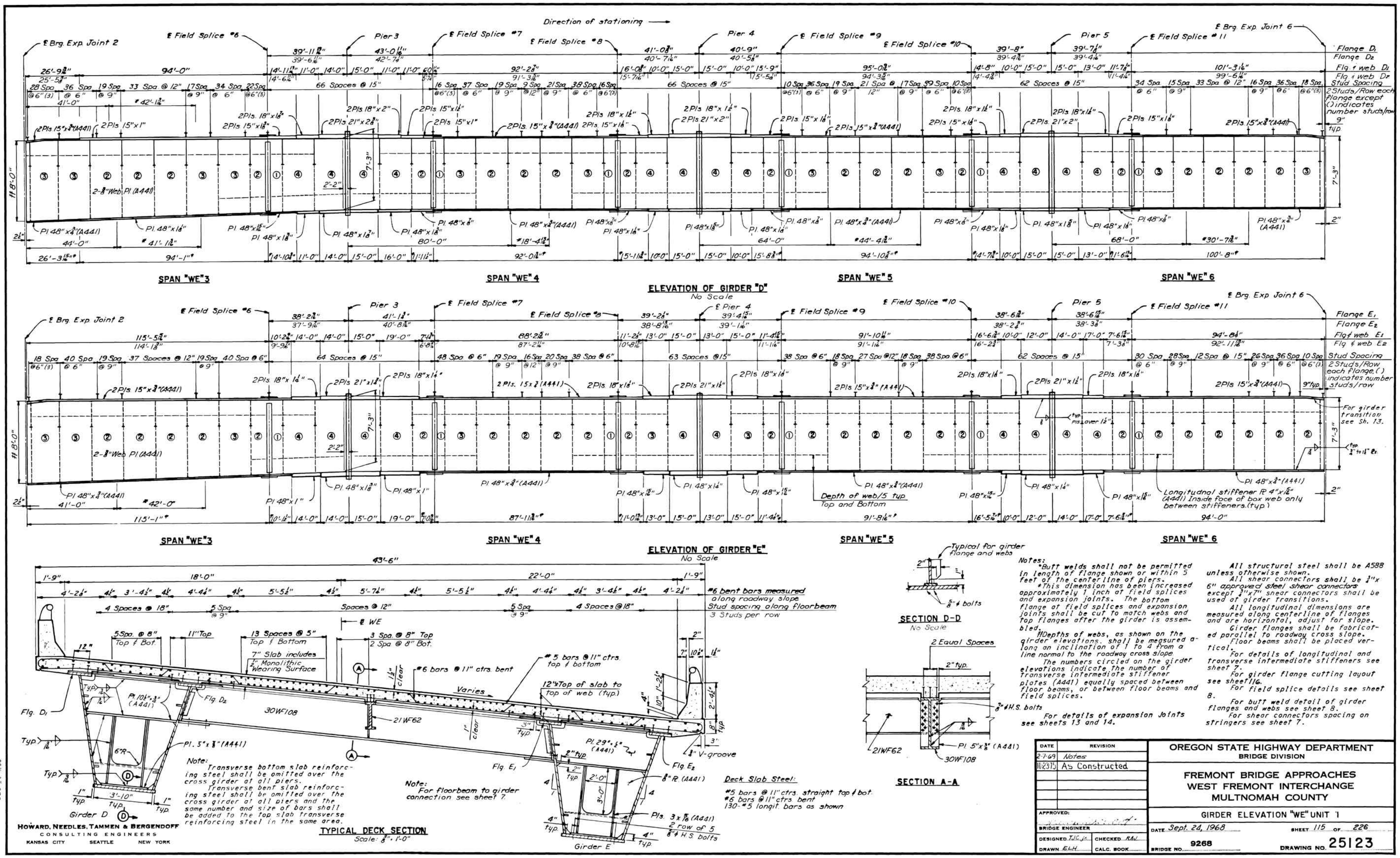

ELEVATION OF GIRDER "A"
No Scale
SPAN "WS" 2
SPAN "WS" 1
ELEVATION OF GIRDER "B"
No Scale
SECTION A-A
SECTION B-B
TYPICAL DECK SECTION
Notes:
*Butt welds shall not be permitted in length of flange shown or within 5 feet of the centerline of piers.
*This dimension has been increased approximately 1 inch at field splices and expansion joints. The bottom flange at field splices and expansion joints shall be cut to match webs and top flanges after the girder is assembled.
##Depths of webs, as shown on the girder elevations, shall be measured along an inclination of 1 to 4 from a line normal the roadway cross slope.
The numbers circled on the girder elevations indicate the number of transverse intermediate stiffener plates (A441) equally spaced between floor beams, or between floor beams and field splices.
All structural steel shall be A588 unless otherwise shown.
All shear connectors shall be ¾"x 6" approved steel shear connectors except ¾"x7" shear connectors shall be used at girder transitions.
All longitudinal dimensions are measured along centerline of flanges and are horizontal, adjust for slope.
Girder flanges shall be fabricated parallel to roadway cross slope.
Floor beams shall be placed vertical.
For details of longitudinal and transverse intermediate stiffeners see sheet 7.
For girder flange cutting layout see sheet 167.
For field splice details see sheet 8.
For butt weld detail of girder flanges and webs see sheet 8.
For shear connectors spacing on stringers see sheet 7.
Notes:
Transverse bottom slab reinforcing steel shall be omitted over the cross girder at all piers.
Transverse bent slab reinforcing steel shall be omitted over the cross girder at all piers and the same number and size of bars shall be added to the top slab transverse reinforcing steel in the same area.
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
DATE REVISION
2-7-69 Notes
12-10-75 As Constructed
APPROVED:
BRIDGE ENGINEER
DESIGNED EED CHECKED WEG
DRAWN RAS-MDW CALC. BOOK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION "WS" UNIT 1
DATE Sept. 24, 1968
SHEET 166 OF 226
BRIDGE NO. 9268
DRAWING NO. 25174

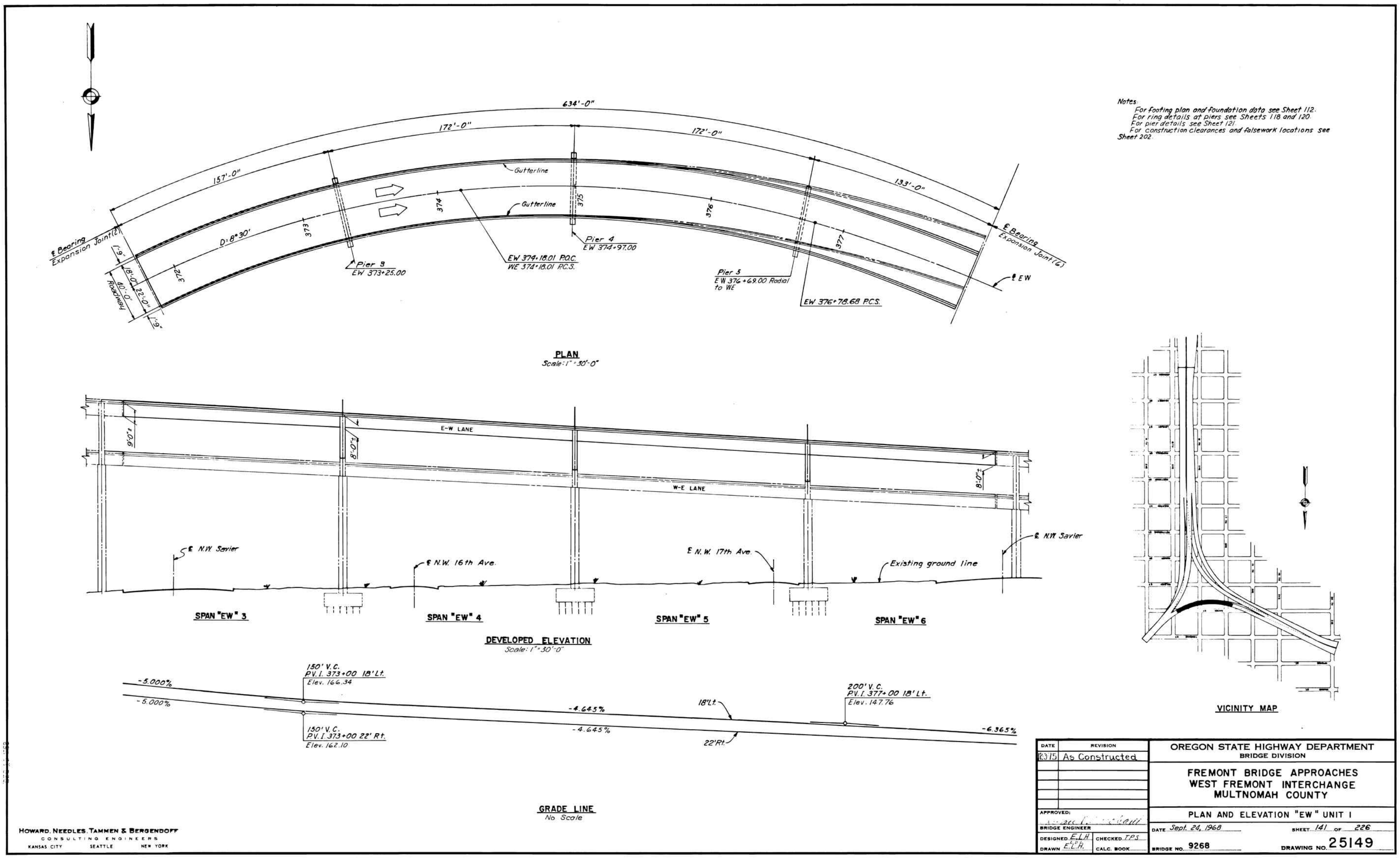

ELEVATION OF GIRDER "A"
No Scale
SPAN "WS" 4
SPAN "WS" 3
ELEVATION OF GIRDER "B"
No Scale
TYPICAL DECK SECTION
SECTION B-B
SECTION A-A
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION "WS" UNIT 2
Sept. 24, 1968
9268
25179

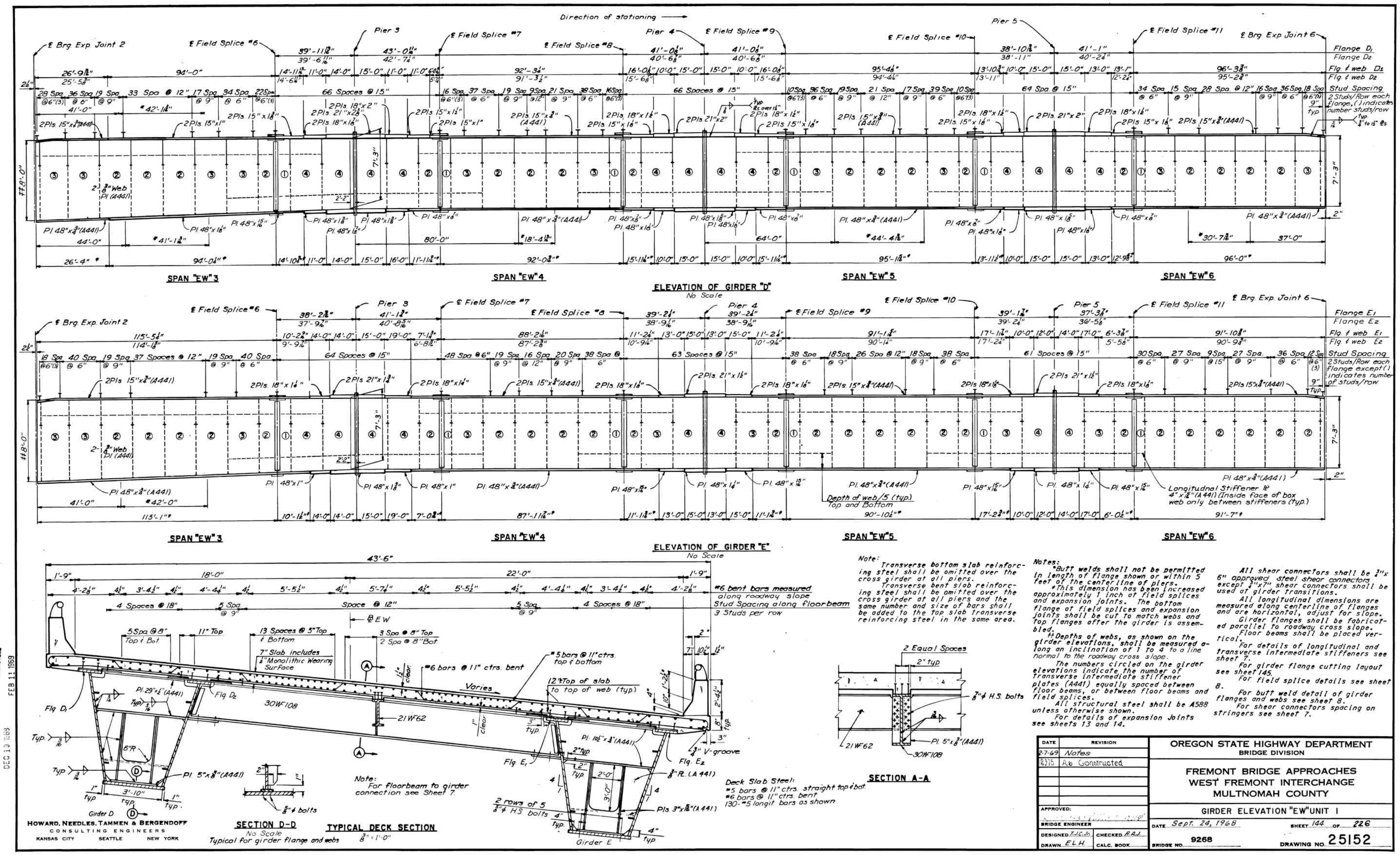

Unit 3 WS=483'-8 3/8"
PLAN
Scale 1"=30'0"
Notes:
For construction clearances and falsework location see Sheet 202.
For footing plan and foundation data see Sheet 185.
For piers 5 and 5A details see Sheet 169.
For pavement elevations and camber see Sheet 168.
SW LANES
WS LANES
Existing ground line
SPAN "WS" 7
SPAN "WS" 6
SPAN "WS" 5
DEVELOPED ELEVATION
Scale 1"=30'-0"
GRADE LINE
No scale
VICINITY MAP
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
As Constructed
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
PLAN AND ELEVATION "WS" UNIT 3
DATE Sept. 24, 1968
SHEET 173 OF 226
BRIDGE NO. 9268
DRAWING NO. 25181


SPAN "WS" 7
ELEVATION GIRDER "A"
No scale
SPAN "WS" 6
SPAN "WS" 5
ELEVATION GIRDER "B"
No Scale
SECTION A-A
SECTION B-B
TYPICAL DECK SECTION
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION 'WS' UNIT 3
DATE Sept 24, 1968
SHEET 176 OF 226
BRIDGE NO. 9268
DRAWING NO. 25184
As Constructed


Note:
For construction clearances and falsework locations see Sheet 202.
For ring details at piers see Sheets 118 and 120.
For pier details see Sheet 121.
637'-8⅝"
172'-0"
172'-3"
157'-0"
136'-5⅝"
Gutterline
Pier 3
WE 373+25.00
Pier 4
WE 374+97.00
Radial to E W
Pier 5
WE 376+69.25
WE 374+18.01 P.C.S.
WE 373+88.83
D=8°30'
℄ Bearing Expansion Joint (2)
℄ Bearing Expansion Joint (6)
℄ WE
PLAN
Scale: 1" = 30'-0"
E-W LANE
W-E LANE
℄ N.W. Savier St.
℄ N.W. 16th Ave.
℄ N.W. 17th Ave.
Existing ground line
SPAN "WE" 3
SPAN "WE" 4
SPAN "WE" 5
SPAN "WE" 6
DEVELOPED ELEVATION
Scale: 1" = 30'-0"
-5.000%
-4.646%
100' V.C.
P.V.I. 373+00 22' Rt.
Elev. 128.10
WE 373+50 Elev. 125.78 22' Rt.
Elev. 126.18 18' Rt.
GRADE LINE
No Scale
VICINITY MAP
DATE
REVISION
10-25-75 As Constructed
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
PLAN AND ELEVATION "WE" UNIT 1
APPROVED:
BRIDGE ENGINEER
DESIGNED ELH
CHECKED TPS
DRAWN ELH-LDL
CALC. BOOK
DATE Sept. 24, 1968
SHEET 111 OF 226
BRIDGE NO. 9268
DRAWING NO. 25119
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK


ELEVATION OF GIRDER "D"
No Scale
SPAN "WE" 3
SPAN "WE" 4
SPAN "WE" 5
SPAN "WE" 6
ELEVATION OF GIRDER "E"
No Scale
TYPICAL DECK SECTION
SECTION D-D
SECTION A-A
Notes:
*Butt welds shall not be permitted in length of flange shown or within 5 feet of the centerline of piers.
*This dimension has been increased approximately 1 inch at field splices and expansion joints. The bottom flange at field splices and expansion joints shall be cut to match webs and top flanges after the girder is assembled.
Depths of webs, as shown on the girder elevations, shall be measured along an inclination of 1 to 4 from a line normal to the roadway cross slope.
The numbers circled on the girder elevations indicate the number of transverse intermediate stiffener plates (A441) equally spaced between floor beams, or between floor beams and field splices.
For details of expansion Joints see sheets 13 and 14.
All structural steel shall be A588 unless otherwise shown.
All shear connectors shall be ¾"x 6" approved steel shear connectors except ¾"x7" shear connectors shall be used at girder transitions.
All longitudinal dimensions are measured along centerline of flanges and are horizontal, adjust for slope.
Girder flanges shall be fabricated parallel to roadway cross slope.
Floor beams shall be placed vertical.
For details of longitudinal and transverse intermediate stiffeners see sheet 7.
For girder flange cutting layout see sheet 116.
For field splice details see sheet 8.
For butt weld detail of girder flanges and webs see sheet 8.
For shear connectors spacing on stringers see sheet 7.
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
DATE REVISION
2-7-69 Notes
10-23-75 As Constructed
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION "WE" UNIT 1
DATE Sept. 24, 1968
SHEET 115 OF 226
BRIDGE NO. 9268
DRAWING NO. 25123


Notes:
For footing plan and foundation data see Sheet 112.
For ring details at piers see Sheets 118 and 120.
For pier details see Sheet 121.
For construction clearances and falsework locations see Sheet 202.
634'-0"
172'-0"
172'-0"
157'-0"
133'-0"
Pier 3
EW 373+25.00
Pier 4
EW 374+97.00
Pier 5
EW 376+78.68 P.C.S.
EW 374+18.01 P.O.C.
WE 374+18.01 P.C.S.
PLAN
Scale: 1" = 30'-0"
E-W LANE
W-E LANE
Existing ground line
SPAN "EW" 3
SPAN "EW" 4
SPAN "EW" 5
SPAN "EW" 6
DEVELOPED ELEVATION
Scale: 1" = 30'-0"
GRADE LINE
No Scale
VICINITY MAP
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
As Constructed
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
PLAN AND ELEVATION "EW" UNIT 1
DATE Sept. 24, 1968
SHEET 141 OF 226
BRIDGE NO. 9268
DRAWING NO. 25149


ELEVATION OF GIRDER "D"
No Scale
SPAN "EW" 3
SPAN "EW" 4
SPAN "EW" 5
SPAN "EW" 6
ELEVATION OF GIRDER "E"
No Scale
TYPICAL DECK SECTION
SECTION D-D
No Scale
Typical for girder flange and webs
SECTION A-A
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY SEATTLE NEW YORK
OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
FREMONT BRIDGE APPROACHES
WEST FREMONT INTERCHANGE
MULTNOMAH COUNTY
GIRDER ELEVATION "EW" UNIT I
Sept. 24, 1968
144 OF 226
9268
DRAWING NO. 25152

TASK 2
STRAIN GAGING

Scope of Work

Task 2 of WOC 5 was to install electric resistance strain gages at Agency-designated locations inside the bridge girders and measure and record the service strains for a period of seven days.

The designated bridges, listed in Table 6, each have two girders. The scope of work required the installation of strain gages on both girders of each bridge, at two cross sections per bridge–one near a pier and the other at midspan.

Figures 7 through 10 show the gage locations on ODOT elevation drawings of the spans.

Table 6. Strain Gage Locations

Bridge	Line	Unit	Span	Cross Section Location		Girders & Lane
				Midspan*	Pier No.	
9268A	SW	3	7	79 feet	4	A Slow, B Fast
9268B	WS	3	6	87 feet	5	A Fast, B Slow
9268E	WE	1	5	87 feet	5	D Slow, E Fast
9268W	EW	1	5	87 feet	5	D Fast, E Slow

*approximate distance from pier

Figure 7. Plan and Elevation "SW" Units 2 & 3 (ODOT Drawing 25192)

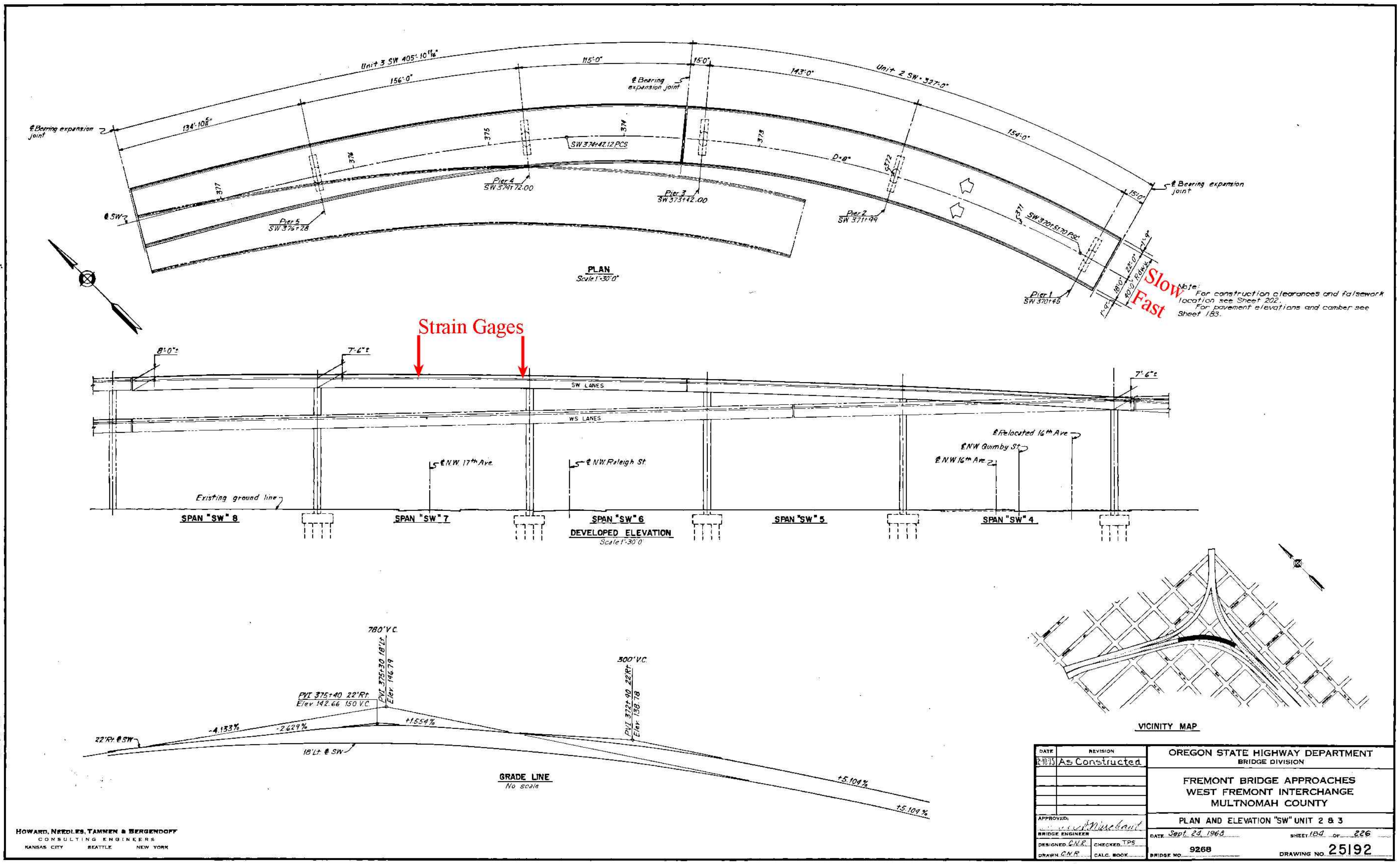


Figure 8. Plan and Elevation "WS" Unit 3 (ODOT Drawing 25181)

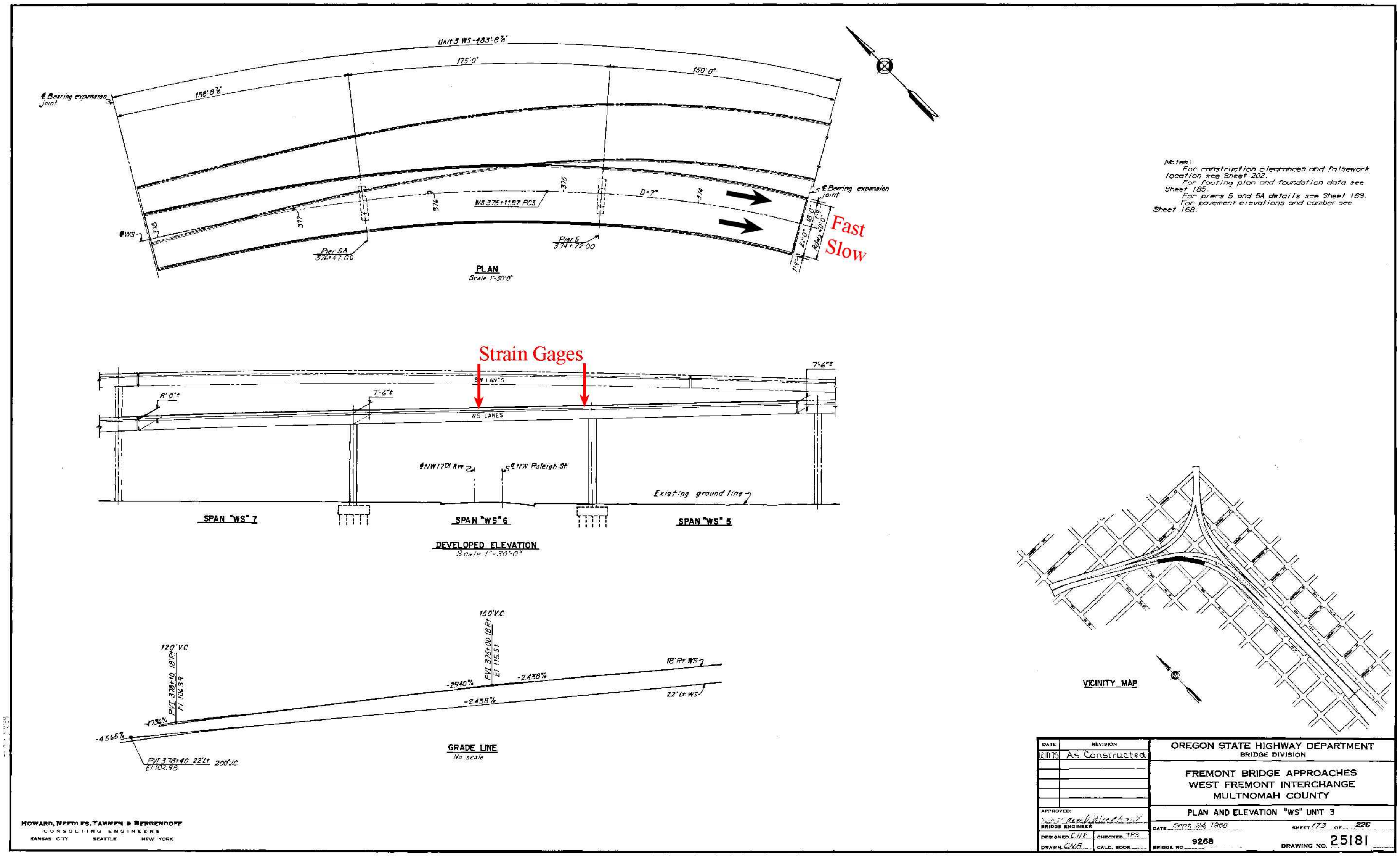


Figure 9. Plan and Elevation "WE" Unit 1(ODOT Drawing 25119)

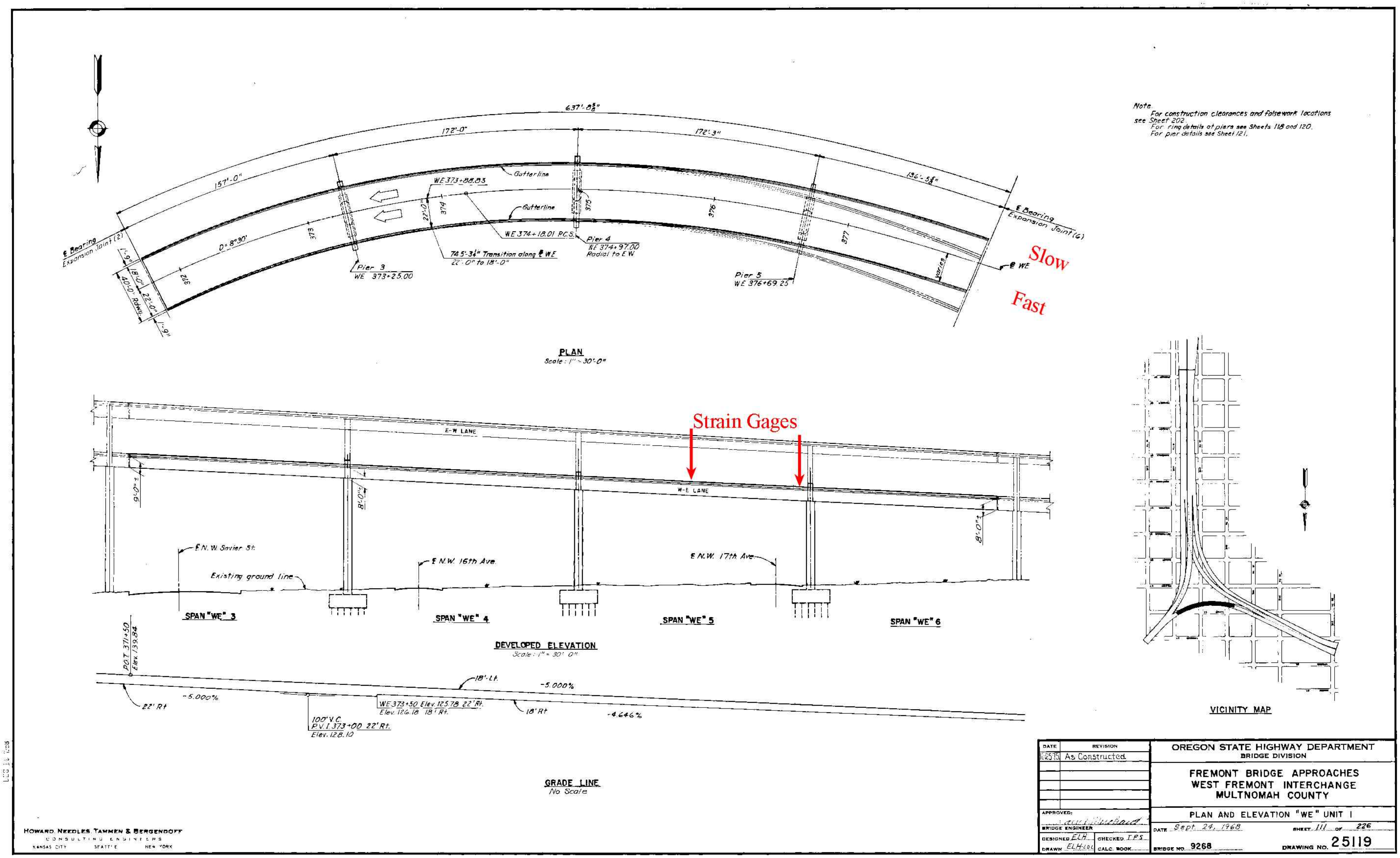
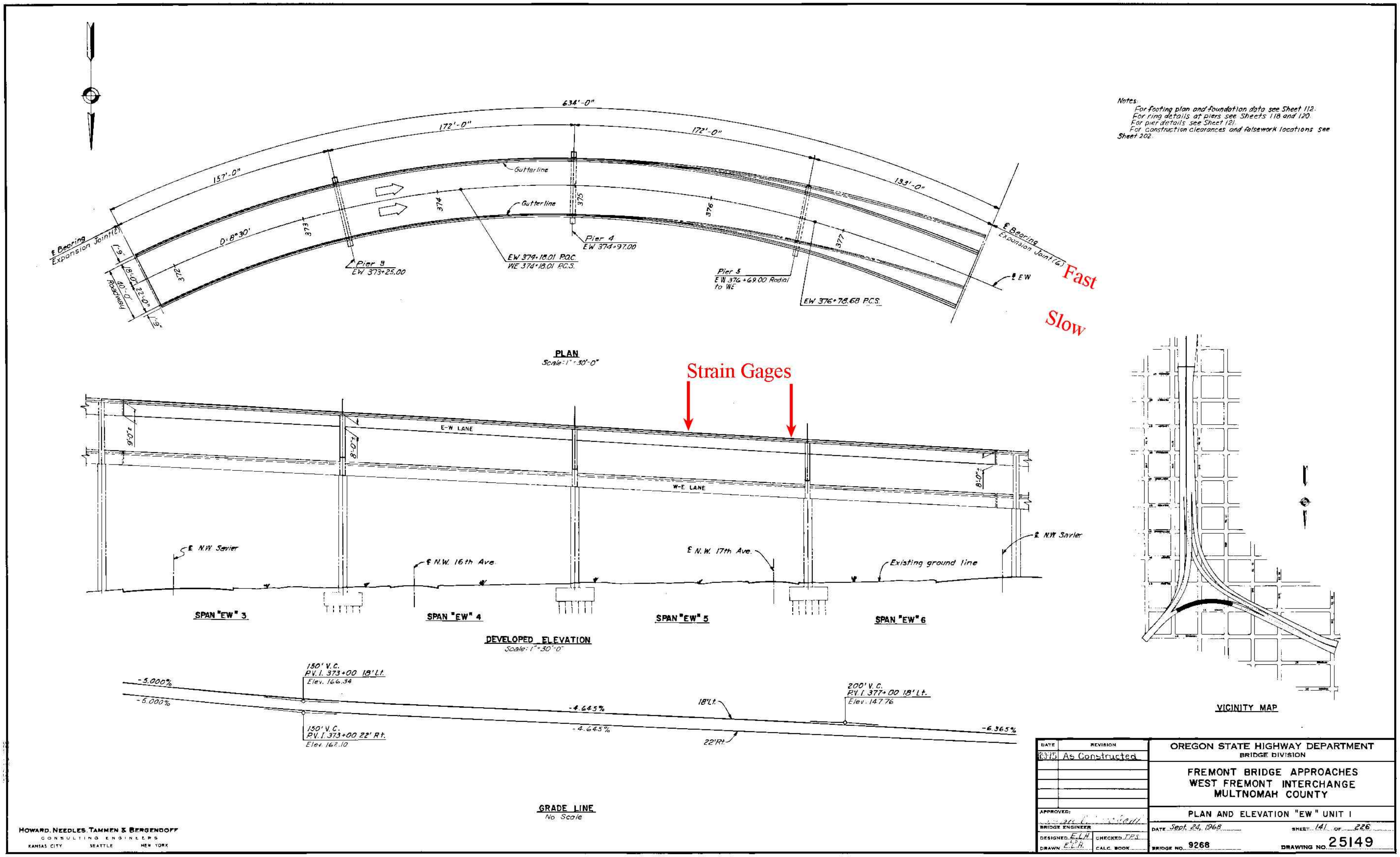


Figure 10. Plan and Elevation "EW" Unit 1 (ODOT Drawing 25149)



Strain Gage Mounting Locations

Figure 11 shows the gage mounting locations within the girders. The scope of work specified the installation of four single-element, uniaxial strain gages in each girder at each cross section, with one gage on each of the two top flanges and one gage on each side of the bottom flange. The gages were installed 3 inches from the web plates at each location.

The gages at piers were mounted 2 feet beyond the cross stiffener bulkhead at the pier (i.e., toward midspan), or approximately 4 feet from the centerline of the pier. Per Agency instructions, the gages were mounted at least 2 feet from any bolted splice plates.

The gages used on this assignment were Measurements Group LWK-06-W250B-350 weld-on, single-element, uniaxial strain gages. The gages had integral lead wires, which were attached to terminal blocks fixed to the girder near the gage. A coating of Measurements Group M-Coat F was applied to the gages to provide environmental protection. Figure 12 shows a typical gage, as-mounted on a top flange (prior to connecting and securing the integral lead wires).

The data acquisition system was a 16-channel, 16-bit, Somat® eDaq™ data acquisition system. Data were recorded at a sampling rate of 100 Hz using a 15-Hz eight-pole Butterworth low pass filter. Strain data from normal, random bridge traffic were recorded for a period of seven days on each bridge, with all 16 gages per bridge read simultaneously. Power for the data acquisition system was provided by four, deep-cycle storage batteries.

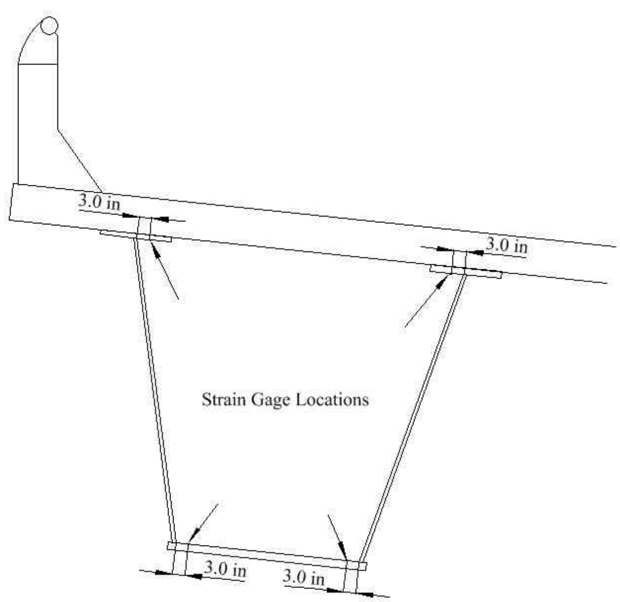


Figure 11. Strain Gage Mounting Locations

Figure 12 (71148-02) Strain Gage on Top Flange

Strain Gage Identification

The strain gages were numbered 1 through 16 on each bridge, with Gages 1 through 8 next to the piers and Gages 9 through 16 at midspan. The same numbering sequence was used for each bridge section, with each gage numbered according to its location on the span. Figure 13 shows cross-sectional views of each bridge, with the girders identified per the ODOT drawings and the gages numbered per our numbering sequence.

The work order specified that data be collected for 7 days on each bridge. Although the data acquisition system we were using had a capacity of 64 channels, which would have allowed us to collect data from all four bridges simultaneously, the practical aspects of connecting the wires from gages on four different bridges to the system all at once made it more practical to collect data from each bridge separately.

The actual data collection times were slightly less than a full 7 days because of the logistics of accessing the bridges. That is, the bridges were only available from 8:00 pm until 5:00 am each night and required partial lane closures (which were done by a traffic control subcontractor). To control costs, each time we moved the data acquisition equipment from one bridge to the next, we did so in a single night, rather than remove the equipment from one bridge one night and install it on the next bridge the following night.

Thus, we would start work disconnecting the equipment at around 8:00 pm. After it was disconnected, traffic control would shift to the next bridge and we would start connecting the equipment to the gages on that bridge, typically finishing around midnight or 1:00 am. This process was then repeated the following week, resulting in a four to five hour time span each week (between disconnecting the equipment on one bridge and reconnecting it on the next bridge), during which no data were taken.

The starting, ending, and elapsed times and dates for the data collection on each bridge are shown in Table 7.

Table 7. Strain Gage Data Collection Dates/Times

Bridge	Line	Start		Stop		Elapsed Time
9268A	SW	5/26/2006	1:00 AM	6/1/2006	8:25 PM	6 d, 19 h, 25 m
9268B	WS	5/19/2006	1:55 AM	5/25/2006	9:00 PM	6 d, 19 h, 5 m
9268E	WE	5/12/2006	12:15 AM	5/18/2006	9:25 PM	6 d, 21 h, 10 m
9268W	EW	5/1/2006	10:40 PM	5/8/2006	8:55 PM	6 d, 22 h, 15 m

TO: Oregon Departmatment of Transportation
SUBJECT: FREMONT BRIDGE; FINAL REPORT
REF. NO.: 7112003

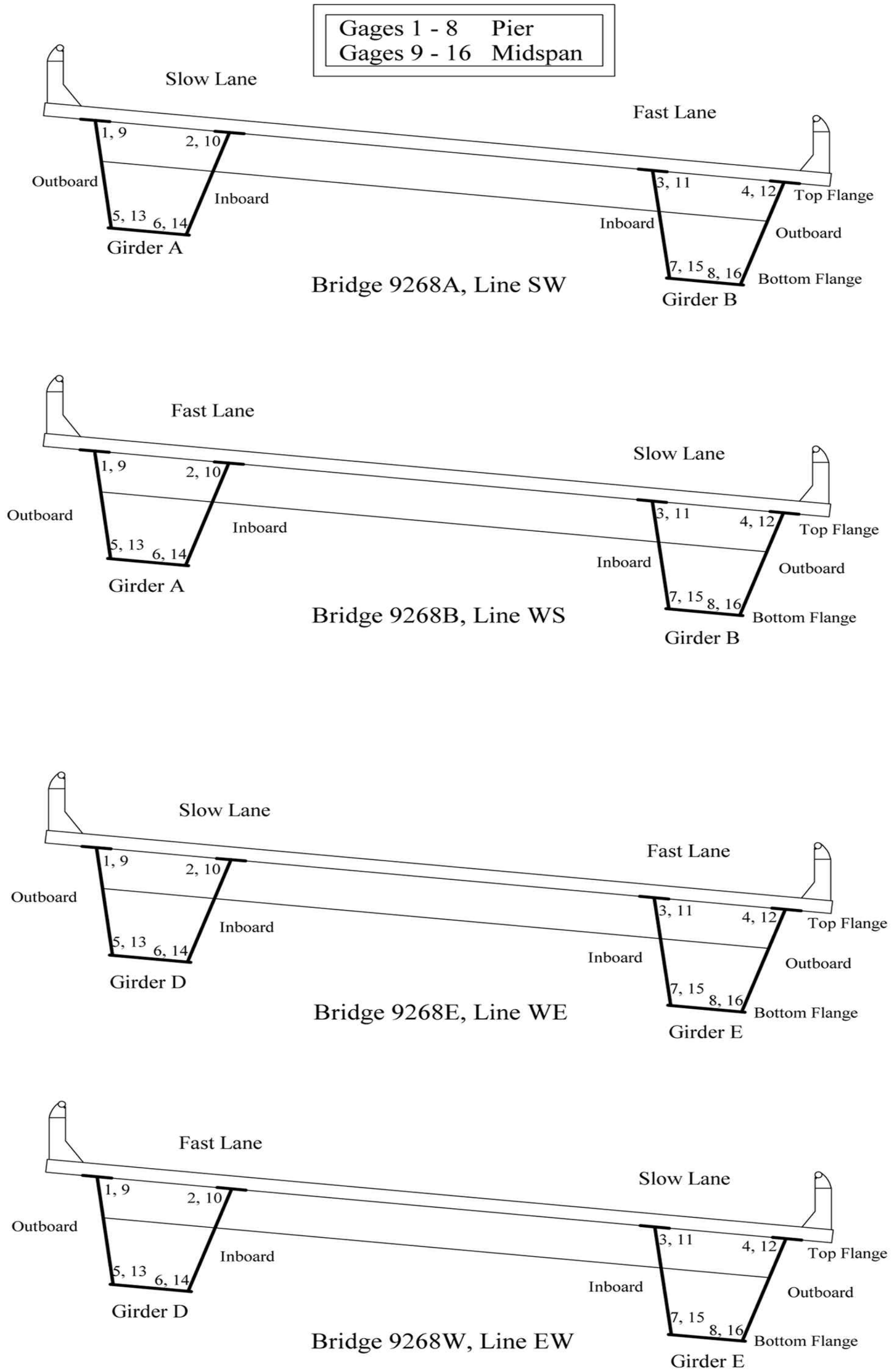


Figure 13. Strain Gage Identification

Strain Gage Data Presentation

The data from the strain gages are presented in this report in three formats: the first being a continuous recording of strain ranges in rain-flow format, the second being a root-mean-cube compilation of effective strain range, and the third being a recording of strain-time histories of the ten largest strain cycles for each gage.

Rainflow Diagram

Figure 14 shows a typical plot of rain-flow data, presented on a semi-log scale so as to accommodate the large difference in counts between the lower and higher strain ranges.The rain-flow data were collected in 200 evenly divided bins, each of which was 1 microstrain wide, using a 2-microstrain peak-valley hysteresis value. As shown in Figure 14, the rain-flow data exhibited a logarithmic decrease in the number of counts as a function of strain range, up to a strain range of about 40 microstrain. However, beyond this strain range, the data continued to exhibit single-strain events all the way to a strain range of about 97 microstrain.

Seven-Day Peak-Valley Strain History Diagram

Figure 15 shows the entire seven days of peak-valley strain data for the same gage shown in Figure 14 The peak-valley histories showed that in addition to the very short-term fluctuations in strain caused by bridge loading, the gages registered a much larger, gradual change in strain over a time period of several hours during the course of each day. A review of the a peak-valley history for the entire seven-day data collection period showed that these gradual strain events occurred on all gages for all bridges. These strains were not due to mechanical loading imposed by traffic, but rather, they were the result of temperature-induced strains imposed by daily, ambient temperature fluctuations. For example, the change in strain from Point A to Point B in Figure 15 is about 97 microstrain and corresponds with the single Rainflow event shown at a range of 97 microstrain in Figure 14.

Note that the X-axis in Figure 14 is number of events, rather than time. Thus, the spacing between major peaks varies, depending on traffic volume. In this case, the data collection started at 1:55 am on Friday, May 19. The first broad maxima represents the afternoon heating of the bridge on Friday afternoon. The next two maximums are much closer together than the others and represent the afternoon heating of the bridge on Saturday and Sunday afternoon when the traffic volume (i.e., number of strain events) was presumably substantially less than on the weekdays.

These large-magnitude, daily fluctuations in strain due to temperature had a significant impact on the root-mean-cube calculation in some instances. That is, the root-mean-cube data were calculated using three different cut-off values: 2, 5, and 10 microstrain. At the higher cutoff values, with a lesser amount of overall data, the inclusion of a few very high strain values had a significant effect on the root-mean-cube value. However, because they do indeed represent actual strain events, these large-magnitude daily strain fluctuations were included when running the root-mean-cube calculations for each bridge.

Individual charts showing rain flow data for each of the strain gages start on page 70 through 101. Individual charts showing seven-day peak-valley histories for each of the gages are on pages 102 through 133. Tables 8 through 11 on pages 134 through 137 show root-mean-cube data for each of the gages.

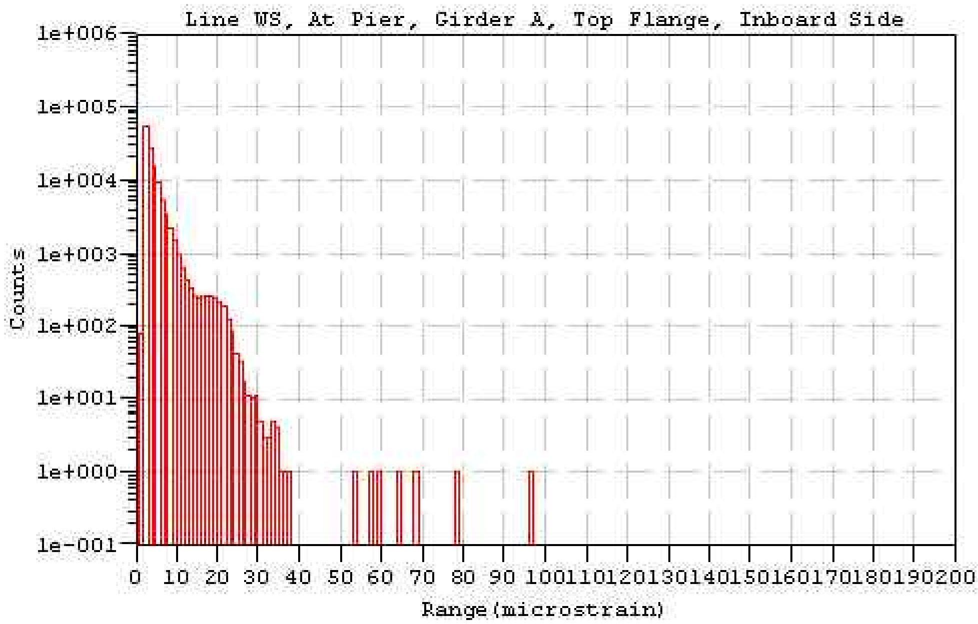


Figure 14. Typical RainFlow Diagram

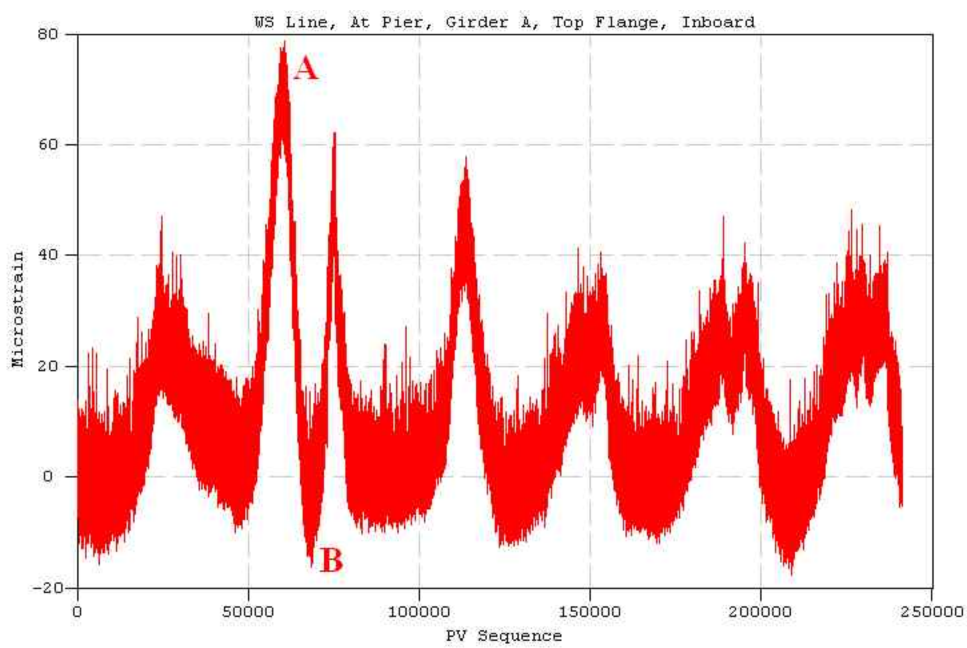


Figure 15. Typical Seven-Day Peak-Valley Strain History

Strain-Time Histories of Largest Strain Events

Figure 16 is a graph of a typical strain-time history showing the first-highest strain cycles that occurred during the seven days of data gathering (in this case, Bridge 9268W (EW Line) at mid-span) for each of the eight gages in the section. Similar plots for each of the eight instrumented bridge sections (four bridges, two sections each) are shown on pages 138 through 217 for each of the ten highest-strain events (i.e., first-highest strain event, second-highest strain event, third-highest strain event, etc.)

The magnitude of the recorded strain for each individual gage during a particular traffic event was dependent on the weight of the truck and its position on the bridge relative to the gage location; thus, only in a few instances did a single traffic event (i.e., a particularly large truck) result in the simultaneous recording of a maximum strain on more than one gage.

To compensate for the fact that the maximum strains did not, in general, occur simultaneously, the plots have been superimposed on each other so that the highest peak of each strain event for each gage is shown at the same position on the graph.

Note the discontinuity in data at about 3¼ seconds on gage No. 9 (see Figure 13 for gage identifications). We had set up the data acquisition equipment to record 10 seconds of data for each of the ten highest strain events for each gage–five seconds before the peak strain and five seconds after. In most cases the data record for the event showed exactly that–5 seconds before the highest strain and 5 seconds after the highest strain; however, in a few instances, the length of the recording was not symmetrical about the highest strain. We did not establish why this occurred, but when it did, the plots do not contain a full 5 seconds of data on both sides of the maximum strain. (i.e., they contain more than 5 seconds on one side and less than 5 seconds on the other.)

Because we wanted to show all the peaks aligned in the plot, it was necessary in these cases to offset the data relative to the peak, leaving a discontinuity in the data either before or after the peak. When this non-symmetrical recording of data occurred, the data shown either before or after the discontinuity are from the adjacent strain event recorded by the equipment. Thus, in the case of Figure 16, the data shown in the plot for gage No. 9 after the discontinuity (beyond 3¼ seconds) is from the next strain event recorded by the instrumentation.

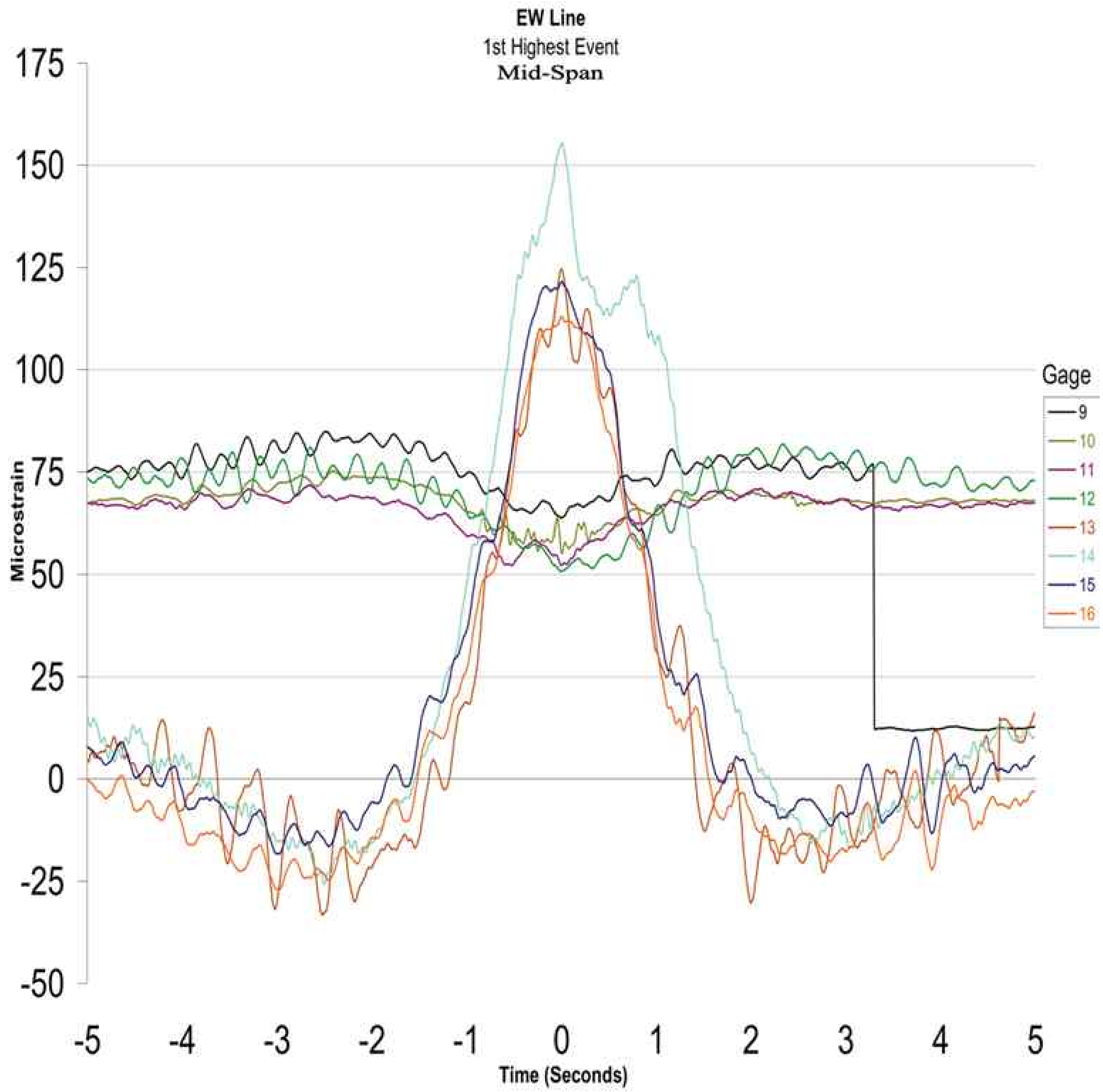
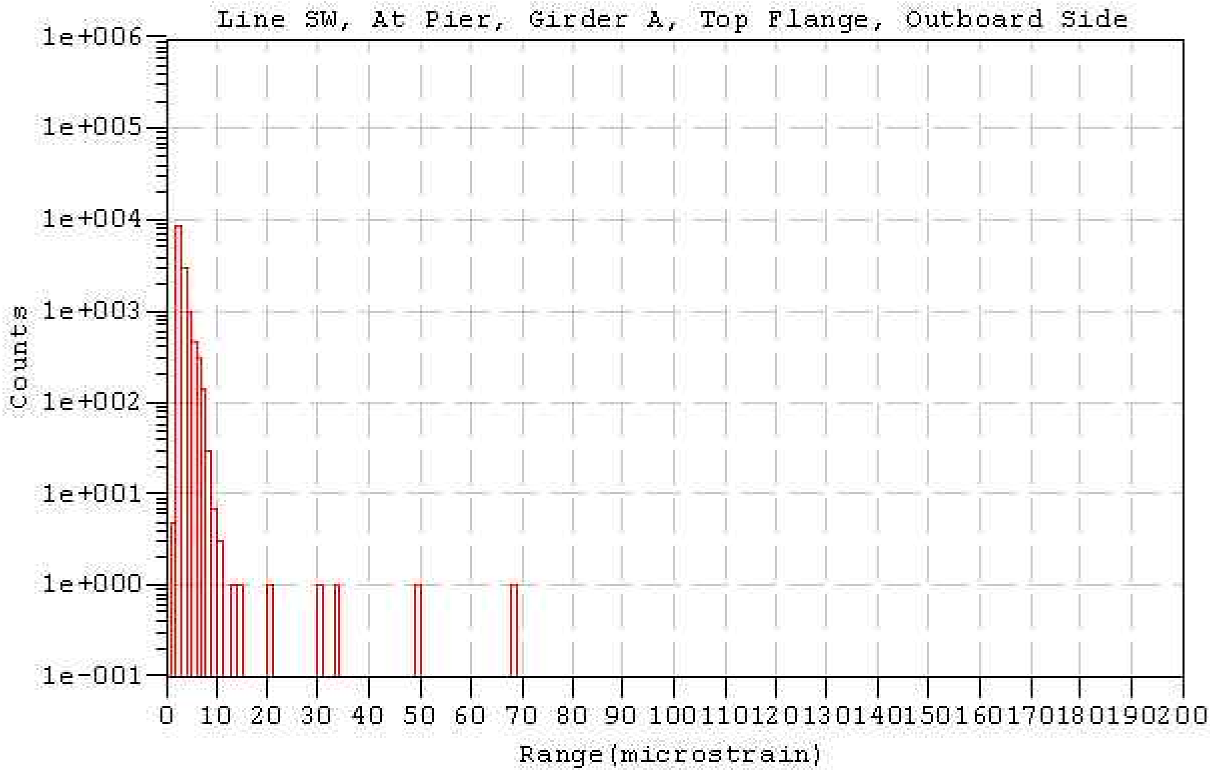
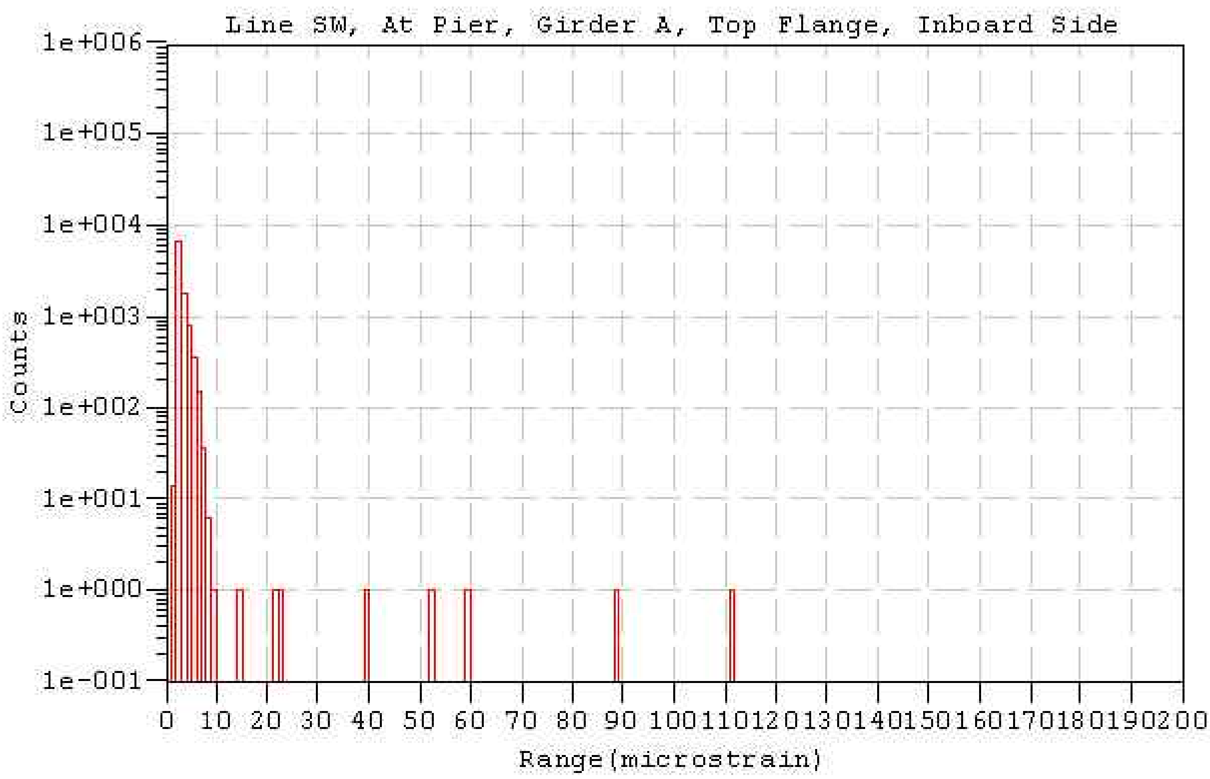


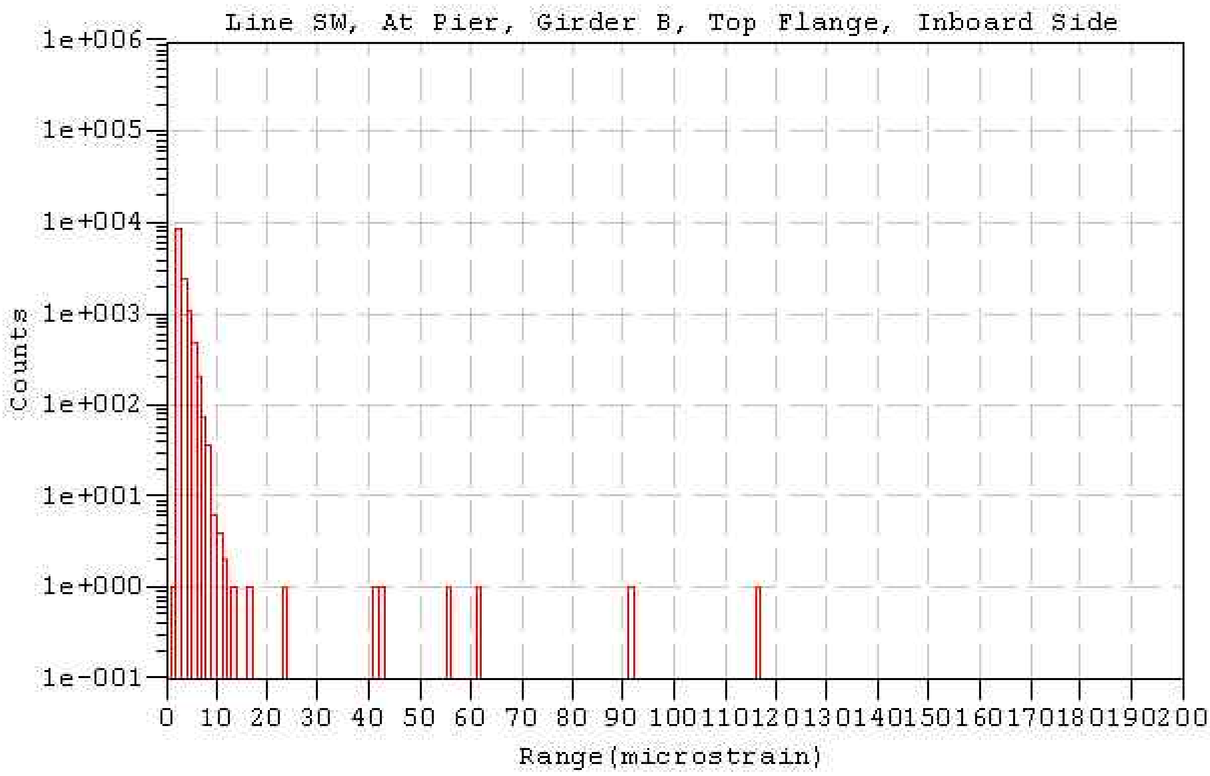
Figure 16. Typical Strain-Time History of Largest Strain Cycles

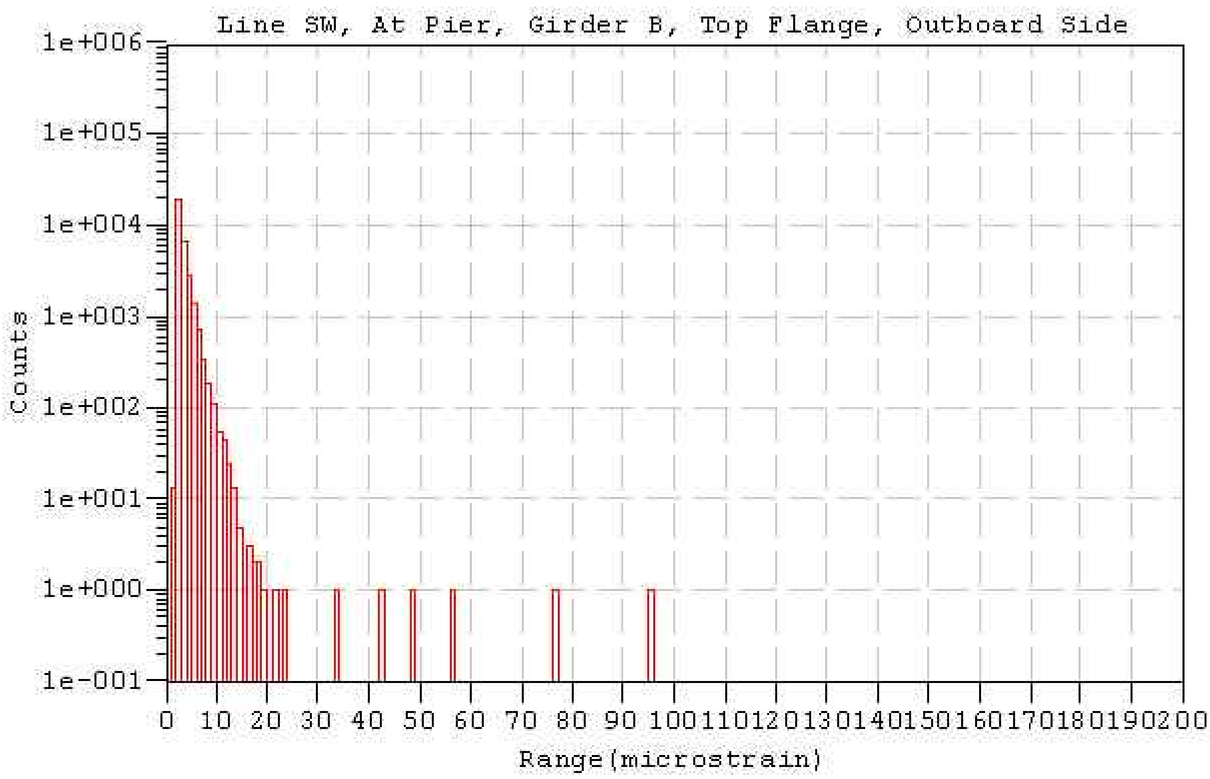
RainFlow Diagrams



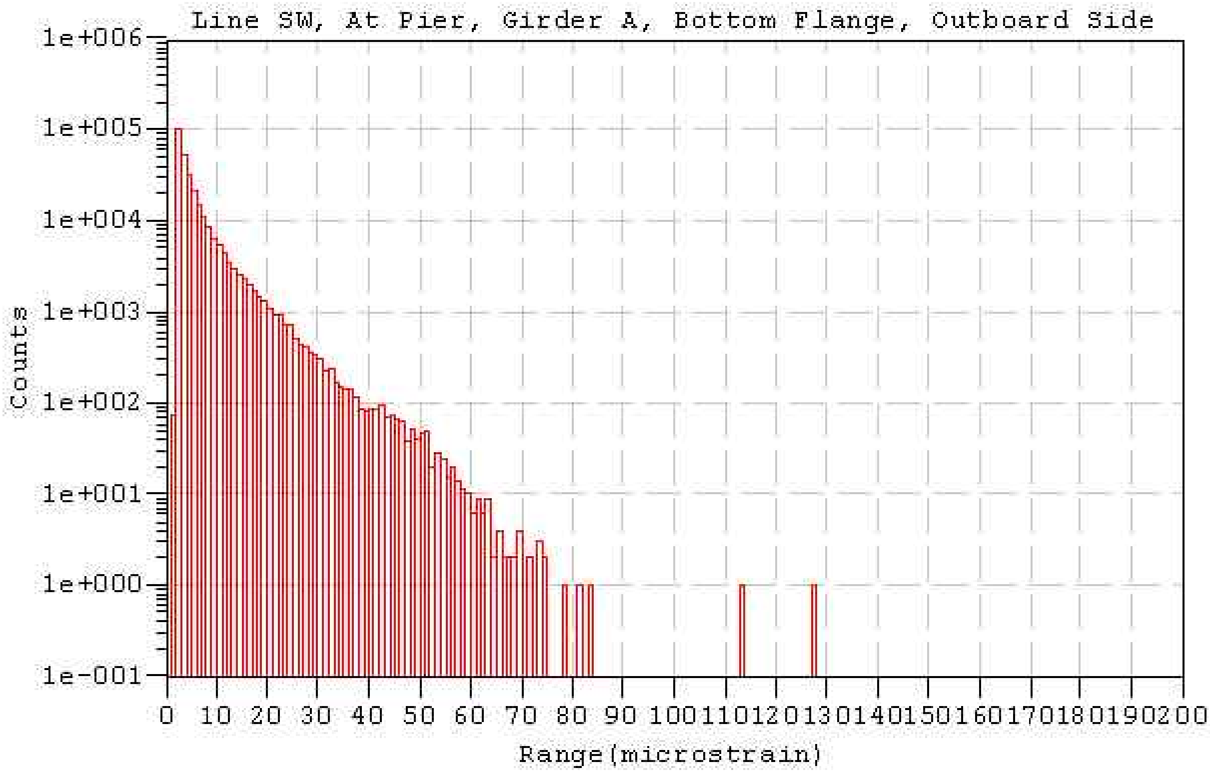


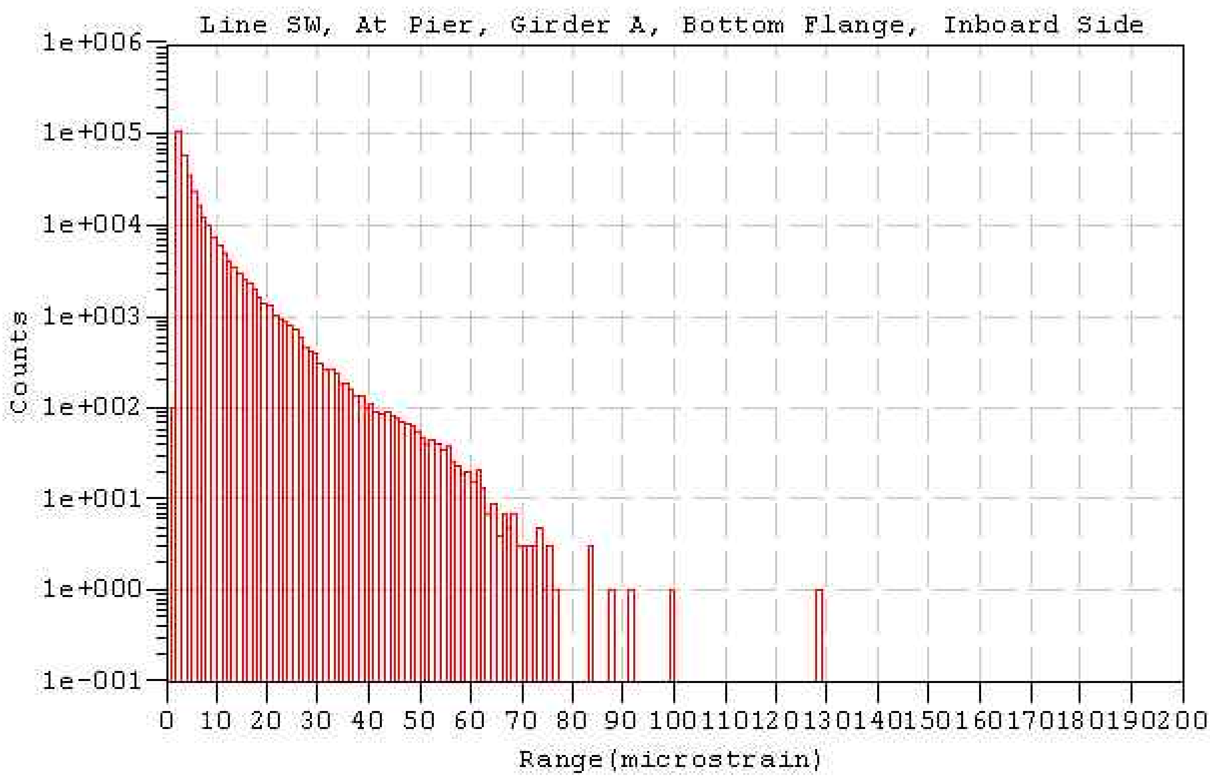
RainFlow Diagrams



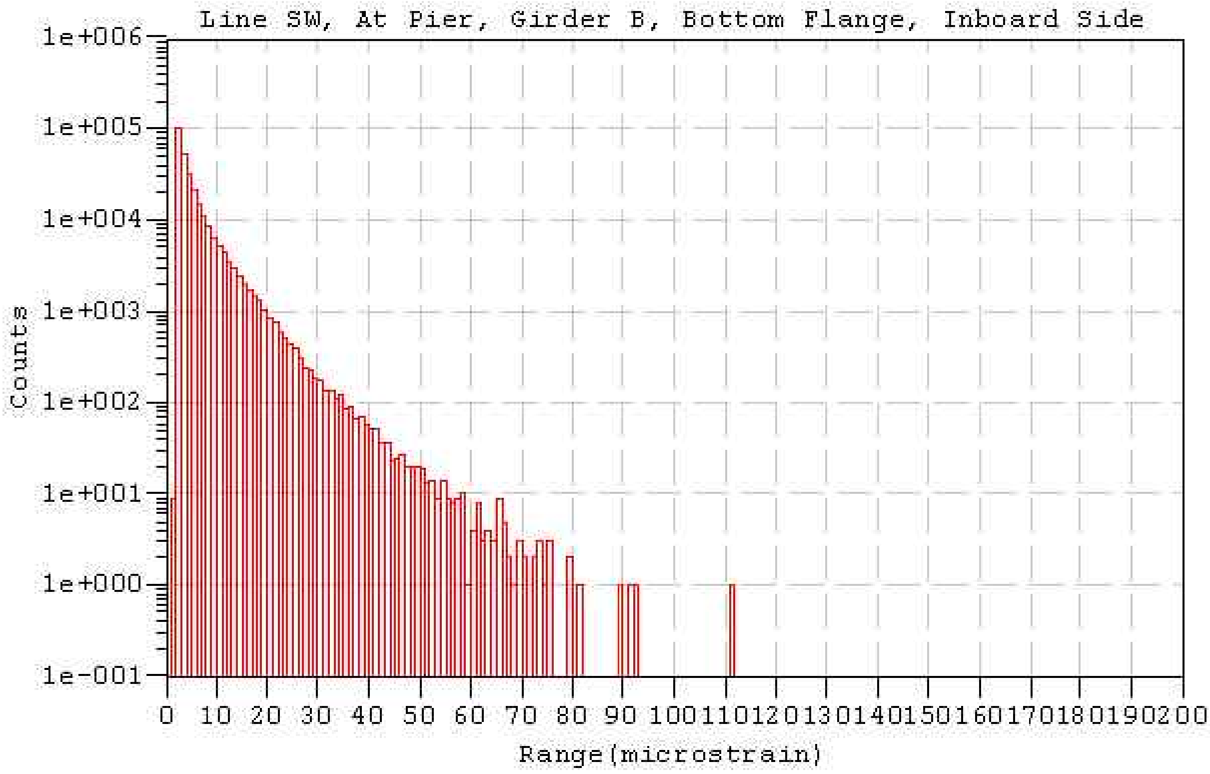


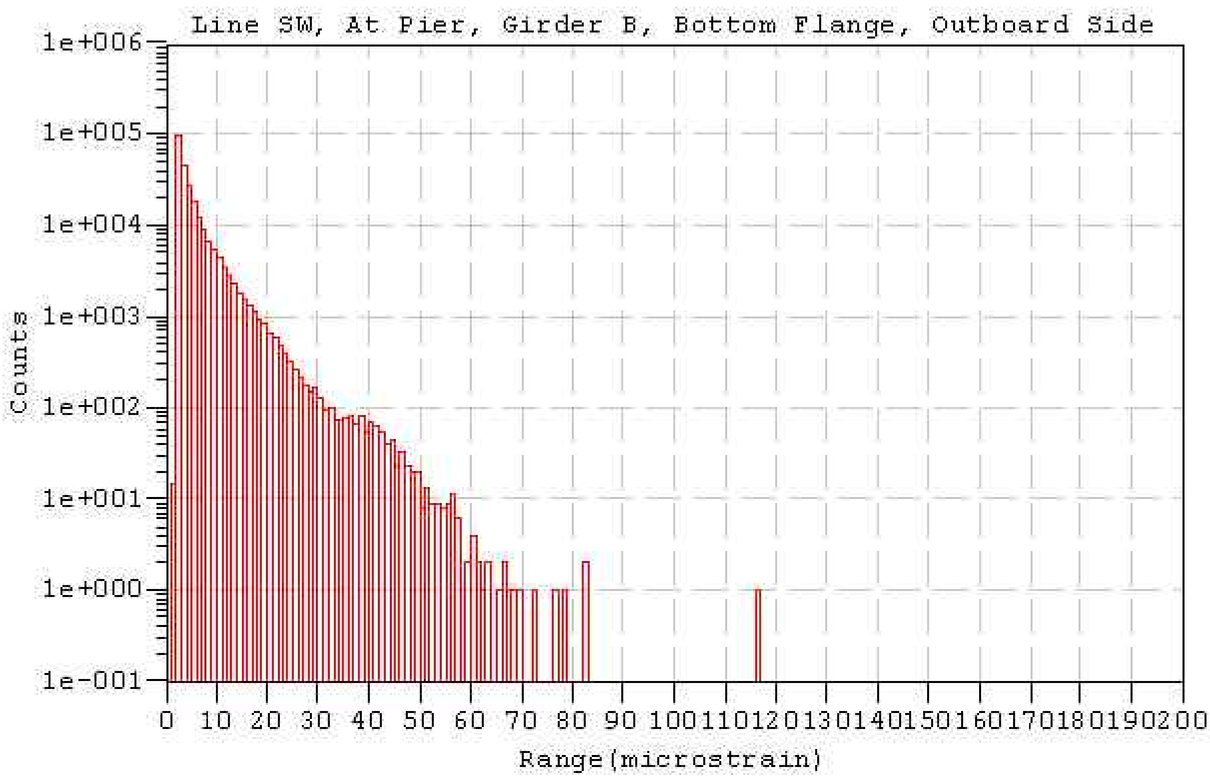
RainFlow Diagrams



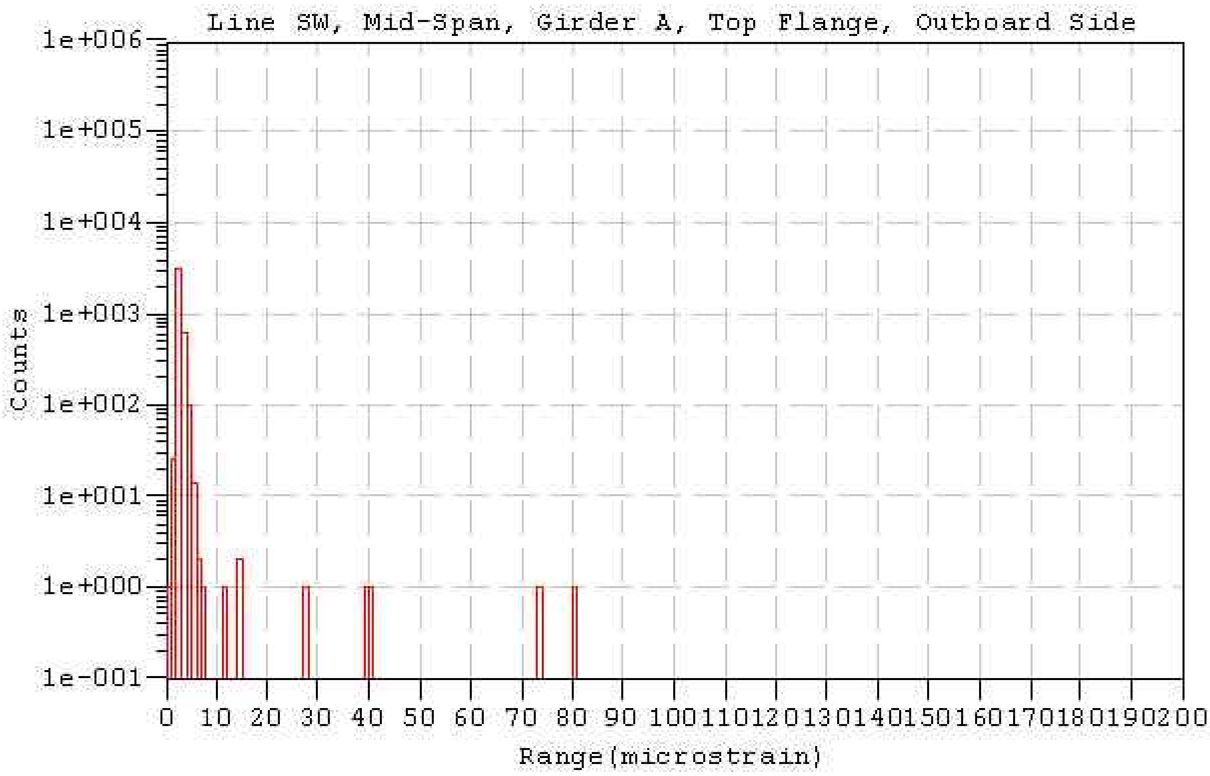


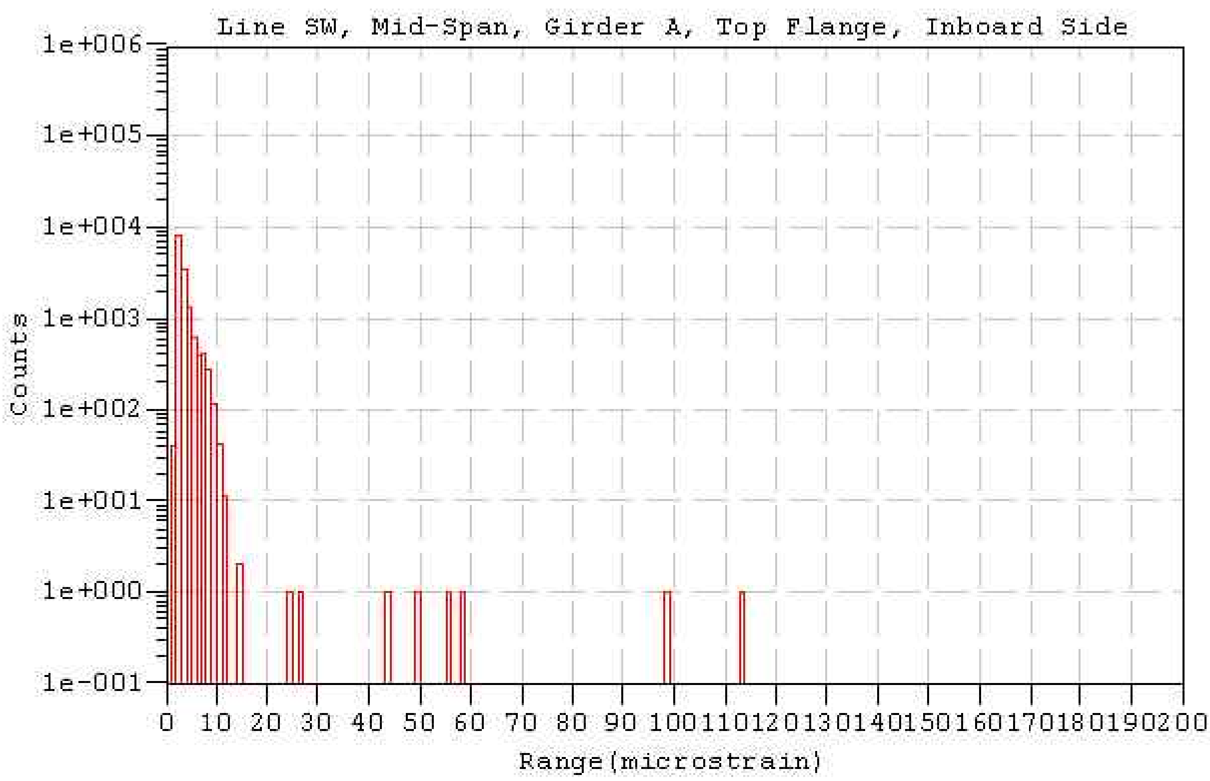
RainFlow Diagrams



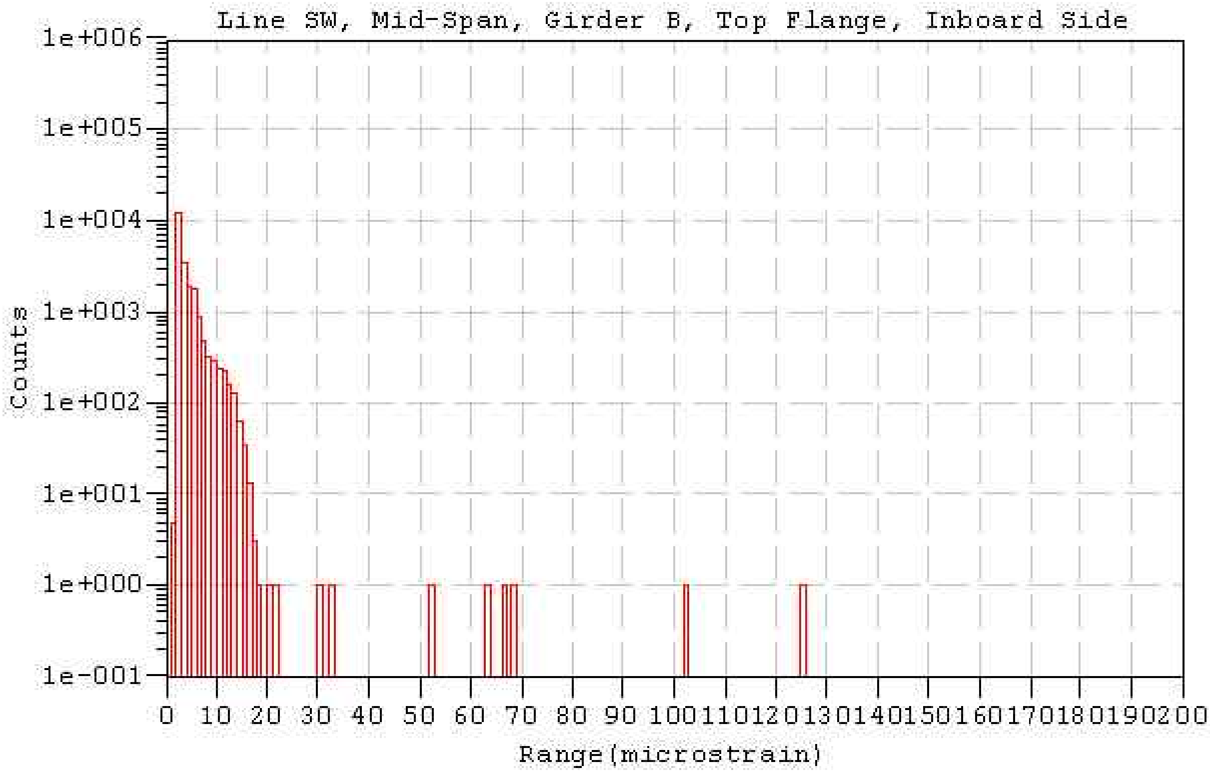


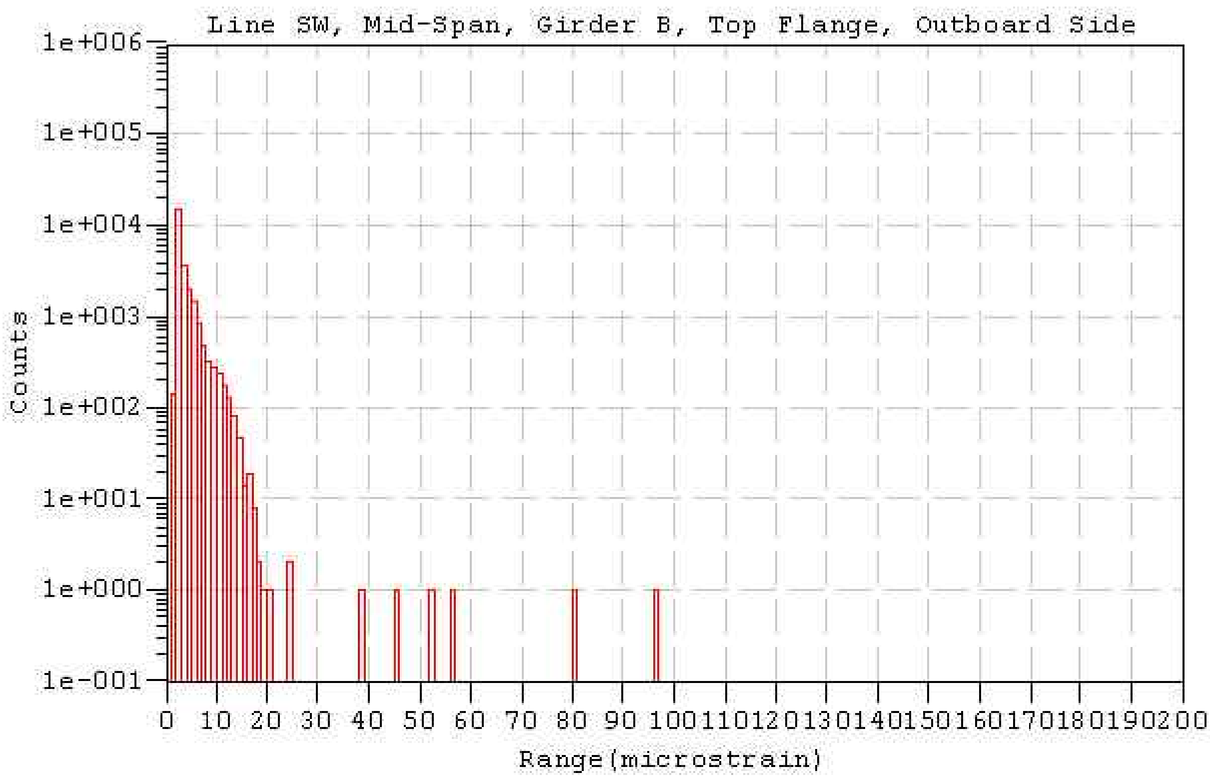
RainFlow Diagrams





RainFlow Diagrams





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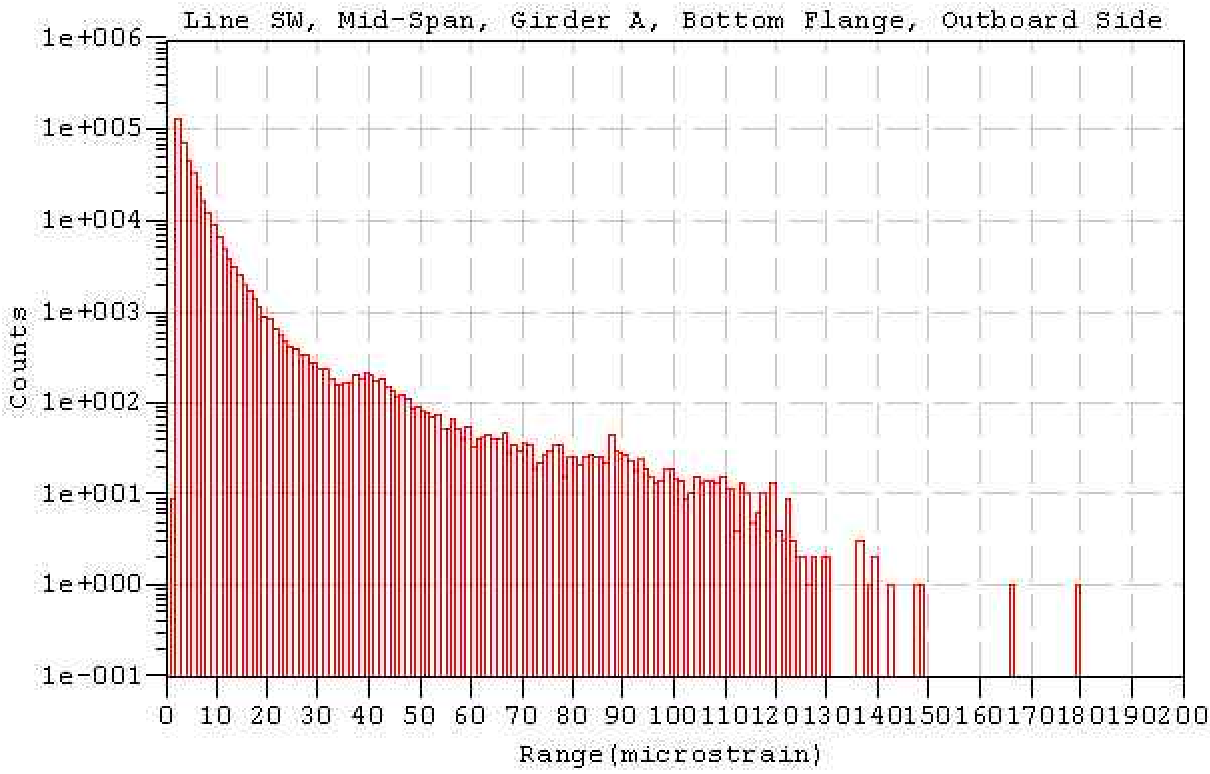

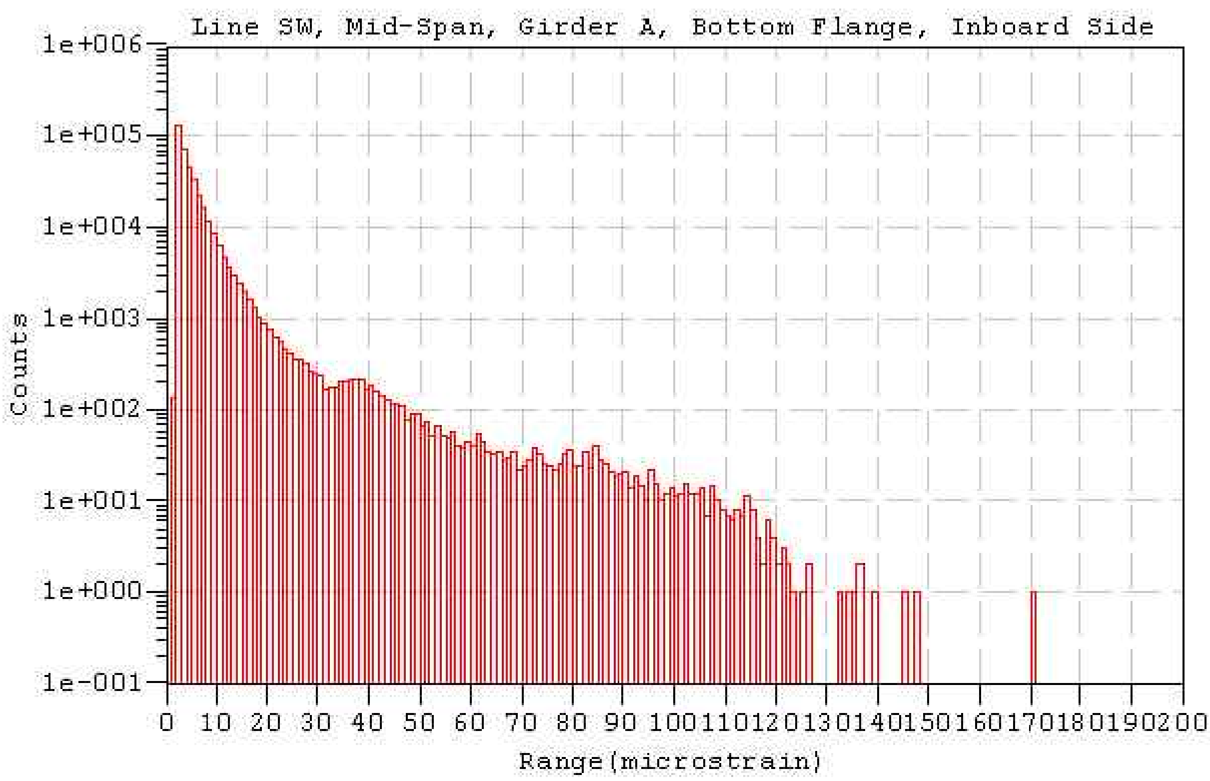

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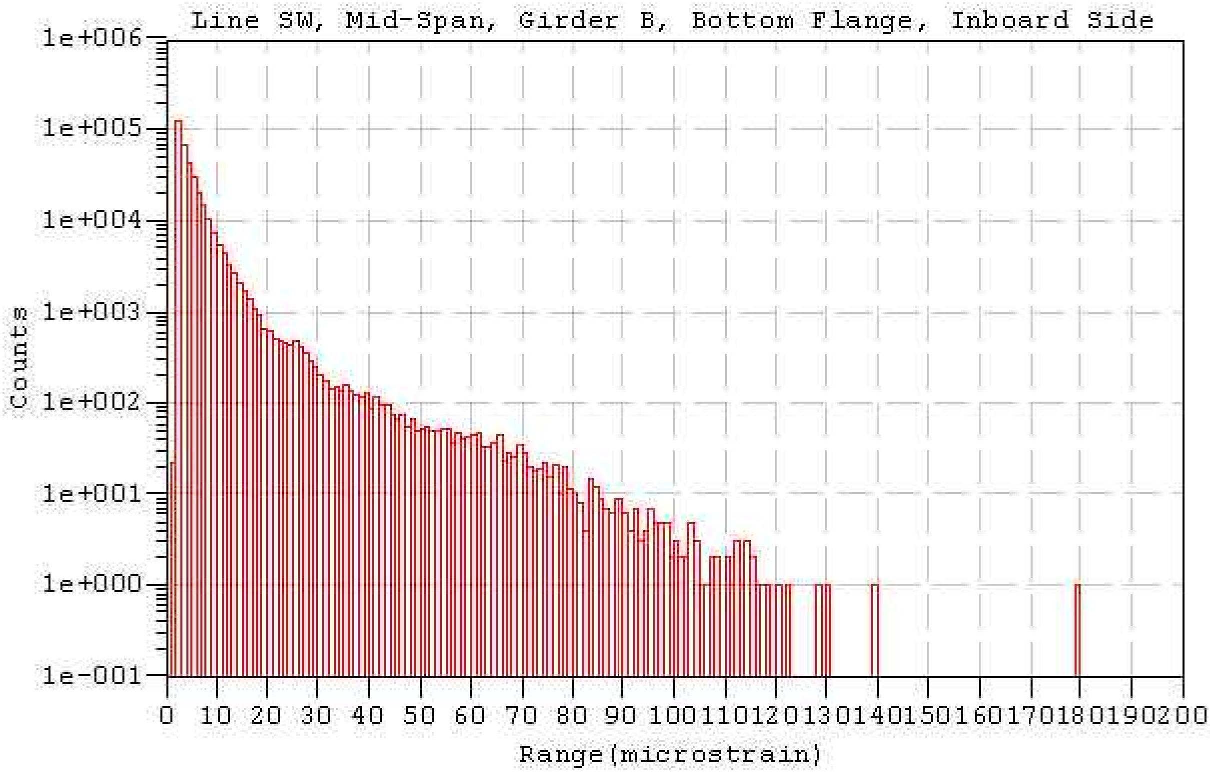

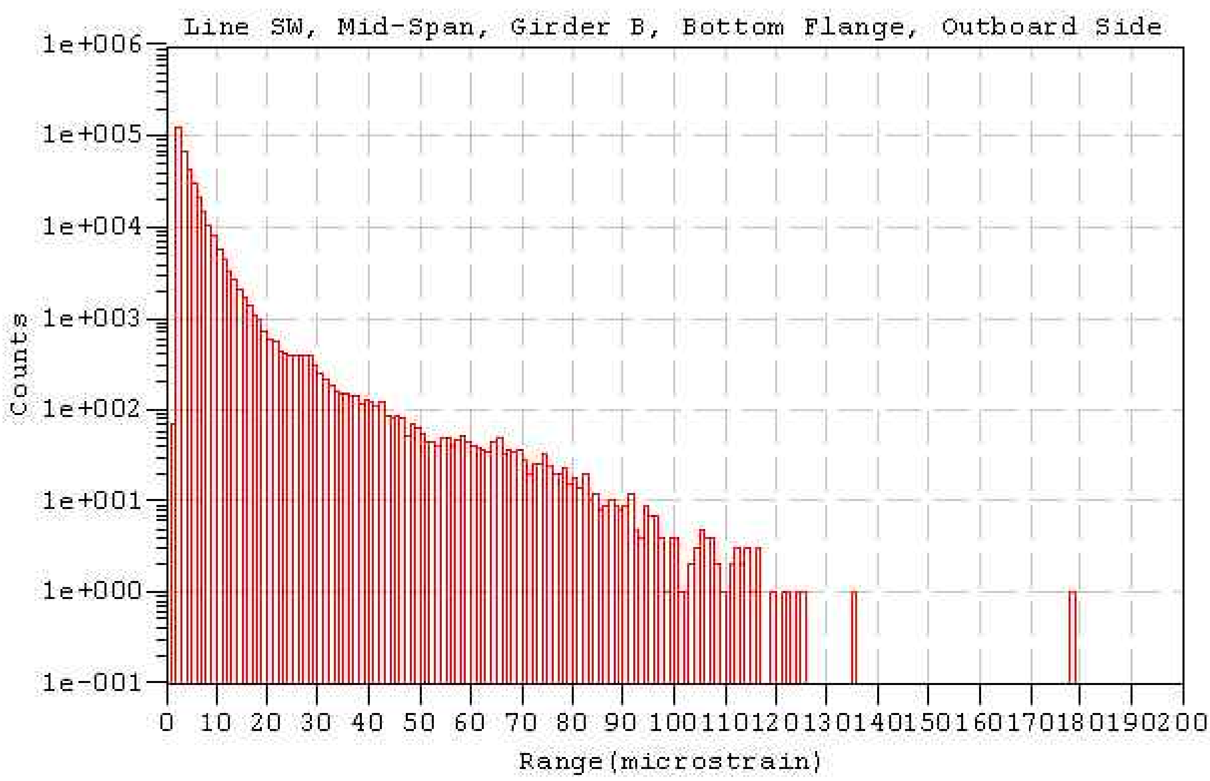

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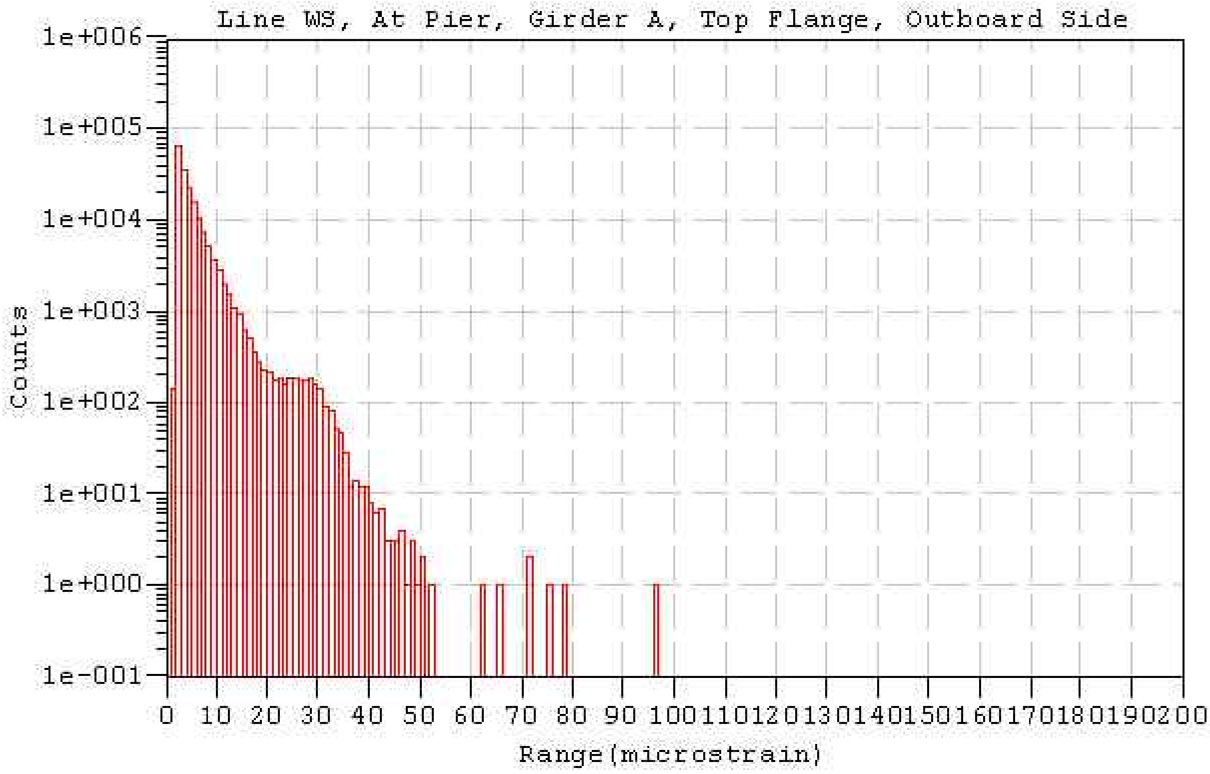

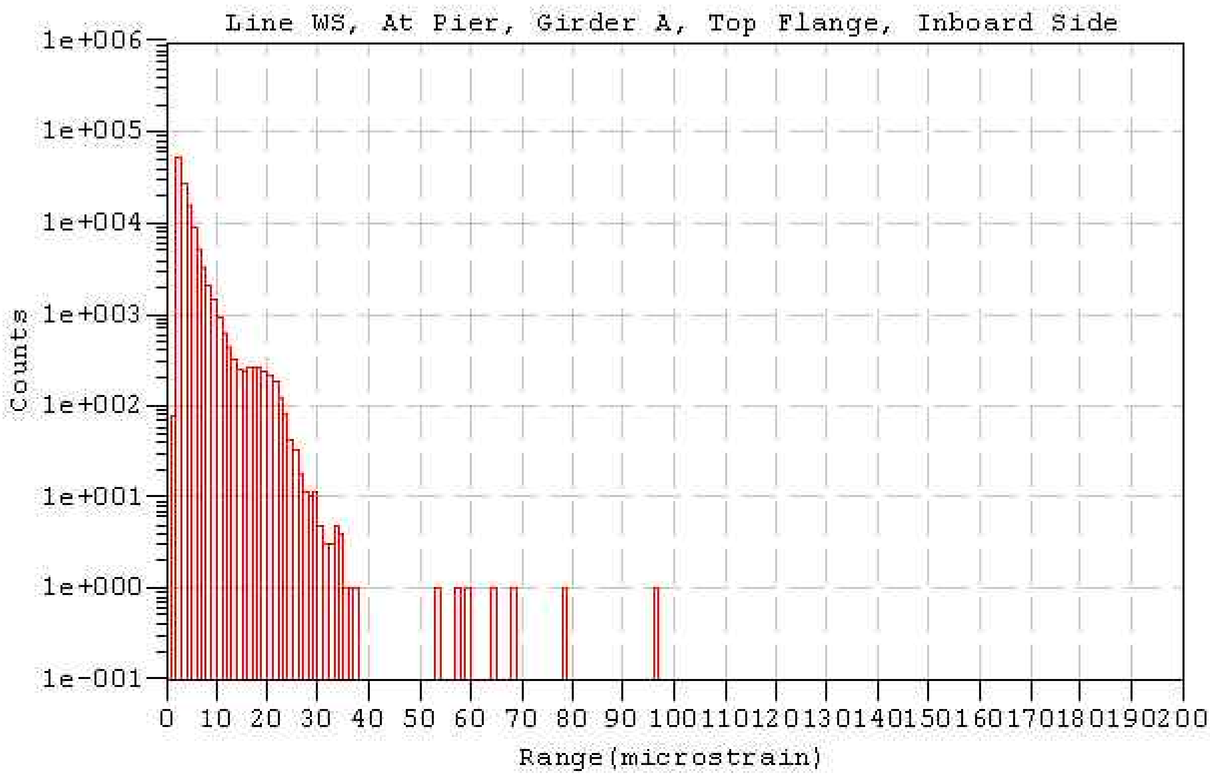

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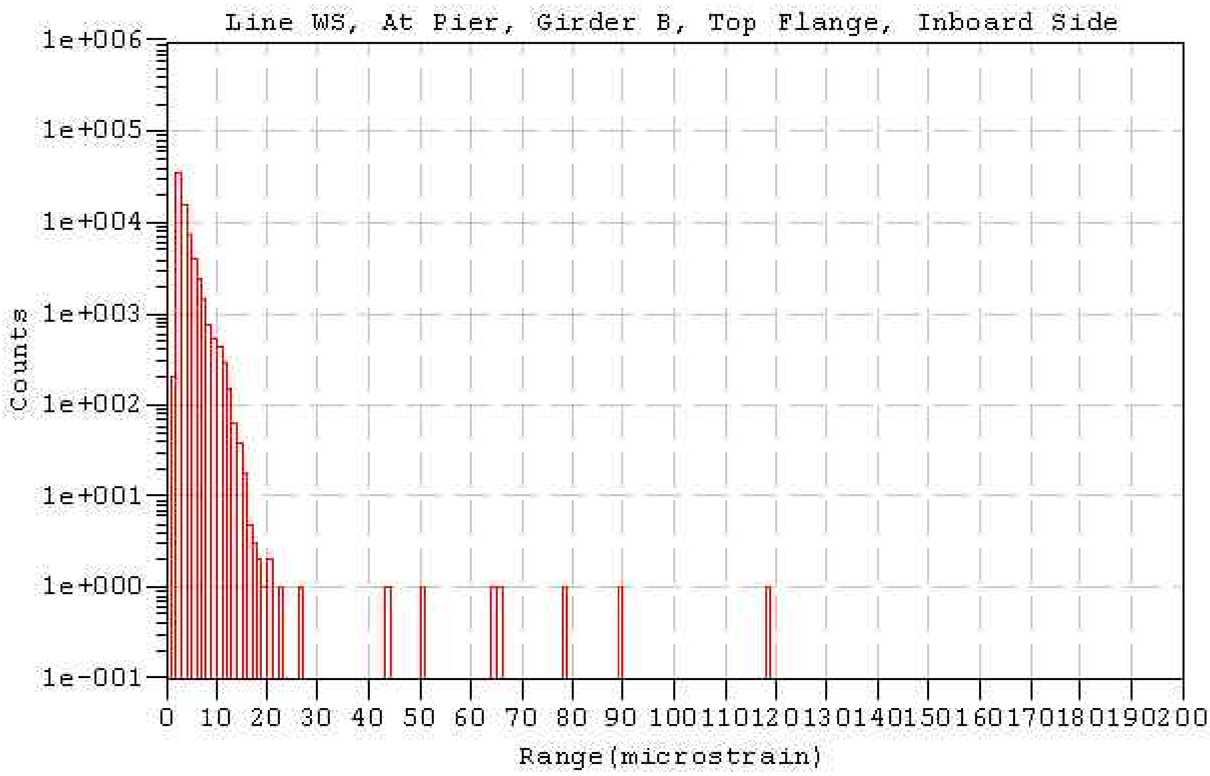

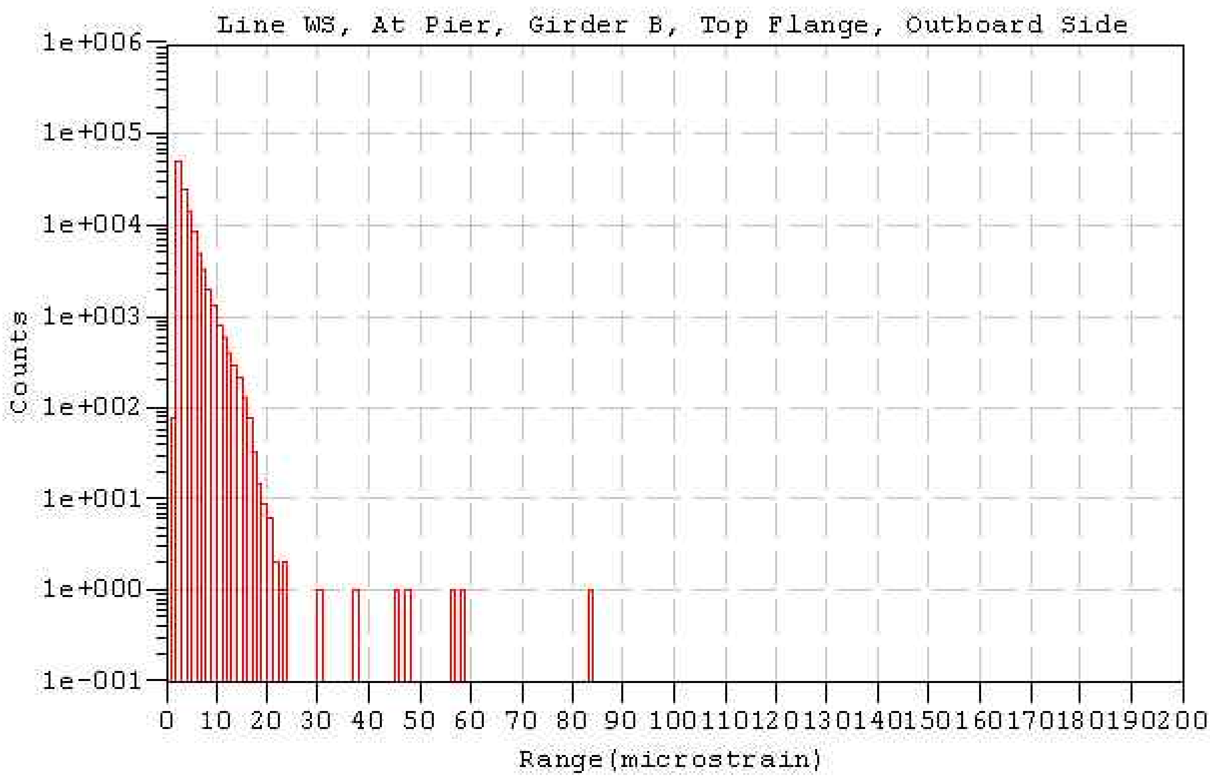

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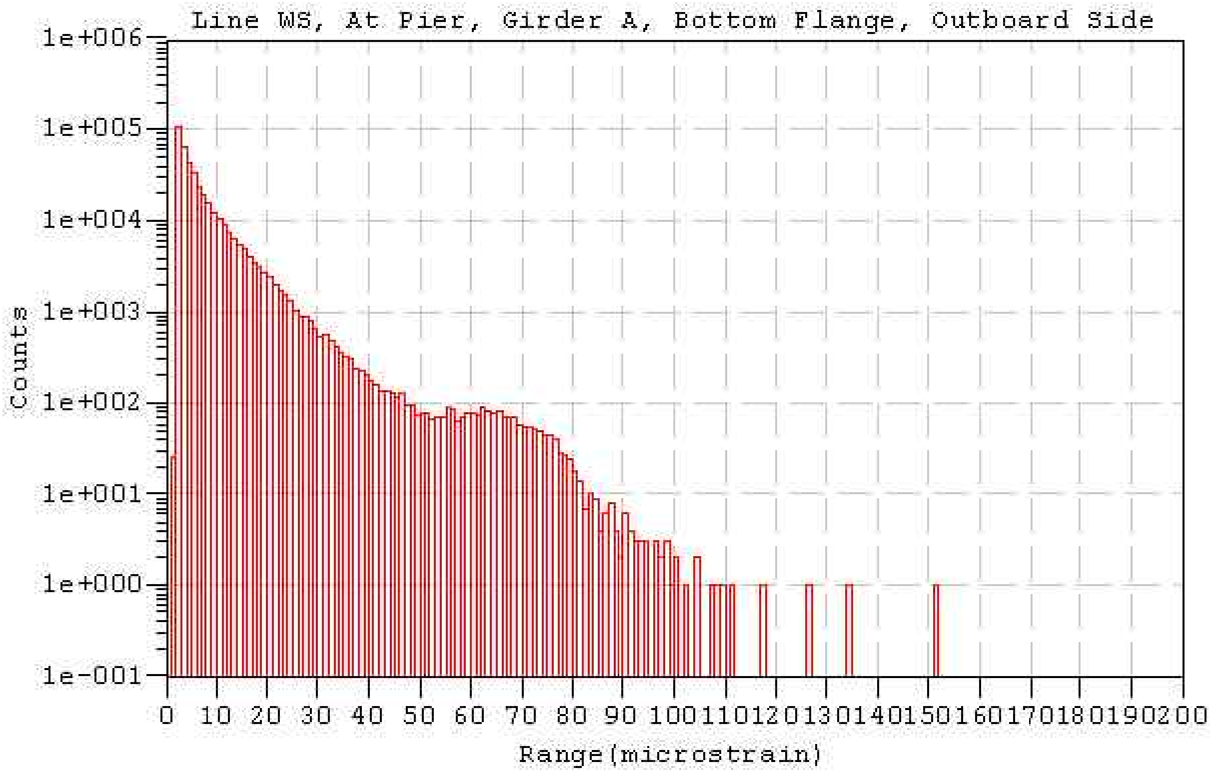

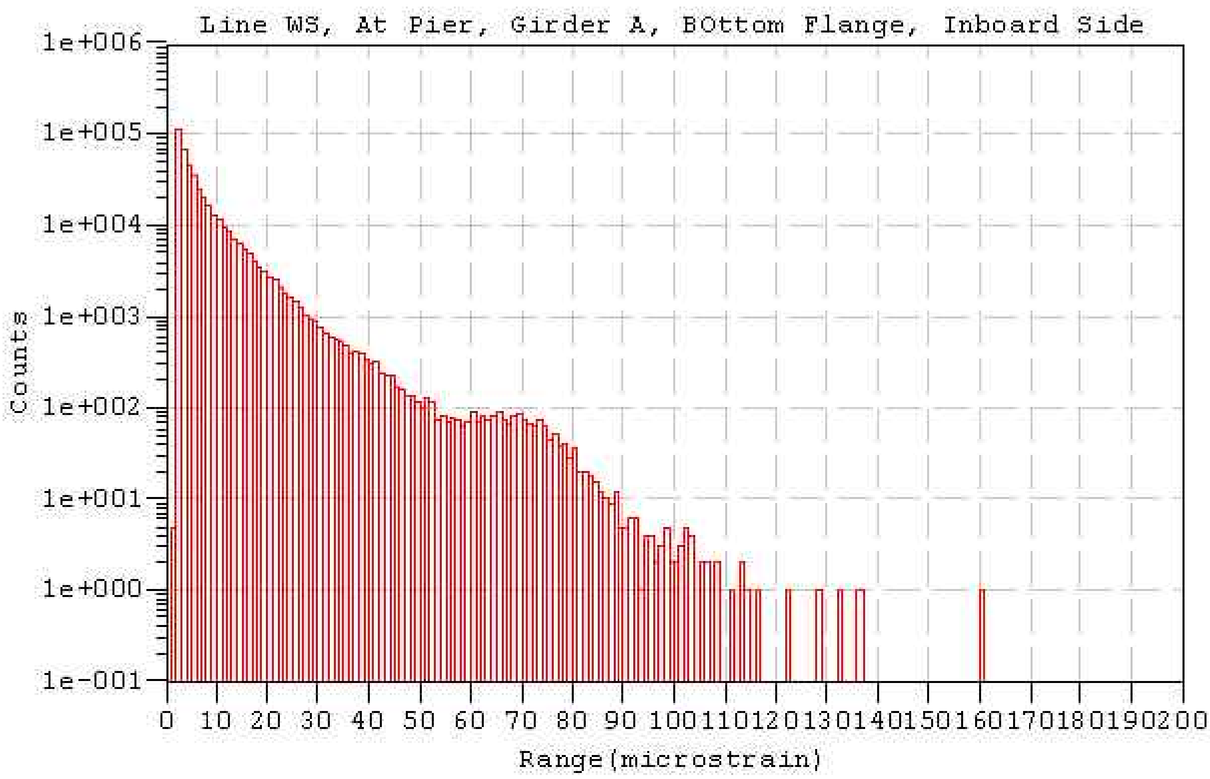

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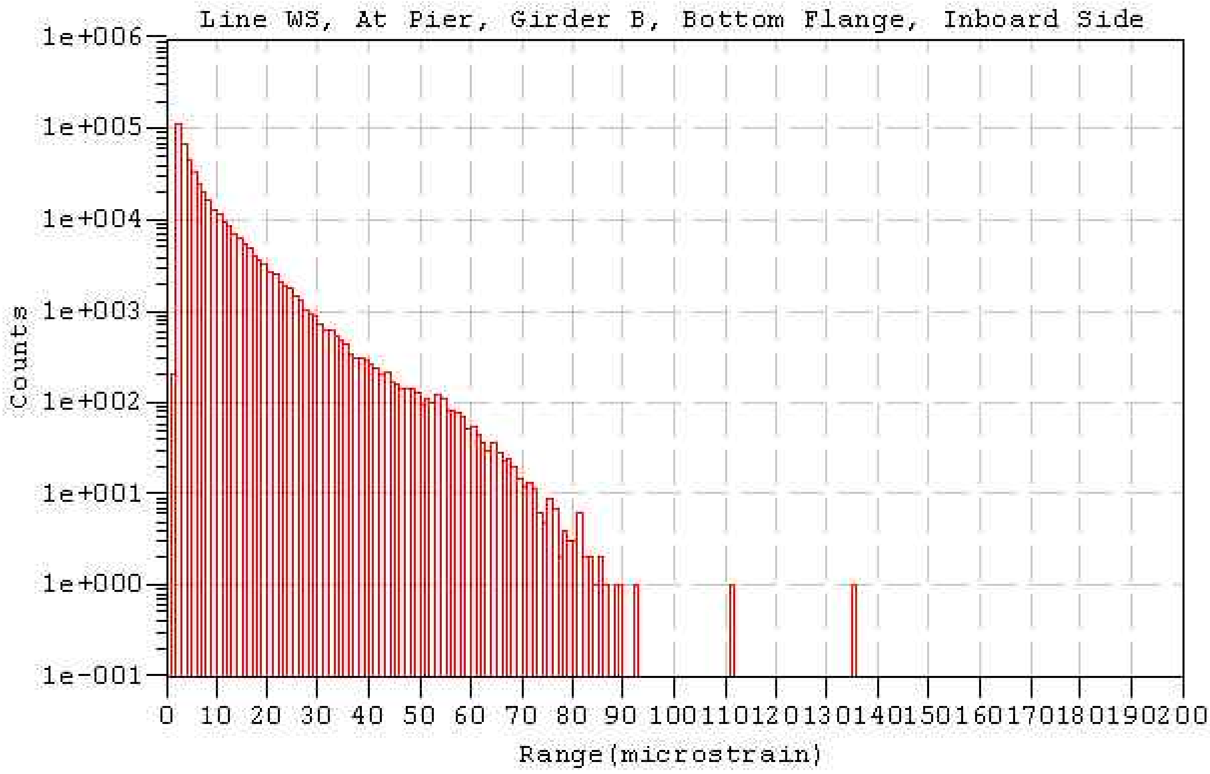

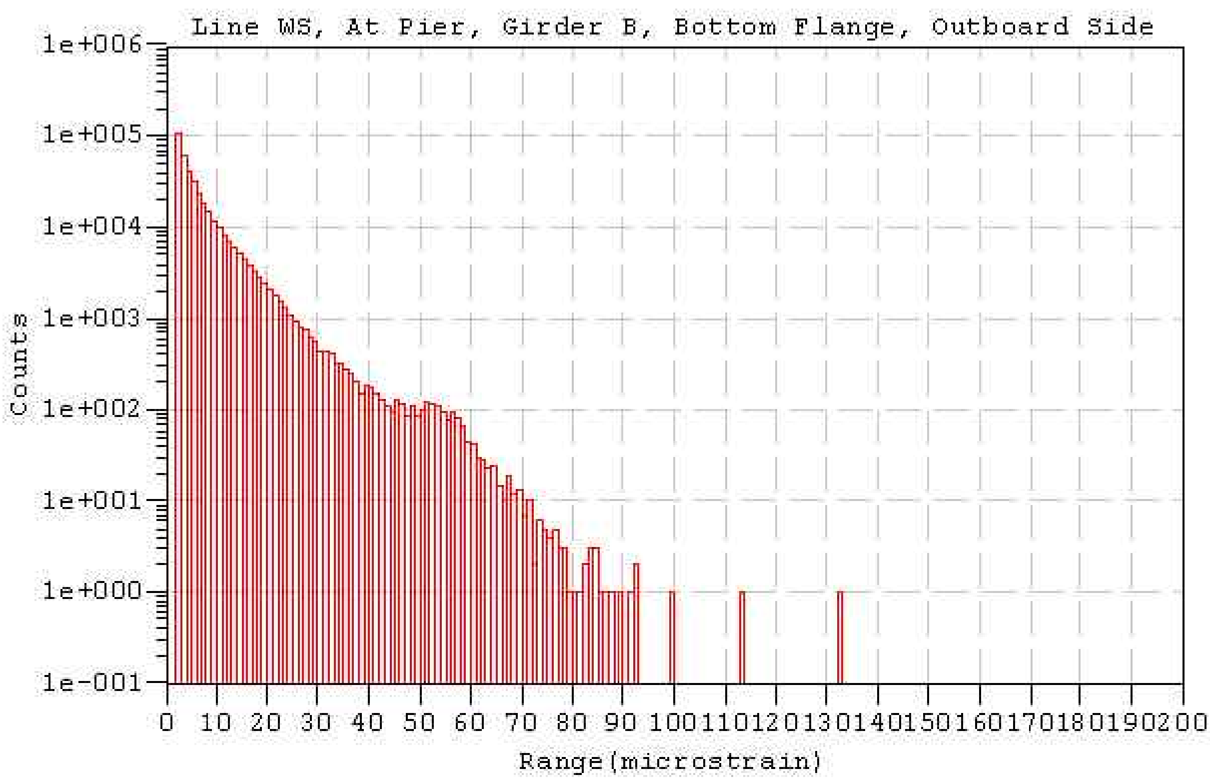

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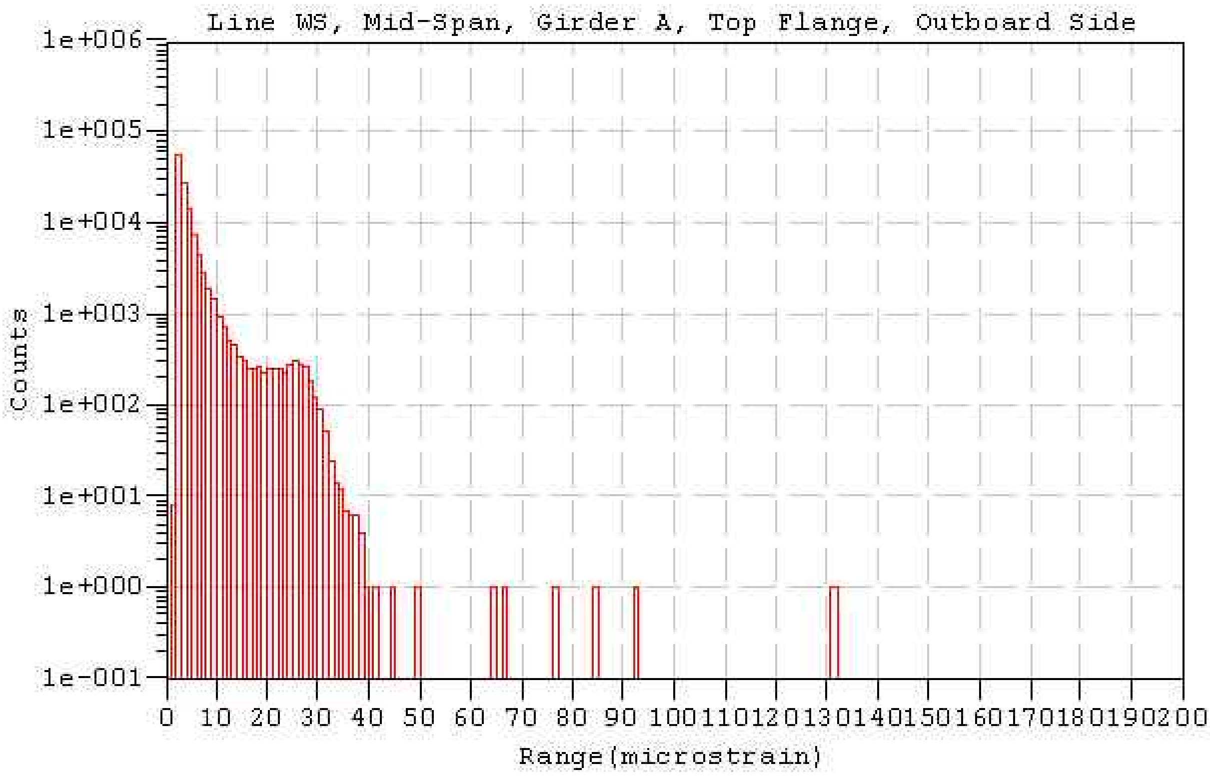

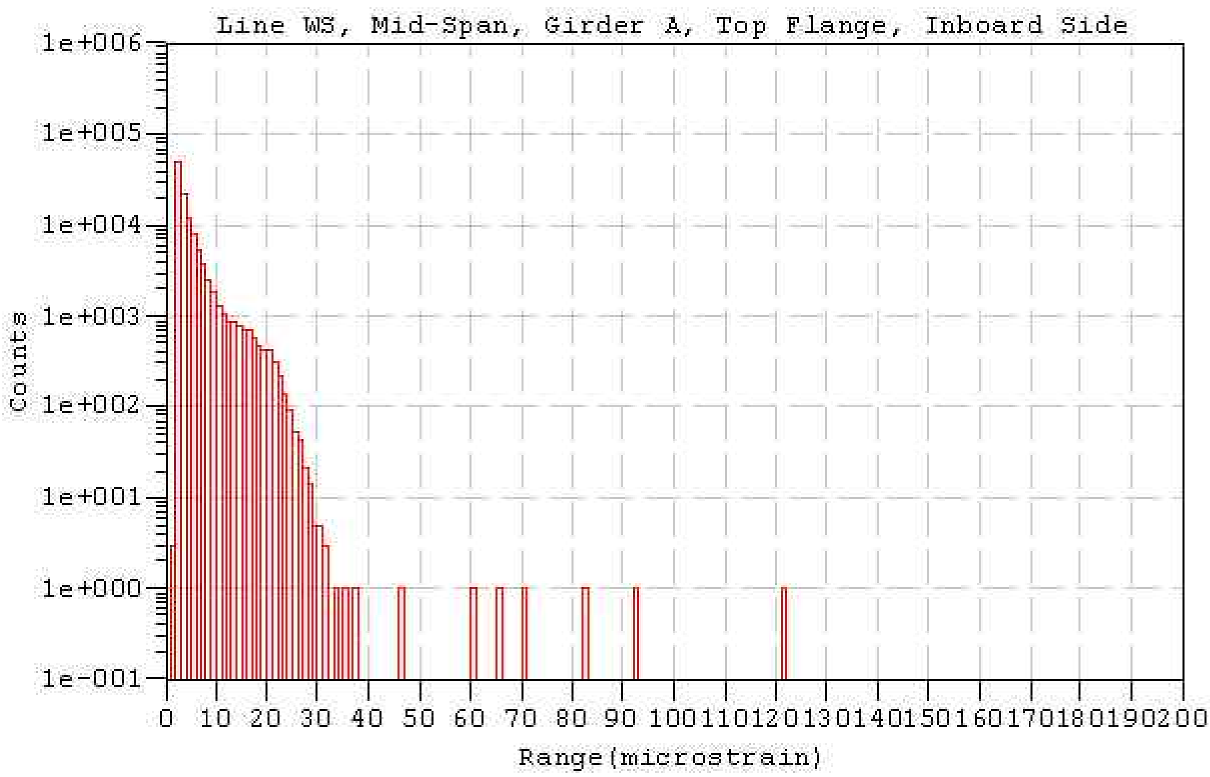

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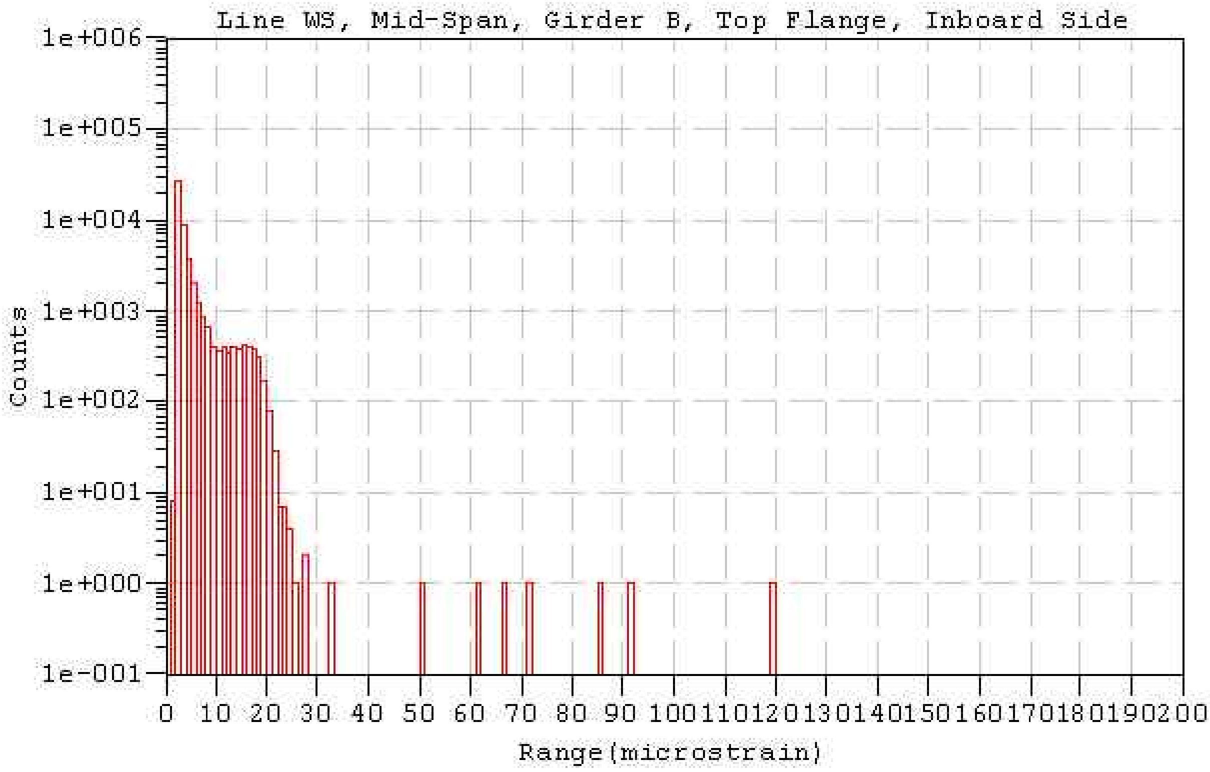

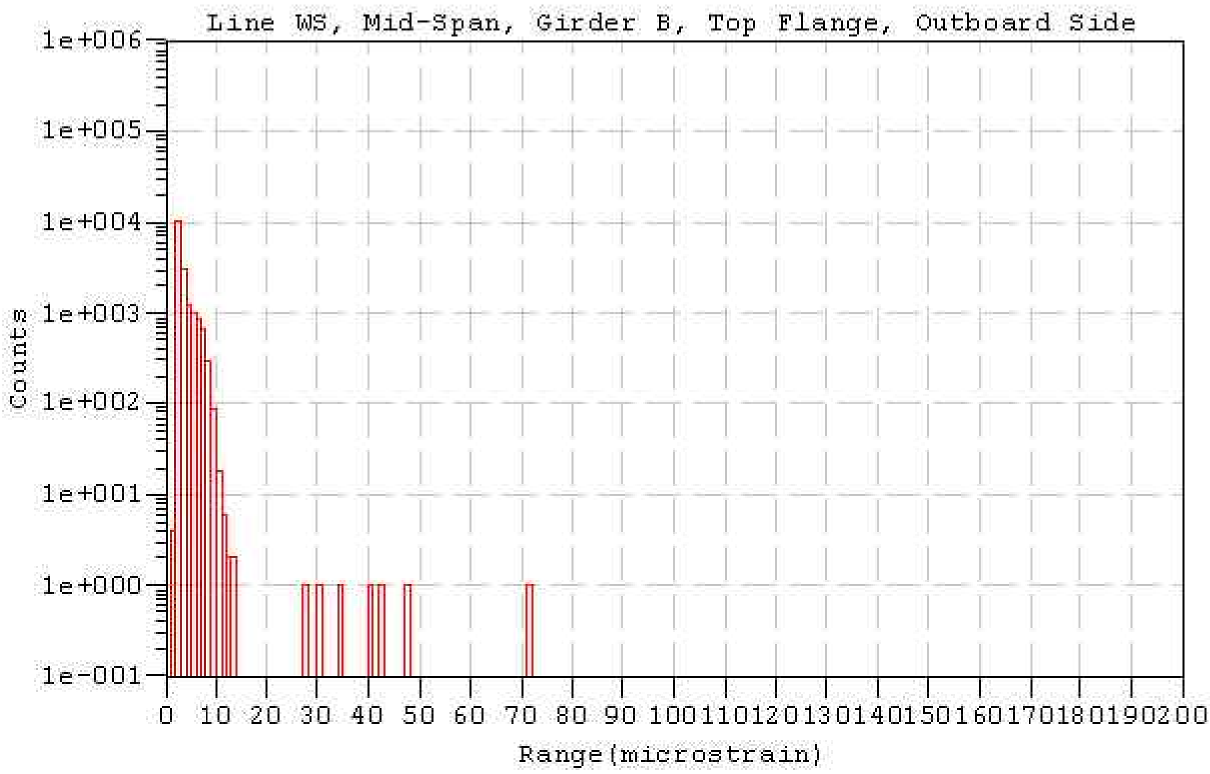

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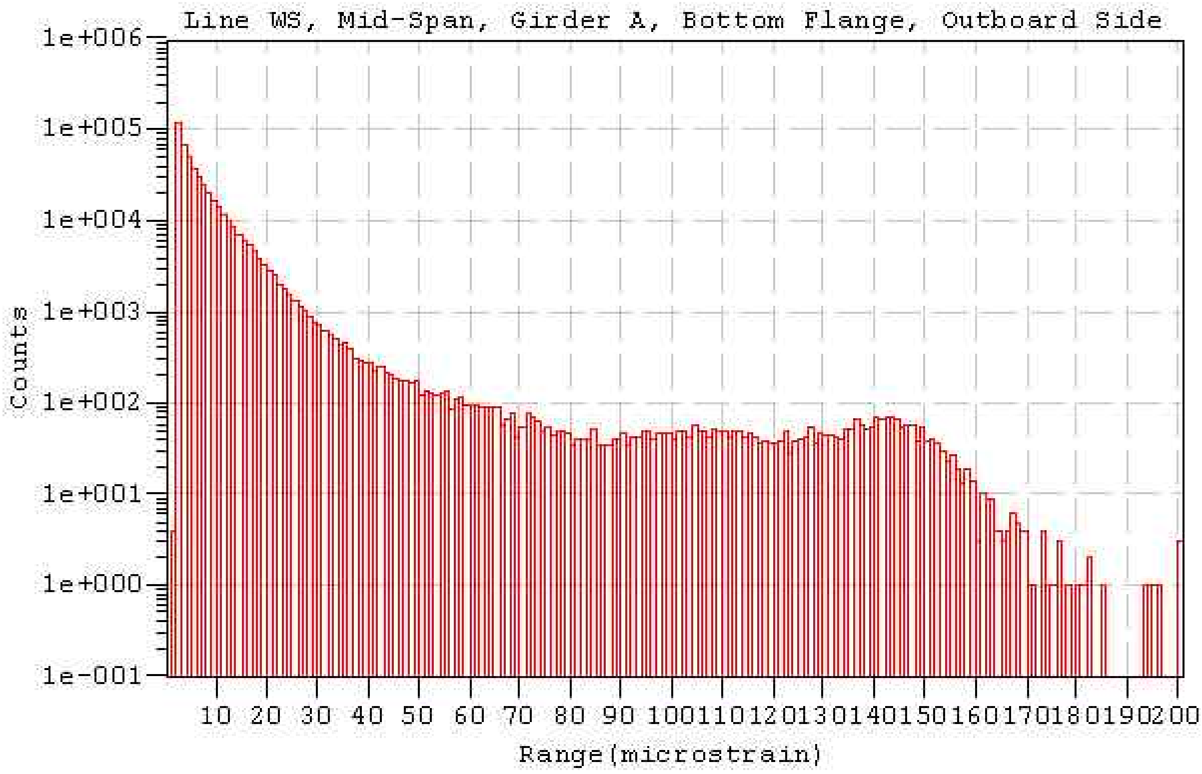

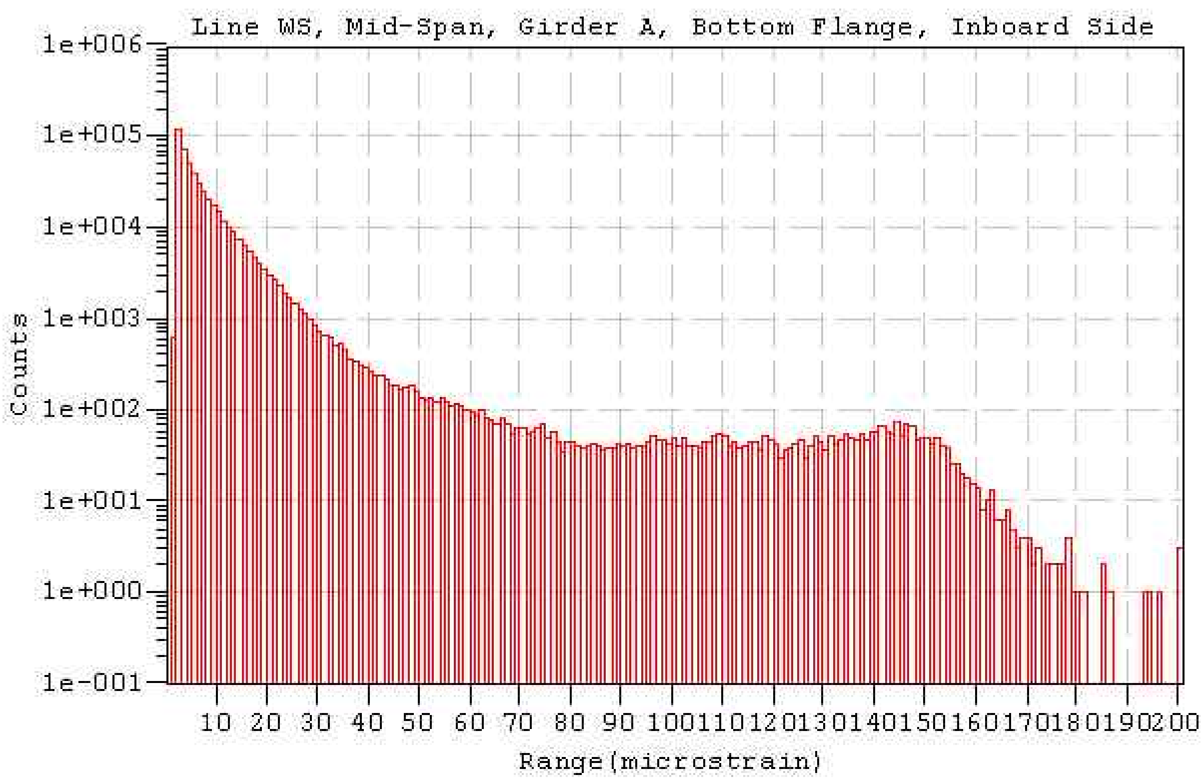

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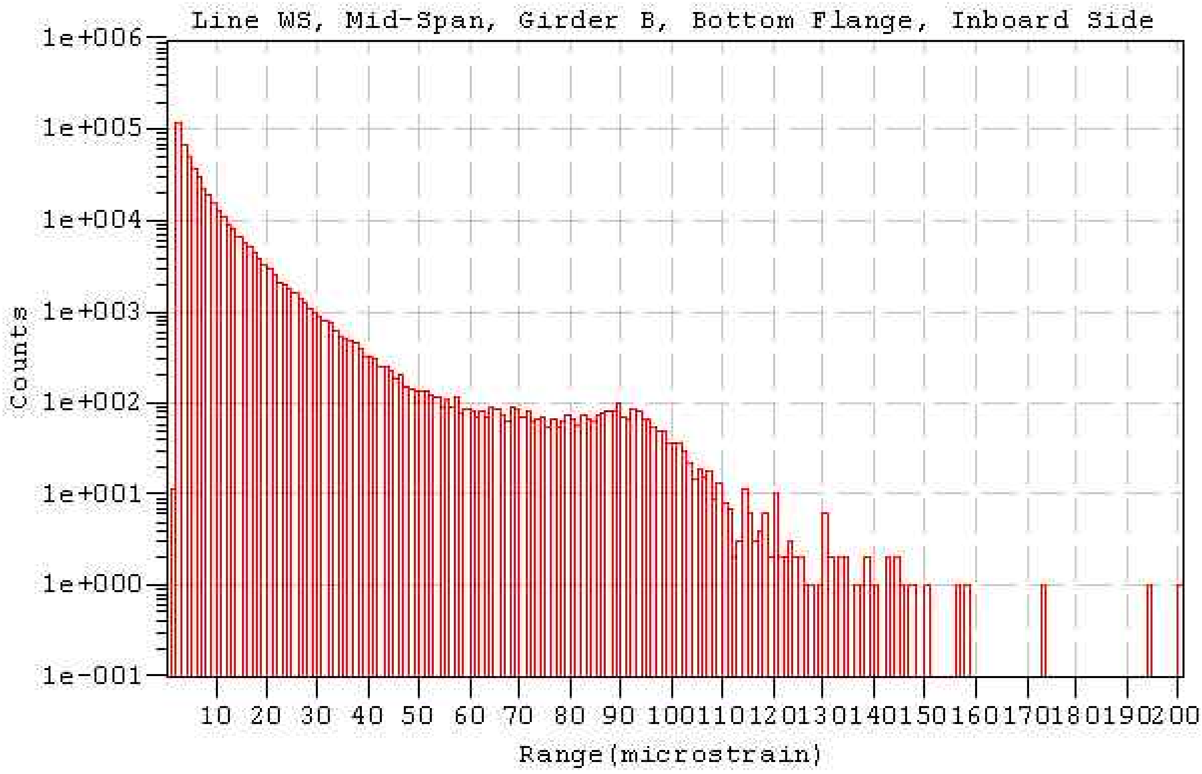

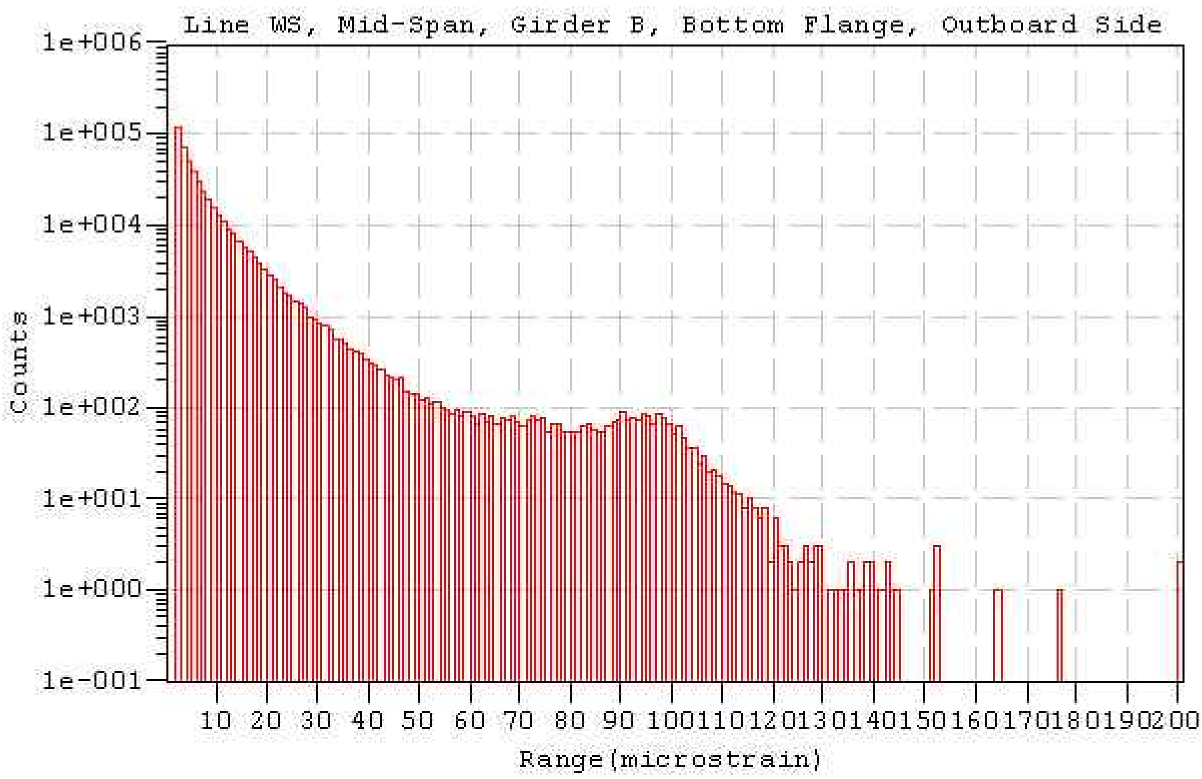

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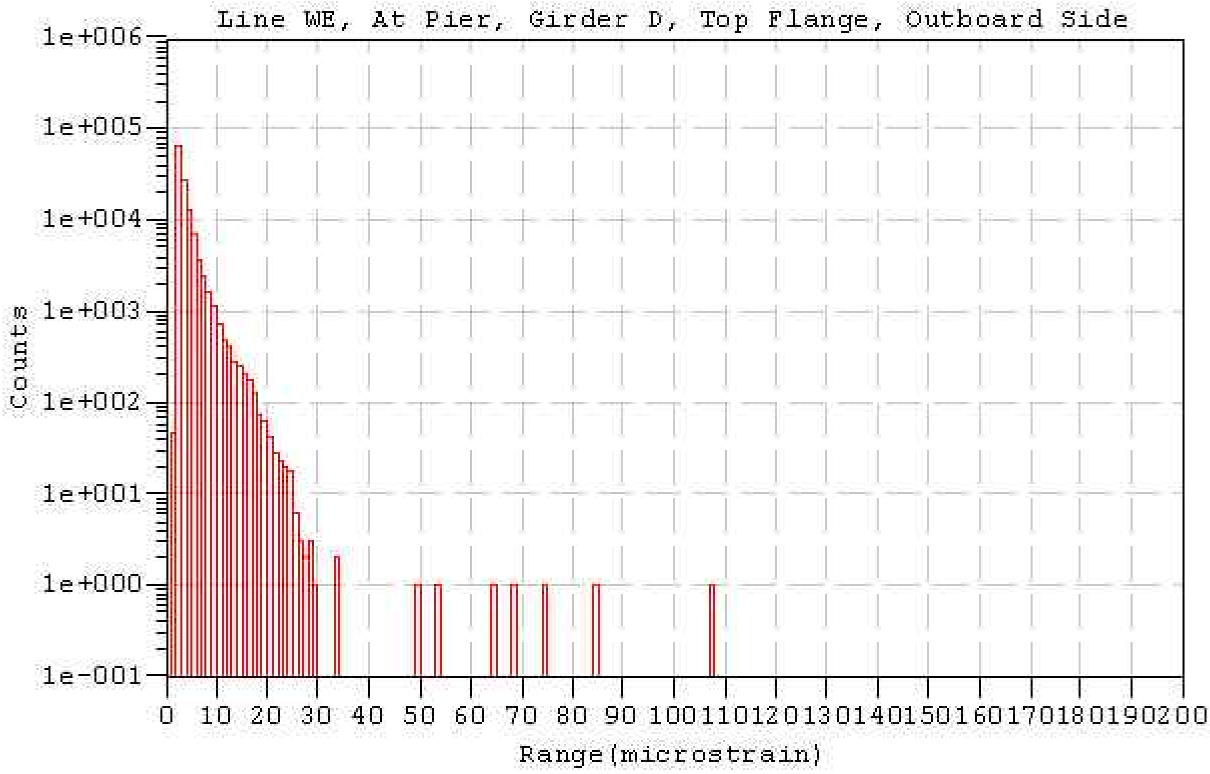

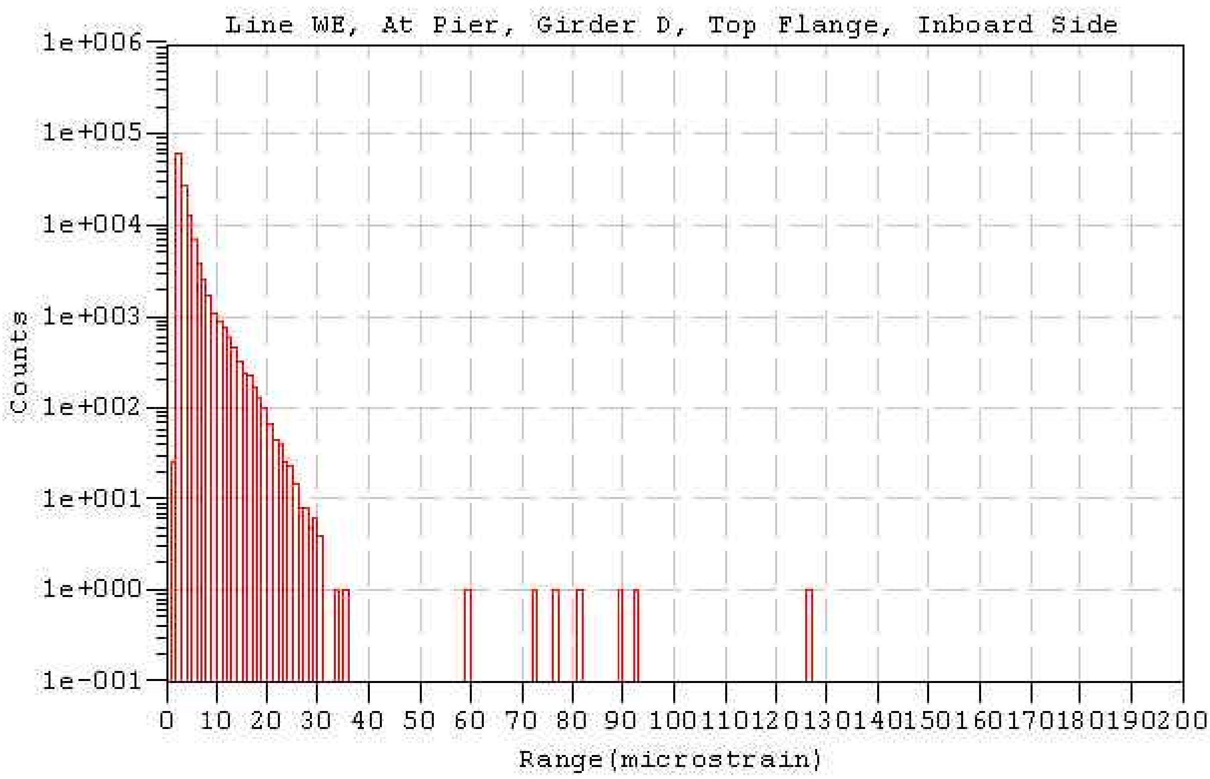

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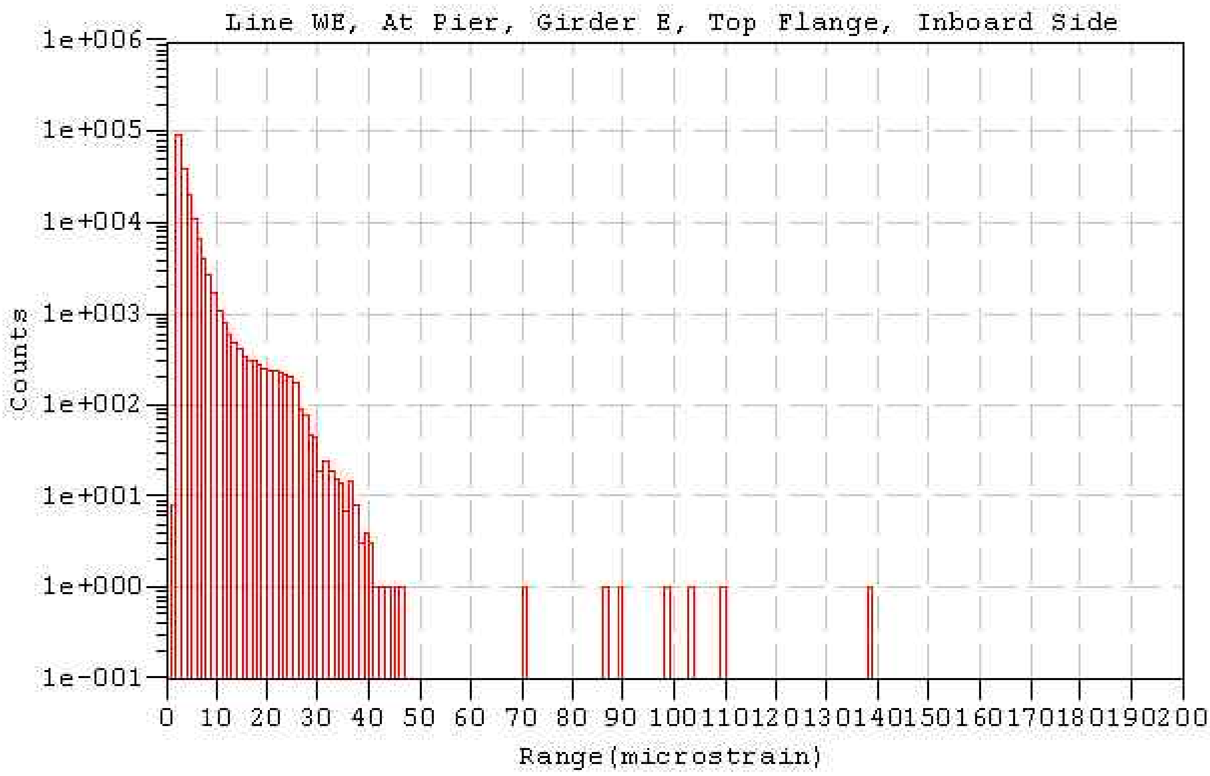

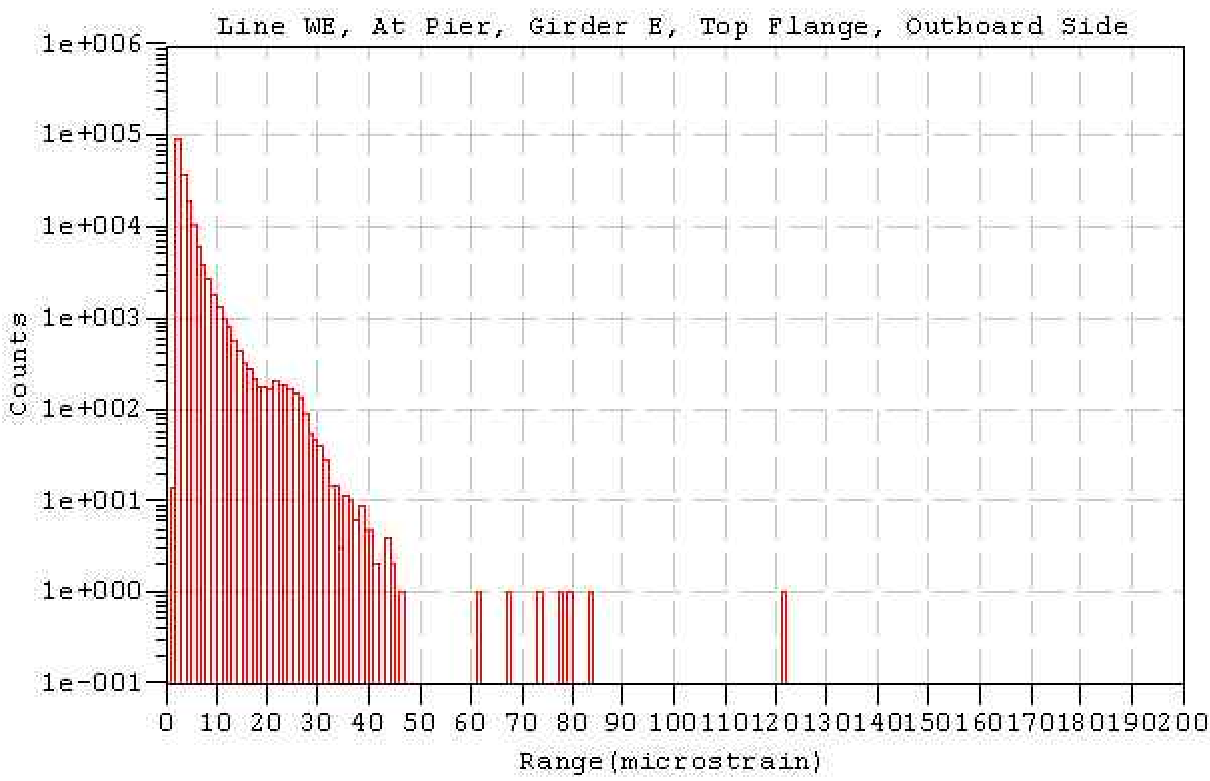

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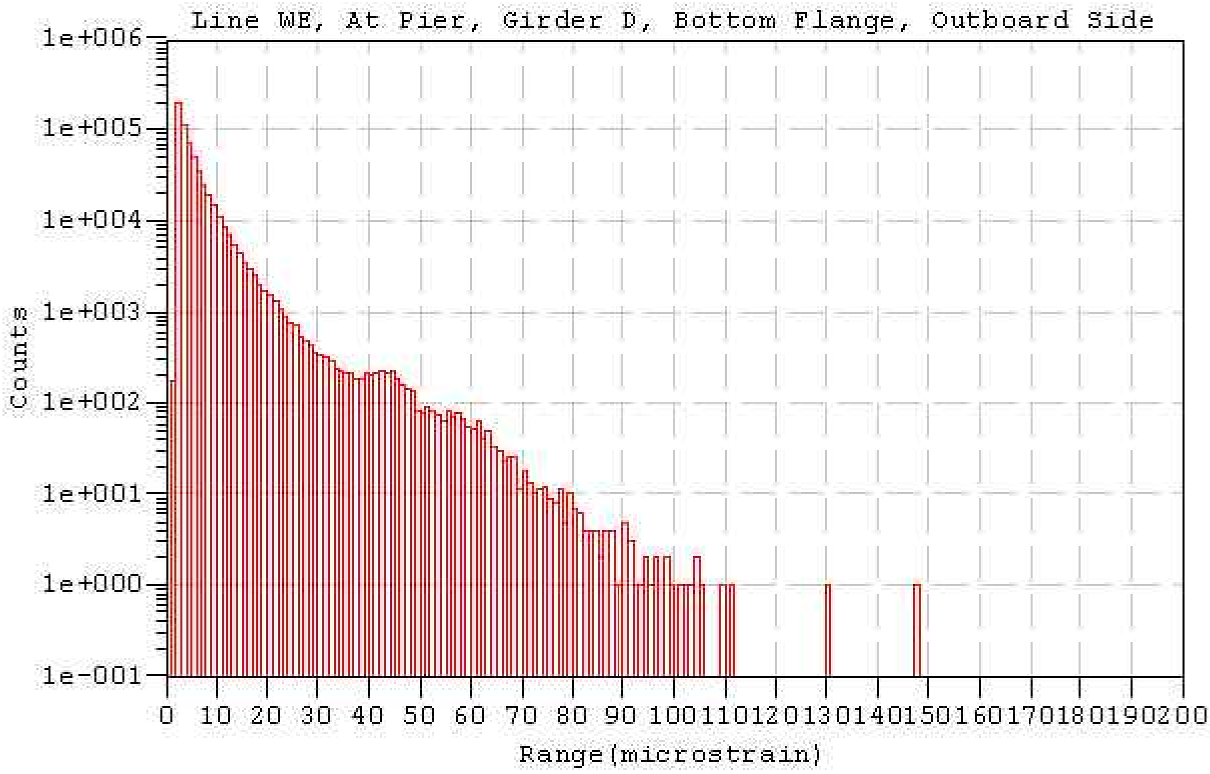

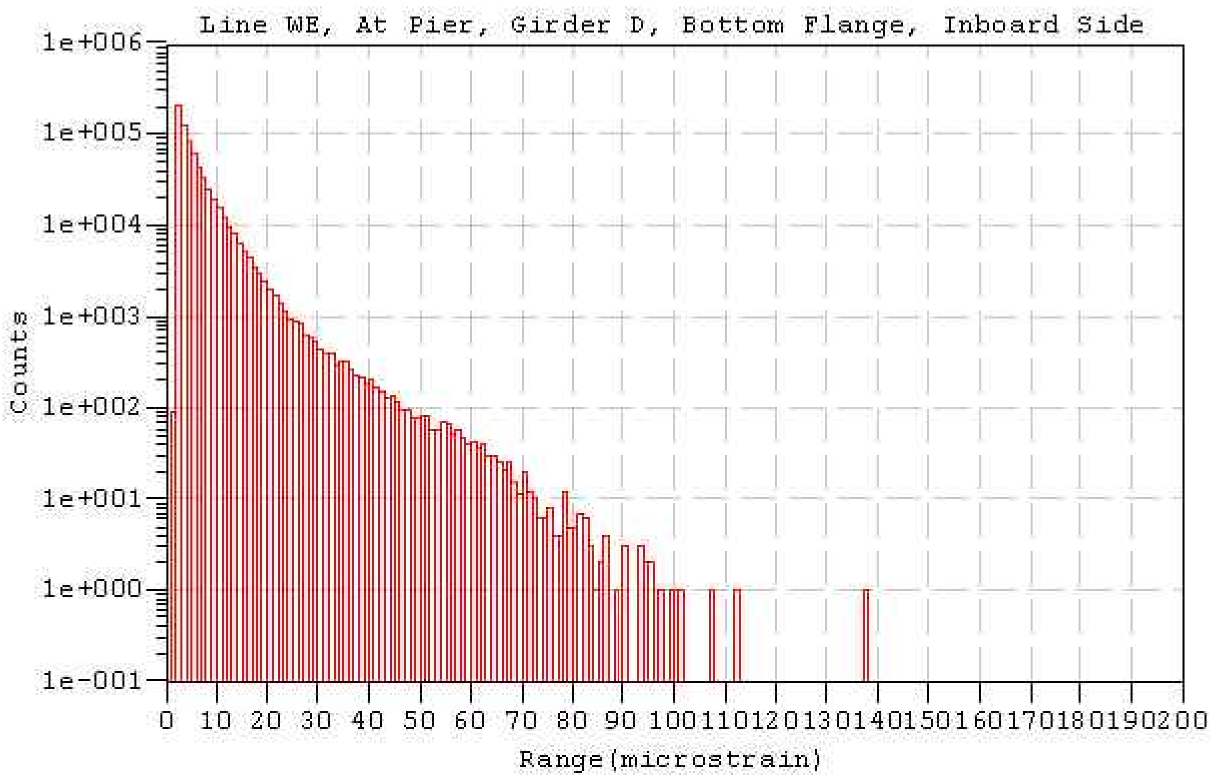

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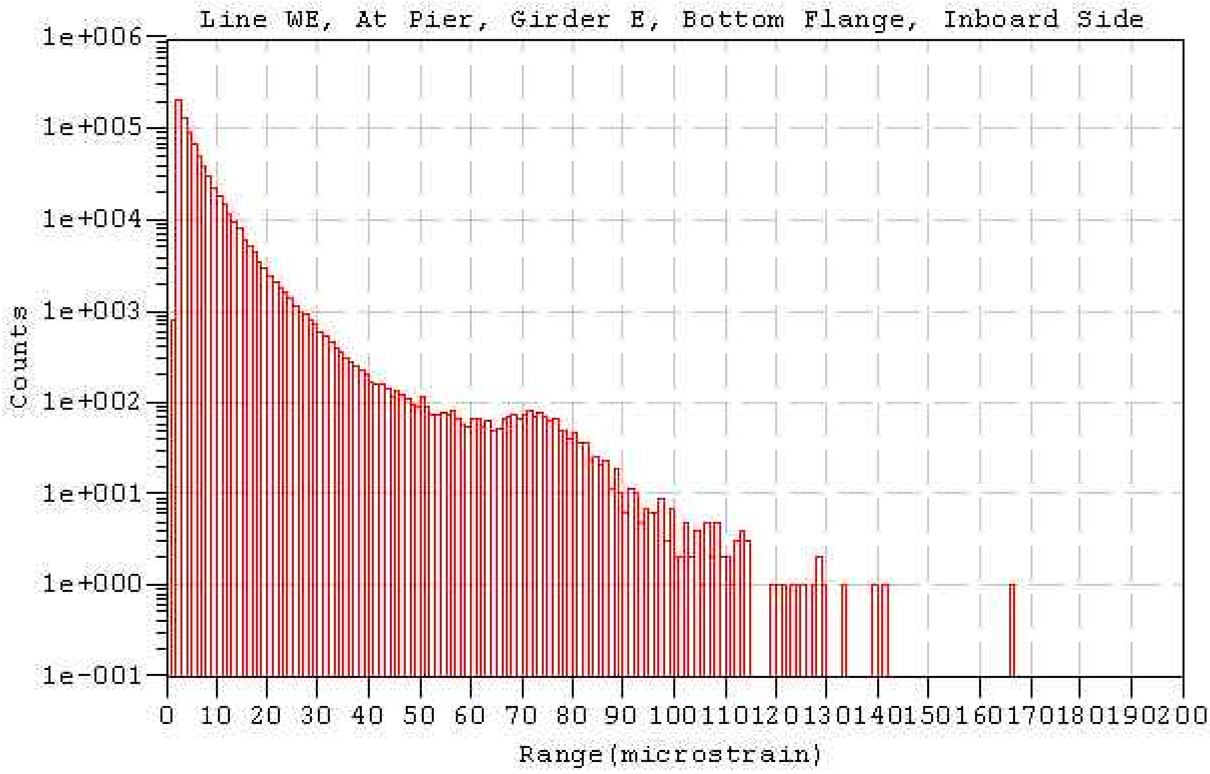

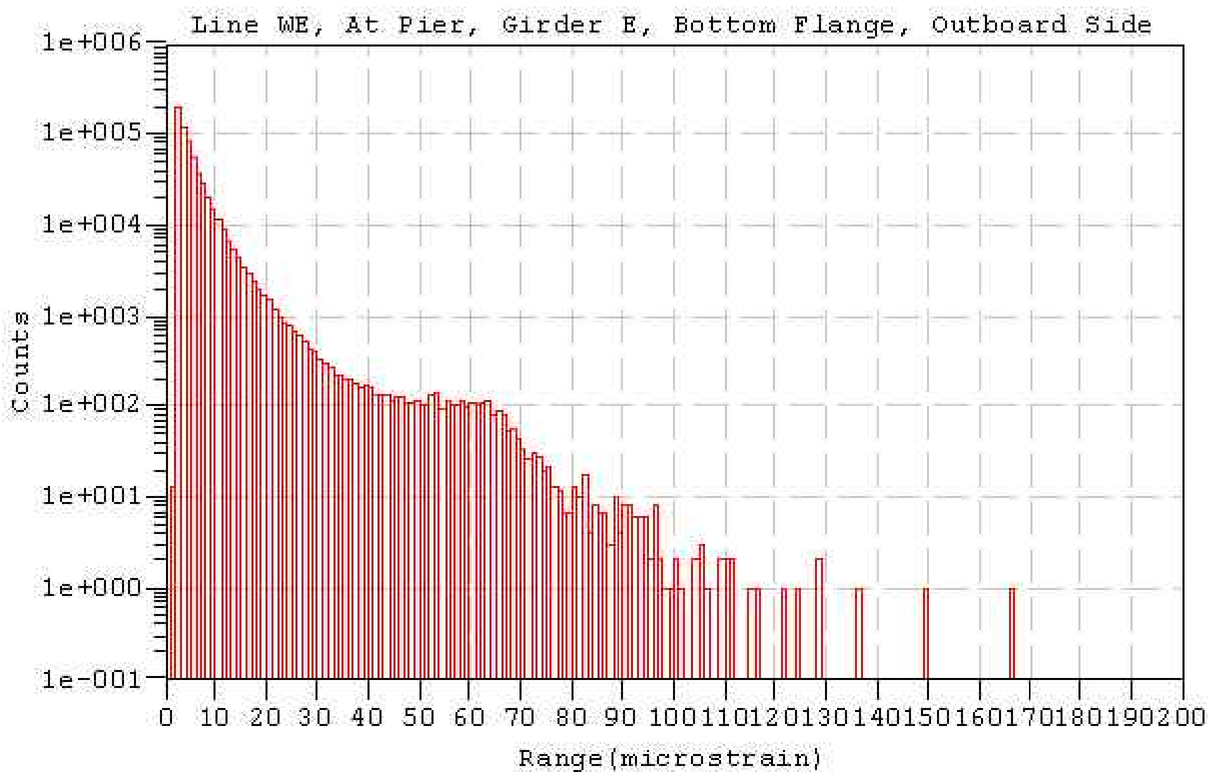

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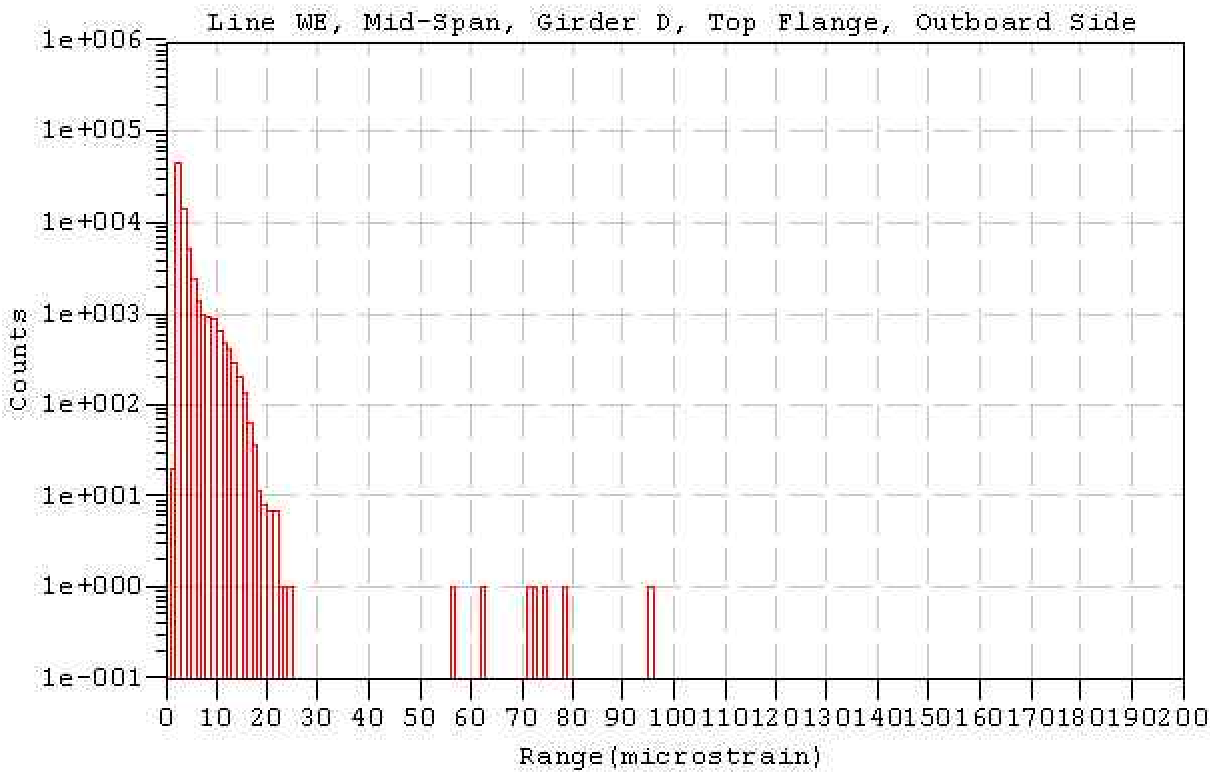

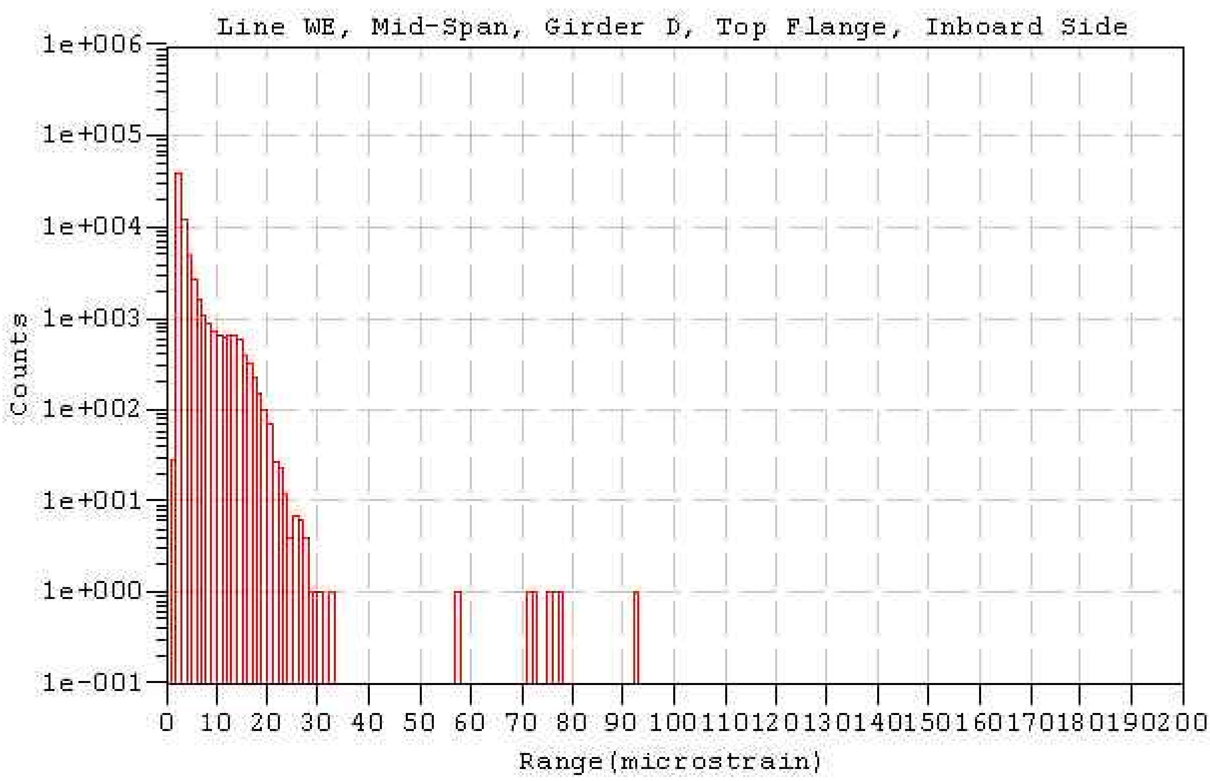

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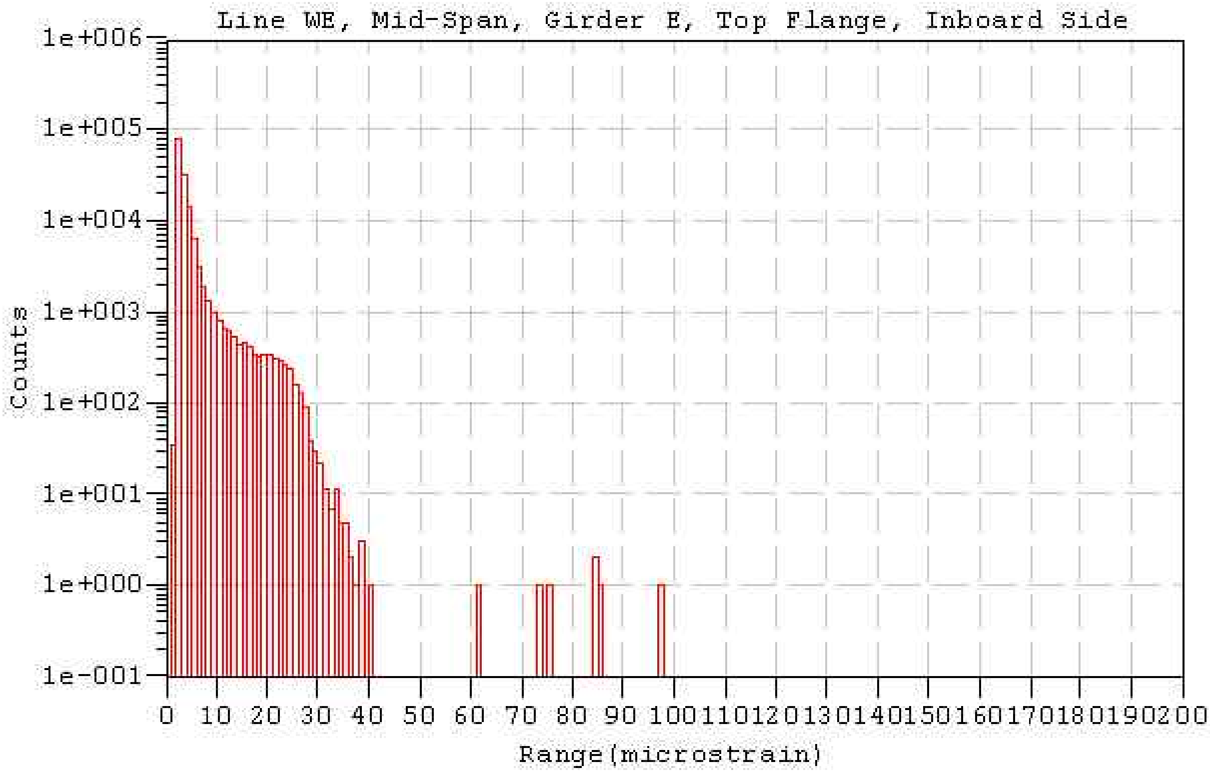

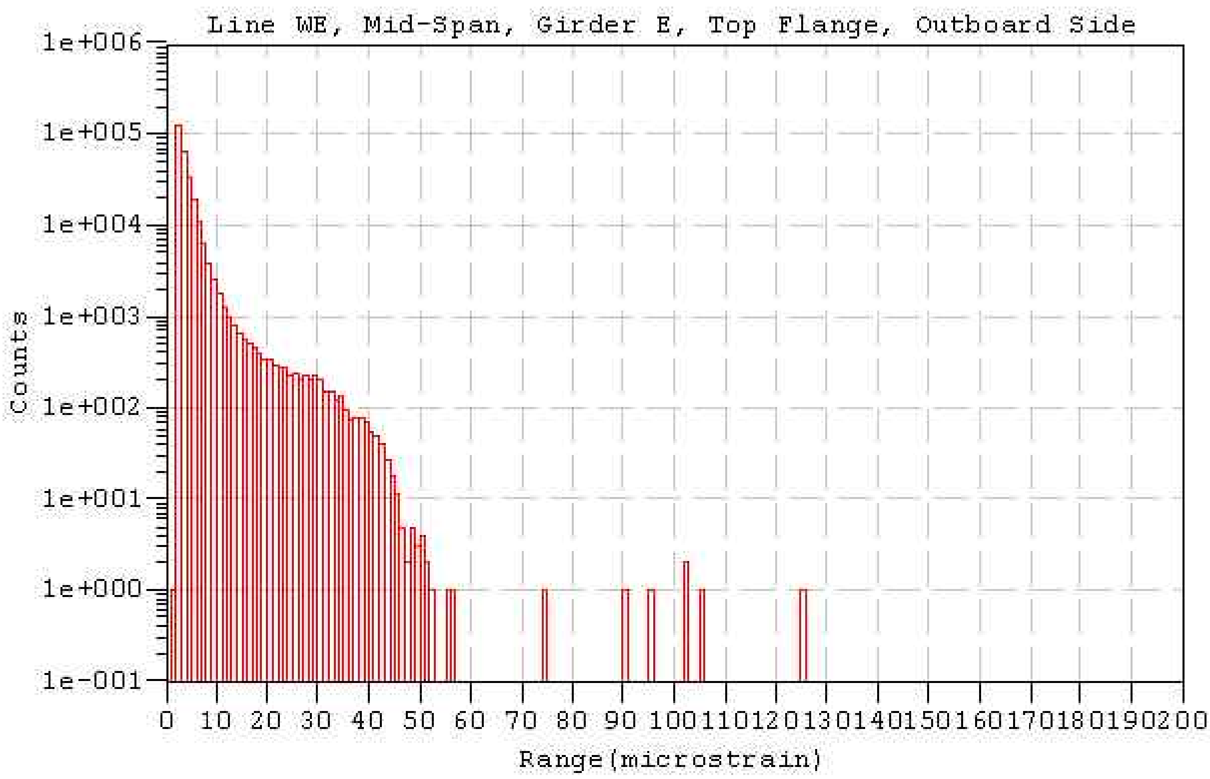

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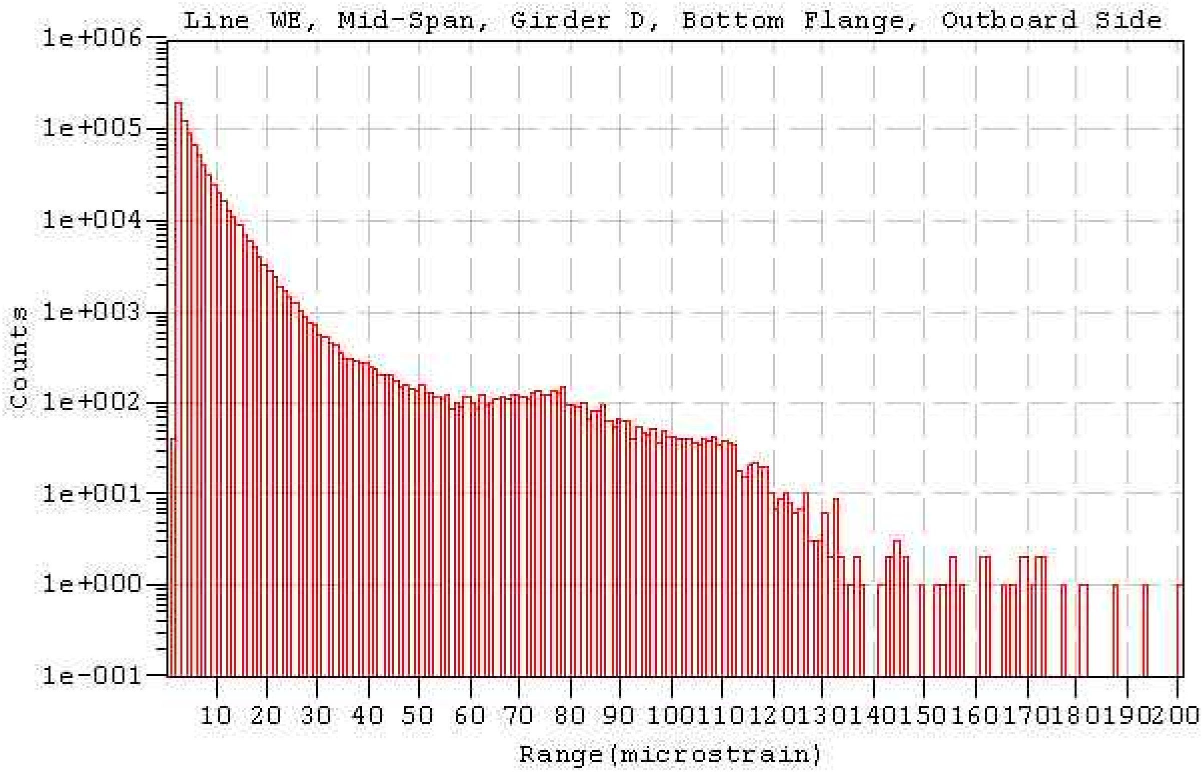

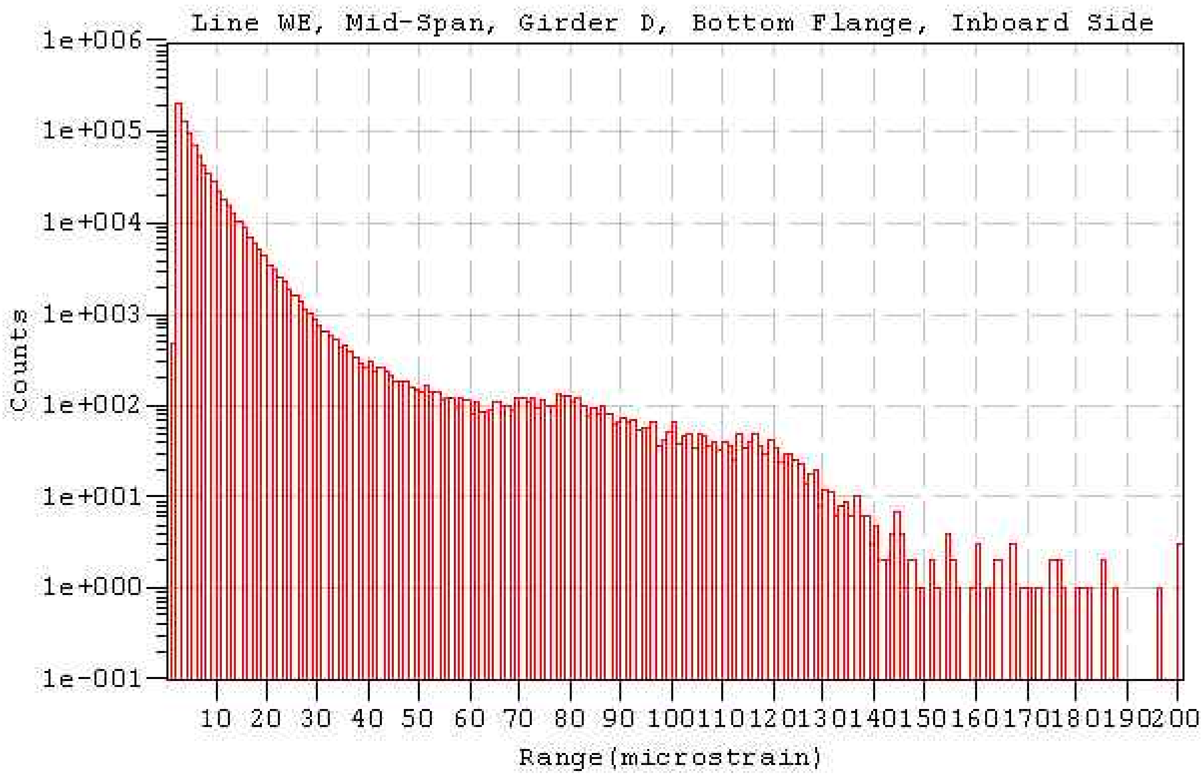

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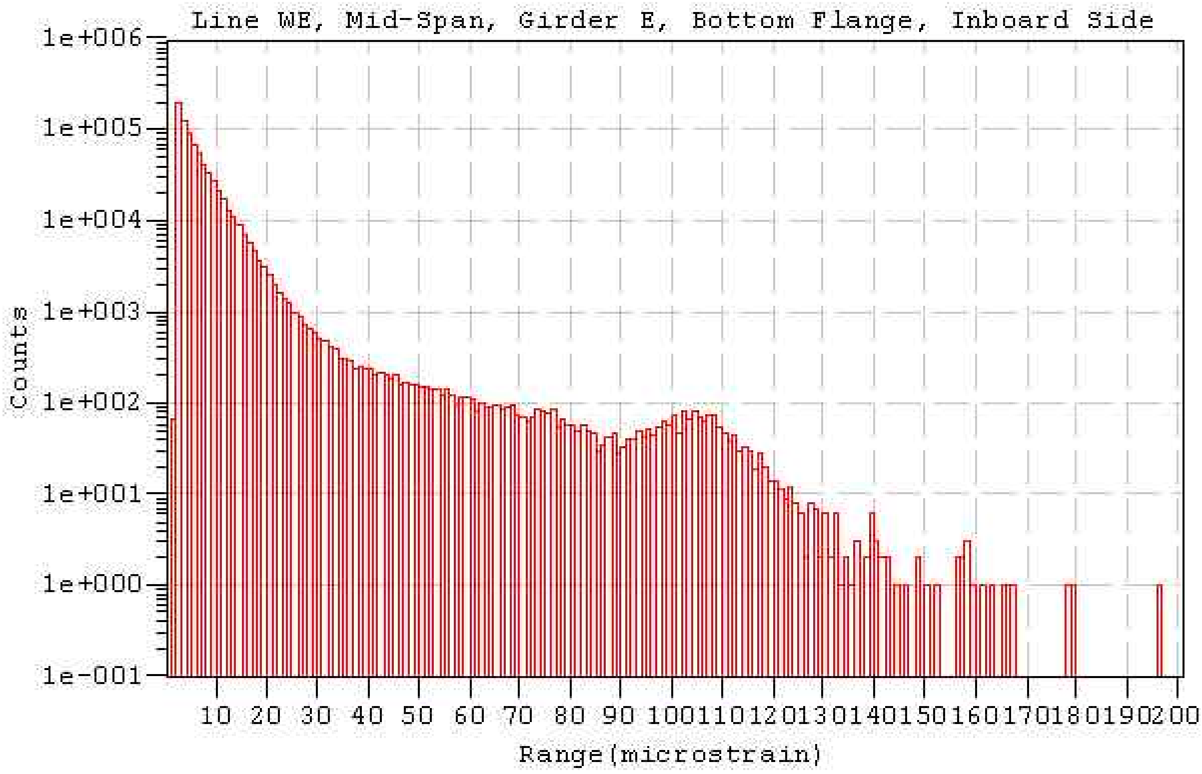

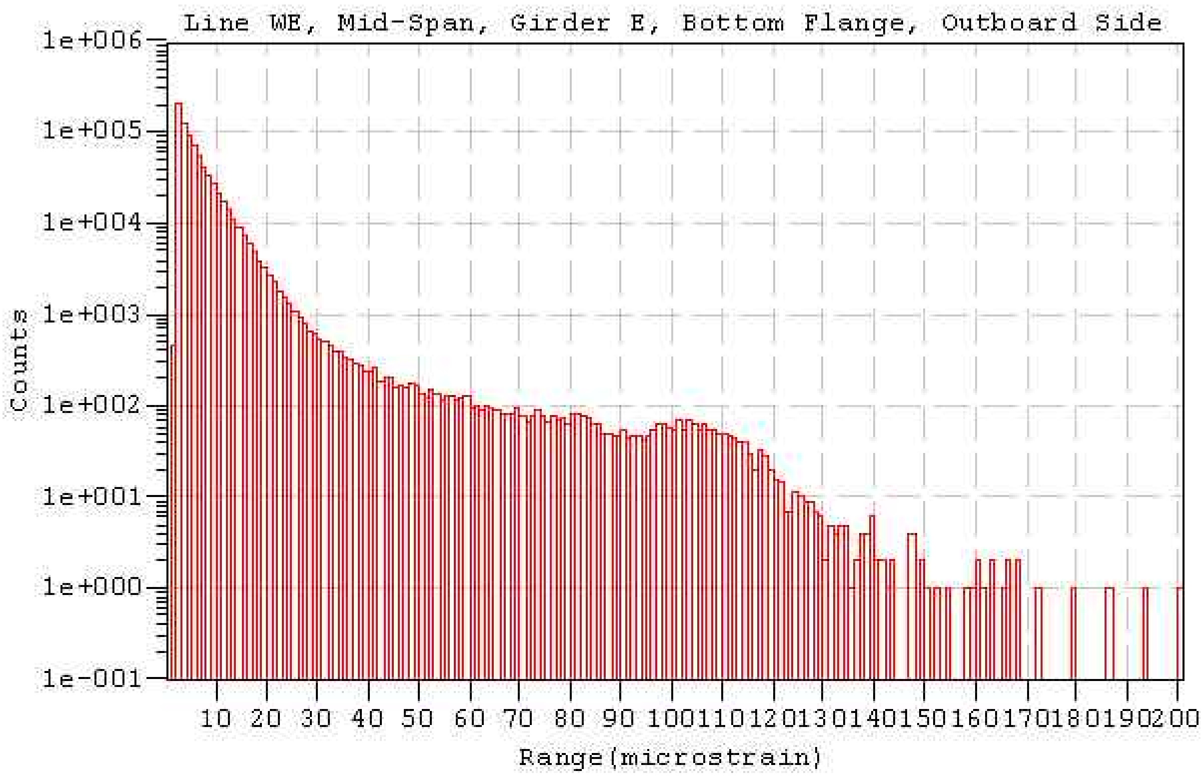

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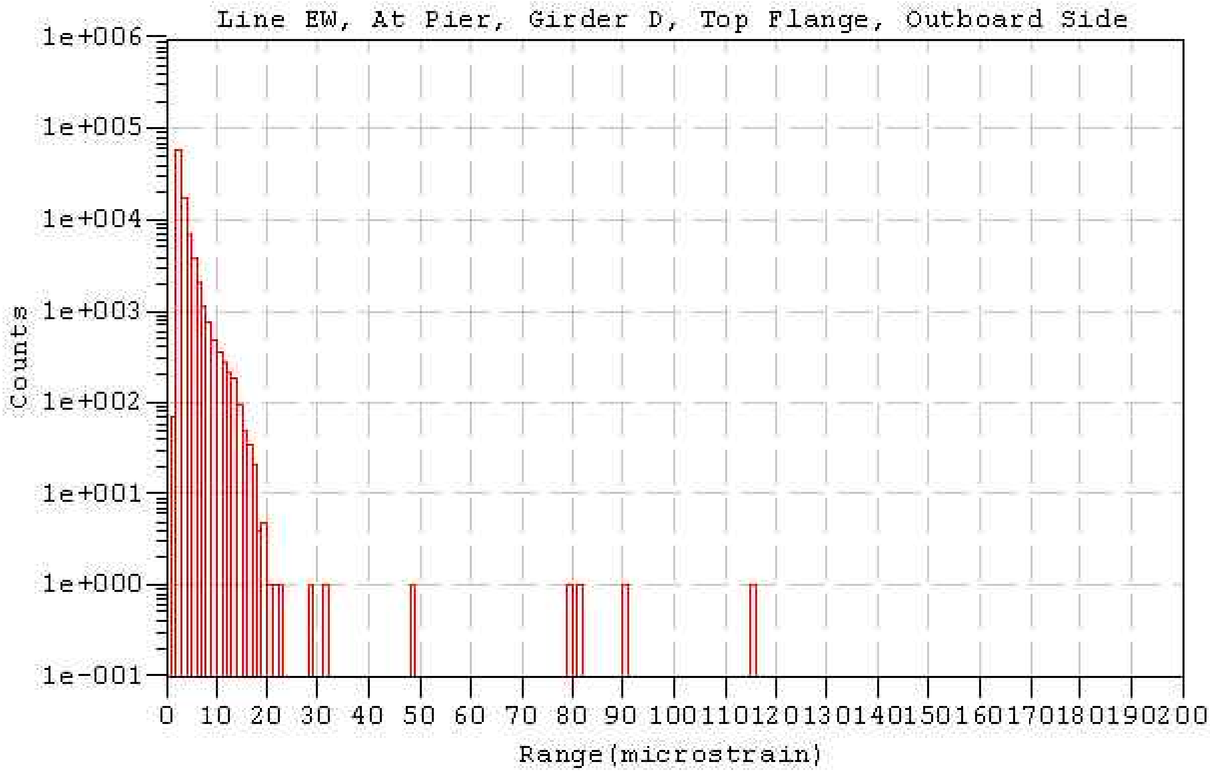

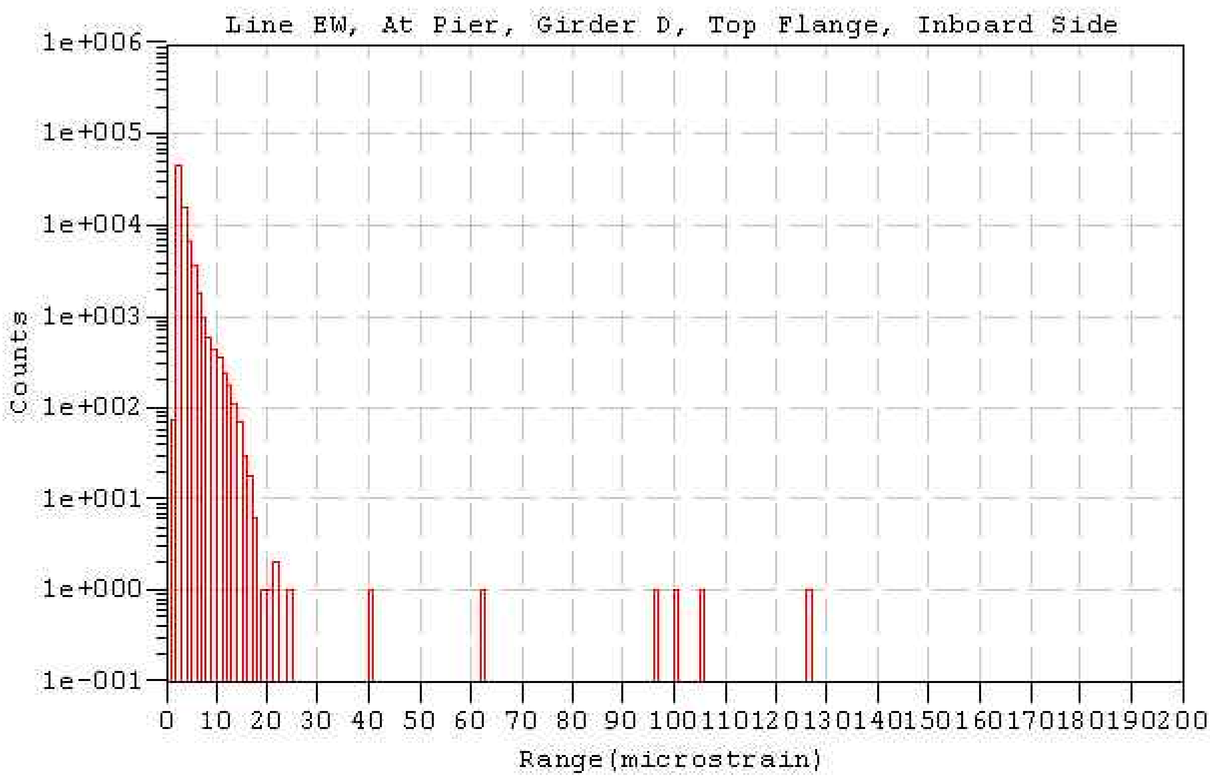

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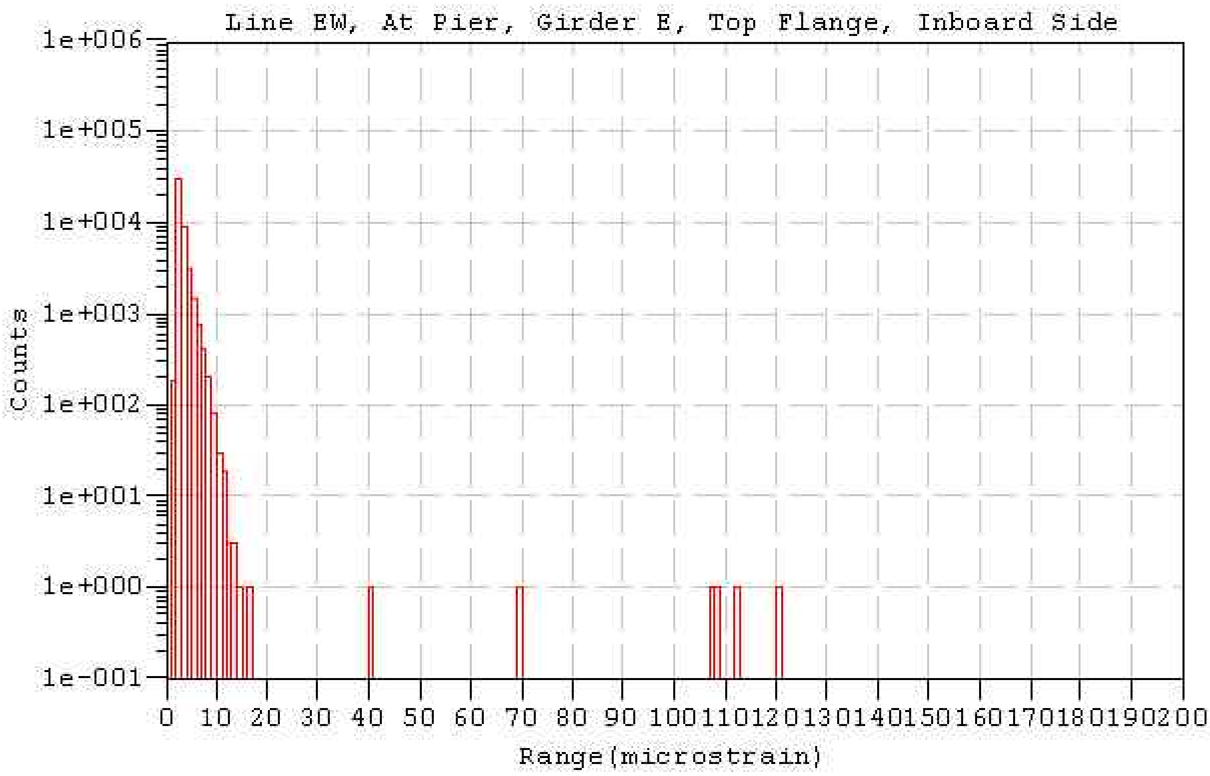

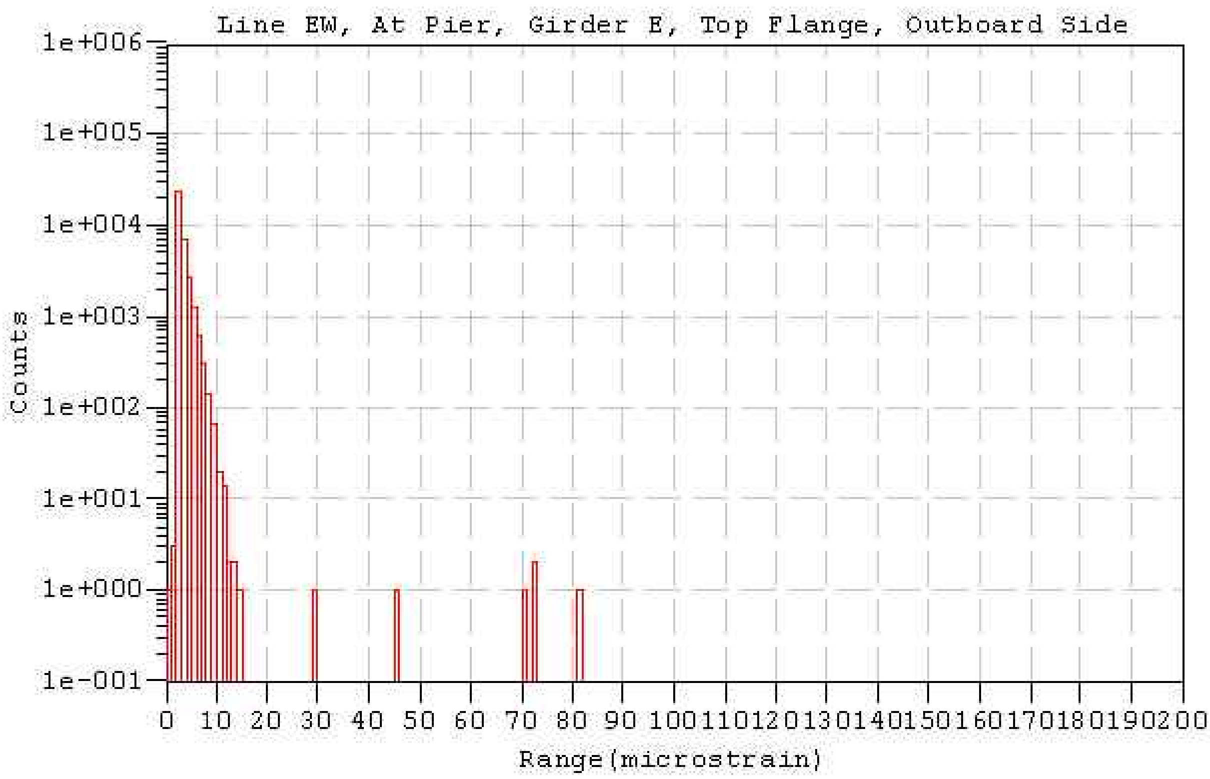

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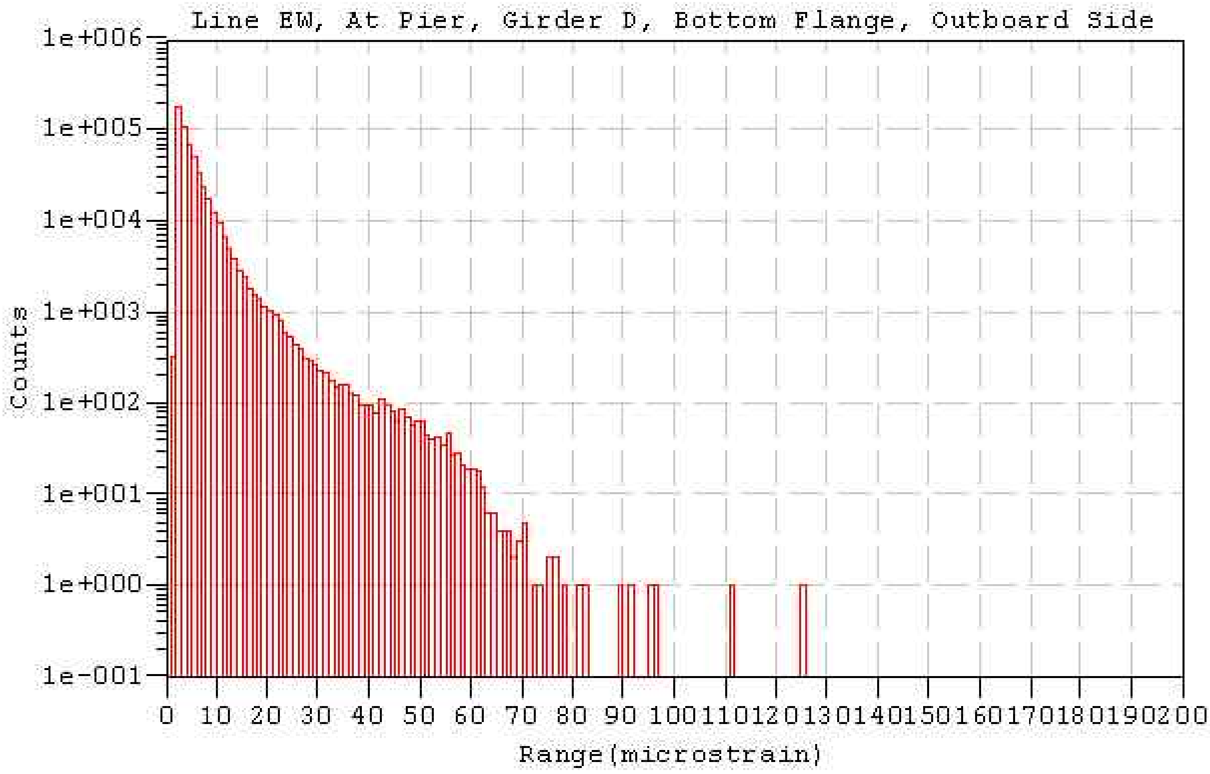

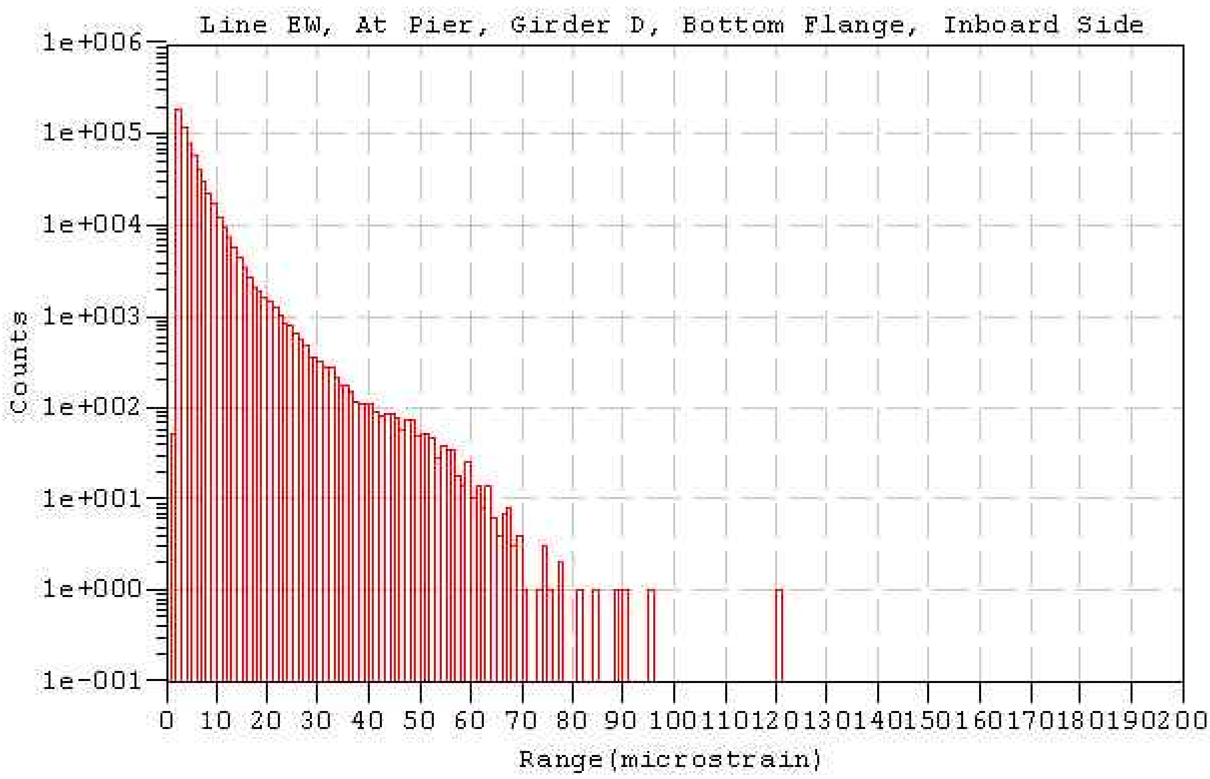

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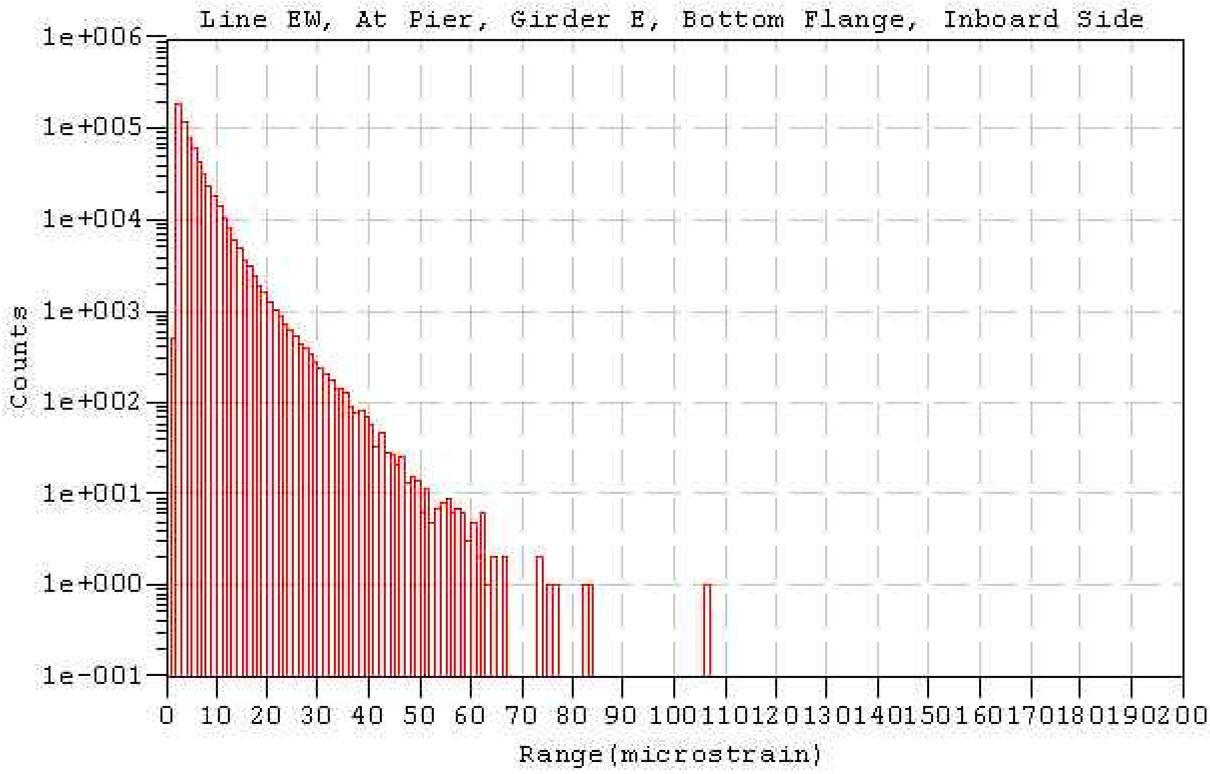

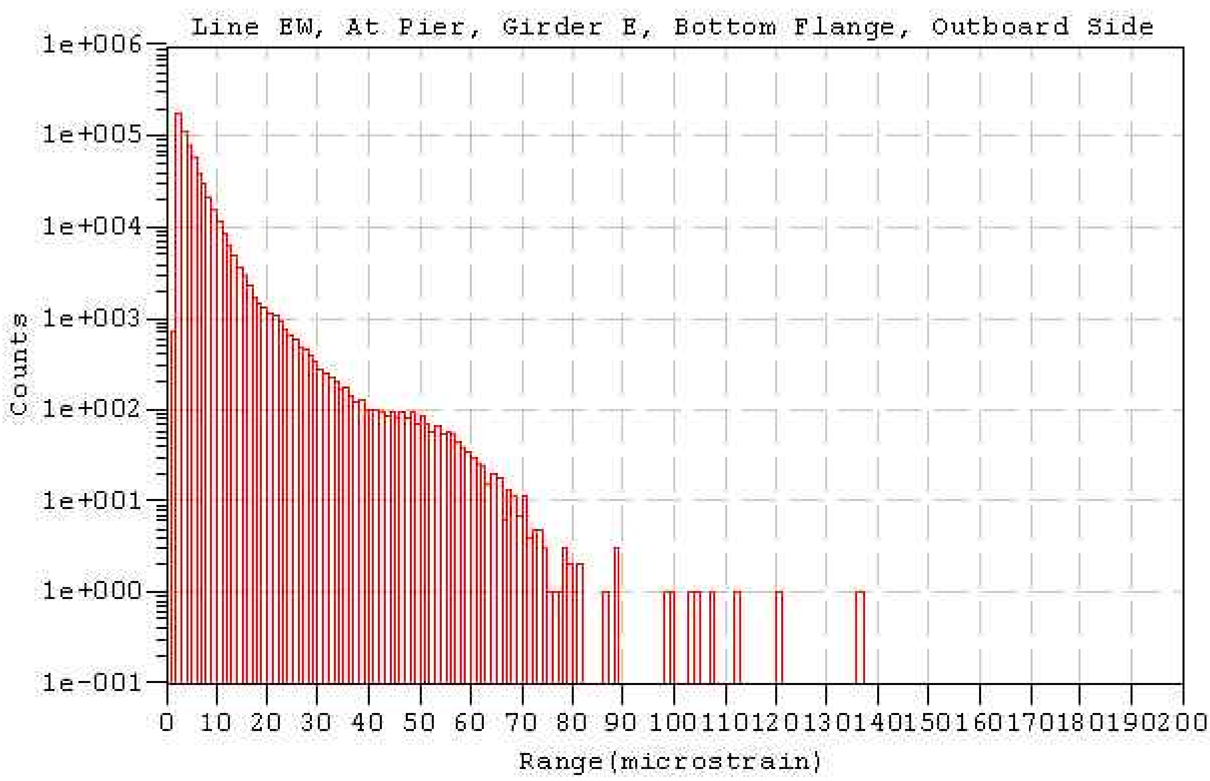

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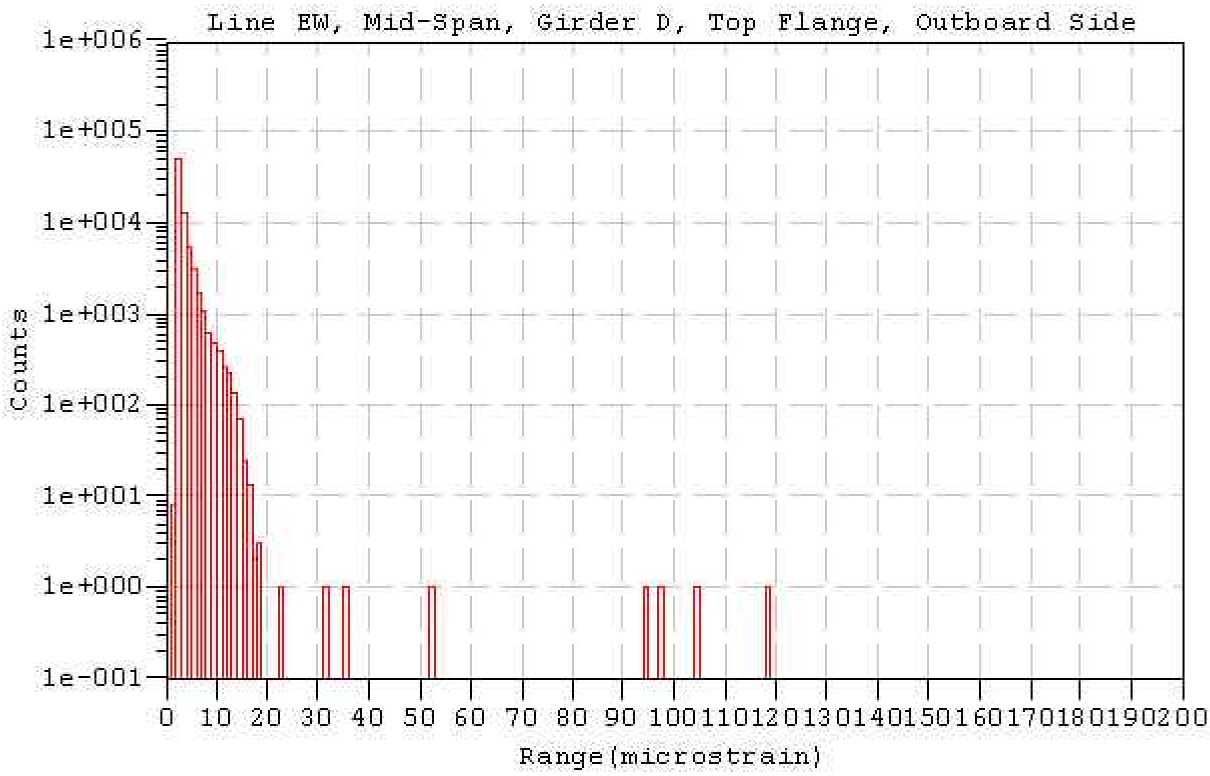

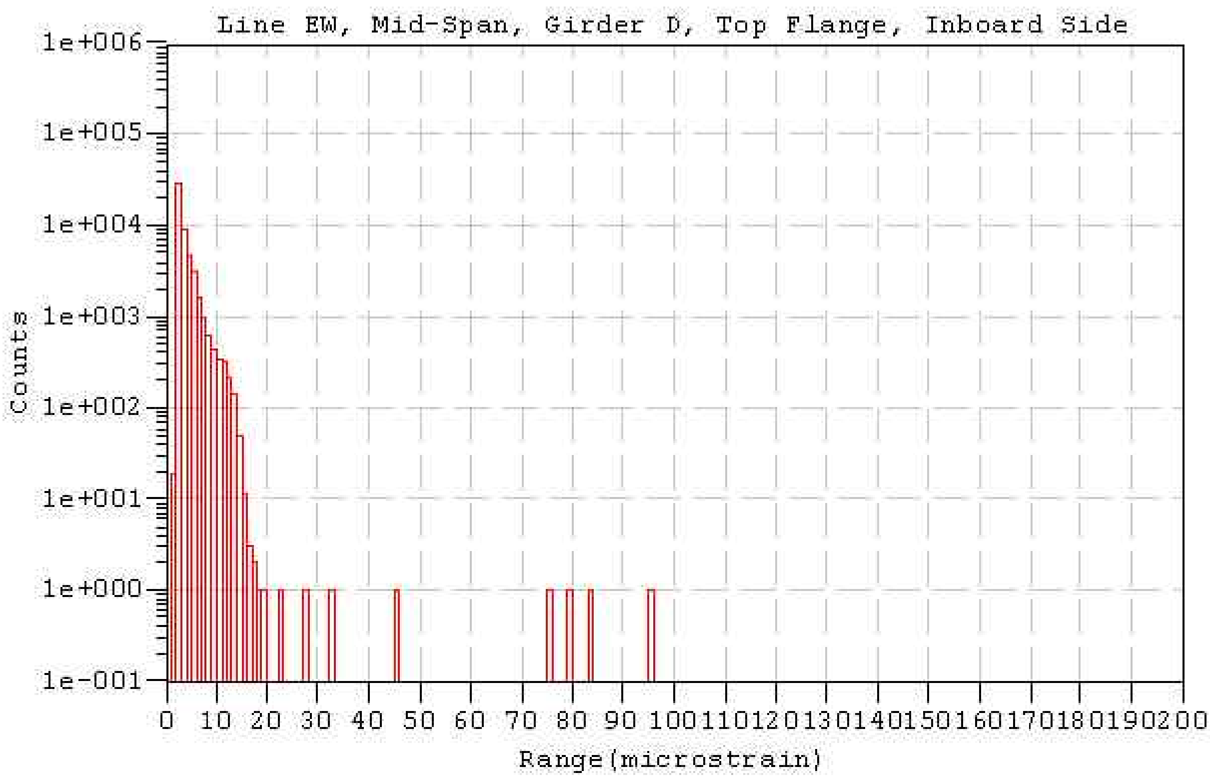

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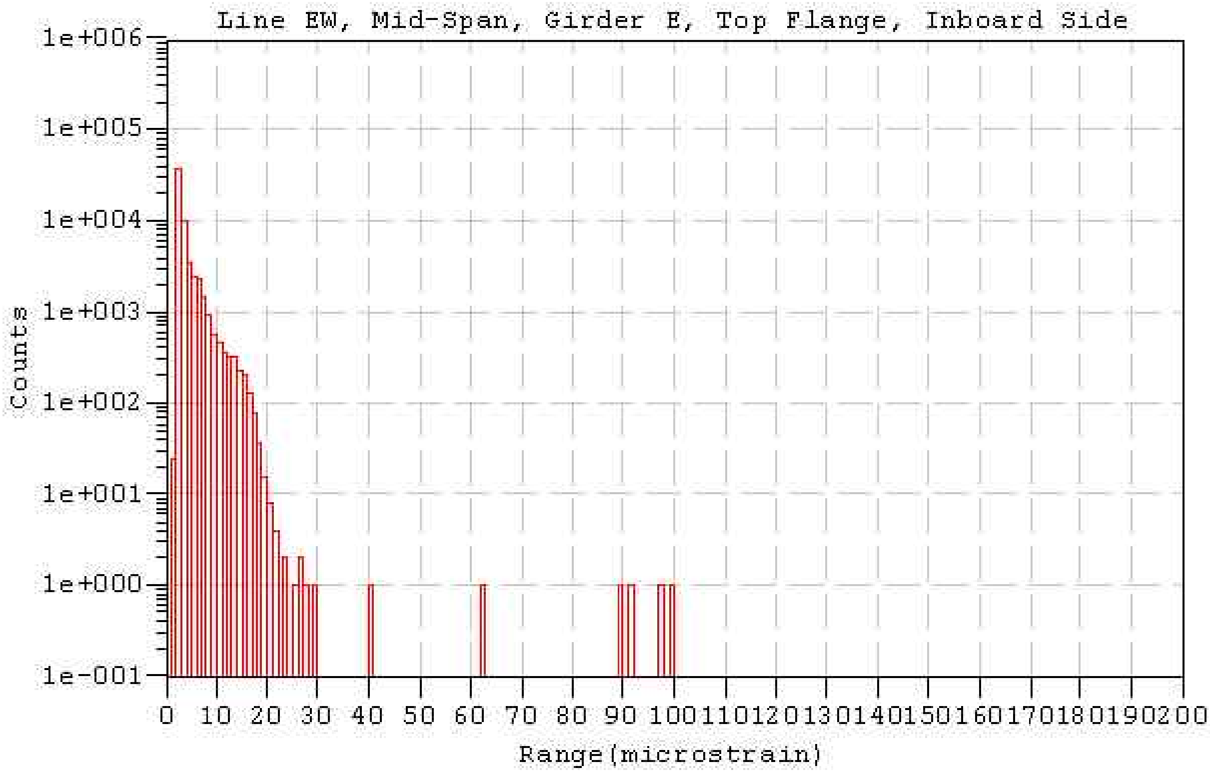

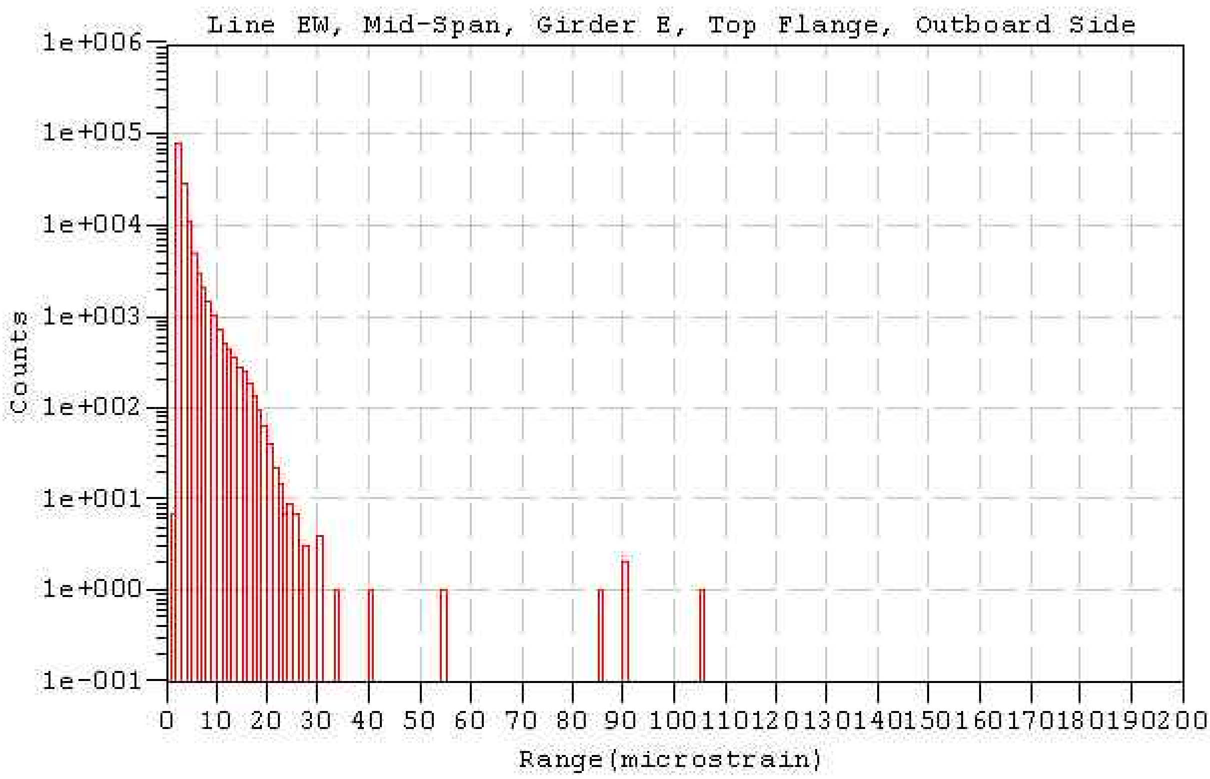

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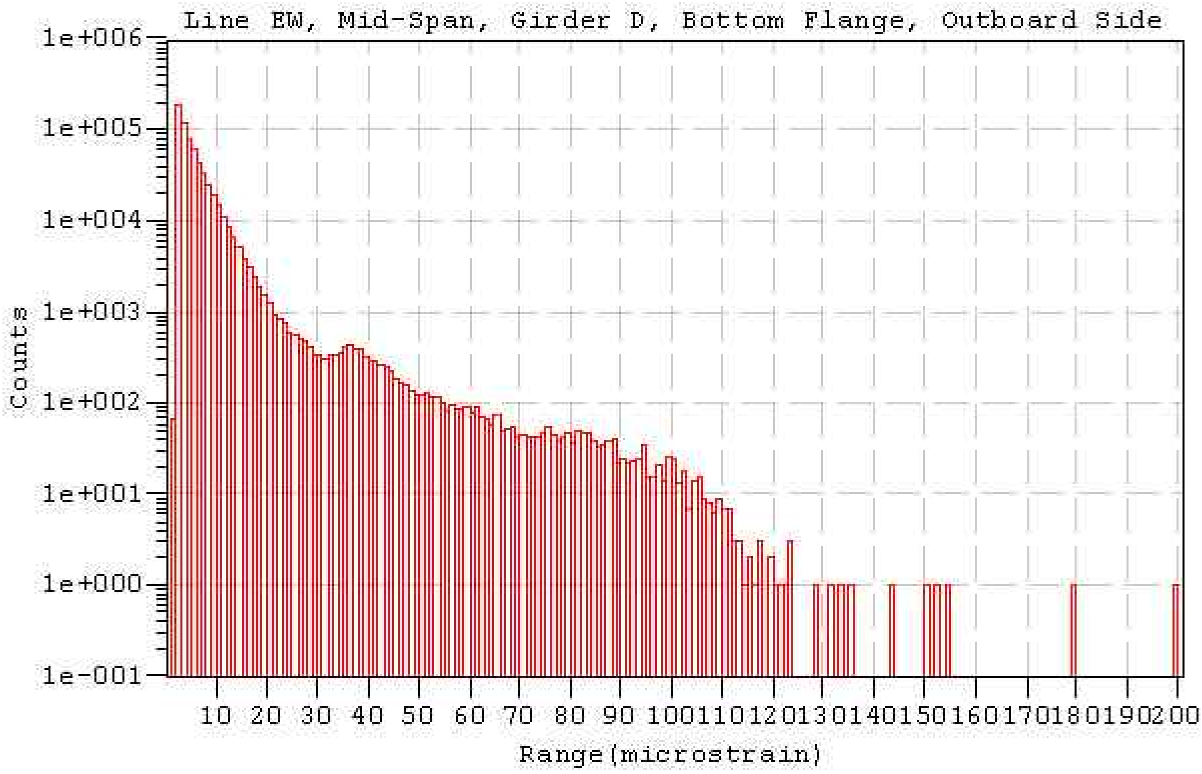

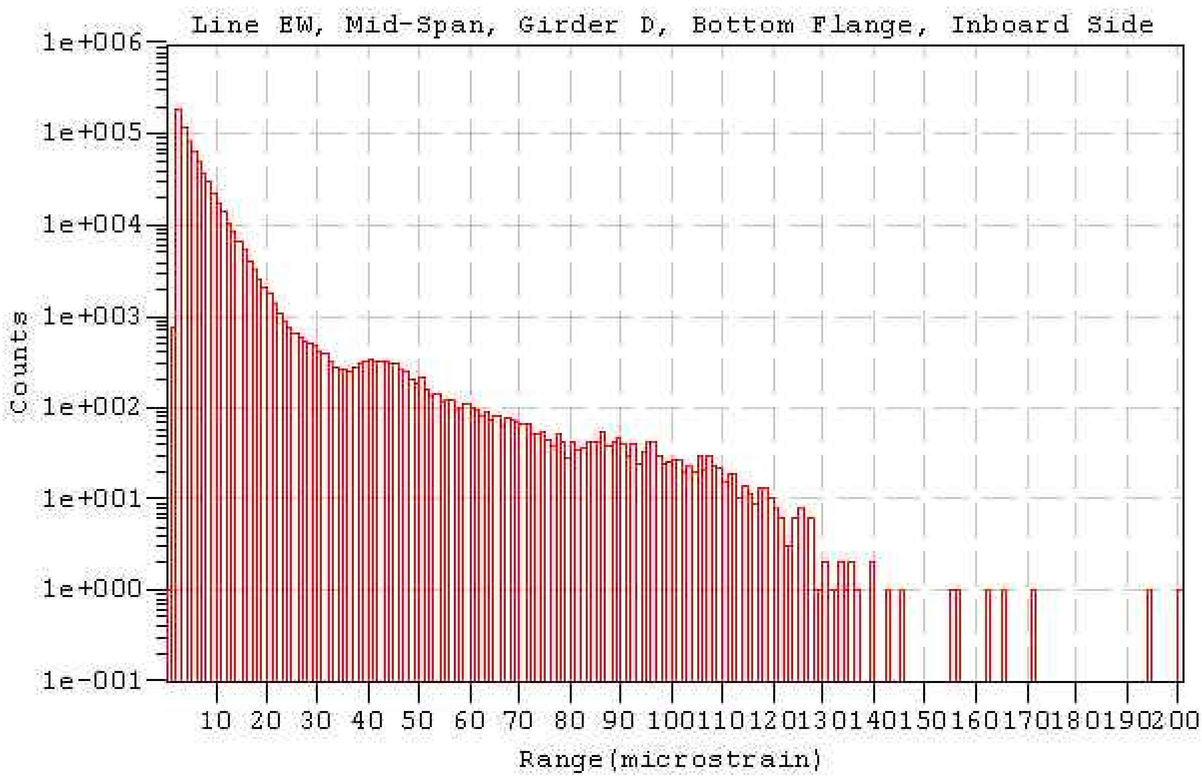

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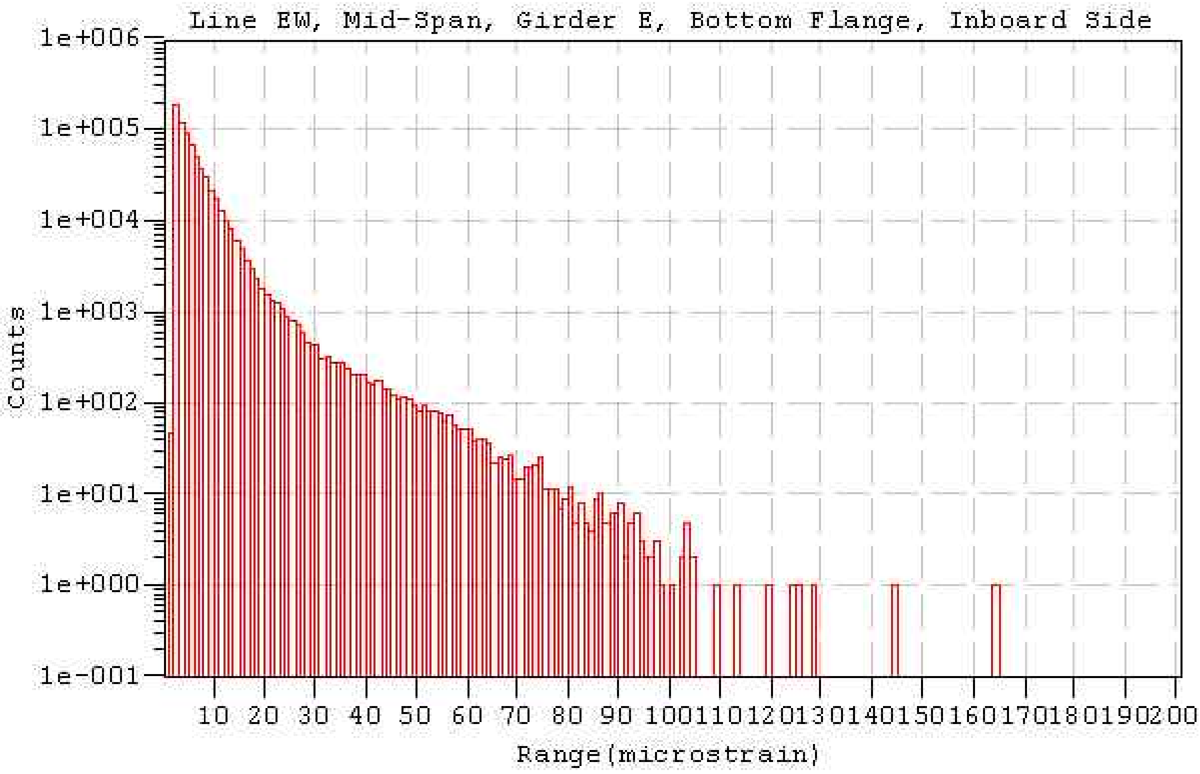

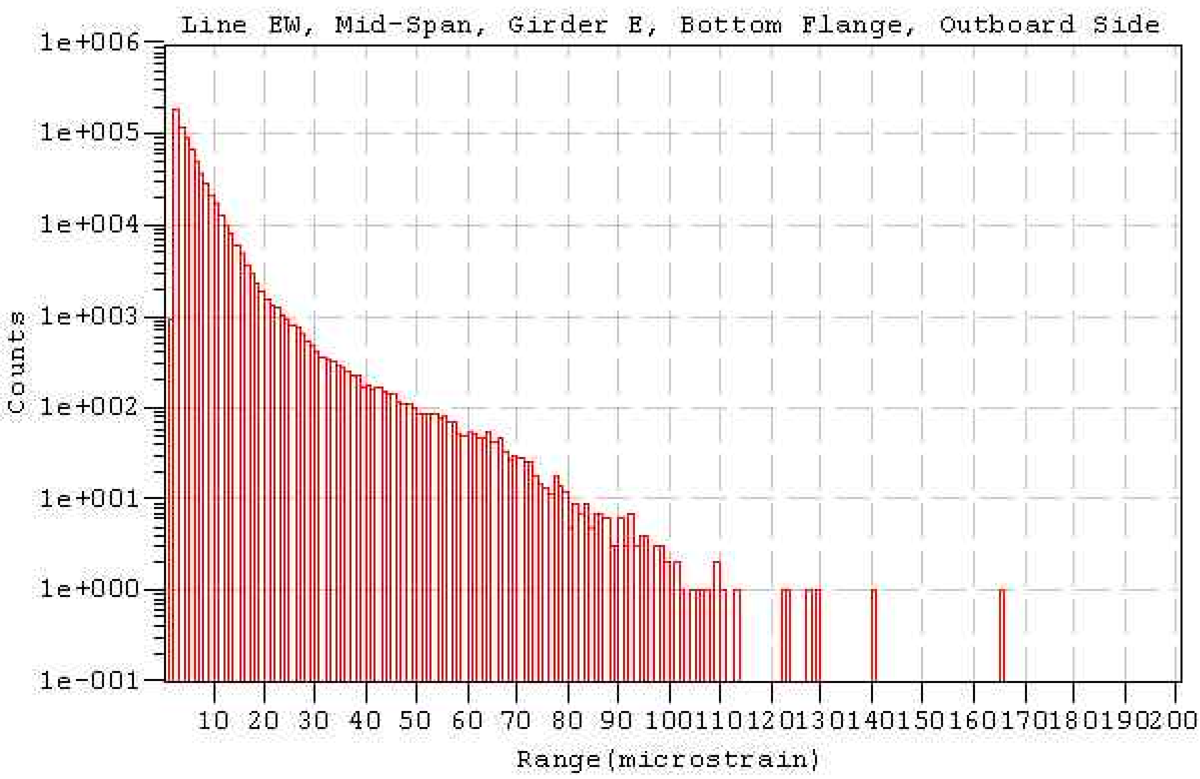

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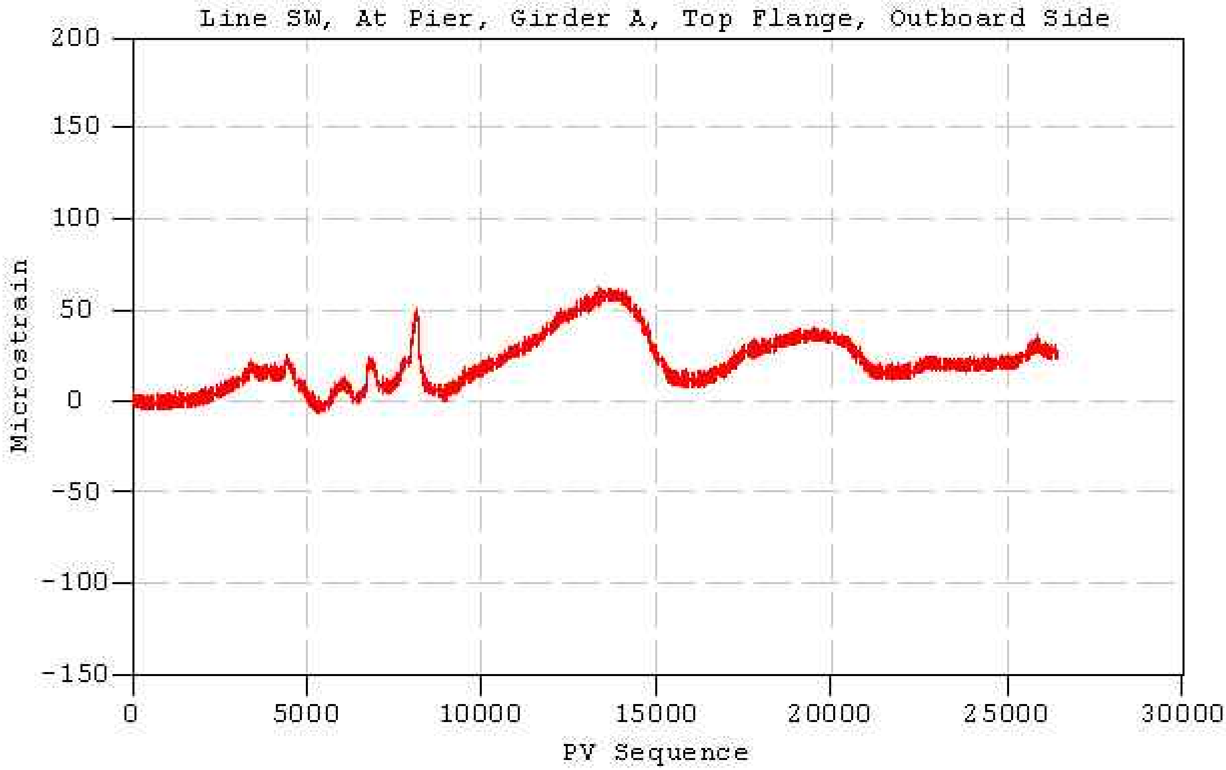

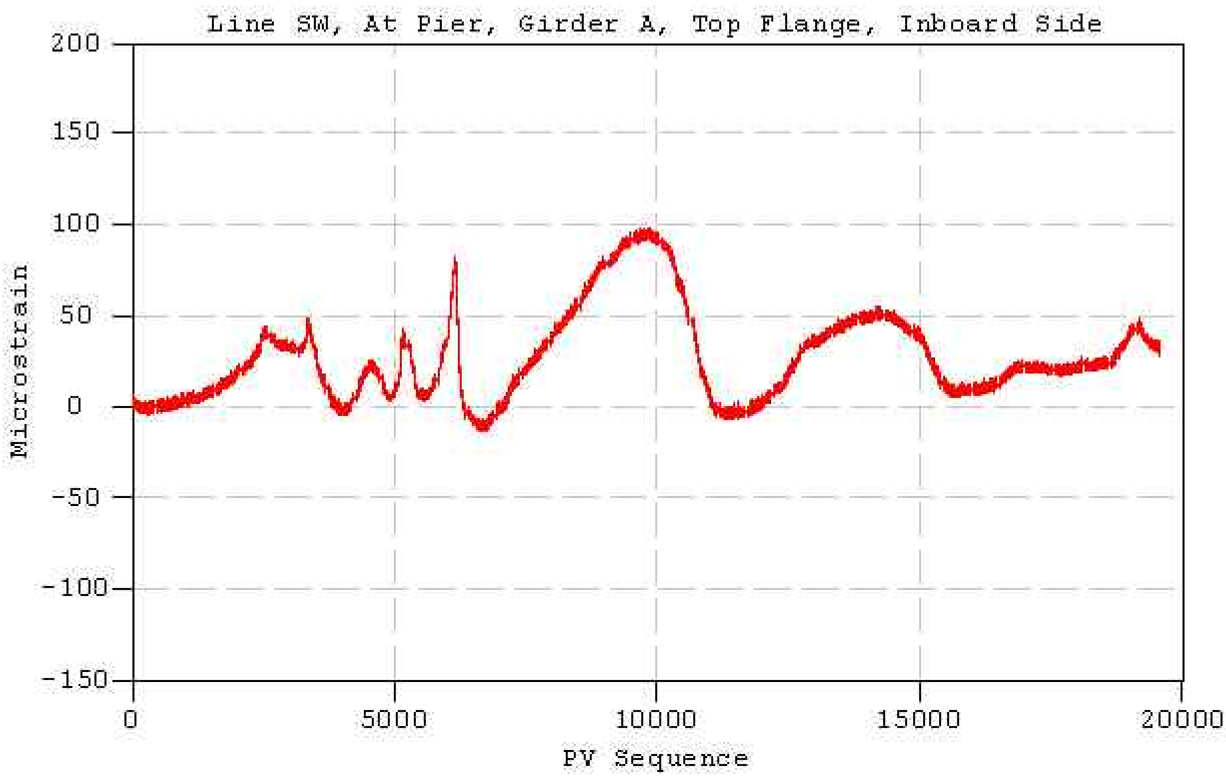

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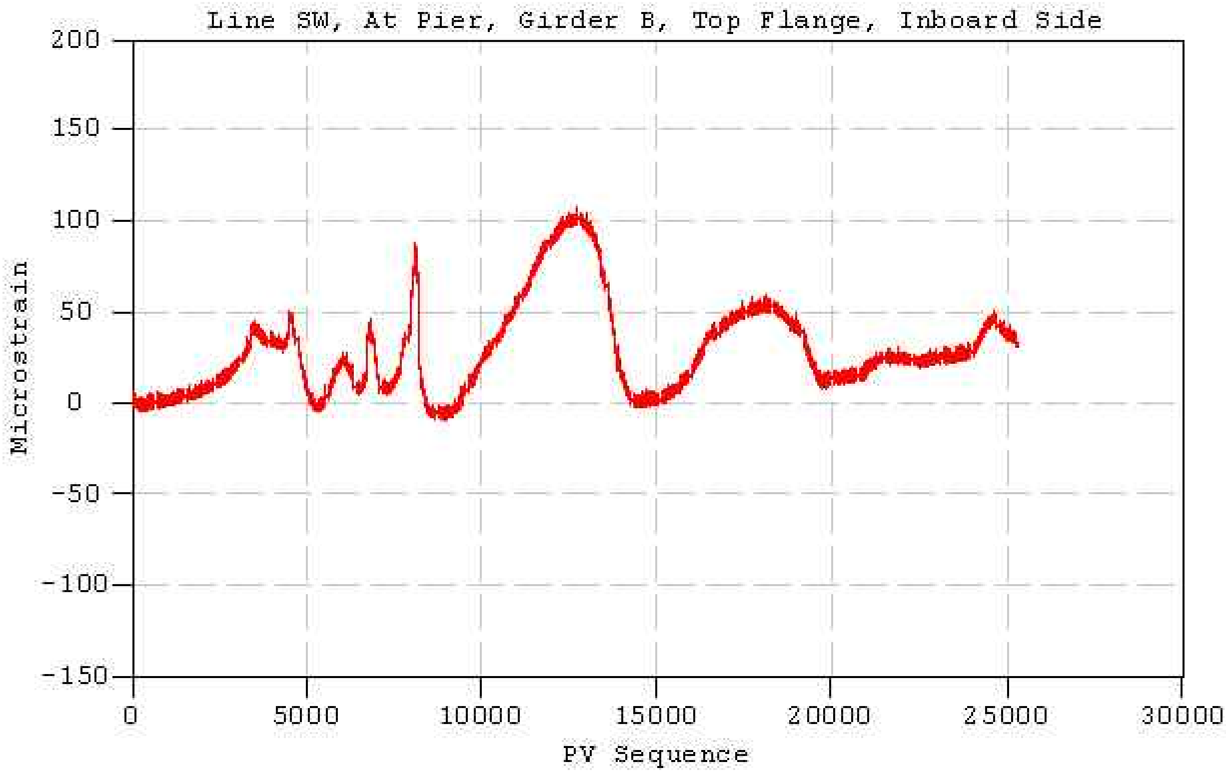

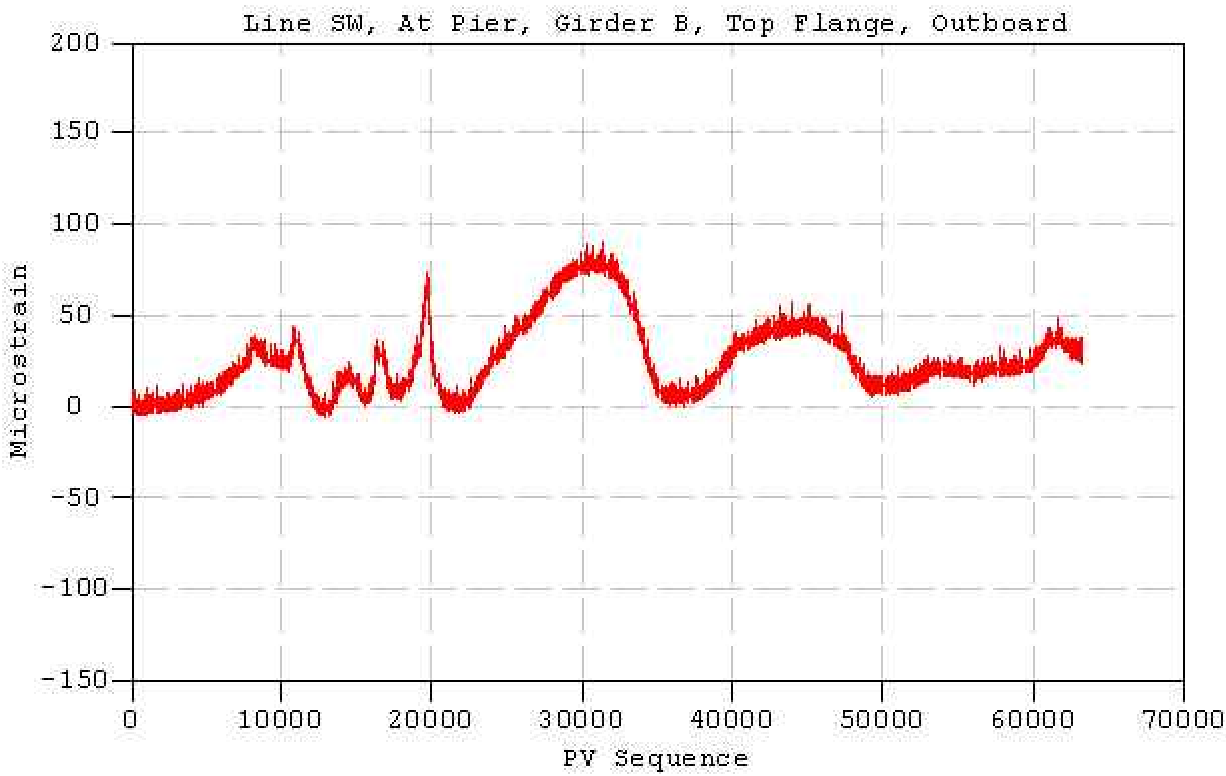

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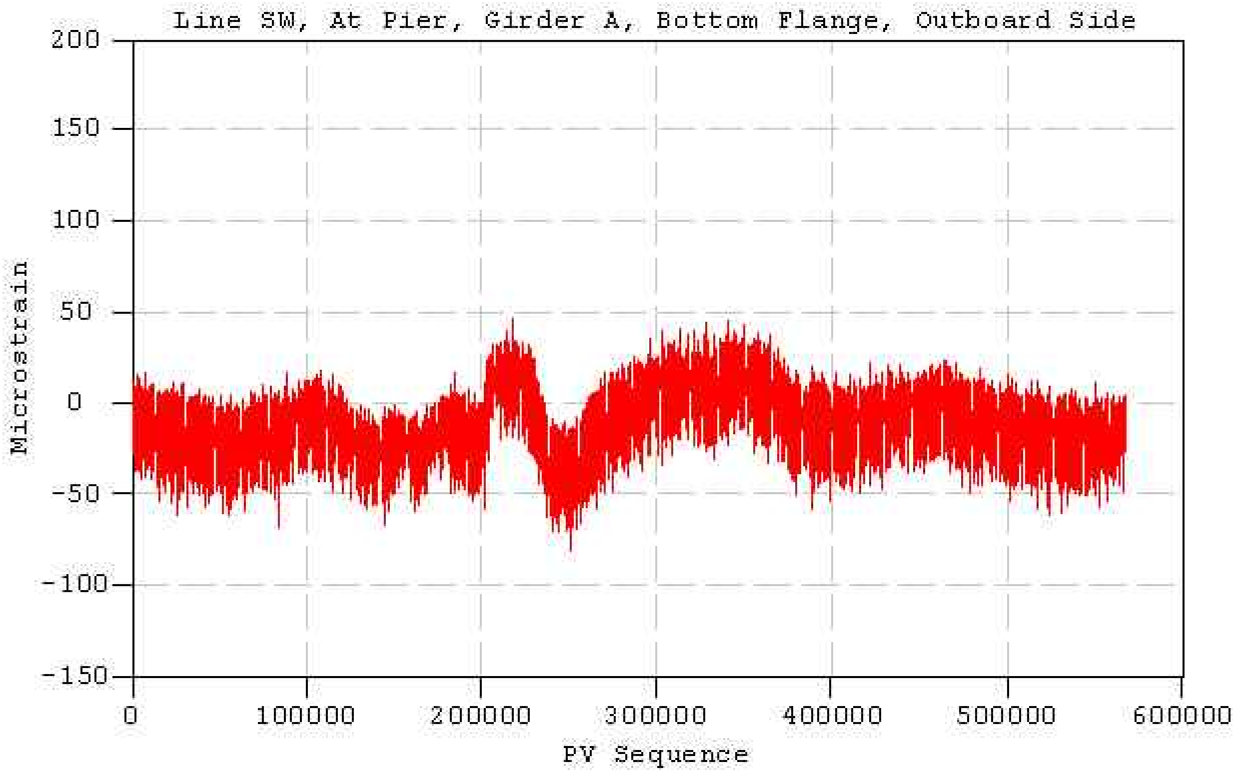

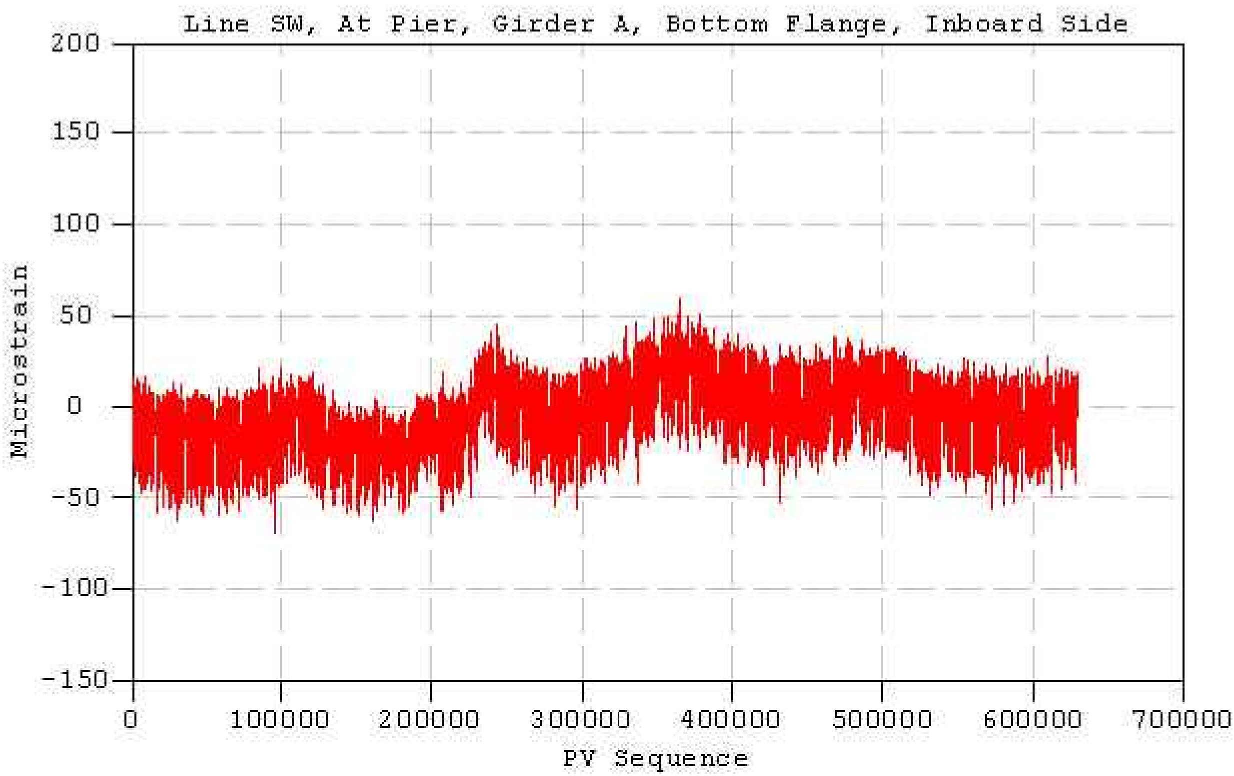

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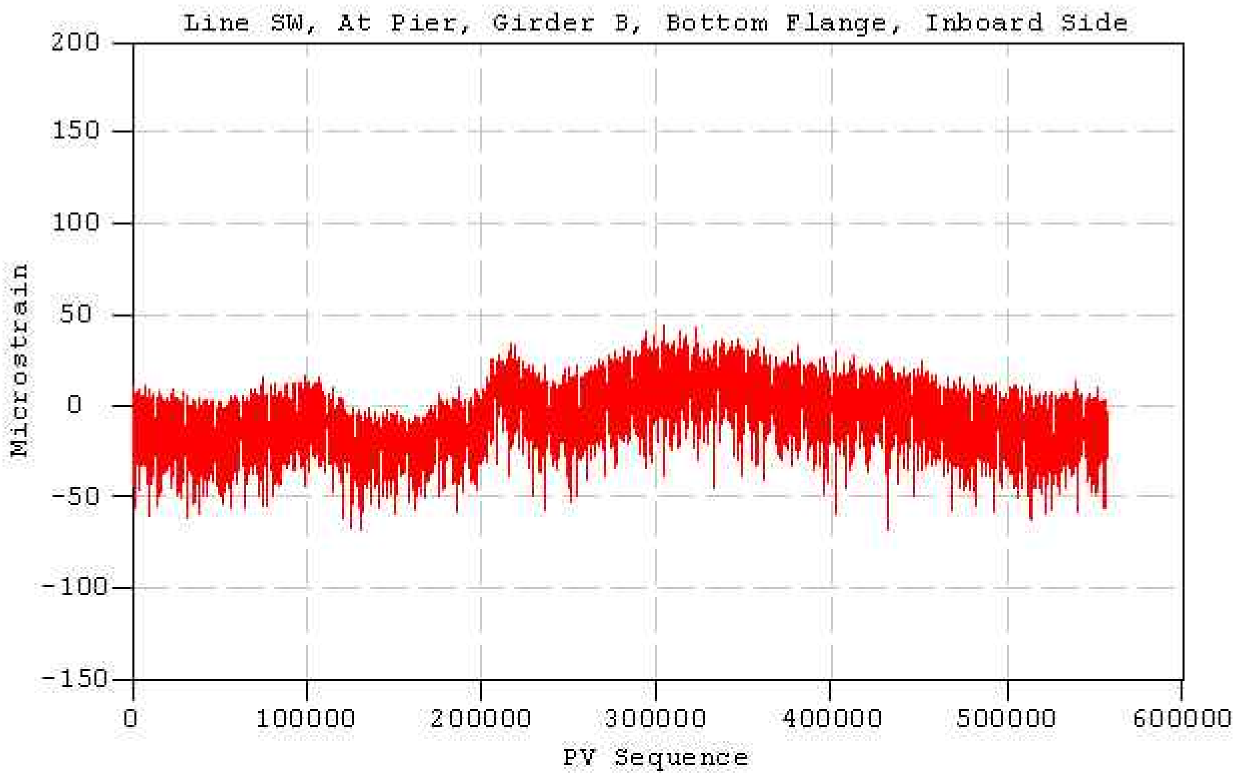

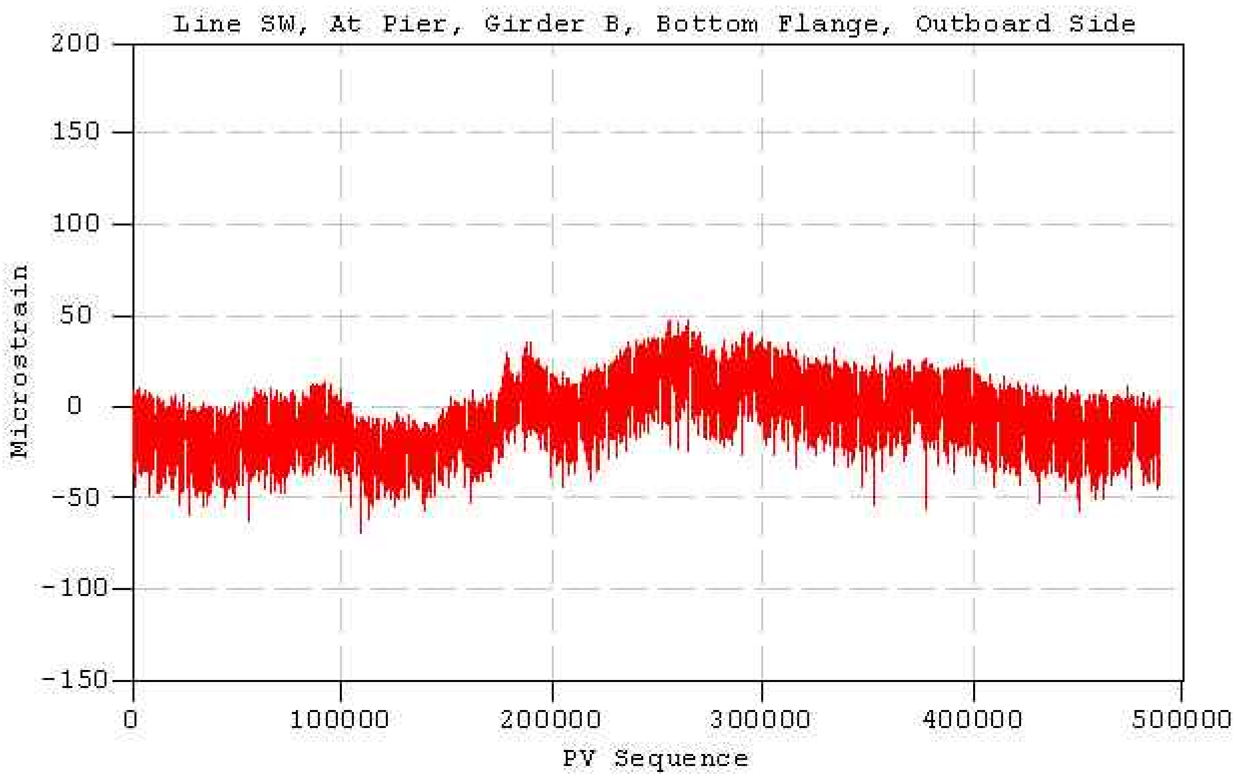

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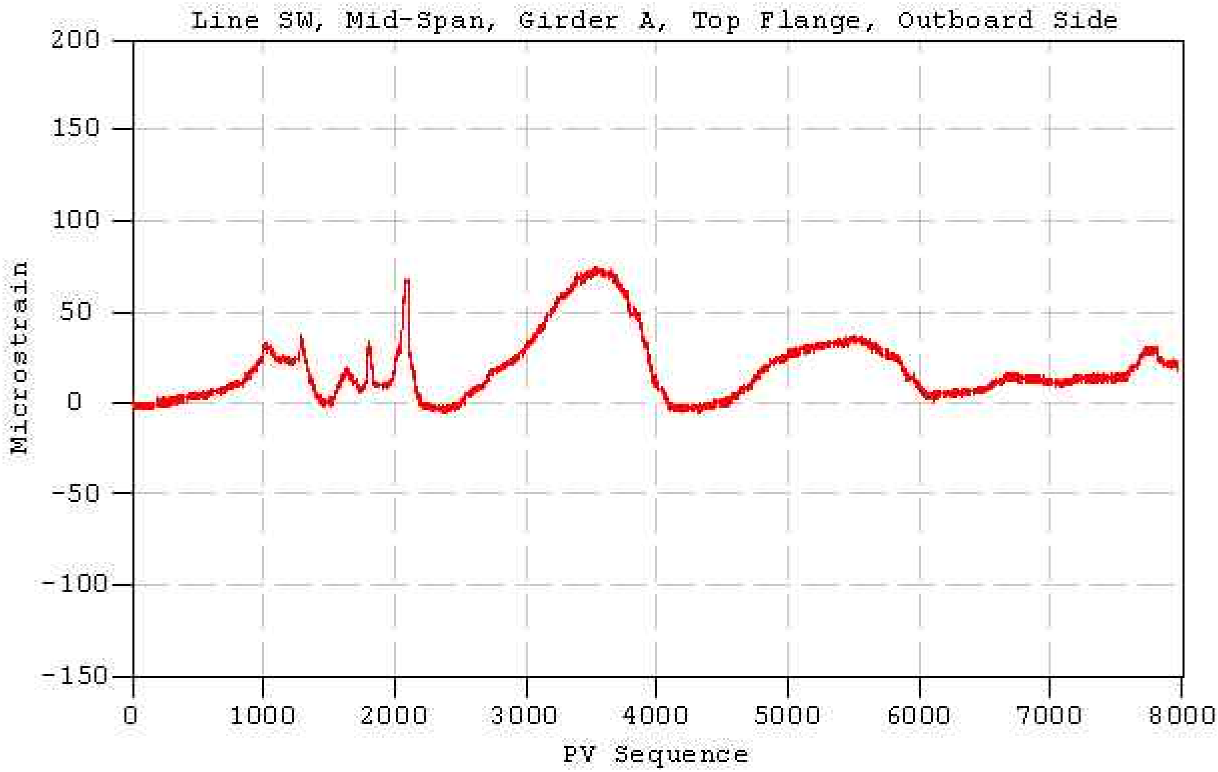

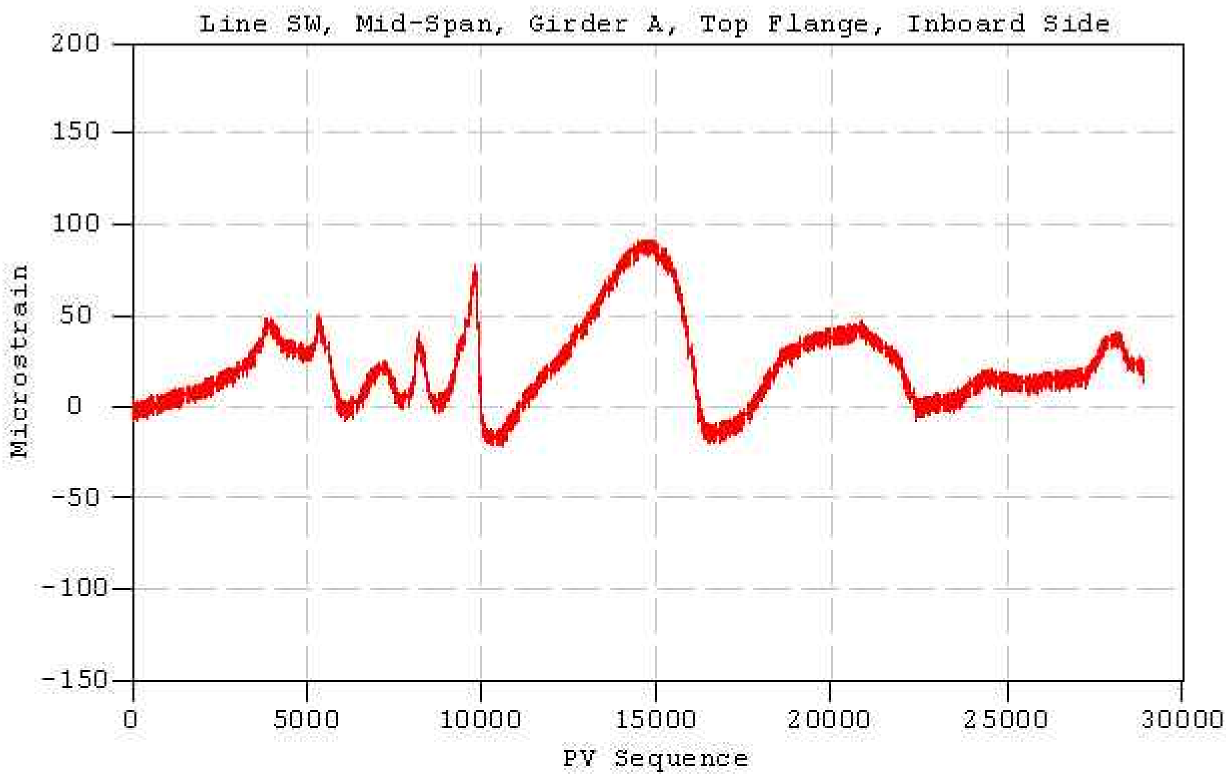

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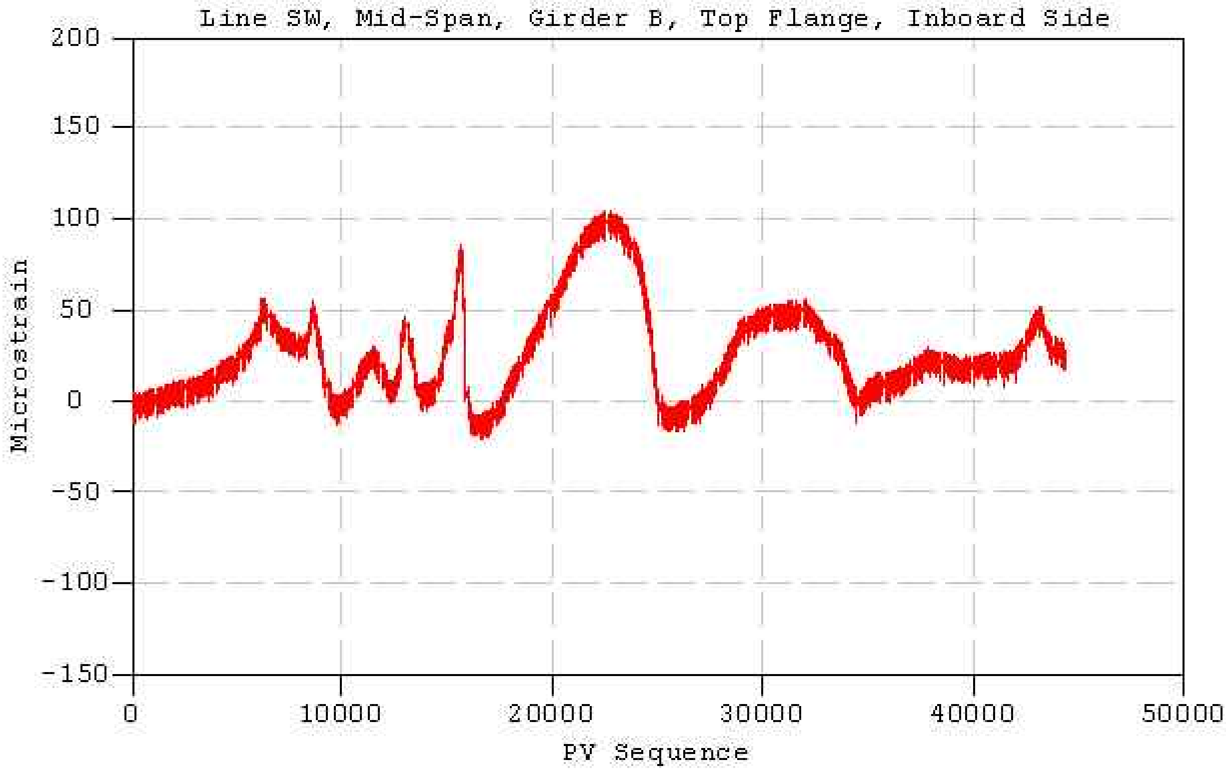

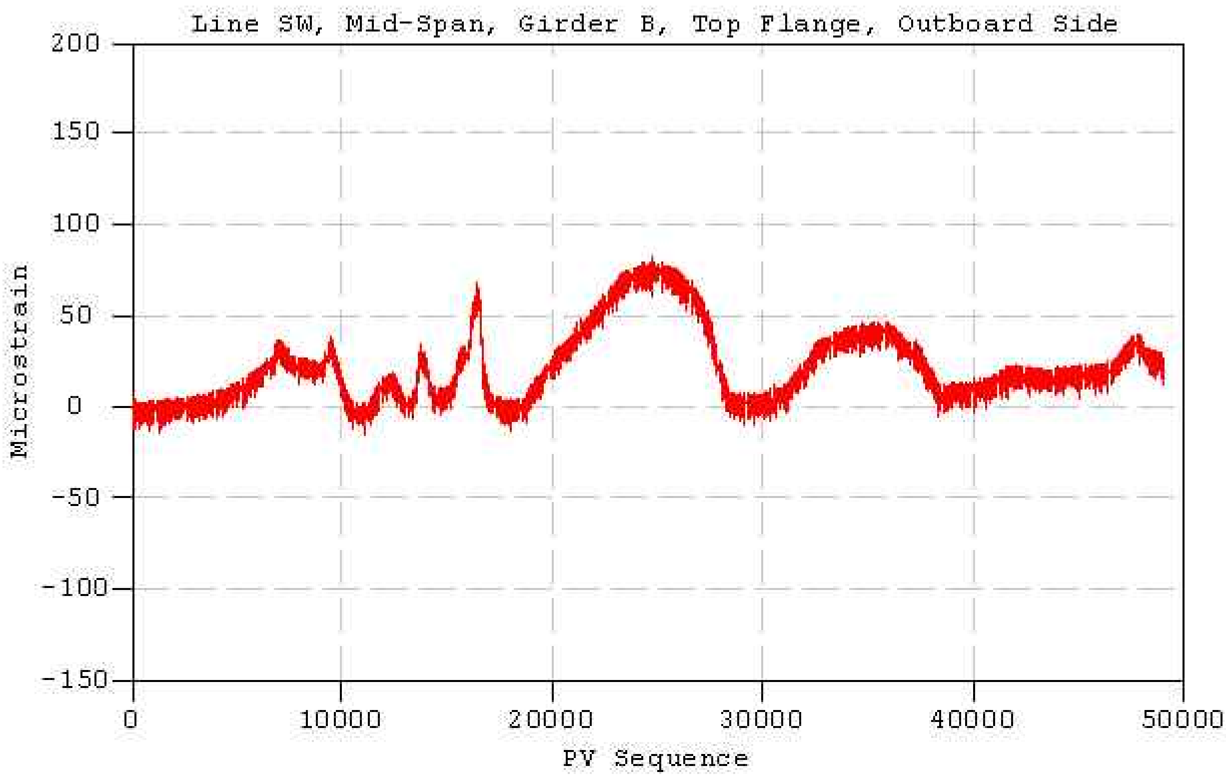

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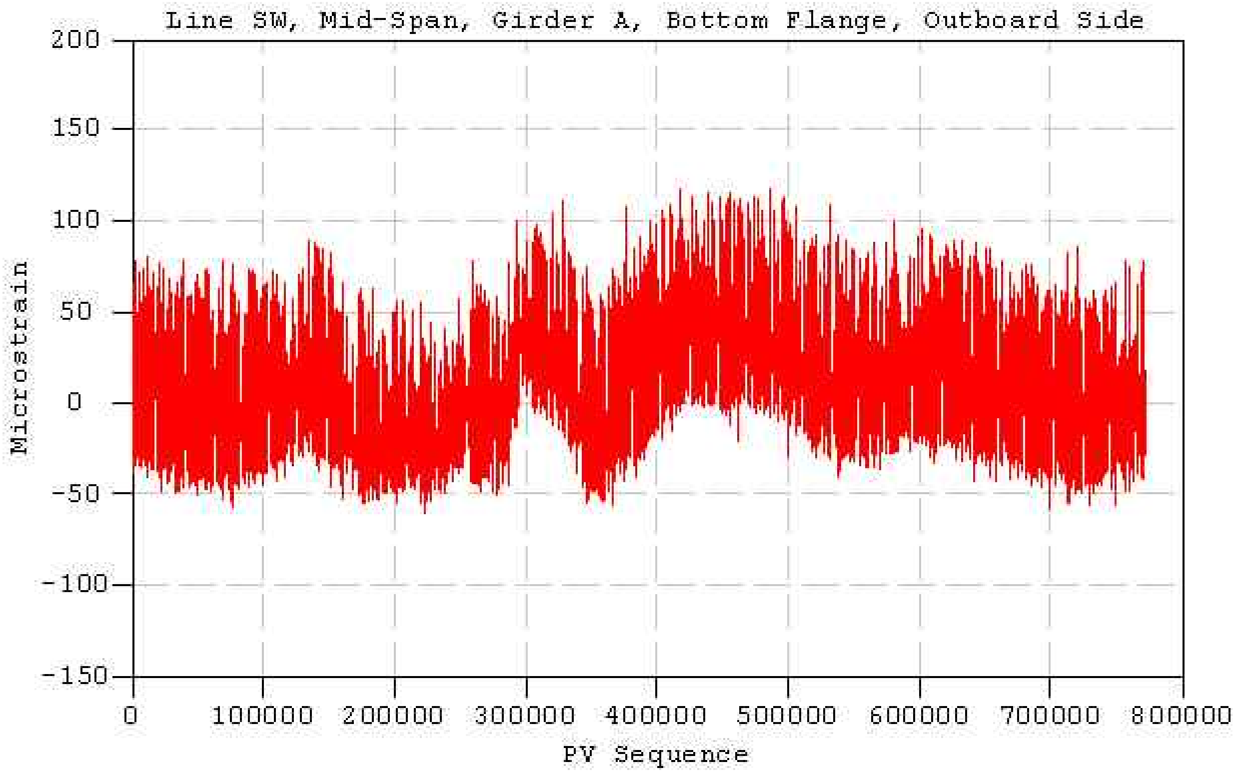

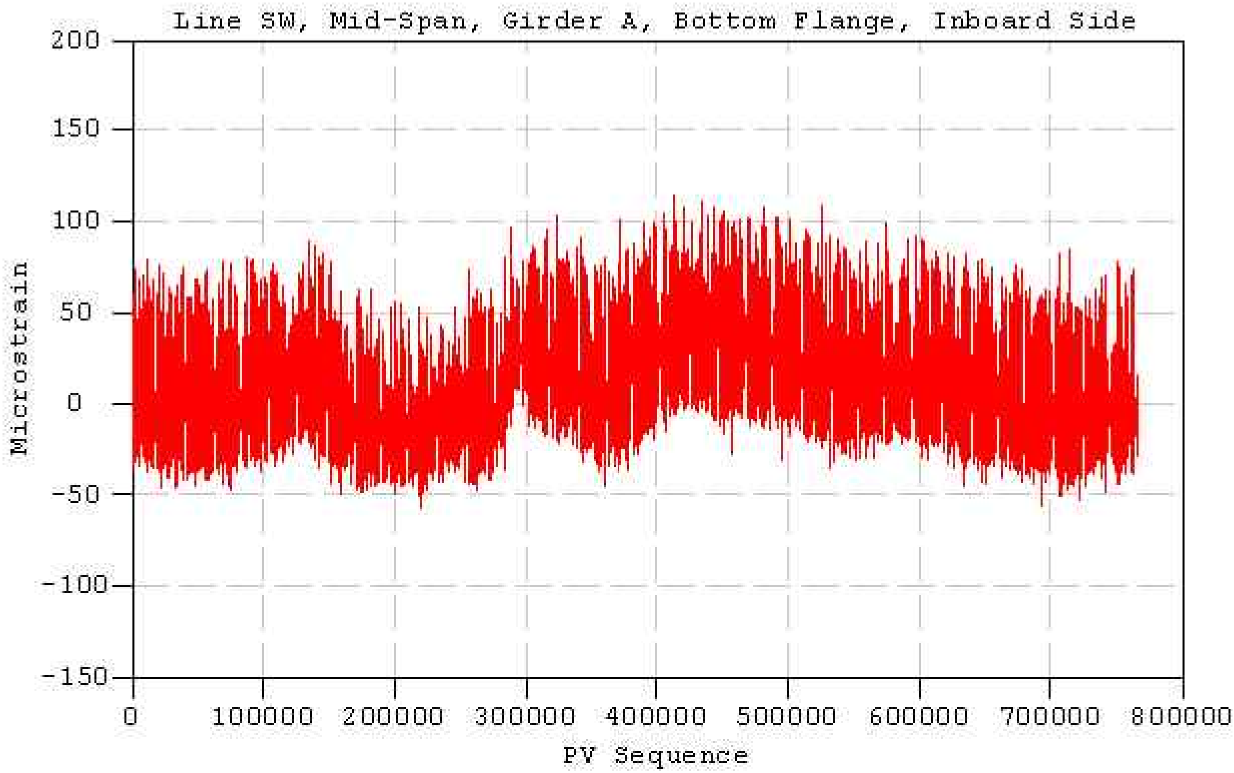

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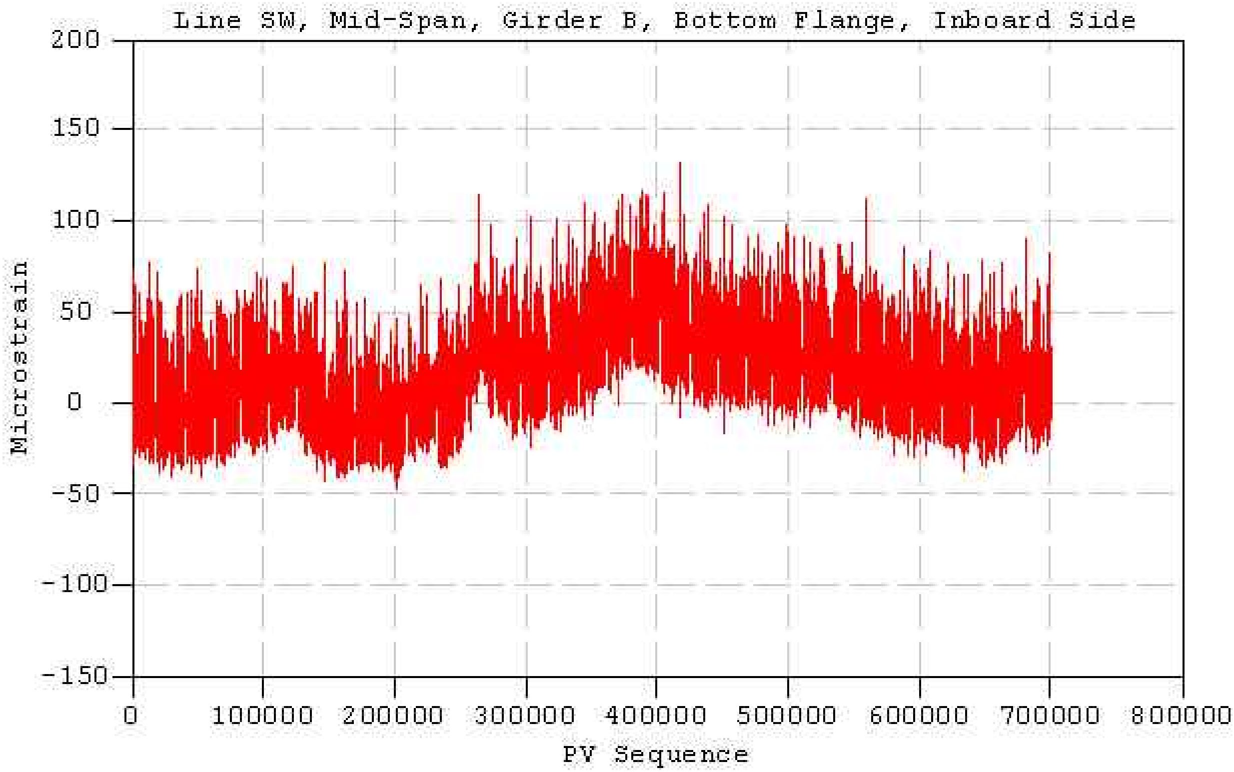

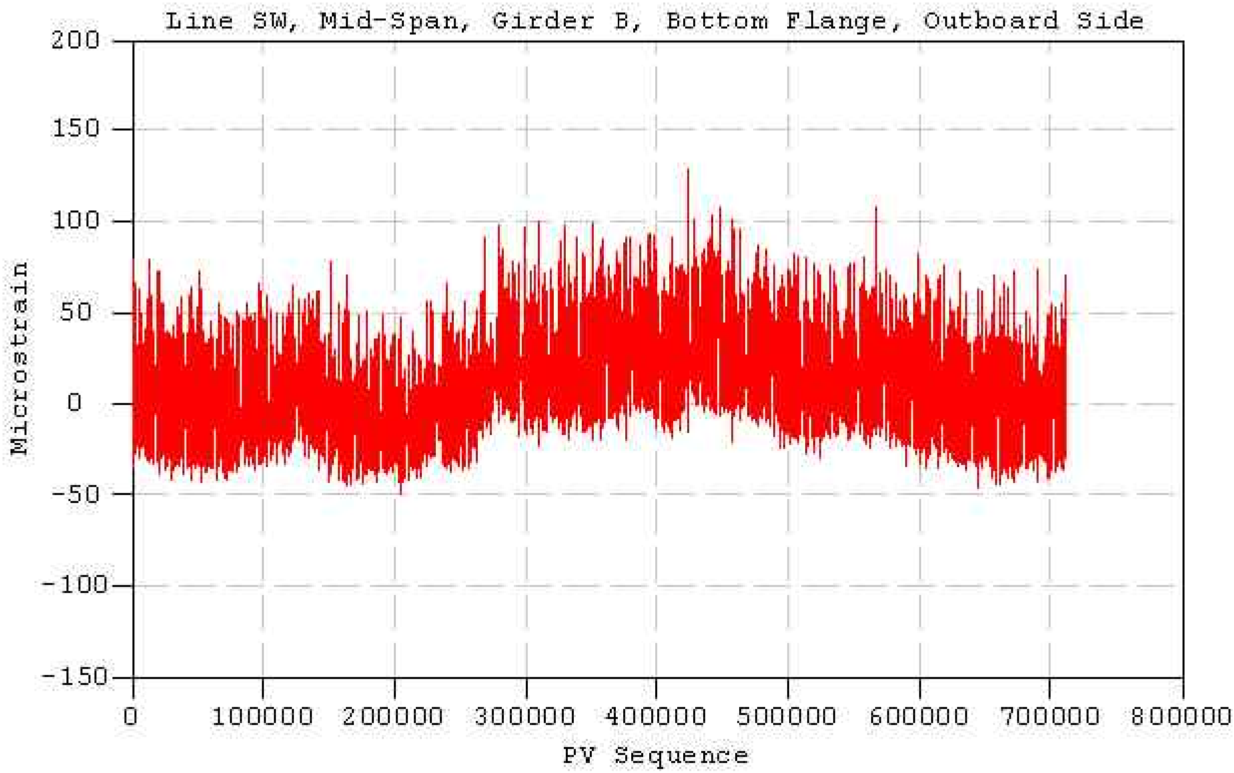

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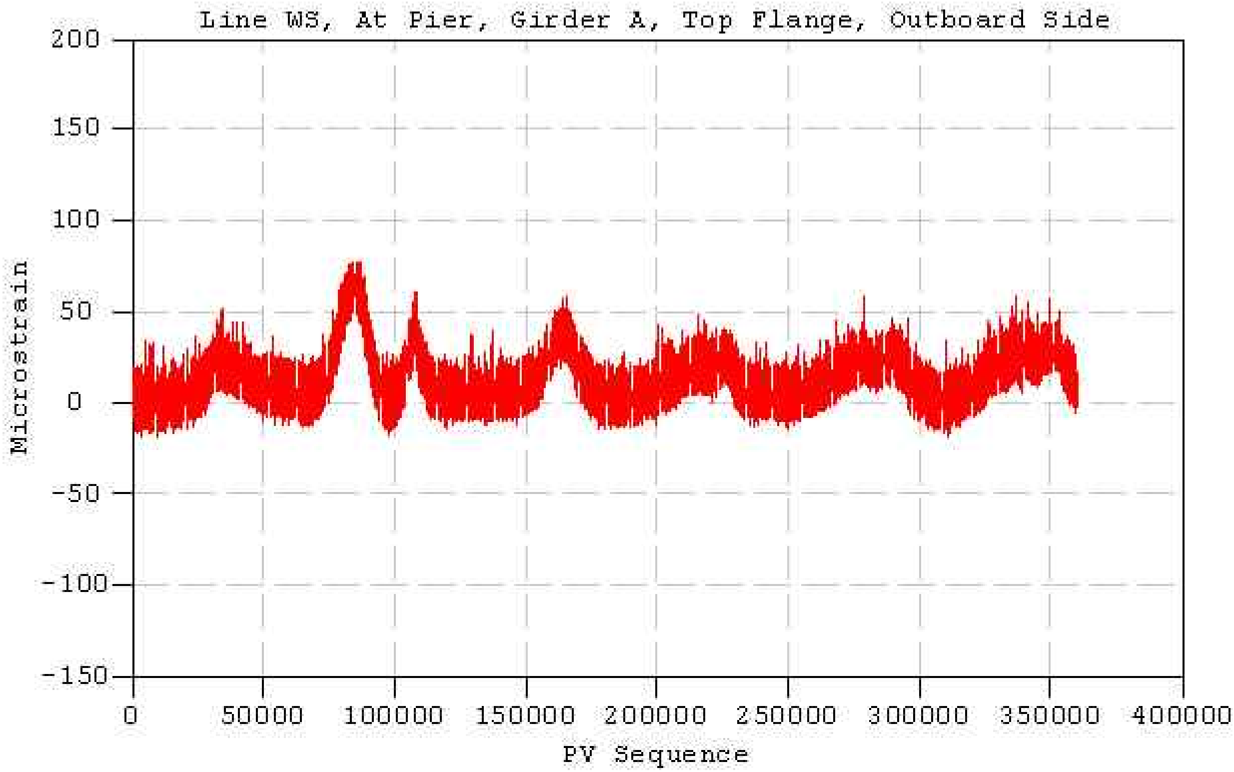

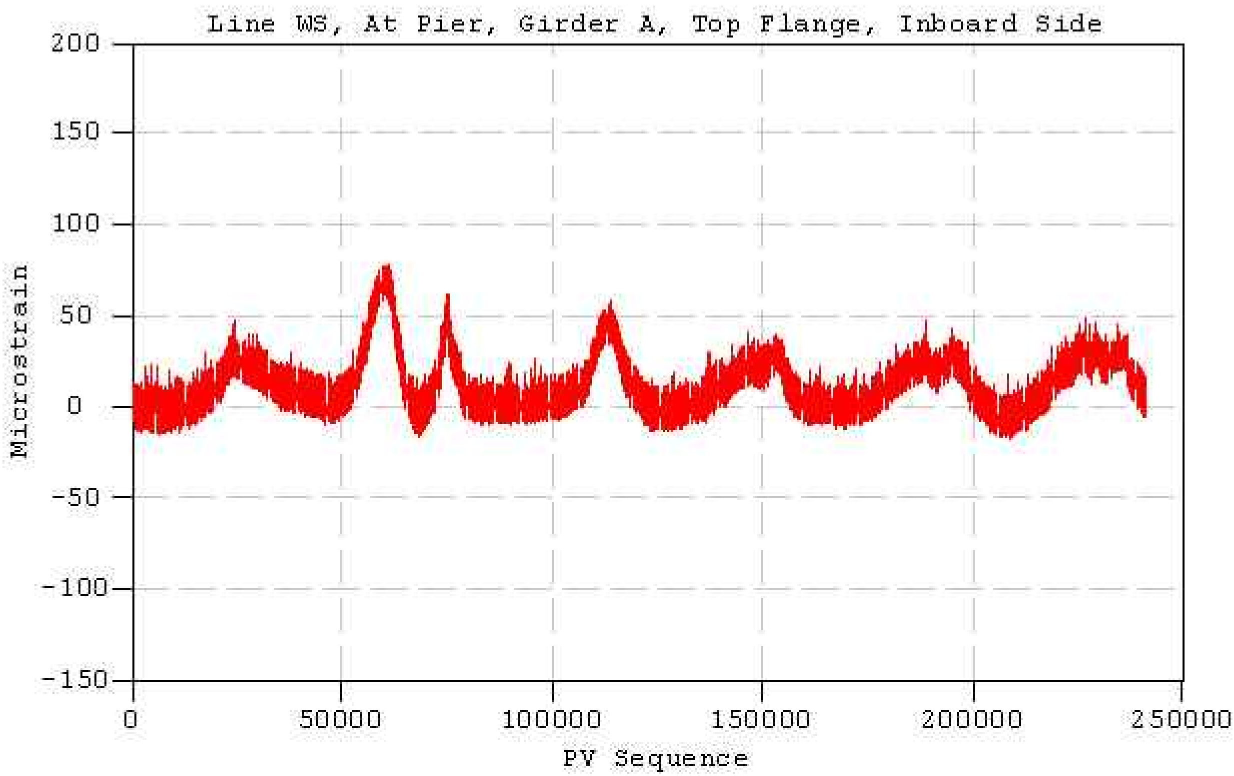

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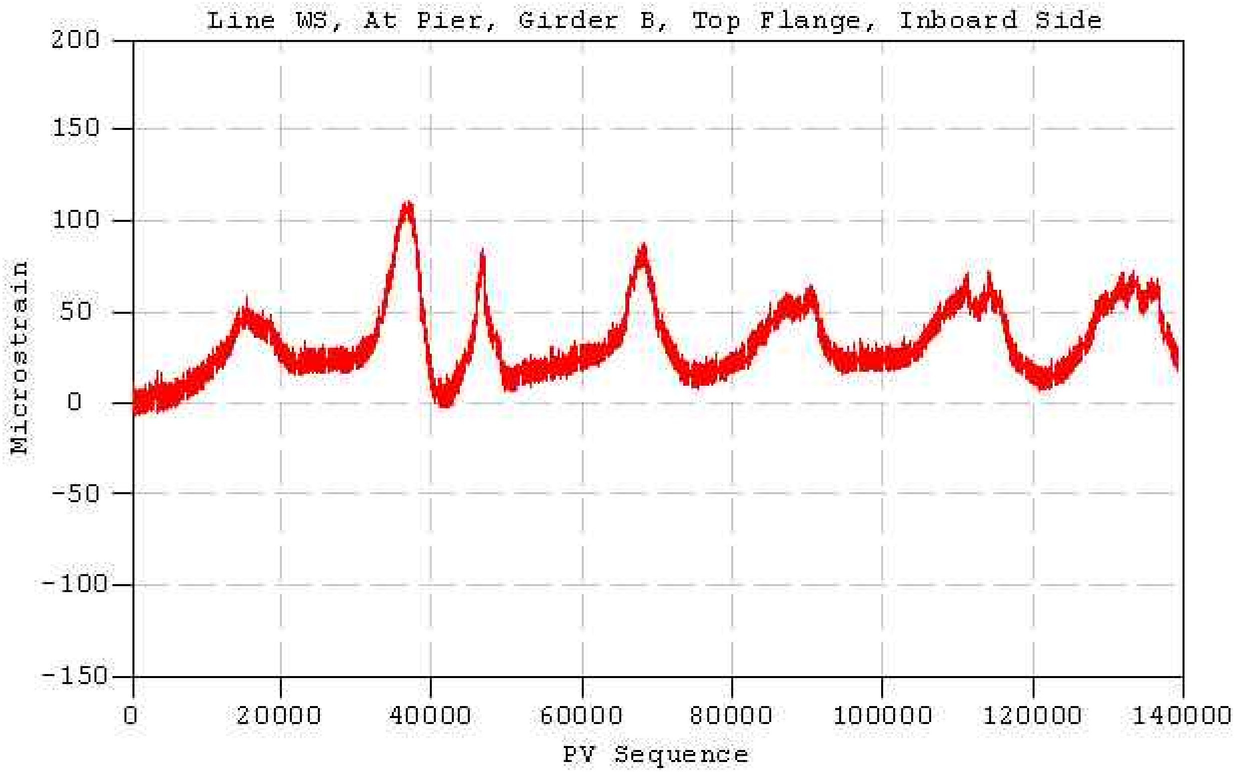
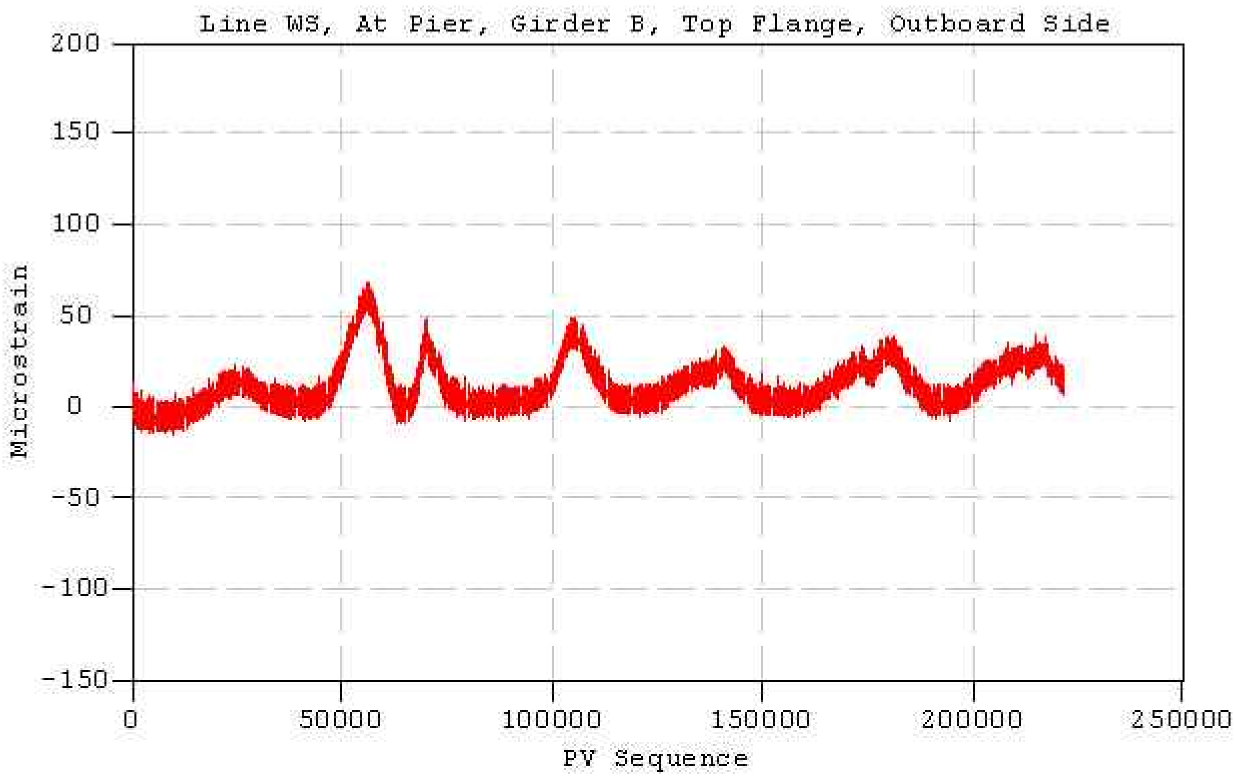
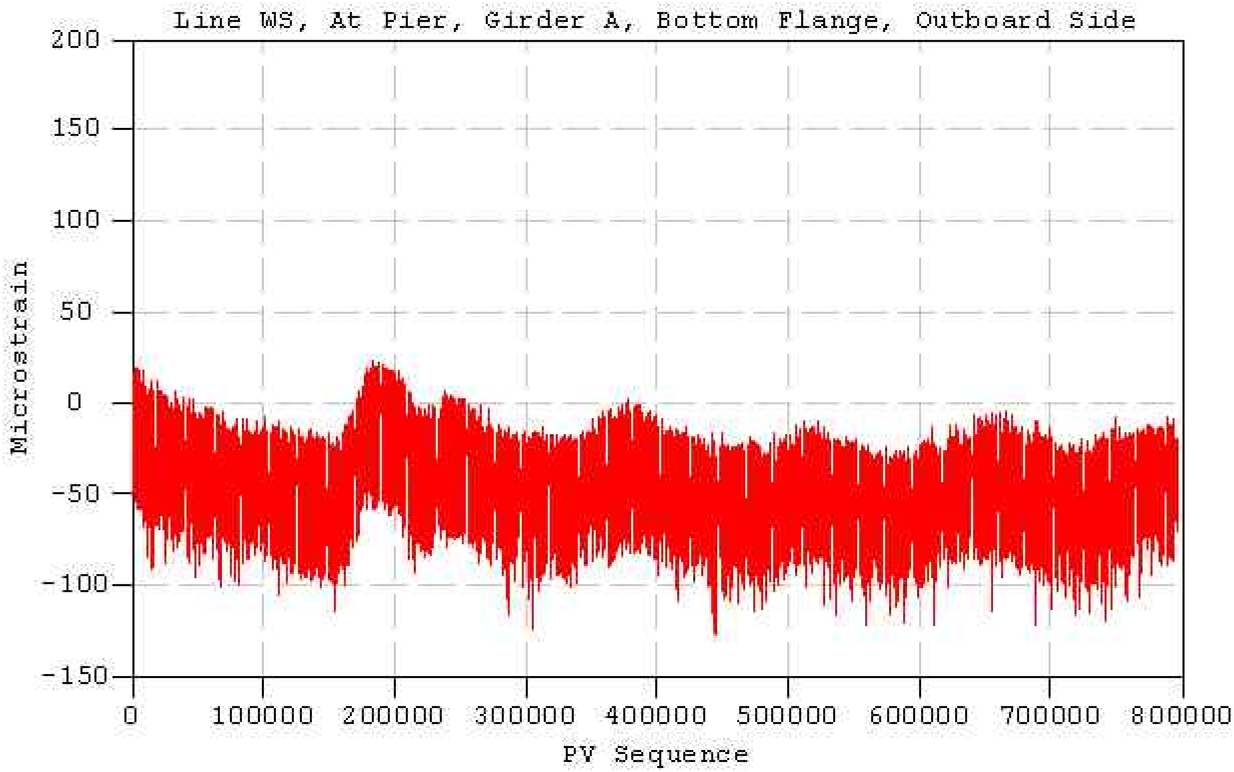
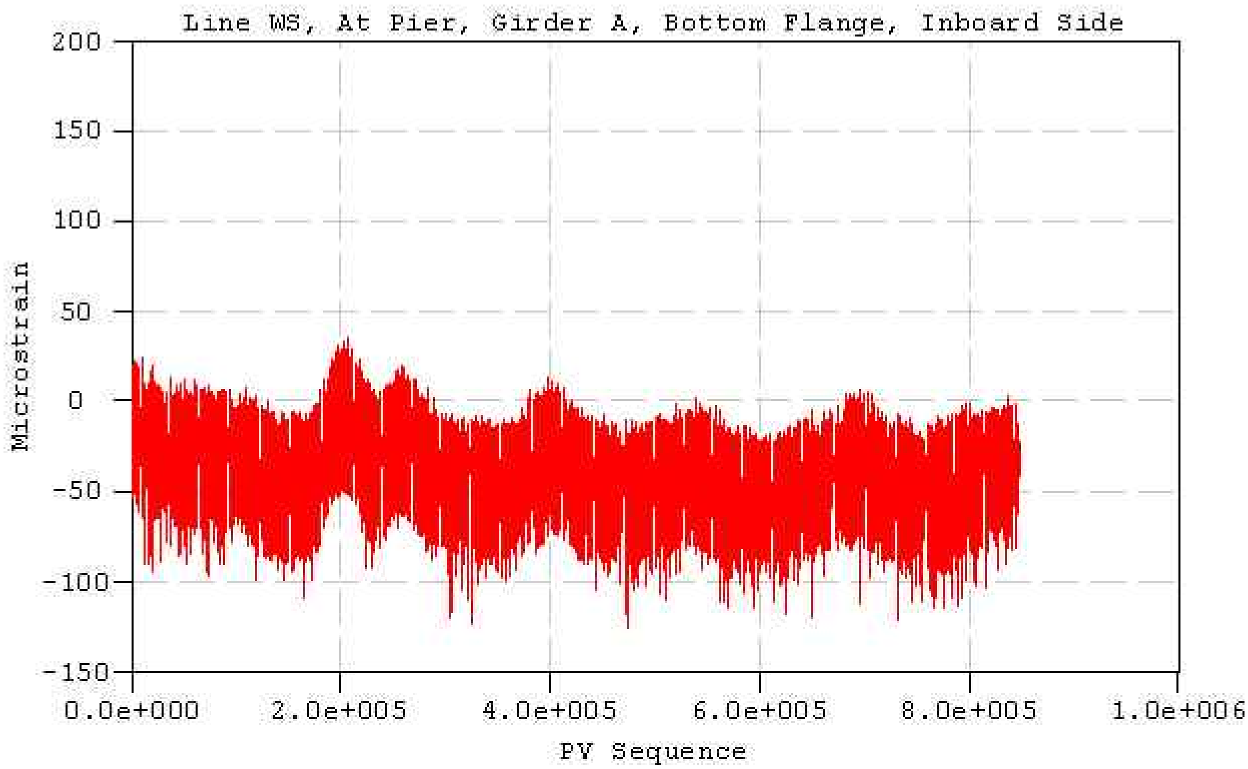


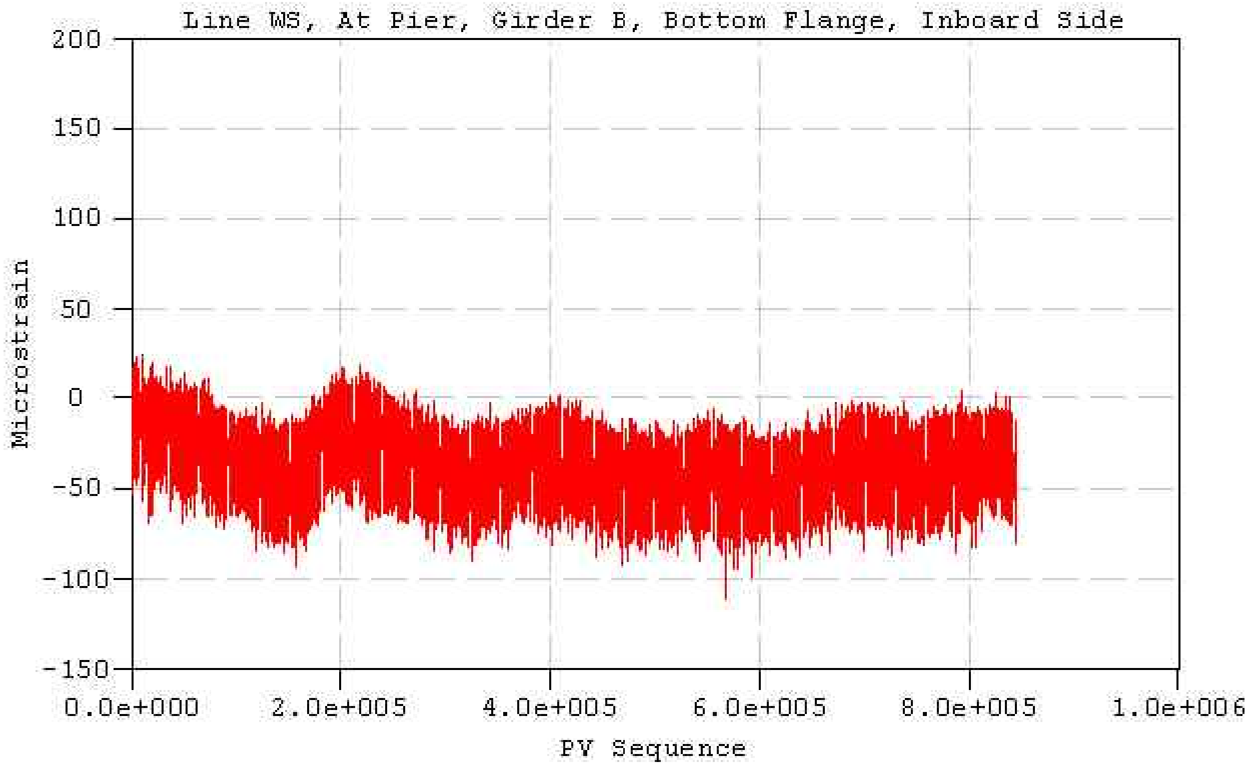


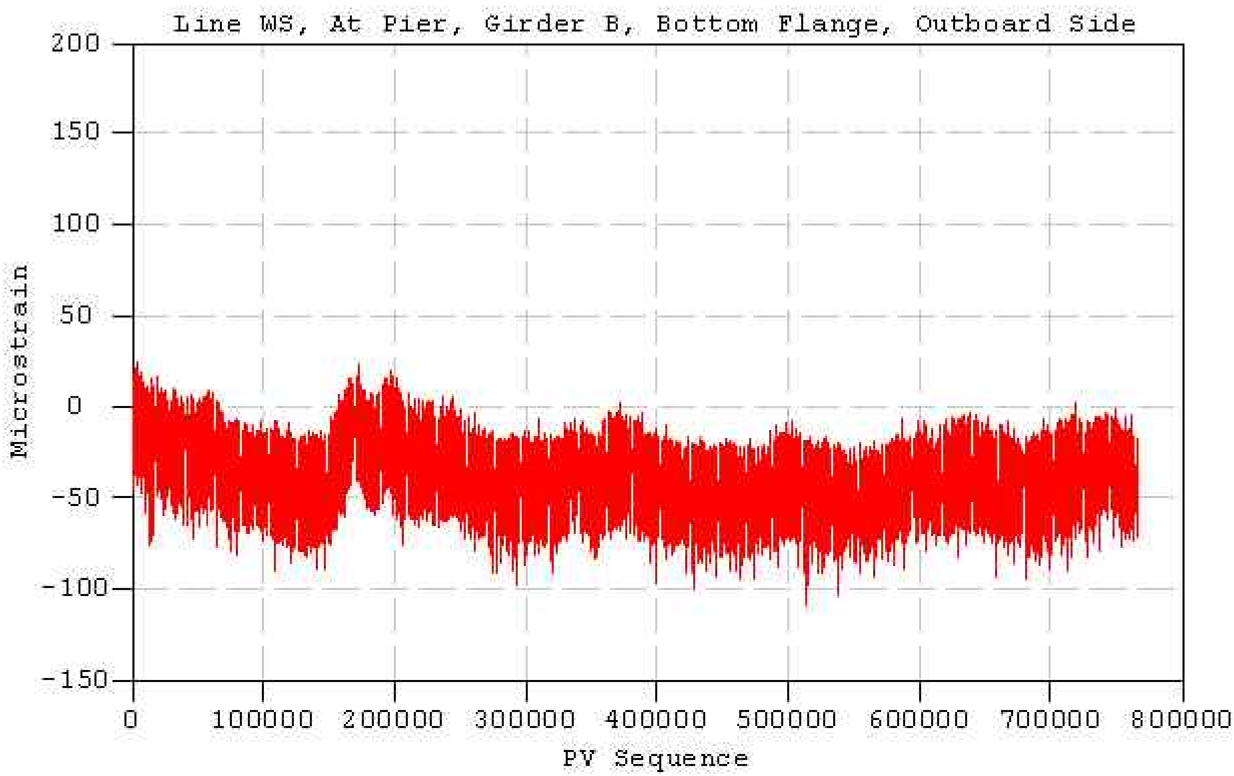
Peak-Valley Strain History Diagrams



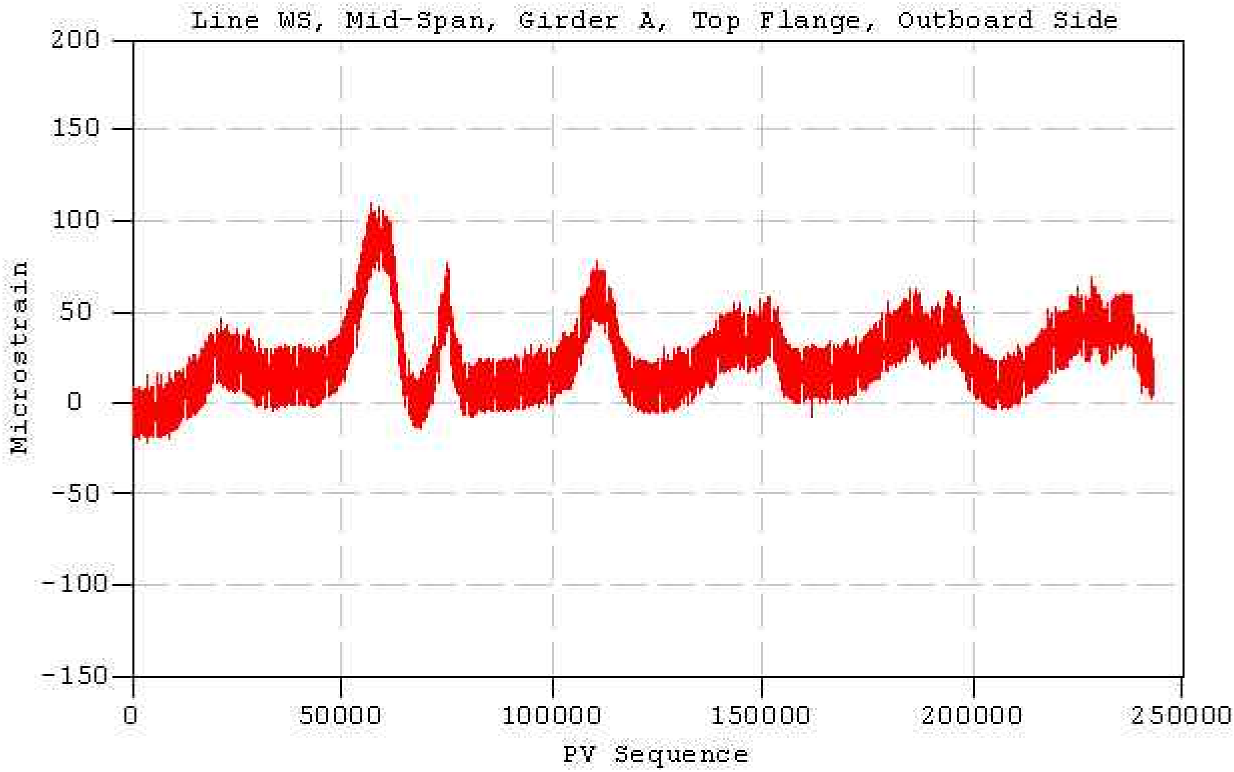


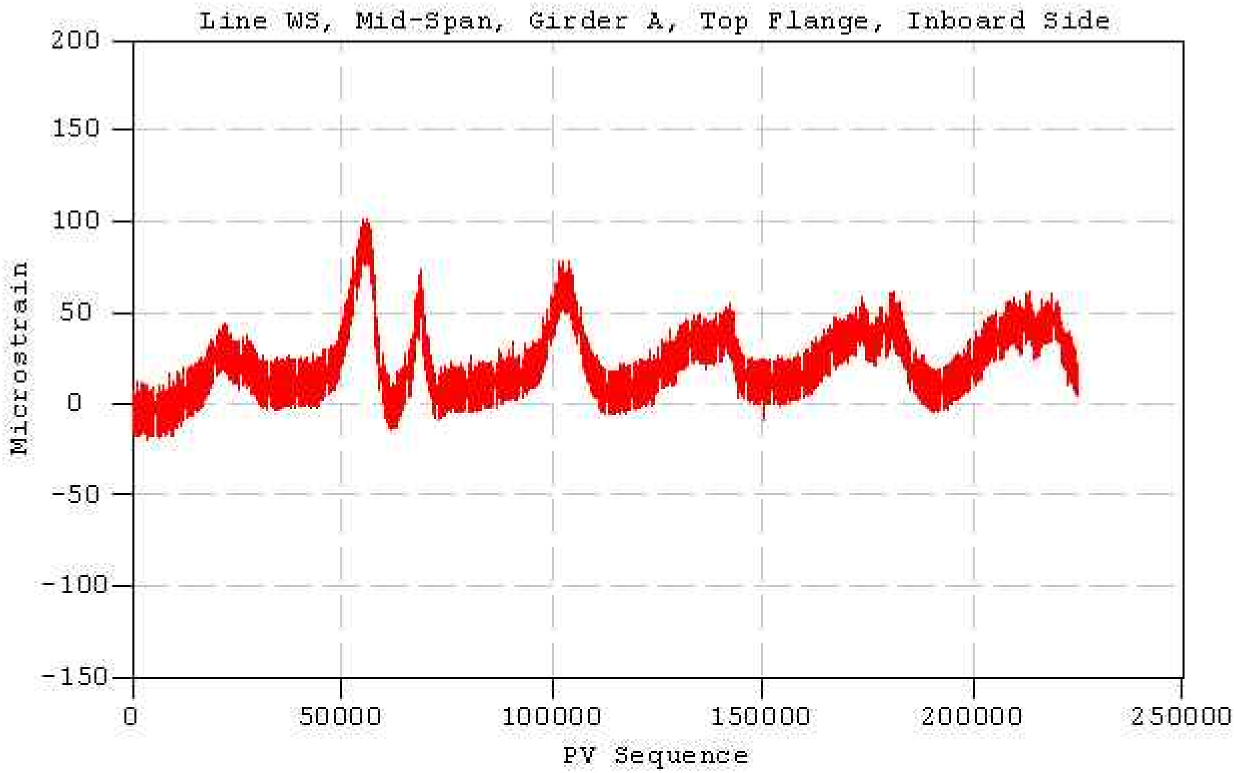
Peak-Valley Strain History Diagrams





Peak-Valley Strain History Diagrams





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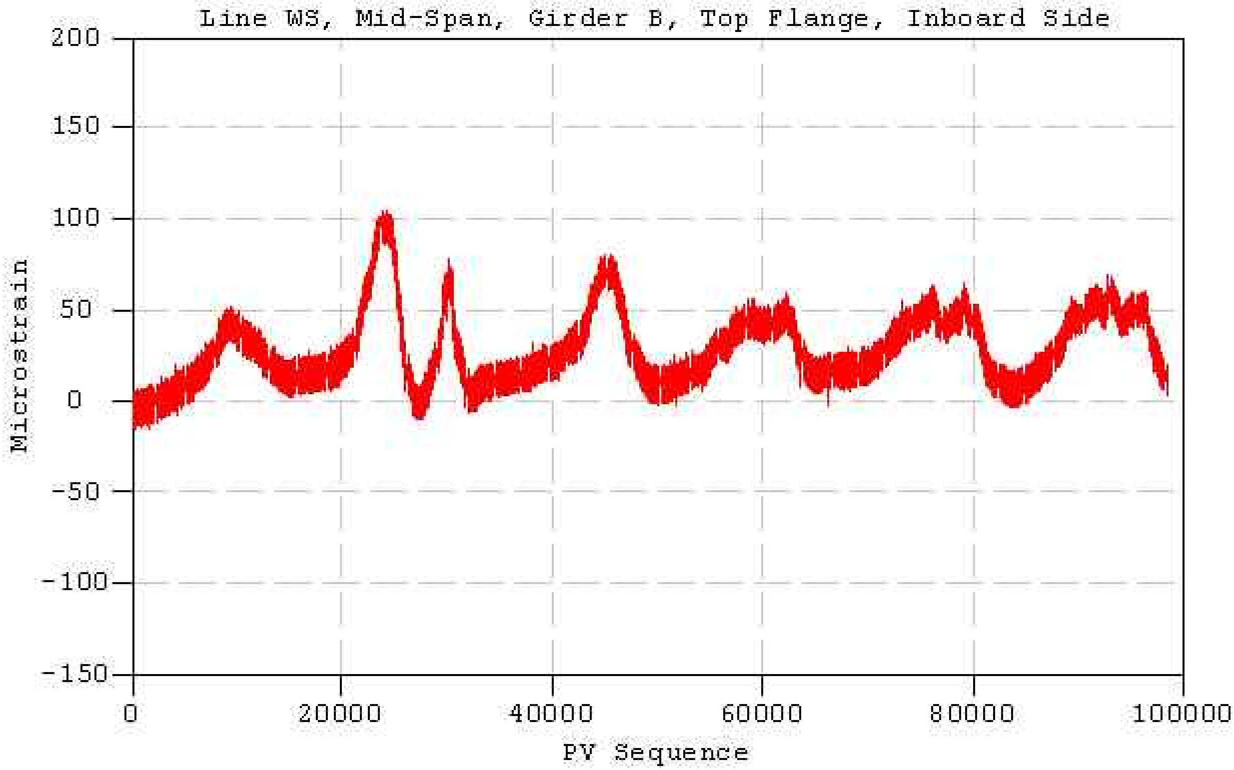

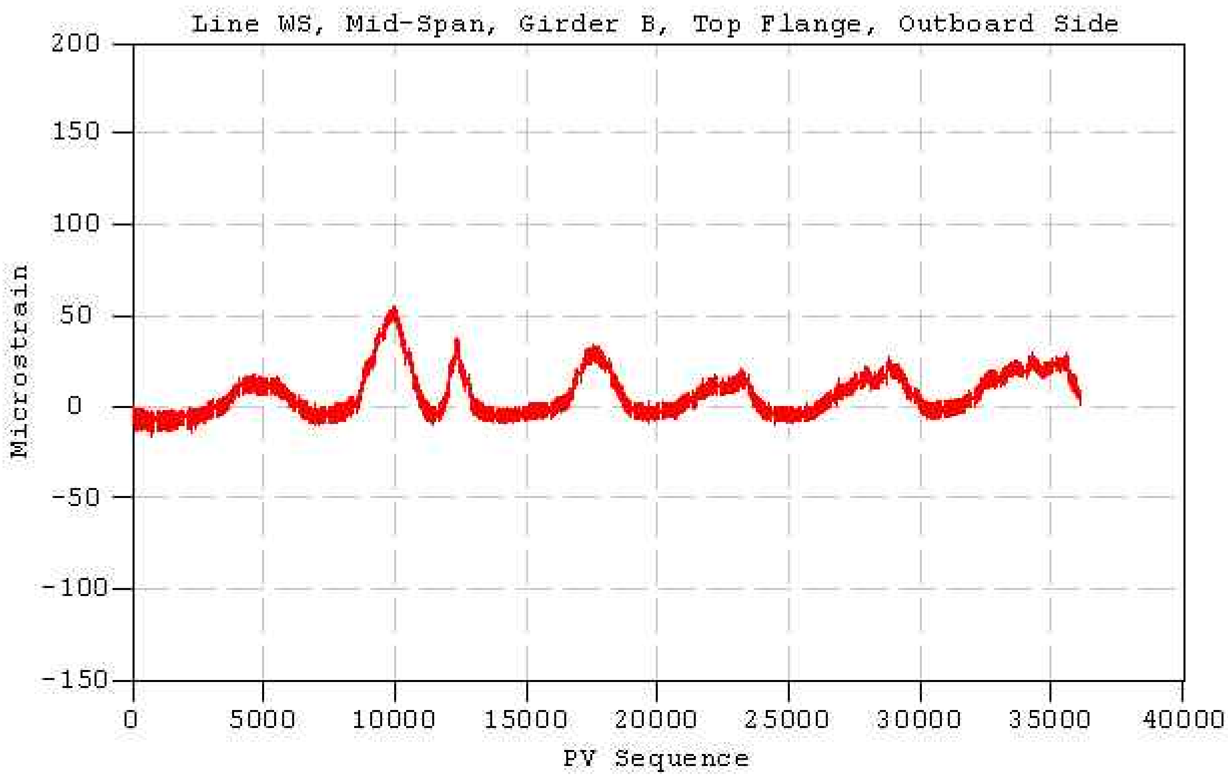

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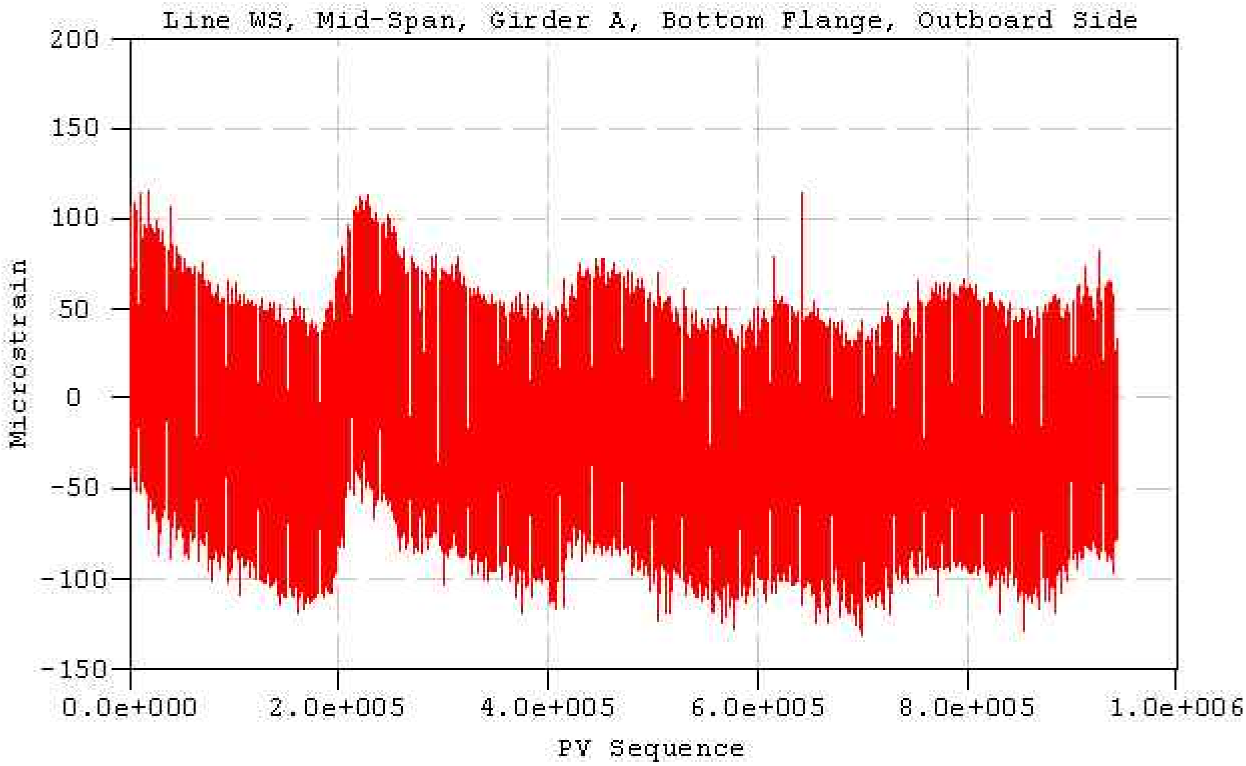

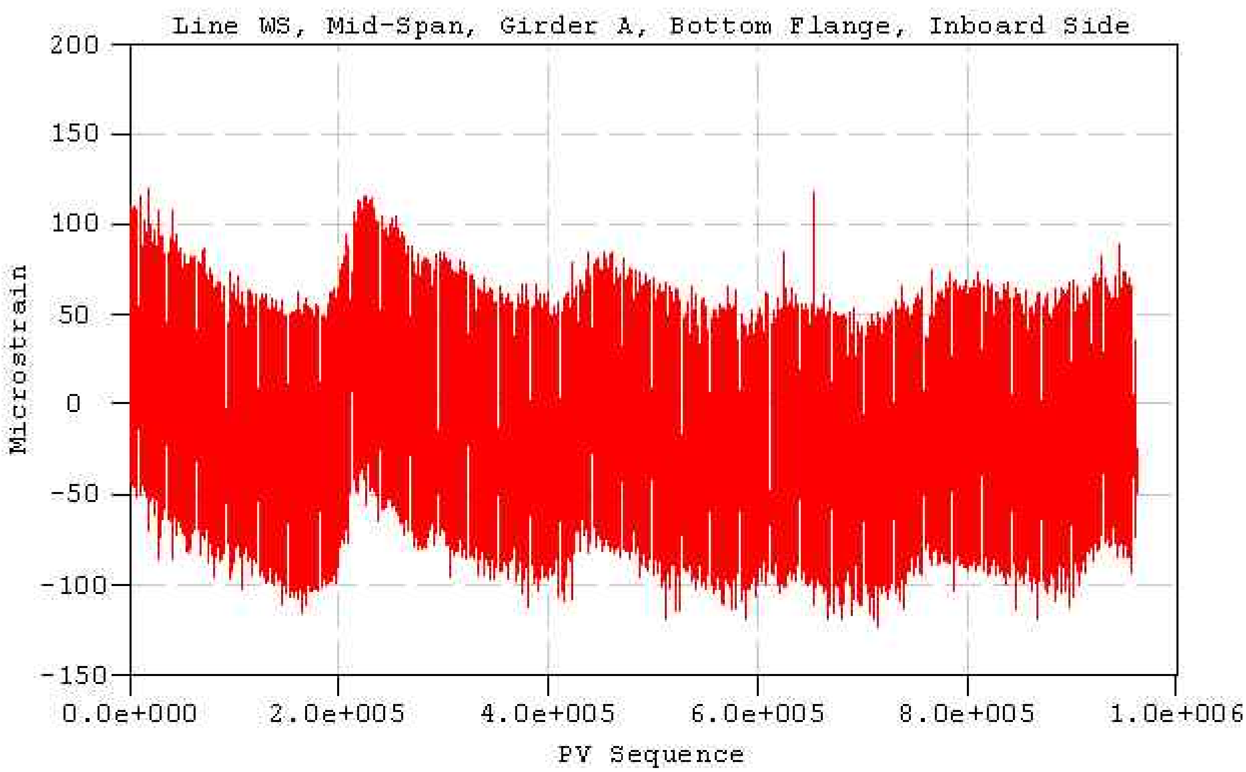

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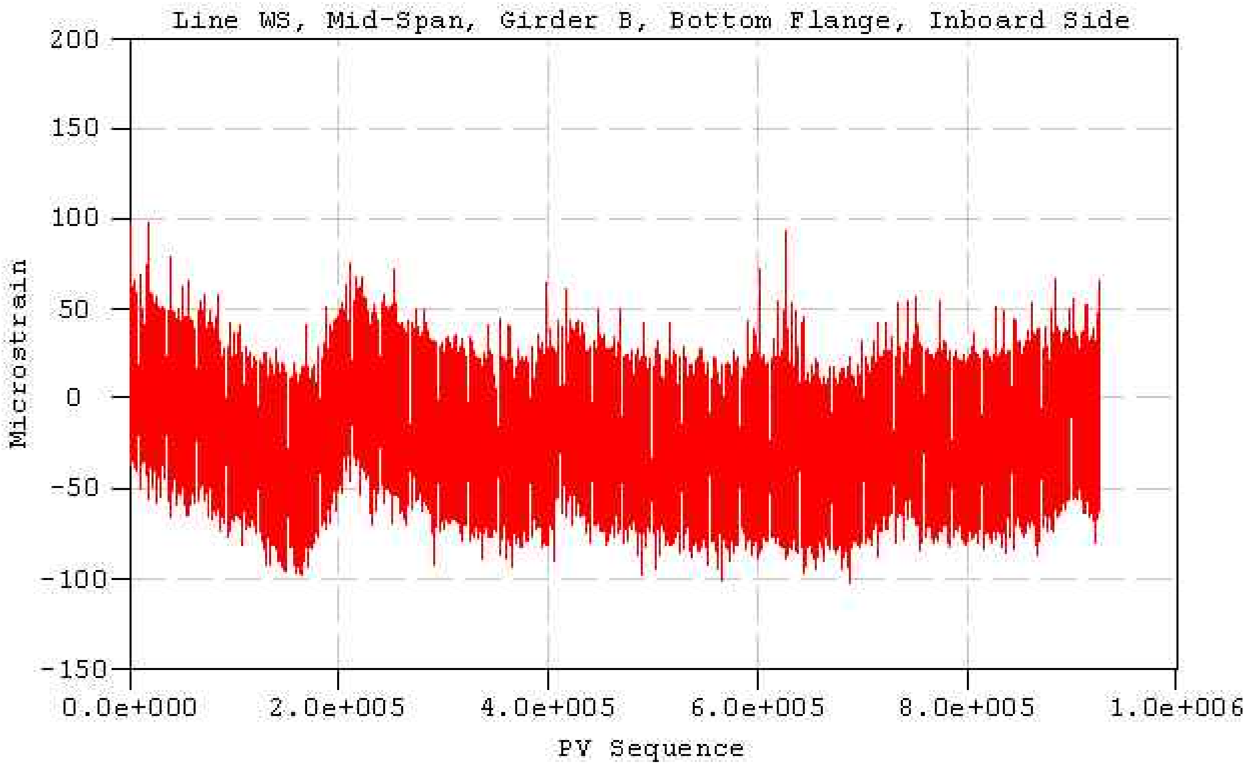

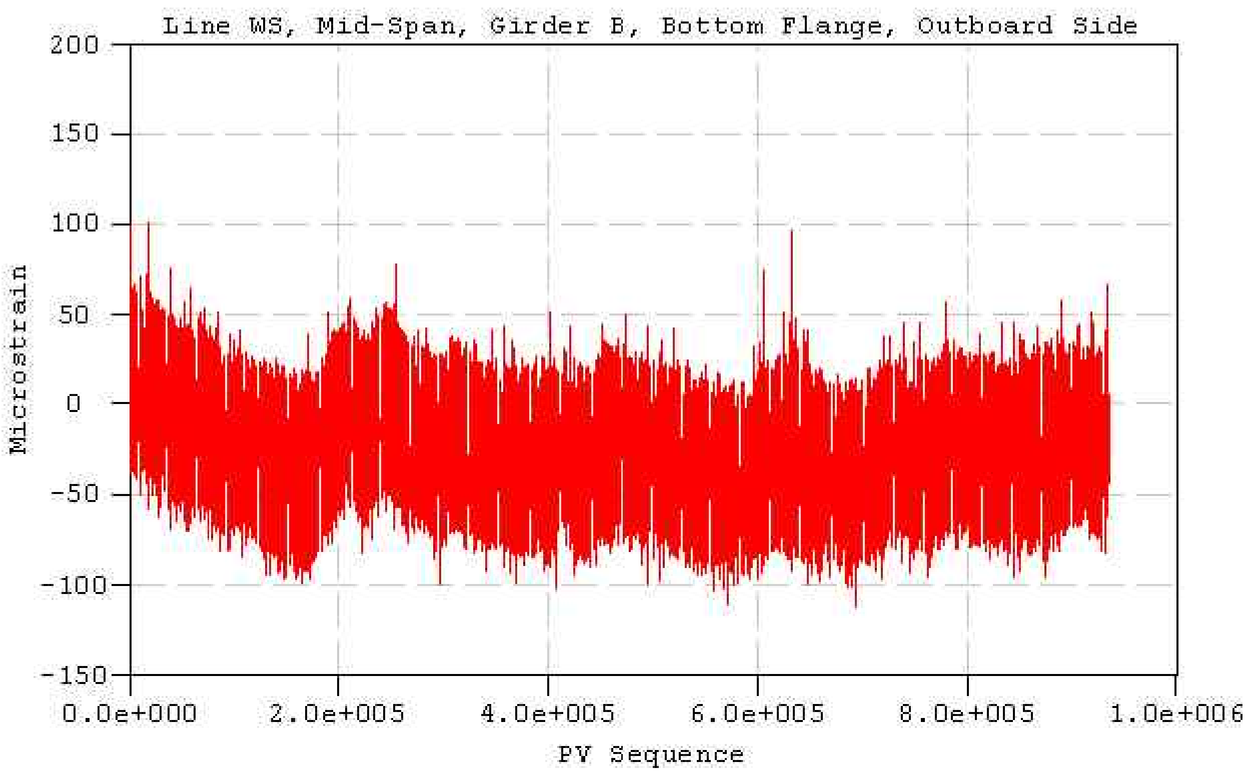

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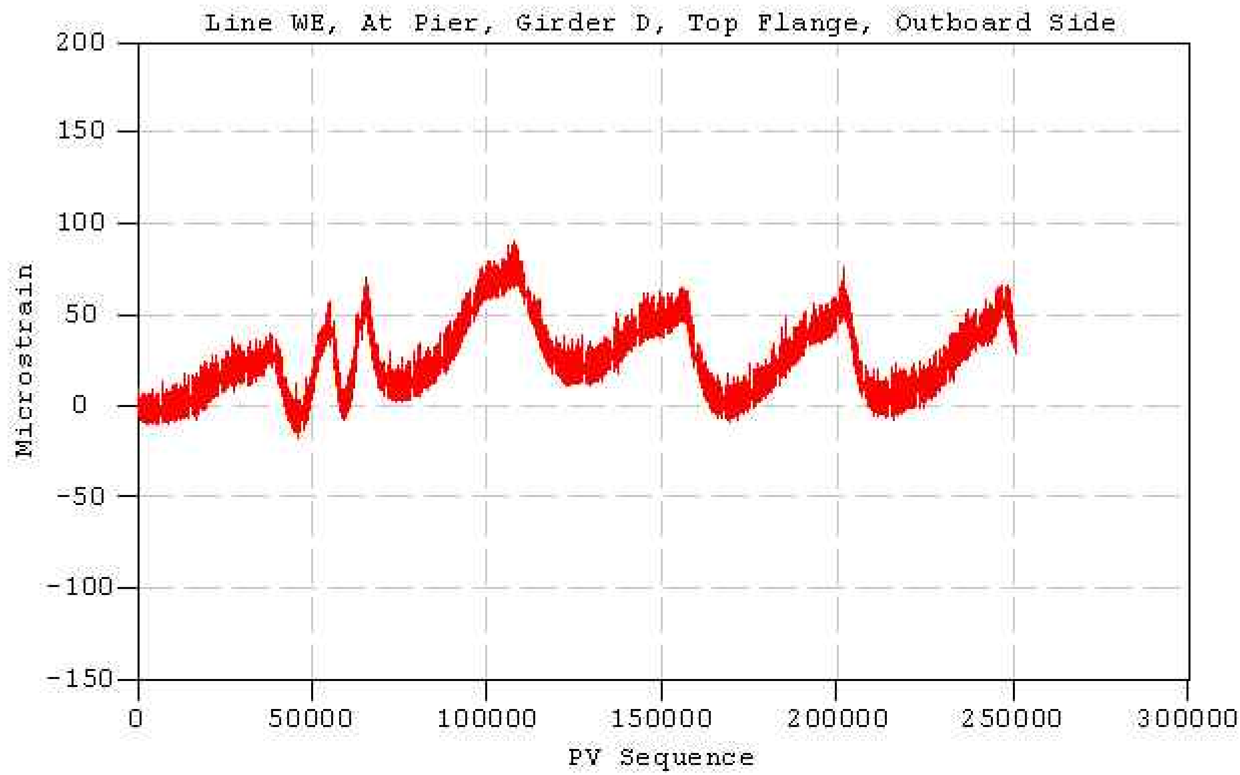

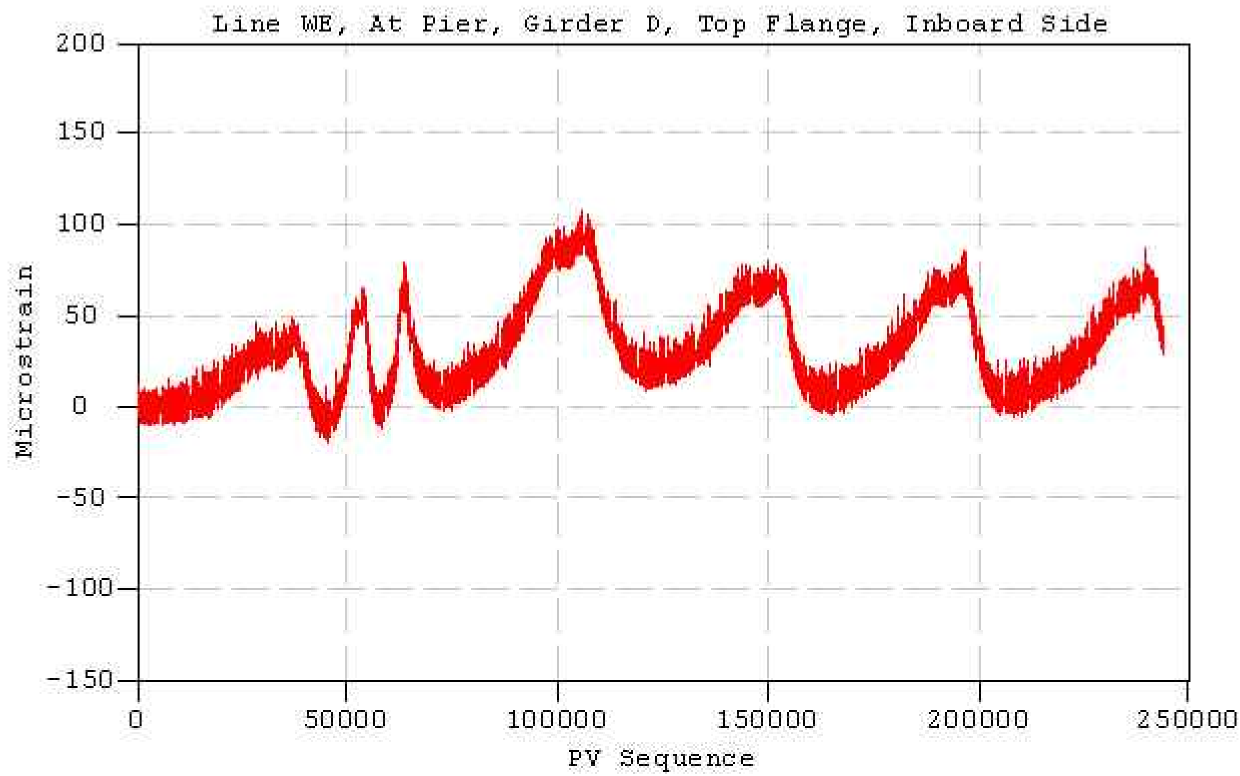

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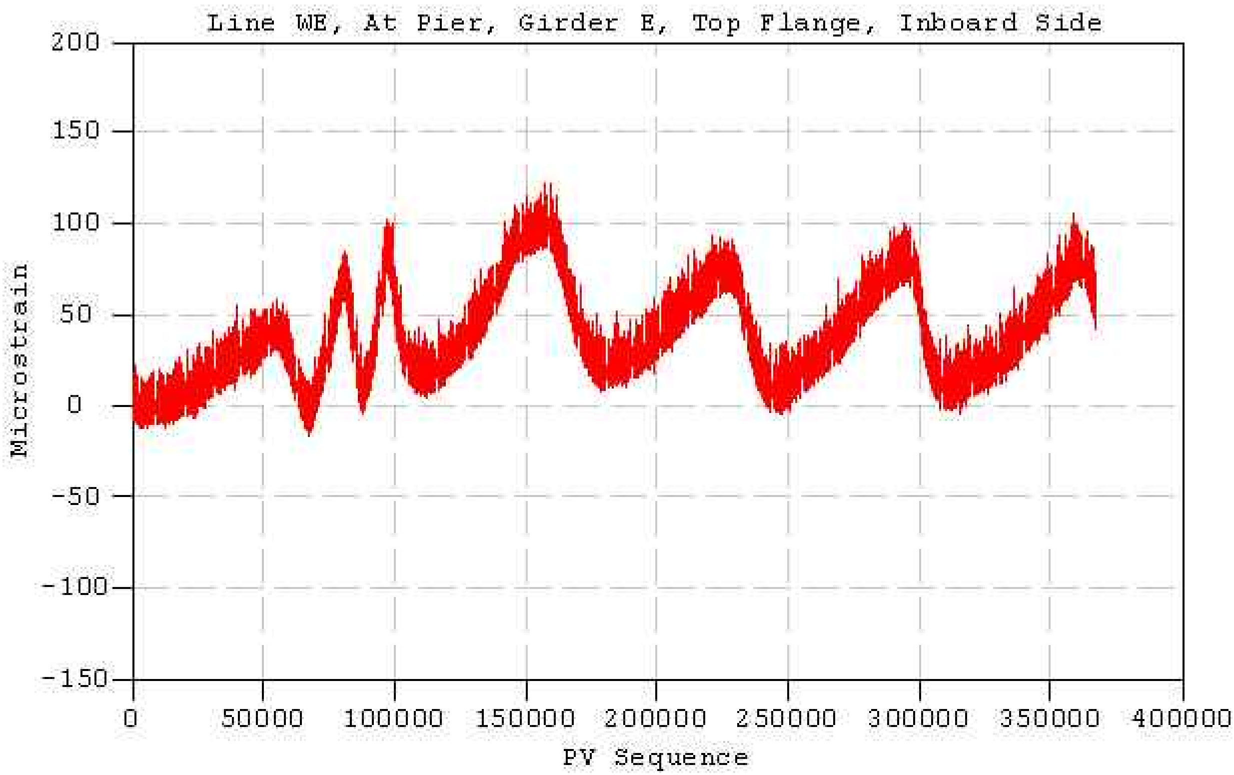

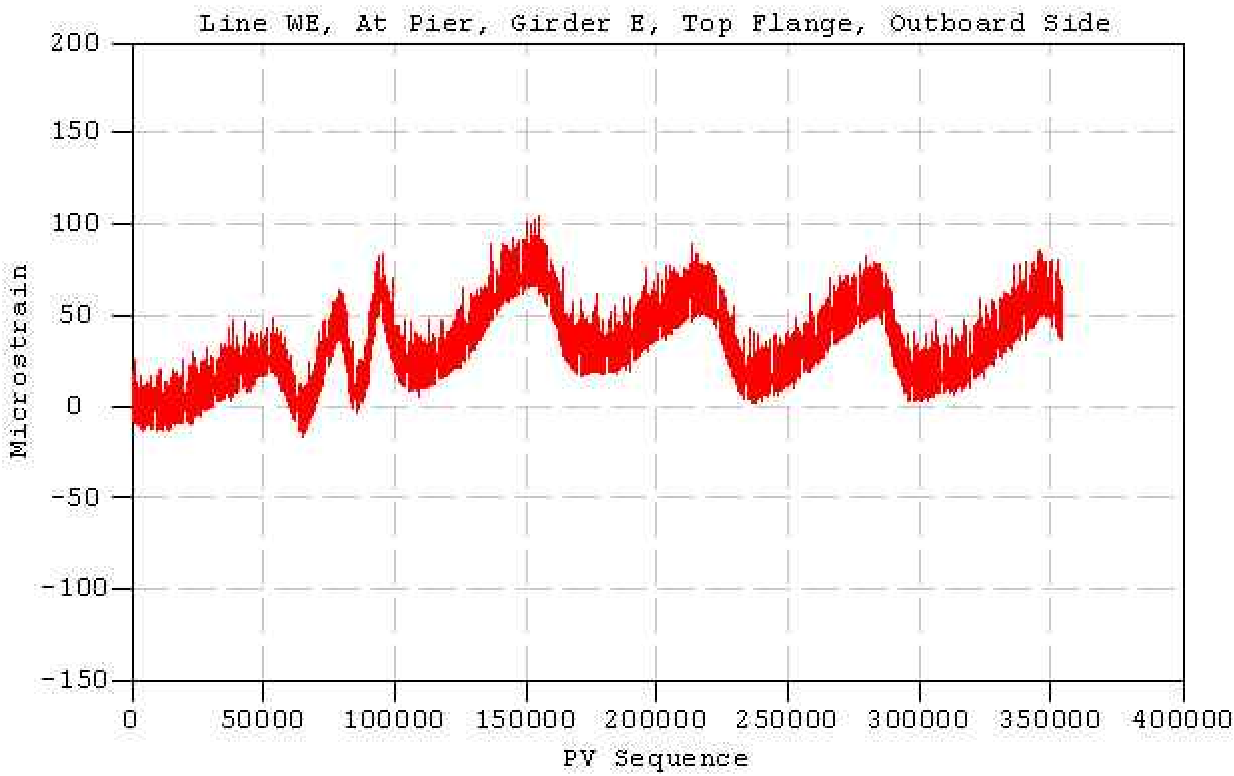

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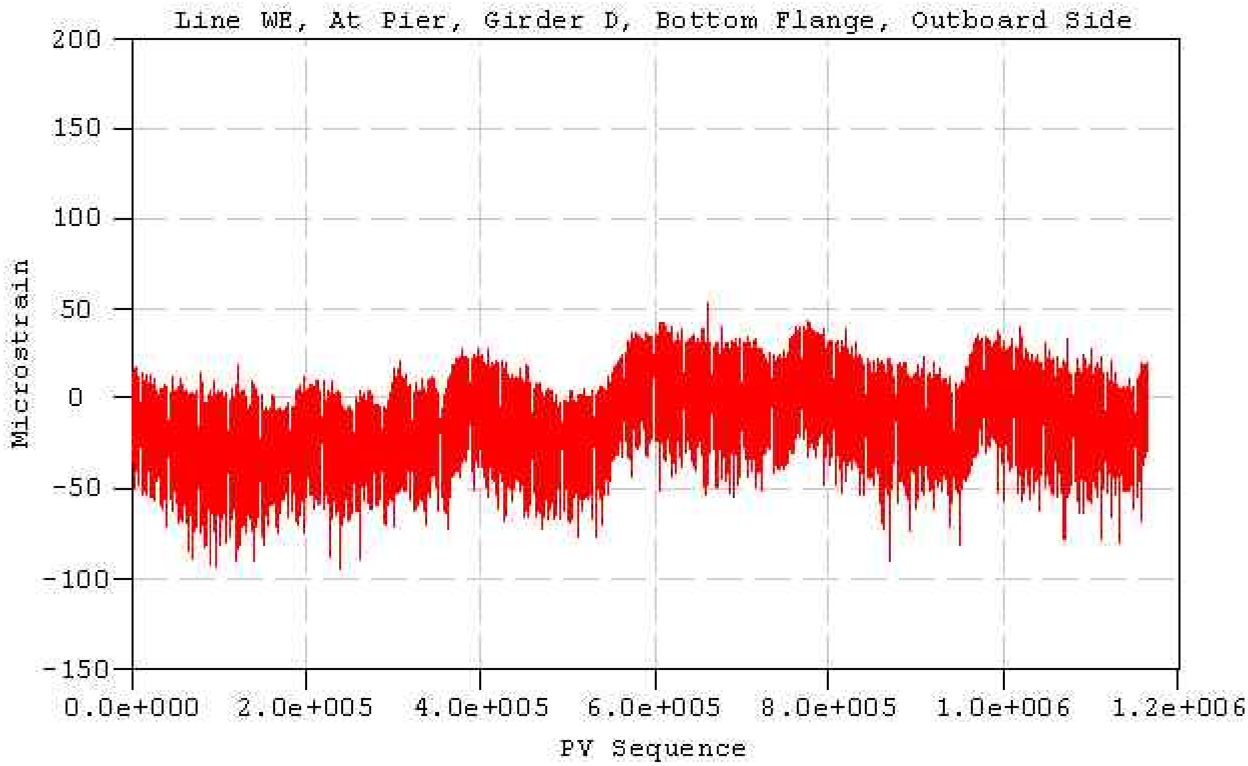

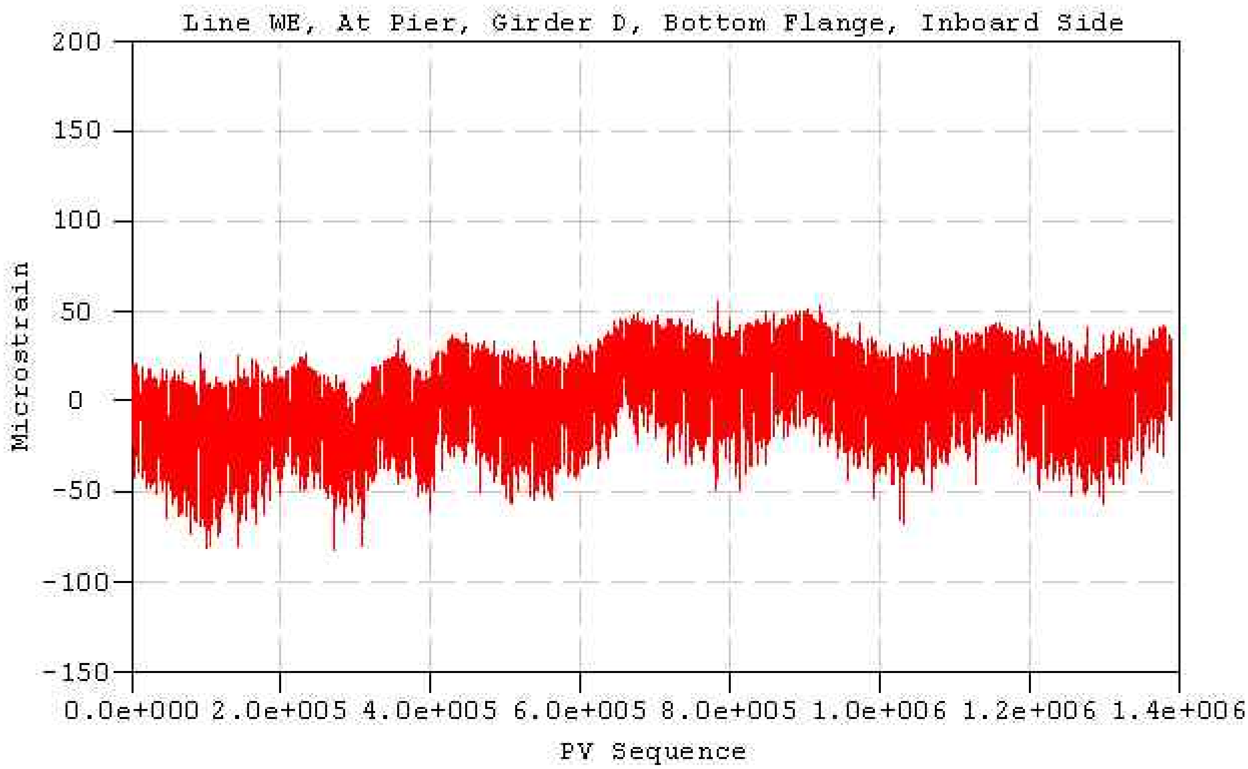

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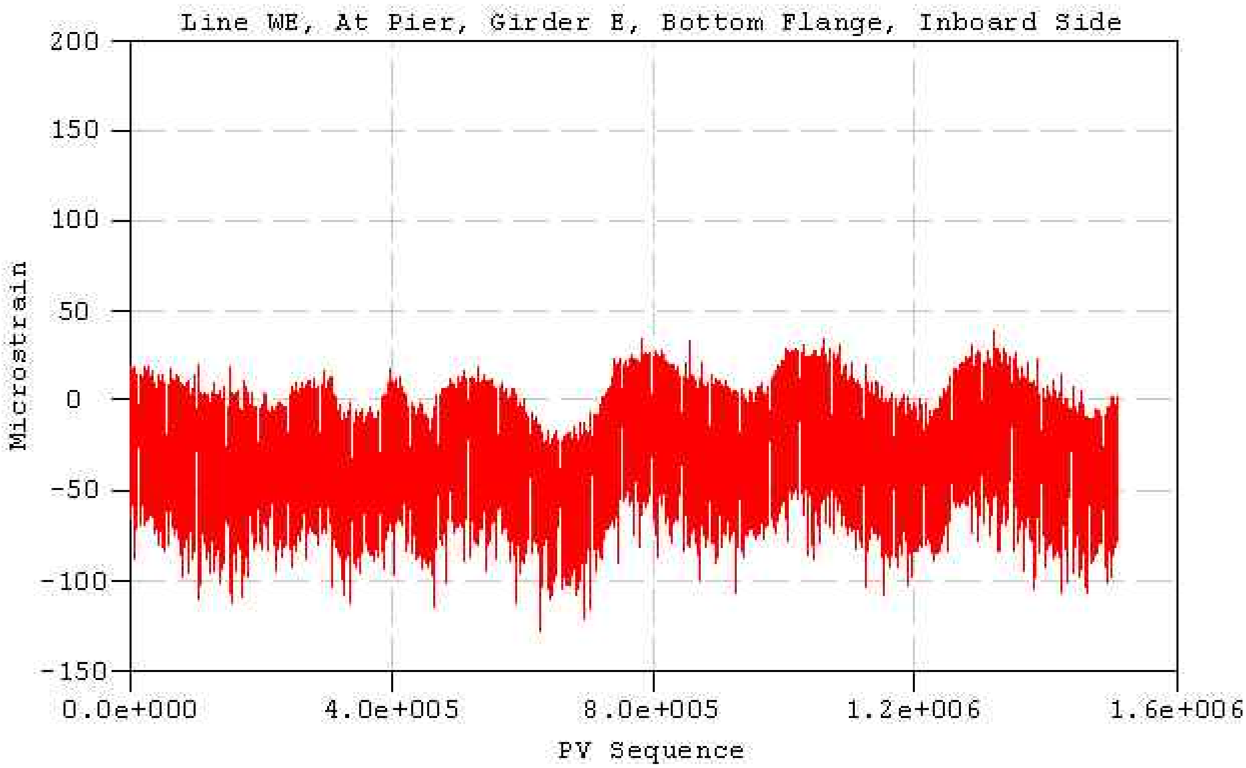

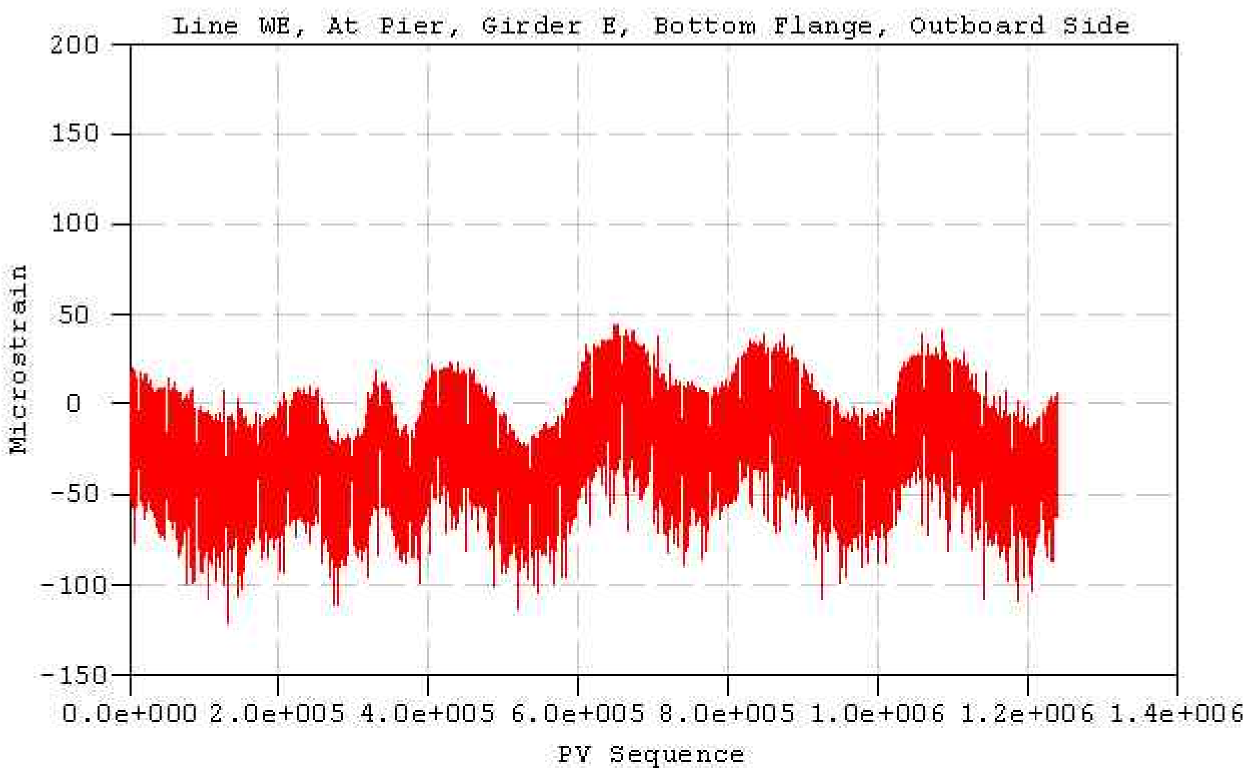

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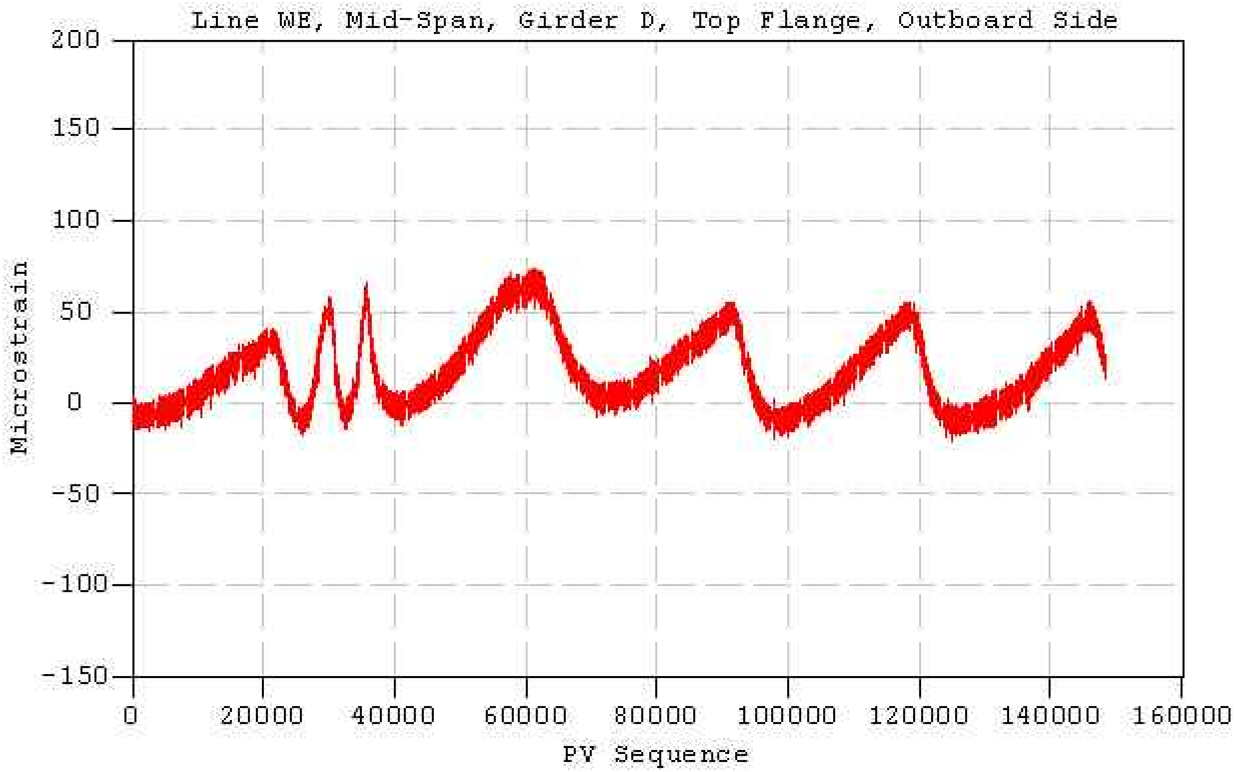

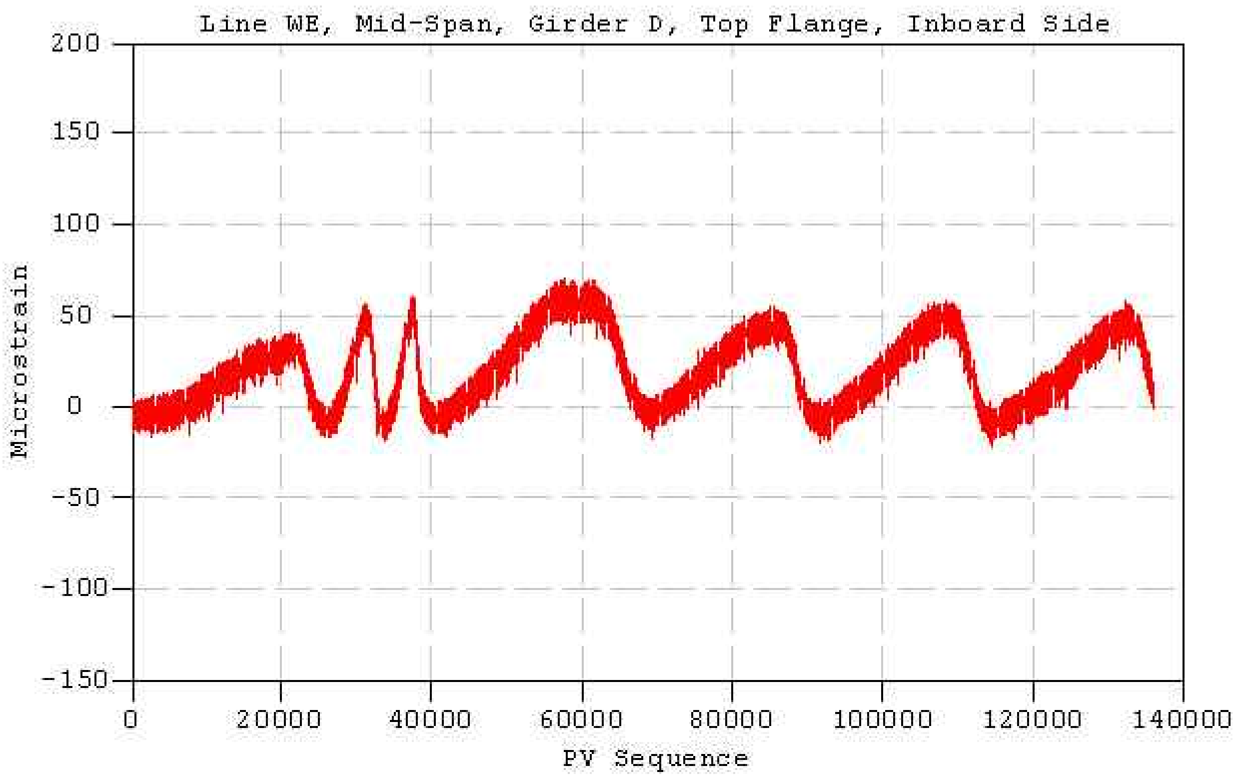

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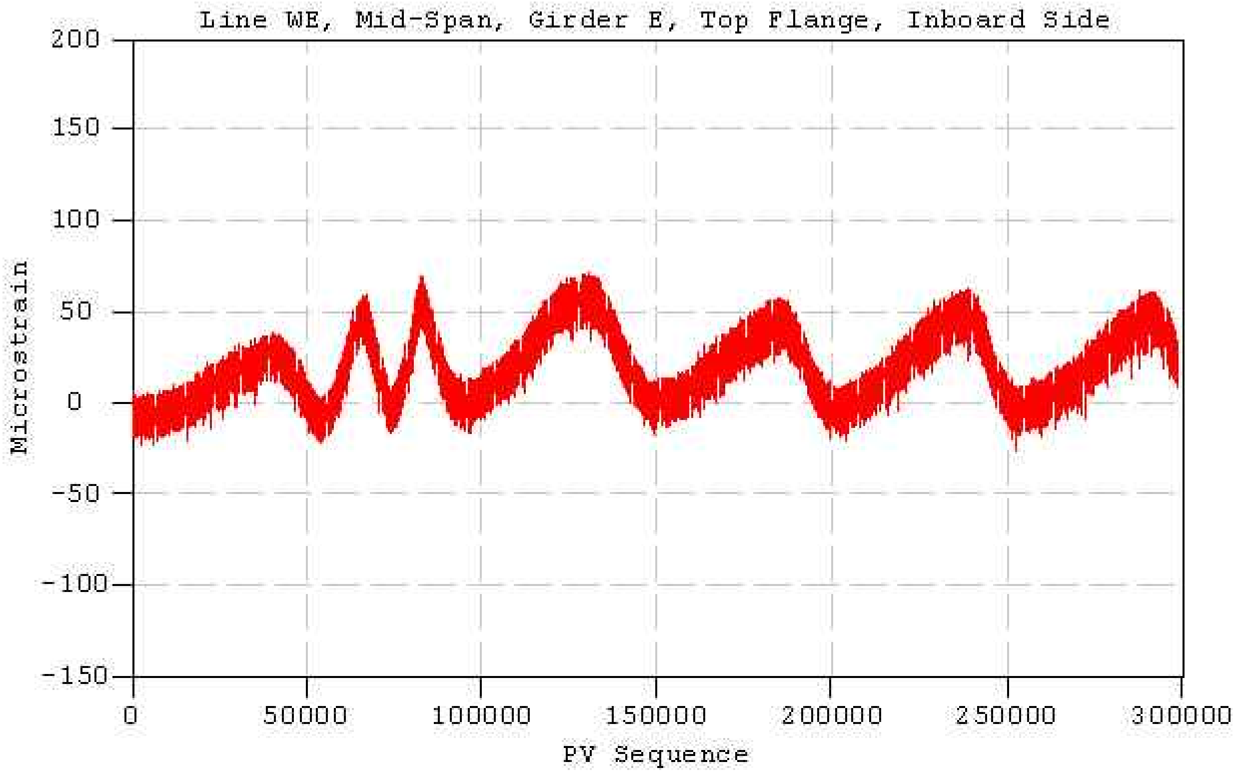

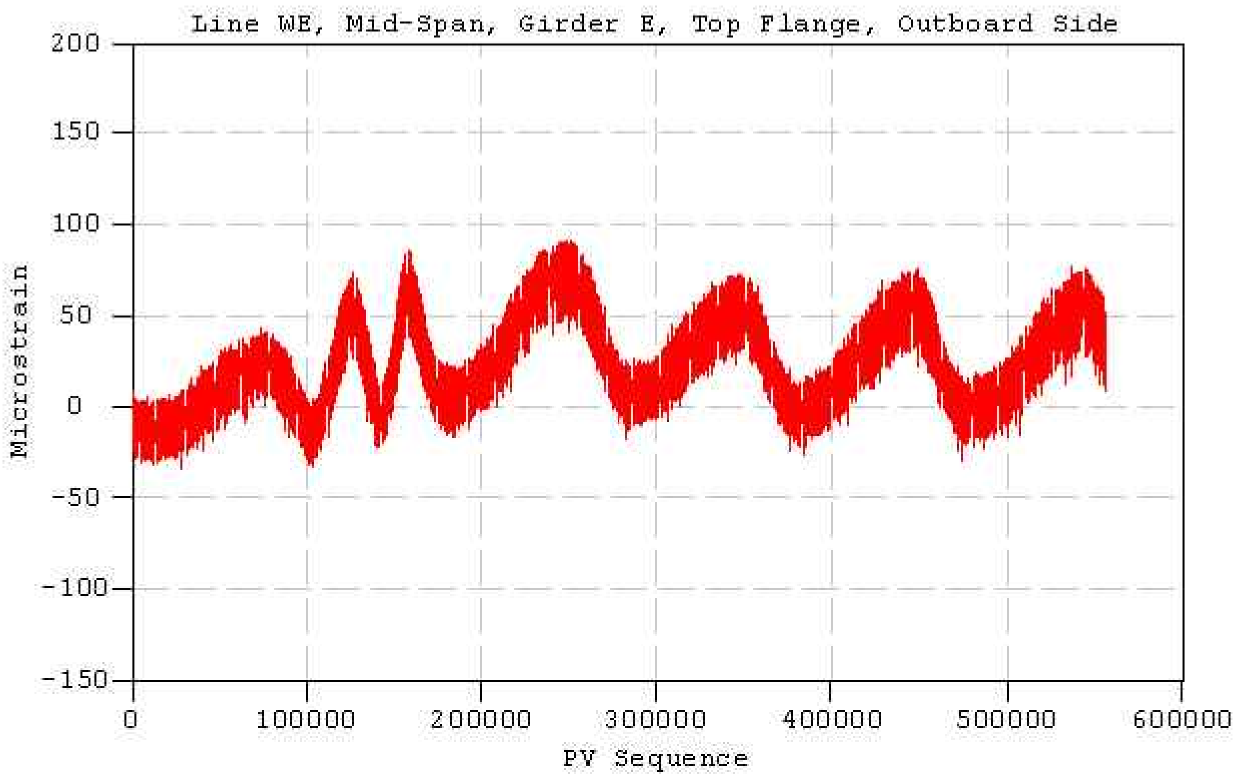

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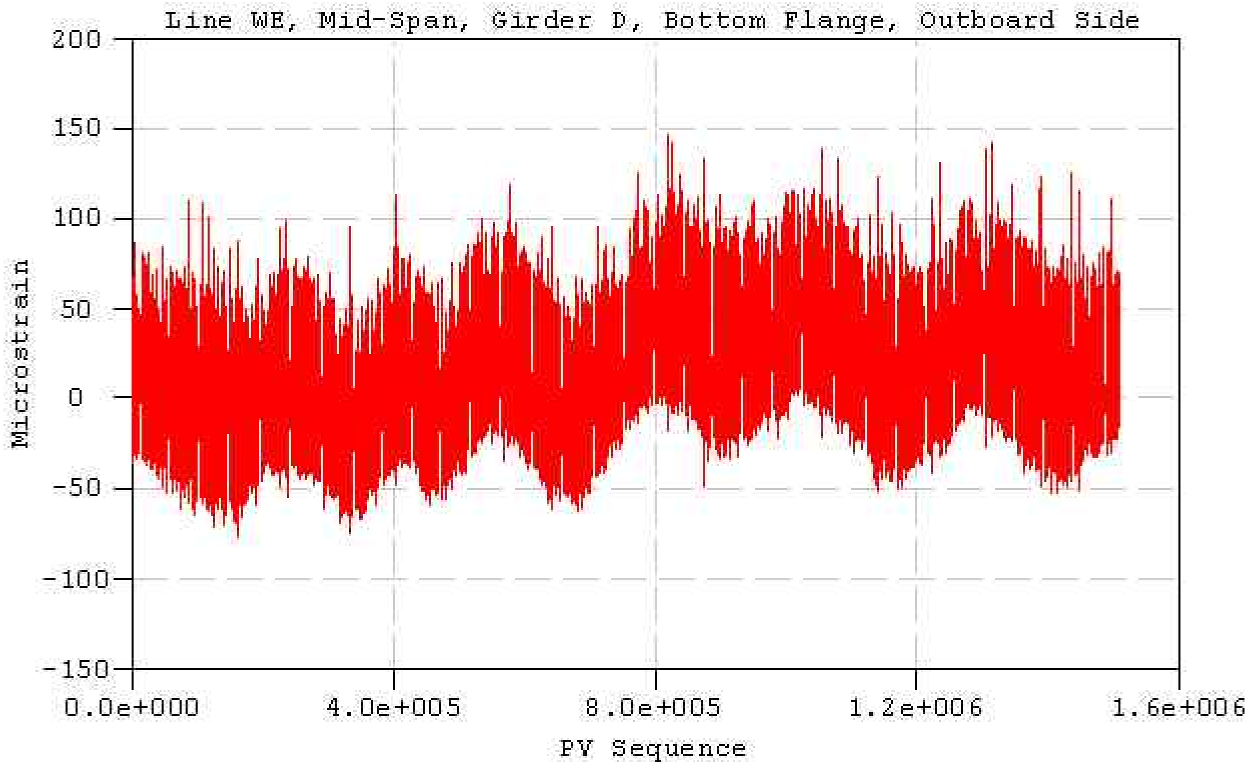

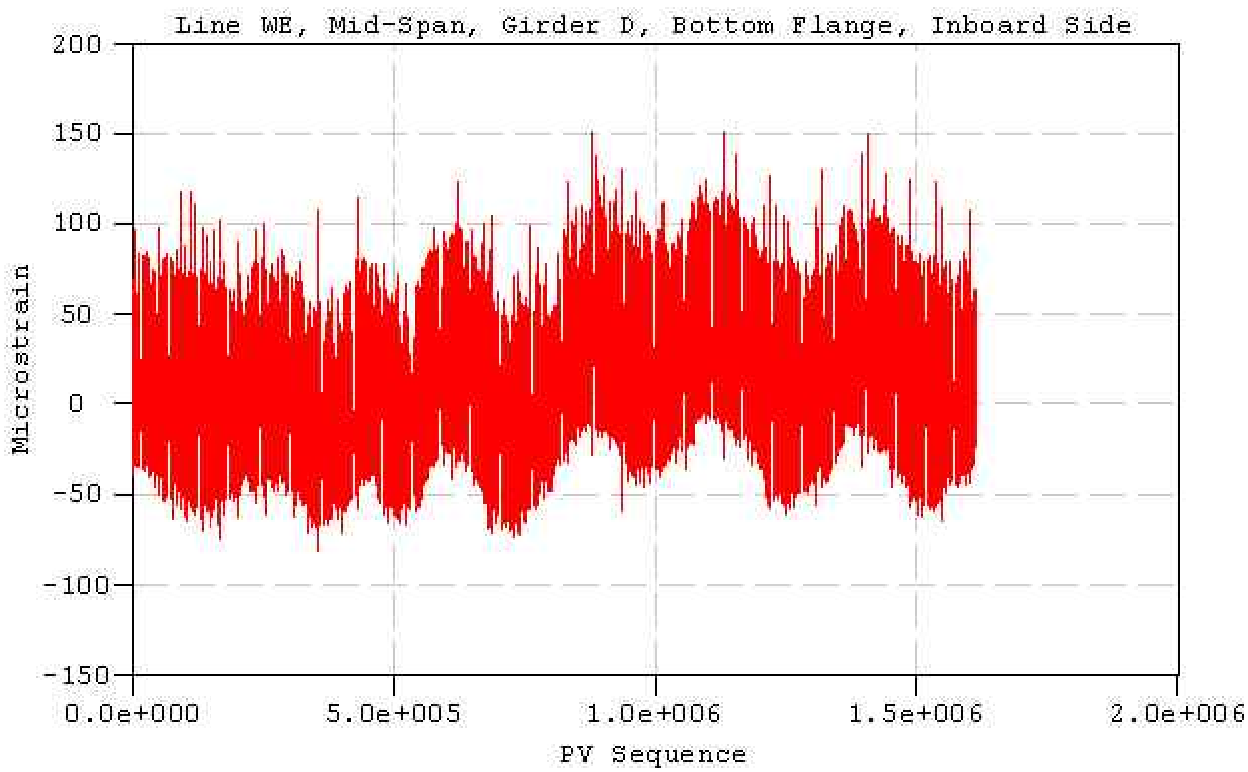

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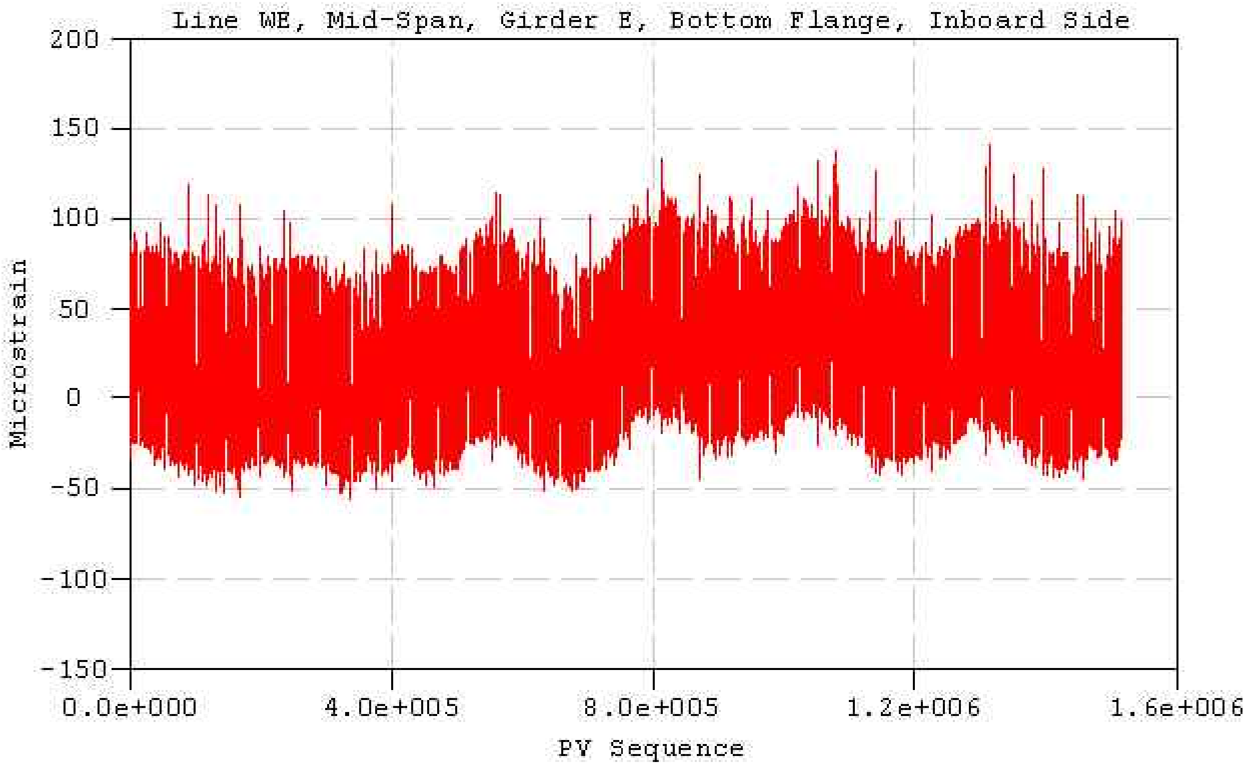

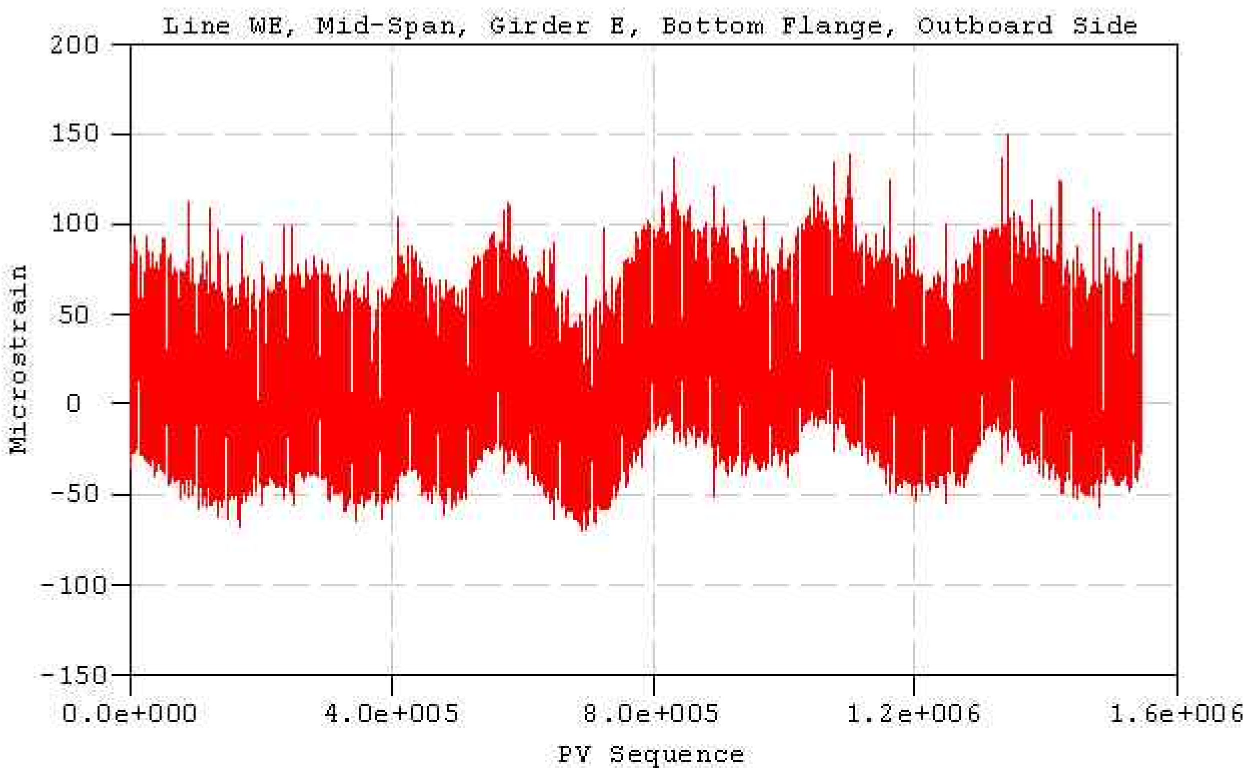

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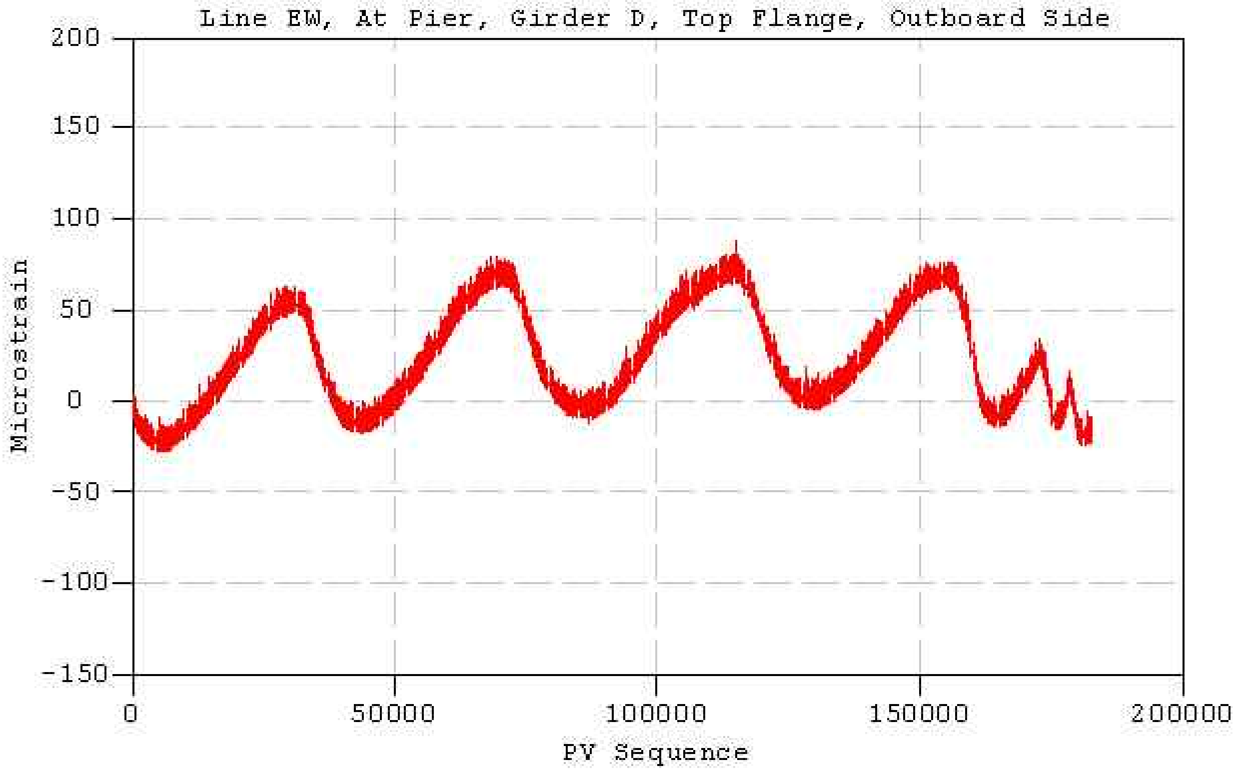

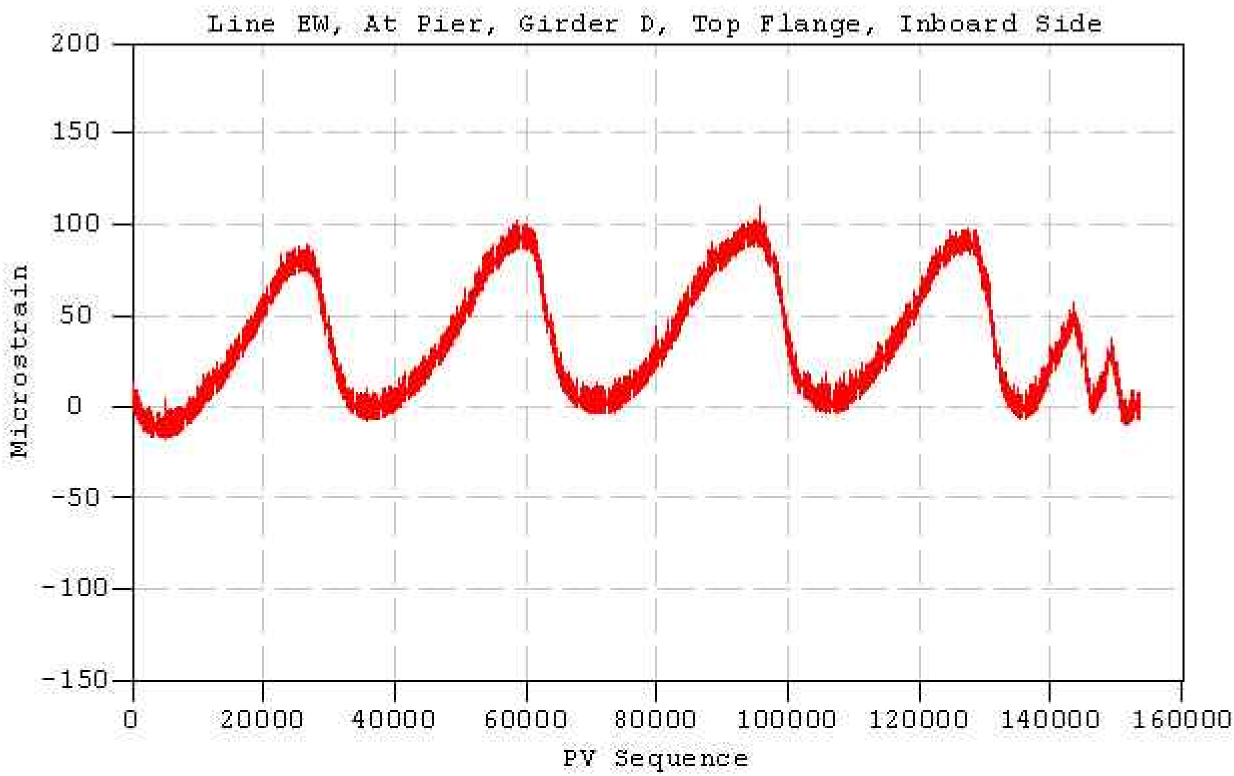

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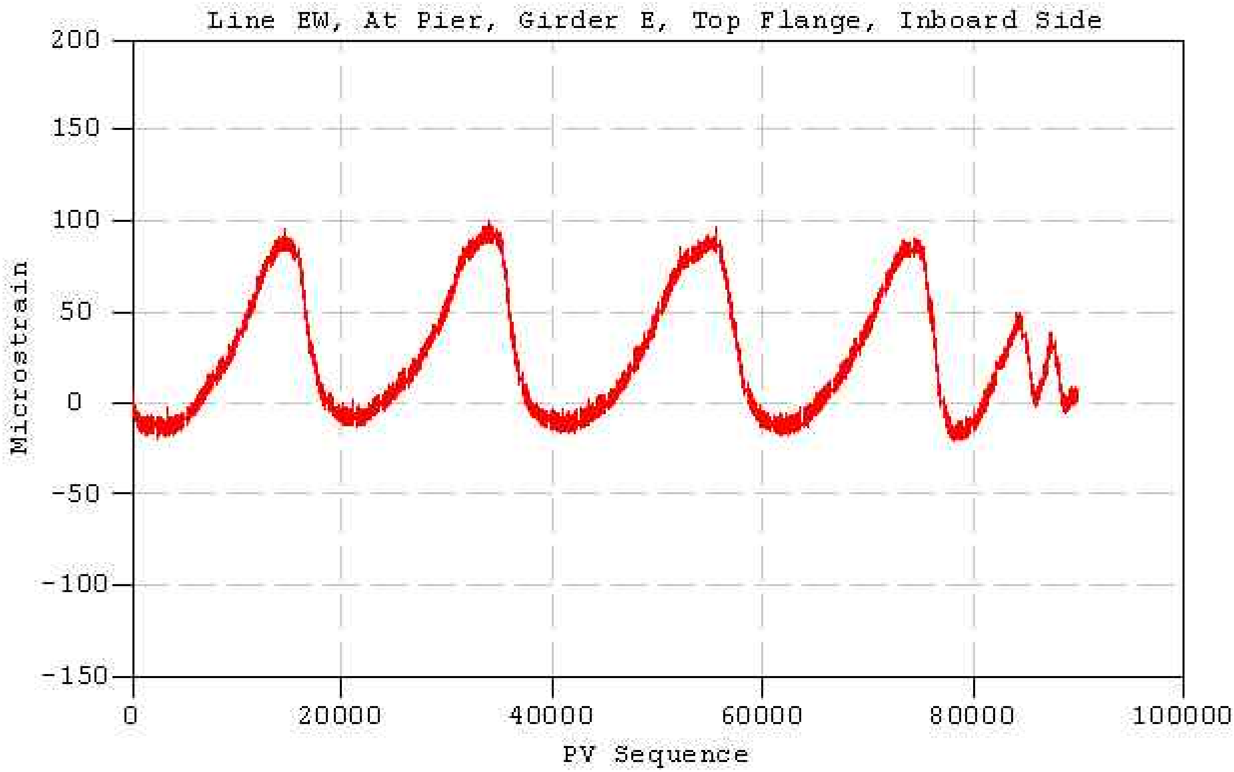

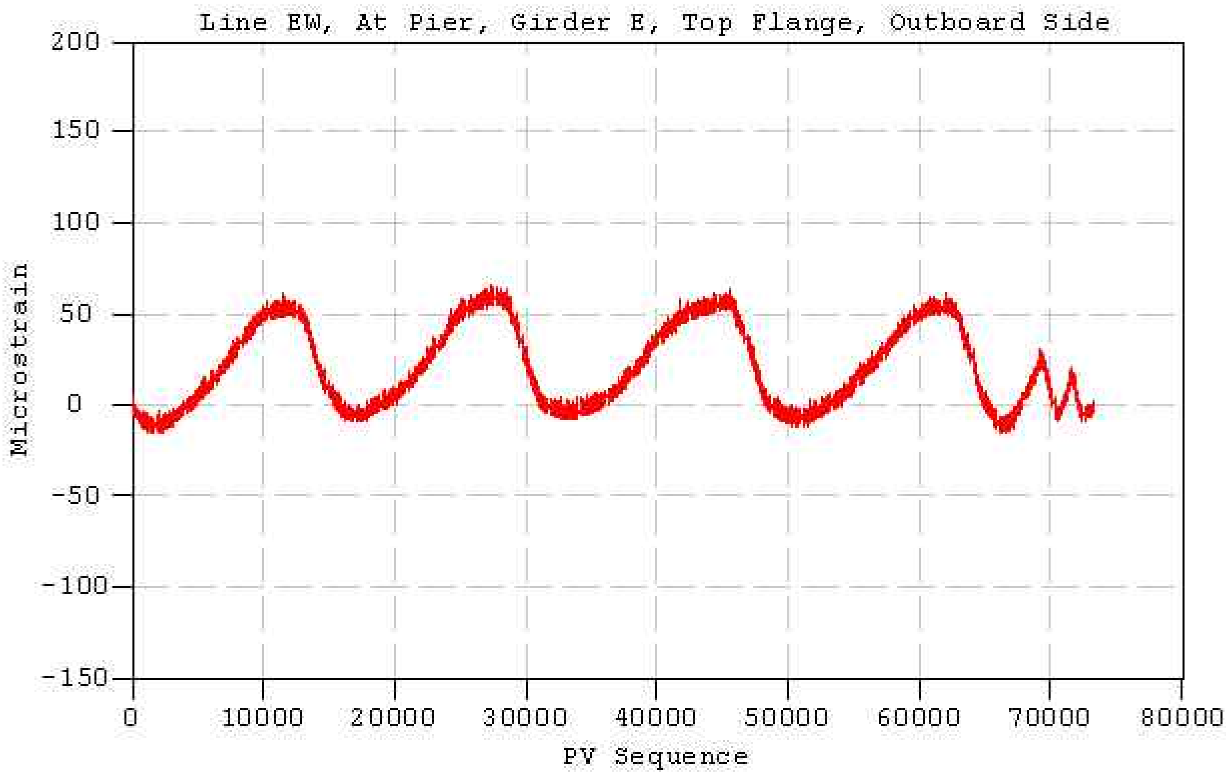

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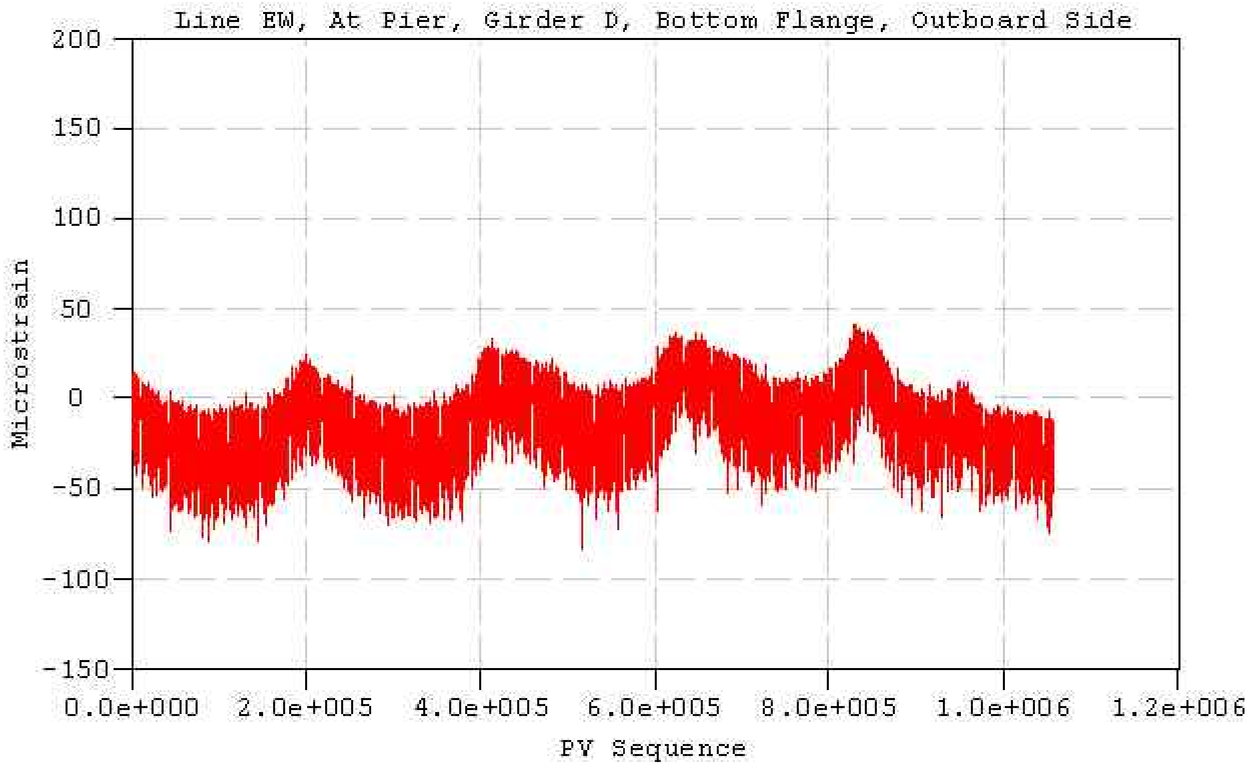

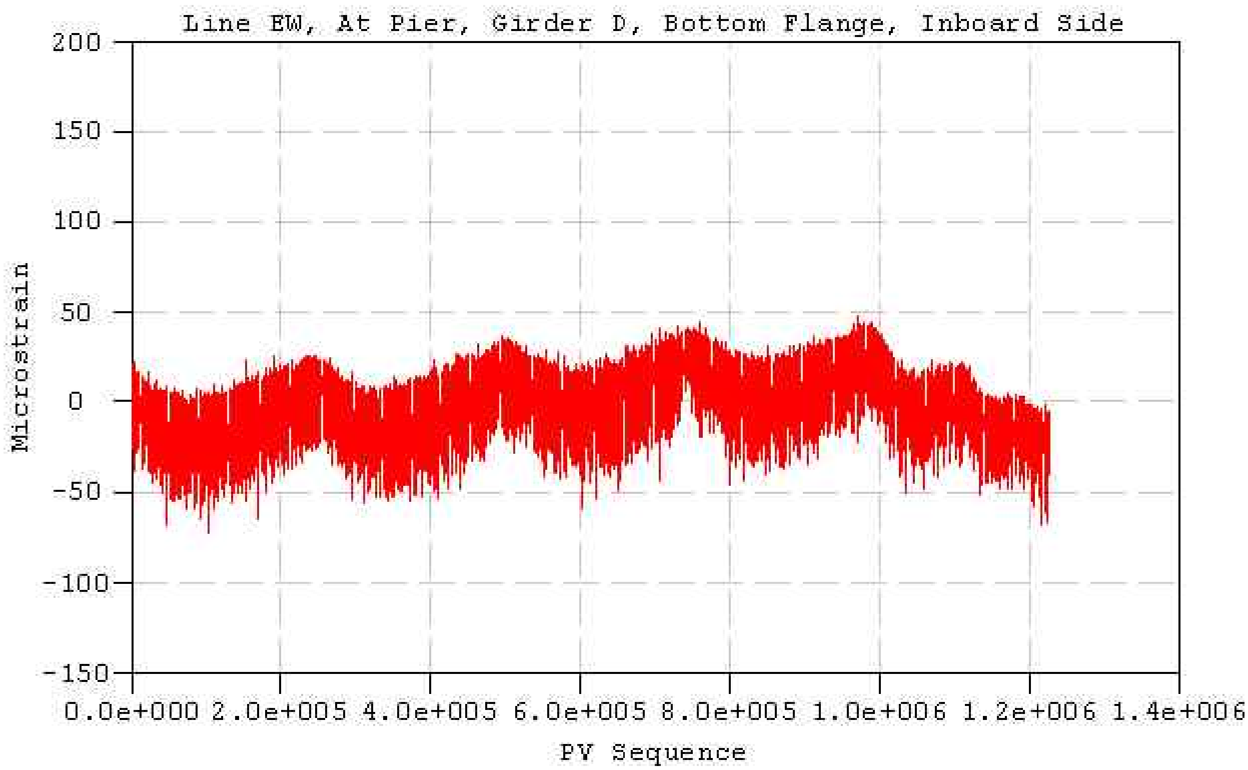

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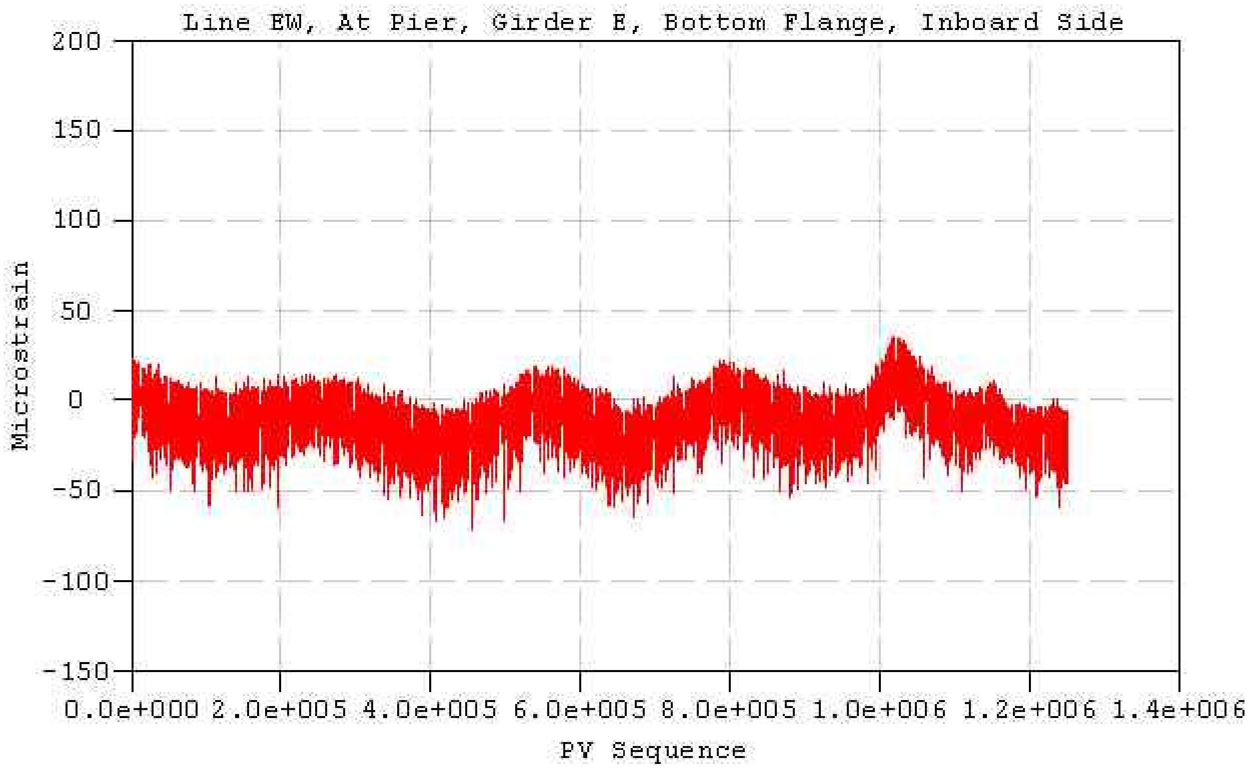

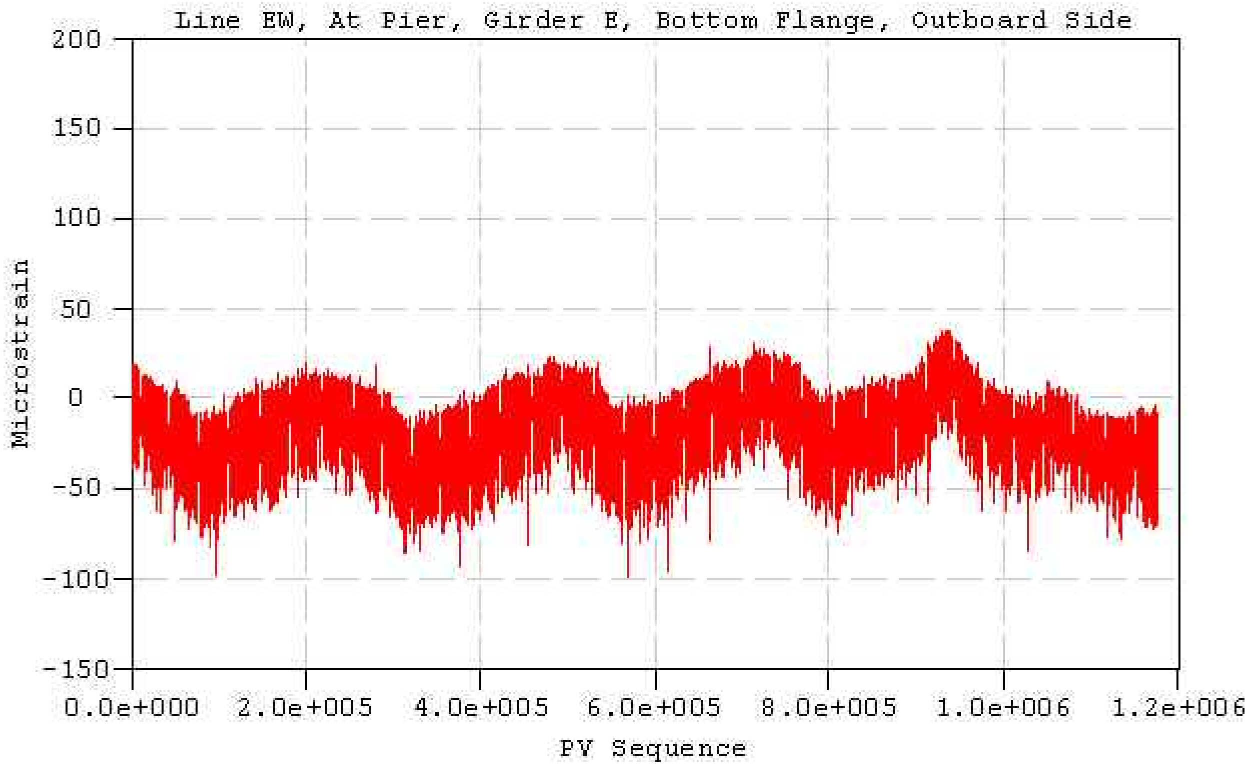

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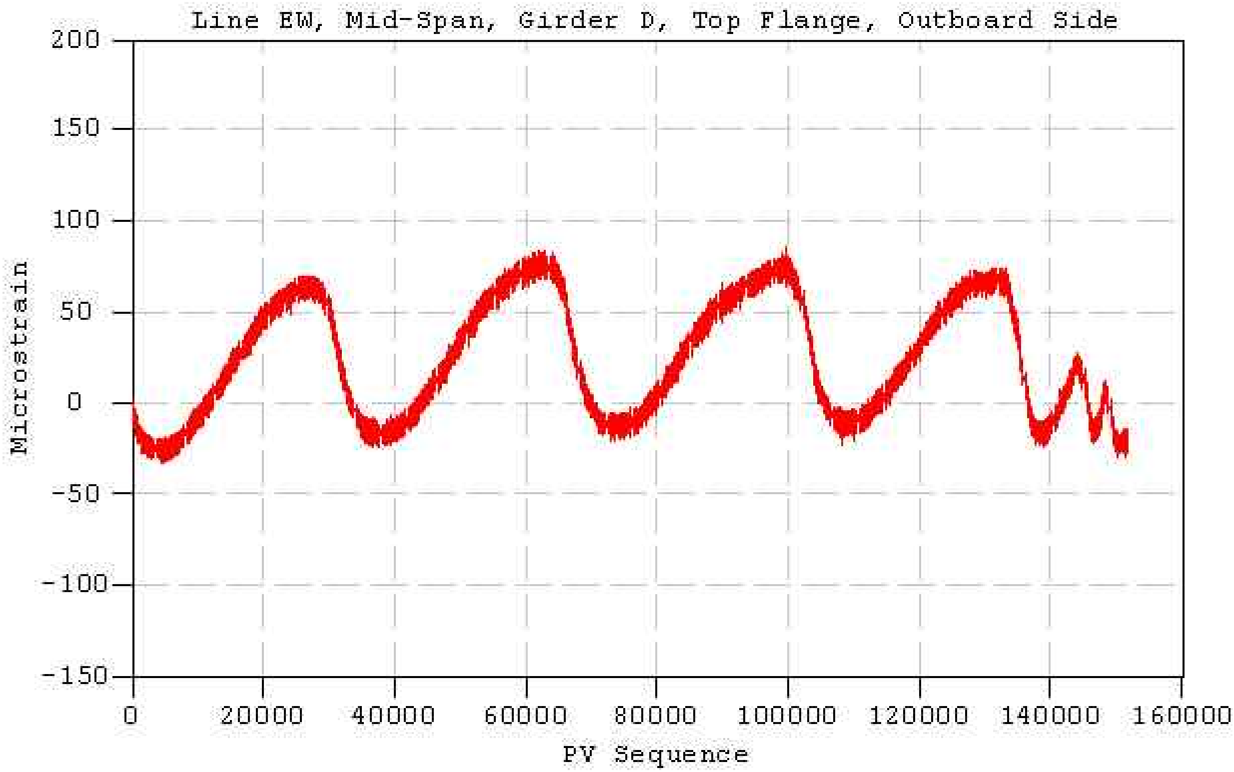

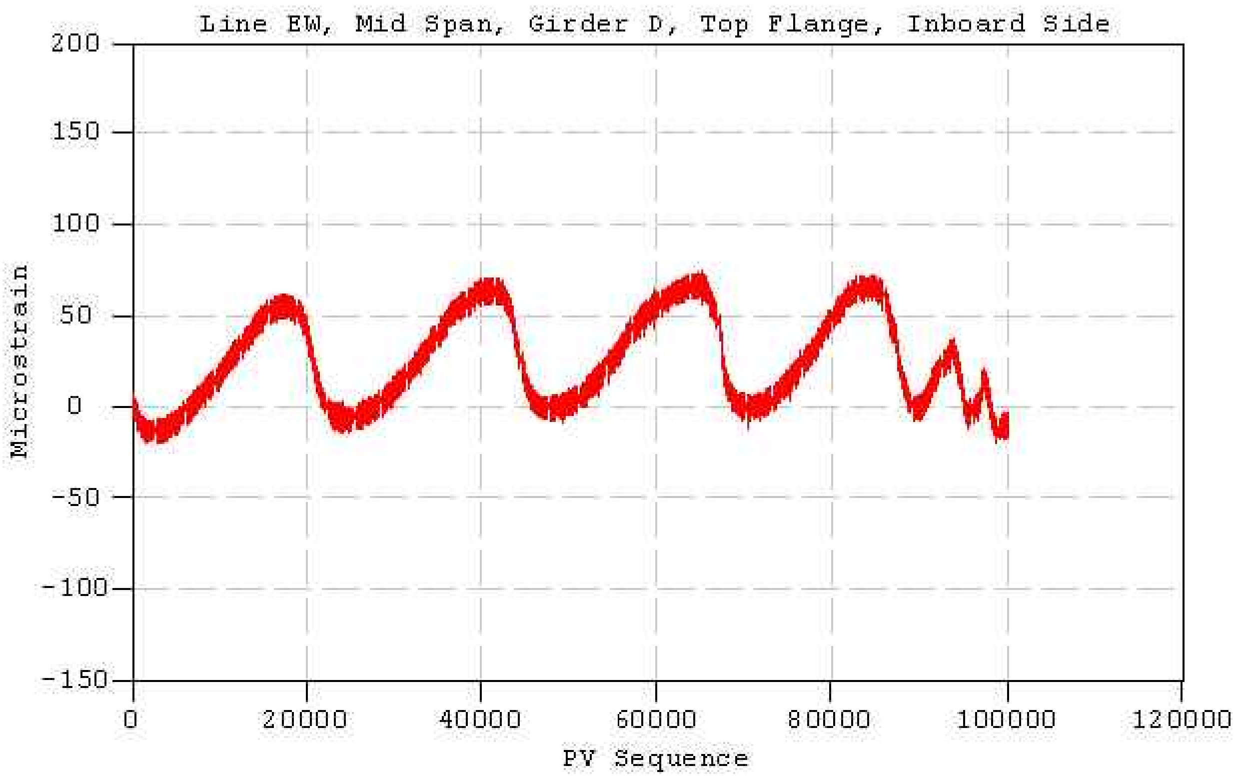

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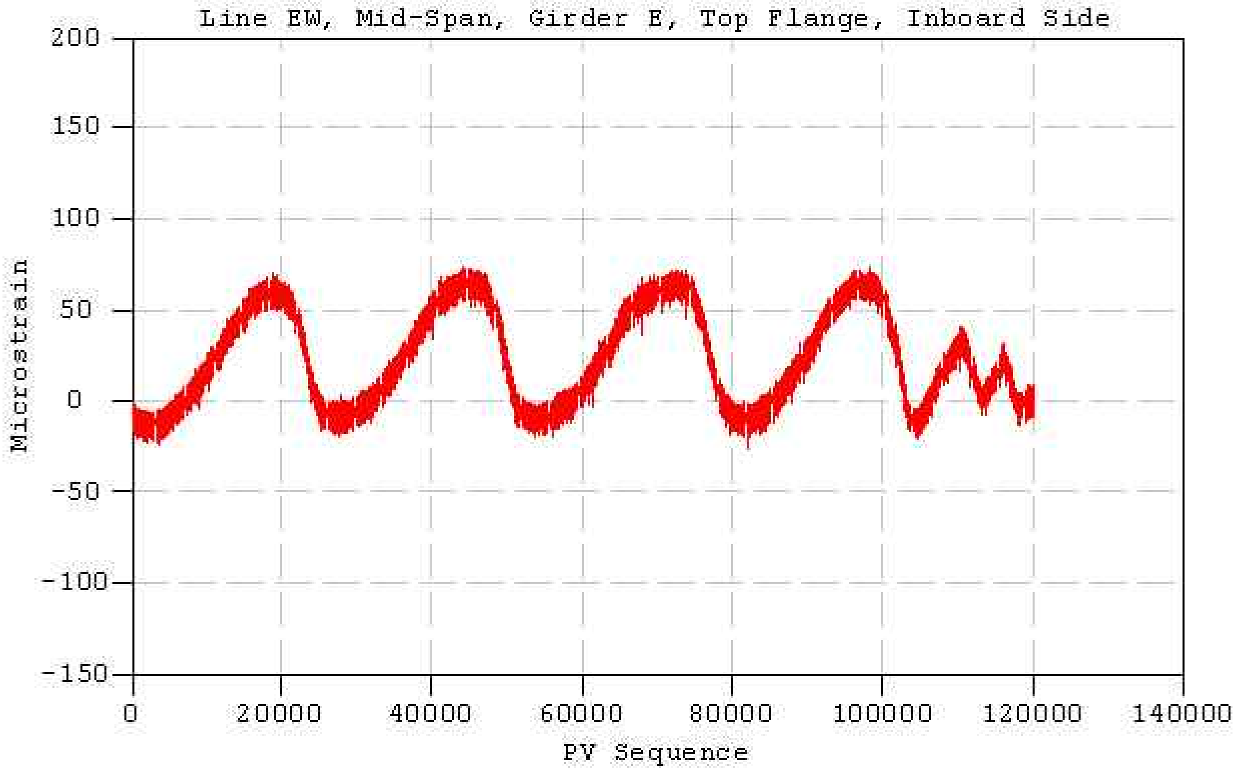

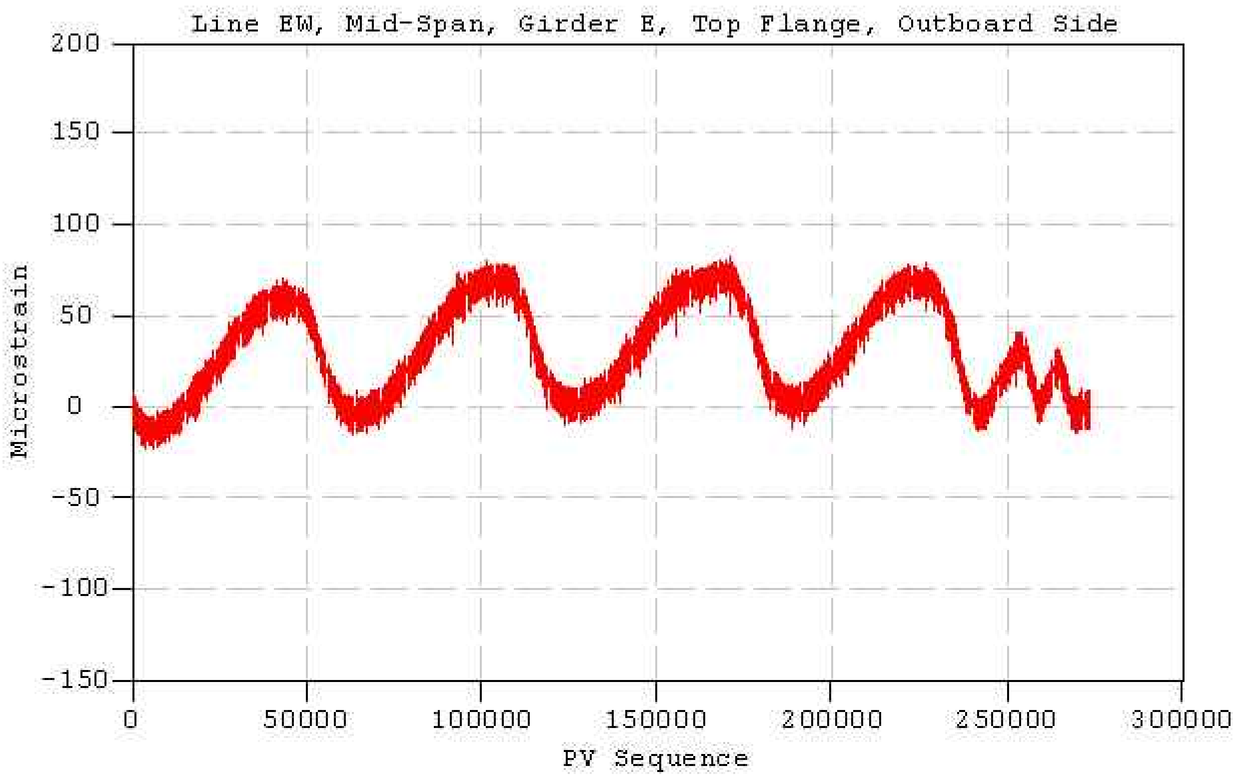

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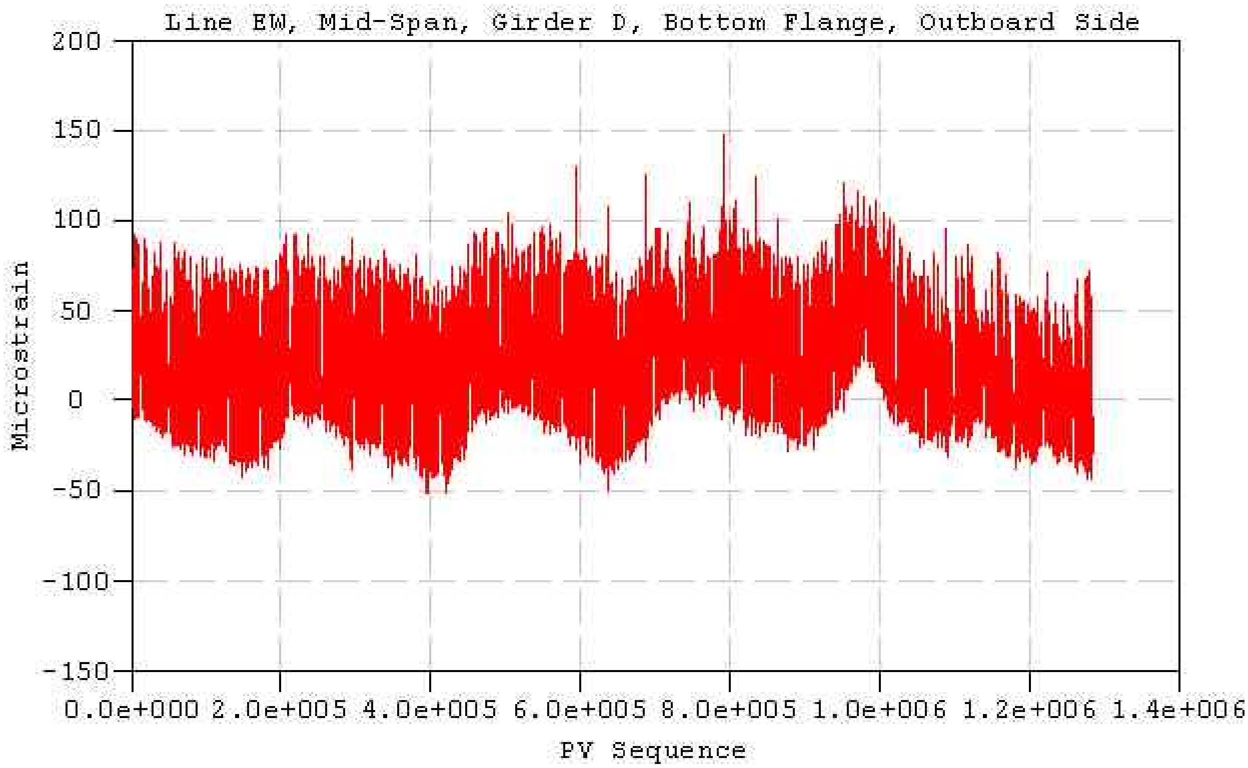

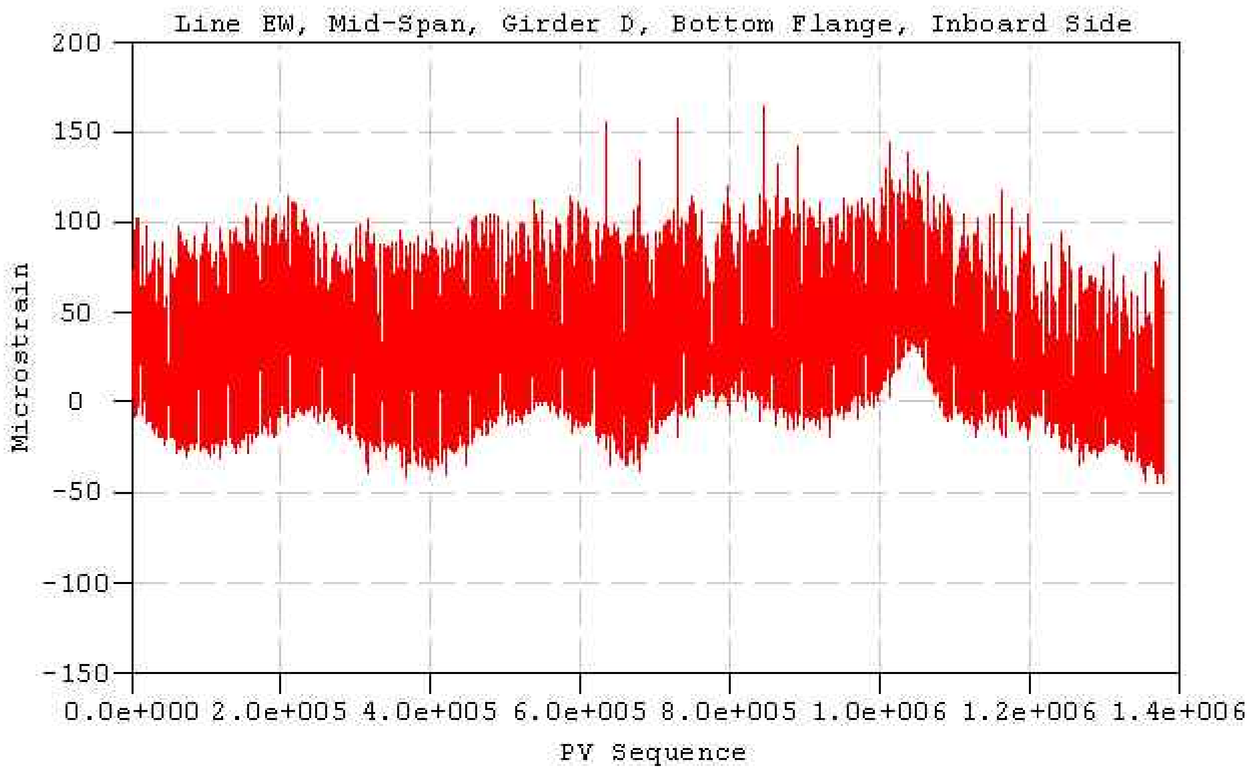

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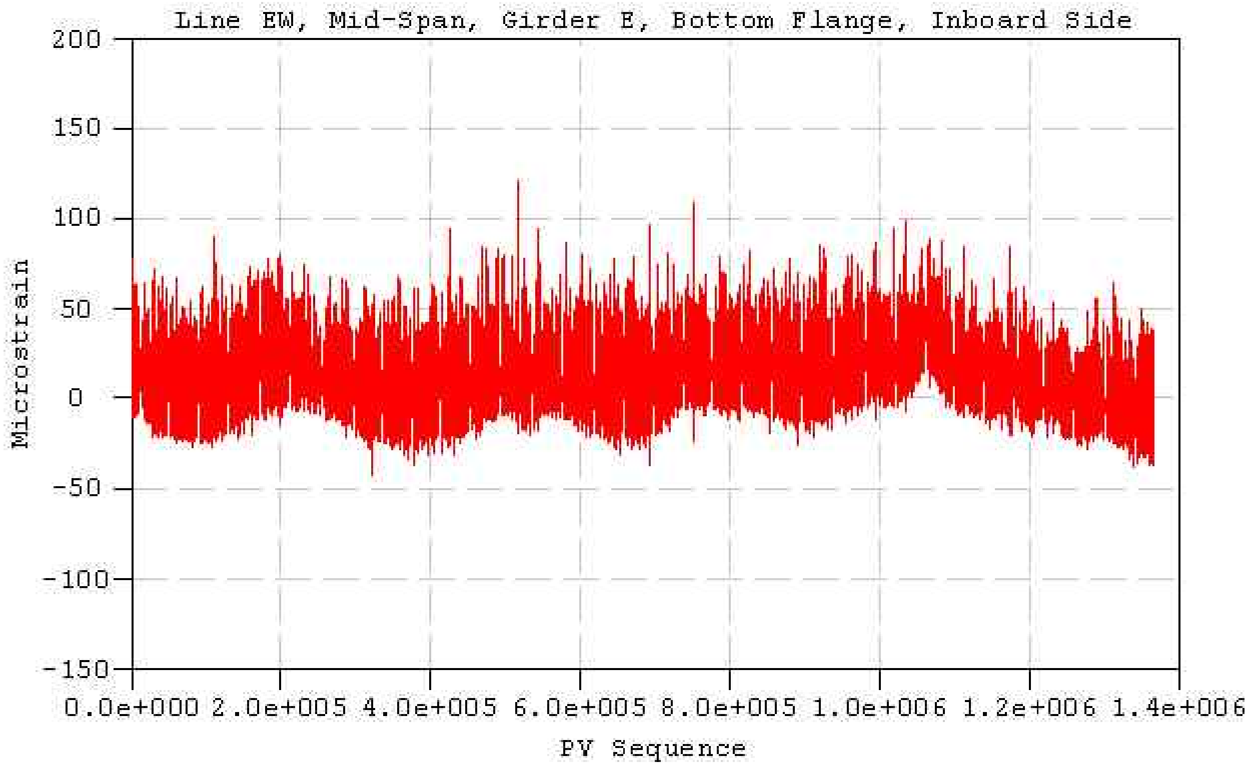


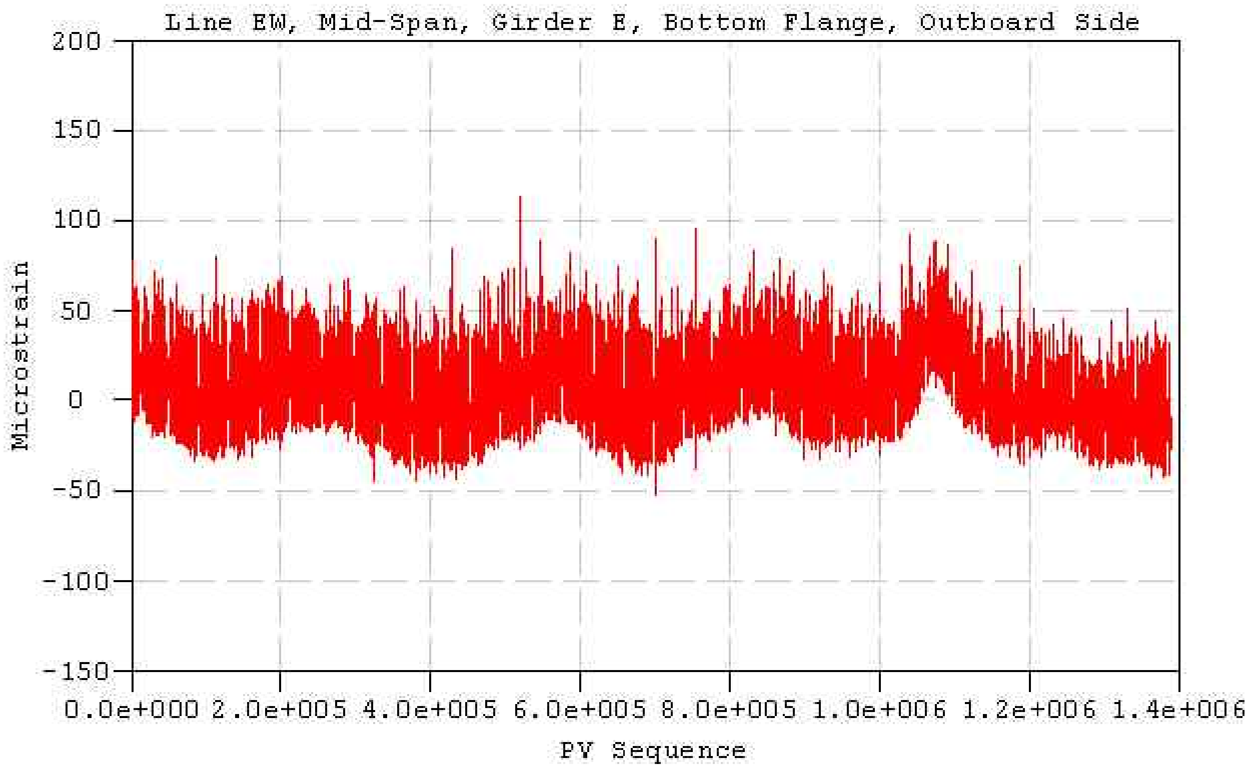


Table 8. Root-Mean-Cube Effective Strain Range (microstrain), Bridge 9268A, Line SW

Channel	Gage No.	Girder	Flange		Strain Cut-Off		
					2 μ strain	5 μ strain	10 μ strain
Next to Pier							
1630	1	A	Top	Outboard	4	9	37
1631	2	A	Top	Inboard	7	17	68
1702	3	B	Top	Inboard	6	16	58
1707	4	B	Top	Outboard	5	10	23
1698	5	A	Bottom	Outboard	11	16	21
1699	6	A	Bottom	Inboard	11	16	22
1708	7	B	Bottom	Inboard	10	14	20
1712	8	B	Bottom	Outboard	10	14	20
Midspan							
1952	9	A	Top	Outboard	7	35	51
1953	10	A	Top	Inboard	7	13	36
1960	11	B	Top	Inboard	7	11	19
1961	12	B	Top	Outboard	6	10	17
1954	13	A	Bottom	Outboard	15	22	33
1959	14	A	Bottom	Inboard	14	21	32
1962	15	B	Bottom	Inboard	13	18	28
1967	16	B	Bottom	Outboard	13	19	28

Table 9. Root-Mean-Cube Effective Strain Range (microstrain), Bridge 9268B, Line WS

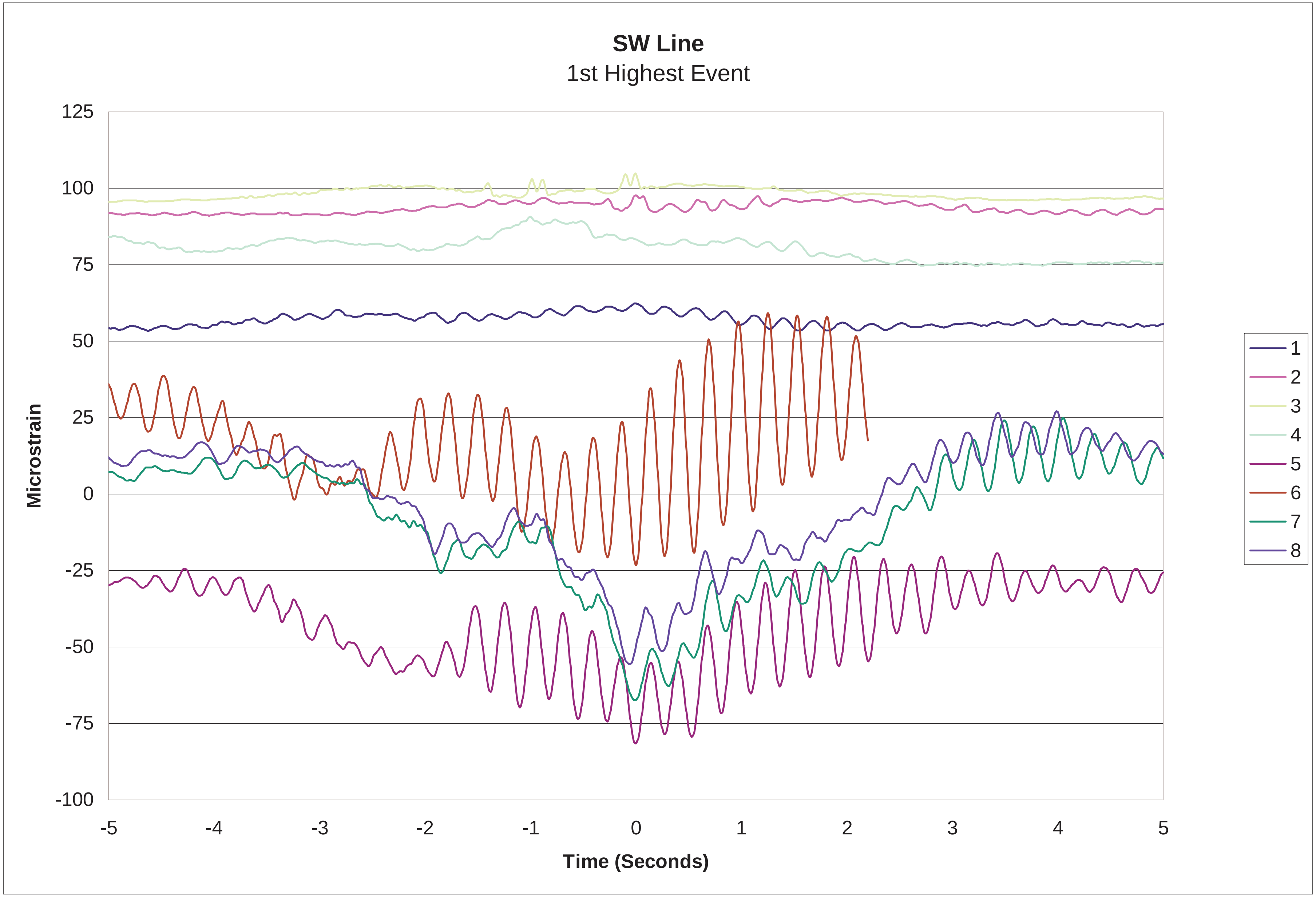
Channel	Gage No.	Girder	Flange		Strain Cut-Off 2 μ strain	5 μ strain	10 μ strain
Next to Pier							
1630	1	A	Top	Outboard	8	12	18
1631	2	A	Top	Inboard	7	10	17
1702	3	B	Top	Inboard	5	9	17
1707	4	B	Top	Outboard	5	8	14
1698	5	A	Bottom	Outboard	15	19	25
1699	6	A	Bottom	Inboard	16	20	26
1708	7	B	Bottom	Inboard	14	18	23
1712	8	B	Bottom	Outboard	13	17	22
Midspan							
1952	9	A	Top	Outboard	8	14	21
1953	10	A	Top	Inboard	8	12	17
1960	11	B	Top	Inboard	8	13	17
1961	12	B	Top	Outboard	5	8	28
1954	13	A	Bottom	Outboard	26	33	44
1959	14	A	Bottom	Inboard	26	33	44
1962	15	B	Bottom	Inboard	19	24	31
1967	16	B	Bottom	Outboard	19	24	32

Table 10. Root-Mean-Cube Effective Strain Range (microstrain), Bridge 9268E, Line WE

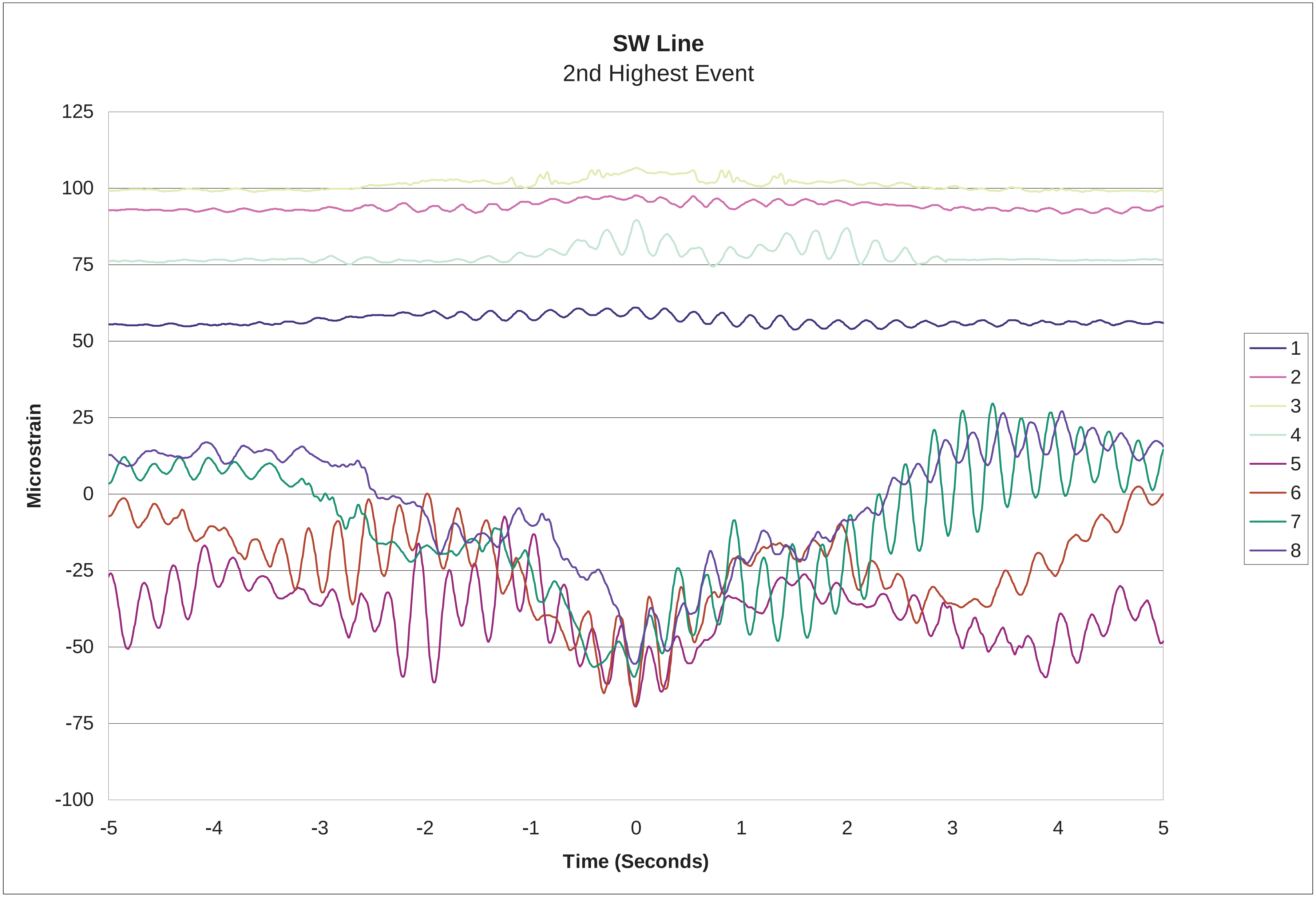
Channel	Gage No.	Girder	Flange		Strain Cut-Off		
					2 μ strain	5 μ strain	10 μ strain
Next to Pier							
1630	1	D	Top	Outboard	5	10	16
1631	2	D	Top	Inboard	6	10	16
1702	3	E	Top	Inboard	7	12	19
1707	4	E	Top	Outboard	7	12	19
1698	5	D	Bottom	Outboard	11	16	23
1699	6	D	Bottom	Inboard	11	15	21
1708	7	E	Bottom	Inboard	13	17	24
1712	8	E	Bottom	Outboard	12	17	26
Midspan							
1952	9	D	Top	Outboard	5	11	15
1953	10	D	Top	Inboard	7	12	16
1960	11	E	Top	Inboard	7	14	19
1961	12	E	Top	Outboard	8	14	22
1954	13	D	Bottom	Outboard	17	23	32
1959	14	D	Bottom	Inboard	18	24	33
1962	15	E	Bottom	Inboard	17	22	32
1967	16	E	Bottom	Outboard	17	23	32

Table 11. Root-Mean-Cube Effective Strain Range (microstrain), Bridge 9268W, Line EW

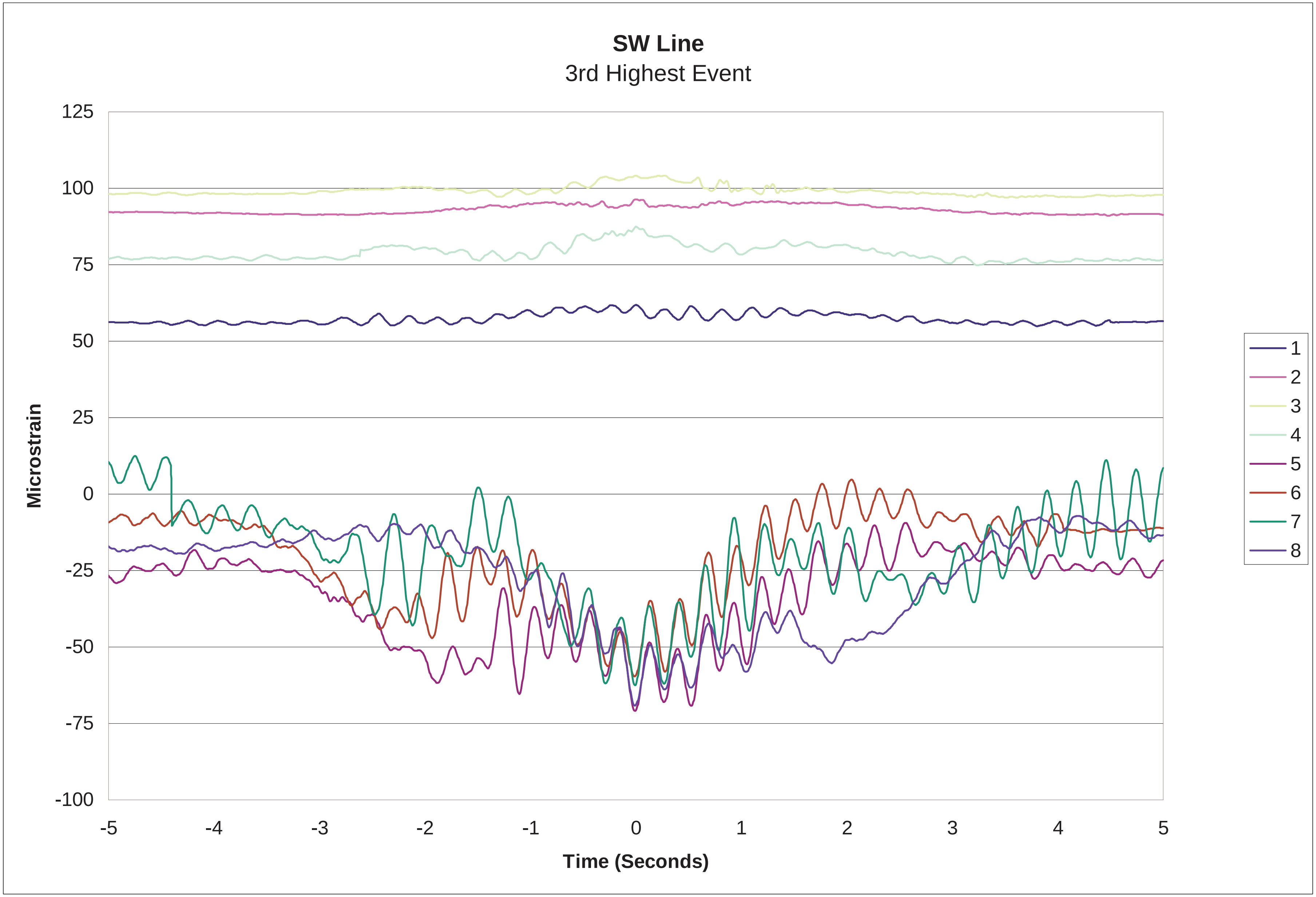
Channel	Gage No.	Girder	Flange		Strain Cut-Off 2 μ strain	5 μ strain	10 μ strain
Next to Pier							
1630	1	D	Top	Outboard	5	10	17
1631	2			Inboard	5	10	19
1702	3	E		Inboard	6	13	46
1707	4			Outboard	4	10	34
1698	5	D	Bottom	Outboard	10	13	21
1699	6			Inboard	10	13	20
1708	7	E		Inboard	9	12	17
1712	8			Outboard	10	14	21
Midspan							
1952	9	D	Top	Outboard	5	10	18
1953	10			Inboard	5	9	16
1960	11	E		Inboard	6	11	16
1961	12			Outboard	5	10	14
1954	13	D	Bottom	Outboard	14	19	29
1959	14			Inboard	16	21	30
1962	15	E		Inboard	11	15	22
1967	16			Outboard	11	15	22



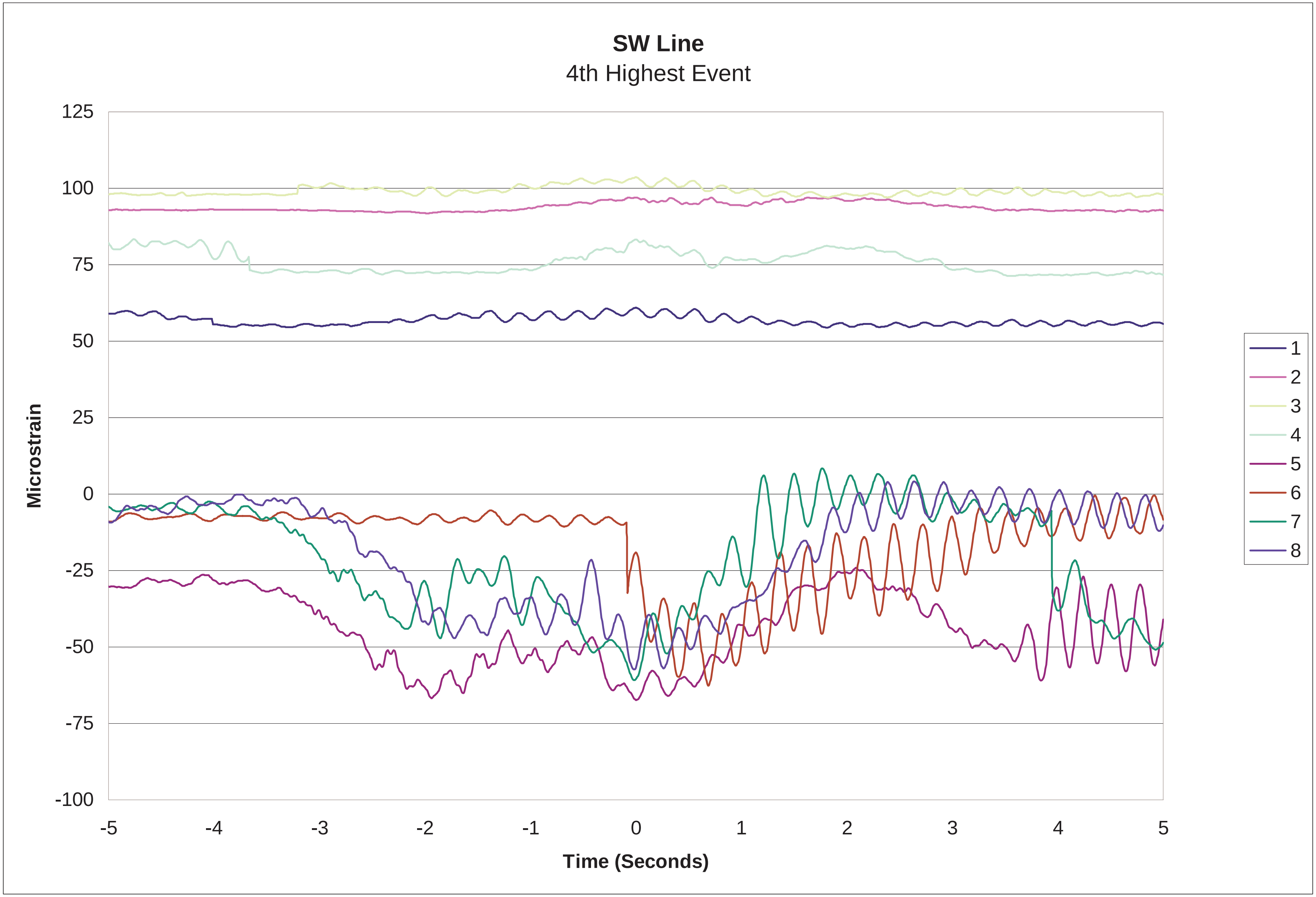
SW Line
1st Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8



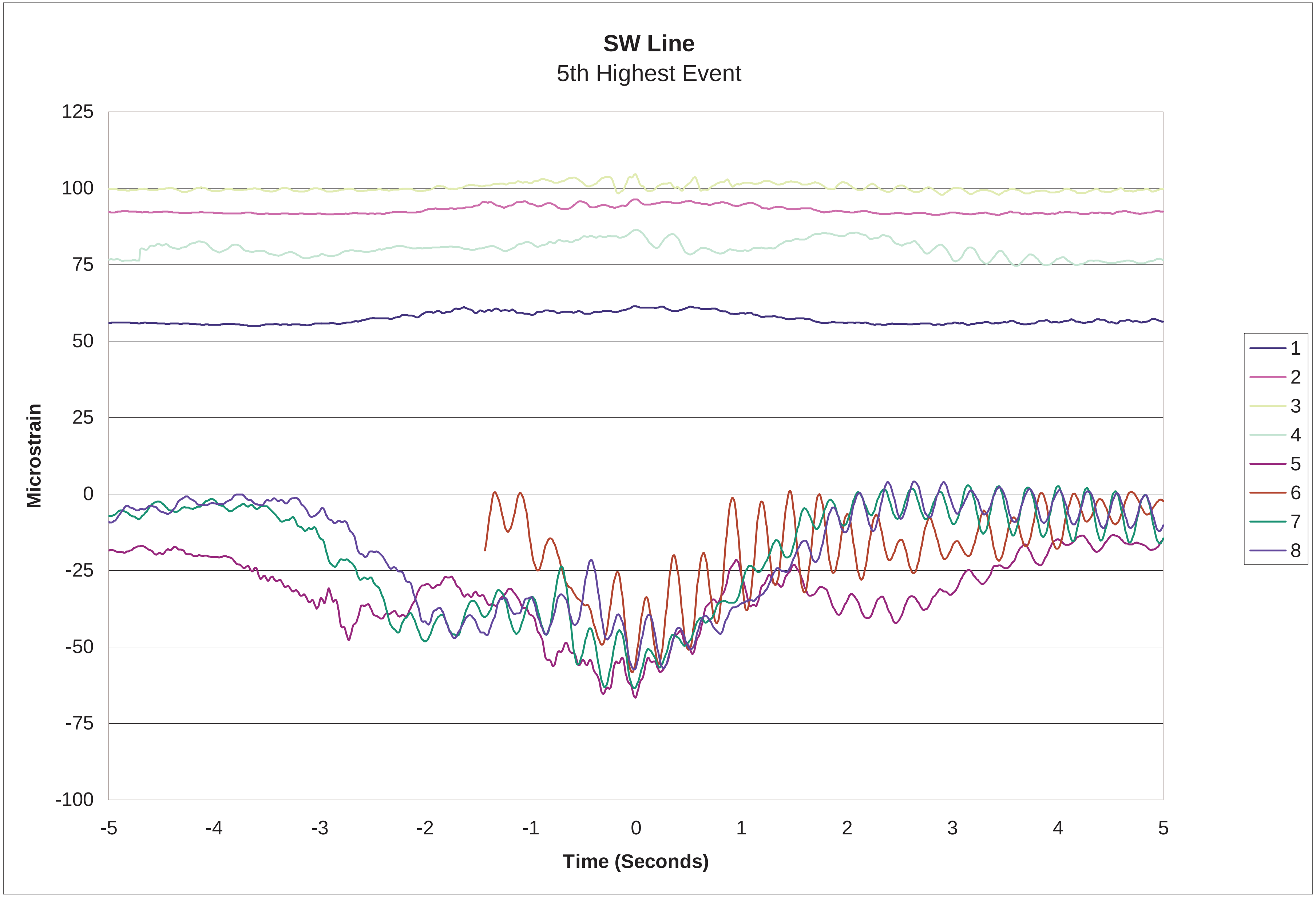
SW Line
2nd Highest Event
125
100
75
50
25
0
-25
-50
-75
-100
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8



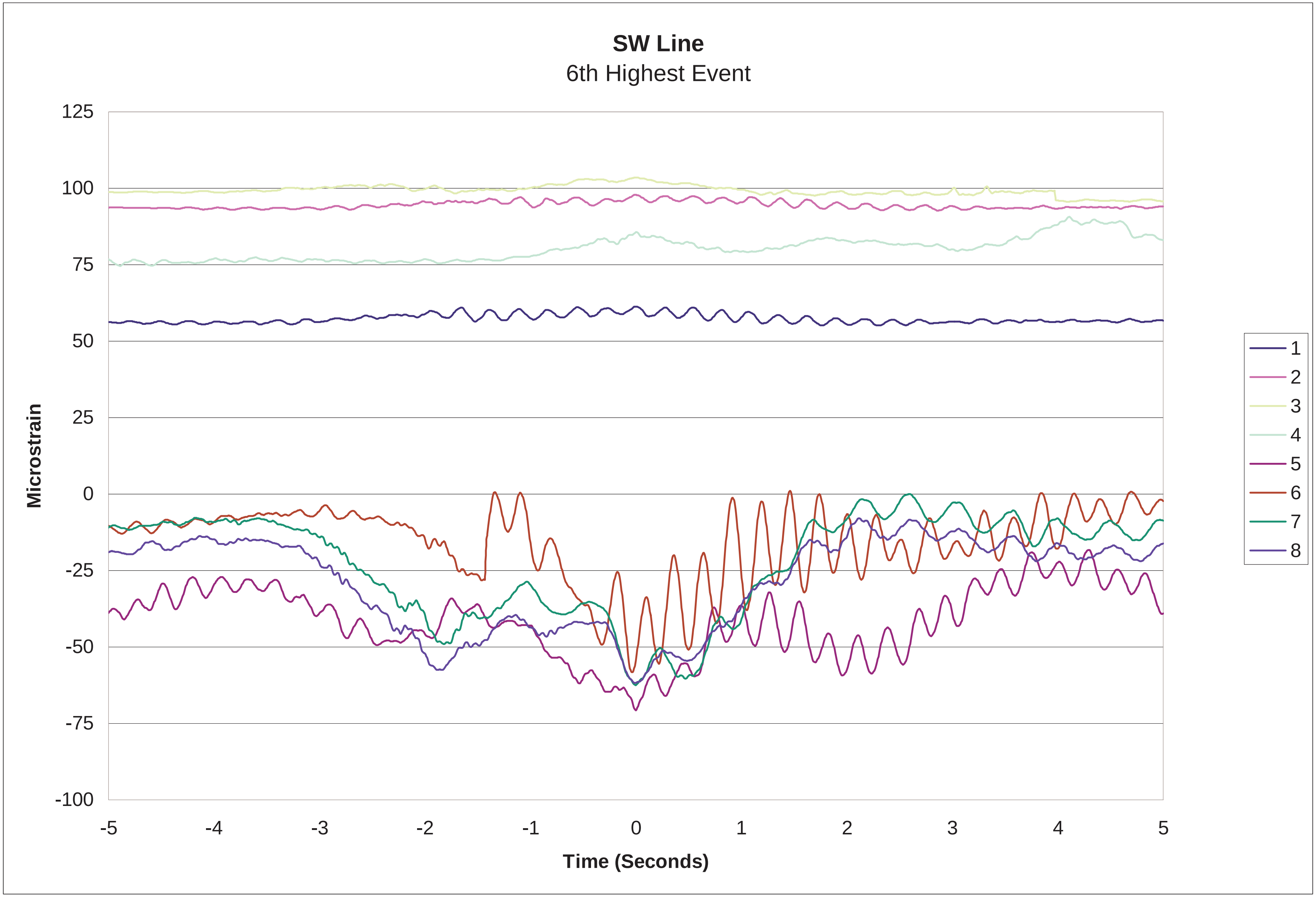
SW Line
3rd Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8

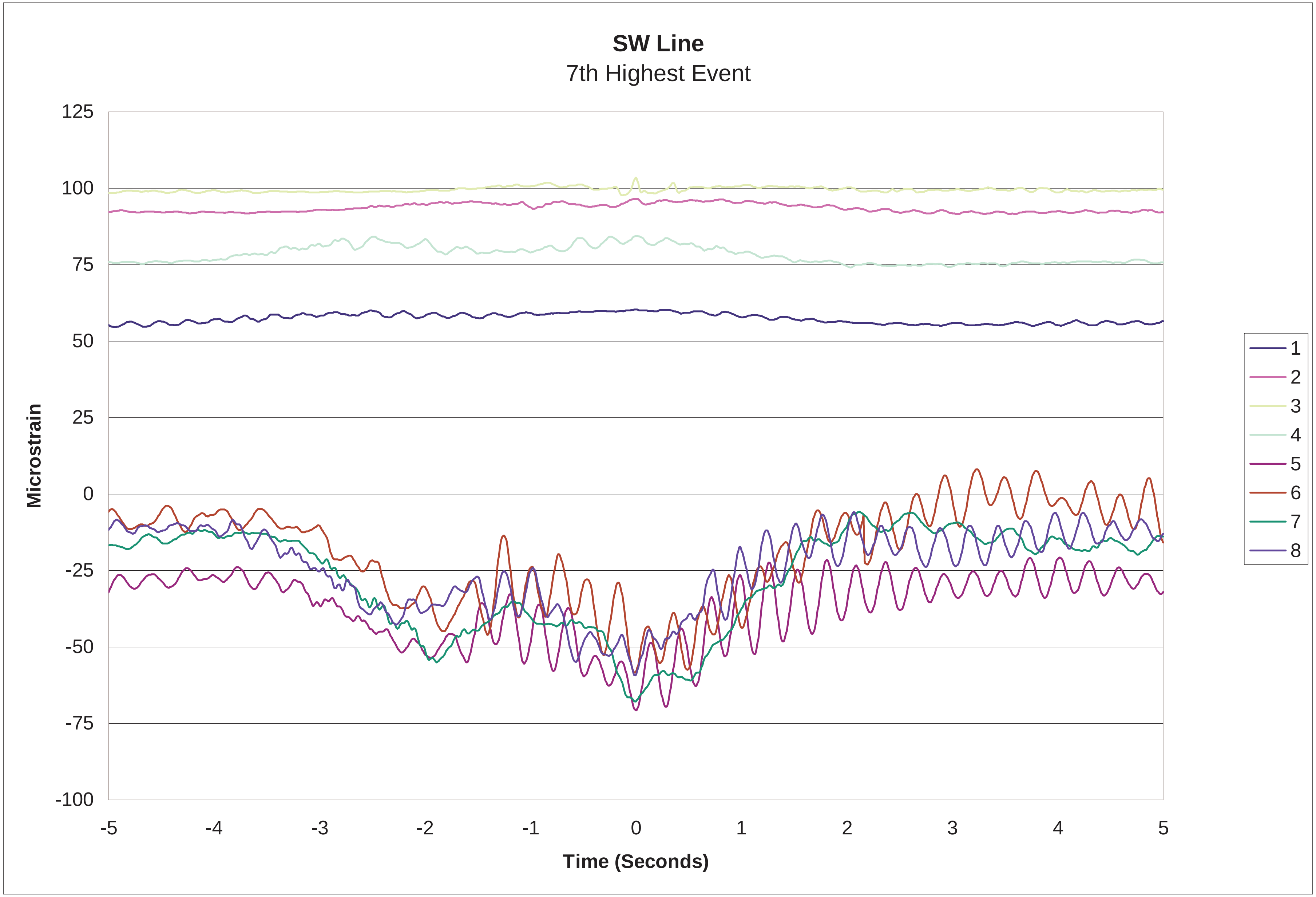


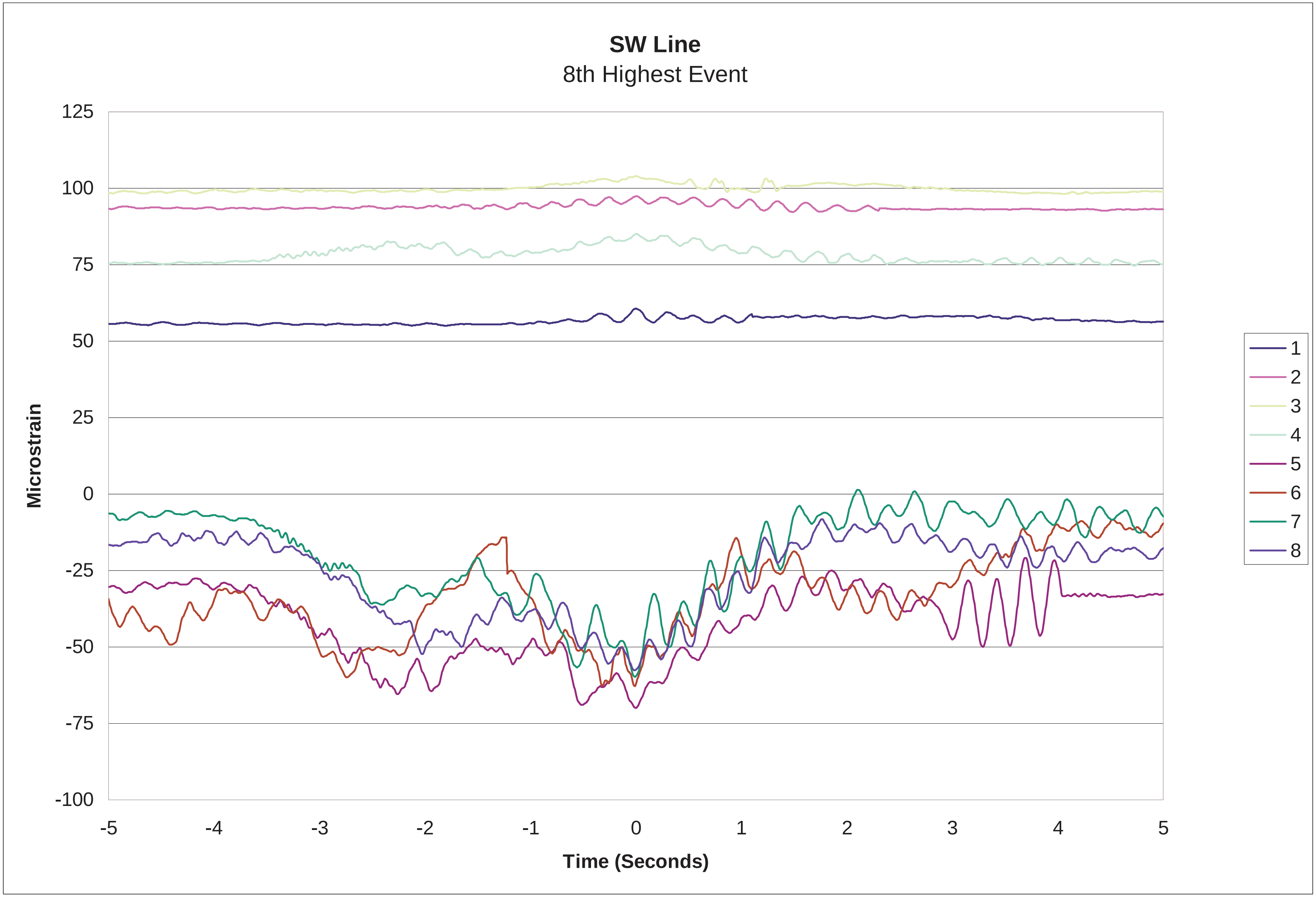
SW Line
4th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8



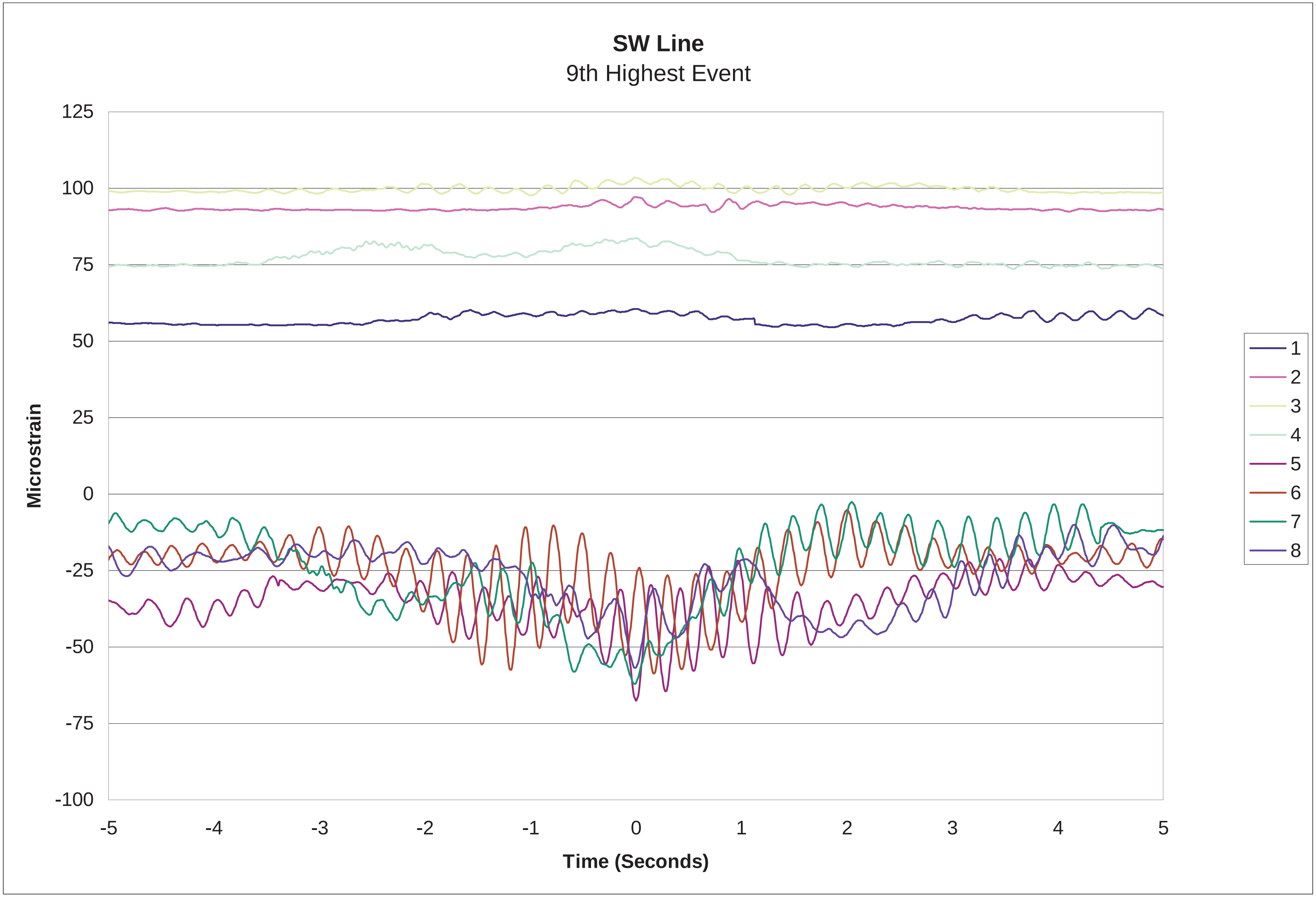
SW Line
5th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8


SW Line
6th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8


SW Line
7th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8



SW Line
8th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
4
5
6
7
8


SW Line
9th Highest Event
Microstrain
Time (Seconds)
125
100
75
50
25
0
-25
-50
-75
-100
-5
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-2
-1
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1
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3
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5
1
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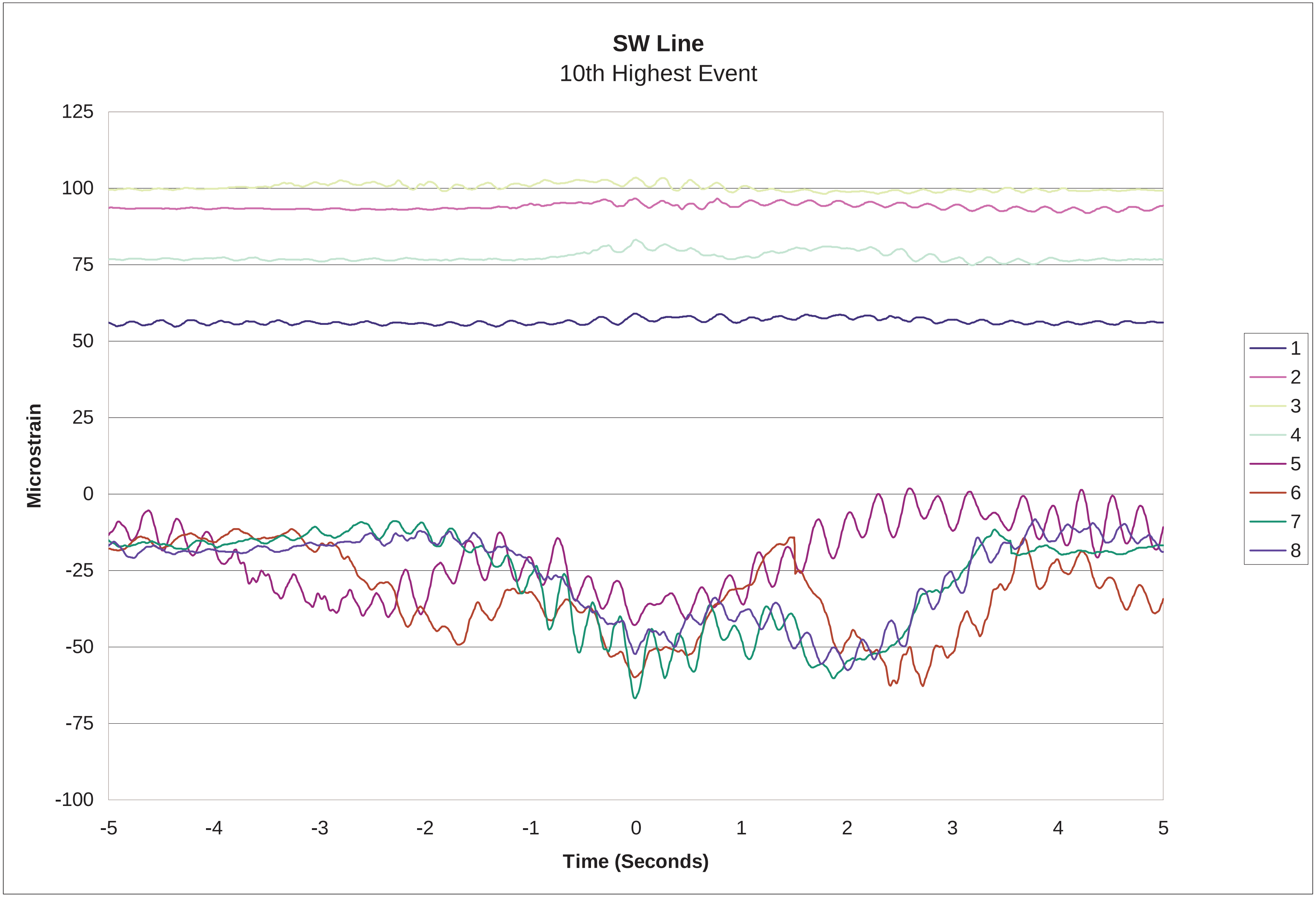
7
8



SW Line
10th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
1
2
3
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7
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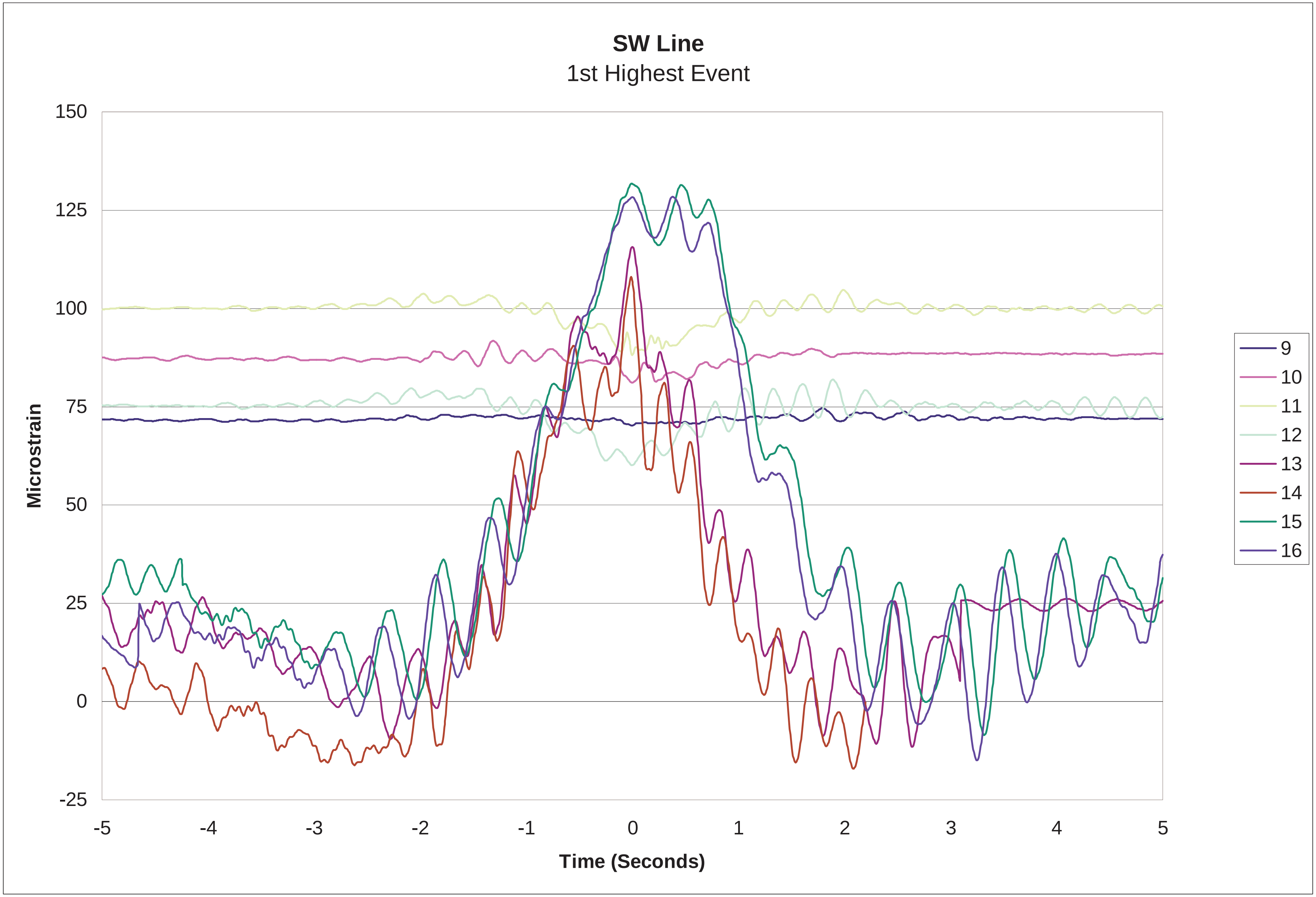
SW Line
1st Highest Event
Microstrain
150
125
100
75
50
25
0
-25
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
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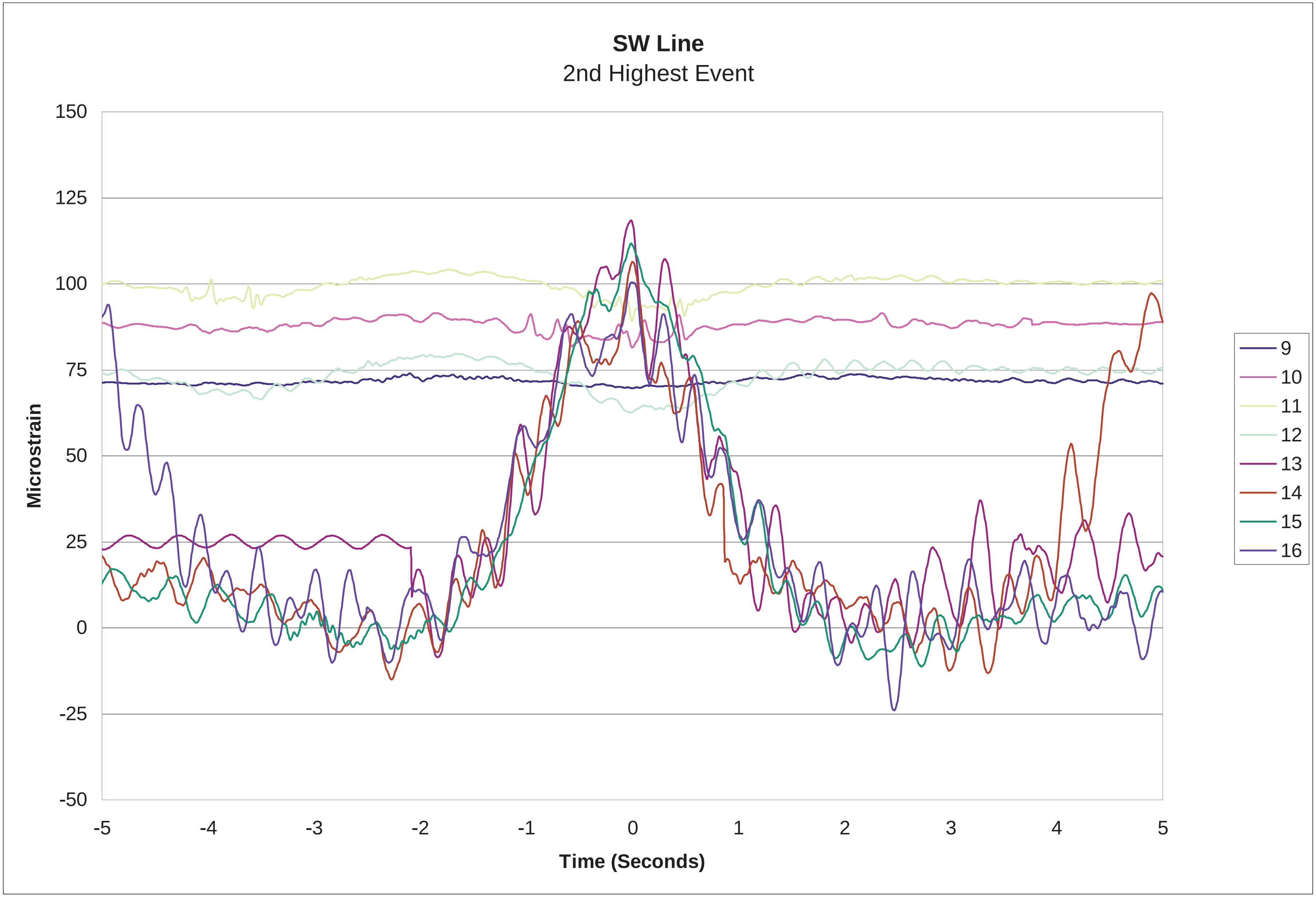
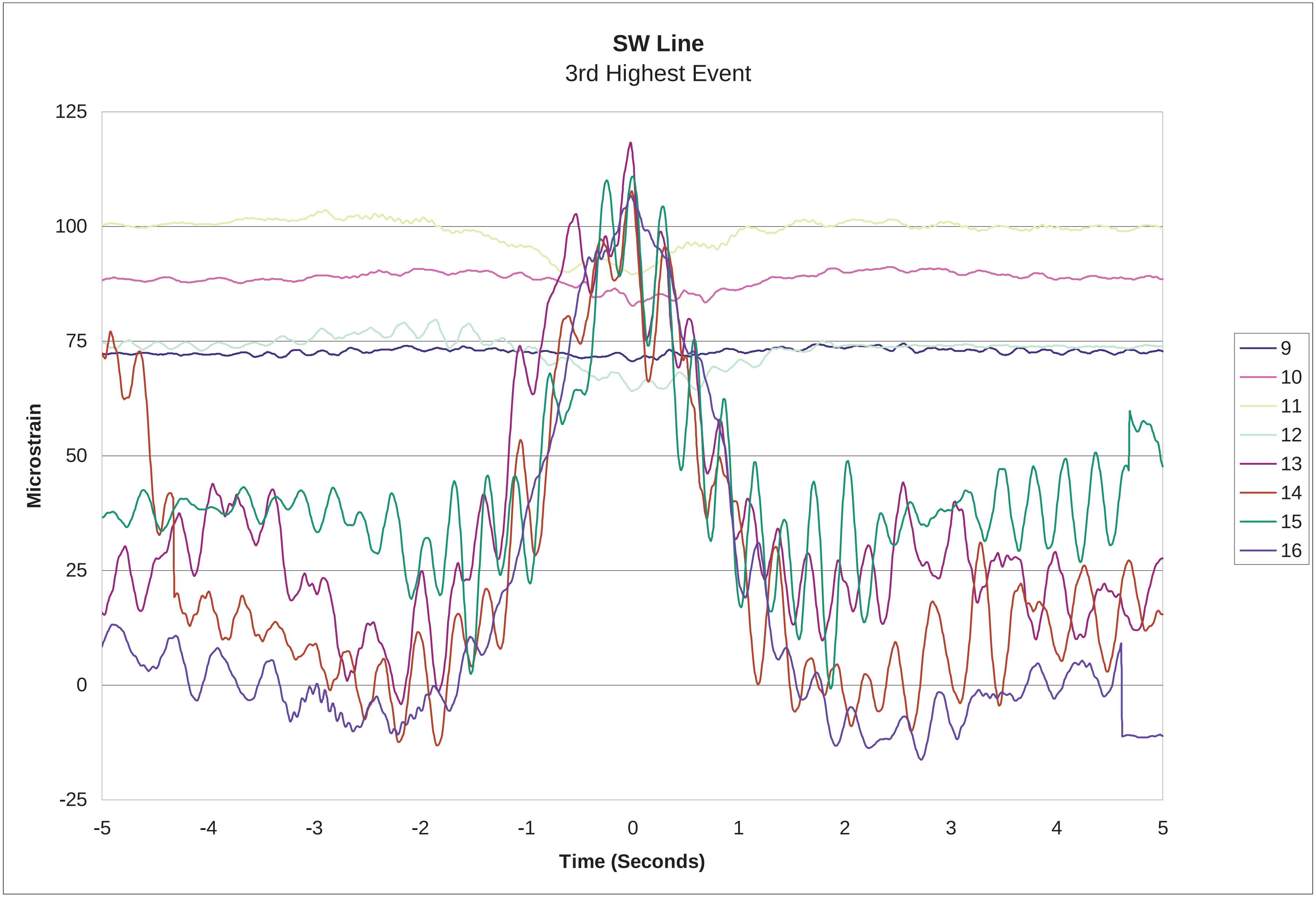
15
16



SW Line
2nd Highest Event
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
14
15
16


SW Line
3rd Highest Event
Microstrain
125
100
75
50
25
0
-25
-5
-4
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-2
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0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
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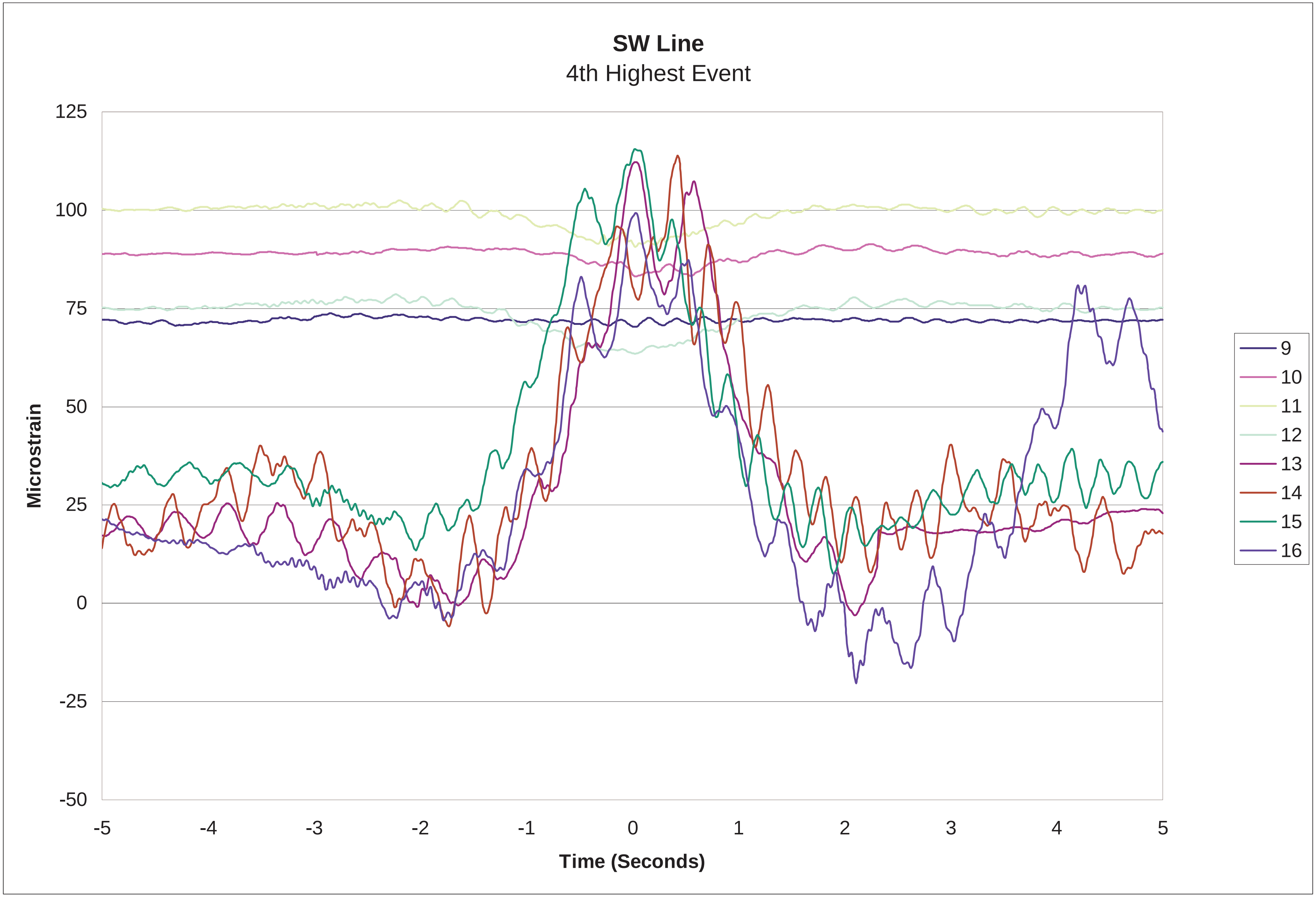
15
16



SW Line
4th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
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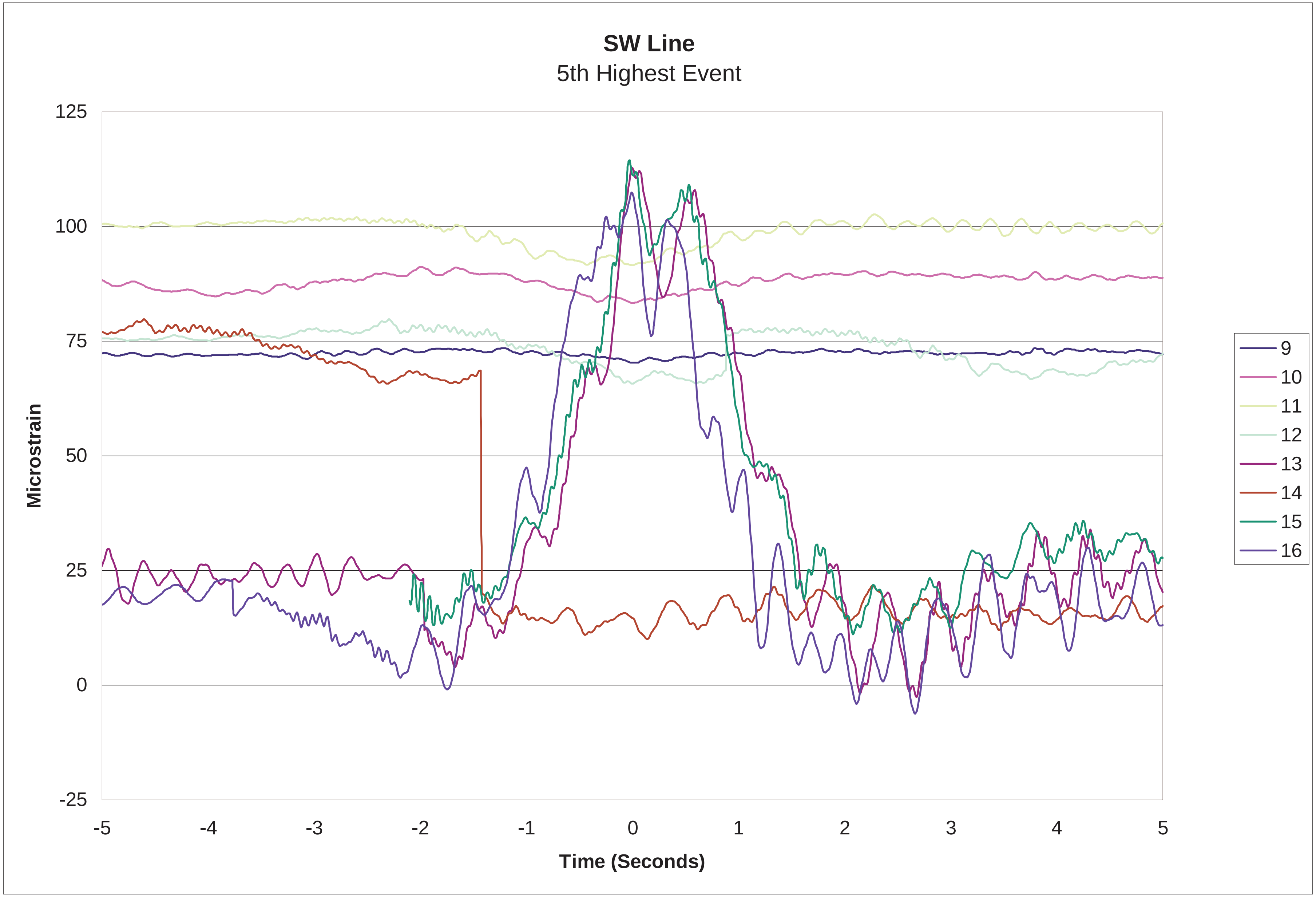
15
16


SW Line
5th Highest Event
Microstrain
125
100
75
50
25
0
-25
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
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15
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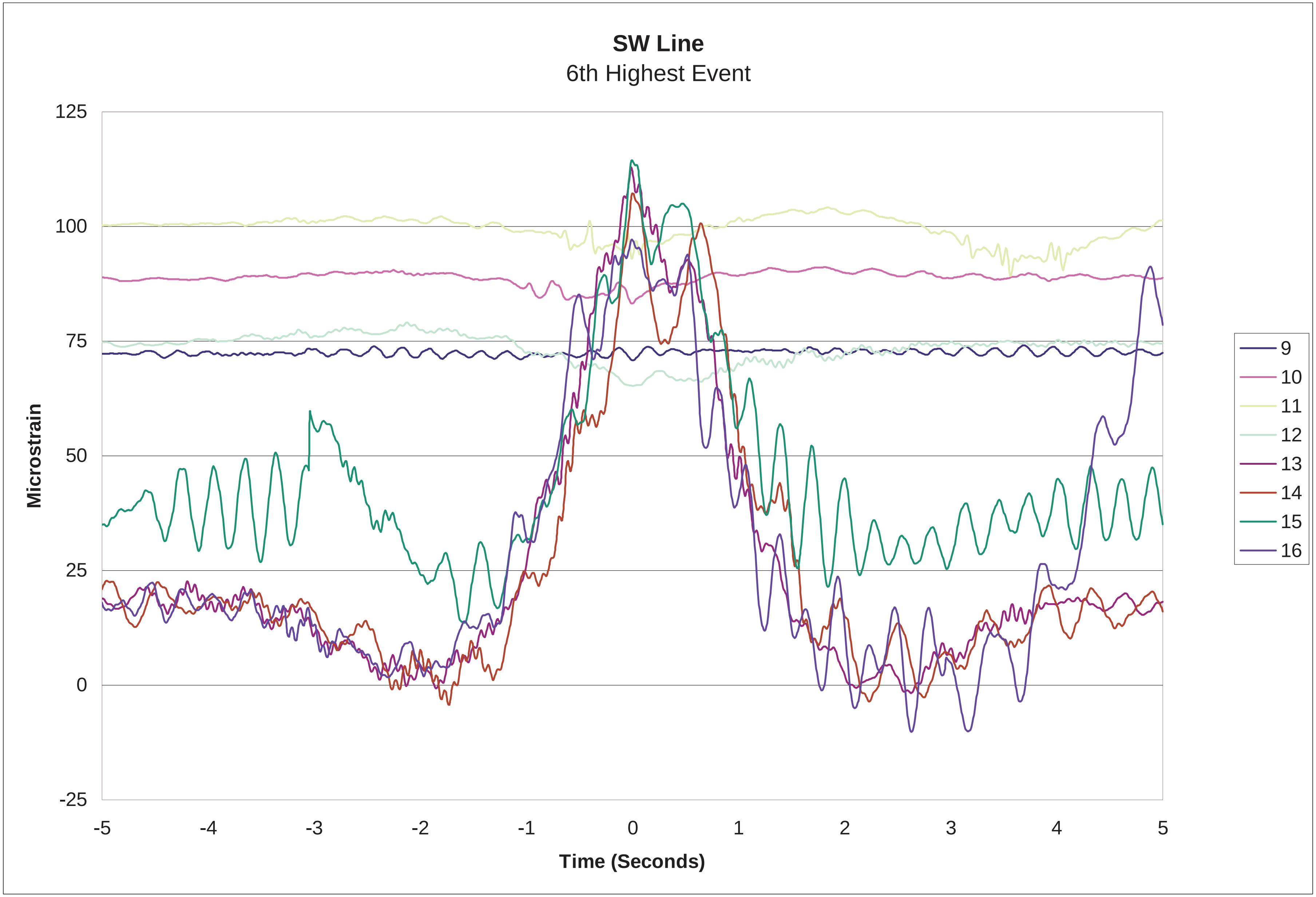

SW Line
6th Highest Event
Microstrain
125
100
75
50
25
0
-25
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
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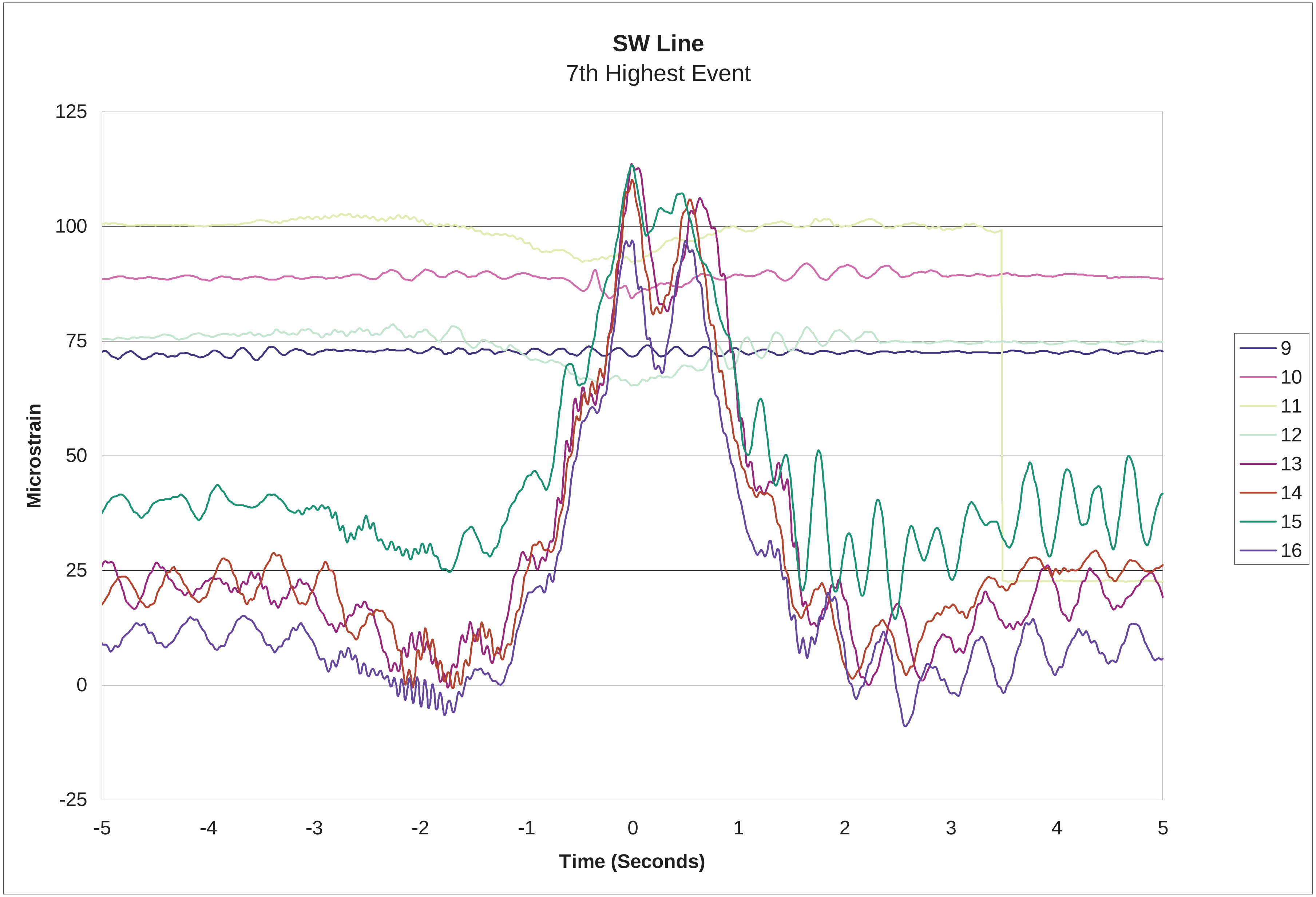
15
16


SW Line
7th Highest Event
Microstrain
125
100
75
50
25
0
-25
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
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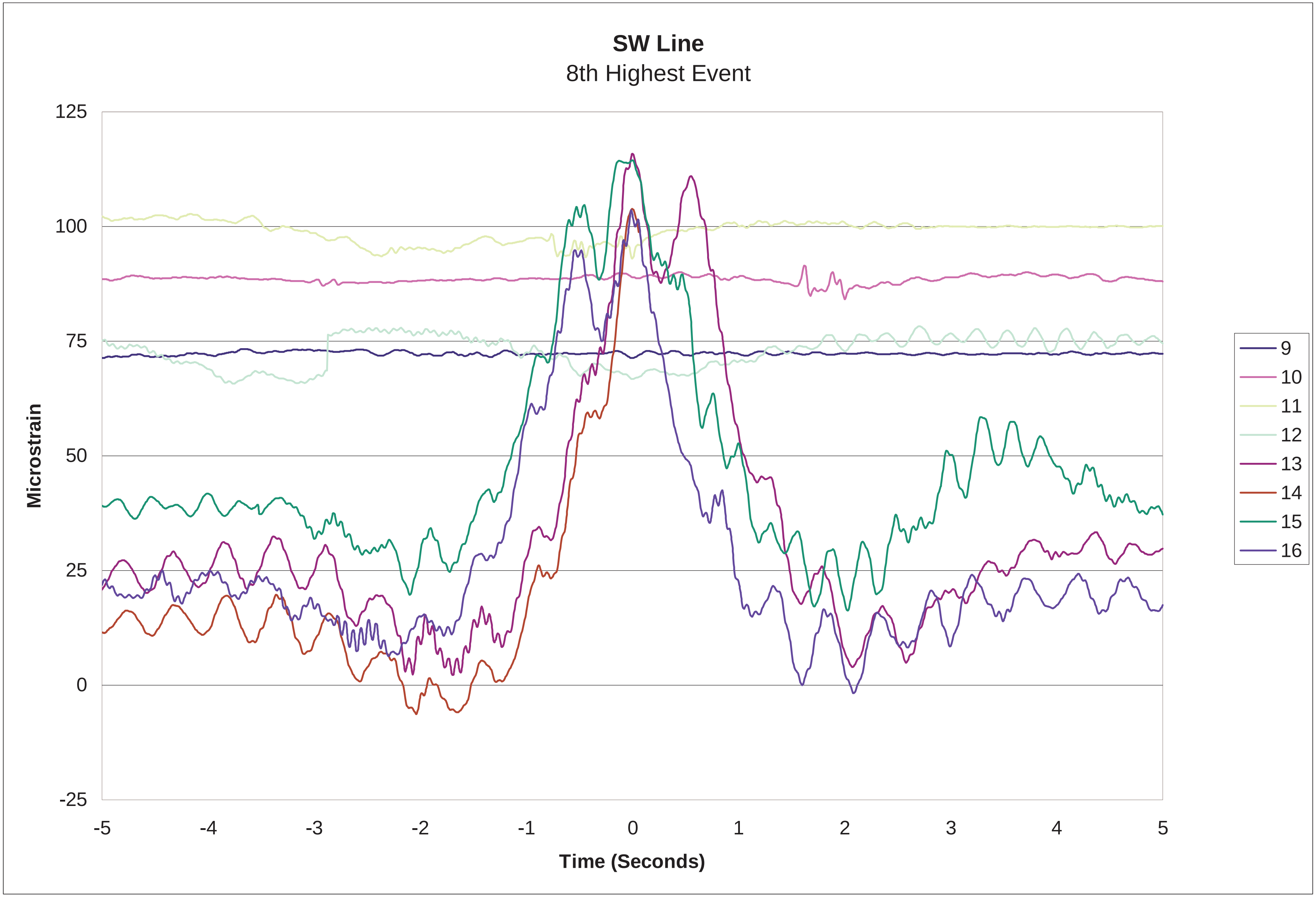
15
16



SW Line
8th Highest Event
125
100
75
50
25
0
-25
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
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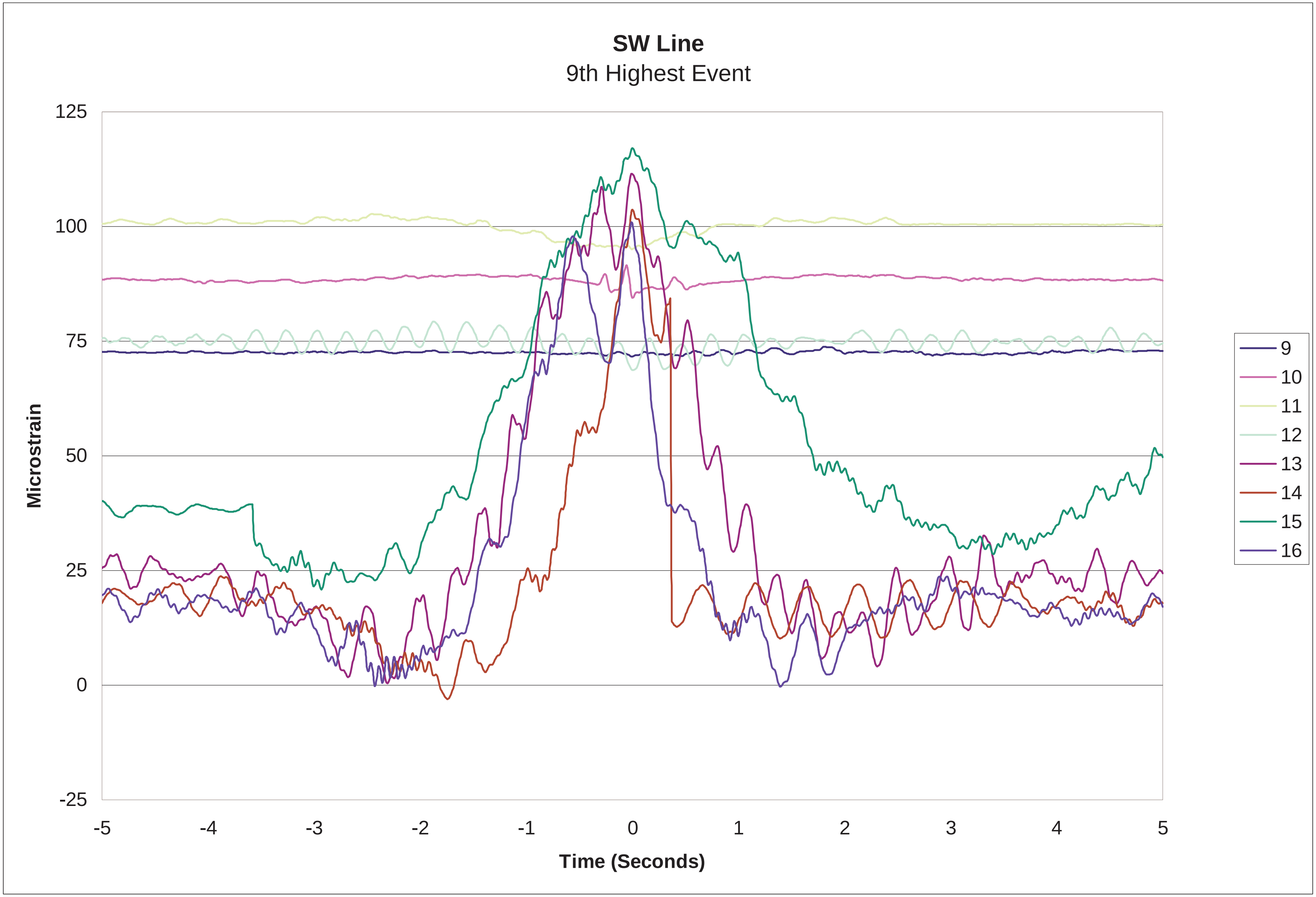
15
16



SW Line
9th Highest Event
Microstrain
125
100
75
50
25
0
-25
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
10
11
12
13
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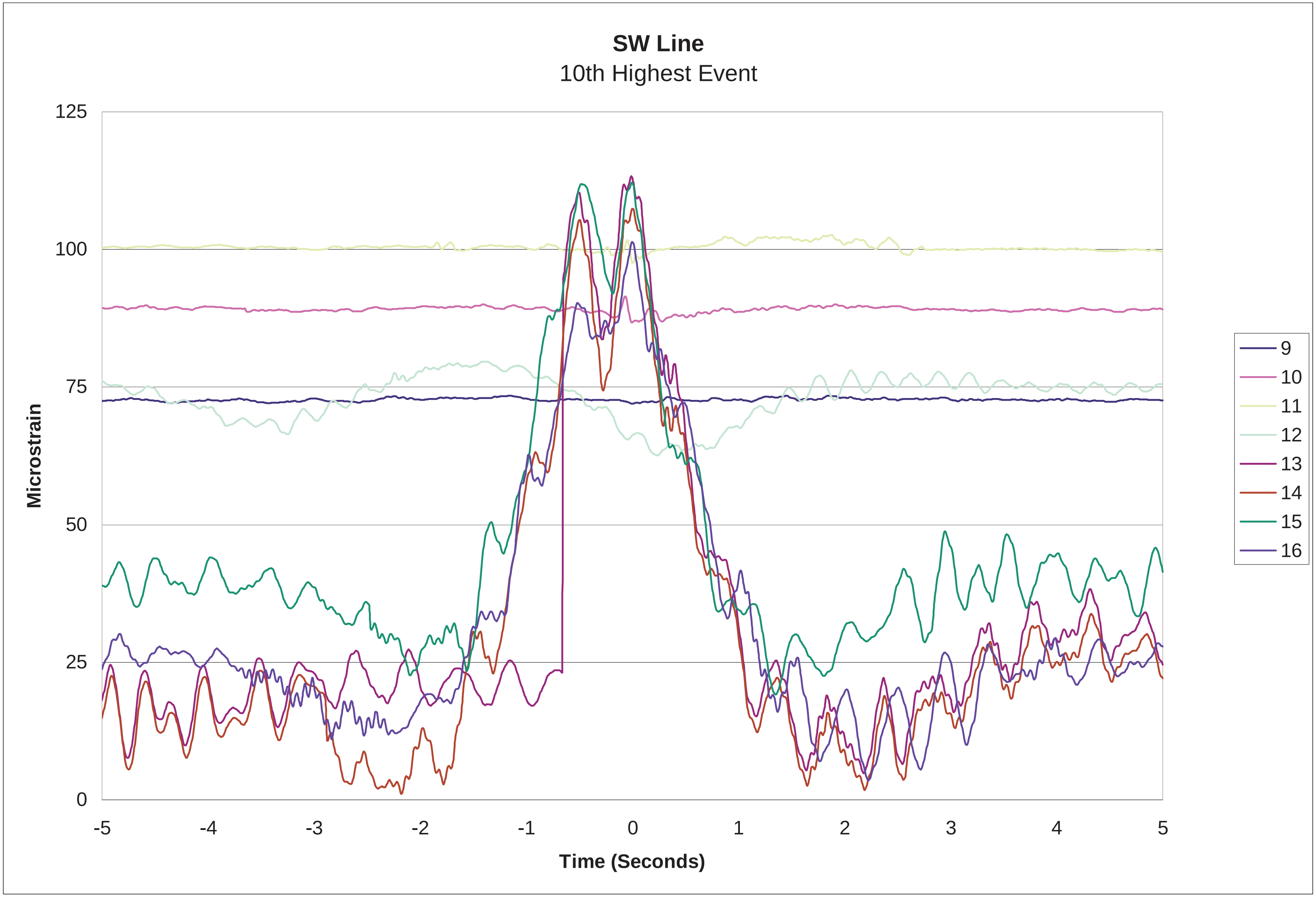
15
16



SW Line
10th Highest Event
Microstrain
125
100
75
50
25
0
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
9
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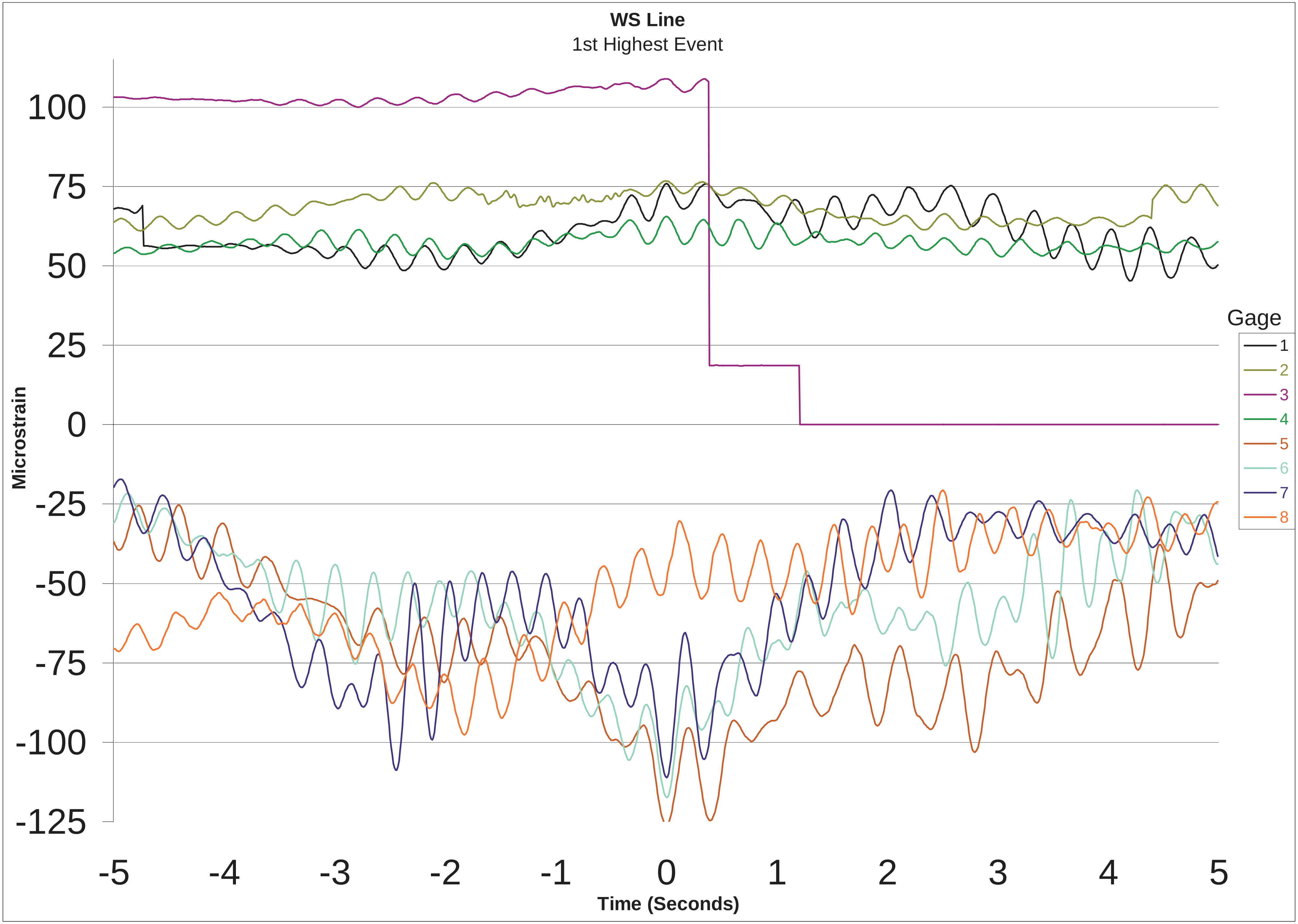
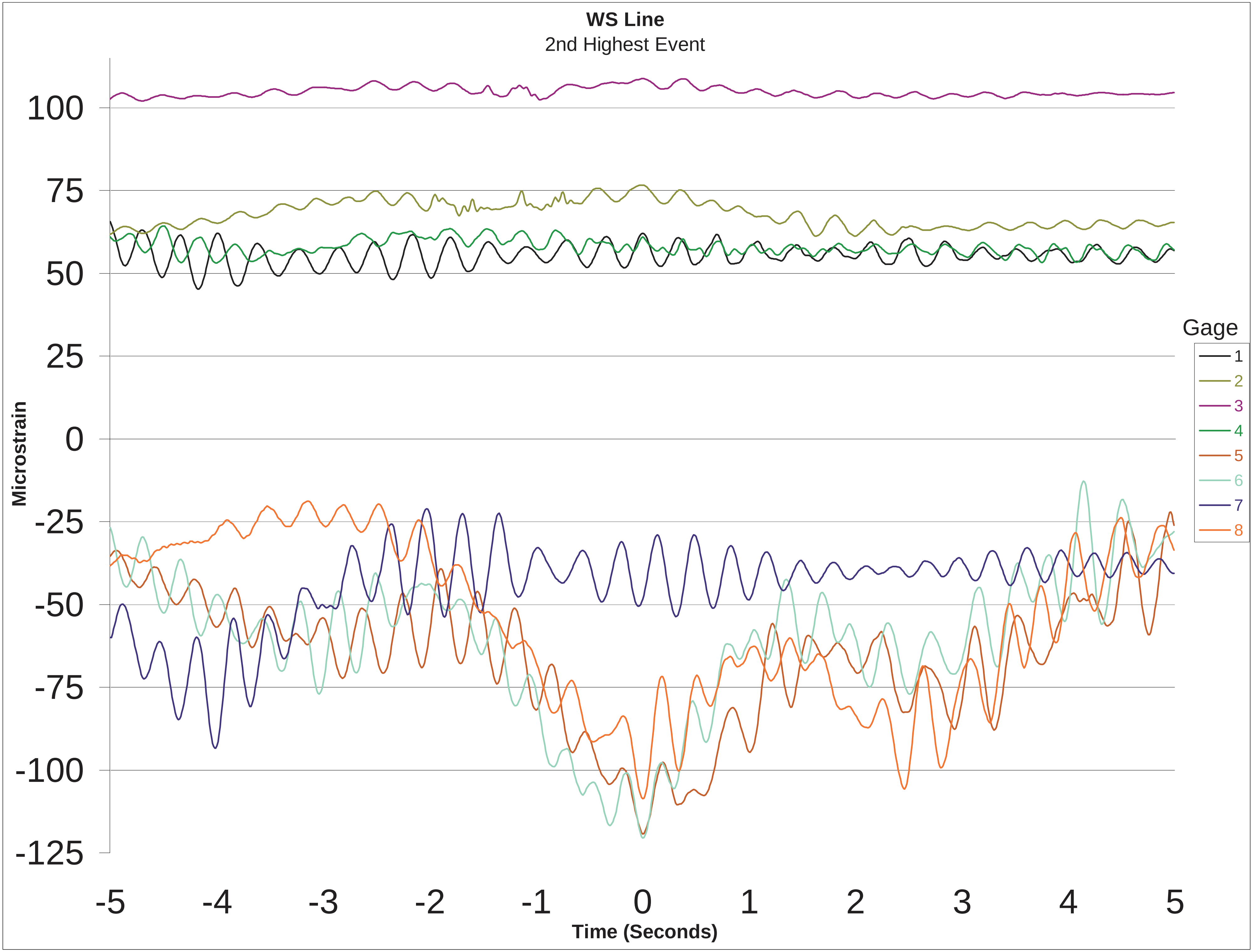
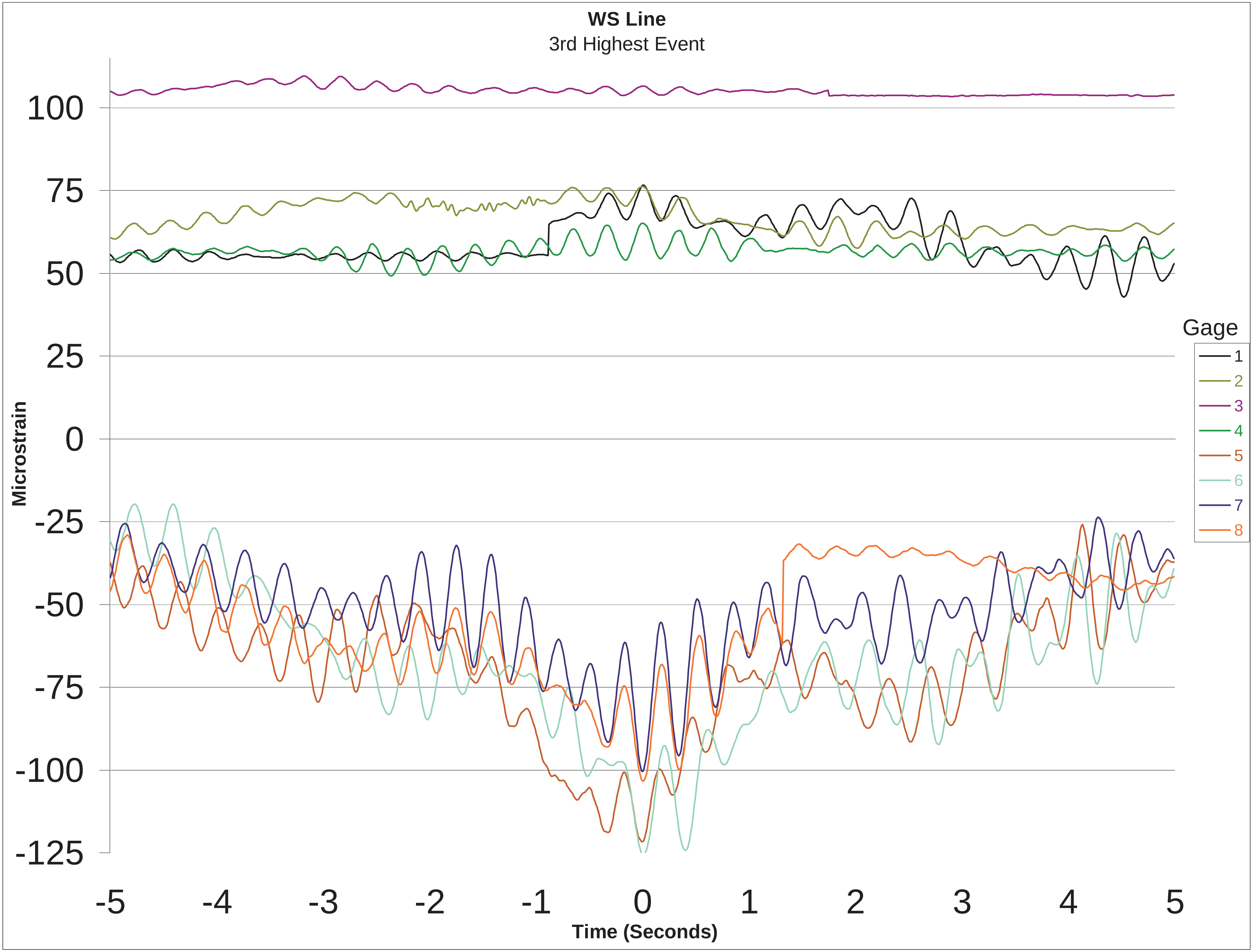
11
12
13
14
15
16



WS Line
1st Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


WS Line
2nd Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


WS Line
3rd Highest Event
100
75
50
25
0
-25
-50
-75
-100
-125
Microstrain
-5
-4
-3
-2
-1
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3
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5
Time (Seconds)
Gage
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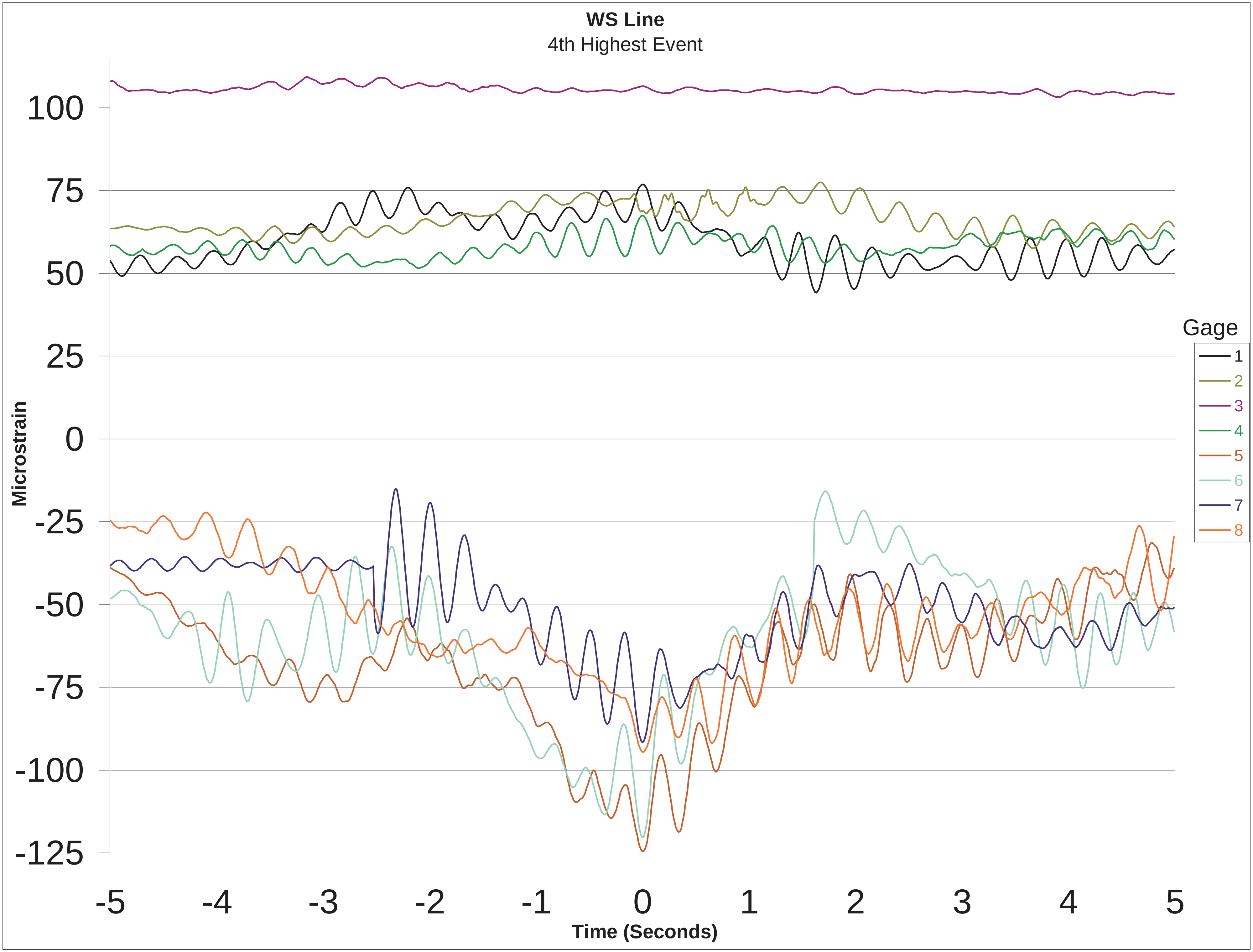
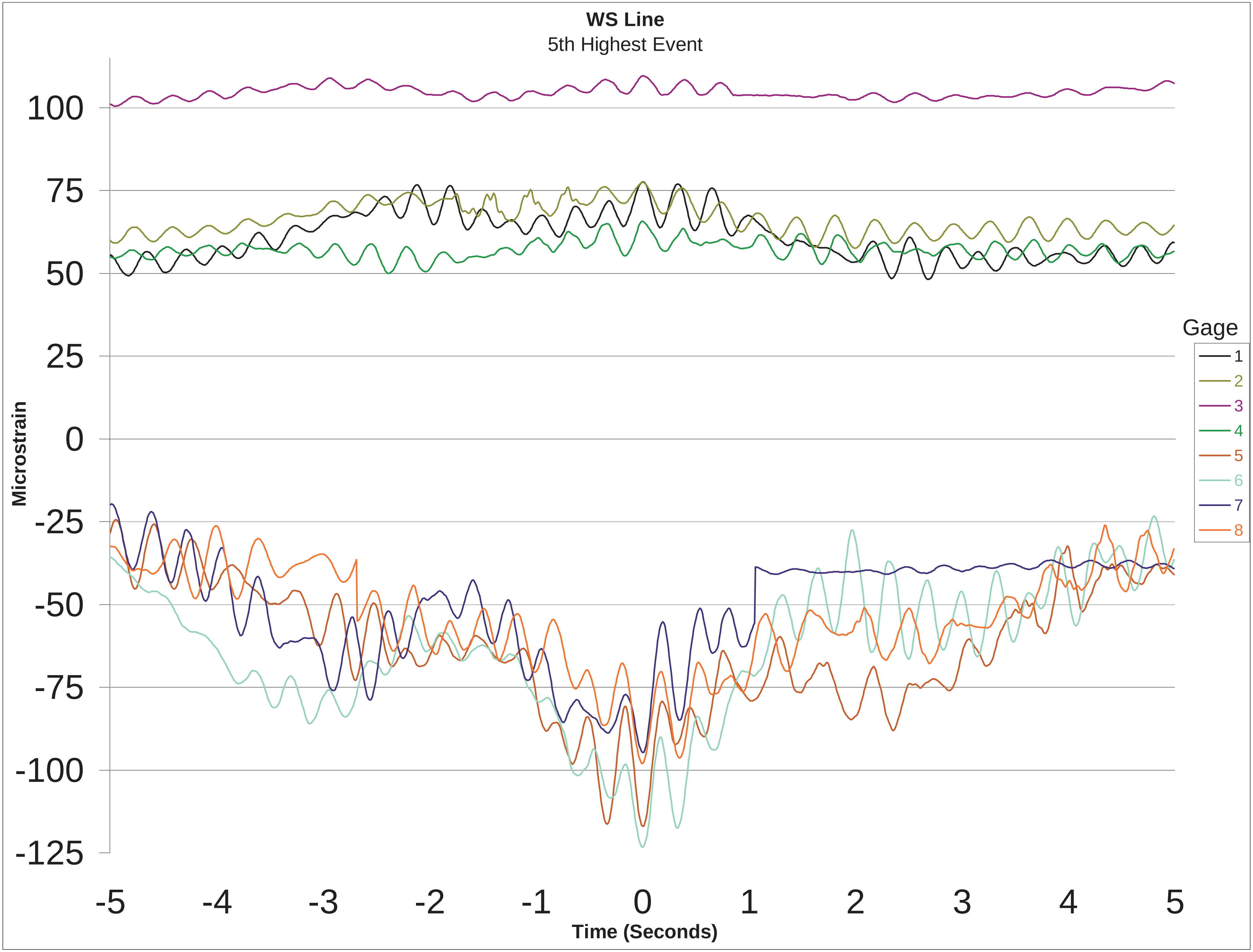
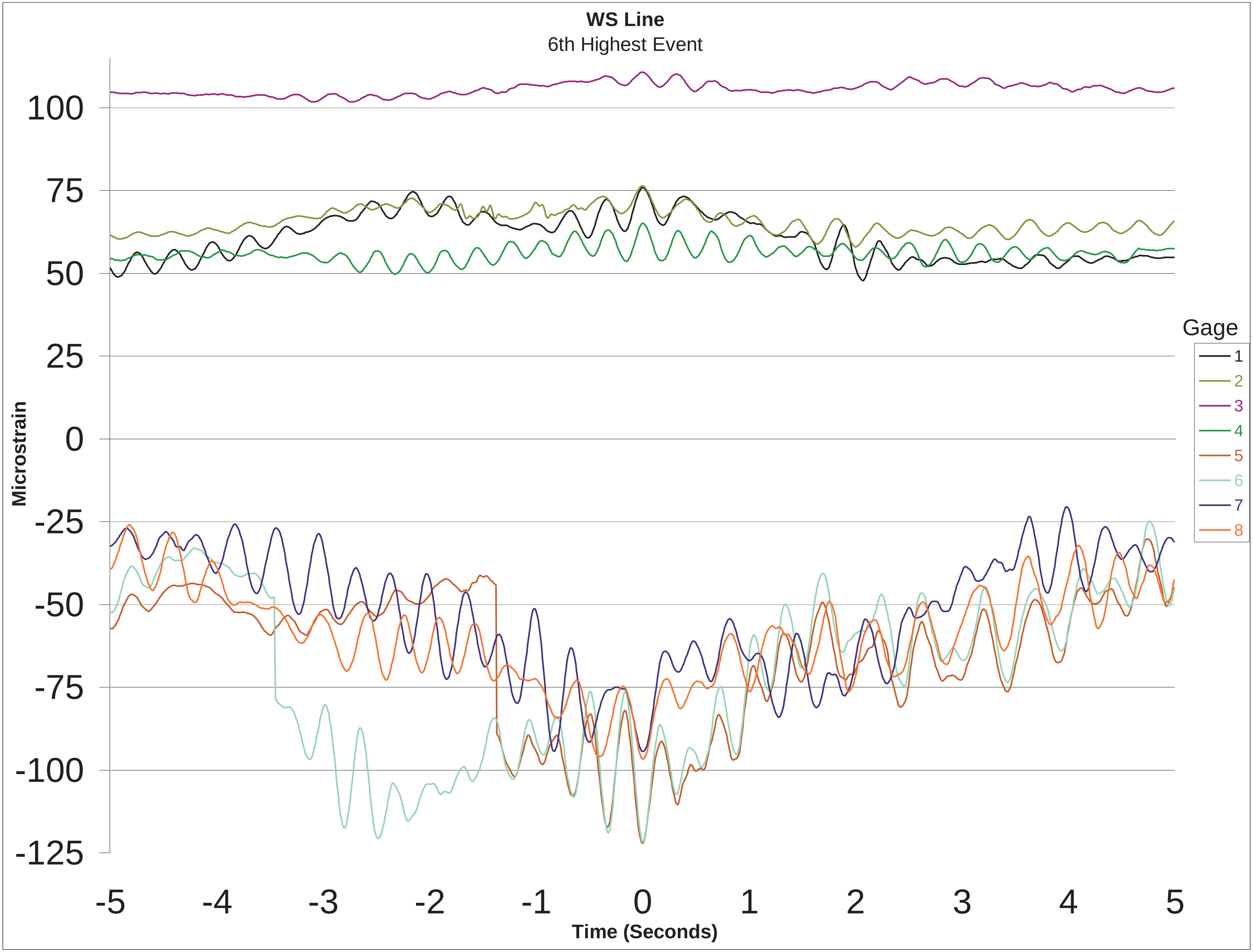
5
6
7
8

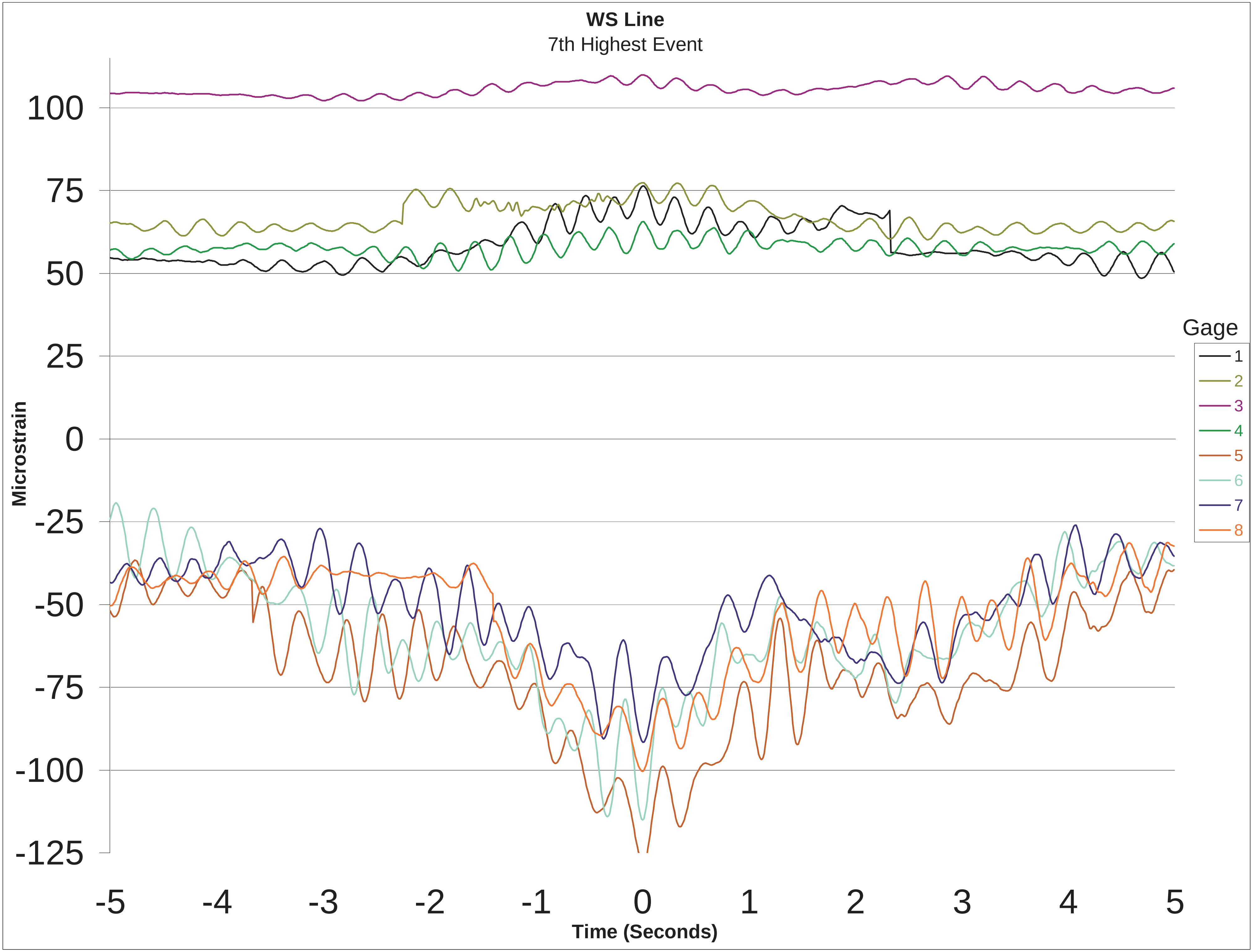

WS Line
4th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



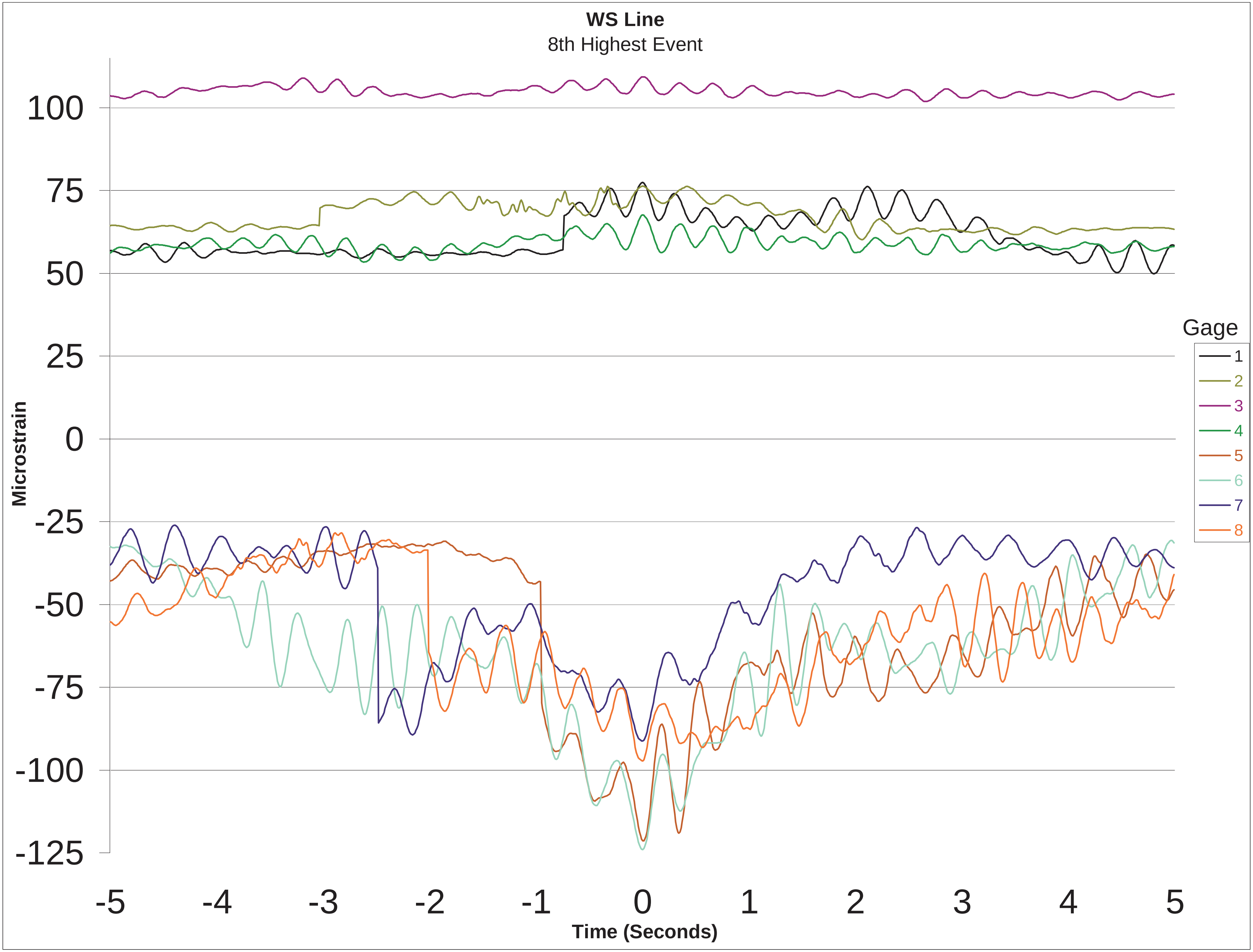
WS Line
5th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



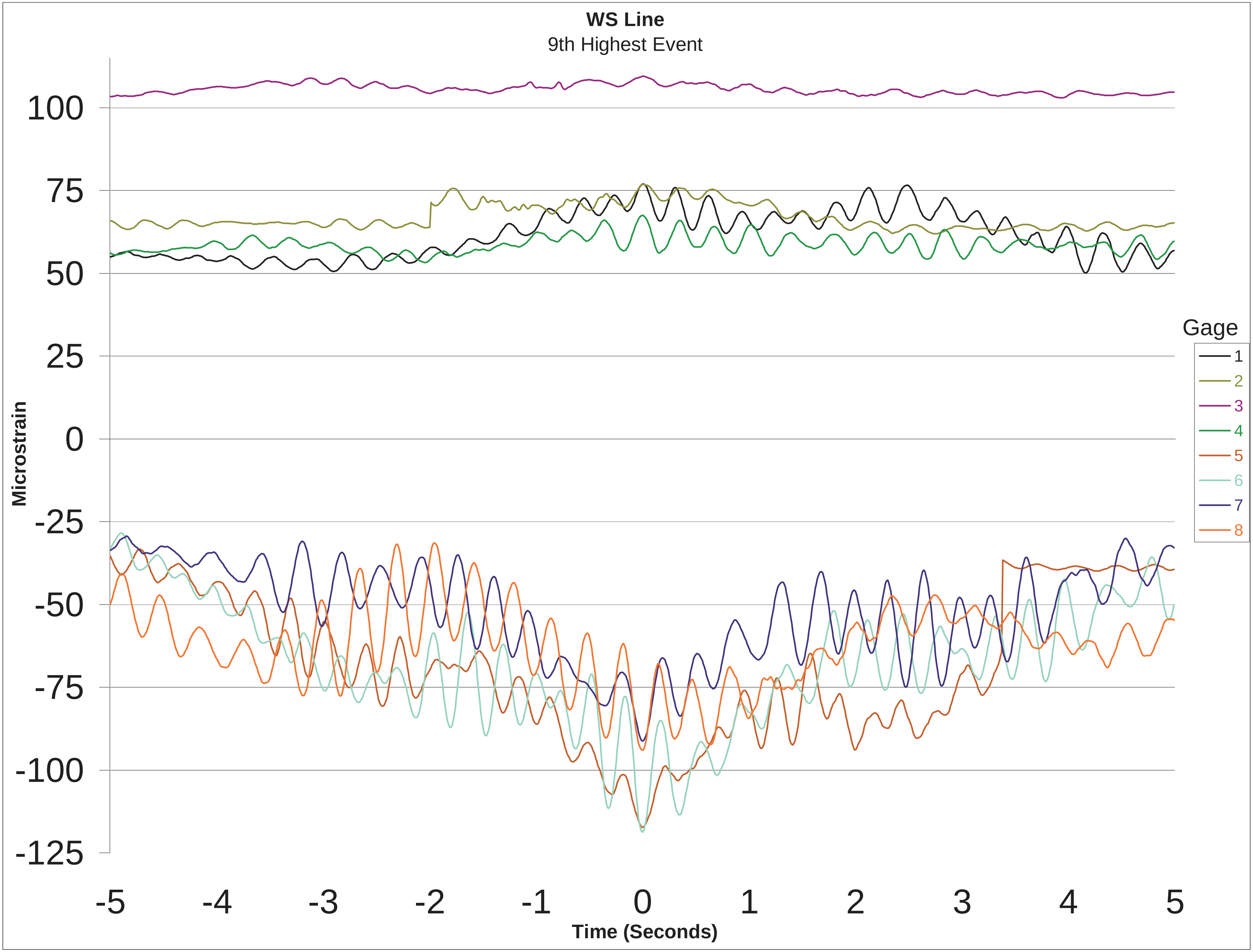
WS Line
6th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



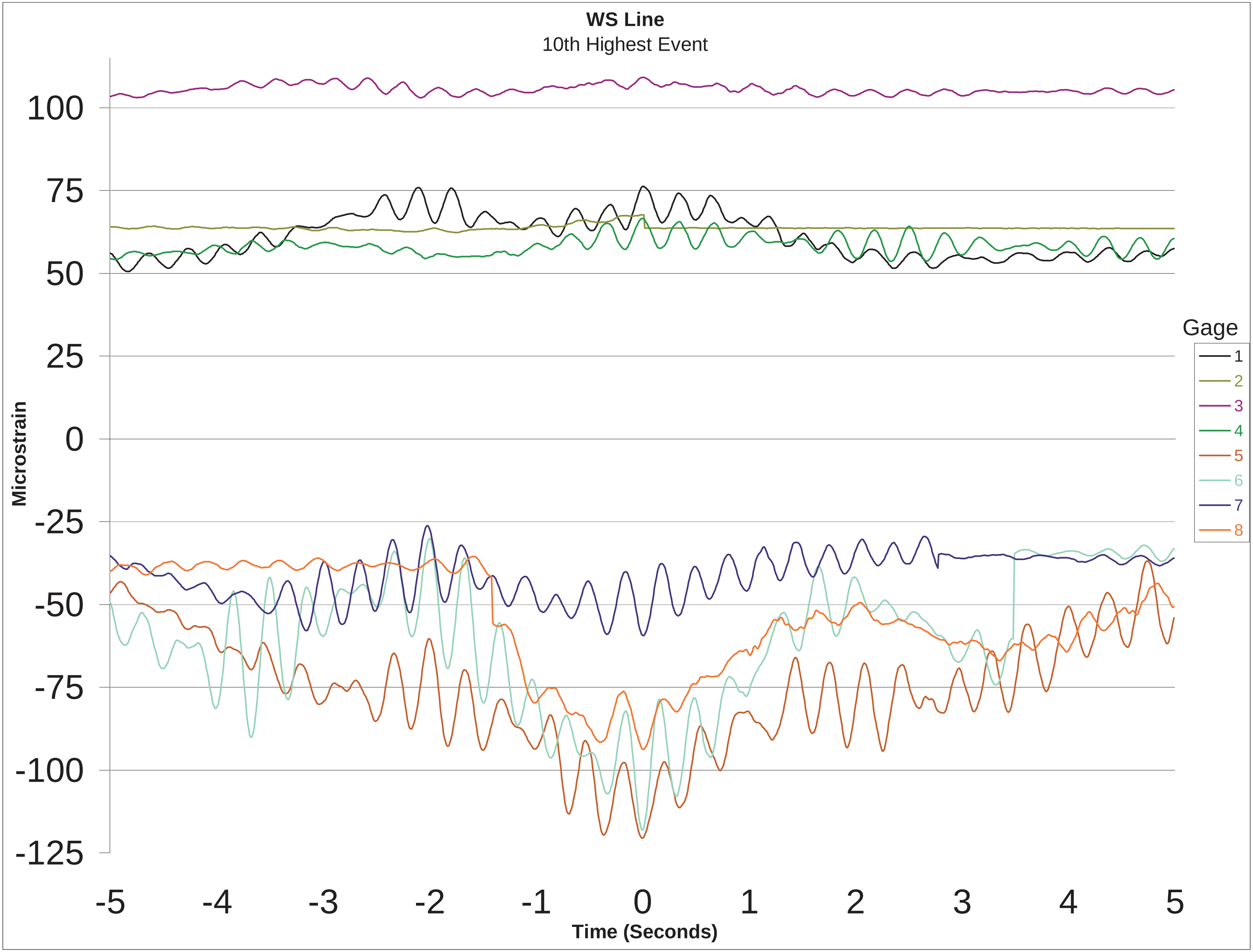
WS Line
7th Highest Event
100
75
50
25
0
-25
-50
-75
-100
-125
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



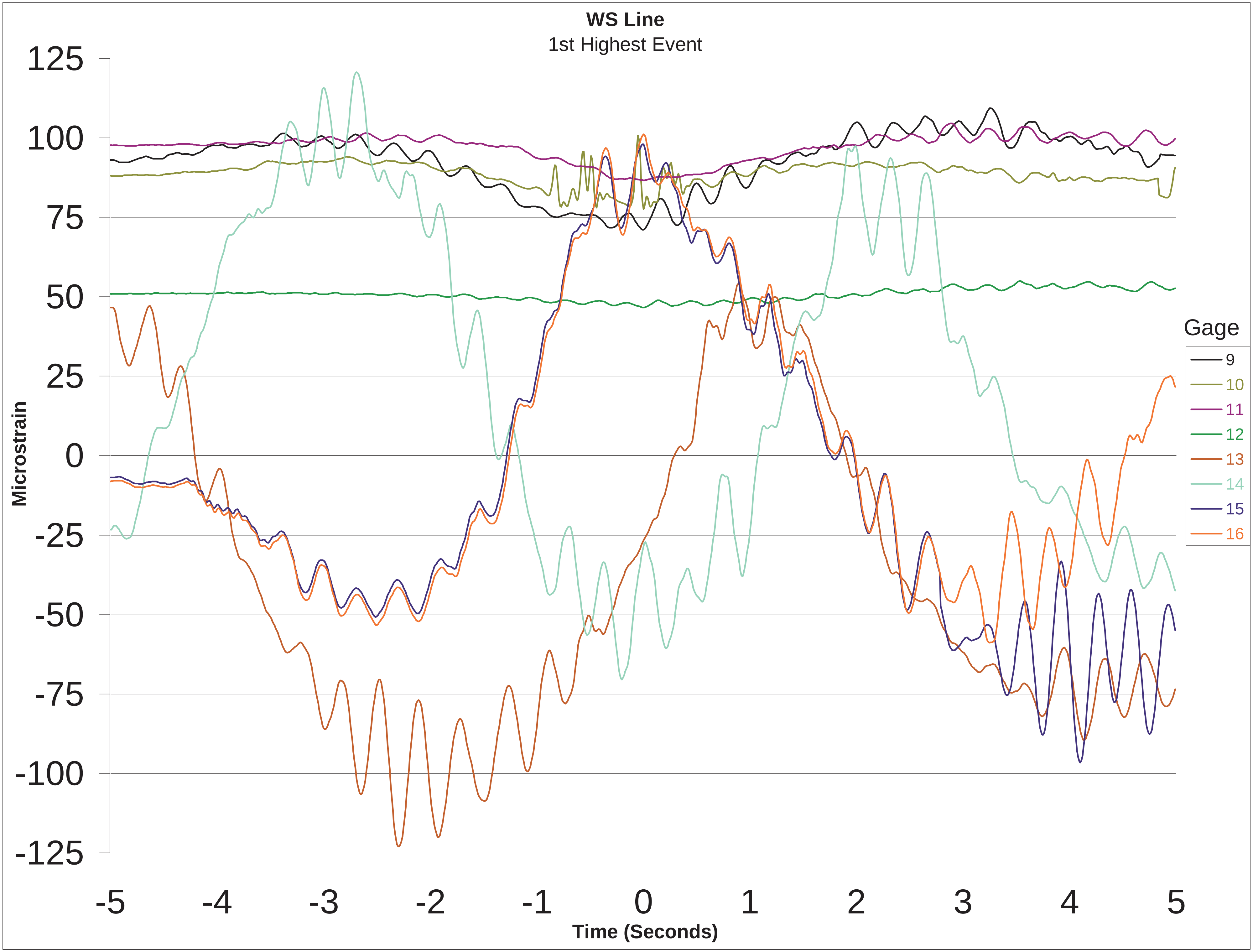
WS Line
8th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

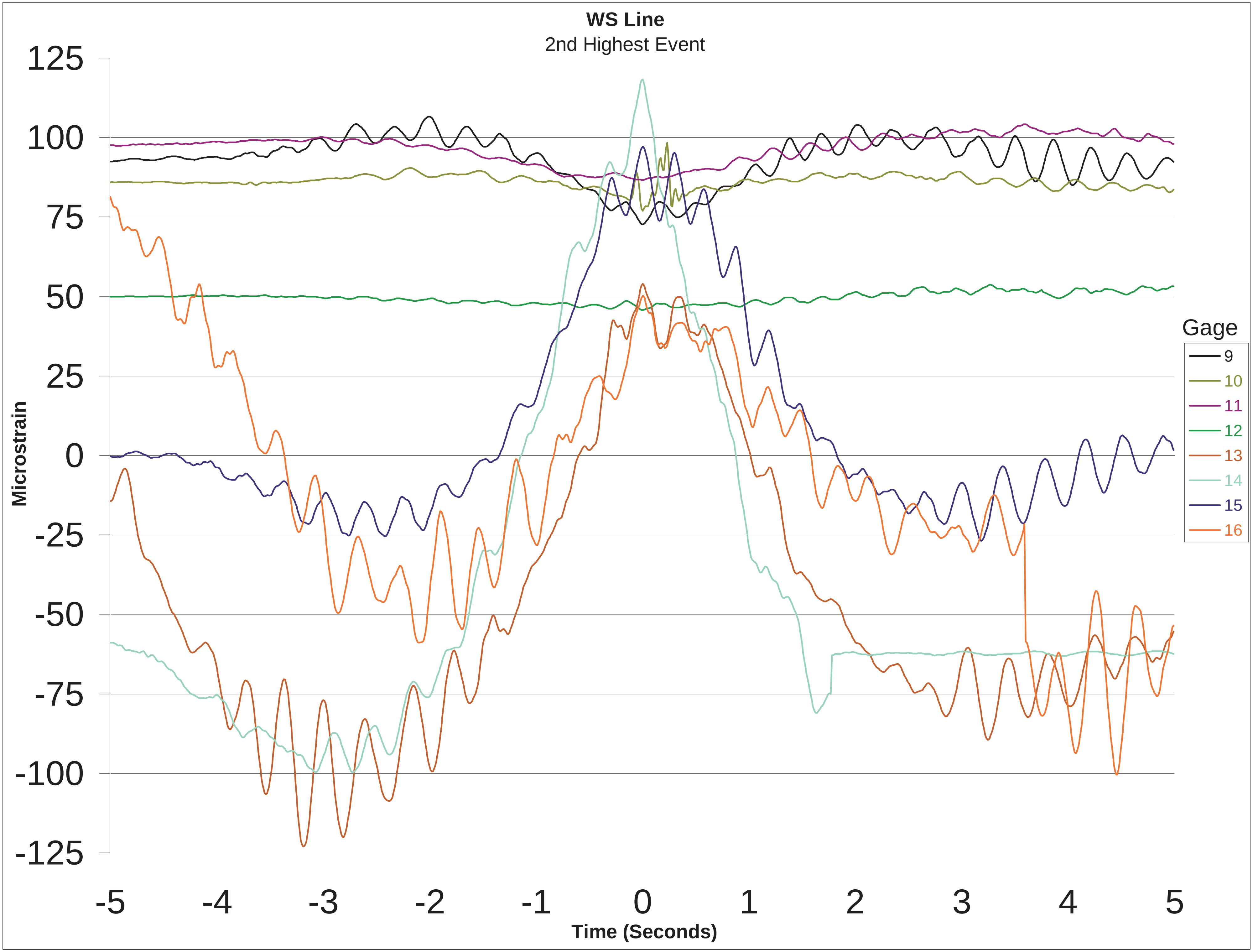


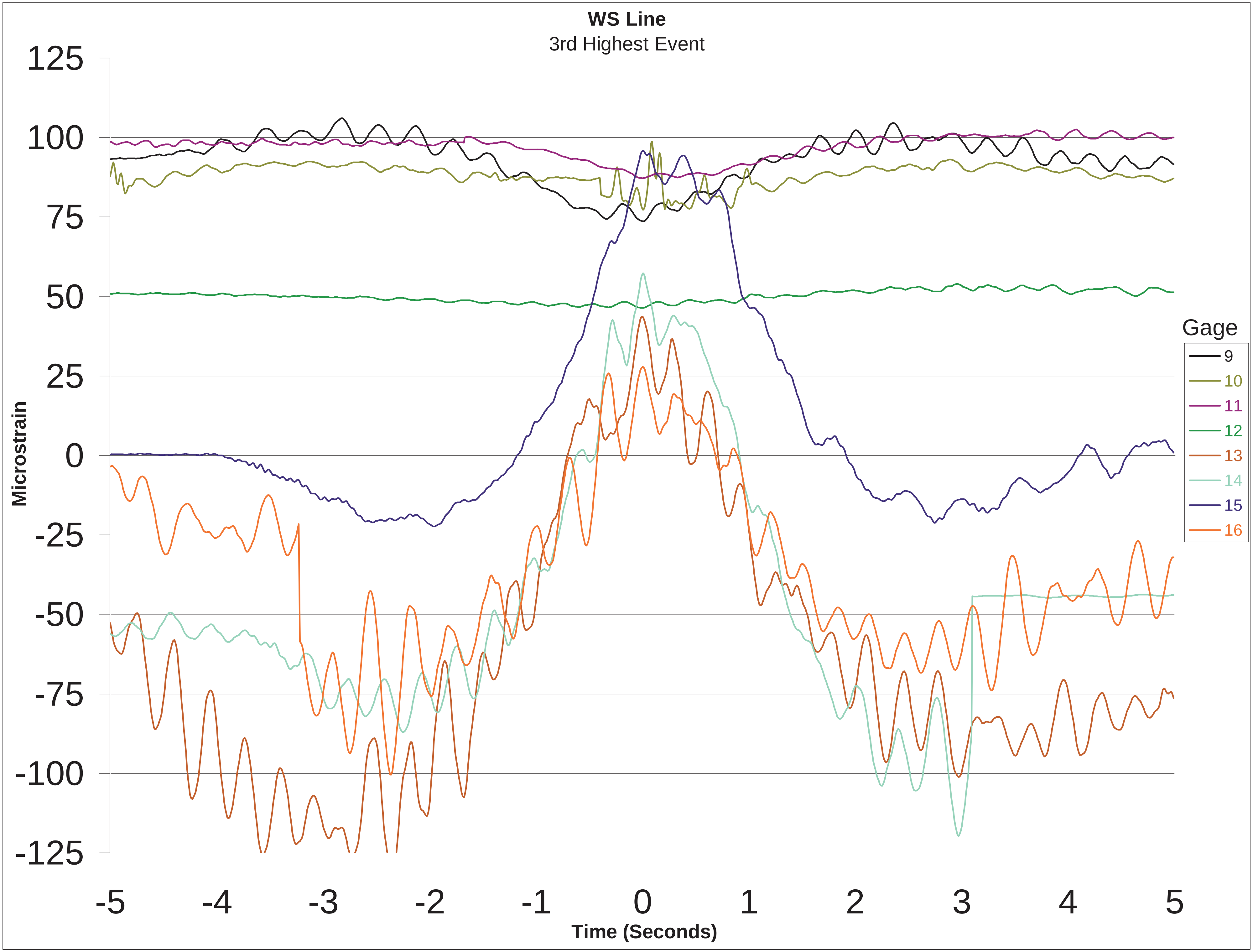
WS Line
9th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



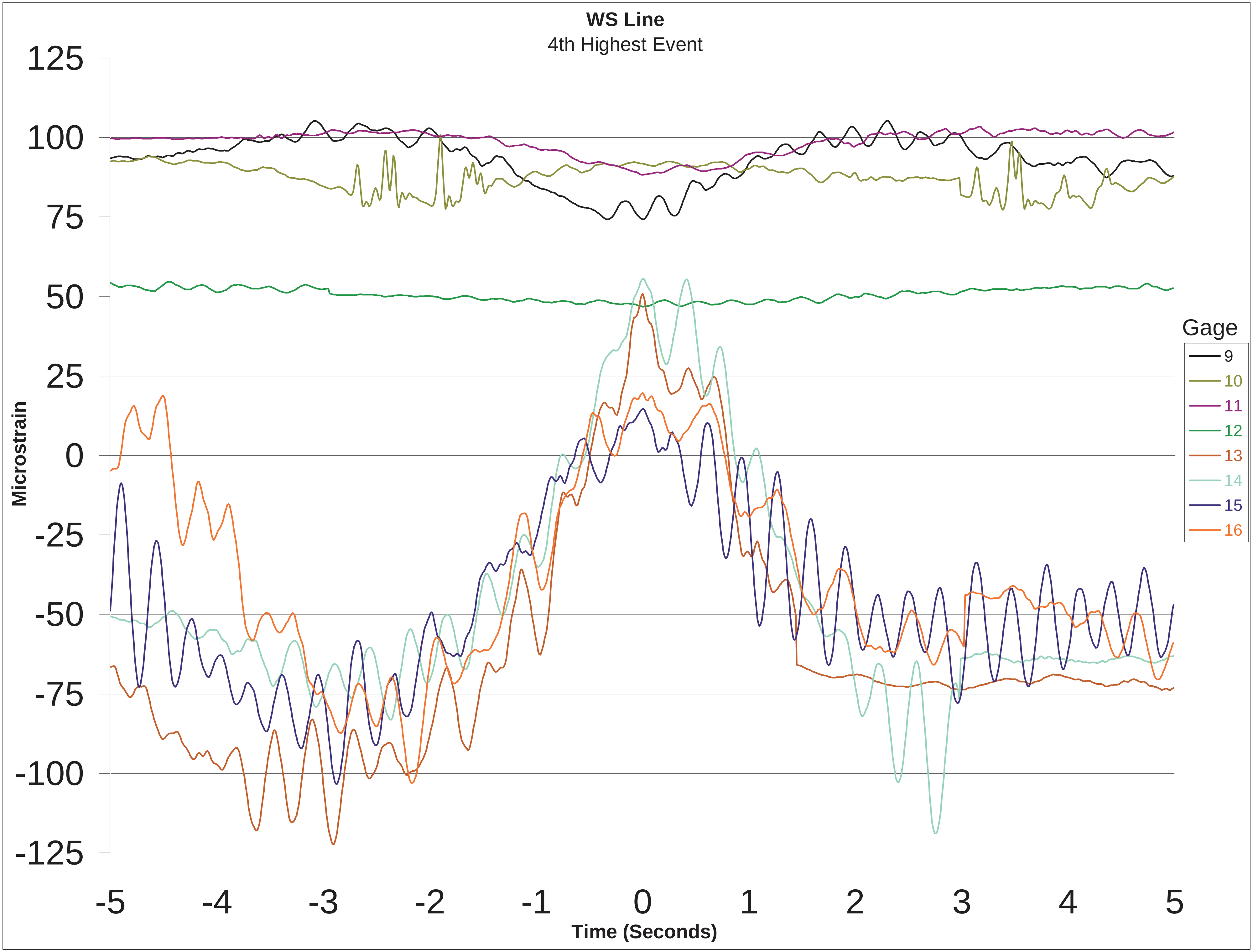
WS Line
10th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


WS Line
1st Highest Event
125
100
75
50
25
0
-25
-50
-75
-100
-125
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

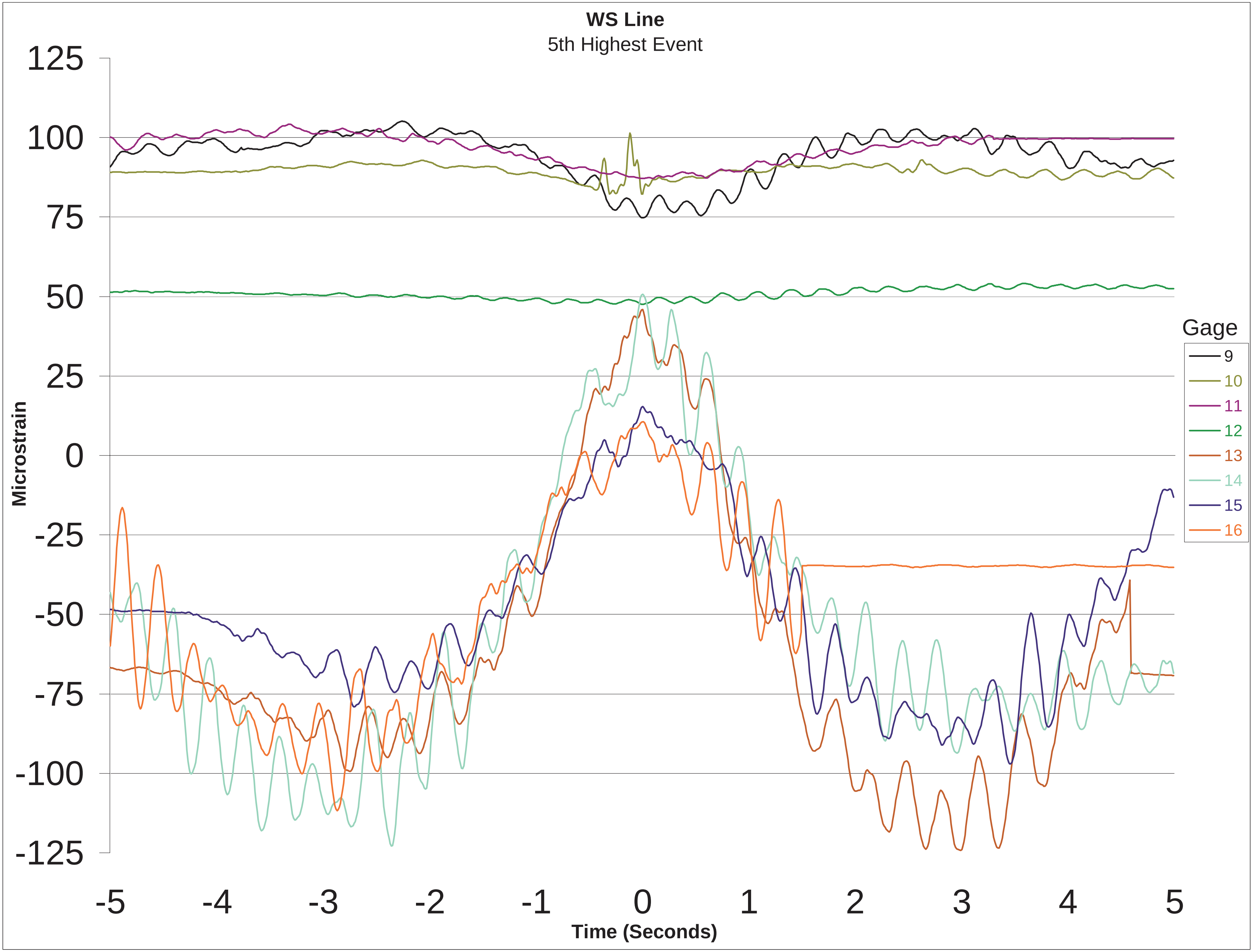

WS Line
2nd Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16



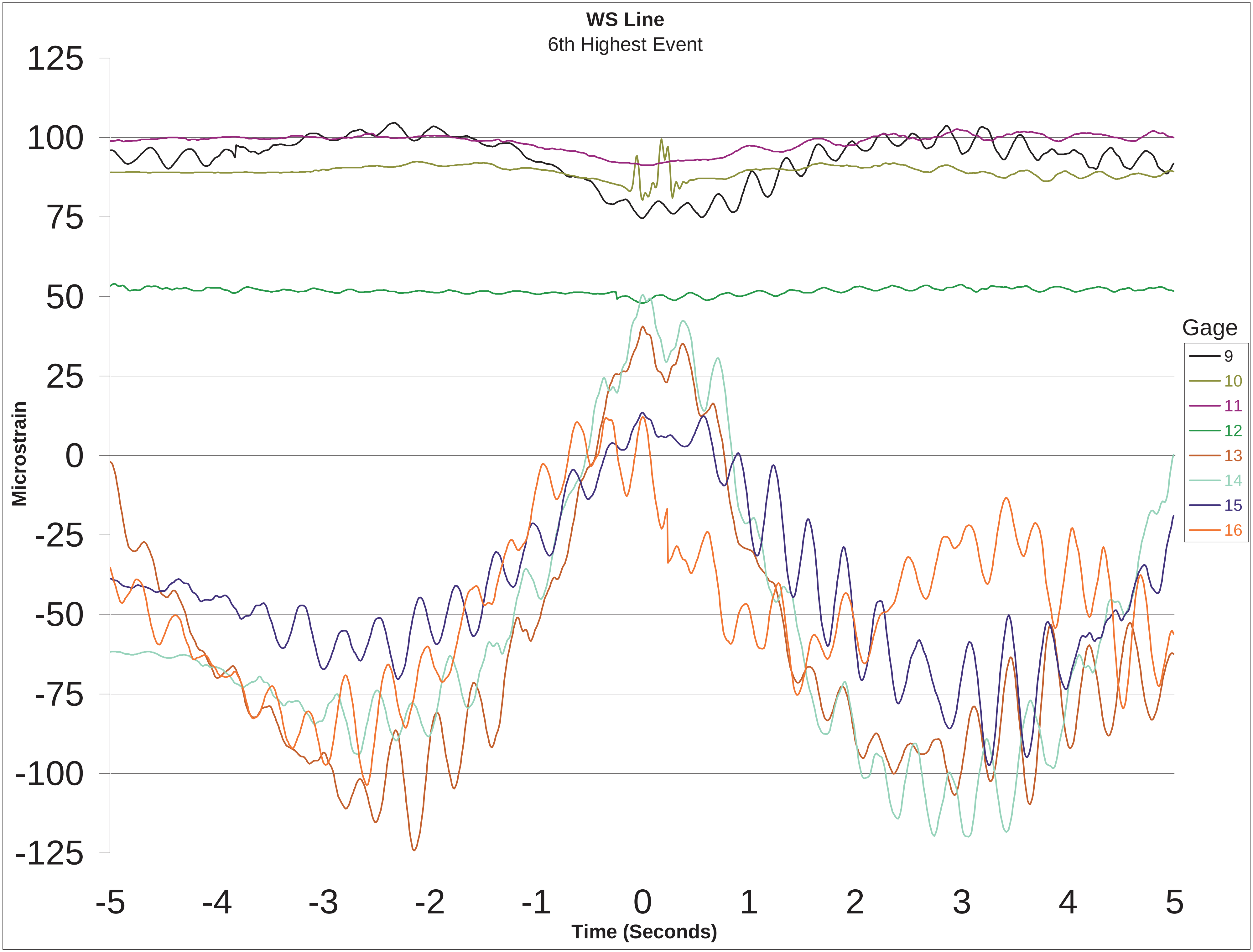
WS Line
3rd Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16



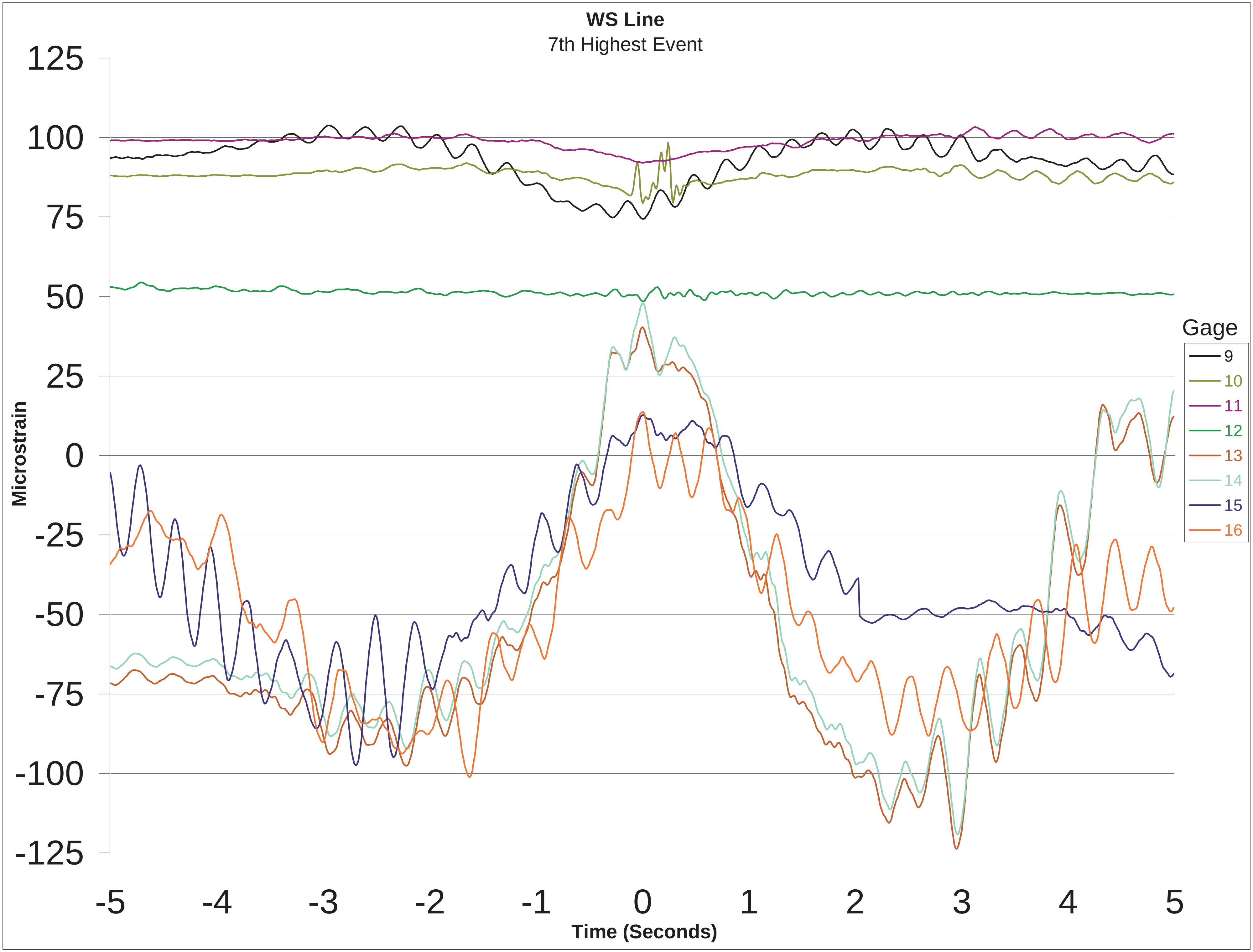
WS Line
4th Highest Event
Microstrain
Time (Seconds)
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Gage
9
10
11
12
13
14
15
16

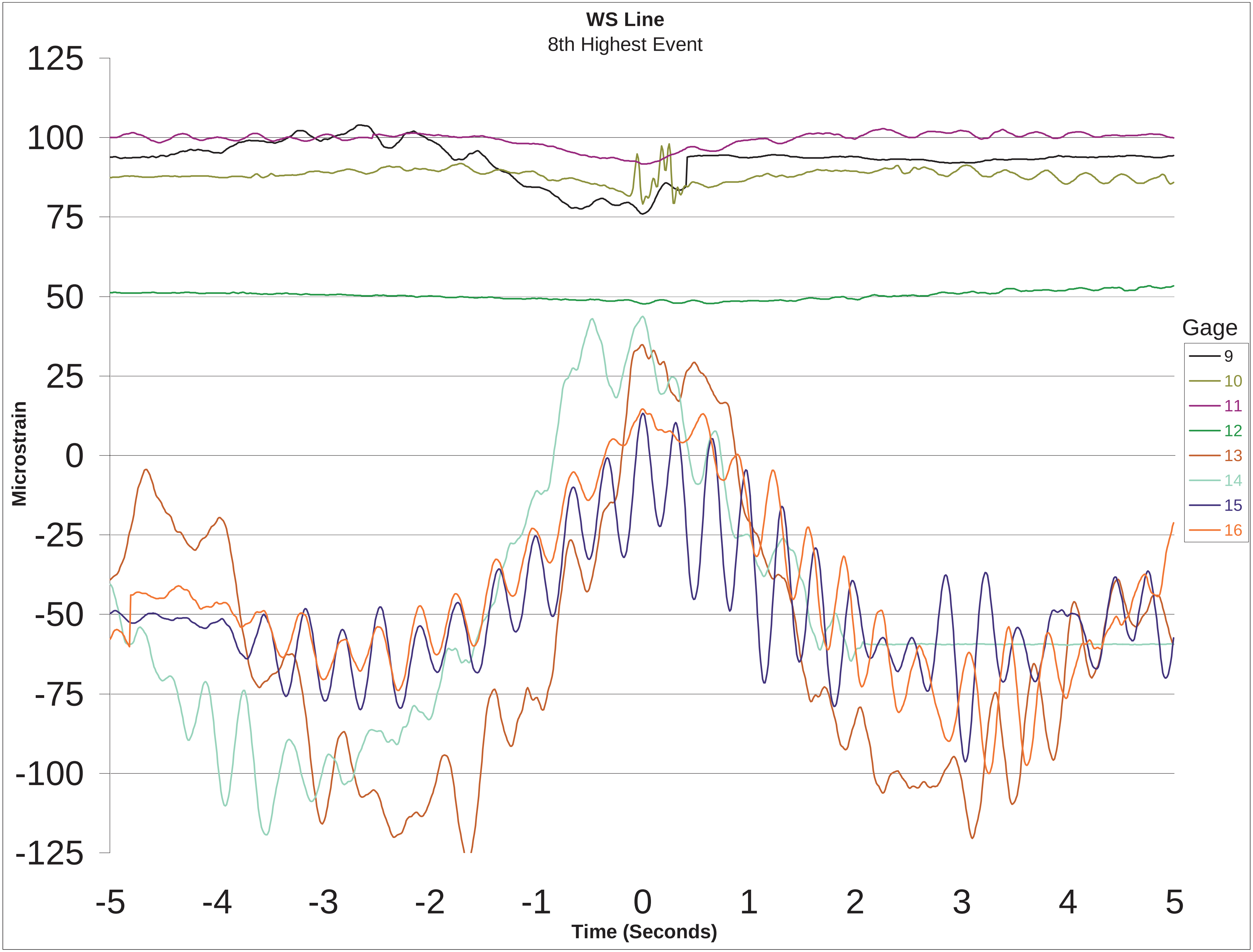


WS Line
5th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16



WS Line
6th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16


WS Line
7th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16


WS Line
8th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
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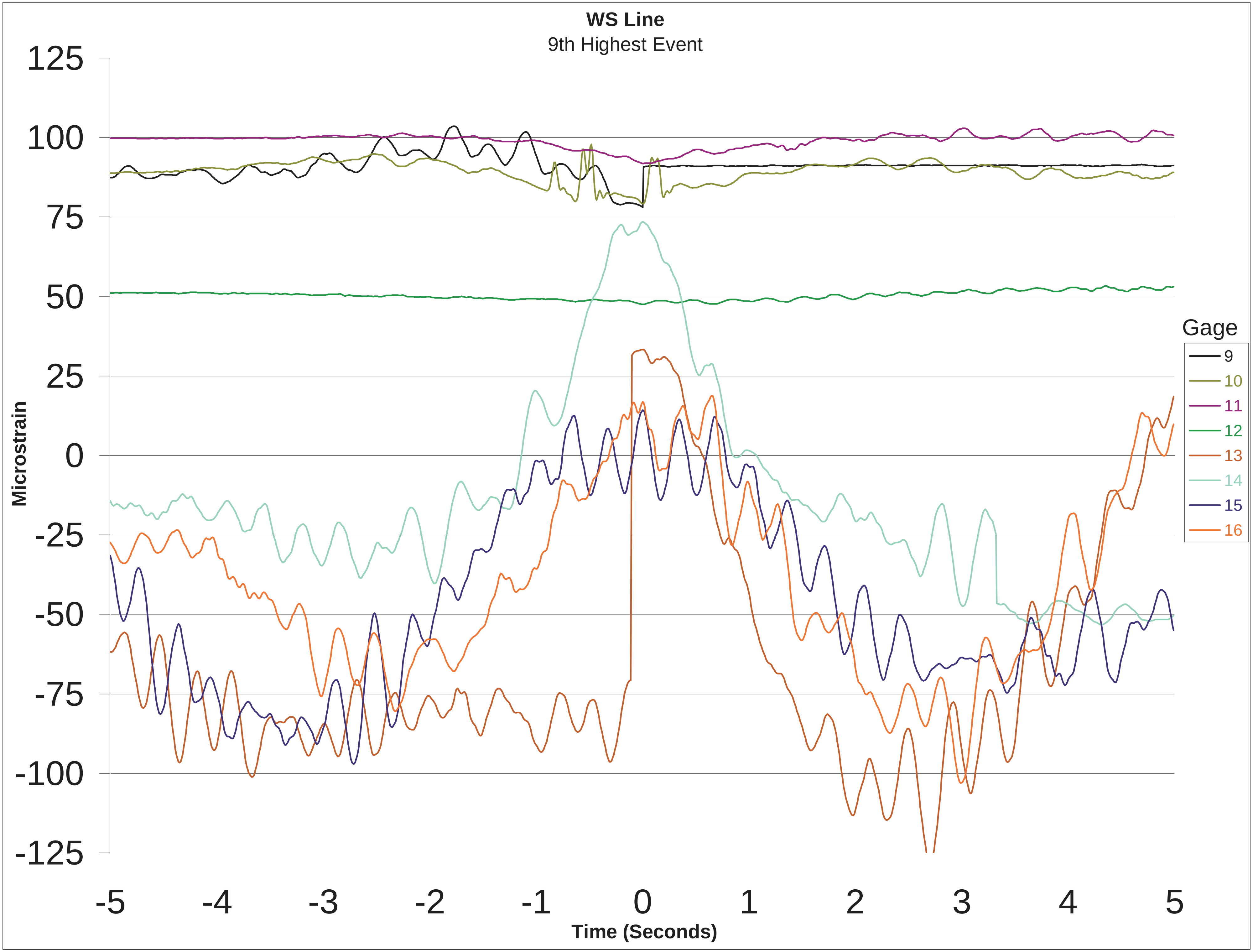
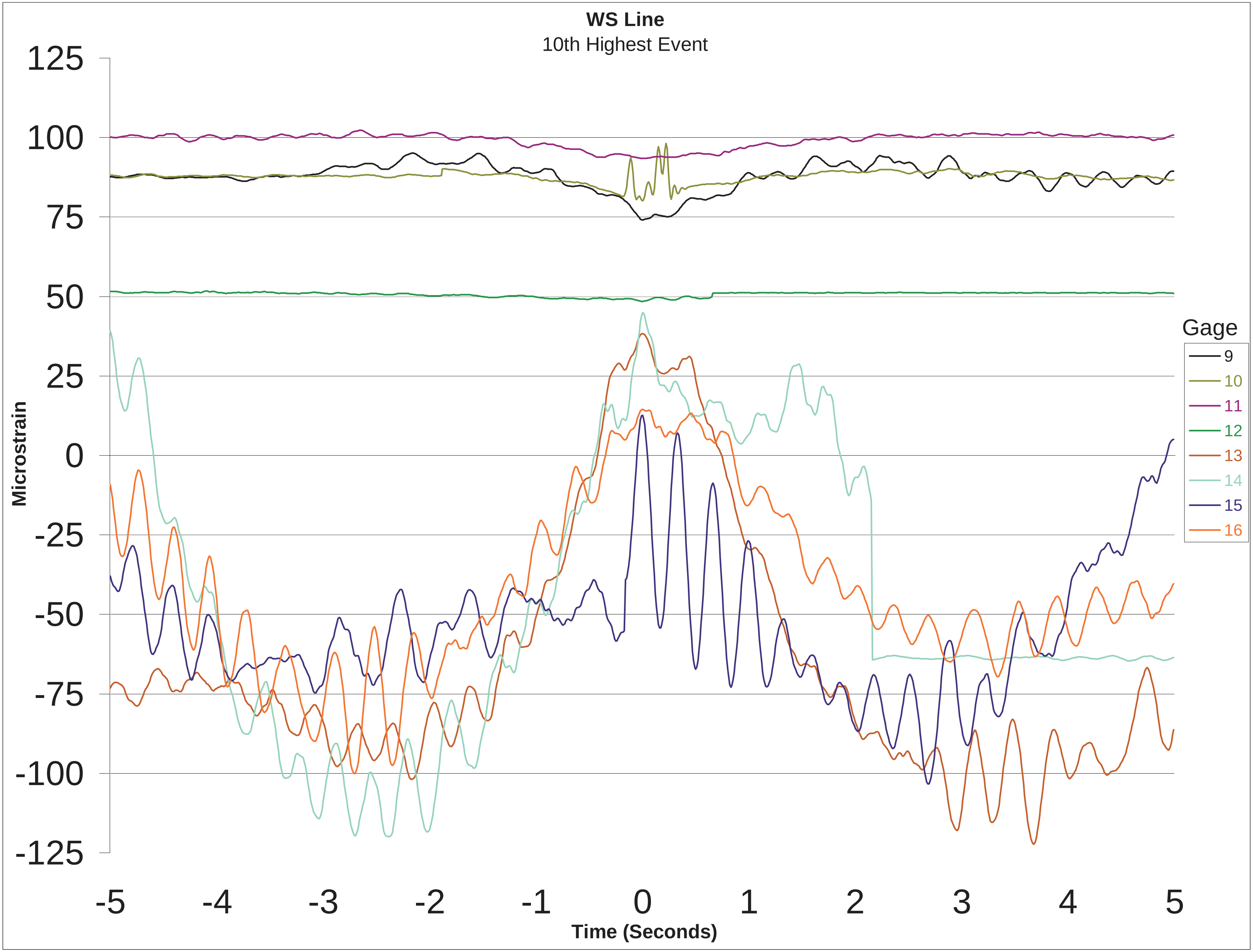
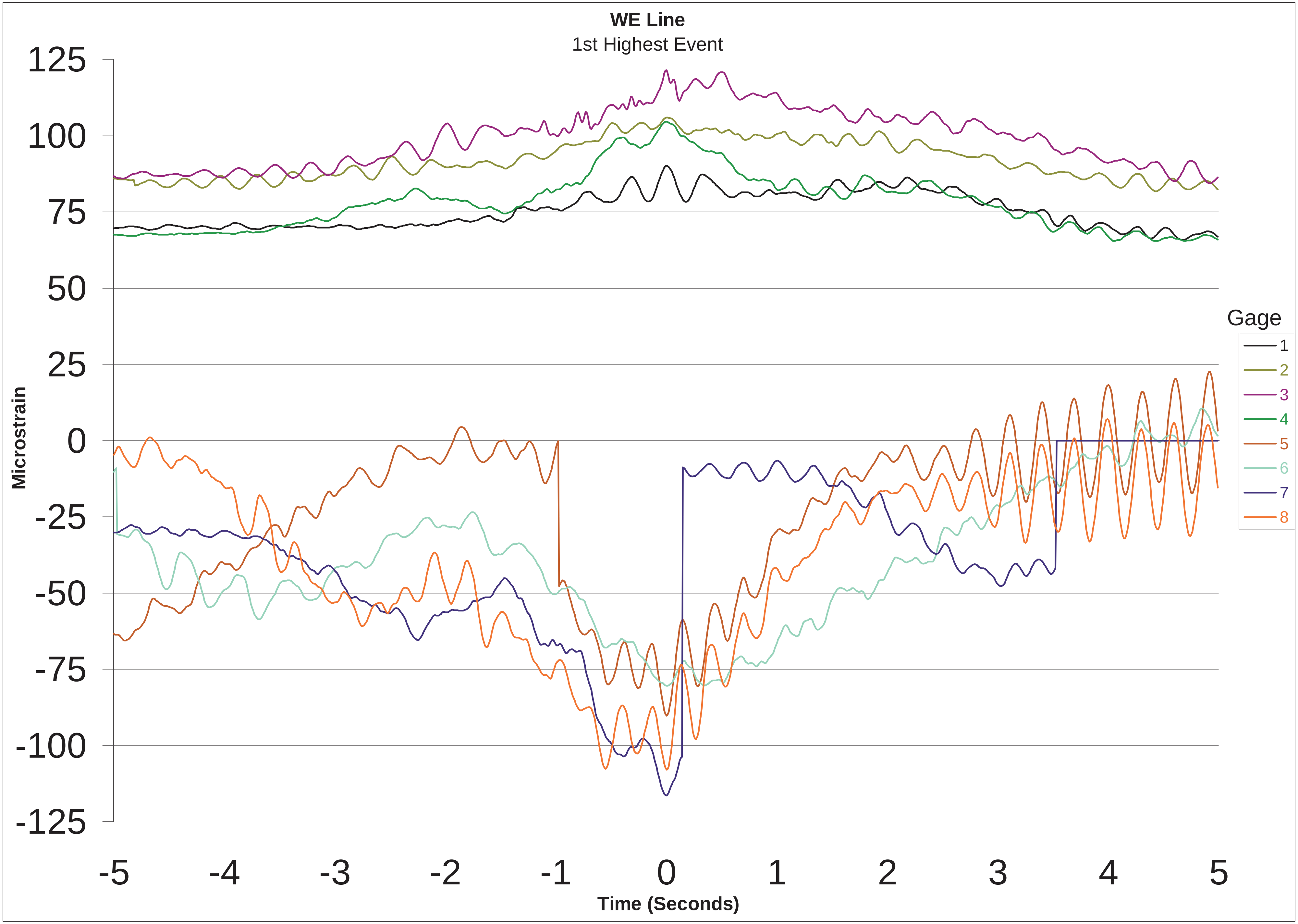
15
16

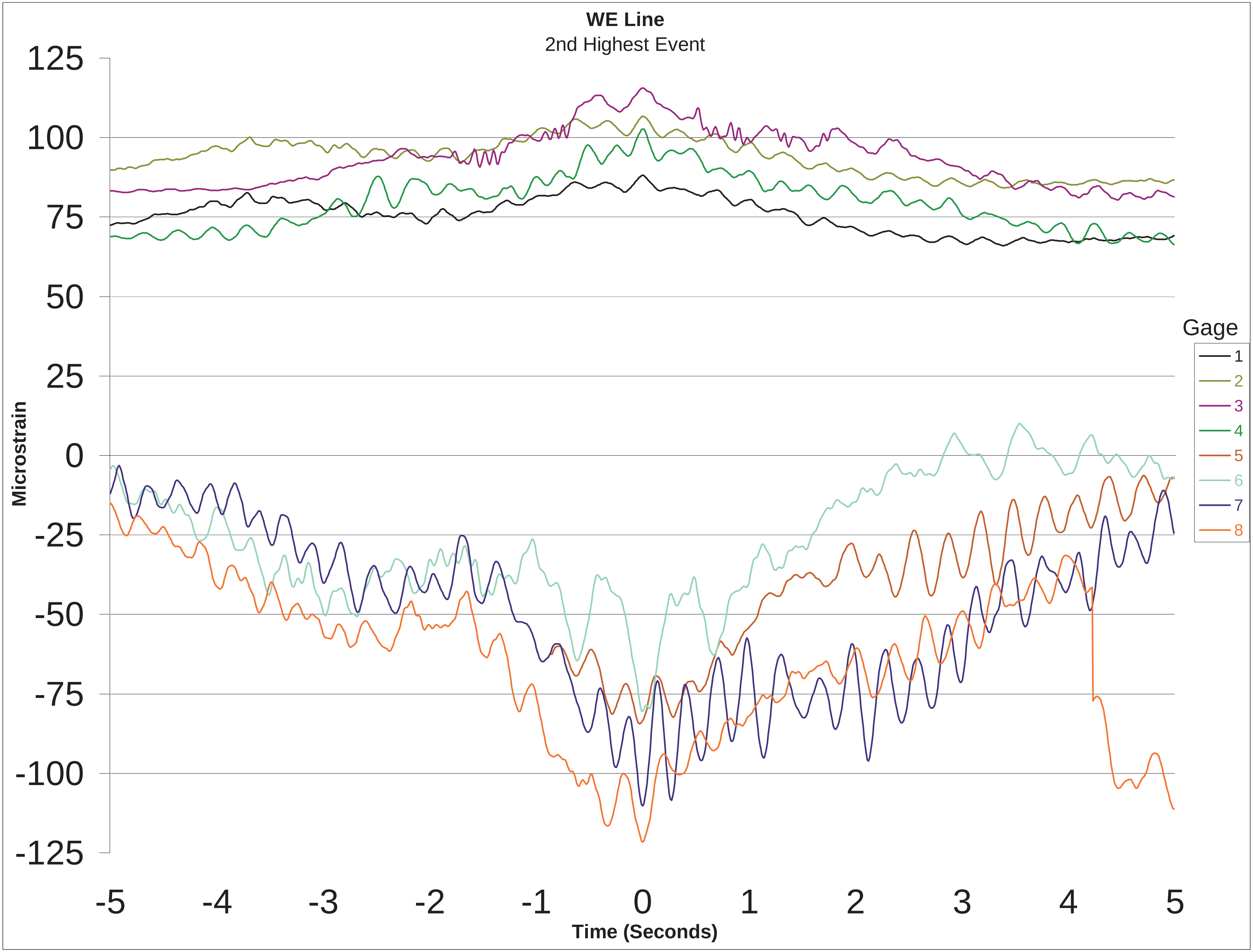


WS Line
9th Highest Event
Microstrain
Time (Seconds)
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Gage
9
10
11
12
13
14
15
16

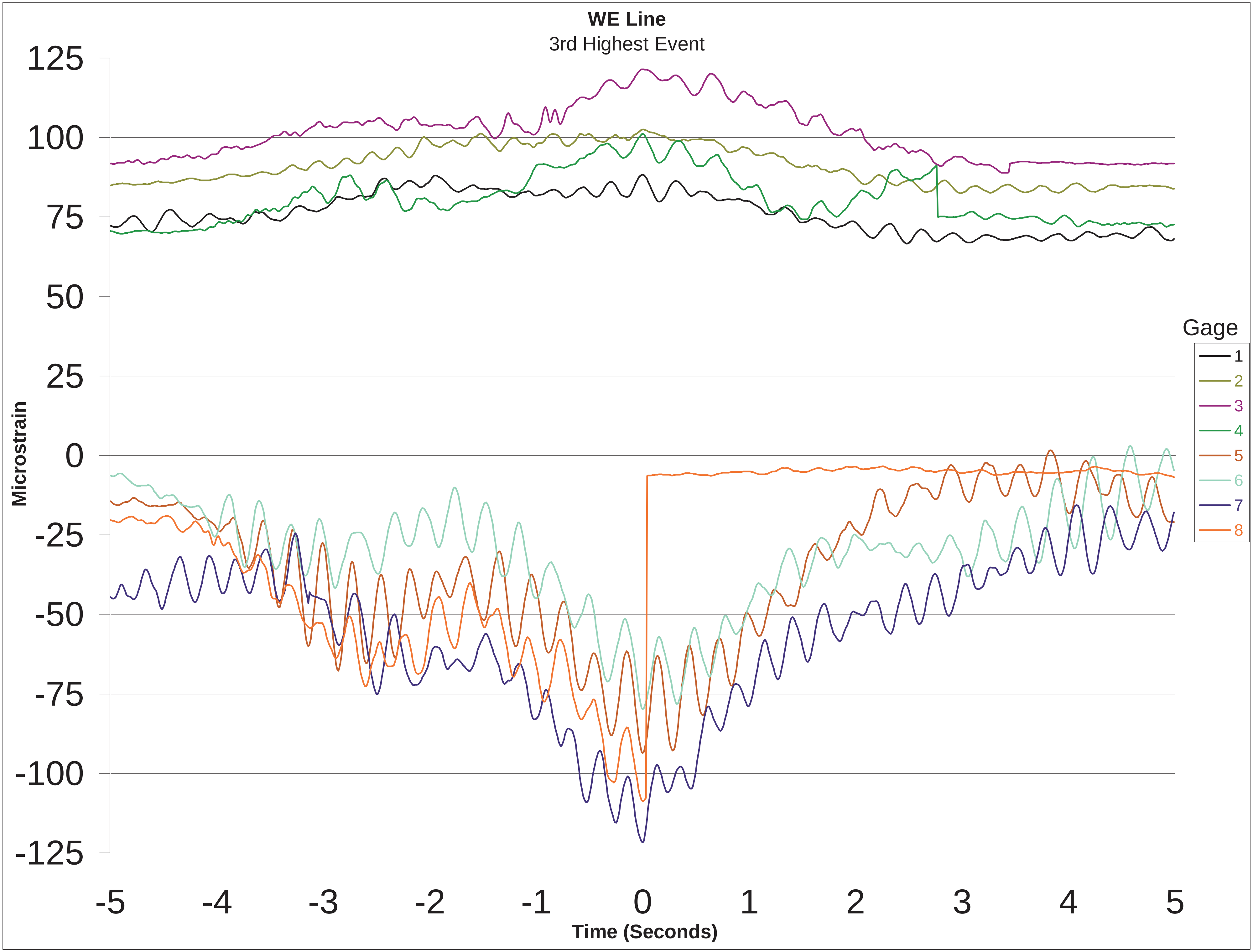


WS Line
10th Highest Event
125
100
75
50
25
0
-25
-50
-75
-100
-125
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

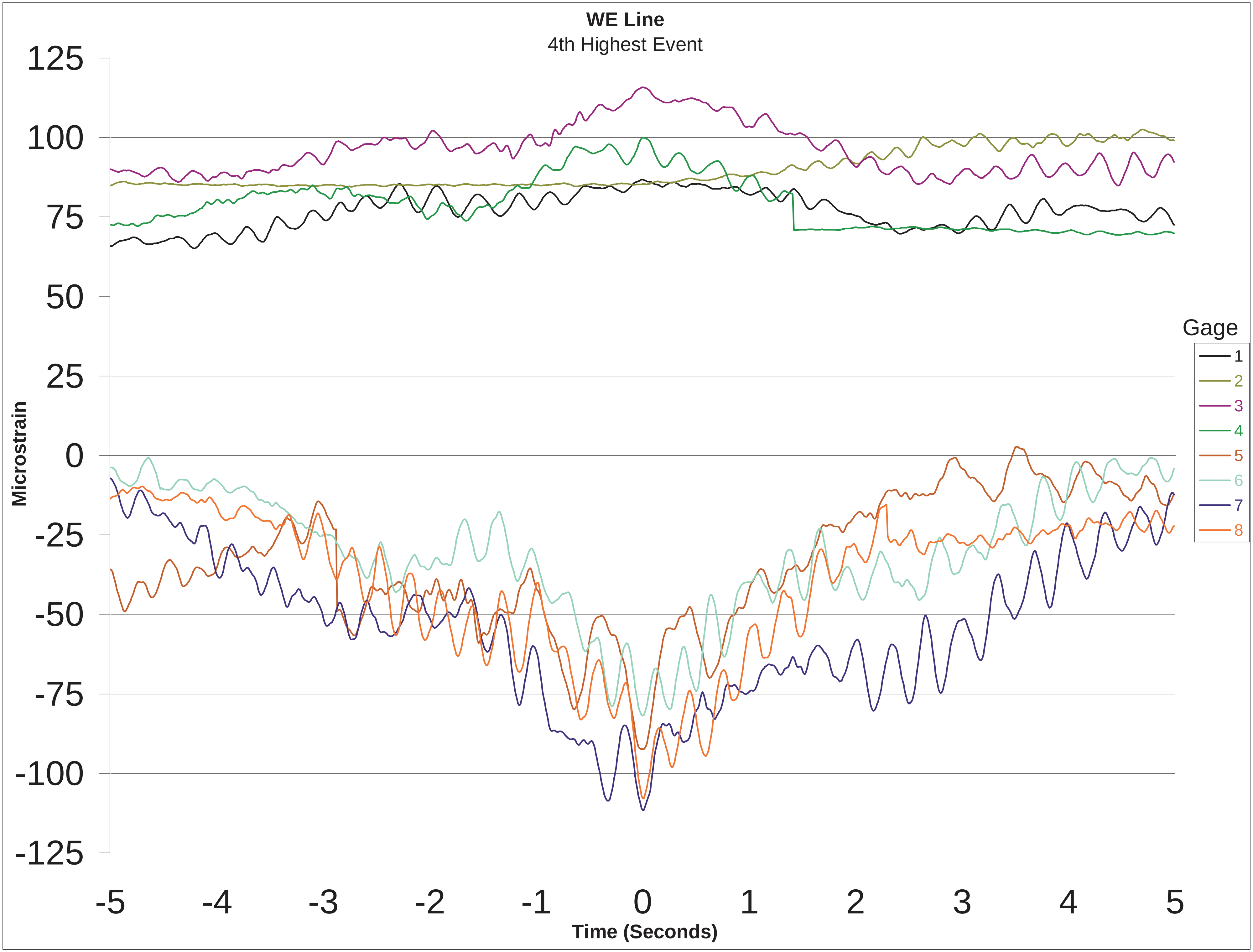

WE Line
1st Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



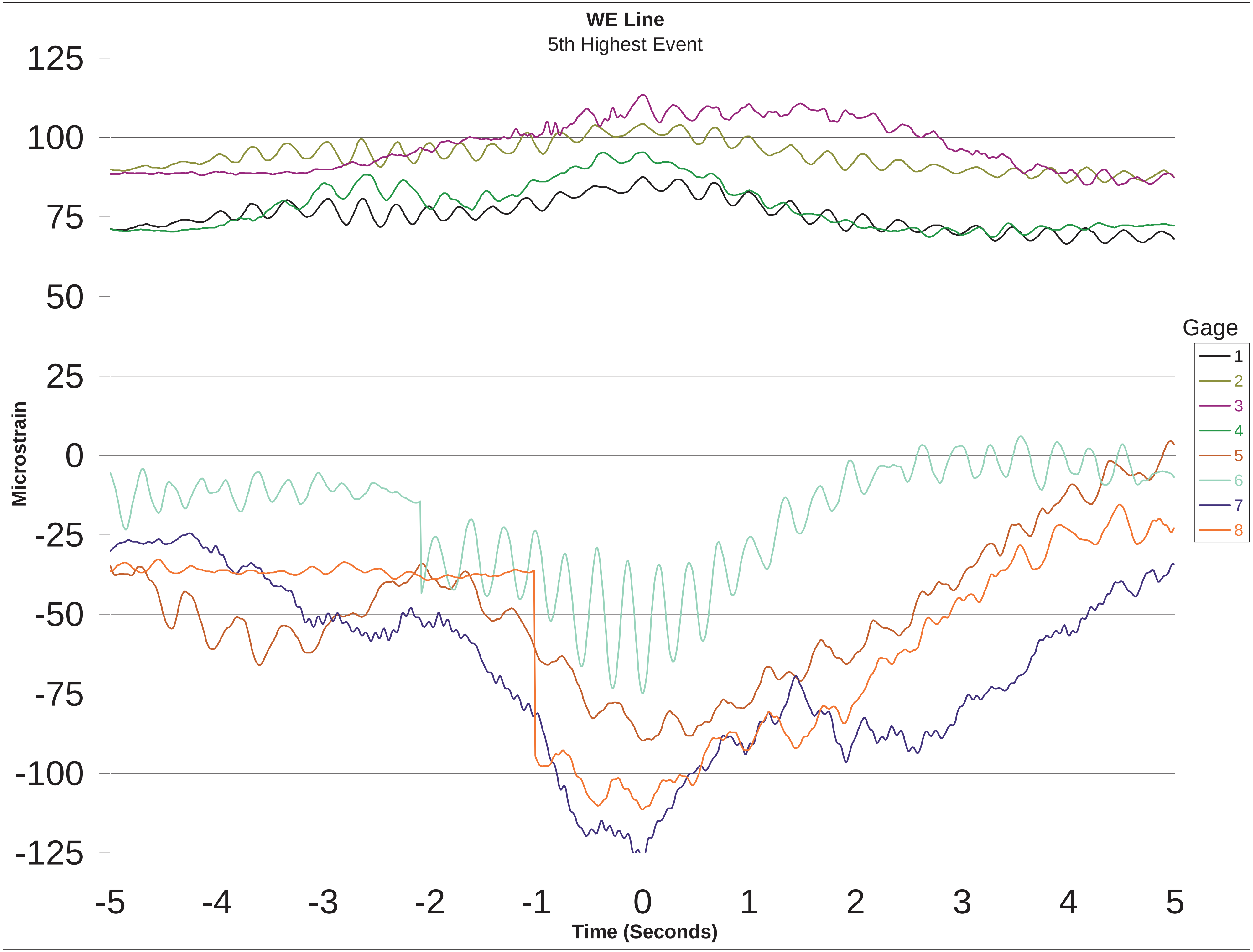
WE Line
2nd Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

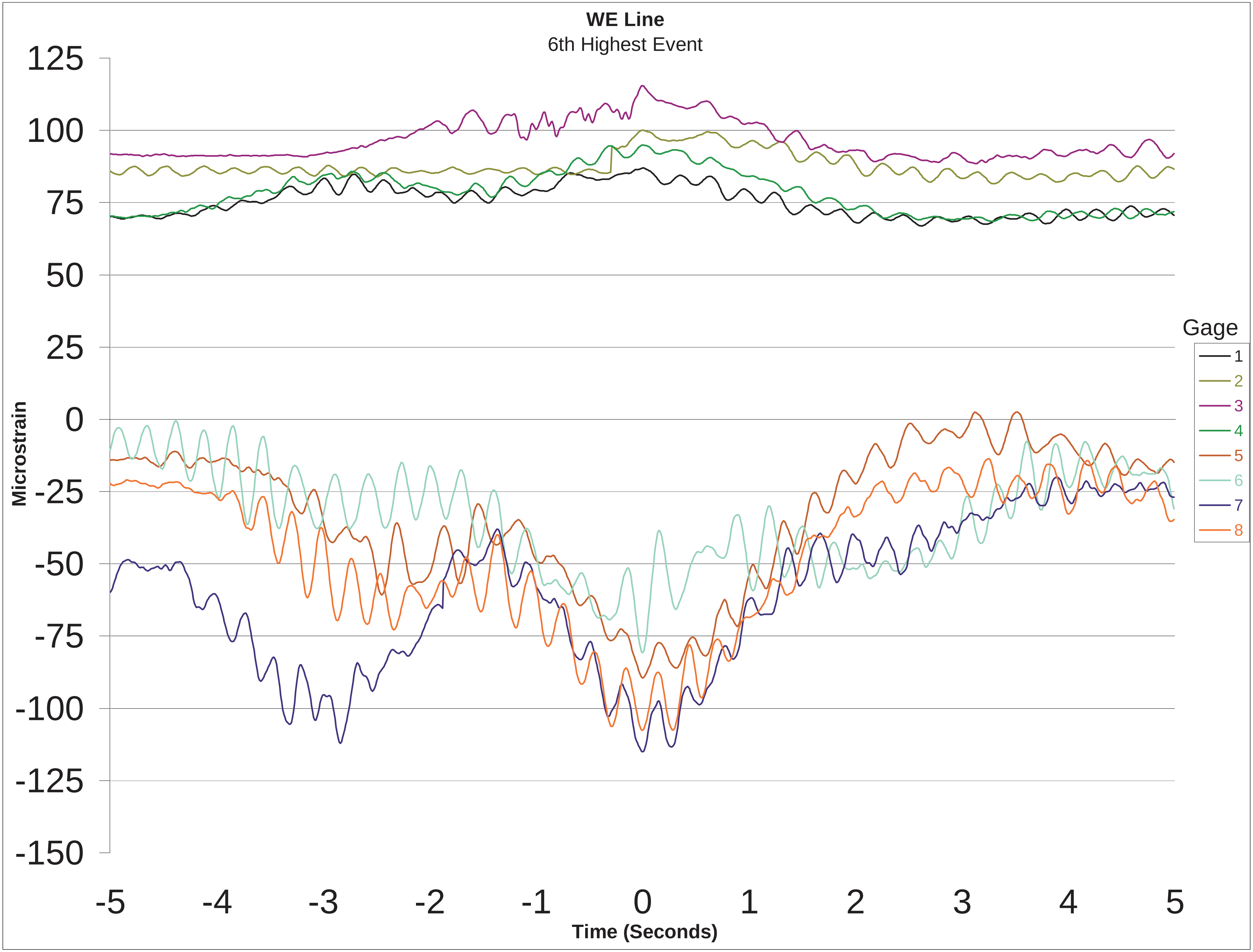


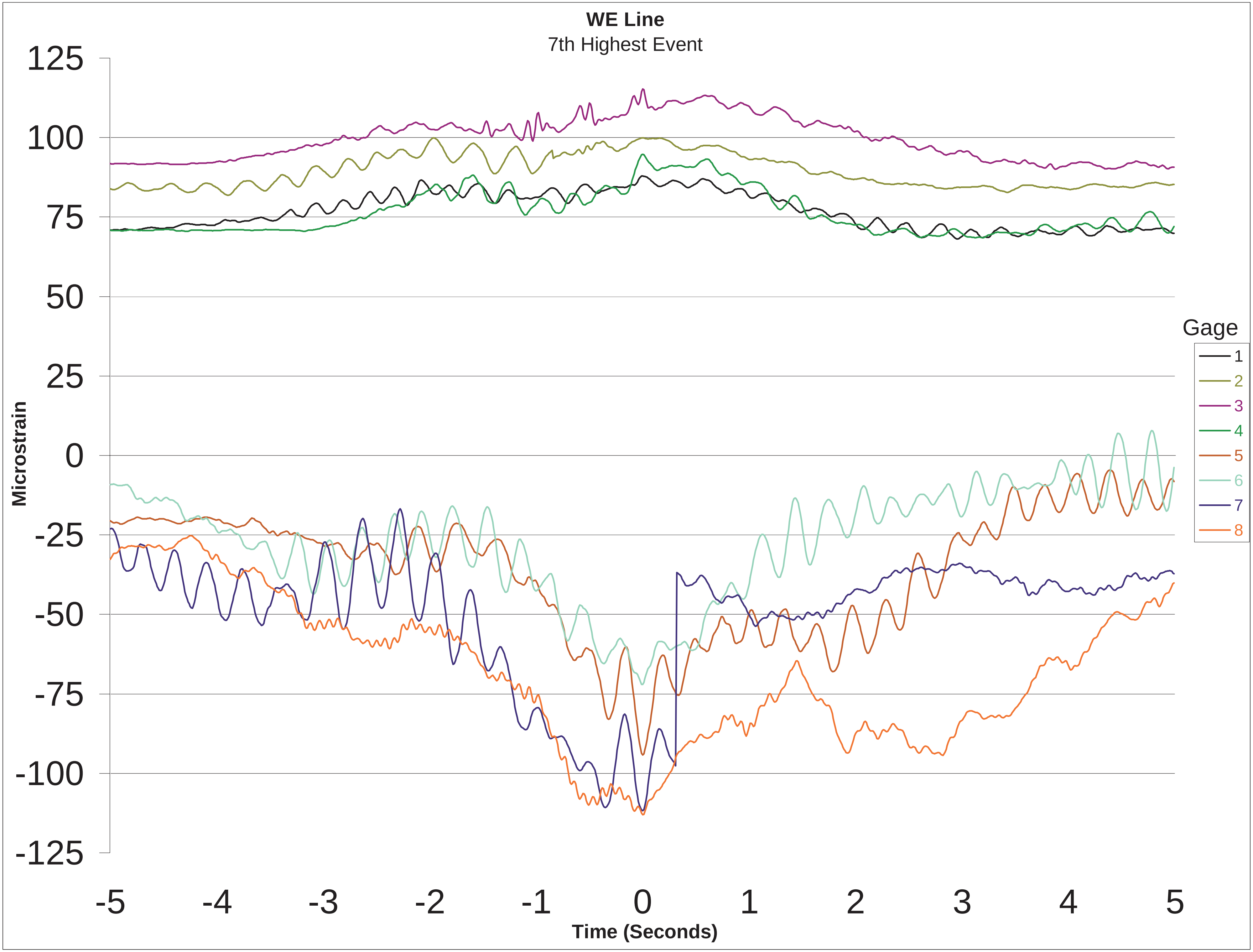
WE Line
3rd Highest Event
125
100
75
50
25
0
-25
-50
-75
-100
-125
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



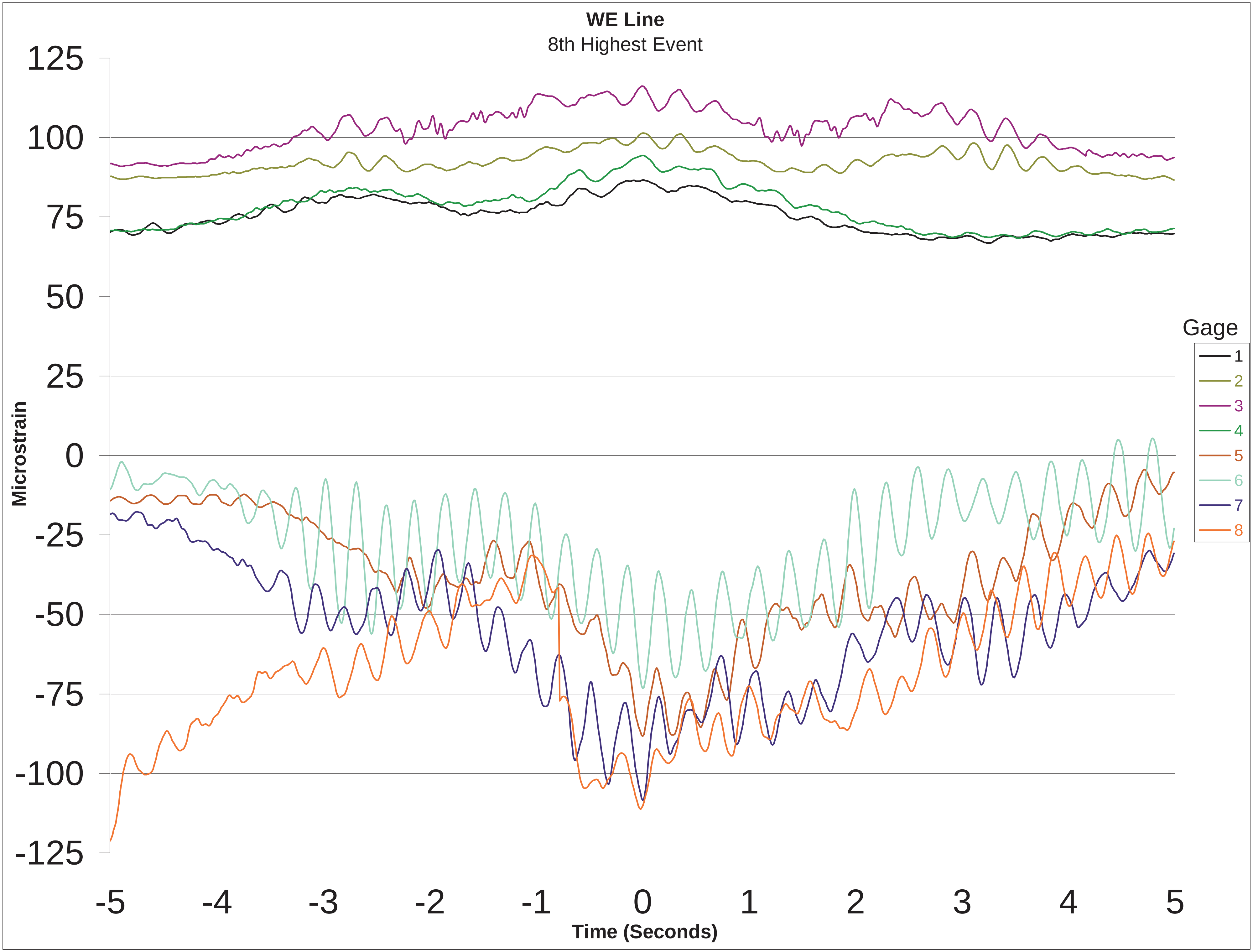
WE Line
4th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


WE Line
5th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

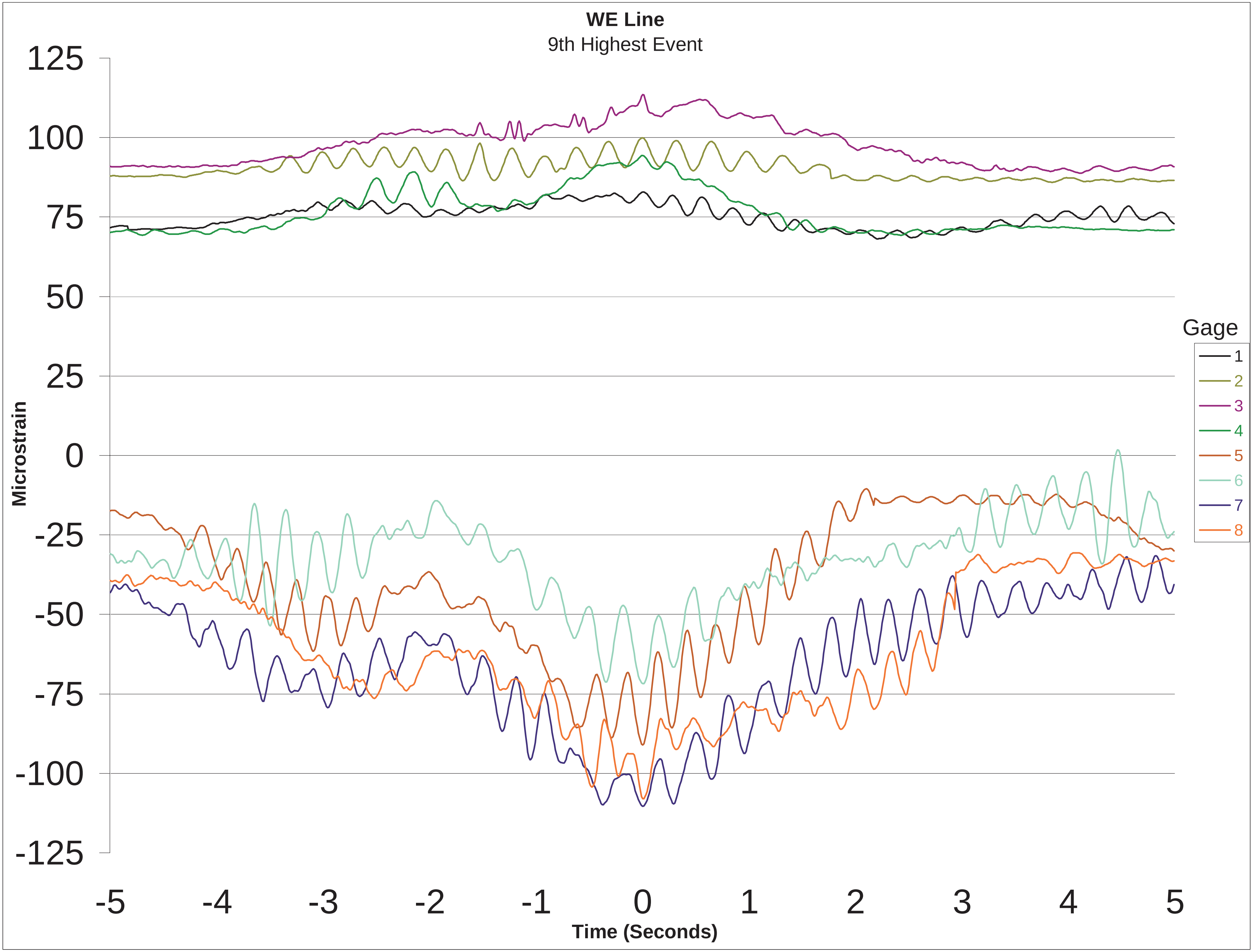

WE Line
6th Highest Event
125
100
75
50
25
0
-25
-50
-75
-100
-125
-150
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

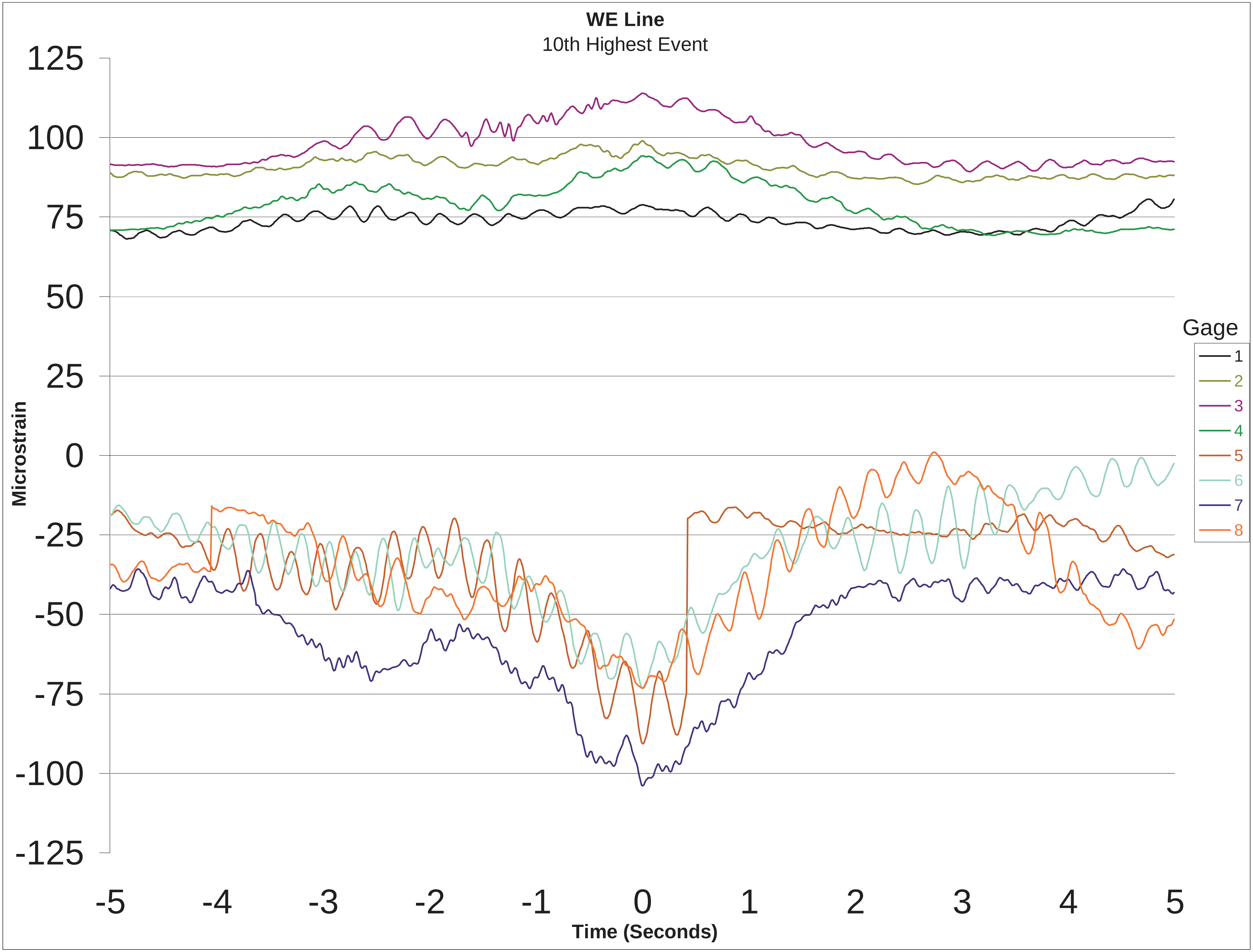


WE Line
7th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

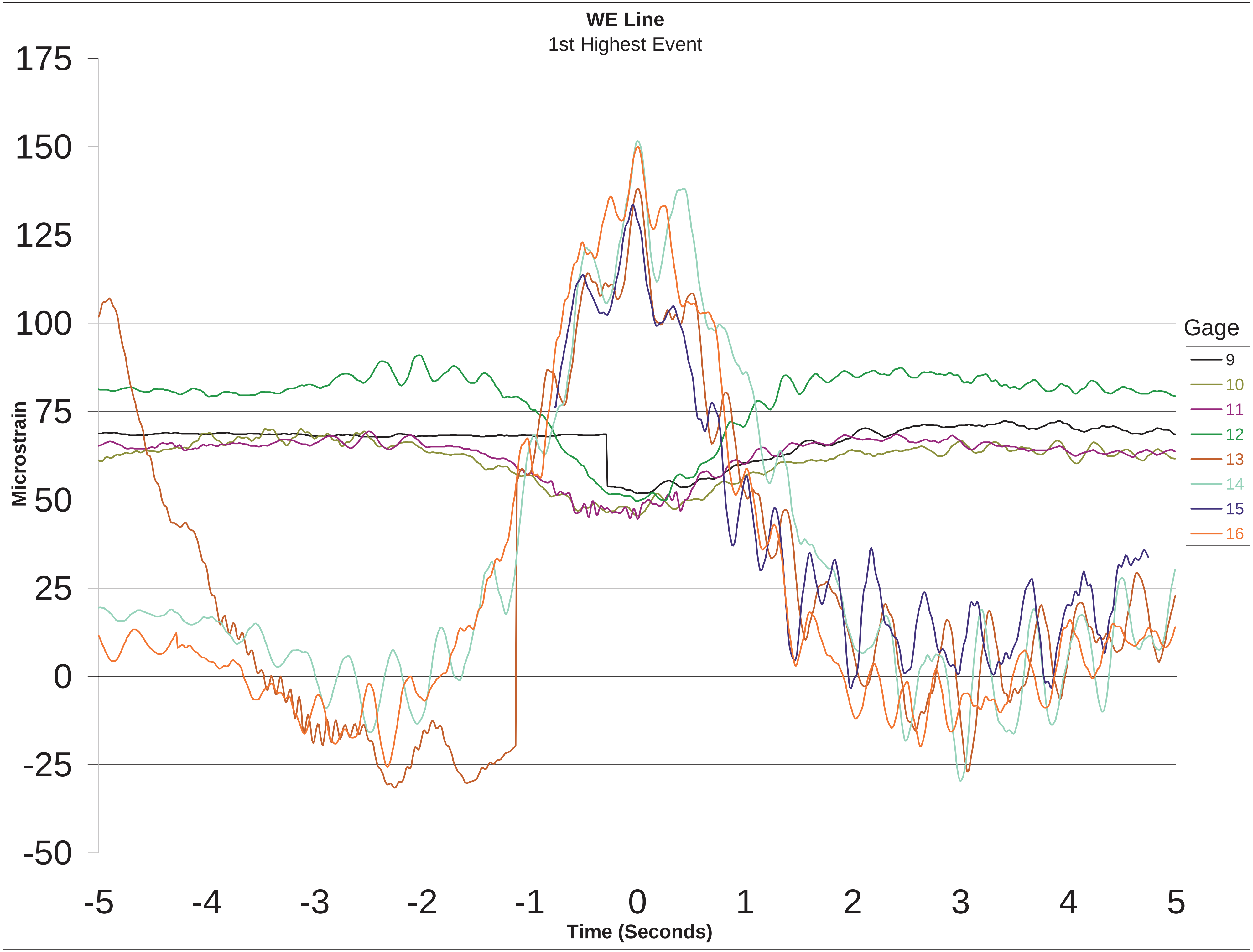


WE Line
8th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

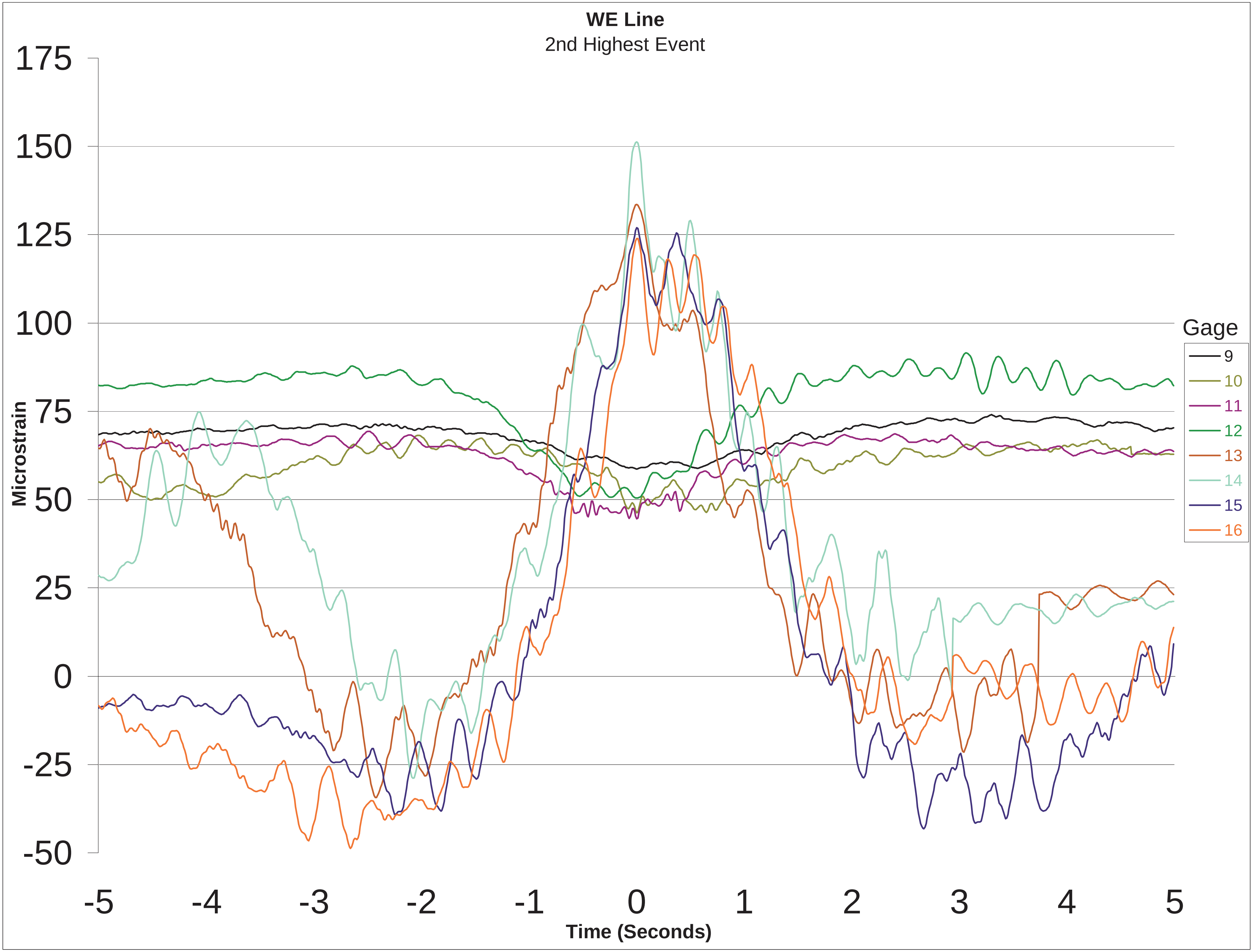

WE Line
9th Highest Event
Microstrain
Time (Seconds)
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Gage
1
2
3
4
5
6
7
8



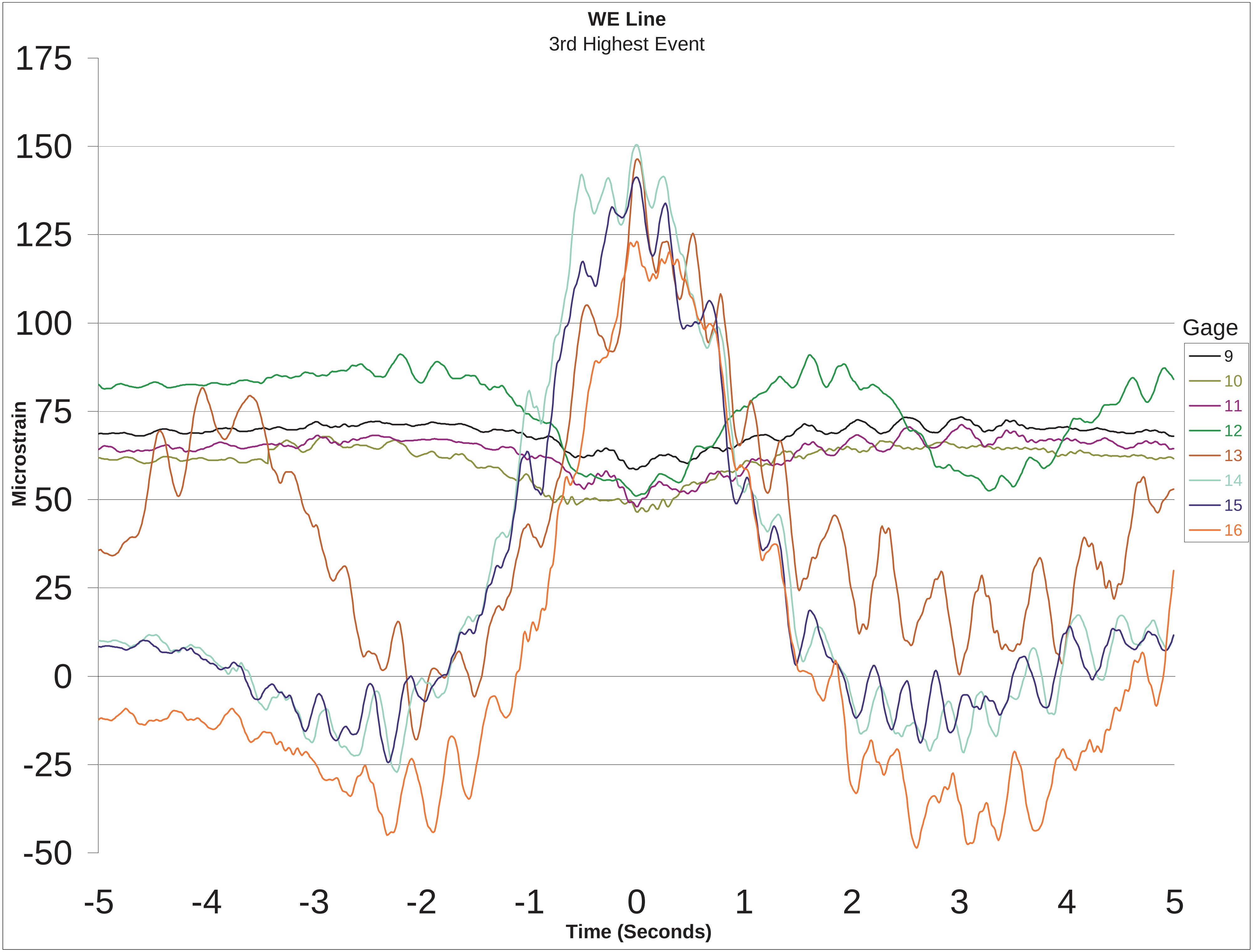
WE Line
10th Highest Event
Microstrain
125
100
75
50
25
0
-25
-50
-75
-100
-125
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

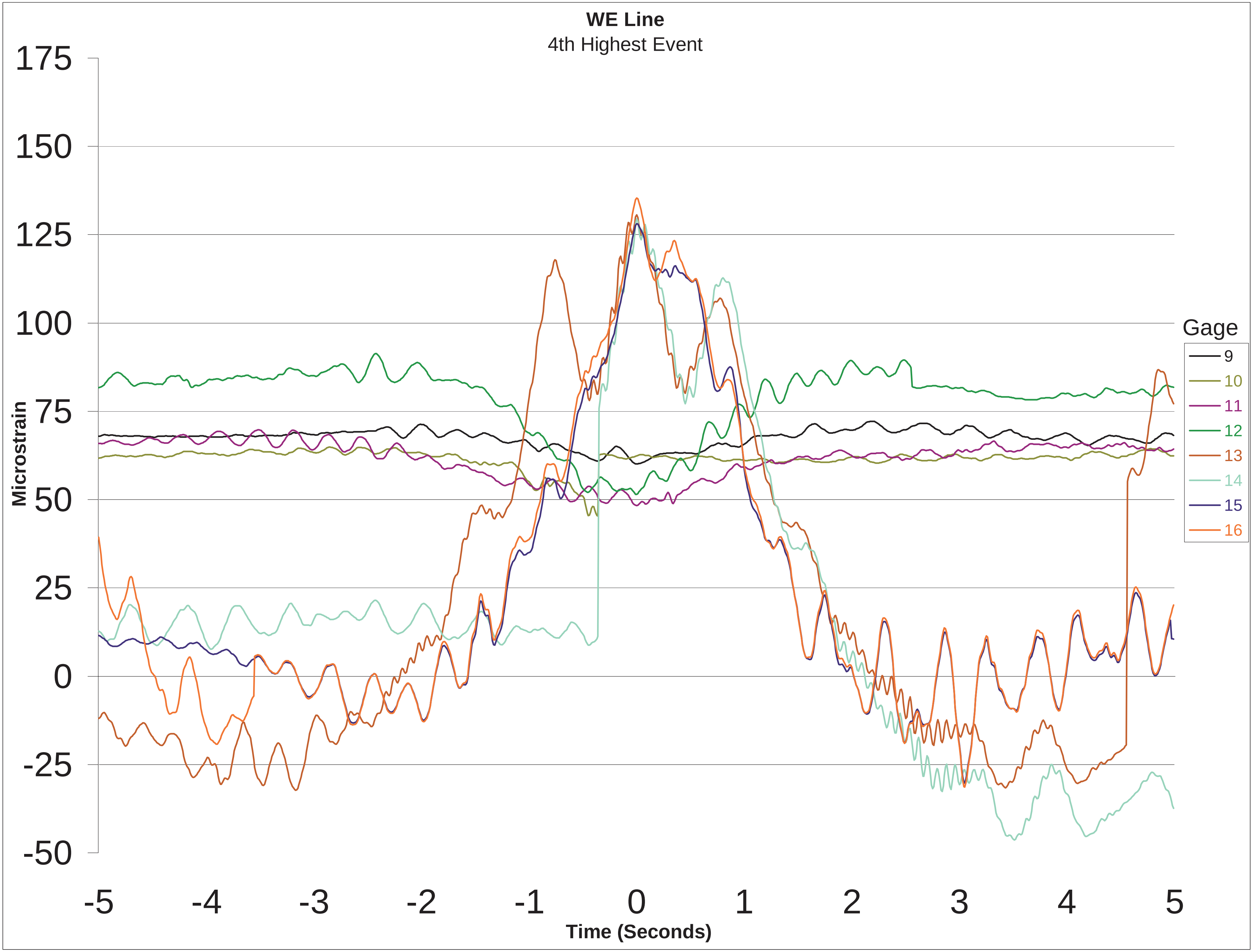


WE Line
1st Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

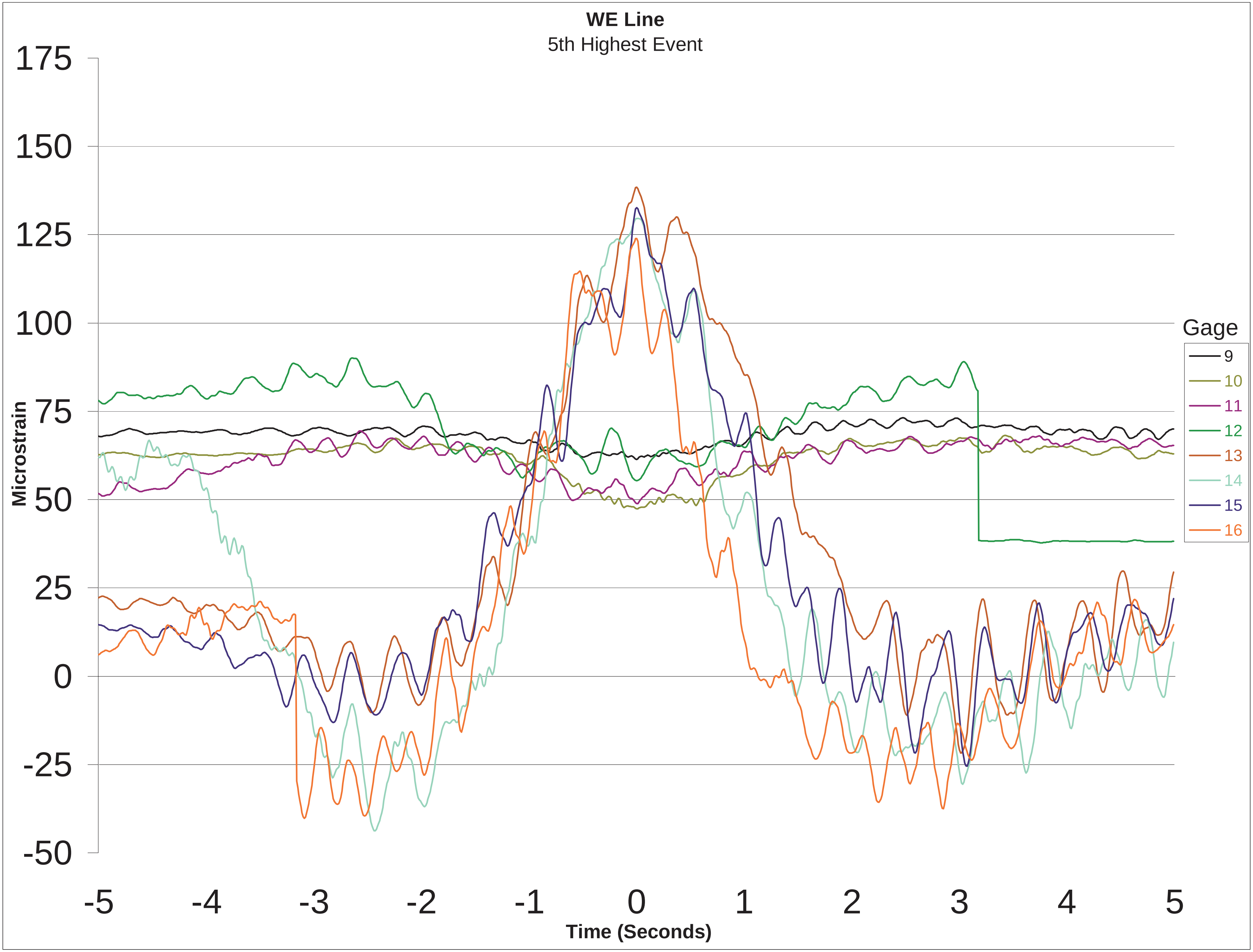


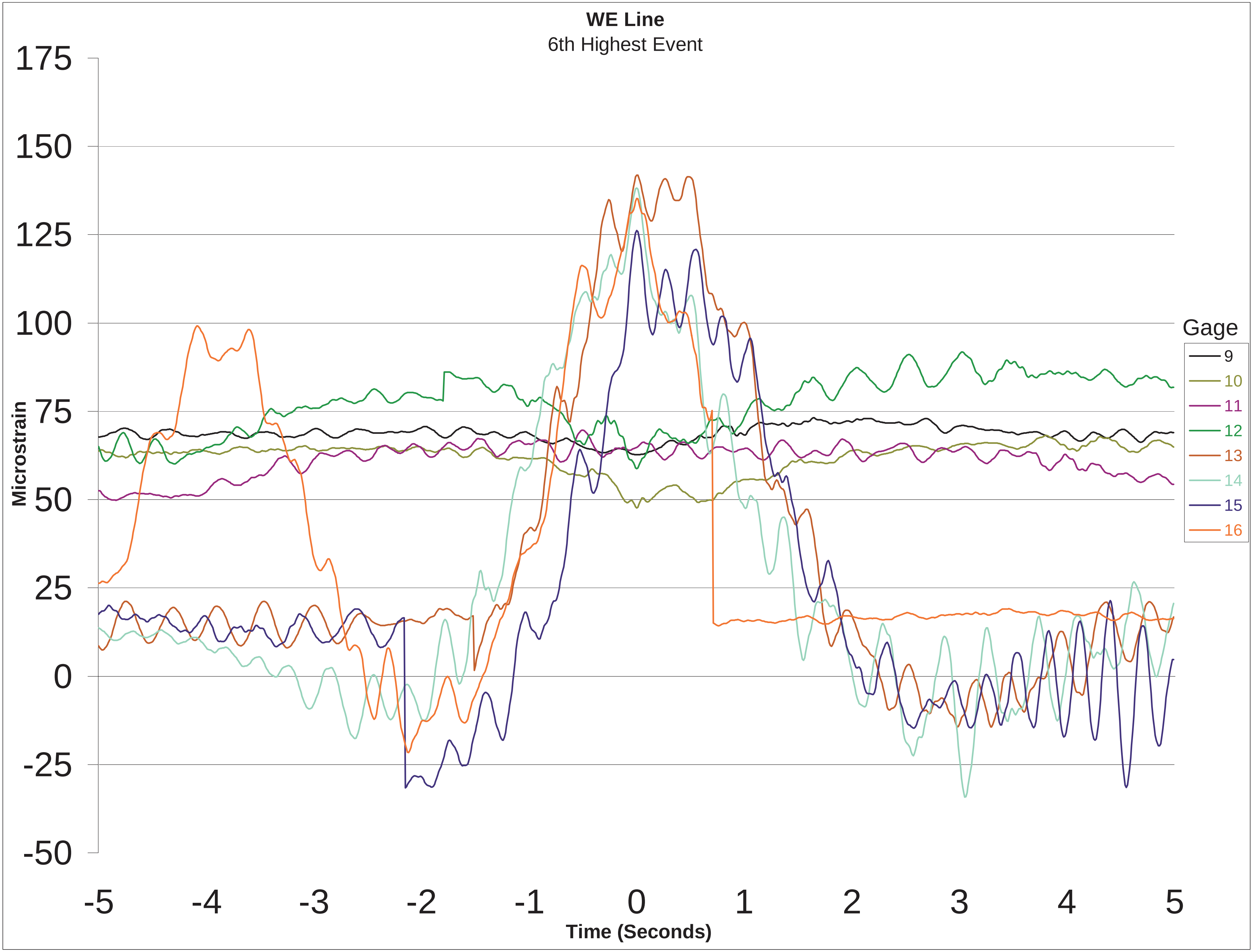
WE Line
2nd Highest Event
175
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16


WE Line
3rd Highest Event
175
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

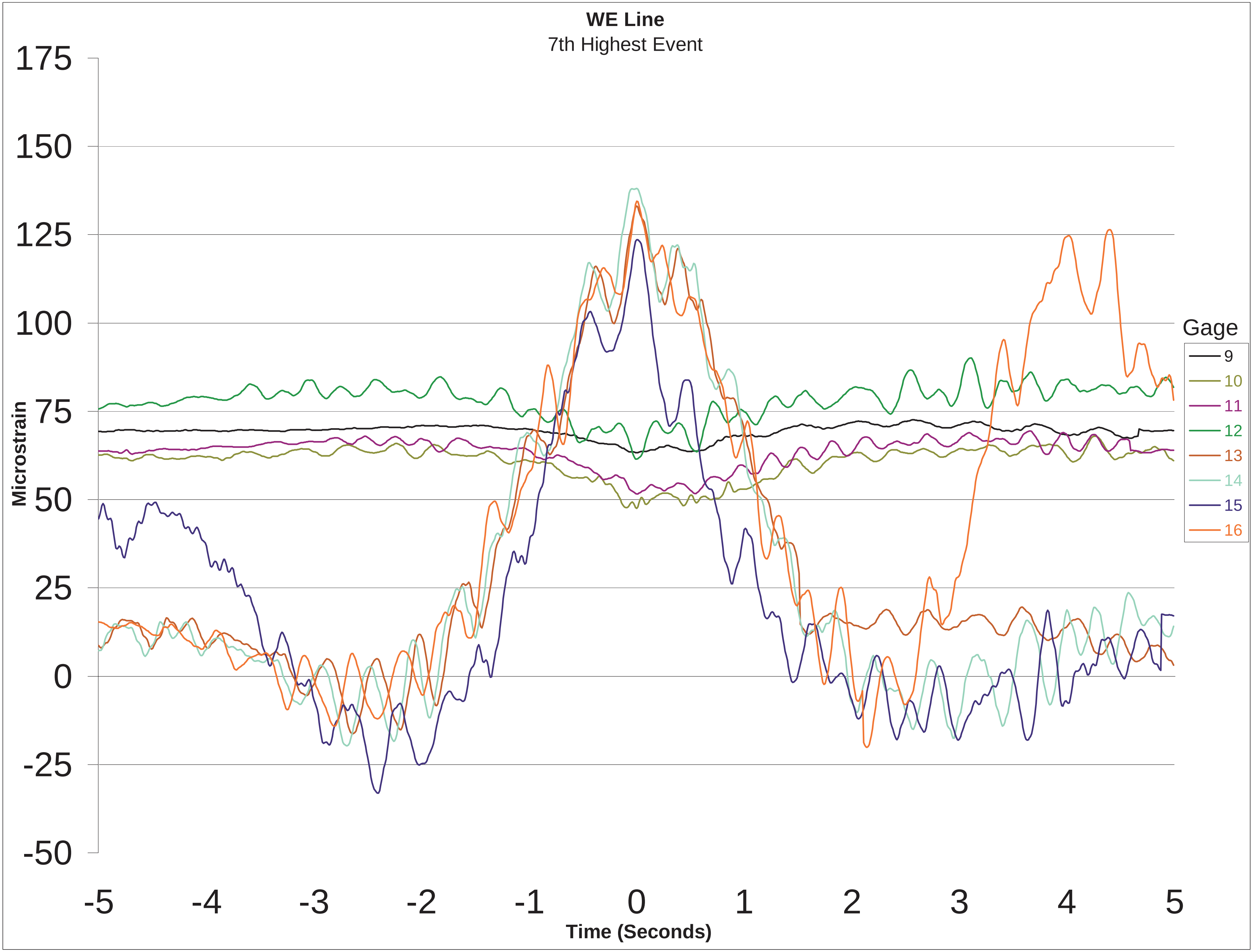


WE Line
4th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

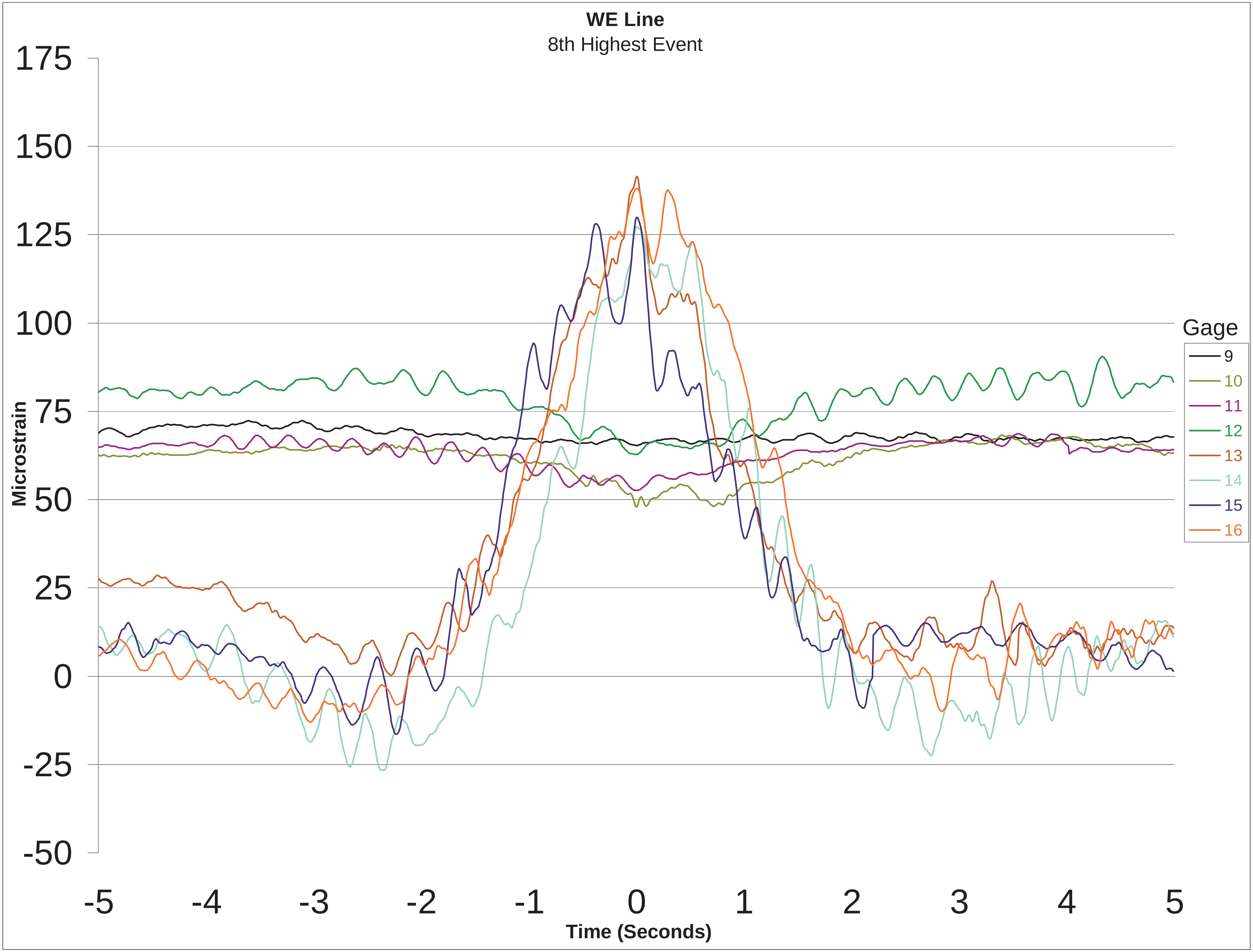

WE Line
5th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16



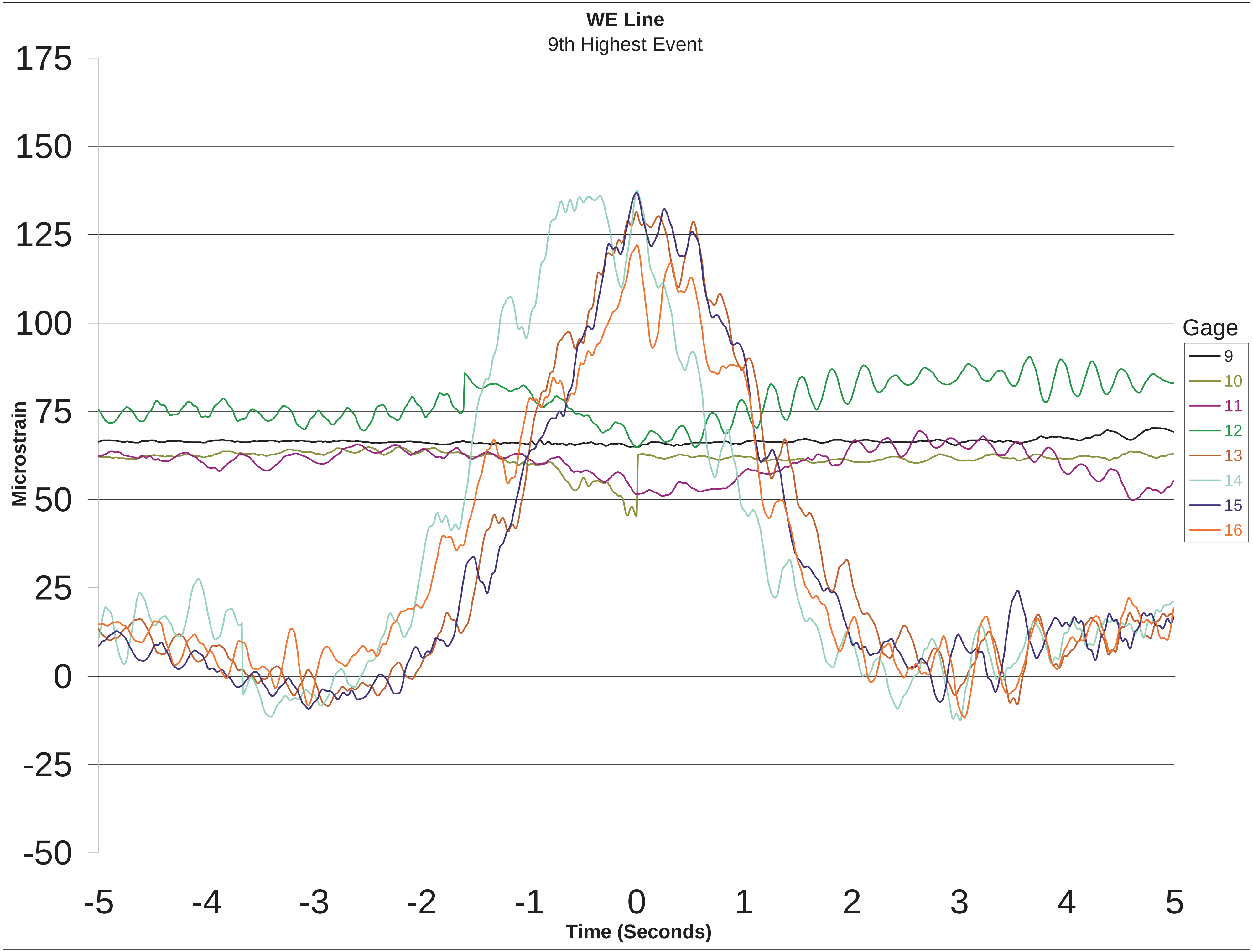
WE Line
6th Highest Event
175
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

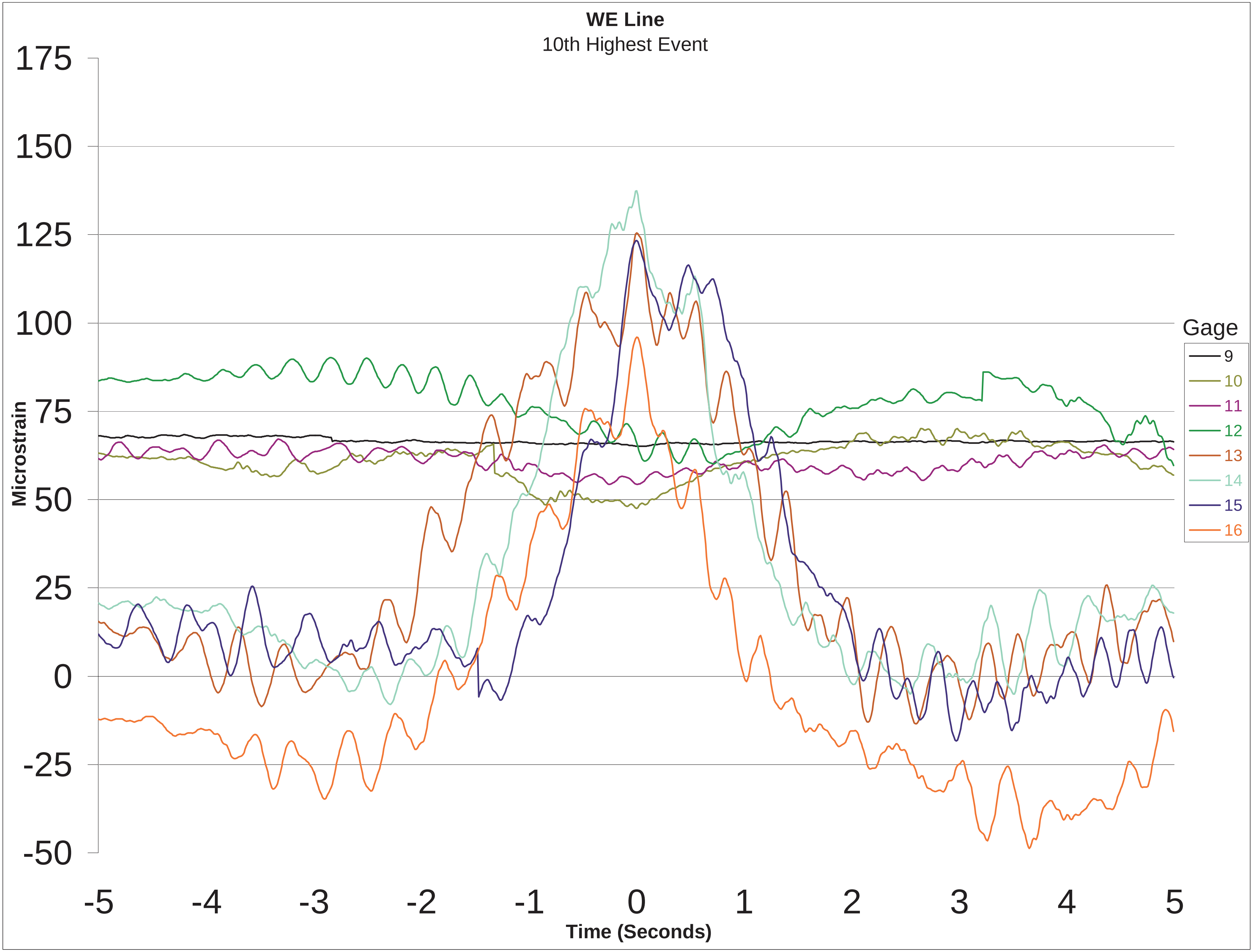


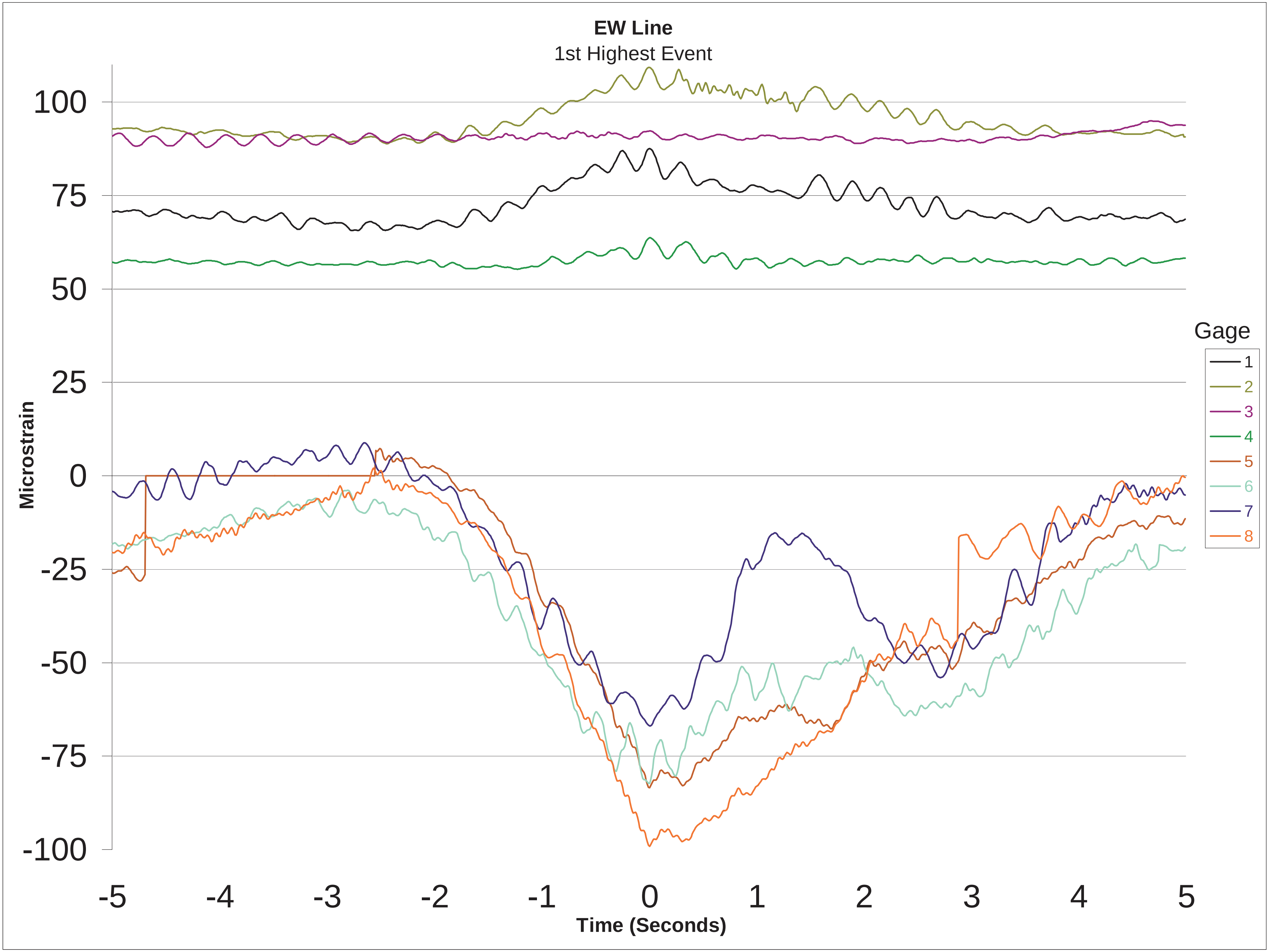
WE Line
7th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16



WE Line
8th Highest Event
175
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16


WE Line
9th Highest Event
175
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16


WE Line
10th Highest Event
175
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16


EW Line
1st Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
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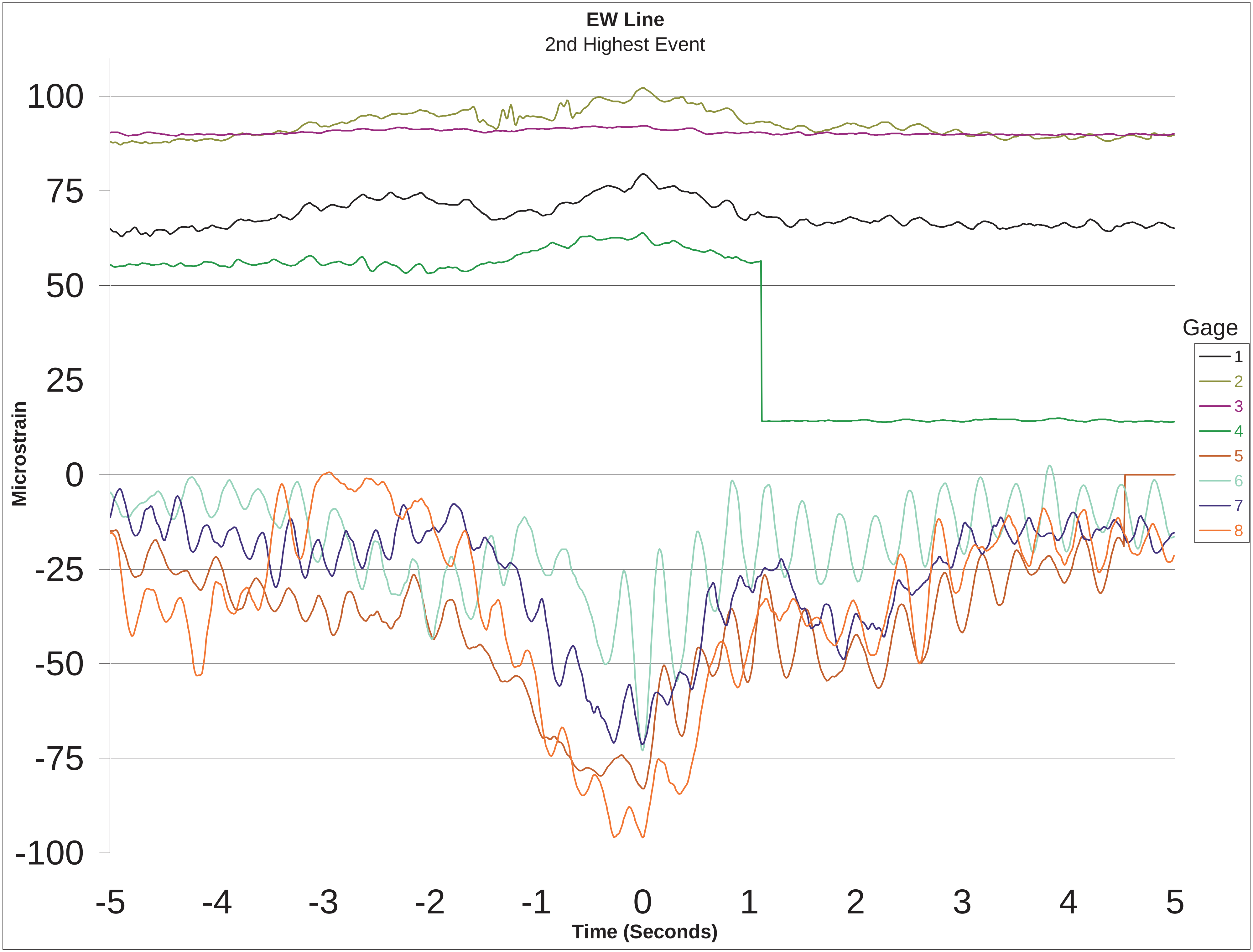
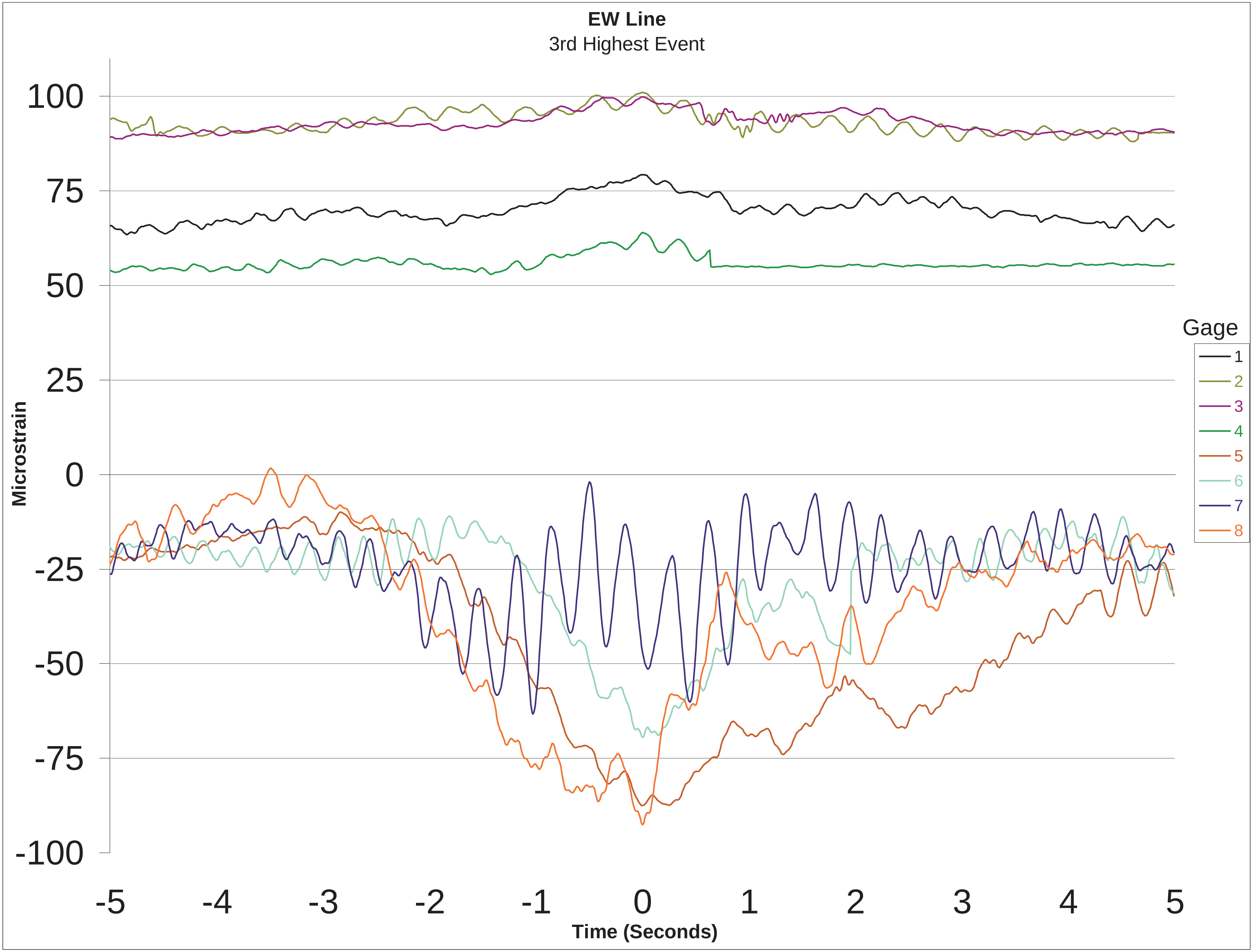
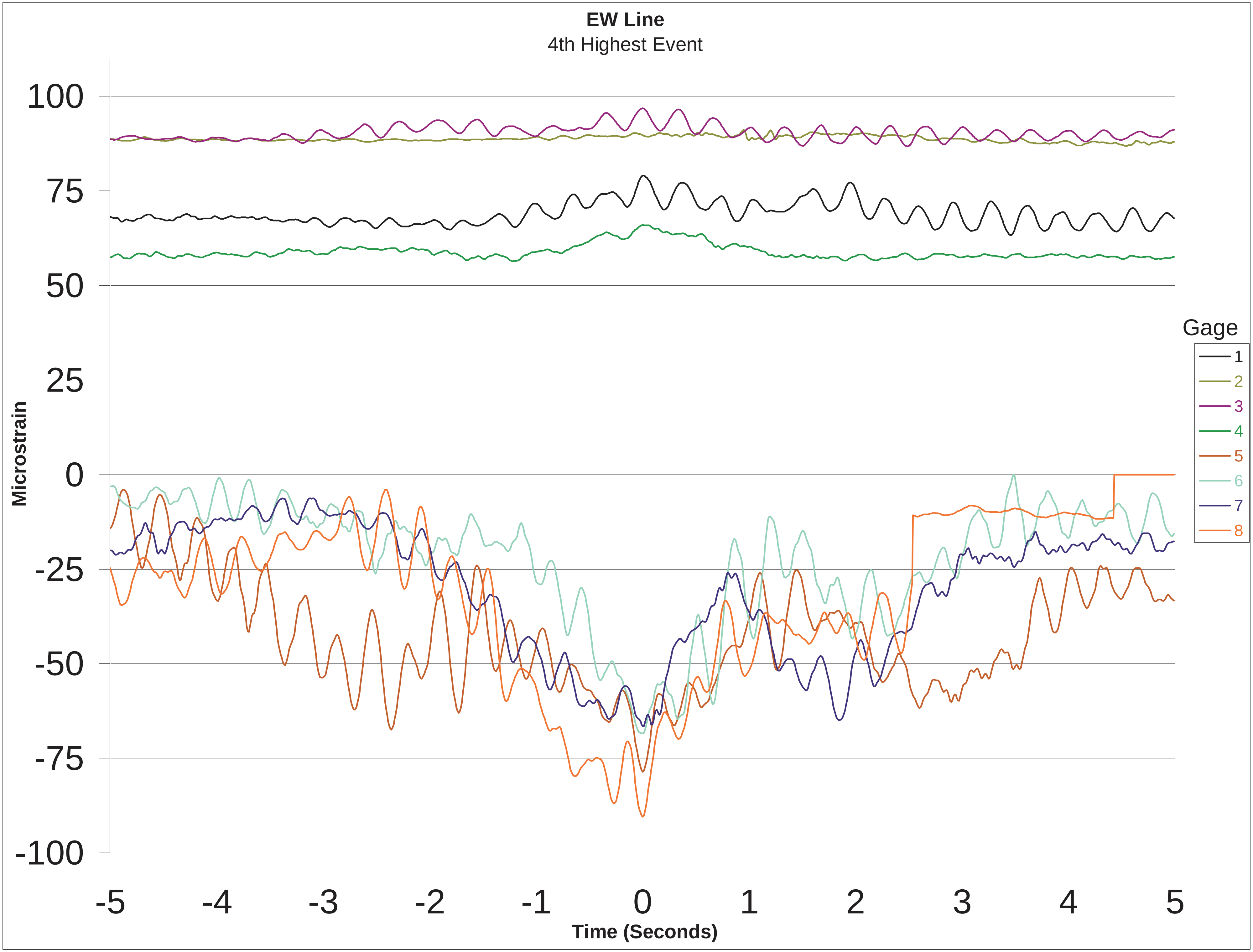
5
6
7
8



EW Line
2nd Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



EW Line
3rd Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


EW Line
4th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
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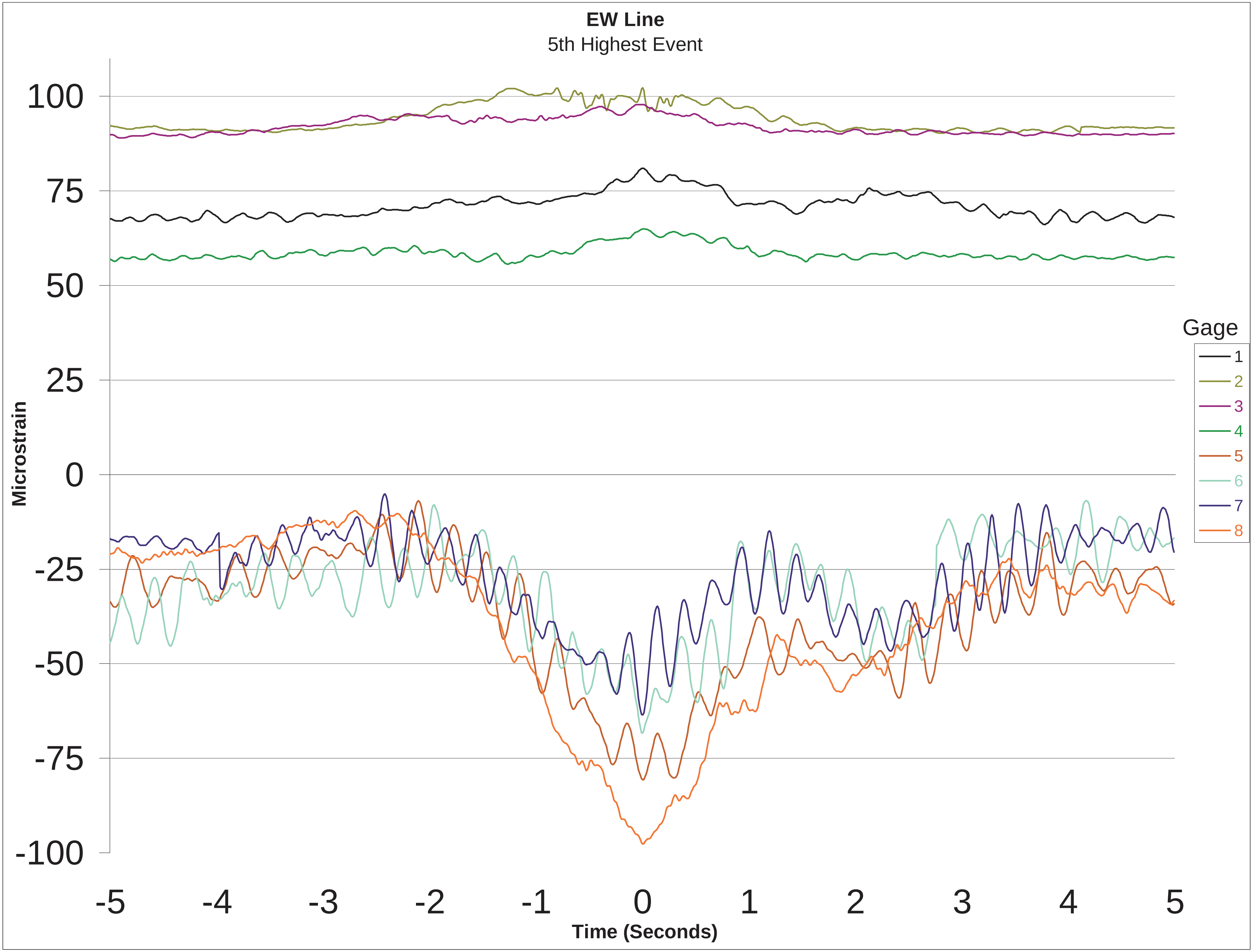
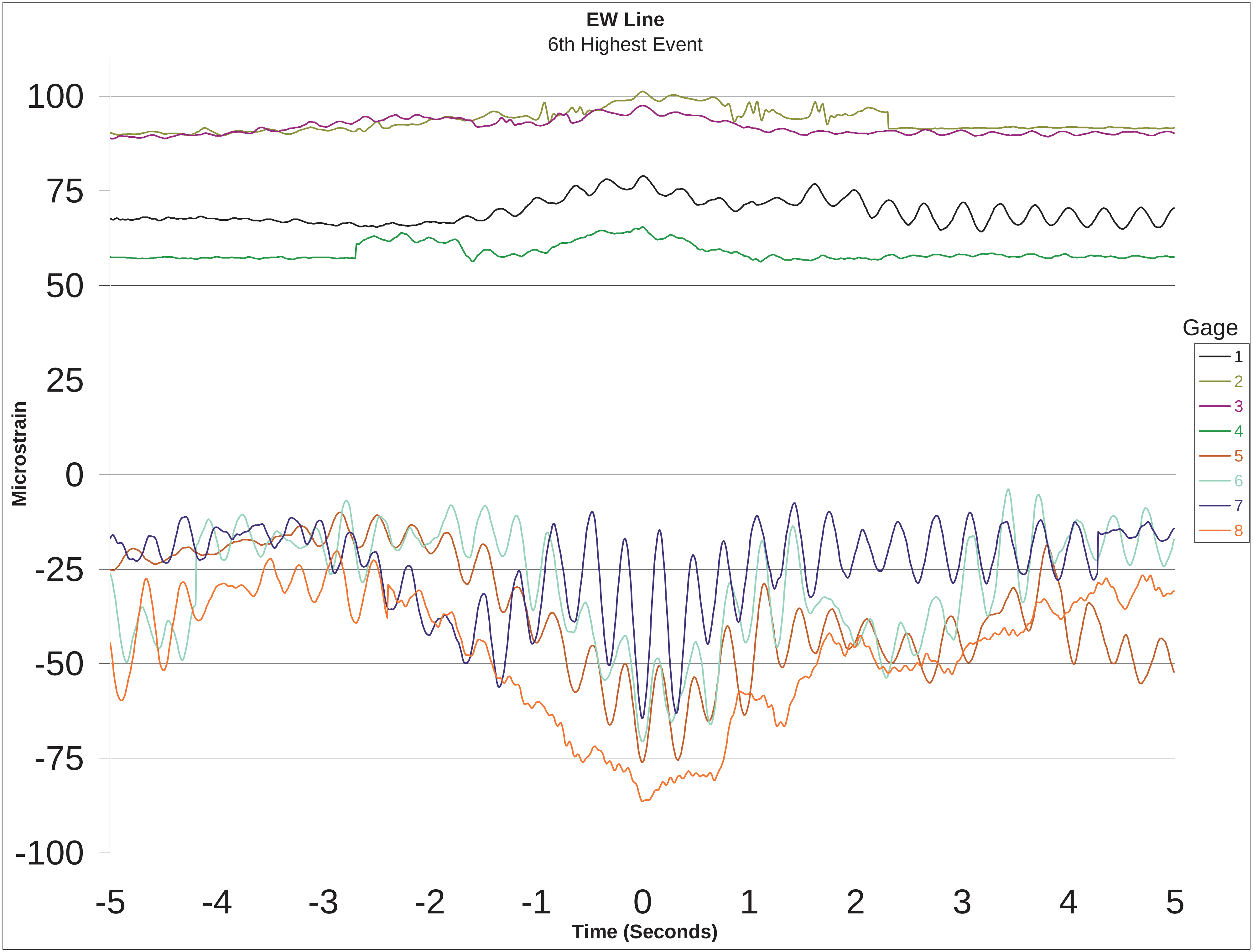
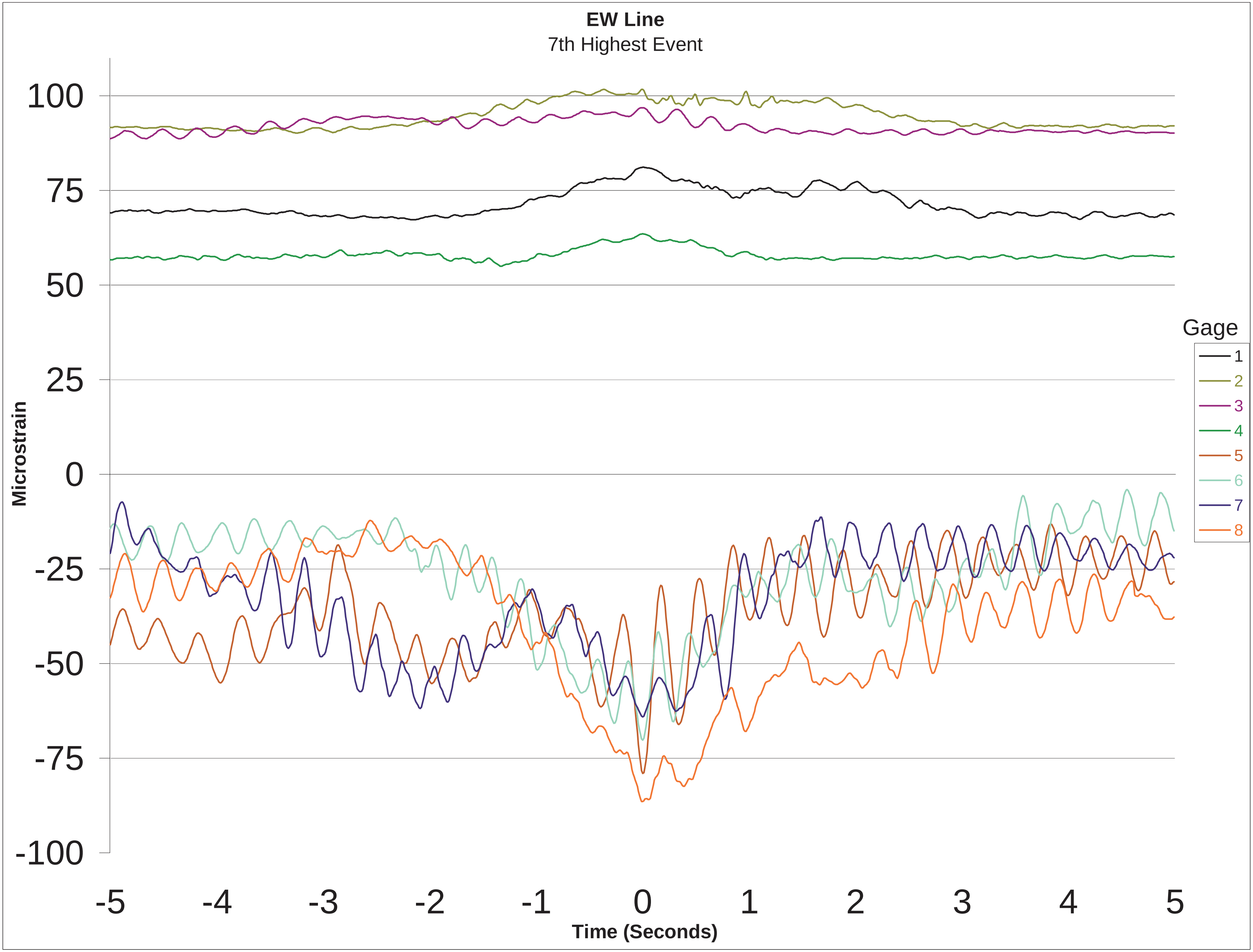
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

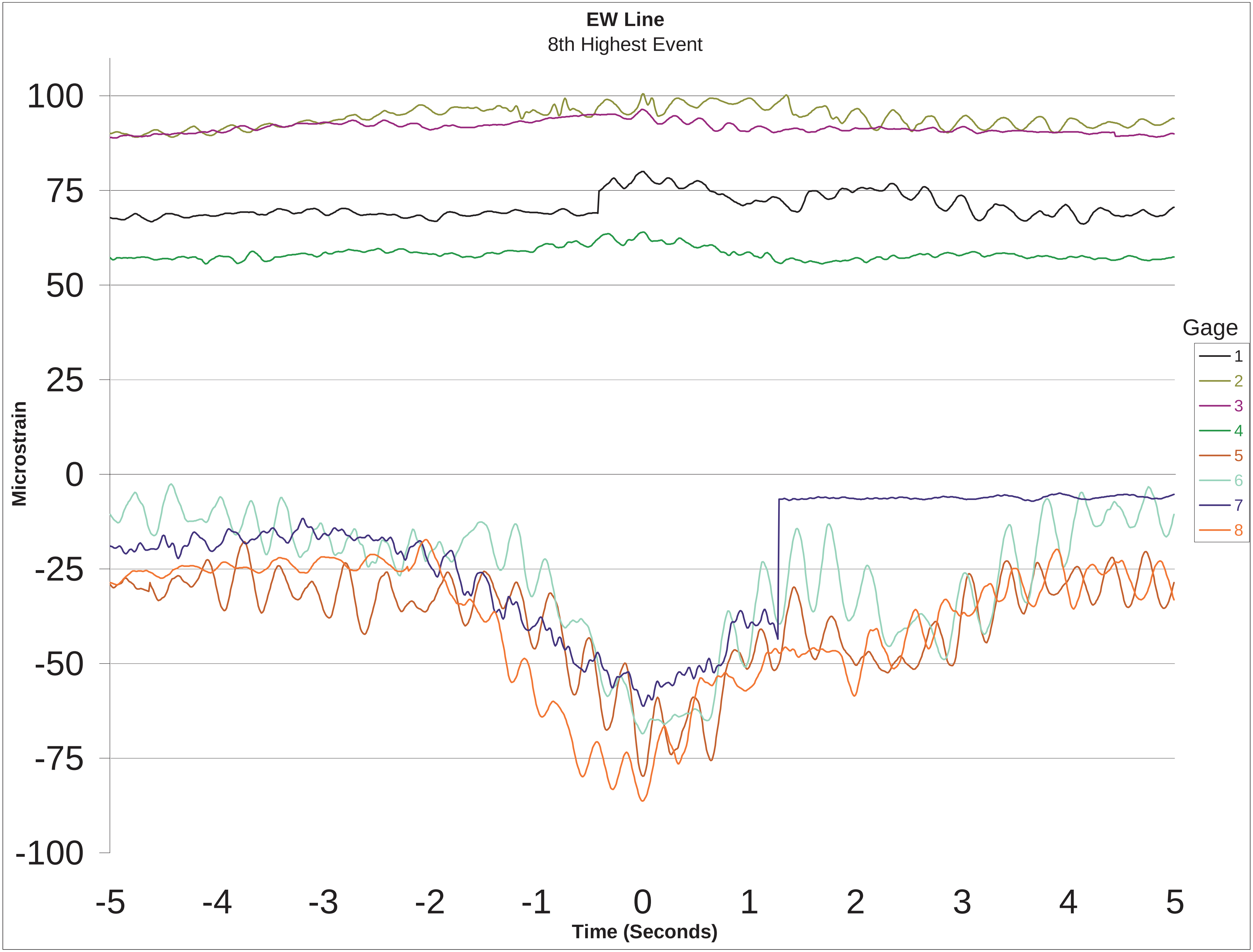


EW Line
5th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

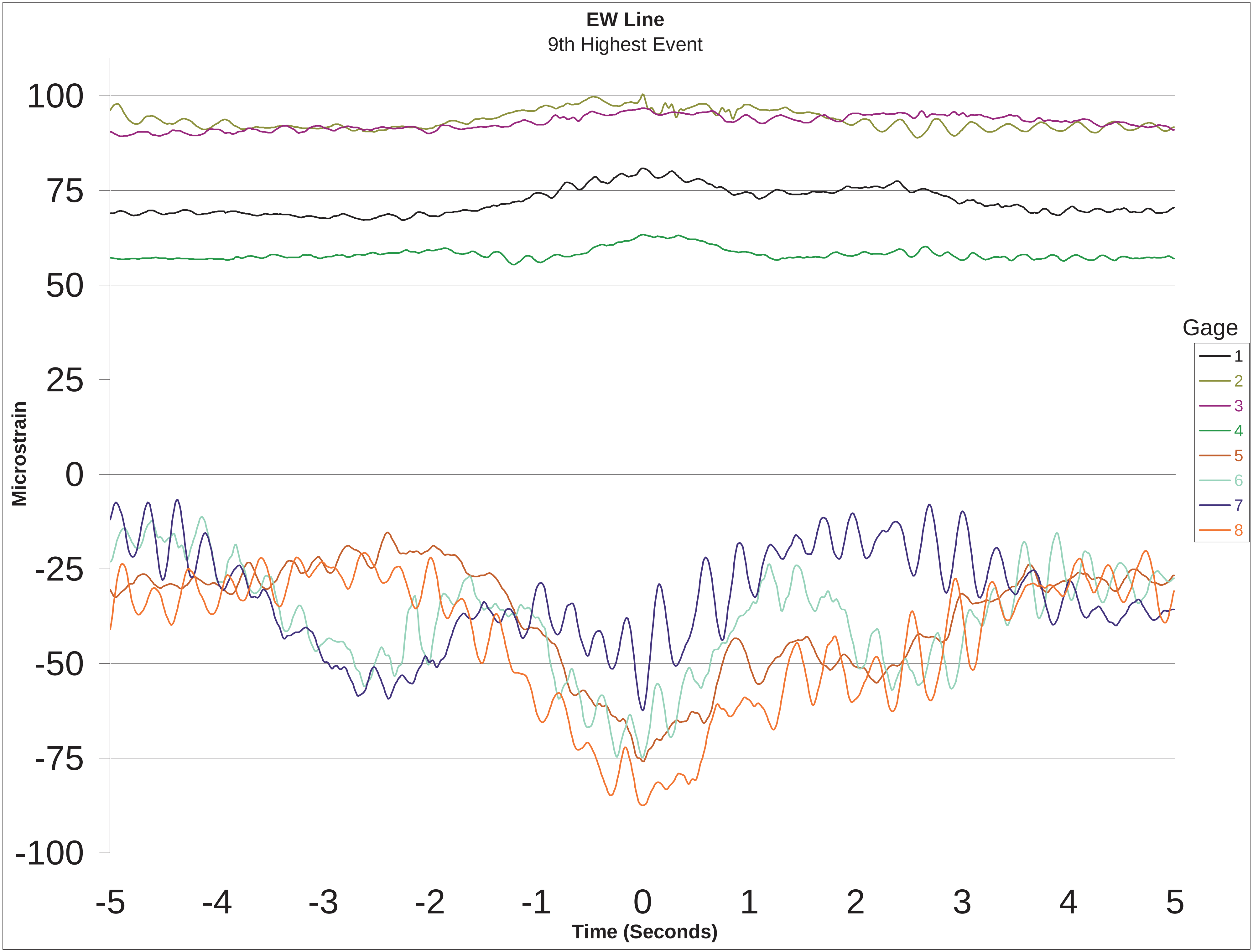


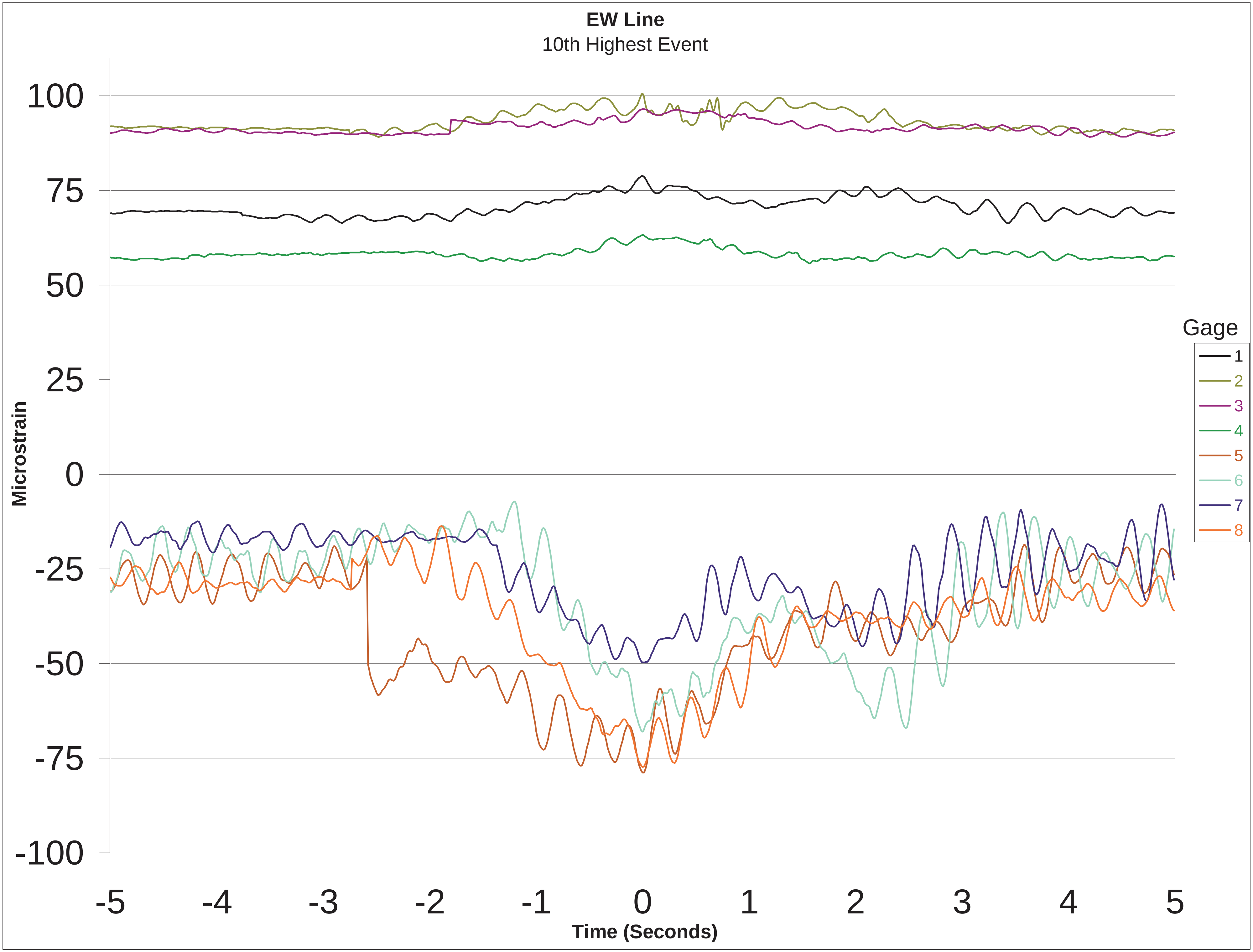
EW Line
6th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


EW Line
7th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8

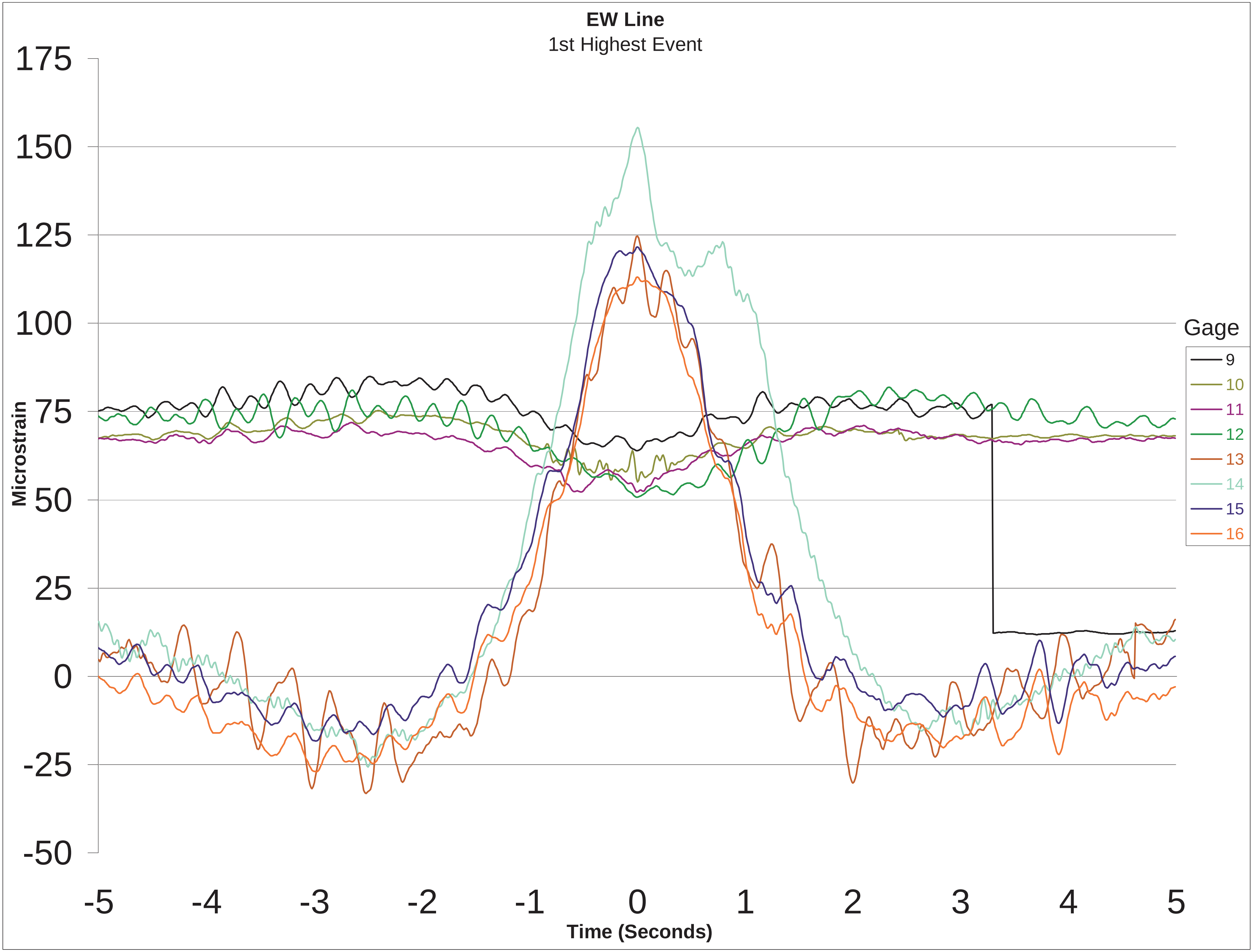


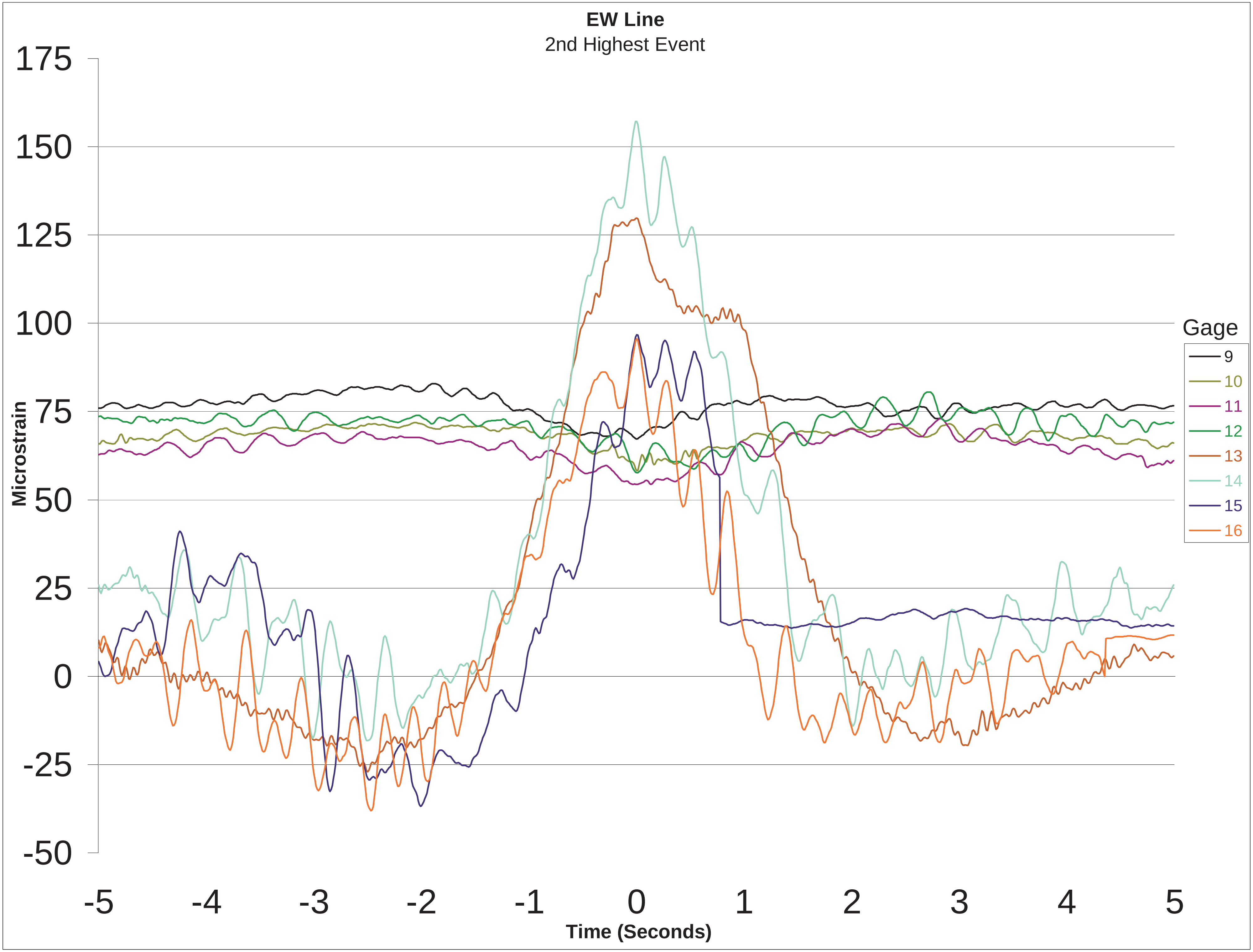
EW Line
8th Highest Event
100
75
50
25
0
-25
-50
-75
-100
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


EW Line
9th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8



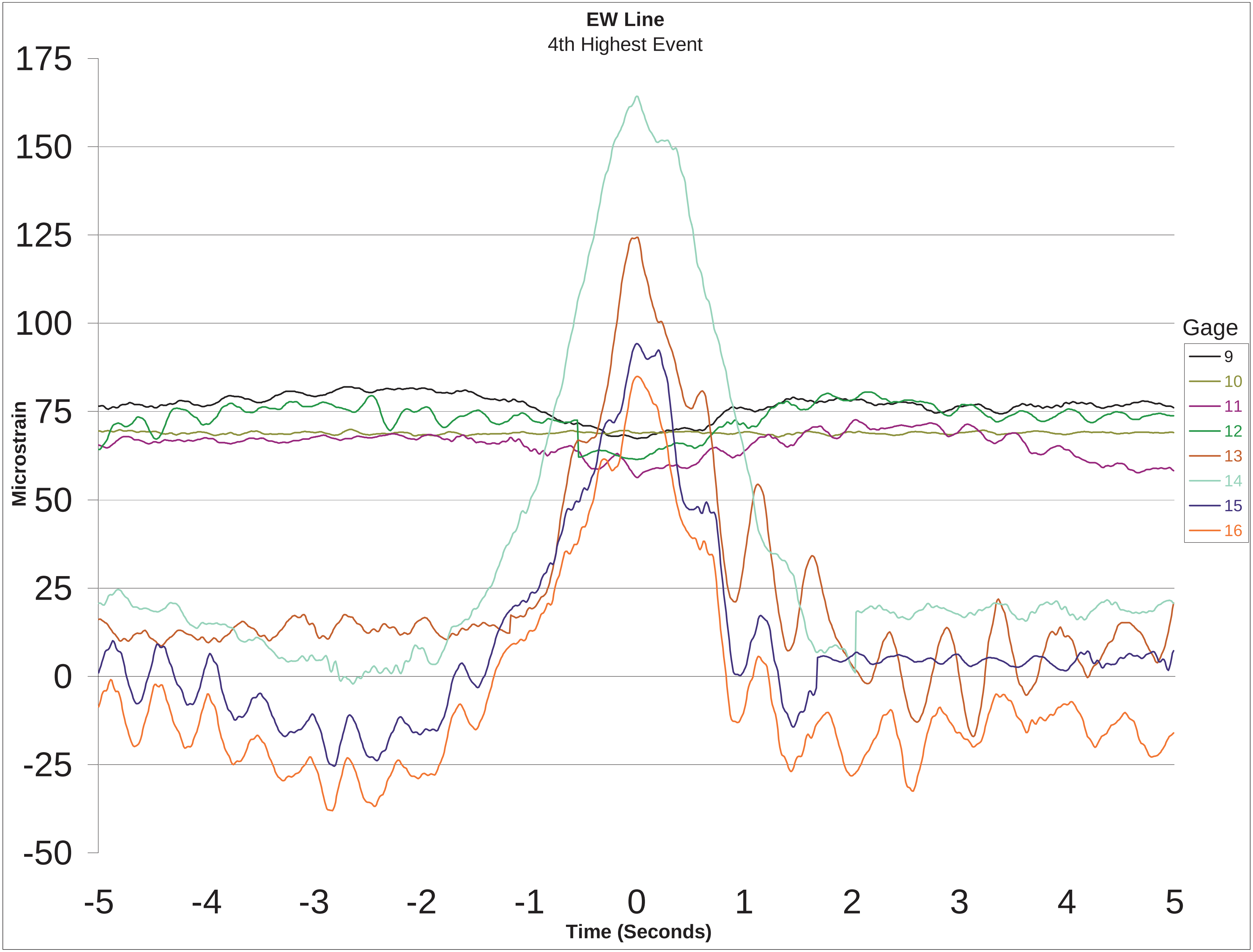
EW Line
10th Highest Event
Microstrain
100
75
50
25
0
-25
-50
-75
-100
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
1
2
3
4
5
6
7
8


EW Line
1st Highest Event
175
150
125
100
75
50
25
0
-25
-50
Microstrain
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

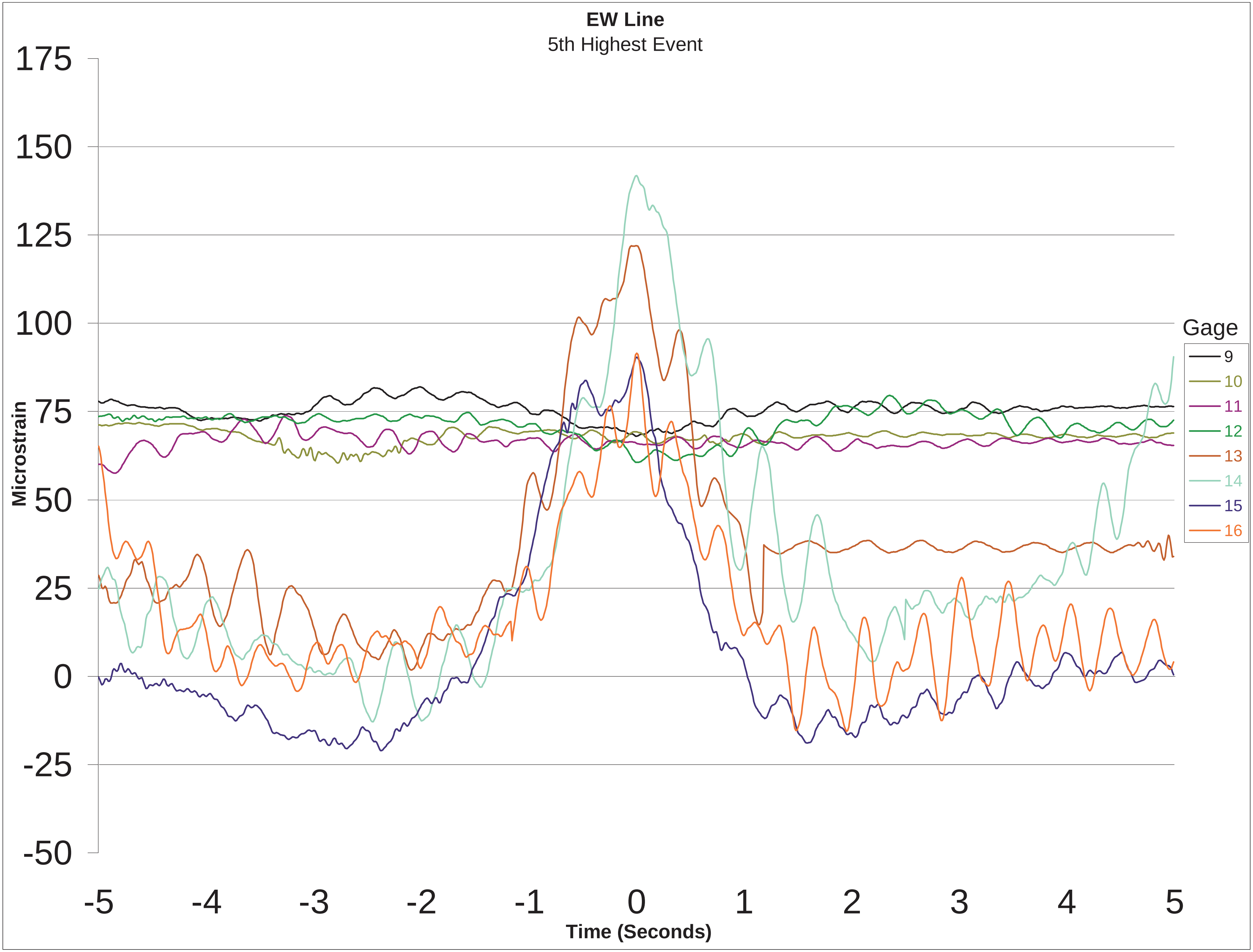


EW Line
2nd Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

EW Line
3rd Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16



EW Line
4th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16



EW Line
5th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
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5
Time (Seconds)
Gage
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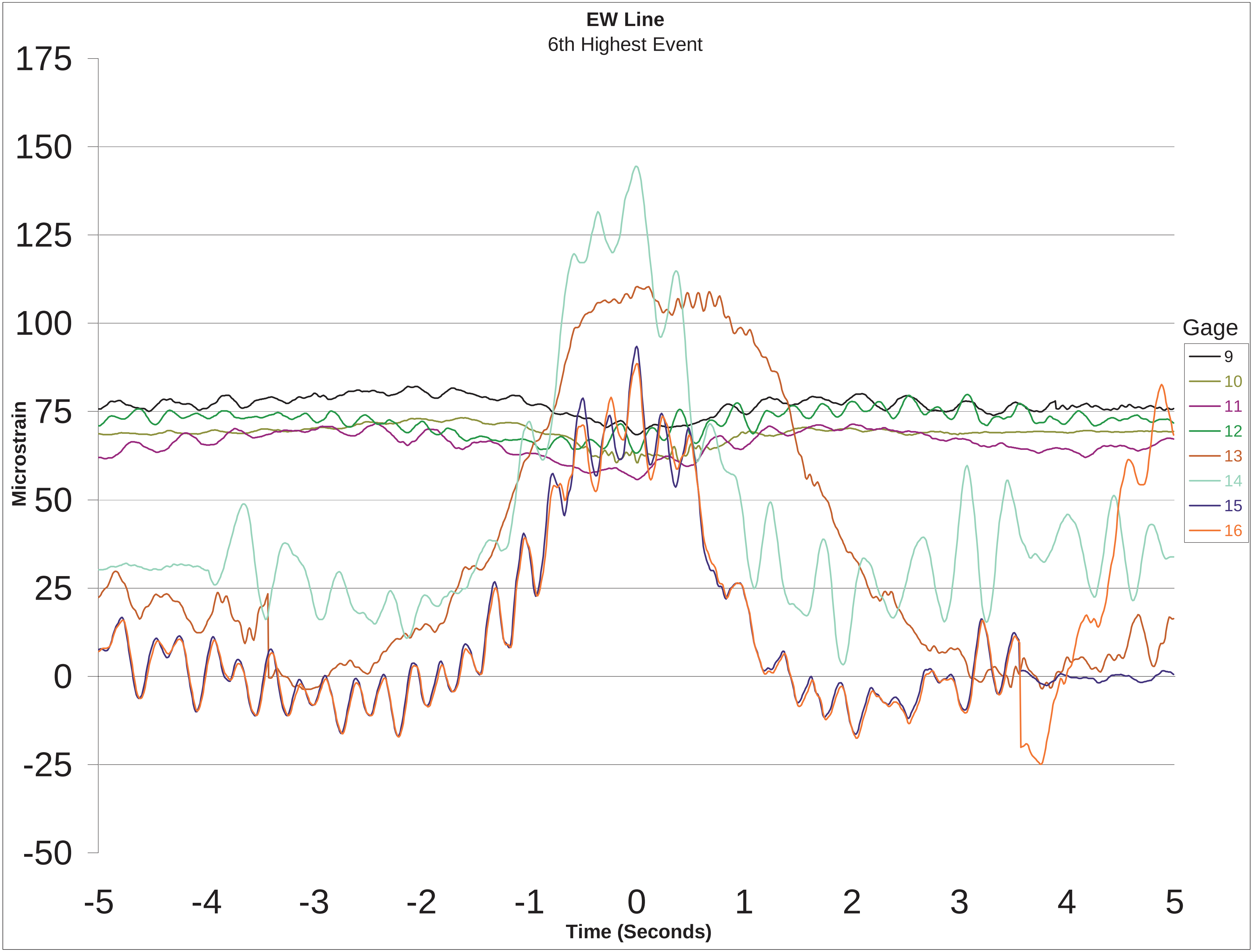

EW Line
6th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
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5
Time (Seconds)
Gage
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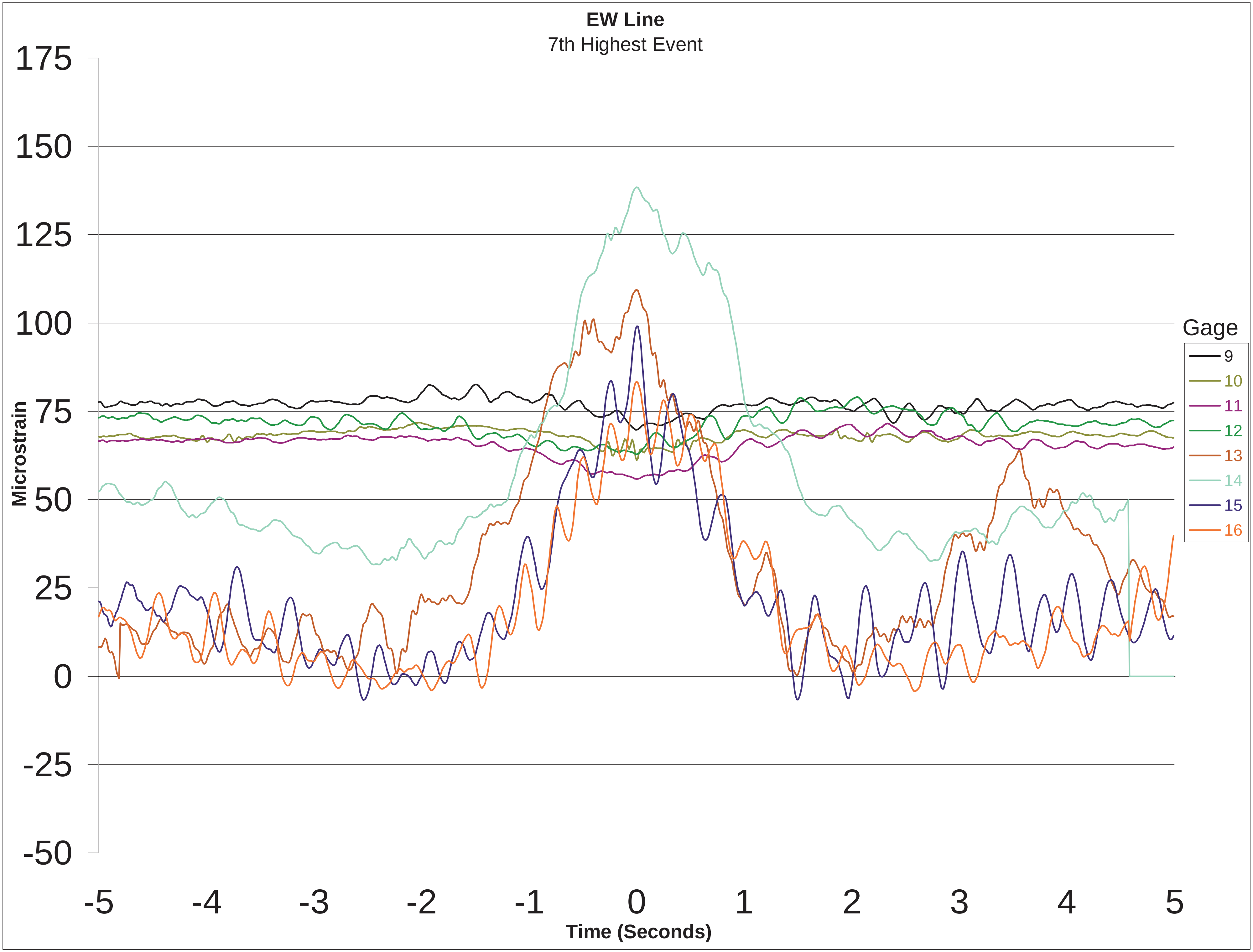
15
16


EW Line
7th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
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4
5
Time (Seconds)
Gage
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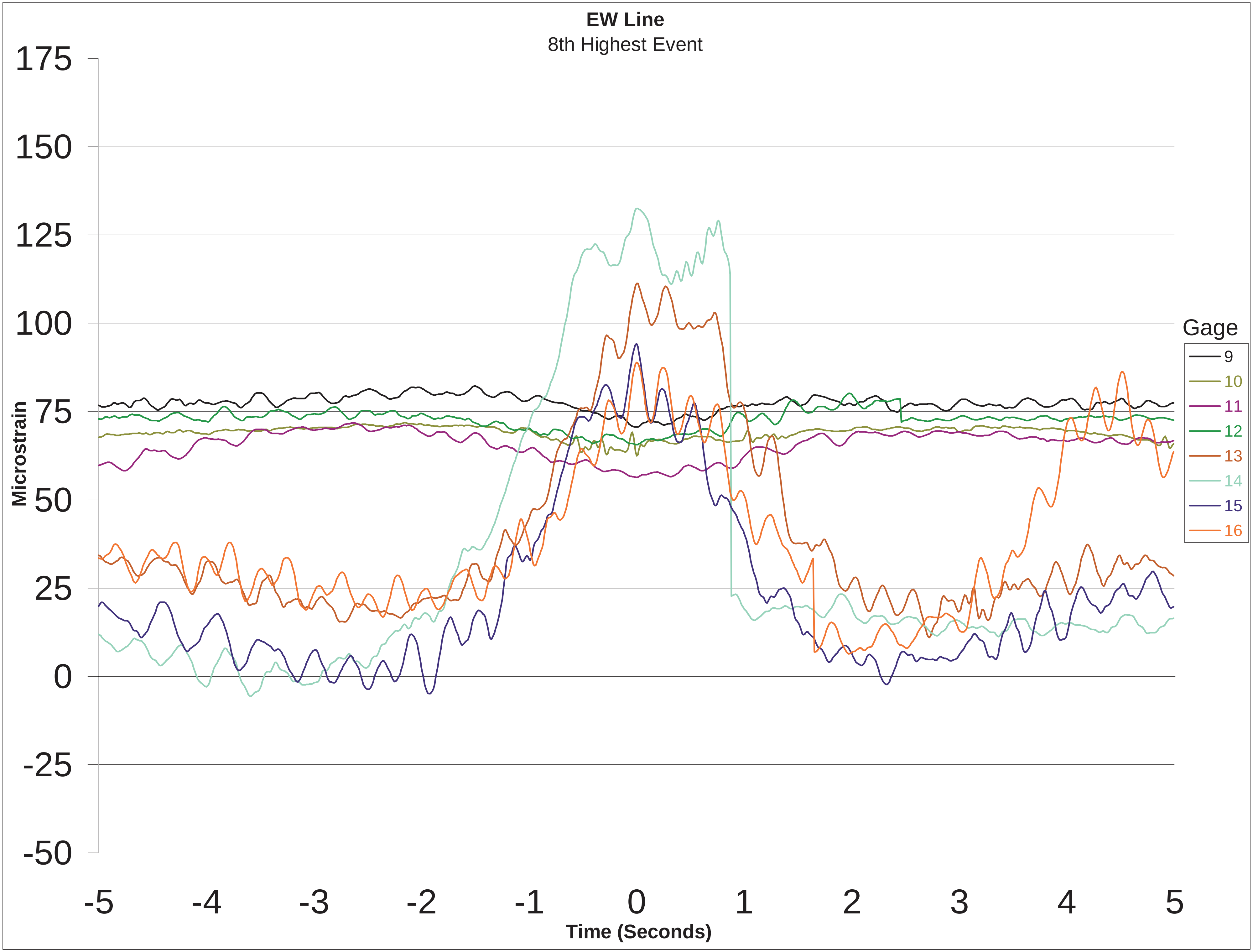
15
16



EW Line
8th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
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3
4
5
Time (Seconds)
Gage
9
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12
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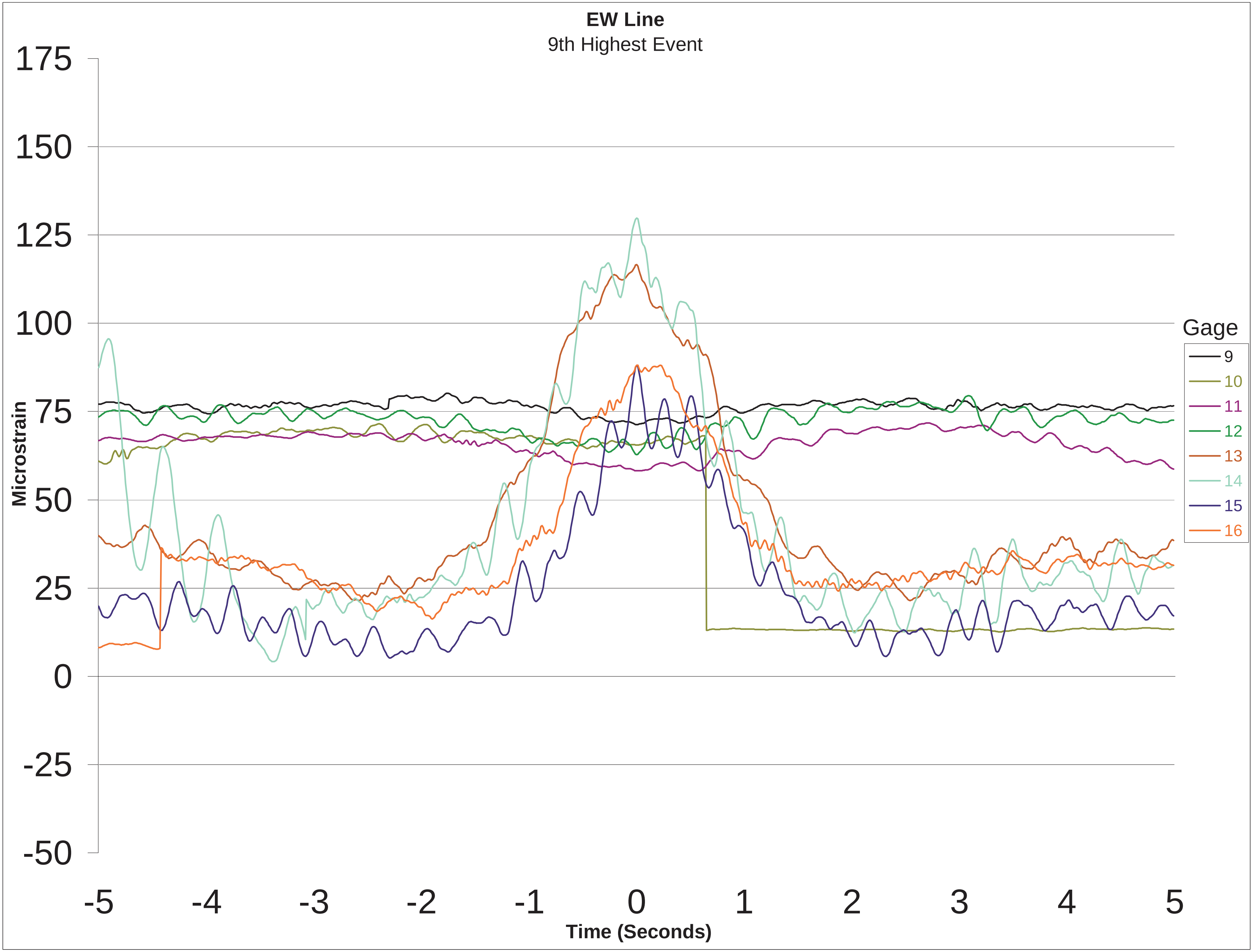
14
15
16



EW Line
9th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
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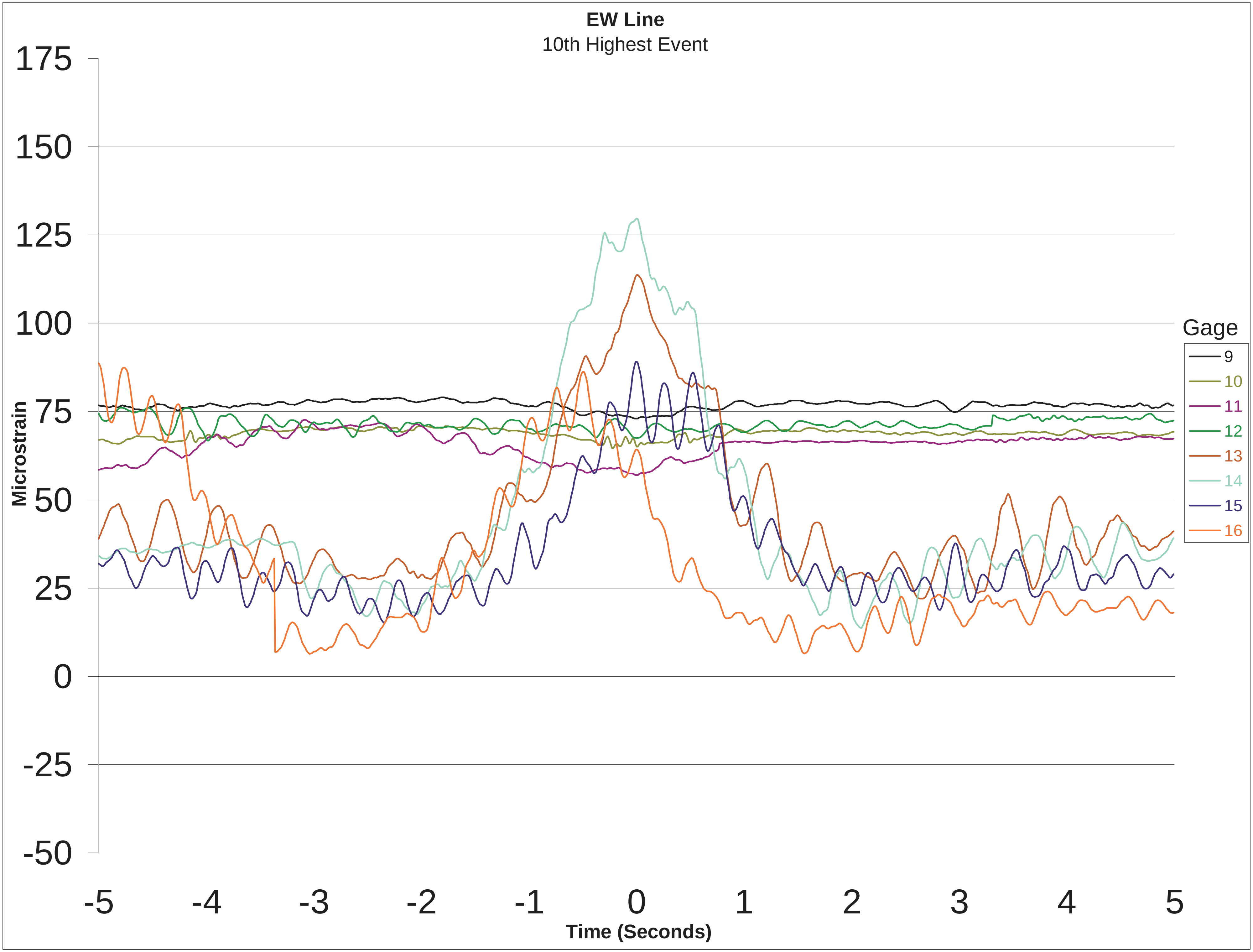
14
15
16



EW Line
10th Highest Event
Microstrain
175
150
125
100
75
50
25
0
-25
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Time (Seconds)
Gage
9
10
11
12
13
14
15
16

TASK 3
CORE REMOVAL FROM
BOTTOM FLANGE PLATES
Phases 1 and 2

Scope of Work

Task 3 of WOC 5 was to remove material testing cores at Agency-specified locations from the bottom flange plates by core drilling through the full thickness of the plates. The task was divided into two phases. Phase 1 specified the removal of five cores for Phase 1 Material Testing. These cores were used for Charpy V-Notch impact bars, which were machined and tested prior to the next phase of core removal. Phase 2 specified the removal of ten additional cores for Phase 2 Material Testing. These cores were used for fracture toughness and fatigue crack growth threshold testing. Per Agency instructions, only seven of the ten authorized Phase 2 cores were removed..

Core Removal Locations

The cores were removed from the girders near piers, where the bottom flange plates are thickest and are loaded in compression. The locations of the cores were selected from the list compiled during the Weld Identification survey conducted by MEIC engineers as Task 1 of the work order.

The cores were removed using 4-inch diameter, bi-metal hole saws and a portable, magnetic base drill by core drilling from the inside of the girders.

Because access and traffic control is more difficult on the “double deck” spans of Bridges 9268E (WE Line) and 9268W (EW Line) than it is on Bridges 9268 A (SW Line) and 9268B (WS Line), all 12 cores were taken from Bridges 9268A and 9268B. Table 12 lists the cores removed from the bridges. All cores (except the base metal core) were removed from weld joints between plates of different thicknesses.

Table 12. Core Removal Locations

Core No.	Weld No.	Position
Phase 1		
1	SW2-A-B-4	Weld
2	SW3-A-B-1	Weld
3	SW3-B-B-1	HAZ
4	SW4-A-B-3	HAZ
5	SW5-A-B-1	HAZ
Phase 2		
6	WS4-A-B-2	HAZ
7	WS4-A-B-1	Weld
8	WS3-A-B-5	Weld
9	WS3-A-B-4	HAZ
10	(WS1-A-B-)*	Base Metal
11	WS1-A-B-5	HAZ
12	WS1-A-B-4	Weld

*Base metal core removed from WS Line, girder A, bottom flange, mid-width of girder, 2 feet from core No. 11, toward core No. 12

Weld Inspection

Prior to removing the cores, the welds were inspected by ultrasonic testing. The inspection during this phase of the WOC was limited to the weld in just the center portion of the girder. (A more detailed inspection of other welds was subsequently performed as Task 5 of the WOC.)

The inspection was preformed under subcontract by NDE Professionals, Inc., by Mr. Bob Hosman and Mr. Lee Garrison, both ASNT Level III Certified UT inspectors. The inspection was done in accordance with the American Welding Society (AWS) Bridge Welding Code, AWS D1.5, except that, per Agency directive, the paint was not removed[1].

The inspection was by straight beam ultrasonic scanning, using a 1-inch diameter, 2.25 MHz transducer, followed by shear wave (angle beam) scanning using both a 70° and a 45°, 2.25 MHz transducer. The test instrument was a Panametrics Epoch™ 3 flaw detector. Prior to inspection, the surface of the inspection area was cleaned by wire brushing and/or scraping with a putty knife, followed by solvent wiping, to remove surface deposits and/or any protrusions in the paint that might otherwise prevent good contact between the transducer and the surface.

[1]AWS D1.5 requires the paint to be removed for code inspections. However, because of the extent of grinding and repainting that would have been required, ODOT directed that the paint not be removed. To compensate for the presence of the paint, the scanning level gain was increased by an amount determined by taking a pitch-catch angle beam reading in an area with paint and another reading next to that, where the paint was removed in two small spots. The amount of gain required to bring the UT signal in the painted area up to the same level as the signal in the bare area was then added to the instrument for the angle beam inspection of the welds through the paint.

Figure 17 (71149-02) Ultrasonic inspection of weld prior to removing core

The ultrasonic inspection revealed a few minor, non-reportable indications but no significant flaws. In two instances (cores Nos. 8 and 9), the indications were present along a considerable length of the weld, but because they were non-reportable in magnitude they were judged insignificant, and the cores were taken at mid-width of the girder. In a third instance (core 6), where a minor indication was encountered at just one location, the core location was shifted off to one side to avoid the area with the indication. Note that even at this location, the indication was judged insignificant, but because it was localized to just one spot, and the core could be easily shifted away from that spot, the spot with the indication was avoided.

The cause for the non-reportable indications was not conclusively established; however, subsequent polishing and etching of core Nos. 8 and 9 revealed the presence of repair welds at the outside surface, along the toe of the electroslag weld, as shown in Figure 60. Wet fluorescent magnetic particle testing of the cores did not reveal any cracks or other defects, either on the outside surface of the weld or on the cross-sectional surface cut by the hole saw; thus, the we suspect the UT signals may have been due to metallurgical variations (such as a difference in grain structure) between the repair weld and the electroslag weld/base metal.

The results of the ultrasonic inspection are in Table 13.

Table 13. UT Inspection Results

Core No.	Weld No.	Thickness, Inch	Results
Phase 1			
1	SW2-A-B-4	1.55/2.05	No reportable indications.
2	SW3-A-B-1	2.05/1.52	No reportable indications.
3	SW3-B-B-1	1.80/1.50	No reportable indications.
4	SW4-A-B-3	1.45/1.85	No reportable indications.
5	SW5-A-B-1	1.83/1.43	No reportable indications.
Phase 2			
6	WS4-A-B-2	1.55/2.08	No reportable indications; slight signal at mid-width of girder; core removed 19" from web to avoid indication.
7	WS4-A-B-1	2.08/2.47	No indications; core removed mid-width of girder.
8	WS3-A-B-5	2.08/2.47	Slight UT signal all along weld toe at outside surface with 45° transducer–interpreted to be non-relevant; core removed from mid-width of girder.
9	WS3-A-B-4	1.55/2.10	Similar signal as at core No. 8 but not as strong; core removed from mid-width of girder.
10	Base Metal*	1.94	No indications; core removed from mid-width of girder.
11	WS1-A-B-5	1.98/2.00	No indications; core removed from mid-width of girder.
12	WS1-A-B-4	1.60/2.00	No indications; core removed from mid-width of girder.

*Base metal coupon removed from WS Line, mid-width of girder,
2 feet from core No. 11, toward core No. 12

Weld Etching

After each weld had been inspected and the general location for the core selected, a strip of paint was removed by grinding, and the surface was sanded and etched with 10 percent Nital (10 percent nitric acid/90 percent methanol) to reveal the precise location of the weld and heat-affected zones.

The etching was done under subcontract by Mr. Bob Turpin of Portland State University, who has considerable experience in the "barreling" profiles exhibited by electroslag welds and was thus retained by MEIC to provide consulting advise on where best to center the core drilling equipment so as to maximize the useable volume of the core. Figure 18 shows the weld being etched by Mr. Turpin, Figure 19 shows the weld after etching, and Figure 20 shows the weld after center-punch marking the precise location where the core drill was to be centered.

Figure 18 (71149-05) Weld being etched

Figure 19 (71149-07) Etched weld (10% Nital)

Figure 20 (71149-08) Punch marked weld

Core Removal Procedure

Figures 21 and 22 show a core being removed from one of the bottom flange welds (core No. 6, weld No. WS4-A-B-2). The cores were removed with 4-inch diameter bi-metal holes saws, using a Milwaukee Model 4297-1/4203, magnetic-base portable drill[2].

Because the thickness of the girder was greater than the cutting depth of a standard hole saw, special cutters were fabricated by welding extension tubes onto standard hole saws, thereby increasing the depth that could be cut.

The photograph shows a standard hole saw in use. To reduce the number of modified hole saws needed for the project, standard hole saws were used until the cut reached the maximum cutting depth of the standard saw (1.4 inches), at which point a hole saw with an extension tube was installed on the drill to complete the cut.

The cores were cut without cutting fluid, because it was found that the presence of cutting fluid made it very difficult to clear the chips from the saw kerf after the cutting depth exceeded about 1/4 inch. That is, once the hole saw reached a depth of about 1/4 inch, the fluid-soaked chips matted together in the bottom of the cut and the hole saw would cease cutting. A laborious process of clearing the chips from the kerf was then necessary before cutting could be resumed. When cuts were made using cutting fluid, several hours were required to remove a single core. By making the cut without fluid, MEIC's engineers were able to blow the chips out of the kerf using a portable air compressor and a needle-shaped nozzle (bottom photograph), and a core could be removed in about 45 minutes.

[2] Initially, it was planned to use a milling cutter to remove the cores, similar to what ODOT had used on the West Linn Bridge. MEIC engineers tested electric, portable drill presses from three manufacturers but found that none had sufficient power to drive the cutter, despite claims to the contrary by the drill/cutter manufacturers. (ODOT had used a large, custom-built, 27-HP hydraulic unit to drive the cutter during the previous sampling.) Subsequently, tests showed that a standard hole saw was capable of making the cut, so the original plan of using a milling cutter was abandoned. An advantage to using hole saws was cost. That is, the hole saws cost less than $20 each, and even with the cost of adding an extension tube, the cost was still less than $75, whereas the milling cutters varied in price from $700 to $1,300 each and reportedly would require sharpening after 4 to 5 cuts, at a cost of $400.

Figure 21 (71149-15) Core drilling equipment

Figure 22 (71149-13) ESW weld during core drilling

Core Plugs

Figures 23 and 24 show one of the girders after removing the core sample. In accordance with the work order contract, machined steel hole plugs were fabricated by MEIC engineers and subsequently installed in the holes. The hole plugs were fabricated in accordance with Agency drawing No. 33043, modified[3] as shown in Figure 25.

The hole plugs consisted of an outer, hollow, split sleeve with a cylindrical profile on the outside and a conical tapered profile on the inside. A mating, tapered plug, forced into the split sleeve with a bolt and nut, causes the split sleeve to expand tightly into the hole. The cores were installed by MEIC engineers using a torque wrench from inside the girder, while the applied torque was being resisted from outside by an MEIC engineer in a boom lift using a wrench. Per Agency instructions, the plugs were installed to a torque of about 350 ft-lbs.

Prior to installing the plugs, the plugs and the inside of the holes were painted with "Zero-Rust" paint, as was the area surrounding the holes, where the paint had been ground off to provide a more secure surface for the magnetic base drill[4].

[3]Agency drawing No. 33043, Electroslag Weld Sample Hole Plug, was for the plugs used on the Willamette River (West Linn) Bridge, where the flanges were substantially thicker and the cores were of a different size than on the Fremont Bridge west side approaches. The drawing was modified by MEIC engineers (with Agency approval) to provide for a shorter plug, sized for the hole diameter of the cores being removed from the Fremont Bridge approaches.

[4]The Agency reported that "Zero-Rust" was the paint used by the Bridge Crews to repair breaks in the paint. According to the manufacture's website, Zero-Rust is a direct metal, phenolic modified alkyd rust and corrosion control coating.

Figure 23
(71149-19)
Core plug ready to be installed

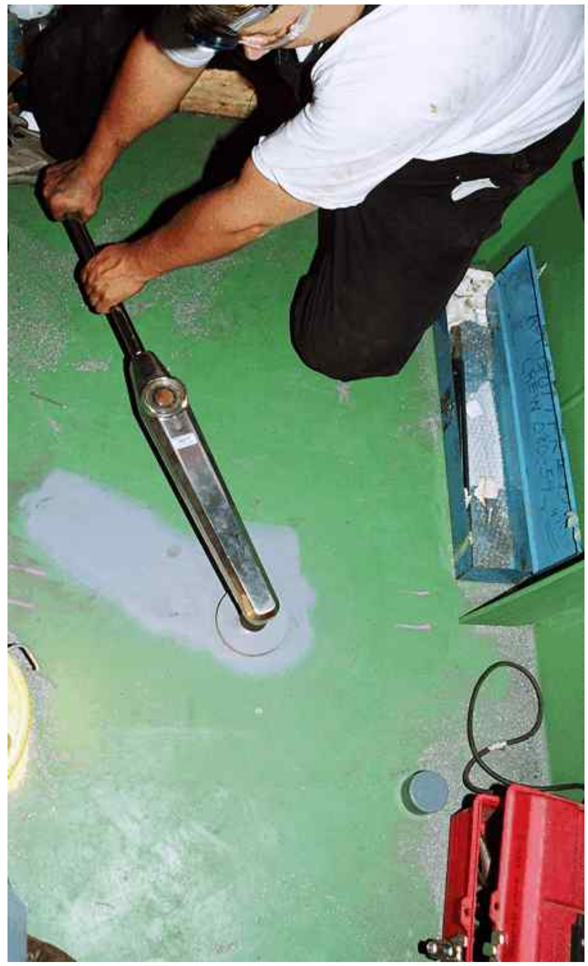
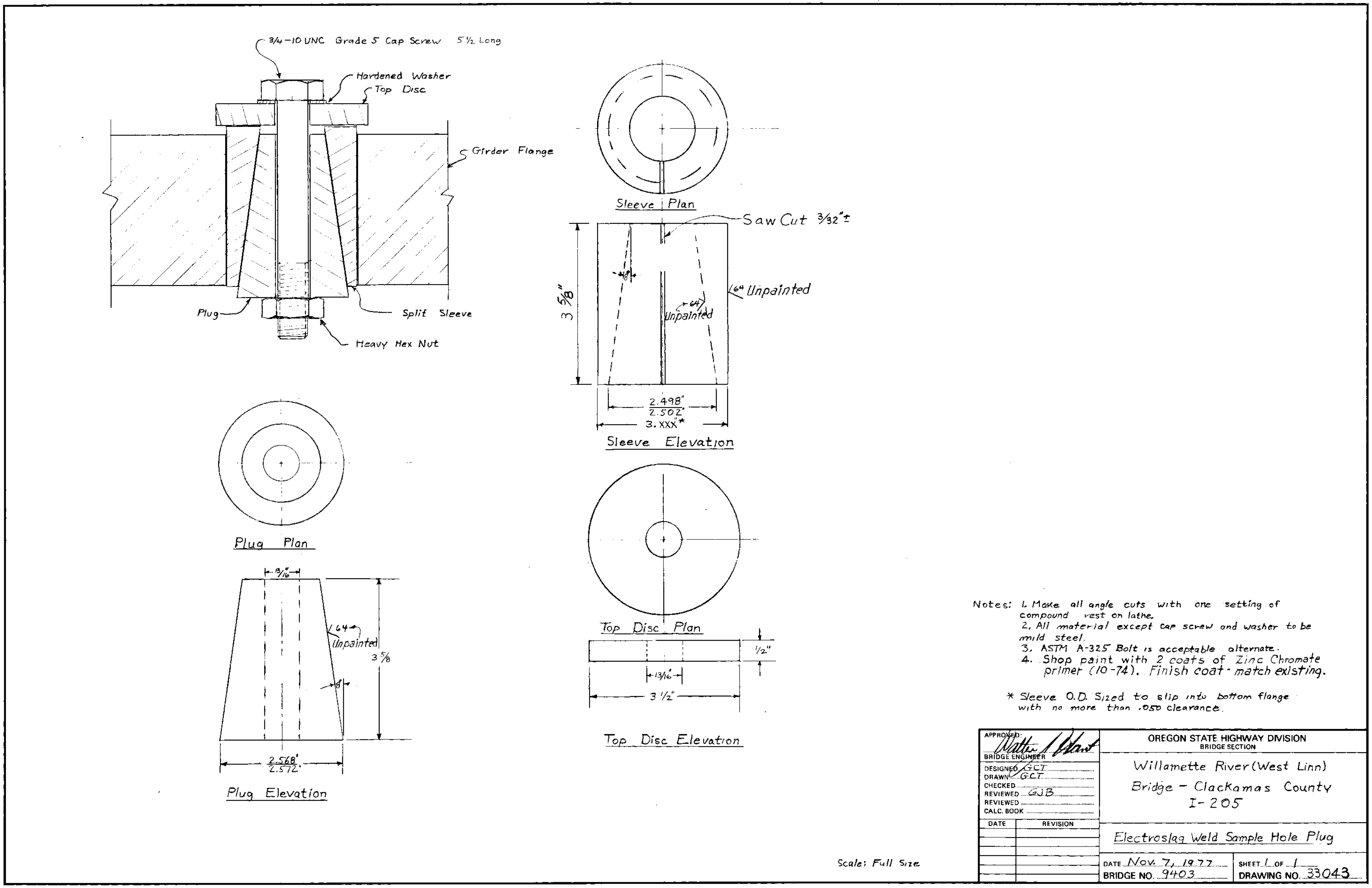


Figure 24
(71149-20)
Core plug being torqued to 350 ft-lbs


3/4–10 UNC Grade 5 Cap Screw 5 1/2 Long
Hardened Washer
Top Disc
Girder Flange
Plug
Split Sleeve
Heavy Hex Nut
Sleeve Plan
Saw Cut 3/32"±
64 Unpainted
Unpainted
3 5/8"
2.498" / 2.502"
3.XXX*
Sleeve Elevation
Plug Plan
9/16"
64 Unpainted
3 5/8
2.568" / 2.572"
Plug Elevation
Top Disc Plan
1/2"
13/16"
3 1/2"
Top Disc Elevation
Notes: 1. Make all angle cuts with one setting of compound rest on lathe.
2. All material except cap screw and washer to be mild steel.
3. ASTM A-325 Bolt is acceptable alternate.
4. Shop paint with 2 coats of Zinc Chromate primer (10-74). Finish coat - match existing.
* Sleeve O.D. Sized to slip into bottom flange with no more than .050 clearance.
Scale: Full Size
APPROVED:
BRIDGE ENGINEER
DESIGNED GCT
DRAWN GCT
CHECKED
REVIEWED GJB
REVIEWED
CALC. BOOK
DATE
REVISION
OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION
Willamette River (West Linn) Bridge - Clackamas County
I-205
Electroslag Weld Sample Hole Plug
DATE Nov. 7, 1977
SHEET 1 OF 1
BRIDGE NO. 9403
DRAWING NO. 33043

TASK 4
MATERIAL TESTING
Phases 1 and 2

Scope of Work

Task 4 of WOC 5 was to perform destructive material testing of the bridge material on the core samples removed from the bridge during Task 3. Task 4 was divided into two phases, 1 and 2. The purpose of Phase 1 testing was to qualitatively determine the fracture toughness of the weld and heat-affected zone (HAZ) metals using Charpy V-Notch (CVN) testing in accordance with ASTM E23-04, Standard Test Methods for Notch Bar Impact Testing of Metallic Materials. The purpose of Phase 2 testing was to quantify the in service fracture toughness and fatigue crack growth threshold of the weld and HAZ metals.

Phase 1 Material Testing

The following tasks were identified in the scope of work for the Phase 1 testing:

- Macro etch the cores per ASTM E340-00e1 and identify the location/profile of the base metal, weld, and HAZ.
- Micro polish and etch the samples and metallurgically characterize the base metal, weld metal, and HAZ.
- Conduct Charpy V-notch testing of the weld and HAZ per ASTM E23-04 over a range of temperatures to produce Impact Strength vs Temperature transition curves which identify the upper and lower transition temperatures.
- Perform chemical analysis of the base metal and weld metal and evaluate conformance with base metal specification.
- Calculate Carbon Equivalent (CE) of base metal and weld metal, using results of chemical analysis.
- Provide recommendations for Phase 2 core extraction and testing.

Weld Profiles

Figures 26 through 31 show the Phase 1cores after macro etching to reveal the location of the weld, HAZ, and base metal. All five Phase 1 cores were taken from welds between plates of different thicknesses. The thickness transition (step) was accommodated on the outside (bottom side) of the girder, with the inside surfaces flush with each other. (The cores are shown in these photographs upside down, with the thickness transition on the top.)

The cores exhibited varying degrees of "barreling," or depth of fusion, a condition that occurs in electroslag welds whereby the location of the fusion zone of the weld metal beneath the surface is not the same as it is at the surface, being either convex (bowing outward) or concave (bowing inward). On the thicker plate side of the weld, the barreling was convex, while on the thinner plate side, it was concave. (The welds were not particularly easy to discern–especially in the photographs–so their locations were marked with a series of dots using a felt tip marker.)

Table 14 shows weld barreling relative to the toe of the weld at the surface.

Table 14. Weld Profiles, Phase 1 Coupons

Core No.	Weld No.	Thicker Plate Side of Weld		Thinner Plate Side of Weld	
		Plate Thickness, Inch	Barreling, Inch	Plate Thickness, Inch	Barreling, Inch
1	SW2-A-B-4	2.05	3/8 to 7/16; convex	1.55	3/8; concave
2	SW3-A-B-1	2.05	1/16 to 1/8; convex	1.52	3/8; concave
3	SW3-B-B-1	1.8	<1/16 to 1/16; convex	1.5	*
4	SW4-A-B-3	1.85	3/16 to 1/4; convex	1.45	*
5	SW5-A-B-1	1.83	5/16; convex	1.43	*

*Core was centered on HAZ of thicker plate; weld and HAZ on thinner plate not present in core coupon.

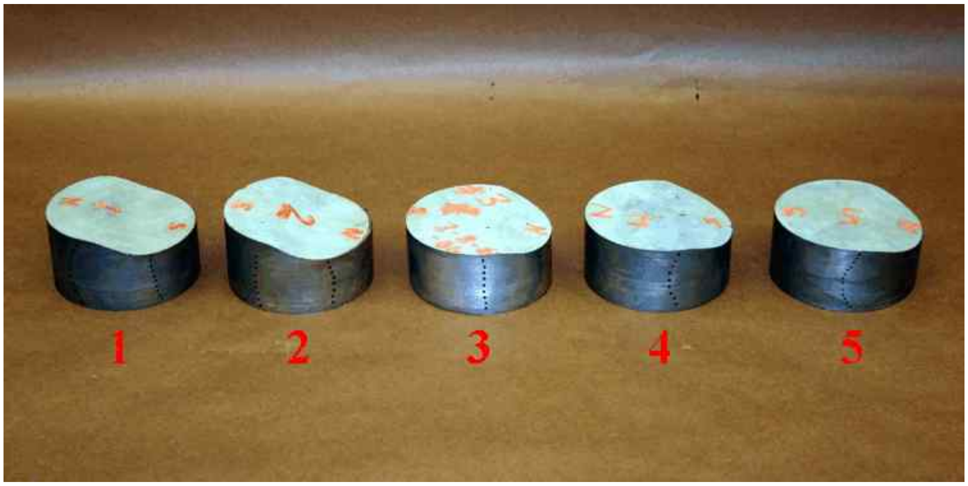


Figure 26 (D71142-01) Phase 1 Cores Nos. 1 through 5

Figure 27 (D71142-05) Core No. 1 (weld metal)

Figure 28 (D71142-06) Core No. 2 (weld metal)

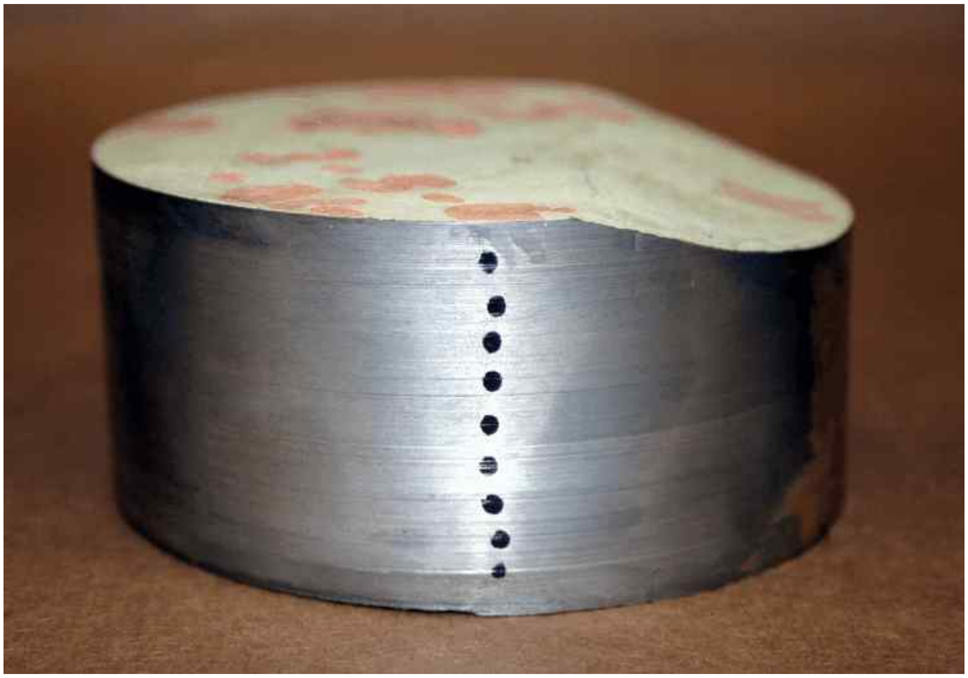


Figure 29 (D71142-07) Core No. 3 (HAZ)

Figure 30 (D71142-08) Core No. 4 (HAZ)

Figure 31 (D71142-09) Core No. 5 (HAZ)

Microstructures

Figures 32 and 33 are micrographs showing the etched microstructure of the base metal and HAZ on the material in core No. 4. The base metal (Figure 32) consisted of equiaxed grains of ferrite and pearlite in a banded structure typical of hot-rolled, low carbon steel plates. The HAZ (Figure 33) exhibited a transition from the banded structure, exhibiting a breakdown of the pearlite due to recrystallization.

Figure 34 shows an area in the HAZ closer to the weld metal.

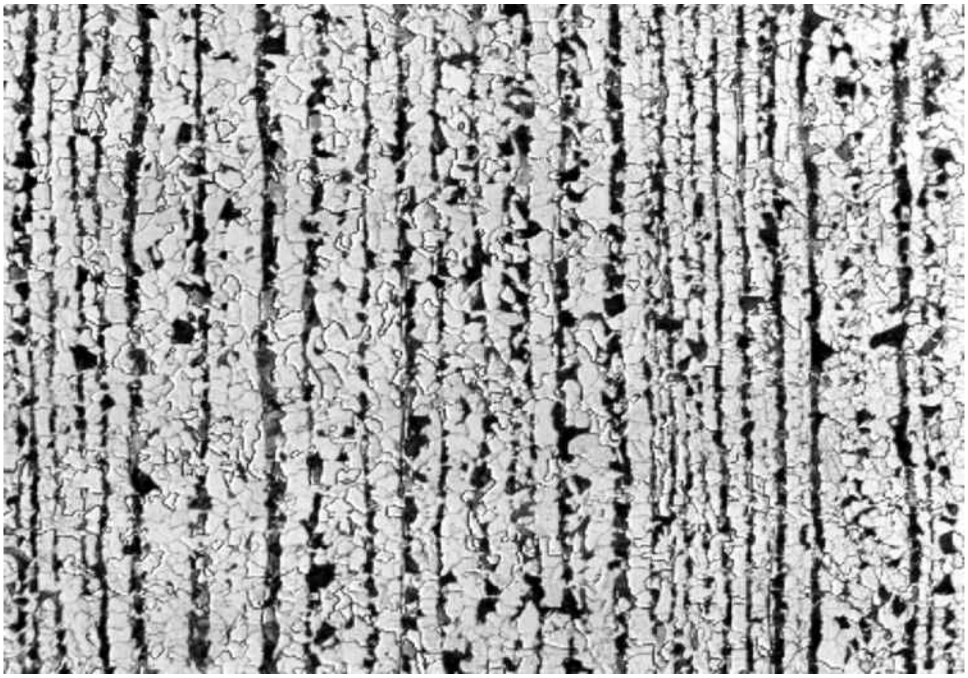


Figure 32 (D34467) 100X Core No. 4, Charpy Bar No. 9, Base Metal

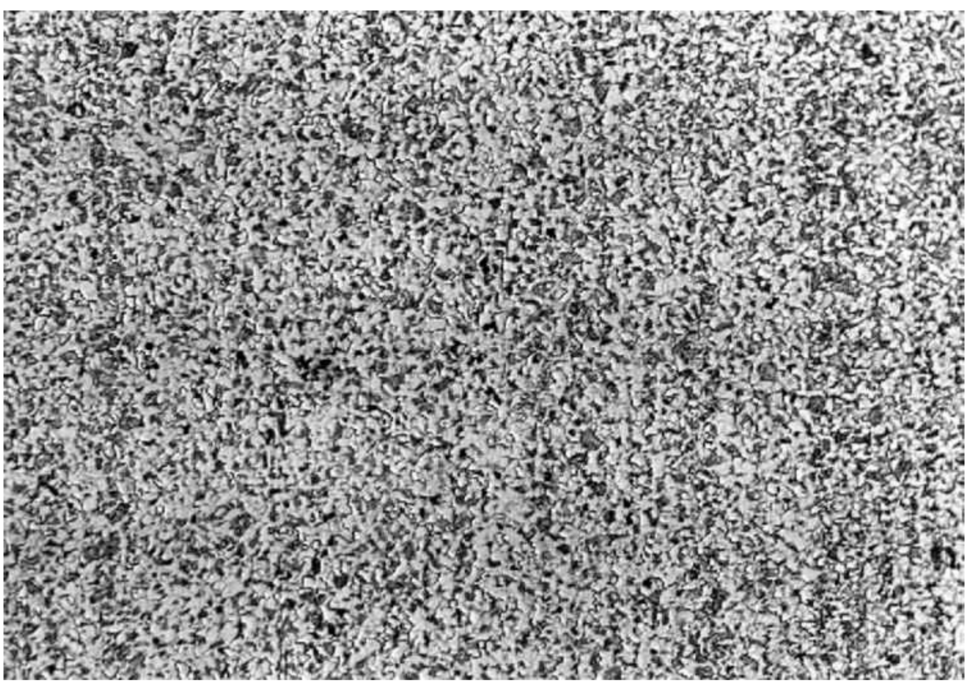


Figure 33 (D34468) 100X Core No. 4, Charpy Bar No. 1, HAZ

Figure 34 shows another view of the HAZ closer to the weld metal than the area shown in Figure 33. In this area, the HAZ exhibited coarse prior austenitic grains, with a grain size of 2.5 to 3.0 as measured per ASTM E 112, with grain boundary ferrite.

Figure 35 shows the weld metal in core No. 4. The structure in the weld metal consisted of large prior austenitic grains with blocky, lath-like grain boundary ferrite, with a grain size of 0.5 to 1.0 per ASTM E 112. Finer Widmanstatten ferrite was present within the large grains.

No significant non-metallic inclusions were observed.

Figure 34 (D34469) 100X Core No. 4, Charpy Bar No. 1, HAZ

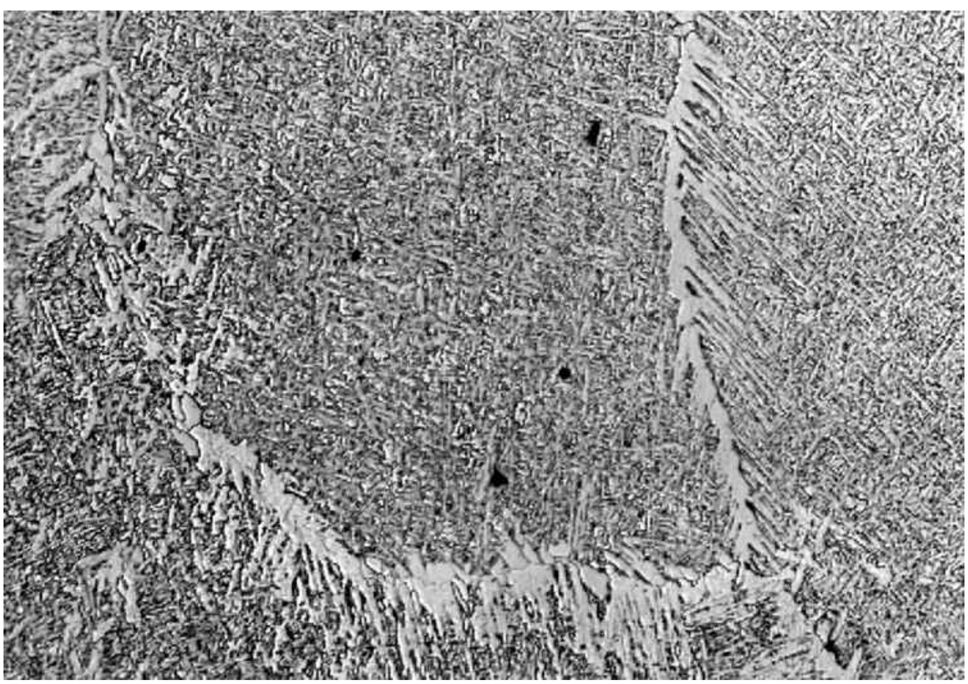


Figure 35 (D34470) 100X Core No. 4, Charpy Bar No. 9, Weld Metal

Impact Testing

The impact bars were machined and the tests conducted under subcontract at Koon-Hall-Adrian Metallurgical in Portland, Oregon. The thickness of the flange plate on the thinner side of the cores was insufficient to obtain three sets (rows) of Charpy bars; thus, only two sets of bars were removed from each core. One set was taken at the surface of the flange (corresponding to the inside of the girder) after first machining off a thin layer of metal to remove repair welds that had been made at the toes of the ESW welds during fabrication. The other set was taken below that, corresponding to the bottom half of the thinner plate or near mid-thickness of the thicker plate.

Each set consisted of five bars, for a total of 10 bars per core, or a total of 50 bars for the five cores. The bars were tested at five temperatures from -20 to +170°F.

Table 15 shows the results of the Charpy V-Notch impact tests, with the data organized according to test temperature and whether the bar was in the weld metal or HAZ[5]. Plots of the data are in the figures that follow.

Table 15. Charpy V-Notch Impact Test Data

Temp (°F)	Core No.	Impact Strength (ft-lb)	
		Surface	**Center**
Center Weld Metal			
-20	1	9.5*	5.0*
	2	6.0*	8.0*
0	1	26	9.0*
	1	11	8.5*
	2	5	18
40	1	34.5	18
	2	16	30
76	2	30.0	30.5
100	2	38.5	52
170	1	68.0	61

[5]A more detailed table showing the Charpy test results with the data organized according to core and bar number is in Table 17.

Temp (°F)	Core No.	Impact Strength (ft-lb)	
		Surface	**Center**
HAZ Metal			
-20	4	58	17
	5	53.0	16
0	3	12.0*	10
	4	60.0	18
	5	11.0	16
40	3	24.5	21
	3	13.5	24
	4	85.5	76
	4	82	68.5
	5	37	23
	5	102	18
76	3	38	63
100	3	32	57
	4	86	86
170	5	98.0	109

*value is outside of machine's verified range of 12.5 to 211.0 ft-lbs; per ASTM E 23-05, Annex A2, impact values outside of verified range are reported as approximate.

Charpy Impact Test Transition Curves

Figures 36 and 37 show plots of the test data from the Charpy V-notch tests. The purpose of the impact testing was to identify the location of the transitions between lower and higher impact strength, and to provide the Agency with data such that an estimate could be made of the in-service fracture toughness by use of the Barsom-Rolfe two-stage fracture correlation.

The curves shown in the two figures were fit to the data using the average impact strength at each test temperature. The curves are a combination of best-fit polynomial and manual plotting; in particular, the flat portions on both ends of the curve were forced manually.

The locations of the notches in the Charpy bars were established on the basis of a macro-etch of the coupons removed from the cores, prior to final machining. Subsequent review of the HAZ test data suggested the notches may not have been precisely in the HAZ; to resolve this, we metallographically mounted, polished, and etched the questionable fractured bars and examined them in more detail.

The examination showed that in several cases, the notches were indeed not precisely coincident with the HAZ, but instead were as much as 1 to 2 mm into the weld metal. Consequently, the HAZ data should be discounted somewhat and viewed as more representative of weld metal near the HAZ rather than precise HAZ values. Photographs of the polished Charpy bars after fracture showing the position of the notch in relation to the weld and HAZ are in the following figures.

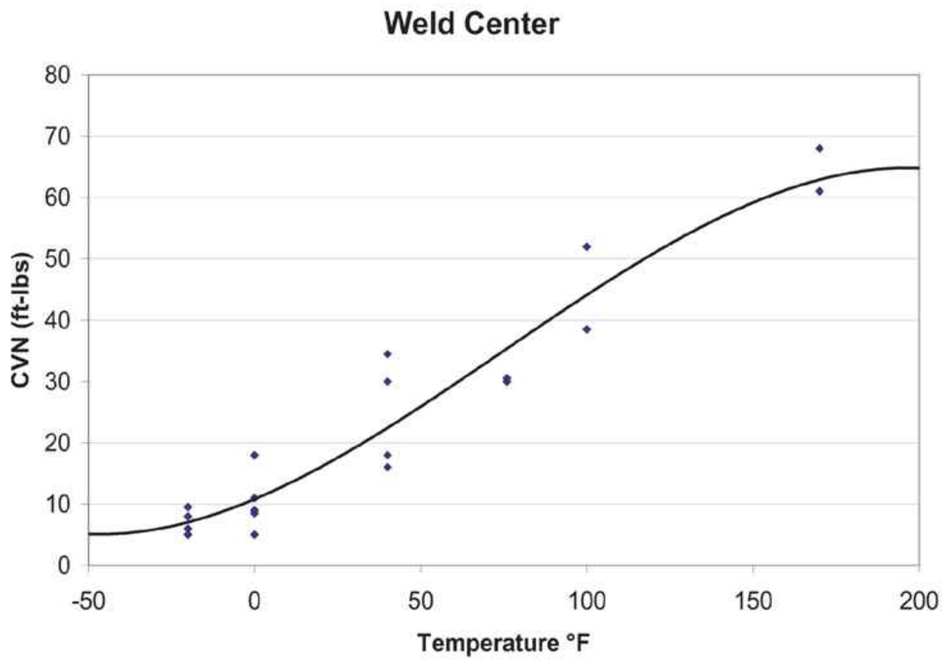


Figure 36 Charpy V-Notch Transition Curve, Weld Metal

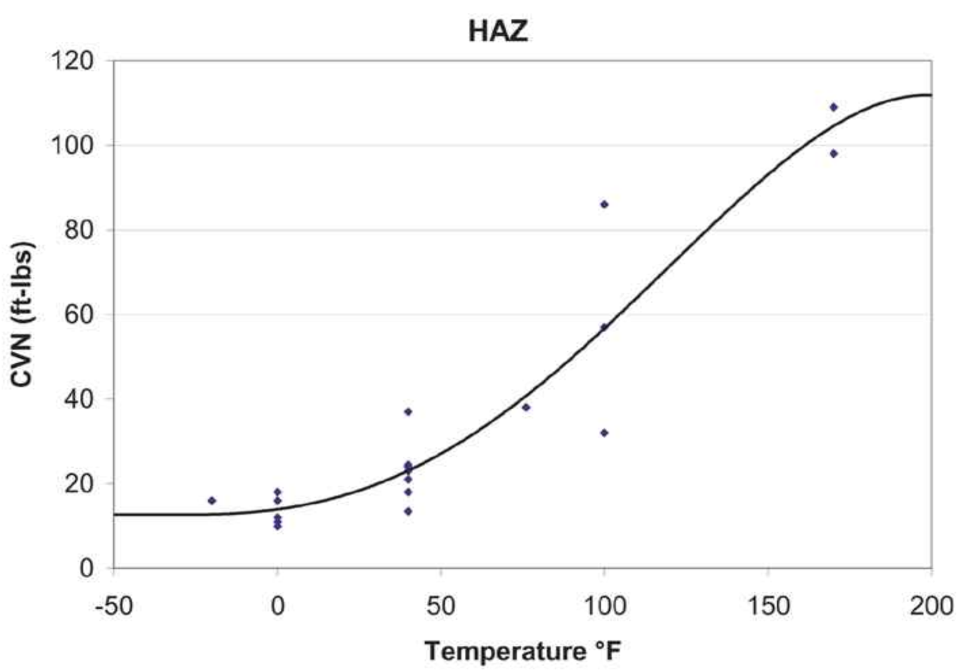


Figure 37 Charpy V-Notch Transition Curve, HAZ

Charpy Bar Notch Locations

Figures 38 and 39 show two of the fractured Charpy V-notch bars after polishing and etching them to determine the precise location of the notch. As noted in the previous caption, a review of the HAZ test data suggested some of the notches may not have been precisely in the HAZ. Microplishing and etching subsequently confirmed this, as shown in these two figures. In Figure 38, the notch is 1 mm beyond the HAZ into the weld metal. In Figure 39 , the notch is 2 mm beyond the HAZ into the weld metal.

Because the notches were not precisely centered on the HAZ, the HAZ impact data should be discounted somewhat and viewed as more representative of weld metal near the HAZ, rather than precise HAZ values.

Figure 38 (D72112-08) Core No. 4, Charpy V-Notch Impact Bar No. 8

Figure 39 (D72112-04) Core No. 4, Charpy V-Notch Impact Bar No. 4

Chemical Analysis

Table 16 summarizes the results of chemical analysis of one base metal sample and three weld metal samples.

The base metal sample was in conformance with the material specification, ASTM A 588, Grade B, “Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4-inches Thick.”

The carbon equivalent (a measure of the weldability of the material) was calculated by using the formula given in AWS D1.5, “Bridge Welding Code,” and AWS D1.1, “Structural Welding Code–Steel,” and is shown in the bottom line of the table.

Phase 1 Summary

The data from the Phase 1 testing did not indicate cause for alteration of the basic plan of the Phase 2 core removal and testing. However, rather than remove all ten Phase 2 cores authorized by the WOC, per Agency instructions, only seven Phase 2 cores were removed.

Table 16. Chemical Analysis Results

Element, weight percent	Material Specification	Sample*			
	ASTM A 588, Grade B	1	2	3	4
Carbon	0.20 max	0.13	0.15	0.11	0.16
Manganese	0.75 - 1.35	1.19	1.37	1.26	1.35
Phosphorus	0.04 max	0.01	0.011	0.012	0.014
Sulfur	0.05 max	0.02	0.014	0.016	0.012
Silicon	0.15 - 0.50	0.25	0.26	0.36	0.23
Chromium	0.40 - 0.70	0.52	0.4	0.18	0.43
Nickel	0.50 max	0.01	0.27	0.11	0.32
Copper	0.20 - 0.40	0.31	0.23	0.12	0.27
Vanadium	0.01 - 0.10	0.04	0.05	0.02	0.05
Molybdenum	--	--	<0.01	<0.01	<0.01
Aluminum	--	--	0.03	0.01	0.02
Niobium	--	--	<0.01	<0.01	<0.01
Titanium	--	--	<0.01	<0.01	<0.01
Boron	--	--	<0.0001	0	<0.0001
Tin	--	--	0.003	0.004	0.001
Iron		Balance	Balance	Balance	Balance
Carbon Equivalent**		0.5	0.55	0.44	0.56

Analysis performed by optical emission spectroscopy; carbon and sulfur by combustion.

*Sample No.	Description
1	Base metal, core No. 2, (next to weld SW3-A-B-1), plate south of weld, bottom of plate
2	Weld metal, core No. 4, weld SW4-A-B-3, bottom of weld
3	Weld metal, core No. 4, weld SW4-A-B-3, top of weld
4	Weld metal, core No. 3, weld SW3-B-B-1, bottom of weld

**Carbon equivalent calculated per AWS D1.1: CE = C +(Mn + Si)/6 + (Cr + Mo + V)/5 + (Ni + Cu)/15

Table 17. Charpy V-Notch Test Data, Arranged by Core No.

Core No.	Notch Location	Bar No.	Location	Temperature, °F	Impact Strength, ft-lbs
1	Weld	1	Surface	0	26
		2		0	11.0*
		3		40	34.5
		4		170	68
		5		-20	9.5*
		6	Center	40	18
		7		170	61
		8		-20	5.0*
		9		0	9.0*
		10		0	8.5*
2	Weld	1	Surface	40	16
		2		76	30
		3		0	5.0*
		4		100	38.5
		5		-20	6.0*
		6	Center	0	18
		7		40	30
		8		76	30.5
		9		100	52
		10		-20	8.0*

*Value is outside of machine's verified range of 12.5 to 211.0 ft-lbs; per ASTM E 23-05, Annex A2, impact values outside of verified range are reported as approximate.

Table 17. Charpy V-Notch Test Data, Arranged by Core No.

Core No.	Notch Location	Bar No.	Location	Temperature, °F	Impact Strength, ft-lbs
3	HAZ	1	Surface	0	12.0*
		2		40	24.5
		3		76	38
		4		100	32
		5		40	13.5
		6	Center	0	10.0*
		7		40	21
		8		76	63
		9		100	57
		10		40	24
4	HAZ	1	Surface	0	60
		2		40	85.5
		3		40	82
		4		100	86
		5		-20	58
		6	Center	0	18
		7		40	76
		8		40	68.5
		9		-20	17
		10		100	86

*Value is outside of machine's verified range of 12.5 to 211.0 ft-lbs; per ASTM E 23-05, Annex A2, impact values outside of verified range are reported as approximate.

Table 17. Charpy V-Notch Test Data, Arranged by Core No.

Core No.	Notch Location	Bar No.	Location	Temperature, °F	Impact Strength, ft-lbs
5	HAZ	1	Surface	0	11.0*
		2		40	37
		3		170	98
		4		40	102
		5		-20	53
		6	Center	0	16
		7		40	23
		8		170	109
		9		40	18
		10		-20	16

*Value is outside of machine's verified range of 12.5 to 211.0 ft-lbs; per ASTM E 23-05, Annex A2, impact values outside of verified range are reported as approximate.

Phase 2 Material Testing Scope of Work

Following the completion of the Phase 1 material evaluation of Task 4, seven additional cores were removed from the bridge girder bottom flanges for Phase 2 testing (Table 18). Of these, six cores were from welds, while the seventh was from the girder base material. The purpose of the Phase 2 testing was to quantify the in service fracture toughness and fatigue crack growth threshold of the weld and HAZ. The following specific tasks were included in the Phase 2 testing[6]:

- Test three weld metal specimens, three HAZ specimens, and one base metal specimen in accordance with ASTM E 1820 using the compact tension specimen configuration, at 0 °F and a target loading rate of zero to P_{max} in 1 to 3 seconds, to establish the fracture toughness.

- Test three weld metal specimens and one base metal specimen in accordance with ASTM E 647 to establish the fatigue crack growth rates and fatigue thresholds, at room temperature.

- Test three weld metal and three base metal specimens in accordance with ASTM E8 to establish tensile properties, at room temperature.

Table 18. Phase 2 Core Samples

Core Number	6	7	8	9	10	11	12
Material	HAZ	AWM	AWM	HAZ	BM	HAZ	HAZ
Weld Identification	WS4-A-B-2	WS4-A-B-1	WS3-A-B-5	WS3-A-B-4	WS1	WS1-A-B-5	WS1-A-B-4

Although not specifically requested in the WOC for Phase 2, the cores were macro etched to reveal the location of the weld, HAZ, and base metal, just has had been done for the Phase 1 cores. Figures 40 through 47 show the cores after macro etching. All six cores from welds were taken at welds between plates of different thicknesses. As noted in the Phase 1 material evaluation report, the thickness transition (step) was accommodated on the outside (bottom side) of the girder, with the inside surfaces flush with each other. (However, unlike in the Phase 1 report, the cores are shown in these photographs right side up, oriented as they would have been in service.)

As with the Phase 1 cores, the Phase 2 cores exhibited varying degrees of “barreling”, which, because the welds were not particularly easy to discern in the photographs, is highlighted here by series of dots along the weld fusion lines.

[6]The official Work Order Contract did not specify fracture toughness or fatigue testing of the base metal. Testing of the base metal was subsequently requested by the Agency after the WOC had been signed and authorized, but prior to the start of actual testing. Per ODOT’s request, the fracture toughness and fatigue testing of the base metal was incorporated into the test program at the same time as the testing of the weld metal and HAZ.

Figure 40 (D71147-01) Phase 2 Cores, Nos. 7 through 12

Figure 41 (D71147-03) Core No. 6 (HAZ)

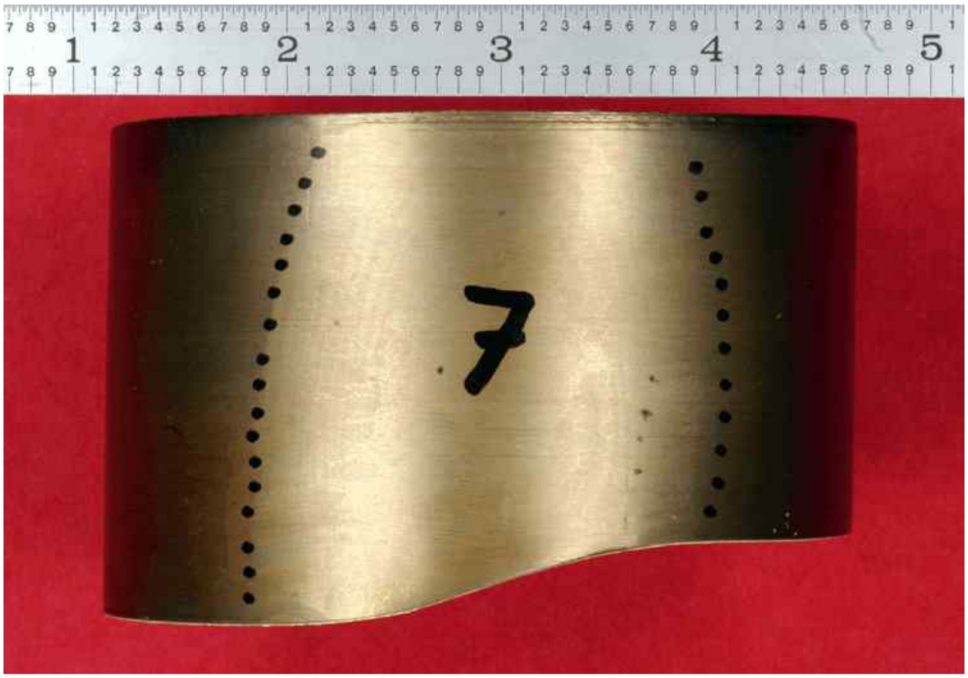


Figure 42 (D71147-05) Core No. 7 (weld metal)

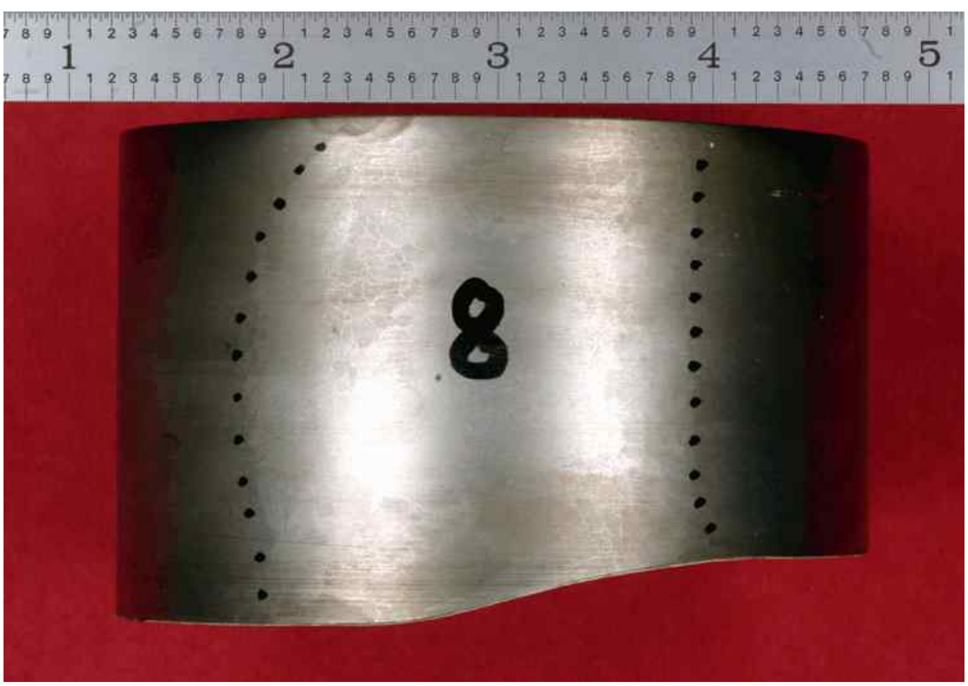


Figure 43 (D71147-08) Core No. 8 (weld metal)

Figure 44 (D71147-10) Core No. 9 (HAZ)

Figure 45 (D71147-11) Core No. 10 (base metal)

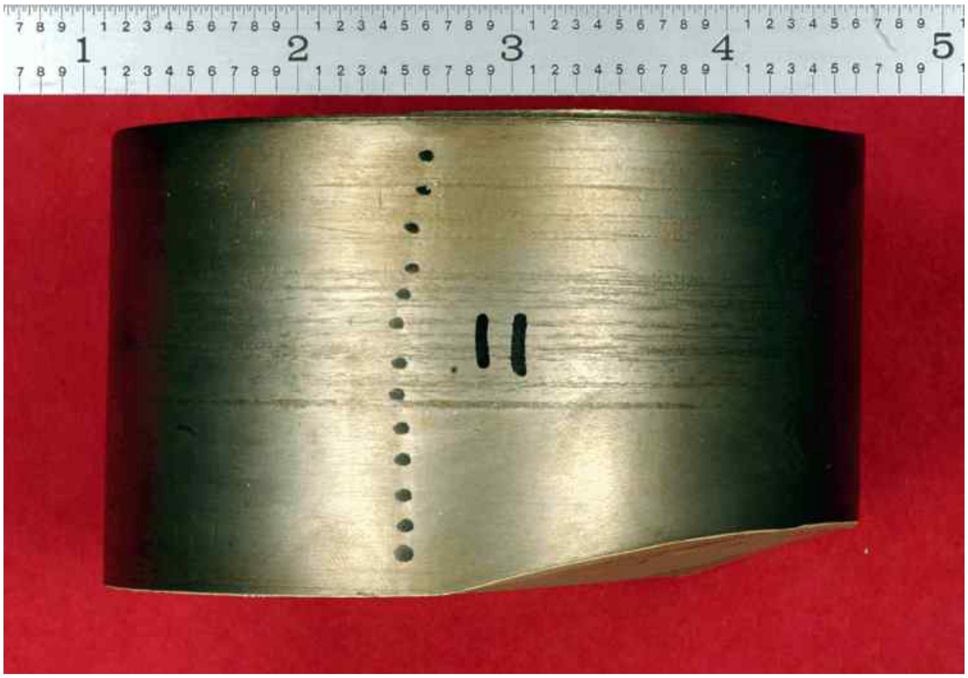


Figure 46 (D71147-12) Core No. 11 (HAZ)

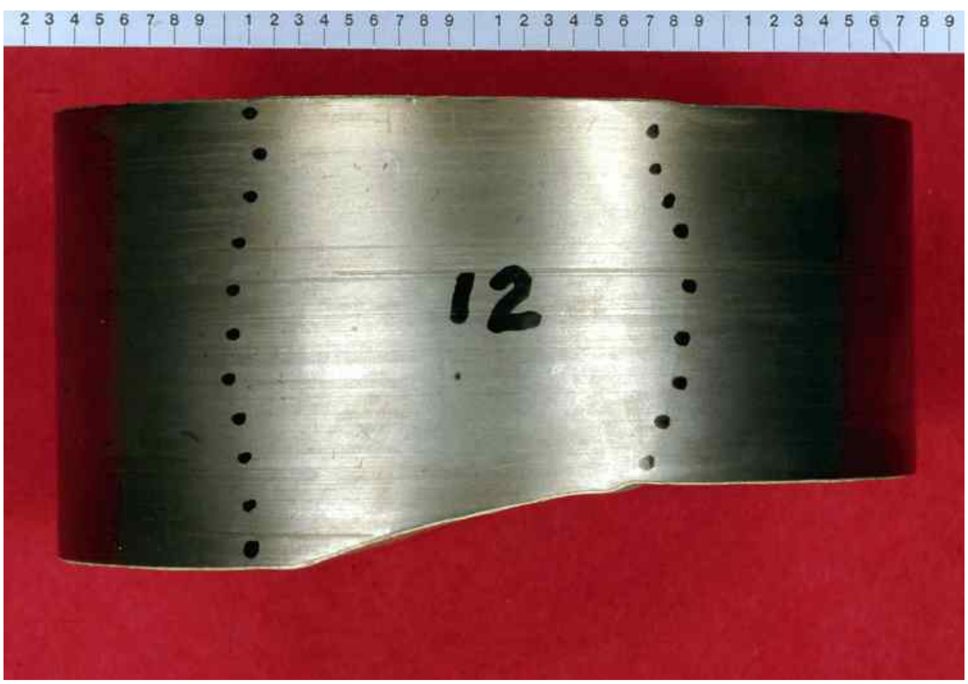


Figure 47 (D71147-15) Core No. 12 (weld metal)

Tensile Tests

The mechanical properties of the weld metal and base metal were measured by tensile tests on specimens machined from the three weld metal cores and the one base metal core. The base metal and weld metal both complied with the ultimate strength, yield strength, and elongation requirements of ASTM A 588 steel. (The specified bridge steel.) The test results are shown in Table 19.

Table 19. Tensile Test Results*

Property	ASTM A 588	Core No.						Average	
		7	8	12	10-1	10-2	10-3	Weld metal	Base metal
		Weld metal			Base metal				
Ultimate Strength, ksi	70	84.6	77.6	85.5	84.6	84.4	83.8	82.6	84.3
Yield Strength, ksi	50	57.6	50.5	58.8	55.4	55.8	56.8	55.6	56
Elongation, % in 4D	19	22	27	24.5	27.5	29	29	24.5	28.5
Reduction of area, %		61.5	66.5	59.5	72.5	72.5	72.5	62.5	72.5

*tensile testing conducted by Koon Hall Adrian Metallurgical (Portland, Oregon).

Fatigue Crack Growth and Fatigue Threshold Test and J-Fracture Toughness Test

The fatigue crack growth and fatigue threshold test per ASTM E467 and the J-fracture toughness testing per ASTM E 1820 was conducted under subcontract by CC Technologies, Inc. (Dublin, Ohio). One compact tension (CT) specimen (Figure 48) was machined from each of the seven core samples.

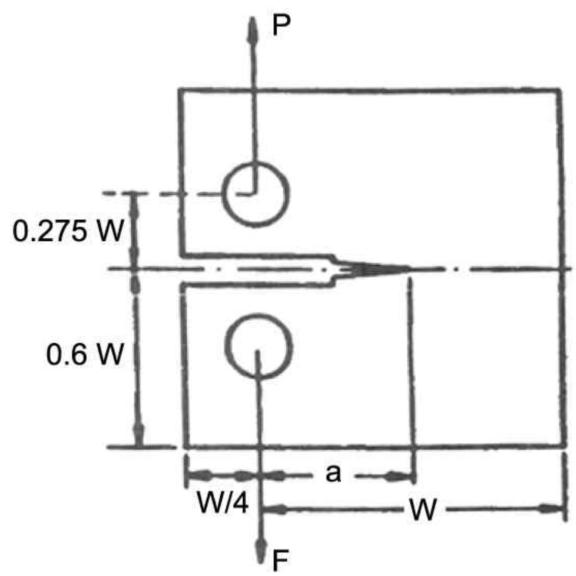


Figure 48. Diagram of CT Specimen

The CT specimens were used to derive both the fatigue crack growth rate data and the fracture toughness data. The testing procedures and the results are described in detail in the report from CC Technologies, Inc., which follows on the next page:

CC Technologies
A DNV COMPANY
Innovative Solutions

5777 Frantz Road, Dublin, OH 43017-1386 USA
TEL 614-761-1214 FAX 614-761-1633

January 31, 2007

Mr. Robert Hodel, P.E.
General Manager
MEI-Charlton, Inc.
2233 SW Canyon Road
Portland, OR

Re: Fatigue Crack Growth and Fatigue Threshold Testing per ASTM E647,[1] and J-Fracture Toughness Testing per ASTM E1820[2] of Steel Base Metal, Heat Affected Zone, and Weld Core Samples from Fremont Bridge. (Project 80631001)

Dear Mr. Hodel:

This letter reports the results of fatigue-crack-growth-rate and fatigue-threshold testing, and J-fracture-toughness testing. The objectives of the work were to measure the fatigue crack growth rates (da/dN) as a function of stress intensity factor range (ΔK), determine the fatigue threshold (ΔK_{th}) at room temperature, and determine the J-fracture-toughness at 0 ^oF (-18 ^oC).

BACKGROUND

The State of Oregon Department of Transportation *(ODOT)* is investigating electroslag welds of an existing highway bridge. The bridge is constructed of 1.5-in. to 2.5-in. thick members of structural carbon steel. The welds are made using a single-pass, cast-in-place process involving an open groove between the members. The groove is filled with flux wire and copper shoes are placed on both sides of the groove to provide the welding current to make the joint. The cross-section of the welds is barrel-shaped, which means that the weld is wider at mid-wall than at either surface. The welds have a very coarse grain structure in the heat affected zone (HAZ). Taking these factors into account, the approximate width of each weld is 2-in. of which about 1-in. is fusion metal and 0.5-in. is HAZ on either side of the weld.

TECHNICAL APPROACH

MEI Charlton *(MEI)* provided seven (7) ASTM A588 steel core samples that were removed by MEI from the Fremont Bridge in Oregon for the work. It was reported that a 4-in. diameter hole cutter was used, which gave samples of approximately 3.5-in. diameter. The middle of

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1 E647-00 Standard Test Method for Measurement of Fatigue Crack Growth Rates, Book of Standards, Volume 03.01, ASTM International.

2 E1820-06 Standard Test Method for Measurement of Fracture Toughness, Book of Standards, Volume 03.01, ASTM International.

the round core samples was positioned over either the centerline of the weld fusion metal or the HAZ. To achieve this, the coating was locally removed from the steel, the steel was polished and etched to reveal the weld, and the core drill was aligned prior to cutting, see Figures 1a and 1b. The 3.5-inch diameter round cores varied in thickness from approximately 1.5 to 2.5-inch, and included one (1) base metal sample, three (3) heat affected zone (HAZ) samples, and three (3) all weld metal (AWM) samples from electroslag welds; see Figure 2. Core identification and the test matrix are summarized in Table 1.

CC Technologies, Inc. *(CC Technologies)* prepared one full size (1-T) compact tension (CT) specimen and one round tensile bar specimen from each core. The tensile bars were sent to MEI for testing, and the CT-specimens were used by CC Technologies for the fatigue and fracture toughness tests. A schematic of the core and specimens geometries with respect to the welds is shown in Figure 3.

The CT specimens were made with the 3.5-in. diagonal dimension (W = 2.0 in. which resulted in an approximately 2.4 × 2.5 in. rectangular specimen) – refer to Figure A2.2 of ASTM E1820. The specimens had a thickness (B-dimension in Figure A2.2) of 1-inch and were side-grooved (0.100-in. deep) on both sides to create a net section thickness (B_N) of 0.800-in.; refer to Figure 4. In the AWM and HAZ specimens, the direction of the notches was along the length direction of the weld.

RESULTS

Fatigue-Crack-Growth-Rate Testing

Fatigue crack growth rate (CGR) and fatigue threshold testing was performed in air and at room temperature using the CT specimens. Three specimens were tested of AWM and one specimen was tested of base metal. Another three specimens of HAZ metal were fatigue pre-cracked, but not tested to the threshold. The pre-cracks had an approximate total length and a ratio a/W = 0.7.

The specimens were subjected to cyclic loading using an electro-hydraulic test system. A load (P) shedding technique was applied to gradually decrease the range of the stress intensity factor (ΔK) for the growing crack. The R-ratio was 0.1 (minimum load divided by maximum load = P_{min}/P_{max}), and the cyclic frequency was in the range of 20 to 39 Hz. As the crack grew, the applied load range was periodically reduced to decrease the applied load. The results were analyzed in accordance with the procedures outlined in ASTM E647 to develop a plot of da/dN versus ΔK. The ΔK_{th} value was then determined from the plot.

The da/dN versus ΔK curves for the base-metal and AWM specimens are shown in Figure 5. The results of the fatigue CGR and fatigue threshold testing are summarized in Table 2. The curves showed two different slopes, and therefore the curve-fit coefficients for both are reported in the table. For the high-growth region, the values represent the Paris Law coefficients.

J-Fracture-Toughness Testing

After the fatigue CGR tests, the same AWM and base metal specimens were re-precracked to prepare them for J-fracture toughness testing. The HAZ specimens were tested in the pre-cracked condition, without CGR testing.

The specimens were cooled to 0 +/- 2 ^{0}F (-18 +/- 1 ^{o}C) inside a test chamber in a servo-hydraulic testing machine by submerging them in methanol and adding dry ice as needed. The specimen temperature was measured from an attached thermocouple. Once the test temperature was reached, the specimen was soaked at that temperature for approximately 20 minutes. A monotonically increasing load from zero to maximum load was then rapidly[(3)] applied to the specimen under displacement control until there was a sudden load drop and a rapid increase in crack length. The specimens were instrumented for DC electric potential drop (EPD) measurements, from which the crack length (and thereby growth rate) was determined during the test.

MEI provided tensile property data for the material; see Table 3. For the present analysis, the average values for the base metal and the weld metal were used. The average weld metal property values were also used for the HAZ specimens.

With the tensile property data, the specimen and final crack size dimensions, the load-displacement-crack length data were analyzed to produce a J-R curve (J-energy per unit of area versus change in crack length) for each specimen; see Figures 6a through 6g. From the curves, J(Q) values were determined for the specimens – refer back to Table 2. All the specimen dimensions and load requirements in ASTM E1820 were fulfilled, except for the outer two measurements of the physical crack lengths, as determined using the 9-point crack sizing method. Therefore, the J(Q) values could not be qualified as J_{Ic}-values. A summary of the specimen dimensions and data values used in the analysis is provided in Table 4. Photographs of the fracture surfaces of the specimens are shown in Figure 7.

The data for the base metal specimen 10-WS1 were well behaved, as seen from the JR curve (Figure 6a) with an initial slope close to the suggested slope from the ASTM standard. The J(Q) value is determined at the point where the curve intersects the 0.2-mm offset line (126.6 kJ/m^2 = 723 lb$_f$/in. for the base metal).

The data for the HAZ specimens revealed a steeper slope and had J(Q) values higher than those for the base-metal specimen. Specimen 9-WS3AB4 and Specimen 11-WS1AB5 failed suddenly within the area of J-R valid values, as seen from the sudden increase in crack length at 171.0 and 339.3 kJ/m^2 (976 and 1,937 lb$_f$/in.), respectively; see Figures 6c and 6d. Specimen 6-WS4AB2 did not fail below the J-limit value for valid test data was reached, and therefore that value (391.6 kJ/m^2 [2,236 lb$_f$/in.]) was reported as J(Q); see Figure 6b.

The data for the AWM specimens was difficult to interpret. The EPD signals during the first phase of the load increase showed an erroneous crack increase, followed by a decrease to approximate initial value for the specimens. This is visible as a “nose” in the JR-curves; see

---

3 The load was applied with a target rate of 1 to 3 seconds to fail the specimen.

Figures 6e, 6f, and 6g. This phenomenon occurred in the nominally elastic deformation range, where the crack is not expected to grow, but the plastic zone size around the crack tip can increase. By ignoring the nose, and proceeding with the standard data analysis of the remainder of the data, a J(Q) value for the specimens could be determined, nonetheless. The three specimens (Specimen 7-WS4AB1, 8-WS3AB5, and 12-WS1AB4) behaved approximately the same, with values of 64.7, 58.2, and 69.6 kJ/m^2 (369, 332, and 397 lb_f/in.), respectively.

SUMMARY OF RESULTS

Base metal had lower fatigue-crack-growth resistance than weld metal, with ΔK_{th} = 7.84 MPa√m (7.13 ksi √ in). for base metal and 9.30 to 10.54 MPa√m (8.45 to 9.58 ksi √ in.) for weld metal. The fatigue-crack-growth-rate for base metal was also higher than for weld metal for given ΔK.

J(Q) for HAZ metal was higher than that for base metal, 171 to 392 kJ/m^2 (976 to 2,236 lb_f/in.) versus 127 kJ/m^2 (723 lb_f/in.). J(Q) for weld metal was significantly lower than for base metal, 58 to 70 kJ/m^2 (332 to 397 lb_f/in.) versus 127 kJ/m^2 (723 lb_f/in.). This is a relatively low value of toughness for weld metal.

Please contact me if you have any questions about these results.

Sincerely,
CC TECHNOLOGIES, INC.

Author:

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Carl E. Jaske, Ph.D., P.E.
Senior Project Manager

Approved by:
Oliver C. Moghissi

Oliver C. Moghissi, Ph.D.
Sr. Vice President - Laboratories

cc: Bill Kovacs, Carl Jaske, Oliver Moghissi (CCT)

Table 1. Identification of Core Samples and Test Matrix.

Sample ID	Region(*)	Core Thickness (inch)		ASTM E647-05 Room Temperature			ASTM E1820-06 0 °F (-18 °C)
		Thin Side	Thick Side	Fatigue Precrack	CGR(*) and ΔK_{th} Test	Fatigue Re-Precrack	JR-Curve Test
10-WS1	Base Metal	1.94		Yes	Yes	Yes	Yes
6-WS4AB2	HAZ	1.55	2.06	Yes	–	–	Yes
9-WS3AB4	HAZ	1.53	2.05	Yes	–	–	Yes
11-WS1AB5	HAZ	1.98	2.27	Yes	–	–	Yes
7-WS4AB1	AWM	2.05	2.43	Yes	Yes	Yes	Yes
8-WS3AB5	AWM	2.04	2.40	Yes	Yes	Yes	Yes
12-WS1AB4	AWM	1.54	1.93	Yes	Yes	Yes	Yes

(*) HAZ = heat affected zone, AWM = all weld metal, CGR = crack growth rate

Table 2. Summary of Fatigue and Fracture Toughness Test Results.

Sample ID	Region	da/dN (in./cycle)		ΔK_{th}		J(Q) (**)	
		Near Threshold	Higher ΔK	(MPa$\sqrt{m}$)	(ksi$\sqrt{inch}$)	(kJ/m^2)	lb$_f$/in.
10-WS1	Base Metal	1.45E-15 × $\Delta K^{8.02}$	9.10E-10 × $\Delta K^{2.44}$	7.84	7.13	126.6	723
6-WS4AB2	HAZ	–	–	-	–	391.6	2,236
9-WS3AB4	HAZ	–	–	-	–	171.0	976
11-WS1AB5	HAZ	–	–	-	–	339.3	1,937
7-WS4AB1	AWM	4.20E-19 × $\Delta K^{11.0}$	1.25E-11 × $\Delta K^{4.04}$	9.80	8.91	64.7	369
8-WS3AB5	AWM	5.74E-21 × $\Delta K^{13.3}$	1.99E-09 × $\Delta K^{1.99}$	9.30	8.45	58.2	332
12-WS1AB4	AWM	1.97E-21 × $\Delta K^{13.1}$	7.94E-11 × $\Delta K^{3.26}$	10.54	9.58	69.6	397

(**) 1 kJ/m^2 = 5.710148 lb$_f$/in.

Table 3. Mechanical Properties [Data provided by MEI Charlton, Inc.].

	ASTM A588	Core No.						Average	
		7	8	12	1-Oct	2-Oct	3-Oct		
		Weld	Weld	Weld	Base metal			Weld metal	Base metal
Ultimate Strength, ksi	70.0 min.	84.6	77.6	85.5	84.6	84.4	83.8	82.6	84.3
Yield Strength, ksi	50.0 min.	57.6	50.5	58.8	55.4	55.8	56.8	55.6	56.0
Flow Strength (YS+UTS)/2 , ksi		71.1	64.1	72.2	70.0	70.1	70.3	69.1	70.1
Elongation, %	19.0 min.	22.0	27.0	24.5	27.5	29.0	29.0	24.5	28.5
Reduction of area, %		61.5	66.5	59.5	72.5	72.5	72.5	62.5	72.5
E-Modulus, ksi								27,817	28,000

Table 4. Specimen and Crack Dimensions Used in Data Analysis.

	10-WS1	7-WS4AB1	8-WS3AB5	12-WS1AB4	6-WS4AB2	9-WS3AB4	11-WS1AB5
Sample Condition	**Base Metal**	**AWM**	**AWM**	**AWM**	**HAZ**	**HAZ**	**HAZ**
Specimen Width (W), inch	1.9921	1.9873	1.9875	1.9905	1.9960	1.9978	1.9963
Crack Length (a), before fatigue	1.1548	1.2360	1.1720	1.1753	N/A	N/A	N/A
Crack Length a(o), after pre-cracking	1.3730	1.3990	1.3540	1.3960	1.3990	1.4000	1.4010
Specimen Thickness B, inch	0.9940	0.9934	0.9948	0.9923	0.9980	0.9984	0.9968
Specimen Net Thickness (B_N), inch	0.800	0.800	0.800	0.800	0.800	0.800	0.800
Physical Crack Size (a(p)), inch	1.3499	1.3365	1.3465	1.3659	1.3928	1.4017	1.3921
Remaining Ligament b(o), inch	0.6422	0.6508	0.6410	0.6246	0.6032	0.5961	0.6042
Yield Strength (YS), ksi	56.0	55.6	55.6	55.6	55.6	55.6	55.6
Ultimate Tensile Strength (UTS), ksi	84.3	82.6	82.6	82.6	82.6	82.6	82.6
Flow Strength (FS), ksi	70.1	69.1	69.1	69.1	69.1	69.1	69.1
Elastic Modulus (E), ksi	28,000	27,817	27,817	27,817	27,817	27,817	27,817
Poisson's Ratio (ν)	0.3	0.3	0.3	0.3	0.3	0.3	0.3

(N/A) = Not applicable

(a) (b)

Figure 1. Photographs of removing core samples by (a) removing Paint and etching the steel, and (b) cutting with hole saw [Courtesy of MEI Charlton, Inc.].

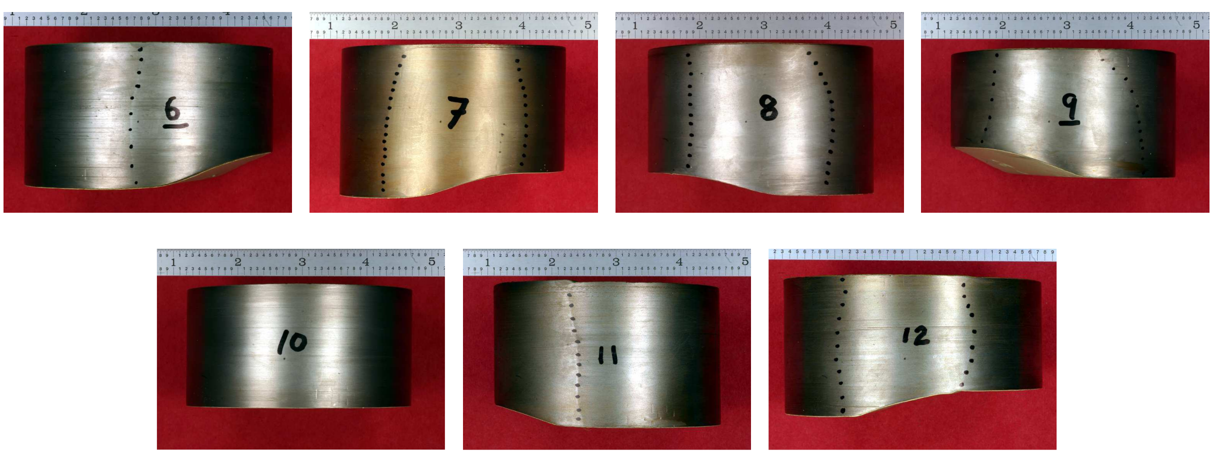


Figure 2. Photographs of core samples with approximate weld location marked as dotted lines [Courtesy of MEI Charlton, Inc.].

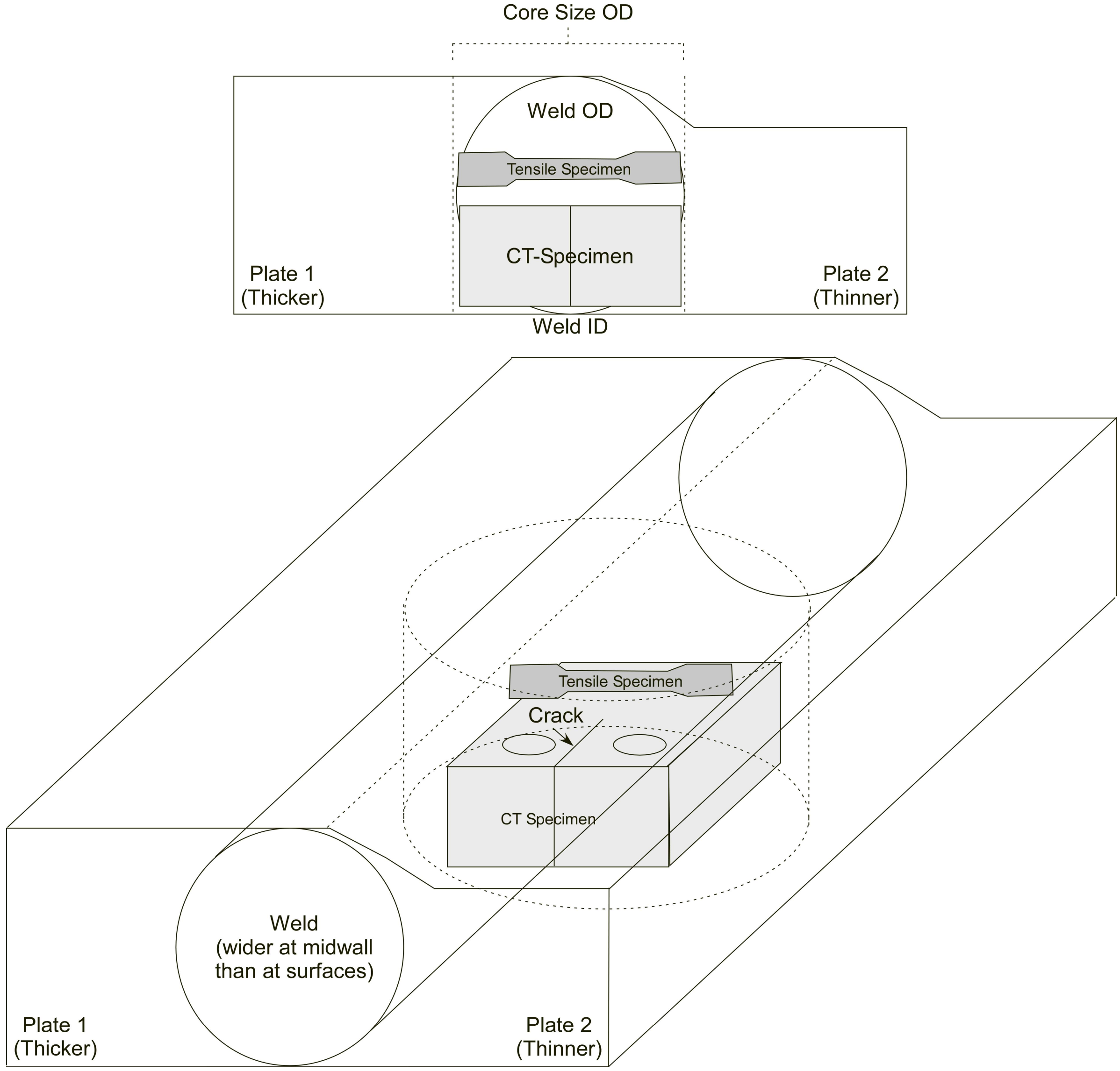


Figure 3. Drawing of specimen orientations.

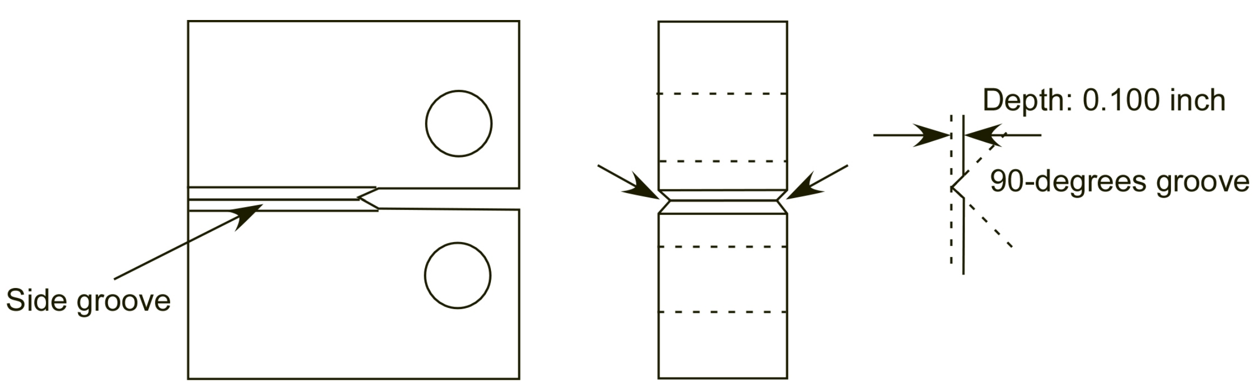


Figure 4. Drawing showing orientation of side grooves in CT-specimen.

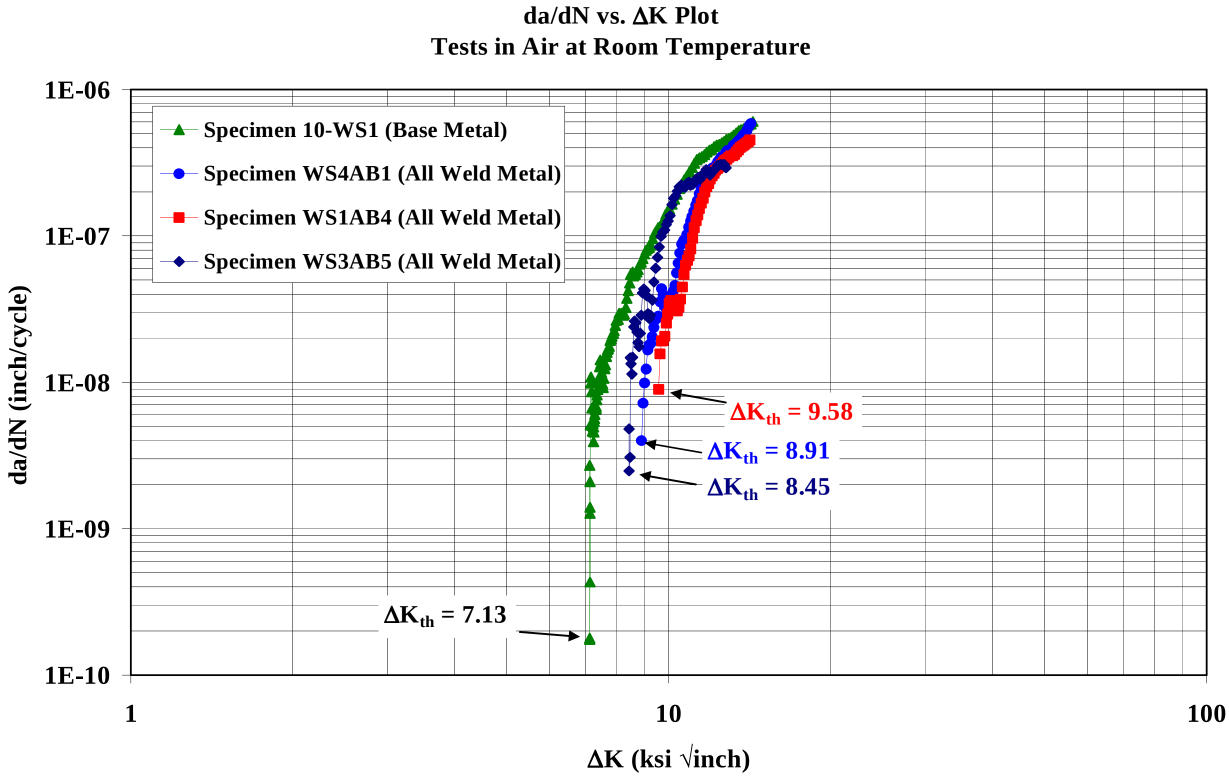


Figure 5. Fatigue crack growth rate curves for tests in air at room temperature.

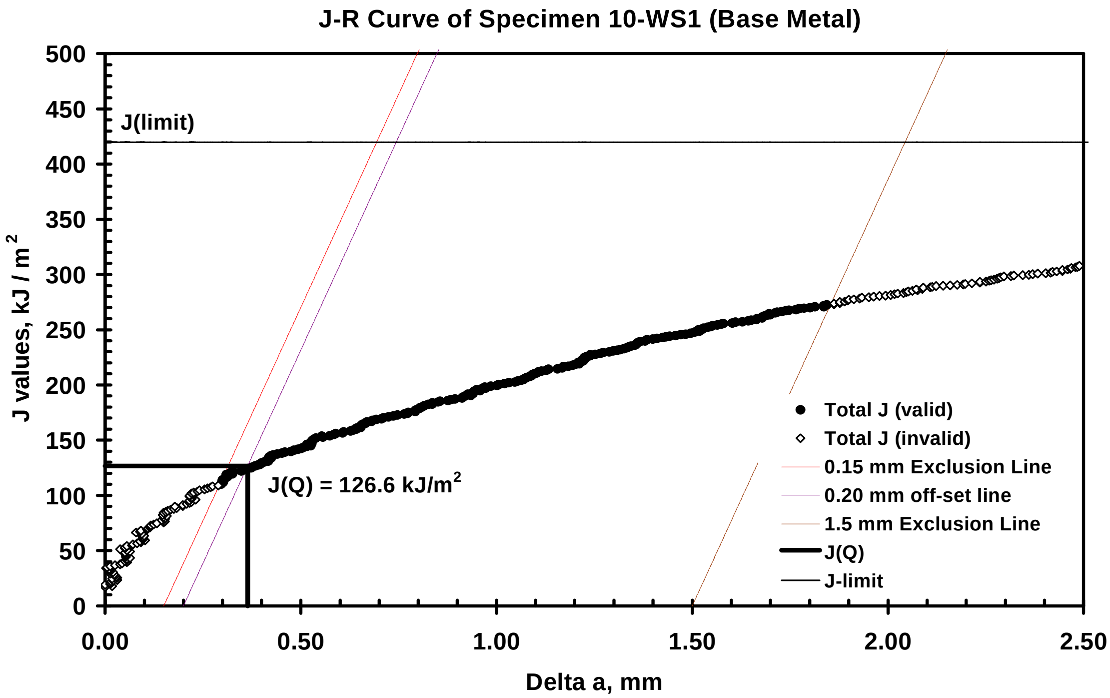


Figure 6a. J-R curve for Specimen 10-WS1 (base metal), at 0 °F (-18 °C).

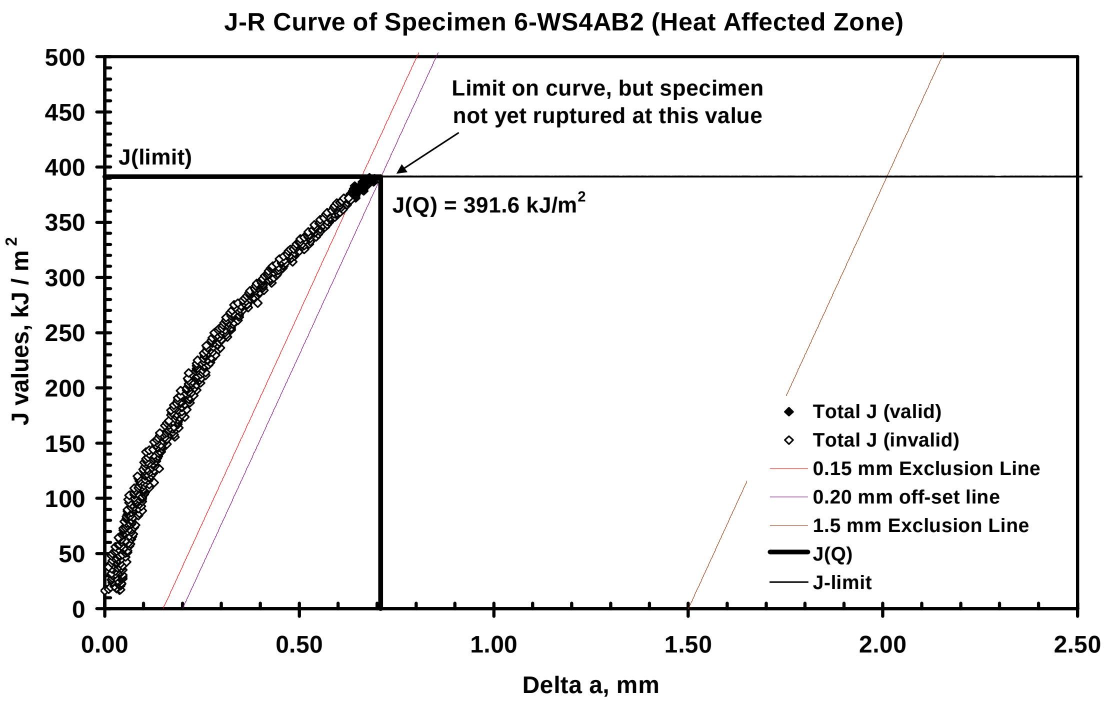


Figure 6b. J-R curve for Specimen 6-WS4AB2 (heat affected zone), at 0 °F (-18 °C).

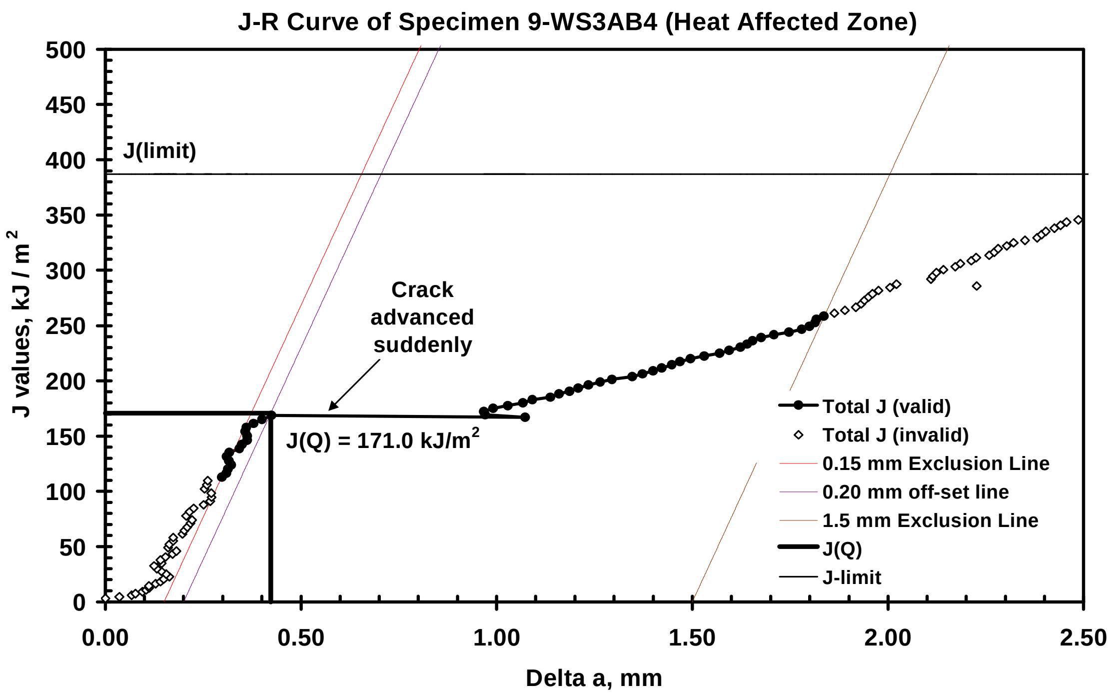


Figure 6c. J-R curve for Specimen 9-WS3AB4 (heat affected zone), at 0 °F (-18 °C).

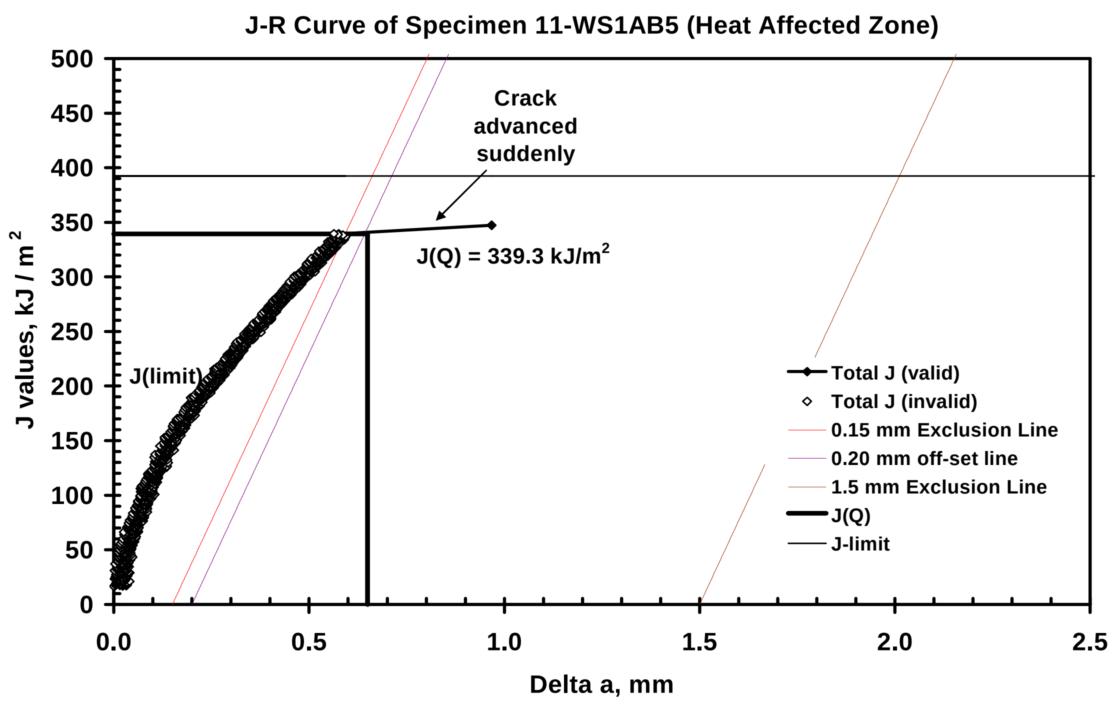


Figure 6d. J-R curve for Specimen 11-WS1AB5 (heat affected zone), at 0 °F (-18 °C).

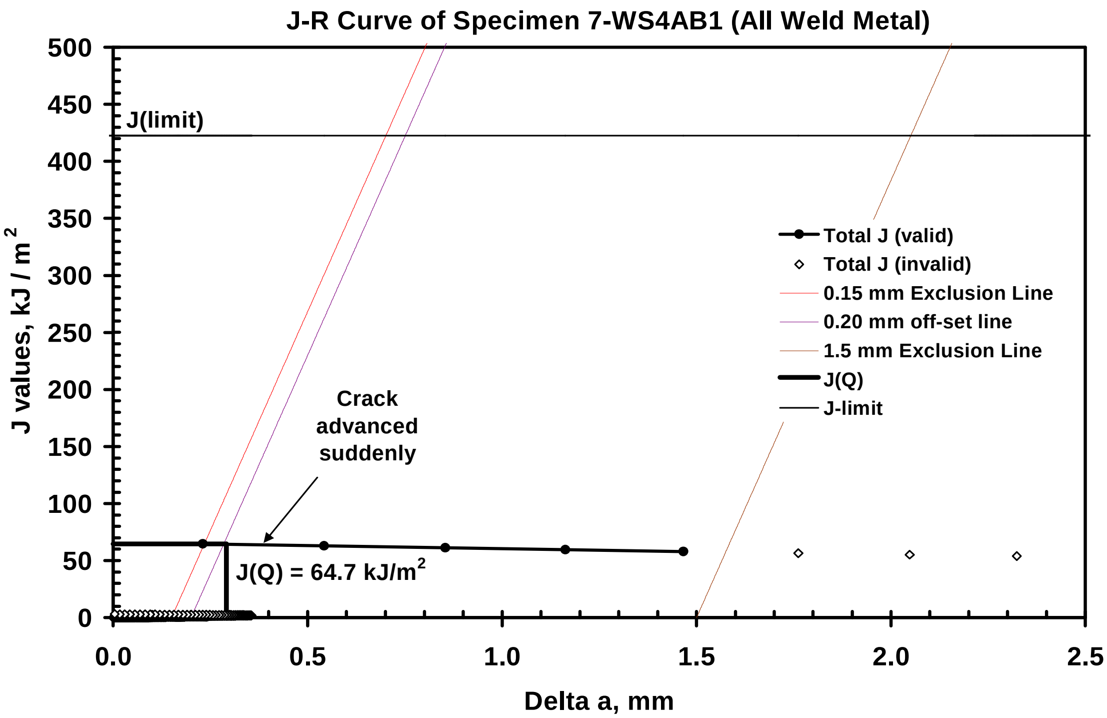


Figure 6e. J-R curve for Specimen 7-WS4AB1 (all weld metal), at 0 ^{o}F (-18 ^{o}C).

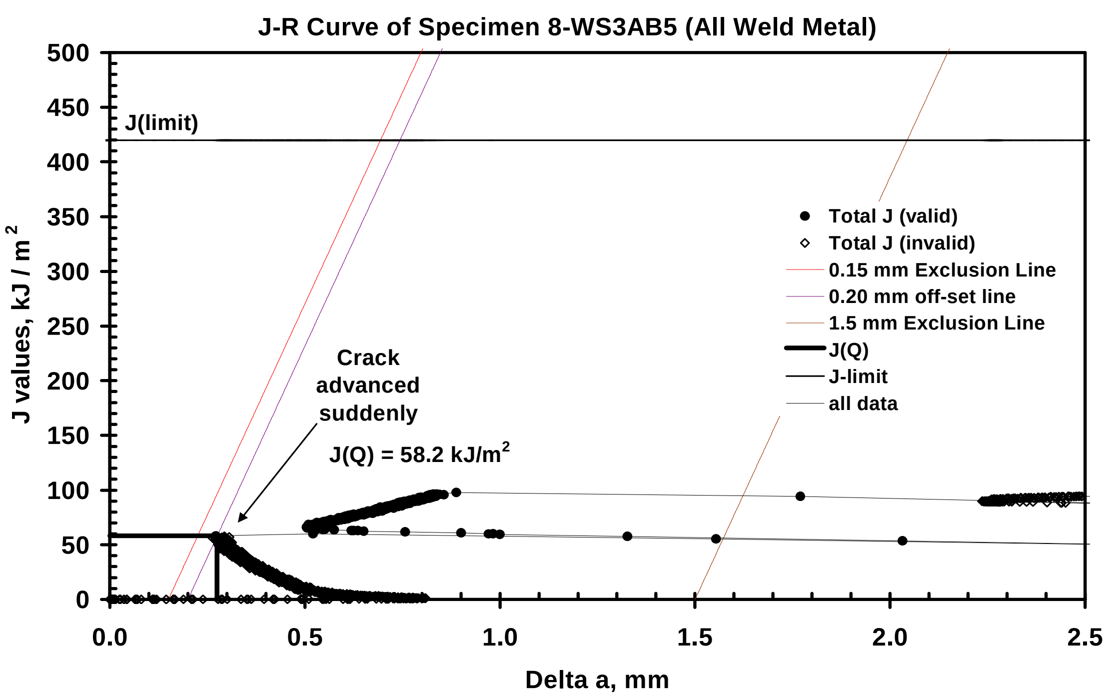


Figure 6f. J-R curve for Specimen 8-WS3AB5 (all weld metal), at 0 ^{o}F (-18 ^{o}C).

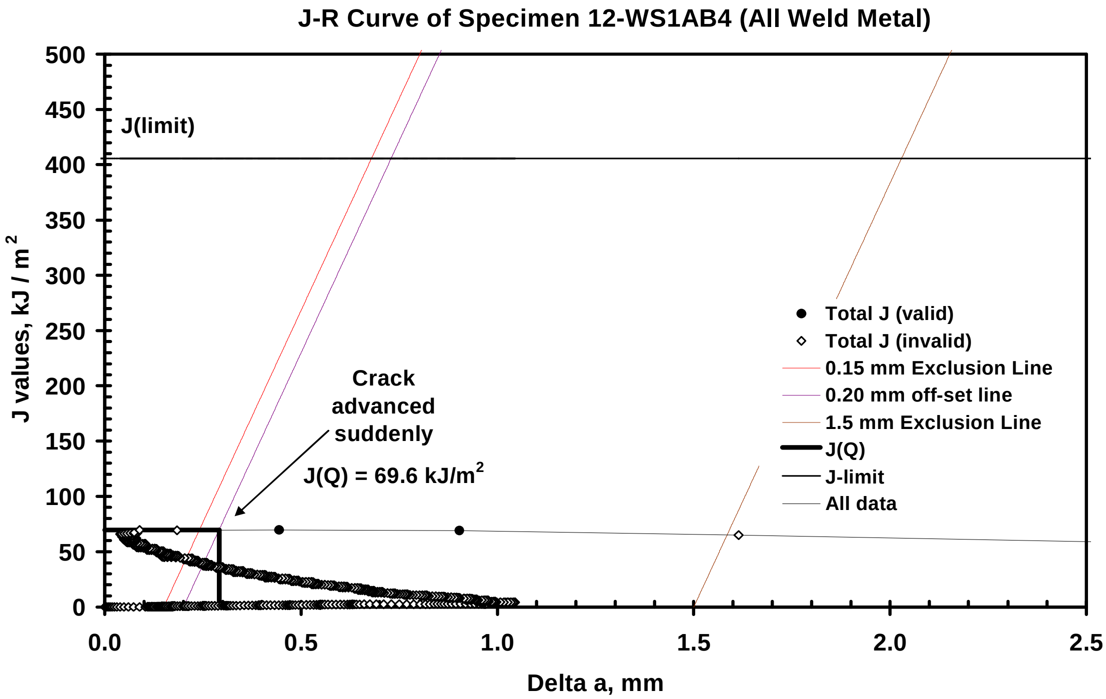
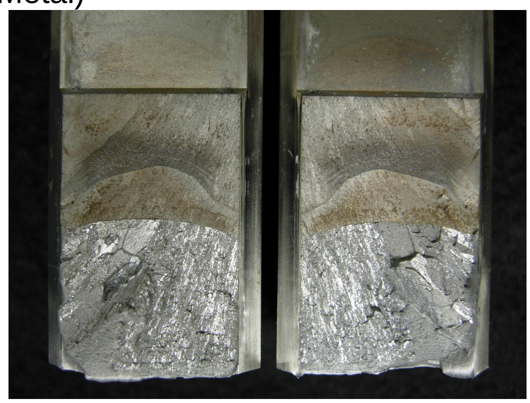


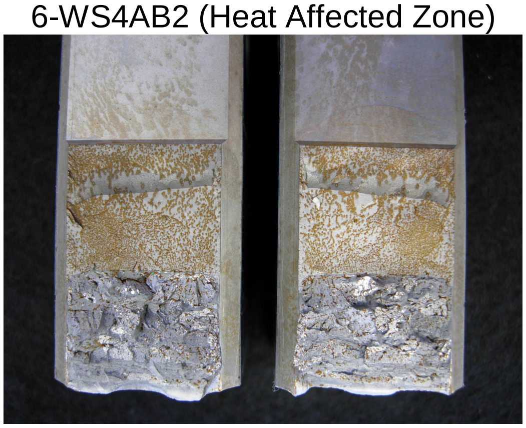
Figure 6g. J-R curve for Specimen 12-WS1AB4 (all weld metal), at 0 ^{o}F (-18 ^{o}C).

10-WS1 (Base Metal)

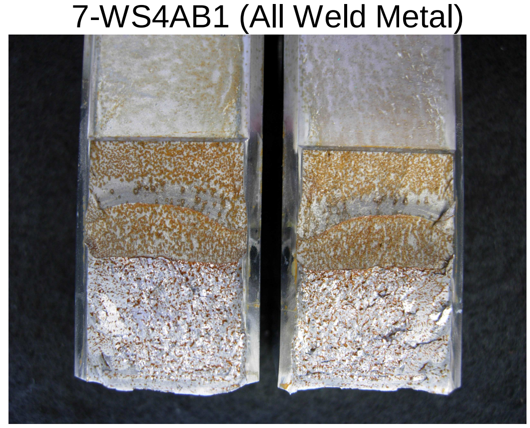
6-WS4AB2 (Heat Affected Zone)



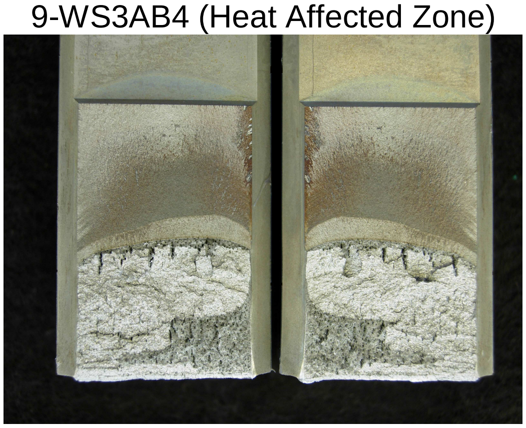
7-WS4AB1 (All Weld Metal)



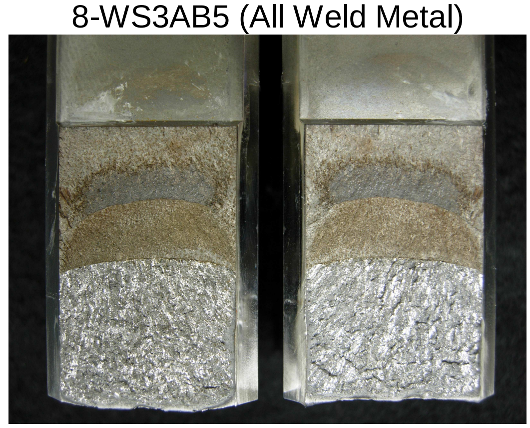
9-WS3AB4 (Heat Affected Zone)



8-WS3AB5 (All Weld Metal)



11-WS1AB5 (Heat Affected Zone)



12-WS1AB4 (All Weld Metal)

Figure 7. Photographs of Fracture Surfaces.

APPENDIX A

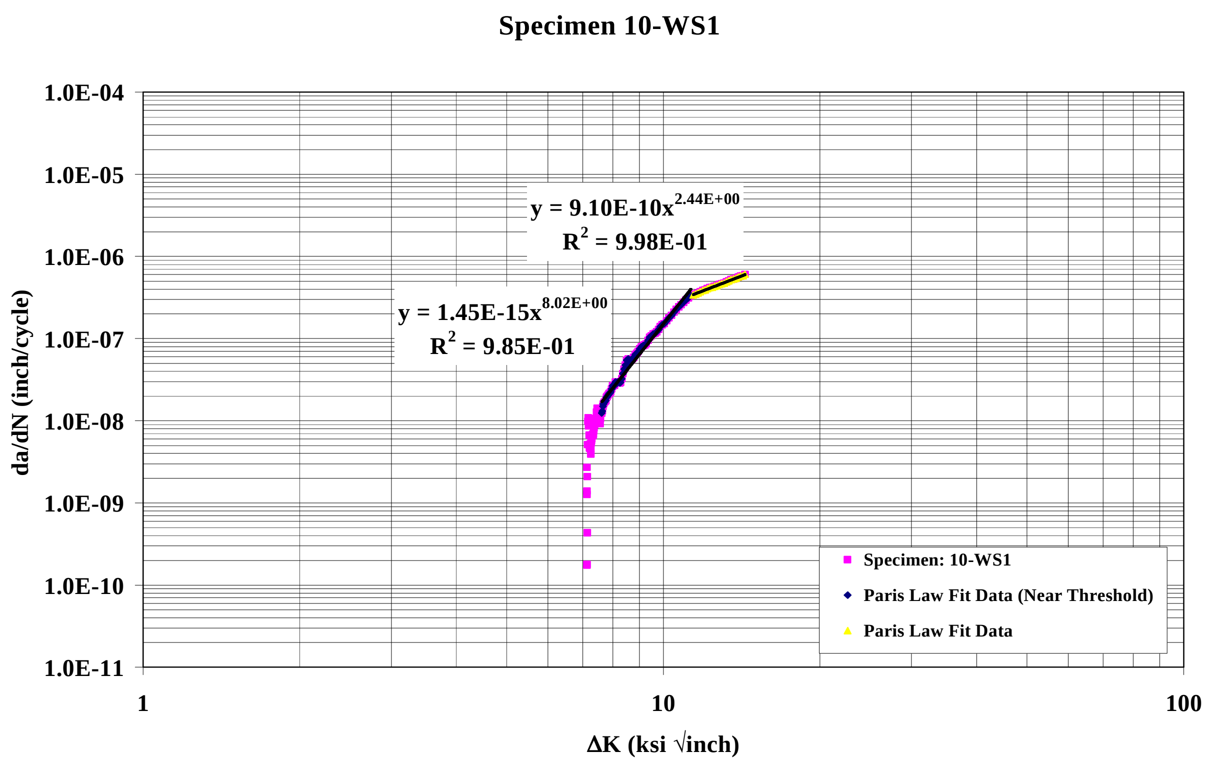
FATIGUE-CRACK-GROWTH-RATE DATA

Specimen 10-WS1 (Steel Base Metal)
Test Temperature: Lab Temperature (75 F°), Test Load Ratio: R = 0.10
Test Frequency/Waveshape: 20 Hz./Sinusoidal
K-Decreasing Threshold da/dN (C = -2.00)

da/dN (inch/Cycle)	ΔK (ksi √ inch)
6.05E-07	14.35
5.82E-07	14.23
5.75E-07	14.08
5.61E-07	13.94
5.41E-07	13.79
5.31E-07	13.65
5.24E-07	13.50
5.05E-07	13.36
4.89E-07	13.22
4.75E-07	13.08
4.65E-07	12.95
4.57E-07	12.81
4.42E-07	12.68
4.34E-07	12.55
4.22E-07	12.42
4.18E-07	12.29
4.08E-07	12.16
3.91E-07	12.03
3.85E-07	11.91
3.73E-07	11.78
3.59E-07	11.66
3.48E-07	11.53
3.41E-07	11.41
3.33E-07	11.29
3.13E-07	11.17
2.94E-07	11.06
2.75E-07	10.94
2.58E-07	10.82
2.43E-07	10.71
2.27E-07	10.59
2.10E-07	10.48
1.92E-07	10.37
1.78E-07	10.26
1.65E-07	10.15
1.52E-07	10.04
1.47E-07	9.95
1.40E-07	9.87
1.29E-07	9.80
1.21E-07	9.72
1.16E-07	9.64
1.14E-07	9.56
1.08E-07	9.48

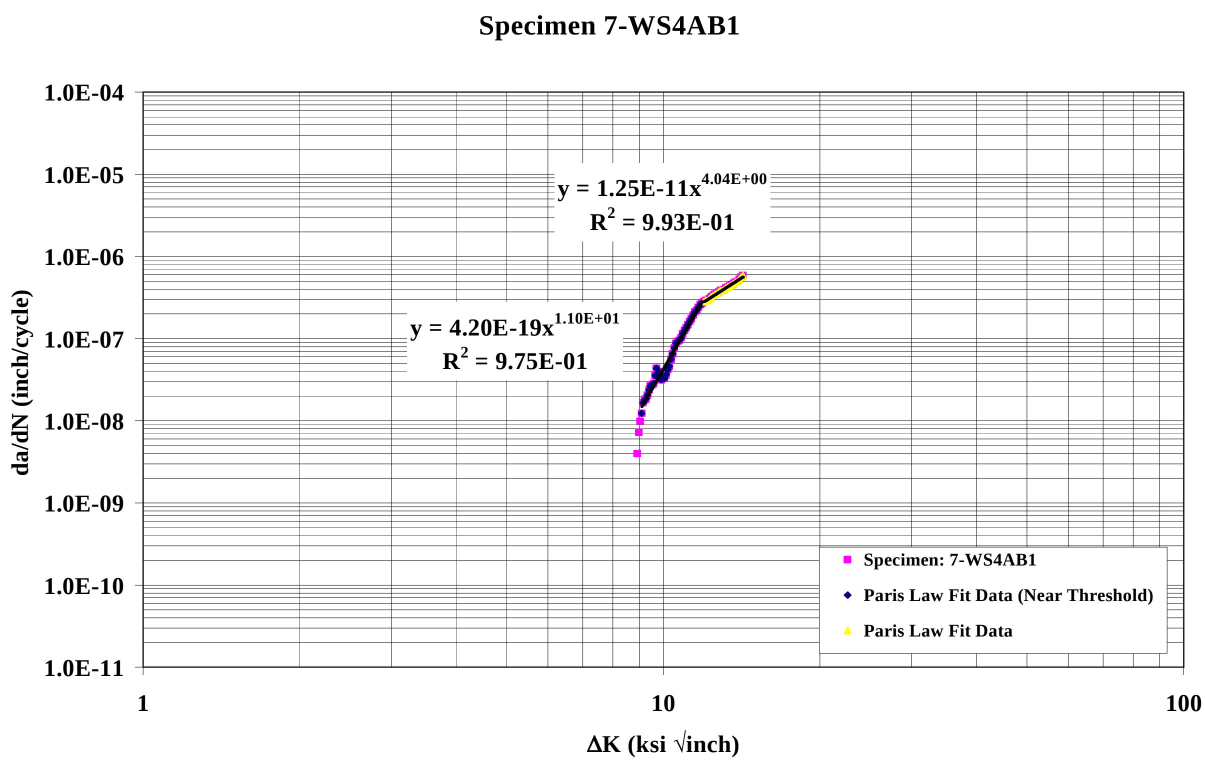
da/dN (inch/Cycle)	ΔK (ksi √ inch)
1.04E-07	9.42
9.54E-08	9.35
8.61E-08	9.27
8.37E-08	9.21
8.22E-08	9.15
8.00E-08	9.07
7.55E-08	9.02
6.98E-08	8.96
6.61E-08	8.89
6.42E-08	8.84
5.90E-08	8.77
5.66E-08	8.72
5.48E-08	8.67
5.33E-08	8.61
5.67E-08	8.57
5.46E-08	8.51
4.77E-08	8.46
4.24E-08	8.41
3.76E-08	8.36
3.25E-08	8.32
2.94E-08	8.28
2.87E-08	8.25
2.97E-08	8.21
2.96E-08	8.17
2.93E-08	8.13
2.97E-08	8.09
2.69E-08	8.06
2.69E-08	8.02
2.66E-08	7.99
2.44E-08	7.96
2.26E-08	7.92
2.17E-08	7.89
2.15E-08	7.87
2.06E-08	7.83
1.98E-08	7.80
1.93E-08	7.78
1.76E-08	7.75
1.70E-08	7.72
1.67E-08	7.70
1.60E-08	7.68
1.50E-08	7.65
1.31E-08	7.63

da/dN (inch/Cycle)	ΔK (ksi √ inch)
1.24E-08	7.61
1.26E-08	7.59
1.06E-08	7.58
9.17E-09	7.56
9.78E-09	7.54
1.08E-08	7.52
1.17E-08	7.51
1.34E-08	7.49
1.42E-08	7.47
1.27E-08	7.45
1.01E-08	7.43
1.06E-08	7.41
1.01E-08	7.40
8.93E-09	7.38
8.19E-09	7.36
7.59E-09	7.35
6.82E-09	7.34
6.59E-09	7.33
6.91E-09	7.32
6.54E-09	7.31
6.02E-09	7.29
5.66E-09	7.28
5.40E-09	7.27
4.56E-09	7.26
3.92E-09	7.25
4.93E-09	7.25
5.22E-09	7.24
4.61E-09	7.23
4.66E-09	7.22
5.06E-09	7.21
6.67E-09	7.20
8.64E-09	7.19
1.09E-08	7.17
9.85E-09	7.16
5.12E-09	7.15
2.10E-09	7.14
4.32E-10	7.14
1.27E-09	7.14
1.40E-09	7.14
1.79E-10	7.14
1.75E-10	7.13
2.71E-09	7.13


Specimen 10-WS1
y = 9.10E-10x^2.44E+00
R^2 = 9.98E-01
y = 1.45E-15x^8.02E+00
R^2 = 9.85E-01
1.0E-04
1.0E-05
1.0E-06
1.0E-07
1.0E-08
1.0E-09
1.0E-10
1.0E-11
da/dN (inch/cycle)
1
10
100
ΔK (ksi √inch)
Specimen: 10-WS1
Paris Law Fit Data (Near Threshold)
Paris Law Fit Data

Specimen 7-WS4AB1 (All Weld Metal)
Test Temperature: Lab Temperature (75 F°)
Test Load Ratio: R = 0.10
Test Frequency/Waveshape: 20 Hz./Sinusoidal
K-Decreasing Threshold da/dN (C = -2.00)

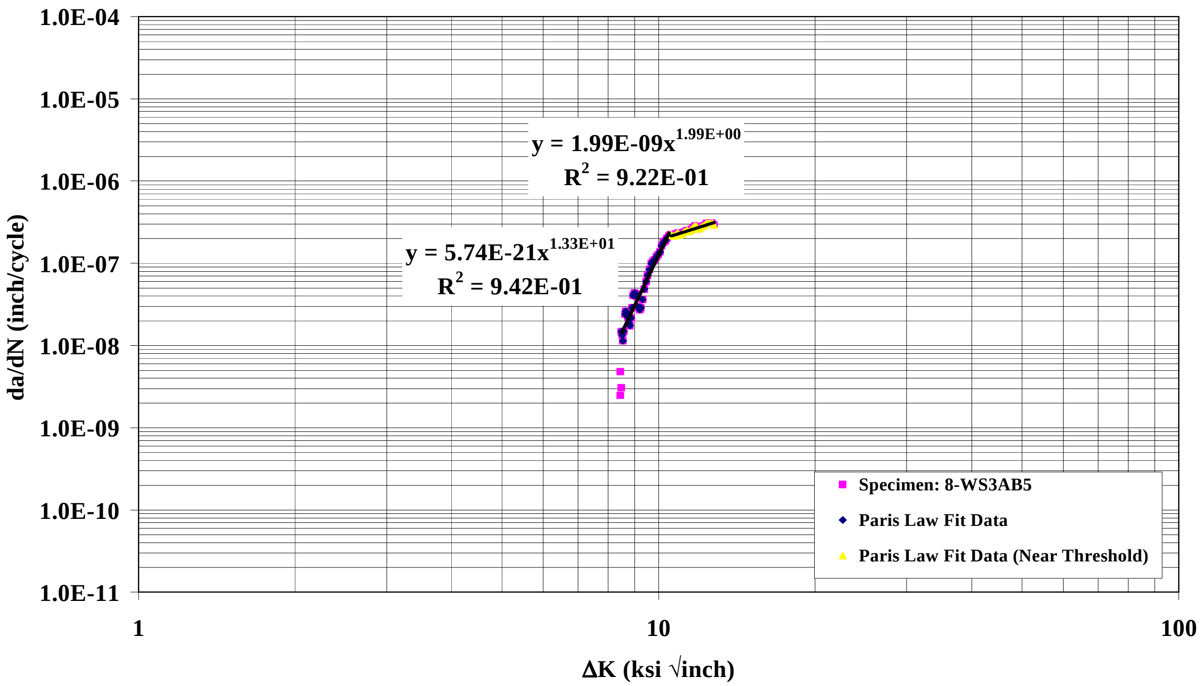
da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)
5.86E-07	14.23	3.35E-07	12.46	1.26E-07	10.98	3.94E-08	9.78
5.63E-07	14.11	3.22E-07	12.36	1.15E-07	10.90	4.36E-08	9.71
5.26E-07	14.00	3.05E-07	12.26	1.02E-07	10.82	3.54E-08	9.64
4.95E-07	13.88	2.93E-07	12.16	9.54E-08	10.74	2.83E-08	9.58
4.88E-07	13.76	2.91E-07	12.06	9.26E-08	10.66	2.74E-08	9.51
4.67E-07	13.65	2.80E-07	11.97	8.78E-08	10.58	2.66E-08	9.45
4.50E-07	13.53	2.68E-07	11.87	7.64E-08	10.50	2.37E-08	9.39
4.38E-07	13.42	2.58E-07	11.78	6.51E-08	10.42	2.05E-08	9.32
4.24E-07	13.31	2.41E-07	11.68	5.57E-08	10.35	1.83E-08	9.26
4.15E-07	13.19	2.22E-07	11.59	4.58E-08	10.27	1.79E-08	9.20
3.94E-07	13.09	2.12E-07	11.50	4.18E-08	10.20	1.66E-08	9.14
3.84E-07	12.98	1.94E-07	11.42	3.63E-08	10.13	1.23E-08	9.09
3.85E-07	12.87	1.75E-07	11.33	3.28E-08	10.05	9.88E-09	9.02
3.68E-07	12.76	1.62E-07	11.24	3.43E-08	9.99	7.19E-09	8.97
3.53E-07	12.66	1.48E-07	11.15	3.16E-08	9.92	3.98E-09	8.91
3.48E-07	12.56	1.36E-07	11.07	3.28E-08	9.84		


Specimen 7-WS4AB1
y = 1.25E-11x^4.04E+00
R^2 = 9.93E-01
y = 4.20E-19x^1.10E+01
R^2 = 9.75E-01
1.0E-04
1.0E-05
1.0E-06
1.0E-07
1.0E-08
1.0E-09
1.0E-10
1.0E-11
da/dN (inch/cycle)
1
10
100
ΔK (ksi √inch)
Specimen: 7-WS4AB1
Paris Law Fit Data (Near Threshold)
Paris Law Fit Data

Specimen 8-WS3AB5 (All Weld Metal)
Test Temperature: Lab Temperature (75 F°)
Test Load Ratio: R = 0.10
Test Frequency/Waveshape: 20 Hz./Sinusoidal
K-Decreasing Threshold da/dN (C = -2.00)

da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)
2.92E-07	12.78	2.37E-07	11.17	1.10E-07	9.83	2.17E-08	8.86
3.07E-07	12.68	2.26E-07	11.08	1.06E-07	9.75	1.75E-08	8.82
3.07E-07	12.57	2.21E-07	10.99	9.92E-08	9.68	1.87E-08	8.77
3.06E-07	12.46	2.33E-07	10.90	8.39E-08	9.61	2.20E-08	8.74
3.04E-07	12.35	2.27E-07	10.81	7.12E-08	9.53	2.55E-08	8.70
2.91E-07	12.25	2.15E-07	10.72	6.01E-08	9.46	2.62E-08	8.65
2.82E-07	12.14	2.17E-07	10.64	4.85E-08	9.39	2.39E-08	8.62
2.72E-07	12.04	2.23E-07	10.55	3.63E-08	9.32	1.48E-08	8.57
2.60E-07	11.94	2.16E-07	10.47	2.88E-08	9.25	1.14E-08	8.55
2.79E-07	11.84	2.02E-07	10.38	2.73E-08	9.20	1.33E-08	8.52
2.84E-07	11.74	1.88E-07	10.30	2.95E-08	9.15	1.47E-08	8.49
2.70E-07	11.64	1.80E-07	10.22	3.90E-08	9.10	3.07E-09	8.48
2.56E-07	11.55	1.64E-07	10.14	4.24E-08	9.05	2.47E-09	8.45
2.44E-07	11.45	1.38E-07	10.06	4.34E-08	8.99	4.80E-09	8.44
2.51E-07	11.36	1.27E-07	9.98	4.08E-08	8.94		
2.45E-07	11.26	1.20E-07	9.91	2.87E-08	8.89		

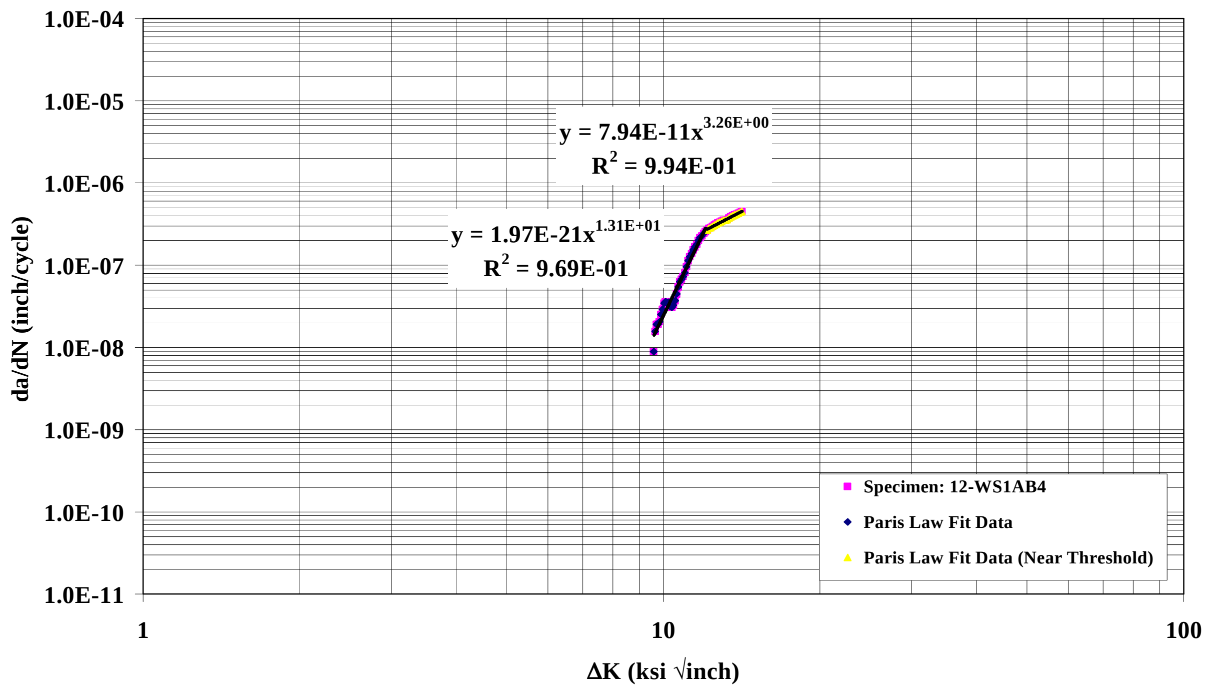
Specimen 8-WS3AB5



Specimen 12-WS1AB4 (All Weld Metal)
Test Temperature: Lab Temperature (75 F°)
Test Load Ratio: R = 0.10
Test Frequency/Waveshape: 20 Hz./Sinusoidal
K-Decreasing Threshold da/dN (C = -2.00)

da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)	da/dN (inch/Cycle)	ΔK (ksi √ inch)
4.52E-07	14.16	3.16E-07	12.63	1.39E-07	11.33	3.56E-08	10.24
4.41E-07	14.06	3.06E-07	12.53	1.27E-07	11.25	3.46E-08	10.17
4.28E-07	13.94	2.96E-07	12.43	1.14E-07	11.16	3.62E-08	10.10
4.19E-07	13.82	2.88E-07	12.33	9.62E-08	11.08	3.47E-08	10.03
4.10E-07	13.71	2.82E-07	12.24	8.13E-08	11.00	2.92E-08	9.96
4.02E-07	13.60	2.67E-07	12.14	7.28E-08	10.92	2.53E-08	9.90
3.87E-07	13.48	2.57E-07	12.05	6.80E-08	10.84	2.06E-08	9.83
3.74E-07	13.37	2.44E-07	11.96	6.29E-08	10.76	1.92E-08	9.77
3.60E-07	13.26	2.27E-07	11.86	5.44E-08	10.68	1.92E-08	9.71
3.52E-07	13.15	2.17E-07	11.77	4.47E-08	10.61	1.56E-08	9.64
3.53E-07	13.05	2.00E-07	11.68	3.69E-08	10.53	8.94E-09	9.58
3.44E-07	12.94	1.80E-07	11.59	3.26E-08	10.46		
3.33E-07	12.84	1.67E-07	11.50	3.06E-08	10.38		
3.28E-07	12.73	1.54E-07	11.42	3.17E-08	10.31		

Specimen 12-WS1AB4



Upon completion of the fracture toughness and fatigue testing, the CC Technology report was submitted to the Agency for review. Subsequently, the Agency requested additional information, which was provided by CC Technologies as follows:

Load-Displacement Curves for J-Fracture Toughness Tests

Figures 49 and 50 show the load-displacement curves provided by CC Technologies for all seven CT specimens; Figure 49 shows the full history of the testing, while Figure 50 only shows the initial stage of the testing. Figures 51 through 57 shown individual load-displacement curves for each specimen.

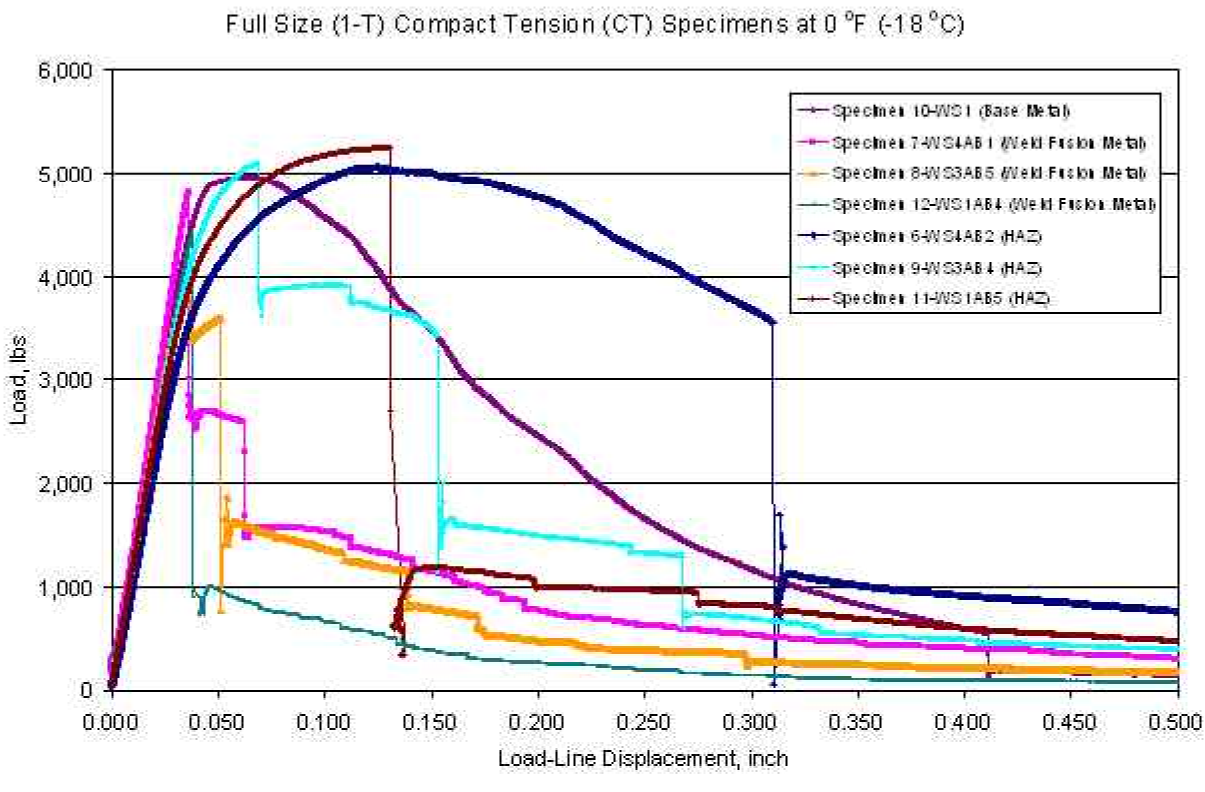


Figure 49. Load-Displacement Curve for Fracture Toughness Tests

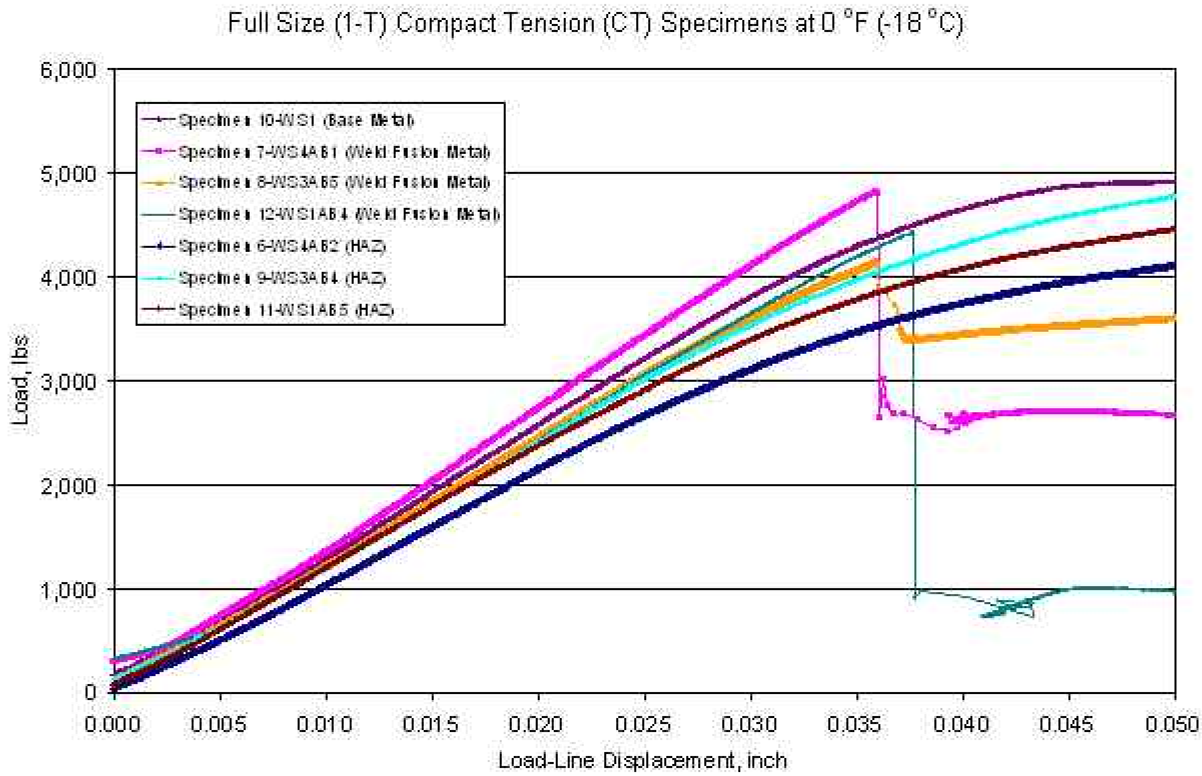


Figure 50. Load-Displacement Curves for Initial Stage of Fracture Toughness Tests

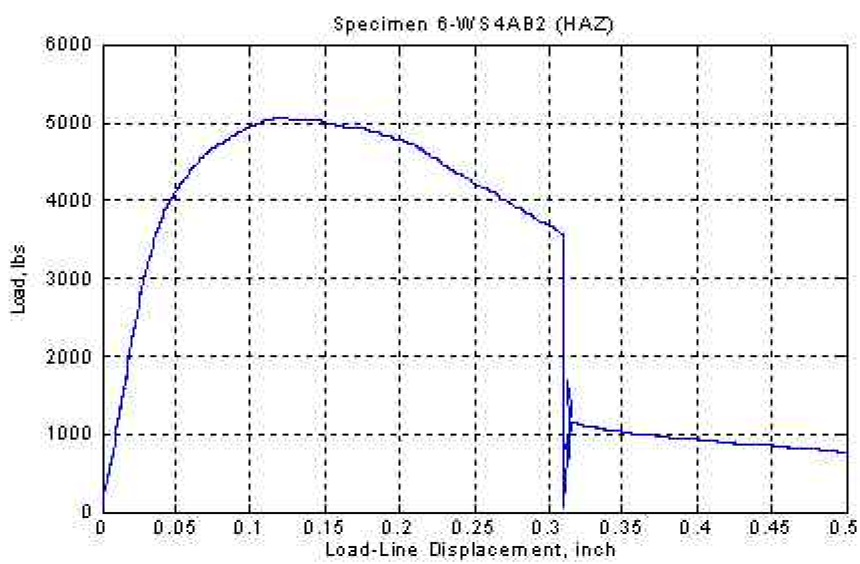


Figure 51. Load-Displacement Curve for Core No. 6. WS4-A-B-2 (HAZ)

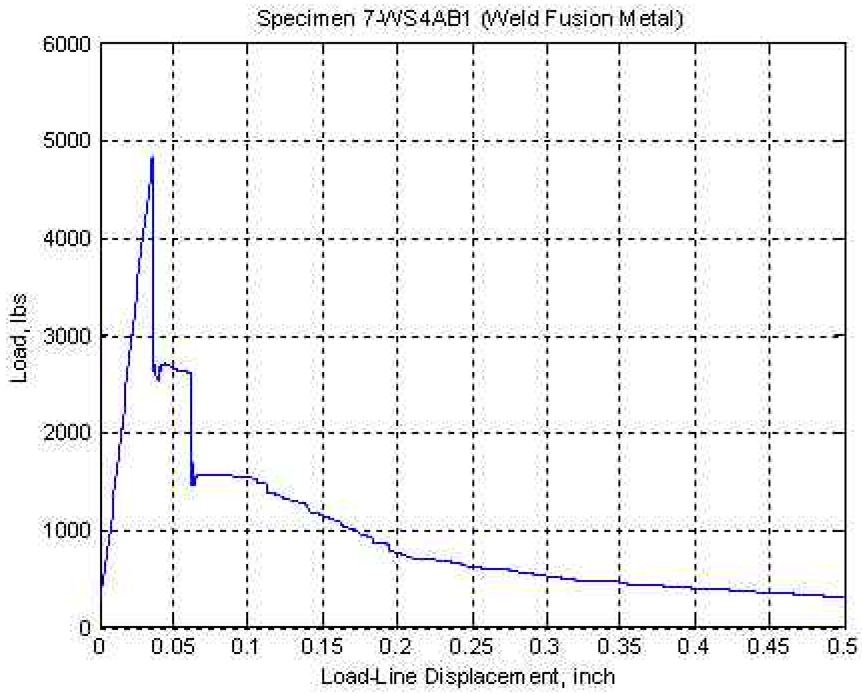


Figure 52. Load-Displacement Curve for Core No. 7. WS4-A-B-1 (AWM)

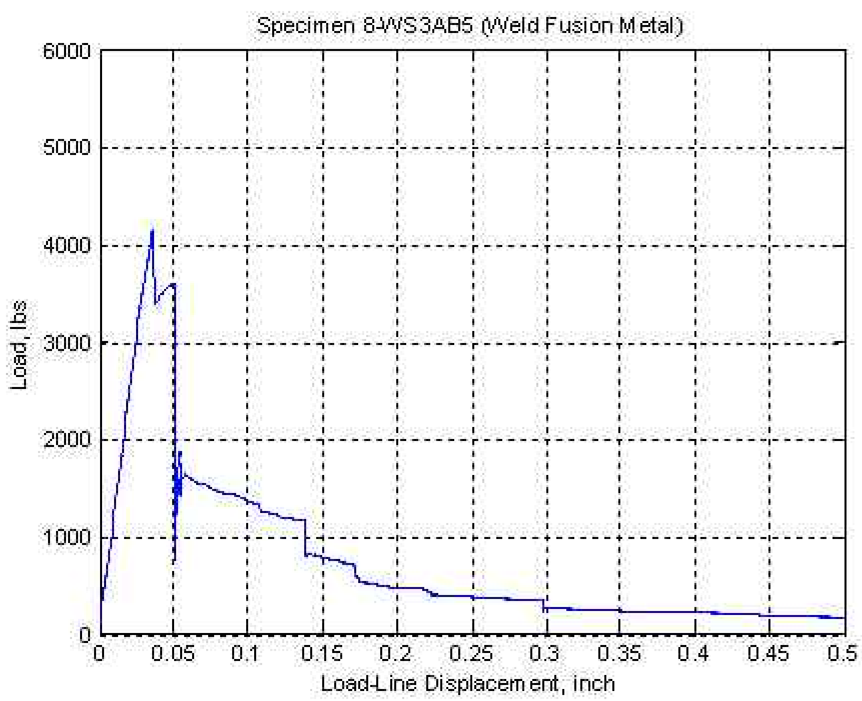


Figure 53. Load-Displacement Curve for Core No. 8. WS5-A-B-5 (AWM)

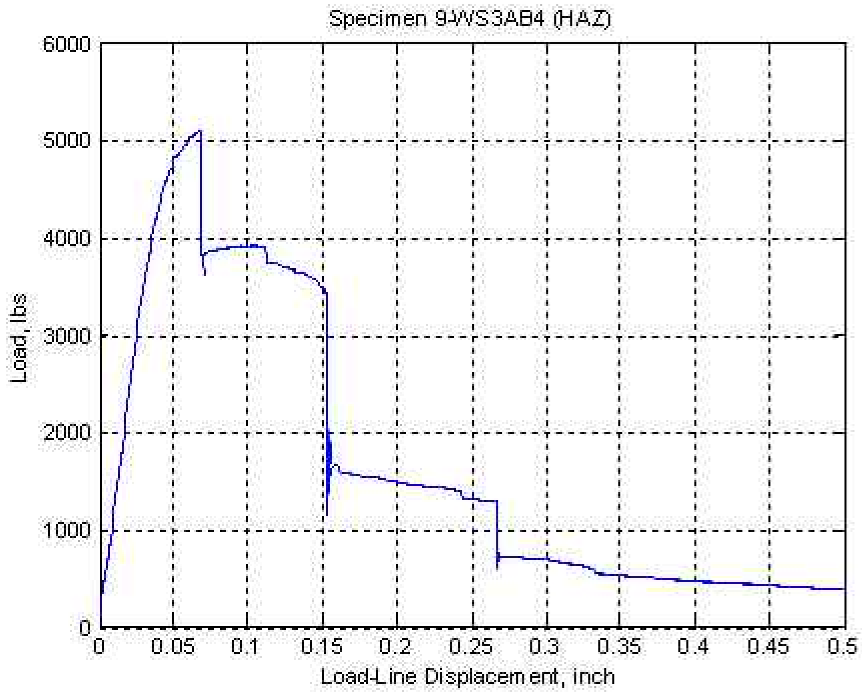


Figure 54. Load-Displacement Curve for Core No. 9. WS3-A-B-4 (HAZ)

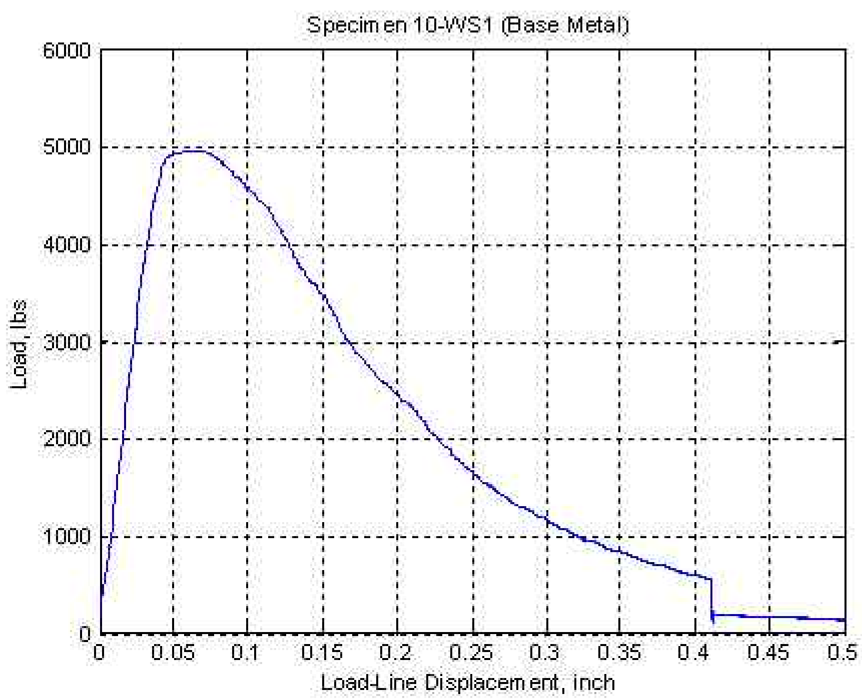


Figure 55. Load-Displacement Curve for Core No. 10. WS1 (Base Metal)

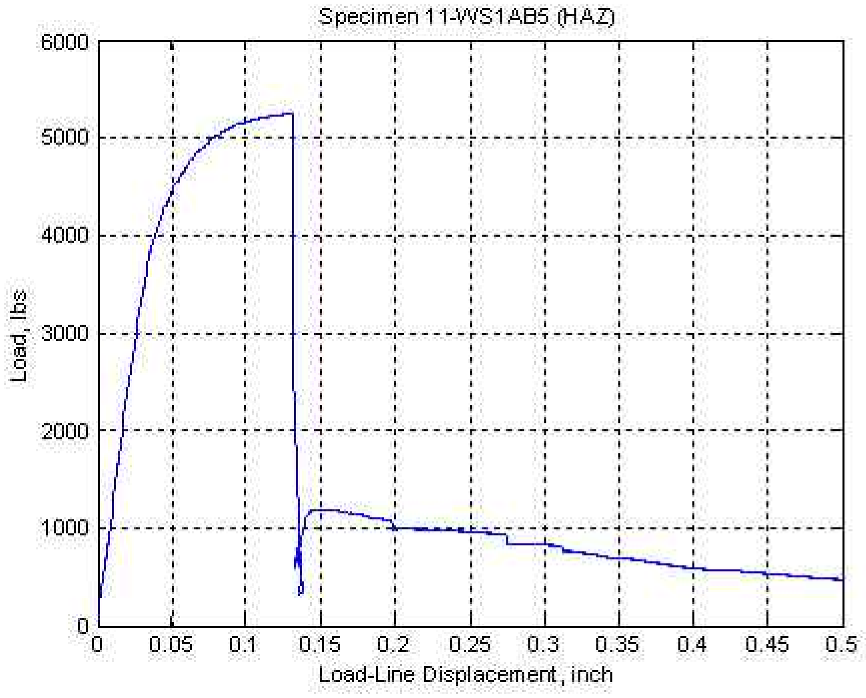


Figure 56. Load-Displacement Curve for Core No. 11. WS1-A-B-5 (HAZ)

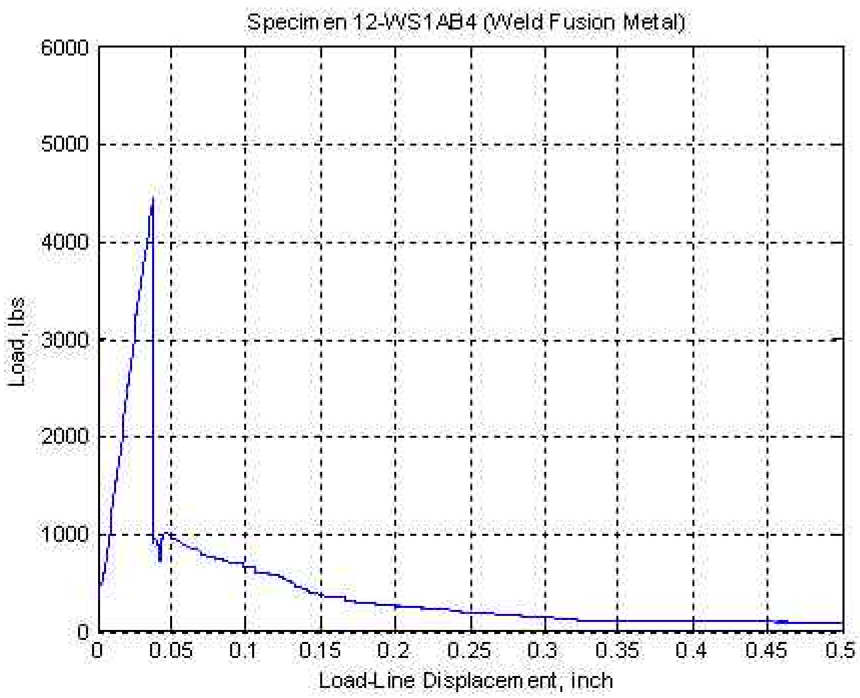


Figure 57. Load-Displacement Curve for Core No. 12. WS1-A-B-4 (AWM)

Based on these curves, CC Technologies calculated the maximum load, Pmax; the load at provisional critical stress, P(Q); and the stress intensity factor at P(Q), K(Q); as listed in Table 20.

Table 20. Parameters Derived From J-Fracture Toughness Test.

Maximum Load, lbs	Pmax	4,962	4,825	4,147	4,431	5,066	5,098	5,255
Crack Length at Pmax, inch	$(a)_{Pmax}$	1.3542	1.2933	1.3572	1.3688	1.4068	1.4184	1.4148
Crack ratio at Pmax	$(a/W)_{Pmax}$	0.6798	0.6508	0.6829	0.6877	0.7048	0.7100	0.7087
	$f(a/W)_{Pmax}$	19.4191	16.9181	19.7220	20.2079	22.1161	22.7527	22.5942
Stress intensity factor at K(max), ksi√in.	K(max)	76.68	64.96	65.08	71.25	89.16	92.30	94.48
Load at provisional stress K(Q), lbs	P(Q)	4,341	4,825	4,044	4,431	3,303	3,935	3,422
Crack Length at P(Q), inch	$(a)_{P(Q)}$	1.3499	1.2933	1.3581	1.3688	1.3928	1.4088	1.3921
Crack ratio at P(Q)	$(a/W)_{P(Q)}$	0.6776	0.6508	0.6833	0.6877	0.6978	0.7052	0.6974
	$f(a/W)_{P(Q)}$	19.2121	16.9181	19.7670	20.2079	21.3010	22.1622	21.2517
Stress intensity factor at P(Q), ksi√in.	$K_{P(Q)}$	66.36	64.96	63.61	71.25	55.99	69.40	57.87

Tensile Test Stress-Strain Curves

Figures 58 and 59 show the stress-strain relationships of the six tensile test specimens, with Figure 57 showing the full stress-strain relationship, and Figure 58 showing only the curves in the plastic area.

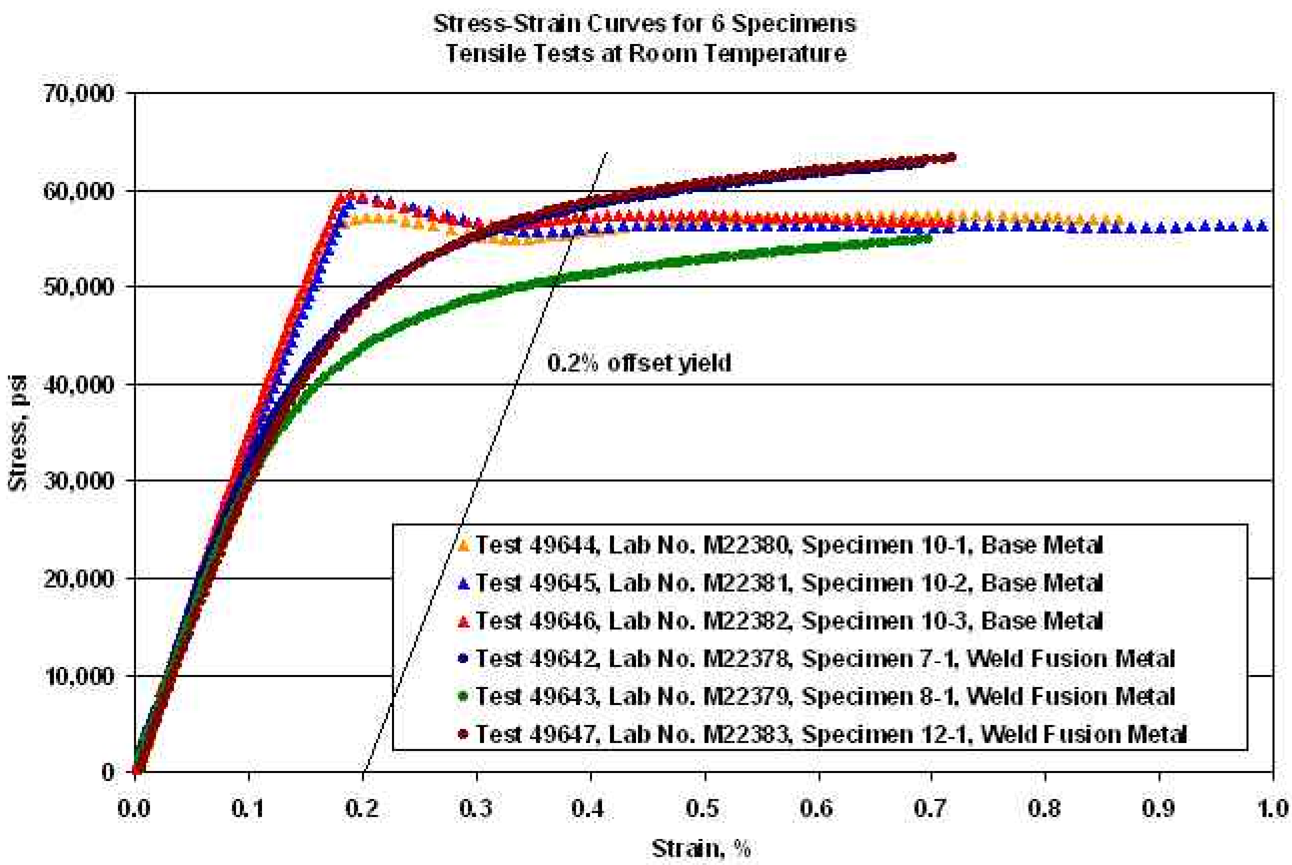


Figure 58. Stress-Strain Curves for Tensile Test Specimens (room temperature)

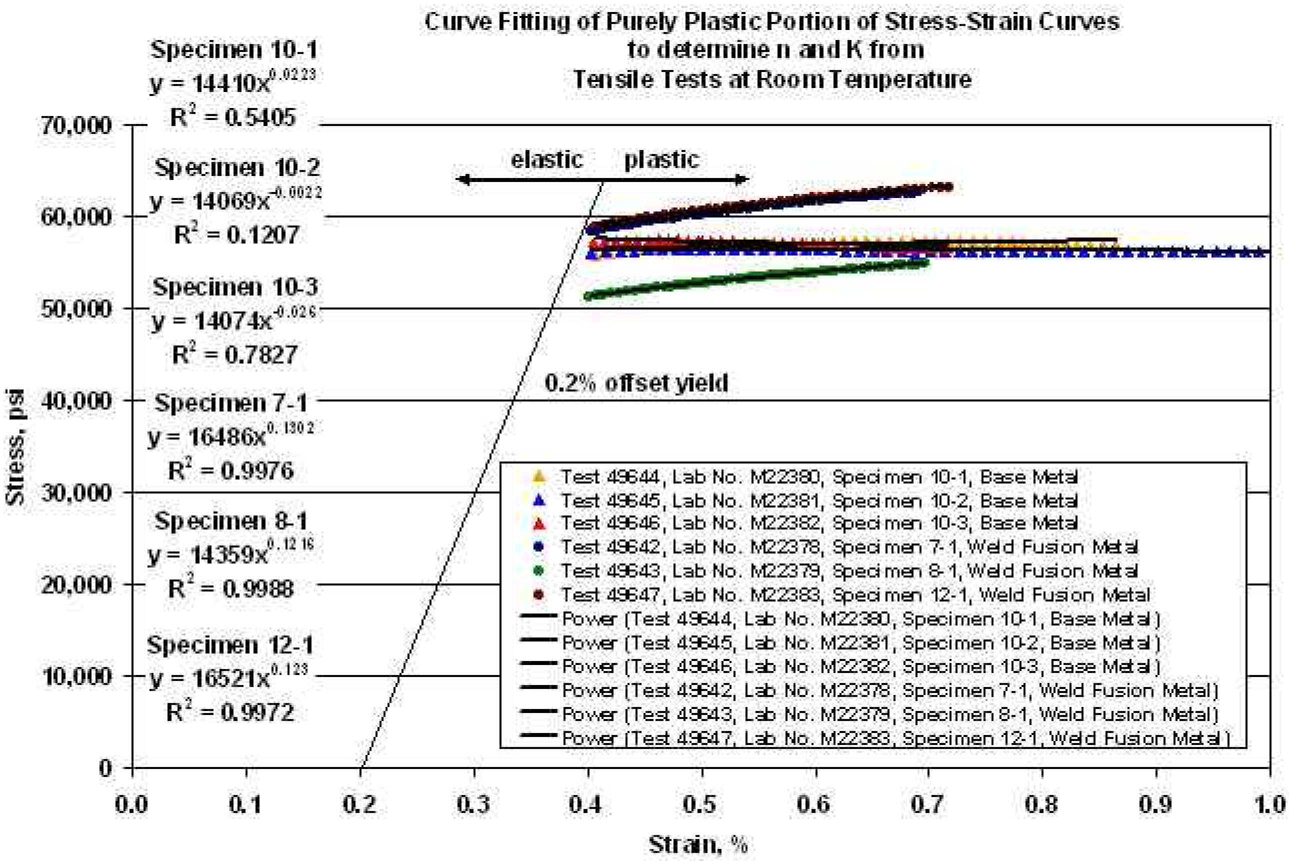


Figure 59. Stress-Strain Curves for Tensile Test Specimens in Plastic Region

Stress-Strain Equation Parameters

From the stress-strain curves for the tensile bars, CC Technologies regressed the parameters in the stress-strain behavior equation, $\sigma = K\epsilon^n$. In this equation, σ represents the applied stress on the material, ϵ is the strain, K is the strain hardening coefficient and n is the strain hardening exponent (also called the strain hardening index). The regressed value of n and K are shown in Table 21.

Table 21. Regressed Stress-Strain Equation Parameters.

	Core No.						Average	
	7	8	12	10-1	10-2	10-3	Weld Metal	Base Metal
	Weld Metal			Base Metal				
K	65,943	57,437	66,083	57,638	56,274	56,297	63,154	56,736
n	0.1302	0.1216	0.123	0.0223	-0.0022	-0.026	0.1249	-0.0020

TASK 5
NONDESTRUCTIVE TESTING

Scope of Work

Task 5 of WOC 5 was to perform both Ultrasonic testing (UT) and Radiographic testing (RT) on Agency selected weldments in the four Westside approach bridges involved in this study. The Work Order Contract (WOC) specified that twenty bottom flange welds were to be tested by both UT and RT and that twenty top flange welds were to be tested by UT only. Although not specifically designated in the WOC, per Agency instructions, these welds were evenly divided among the four bridges. Although also not specified in the WOC, all the inspected welds were selected at locations in tension.

The top flanges vary in width from approximately 14 to 20 inches, and the webs are welded to the flanges at about mid width of the flange. Thus, the flanges extend about 7 to 10 inches on the inside of the girder and about the same amount on the outside. The WOC specified that the ultrasonic inspection on the top flanges was to be done on both the portion inside the girder and the portion outside of the girder.

The bottom flanges are approximately 48 inches wide, with only about 1 inch on the outside of the girder beyond each web. The WOC specified that the ultrasonic inspection on the bottom flanges was to be done only on the portion inside the girder. (That is, the small portions of the flange on the outside were not specified to be inspected.)

The WOC specified that the radiographic inspection was to be done only on the bottom flanges, because the bridge deck is in direct contact with the top surface of the top flanges, which precludes access to the top side of the flanges. (Access to both sides is necessary for radiographic inspection. i.e., the film must be placed on one side, and the radioisotope source on the other.) The radiographic inspection was done on the entire length of the bottom flange welds. Just as with the UT inspection on the bottom flanges, the RT inspection was limited to the portion on the inside of the girder.

Ultrasonic Inspection

The ultrasonic inspection was preformed under subcontract by NDE Professionals, Inc., under the direct supervision of Mr. Nick McCoy of NDE Professionals. (Mr. McCoy is a Level II UT inspector and an ASNT Level III Certified RT and PT inspector). The inspection was done in accordance with the American Welding Society (AWS) Bridge Welding Code, AWS D1.5, except that, per Agency directive, the paint was not removed.

The inspection was by shear wave (angle beam) scanning using both a 70° and a 60°, 2.25 MHz transducer. The test instrument was a Panametrics Epoch™ 3 flaw detector. Prior to inspection, the surface of the inspection area was wiped clean to remove loose surface deposits, but otherwise, the inspection was done through the paint.

It is important to note that AWS D1.5 requires the paint to be removed for code inspections. However, because of the extent of grinding and repainting that would have been required, ODOT directed that the paint not be removed. To compensate for the presence of the paint, the scanning level gain was increased in accordance with the procedure in AWS D1.1-2004, Paragraph K6.1, whereby a reading is taken with pitch-catch angle beam transducers in an area with paint and another reading is taken next to that, where the paint has been removed in two small spots so the transducer can contact the bare steel. The amount of gain required to bring the UT signal in the painted area up to the same level as the signal in the bare area (in this case, 6 dB) is then added to the instrument for the angle beam inspection of the welds through the paint.

Radiographic Inspection

The radiographic inspection was done under subcontract by International Inspection, Inc., under the supervision of Mr. McCoy of NDE Professionals, Inc. The radiographs were taken using an Iridium 192 source, with the source on the inside of the girder and the film on the outside. Each of the inspected welds was radiographed in four, overlapping shots.

Inspection Access

Access to the outside of the bridge girders was provided by a 135-foot boom truck (rented by MEIC from United Rentals, Inc.). Access to the inside was via the manholes in the bridge deck, which were made available by closing the appropriate traffic lane in each bridge. Because the Westside approaches are part of the I-405 freeway system, access was restricted to the hours between 8:00 pm and 5:00 am. Traffic control was provided by Northwest Traffic Control, who was retained by the Agency, but whose activities were directed by MEIC.

The inspection access on Bridge 9268W (EW line) was significantly more restricted than on the other bridges, to the extent that the original inspection scope in the WOC was reduced and did not include either radiographic inspection of the flanges in that bridge or ultrasonic inspection of the external portion of the top flanges.

The access difficulty stems from the fact that Bridge 9268W is an upper deck bridge, directly above Bridge 9268E. Thus, in order to access the outside of the girders in Bridge 9268W it would have been necessary to close the freeway traffic lane below, on Bridge 9268E Although closing of a freeway traffic lane was already a necessity for accessing the inside of the girders on any of the bridges (through the manholes in the emergency lane), inspection of the welds in Girder E on the outside of Bridge 9268W would have required closing a traffic lane on both bridges. (i.e., closing Bridge 9268E for access to the underside of the bridge above, and closing Bridge 9268W, for access to the inside of the girders[7].) Additionally, the boom truck needed for access to the outside of the girder would need to be delivered and picked up each night, which, although possible would have entailed additional cost and would have further restricted the time available for inspection each night.

Consequently, in consultation with the Agency, the decision was made to inspect the welds on the inside of Bridge 9268W by UT only, and forego the radiographic inspection on the bottom flanges and the

---

[7]Bridges 9268E (lower deck) and 9268W (upper deck) are supported by encircling Ring Girders, which contain ladders in the vertical sections, thereby providing access from one bridge to the other. Thus, both bridges can be accessed by closing the traffic lane containing the access manholes in just one of the bridges. The access manholes in Bridge 9268E (lower deck) are in the slow lane, below Girder D in Bridge 9268W (upper deck). Thus, it would be possible to inspect Girder D in Bridge 9268W (upper deck) by closing just a single lane of traffic (i.e., the slow lane in Bridge 9268E (lower deck)). However, two traffic lanes would need to be closed in order to inspect Girder E in Bridge 9268W (upper deck), namely the fast lane in Bridge 9268E (lower deck) to provide access to the underside of Girder E in Bridge 9268W, and the fast lane in Bridge 9268W (upper deck) to provide access to the manholes and inside of the bridge.

external ultrasonic inspection on the upper flanges. Thus, no radiography was been on the bottom flange welds of Bridge 9268W (EW line), and the ultrasonic inspection on the top flange welds of that bridge was been limited to the portions on the inside of the girders.

One other weld was not fully inspected in accordance with the original WOC inspection scope. Specifically, one of the top flange welds on Bridge 9268B (WS Line) was UT inspected on the inside of the girder only, because power lines in the vicinity of the girder made inspection access on the outside hazardous.

Inspection Results

Of the 20 bottom flange welds and 20 top flange welds (six top flange welds, inside only) inspected ultrasonically, five welds–all on bottom flanges--had relevant ultrasonic indications per the reporting requirements of AWS D1.5. Of these, the indications on three of the welds were reportable but not rejectable per the requirements of AWS D1.5. The indications on the other two welds were rejectable per the requirements of AWS D1.5.

Of the 15 bottom flanges inspected radiographically, none had relevant or reportable indications. It is important to note that these 15 welds included the three welds with reportable-but-not-rejectable UT indications and one of the two welds with rejectable UT indications. That is, no reportable radiographic indications were found in any of the welds with UT indications which were also radiographed (amounting to 4 of the 5 welds with UT indications). Thus, the ultrasonic indications in these welds can likely be attributed to either paint-related coupling issues or non-significant discontinuities in ultrasonic properties, rather than actual flaws in the material. (see further discussion below)

The one weld with rejectable ultrasonic indications that has not been subjected to radiography was on Bridge 9268W (EW Line). Thus, these indications have not been confirmed or otherwise evaluated by radiographic inspection.

The weld in question was EW5-E-B-2, and it had two, separate, rejectable UT indications, one of which was 3 inches long and the other of which was 3 ½ inches long. One of the indications was about 1/32 inch from the outside surface of the girder and the other was about 1/16 inch from the outside surface. It is important to note here that all the ultrasonic indications found during the inspection were either at the outside surface or within 1/10 inch of the outside surface.

The ultrasonic signals from the two rejectable indications on weld EW5-E-B-2, as well those from the other reportable and/or rejectable indications found during the inspection, were broad and relatively indistinct in character, which suggests they were not due to cracks or similar, planer, crack-like features (such as incomplete fusion). Although it is possible that the indications were caused by shallow undercutting or other geometric features at the outside surface, we consider this somewhat unlikely for two reasons: for one, the broad nature of the indications is not consistent with the signal expected for a geometric feature, and two, given the surface finish requirements for the tension welds, it appears somewhat unlikely that surface geometric features sufficient to produce an ultrasonic signal would have been left in the structure during construction.

There are two other possible causes for the indications which we consider likely. One relates to the presence of the paint on the inside and outside surfaces. Specifically, the paint on the inside surface was rough and uneven, with drips and bubbles at some locations, which compromises the consistency of the UT inspection because of transducer "rock" and variations in sound attenuation. Additionally, the presence of paint on the outside surface of the girder suggests the possibility of "sound coupling" at the outside surface. This condition can occur if the interface between the paint and the metal is sufficiently tight to allow the sound to propagate into the paint layer and reflect back to the transducer.

Another possibility is the presence of repair welds at the toes of the electroslag welds. This possibility is suggested by the conditions we observed on the coupons removed from the bottom flanges during Task 3. That is, prior to removal of the coupons, the areas of the welds where the coupons were to be removed were inspected ultrasonically. The welds where two of the coupons were removed (weld WS3-A-B-5, Core No. 8 and weld WS3-A-B-4, Core No. 9) both contained continuous ultrasonic indications along the full length of the inspection area (approximately 12 inches). Although the magnitude of the indications was smaller than the indications observed on weld EW5-E-B-2, to the extent they were non-reportable, their appearance was similar to the indications observed during the present inspection, namely, they were broad and somewhat indistinct.

Subsequently, after the coupons had been removed from the girder and returned to our lab, we polished and etched the outside surface of the coupons (i.e., the outside surface of the flange) and the saw-cut surfaces (i.e., the cross sections through the flange). The toes of the electroslag welds contained repair welds that were approximately 1/8 inch deep by 1/4 inch wide and consisted of several overlapping (lengthwise) individual weld beads, likely made by the Shielded Metal Arc Welding (SMAW) process, as shown in Figure 60 (specimen was etched and photographed after removal of the fracture toughness test specimen).

Figure 60 (D72040-02) Core No. 8

Wet fluorescent magnetic particle testing of the coupons did not reveal any indications of flaws, either on the outside surface or on the saw-cut cross section. Thus, we consider it likely that the ultrasonic indications on those welds were due to metallurgical differences between the electroslag weld, the SMAW repair weld, and the girder base material.

Hence it is possible that the indications observed during the current inspections were due to similar conditions (i.e., repair welds). As noted above, this possibility is supported by the fact that on the four welds with ultrasonic indications that were also inspected radiographically, no indications were found in the radiographs. (Radiographic inspection would not reveal metallurgical differences between welds and base metal unless physical discontinuities were present, such as slag, porosity, cracks, or fusion defects.)

Weld Acceptance Criteria

The weld inspection results are summarized in Table 23. The footnotes to the table list the indications found by the ultrasonic inspection and identify whether they were acceptable or rejectable per the requirements of AWS D1.5. The rejectable indications are highlighted in red. The inspection report produced by NDE Professionals, Inc., from which Table 23 was derived, is on the page24 April 2007 following the table.

The acceptance/rejection criteria of AWS D1.5 are based on the length of the indication and the strength of the ultrasonic signal (or more precisely, the algebraic difference in decibels between the indication level and the reference level with correction for attenuation.) The acceptance/rejection criteria are given in Table 6.3 of AWS D1.5, whereby a "Flaw Severity Class" (A, B, C, or D) is assigned to each indication (for a particular thickness range of material), depending on the strength of the ultrasonic signal. Acceptance or rejection is then determined by the Flaw Severity Classification and the length of the indication, as shown in Table 22 (excerpted from AWS D1.5).

Table 22. AWS D1.5 Ultrasonic Inspection Acceptance Criteria

Flaw Severity Class	Acceptance/Rejection Criteria
A	Any indication in this category shall be rejected (regardless of length).
B	Any indication in this category having a length greater than 3/4 inch shall be rejected
C	Any indication in this category having a length greater than 2 inches in the middle half or 3/4 inch length in the top or bottom quarter of the weld thickness shall be rejected
D	Any indication in this category shall be accepted regardless of length or location in the weld

Table 23. Weld Inspection Results

Bridge	Line	Span	Top Flange (UT)			Bottom Flange (UT & RT)		
			Weld ID	Inside	Outside	Weld ID	UT	RT
9268A	SW	2	SW2-A-T1-8	NRI	NRI			
			SW2-B-T1-4	NRI	NRI			
			SW2-B-T1-5	NRI	NRI			
		4	SW4-A-T1-5	NRI	NRI			
			SW4-A-T2-5	NRI	NRI			
		3				SW3-A-B-2	NRI	NRI
						SW3-B-B-2	NRI	NRI
		4				SW4-A-B-2	NRI	NRI
						SW4-B-B-2	note [8]	NRI
		5				SW5-B-B-2	NRI	NRI
9268B	WS	1	WS1-A-T2-4	NRI	NRI			
		3	WS3-A-T1-5	NRI	NRI			
			WS3-B-T1-4	NRI	NRI			
			WS3-B-T2-4	NRI	note [9]			
		4	WS4-A-T2- 2	NRI	NRI			
		1				WS1-A-B-2	NRI	NRI
		3				WS3-A-B-2	note [10]	NRI
						WS3-B-B-2	note [11]	NRI
		5				WS5-A-B-1	NRI	NRI
						WS5-B-B-1	NRI	NRI

[8] **One Class D reportable (Acceptable) indication (2 1/4 inches long)**

[9] **Not inspected due to interference from power lines**

[10] **One Class C rejectable indication (1 1/4 inches long), one Class D reportable (Acceptable) indication (3/4 inch long); one Class C reportable (Acceptable) indication (5/8 inch long)**

[11] **One Class D reportable (Acceptable) indication (1/8 inch long)**

Table 23. Weld Inspection Results

Bridge	Line	Span	Top Flange (UT)			Bottom Flange (UT & RT)		
			Weld ID	Inside	Outside	Weld ID	UT	RT
9268	WE	3	WE3-D-T2-3	NRI	NRI			
			WE3-E-T1-3	NRI	NRI			
		4	WE4-D-T2-1	NRI	NRI			
			WE4-E-T1-1	NRI	NRI			
		6	WE6-D-T2-1	NRI	NRI			
		3				WE3-D-B-1	note [12]	NRI
						WE3-E-B-2	NRI	NRI
		4				WE4-D-B-2	NRI	NRI
						WE4-E-B-2	NRI	NRI
		6				WE6-D-B-2	NRI	NRI
9268W	EW	3	EW3-D-T1-3	NRI	not insp			
		5	EW5-D-T1-1	NRI	not insp			
			EW5-D-T2-1	NRI	not insp			
			EW5-E-T2-4	NRI	not insp			
		6	EW6-E-T2-1	NRI	not insp			
		3				EW3-D-B-3	NRI	not insp
						EW3-E-B-2	NRI	not insp
		5				EW5-D-B-2	NRI	not insp
						EW5-E-B-2	note [13]	not insp
		6				EW6-D-B-2	NRI	not insp

[12] **One Class D reportable (Acceptable) indication (1 inch long)**

[13] **One Class A rejectable indication (3 inches long), one Class B rejectable indication (3 ½ inches long)**

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December 13, 2006

MEI-CHARLTON, Inc.
7220 N. Lombard Rd.
Portland, Oregon

Attn: Rob Rutledge
Bob Hodel

Subject: ODOT- Freemont Bridge Ultrasonic and Radiographic Inspection of Selected Electroslag Welds.

I have completed the review of the Radiographs and Ultrasonic Inspection reports.
The summary of the findings are as follows.

Ultrasonic Inspection Results

20 Top flange Welds Ultrasonically Inspected.
20 Bottom Flange Welds Ultrasonically Inspected.
5 -Welds total with Indications.

SW4BB2: 1- Class D Reportable.
WE3DB1: 1- Class D Reportable.
EW5EB2: 1- Class A Rejectable, 1-Class B Rejectable.
WS3AB2: 1- Reportable Class D, 1-Reportable Class C, 1- Rejectable Class C.
WS3BB2: 1- Class D Reportable.

The Five (5) welds listed above, NPI is of the opinion that a number of factors are contributing to these near surface indications. These are uneven rough paint thickness with, bubbles and paint drips on the scanning surface. The probability of some localized weld repair during fabrication could contribute grain boundary reflectors between different welding processes. However at the time of this inspection this could not be verified through the paint. NPI is confident if the paint is removed the reinspection results would find no reportable or rejectable indications.

Radiographic Inspection Results

15 Bottom Flange Welds were selected for Radiographic Inspection.
No relevant or Reportable Indications were found. These include the above referenced Ultrasonic Welds with indications, except for Weld EW5EB2 which was not Radiographed.

Nicholas D. McCoy
ASNT L III # 48422-RT

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ULTRASONIC TEST REPORT

CUSTOMER: **MEI-Charlton, Inc.** DATE : **10-20-06**

DESCRIPTION: **Freemont Bridge I-405, SR-30 approaches** JOB#:

PART SN: **See Weld Numbers Below**

Technique

SPEC:	**AWS D1.5:2002**	ACCEPTANCE STANDARD:	**Table 6.3 Tension (Fracture Critical)**
PROCEDURE:	**TIP-UT-103.0**	TECHNIQUE:	**Table 6.2 from Face A**
EQUIPMENT:	**Epoch III**	SERIAL # :	**95064807** CAL. DUE: **11-5-06**
CAL. BLOCK:	**IIW**	COUPLANT:	**Ultex**
TRANSDUCER	**Panametrics**	ANGLE/ FREQ.:	**0°,60°,70°**
MATERIAL:	**Carbon Steel**	MATERIAL THICKNESS:	**0.80" up to 2.85 inches**
JOINT TYPE:	**Butt-ESW**	WELD IDENTIFICATION:	**See below**

RESULTS

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	From X	From Y	Discontinuity. Evaluation	Remarks
SW2ATI8														No Indications/Outer Flange
SW2BT14														No Indications/Outer Flange
SW2BT15														No Indications/Outer Flange
SW4AT15														No Indications/Outer Flange
SW4AT25														No Indications/Outer Flange
WS4AT22														No Indications/Outer Flange
WE3DT23														No Indications/Outer /Inner Flange
WE3ET13														No Indications/Outer Flange
WE4DT21														No Indications/Outer Flange/Limited scan bolt
WE4ET11														No Indications/Outer/Inner Flange/Limited scan bolt
WE4EB2														No Indications//Limited scan Stiffener
WE6DT21														No Indications/Outer Flange/Limited scan bolt

Remarks: All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ Db was determined by procedure in AWS Ref: D1.1-2004- para. K6.1 .

UT Level II Technician: [signature] **Date:** 10/27/06

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ULTRASONIC TEST REPORT

CUSTOMER: **MEI-Charlton, Inc.** DATE : **10-18-06**

DESCRIPTION: **Freemont Bridge I-405, SR-30 approaches** JOB#: ______

PART SN: **See Weld Numbers Below**

Technique

SPEC: **AWS D1.5:2002** ACCEPTANCE STANDARD: **Table 6.3 Tension (Fracture Critical)**

PROCEDURE: **TIP-UT-103.0** TECHNIQUE: **Table 6.2 from Face A**

EQUIPMENT: **Epoch III** SERIAL # : **95064807** CAL. DUE: **11-5-06**

CAL. BLOCK: **IIW** COUPLANT: **Ultex**

TRANSDUCER **Panametrics** ANGLE/ FREQ.: **0°,60°,70°**

MATERIAL: **Carbon Steel** MATERIAL THICKNESS: **0.80" up to 2.85 inches**

JOINT TYPE: **Butt-ESW** WELD IDENTIFICATION: **See below**

RESULTS

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	From X	From Y	Discontinuity. Evaluation	Remarks
WS3AT15														No Indications/Outer Flange
WS3BT14														No Indications/Outer Flange

Remarks: All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ Db was determined by procedure in AWS Ref: D1.1-2004- para. K6.1 .

UT Level II Technician: [signature] **Date:** 10/27/06

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ULTRASONIC TEST REPORT

CUSTOMER: **MEI-Charlton, Inc.** DATE : **10-17-06**

DESCRIPTION: **Freemont Bridge I-405, SR-30 approaches** JOB#:

PART SN: **See Weld Numbers Below**

Technique

SPEC: **AWS D1.5:2002**

ACCEPTANCE STANDARD: **Table 6.3 Tension (Fracture Critical)**

PROCEDURE: **TIP-UT-103.0**

TECHNIQUE: **Table 6.2 from Face A**

EQUIPMENT: **Epoch III**

SERIAL # : **95064807** CAL. DUE: **11-5-06**

CAL. BLOCK: **IIW**

COUPLANT: **Ultex**

TRANSDUCER **Panametrics**

ANGLE/ FREQ.: **0°,60°,70°**

MATERIAL: **Carbon Steel**

MATERIAL THICKNESS: **0.80" up to 2.85 inches**

JOINT TYPE: **Butt-ESW**

WELD IDENTIFICATION: **See below**

RESULTS

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	From X	From Y	Discontinuity. Evaluation	Remarks
WS1AT2 4														No Indications/Outer Flange

Remarks: All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ Db was determined by procedure in AWS Ref: D1.1-2004- para. K6.1.

UT Level II Technician: [signature] **Date:** 10/27/06

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ULTRASONIC TEST REPORT

CUSTOMER: **MEI-Charlton, Inc.** DATE : **10-13-06**

DESCRIPTION: **Freemont Bridge I-405, SR-30 approaches** JOB#:

PART SN: **See Weld Numbers Below**

Technique

SPEC: **AWS D1.5:2002** ACCEPTANCE STANDARD: **Table 6.3 Tension (Fracture Critical)**

PROCEDURE: **TIP-UT-103.0** TECHNIQUE: **Contact Table 6.2 from Face A**

EQUIPMENT: **Epoch III** SERIAL # : **95064807** CAL. DUE: **11-5-06**

CAL. BLOCK: **IIW** COUPLANT: **Ultex**

TRANSDUCER **Panametrics** ANGLE/ FREQ.: **0°,60°,70°**

MATERIAL: **Carbon Steel** MATERIAL THICKNESS: **0.80" up to 2.85 inches**

JOINT TYPE: **ESW- Butt** WELD IDENTIFICATION: **See below**

RESULTS

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	From X	From Y	Discontinuity. Evaluation	Remarks
SW2AT18														No Indications/Inner flange
SW4AT15														No Indications/Inner flange
SW4AT25														No Indications/Inner flange
WE3ET13														No Indications/Inner flange
SW3AB2														No Indications
SW4AB2														No Indications
EW6ET21														No Indications/Inner flange
EW5ET24														No Indications/Inner flange
EW3EB2														No Indications
WE3EB2														No Indications
EW3DB3														No Indications
EW5DB2														No Indications
WS3AT15														No Indications/Inner flange
EW5DT21														No Indications/Inner flange
EW6DB2														No Indications
WE4DT21														No Indications/Inner flange/Limited scan bolt
WE6DT21														No Indications/Inner flange/Limited scan bolt

Remarks: All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ Db was determined by procedure in AWS Ref: D1.1-2004- para. K6.1.

Weld EW5-D-T2-4 – No Inspection due to through bolt holes on both sides of welds

UT Level II Technician: **Date:** 10/27/06

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ULTRASONIC TEST REPORT

CUSTOMER: **MEI Charlton, Inc.** DATE : **10/13/06**

DESCRIPTION **Electorslag Welds** JOB#:

PART SN: **405 Bridge Portland Oregon – Freemont Bridge**

Technique

SPEC:	**AWS D1.5 - 2002**	ACCEPTANCE STANDARD:	**Tension Criteria (Fracture Critical)**
PROCEDURE:	**TIP-UT-103.0**	TECHNIQUE:	**Contact Table 6.2 from Face A**
EQUIPMENT:	**Panametrics Epoch IV**	SERIAL #: **011362910**	CAL. DUE: **10/30/06**
CAL. BLOCK:	**IIW**	COUPLANT:	**UT-X**
TRANSDUCER	**Panametrics**	ANGLE/ FREQ.:	**0°,60°,70°**
MATERIAL:	**Carbon Steel**	MATERIAL THICKNESS:	**0.80" to 2.85"**
JOINT TYPE:	**BUTT**	WELD IDENTIFICATION:	**SEE BELOW**

RESULTS

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	From X	From Y	Discontinuity. Evaluation	Remarks
SW2 BT14														No Indications/Inner flange
SW2 BT15														No Indications/Inner Flange
SW3 BB2														No Indications
SW4 BB2	1	70	A	59	41.5	3	14.5	2-1/4"	2.429"	.831"	+1/4"	1-3/8"	Class D	Acceptable
SW5 BB2														No Indications
WE3 DB1	1	70	A	60.2	41.5	5	13.7	1"	3.348"	1.145"	+1/2"	1-3/4"	Class D	Acceptable
EW5 EB2	1	70	A	52.7	41.5	2	9.2	3-1/2"	2.185"	.747"	+3/8"	0	Class B	Rejectable
	2	70	A	52.4	41.5	3	7.9	3"	4.496"	.806"	0	43"	Class A	Rejectable
EW5 DT11														No Indications/Inner Flange
EW3 DT13														No Indications/Inner Flange
WE6 DB2														No Indications
WE4 DB2														No Indications

Remarks: All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ dB was determined by procedure in AWS Ref: D1.1-2004- para. K6.1 .

UT Level II Technician: [signature] **Date:** 10/27/06

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ULTRASONIC TEST REPORT

CUSTOMER: **MEI Charlton,Inc.** DATE : **10/12/06**

DESCRIPTION: **Electorslag Welds** JOB#:

PART SN: **405 Bridge Portland Oregon – Freemont Bridge**

Technique

SPEC: **AWS D1.5 - 2002** ACCEPTANCE STANDARD: **Tension Criteria (Fracture Critical)**

PROCEDURE: **TIP-UT-103.0** TECHNIQUE: **Contact Table 6.2 from Face A**

EQUIPMENT: **Panametrics Epoch IV** SERIAL #: **011362910** CAL. DUE: **10/30/06**

CAL. BLOCK: **IIW** COUPLANT: **ULTEX**

TRANSDUCER **Panametrics** ANGLE/ FREQ.: **0°,60°,70°**

MATERIAL: **Carbon Steel** MATERIAL THICKNESS: **0.80" to 2.85"**

JOINT TYPE: **BUTT** WELD IDENTIFICATION: **SEE BELOW**

RESULTS

Weld ID	Ind. #	Tran. Angle	Face	Ind. Lev.	Ref. Lev.	Atten. Factor	Ind. Rate	Length	Angular Distance	Depth From A	from X	From Y	Discontinuity. Evaluation	Remarks
WS3 AB2	1	70	A	65.9	45.1	7	13.8	¾"	4.297"	1.413	0	12-7/8"	Class D	Acceptable
	2	70	A	62.3	45.1	7	10.2	5/8"	4.262"	1.402	0	2-1/4"	Class C	Acceptable
	3	70	A	62.1	45.1	7	10	1-1/4"	4.285"	1.409	0	4"	Class C	Rejectable
WS3 BB2	1	70	A	60.1	45.1	4	11	1/8"	3.131"	.970	0	4-3/4"	Class D	Acceptable
WS1 AB2														No Indications
WS5 AB1														No Indications
WS5 BB1														No Indications
WS1 AT24														No indications inner flange
WS3 BT14														No indications inner flange
WS3 BT24														No indications inner flange
WS4 AT22														No indications inner flange

Remarks: **All weld surfaces are in the as painted condition (smooth to dried drips and bubbles) while UT inspection was performed. Sensitivity transfer correction value of adding 6+ dB was determined by procedure in AWS Ref: D1.1-2004- para. K6.1 .**

Level II Technician: [signature] **Date:** 10/27/06

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RADIOGRAPHIC TEST REPORT

CLIENT MEI

PAGE 1 OF 1

PROJECT FREMONT BRIDGE

DATE / TIME 10-19-06

REPORT No. P4505

PROCEDURE 11 12 4003 REVISION 6

P.O. NUMBER

JOB No. 9268

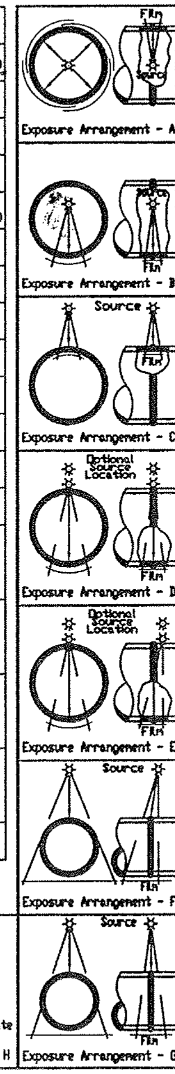
SPECIFICATION AWS D1.5-02

JOB DESCRIPTION RT BRIDGE MEMBERS

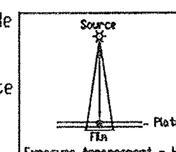
ACCEPTANCE CRITERIA AWS D1.5-02

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
WE3 DB1	A-B	/									
1.20" THK	B-C	/									
	C-D	/									
	D-E	/									
WE3 EB2	A-B	/									
0.83" THK	B-C	/									
	C-D	/									
	D-E	/									
WE4 DB2	A-B	/									
0.83" THK	B-C	/									
	C-D	/									
	D-E	/									
WE4 EB2	A-B	/									
0.80" THK	B-C	/									
	C-D	/								FA	
	D-E	/								FA	

Radiographic Technique	
X-Ray/Gamma	IR192
K.V./Source	72
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./Ci Min.	3:00
I.Q.I. Type (2) ASME	20
I.Q.I. Film Side/Source Side	
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	.80 — 1.2
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	PB



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	16

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER R. KOSKI LVII SIGNATURE [signature]
SNT-TC-1A

CLIENT/MANUFACTURERS REP. Nicholas D. McCoy L III SIGNATURE [signature]

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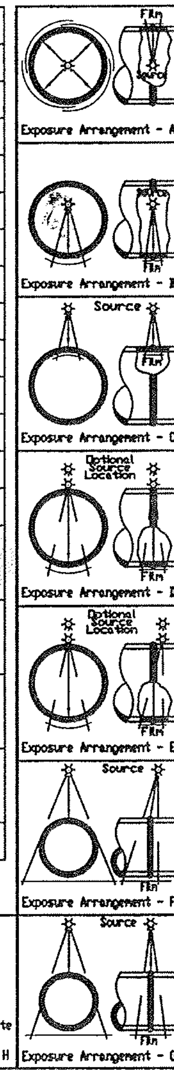
RADIOGRAPHIC TEST REPORT

CLIENT MEI
PAGE 1 OF 1
PROJECT FREMONT BRIDGE
DATE / TIME 10-20-06
REPORT No. P4505
PROCEDURE II 12 4003
REVISION 6
P.O. NUMBER
JOB No. 9268
SPECIFICATION AWS D1.5-02
JOB DESCRIPTION RT BRIDGE MEMBERS
ACCEPTANCE CRITERIA AWS D1.5-02

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
WE6DB2	A-B	✓									
9268E	B-C	✓									
0.80" THK	C-D	✓									
	D-E	✓									
NDM For Above											

Radiographic Technique	
X-Ray/Gamma	IR192
K.V./Source	72
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./Ci Min.	2.10
I.Q.I. Type (2) ASME	20
I.Q.I. Film Side/Source Side	Source
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	.85
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	Pb



Defect Code
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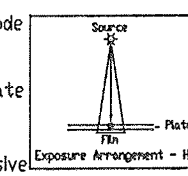
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	4

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER SNT-TC-1A R. KOSKI LVII SIGNATURE [signature]

CLIENT/MANUFACTURERS REP. Nicholas D McCoy LIII SIGNATURE [signature]

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RADIOGRAPHIC TEST REPORT

CLIENT: MEI

PAGE 1 OF 1

PROJECT: FREMONT BRIDGE

DATE / TIME: 10-19-06

REPORT No.: P4505

PROCEDURE: 11 12 4003

REVISION: 6

P.O. NUMBER:

JOB No.: 9268

SPECIFICATION: AWS D1.5-02

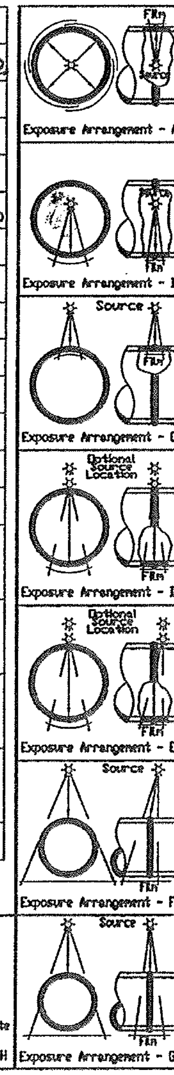
JOB DESCRIPTION: RT BRIDGE MEMBERS

ACCEPTANCE CRITERIA: AWS D1.5-02

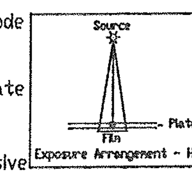
Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
WE3 DB1	A-B	✓									
	B-C	✓									
	C-D	✓									
	D-E	✓									
WE3 EB2	A-B	✓									
	B-C	✓									
	C-D	✓									
	D-E	✓									
WE4 DB2	A-B	✓									
	B-C	✓									
	C-D	✓									
	D-E	✓									
WE4 EB2	A-B	✓									
	B-C	✓									
	C-D	✓								FA	
	D-E	✓								FA	

Not shot

Radiographic Technique	
X-Ray/Gamma	IR192
K.V./Source	72
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./Ci Min.	3:00
I.Q.I. Type (2) ASME	20
I.Q.I. Film Side/Source Side	
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	.80 - 1.2
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	PB



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	16

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER SNT-TC-1A: R. KOSKI LVII

SIGNATURE: [signature]

CLIENT/MANUFACTURERS REP.: Nicholas D. McCoy LIII

SIGNATURE: [signature]

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RADIOGRAPHIC TEST REPORT

CLIENT MEI

PAGE 1 OF 1

PROJECT FREMONT BRIDGE

DATE / TIME 10-20-06 REPORT No. P4505

PROCEDURE 11 12 4003 REVISION 6

P.O. NUMBER

JOB No. 9268

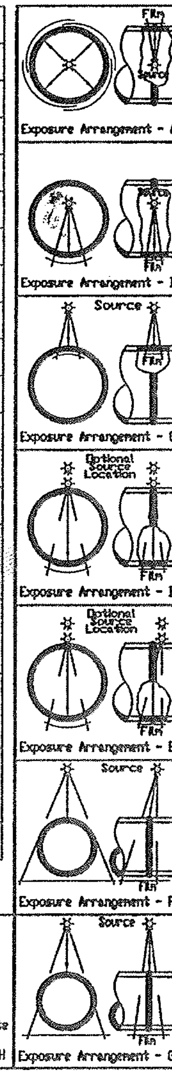
SPECIFICATION AWS D1.5-02

JOB DESCRIPTION RT BRIDGE MEMBERS

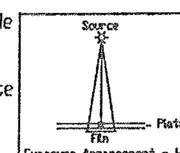
ACCEPTANCE CRITERIA AWS D1.5-02

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
WE6DB2	A-B	✓									
9268E	B-C	✓									
	C-D	✓									
	D-E	✓									

Radiographic Technique	
X-Ray/Gamma	IR192
K.V./Source	72
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./Ci Min.	2:10
I.Q.I. Type (2) ASME	20
I.Q.I. Film Side/Source Side	
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	.85
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	Pb



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	4

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER R. KOSKI LVII SIGNATURE [signature]
SNT-TC-1A

CLIENT/MANUFACTURERS REP. Nicholas D McCoy LII SIGNATURE [signature]

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RADIOGRAPHIC TEST REPORT

CLIENT: MEI
PAGE 1 OF 1

PROJECT: FREMONT BRIDGE
DATE / TIME: 10-16-06
REPORT No.: P4505

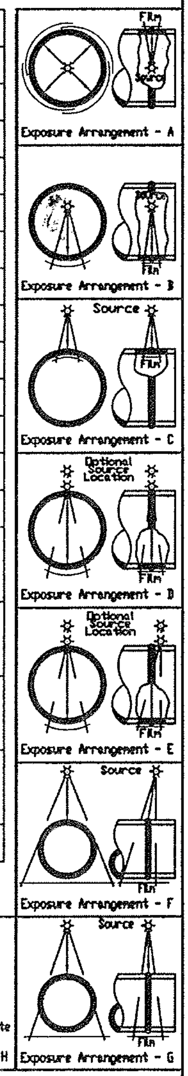
PROCEDURE: II 12 4003
REVISION: 6
P.O. NUMBER:
JOB No.: 9268B

SPECIFICATION: AWS D1.5-02
JOB DESCRIPTION: RT BRIDGE MEMBERS

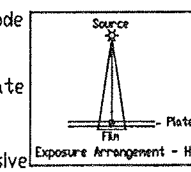
ACCEPTANCE CRITERIA: AWS D1.5-02

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
WS5 AB1	A-B	✓									
9268B	B-C	✓									
1.2" THK	C-D	✓									
	D-E	✓									
WS5 BB1	A-B	✓									
9268B	B-C	✓									
0.93" THK	C-D	✓									
	D-E	✓									
WS3AB2	A-B	✓									
1.50" THK	B-C	✓									
	C-D	✓									
	D-E	✓									
WS3 BB2	A-B	✓									
1.05" THK	B-C	✓									
	C-D	✓									
NDM for Above.	D-E	✓									

Radiographic Technique	
X-Ray/Gamma	IR192
K.V./Source	73
F-Spot/Size	.136
F.F.D./S.F.D.	26"
M.A.S./Ci Min.	3:00
I.Q.I. Type ASME (2)	20 25
I.Q.I. Film Side/Source Side	Source
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	1.2"
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	Pb



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	16

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER SNT-TC-1A: R. KOSKI LVII
SIGNATURE: [signature]

CLIENT/MANUFACTURERS REP.: Nicholas D McCoy L III
SIGNATURE: [signature]

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RADIOGRAPHIC TEST REPORT

CLIENT: MEI
PAGE 1 OF 1

PROJECT: FREMONT BRIDGE
DATE / TIME: 10-17-06
REPORT No.: P4505

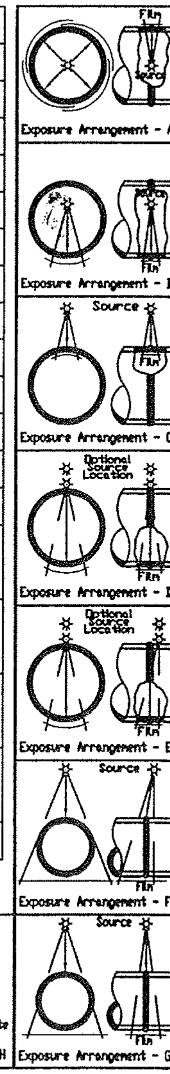
PROCEDURE: 11 124003
REVISION: 6
P.O. NUMBER:
JOB No.: 9268

SPECIFICATION: AWS D1.5-02
JOB DESCRIPTION: RT BRIDGE MEMBERS

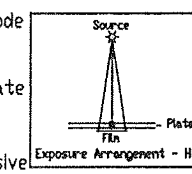
ACCEPTANCE CRITERIA: AWS D1.5-02

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
WS1AB2	A-B	✓									
9268B	B-C	✓									
1.10" THK	C-D	✓									
	D-E	✓									
SW3AB2	A-B	✓									
9268A	B-C	✓									
1.50" THK	C-D	✓									
	D-E	✓									
SW3BB2	A-B	✓									
9268A	B-C	✓									
1.20" THK	C-D	✓									
	D-E	✓									
NDM for above.											

Radiographic Technique	
X-Ray/Gamma	IR 192
K.V./Source	73
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./Ci Min.	3:00
I.Q.I. Type ASME (2)	20
I.Q.I. Film Side/Source Side	Source Side
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	1.1" – 1.5"
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	Pb



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	12

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER SNT-TC-1A: R. KOSKI LVII
SIGNATURE: [signature]

CLIENT/MANUFACTURERS REP.: Nicholas D McCoy
SIGNATURE: [signature]

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RADIOGRAPHIC TEST REPORT

PAGE 1 OF 1

CLIENT MEI

PROJECT FREMONT BRIDGE

PROCEDURE 11 12 4003 REVISION 6

SPECIFICATION AWS D1.5-02

ACCEPTANCE CRITERIA AWS D1.5-02

DATE / TIME 10-18-06

REPORT No. P4505

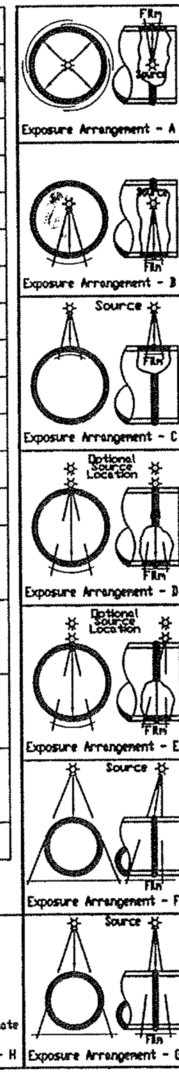
P.O. NUMBER

JOB No. 9268

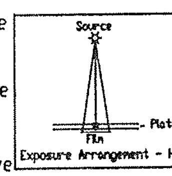
JOB DESCRIPTION RT BRIDGE MEMBERS

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
SW4AB2	A-B	✓									
1.05" THK	B-C	✓									
	C-D	✓									
	D-E	✓									
SW4 BB2	A-B	✓									
0.83" THK	B-C	✓									
	C-D	✓									
	D-E	✓									
SW5 BB2	A-B	✓									
0.83" THK	B-C	✓									
	C-D	✓									
	D-E	✓									
NDM for Above											

Radiographic Technique	
X-Ray/Gamma	IR192
K.V./Source	72
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./Ci Min.	3:00
I.Q.I. Type (2) ASME	20
I.Q.I. Film Side/Source Side	
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	.80
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	Pb



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	12

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER SNT-TC-1A R. KOSKI LVII SIGNATURE [illegible]

CLIENT/MANUFACTURERS REP. Nicholas D McCoy L.III SIGNATURE [illegible]

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RADIOGRAPHIC TEST REPORT

CLIENT: MEI
PAGE 1 OF 1

PROJECT: FREMONT BRIDGE
DATE / TIME: 10-17-06
REPORT No.: P4505

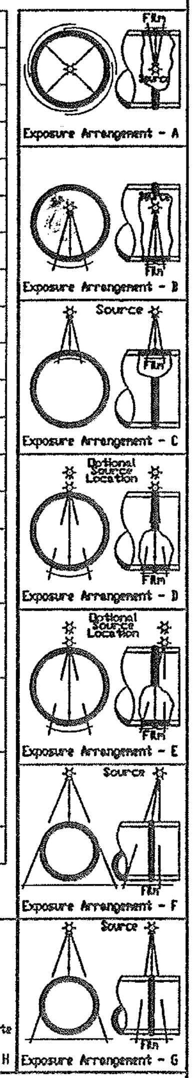
PROCEDURE: 11 12 4003
REVISION: 6
P.O. NUMBER:
JOB No.: 9268

SPECIFICATION: AWS D1.5-02
JOB DESCRIPTION: RT BRIDGE MEMBERS

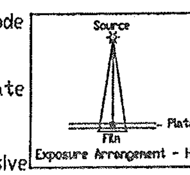
ACCEPTANCE CRITERIA: AWS D1.5-02

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
WS1AB2	A-B	/									
9268B	B-C	/									
	C-D	/									
	D-E	/									
SW3AB2	A-B	/									
9268A	B-C	/									
	C-D	/									
	D-E	/									
SW3BB2	A-B	/									
9268A	B-C	/									
	C-D	/									
	D-E	/									

Radiographic Technique	
X-Ray/Gamma	IR 192
K.V./Source	73
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./CI Min.	3:00
I.Q.I. Type ASME (2)	20
I.Q.I. Film Side/Source Side	Source Side
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	1.1" - 1.5"
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	25
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	Pb



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7X17	12

WE HEREBY CERTIFY THAT THE ABOVE INFORMATION WAS EXAMINED IN ACCORDANCE WITH THE SPECIFIED REQUIREMENTS, AND THAT THE RESULTS ARE THE ACCURATE INTERPRETATION OF THE UNDERSIGNED INSPECTOR TO THE BEST OF HIS/HERS KNOWLEDGE, ABILITY AND INTEGRITY.

FILM INTERPRETER SNT-TC-1A: R. KOSKI LVII
SIGNATURE: [signature]

CLIENT/MANUFACTURERS REP.: Nicholas D McCoy
SIGNATURE: [signature]

INTERNATIONAL INSPECTION, Inc.

Specialists in Nondestructive Examination

Los Angeles (562) 944-3166 Portland (503) 283-2668 Seattle (206) 766-8180

RADIOGRAPHIC TEST REPORT

CLIENT MEI

PROJECT FREMONT BRIDGE

PROCEDURE 11 12 4003 REVISION 6

SPECIFICATION AWS D1.5-02

ACCEPTANCE CRITERIA AWS D1.5-02

PAGE 1 OF 1

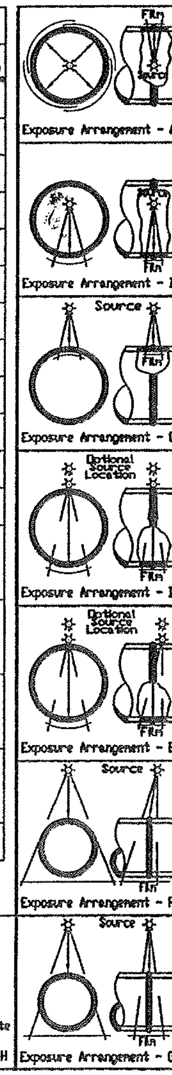
DATE / TIME 10-18-06 REPORT No. P4505

P.O. NUMBER JOB No. 9268

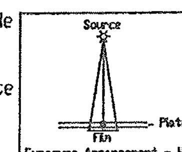
JOB DESCRIPTION RT BRIDGE MEMBERS

Identification	View	Accept	Reject	Crack	Inc. Pen.	Inc. Fusion	Porosity	Inclusion	Undercut	Density	Remarks
SW4AB2	A-B	✓									
	B-C	✓									
	C-D	✓									
	D-E	✓									
SW4 BB2	A-B	✓									
	B-C	✓									
	C-D	✓									
	D-E	✓									
SW5BB2	A-B	✓									
	B-C	✓									
	C-D	✓									
	D-E	✓									

Radiographic Technique	
X-Ray/Gamma	IR192
K.V./Source	72
F-Spot/Size	.136
F.F.D./S.F.D.	26
M.A.S./Ci Min.	3:00
I.Q.I. Type (2) ASME	20
I.Q.I. Film Side/Source Side	
Shims	N/A
Material	CS
Nominal I.D.	N/A
W.T./Sch	.80
Film Type AGFA	D7
Single/Double Load	S
Single/Double Viewed	S
Minimum Source to Object Distance	26
Maximum Distance Source Side of Object to Film	1
Applicable Arrangement	H
Front/Back Screen	Pb



Defect Code
1 Small
2 Moderate
3 Large
4 Excessive



Film Size	Quantity
7x17	12

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CLIENT/MANUFACTURERS REP. Nicholas D McCoy L.III SIGNATURE [signature]