Investigation of the June 3, 2006, Collapse of Grandview Triangle Bridge in Kansas City, MO

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REPORT

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REPORT

On June 3, 2006 at about 1:30 p.m., one construction employee was killed and another was injured when two spans of a bridge under demolition suddenly collapsed. The employees were part of a crew engaged in demolition of the last two spans of an 850' long steel bridge. The injured employee was treated and later released.

Description of the bridge:

A-2195 is a single-lane bridge connecting the westbound I-470 to the southbound US-71. It was constructed in 1974 and is owned by the Missouri Department of Transportation (MODOT). The bridge is a steel-framed structure with welded plate girders and steel floor beams. The bridge deck consisted of a 9" thick concrete slab composite with the plate girders through shear studs. The two girders had lateral and transverse bracings, generally consisting of steel angles. The bridge essentially ran in an east-west direction with multiple spans for a total bridge length of 850 ft. There were twelve spans of varying lengths. The main girders were supported over concrete piers, known as "bents". The east and west abutments were known as bent No.1 and bent No.13, respectively. There were eleven intermediate bents. The bents were sequentially numbered from the east abutment as No.1 to the west abutment as No.13. The overall width of the bridge measured from out to out of the rails was 27'-9". The clear width of the roadway was 25'-0". The steel girders' depths varied from 48" to 114" between the flanges. The bridge was generally designed as a continuous beam over bearings placed over concrete bents, with alternate fixed and expansion supports. The girders were not positively connected to the bents either at the fixed or the expansion bearings.

Demolition of the bridge:

In 2005, MODOT awarded a series of contracts to demolish old bridges and construct new bridges as part of a larger project called Grandview Triangle. MODOT awarded a contract to APAC-Kansas (APAC) to demolish the A-2195 bridge from the west abutment to approximately 49'-6" west of the bent No. 3. It included demolition of the entire bridge including bents.

MODOT awarded the contract to demolish the remaining spans (see figures 1 & 2) to a different company, Clarkson Construction (Clarkson) of Kansas. Clarkson retained APAC as a subcontractor to demolish the remaining bridge. Therefore, APAC acted as a prime contractor for the first nine spans and as a subcontractor for the last two spans.

Whether as a prime contractor or a subcontractor, APAC employed similar procedures to demolish the bridge. Using an excavator, also known as a track hoe, APAC first chipped away the entire width of the concrete deck, approximately 20' at a time; then two crew members flame-cut the exposed deck reinforcing bars. The next deck section was then chipped away and then the reinforcing steel was flame-cut, and the process continued until approximately a span and a half of the deck was cleared. Then the floor beams running transverse to the bridge girders were flame-cut, including all the lateral bracings. A location to flame-cut the girders was then selected, about 40'-45' away from the concrete bents, and the girders were then brought down by two cranes, each crane holding one girder.

Incident:

In the first contract as a prime contractor, APAC demolished the bridge, up to about 8'-10' west of bent No. 4. It also demolished the concrete deck up to bent No. 3. However, the steel girders, steel beams and bracings were still intact up to approximately 8' west of bent No. 4. Therefore, with the exception of the concrete deck, the bridge was still intact up to approximately bent No.4 at the end of the first contract.

On the evening of June 1, 2006, the work began to demolish the bridge from bent No. 4, proceeding east to bent No. 1. The crew worked all night and by the morning of June 2, 2006, they had removed the steel from bent No. 4 to approximately 49'-6" west of bent No. 3. As explained earlier, the steel floor beams and bracings were flame-cut and brought down and then the girders were flame-cut and brought down by two cranes, each crane hoisting one girder. The crew did not work late on June 2, 2006.

The crew began working on Saturday, June 3, 2006 at approximately 7:30 a.m. There were three APAC employees on the deck. One was the operator of the track hoe equipped with the hoe ram. The other two were with the reinforcing steel cutters. As stated earlier, on June 3, 2006, the steel framing of the bridge was intact from the east bent No. 1 through approximately 49'-6" west of bent No. 3. The concrete deck, however, extended only up to bent No. 3. The steel girders were cantilevering approximately 49'-6" west of bent No. 3. The operator began chipping away the concrete deck from bent No. 3, proceeding east for a depth of 15'-20' at a time. After each section of concrete was demolished, the two employees flame-cut the reinforcing steel. The operator then proceeded to cut the next section of the deck and the process continued until the operator had demolished the concrete deck up to approximately 50'-55' east of bent No. 3. The two employees were then to flame-cut the reinforcing steel. The first employee began preparing the torch, but the second employee, feeling thirsty, went to get a drink from his car parked on the bridge between bent No. 1 and bent No. 2. As the second employee stepped on to the span between bent No.1 and No. 2, and was walking to his car, the bridge between bent No. 2 and 3, and the bridge between bent No.1 and No. 2 suddenly collapsed. The span between bent No. 1 and No. 2 dropped down over the sloping embankment, killing the second employee. The span between bent No. 2 and No. 3 collapsed in a V-shape, approximately 80'-0" west of bent No. 2 (herein called failure point), trapping the first employee who was later rescued by the first responders. See figures 3 to 35 for the photographs of collapsed bridge girder.

Two days later, the hanging spans, including the concrete bents, were charged with explosives and imploded. The steel girders at the junction of the V-shape were salvaged and taken to APAC's storage yard for later examination.

The demolition contractor, APAC-Kansas, Inc., retained Stanley T. Rolfe, professor of civil engineering, University of Kansas in Lawrence, KS, as their structural engineering consultant after the collapse. APAC also contracted Robert S. Vecchio, principal of Lucius Pitkin, Inc. in New York, NY, to collect and test the steel samples from the failed structural members. On July 26, 2006, OSHA personnel met with APAC and his consultants at the storage site of the failed structural members. All parties agreed on a laboratory testing program and steel samples were collected on that same day. Multiple samples were tested, see attached report on pages 27 to 34.

On September 29, 2006, OSHA received the tabulated test results, without text, from APAC. The tests were conducted by Lucius Pitkin, Inc. Based on the review of the test results, we noted the following:

- Based on the scanning electron microscope examination, all fractured surfaces exhibited ductile overloading. No fatigue fractures were observed.
- For the top flange of the failed girder, the minimum yield strength was reported to be 40 ksi. The increase from 36 ksi to 40 ksi was probably due to the strain hardening effect which occurred during the collapse.

Engineering Analysis:

An analysis was done to determine whether the existing girder without the composite action of the concrete deck slab, but with cantilevers at each end, was adequate to support the loads placed over it immediately prior to the collapse. The span of the girder between bents No. 2 and No. 3 was considered to be 130' with an overhang on the east end of 3'-10" and 49'-6" at the west end. The cantilever at the west end was due to the fact that the girder was partially demolished up to 49'-6" west of bent No. 3. Under the conditions stated above, the girder was a statically determinate structure.

The load on the east cantilever was determined based upon the 9" thick concrete deck with 2" asphalt concrete topping, a ¼" thick coal tar, parapets and handrail between bent No. 1 and No. 2. Dead load of a 9" thick concrete deck with 2" asphalt concrete topping, parapets and handrails extending approximately 78'-10" west of bent No. 2 was considered. In addition, a weight of 72,800 pounds of track hoe uniformly distributed over its footprint was considered.

As discussed earlier, the concrete deck was already removed 78'-10" west of bent No. 2, which significantly reduced the flexural capacity of the girder. However, the west and east cantilever reduced the positive flexural moment of the girder to some extent.

Industry practice is to apply a load factor of 1.4 and a capacity reduction factor of 0.9 when evaluating the load carrying capacity, as per ASCE-37. Accordingly, factors of 1.4 and 0.9 were considered during our evaluation. We also evaluated the girder without these factors to determine the failure load.

The as-built drawings indicated the steel to be A-36. Tests conducted after the collapse, however, indicated a slightly higher yield strength of the top flange to be approximately 40 ksi. We evaluated using both yield values. The location of the track hoe was critical to the load carrying capacity of the girder. Conservatively, it was estimated that the track hoe weighing 72,800 pounds was located approximately near the mid-span between bent No. 2 and No. 3.

AISC LRFD method was used to verify the adequacy of the girder for the loads placed over it immediately before the collapse. The flexural moment at the failure point was calculated using STAAD.Pro 2005. Since the girder was statically determinate, prismatic section was used to compute the flexural demand. Deflections were not computed. The nominal flexural capacity of the girder was computed and then compared against the demand moment. The nominal moment was governed by the limit state of flange local buckling.

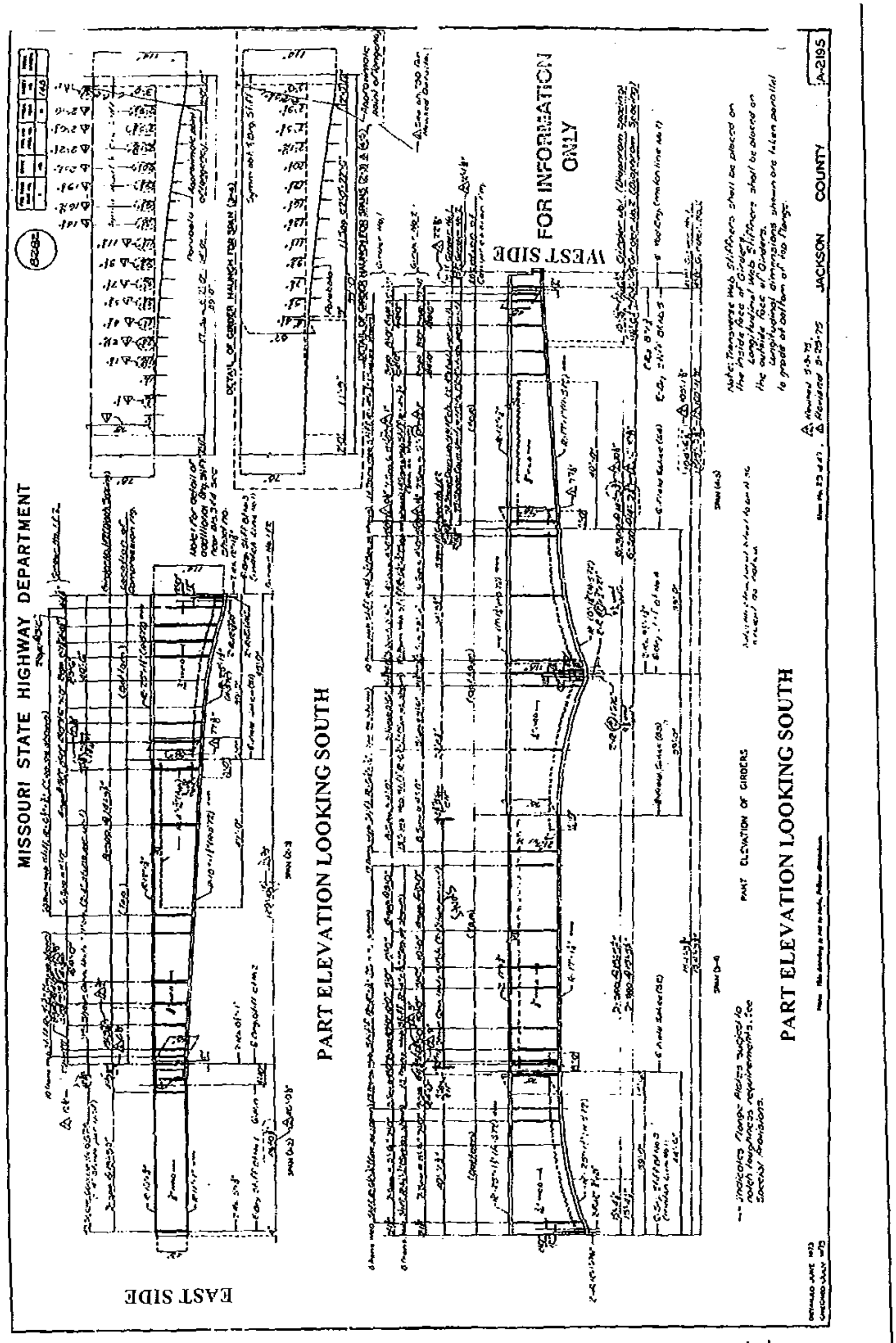
Computations indicated that considering the A-36 steel yield strength and the usual load and capacity reduction factors of 1.4 and 0.9, respectively, the stress on the girder was approximately 85% above the allowable value at the time of the collapse. Even if the load and capacity reduction factors were not considered, the stresses were 18% above the yield strength, indicating that failure could be imminent. It appears that the contractor failed to recognize the reduced load carrying capacity of the girder due to the removal of the concrete deck. The span between bent No. 2 and No. 3 was the longest span the contractor had encountered in this project. In addition to the longer span and the absence of the deck, the weight of the track hoe at its critical location contributed to the collapse.

Conclusion:

Based upon the above, we conclude that:

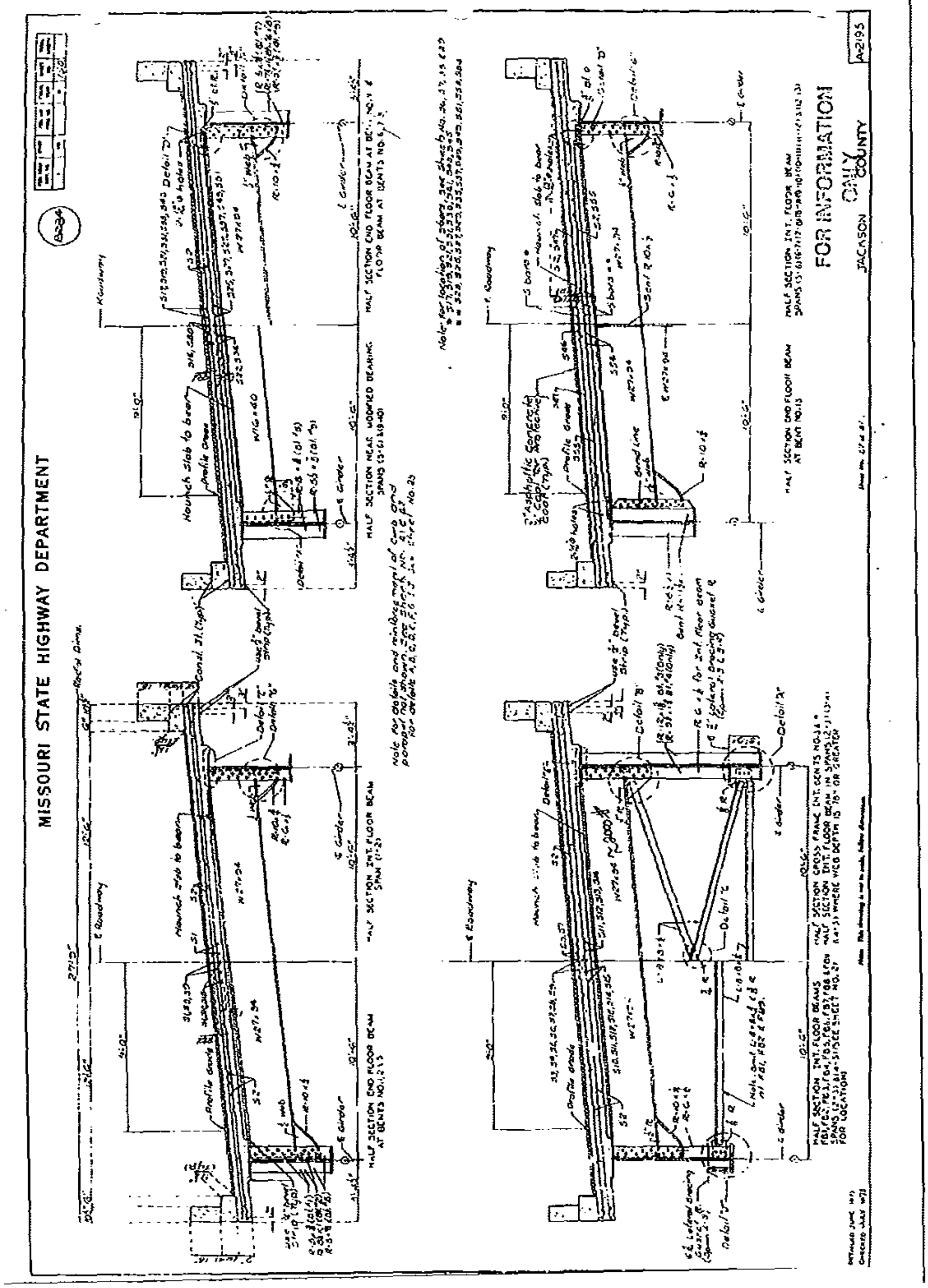
1. The demolition operation of the last two spans was carried out by the contractor in such a way that the structural member was overstressed beyond its failure load, which resulted in the bridge collapse.

2. Wind was not a contributing factor to the collapse of the bridge girder.



PARTIAL LONGITUDINAL ELEVATION OF BRIDGE GIRDER

FIGURE 1



CROSS SECTION OF BRIDGE GIRDER

<u>FIGURE 2</u>

FIGURE 3
(Looking north, collapsed south girder between bent no. 2 & 3)

FIGURE 4
(Looking south, collapsed girder framing west of bent no. 3)

FIGURE 5
(Looking south, collapsed bridge girder between bent no. 1 & 2)

FIGURE 6
(Looking south, collapsed girder framing bent no. 3)

FIGURE 7
(Looking south, collapsed bridge girder between bent no. 1 & 2)

FIGURE 8
(Looking north, collapsed bridge girders framing at bent no. 3)

FIGURE 9
(Looking north, V-shape collapse of bridge girder span between bent no. 2 & 3)

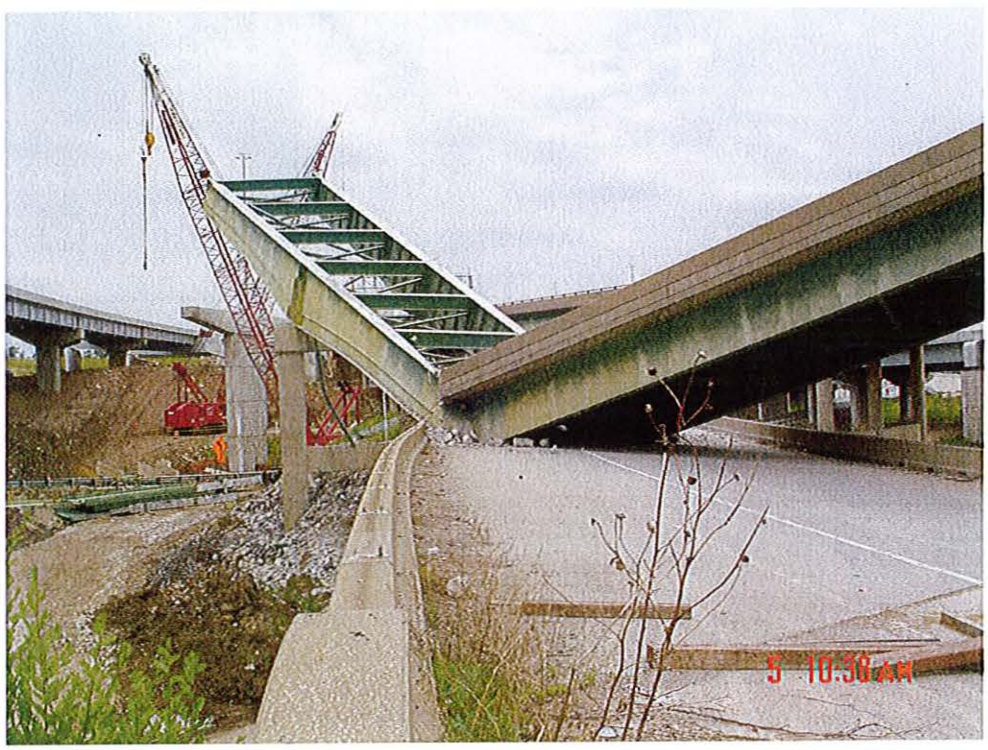


FIGURE 10
(Looking north, collapsed bridge girder span 2-3 fell on lower bridge)

FIGURE 11
(Looking north, bridge girder separated from bent no. 1 abutment)

FIGURE 12
(Looking north, bridge girder at span 1 separated from pinned connection near bent no. 2)

FIGURE 13
(Looking north, bridge girder separated from bent no. 1 abutment)

FIGURE 14
(Looking south, collapsed bridge girder span 2-3 fell on lower bridge into V-shape)

FIGURE 15
(Looking south, Separation of bridge girder near bent no. 2)

FIGURE 16
(Looking south, collapsed bridge girder span 2-3 fell on lower bridge into V-shape)

FIGURE 17
(Track hoe equipment used for demolition)

FIGURE 18
(Looking south, Separation of bridge girder near bent no. 2)

FIGURE 19
(damage to lower bridge girder framing due to upper bridge collapse)

FIGURE 20
(Damaged top flange of the bridge girder)

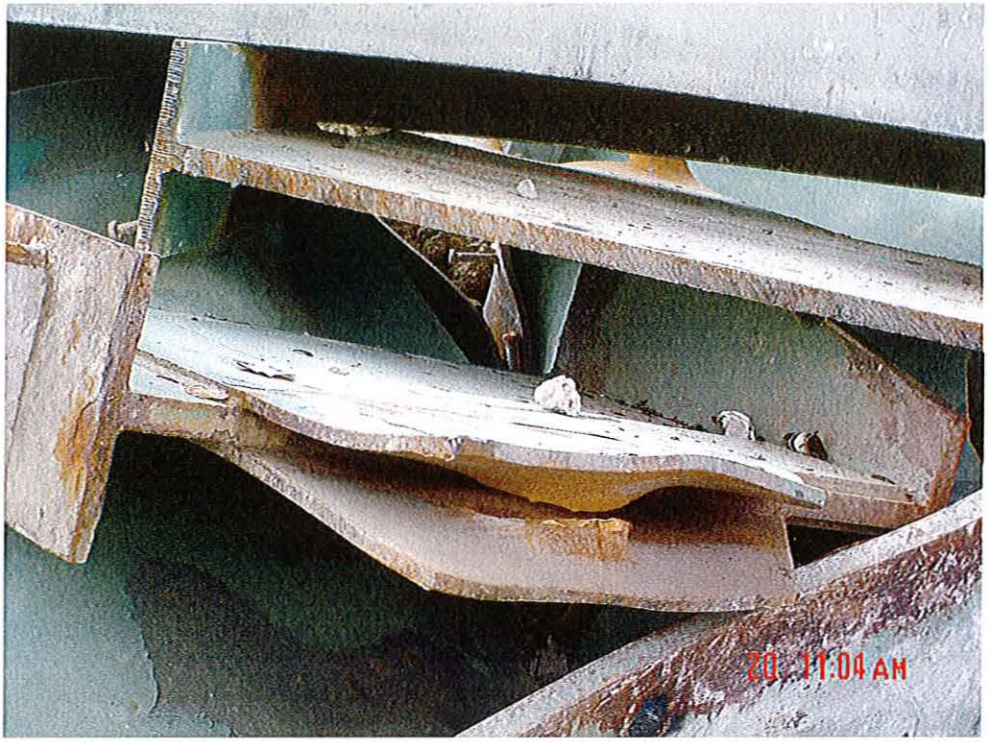
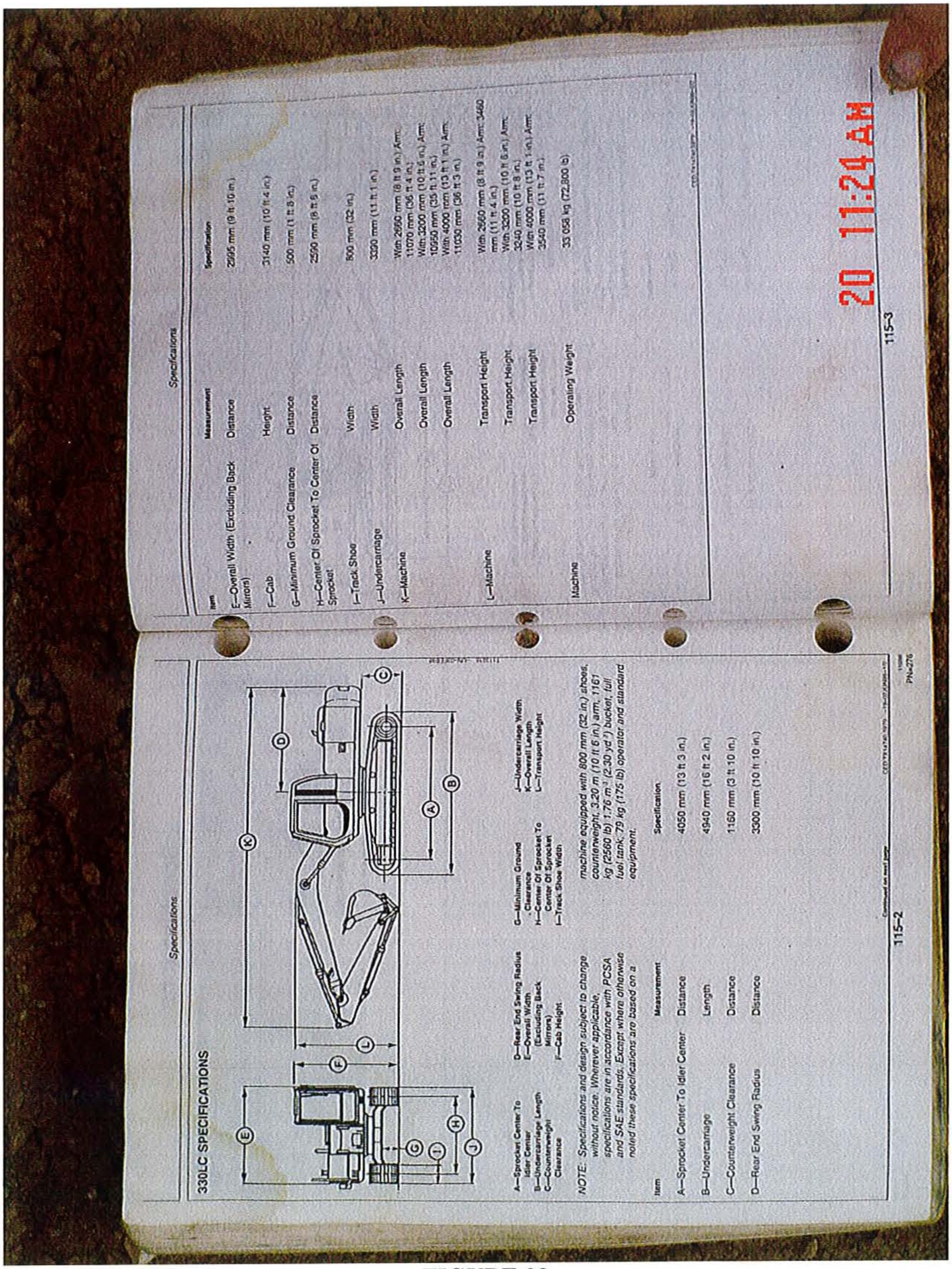


FIGURE 21
(Damaged top flange of the bridge girder)

FIGURE 22
(Damaged top flange of the bridge girder)


Specifications

330LC SPECIFICATIONS

A—Sprocket Center To Idler Center
B—Undercarriage Length
C—Counterweight Clearance
D—Rear End Swing Radius
E—Overall Width (Excluding Back Mirrors)
F—Cab Height
G—Minimum Ground Clearance
H—Center Of Sprocket To Center Of Sprocket
I—Track Shoe Width
J—Undercarriage Width
K—Overall Length
L—Transport Height

NOTE: Specifications and design subject to change without notice. Wherever applicable, specifications are in accordance with PCSA and SAE standards. Except where otherwise noted these specifications are based on a machine equipped with 800 mm (32 in.) shoes, counterweight, 3.20 m (10 ft 6 in.) arm, 1161 kg (2560 lb) 1.76 m³ (2.30 yd³) bucket, full fuel tank, 79 kg (175 lb) operator and standard equipment.

Item	Measurement	Specification
A—Sprocket Center To Idler Center	Distance	4050 mm (13 ft 3 in.)
B—Undercarriage	Length	4940 mm (16 ft 2 in.)
C—Counterweight Clearance	Distance	1160 mm (3 ft 10 in.)
D—Rear End Swing Radius	Distance	3300 mm (10 ft 10 in.)

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PN=276

Specifications

Item	Measurement	Specification
E—Overall Width (Excluding Back Mirrors)	Distance	2995 mm (9 ft 10 in.)
F—Cab	Height	3140 mm (10 ft 4 in.)
G—Minimum Ground Clearance	Distance	500 mm (1 ft 8 in.)
H—Center Of Sprocket To Center Of Sprocket	Distance	2590 mm (8 ft 6 in.)
I—Track Shoe	Width	800 mm (32 in.)
J—Undercarriage	Width	3390 mm (11 ft 1 in.)
K—Machine	Overall Length	With 2660 mm (8 ft 9 in.) Arm: 11070 mm (36 ft 4 in.)
	Overall Length	With 3200 mm (10 ft 6 in.) Arm: 10950 mm (35 ft 11 in.)
	Overall Length	With 4000 mm (13 ft 1 in.) Arm: 11030 mm (36 ft 3 in.)
L—Machine	Transport Height	With 2660 mm (8 ft 9 in.) Arm: 3460 mm (11 ft 4 in.)
	Transport Height	With 3200 mm (10 ft 6 in.) Arm: 3240 mm (10 ft 8 in.)
	Transport Height	With 4000 mm (13 ft 1 in.) Arm: 3540 mm (11 ft 7 in.)
Machine	Operating Weight	33 058 kg (72,800 lb)

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FIGURE 23
(Specification of 330LC Track Hoe)

FIGURE 24
(Twisted top flange of the girder)

FIGURE 25
(Twisted top flange of the girder)

FIGURE 26
(Twisted top flange of the south girder)

FIGURE 27
(Twisted top flange of the north girder)

FIGURE 28
(South girder bottom flange)

FIGURE 29
(North girder bottom flange)

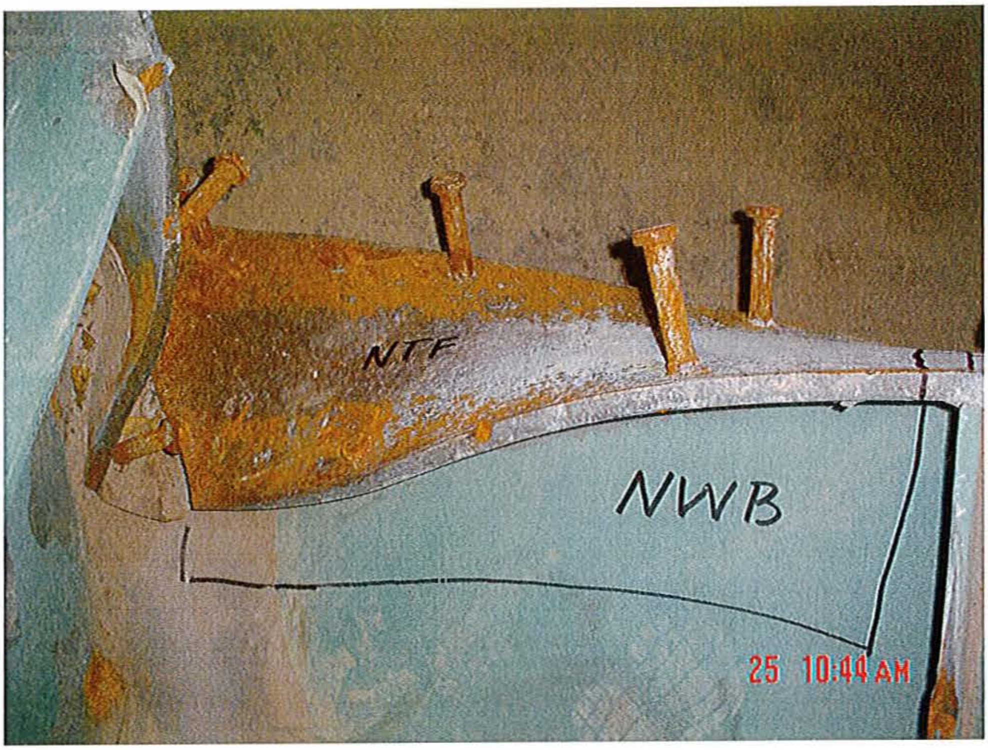

FIGURE 30
(North girder top flange with web)

FIGURE 31
((south girder top flange)

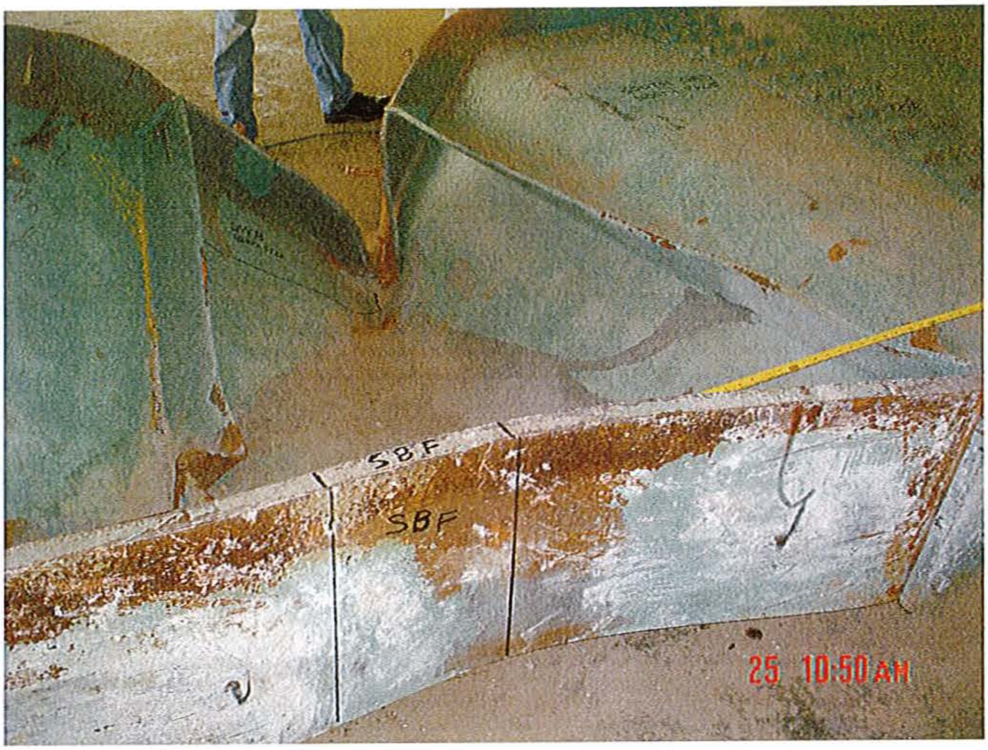

FIGURE 32
(South girder bottom flange)

FIGURE 33
(North girder south face bottom flange)

FIGURE 34
(North girder bottom flange)

FIGURE 35
(South girder bottom flange)

TABLE 1
SAMPLE IDENTIFICATION AND TEST SCHEDULE
GRANDVIEW TRIANGLE BRIDGE – KANSAS CITY

LPI Identification	Structural Member	Description	Sample Location	Tensile Test Specimens	CVN Specimens
1	NTF	North Girder at Failure Pt.	Top Flange	2	3
2	NWB	North Girder at Failure Pt.	Web	2	3
3	NBF	North Girder at Failure Pt.	Bottom Flange	2	3
4	STF	South Girder at Failure Pt.	Top Flange	2	3
5	SWB	South Girder at Failure Pt.	Web	2	3
6	SBF	South Girder at Failure Pt.	Bottom Flange	2	3

TABLE 2
COMPOSITIONAL ANALYSIS (Wt %)
GRANDVIEW TRIANGLE BRIDGE

Element	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
C	0.20	0.17	019	0.18	0.16	0.15
Mn	0.49	1.10	1.20	0.49	1.11	1.11
Cr	0.086	0.002	0.014	0.086	0.014	0.014
Mo	0.022	0.006	0.024	0.022	0.026	0.023
Ni	0.034	0.014	0.014	0.036	0.017	0.013
Cu	0.024	0.008	0.27	0.025	0.003	0.24
V	0.001	0.002	0.053	0.001	0.003	0.050
P	0.007	0.006	0.012	0.005	0.009	0.010
S	0.020	0.017	0.030	0.017	0.023	0.017
Si	0.077	0.24	0.22	0.077	0.26	0.21

HARDNESS SURVEY – GRANDVIEW TRIANGLE BRIDGE
Sketch of Hardness Survey

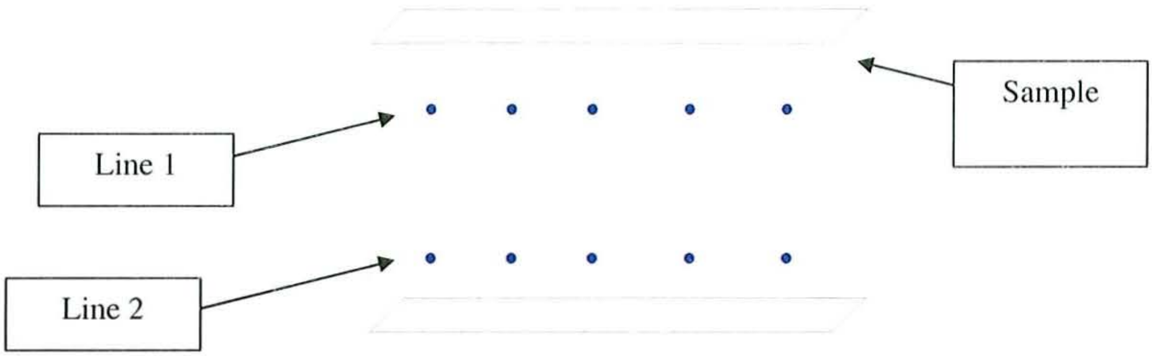

TABLE 3
HARDNESS SURVEY OF SAMPLE 1

Reading	Line1 (HRB)	Line 2 (HRB)
1	79.0	81.0
2	79.0	79.0
3	78.5	77.5
4	79.0	77.5
5	79.0	78.0
6	78.0	77.0
7	78.0	77.0
8	78.0	78.0
Average	***78.5***	***78.1***

Equivalent tensile strength line 1 = 70 ksi
Equivalent tensile strength line 2 = 70 ksi

TABLE 4
HARDNESS SURVEY OF SAMPLE 2

Reading	Line1 (HRB)	Line 2 (HRB)
1	80.0	79.5
2	80.0	79.0
3	79.0	79.0
4	80.5	80.0
5	79.5	79.5
Average	***79.8***	***79.4***

Equivalent tensile strength line 1 = 72 ksi
Equivalent tensile strength line 2 = 72 ksi

TABLE 5
HARDNESS SURVEY OF SAMPLE 3

Reading	Line1 (HRB)	Line 2 (HRB)
1	79.0	87.5
2	86.5	87.0
3	86.5	87.5
4	86.0	87.0
5	87.0	86.5
6	88.0	88.0
7	89.0	87.0
8	88.5	88.0
9	87.5	87.0
10	87.0	87.0
11	86.5	87.0
12	86.5	86.0
Average	***86.5***	***87.1***

Equivalent tensile strength line 1 = 84 ksi
Equivalent tensile strength line 2 = 84 ksi

TABLE 6
HARDNESS SURVEY OF SAMPLE 4

Reading	Line1 (HRB)	Line 2 (HRB)
1	72.5	74.5
2	74.0	76.0
3	75.0	74.5
4	75.0	76.0
5	74.5	75.0
6	75.0	77.0
Average	***74.3***	***75.5***

Equivalent tensile strength line 1 = 65 ksi
Equivalent tensile strength line 2 = 67 ksi

TABLE 7
HARDNESS SURVEY OF SAMPLE 5

Reading	Line1 (HRB)	Line 2 (HRB)
1	77.5	79.0
2	79.0	80.5
3	78.0	81.0
4	78.0	81.0
5	78.5	81.5
6	78.5	78.0
Average	***78.3***	***80.2***

Equivalent tensile strength line 1 = 69 ksi
Equivalent tensile strength line 2 = 72 ksi

TABLE 8
HARDNESS SURVEY OF SAMPLE 6

Reading	Line1 (HRB)	Line 2 (HRB)
1	82.0	90.0
2	84.0	89.5
3	84.5	89.5
4	85.0	89.0
5	85.5	90.0
6	86.5	89.0
7	87.0	88.0
8	86.0	89.0
9	86.0	90.0
10	86.5	90.0
11	86.0	88.0
12	85.5	87.0
Average	***85.4***	***89.1***

Equivalent tensile strength line 1 = 82 ksi
Equivalent tensile strength line 2 = 88 ksi

TABLE 9
TENSILE STRENGTH TESTING – GRANDVIEW TRIANGLE BRIDGE

Sample Identification	Yield Strength (0.2% offset) (ksi)	Ultimate Tensile Strength (ksi)	Elongation (%)	Reduction in Area (%)
1-1	40	65	32	59
1-2	40	64	31	60
2-1	50	74	33	--[a]
2-2	50	73	34	--[a]
3-1	51	81	27	27
3-2	49	81	29	29
4-1	43	66	29	60
4-2	46	66	30	61
5-1	50	73	27	--[a]
5-2	51	73	28	--[a]
6-1	48	79	28	62
6-2	56	80	29	61

Note: [a] Samples 2-1, 2-2, 5-1, 5-2 are flat specimens.

TABLE 10
CHARPY V-NOTCH IMPACT RESULTS
GRANDVIEW TRAINGLE BRIDGE

Specimen Identification	Size	Test Temp. (°F)	Absorbed Energy (ft-lb)	Lateral Expansion (in.)	Percent Shear
1-1	Standard size (10 mm x 10 mm x 55mm)	75	10	0.010	20
1-2		75	6	0.009	10
1-3		75	7	0.010	10
3-1		75	23	0.024	50
3-2		75	21	0.020	40
3-3		75	23	0.023	50
4-1		75	11	0.015	20
4-2		75	19	0.019	20
4-3		75	11	0.014	20
6-1		75	21	0.022	50
6-2		75	25	0.030	70
6-3		75	31	0.032	80
2-1	Sub size (10mm x 7.5mm x 55mm)	75	29	0.044	100
2-2		75	29	0.046	100
2-3		75	29	0.046	100
5-1		75	29	0.046	100
5-2		75	28	0.044	100
5-3		75	27	0.043	100

TABLE 11
FRACTOGRAPHY EXAMINATION – STEREO MICROSCOPE & SCANNING ELECTRON MICROSCOPE – GRANDVIEW TRIANGLE BRIDGE

Fractured Specimen	No. of Samples	Stereo Microscopy	Scanning Electron Microscopy [(a)]
NWB-F2	4	Ductile fracture	Ductile dimples-Microvoid coalescence - No fatigue
NWB-F3	2	Ductile fracture	"
NORTH GIRDER-FB	3	Ductile fracture	"
SWB-F2	3	Ductile fracture	"
SWB-F3	3	Ductile fracture	"
SOUTH GIRDER-FB	3	Ductile fracture	"

Note: [(a)] SEM examination of all fractured surfaces exhibited the ductile overloading. No fatigue was observed.