# APPENDIX H

## Bridge Assessments

# LUDLAM TRAIL CORRIDOR BRIDGE INSPECTION REPORT

Structure No: **Ludlam Bridge-1** Date: **9/19/2018**

MIAMI-DADE COUNTY

## ROUTINE INSPECTION REPORT

**Prepared by: Marlin Engineering, Inc.**

MARLIN
1700 NW 66 Avenue - Ste. 106
Phone: 305-477-7575

## CONTENTS OF REPORT



## PREPARED FOR: MIAMI-DADE COUNTY

### REPORT IDENTIFICATION

Bridge No.: N/A Topside Inspection Date: 9/19/2018 Underwater Inspection Date: 9/19/2018

Structure Name: Ludlam Trail Corridor Bridge-1

Road Name/Number: N/A

Feature Intersected: Coral Gables (C-3) Canal

Location: 0.5 Miles North of SR-976 (SW 40th St.)

Type of Inspection: [X] Routine [ ] Interim [ ] Initial [ ] Special

### INSPECTION CONDITIONS

Superstr. NBI Rating: 4 Poor — Deck NBI Rating: N/A — Equipment Used: Camera, Inspection Hammer, Wrenches

Substruct. NBI Rating: 4 Poor — Channel NBI Rating: 6 Fair

Plumb: Yes [X] No [ ]

Min Lateral Clear. (ft): ____

Vertical Clearance (ft): ____

Special Equipment: No

Elements Inspected: Timber Deck, Steel Beams, Slide Bearing Steel Pier Caps, Steel Abutment Timber Piles, Concrete Wall, and Channel.

MOT Required: Yes [ ] No [x]

Special Crew Hours: 10 hrs x 5 inspectors — Hazards: Marine Life

**Critical Deficiency Notes:** None

**Personnel / Title / Number** — **Initials**

Hays, Stephen - Inspector/Commercial Diver (CBI. #00438), Lead — SH

Spinola Abdel - Bridge Inspector Assistance

Rodriguez Carlos - Bridge Inspector Assistance

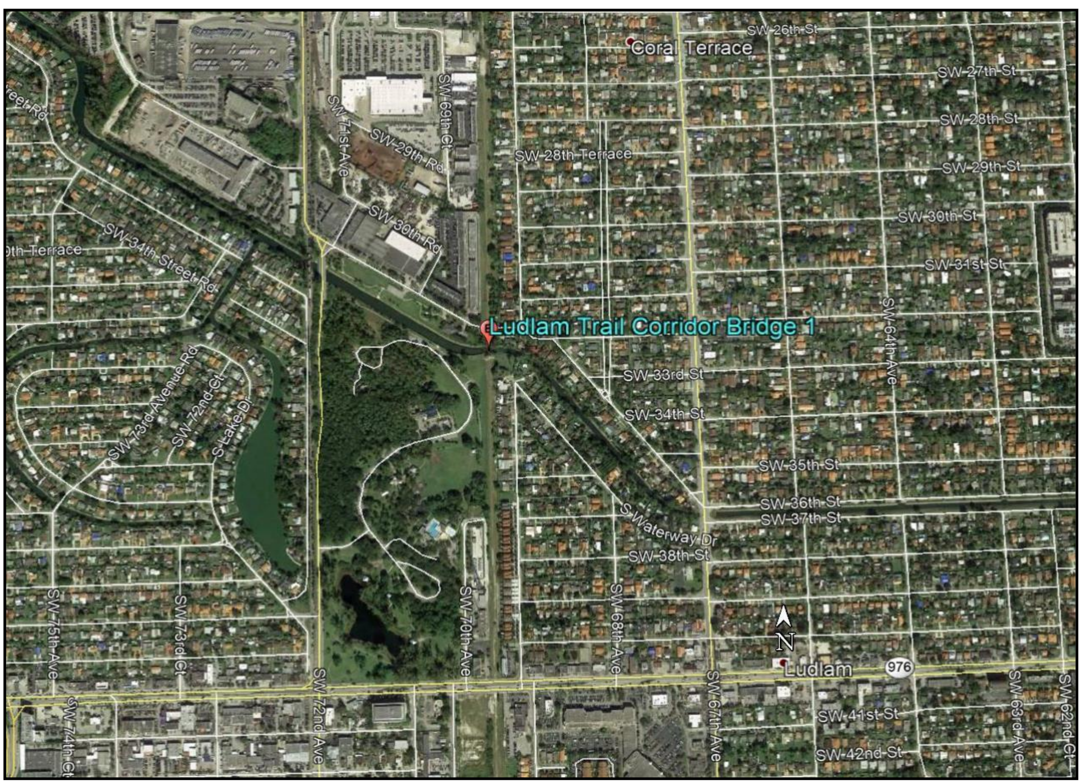
Rego, Alexis - Bridge Inspection Supervisor (CBI # 409)

Quintana, Jose - Professional Engineer (P.E. #47072)

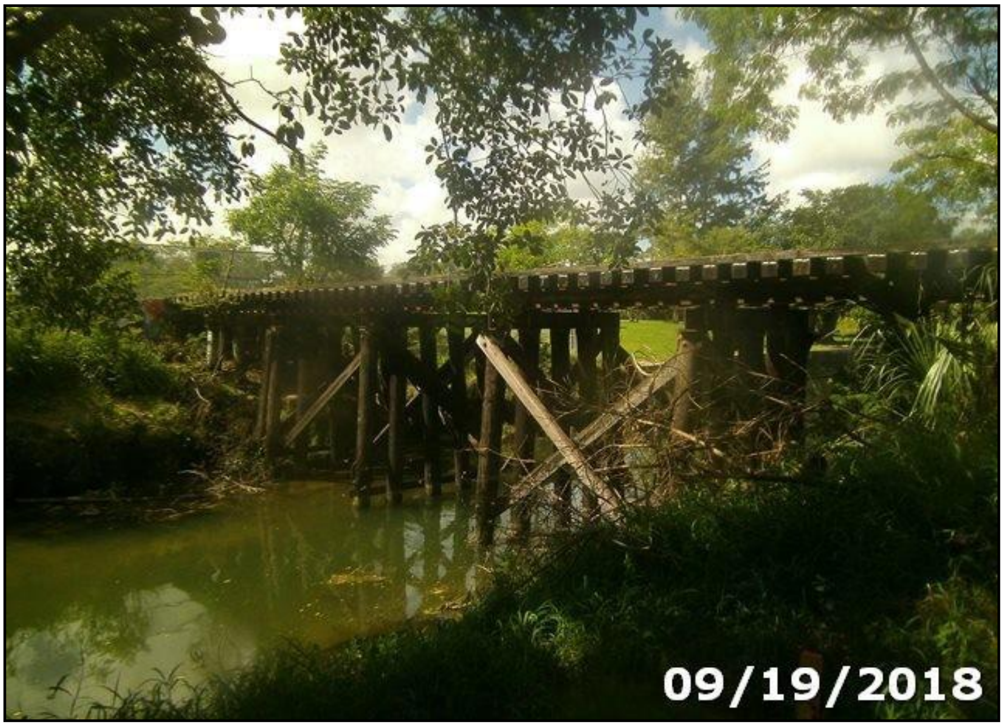
JOSE ANTONIO QUINTANA LICENSE No 47072 ★ STATE OF FLORIDA PROFESSIONAL ENGINEER

## A: ELEVATION & LOCATION MAP

Ludlam Trail Corridor Bridge-1 over Coral Gables Canal, located 0.5 Miles North of SR-976



## B: STRUCTURE LEVEL INVENTORY REPORT




**Inventory Photo 01: West Elevation**


**Inventory Photo 02: Timber Deck Overview (Railroad Ties and Timber Guards)**

## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 03: South Approach Looking South**


**Inventory Photo 04: North Approach Looking North**

## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 05: Typical Elastomeric Bearing**


**Inventory Photo 06: Typical Underside View**

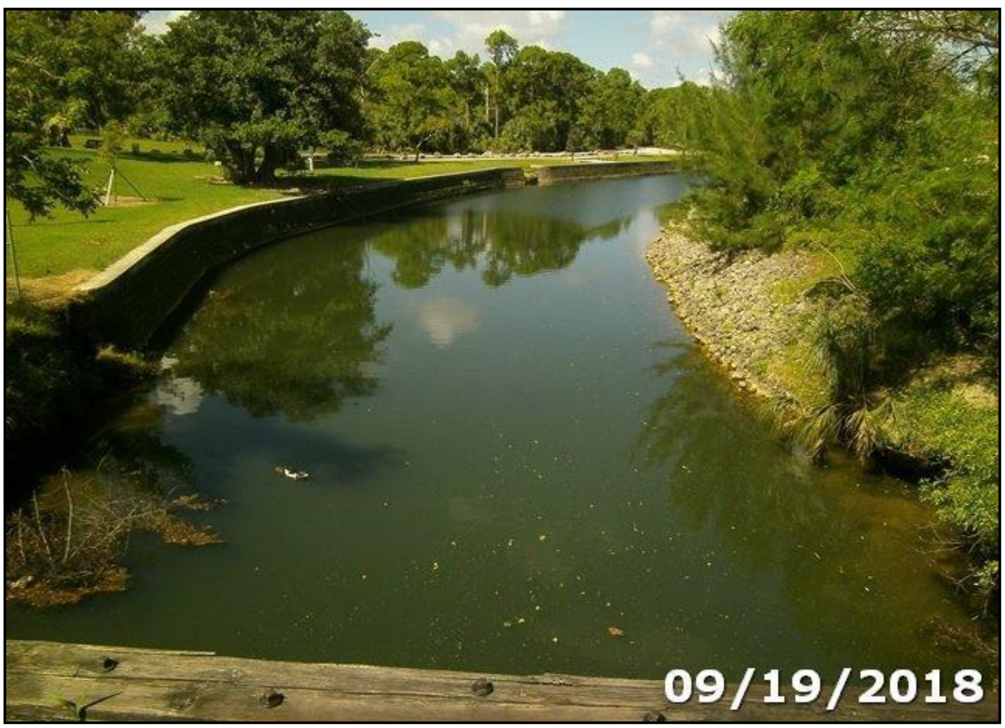
## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 07: Typical Timber Piles and Cross Bracing**


**Inventory Photo 08: Abutment 1 Overview**

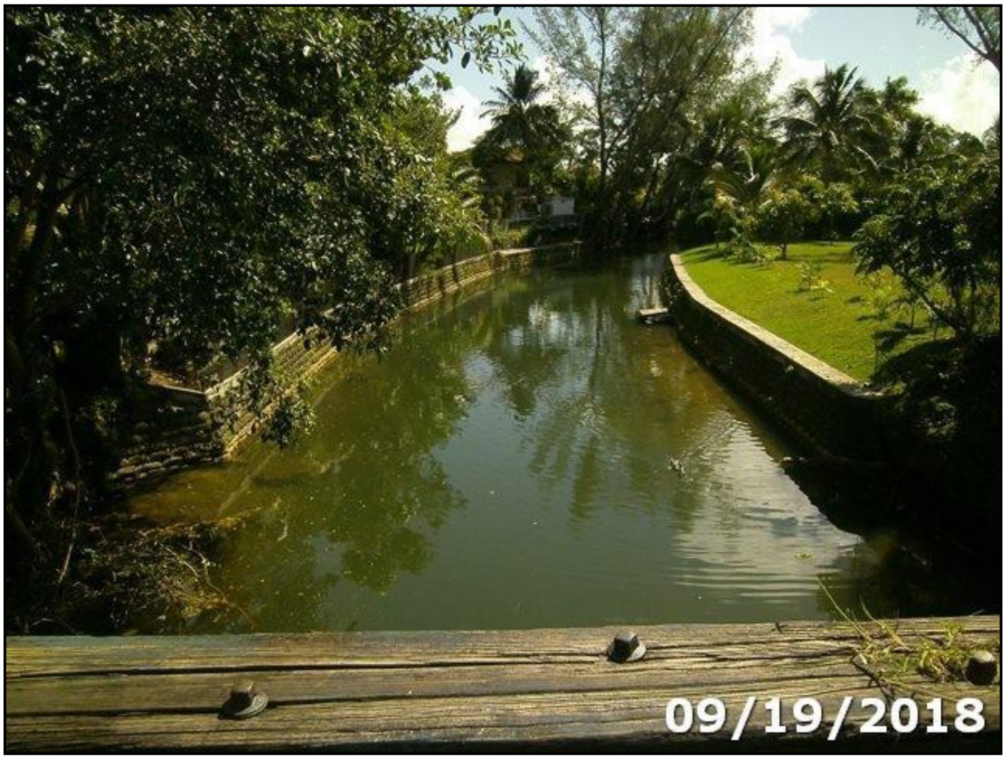
## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 09: Abutment 8 Overview**




**Inventory Photo 10: West Channel View**

## B: STRUCTURE LEVEL INVENTORY REPORT




**Inventory Photo 11: East Channel View**

## C: STRUCTURE NOTES

**General Notes:**

1. The structure is inventoried from South to North.
2. The NBI Ratings are as follows: Deck - N/A
   Superstructure - 4 (Poor)
   Substructure - 4 (Poor)
   Channel - 6 (Satisfactory)
3. The bridge is in an overall poor structural condition and it should remain closed.

**Future Use Consideration:**
The structure's intended future use is as a shared-use path with light maintenance and emergency vehicle access. Though the structure can be rehabilitated and restored to match the historical significance and aesthetics of the existing bridge, full replacement is the most viable option. Replacement is the recommended preferred option due to the following: significantly lower costs, the design of a new bridge would not have the structural and aesthetic constraints of reusing the existing bridge, and potential time savings if Miami-Dade County considers use of a prefab bridge design.

## C: STRUCTURE NOTES

**Estimated Opinion of Costs**

Costs estimates have been prepared based on structural bridge systems that meet current code and standards for the intended future use of the structure as a shared-use path with limited light maintenance and emergency vehicle access.

**LUDLAM - 1**

**REPAIR ESTIMATE**

| Activity | Unit Cost | Qty | Total |
|---|---|---|---|
| PM (12%) | $75,232.80 | **1** | $75,232.80 |
| Design/A/E (15%) | $94,041.00 | 1 | $94,041.00 |
| General Req. (6% | $37,616.40 | **1** | $37,616.40 |
| Bond Ins (1%) | $6,269.40 | **1** | $6,269.40 |
| Contingency (10%) | $53,100.00 | **1** | $53,100.00 |
| Mobilization (10%) | $62,694.00 | 1 | $62,694.00 |
| Overdecking/Railing | $65/Sq. Ft | 1,476 Sq. Ft | $95,940.00 |
| Pile Jackets | $10,000/Pile | 40 | $400,000.00 |
| Embankment | $28,000/LS | 1 | $28,000.00 |
| Concrete Repairs | $7,000/LS | 1 | $7,000.00 |
| Steel Cap Reinforcement | $41,000/LS | 1 | $41,000.00 |
| Cleaning/Coating Bearings | $18,000/LS | 1 | $18,000.00 |
| Timber Work (Cross Bracing) | $37,000/LS | 1 | $37,000.00 |
| | | **Total** | **$955,893.60** |

**LUDLAM - 1**

**REPLACEMENT ESTIMATE (Shared-used path bridge with light maintenance/emergency access)**

| Activity | Unit Cost | Qty | Total |
|---|---|---|---|
| PM (12%) | $51,364.80 | **1** | $51,364.80 |
| Design/A/E (15%) | $64,206.00 | 1 | $64,206.00 |
| General Req. (6% | $25,682.40 | **1** | $25,682.40 |
| Bond Ins (1%) | $4,280.40 | **1** | $4,280.40 |
| Contingency (10%) | $42,804.00 | **1** | $42,804.00 |
| Mobilization (10%) | $42,804.00 | 1 | $42,804.00 |
| Construction | $250/Sq. Ft | 1,476 Sq. Ft | $369,000.00 |
| Demolition | $40/Sq. Ft. | 1,476 Sq. Ft | $59,040.00 |
| | | **Total** | **$659,181.60** |

## D: ELEMENT NOTES

| **Element** | **Quantity** |
|---|---|
| **<u>Timber Deck</u>** | **836 sf.** |

Note: The deck is composed of transverse railroad ties (10ft. L x 8in. W x 8in. H) tied together by two full length longitudinal timber guards (82ft. L x 8in. W x 8in. H) at both sides of the structure. The rails are missing throughout the structure. Refer to the Addendum for elements layout and numbering.

**CS-4:**

**1.** The right (east) timber guard is decayed and rotted up to 100% section loss at north and south ends, missing up to 15ft. L at the north end, also, the 4th railroad tie from the north is not attached and displaced. **See Photos 01 and 02.**

**2.** There is a missing section 16ft. L on the left timber guard at Span 2. **See Photo 03.**

3. The railroad ties typically exhibit splits and checks, some have severe decay with areas up to 5ft. L x 3in. W x 3in. D having 100% section loss and vegetation growth. Worst cases are Ties 20, 21, 24 ,26, 28, 31, 32, 33, and 37. **See Photos 04 and 05.**

4. There is a tree growing between the railroad ties at north of Bent 7. **See Photo 06.**

**<u>Steel Beams</u>** **164 ft.**

Note: There are two parallel steel I-beams (13in. H x 12.25in. W x 1in. of thickness), 82ft. L each, along the structure, supporting the railroad ties. Refer to the Addendum for elements layout and numbering.

**CS-3:**

1. Both beams exhibit severe corrosion throughout with pitting at the lower portion of the web, having **areas** of 95% section remaining. **See Photo 07.**

**2.** The outboard faces of the beams exhibit graffiti. **See Photo 08.**

**<u>Slide Bearing</u>** **16 ea.**

Note: This element represents the slide bearing plates and hardware under the steel beams over abutment caps and intermediate bent caps.

**CS-3:**

1. The bearing plates and anchorage hardware have severe corrosion with section loss up to 100%. There are neoprene pads at the abutments bearings only. **See Photos 09 and 10.**

## D: ELEMENT NOTES

| **Element** | **Quantity** |
|---|---|
| **<u>Steel Pier Caps</u>** | **66 ft.** |

Note: This element represents the steel caps at Piers 2 thru 7. There are six steel I-Beams 11ft. L each. (dimensions: 12in. H x 12in. W). Refer to the Addendum for elements layout and numbering.

**CS-3:**

**1.** All steel caps exhibit moderate to severe corrosion, having areas of up to 100% section loss at random locations. Worst cases are Pier caps 2 and 7. **See Photos 11 and 12.**

2. The pier caps exhibit graffiti throughout.

**Steel Abutment** — **22 ft.**

Note: This element represents the steel I-Beams caps at end bents, 11ft. L each. (dimensions: 12in. H x 12in. W).

**CS-3:**

1. The abutment caps exhibit moderate to severe corrosion, having areas with section loss up to 100% at the ends of Abutment 1 cap. **See Photos 13 and 14.**

**Timber Piles** — **40 ea.**

Note: This element represents the timber piles at all bents including the abutments, with five piles each, and the timber cross bracings tied to them. Refer to the Addendum for elements layout and numbering.

**CS-3:**

1. All piles exhibit severe decay below the groundline, having soft areas with up to 70% section loss. **See Photos 15, 16, and 17.**

**2.** Pile 4-1 and the cross bracing have severe decay, having up to 100% section loss at the west fascia cross bracing connection. **See Photo 18.**

3. The piles of Bents 3 thru 6 exhibit decay with soft areas up to 3/4in. D at the tide zone, having up to 5% section loss. Total 20 piles. **See Photo 19**.

3. The piles typically exhibit checks and splits.

4. Bent 2 piles have evidence of fire damage.

5. Spans 2, 3, and 5 are missing one of two fascia cross bracings.

**6.** Pier 6 has one broken cross bracing at Pile 6-4, and the horizontal is missing. **See Photo 20.**

**7.** The cross bracing typically exhibit decay with section loss at the bottom ends. **See Photo 21.**

## D: ELEMENT NOTES

**Element** **Quantity**

**<u>Concrete Wall</u>** **80 ft.**

Note: This element represents the 40ft. L concrete retaining walls behind both abutments.

**CS-3:**

**1.** Abutment 1 concrete wall has a spall 30in. L x 15in. H x 3in. D with two exposed rebar, having corrosion with up to 50% section loss, 2ft. west of Pile 1-5. **See Photo 22.**

**2.** Both concrete walls exhibit graffiti throughout. **See Photo 23.**

**<u>Channel</u>** **1 ea.**

**CS-3:**

**1.** Both channel embankments have erosion throughout. **See Photo 24.**

**2.** There is drift and debris scattered on the channel. **See Photo 25.**

**3.** There is heavy vegetation growth at both channel embankments below the structure. **See Photo 26.**

## E: PHOTO SECTION


**Photo 01:** The right (east) timber guard is decayed up to 100%, having the north end missing. The 4th railroad tie from the north is not attached and displaced.


**Photo 02: Decayed and missing south end of the right (east) timber guard.**

## E: PHOTO SECTION


**Photo 03:** Missing section 16ft. L on the left timber guard at Span 2.


**Photo 04:** Typical splits and checks on the railroad ties.

## E: PHOTO SECTION


**Photo 05:** Typical severe decay and vegetation growth on some railroad ties.


**Photo 06:** There is a tree growing between the railroad ties at north of Bent 7.

## E: PHOTO SECTION


**Photo 07:** Both beams exhibit severe corrosion throughout with pitting.


**Photo 08:** The outboard faces of the beams exhibit graffiti.

## E: PHOTO SECTION


**Photo 09:** The bearing plates and anchorage hardware are heavily corroded with up to 100% section loss. Neoprene pads at abutment bearings.


**Photo 10:** Typical corrosion with pitting at the intermediate bents bearing plates.

## E: PHOTO SECTION


**Photo 11:** All steel caps exhibit moderate to severe corrosion, having areas of up to 100% section loss at random locations. Shown Pier cap 7, east end.


**Photo 12:** Typical corrosion with pitting on the steel pier caps.

## E: PHOTO SECTION


**Photo 13:** Severe corrosion with section loss up to 100% at east end of Abutment 1 cap. (Front view)


**Photo 14:** Severe corrosion with section loss up to 100% at east end of Abutment 1 cap. (East view)

## E: PHOTO SECTION


**Photo 15:** Pile 8-3 exhibits severe decay with section loss up to 70% below the groundline.


**Photo 16:** Severe decay with section loss up to 40% below the groundline on Pile 1-2.

## E: PHOTO SECTION


**Photo 17:** Severe decay with section loss up to 50% below the groundline on Pile 2-3.


**Photo 18:** Pile 4-1 and the cross bracing have severe decay, having up to 100% section loss at the cross bracing connection.

## E: PHOTO SECTION


**Photo 19:** Typical decay on the piles of Bents 3 thru 6 at the tide zone.


**Photo 20:** Pier 6 has one broken cross bracing at Pile 6-4, and the horizontal is missing.

**E: PHOTO SECTION**


**Photo 21:** Typical decay with section loss at the bottom ends of the cross bracing members.


**Photo 22:** Spall 30in. L x 15in. H x 3in. D with two exposed rebar at Abutment 1 concrete wall, 2ft. west of Pile 1-5.

## E: PHOTO SECTION


**Photo 23:** Both concrete walls exhibit graffiti throughout.


**Photo 24:** Typical erosion at the channel slopes.

## E: PHOTO SECTION


**Photo 25:** There is drift and debris scattered on the channel.


**Photo 26:** There is heavy vegetation growth at both channel slopes below the structure.

## F: RECOMMENDED REPAIRS

**Element**

**Timber Deck**
_Replace all timber deck railroad ties and timber guards. Photos 01 thru 05.
_Remove tree growing between railroad ties north of Bent 7. Photo 06.

**Steel Beams**
_Clean and coat corrosion at the steel beams. Photo 07.

**Slide Bearing**
_Clean and coat corrosion on bearing plates and hardware. Photos 09 and 10.

**Steel Pier Caps**
_Reinforce web to bottom flange section on all steel caps. Photos 11 and 12.

**Steel Abutm**
_Reinforce web to bottom flange section of the abutment caps. Photos 13 and 14.

**Timber Piles**
_Replace or jacket all timber piles. Photos 15 thru 19.
_Replace all cross bracing members. Photos 20 and 21.

**Concrete Wall**
_Clean and coat exposed rebar and repair spall at Abutment 1 concrete wall. Photo 22.

**Channel**
_Fill and stabilize eroded areas at the channel embankments. Photo 24.
_Remove debris and drift from the channel. Photo 25.
_Remove heavy vegetation from the channel slopes below the structure. Photo 26.

## G: SCOUR EVALUATION

Profile Data - Numerical Summary

| Bent # | Left Height (ft.) | Right Height (ft.) |
|---|---|---|
| 1 | 4.5 | 4.4 |
| 2 | 8.1 | 8.6 |
| 3 | 14.9 | 15.5 |
| 4 | 19 | 18.9 |
| 4.5 | 19.8 | 19.3 |
| 5 | 18.9 | 18.8 |
| 6 | 15.9 | 15.6 |
| 7 | 8.6 | 8.7 |
| 8 | 4.2 | 4.3 |

Notes:

1. Measurements were taken from the top of the deck timber guards. Waterline measurement at mid-channel: Left 15.4 ft. Right 15.4 ft. Maximum Depth: 4.4 ft.
2. There are no previous documented soundings to evaluate any significant scour issues.
3. Current sounding and depth of channel do not reflect any potential scour concerns.

# PEDESTRIAN BRIDGE INSPECTION REPORT

Structure No: **Ludlam Bridge-1**

## H. ADDENDUM

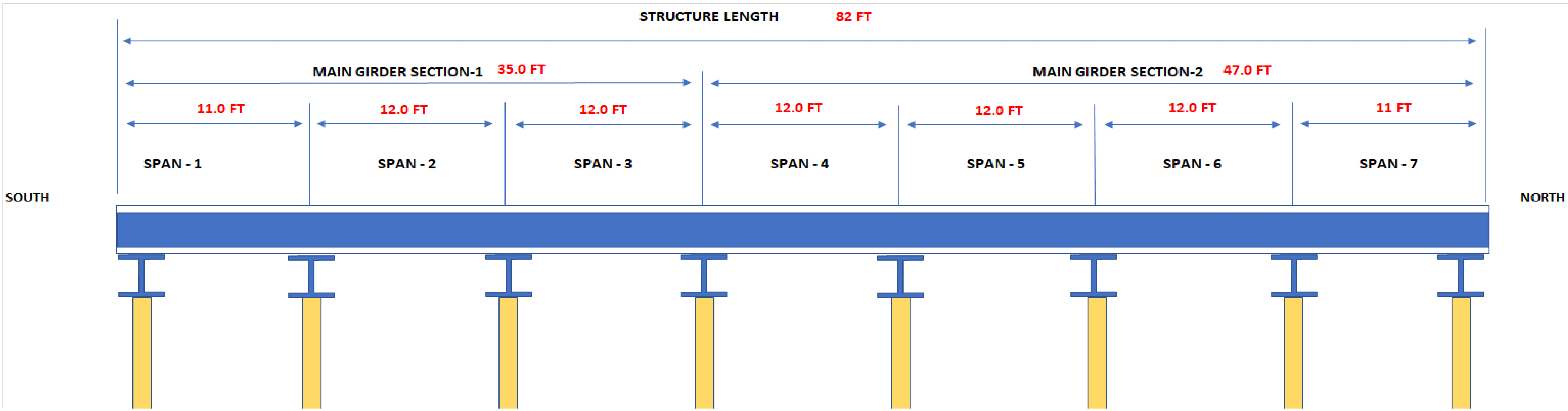



**Diagram 01:** Elevation View

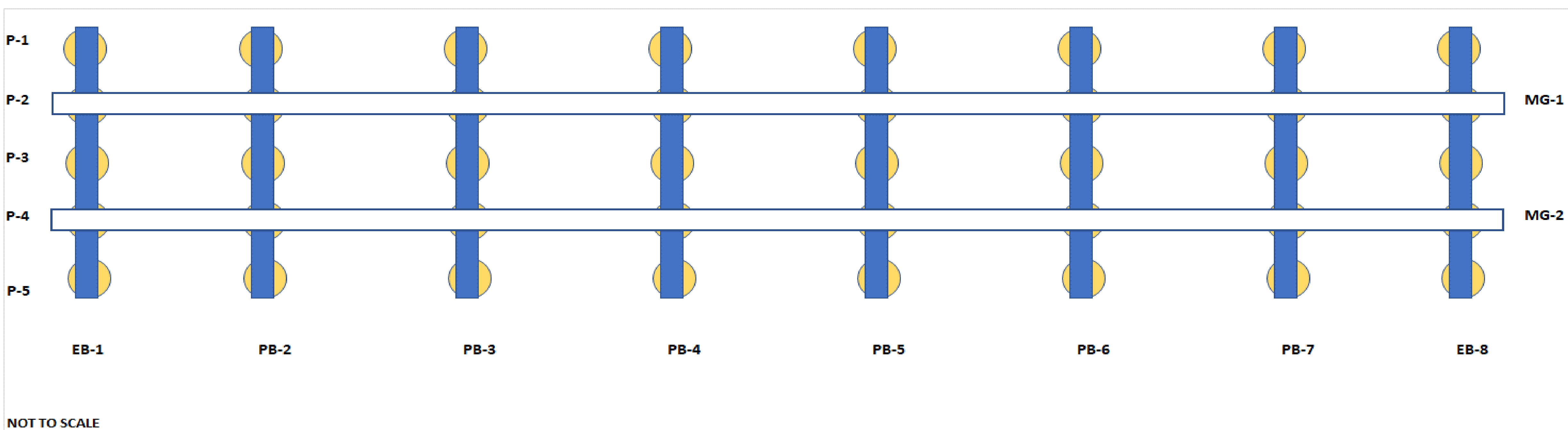
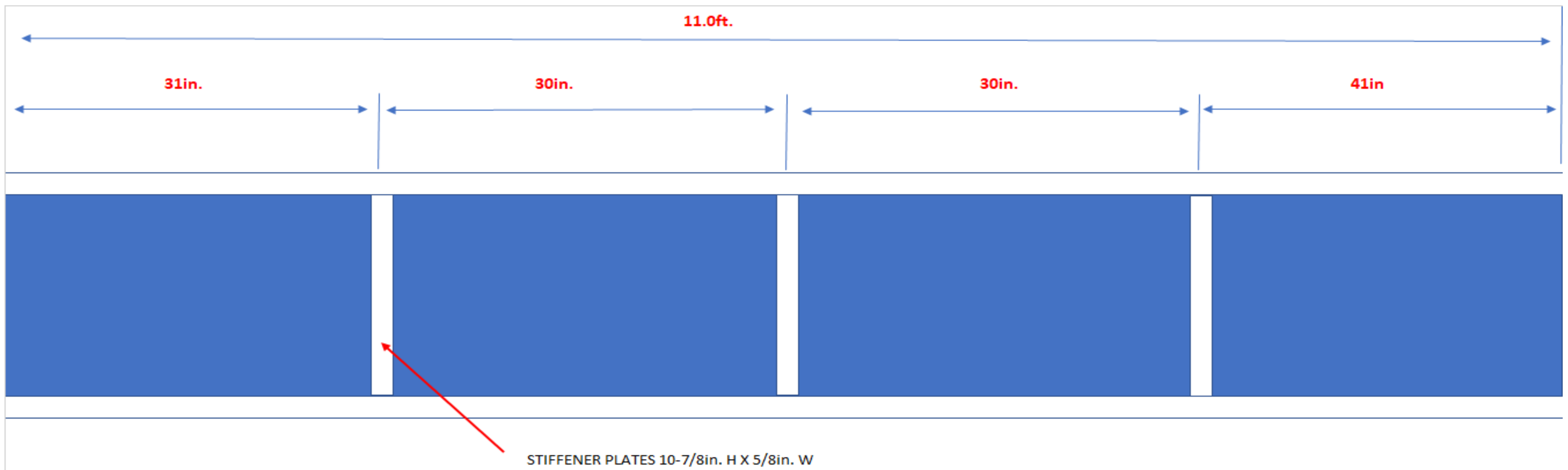



**Diagram 02:** Top View. Main Elements Layout and Numbering.

# PEDESTRIAN BRIDGE INSPECTION REPORT

Structure No: **Ludlam Bridge-1**

## H. ADDENDUM



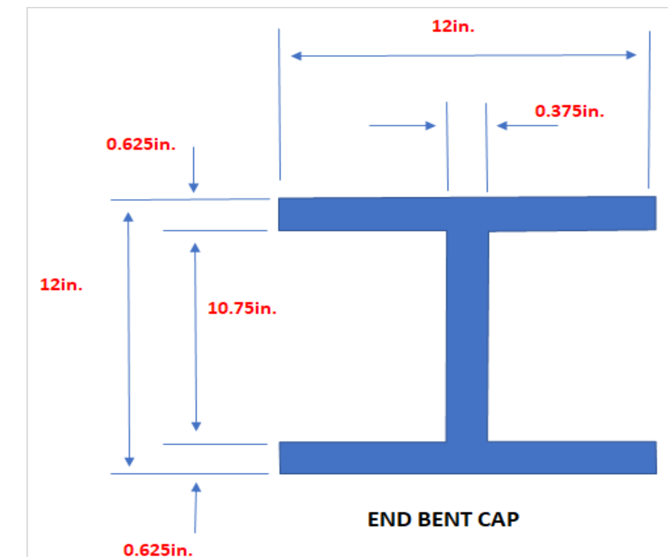
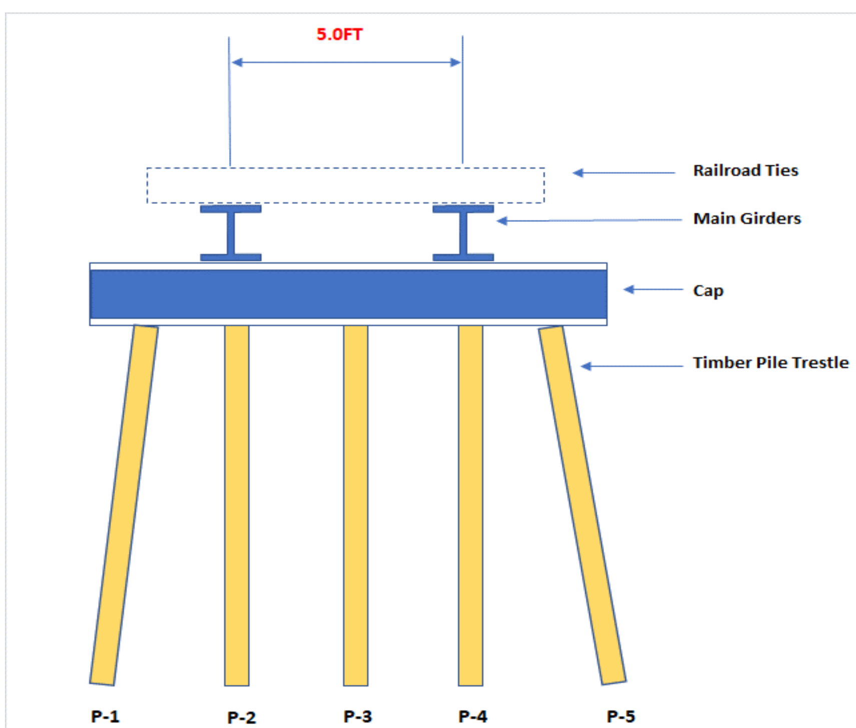
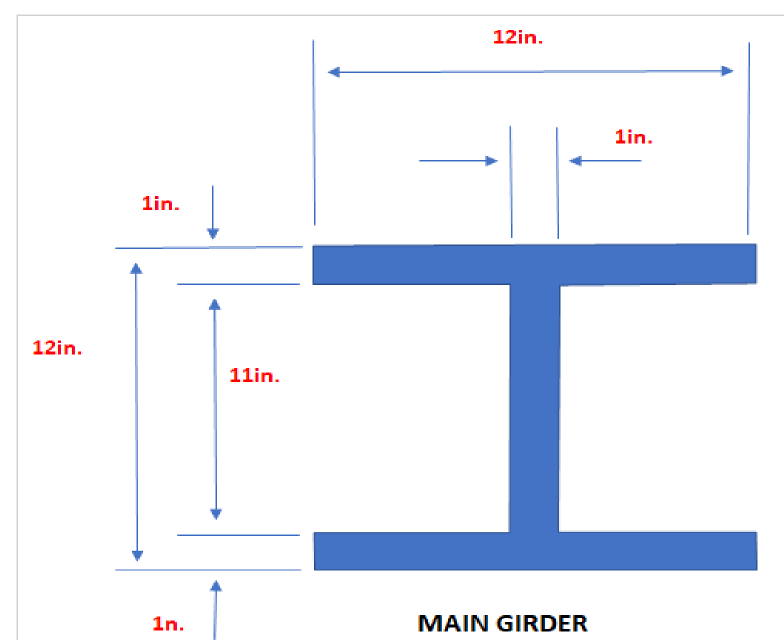




**Diagram 01:** End Bent Cap



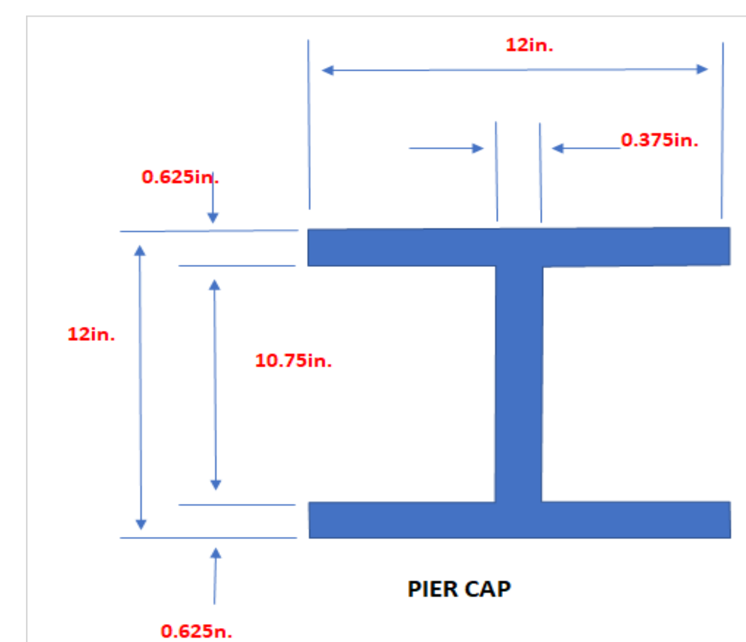




**Diagram 03:** Typical Bent Elevation

**LUDLAM TRAIL CORRIDIOR BRIDGE 1**
**LOAD RATING ANALYSIS**

**NOVEMBER, 2018**

# LOAD RATING ANALYSIS RESULTS

**PREPARED FOR**
AECOM


**PREPARED BY**
MARLIN Engineering, Inc.
1700 NW 66th Avenue, Suite 106
Plantation, FL 33313
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MARLIN

# Table of Contents

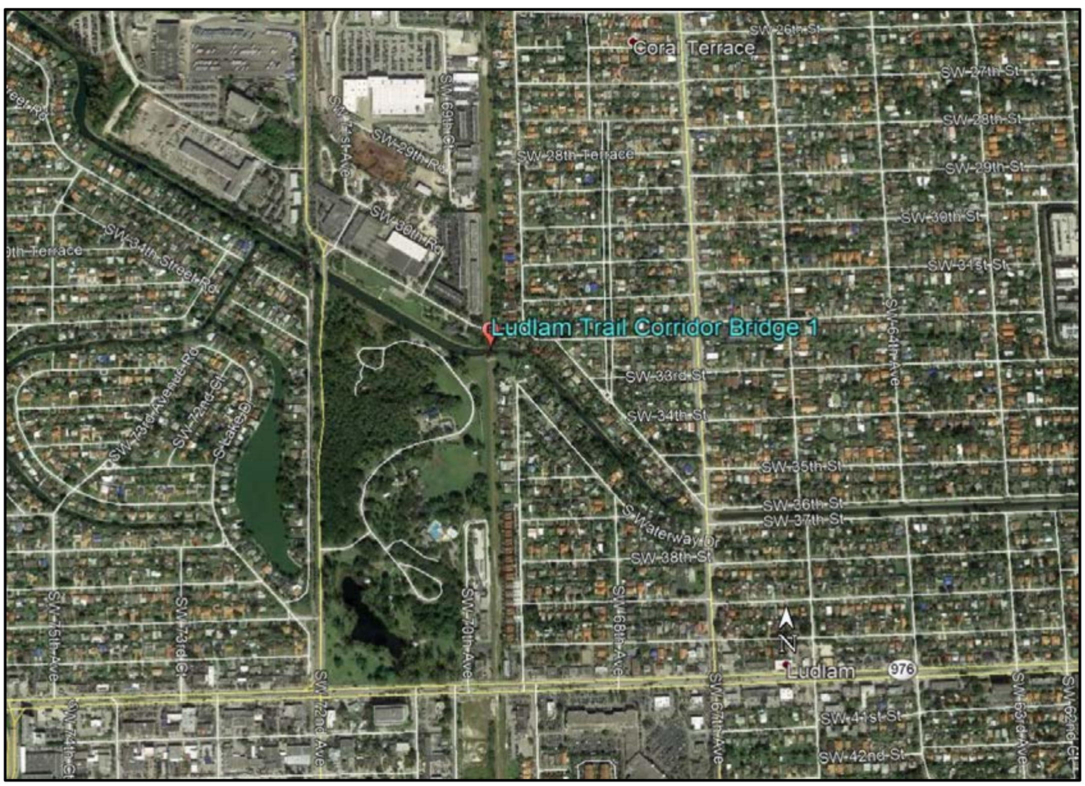
# Executive Summary

Ludlum Trail Corridor Bridge 1 crossing the Coral Gables (C-3) Canal was load rated for future potential use a s a shared-use path with light maintenance emergency vehicle access. The bridge was originally an FEC rail line that has been abandoned. No existing plans were provided for this analysis.

A load rating analyses of the superstructure, based on assumptions outlined in this report, and on findings of a bridge inspection performed by Marlin Engineering indicate that with the removal of the exiting rail road tie deck and installation of a properly designed reinforced concrete deck, the bridge would not have Florida Legal Load restrictions on the superstructure.

# Bridge Location

Ludlam Trail Corridor Bridge is located approximately 0.5 mile north of SR-976 (SW 40th Street) in the northeast corner of AD Barnes Park.




**Location of Bridge 1**

## Load Rating Assumptions

The following assumptions were made for this load rating analysis:

1. The load rating was performed using LRFR method.
2. The load rating analysis is limited to one two-girder 12-0" span, conservatively taken to be simply supported.
3. Reduction in girder section properties was taken to account for documented section loss.
4. A non-composite reinforced concrete deck was analyzed in lieu of the existing timber rail tie deck.
5. No allowance for the weight of stay-in-place forms is included in the load rating.
6. No allowance for future wearing surface is included in the load rating.
7. A fictitious concrete deck with a width of 18.0 ft. was used for the computation of Dead Load ONLY. **The existing superstructure CANNOT be used to support an 18 ft. wide deck. Increasing** the deck width to 18 ft. would require the design of a new superstructure and substructure, and subsequent post-design load rating.

## Bridge Information

| | |
|---|---|
| Max Span Length: | 12.0 ft. |
| Bridge Width: | 18.0 ft.* |
| Deck Thickness: | 10 in. |
| Girder Dimensions: | As shown in Load Rating calculations |
| Skew Angle: | 0 deg. |
| Concrete Unit Wight: | 150 pcf for deck concrete |
| Steel Unit Weight; | 490 pcf |
| Steel Yield Strength: | 36 ksi |

*Used in the computation of dead load only. **The existing superstructure cannot be used to support an 18 ft. wide deck.**

# Load Rating Summary Sheet:

| | | | | |
|---|---|---|---|---|
| **Bridge No.** | **Ludlam Bridge 1** | **Analysis Method:** | **LRFR-LRFD** | **FDOT Bridge Load Rating Summary Form (Page 1 of 1)** |
| **Location** | **Ludlam Trail over Coral Gables (C-3) Canal** | | | |
| **Description** | **Single Span - Two Girder Bridge** | | | |

| Rating Type | Rating Type | Gross Axle Weight (tons) | Moment/Shear/Service | | Dead Load Factor | Live Load Factor | Live Load Distrib. Factor (axles) | Rating Factor | Span No. - Girder No., Interior/Exterior, %Span·L | RF·Weight (tons) |
|---|---|---|---|---|---|---|---|---|---|---|
| **Level** | **Vehicle** | **Weight** | **Member Type** | **Limit** | **DC** | **LL** | **LLDF** | **RF** | **Governing Location** | **RATING** |
| Inventory | HL93 | 36 | Steel | Strength, Moment | 1.25/0.90 | 1.75 | 1.000 | 1.664 | 0.5L | 59.9 |
| Operating | HL93 | 36 | Steel | Strength, Moment | 1.25/0.90 | 1.35 | 1.000 | 2.157 | 0.5L | 77.7 |
| Permit | FL120 | 60 | Steel | Service | 1.00 | 0.90 | 1.000 | 1.256 | 0.5L | 75.4 |
| Permit Max Span | FL120 | 60 | Steel | Service | 1.00 | 0.90 | 1.000 | 1.256 | 0.5L | 75.4 |
| Legal | SU2 | 17 | Steel | NA | NA | NA | | | | -1 |
| | SU3 | 33 | Steel | NA | NA | NA | | | | -1 |
| | SU4 | 35 | Steel | Limit Test | NA | NA | | | | -1 |
| | C3 | 28 | Steel | Limit Test | NA | NA | | | | -1 |
| | C4 | 36.7 | Steel | Limit Test | NA | NA | | | | -1 |
| | C5 | 40 | Steel | Limit Test | NA | NA | | | | -1 |
| | ST5 | 40 | Steel | Limit Test | NA | NA | | | | -1 |

| | | |
|---|---|---|
| *Original* Design Load | Rail | |
| Rating Type, *Analysis* | LRFR-LRFD | |
| Distribution Method | Others | |
| Impact Factor | 33.0% | (axle loading) |
| FL120 Gov. Span Length | N/A | (feet) |
| Recommended Posting | At/Above legal loads. Posting Not Required. | |
| Recommended SU Posting* | 99 | (tons) |
| Recommended C Posting | 99 | (tons) |
| Recommended ST5 Posting | 99 | (tons) |
| Floor Beam Present? | No | |
| Segmental Bridge? | No | |
| Project No. & Reason | N/A | Other |
| Plans Status | NA (use field measurements) | |
| Software Name, Version | Mathcad | |

| | | | |
|---|---|---|---|
| Performed by: | JAV | Date: | 10.25.18 |
| Checked by: | BKR | Date: | 11.03.18 |
| Sealed By: | JAV | Date: | 10.25.18 |
| FL P.E. No.: | 77896 | | |
| Cert. Auth. No.: | 6104 | | |
| Phone & email: | 561.229.0239 - jvers@marlinengineering.com | | |
| Company: | MARLIN Eningeering, Inc. | | |
| Address: | 10415 Riversife Drive, Palm Beach Gardens, FL 33410 | | |

P.E. Seal

JULIE ANN VERS
LICENSE
11.08.18
No. 77896
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

COMMENTS BY THE ENGINEER

This 12-01-2017 summary follows the FDOT Bridge Load Rating Manual (BLRM), and the FDOT BMS Coding Guide.
*Recommended SU Posting levels for Florida SU trucks adequately restricts AASHTO SU trucks; see BLRM Chapter 7.

fdot.gov/maintenance/LoadRating.shtm

# Load Rating Calculations:

# Load and Resistance Factor Rating (LRFR) for Noncomposite Steel Bridge

SUBJECT: Ludlam Bridge - 1
PROJECT#: 2018038.000

DESIGNED BY: JAV — DATE: 10.27.18
CHECKED BY: BKR — DATE: 11.03.18

***References:***

- ***FDOT "Bridge Load Rating Manual, 2018"***
- ***AASHTO "Manual for Bridge Evaluation, 2nd Edition with 2015 Revisions"***
- ***AASHTO "Bridge Design Specifications, Seventh Edition"***
- ***Bridge Inspection Report by MARLIN dated 09.19.18***

**Girder Section Properties:**

Width of Flange: $b_f := 12in$

Reduction Factor for Section Loss: $SL := 0.75$

Thickness of Flange: $t_f := SL \cdot 1.00in = 0.75 \cdot in$

Thickness of Web: $t_w := SL \cdot 1.00in = 0.75 \cdot in$

Height of Member Minus the Flange and Curved Radius: $h := 10.in$

Area of Girder: $A_g := b_f \cdot 2 \cdot t_f + h \cdot t_w = 25.5 \cdot in^2$

Plastic Section Modulus: $Z_x := 150.2in^3$

Elastic Section Modulus: $S_x := 129.9in^3$

Depth of Member: $D := 12in$

Torsional Constant: $J := 10.19in^4$

**Geometry:**

Span Length of Girders: $Span := 12ft + 0in$

Number of Girders: $n_{girders} := 2$

Unbraced Span Length of Girders: $Span_{unbraced} := 12ft + 0in$

Unbraced Length: $L_b := Span_{unbraced}$

Span Length of Deck: $Span_{deck} := 5ft + 0in$

Overhang Length of Deck: $Overhang := 6ft + 6in$

Thickness of Deck: $t_{deck} := 10in$

Width of Bridge: $Width := 18ft + 0in$

**Material Properties:**

| | |
|---|---|
| Year Built: | Year := "unknown" |
| Steel Yield Strength: | $F_y := 36ksi$ |
| Steel Moduus of Elasticity: | $E := 29000ksi$ |
| Density of Steel: | $\gamma_{steel} := 490pcf$ |
| Density of Concrete: | $\gamma_{conc} := 150pcf$ |

**Load Factors:**

| | | |
|---|---|---|
| Wearing Course Load Factors: | $\gamma_{DW} := 1.25$ | $\gamma_{DW.o} := 1.25$ |
| Component Load Factors: | $\gamma_{DC} := 1.25$ | $\gamma_{DC.o} := 1.25$ |
| Live and Impact Load Factors: | $\gamma_{LL.IM} := 1.75$ | $\gamma_{LL.IM.o} := 1.35$ |
| Service Dead Load Factors: | $\gamma_D := 1.0$ | |
| Service Live Load Factors: | $\gamma_{LL.IMs} := 1.3$ | $\gamma_{LL.IMso} := 1.0$ |
| Strength Legal Live Load Factor: | $\gamma_L := 1.40$ | |
| Service Legal Live Load Factor: | $\gamma_{Ls} := 1.30$ | |

**Evaluation Factors for Strength Limit States:**

| | | |
|---|---|---|
| Resistance Factor for Flexure: | $\varphi := 1.0$ | |
| Condition Factor for Poor Condition: | $\varphi_c := 1.0$ | |
| System Factor for Slab Bridges: | $\varphi_s := 0.85$ | (For 2 girders) |
| Tire Load from Design Truck: | $P_{truck} := 16kip$ | |
| Tire load from the design tandem | $P_{tandem} := 12.5kip$ | |
| Multiple Presence Factor, Single Lane Loaded: | $m_{single} := 1.0$ | (Use 1.0 as MPF's not applicable to this one-lane bridge) |
| Multiple Presence Factor, Double Lanes Loaded: | $m_{double} := 1.0$ | |
| Dynamic Load Allowance: | $IM := 33\%$ | |

**DC Component:**

Weight of Deck Units: $wt_{deck} := \gamma_{conc} \cdot t_{deck}$

Deck Dead Load: $DC_{deck} := wt_{deck} \cdot (0.5 \cdot Span_{deck} + Overhang) = 1.125 \cdot klf$

Weight of Girder: $wt_{girder} := \gamma_{steel} \cdot A_g = 0.087 \cdot klf$

Girder Dead Load: $DC_{girder} := wt_{girder} = 0.087 \cdot klf$

Height of Curb: $h_{curb} := 8in$

Width of Curb: $b_{curb} := 7in$

Weight of Curb: $wt_{curb} := h_{curb} \cdot b_{curb} \cdot \gamma_{conc} = 0.058 \cdot klf$

Number of Curbs: $n_{curb} := 2$

Curb Dead Load: $DC_{curb} := \dfrac{n_{curb} \cdot wt_{curb}}{n_{girders}} = 0.058 \cdot klf$

Moment due to Component Loads: $M_{DC} := \dfrac{(DC_{deck} + DC_{girder} + DC_{curb}) \cdot Span^2}{8} = 22.862 \cdot kip \cdot ft$

Shear due to Component Loads: $V_{DC} := \dfrac{(DC_{deck} + DC_{girder} + DC_{curb}) \cdot Span}{2} = 7.621 \cdot kip$

**DW Component:**

Weight of Wearing Layer: $wt_{wearing} := 0psf$

Load of Wearing Layer: $DW_{wearing} := wt_{wearing} \cdot (0.5 \cdot Span_{deck} + Overhang) = 0$

Moment due to Wearing Loads: $M_{DW} := \dfrac{DW_{wearing} \cdot Span^2}{8} = 0$

Shear due to Wearing Loads: $V_{DW} := \dfrac{DW_{wearing} \cdot Span}{2} = 0$

**Live Load Analysis**

*Conservatively assume that wheel is directly over girder and less than 2' away from curb.

Design Lane Load: $Lane := 64psf \cdot (0.5 \cdot Span_{deck} + Overhang) = 0.576 \cdot klf$

Reaction from the Design Truck: $R_{truck} := m_{single} \cdot P_{truck} = 16 \cdot kip$

Reaction from the Design Tandem: $R_{tandem} := m_{single} \cdot P_{tandem} = 12.5 \cdot kip$

**Controlling Live Load on the Girders:**

Design Live Load Moment on the Exterior Beam including Impact and Lane Loads: $M_{LL.IM} := \frac{205}{2} kip \cdot ft$

(FDOT Load Rating Manual, 2018)

Design Live Load Shear on the Exterior Beam including Impact and Lane Loads: $V_{LL.IM} := \frac{62.4}{2} kip$

(FDOT Load Rating Manual, 2018)

## Nominal Flexural Resistance of the Section:

### Check that Appendix A6 of AASHTO LRFD is appropriate:

Depth of the web in compression in the elastic range — $D_{cp} := 0.5D$

Depth of the web in compression at the plastic moment — $D_c := D_{cp}$

Check that the web satisfies the noncompact slenderness limit [A6.1-1] — $\frac{2 \cdot D_{cp}}{t_w} \le 5.7 \cdot \sqrt{\frac{E}{F_y}} = 1$ — 6.10.6.2.3-1

$$Check_{NC.slenderness} := \text{if}\left(\frac{2 \cdot D_{cp}}{t_w} \le 5.7 \cdot \sqrt{\frac{E}{F_y}}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

With the girder having symmetric flanges the check of $I_{yc} / I_{yt} > 0.3$ is met. Appendix A6 is appropriate.

Limiting slenderness ratio for noncompact web [A6.2.1-3] — $\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E}{F_y}}$

Hybrid factor for rolled shapes [6.10.1.10.1] — $R_h := 1.0$

Redefinition of variables — $b_c := b_f \quad t_c := t_f \quad b_t := b_f \quad t_t := t_f$

Plastic compression force in the compression flange — $P_c := F_y \cdot b_c \cdot t_c$

Plastic compression force in the web — $P_w := F_y \cdot D \cdot t_w$

Plastic compression force in the tension flange — $P_t := F_y \cdot b_t \cdot t_t$

Distance from the plastic neutral axis to the top of the element where the plastic neutral axis is located [D6.1] — $Y_{bar} := \frac{D}{2} \cdot \left(\frac{P_t - P_c}{P_w} + 1\right) = 6 \cdot in$

Distance from the plastic neutral axis to the midthickness of the compression flange — $d_c := Y_{bar} - 0.5 \cdot t_f$

Distance from the plastic neutral axis to the midthickness of the tension flange — $d_t := Y_{bar} - 0.5 \cdot t_f$

Plastic moment — $M_p := \frac{P_w}{2 \cdot D} \cdot \left[Y_{bar}^{\ 2} + \left(D - Y_{bar}\right)^2\right] + P_c \cdot d_c + P_t \cdot d_t = 384.75 \cdot kip \cdot ft$

Yield moment — $M_y := F_y \cdot S_x = 389.7 \cdot kip \cdot ft$

Limiting slenderness ratio for compact web [A6.2.1-2]

$$\lambda_{pw} := \frac{\sqrt{E \div F_y}}{\left(0.54 \cdot \frac{M_p}{R_h \cdot M_y} - 0.09\right)^2}$$

$$\lambda_{pw} \le \lambda_{rw} \cdot \frac{D_{cp}}{D_c} = 1$$

$$\text{Check}_{compact.web} := \text{if}\left(\lambda_{pw} \le \lambda_{rw} \cdot \frac{D_{cp}}{D_c}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

The web is compact.

**Check Compression Flange Buckling:**

Moment of inertia of the compression flange plus one-third of the web depth

$$I_{yc} := \frac{t_f \cdot b_f^{\,3}}{12} + \frac{(D \div 3) \cdot t_w^{\,3}}{12} = 108.141 \cdot in^4$$

Area of the compression flange plus one-third of the web depth

$$A_c := t_f \cdot b_f + (D \div 3) \cdot t_w = 12 \cdot in^2$$

Radius of gyration of the compression flange plus one-third of the web depth

$$r_t := \sqrt{I_{yc} \div A_c} = 3.002 \cdot in$$

Check to see if the compression flange is adequately braced

$$1.76 \cdot r_t \cdot \sqrt{\frac{E}{F_y}} \ge L_b = 1$$

$$\text{Check}_{CF.braced} := \text{if}\left(1.76 \cdot r_t \cdot \sqrt{\frac{E}{F_y}} \ge L_b, \text{"CF Adequately Braced"}, \text{"CF NOT Adequately Braced"}\right)$$

$$\text{Check}_{CF.braced} = \text{"CF Adequately Braced"}$$

The compression flange is not adequately braced.

**Compression Flange Local Buckling:**

Slenderness ratio of the flange [A6.3.2-3]

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 8$$

Limiting slenderness ratio for a compact flange [A6.3.2-4]

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785$$

Flange local buckling coefficient for rolled shapes [A6.3.2]

$$k_c := 0.76$$

Limiting slenderness ratio for a noncompact flange [A6.3.2-5]

$$\lambda_{rf} := 0.95 \cdot \sqrt{\frac{E \cdot k_c}{F_y}}$$

Web plastification factor for the compression flange [A6.2.1-4]

$$R_{pc} := M_p \div M_y$$

Elastic section modulus about the major axis of the section to the compression flange

$$S_{xc} := M_y \div F_y$$

Nominal flexural resistance based on compression flange local buckling [A6.3.2-2]

$$M_{n.FLB} := \begin{cases} R_{pc} \cdot M_y & \text{if } \lambda_f \le \lambda_{pf} \\ \left[\left[1 - \left(1 - \frac{F_y \cdot S_{xc}}{R_{pc} \cdot M_y}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \cdot R_{pc} \cdot M_y\right] & \text{otherwise} \end{cases}$$

$$M_{n.FLB} = 384.75 \cdot ft \cdot kip$$

**Lateral Torsional Buckling:**

Limiting unbraced length to achieve nominal flexural resistance [A6.3.3-4]

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_y}} = 85.202 \cdot in$$

Limiting unbraced length to achieve nominal onset of yielding [A6.3.3-5]

$$L_r := 1.95 \cdot r_t \cdot \frac{E}{F_y} \cdot \sqrt{\frac{J}{S_{xc} \cdot h}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{F_y}{E} \cdot \frac{S_{xc} \cdot h}{J}\right)^2}} = 602.542 \cdot i$$

Moment gradient modifier

$$C_b := 1.0$$

Elastic lateral torsional buckling stress [A6.3.3-8]

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(L_b \div r_t\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J}{S_{xc} \cdot h} \cdot \left(\frac{L_b}{r_t}\right)^2}$$

Flexural resistance based on lateral torsional buckling [A6.3.3]

$$M_{n.LTB} := \begin{cases} \left(R_{pc} \cdot M_y\right) & \text{if } L_b \le L_p \\ \min\left(F_{cr} \cdot S_{xc}, R_{pc} \cdot M_y\right) & \text{if } L_b > L_r \\ \left[C_b \cdot \left[1 - \left(1 - \frac{F_y \cdot S_{xc}}{R_{pc} \cdot M_y}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_{pc} \cdot M_y\right] & \text{otherwise} \end{cases}$$

$$M_{n.LTB} = 385.313 \cdot ft \cdot kip$$

Nominal flexural resistance based on the compression flange

$$M_n := \min\left(M_{n.FLB}, M_{n.LTB}\right) = 384.75 \cdot kip \cdot ft$$

**Shear Resistance:**

Shear buckling coefficient $k := 5$

Ratio of shear-buckling resistance to shear yield strength [6.10.9.2.3-4,5and6]

$$C := \begin{cases} 1.0 & \text{if } \frac{D}{t_w} \le 1.12 \cdot \sqrt{\frac{E \cdot k}{F_y}} \\ \left[\frac{1.57}{(D \div t_w)^2} \cdot \left(\frac{E \cdot k}{F_y}\right)\right] & \text{if } \frac{D}{t_w} > 1.4 \cdot \sqrt{\frac{E \cdot k}{F_y}} \\ \left(\frac{1.12}{D \div t_w} \cdot \sqrt{\frac{E \cdot k}{F_y}}\right) & \text{otherwise} \end{cases}$$

Nominal shear resistance of unstiffendd web [6.10.9.2-1and2] $V_n := C \cdot 0.58 \cdot F_y \cdot D \cdot t_w = 187.92 \cdot kip$

**Service II Stresses:**

Allowable stress for service limit state specified in the LRFD code for non-composite section: $f_r := 0.80 \cdot F_y$

Component Dead Load Stress: $f_{DC} := M_{DC} \div S_x = 2.112 \cdot ksi$

Wearing Dead Load Stress: $f_{DW} := M_{DW} \div S_x = 0$

Live Load Stress: $f_{LL.IM} := M_{LL.IM} \div S_x = 9.469 \cdot ksi$

## Design Load Rating:

### General Load-Rating Equation:

General Load Rating Equation:

$$RF = \frac{C - \gamma_{DC} \cdot DC - \gamma_{DW} \cdot DW}{\gamma_L \cdot (LL + IM)} + -\frac{\gamma_p \cdot P}{\gamma_L \cdot (LL + IM)}$$

### Strength I Limit State:

Inventory Rating Factor:

$$M_{RF.I} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL.IM} \cdot M_{LL.IM}} = 1.664$$

Operating Rating Factor:

$$M_{RF.O} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC.o} \cdot M_{DC} - \gamma_{DW.o} \cdot M_{DW}}{\gamma_{LL.IM.o} \cdot M_{LL.IM}} = 2.157$$

Inventory Rating Factor:

$$V_{RF.I} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot V_n - \gamma_{DC} \cdot V_{DC} - \gamma_{DW} \cdot V_{DW}}{\gamma_{LL.IM} \cdot V_{LL.IM}} = 2.751$$

Operating Rating Factor:

$$V_{RF.O} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot V_n - \gamma_{DC.o} \cdot V_{DC} - \gamma_{DW.o} \cdot V_{DW}}{\gamma_{LL.IM.o} \cdot V_{LL.IM}} = 3.566$$

### Service II Limit State:

Inventory Rating Factor:

$$RF_I := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{LL.IMs} \cdot f_{LL.IM}} = 2.168$$

Operating Rating Factor:

$$RF_O := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{LL.IMso} \cdot f_{LL.IM}} = 2.819$$

### Summary of Rating Factors:

RATING_FACTORS =

| "Limit State" | "Inventory Design Load Rating" | "Operating Design Load Rating" |
|---|---|---|
| "Strength I Flexure" | 1.664 | 2.157 |
| "Strength I Shear" | 2.751 | 3.566 |
| "Service II" | 2.168 | 2.819 |

DESIGN = "Legal load ratings are not required"

**Legal Load Rating:**

Maximum factor used earlier in lever rule calculations to distribute tire loads onto the girders

$\varphi_{lever} := 1.00$

Moment from SU4 permit vehicle

$P_1 := \varphi_{lever} \cdot 18.7kip \div 2$ $P_2 := \varphi_{lever} \cdot 13.9kip \div 2$

(FDOT Load Rating Manual, 2018)

$M_{SU4} := \frac{176.2}{2} \cdot kip \cdot ft$

(FDOT Load Rating Manual, 2018)

$P := \varphi_{lever} \cdot 22kip \div 2$

Moment from the C5 permit vehicle

$M_{C5} := \frac{147.9}{2} \cdot kip \cdot ft$

Moment from the ST5 permit vehicle

$P := \varphi_{lever} \cdot 18kip \div 2$

(FDOT Load Rating Manual, 2018)

$M_{ST5} := \frac{134.9}{2} \cdot kip \cdot ft$

Moment from the FL120 permit vehicle
(FDOT Load Rating Manual, 2018)

$P := \varphi_{lever} \cdot 53.3kip \div 2$

$M_{FL120} := \frac{266.0}{2} \cdot kip \cdot ft$

**Strength I Limit State:**

Flexure rating factor for SU4 loading

$$RF_{SU4.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_L \cdot \left[(1 + IM) M_{SU4}\right]}$$

$RF_{SU4.SI} = 1.819$

Flexure rating factor for C5 loading

$$RF_{C5.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_L \cdot \left[(1 + IM) M_{C5}\right]}$$

$RF_{C5.SI} = 2.168$

Flexure rating factor for ST5 loading

$$RF_{ST5.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_L \cdot \left[(1 + IM) M_{ST5}\right]}$$

$RF_{ST5.SI} = 2.376$

Flexure rating factor for FL120 loading

$$RF_{FL120.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{1.0 \cdot \left[(1 + IM) M_{FL120}\right]}$$

$RF_{FL120.SI} = 1.687$

**Service II Limit State**

Flexure rating factor for SU4 loading

$$RF_{SU4.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{SU4} \div S_x\right]}$$

$RF_{SU4.SII} = 1.897$

Flexure rating factor for C5 loading

$$RF_{C5.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{C5} \div S_x\right]}$$

$RF_{C5.SII} = 2.259$

Flexure rating factor for ST5 loading

$$RF_{ST5.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{ST5} \div S_x\right]}$$

$RF_{ST5.SII} = 2.477$

Flexure rating factor for FL120 loading

$$RF_{FL120.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{FL120} \div S_x\right]}$$

$RF_{FL120.SII} = 1.256$

**Posted Load Rating:**

SU4 Posting:

$$P_{RF.SU4} := 70kip \cdot \left(\min\left(RF_{SU4.SI}, RF_{SU4.SII}\right)\right) = 63.679\ tonf$$

C5 Posting:

$$P_{RF.C5} := 56kip \cdot \left(\min\left(RF_{C5.SI}, RF_{C5.SII}\right)\right) = 121.383\ kip$$

ST5 Posting:

$$P_{RF.ST5} := 80kip \cdot \left(\min\left(RF_{ST5.SI}, RF_{ST5.SII}\right)\right) = 190.114\ kip$$

FL120 Posting:

$$P_{RF.FL120} := 120kip \cdot \left(\min\left(RF_{FL120.SI}, RF_{FL120.SII}\right)\right) = 150.758\ kip$$

$$POSTED\_RATING := \begin{pmatrix} \text{"SU4"} & if\left(\min\left(RF_{SU4.SI}, RF_{SU4.SII}\right) > 1, \text{"Not Required"}, P_{RF.SU4}\right) \\ \text{"C5"} & if\left(\min\left(RF_{C5.SI}, RF_{C5.SII}\right) > 1, \text{"Not Required"}, P_{RF.C5}\right) \\ \text{"ST5"} & if\left(\min\left(RF_{ST5.SI}, RF_{ST5.SII}\right) > 1, \text{"Not Required"}, P_{RF.ST5}\right) \\ \text{"FL120"} & if\left(\min\left(RF_{FL120.SI}, RF_{FL120.SII}\right) > 1, \text{"Not Required"}, P_{RF.FL120}\right) \end{pmatrix}$$

Required Posted Load Rating:

$$POSTED\_RATING = \begin{pmatrix} \text{"SU4"} & \text{"Not Required"} \\ \text{"C5"} & \text{"Not Required"} \\ \text{"ST5"} & \text{"Not Required"} \\ \text{"FL120"} & \text{"Not Required"} \end{pmatrix} \cdot tonf$$

MIAMI-DADE COUNTY

# ROUTINE INSPECTION REPORT

**Prepared by: Marlin Engineering, Inc.**

MARLIN
1700 NW 66 Avenue - Ste. 106
Phone: 305-477-7575

## CONTENTS OF REPORT



## PREPARED FOR: MIAMI-DADE COUNTY

### REPORT IDENTIFICATION

Bridge No.: N/A  Inspection Date: 9/20/2018  Underwater Inspection Date: 9/20/18

Structure Name: Ludlam Trail Corridor Bridge-2

Road Name/Number: N/A

Feature Intersected: Tamiami (C- 4 ) Canal

Location: 0.1 Miles North of SR-968 (W Flagler St.)

Type of Inspection: [X] Routine [ ] Interim [ ] Initial [ ] Special

### INSPECTION CONDITIONS

Superstr. NBI Rating: 4 Poor  Deck NBI Rating: N/A  Equipment Used: Camera, Inspection Hammer, Wrenches
Substruct. NBI Rating: 4 Poor  Channel NBI Rating: 6 Fair
Plumb: Yes X N
Min Lateral Clear. (ft):
Vertical Clearance (ft):
Special Equipment: No
MOT Required: Yes [ ] No [x]
Special Crew Hours: 10 hrs x 5 inspectors  Hazards: Marine Life

Elements Inspected: Timber Deck, Steel Beams, Slide Bearing Steel Pier Caps, Steel Abutment Timber Piles, Concrete Wall, and Channel.

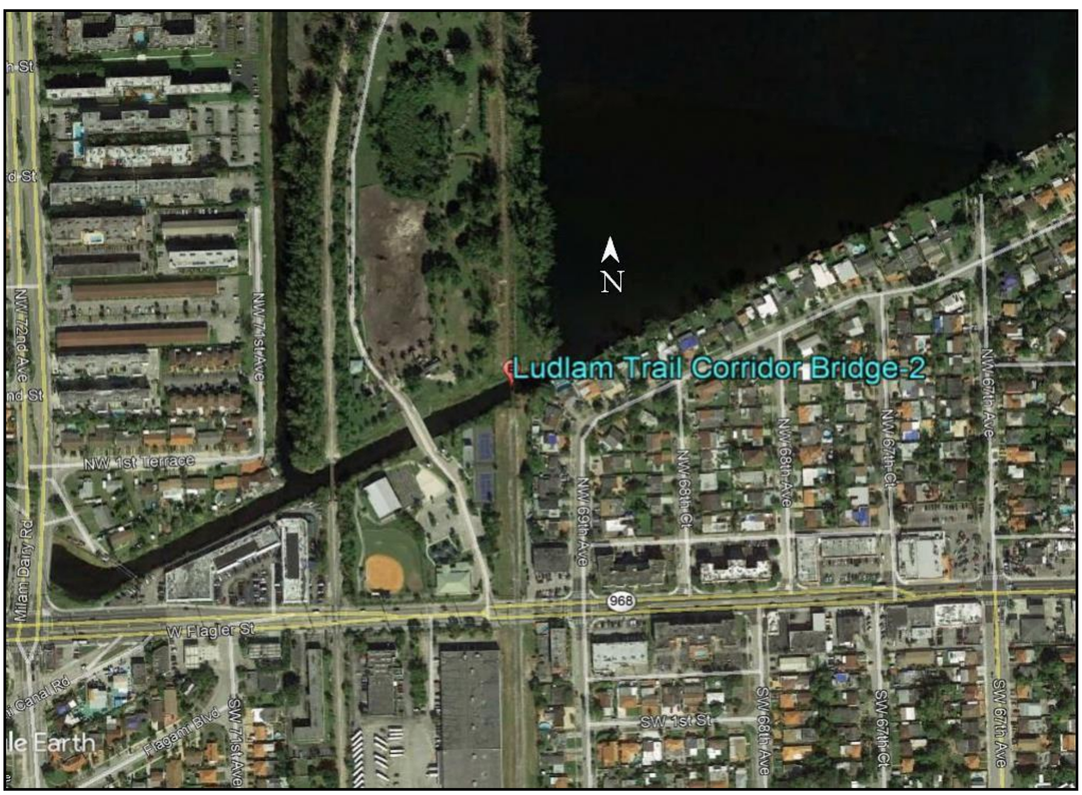
**Critical Deficiency Notes:** None

| **Personnel / Title / Number** | **Initials** |
|---|---|
| Hays, Stephen - Inspector/Commercial Diver (CBI. #00438), Lead | SH |
| Spinola Abdel - Bridge Inspector Assistance | |
| Rodriguez Carlos - Bridge Inspector Assistance | |
| Rego, Alexis - Bridge Inspection Supervisor (CBI # 409) | |
| Quintana, Jose - Professional Engineer (P.E. #47072) | |

JOSE ANTONIO QUINTANA LICENSE No 47072 STATE OF FLORIDA PROFESSIONAL ENGINEER

## A: ELEVATION & LOCATION MAP

Ludlam Trail Corridor Bridge-2 over Tamiami Canal, located 0.1 Miles North of SR-968



## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 01: East Elevation**


**Inventory Photo 02: West Elevation**

## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 03: Deck Overview**


**Inventory Photo 04: South Approach Looking South**

## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 05: North Approach Looking North**


**Inventory Photo 06: Typical Bearing**

## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 07: Typical Intermediate Pier**


**Inventory Photo 08: Main Span Bent 6 Overview**

## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 09: Abutment 1 Overview**


**Inventory Photo 10: Abutment 10 Overview**

## B: STRUCTURE LEVEL INVENTORY REPORT


**Inventory Photo 11: West Channel View**


**Inventory Photo 12: East Channel View**

## C: STRUCTURE NOTES

**General Notes:**

1. The structure is inventoried from South to North.
2. The NBI Ratings are as follows:
   Deck - N/A
   Superstructure - 4 (Poor)
   Substructure - 4 (Poor)
   Channel - 6 (Satisfactory)
3. The bridge is in an overall poor structural condition and it should remain closed.

**Future Use Consideration:**
The structure's intended future use is as a shared-use path with light maintenance and emergency vehicle access. Though the structure can be rehabilitated and restored to match the historical significance and aesthetics of the existing bridge, full replacement is the most viable option. Replacement is the recommended preferred option due to the following: significantly lower costs, the design of a new bridge would not have the structural and aesthetic constraints of reusing the existing bridge, and potential time savings if Miami-Dade County considers use of a prefab bridge design.
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## C: STRUCTURE NOTES

**Estimated Opinion of Costs:**
Costs estimates have been prepared based on structural bridge systems that meet current code and standards for the intended future use of the structure as a shared-use path with limited light maintenance and emergency vehicle access.

**LUDLAM - 2**

**REPAIR ESTIMATE**

| Activity | Unit Cost | Qty | Total |
|---|---|---|---|
| PM (12%) | $81,842.40 | **1** | $81,842.40 |
| Design/A/E (15%) | $102,303.00 | 1 | $102,303.00 |
| General Req. (6% | $40,921.20 | **1** | $40,921.20 |
| Bond Ins (1%) | $6,820.20 | **1** | $6,820.20 |
| Contingency (10%) | $68,202.00 | **1** | $68,202.00 |
| Mobilization (10%) | $68,202.00 | 1 | $68,202.00 |
| Overdecking/Railing | $65/Sq. Ft | 1,908 Sq. Ft | $124,020.00 |
| Pile Jackets | $10,000/Pile | 56 | $400,000.00 |
| Embankment | $28,000/LS | 1 | $28,000.00 |
| Concrete Repairs | $1,000/LS | 1 | $1,000.00 |
| Steel Cap Reinforcement | $55,000/LS | 1 | $55,000.00 |
| Cleaning/Coating Bearings | $28,000/LS | 1 | $28,000.00 |
| Timber Work (Cross Bracing) | $46,000/LS | 1 | $46,000.00 |
| | | **Total** | **$1,050,310.80** |

**LUDLAM - 2**
**REPLACEMENT ESTIMATE (Shared-used path bridge with light maintenance/emergency access)**

| Activity | Unit Cost | Qty | Total |
|---|---|---|---|
| PM (12%) | $66,398.40 | **1** | $66,398.40 |
| Design/A/E (15%) | $82,998.00 | 1 | $82,998.00 |
| General Req. (6% | $33,199.20 | **1** | $33,199.20 |
| Bond Ins (1%) | $5,533.20 | **1** | $5,533.20 |
| Contingency (10%) | $55,332.00 | **1** | $55,332.00 |
| Mobilization (10%) | $55,332.00 | 1 | $55,332.00 |
| Construction | $250/Sq. Ft | 1,908 Sq. Ft | $477,000.00 |
| Demolition | $40/Sq. Ft. | 1,908 Sq. Ft | $76,320.00 |
| | | **Total** | **$852,112.80** |

## D: ELEMENT NOTES

| **Element** | **Quantity** |
|---|---|
| **Timber Deck** | **1060 sf.** |

Note: The deck is composed of steel rails, transverse railroad ties (10ft. L x 8in. W x 8in. H) tied together by two full length longitudinal timber guards (82ft. L x 8in. W x 4in. H) at both sides of the structure.

**CS-4:**

1. The railroad ties typically exhibit splits and checks, some have severe decay with soft and hollow areas, and vegetation growth. Worst cases are Ties 10, 11, 22 ,28, 29, 30, 40, 41, 44, 49, 52, 53, 54, 69, 71, 76, 79, and 80. **See Photos 01 and 02.**
**2.** Railroad tie 36 is missing. **See Photo 03.**
**3.** The timber guards exhibit splits and checks, having areas of severe decay and missing sections over Bent 4. **See Photo 04.**

| | |
|---|---|
| **Steel Beams** | **1024 ft.** |

Note: There are two parallel steel I-beams (12in. H x 12in. W) at spans 1 thru 4, and 6 thru 9. Span 5 (main span) has four parallel girders I-beams (30in. H x 10-3/4in. W). Refer to the Addendum for elements layout and numbering.

**CS-3:**

**1.** Steel beams exhibit light to moderate corrosion throughout with pitting on some areas at the bottom flanges. **See Photo 05.**
**2.** The faces of the beams exhibit graffiti. **See Photo 06.**

| | |
|---|---|
| **Slide Bearing** | **32 ea.** |

Note: This element represents the bearing plates and hardware under the steel beams over the pier caps.

**CS-3:**

1. The bearing plates and anchorage hardware have severe corrosion with section loss up to 50%. **See Photo 07.**

| | |
|---|---|
| **Steel Pier Caps** | **104 ft.** |

Note: This element represents the steel caps (11ft. L) at Piers 2 thru 4 and 7 thru 9, and Piers 5 and 6 have an array of steel caps, each totaling 8ft. L. Refer to the Addendum for elements layout and numbering.

**CS-3:**

**1.** The steel caps exhibit moderate to severe corrosion, having areas of up to 40% section remaining at webs, bottom flanges and stiffeners. **See Photos 08 and 09.**
**2.** The pier caps exhibit graffiti on the north and south faces. **See Photo 08.**

## D: ELEMENT NOTES

| Element | Quantity |
|---|---|
| **Steel Abutment** | **22 ft.** |

Note: This element represents the steel I-Beams caps at abutments, 11ft. L each. (dimensions: 12in. H x 12in. W).

**CS-3:**
1. The abutment caps exhibit moderate to severe corrosion on the webs and bottom flanges, having areas of section loss up to 100% at both ends. **See Photo 10.**

| | |
|---|---|
| **Timber Piles** | **56 ea.** |

Note: This element represents the timber piles at all bents including the abutments. The abutments and bents 2 thru 4, and 7 thru 9 have four piles each. Bents 5 and 6 have 12 piles each. Also, the timber cross bracings are considered under this element. Refer to the Addendum for elements layout and numbering.

**CS-3:**
**1.** Piles 1-2, 2-1, 3-4, 9-1, 9-3, and 10-3 exhibits areas up to 24in. H x 12in. W of severe decay with up to 0% section remaining at the cap. **See Photos 11 and 12.**
**2.** The timber piles exhibit checks and splits throughout, and decay with soft areas below the ground line with up to 1-1/2in. D. **See Photo 13.**
**3.** Piles 6-1, 6-3, 6-11, and 6-12 exhibit severe decay and insect activity with up to 20% section remaining, most of them approximately 5ft. below the cap. **See Photos 14 and 15.**
**4.** The timber piles at Bents 2 and 3 exhibit minor fire damage. **See Photo 16.**
**5.** The cross bracing members exhibit areas of severe decay with up to 0% section remaining at the bottom ends. Worst cases are located in Bents 3, 4, 5, 6, 7, and 8. **See Photo 17.**

| | |
|---|---|
| **Concrete Wall** | **80 ft.** |

Note: This element represents the 40ft. L concrete retaining walls behind both abutments.

**SECONDARY:**
1. Both concrete walls exhibit graffiti throughout. **See Photo 18.**

| | |
|---|---|
| **Channel** | **1 ea.** |

**CS-2:**
**1.** The channel embankments exhibit erosion at both sides of the bridge. **See Photo 19.**
**2.** There is drift scattered throughout the channel. **See Photo 20.**
**3.** There are numerous cut-off piles up to 3ft. L on the channel below the structure, not affecting the water flow.
**4.** There is heavy vegetation growth at both channel slopes below the structure. **See Photo 21.**

## E: PHOTO SECTION


**Photo 01:** Typical splits and checks on railroad ties. Severe decay at Ties 10, 11, 22 ,28, 29, 30, 40, 41, 44, 49, 52, 53, 54, 69, 71, 76, 79, and 80.


**Photo 02:** Typical decay with vegetation growth on railroad ties.

## E: PHOTO SECTION

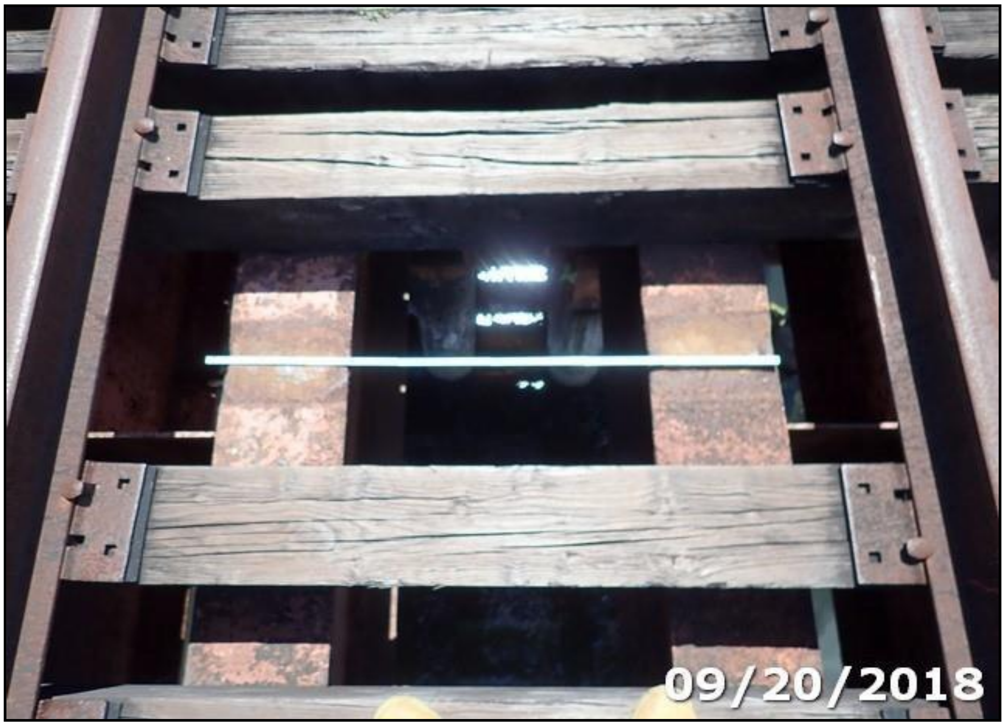



**Photo 03:** Railroad tie 36 is missing.


**Photo 04:** Severe decay and missing sections of the timber guards at both sides of the deck.

## E: PHOTO SECTION


**Photo 05:** Typical corrosion on the bottom flanges of the steel beams.


**Photo 06:** There is graffiti on the steel beams.

**E: PHOTO SECTION**


**Photo 07:** Typical corrosion with section loss up to 50% on the bearing plates and anchorage hardware.


**Photo 08:** Severe corrosion with up to 40% section remaining on the steel pier caps. There is graffiti on the faces of the pier caps. Shown Pier cap 2, north face.

## E: PHOTO SECTION


**Photo 09:** Severe corrosion with up to 40% section remaining on the steel pier caps. There is graffiti on the faces of the pier caps. Shown Pier cap 3, south face.


**Photo 10:** Severe corrosion on the webs and bottom flanges of the abutment caps with up to 100% section loss at the ends.

## E: PHOTO SECTION


**Photo 11:** Severe decay with section loss at the top of the timber piles. Shown Pile 1-2.


**Photo 12:** Severe decay with section loss at the top of the timber piles. Shown Pile 2-1.

## E: PHOTO SECTION


**Photo 13:** The timber piles typically exhibit checks and splits, and below the groundline have decay with soft areas up to 1-1/2in. D.


**Photo 14:** Piles 6-1, 6-3, 6-11, and 6-12 exhibit severe decay and insect activity with up to 20% section remaining. Shown Pile 6-1.

## E: PHOTO SECTION


**Photo 15:** Piles 6-1, 6-3, 6-11, and 6-12 exhibit severe decay and insect activity with up to 20% section remaining. Shown Pile 6-11.


**Photo 16:** The timber piles at Bents 2 and 3 exhibit minor fire damage.

## E: PHOTO SECTION


**Photo 17:** The cross bracing members exhibit areas of severe decay with up to 0% section remaining at the bottom ends.

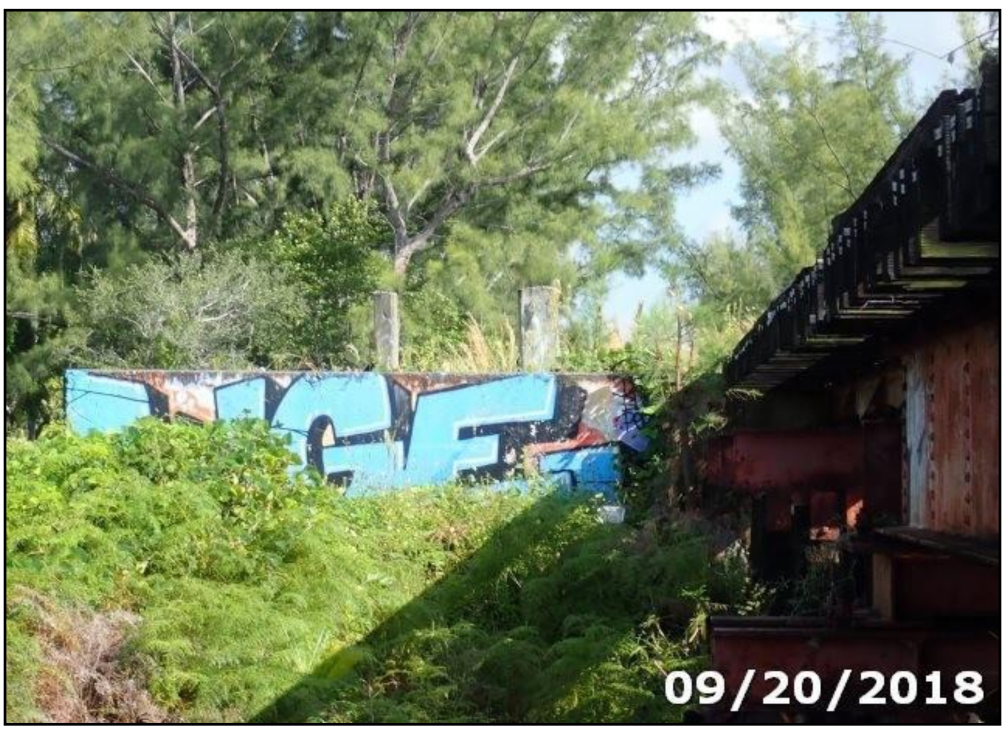



**Photo 18:** Both concrete walls exhibit graffiti throughout.

## E: PHOTO SECTION


**Photo 19:** The channel embankments exhibit erosion at both sides of the bridge.


**Photo 20:** There is drift scattered throughout the channel.

## E: PHOTO SECTION

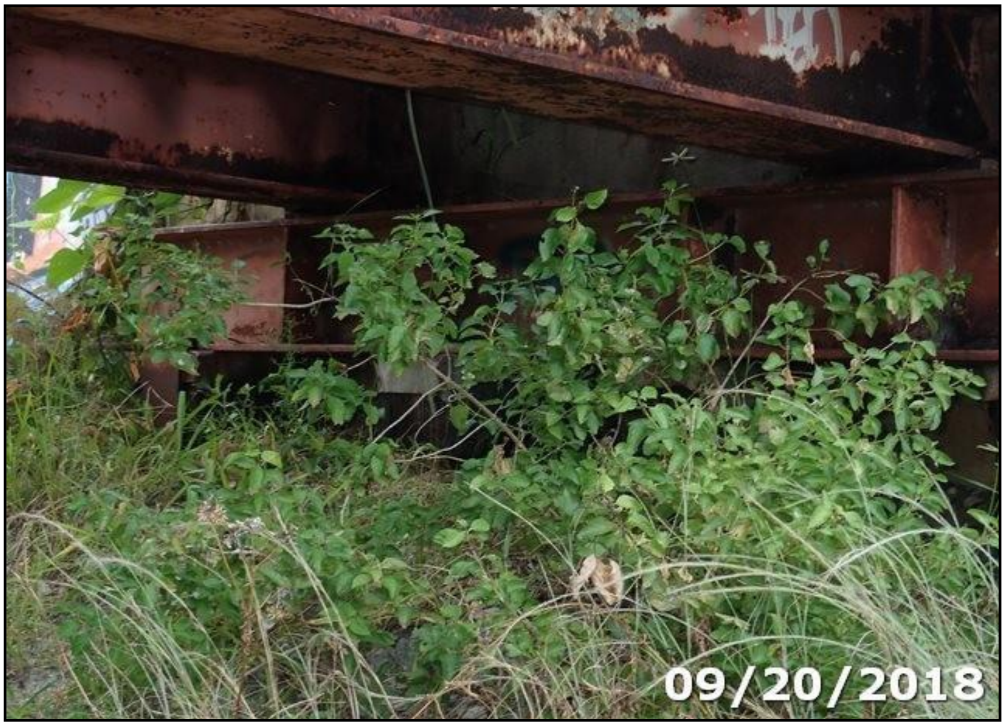



**Photo 21:** Heavy vegetation growth at both channel slopes below the structure.

## F. RECOMMENDED REPAIRS

**Element**

**Timber Deck**
_Replace all timber deck railroad ties and timber guards. Photos 01 thru 04.

**Steel Beams**
_Clean and coat corrosion on the steel beams. Photo 05.

**Slide Bearing**
_Clean and coat corrosion on bearing plates and anchorage hardware. Photo 07.

**Steel Pier Caps**
_Reinforce web to bottom flange section on all steel caps. Photos 08 and 09.

**Steel Abutment**
_Reinforce web to bottom flange section on abutment caps. Photo 10.

**Timber Piles**
_Replace or jacket all timber piles. Photos 11 thru 16.
_Replace all cross bracing members. Photo 17.

**Concrete Wall**
_No corrective action required at the time of this inspection.

**Channel**
_Fill and stabilize eroded areas at the channel embankments. Photo 19.
_Remove drift scattered on the channel. Photo 20.
_Remove heavy vegetation from the channel slopes below the structure. Photo 21.

## G: SCOUR EVALUATION

Profile Data - Numerical Summary

| Bent # | Left Height (ft.) | Right Height (ft.) |
|---|---|---|
| 1 | 4.4 | 3.5 |
| 2 | 7.5 | 7.3 |
| 3 | 11.4 | 9.9 |
| 4 | 12.4 | 12.0 |
| 5 | 17.9 | 15.0 |
| 5.5 | 19.2 | 18.3 |
| 6 | 17.7 | 19.0 |
| 7 | 14.0 | 14.2 |
| 8 | 11.7 | 12.0 |
| 9 | 7.8 | 8.5 |
| 10 | 3.6 | 4.3 |

Notes:

1. Measurements were taken from the top of the deck timber guards. Waterline measurement at mid-channel: Left 12.0 ft. Right 12.3 ft. Maximum Depth: 7.2 ft.
2. There are no previous documented soundings to evaluate any significant scour issues.
3. Current sounding and depth of channel do not reflect any potential scour concerns.

## H. ADDENDUM

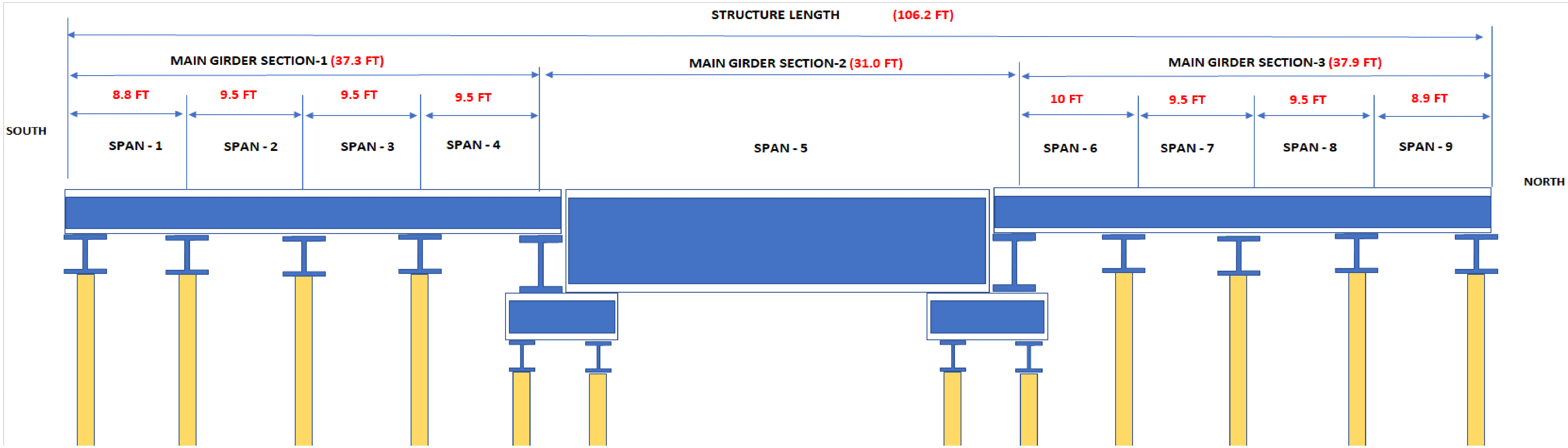



**Diagram 01:** Elevation View

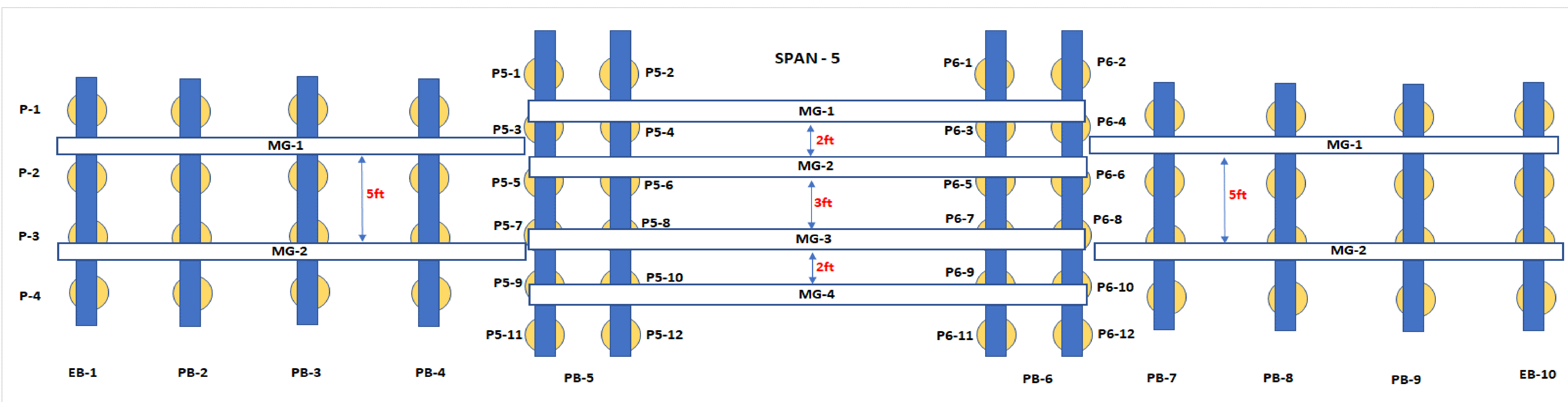



**Diagram 02:** Top View. Main Elements Layout and Numbering.

# PEDESTRIAN BRIDGE INSPECTION REPORT

Structure No: **Ludlam Bridge-2**

## H. ADDENDUM

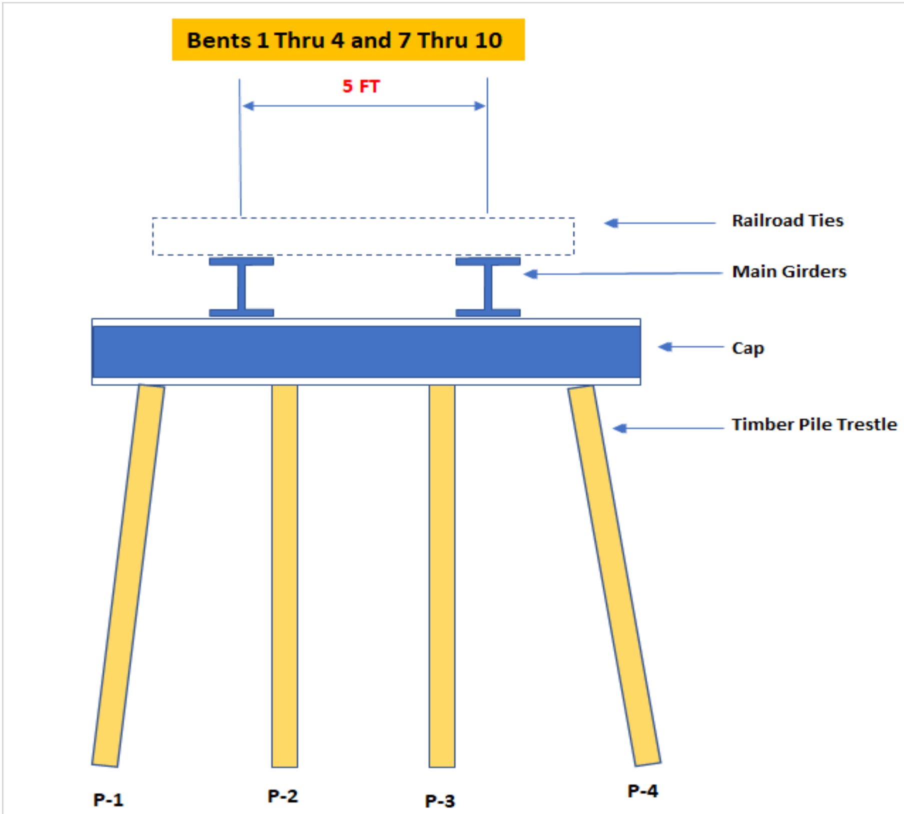



**Diagram 03:** Typical Bent Elevation

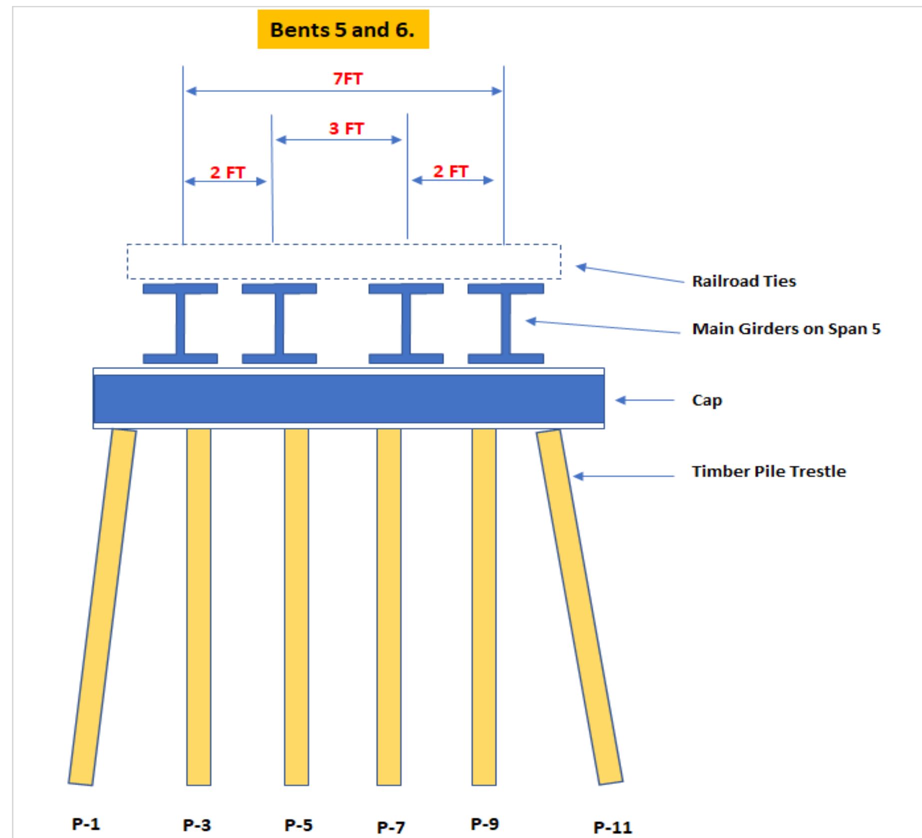
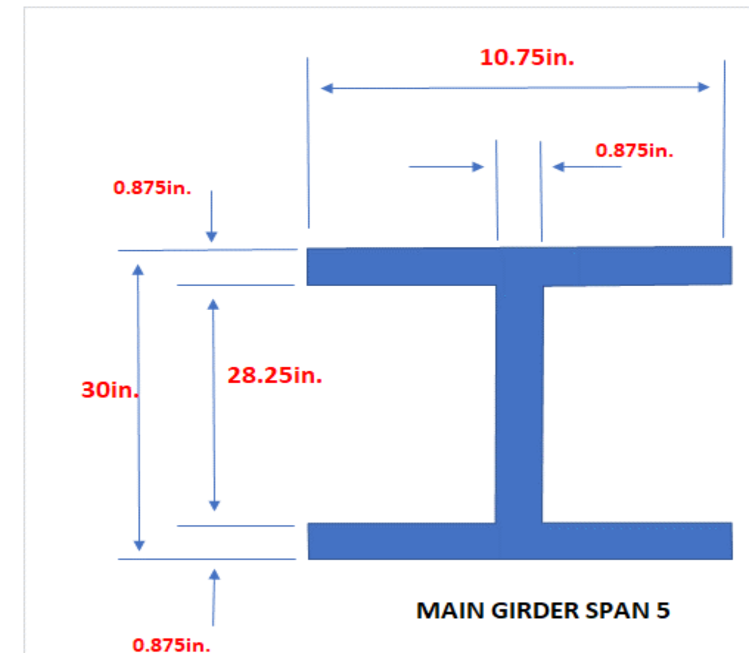



**Diagram 04:** Bents 5 and 6 Elevation



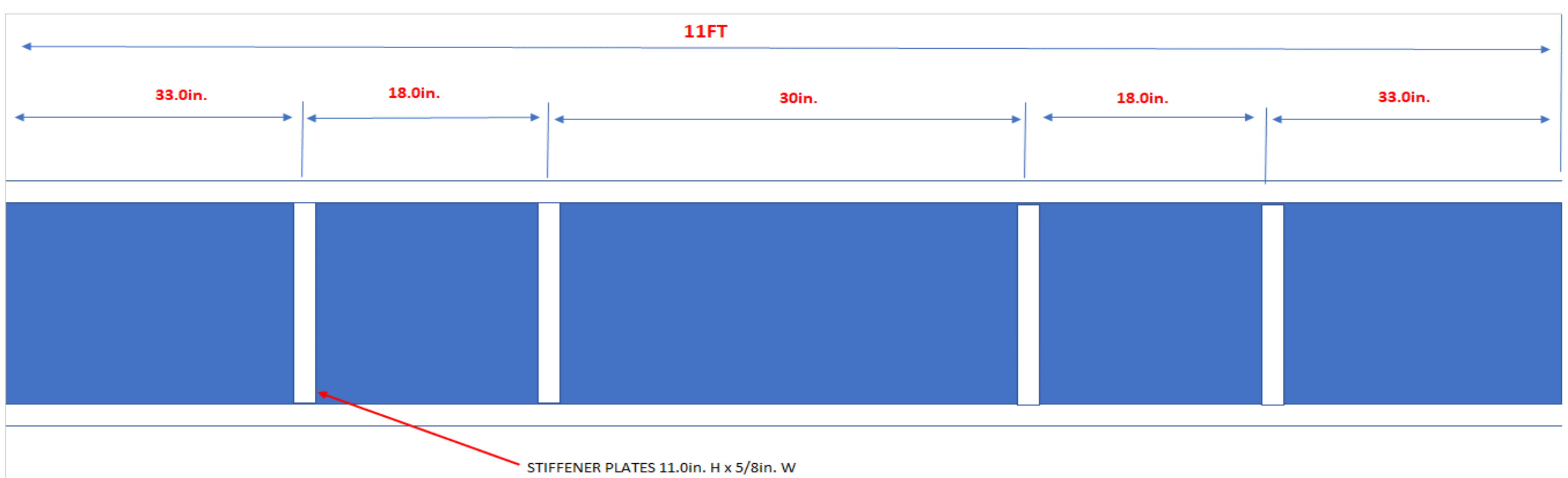
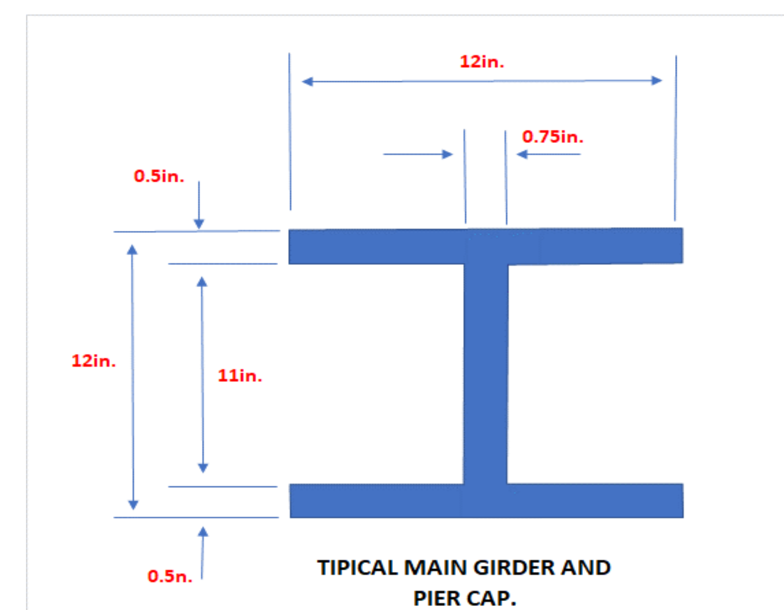
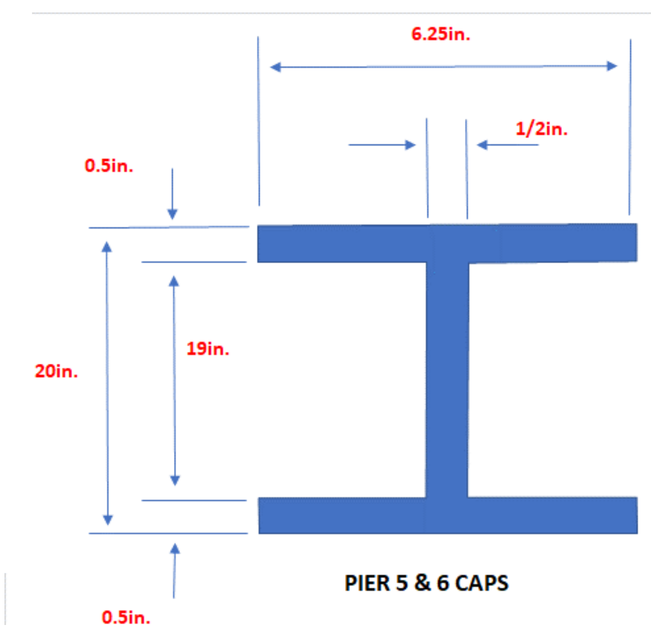




Diagram 05: Typical Pier Cap (Including Abutment Caps)



## H. ADDENDUM


Diagram 06: Bents 5 and 6 Pier Cap



**LUDLAM TRAIL CORRIDIOR BRIDGE 2**
**LOAD RATING ANALYSIS**

**NOVEMBER, 2018**

# LOAD RATING ANALYSIS RESULTS

**PREPARED FOR**
AECOM


**PREPARED BY**
MARLIN Engineering, Inc.
1700 NW 66th Avenue, Suite 106
Plantation, FL 33313
954.870.5070 | www.marlinengineering.com
MARLIN

# Table of Contents

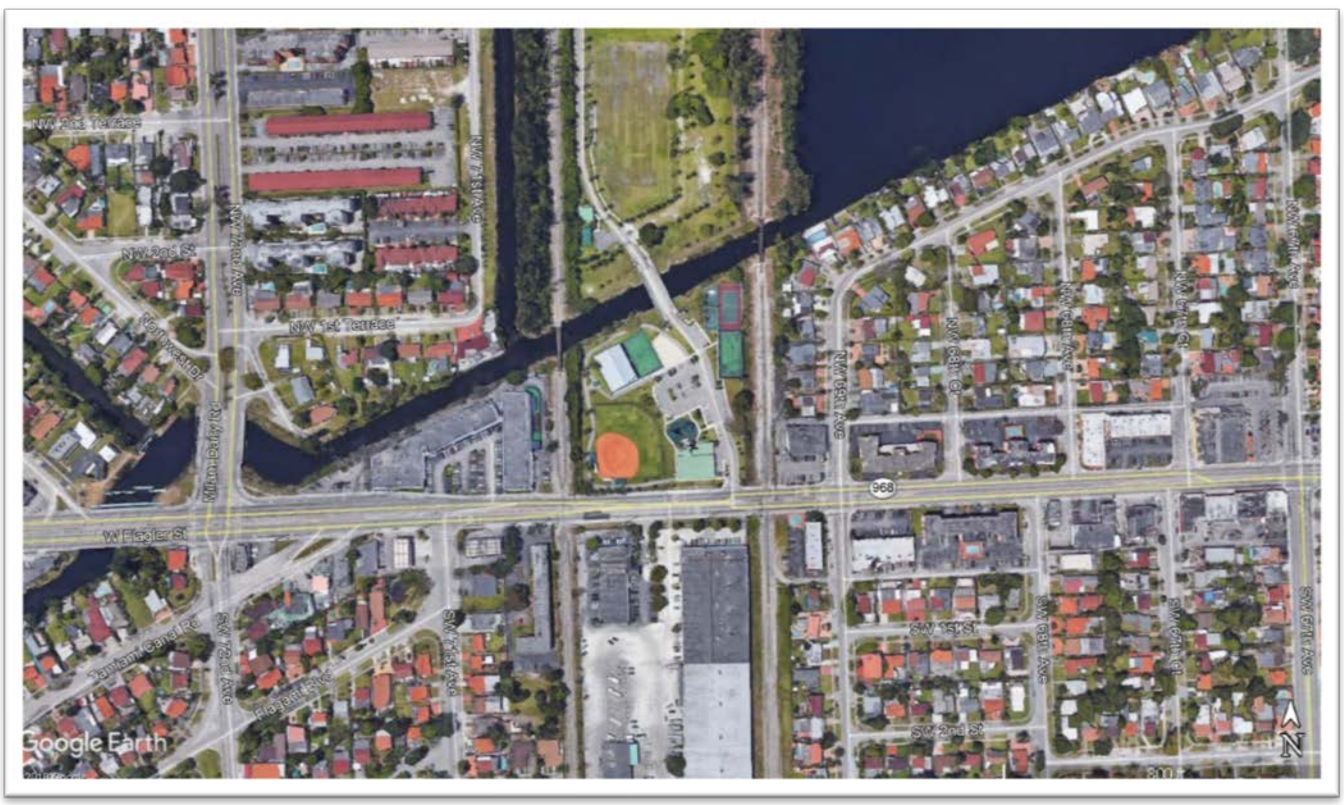
# Executive Summary

Ludlum Trail Corridor Bridge 2 crossing the Tamiami (C-4) Canal was load rated for future potential use as a shared-use path with light maintenance emergency vehicle access. The bridge was originally an FEC rail line that has been abandoned. No existing plans were provided for this analysis.

A load rating analyses of the superstructure, based on assumptions outlined in this report, and on findings of a bridge inspection performed by Marlin Engineering indicate that with the removal of the exiting rail road tie deck and installation of a properly designed reinforced concrete deck, the bridge would have restrictions for Florida Legal Load SU4 on the superstructure.

# Bridge Location

Ludlam Trail Corridor Bridge is located approximately 0.1 mile north of SR-986 (W Flagler Street).




**Location of Bridge 2**

## Load Rating Assumptions

The following assumptions were made for this load rating analysis:

1. The load rating was performed using LRFR method.
2. The load rating analysis was performed for an approach span and the main span. The load rating is controlled by the four-girder 31'-0" main span which is taken to be simply supported.
3. Reduction in girder section properties was taken to account for documented section loss.
4. A non-composite reinforced concrete deck was analyzed in lieu of the existing timber rail tie deck.
5. No allowance for the weight of stay-in-place forms is included in the load rating.
6. No allowance for future wearing surface is included in the load rating.
7. A fictitious concrete deck with a width of 18.0 ft. was used for the computation of Dead Load ONLY. **The existing superstructure CANNOT be used to support an 18 ft. wide deck. Increasing** the deck width to 18 ft. would require the design of a new superstructure and substructure, and subsequent post-design load rating.

## Bridge Information

| | |
|---|---|
| Max Span Length: | 31.0 ft. |
| Bridge Width: | 18.0 ft.* |
| Deck Thickness: | 10 in. |
| Girder Dimensions: | As shown in Load Rating calculations |
| Skew Angle: | 0 deg. |
| Concrete Unit Wight: | 150 pcf for deck concrete |
| Steel Unit Weight; | 490 pcf |
| Steel Yield Strength: | 36 ksi |

*Used in the computation of dead load only. **The existing superstructure cannot be used to support an 18 ft. wide deck.**

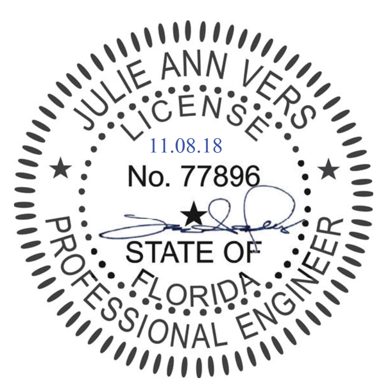
| | | | | |
|---|---|---|---|---|
| **Bridge No.** | **Ludlam Bridge 2** | **Analysis Method:** | **LRFR-LRFD** | **FDOT Bridge Load Rating Summary Form (Page 1 of 1)** |
| **Location** | **Ludlam Trail over Tamiami (C-4) Canal** | | | |
| **Description** | **Single Span - Four Girder Bridge** | | | |

| Rating Type | Rating Type | Gross Axle Weight (tons) | Moment/Shear/Service | | Dead Load Factor | Live Load Factor | Live Load Distrib. Factor (axles) | Rating Factor | Span No. - Girder No., Interior/Exterior, %Span·L | RF·Weight (tons) |
|---|---|---|---|---|---|---|---|---|---|---|
| **Level** | **Vehicle** | **Weight** | **Member Type** | **Limit** | **DC** | **LL** | **LLDF** | **RF** | **Governing Location** | **RATING** |
| Inventory | HL93 | 36 | Steel | Strength, Moment | 1.25/0.90 | 1.75 | 1.000 | 0.883 | 0.5L | 31.8 |
| Operating | HL93 | 36 | Steel | Strength, Moment | 1.25/0.90 | 1.35 | 1.000 | 1.145 | 0.5L | 41.2 |
| Permit | FL120 | 60 | Steel | Strength, Moment | 1.25/0.90 | 1.35 | 1.000 | 0.941 | 0.5L | 56.5 |
| Permit Max Span | FL120 | 60 | Steel | Strength, Moment | 1.25/0.90 | 1.35 | 1.000 | 0.941 | 0.5L | 56.5 |
| Legal | SU2 | 17 | Steel | NA | NA | NA | | | | -1 |
| | SU3 | 33 | Steel | NA | NA | NA | | | | -1 |
| | SU4 | 35 | Steel | Strength, Moment | 1.25/0.90 | 1.35 | 1.000 | 0.881 | 0.5L | 30.8 |
| | C3 | 28 | Steel | Limit Test | NA | NA | | | | -1 |
| | C4 | 36.7 | Steel | Limit Test | NA | NA | | | | -1 |
| | C5 | 40 | Steel | Strength, Moment | 1.25/0.90 | 1.35 | 1.000 | 1.115 | 0.5L | 44.6 |
| | ST5 | 40 | Steel | Strength, Moment | 1.25/0.90 | 1.35 | 1.000 | 1.196 | 0.5L | 47.8 |

| | | |
|---|---|---|
| *Original* Design Load | Rail | |
| Rating Type, *Analysis* | LRFR-LRFD | |
| Distribution Method | Others | |
| Impact Factor | 33.0% | (axle loading) |
| FL120 Gov. Span Length | N/A | (feet) |
| Recommended Posting | 10.0 to 19.9% below (0.801-0.900) (Required) | |
| Recommended SU Posting* | 30 | (tons) |
| Recommended C Posting | 99 | (tons) |
| Recommended ST5 Posting | 99 | (tons) |
| Floor Beam Present? | No | |
| Segmental Bridge? | No | |
| Project No. & Reason | N/A | Other |
| Plans Status | NA (use field measurements) | |
| Software Name, Version | Mathcad | |

| | | | |
|---|---|---|---|
| Performed by: | JAV | Date: | 10.25.18 |
| Checked by: | BKR | Date: | 11.03.18 |
| Sealed By: | JAV | Date: | 10.25.18 |
| FL P.E. No.: | 77896 | | |
| Cert. Auth. No.: | 6104 | | |
| Phone & email: | 561.229.0239 - jvers@marlinengineering.com | | |
| Company: | MARLIN Eningeering, Inc. | | |
| Address: | 10415 Riversife Drive, Palm Beach Gardens, FL 33410 | | |

P.E. Seal




COMMENTS BY THE ENGINEER

This 12-01-2017 summary follows the FDOT Bridge Load Rating Manual (BLRM), and the FDOT BMS Coding Guide.
*Recommended SU Posting levels for Florida SU trucks adequately restricts AASHTO SU trucks; see BLRM Chapter 7.

fdot.gov/maintenance/LoadRating.shtm

# Load Rating Calculations:

# Load and Resistance Factor Rating (LRFR) for Noncomposite Steel Bridge

| | | | |
|---|---|---|---|
| SUBJECT | Ludlam Bridge - 2 | | |
| PROJECT# | 2018038.000 | | |
| DESIGNED BY | JAV | DATE | 10.27.18 |
| CHECKED BY | BKR | DATE | 11.03.18 |

***References:***

- ***FDOT "Bridge Load Ratin Manual, 2018"***
- ***AASHTO "Manual for Bridge Evaluation, 2nd Edition with 2015 Revisions"***
- ***AASHTO "Bridge Design Specifications, Seventh Edition"***
- ***Bridge Inspection Report by MARLIN dated 09.19.18***

**Girder Section Properties:**

Width of Flange: $b_f := 10.75in$

Reduction Factor for Section Loss: $SL := 0.95$

Thickness of Flange: $t_f := SL \cdot 0.875in = 0.831 \cdot in$

Thickness of Web: $t_w := SL \cdot 0.875in = 0.831 \cdot in$

Height of Member Minus the Flange and Curved Radius: $h := 28.25in$

Area of Girder: $A_g := b_f \cdot 2 \cdot t_f + h \cdot t_w = 41.355 \cdot in^2$

Plastic Section Modulus: $Z_x := 427.4in^3$

Elastic Section Modulus: $S_x := 358.5in^3$

Depth of Member: $D := 30.0in$

Torsional Constant: $J := 10.01in^4$

**Geometry:**

Span Length of Girders: $Span := 31ft + 0in$

Number of Girders: $n_{girders} := 4$

Unbraced Span Length of Girders: $Span_{unbraced} := 31ft + 0in$

Unbraced Length: $L_b := Span_{unbraced}$

Span Length of Deck: $Span_{deck} := 5ft + 0in$

Span Length of Deck: $Span_{deck2} := 2ft + 0in$

Overhang Length of Deck: $Overhang := 4ft + 6in$

Thickness of Deck: $t_{deck} := 10in$

Width of Bridge: $Width := 18ft + 0in$

## Material Properties:

Year Built: Year := "unknown"

Steel Yield Strength: $F_y := 36ksi$

Steel Moduus of Elasticity: $E := 29000ksi$

Density of Steel: $\gamma_{steel} := 490pcf$

Density of Concrete: $\gamma_{conc} := 150pcf$

## Load Factors:

Wearing Course Load Factors: $\gamma_{DW} := 1.25$ $\gamma_{DW.o} := 1.25$

Component Load Factors: $\gamma_{DC} := 1.25$ $\gamma_{DC.o} := 1.25$

Live and Impact Load Factors: $\gamma_{LL.IM} := 1.75$ $\gamma_{LL.IM.o} := 1.35$

Service Dead Load Factors: $\gamma_D := 1.0$

Service Live Load Factors: $\gamma_{LL.IMs} := 1.3$ $\gamma_{LL.IMso} := 1.0$

Strength Legal Live Load Factor: $\gamma_L := 1.40$

Service Legal Live Load Factor: $\gamma_{Ls} := 1.30$

## Evaluation Factors for Strength Limit States:

Resistance Factor for Flexure: $\varphi := 1.0$

Condition Factor for Poor Condition: $\varphi_c := 1.0$

System Factor for Slab Bridges: $\varphi_s := 1.0$

Tire Load from Design Truck: $P_{truck} := 16kip$

Tire load from the design tandem $P_{tandem} := 12.5kip$

Multiple Presence Factor, Single Lane Loaded: $m_{single} := 1.0$ (Use 1.0 as MPF's not applicable to this one-lane bridge)

Multiple Presence Factor, Double Lanes Loaded: $m_{double} := 1.0$

Dynamic Load Allowance: $IM := 33\%$

## DC Component:

Weight of Deck Units: $wt_{deck} := \gamma_{conc} \cdot t_{deck}$

Deck Dead Load: $DC_{deck} := wt_{deck} \cdot (0.5 \cdot Span_{deck} + Overhang) = 0.875 \cdot klf$

Weight of Girder: $wt_{girder} := \gamma_{steel} \cdot A_g = 0.141 \cdot klf$

Girder Dead Load: $DC_{girder} := wt_{girder} = 0.141 \cdot klf$

Height of Curb: $h_{curb} := 8in$

Width of Curb: $b_{curb} := 7in$

Weight of Curb: $wt_{curb} := h_{curb} \cdot b_{curb} \cdot \gamma_{conc} = 0.058 \cdot klf$

Number of Curbs: $n_{curb} := 2$

Curb Dead Load: $DC_{curb} := \dfrac{n_{curb} \cdot wt_{curb}}{n_{girders}} = 0.029 \cdot klf$

Moment due to Component Loads: $M_{DC} := \dfrac{(DC_{deck} + DC_{girder} + DC_{curb}) \cdot Span^2}{8} = 125.517 \cdot kip \cdot ft$

Shear due to Component Loads: $V_{DC} := \dfrac{(DC_{deck} + DC_{girder} + DC_{curb}) \cdot Span}{2} = 16.196 \cdot kip$

## DW Component:

Weight of Wearing Layer: $wt_{wearing} := 0psf$

Load of Wearing Layer: $DW_{wearing} := wt_{wearing} \cdot (0.5 \cdot Span_{deck} + Overhang) = 0$

Moment due to Wearing Loads: $M_{DW} := \dfrac{DW_{wearing} \cdot Span^2}{8} = 0$

Shear due to Wearing Loads: $V_{DW} := \dfrac{DW_{wearing} \cdot Span}{2} = 0$

## Live Load Analysis

*Conservatively assume that wheel is directly over girder and less than 2' away from curb.

Design Lane Load: $Lane := 64psf \cdot (0.5 \cdot Span_{deck2} + Overhang) = 0.352 \cdot klf$

Reaction from the Design Truck: $R_{truck} := m_{single} \cdot P_{truck} = 16 \cdot kip$

Reaction from the Design Tandem: $R_{tandem} := m_{single} \cdot P_{tandem} = 12.5 \cdot kip$

## Controlling Live Load on the Girders:

Design Live Load Moment on the Exterior Beam including Impact and Lane Loads: $M_{LL.IM} := \frac{506.2}{2} kip \cdot ft = 253.1 \cdot kip \cdot ft$

(FDOT Load Rating Manual, 2018)

Design Live Load Shear on the Exterior Beam including Impact and Lane Loads: $V_{LL.IM} := \frac{75.6}{2} kip$

(FDOT Load Rating Manual, 2018)

## Nominal Flexural Resistance of the Section:

### Check that Appendix A6 of AASHTO LRFD is appropriate:

Depth of the web in compression in the elastic range $D_{cp} := 0.5D$

Depth of the web in compression at the plastic moment $D_c := D_{cp}$

Check that the web satisfies the noncompact slenderness limit [A6.1-1] $\frac{2 \cdot D_{cp}}{t_w} \le 5.7 \cdot \sqrt{\frac{E}{F_y}} = 1$ 6.10.6.2.3-1

$$Check_{NC.slenderness} := if\left(\frac{2 \cdot D_{cp}}{t_w} \le 5.7 \cdot \sqrt{\frac{E}{F_y}}, "OK", "NG"\right) = "OK"$$

With the girder having symmetric flanges the check of $I_{yc} / I_{yt} > 0.3$ is met. Appendix A6 is appropriate.

Limiting slenderness ratio for noncompact web [A6.2.1-3] $\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E}{F_y}}$

Hybrid factor for rolled shapes [6.10.1.10.1] $R_h := 1.0$

Redefinition of variables $b_c := b_f \quad t_c := t_f \quad b_t := b_f \quad t_t := t_f$

Plastic compression force in the compression flange $P_c := F_y \cdot b_c \cdot t_c$

Plastic compression force in the web $P_w := F_y \cdot D \cdot t_w$

Plastic compression force in the tension flange $P_t := F_y \cdot b_t \cdot t_t$

Distance from the plastic neutral axis to the top of the element where the plastic neutral axis is located [D6.1] $Y_{bar} := \frac{D}{2} \cdot \left(\frac{P_t - P_c}{P_w} + 1\right) = 15 \cdot in$

Distance from the plastic neutral axis to the midthickness of the compression flange $d_c := Y_{bar} - 0.5 \cdot t_f$

Distance from the plastic neutral axis to the midthickness of the tension flange $d_t := Y_{bar} - 0.5 \cdot t_f$

Plastic moment $M_p := \frac{P_w}{2 \cdot D} \cdot \left[Y_{bar}^2 + (D - Y_{bar})^2\right] + P_c \cdot d_c + P_t \cdot d_t = 1.343 \times 10^3 \cdot kip \cdot$

Yield moment $M_y := F_y \cdot S_x = 1.075 \times 10^3 \cdot kip \cdot ft$

Limiting slenderness ratio for compact web [A6.2.1-2]

$$\lambda_{pw} := \frac{\sqrt{E \div F_y}}{\left(0.54 \cdot \frac{M_p}{R_h \cdot M_y} - 0.09\right)^2}$$

$$\lambda_{pw} \le \lambda_{rw} \cdot \frac{D_{cp}}{D_c} = 1$$

$$Check_{compact.web} := if\left(\lambda_{pw} \le \lambda_{rw} \cdot \frac{D_{cp}}{D_c}, "OK", "NG"\right) = "OK"$$

The web is compact.

**Check Compression Flange Buckling:**

Moment of inertia of the compression flange plus one-third of the web depth

$$I_{yc} := \frac{t_f \cdot b_f^{\,3}}{12} + \frac{(D \div 3) \cdot t_w^{\,3}}{12} = 86.534 \cdot in^4$$

Area of the compression flange plus one-third of the web depth

$$A_c := t_f \cdot b_f + (D \div 3) \cdot t_w = 17.248 \cdot in^2$$

Radius of gyration of the compression flange plus one-third of the web depth

$$r_t := \sqrt{I_{yc} \div A_c} = 2.24 \cdot in$$

Check to see if the compression flange is adequately braced

$$1.76 \cdot r_t \cdot \sqrt{\frac{E}{F_y}} \ge L_b = 0$$

$$Check_{CF.braced} := if\left(1.76 \cdot r_t \cdot \sqrt{\frac{E}{F_y}} \ge L_b, "CF\ Adequately\ Braced", "CF\ NOT\ Adequately\ Braced"\right)$$

$$Check_{CF.braced} = "CF\ NOT\ Adequately\ Braced"$$

The compression flange is not adequately braced.

**Compression Flange Local Buckling:**

Slenderness ratio of the flange [A6.3.2-3]

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 6.466$$

Limiting slenderness ratio for a compact flange [A6.3.2-4]

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785$$

Flange local buckling coefficient for rolled shapes [A6.3.2]

$k_c := 0.76$

Limiting slenderness ratio for a noncompact flange [A6.3.2-5]

$\lambda_{rf} := 0.95 \cdot \sqrt{\frac{E \cdot k_c}{F_y}}$

Web plastification factor for the compression flange [A6.2.1-4]

$R_{pc} := M_p \div M_y$

Elastic section modulus about the major axis of the section to the compression flange

$S_{xc} := M_y \div F_y$

Nominal flexural resistance based on compression flange local buckling [A6.3.2-2]

$$M_{n.FLB} := \begin{cases} R_{pc} \cdot M_y & \text{if } \lambda_f \le \lambda_{pf} \\ \left[\left[1 - \left(1 - \frac{F_y \cdot S_{xc}}{R_{pc} \cdot M_y}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \cdot R_{pc} \cdot M_y\right] & \text{otherwise} \end{cases}$$

$M_{n.FLB} = 1.343 \times 10^3 \cdot ft \cdot kip$

**Lateral Torsional Buckling:**

Limiting unbraced length to achieve nominal flexural resistance [A6.3.3-4]

$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_y}} = 63.572 \cdot in$

Limiting unbraced length to achieve nominal onset of yielding [A6.3.3-5]

$L_r := 1.95 \cdot r_t \cdot \frac{E}{F_y} \cdot \sqrt{\frac{J}{S_{xc} \cdot h}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{F_y}{E} \cdot \frac{S_{xc} \cdot h}{J}\right)^2}} = 232.427 \cdot i$

Moment gradient modifier

$C_b := 1.0$

Elastic lateral torsional buckling stress [A6.3.3-8]

$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{(L_b \div r_t)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J}{S_{xc} \cdot h} \cdot \left(\frac{L_b}{r_t}\right)^2}$

Flexural resistance based on lateral torsional buckling [A6.3.3]

$$M_{n.LTB} := \begin{cases} (R_{pc} \cdot M_y) & \text{if } L_b \le L_p \\ \min(F_{cr} \cdot S_{xc}, R_{pc} \cdot M_y) & \text{if } L_b > L_r \\ \left[C_b \cdot \left[1 - \left(1 - \frac{F_y \cdot S_{xc}}{R_{pc} \cdot M_y}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_{pc} \cdot M_y\right] & \text{otherwise} \end{cases}$$

$M_{n.LTB} = 548.133 \cdot ft \cdot kip$

Nominal flexural resistance based on the compression flange

$M_n := \min(M_{n.FLB}, M_{n.LTB}) = 548.133 \cdot kip \cdot ft$

**Shear Resistance:**

Shear buckling coefficient $k := 5$

Ratio of shear-buckling resistance to shear yield strength [6.10.9.2.3-4,5and6]

$$C := \begin{cases} 1.0 & \text{if } \frac{D}{t_w} \le 1.12 \cdot \sqrt{\frac{E \cdot k}{F_y}} \\ \left[\frac{1.57}{\left(D \div t_w\right)^2} \cdot \left(\frac{E \cdot k}{F_y}\right)\right] & \text{if } \frac{D}{t_w} > 1.4 \cdot \sqrt{\frac{E \cdot k}{F_y}} \\ \left(\frac{1.12}{D \div t_w} \cdot \sqrt{\frac{E \cdot k}{F_y}}\right) & \text{otherwise} \end{cases}$$

Nominal shear resistance of unstiffendd web [6.10.9.2-1and2]

$$V_n := C \cdot 0.58 \cdot F_y \cdot D \cdot t_w = 520.695 \cdot kip$$

**Service II Stresses:**

Allowable stress for service limit state specified in the LRFD code for non-composite section:

$$f_r := 0.80 \cdot F_y$$

Component Dead Load Stress:

$$f_{DC} := M_{DC} \div S_x = 4.201 \cdot ksi$$

Wearing Dead Load Stress:

$$f_{DW} := M_{DW} \div S_x = 0$$

Live Load Stress:

$$f_{LL.IM} := M_{LL.IM} \div S_x = 8.472 \cdot ksi$$

## Design Load Rating:

### General Load-Rating Equation:

General Load Rating Equation: $$RF = \frac{C - \gamma_{DC} \cdot DC - \gamma_{DW} \cdot DW}{\gamma_L \cdot (LL + IM)} + -\frac{\gamma_p \cdot P}{\gamma_L \cdot (LL + IM)}$$

### Strength I Limit State:

Inventory Rating Factor: $$M_{RF.I} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL.IM} \cdot M_{LL.IM}} = 0.883$$

Operating Rating Factor: $$M_{RF.O} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC.o} \cdot M_{DC} - \gamma_{DW.o} \cdot M_{DW}}{\gamma_{LL.IM.o} \cdot M_{LL.IM}} = 1.145$$

Inventory Rating Factor: $$V_{RF.I} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot V_n - \gamma_{DC} \cdot V_{DC} - \gamma_{DW} \cdot V_{DW}}{\gamma_{LL.IM} \cdot V_{LL.IM}} = 7.565$$

Operating Rating Factor: $$V_{RF.O} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot V_n - \gamma_{DC.o} \cdot V_{DC} - \gamma_{DW.o} \cdot V_{DW}}{\gamma_{LL.IM.o} \cdot V_{LL.IM}} = 9.807$$

### Service II Limit State:

Inventory Rating Factor: $$RF_I := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{LL.IMs} \cdot f_{LL.IM}} = 2.233$$

Operating Rating Factor: $$RF_O := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{LL.IMso} \cdot f_{LL.IM}} = 2.904$$

## Summary of Rating Factors:

RATING_FACTORS :=

| "Limit State" | "Inventory Design Load Rating" | "Operating Design Load Rating" |
|---|---|---|
| "Strength I Flexure" | $M_{RF.I}$ | $M_{RF.O}$ |
| "Strength I Shear" | $V_{RF.I}$ | $V_{RF.O}$ |
| "Service II" | $RF_I$ | $RF_O$ |

DESIGN := | "Perform legal load rating" if $\min(M_{RF.I}, M_{RF.O}, V_{RF.I}, V_{RF.O}, RF_I, RF_O) < 1$
| "Legal load ratings are not required" otherwise

RATING_FACTORS =

| "Limit State" | "Inventory Design Load Rating" | "Operating Design Load Rating" |
|---|---|---|
| "Strength I Flexure" | 0.883 | 1.145 |
| "Strength I Shear" | 7.565 | 9.807 |
| "Service II" | 2.233 | 2.904 |

DESIGN = "Perform legal load rating"

**Legal Load Rating:**

Maximum factor used earlier in lever rule calculations to distribute tire loads onto the girders

$\varphi_{lever} := 1.00$

Moment from SU4 permit vehicle

(FDOT Load Rating Manual, 2018)

$P_1 := \varphi_{lever} \cdot 18.7kip \div 2 \qquad P_2 := \varphi_{lever} \cdot 13.9kip \div 2$

$M_{SU4} := \frac{476.82}{2} \cdot kip \cdot ft = 238.41 \cdot kip \cdot ft$

(FDOT Load Rating Manual, 2018)

Moment from the C5 permit vehicle

$P := \varphi_{lever} \cdot 22kip \div 2$

$M_{C5} := \frac{376.9}{2} \cdot kip \cdot ft = 188.45 \cdot kip \cdot ft$

Moment from the ST5 permit vehicle

(FDOT Load Rating Manual, 2018)

$P := \varphi_{lever} \cdot 18kip \div 2$

$M_{ST5} := \frac{351.4}{2} \cdot kip \cdot ft = 175.7 \cdot kip \cdot ft$

Moment from the FL120 permit vehicle
(FDOT Load Rating Manual, 2018)

$P := \varphi_{lever} \cdot 53.3kip \div 2$

$M_{FL120} := \frac{625.4}{2} \cdot kip \cdot ft = 312.7 \cdot kip \cdot ft$

**Strength I Limit State:**

Flexure rating factor for SU4 loading

$$RF_{SU4.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_L \cdot \left[(1 + IM) M_{SU4}\right]}$$

$RF_{SU4.SI} = 0.881$

Flexure rating factor for C5 loading

$$RF_{C5.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_L \cdot \left[(1 + IM) M_{C5}\right]}$$

$RF_{C5.SI} = 1.115$

Flexure rating factor for ST5 loading

$$RF_{ST5.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_L \cdot \left[(1 + IM) M_{ST5}\right]}$$

$RF_{ST5.SI} = 1.196$

Flexure rating factor for FL120 loading

$$RF_{FL120.SI} := \frac{\varphi_c \cdot \varphi_s \cdot \varphi \cdot M_n - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{1.0 \cdot \left[(1 + IM) M_{FL120}\right]}$$

$RF_{FL120.SI} = 0.941$

**Service II Limit State**

Flexure rating factor for SU4 loading

$$RF_{SU4.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{SU4} \div S_x\right]}$$

$RF_{SU4.SII} = 1.783$

Flexure rating factor for C5 loading

$$RF_{C5.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{C5} \div S_x\right]}$$

$RF_{C5.SII} = 2.255$

Flexure rating factor for ST5 loading

$$RF_{ST5.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{ST5} \div S_x\right]}$$

$RF_{ST5.SII} = 2.419$

Flexure rating factor for FL120 loading

$$RF_{FL120.SII} := \frac{f_r - \gamma_D \cdot f_{DC} - \gamma_D \cdot f_{DW}}{\gamma_{Ls} \cdot \left[(1 + IM) M_{FL120} \div S_x\right]}$$

$RF_{FL120.SII} = 1.359$

**Posted Load Rating:**

SU4 Posting: $P_{RF.SU4} := 70kip \cdot \left(\min\left(RF_{SU4.SI}, RF_{SU4.SII}\right)\right) = 30.846\,tonf$

C5 Posting: $P_{RF.C5} := 56kip \cdot \left(\min\left(RF_{C5.SI}, RF_{C5.SII}\right)\right) = 62.438\,kip$

ST5 Posting: $P_{RF.ST5} := 80kip \cdot \left(\min\left(RF_{ST5.SI}, RF_{ST5.SII}\right)\right) = 95.671\,kip$

FL120 Posting: $P_{RF.FL120} := 120kip \cdot \left(\min\left(RF_{FL120.SI}, RF_{FL120.SII}\right)\right) = 112.886\,kip$

$$POSTED\_RATING := \begin{pmatrix} \text{"SU4"} & if\left(\min\left(RF_{SU4.SI}, RF_{SU4.SII}\right) > 1, \text{"Not Required"}, P_{RF.SU4}\right) \\ \text{"C5"} & if\left(\min\left(RF_{C5.SI}, RF_{C5.SII}\right) > 1, \text{"Not Required"}, P_{RF.C5}\right) \\ \text{"ST5"} & if\left(\min\left(RF_{ST5.SI}, RF_{ST5.SII}\right) > 1, \text{"Not Required"}, P_{RF.ST5}\right) \\ \text{"FL120"} & if\left(\min\left(RF_{FL120.SI}, RF_{FL120.SII}\right) > 1, \text{"Not Required"}, P_{RF.FL120}\right) \end{pmatrix}$$

Required Posted Load Rating:

$$POSTED\_RATING = \begin{pmatrix} \text{"SU4"} & 30.8 \\ \text{"C5"} & \text{"Not Required"} \\ \text{"ST5"} & \text{"Not Required"} \\ \text{"FL120"} & 56.4 \end{pmatrix} \cdot tonf$$