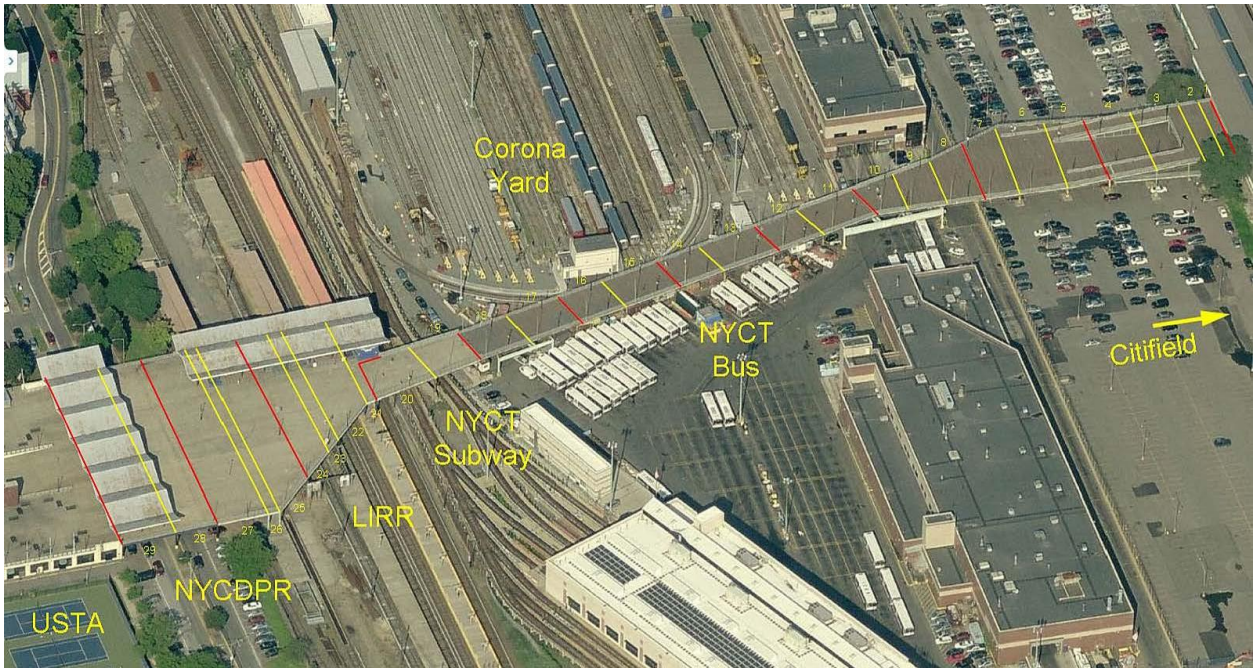
NYC DEPARTMENT OF DESIGN AND CONSTRUCTION
DIVISION OF INFRASTRUCTURE

PROJECT ID:
HBPED700Q

DESIGN REPORT
FOR PRE-SCOPING SERVICES

BOOK 2 OF 2



PASSERELLE PEDESTRIAN BRIDGE
BOROUGH OF QUEENS

OCTOBER 2014

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

Appendices

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1.0 Introduction

1.1 Engineering Agreement

On behalf of the New York City Department of Design and Construction (NYCDDC), HAKS and Weidlinger Associates, Inc., who have teamed up as a joint venture, conducted preliminary engineering services for the Passerelle Pedestrian Bridge located in Flushing Meadows-Corona Park, Flushing, New York. The purpose of the pre-scoping services is to collect information and perform the work necessary to determine the integrity of existing structural condition, especially deck deficiencies, so that a recommendation to rehabilitate and/or replace the bridge and its components can be provided. The assigned executive Contract Number is 20131423135.

The original scope of work agreed upon in May 2013 included tasks that were required to enable joint venture and its subconsultants to provide studies as required and recommendations to rehabilitate or replace the bridge. The studies include topographic survey and right of way (ROW) determination, existing deck cores testing, hazardous material investigation, soil investigation, pedestrian count program, in-depth inspection, and structural evaluation. The project deliverables combine all the above studies accompanied by conclusion and recommendations at the end including schematic design alternatives, preliminary design drawings, and cost estimates to be carried over into the final design contract (not included in this contract) at the end of year 2014.

The purpose of this project is to develop alternatives for the full reconstruction of the Passerelle Pedestrian Bridge.

1.2 Project Area Description and History

The Passerelle Pedestrian Bridge is owned by New York City Department of Parks & Recreation (NYCDPR). The pedestrian overpass was originally built for the 1939 World's Fair to transport people between Roosevelt Avenue and the fair entrance and underwent major reconstruction for the 1964 World's Fair that involved the complete replacement of the superstructure. It also added more foundations and built two new buildings called the Passerelle Building (now functions as a NYCDPR maintenance building). The structure is a multi-span steel trestle frame with timber and concrete/timber decking with a total length of approximately 1,100 feet and widths ranging from 40 feet at the main walkway to 230 feet over the LIRR area. The condition of the structure has visibly deteriorated since its last rehabilitation for the 1964 World's Fair.

The bridge spans over (from north to south) the NY Mets Southfield parking lot managed by IMPARK; the New York City Transit (NYCT) Corona maintenance yard for subway trains and buses; LIRR Port Washington Line tracks and platforms that are part of the Mets-Willets Point station; and NYCDPR property that consists of a parking area, two park roads (Perimeter Road), and a sidewalk.

The north end is the beginning of the bridge, where there is an elevated MTA No. 7 (Roosevelt Avenue) subway track structure adjacent to the bridge which is perpendicular in orientation. The MTA station is not part of the bridge. The Bridge has three levels from spans 1 through 5; the

upper, mezzanine, and lower levels. The upper and mezzanine levels are directly connected to the MTA structure with an expansion joint separating these two structures, both levels have gates that are closed most of the time but are occasionally open for special events. The lower level is a ramp that leads directly into the mezzanine of the bridge and to the sidewalk of Roosevelt Avenue and entrance to the station. In addition, NYCT improved the station by adding a disability ramp adjacent to the lower level ramp of the bridge in 2009.

Underneath spans 1 to 2 of the bridge are occupied by the NYCT as storage, power operation and electrical meter boxes used for the NYCT station. Both bents 1 and 2 are covered within a fence. Underneath spans 3 through 7 is a vehicular parking area managed by IMPARK.

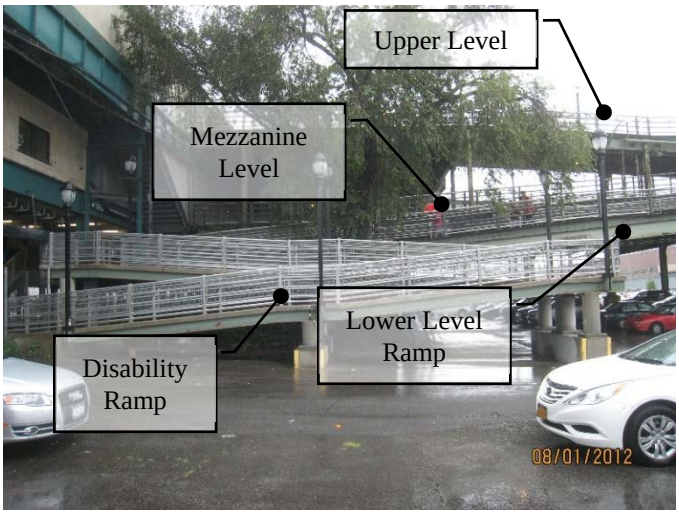

Figure 1.2.1: Bents 1 and 2 (Looking East)

Figure 1.2.2: Bents 1 and 2 (Under Bridge)

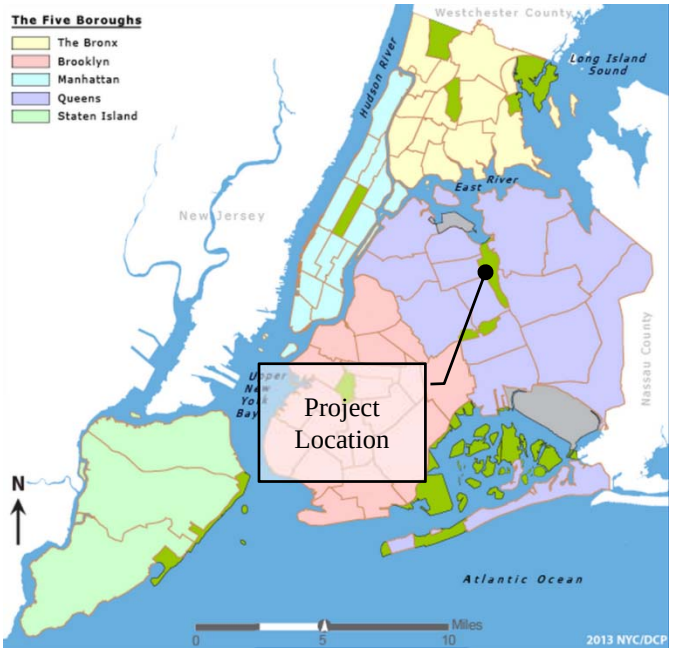

Figure 1.2.3: Key Map

Underneath spans 8 to 26 of the bridge are the NYCT Corona maintenance yard for subway trains and buses, and the LIRR Port Washington Line tracks and platforms.

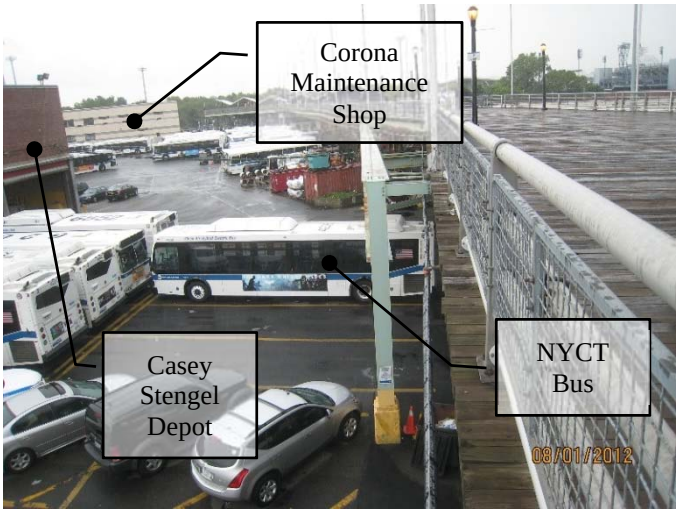

Figure 1.2.4: East Fascia (Looking South)

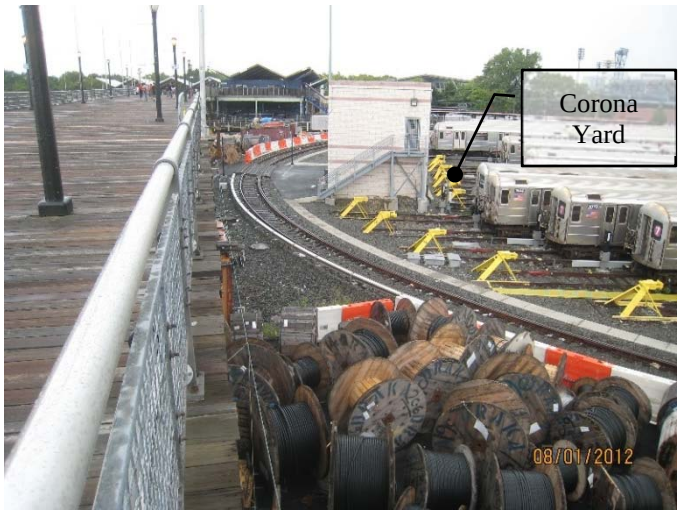

Figure 1.2.5: West Fascia (Looking South)

The south end is the end of the bridge, where there is an approach path leading to the pedestrian bridge in between NYCDPR maintenance buildings (out of scope of this project). The roofs of the buildings are public space areas with benches opened to the public. Beyond these buildings are David Dinkins Circle, and the US Tennis Association's (USTA) complex. Underneath the bridge from spans 27 through 29 exists a sidewalk, park roads, and parking area owned by NYCDPR and above the bridge from spans 27 through 29 exists two canopy structures.

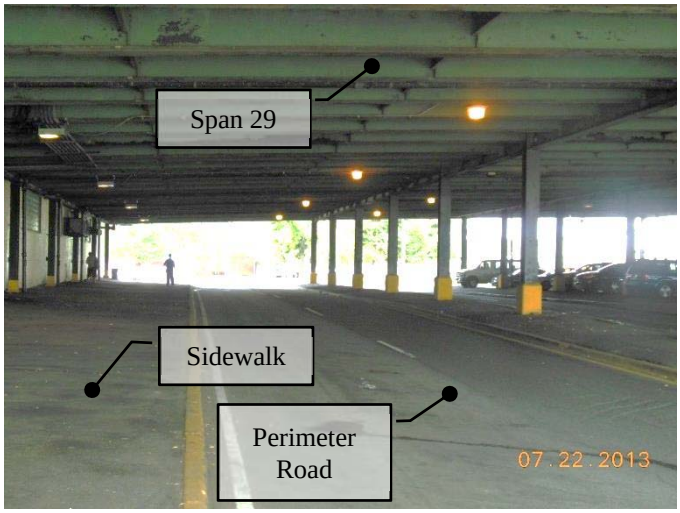

Figure 1.2.6: Span 29 Underdeck (Looking East)

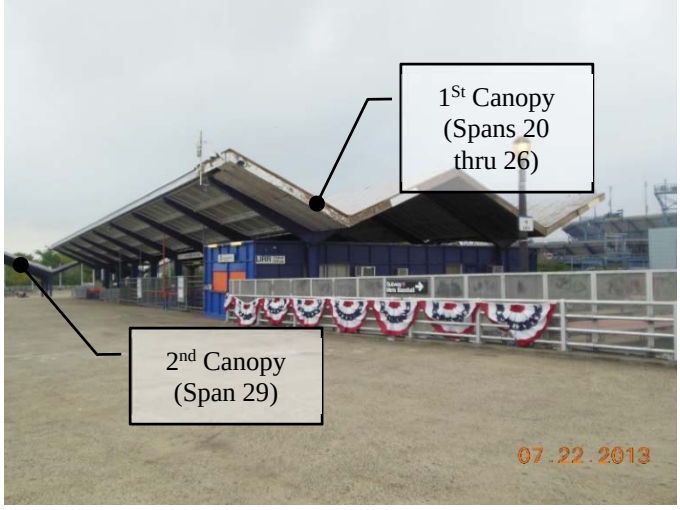

Figure 1.2.7: 1st Canopy at Spans 20 thru 26 (Looking South)

The bridge's 29 spans consist entirely of steel superstructure framing (combination of bolt and rivet) supported on substructure steel pier columns. From the north end of the bridge counting south, there are 18 spans (Spans 1-18) that have boardwalk timber decking. Spans 1 to 7 (over the Southfield parking lot) are steel girder-floorbeam-stringer framing systems which are framed into steel pier cap beams that are supported on multi-steel columns. In these spans, there are

ramps on each side of the Bridge, creating a few levels of framing. Spans 8 through 18 (the middle portion of the bridge that spans over the NYCT Corona maintenance yard for subway trains and buses, has steel girder-floorbeam-stringer framing systems framed into steel pier cap beams that are supported on two-steel columns. The remaining 11 spans at the south end of the Bridge (Spans 19-29) have a concrete deck. These spans are all steel multi-stringer systems framed into steel pier cap beams supported on multi-steel columns. Steel columns are founded on footings with piles throughout the bridge.

Since the entire superstructure was replaced circa 1964, it is approximately 50 years old, with all foundations (constructed circa 1939) approximately 74 years old. It should be noted that at two locations within the NYCT Corona maintenance yard (at spans 9-11 and 17-18), there are three large span transfer girders that were installed to allow larger horizontal clearances for buses to pass underneath the bridge. This construction is more recent, and involved replacing original pier columns with newer, widely spaced columns to facilitate the larger desired spans, while maintaining the existing superstructure framing.

2.0 In-Depth Inspection

From August to October, 2013, HAKS/WAI JV Firm performed an In-depth Inspection of the Passerelle Pedestrian Bridge as part of the Pre-Scoping Services, Project ID. HBPED700Q. Based on the inspection, approximately twenty two (22) structural steel members, (stringers, floorbeams, and columns) were found to be in a severely deteriorated condition. These members are not capable of supporting any pedestrian load. To prevent the collapse of these members and to keep the public safe, several sections of the bridge deck above the deteriorated members are barricaded and closed for public use. The Passerelle Pedestrian Bridge is currently not included for rehabilitation in any capital programs. To restore the bridge structure to an acceptable level of service, these deteriorated steel members must be repaired or replaced as necessary. For further details of the inspection see attached Appendix A: In-Depth Inspection Report.

2.1 Bridge Deck

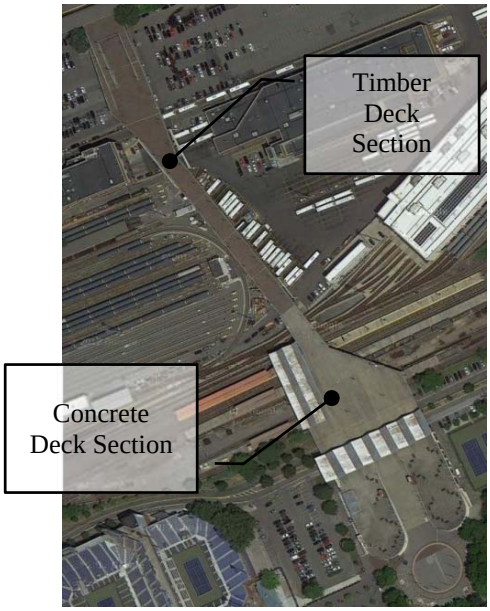

Figure 2.1.1: Arial View

2.1.1 Timber Deck Section

The Timber Deck Section of the Bridge runs from Span 1 to Span 18. The length of this section is 732ft and the width ranges from 40ft to 96.5ft. The total area is approximately 35,000 sf. The timber plank decking consists of 3" thick x 6" wide pressure treated lumber (original design) and pine lumber (replaced planks). The timber deck is in an advanced stage of deterioration including, splitting, checking, cracking and decaying. Some screws or nails are missing throughout the deck. There is a general unevenness that is typical of boardwalk planking. At some locations, vertical differential between planks and metal joints up to an inch were noted and may pose a tripping hazard for pedestrians, but this was not observed to be a prevalent condition. The New York City Department of Parks & Recreation continuously performs maintenance work on the timber deck to provide safe conditions for the public.

A safety flag was issued regarding the timber deck located between floor beams FB11 and FB11A. The planks are cracked and soft. For further details see the Inspection Report in Appendix A.

2.1.2 Concrete Deck Section

The Concrete Deck Section of the Bridge runs from Span 19 to Span 29. The length of this section is 390ft and the width ranges from 40ft to 230ft. The total area is approximately 60,000 sf. The concrete deck is a covering of a 5.5inch thick slab. The concrete deck is in fair to good condition. There are some defects noted, such as cracking and patches along the expansion joints. These defects are estimated to cover approximately 2% of the total area. The expansion joints are 3/4" wide joints filled with pourable sealer that allows water to infiltrate below. These joints are located directly above longitudinal stringers and transverse floorbeams and the constant water leakage and wetness has resulted in corrosion and section loss of the steel members below them.

Figure 2.1.2.1: Typical Timber Decking

Figure 2.1.2.2: Typical Concrete Decking

2.2 Superstructure and Substructure

2.2.1 Timber Deck Section

A. Stringers

There are approximately five-hundred thirty-five (535) stringers, totaling approximately 230,000 pounds, located in the Timber Deck Section. These members vary in size: W10X21, W10X25, W16X36, and W16X76. In general stringers are in fair to good condition but exhibit heavy corrosion and section loss. Especially, some stringers from spans 8 to18 in the Timber Deck Section are severely deteriorated. Fifteen (15) stringers were flagged due to insufficient load capacity: four (4) Red Flags and eleven (11) Yellow Flags, seen in Table 2.2.1A below. For further details see the Inspection Report in Appendix A.

Table 2.2.1A: Flagged Stringers in Timber Deck Section

Structural Flag	Location		Structural Flag	Location
Yellow	Span 11 - S6B		Yellow	Span 17 - S6B
Yellow	Span 12 - S6B		Yellow	Span 17 - S7B
Yellow	Span 13 - S7C		Red	Span 18 S3C
Yellow	Span 15 - S7C		Yellow	Span 18 - S4A
Yellow	Span 16 - S5A		Red	Span 18 - S5C and S6C
Yellow	Span 16 - S6A		Red	Span 18 - S7B
Yellow	Span 16 S5B and S6B		Red	Span 18 - S7C
Yellow	Span 17 - S6A and S7A			

Figure 2.2.1.1: Span 18 Stringer S3C (Looking Southeast)

Figure 2.2.1.2: Span 18 Stringer S7C (Looking West)

B. Floorbeams

There are approximately one-hundred twenty (120) floorbeams, totaling approximately 290,000 pounds, in the Timber Deck Section. These members vary in depth from 10 inches to 36 inches, and in weight from 21 pounds per foot to 300 pounds per foot. In general, the floorbeams are in fair to good condition except at spans 7 to 17, these members exhibit heavy corrosion and section loss throughout. In the Timber Deck Section two (2) floorbeams were flagged due to insufficient load capacity: Two Yellow Flags, seen in Table 2.2.1B below. For further details see the Inspection Report in Appendix A.

Table 2.2.1B: Flagged Floorbeams in Timber Deck Section

Structural Flag	Location
Yellow	FB17B
Yellow	FB7A(3)

Figure 2.2.1.3: FB17B (Looking South)

Figure 2.2.1.4: FB7A(3)

C. Girders

There are approximately eighty (80) girders, totaling approximately 425,000 pounds, in the Timber Deck Section. These members vary in depth from 24 inches to 36 inches, and in weight from 76 pounds per foot to 230 pounds per foot. In general, the girders are in good condition, no girders in the Timber Deck Section were flagged due to insufficient load capacity, however, the majority of these members exhibit paint deterioration resulting from mild corrosion.

D. Columns

There are approximately forty-five (45) columns in the Timber Deck Section. All columns have a depth of 14 inches, but vary in weight from 38 pounds per foot to 119 pounds per foot. In general, steel columns are in good condition. Some defects such as minor section loss at flanges near the

bottom base plate, missing bolt nuts, and paint deterioration were noted. No columns in the Timber Deck Section were flagged due to insufficient load capacity.

2.2.2 Concrete Deck Section

A. Stringers

There are approximately two-hundred seventy (270) stringers, totaling approximately 885,000 pounds, in the Concrete Deck Section. These members vary in depth from 12 inches to 33 inches, and in weight from 25 pounds per foot to 160 pounds per foot. In general, majority of the stringers in Concrete Deck Section are in good condition with exception of S18, S25 and S23 which exhibit severe corrosion with significant section loss due to water leaking from the longitudinal expansion joint above. In the Concrete Deck Section four (4) stringers were flagged due to insufficient load capacity: three (3) Red Flags and one (1) Yellow Flag, seen in Table 2.2.2A below. For further details see the Inspection Report in Appendix A.

Table 2.2.1B: Flagged Stringers in Concrete Deck Section

Structural Flag	Location
Red	Span 24 - S18
Red	Span 25 - S23
Red	Span 26 - S25
Yellow	Span 28 - S18

Figure 2.2.2.1: S25

Figure 2.2.2.2: S18

B. Floorbeams

There are approximately seventy (70) floorbeams, totaling approximately 315,000 pounds in the Concrete Deck Section. These members vary in depth from 30 inches to 33 inches, and in weight

from 116 pounds per foot to 194 pounds per foot. Generally floorbeams in Concrete Deck Section are in good condition with exception of floorbeams at bents 24 and 27 under transverse joints. These floorbeams exhibit heavy to severe corrosion with thru holes at few locations. No floorbeams in the Concrete Deck Section were flagged due to insufficient load capacity.

C. Girders

There are six (6) girders, totaling approximately 45,000 pounds, in the Concrete Deck Section. These members vary in size: W33X130 and W33X152. The girders in the Concrete Deck Section are in good condition. No girders in the Concrete Deck Section were flagged due to insufficient load capacity.

D. Columns

There are approximately eighty (80) columns in the Concrete Deck Section. All columns have a depth of 14 inches, but vary in weight from 61 pounds per foot to 95 pounds per foot. In general, the majority of the columns are in good conditions, typical defects including minor section loss at column flanges, cracked or spalled concrete encasement at lower part of the columns, and paint deterioration. In the Concrete Deck Section one (1) column were flagged due to insufficient load capacity: one (1) Red flag seen in Table 2.2.2D below.

Table 2.2.1D: Flagged Columns in Concrete Deck Section

Structural Flag	Location
Red	Column Line G at Bent 23
Red	Column Line G at Bent 24
Red	Column Line F at Bent 24

Figure 2.2.2.3: COL. G23 UPPER SECTION

Figure 2.2.2.4: COL. G23 LOWER SECTION

2.3 Canopy Structure

The canopy structure in the Concrete Deck Section consist of a roof structure supported on steel V-shaped beams that are supported on steel columns which are anchored into the bridge superstructure framing. There are two canopy structures, one located between spans 20 to 27 and the other one located at span 29. The tapered steel beams and columns were observed to be in good condition with minor surface rust. The roof structures are undergoing reconstruction at the time of the in-depth inspection. Approximately 90% of the roof has been replaced; new sidings and gutter sleeves were also observed. In general, the roof structures are in good condition except at few locations where the steel roof battens are disconnected due to water leakage from roof gutter. In addition, there is a section of the roof is missing between Spans 21 to 22 above the entrance to the LIRR track No. 1 and No. 2 train platforms.

Figure 2.5.1: Canopy at Spans 20 to 27

Figure 2.5.2: Canopy at Span 23-26

3.0 Structural Analysis

3.1 Introduction

Load ratings were initially rated by using the Allowable Stress Method (ASD). If the member did not meet the minimum required inventory rating for either pedestrian load or vehicle load H10, then it was re-rated by using the Load Factor Method (LFD), in accordance with the guidelines of Appendix C of NYCDOT's Procedure for Bridge Reconstruction Project Report (BRPR). The as-built and as-inspected ratings were calculated according to the latest provisions of AASHTO Manual for Bridge Evaluation, 2nd Edition (2011) and AASHTO Standard Specification for Highway Bridge, 17th Edition (2002), and LRFD Guide Specifications for the Design of Pedestrian Bridges (2009).

The following loads were applied to the structure, including the canopies at the south end:

- Steel self-weight;
- Concrete 150 lb/ft^3 and wet timber plank 50 lb/ft^2;
- 10 psf superimposed dead load;
- 100 psf floor live load;
- Vehicle load H10; (Single Truck, 8 kips wheel load)
- Self-weight and snow load on the canopies; (Based on the latest NYC Building code)
  P_g = 25 lb/ft^2 at NYC region

The material properties were defined for the structure:

- Structural steel: F_y =33,000 psi (Year of construction between 1936 to 1963)
- Concrete: f'_c = 2,500 psi

Assumption made in the calculations:

- All superstructures were non-composite sections;

The superimposed dead load and pedestrian load are distributed based on the tributary area from the framing plan. The live load (H10 maintenance truck) was computed based on the moving load to produce maximum result typically at the mid-spans. Since most of the structural members were simple spans, the maximum moving load results were compared to the pedestrian load results to find which load governs.

Original Design Criteria / References

- Bridge built from plans dated in 1961;
- No reference to design code used;
- Live Load = 100 psf shown on plans.

3.2 Summary of Findings

As-Built load ratings findings (timber deck spans)
Generally, the stringers cannot carry vehicle wheel load of 8 kips. In comparison to the existing design, these timber deck spans were only analyzed for pedestrian live load of 100psf and found most of the stringers are okay. However, the floorbeams and girders are found that they do not rate for pedestrian live load of 100psf. The lowest load ratings for the floorbeam is 0.46 (ASD) and girder is 0.21 (ASD). Columns are okay for pedestrian live load of 100psf.

As-Built load ratings findings (concrete deck spans)
The stringers can carry vehicle wheel load of 8 kips. The stringers, floorbeam, and column are found to be okay for pedestrian live load of 100psf, except for one location at the floorbeam (bent 24), structural component '24-6'. This floorbeam which supports the base of the existing canapy has a very low rating. No girders are found in the concrete deck spans.

As-Inspected load ratings findings (both timber and deck spans)
Only structural components with deteriorations found were re-computed for section properties based on the field measurements, then incorporated these numbers to calculate the As-Inspected load ratings.

- Incorporates inspected section losses of steel members;
- Spans 1-7 (timber deck): Very few members deteriorated and need repair. The lowest load ratings for the floorbeam is 0.46 (ASD) and girder is 0.21 (ASD);
- Spans 8-18 (timber deck): More advanced corrosion in stringers and floorbeams need repair (see Note 1). Taking the average losses of 20% reduction to compute the as-inspected load ratings, the lowest load ratings for the floorbeam is 0.64 (ASD). Stringers and girders are okay;
- Spans 19-29 (concrete deck): Members under leaking expansion joints deteriorated, and need repair (see Note 1).

Note 1: For deteriorated locations, see Appendix A (In-depth Inspection Report).
Note 2: For load rating calculation, see Appendix B.

3.3 Conclusion

The bridge appears that the structure was designed to "Minimum Design Loads in Buildings and Other Structures" from American Standard Building Code Requirements A58.1 (1945) but low in AASHTO bridge ratings especially from spans 1-18. The ratings are low for stringers, floorbeams, and girders except columns. It is not clear whether the original design had used live load reduction to design the bridge based on the 1961 code. Corrosion is not severe for spans 1-18 and spans 19-29. On the other hand, spans 8-18 have more advanced corrosion in stringers and floorbeams and these spans are typically carrying the NYTA cables. The load ratings in these areas were estimated due to the fact that top flanges are not currently exposed with timber planks.

1.0 NYSDOT Inspection Forms

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

TP349 (12/97)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 1 OF

RC - BIN (1 2) (3 4 5 6 7 8 9)

TEAM LEADER **Mike Perng, P.E. & Jie Sun, P.E.**

Signature

P.E. NUMBER **068942& 092407** STATE **NY**

ASST TEAM LEADER **Tung For Ko & Charles Diamond**

DATE MO (13 14) DAY (15 16) YEAR (17 18)

RAMP BRIDGE ATTACHED TO SPAN _____ BIN _____

INSPECTION AGENCY **1 3** (19 20) TYPE OF INSPECTION **3** (21)

1-BIENNIAL 2-INTERIM 3-IN-DEPTH 4-NONE(UNDER CONTRACT) 5-SPECIAL

STATE HWY. NO. **000000** MILEPOINT: **00000** POLIT. UNIT: **New York**

FEATURE(S) CARRIED: **PASSERELLE PED. BRIDGE**

FEATURE(S) CROSSED: **NYCT**

TOTAL SPANS: **29** BRIDGE ORIENTED: **Southeast** YEAR BUILT **1964**

BRIDGE TYPE : **Steel Girder and Floorbeam System** AADT/YR:

VERTICAL CLEARANCE AND LOAD POSTINGS	ON : NOT POSTED	UNDER : Span 3	LOADING : NONE	
	FT (19) IN (21)	8 FT (23) 6 IN (25)	TONS (27)	062 (118)

ABUTMENTS:	Begin	End	WINGWALLS:	Begin	End	APPROACHES:	
Joint with deck	8 (22)	6 (23)	Walls	8 (40)	8 (41)	Drainage	5 (53)
Bearings, anchor bolts, pads	8 (24)	8 (25)	Footings	8 (42)	8 (43)	Embankment	8 (54)
Bridge seat and pedestals	8 (26)	8 (27)	Erosion or Scour	8 (44)	8 (45)	Settlement	6 (55)
Backwall	8 (28)	8 (29)	Piles	8 (46)	8 (47)	Erosion	8 (56)
Stem (breastwall)	8 (30)	8 (31)	**STREAM CHANNEL:**			Pavement	5 (57)
Erosion or scour	8 (32)	8 (33)	Stream Alignment	8 (48)		Guide Railing	5 (58)
Footings	8 (34)	8 (35)	Erosion and Scour	8 (49)			
Piles	8 (36)	8 (37)	Waterway Opening	8 (50)		GENERAL RECOMMEND	5 (60)
Recommendation	8 (38)	6 (39)	Bank Protection	8 (51)			

ACCESS CATEGORY:

BUCKET TRUCK
45' LIFT
WALKING
LANE CLOSURE W/ SHADOW VEHICLE

FLAG ISSUED?			
NONE		(61)	
RED STRUCTURAL	X	(62)	Total of 8 flags in varies locations.
YELLOW STRUCTURAL	X	(63)	Total of 15 flags in varies locations.
SAFETY	X	(64)	Total of 1 flag.

Vulnerability Reassessment Review Recommended

HYD	OVL	STL	COL	CON	SMC
3	X	X	X	X	X
(65)					(70)

1 = YES
2 = NO
3 = NA
X = NOT USED THIS CYCLE

REVIEWED BY _____
P.E. NUMBER _____
DATE _____

TP350g (10/95)

RC-BIN 0 0 (1 2) | (3 4 5 6 7 8 9)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 2 OF _____

TEAM LEADERS: **Mike Perng, P.E. & Jie Sun, P.E.**

ASST TEAM LEADER: **Tung For Ko & Charles Diamond**

DATE MO 0 0 (13 14) DAY 0 0 (15 16) YEAR 0 0 (17 18)

Features carried: **PASSERELLE PED. BRIDGE**
Features crossed: **NYCT**

SPAN NO.			DECK ELEMENTS								SUPERSTRUCTURE						PIER										UTILITIES		
			Wearing Surface	Curbs	Sidewalks & Fascias	Railings & Parapets	Scuppers	Gratings	Median	Mono Deck Surf.	Deck Structural	Primary Members	Secondary Members	Paint	Joints	Recom-mendation	Brg. anchor bolt, pads	Pedestals	Top of pier cap or beam	Stem solid pier	Cap beam	Pier columns	Footings	Erosion or scour	Piles	Recom-mendation	Lighting stds. and fixtures	Sign structure	Utilities and utility supp
10	11	12	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
0	0	1	4	8	8	6	8	8	8	8	5	5	5	2	5	5	6	8	8	4	8	4	5	8	9	4	1	8	5
0	0	2	4	8	8	5	5	8	8	8	5	5	5	2	8	4	8	8	8	4	8	4	4	8	5	5	4	8	5
0	0	3	4	8	8	5	3	8	8	8	4	5	5	2	8	4	8	8	8	5	8	5	5	8	5	5	4	8	5
0	0	4	4	8	8	5	8	8	8	8	4	5	5	2	5	4	8	8	8	5	8	5	5	8	9	5	4	8	5
0	0	5	4	8	8	4	8	8	8	8	4	4	4	2	8	4	8	8	8	5	8	5	5	8	9	5	4	8	5
0	0	6	4	8	8	5	8	8	8	8	4	4	4	2	8	4	8	8	8	5	8	5	3	8	9	4	5	8	5
0	0	7	4	8	8	5	8	8	8	8	4	4	4	2	5	4	8	8	8	5	8	4	9	8	9	4	5	8	3
0	0	8	3	8	8	5	8	8	8	8	4	3	3	1	8	3	8	8	8	5	8	4	9	5	9	4	5	8	5
0	0	9	3	8	8	5	8	8	8	8	4	4	3	1	8	4	8	8	6	8	6	8	8	8	8	5	4	8	5
0	1	0	3	8	8	5	8	8	8	8	4	4	3	1	5	4	8	8	6	8	4	8	8	8	8	5	5	8	5
0	1	1	4	8	8	5	8	8	8	8	4	3	3	1	8	3	8	8	8	4	8	5	9	5	9	5	5	8	4

DIVING INSPECTION REQUIRED? [ ] Yes [X] No *If yes, indicate year of last diving inspection* [ ][ ]

SPECIAL EMPHASIS INSPECTION REQUIRED: [X] Yes [ ] No
If yes, indicate type below.

NON REDUNDANT/FRACTURE CRITICAL [X]
1. Two steel main girders on Spans 1~11.
2. Two steel exterior girders on spans 1-4 and 6-11

PIN AND HANGERS [ ]
2. Steel columns on Piers 1~9 & Pier 11.
3. Steel floorbeams spaced more than 12' on Spans 1~11.

FATIGUE-PRONE WELDS (AASHTO D, E, OR E') [ ]
4. Added double steel capbeam on Piers 9 & 10

NON-CATEGORIZED FATIGUE-PRONE DETAILS [ ]

OTHERS (SPECIFY) _____ [ ]

RECOMMEND FURTHER INVESTIGATION [1]
1 = NO
2 = YES

REMARKS
20
50

Field Notes:

Date	Time of Arrival	Time of Departure	Temp (F)	Weather Conditions
				See attached sheet for inspection dates

TP350g (10/95)

RC-BIN 0 0 (1 2) | (3 4 5 6 7 8 9)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 3 OF _____

TEAM LEADERS: **Mike Perng, P.E.**

ASST TEAM LEADER: **Charles Diamond & Tim Viani**

DATE MO 0 0 (13 14) DAY 0 0 (15 16) YEAR 0 0 (17 18)

Features carried: **PASSERELLE PED. BRIDGE**
Features crossed: **NYCT**

SPAN NO.			DECK ELEMENTS								SUPERSTRUCTURE						PIER										UTILITIES		
			Wearing Surface	Curbs	Sidewalks & Fascias	Railings & Parapets	Scuppers	Gratings	Median	Mono Deck Surf.	Deck Structural	Primary Members	Secondary Members	Paint	Joints	Recom-mendation	Brg. anchor bolt, pads	Pedestals	Top of pier cap or beam	Stem solid pier	Cap beam	Pier columns	Footings	Erosion or scour	Piles	Recom-mendation	Lighting stds. and fixtures	Sign structure	Utilities and utility supp
10	11	12	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
0	1	2	3	8	8	6	8	8	8	8	4	3	3	1	5	3	8	8	8	5	8	4	9	5	9	4	5	8	5
0	1	3	4	8	8	6	8	8	8	8	4	3	3	1	8	3	8	8	8	5	8	4	9	5	9	4	5	8	5
0	1	4	4	8	8	6	8	8	8	8	4	4	3	1	5	4	8	8	8	5	8	4	9	5	9	4	5	8	5
0	1	5	3	8	8	6	8	8	8	8	4	3	3	1	8	3	8	8	8	5	8	5	9	5	9	5	5	8	5
0	1	6	4	8	8	6	8	8	8	8	4	2	3	1	5	3	8	8	8	4	8	5	9	5	9	5	5	8	5
0	1	7	4	8	8	6	8	8	8	8	4	2	3	1	8	3	8	8	8	4	6	5	9	5	8	5	5	8	4
0	1	8	3	8	8	6	8	8	8	8	4	1	3	1	5	2	8	8	8	5	8	5	9	5	9	5	5	8	5
0	1	9	3	8	8	6	8	8	8	8	4	4	4	1	4	2	8	8	8	5	8	5	9	5	9	5	5	8	5
0	2	0	3	8	8	6	8	8	8	8	4	4	4	1	4	2	8	8	8	5	8	5	9	5	9	5	5	8	5
0	2	1	5	8	8	8	8	8	8	8	4	4	5	4	3	4	8	8	8	5	8	5	9	5	9	5	5	8	5
0	2	2	5	8	8	8	8	8	8	8	4	4	5	4	3	4	8	8	8	8	8	5	9	9	9	5	5	8	5

DIVING INSPECTION REQUIRED? [ ] Yes [X] No *If yes, indicate year of last diving inspection* [ ][ ]

SPECIAL EMPHASIS INSPECTION REQUIRED: [X] Yes [ ] No
If yes, indicate type below.

NON REDUNDANT/FRACTURE CRITICAL [X]

PIN AND HANGERS [ ]

FATIGUE-PRONE WELDS (AASHTO D, E, OR E') [ ]

NON-CATEGORIZED FATIGUE-PRONE DETAILS [ ]

OTHERS (SPECIFY) Square edge concrete haunch [X]

1. Two steel main girders on Spans 12~18.
2. Steel columns on Piers 12~18.
3. Steel floorbeams spaced more than 12' on Spans 12~19.
4. Added double steel capbeam on Piers 17

Note: Inspection of spans 19 to 20 is pending

Along all stringers/floorbeams on Spans 19-22.

RECOMMEND FURTHER INVESTIGATION [1]
1 = NO
2 = YES

REMARKS
20
50

Field Notes:

Date	Time of Arrival	Time of Departure	Temp (F)	Weather Conditions
				See attached sheet for inspection dates

TP350g (10/95)

RC-BIN 0 0 (1 2) | (3 4 5 6 7 8 9)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 4 OF ____

TEAM LEADERS: **Jie Sun P.E.**

ASST TEAM LEADER: **Tung For Ko**

DATE MO 0 0 (13 14) DAY 0 0 (15 16) YEAR 0 0 (17 18)

Features carried: **PASSERELLE PED. BRIDGE**
Features crossed: **NYCT**

SPAN NO.			DECK ELEMENTS								SUPERSTRUCTURE						PIER										UTILITIES		
			Wearing Surface	Curbs	Sidewalks & Fascias	Railings & Parapets	Scuppers	Gratings	Median	Mono Deck Surf.	Deck Structural	Primary Members	Secondary Members	Paint	Joints	Recom-mendation	Brg. anchor bolt, pads	Pedestals	Top of pier cap or beam	Stem solid pier	Cap beam	Pier columns	Footings	Erosion or scour	Piles	Recom-mendation	Lighting stds. and fixtures	Sign structure	Utilities and utility supp
10	11	12	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
0	2	3	5	8	8	6	5	8	8	8	4	5	5	3	5	4	8	8	8	8	8	3	9	9	9	3	8	8	4
0	2	4	4	8	8	6	3	8	8	8	4	3	5	2	5	3	8	8	8	5	8	5	9	8	9	3	6	8	5
0	2	5	5	8	8	6	8	8	8	8	4	4	5	3	5	3	8	8	8	8	8	5	9	8	9	5	5	8	8
0	2	6	5	8	8	6	8	8	8	8	4	4	5	3	5	4	8	8	8	8	8	5	9	8	9	5	8	8	5
0	2	7	4	8	8	6	8	8	8	8	4	4	5	2	5	4	8	8	8	8	8	4	9	8	9	4	6	8	8
0	2	8	4	8	8	6	3	5	8	8	4	3	5	3	5	4	8	8	8	8	8	4	9	8	9	4	4	8	4
0	2	9	5	8	8	6	4	5	8	8	4	5	5	3	5	4	8	8	8	8	8	4	9	8	9	4	6	8	4

DIVING INSPECTION REQUIRED? [ ] Yes [X] No *If yes, indicate year of last diving inspection* [ ][ ]

SPECIAL EMPHASIS INSPECTION REQUIRED: [X] Yes [ ] No
If yes, indicate type below.

NON REDUNDANT/FRACTURE CRITICAL [ ] 1. Steel floorbeams spaced more than 12' on Spans 23 to27

PIN AND HANGERS [ ] 2. Steel columns on Piers 23~29.

FATIGUE-PRONE WELDS (AASHTO D, E, OR E') [X] 1. Partial length welded cover plates E' weld detail at Span 29
Stringer S2, S3, S8, S14, S20, S26, S31 and S32

NON-CATEGORIZED FATIGUE-PRONE DETAILS [ ]

OTHERS (SPECIFY) Square edge concrete haunch [X] Along all stringers/floorbeams on Span 23-29.

RECOMMEND FURTHER INVESTIGATION **1** 1 = NO 2 = YES

REMARKS (20) (50)

Field Notes:

Date	Time of Arrival	Time of Departure	Temp (F)	Weather Conditions
				See attached sheet for inspection dates

FEDERAL RATING FORM

NYS DEPT. OF TRANSPORTATION

RC - BIN | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0
(1 2 3 4 5 6 7 8 9)

BRIDGE INSPECTION REPORT

TEAM LEADER **Mike Perng, P.E. & Jie Sun, P.E.**

SHEET **5** OF _____

ASST TEAM LEADER **Tung For Ko & Charles Diamond**

DATE

MO		DAY		YEAR	
0	0	0	0	0	0
13	14	15	16	17	18

FEATURE CARRIED: **PASSERELLE PED. BRIDGE**

FEATURE CROSSED: **NYCT**

Description	Deck	Superstructure	Substructure	Channel	Culvert
Fed. Item #	58	59	60	61	62
Rating	5	4	6	N	N

Comments:

Notes:

1) See attached explanations for Federal Items Nos. a) 58 - Deck, 59 - Superstructure, 60 - Substructure; b) 61 - Channel and Channel Protection; c) 62 - Culverts.

2) Items Nos. 58, 59 and 60 shall be coded N for all culverts.

3) A rating or an N must be entered for all Federal Items. Blanks are not acceptable.

SHEET 6 OF _____

Field Notes:

Date	Time of Arrival	Time of Departure	Temp (F)	Weather
7/29/2013	8:00 AM	4:00 PM	75°	sunny
7/30/2013	8:00 AM	4:00 PM	74°	sunny
7/31/2013	7:00 AM	3:30 PM	75°	sunny
8/1/2013	7:00 AM	3:00 PM	72°	rainning
8/2/2013	7:00 AM	3:20 PM	74°	sunny
8/5/2013	7:00 AM	3:00 PM	70°	sunny
8/6/2013	7:00 AM	3:00 PM	73°	Partly sunny
8/12/2013	8:00 AM	4:00 PM	74°	sunny
8/13/2013	8:00 AM	4:00 PM	73°	sunny
8/14/2013	8:00 AM	4:00 PM	70°	sunny
8/15/2013	8:00 AM	4:00 PM	69°	sunny
8/16/2013	8:00 AM	4:00 PM	70°	sunny
9/11/2013	7:00 AM	3:00 PM	81°	sunny
9/12/2013	7:00 AM	3:00 PM	78°	sunny
9/13/2013	8:00 AM	4:00 PM	70°	sunny
9/16/2013	8:00 AM	4:00 PM	65°	sunny
9/17/2013	7:00 AM	3:30 PM	59°	sunny
9/18/2013	7:00 AM	3:00 PM	60°	sunny
10/11/2013	8:00 AM	4:00 PM	68°	Light rain
10/14/2013	8:00 AM	4:00 PM	66°	Partly sunny
10/15/2013	8:00 AM	4:00 PM	72°	Sunny
10/16/2013	8:00 AM	4:00 PM	67°	Partly sunny
10/17/2013	8:00 AM	4:00 PM	73°	Light rain
10/18/2013	8:00 AM	4:00 PM	68°	Partly sunny
10/21/2013	8:00 AM	4:00 PM	66°	Sunny
10/22/2013	8:00 AM	4:00 PM	67°	Partly sunny
2/12/2014	8:00 AM	4:00 PM	25°	Partly sunny

BD 188 (1/96)

BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 7 OF _____

TEAM LEADER: Mike Perng, P.E. & Jie Sun, P.E.

ASST.TEAM LEADER: Tung For Ko & Charles Diamond

DATE //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS		PHOTO	REMARKS
NEW	PREV	NO.	
			GENERAL NOTES
			1) No NYSDOT BIN has been assigned to this bridge. However, MTA has assigned
			#42-B-067 for this bridge.
			2) No inventory data has been collected for this inspection.
			3) Span 1-5 has three levels (upper, main and lower levels) and Span 6 has two levels
			(main and lower)
			4) This inspection was an in-depth inspection, and is the first inspection documented.
			No previous inspection reports are available

BD 188 (4/95)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 8 OF

BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350
			DECK ELEMENTS
			[19] WEARING SURFACE
4	N/A		Spans1-7:
		1-3	Top surface of timber planks at these spans exhibit deterioration, decay and
		5-5	checking at various locations.
		5-1	At span 5, between FB4-1 and FB4A, S4A and S5A, timber plank exhibits a
			8” L x ½” W crack. (Typical at few other locations)
4	N/A		Spans11, 13, 14, 16 & 17:
		11-5, 11-6	Top of Timber deck shows checking, decay and partially loose connection to
		16-4, 18-2	Supporting stringers in general.
3	N/A	15-3, 15-4	Spans 8 -10, 12, 15 & 18:
		18-2	In addition, some of timber boards are soft and deflected up to 1/2" under
			loading. On the other hand, few timber boards are warping up to 1/2".
		12-3	At span 12, top of timber deck at 11' after FB11 between S4A & S5A,
			One timber board has a fine 1/2"Dp crack crossing, the fibers at 1.5' from
			S4A. The board deflects about 1" when stepped on.
			There is a potential of a punch through. The adjacent timber board before
			the board described above is soft as well and exhibits ½ inch deflection
			under loading See Safety Flag Report #P-011 for details.
4	N/A	24-1	Spans 24, 27 & 28:
			Top of the concrete deck exhibit multiple hairline cracks and concrete
			patches along the transverse joints and other isolated locations
			Area of defects is approx. 10% of the total deck area.
		27-1	Span 27, multiple cracked concrete patches along the rubber joint.
			[22] Railings & Parapets
4	N/A		Span 5
		5-2	One railing post is disconnected from its base plate but stable, broken welds
			were observed. The rest four railings at this side are in good condition.
		5-4	Missing rail at the lower part of the railing.

BD 188 (4/95)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 9 OF

BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
3	N/A		**[23] Scuppers**
			Span 3, 24 and 28
		3-1	At span 3, disconnected drainage downpipe, possibly abandoned at FB2A.
		24-8	At span 24, drainage downpipe is disconnected near bent 24&S25 and
			discharge directly below.
		28-2	At span 28, disconnected and severe corroded drainage pipe between S17
			to S18, near the first diaphragm.
			[27] Deck Structural
4	N/A		Spans 3-7
		3-5	Underside of timber deck shows checking, decay in general. Approx. 30%
			of the timber planks are deteriorated, 10% has hairline cracks.
		3-2	At Span 3, between FB2A-2 & FB2B-2 at S4B, rotted timber nailer with
			severe deterioration
		4-2	At Span 4, between FB3-2 & FB3A-2, S6A and S7A, cracked timber plank
			exhibits 2" separation.
4	N/A		Spans 8-18
		18-1	Underside of timber deck shows checking, decay in general. Replacement
			noted in several locations
4	N/A		Spans 20-22
		21-2	Underside of deck exhibits concrete spall (up to 1" depth) with exposed
			rebar
4	N/A		Spans 23-29
		29-1	The underside of the deck exhibits numerous concrete spalls ranging from
		29-4	1" to 3" deep with exposed corroded reinforcement bars estimated to cover
		26-1	approximately 2% of the span area. The majority of concrete spalls are
			located at the concrete square haunches along FB (special emphasis).
		23-3	At Span 23, G1 east face, concrete deck exhibits a 3'L x 3½" W x 5" D spall.
			with exposed rebar.
		23-4	At span 23, S19 east face, concrete deck at haunch exhibits a 10' x 7" W
			spall, loose concrete is resting at the bottom flange of the C channel.

BD 188 (4/95)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 10 OF

BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[27] Deck Structural
		28-5	At Span 28, concrete spalls along the top flange of S13
			[28] Primary Members
			Spans 1-7
		5-6	Spans 1-7 consist steel stringer-floorbeam system support on four
			girders. Stringers in between girders are then framed into floorbeams by
			shear connections except at span 1 where the stringers are sit on top of the
			floorbeam create a short cantilever span.
			Since timber planks have gaps in between them, the steel members below
			them exhibit heavy corrosion with localized section loss at top flange ranging
			from 5% to 40%. Most top flange localized section loss is at lower level of
			spans 5 and 6 and main level of span 7 near the south exterior girder.
			See attached steel framing deterioration plan
4	N/A		Spans 5-7
		6-1	At Span 6 Lower Level, FBL5A North Face between SL1A &SL2A, top flange
			exhibits severe corrosion with 15-20% section loss.
		7-2	At Span 7, FB6B-3 North Face, between S12B & 13B, top flange exhibits
			severe corrosion with up to 50% section loss.
			Spans 8-19
		10-1	The steel framing consists of stringer-floorbeam system supported by two
		10-2	main girders. The stringers are framed to web of floorbeam, and the
		12-2	the floorbeams are spanned over main girders with four connection bolts
		14-2	to top flange of girders. Since there is no separate wearing course to prevent
			water dripping thru the gap between timber deck boards, all steel members
			below exhibit heavy corrosion with varies degree of section loss.
			In addition, all connection bolt nuts between floorbeam and girder show
			heavy section loss. It is also noticed that right half of spans have more
			deterioration than left half. This may be the result of melting salt used
			in winter were piled up on right half in order to leave left half open to
			pedestrian. See attached framing plan for detail deterioration of each

BD 188 (4/95)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 11 OF

BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[28] Primary Members
			member
4	N/A		Spans 9, 10 & 14:
		9-3,9-4	Main girders and floorbeams are in fair condition in general with minor
		9-2	surface loss up to 5% locally. However, stringers exhibit varies degree of
		14-1	section loss up to 15%, but, mostly occurred on top flanges. No significant
			section loss is noted on critical zone of members.
3	N/A		Spans 8, 11 to13 & 15:
		9-3,9-4	Main girders and floorbeams are in deterioration with up to 10%
		15-2	section loss at the areas under the member been supported. Stringers
			exhibit at least 15% section loss almost throughout. Perforation and
			broken flange edge are also noted in several members. Some of them are
			happened on the critical zone of the members, and, yellow flags have been
			issued:
		8-4	At span 8, FB7A, perforation in the floorbeam web near the connection to
			G3. The surrounding portion of the web near the perforation is paper thin
			exhibiting extreme corrosion. Total section loss for the member is 40%.
		11-1	See Yellow Flag Report #P-012 for details. At span 11, S6B between FB10A
			& FB10B, Extreme corrosion with large perforation and paper thin in the
			lower 4" of web along bottom flange for 12' long starting from 3' away
			from FB10A.The remaining web shows up to 10% section loss. Also, the
			flanges exhibit up to 15% section loss with broken 1/8" of top flange edge
			throughout. The worst total section loss at 6' away from floorbeam FB10A
			with 45% web loss and 20% flange loss. See Yellow Flag Report #P-014 for
			details.
		12-1	At span 12, S6B between FB11A & FB11B, extreme corrosion with large
			perforation and paper thin in the lower 3" of web along bottom flange
			throughout. Remaining of web shows up to 10% section loss. Also, the top
			flange exhibits up to 50% section loss with broken 1" edge through
			out. The bottom flange has less than 5% section loss. The worst total
			section loss is at 2' away from Floorbeam FB11B with 35% web loss and

BD 188 (4/95)

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 12 OF

BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E. **ASST. TEAM LEADER:** Tung For Ko & Charles Diamond **DATE:** //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[28] Primary Members
			30% flange loss. See Yellow Flag Report #P-015 for details.
3	N/A		Spans 8, 11 to13 & 15: (continue)
		13-2	At span 13, S7C between FB12B & FB13, extreme corrosion with large
			perforation and paper thin in the lower 4" of web along bottom flange for
			4' long starting from 5' away from FB13. Remaining web shows up to 20%
			section loss. Also, the top flange exhibits a range of 5% to 20% section
			loss with up to 1/8" broken edge throughout. Bottom flange shows up to 5%
			section loss. The worst total section loss is at 8' away from FB13 with
			50% web loss and 15% flange loss. See Yellow Flag Report #P-016 for
			Details.
		15-1	At span 15, S7C between FB14B & FB15, extreme corrosion with large
			perforation and paper thin in the lower 2.5" of web along bottom flange
			for 12' long starting 3' away from FB15. Remaining web shows up to 20%
			section loss. Also, the top flange exhibits up to 20% section loss with up
			to 1/2" broken edge locally. Bottom flange shows up to 5% section loss.
			The worst total section loss is at 3' away from FB15 with 30% web loss and
			20% flange loss. See Yellow Flag Report #P-017 for details.
2	N/A		Spans 16 & 17:
		17-4	Main girders and floorbeams are in deterioration with up to 10%
			section loss at the areas under the member been supported. Stringers
			exhibit at least 20% section loss almost throughout. There are several
			stringer replacement done at span 17. However, isolated smaller
			perforation and broken flange edge are noted in many remaining ones.
			of them are large, in adjacent members, and, happened on the critical zone
			of the members. Yellow flags have been issued for these conditions:
		16-1	At span 16, S5A between FB15 &FB15A, extreme corrosion with
			in size of 2'L x 2"H & 6"H x 2"H, and, paper thin in the lower 2.5" of web
			along bottom flange for most of the member's length. Remaining web shows
			up to 5% section loss. Also, the flanges exhibit up to 15% section loss

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BRIDGE INSPECTION REPORT
SHEET 13 OF

BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[28] Primary Members
2	N/A		Spans 16 & 17: (continue)
			with up to 1/2" broken top flange at mid-span. The worst section loss is
			at mid-span with 30% web loss and 20% flange loss. See Yellow Flag
			#P-018 for detail.
		16-2	At span 16, S6A between FB15 & FB15A, extreme corrosion with large
			perforation and paper thin in the lower 4" of web along bottom flange for
			4' long near the center of the span. Another corroded hole of 1.5"
			diameter is noted on the web below the connection to FB15A. Also, both
			flanges exhibit 15% section loss throughout. The worst cross section loss
			is at mid-span with 40% web loss and 15% flange loss. See Yellow Flag
			Report #P-019 for details.
		16-3	At span 16, S5B &S6B between FB15A&FB15B, both redundant Stringers
			S5B & S6B exhibits extreme corrosion with local perforation and paper thin
			the lower 2" of web along bottom flange for up to 3' long starting 1.0'
			away from either ends. Remaining of web shows average of 5% section
			Also, the top flanges exhibits average of 20% section loss. Bottom
			flange shows up to 5% section loss. The worst total section loss is at 1.5'
			away from Floorbeam FB15A with 25% web loss and 12% flange loss. See
			Yellow Flag Report #P-020 for details.
		17-2,17-3	At span 17, S6A & S7A between FB16 & FB16A, both redundant adjacent
			Stringer S6A & S7A exhibit extreme corrosion near the connection to FB16A.
			The Stringer S6A exhibits perforation and paper thin in the lower 2" of
			web along bottom flange for 7' long. Remaining of web shows up to 10%
			section loss. Also, the top flange exhibits average of 10% section loss
			with broken 1" edge in 1' long . Bottom flange shows up to 5% section loss.
			The worst total section loss is at 2.5' away from Floorbeam FB16A with 25%
			web loss and 10% flange loss. The Stringer S7A also exhibits perforation
			and paper thin in the lower 1" of web along bottom flange for 4' long, and,
			the top flange exhibits broken 1/2" edge in 1.5' long. The worst total
			section is at 3' away from Floorbeam FB16A with 15% web loss and10%
			Flange loss. See Yellow Flag Report#P-021 for details.

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BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[28] Primary Members
2	N/A	17-5	Spans 16 & 17: (continue)
			At span 17, S6B between FB16A & FB16B, extreme corrosion with large
			perforation and paper thin on the lower web along bottom flange in size of
			4"L x 3"H at quarter point from FB16A and 2'L x 4"H at quarter point from
			FB16B. Remaining of web shows up to 10% section loss. Also, the top
			exhibits up to 15% section loss with up to 1" broken edge randomly
			throughout. The worst cross section loss is at 4' away from Floorbeam
			FB16B with 45% web loss and 30% flange loss.
			See Yellow Flag Report #P-022 for details.
		17-6	At span 17, S7B between FB16A & FB16B, extreme corrosion with large
			perforation and paper thin on the lower 3" web along bottom flange for 10'
			long starting from FB16A. Remaining of web shows up to 15% section loss.
			Also, the top flange exhibits up to 25% section loss with up to 1/2" broken
			edge throughout. The worst cross section loss is at connection to FB16A
			with 40% web loss and 20% flange loss. See Yellow Flag Report #P-023 for
			Details.
1	N/A		Span 18:
		18-5	Main girders and floorbeams are in deterioration with up to 10%
			section loss at the areas under the member been supported. Stringers
			exhibit at least 25% section loss almost throughout. There are many
			stringer replacement done at this span. However, isolated smaller
			perforation and broken flange edge are noted in many remaining ones.
			of them are large, in adjacent members, and, happened on the critical zone
			of the members. Yellow flags and red flags have been issued for these
		18-14	conditions: (Barricaded off the sever deteriorated framing area from top.)
		18-9	S3C between FB17B & FB18, completely sheared off web at the connection
		18-10	to FB17B and the stringer is side swinging approximately 1/8". See Red Flag
			Report #P-006 for details.
		18-12	S7C between FB17B& FB18, completely sheared off web at the connection
		18-13	to FB17B, and,the stringer is sagging approximately 1/8". Remaining web
			exhibits extreme corrosion with up to 2" high perforation along the bottom

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BIN

TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[28] Primary Members Span 18: (continue)
1	N/A		flange in various locations. See Red Flag Report #P-007 for details.
		18-11	S5C & S6C between FB17B & FB18, both adjacent stringers exhibit extreme
			corrosion with up to 2" high perforation on lower web along the bottom
			flange in numerous locations. See Red Flag Report #P-008 for details.
		18-6	S7B between FB17A &FB17B, up to 1.5" high perforation on lower web
		18-7	the bottom flange for 90% of the span length. Other portions of the web
			are paper thin. The top flange has a broken edge up to ¼". The bottom
			flange exhibits up to 15% section loss—it is shaky and is unable to
			function as designed. See Red Flag Report # P-009 for details.
		18-8	FB17B above G3, perforation on the bottom 1/3 of the web along the web
			stiffener above its supporting member G3. The remaining portion of the web
			also exhibits extreme corrosion with up to 2" diameter holes and a few
			pinholes. See Yellow Flag Report # P-010 for details.
		18-4	S4A between FB17 & FB17A, extreme corrosion with large perforation and
			paper thin on the lower 3" web along bottom flange throughout. Remaining
			of web shows 25% section loss. Also, the top flange exhibits up to 30%
			section loss with up to 1/4" broken edge throughout. Bottom flange shows
			10% section loss as well. The cross section loss has 45% web loss and 25%
			flange loss throughout. See Yellow Flag Report #P-024 for details.
4	N/A		Spans 20-22
		22-2	FB223 connecting to column 22G — 1 out of 9 connection bolt at
			bottom has been sheared off.
		24-2	Spans 23-29 consist steel stringer-floorbeam system support by steel
		29-3	columns. Stringers are framed into floorbeams by shear connections
			bolted to the web of the floorbeams. The majority of the steel members are
			fair to good condition with normal rust and paint peeling except stringers and
			floorbeams which located directly under the longitudinal and transverse
			expansion joints. At those areas, steel members exhibit severe corrosion
			up to 100% section loss at web and up to 40% section loss at flanges. For
			these conditions at spans 24, 25 and 26 RED FLAGS have been issued.
			See flag reports for more details. Other areas, steel members exhibit
			significant corrosion with section loss at Mid-span and at connections
			ranging from 5% to 18% at top flanges, 5% to 15% section loss at bottom

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E. **ASST. TEAM LEADER:** Tung For Ko & Charles Diamond **DATE:** //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[28] Primary Members
			flange and 5% to 15% section loss at webs. Impact damages were also
			observed at span 28 stringers' bottom flanges and webs at twelve
			different locations. See attached steel framing deterioration plan.
3	N/A		Span 24
			Stringers and floorbeams in span 24 are generally in good condition except
		24-5	FB24-4 to FB24-7 under transverse expansion joint exhibits severe
		24-3	with section loss up to 30% including corrosion hole at the web and upto12%
		24-4	at flanges and web corrosion holes at S17.
		24-6	At span 24, S18 stringer exhibits approximately 65% web section loss at its
		24-7	its end connections and severe delaminated corrosion at mid-span, which
			could not inspected hands-on due to the presence of LIRR power line near
			Mid-span. Based on the severely corroded condition of stringers along this
			Column line G in spans 25 and 26, we expect similar advanced corrosion in
			the web and flanges at midspan. See RED FLAG P-003 for details.
4	N/A		Span 25
		25-1	Stringers and floorbeams in span 25 are generally in good condition with
			normal corrosion and minor section loss expect S23 exhibits approximately
		25-2	80% web section loss and 60% bottom flange section loss at mid-span
		25-3	and approximately 60% web section loss at the ends.
		25-4	See RED FLAG P-002 for details.
4	N/A		Span 26
		26-2	Stringers and floorbeams in span 26 are generally in good condition with
		26-3	normal corrosion expect S25 exhibits 100% section loss of the web for most
			of its length. See RED FLAG P-001 for details.
4	N/A		Span 27
		27-1	S25 between columns 26G & 27G near 1st interior diaphragm
			The 46'-0" long stringer exhibits 60% loss of the web for most of the span
			and increase to 80% loss at approximately one-third of the span due to
			2-4"x4" perforations located near a diaphragm connection.
		27-2	Most of the stringers are in good condition with normal corrosion and minor
		27-5	section loss. FB 27-1 to FB27-8 and S25 under the exp. joint exhibit severe
			corrosion with section loss at midspan and at connections ranging from 5%

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E. **ASST. TEAM LEADER:** Tung For Ko & Charles Diamond **DATE:** //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[28] Primary Members
4	N/A		Span 27 (continue)
			to 15% at top flanges and 5% to 15% section loss at webs.
		27-4	At Span 27, connection plate at S18 and FB27-5 near the bottom flange
			exhibits severe corrosion with a 6.5" H x 3.5" W rusted through hole.
		27-7	At span 28, the first three bolts on the connection plate at FB27-6 and S25
			exhibit up to 90% section loss, Column G-27 East Flange exhibits 5%.
3	N/A		Span 28
		28-8	The majority of the steel members are in fair to good condition with
		28-5	minor rust and paint peeling except S13 and S25 which located directly
		28-9	under the longitudinal expansion joints. At those areas, steel stringers
			exhibit severe corrosion with section loss at midspan and at connections
			ranging from 5% to 16% at top flanges, 5% to 15% at bottom flange
			and 5% to 15% section loss at webs. Impact damages were also
			observed at span 28 stringer bottom flanges and webs at twelve
			different locations. The most significant damage was seen at S18
		28-6	between the first and second diaphragms. This impact damage is 5'- 4" long
		28-3	with distorted stringer bottom flange bent upward by 5" and cause
			the center line of the web to bent eastward by 6-1/4"
		28-7	At Span 28, S25 midspan exhibits severe corrosion. Top flange exhibits 10-
			12% section loss, web exhibits 10-25% and bottom flange exhibits 24%
			section loss.
			[29] Secondary Members
		3-3	Span 1-18
			All lateral diagonal bracing are in fair condition in general.
			However, the portion at gusset plate connection to primary members
			shows up to 40% section loss. The gusset plates exhibit up to 20%
			section loss. But, connection bolt nuts to gusset plate are 100%
			corroded. Also, the bolt show up to 15% section loss. This is typical
			at all gusset plates

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[29] Secondary Members
4	N/A	5-3	Spans 5-7
			Lateral diagonal bracing gusset plate exhibit severe corrosion on the top
			with section loss of up to 50%, connection bolt nuts to gusset plate are
			section loss. Lateral diagonal bracings are in good condition.
3	N/A		Spans 8 -18
		8-3	All lateral diagonal bracing are in fair condition in general. However, the
			portion at gusset plate connection to primary members shows up to 10%
			section loss. The gusset plates exhibit up to 10% section loss. But,
			connection bolt nuts to gusset plate are 100% corroded. Also, the bolt show
			up to 15% section loss. This is typical condition at all gusset plates.
			[30] Paint
2	N/A	6-2	Spans 1-7
			All steel members exhibit peeling paint in some degree. Especially members
			under timber decks and under the expansion joints where corrosion cause
			most of the paint to be separated from the base metal. Almost all the
			are faded. The inspector also noticed that there is only one layer of coating
			applied to the members
1	N/A		Spans 8-19
		10-2	Steel members show large areas have no paint/coating remaining and
			it is present, paint/coating is faded, peeling, and/or chalking.
4	N/A	20-1	Spans 20-22
			Some localized areas with no paint
3	N/A		Spans 23, 25, 26, 28 &29
		26-4	Steel members show some areas of no paint at top flanges and lower webs.
			All paint coatings are faded.
2	N/A	24-9	Spans 24 and 27
			Steel members below the expansion joints exhibits large area of no paint.
			The rest area in these spans has some paint peeling and chalking.

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[31] Joints
3	N/A	21-2	Span 21-22
		21-3	Underdeck framing between S11 & S12 — 3.5'L x 1'W x 1"Dp spall with
			rebar along top flange of S11. Adjacent deck area shows rusted rebar
			Steel framing shows light rust mostly on top flanges. Conditions occurred
			as a result of joint leakage.
			Spans 23-29
5	N/A	27-6	Both longitudinal and transverse joints are in good condition
			because of joint repair which appears to be done in recently.

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			PIER
			[36] Stem solid pier
4	N /A		Spans 1-2
			Concrete pedestal, which the base plate of steel columns anchored to, are
			rated as "stem solid pier"
		1-5	Two out of four concrete pedestals exhibit concrete spall up to 2"H x 2 ½"
		2-2	W x 26" L. One concrete pedestal exhibits vertical crack 21" L x ¼" W crack.
4	N/A		Spans 11,16 &17
			Most of concrete pedestals exhibit minor corner spall and honeycomb on
			vertical faces. But larger spalls are noted on:
		11-2	Column11/1 with 1.5'H x up to 8"W x 2"Dp corner spall,
		17-7	Added column 16B.1/1 with up to 1.5"Dp spall at all corners,
			Added column 17B.1/1 with 2'H x up to 6'W x 2"Dp corner spall.
			[37] Cap beam
6	N/A		Spans 9 and 17
			Lower portion of both old columns of pier 9 and entire left old column
			of pier 17 had been removed. The load path has been replaced with added
			double capbeams at original pier line and supported by added
			longitudinal exterior girder which in turn supported by added column.
			The added double capbeams have no defect noted.
4	N/A	11-3	Span 10
		11-4	At pier 10, similar to pier 9, there are added double capbeams. The one
			at span 11 side between S4A & S5A, has up to 1" impact damage to the
			web, mainly at upper 1' diameter. No crack is noted.

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ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			PIER (continue)
			[38] Pier columns
			Spans 1-7
			There are four steel columns in each span, common defects include section
			loss up to 10% at the lower column flanges near the base plate anchored to
			concrete pedestal. Connection bolt nuts exhibit corrosion and section loss.
			Top of the column exhibit severe corrosion but no section loss at few
			locations.
4	N/A		Span 1,2 7
		1-4	At Span 1, Column K-1, Column baseplate missing one bolt. The remaining
			3 nuts exhibit 40%, 60% and 90% section loss. Column west flange
			exhibits 5 – 10% section loss.
		7-3	At Span 7, Column L-7, all four column baseplate nuts exhibit section loss
			Up to 90%. Column flange exhibits minor section loss with corrosion and
			Paint peeling.
4	N/A		Span 7:
		8-1	Right column exhibits up to 10% web loss and 5% flange loss at base.
			Flanges show up to 1/8" kink. Also, 2 out of 4 anchor bolt nuts are 100%
			corroded.
			Span 8:
4	N/A	8-2	Right column exhibits up to 5% web loss at base. All 4 out of 4 anchor
			bolt nuts are 100% corroded. 1 out of 4 anchor bolt has 10% section
			loss. Also, 2 out of 2 connection bolts to base plate show 30% section loss.
5	N/A		Spans 9, 10 & 17:
			Lower portion of both old columns of pier 9 & 10 and entire left old
			column of pier 17 had been removed. The load path has been replaced
			with added double capbeams at original pier line and supported by added
			longitudinal exterior girder which in turn supported by added column.
		8-6	The added vertical diagonal bracing between added column 7B.1/2 & old
		9-1	column 8/2 is bent. No other distress is noted. Also, added Column 8.1/1
			shows 1 out of 2 anchor bolt nut not fully engaged with 3/16" gap. But,
			these conditions do not affect the load capacity of the columns, and,

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ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[38] Pier columns(continue)
			no defect noted on other added columns and old columns.
4	N/A	12-4	Span 12:
			At left column, the Inner flange exhibits up to 1" kink for 1'L at base.
			2 out of 4 anchor bolts are missing. Also, the remaining 2 anchor bolt
			nuts show 25% section loss. At right column, 2 out of 4 anchor bolt
			nuts exhibit 15% section loss.
4	N/A	13-1	Span 13:
		13-3	At Left column, 1 out of 4 anchor bolt has been sheared off. At right
			column, 2 out of 4 anchor bolts at inner side are missing. Also, the
			base plate has kinked corner at outer side up to 2".
4	N/A		Span 14:
		14-3	At right column, 2 out of 4 anchor bolts at inner side are missing.
			Span 23-29:
			Columns at these spans are generally in good condition except few columns
			exhibit severe corrosion with section loss up to 60%. Approximate 40% of
			of all the columns from spans 27-29 have spalls at the concrete encasement
			at lower portion of the columns, possibly due to water leakage inside the gap
			between the steel columns and encasement.
3	N/A		Span 23:
		23-1	At Span 23, column G-23, lower Col. exhibits advanced corrosion in web
		23-2	flanges. Estimated 59% overall column section loss. SEE RED FLAG P-004
4	N/A		Span 27, 28 and 29
		27-3	At Span 27, Column E-27, concrete encasement exhibits several cracks up
			to ½" wide and 5" long and concrete spalls 4" long x 1 ½" wide.

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[39] Footing
4	N/A		Span 2:
		2-3	Footing below Column K-2 exhibit scaling and spalling. Full length crack
			around the perimeter of the footing was noted at the lower portion.
			Vegetation was also noted at east face of the footing.
3	N/A	6-3	At Span 6, Column L-6, footing exhibits large area of damage. Approximate
			spall size 4'L x 3'W x 7 ½" H. No movement of substructure was observed.
			[41] Piles
5	N/A	3-6	Span 2 and 3:
			At span 3, Column Line K at bent 3, footing has exposed piles due to
			ground settlement, piles are in good condition.

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			UTILITIES
			[43] Lighting stands and fixtures
1	N/A		Span 1
		1-1	Span 1 upper level above west exterior girder, the lighting pole is missing.
		1-2	Exposed wires was noted, since it is outside the railing, pedestrian access to
			this area is very limited.
4	N/A	3-4	Spans 2-5:
			Street lightings at upper and main levels are in good conditions except only
			a few of them are functioning.
4	N/A		Span 9:
		9-5	Lighting standards above timber deck are in fair condition. But electrical
			junction box below G3 between FB8A & FB8B has missing cover
			with exposing insulated wiring. This area is not reachable by public.
			Span 28:
4	N/A	28-4	At span 28, betweenS24&S25, concrete haunch above the underside light
			fixture exhibits cracks and spalls with exposed rebar up to 2'L x 8" W
			[45] Utilities and utility support
3	N/A		Span 7:
		7-1	Disconnected west end of the utility hanger at span 7 between S13C and
			G4. Conduits exhibit slightly sag, the adjacent hangers are also broken but
			not disconnected.
3	N/A		Span 8:
		8-5	The two consecutive utility hangers cross below G3 between FB7B & FB8
			Show 90% section loss. The hanger spacing is 5'. No sign of sagging of
			utility pipes.
4	N/A		Span 17:
		17-8	Communication cable adjacent to added column 16B.1/1 is broken. The
			area is not reachable by public.

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TEAM LEADER: Mike Perng, P.E.& Jie Sun P.E.

ASST. TEAM LEADER: Tung For Ko & Charles Diamond

DATE: //

Feature Carried: PASSERELLE PED. BRIDGE

Feature Crossed: NYCT

RATINGS:		PHOTO	Remarks:
NEW	PREV	NO.	
			TP 350 Cont'd
			[45] Utilities and utility support (continue)
			Span 23:
4	N/A	23-5	Under S16, Junction box is missing a cover, insulated wires are exposed.
			This not reachable by the public.
4	N/A	28-1	At span 28 top of the deck, close up view of the broken conduit cover above
			S1 and FB28-1
4	N/A		Span 29:
		29-2	3' x 3' utility box at span 29 between S11 and S12 with missing cover and
			Wires. This area is not reachable by the public.

2.0 Steel Framing Deterioration Plans

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

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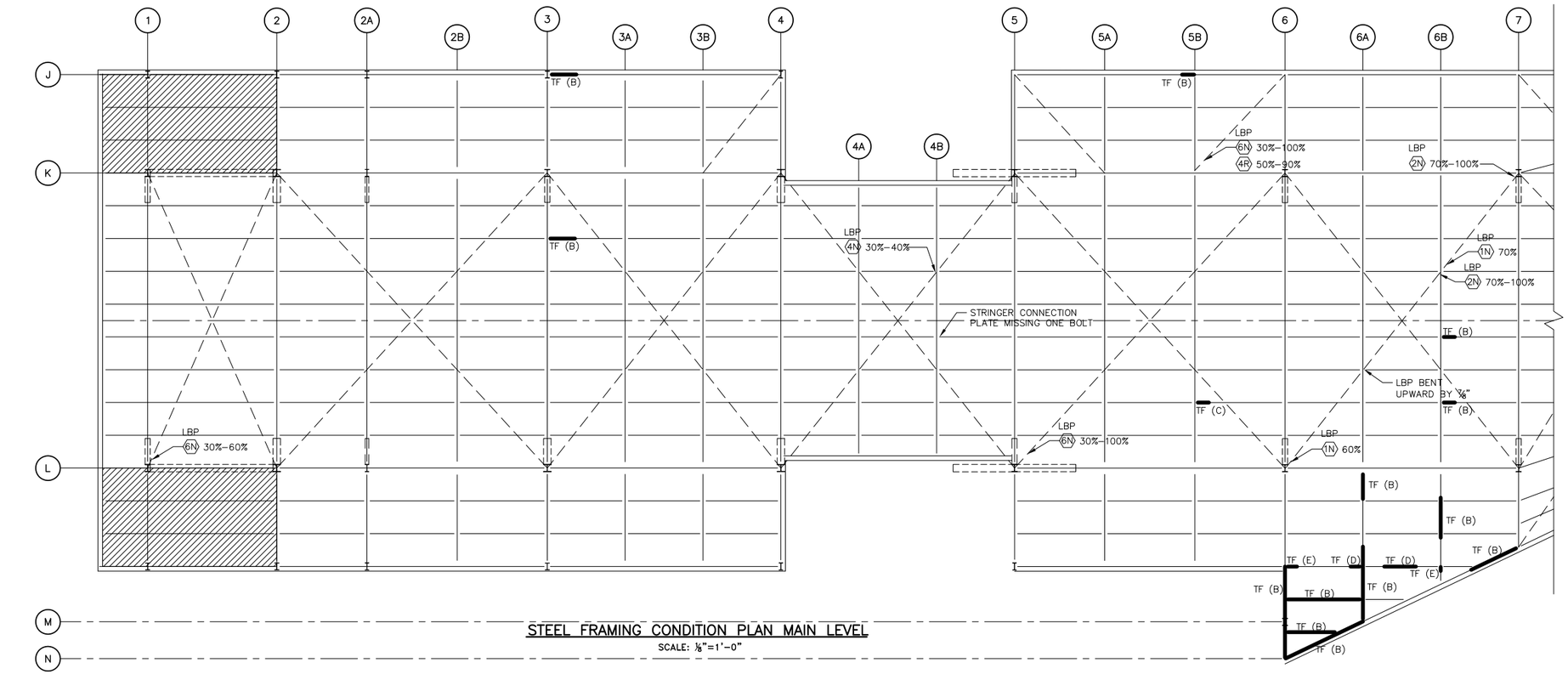

STEEL FRAMING CONDITION PLAN MAIN LEVEL

SCALE: ⅛"=1'-0"

LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
- C – SIGNIFICANT CORROSION WITH SECTION LOSS 15%–20%
- D – SEVERE CORROSION WITH SECTION LOSS UP TO 30%
- E – EXTREME CORROSION WITH SECTION LOSS 31% – 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%
- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

#R – NUMBER OF RIVET HEAD DETERIORATED

#N – NUMBER OF HEX NUTS DETERIORATED

% – PERCENTAGE OF SECTION LOSS

LBP – LATERAL BRACING PLATE DETERIORATION

– NO INSPECTION ACCESS

LBP – LATERAL BRACING PLATE

DRAFT

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SCALE AS SHOWN

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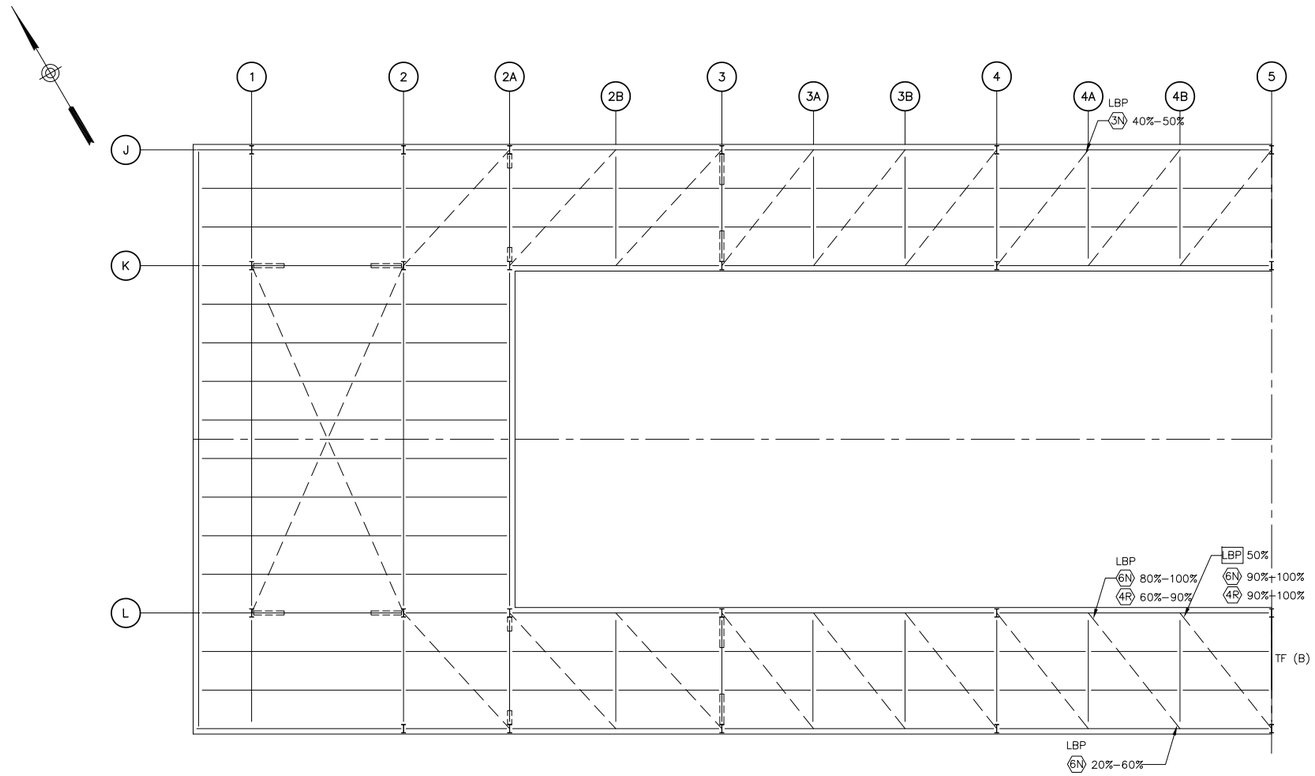
STEEL FRAMING PLAN EXISTING CONDITION
SPAN 1–7 BETWEEN COLUMN J TO N
MAIN LEVEL

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 09/27/13 | SHEET XX OF – | DWG No. F-1-7

CONSULTANT DESIGN

File name: G:\civil\21213014 – NYCDDC Passerelle Bridge\Engineering\CAD\Autocad\F-Span 1-Span 7 Wol.dwg Sep 30, 2013-1:52pm sun



STEEL FRAMING CONDITION PLAN UPPER LEVEL

SCALE: ⅛"=1'–0"

LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
- C – SIGNIFICANT CORROSION WITH SECTION LOSS 15%–20%
- D – SEVERE CORROSION WITH SECTION LOSS UP TO 30%
- E – EXTREME CORROSION WITH SECTION LOSS 31% – 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%

- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

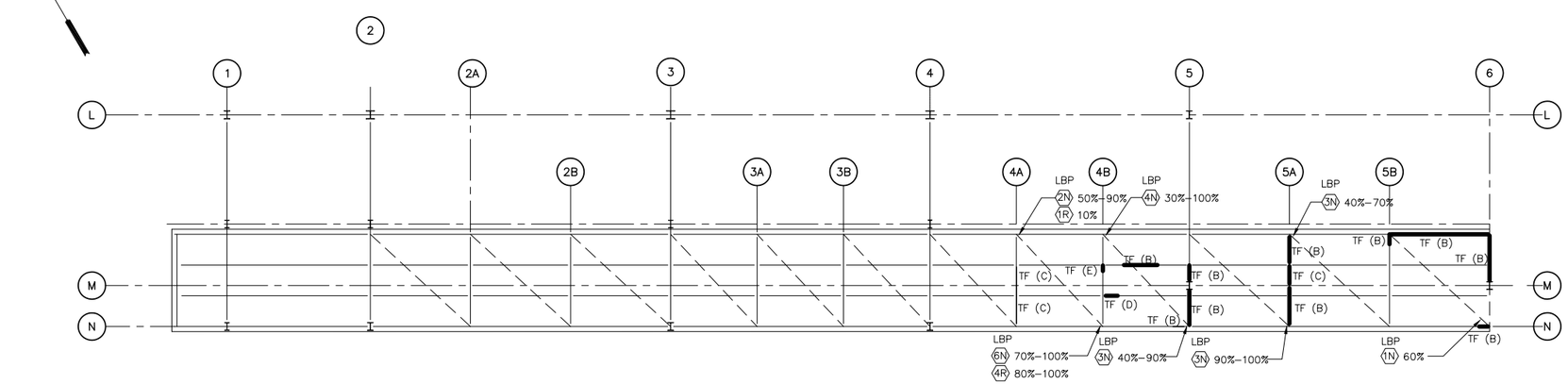
#R – NUMBER OF RIVET HEAD DETERIORATED

#N – NUMBER OF HEX NUTS DETERIORATED

% – PERCENTAGE OF SECTION LOSS

LBP – LATERAL BRACING PLATE DETERIORATION

LBP – LATERAL BRACING PLATE



STEEL FRAMING CONDITION PLAN LOWER LEVEL

SCALE: ⅛"=1'–0"

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STEEL FRAMING PLAN
EXISTING CONDITION SPAN 1–6
UPPER AND LOWER LEVEL

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 09/27/13 | SHEET XX OF – | DWG No. F-1-7 U&L

CONSULTANT DESIGN

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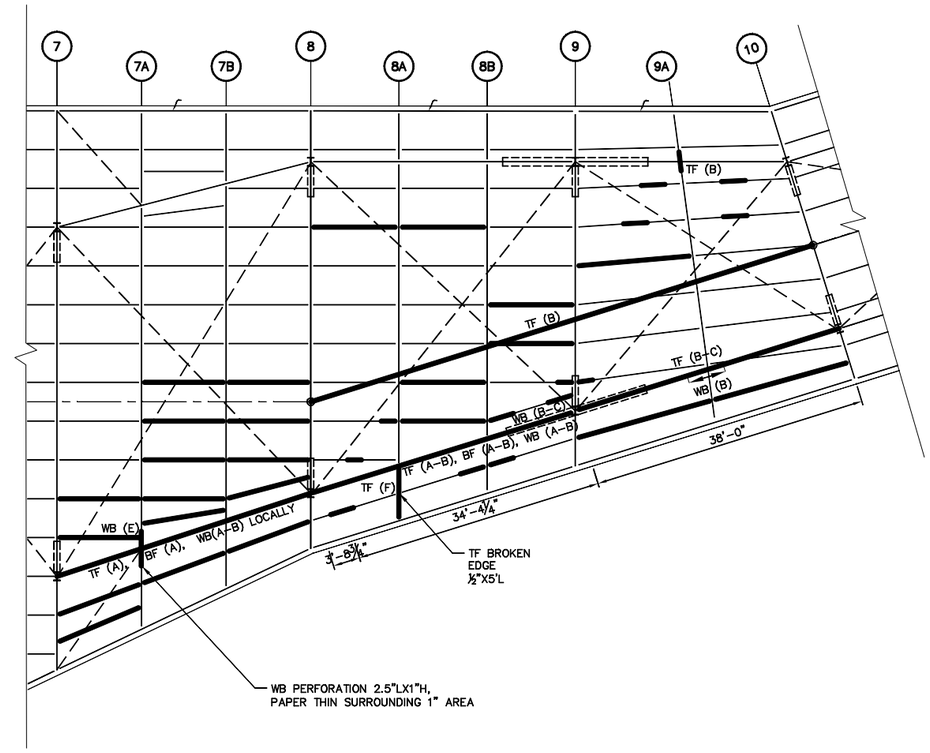

STEEL FRAMING CONDITION PLAN MAIN LEVEL

SCALE: ⅛"=1'-0"

LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
- C – SIGNIFICANT CORROSION WITH SECTION LOSS 15%–20%
- D – SEVERE CORROSION WITH SECTION LOSS UP TO 30%
- E – EXTREME CORROSION WITH SECTION LOSS 31% – 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%

- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

(#R) – NUMBER OF RIVET HEAD DETERIORATED

(#N) – NUMBER OF HEX NUTS DETERIORATED

% – PERCENTAGE OF SECTION LOSS

LBP – LATERAL BRACING PLATE DETERIORATION

– NO INSPECTION ACCESS

LBP – LATERAL BRACING PLATE

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STEEL FRAMING PLAN EXISTING CONDITION
SPAN 8–10 BETWEEN COLUMN J TO N
MAIN LEVEL

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 04/02/14 | SHEET XX OF – | DWG No. F-8-10

CONSULTANT DESIGN

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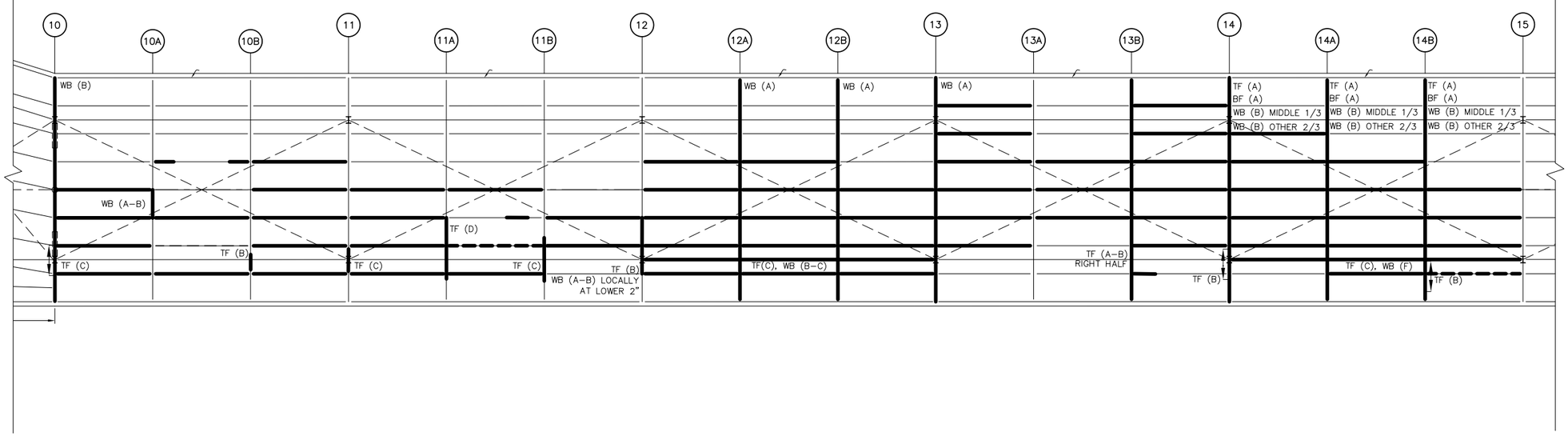

STEEL FRAMING CONDITION PLAN MAIN LEVEL

SCALE: ⅛"=1'–0"

LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
- C – SIGNIFICANT CORROSION WITH SECTION LOSS 15%–20%
- D – SEVERE CORROSION WITH SECTION LOSS UP TO 30%
- E – EXTREME CORROSION WITH SECTION LOSS 31% – 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%
- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

— — – REPAIRED/REPLACED IN EMERGENCY CONTRACT

(#R) – NUMBER OF RIVET HEAD DETERIORATED

(#N) – NUMBER OF HEX NUTS DETERIORATED

% – PERCENTAGE OF SECTION LOSS

LBP – LATERAL BRACING PLATE DETERIORATION

– NO INSPECTION ACCESS

LBP – LATERAL BRACING PLATE

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STEEL FRAMING PLAN EXISTING CONDITION
SPAN 11–15 BETWEEN COLUMN J TO N
MAIN LEVEL

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 04/02/14 | SHEET XX OF – | DWG No. F-11-15

CONSULTANT DESIGN

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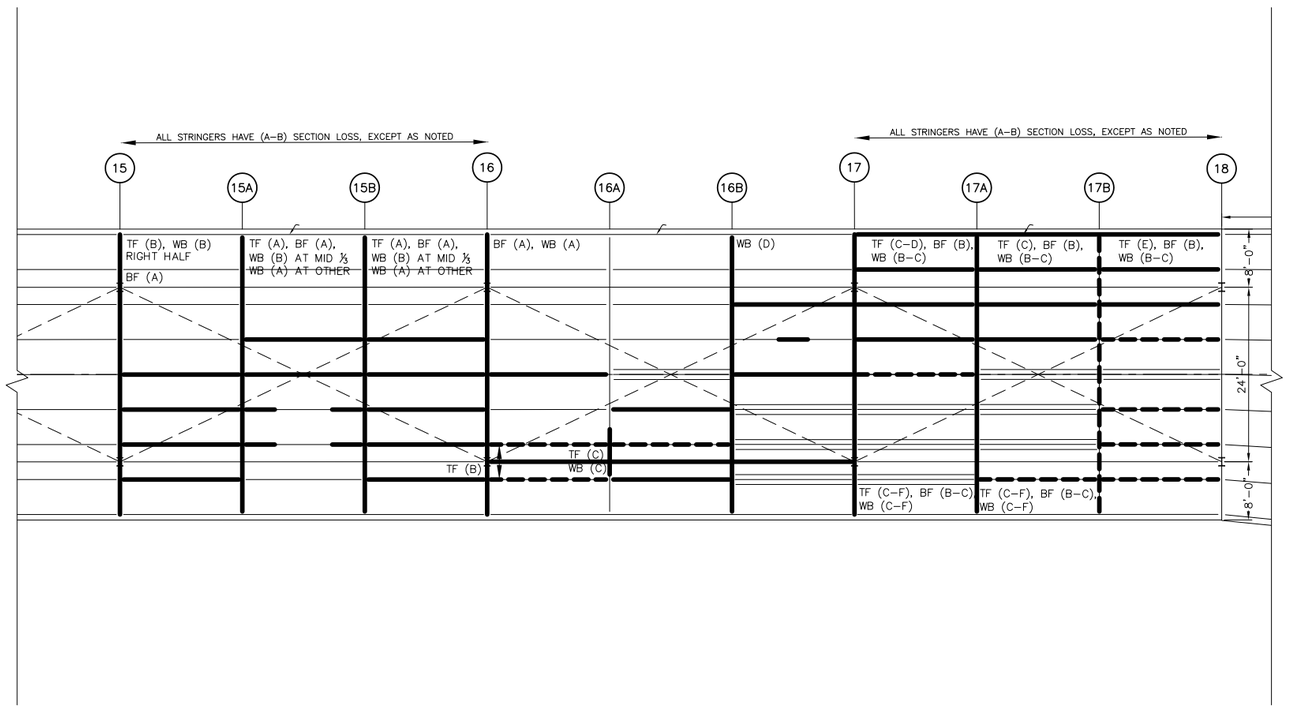

STEEL FRAMING CONDITION PLAN MAIN LEVEL

SCALE: ⅛"=1'–0"

LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
- C – SIGNIFICANT CORROSION WITH SECTION LOSS 15%–20%
- D – SEVERE CORROSION WITH SECTION LOSS UP TO 30%
- E – EXTREME CORROSION WITH SECTION LOSS 31% – 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%

- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

— — – REPAIRED/REPLACED IN EMERGENCY CONTRACT

#R – NUMBER OF RIVET HEAD DETERIORATED

#N – NUMBER OF HEX NUTS DETERIORATED

% – PERCENTAGE OF SECTION LOSS

LBP – LATERAL BRACING PLATE DETERIORATION

– NO INSPECTION ACCESS

LBP – LATERAL BRACING PLATE

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STEEL FRAMING PLAN EXISTING CONDITION
SPAN 16–18 BETWEEN COLUMN J TO N
MAIN LEVEL

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 04/02/14 | SHEET XX OF – | DWG No. F-16-18

CONSULTANT DESIGN

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LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
- C – SIGNIFICANT CORROSION WITH SECTION LOSS 15%–20%
- D – SEVERE CORROSION WITH SECTION LOSS UP TO 30%
- E – EXTREME CORROSION WITH SECTION LOSS 31% – 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%

- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

(#R) – NUMBER OF RIVET HEAD DETERIORATED

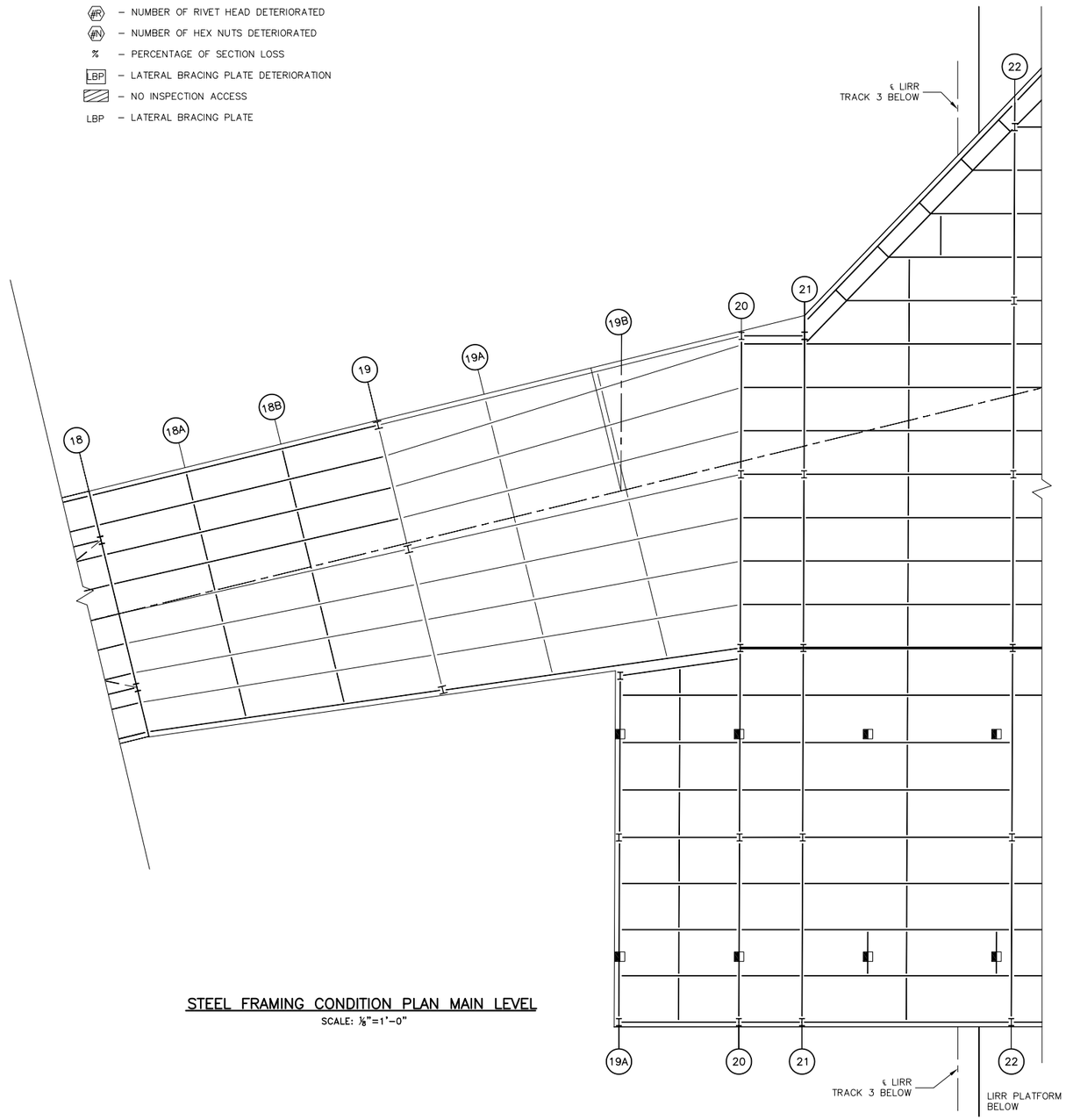
(#N) – NUMBER OF HEX NUTS DETERIORATED

% – PERCENTAGE OF SECTION LOSS

LBP – LATERAL BRACING PLATE DETERIORATION

– NO INSPECTION ACCESS

LBP – LATERAL BRACING PLATE



STEEL FRAMING CONDITION PLAN MAIN LEVEL

SCALE: ⅛"=1'–0"

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STEEL FRAMING PLAN EXISTING CONDITION
SPANS 19–22 BETWEEN COLUMN J TO N
MAIN LEVEL

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 04/02/14 | SHEET XX OF – | DWG No. F-19-22

CONSULTANT DESIGN

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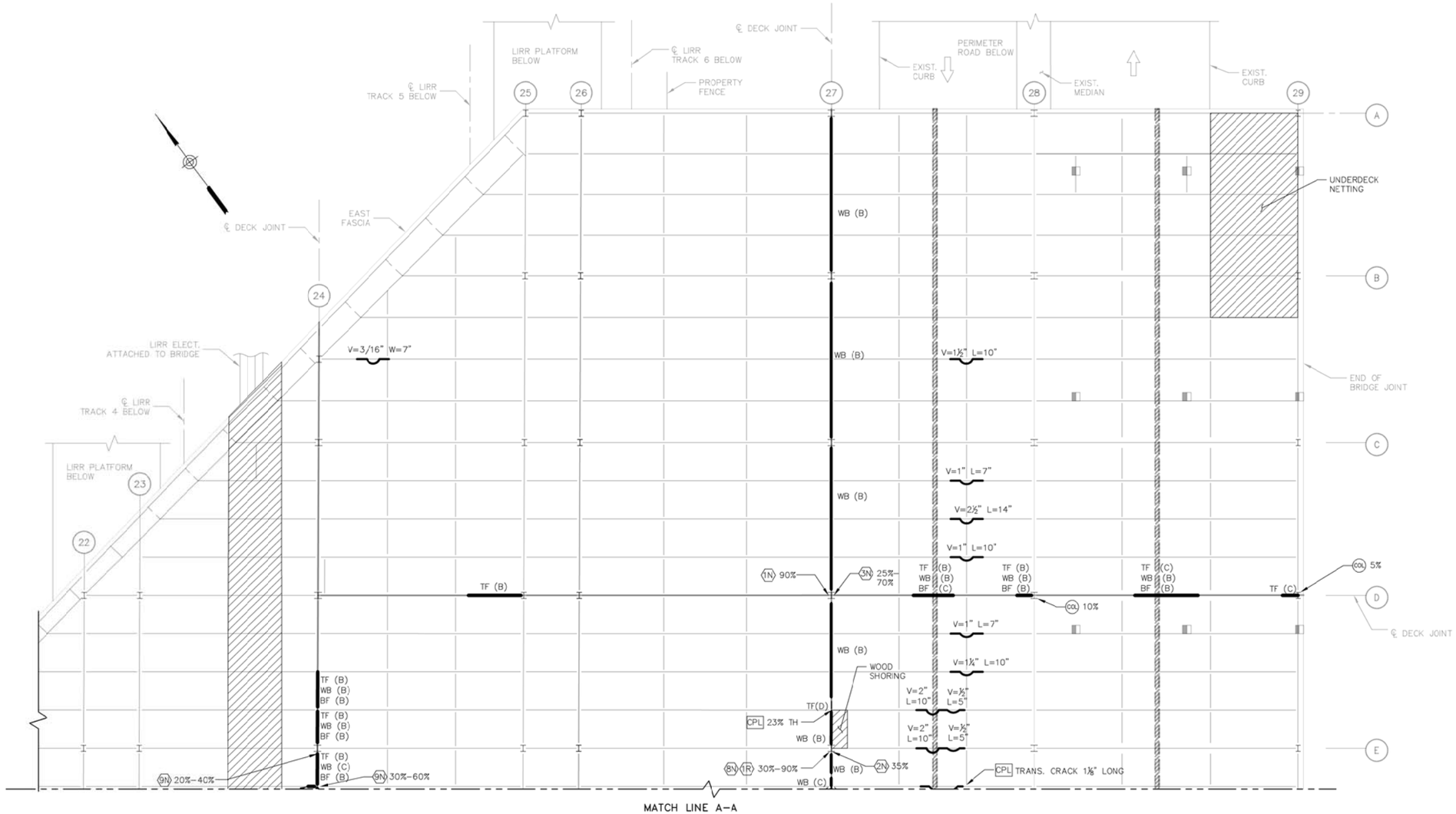

STEEL FRAMING CONDITION PLAN

SCALE: ⅛"=1'-0"

LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
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- D – SEVERE CORROSION WITH SECTION LOSS UP TO 30%
- E – EXTREME CORROSION WITH SECTION LOSS 31–TO 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%

- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

- H – HORIZONTAL DISPLACEMENT FROM CENTERLINE OF THE WEB DUE TO IMPACT
- V – VERTICAL DISPLACEMENT FROM CENTERLINE OF THE FLANGE DUE TO IMPACT
- L – LENGTH OF THE DAMAGE DUE TO IMPACT

- #R – NUMBER OF RIVET HEAD DETERIORATED
- #N – NUMBER OF HEX NUTS DETERIORATED
- % – PERCENTAGE OF SECTION LOSS
- COL – COLUMN DETERIORATION
- CPL – CONNECTION PLATE DETERIORATION
- NO INSPECTION ACCESS
- CANOPY COLUMN
- DRAINAGE SCUPPER
- TH – THROUGH HOLE

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NO.	DATE	DESCRIPTIONS	BY	APPR'D
		REVISIONS		

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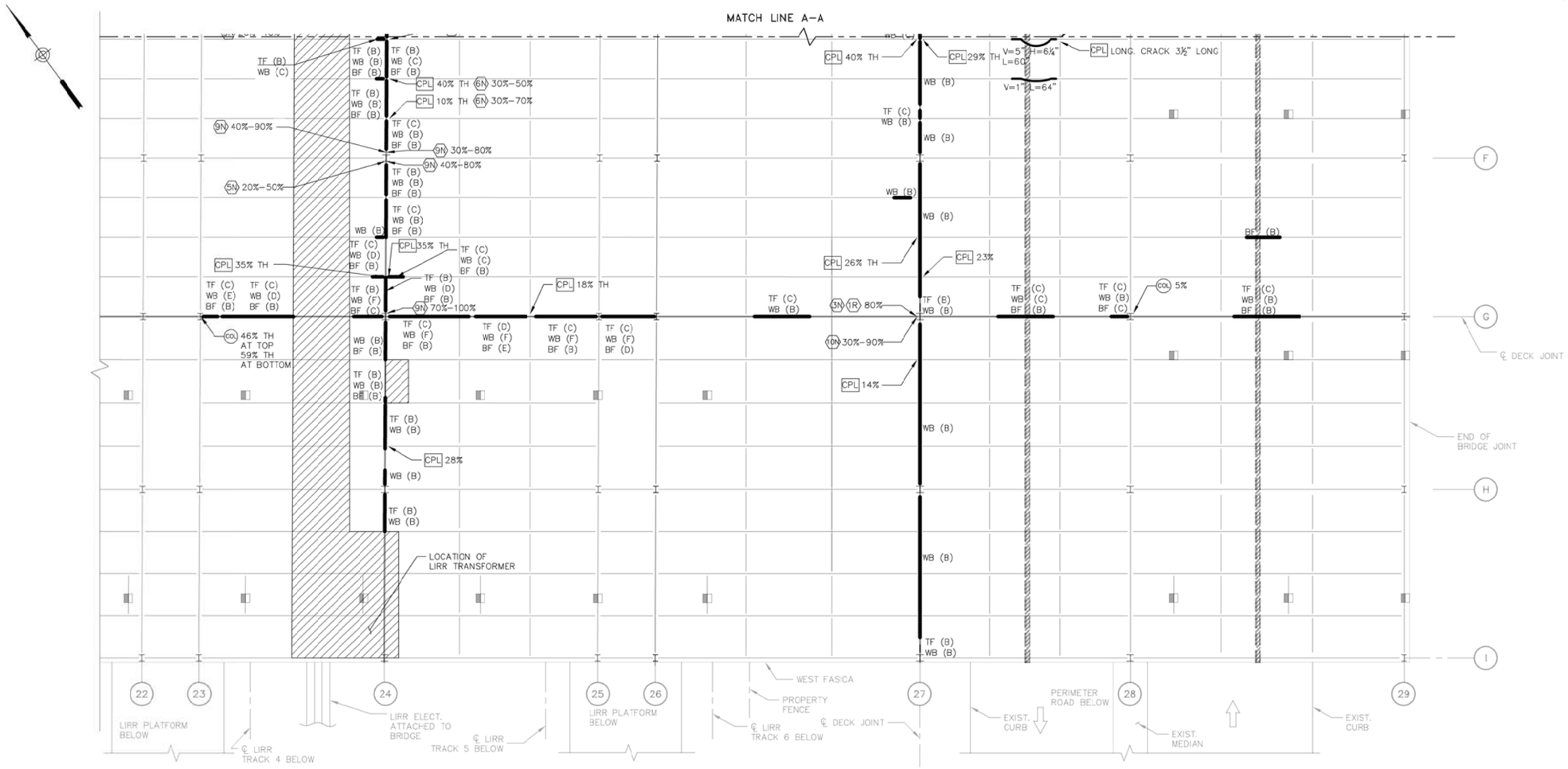
STEEL FRAMING PLAN
EXISTING CONDITION SPAN 23 –29
BETWEEN COLUMN LINES A TO E

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 09/27/13 | SHEET XX OF-- | DWG No. F-23-29

CONSULTANT DESIGN

File name: G:\civil\21213014 - NYCDDC Passerelle Bridge\Engineering\CAD\from HAKS 09-06-2013\F-Span 23-Span 29 Steel Framing Wsl.dwg Sep 30, 2013-9:47am sun



STEEL FRAMING CONDITION PLAN

SCALE: ⅛"=1'–0"

LEGEND

XX (X) – STRUCTURAL STEEL DETERIORATION

- A – SURFACE RUST
- B – NORMAL CORROSION WITH SECTION LOSS 5% – 14%
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- E – EXTREME CORROSION WITH SECTION LOSS 31–TO 50%
- F – EXTREME CORROSION WITH SECTION LOSS UP TO 100%
- BF – BOTTOM FLANGE
- TF – TOP FLANGE
- WB – WEB

- ⌒ – LIMITS OF IMPACT DAMAGE AT BOTTOM FLANGE
- H – HORIZONTAL DISPLACEMENT FROM CENTERLINE OF THE WEB DUE TO IMPACT
- V – VERTICAL DISPLACEMENT FROM CENTERLINE OF THE FLANGE DUE TO IMPACT
- L – LENGTH OF THE DAMAGE DUE TO IMPACT

- 1R – NUMBER OF RIVET HEAD DETERIORATED
- 1N – NUMBER OF HEX NUTS DETERIORATED
- % – PERCENTAGE OF SECTION LOSS
- COL – COLUMN DETERIORATION
- CPL – CONNECTION PLATE DETERIORATION
- NO INSPECTION ACCESS
- CANOPY COLUMN
- DRAINAGE SCUPPER
- TH – THROUGH HOLE

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STEEL FRAMING PLAN
EXISTING CONDITION SPAN 23–29
BETWEEN COLUMN LINES E TO I

PASSERELLE PEDESTRIAN BRIDGE

PROJECT ID: HBPED700Q | DATE: 09/27/13 | SHEET XX OF – | DWG No. F-23-29

CONSULTANT DESIGN

File name: G:\civil\21213014 – NYCEDC Passerelle Bridge\Engineering\CAD\from HAKS 09-06-2013\F-Span 23-Span 29 Steel Framing Wei.dwg Sep 30, 2013-9:49am sun

3.0 Photo Location Plans

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

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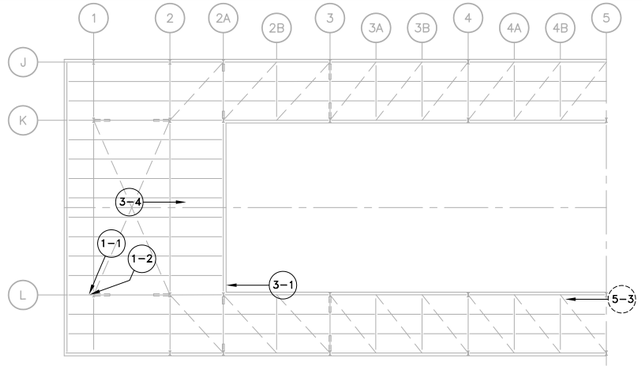
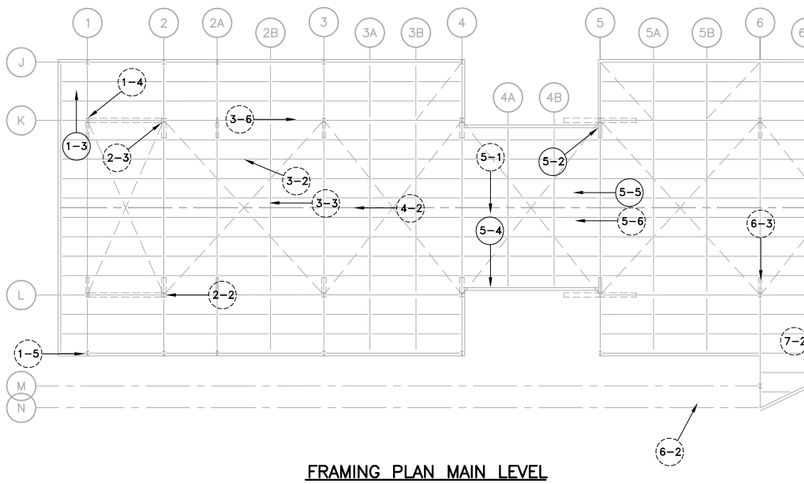


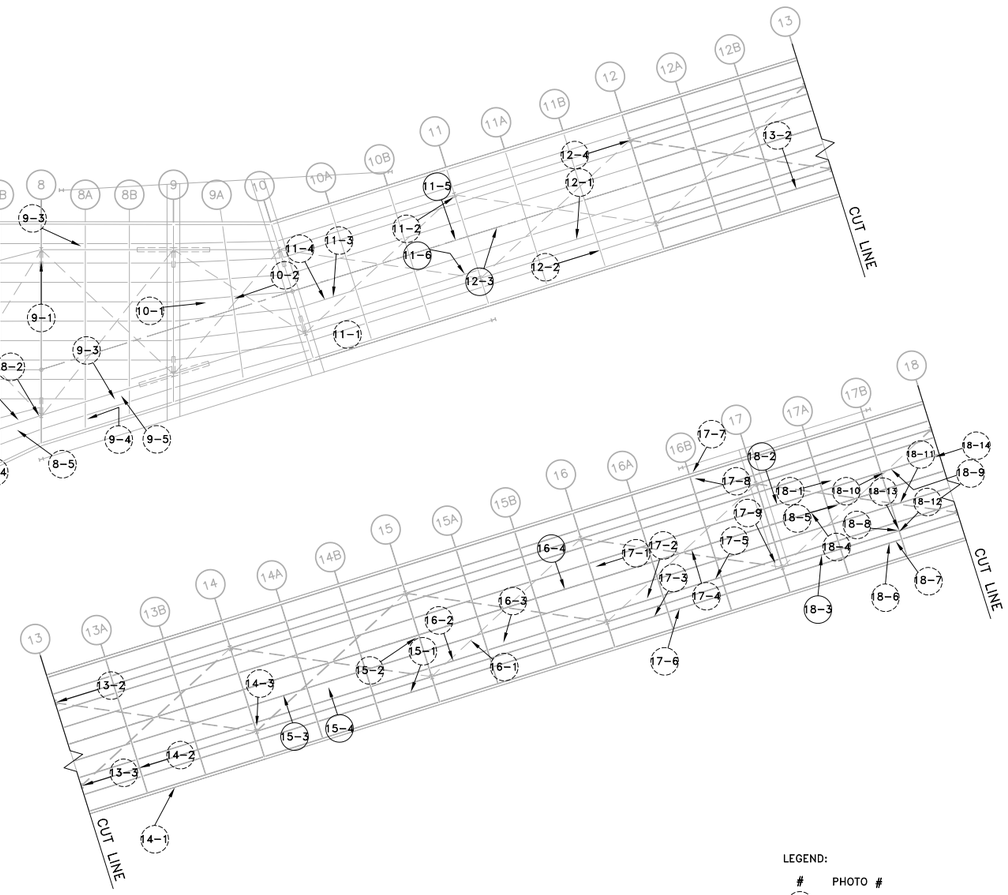
FRAMING PLAN UPPER LEVEL AND RAMPS

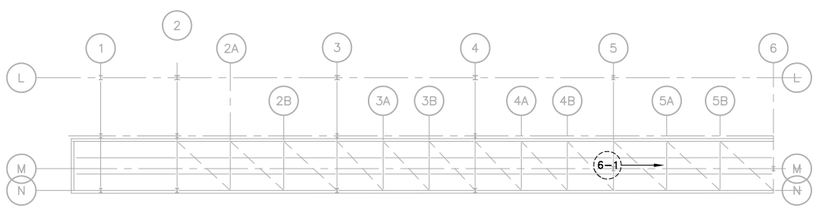
SCALE: $\frac{1}{16}$"=1'-0"



FRAMING PLAN MAIN LEVEL

SCALE: $\frac{1}{16}$"=1'-0"





FRAMING PLAN LOWER LEVEL RAMP

SCALE: $\frac{1}{16}$"=1'-0"

LEGEND:

#	PHOTO #
(#)	UNDER DECK
(#)	ABOVE DECK

NO.	DATE	DESCRIPTIONS	BY	APPR'D

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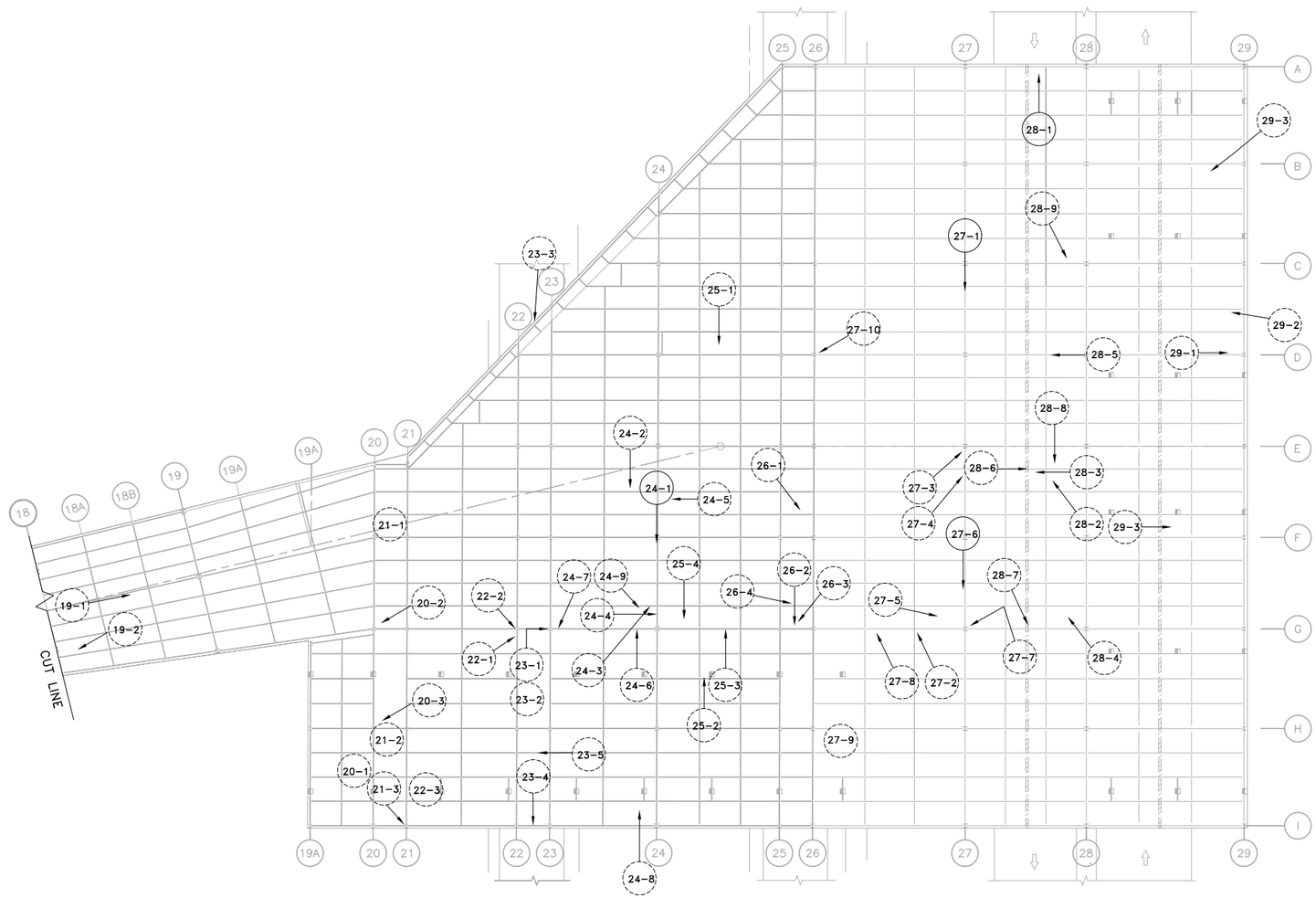
PASSERELLE PEDESTRIAN BRIDGE
PHOTO LOCATION PLAN I

PROJECT ID:	HBPED700E	DATE: 04/07/2014	SHEET OF –	DWG No.

CONSULTANT DESIGN

File name: G:\civil\21213014 – NYCDDC Passerelle Bridge\Engineering\CAD\Photo location plan.dwg Apr 07, 2014-3:36pm sun



FRAMING PLAN MAIN LEVEL

SCALE: $\frac{1}{16}$"=1'-0"

LEGEND:

- # PHOTO #
- (#) UNDER DECK
- (#) ABOVE DECK

NO.	DATE	DESCRIPTIONS	BY	APPR'D

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BUREAU OF DESIGN

PASSERELLE PEDESTRIAN BRIDGE
PHOTO LOCATION PLAN II

PROJECT ID: HBPED700E | DATE: 03/30/2014 | SHEET OF – | DWG No.

CONSULTANT DESIGN

File name: G:\civil\21213014 – NYCDDC Passerelle Bridge\Engineering\CADD\Photo location plan.dwg Mar 31, 2014-4:50pm sun

4.0 Photo Sheets

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

BRIDGE INSPECTION PHOTOS

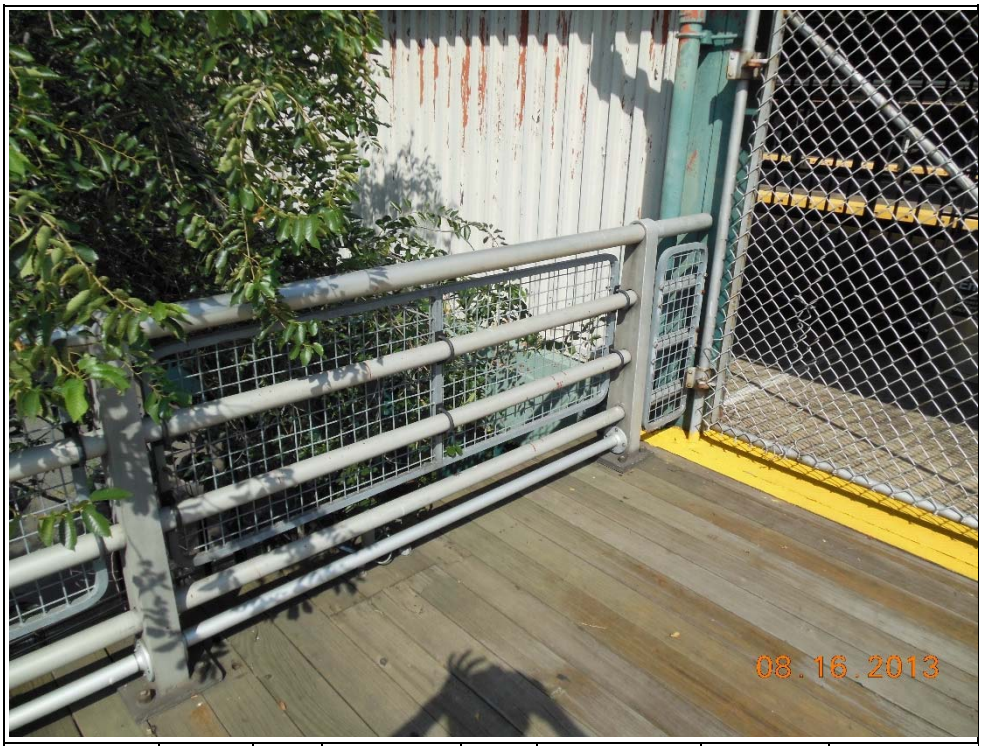


Photo no.	1-1	**BIN:**	N/A	**Type:**	Lighting	**Looking:**	Northwest
Description:	Span 1 upper level above west exterior girder, the lighting pole is missing.						

Photo no.	1-2	**BIN:**	N/A	**Type:**	Lighting	**Looking:**	West & down
Description:	Span 1 upper level above west exterior girder, the lighting pole is missing. Exposed wires was noted, since it is outside the railing, pedestrian access to this area is very limited.						

BRIDGE INSPECTION PHOTOS

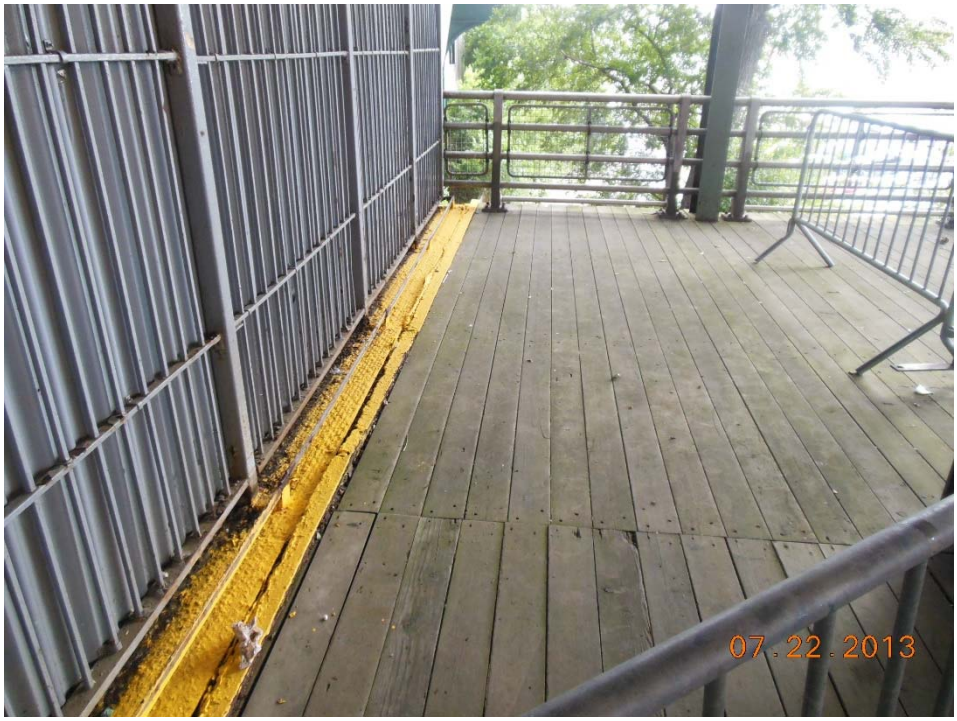


Photo no.	1-3	**BIN:**	N/A	**Type:**	Wearing Surf.	**Looking:**	East
Description:	Top surface of timber planks at span 1 exhibit deterioration, decay and checking at various locations.						

Photo no.	1-4	**BIN:**	N/A	**Type:**	Pier Column	**Looking:**	Northwest
Description:	At Span 1, Column K-1, Column baseplate missing one bolt. The remaining 3 nuts exhibit 40%, 60% and 90% section loss. Column West flange exhibits 5 – 10% section loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	1-5	**BIN:**	N/A	**Type:**	Stem solid pier	**Looking:**	South
Description:	At Span 1, Column LL-1, Column Pedestal exhibits concrete spall, 2”H x 2 ½” W x 26” L at the North Face.						

Photo no.	2-2	**BIN:**	N/A	**Type:**	Secondary	**Looking:**	North
Description:	At Span 2, Column L-2, column pedestal exhibits a 21” L x ¼” W crack on the west face of the footing.						

BRIDGE INSPECTION PHOTOS

Photo no.	2-3	**BIN:**	N/A	**Type:**	Footing	**Looking:**	Southeast
Description:	Footing below Column K-2 exhibit scaling and spalling. Full length crack around the perimeter of the footing was noted at the lower portion. Vegetation was also noted at east face of the footing.						

Photo no.	3-1	**BIN:**	N/A	**Type:**	Scuppers	**Looking:**	North
Description:	Closed up view of the disconnected drainage downpipe at column L at bent 2A.						

BRIDGE INSPECTION PHOTOS

Photo no.	3-2	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	Northeast
Description:	At Span 3, between FB2A-2 & FB2B-2 at S4B, rotted timber nailer with severe deterioration.						

Photo no.	3-3	**BIN:**	N/A	**Type:**	Secondary	**Looking:**	North
Description:	General view of lateral bracing under span 3, typical.						

BRIDGE INSPECTION PHOTOS

Photo no.	3-4	**BIN:**	N/A	**Type:**	Lighting	**Looking:**	South
Description:	Street lightings at upper and main levels are in good conditions except only a few of them are functioning.						

Photo no.	3-5	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	East
Description:	At Span 3, underside of timber deck shows checking, decay in general. Approx. 30% of the timber planks are deteriorated, 10% has hairline cracks.						

BRIDGE INSPECTION PHOTOS

Photo no.	3-6	**BIN:**	N/A	**Type:**	Pile	**Looking:**	South
Description:	At span 3, Column Line K at bent 3, footing has exposed piles due to ground settlement, piles are in good condition.						

Photo no.	4-2	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	North and up
Description:	At Span 4, between FB3-2 & FB3A-2, S6A and S7A, cracked timber plank exhibits 2” separation.						

BRIDGE INSPECTION PHOTOS

Photo no.	5-1	**BIN:**	N/A	**Type:**	Wearing Surf.	**Looking:**	West
Description:	At Span 5, between FB4-1 and FB4A, S4A and S5A, Timber plank exhibits a 8" L x ½" W crack. (Typical at few locations)						

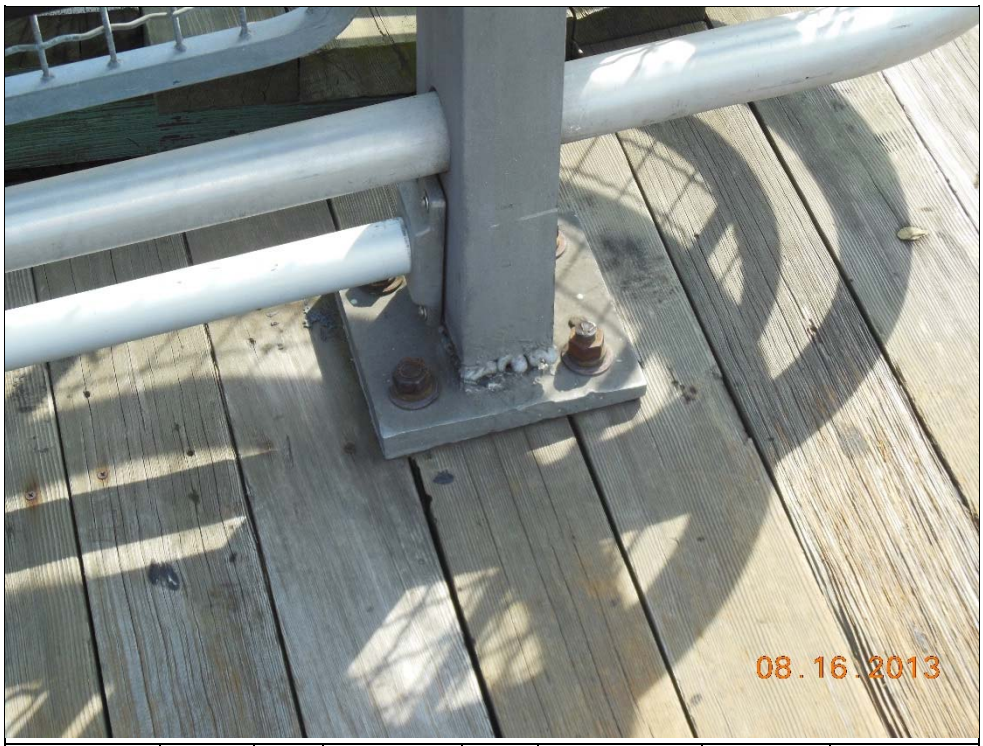


Photo no.	5-2	**BIN:**	N/A	**Type:**	railings	**Looking:**	Southeast
Description:	Span 5 main level, One railing post is disconnected from its base plate but stable, broken welds were observed. The rest four railings at this side are in good condition.						

BRIDGE INSPECTION PHOTOS

Photo no.	5-3	**BIN:**	N/A	**Type:**	Second. Memb.	**Looking:**	North
Description:	At Span 5 Upper Level, FBU4B-2 South Face and GU3, Bracing Plate exhibits up to 50% section loss, the nuts and rivets exhibit up to 100% section loss. (Typical)						

Photo no.	5-4	**BIN:**	N/A	**Type:**	Railings	**Looking:**	West
Description:	At Span 5, between FB4-1 and FB4A-1, Missing one rail at the lower part of the west railing.						

BRIDGE INSPECTION PHOTOS

Photo no.	5-5	**BIN:**	N/A	**Type:**	Wearing surface	**Looking:**	
Description:	General condition of timber decks at span 1-5, some deterioration and decay.						

Photo no.	5-6	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	North
Description:	General view of primary members from Spans 1 to 5.						

BRIDGE INSPECTION PHOTOS

Photo no.	6-1	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	South
Description:	At Span 6 Lower Level, FBL5A North Face between SL1A & SL2A, top flange exhibits severe corrosion with 15-20% section loss.						

Photo no.	6-2	**BIN:**	N/A	**Type:**	Paint	**Looking:**	East
Description:	General condition of the paint at span 6-7						

BRIDGE INSPECTION PHOTOS

Photo no.	6-3	**BIN:**	N/A	**Type:**	Footing	**Looking:**	West
Description:	At Span 6, Column L-6, footing exhibits large area of damage. Approximate spall size 4'L x 3'W x 7 ½" H. No movement of substructure was observed.						

Photo no.	7-1	**BIN:**	N/A	**Type:**	Utilities	**Looking:**	Southeast
Description:	Close up view of disconnected west end of the utility hanger at span 7 between S13C and G4.						

BRIDGE INSPECTION PHOTOS

Photo no.	7-2	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	South
Description:	At Span 7, FB6B-3 North Face, between S12B & 13B, top flange exhibits severe corrosion with localized 50% section loss						

Photo no.	7-3	**BIN:**	N/A	**Type:**	Pier Column	**Looking:**	South
Description:	At Span 7, Column L-7, all four column baseplate nuts exhibit section loss Upto 90%. Column flange exhibits minor section loss with corrosion and paint peeling.						

BRIDGE INSPECTION PHOTOS

Photo no.	8-1	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	Northwest
Description:	Column 7/2 — Up to 10% web loss and 5% flange loss at base. Flanges show up to 1/8" kink. Also, 2 out of 4 anchor bolt nuts are 100% corroded.						

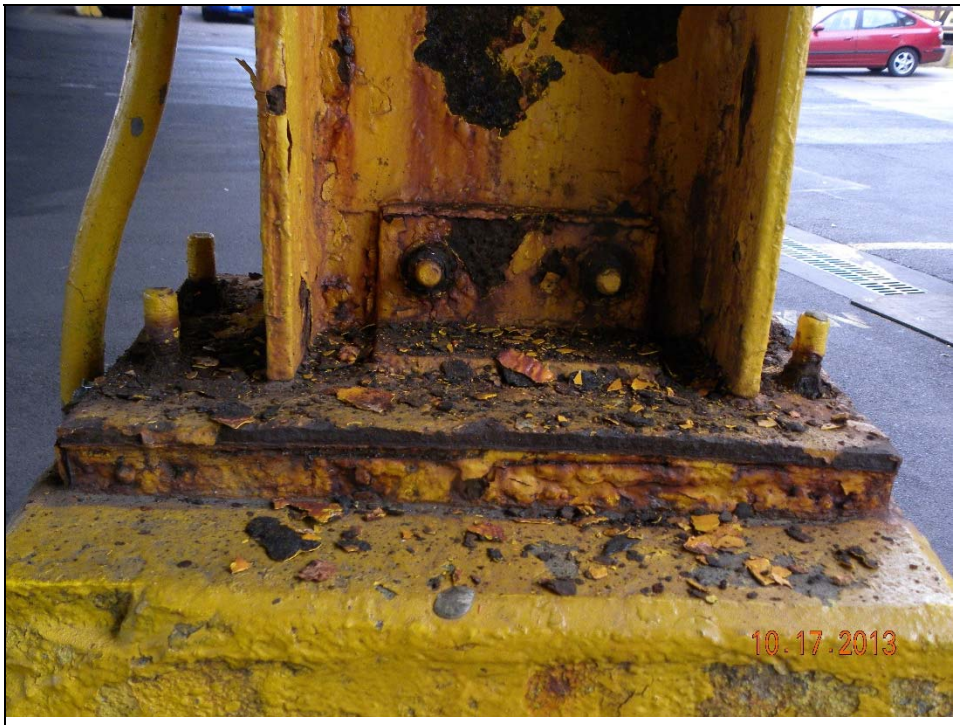


Photo no.	8-2	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	South
Description:	Column 8/2 — Up to 5% web loss at base. 4 out of 4 anchor bolt nuts are 100% corroded. 1 out of 4 anchor bolt has 10% section loss. Also, 2 out of 2 connection bolts to base plate show 30% section loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	8-3	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	West
Description:	Gusset plate to horizontal diagonal bracing between FB7 & G3 — All connection bolt nuts to gusset plate are 100% corroded. Also, connection bolts show up to 15% section loss. Typical condition at gusset plates.						

Photo no.	8-4	**BIN:**	N/A	**Type:**	Deck	**Looking:**	West
Description:	FB7A — Perforation in the floor beam web near the connection to G3. The surrounding portion of the web near the perforations is paper thin exhibiting extreme corrosion. Total section loss for the member is 40%.						

BRIDGE INSPECTION PHOTOS

Photo no.	8-5	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	North
Description:	Utility hanger cross below G3 between FB7B & FB8 — Two consecutive hangers show 90% section loss.						

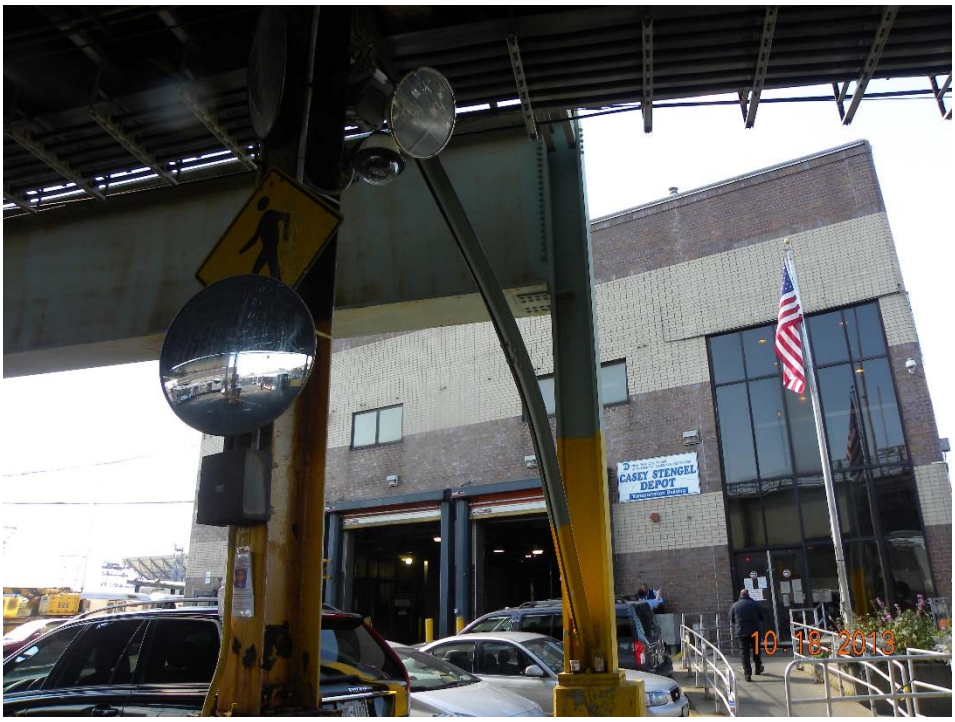


Photo no.	8-6	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	Southeast
Description:	Vertical diagonal bracing between added column 7B.1/2 & column 8/2 — Diagonal bracing is bent. No other distress is noted.						

BRIDGE INSPECTION PHOTOS

Photo no.	9-1	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	Northeast
Description:	Added Column 8.1/1 — 1 out of 2 anchor bolt nut is not fully engaged with 3/16" gap.						

Photo no.	9-2	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	South
Description:	Web connection plate between FB8A & G2 — Bottom 4" of free end is bent up to 1", no other distress is noted.						

BRIDGE INSPECTION PHOTOS

Photo no.	9-3	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	South
Description:	S9B between FB8A & FB8B — Top flange of stringer exhibits broken edge of 1/4"W x 7'L.						

Photo no.	9-4	**BIN:**	N/A	**Type:**	Deck	**Looking:**	West
Description:	FB8A between G3 & S9B — Top flange of floor beam exhibits broken edge of 1/2"W x 5'L.						

BRIDGE INSPECTION PHOTOS

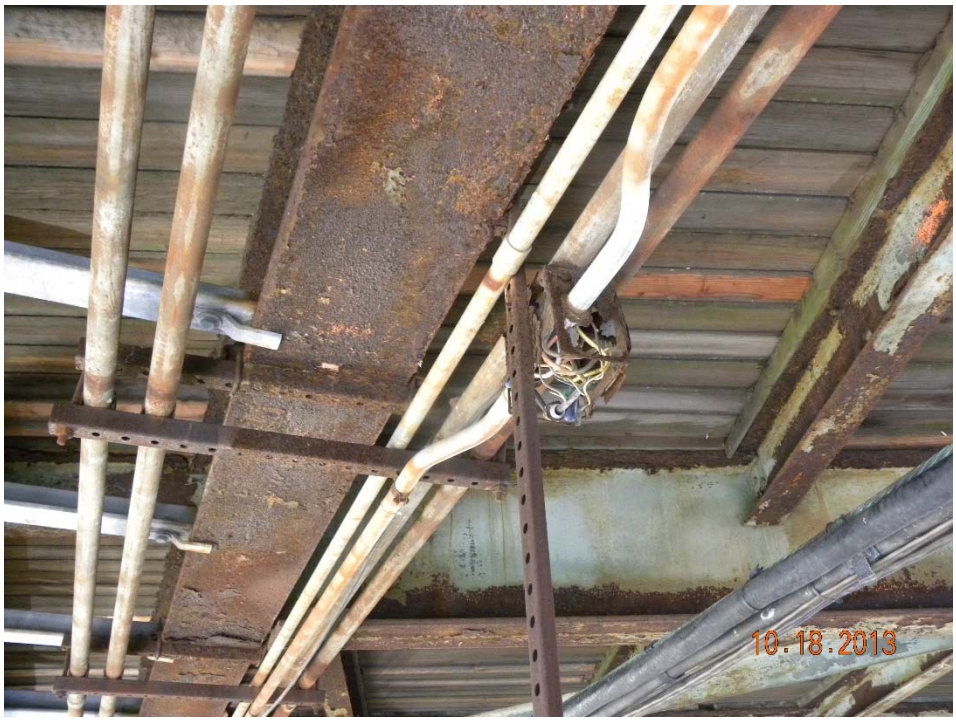


Photo no.	9-5	**BIN:**	N/A	**Type:**	Utilities	**Looking:**	North
Description:	Electrical junction box below G3 between FB8A & FB8B — Junction box has missing cover with exposing insulted wiring. Not reachable by public.						

Photo no.	10-1	**BIN:**	N/A	**Type:**	General	**Looking:**	East
Description:	Underdeck framing — General view. No significant defect is noted.						

BRIDGE INSPECTION PHOTOS

Photo no.	10-2	**BIN:**	N/A	**Type:**	General	**Looking:**	West
Description:	Underdeck framing — General view. No significant defect is noted.						

Photo no.	11-1	**BIN:**	N/A	**Type:**	Deck	**Looking:**	East
Description:	S6B between FB10A & FB10B — Extreme corrosion with large perforation and paper thin in the lower 4" of web along bottom flange for 12' long starting from 3' away from Floorbeam FB10A. The remaining web shows up to 10% section loss. Also, the flanges exhibit up to 15% section loss with broken 1/8" of top flange edge throughout. The worst total section loss at 6' away from floorbeam FB10A with 45% web loss and 20% flange loss.						

BRIDGE INSPECTION PHOTOS

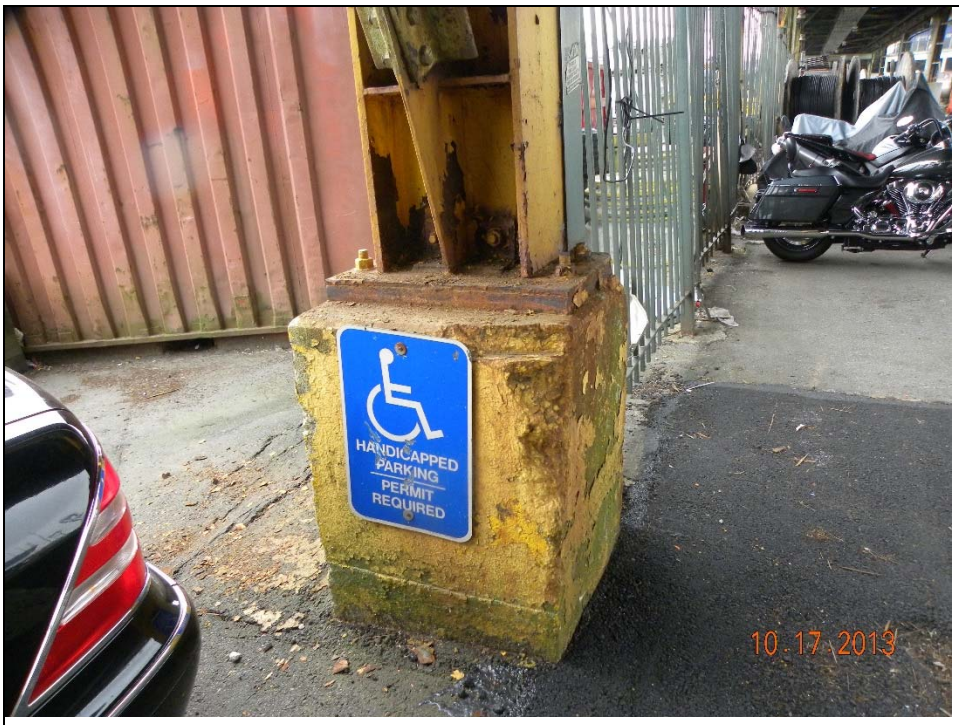


Photo no.	11-2	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	East
Description:	Column 11/1 — Minor web loss at base. Concrete pedestal shows 1.5'H x 8"W x 2"Dp corner spall.						

Photo no.	11-3	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	West
Description:	Added Cross Girder CG10.2 between S4A & S5A — Up to 1" impact damage to the web, mainly at upper 1' diameter area. No crack is noted.						

BRIDGE INSPECTION PHOTOS

Photo no.	11-4	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	South
Description:	Added Cross Girder CG10.2 between S4A & S5A — Up to 1" impact damage to the web, mainly at upper 1' diameter area. No crack is noted.						

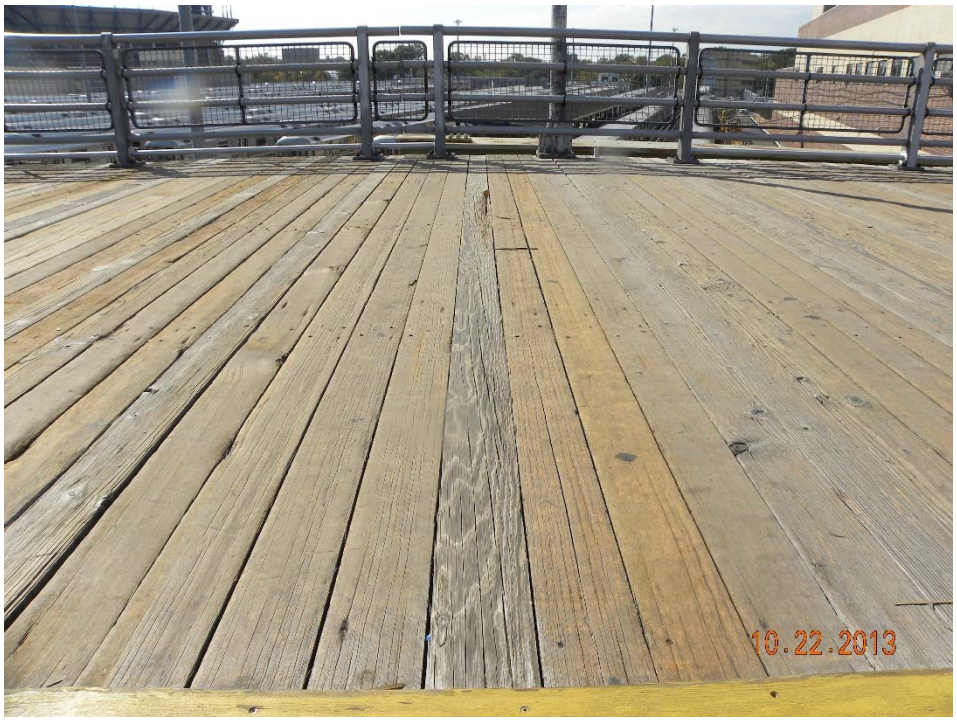


Photo no.	11-5	**BIN:**	N/A	**Type:**	Deck	**Looking:**	Southwest
Description:	Top of timber deck between FB108 & FB11 and S4C & G4 — Timber deck shows checking and partially loose connection to supporting stringers in general.						

BRIDGE INSPECTION PHOTOS

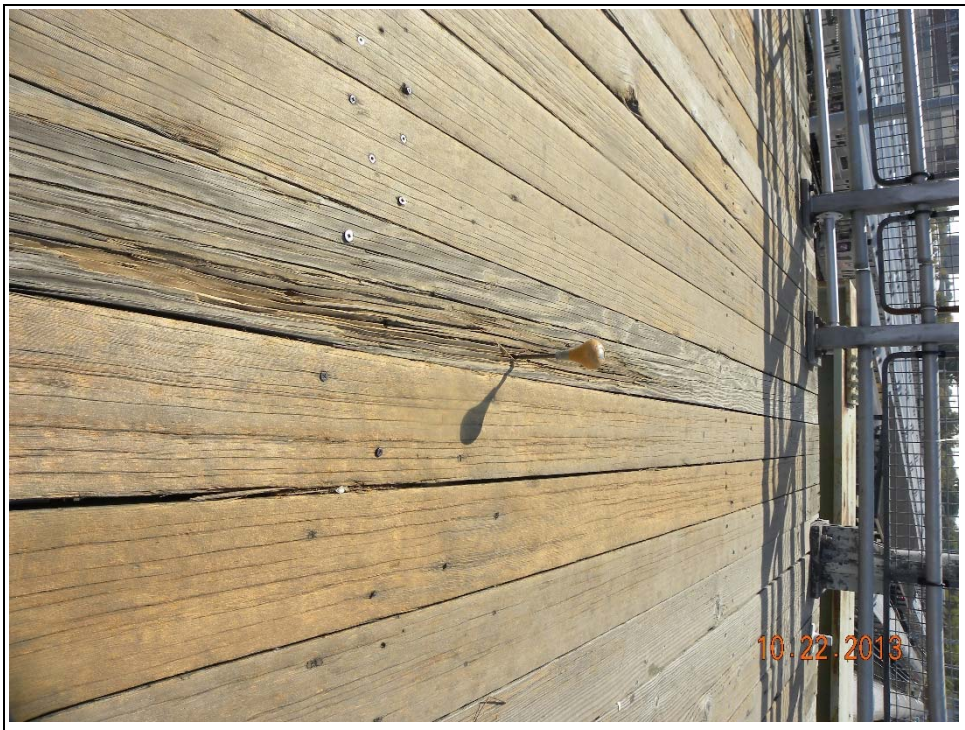


Photo no.	11-6	**BIN:**	N/A	**Type:**	Deck	**Looking:**	Southwest
Description:	Top of timber deck between FB108 & FB11 and S5C & S7C — Timber deck shows checking & decay.						

Photo no.	12-1	**BIN:**	N/A	**Type:**	Deck	**Looking:**	West
Description:	S6B between FB11A & FB11B — Extreme corrosion with large perforation and paper thin in the lower 3" of web along bottom flange throughout. Remaining of web shows up to 10% section loss. Also, the top flange exhibits up to 50% section loss with broken 1" edge throughout. The bottom flange has less than 5% section loss. The worst total section loss is at 2' away from Floorbeam FB11B with 35% web loss and 30% flange loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	12-2	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Southeast
Description:	Connection between FB11B & G3 — Both connection bolt nuts exhibit 70% section loss.						

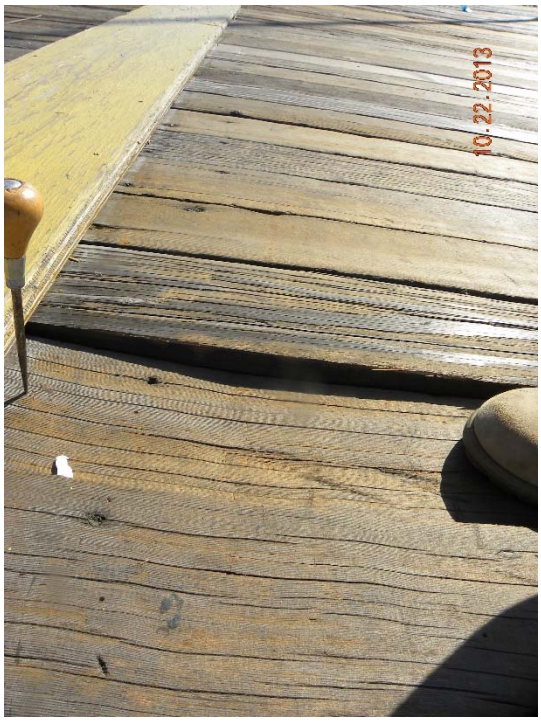


Photo no.	12-3	**BIN:**	N/A	**Type:**	Deck	**Looking:**	East
Description:	Top of timber Deck at 11' after FB11 between S4A & S5A — One timber board has a fine 1/2"Dp crack crossing the fibers at 1.5' from S4A. The board deflects about 1" when stepped on. There is a potential of a punch through. The adjacent timber board before the board described above is soft as well and exhibits ½ inch deflection under loading.						

BRIDGE INSPECTION PHOTOS

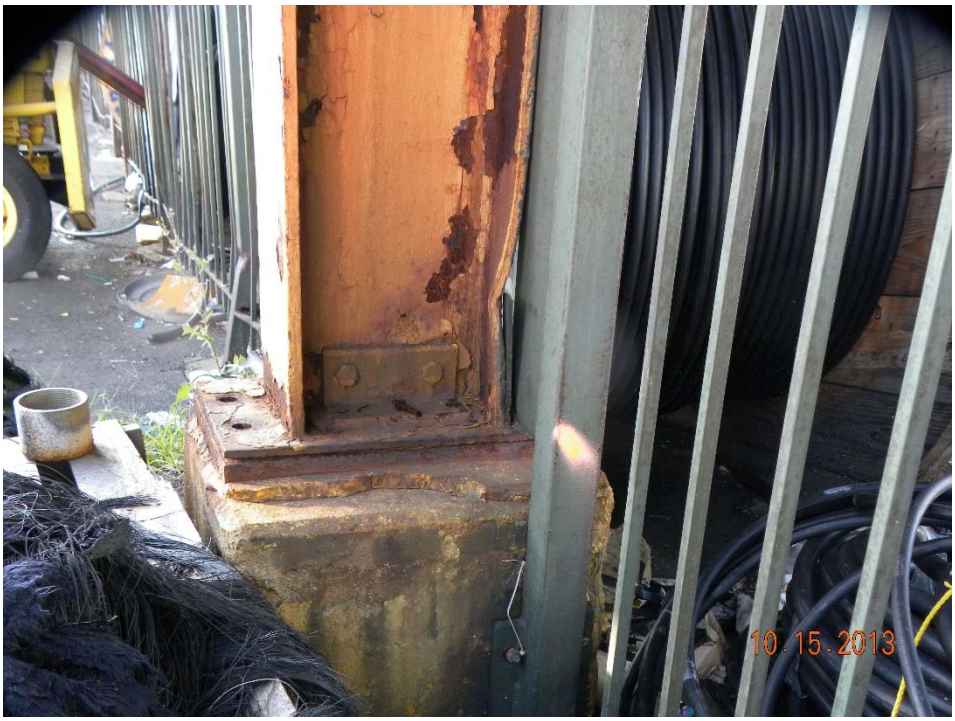


Photo no.	12-4	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	East
Description:	Column12/1 — Inner column flange exhibits up to 1" kink for 1'L at base. 2 out of 4 anchor bolts are missing. Also, the remaining 2 anchor bolt nuts show 25% section loss.						

Photo no.	13-1	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	West
Description:	Column13/1 — 1 out of 4 anchor bolt has been sheared off.						

BRIDGE INSPECTION PHOTOS

Photo no.	13-2	**BIN:**	N/A	**Type:**	Deck	**Looking:**	South
Description:	S7C between FB12B & FB13 — Extreme corrosion with large perforation and paper thin in the lower 4" of web along bottom flange for 4' long starting from 5' away from Floorbeam FB13. Remaining web shows up to 20% section loss. Also, the top flange exhibits a range of 5% to 20% section loss with up to 1/8" broken edge throughout. Bottom flange shows up to 5% section loss. The worst total section loss is at 8' away from Floorbeam FB13 with 50% web loss and 15% flange loss.						

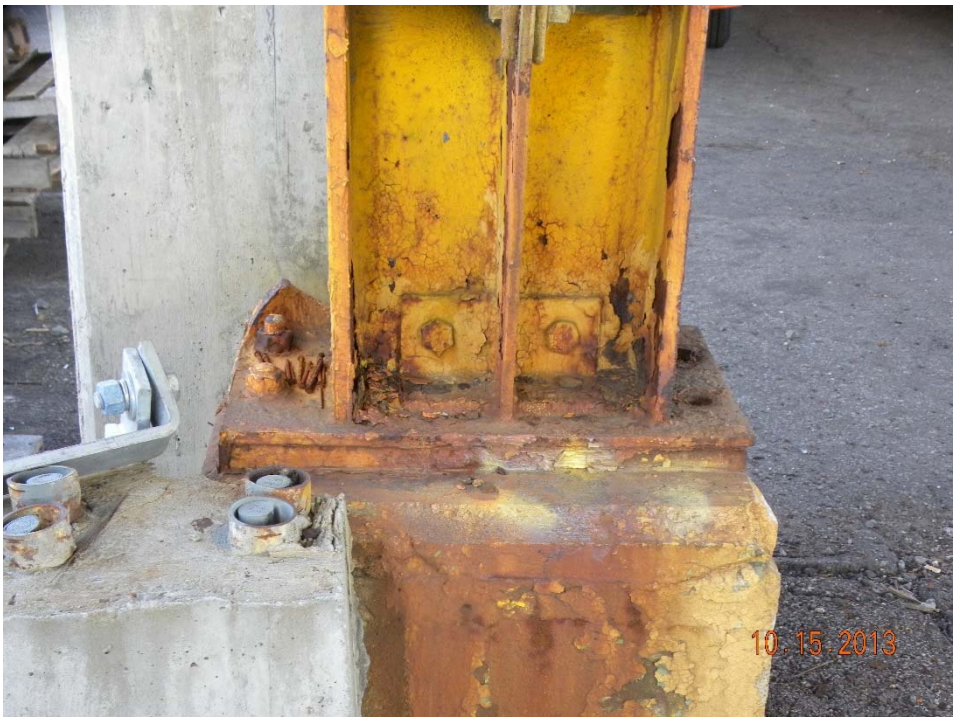


Photo no.	13-3	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	West
Description:	Column 13/2 — 2 out of 4 anchor bolts at inner side are missing. Also, the base plate has kinked corner at outer side up to 2".						

BRIDGE INSPECTION PHOTOS

Photo no.	14-1	**BIN:**	N/A	**Type:**	General	**Looking:**	East
Description:	Underside of framing — Inspector at work. No significant defect is noted.						

Photo no.	14-2	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	West
Description:	Connection between FB13A & G3 — Both connection bolt nuts exhibit 40% section loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	14-3	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	Southwest
Description:	Column 14/2 — 2 out of 4 anchor bolts at inner side are missing.						

BRIDGE INSPECTION PHOTOS

Photo no.	15-1	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Southwest
Description:	S7C between FB14B & FB15 — Extreme corrosion with large perforation and paper thin in the lower 2.5" of web along bottom flange for 12' long starting 3' away from Floorbeam FB15. Remaining web shows up to 20% section loss. Also, the top flange exhibits up to 20% section loss with up to 1/2" broken edge locally. Bottom flange shows up to 5% section loss. The worst total section loss is at 3' away from Floorbeam FB15 with 30% web loss and 20% flange loss.						

Photo no.	15-2	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	East
Description:	FB15 at S4C — Lower web at mid-third exhibits up to 10% section loss. Isolated minor surface loss noted at other areas.						

BRIDGE INSPECTION PHOTOS

Photo no.	15-3	**BIN:**	N/A	**Type:**	Deck	**Looking:**	Northeast
Description:	Top of timber deck at 4' after FB14 between S4A & S5A — One timber board has a fine surface crack crossing the fibers above S5A. The board is soft and exhibits ½ inch deflection under loading.						

Photo no.	15-4	**BIN:**	N/A	**Type:**	Deck	**Looking:**	Northeast
Description:	Top of timber deck at 2.5' after FB14A between S4B & S5B — One timber board has a fine surface crack crossing the fibers above S5B. The board is soft and shows up to 3/8"W check with ½ inch deflection under loading.						

BRIDGE INSPECTION PHOTOS

Photo no.	16-1	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	North
Description:	S5A between FB15 & FB15A — Extreme corrosion with perforations in size of 2'L x 2"H & 6"H x 2"H, and, paper thin in the lower 2.5" of web along bottom flange for most of the member's length. Remaining web shows up to 5% section loss. Also, the flanges exhibit up to 15% section loss with up to 1/2" broken top flange at mid-span. The worst section loss is at mid-span with 30% web loss and 20% flange loss.						

Photo no.	16-2	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	South
Description:	S6A between FB15 & FB15A — Extreme corrosion with large perforation and paper thin in the lower 4" of web along bottom flange for 4' long near the center of the span. Another corroded hole of 1.5" diameter is noted on the web below the connection to FB15A. Also, both flanges exhibit 15% section loss throughout. The worst cross section loss is at mid-span with 40% web loss and 15% flange loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	16-3	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Southwest
Description:	S6B between FB15A & FB15B — Extreme corrosion with local perforation and paper thin in the lower 2" of web along bottom flange for up to 3' long starting 1.0' away from either ends. Remaining of web shows average of 5% section loss. Also, the top flanges exhibits average of 20% section loss. Bottom flange shows up to 5% section loss. The worst total section loss is at 1.5' away from Floorbeam FB15A with 25% web loss and 12% flange loss						

Photo no.	16-4	**BIN:**	N/A	**Type:**	Deck	**Looking:**	Southwest
Description:	Top of timber deck at 2' after FB15B between S3C & S4C — Timber deck shows checking & decay.						

BRIDGE INSPECTION PHOTOS

Photo no.	17-1	**BIN:**	N/A	**Type:**	Deck	**Looking:**	West
Description:	Bottom of timber deck between FB16 & FB16A and S3A & S4A — Timber deck shows fine checking in random location.						

Photo no.	17-2	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	West
Description:	S7A between FB16 & FB16A — Stringer exhibits extreme corrosion near the connection to FB16A. Perforation and paper thin in the lower 2" of web along bottom flange for 7' long. Remaining of web shows up to 10% section loss. Also, the top flange exhibits average of 10% section loss with broken 1" edge in 1' long. Bottom flange shows up to 5% section loss. The worst total section loss is at 2.5' away from Floorbeam FB16A with 25% web loss and 10% flange loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	17-3	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	West
Description:	S6A between FB16 & FB16A — Stringer exhibits extreme corrosion near the connection to FB16A. Perforation and paper thin in the lower 1" of web along bottom flange for 4' long, and, the top flange exhibits broken 1/2" edge in 1.5' long. The worst total section is at 3' away from Floorbeam FB16A with 15% web loss and 10% flange loss.						

Photo no.	17-4	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	North
Description:	S5B between FB16A & FB16B — Stringer exhibits extreme corrosion near the connection to FB16A. Perforation and paper thin in the lower 2" of web along bottom flange for 1' long, and, remaining 50% of web shows up to 25% section loss with few random pin holes. The top flange exhibits 10% section loss. The worst total section is at connection to FB16A with 30% web loss and 10% flange loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	17-5	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Southwest
Description:	S6B between FB16A & FB16B — Extreme corrosion with large perforation and paper thin on the lower web along bottom flange in size of 4"L x 3"H at quarter point from FB16A and 2'L x 4"H at quarter point from FB16B. Remaining of web shows up to 10% section loss. Also, the top flange exhibits up to 15% section loss with up to 1" broken edge randomly throughout. The worst cross section loss is at 4' away from Floorbeam FB16B with 45% web						

Photo no.	17-6	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	East
Description:	S7B between FB16A & FB16B — Extreme corrosion with large perforation and paper thin on the lower 3" web along bottom flange for 10' long starting from FB16A. Remaining of web shows up to 15% section loss. Also, the top flange exhibits up to 25% section loss with up to 1/2" broken edge throughout. The worst cross section loss is at connection to Floorbeam FB16A with 40% web loss and 20% flange loss.						

BRIDGE INSPECTION PHOTOS

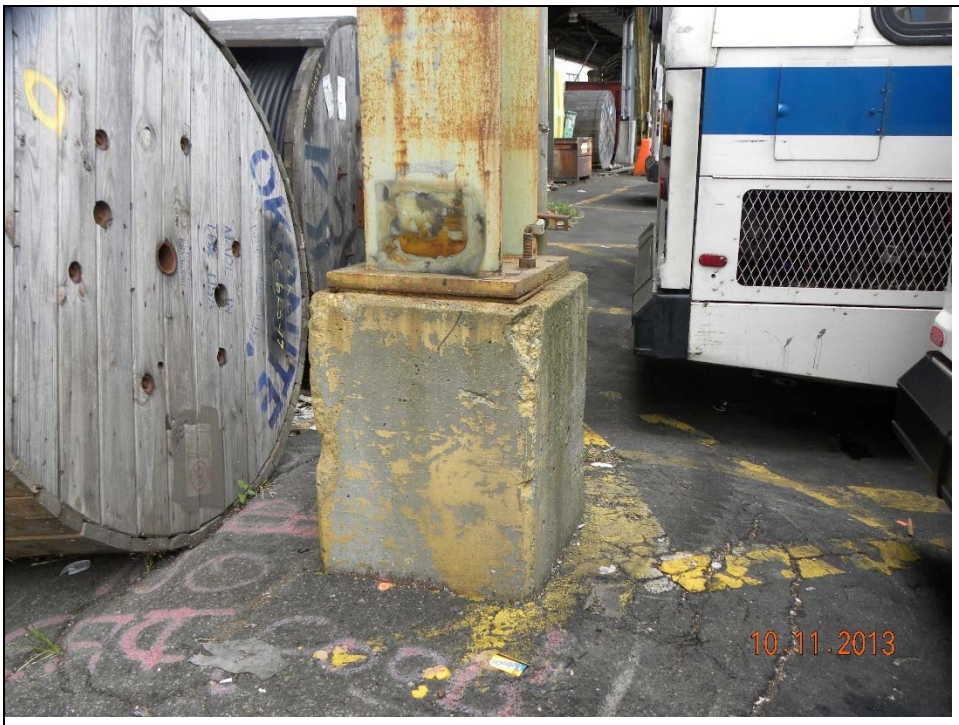


Photo no.	17-7	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	West
Description:	Added column 16B.1/1 — Up to 1.5" Dp corner spall on concrete pedestal.						

Photo no.	17-8	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Northwest
Description:	Communication cable adjacent to added column 16B.1/1 — Broken cable. Not reachable by public.						

BRIDGE INSPECTION PHOTOS

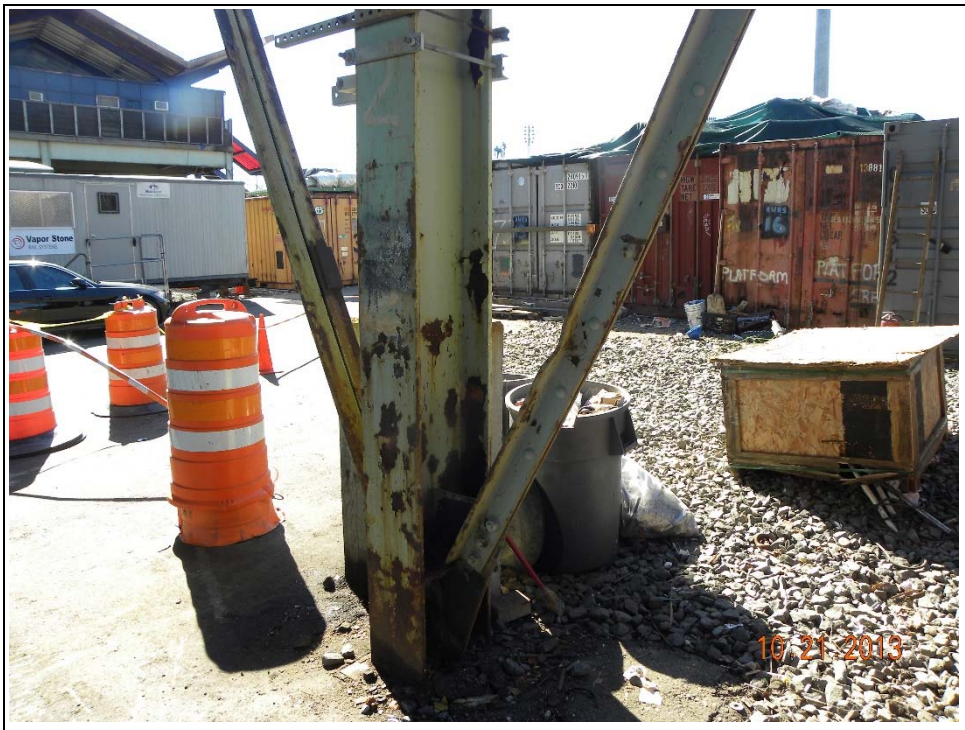


Photo no.	17-9	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	South
Description:	Vertical diagonal bracing between G3 & column 17/2 — Impact damage to the bracing causing lower 5' of member bend outward up to 1" with a 1'L x 1" maximum kink.						

Photo no.	18-1	**BIN:**	N/A	**Type:**	Deck	**Looking:**	East
Description:	Underside of timber deck between FB17A & FB17B — Timber deck shows fine checking in random location.						

BRIDGE INSPECTION PHOTOS

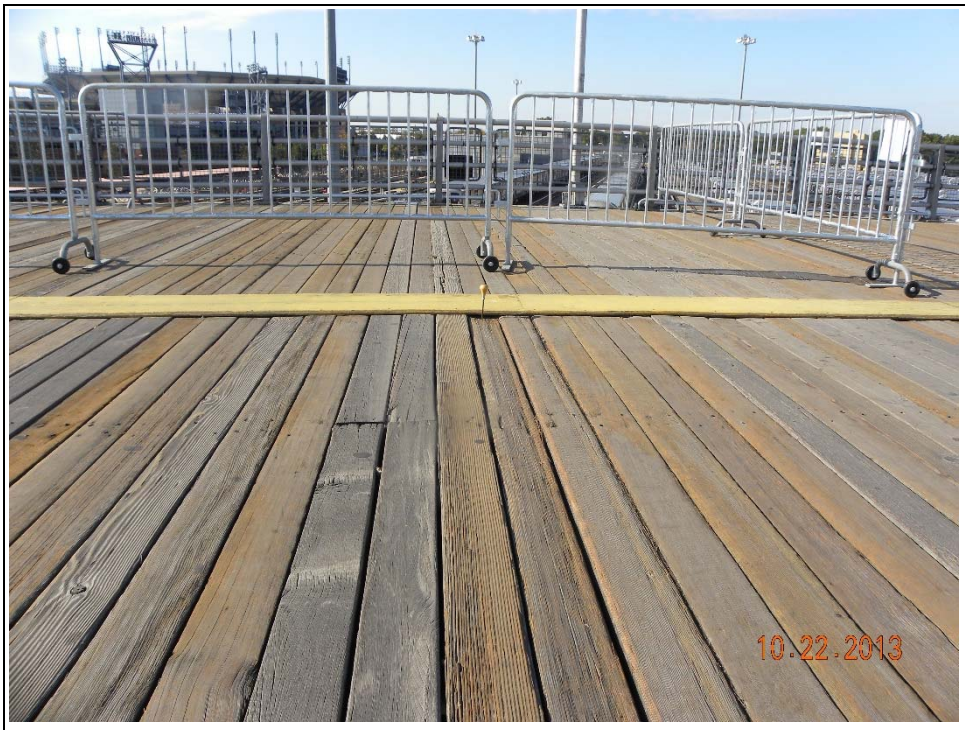


Photo no.	18-2	**BIN:**	N/A	**Type:**	Deck	**Looking:**	South
Description:	Top of timber deck between FB17 & FB17A — Timber deck shows fine checking in random location.						

Photo no.	18-3	**BIN:**	N/A	**Type:**	Deck	**Looking:**	Northeast
Description:	Top of timber deck at 2' before FB17A between S4A & G4 — A warping timber board has partially loose connection to supporting stringers. Remaining timber deck shows fine checking in random location.						

BRIDGE INSPECTION PHOTOS

Photo no.	18-4	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	North
Description:	S4A between FB17 & FB17A — Extreme corrosion with large perforation and paper thin on the lower 3" web along bottom flange throughout. Remaining of web shows 25% section loss. Also, the top flange exhibits up to 30% section loss with up to 1/4" broken edge throughout. Bottom flange shows 10% section loss as well. The cross section loss has 45% web loss and 25% flange loss throughout.						

Photo no.	18-5	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	East
Description:	Underdeck framing between FB17A & FB18 — Replacement work done for S4B, S5B, S6B & S4C by installing a channel on each side of the deteriorated stringers.						

BRIDGE INSPECTION PHOTOS

Photo no.	18-6	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Northeast
Description:	S7B between FB17A & FB17B — Up to 1.5" high perforation on lower web along the bottom flange for 90% of the span length. Other portions of the web are paper thin. The top flange has a broken edge up to ¼". The bottom flange exhibits up to 15% section loss—it is shaky and is unable to function as designed. — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member (See photo 18-14)						

Photo no.	18-7	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Northwest
Description:	S7B between FB17A & FB17B — Up to 1.5" high perforation on lower web along the bottom flange for 90% of the span length. Other portions of the web are paper thin. The top flange has a broken edge up to ¼". The bottom flange exhibits up to 15% section loss—it is shaky and is unable to function as designed. — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member (See photo 18-14\)						

BRIDGE INSPECTION PHOTOS

Photo no.	18-8	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	Southeast
Description:	FB17B above G3 — Perforation on the bottom 1/3 of the web along the web stiffener above its supporting member G3. The remaining portion of the web also exhibits extreme corrosion with up to 2" diameter holes and a few pinholes. — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member. (See photo 18-14)						

Photo no.	18-9	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	North
Description:	S3C between FB17B & FB18 — Completely sheared off web at the connection to FB17B and the stringer is side swinging approximately 1/8". — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member. (See photo 18-14)						

BRIDGE INSPECTION PHOTOS

Photo no.	18-10	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	East
Description:	S3C between FB17B & FB18 — Completely sheared off web at the connection to FB17B and the stringer is side swinging approximately 1/8". — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member. (See photo 18-14)						

Photo no.	18-11	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	South
Description:	S5C & S6C between FB17B & FB18 — Extreme corrosion with up to 2" high perforation on lower web along the bottom flange in various locations. — Action taken: Area has been barricaded off to prevent any pedestrian loading on these members. (See photo 18-14)						

BRIDGE INSPECTION PHOTOS

Photo no.	18-12	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	West
Description:	S7C between FB17B & FB18 — Completely sheared off web at the connection to FB17B, and, the stringer is sagging approximately 1/8". Remaining web exhibits extreme corrosion with up to 2" high perforation along the bottom flange in various locations. — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member. (See photo 18-14)						

Photo no.	18-13	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	South
Description:	S7C between FB17B & FB18 — Completely sheared off web at the connection to FB17B, and, the stringer is sagging approximately 1/8". Remaining web exhibits extreme corrosion with up to 2" high perforation along the bottom flange in various locations. — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member. (See photo 18-14)						

BRIDGE INSPECTION PHOTOS

Photo no.	18-14	**BIN:**	N/A	**Type:**	Deck	**Looking:**	West
Description:	Top of span 18 toward span 17 — Barricaded off the severely deteriorated framing area from top.						

BRIDGE INSPECTION PHOTOS

Photo no.	19-1	**BIN:**	N/A	**Type:**	Deck	**Looking:**	East
Description:	Underdeck framing — Concrete underdeck exhibits minor deterioration in isolated and random area. Steel framing shows light rust with peeling paint in general.						

Photo no.	19-2	**BIN:**	N/A	**Type:**	Deck	**Looking:**	West
Description:	Underdeck at bay 8 at 2' ahead of FB18 — Square edge concrete haunch exhibits a 4"L x 1"Dp spall along top flange of S8. Also, adjacent underdeck shows fine map cracking with leaching for 3'L x full bay.						

BRIDGE INSPECTION PHOTOS

Photo no.	20-1	**BIN:**	N/A	**Type:**	Deck	**Looking:**	N/A
Description:	Underdeck framing between S11 & S18 — Concrete underdeck exhibits map cracking with leaching and rusted rebar chairs in general. Steel framing shows spot rust randomly and mostly on top flanges.						

Photo no.	20-2	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	West
Description:	Column 20G — Column flange exhibits up to 20% thickness loss locally. Also, column web shows up to 30% section loss with few pinholes.						

BRIDGE INSPECTION PHOTOS

Photo no.	20-3	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	West
Description:	Column 20H — A 1/2"H x 1"L thru hole on column web near the base, and, adjacent area shows up to 40% thickness loss.						

Photo no.	21-1	**BIN:**	N/A	**Type:**	Deck	**Looking:**	East
Description:	Underdeck framing between S4 & S5 — Concrete underdeck exhibits wet stain, map cracking with leaching and rusted rebar chairs. Steel framing shows spot rust randomly and mostly on top flanges.						

BRIDGE INSPECTION PHOTOS

Photo no.	21-2	**BIN:**	N/A	**Type:**	Deck	**Looking:**	N/A
Description:	Underdeck framing between S11 & S12 — 3.5'L x 1'W x 1"Dp spall with exposed rebar along top flange of S11. Adjacent deck area shows rusted rebar chairs. Steel framing shows light rust mostly on top flanges.						

Photo no.	21-3	**BIN:**	N/A	**Type:**	Deck	**Looking:**	South
Description:	Square edge concrete haunch along FB214 & S15 over column 21I — Loose concrete has been removed by inspector.						

BRIDGE INSPECTION PHOTOS

Photo no.	22-1	**BIN:**	N/A	**Type:**	Substructure	**Looking:**	East
Description:	Column 22G — Up to 40% thickness loss locally on both web and flanges.						

Photo no.	22-2	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	South
Description:	FB223 connecting to column 22G — 1 out of 9 connection bolt at bottom has been sheared off.						

BRIDGE INSPECTION PHOTOS

Photo no.	22-3	**BIN:**	N/A	**Type:**	Deck	**Looking:**	N/A
Description:	Underdeck framing between S16 & S19 near FB214 — Concrete underdeck exhibits wet stain, map cracking with leaching, hollowness and rusted rebar chairs crossed entire bays. Steel framing shows spot rust randomly.						

Photo no.	27-8	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	North
Description:	S25 between columns 26G & 27G near 1st interior diaphragm — The 46'-0" long stringer exhibits 60% loss of the web for most of the span and increase to 80% loss at approximately one-third of the span due to 2-4"x4" perforations located near a diaphragm connection. — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member.						

BRIDGE INSPECTION PHOTOS

Photo no.	23-1	**BIN:**	N/A	**Type:**	Pier Column	**Looking:**	South
Description:	At Span 23, Column G-23, Col. top portion exhibits advanced corrosion in web and flanges. Estimated 46% overall column section loss. RED FLAG P-004.						

Photo no.	23-2	**BIN:**	N/A	**Type:**	Pier Column	**Looking:**	South
Description:	At Span 23, column G-23, lower Col. exhibits advanced corrosion in web and flanges. Estimated 59% overall column section loss. RED FLAG P-004						

BRIDGE INSPECTION PHOTOS

Photo no.	23-3	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	Northwest
Description:	At Span 23, G1 east face, concrete deck exhibits a 3'L x 3½" W x 5" D spall with exposed rebar.						

Photo no.	23-4	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	Northwest
Description:	At span 23, S19 east face, concrete deck at haunch exhibits a 10' x 7" W spall, loose concrete is resting at the bottom flange of the C channel.						

BRIDGE INSPECTION PHOTOS

Photo no.	23-5	**BIN:**	N/A	**Type:**	Utilities	**Looking:**	North
Description:	Under S16, junction box is missing a cover, insulated wires are exposed. This area is not reachable by the public.						

Photo no.	24-1	**BIN:**	N/A	**Type:**	Wearing Surf.	**Looking:**	West
Description:	Span 24, top of the concrete deck exhibit multiple hairline cracks and concrete patches along the transverse joints and other isolated locations						

BRIDGE INSPECTION PHOTOS

Photo no.	24-2	**BIN:**	N/A	**Type:**	Primary Memb	**Looking:**	West
Description:	General view of the superstructure under spans 23-26						

Photo no.	24-3	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Southeast
Description:	At Span 24, S17 lower web near the connection exhibits 7 ½"L x 2 ½" corrosion hole.						

BRIDGE INSPECTION PHOTOS

Photo no.	24-4	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	South
Description:	At Span 24, between S17 & S18, FB24-5 web exhibits severe corrosion with four thru holes up11" L x 3" H.						

Photo no.	24-5	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	north
Description:	Span 24, FB 24-4 exhibits severe corrosion at flanges and web under the joint.						

BRIDGE INSPECTION PHOTOS

Photo no.	24-6	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	East
Description:	S18, Advanced corrosion with approximate 64% web section loss at connection to Column 24-G RED Flag P-003						

Photo no.	24-7	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Northwest
Description:	Span 24, close-up view of S18 web at connection to Column 23-G Estimated 65% overall web section loss including thru hole RED Flag P-003						

BRIDGE INSPECTION PHOTOS

Photo no.	24-8	**BIN:**	N/A	**Type:**	Scuppers	**Looking:**	East &up
Description:	At span 24, drainage downpipe is disconnected near bent 24 &S25 and water is discharging directly below.						

Photo no.	24-9	**BIN:**	N/A	**Type:**	Paint	**Looking:**	Southwest
Description:	At Span24, Steel members below the expansion joints exhibits large area of no paint.						

BRIDGE INSPECTION PHOTOS

Photo no.	25-1	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	West
Description:	At Span 25, S11 mid-span east face, top flange (TF) exhibits signs of water leakage. TF, web, BF all exhibit surface corrosion. (Typical throughout)						

Photo no.	25-2	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	East
Description:	Advanced corrosion of S25 web and flanges throughout. RED Flag P-002						

BRIDGE INSPECTION PHOTOS

Photo no.	25-3	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	East
Description:	Close-up of corrosion on S25 web and flanges at midspan, approx. section loss is 55%. RED Flag P-002						

Photo no.	25-4	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Southwest
Description:	Close-up view of web at connection to Column 24 G. Estimated 60% overall web section loss. RED Flag P-002						

BRIDGE INSPECTION PHOTOS

Photo no.	26-1	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	Southwest
Description:	At Span 26, between S19 and S20 underside deck exhibits a 6'L x 1½" W x 1" D concrete spall with exposed rebar.						

Photo no.	26-2	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	West
Description:	At span 26, S25 total 100% corrosion of stringer web. RED Flag P-001						

BRIDGE INSPECTION PHOTOS

Photo no.	26-3	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Northwest
Description:	At span 26, S25 total 100% corrosion of stringer web. RED Flag P-001						

Photo no.	26-4	**BIN:**	N/A	**Type:**	Paint	**Looking:**	Southwest
Description:	General condition of the paint under span 28 and 29.						

BRIDGE INSPECTION PHOTOS

Photo no.	27-1	**BIN:**	N/A	**Type:**	Wearing Surf.	**Looking:**	West
Description:	Top of the concrete deck, joint along bent 27, multiple cracked concrete patches.						

Photo no.	27-2	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Northeast
Description:	At span 27, S25 South face exhibits severe corrosion on the lower part of the web with approximately 5 to 10% section loss.						

BRIDGE INSPECTION PHOTOS

Photo no.	27-3	**BIN:**	N/A	**Type:**	Pier Column	**Looking:**	Southeast &Dn
Description:	At Span 27, Column E-27, concrete encasement exhibits several cracks up to ½" wide and 5" long and concrete spalls 4" long x 1 ½" wide.						

Photo no.	27-4	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Southeast
Description:	At Span 27, connection plate at S18 and FB27-5 near the bottom flange exhibits severe corrosion with a 6.5" H x 3.5" W rusted through hole.						

BRIDGE INSPECTION PHOTOS

Photo no.	27-5	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Southwest
Description:	At Span 27, between S23 and S25, FB27-6 North face exhibits severe corrosion on the lower part of the web with 10 to 15% thickness loss. This is a typical condition throughout FB27.						

Photo no.	27-6	**BIN:**	N/A	**Type:**	Joint	**Looking:**	East
Description:	At Span 27, both longitudinal and transverse joints have been repaired.						

BRIDGE INSPECTION PHOTOS

Photo no.	27-7	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Northwest
Description:	At Span 28, the first three bolts on the connection plate at FB27-6 and S25 exhibit up to 90% section loss, Column G-27 East Flange exhibits 5%.						

Photo no.	27-8	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	North
Description:	S25 between columns 26G & 27G near 1st interior diaphragm — The 46'-0" long stringer exhibits 60% loss of the web for most of the span and increase to 80% loss at approximately one-third of the span due to 2-4"x4" perforations located near a diaphragm connection. — Action taken: Area has been barricaded off to prevent any pedestrian loading on this member.						

BRIDGE INSPECTION PHOTOS

Photo no.	27-9	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	N/A
Description:	Underdeck framing at1st sub-bay between S29 & S30 — Concrete underdeck exhibits fine cracks with leaching and shallow spall. Steel framing shows spot rust randomly.						

Photo no.	27-10	**BIN:**	N/A	**Type:**	Superstructure	**Looking:**	West
Description:	S13 at 1st sub-bay — Top flange exhibits up to 20% section loss along the edge, and, bottom flange has 5% to 10% section loss. The lower web shows 15% section loss along bottom flange.						

BRIDGE INSPECTION PHOTOS

Photo no.	28-1	**BIN:**	N/A	**Type:**	Utilities	**Looking:**	down
Description:	Close up view of the broken conduit cover at span 28 above S1 and FB28-1.						

Photo no.	28-2	**BIN:**	N/A	**Type:**	Scuppers	**Looking:**	Northeast
Description:	At span 28, disconnected and severe corroded drainage pipe at Span 28 between S17 to S18, near the first diaphragm.						

BRIDGE INSPECTION PHOTOS

Photo no.	28-3	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	North
Description:	At span 28, diaphragm connection plate at S18 and 2nd diaphragm exhibits impact damage, approximately 7” long broken weld and 3” long transverse crack.						

Photo no.	28-4	**BIN:**	N/A	**Type:**	Lighting	**Looking:**	Northeast
Description:	At span 28, near 2nd diaphragm between S24 & S25, concrete haunch above the underside light fixture exhibits cracks, loose concrete and spalls with exposed rebar up to 2’ L x 8” W.						

BRIDGE INSPECTION PHOTOS

Photo no.	28-5	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	North
Description:	At Span 28, top flange of s13 near column D-28 exhibits severe corrosion with 10 -15% section loss. Concrete spalls are typical at these locations.						

Photo no.	28-6	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	South
Description:	At span 28, S18 exhibits impact damage, bent outward by approximately 6.25" from its centerline.						

BRIDGE INSPECTION PHOTOS

Photo no.	28-7	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Southwest
Description:	At Span 28, S25 midspan exhibits severe corrosion. Top flange exhibits 10-12% section loss, web exhibits 10-25% and bottom flange exhibits 24% section loss.						

Photo no.	28-8	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Southwest
Description:	At span 28, near 2nd diaphragm, S18 exhibits impact damage with up to 5’-4” W x 5” high.						

BRIDGE INSPECTION PHOTOS

Photo no.	28-9	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Northwest
Description:	At Span 28 mid span, S13 exhibits severe corrosion on the lower part of the web with approximately 5 to 10% thickness loss.						

Photo no.	29-1	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	South
Description:	At span 29, concrete haunch above S13 & FB29 exhibits spalls 9'-4" long.						

BRIDGE INSPECTION PHOTOS

Photo no.	29-2	**BIN:**	N/A	**Type:**	Utilities	**Looking:**	Northeast
Description:	Close up view of utility box at span 29 between S11 and S12 with missing cover and exposed wires,						

Photo no.	29-3	**BIN:**	N/A	**Type:**	Primary Memb.	**Looking:**	Northwest
Description:	General view of primary members from Spans 27 to 29.						

BRIDGE INSPECTION PHOTOS

Photo no.	29-4	**BIN:**	N/A	**Type:**	Deck Structural	**Looking:**	South & Up
Description:	At Span 29 mid span, the main deck between S20 and S21 exhibit 1' W x 5'L concrete spall with exposed rebar.						

Photo no.		**BIN:**		**Type:**		**Looking:**	
Description:							

Appendix B: Flag Report

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

Passerelle Pedestrian Bridge
Contract : HBPE6700Q

Flag Report #	Repair Priority	Flag	Location	Description
13-42B067-2 Report by LIRR	1	Red	Column G24	Pier column 5C3 web, near the base, exhibits severe corrosion and section loss with a thru hole up to 9" wide x 2" high. The web exhibits up to 80% section loss surrounding thru hole. The column flanges adjacent to web hole exhibits up to 75% section loss. Total section loss at cross section where the hole is located is approximately 78%. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 1 on key plan. Include this in emergency repair contract.**
13-42B067-3 Report by LIRR	1	Red	Columns F24	Pier column 5C4 at the base exhibits severe corrosion and section loss with corrosion holes 4" wide x 2" high and 4" wide x 2.5" high on the web. The web exhibits up to 70% section loss surrounding thru holes. The column flanges adjacent to web holes exhibits up to 50% section loss. Total section loss at cross section where the holes are located is approximately 60%. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 1 on key plan. Include this in emergency repair contract.**
P-001	1	Red	Span 26 - Stringer along Column Line G	The 10'-0" long stringer exhibits 100% loss of the web for most of its length. This beam is currently carrying little, if any, load. There is no structural distress observed in the deck or surrounding structure. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 1 on key plan. Include this in emergency repair contract.**
P-002	1	Red	Span 25 - Stringer along Column Line G	The 37'-0" long stringer exhibits approximately 80% web section loss and 60% bottom flange section loss at mid-span and approximately 60% web section loss at the ends. This beam is currently carrying little, if any, load. There is no structural distress observed in the deck or surrounding structure. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 1 on key plan. Include this in emergency repair contract.**
P-003	1	Red	Span 24 - Stringer along Column Line G	The 32'-2" long stringer exhibits approximately 65% web section loss at its end connections and severe delaminated corrosion at mid-span, which could not inspected hands-on due to the presence of LIRR power lines near mid-span. Based on the severely corroded condition of stringers along this column line G in spans 25 and 26, we expect similar advanced corrosion in the web and flanges at midspan. There is no structural distress observed in the deck or surrounding structure. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 1 on key plan. Include this in emergency repair contract.**
P-004	1	Red	Column G23	Column exhibits severe corrosion with nearly 50% section loss at its top and approximately 60% section loss at its bottom. There is no indication of buckling or structural distress observed at the time of the inspection. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 1 on key plan. Include this in emergency repair contract.**
P-005	3	Yellow	Span 28 - Stringer S18	Severe impact damage was observed at Stringer 18 bottom flange, web, connecting diaphragms, and diaphragm connection plates. The length of the impact damage is 5'-4" long. Bottom flange is bent upward by 5", lower web bent westward by 6-1/4" from the center line, and west diaphragm is twisted. In addition, the diaphragm connection plate at east face of Stringer 18 exhibits a 7" long broken weld and a 3-1/2" separation from the stringer web due to the impact. No structural crack in the stringer was observed.
P-006	1	Red	Span 18 - Stringer S3C	Stringer S3C has completely sheared off web at the connection to floor beam 17B and the stringer is side swinging approximately 1/8". **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 3 on key plan. Include this in emergency repair contract.**
P-007	1	Red	Span 18 - Stringer S7C	The redundant stringer S7C exhibits extreme corrosion with Up to 2" high perforation on its web along the bottom flange in various locations. Also, the web of stringer S7C at the connection to floor beam 17B is completely sheared off—the stringer is sagging approximately 1/8". **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 2 on key plan. Include this in emergency repair contract.**
P-008	1	Red	Span 18 - Stringers S5C and S6C	The redundant stringers S5C and S6C exhibit extreme corrosion with Up to 2" high perforation on its web along the bottom flange in various locations. **Action taken: Area has been barricaded off to prevent any pedestrian loading on these members - See location 2 on key plan. Include this in emergency repair contract.**

Flag Report #	Repair Priority	Flag	Location	Description
P-009	1	Red	Span 18 - Stringer S7B	Stringer S7B exhibits up to 1.5” high perforation on its web along the bottom flange for 90% of the span length. Other portions of the web are paper-thin. The top flange has a broken edge up to ¼”. The bottom flange exhibits up to 15% section loss—it is shaky and is unable to function as designed. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 2 on key plan. Include this in emergency repair contract.**
P-010	1	Yellow	Span 18 - Floor Beam 17B	The non-redundant floor beam 17B exhibits perforation on the bottom 1/3 of its web along the web stiffener above its supporting member G3. The remaining portion of the web also exhibits extreme corrosion with up to 2” diameter holes and a few pinholes. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 2 on key plan. Include this in emergency repair contract.**
P-011	1	Safety	Span 12 - Timber Deck between stringers S4A and S5A	The timber deck located between floor beams FB11 and FB11A, at 11 feet South of FB11 and between stringers S4A and S5A exhibits two soft boards. One board has a fine crack crossing the fibers at 1.5 feet from S4A. The board deflects about 1 inch when stepped on. There is a potential of a punch through. The adjacent floor board west of the board described above is soft as well and exhibits ½ inch deflection under loading. **Include this in emergency repair contract.**
P-012	3	Yellow	Span 8 - Floor Beam 7A3 between Girder G3 & G4	The non-redundant floor beam 7A3 exhibits perforation in the web near the connection to girder G3. The surrounding portion of the web near the perforations is paper thin exhibiting extreme corrosion. Total section loss for the member is 40%.
P-013	1	Yellow	Span 27 - Stringer along Columns Line G	The 46’-0” long stringer exhibits 60% loss of the web for most of the span and up to 80% loss at approximately one-third of the span. There are also some through holes located near a diaphragm connection at about one-third of the span. This beam is currently carrying little load. There is no structural distress observed in the deck or surrounding structure. **Action taken: Area has been barricaded off to prevent any pedestrian loading on this member - See location 1 on key plan. Include this in emergency repair contract.**
P-014	1	Yellow	Span 11 - Stringer S6B between Floorbeam FB10A & FB10B	The redundant Stringer S6B exhibits extreme corrosion with large perforation and paper thin in the lower 4” of web along bottom flange for 12’ long starting from 3’ away from Floorbeam FB10A. The remaining web shows up to 10% section loss. Also, the flanges exhibit up to 15% section loss with broken 1/8” of top flange edge through out. The worst total section loss at 6’ away from floorbeam FB10A with 45% web loss and 20% flange loss. **Include this in emergency repair contract.**
P-015	2	Yellow	Span 12 - Stringer S6B between Floorbeam FB11A & FB11B	The redundant Stringer S6B exhibits extreme corrosion with large perforation and paper thin in the lower 3” of web along bottom flange through out. Remaining of web shows up to 10% section loss. Also, the top flange exhibits up to 50% section loss with broken 1” edge through out. The bottom flange has less than 5% section loss. The worst total section loss is at 2’ away from Floorbeam FB11B with 35% web loss and 30% flange loss. **Include this in emergency repair contract.**
P-016	3	Yellow	Span 13 - Stringer S7C between Floorbeam FB12B & FB13	The redundant Stringer S7C exhibits extreme corrosion with large perforation and paper thin in the lower 4” of web along bottom flange for 4’ long starting from 5’ away from Floorbeam FB13. Remaining web shows up to 20% section loss. Also, the top flange exhibits a range of 5% to 20% section loss with up to 1/8” broken edge through out. Bottom flange shows up to 5% section loss. The worst total section loss is at 8’ away from Floorbeam FB13 with 50% web loss and 15% flange loss.
P-017	2	Yellow	Span 15 - Stringer S7C between Floorbeam FB14B & FB15	The redundant Stringer S7C exhibits extreme corrosion with large perforation and paper thin in the lower 2.5” of web along bottom flange for 12’ long starting 3’ away from Floorbeam FB15. Remaining web shows up to 20% section loss. Also, the top flange exhibits up to 20% section loss with up to 1/2” broken edge locally. Bottom flange shows up to 5% section loss. The worst total section loss is at 3’ away from Floorbeam FB15 with 30% web loss and 20% flange loss. **Include this in emergency repair contract.**
P-018	3	Yellow	Span 16 - Stringer S5A between Floorbeams FB15 & FB15A	The redundant Stringer S5A exhibits extreme corrosion with perforations in size of 2’L x 2”H & 6”H x 2”H, and, paper thin in the lower 2.5” of web along bottom flange for most of the member’s length. Remaining web shows up to 5% section loss. Also, the flanges exhibit up to 15% section loss with up to 1/2” broken top flange at mid-span. The worst section loss is at mid-span with 30% web loss and 20% flange loss.
P-019	2	Yellow	Span 16 - Stringer S6A between Floorbeams FB15 & FB15A	The redundant Stringer S6A exhibits extreme corrosion with large perforation and paper thin in the lower 4” of web along bottom flange for 4’ long near the center of the span. Another corroded hole of 1.5” diameter is noted on the web below the connection to FB15A. Also, both flanges exhibit 15% section loss through out. The worst cross section loss is at mid-span with 40% web loss and 15% flange loss. **Include this in emergency repair contract.**

Flag Report #	Repair Priority	Flag	Location	Description
P-020	1	Yellow	Span 16 - Stringers S5B and S6B between Floorbeams FB15A & FB15B.	Both redundant Stringers S5B & S6B exhibits extreme corrosion with local perforation and paper thin in the lower 2" of web along bottom flange for upto 3' long starting 1.0' away from either ends. Remaining of web shows average of 5% section loss. Also, the top flanges exhibits average of 20% section loss. Bottom flange shows upto 5% section loss. The worst total section loss is at 1.5' away from Floorbeam FB15A with 25% web loss and 12% flange loss. **Include this in emergency repair contract.**
P-021	1 and 2	Yellow	Span 17 - Stringers S6A and S7A between Floorbeams FB16 & FB16A.	Both redundant adjacent Stringers S6A & S7A exhibit extreme corrosion near the connection to FB16A. The Stringer S6A exhibits perforation and paper thin in the lower 2" of web along bottom flange for 7' long. Remaining of web shows upto 10% section loss. Also, the top flange exhibits average of 10% section loss with broken 1" edge in 1' long . Bottom flange shows upto 5% section loss. The worst total section loss is at 2.5' away from Floorbeam FB16A with 25% web loss and 10% flange loss. The Stringer S7A also exhibits perforation and paper thin in the lower 1" of web along bottom flange for 4' long, and, the top flange exhibits broken 1/2" edge in 1.5' long. The worst total section is at 3' away from Floorbeam FB16A with 15% web loss and 10% flange loss. **Stringer S6A is priority 1. Stringer S7A is Priority 2. Include this in emergency repair contract.**
P-022	1	Yellow	Span 17 - Stringer S6B between Floorbeam FB16A & FB16B.	The redundant Stringer S6B exhibits extreme corrosion with large perforation and paper thin on the lower web along bottom flange in size of 4"L x 3"H at quarter point from FB16A and 2'L x 4"H at quarter point from FB16B. Remaining of web shows upto 10% section loss. Also, the top flange exhibits upto 15% section loss with upto 1" broken edge randomly throughout. The worst cross section loss is at 4' away from Floorbeam FB16B with 45% web loss and 30% flange loss. **Include this in emergency repair contract.**
P-023	1	Yellow	Span 17 - Stringer S7B between Floorbeam FB16A & FB16B.	The redundant Stringer S7B exhibits extreme corrosion with large perforation and paper thin on the lower 3" web along bottom flange for 10' long starting from FB16A. Remaining of web shows upto 15% section loss. Also, the top flange exhibits upto 25% section loss with upto 1/2" broken edge throughout. The worst cross section loss is at connection to Floorbeam FB16A with 40% web loss and 20% flange loss. **Include this in emergency repair contract.**
P-024	2	Yellow	Span 18 - Stringer S4A between Floorbeam FB17 & FB17A.	The redundant Stringer S4A exhibits extreme corrosion with large perforation and paper thin on the lower 3" web along bottom flange throughout. Remaining of web shows 25% section loss. Also, the top flange exhibits upto 30% section loss with upto 1/4" broken edge throughout. Bottom flange shows 10% section loss as well. The cross section loss has 45% web loss and 25% flange loss through out. **Include this in emergency repair contract.**

NOTE:

1) SEE ATTACHED INDIVIDUAL REPORTS FOR EXACT FLAG LOCATIONS

LEGEND:

P-003 FLAG LOCATION

BARRICADED AREAS (3 LOCATIONS)

KEY PLAN

N.T.S.

FINAL DESIGN PREPARED BY:

HAKS | WEIDLINGER ASSOCIATES INC

Joint Venture

P.E. NO. DATE

NO.	DATE	DESCRIPTIONS	BY	APPR'D
		REVISIONS		

STRUCTURAL AND SAFETY FLAGS LOCATION PLAN

PASSERELLE PEDESTRIAN BRIDGE

BOROUGH OF QUEENS

PROJECT ID:	DATE:	SHEET
HBPED700Q	NOVEMBER, 2013	G1

CAPITAL PROJECT ID: HBPED700Q

CONSULTANT DESIGN

Long Island Rail Road

FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*

YELLOW FLAG*

SAFETY FLAG*

INSPECTOR: Mohammadreza Gramy

FLAG NUMBER: 13-42B067-2

DATE OF INSPECTION: 10/17/2013

PIA Status: No

BRIDGE DESCRIPTION:

NAME: Shea Stadium (Station Overpass) BRIDGE #: 42-B-067 BIN: N/A

BRANCH: Pt. Washington LOCATION: Flushing COUNTY: Queens

DESCRIPTION OF FLAGGED CONDITION:

LOCATION: Pier 5, Column 5C3 (Column G24)

DESCRIPTION: Pier column 5C3 web, near the base, exhibits severe corrosion and section loss with a thru hole up to 9" wide x 2" high. The web exhibits up to 80% section loss surrounding thru hole. The column flanges adjacent to web hole exhibits up to 75% section loss. Total section loss at cross section where the hole is located is approximately 78%.

ACCESS: Walking on platform between tracks, extension ladder, foul time or track outage may be required.

Two (2) photos and two (2) sketches attached for clarification.

*The appropriate flag type in the upper left of this form shall be initialed by the individual who is the signatory

Long Island Rail Road

FLAGGED BRIDGE REPORT
PHOTO RECORD FORM

Bridge #: 42-B-067 **Bridge Name:** Shea Stadium (Station Overpass) **Sheet** 2 **of** 2

Inspector: Mohammadreza Gramy **Branch:** Pt. Washington **Date:** 10/17/2013

Photo No.: 1

Location: Pier 5, Column 5C3, looking ahead.

Description: General view of column. Column exhibits severe corrosion and section loss to web and flanges near the base.

Reference: 13-42B067-2 RED FLAG

Photo No.: 2

Location: Pier 5, Column 5C3 near the base, looking ahead.

Description: Corrosion hole on column web at base up to 9" wide x 2" high. Web exhibits up to 80% section loss surrounding thru hole. The column flanges adjacent to hole exhibits up to 75% section loss.

Reference: 13-42B067-2 RED FLAG

ASSET 42-B-067 **2013-2014 Comprehensive Inspection Program**

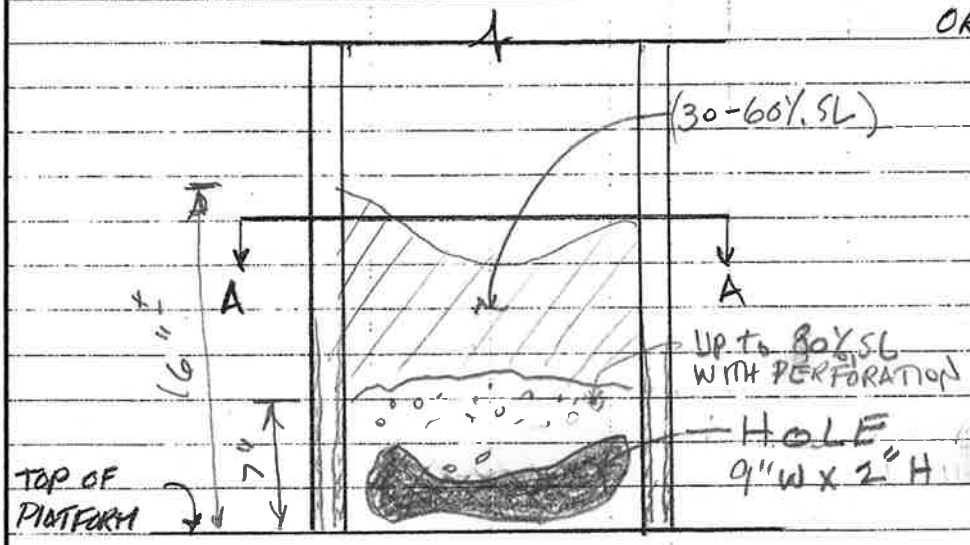
DATE 10/17/13 SHEET 1 OF 1

TEAM LEADER M. Gramy ASSISTANT TEAM LEADER & TRAINEE M. Cahill

FEATURE CARRIED: SHEA STADIUM (STATION OVERPASS)
FEATURE CROSSED: PT. WASHINGTON

ORIGINAL SIZE 14WF87

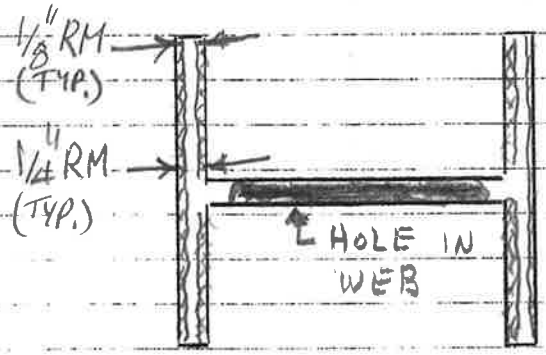
$A = 25.57\ in^2$
$d = 14''$
$t_w = 0.42''$
$b_f = 14.5''$
$t_f = 0.688''$



ELEVATION

FLANGE (RM) $= \frac{1/8'' + 1/4''}{2} \times 14.5'' \times 2$
$= 5.44\ in^2$

WEB (RM) $= (14'' - 2 \times 0.688'' - 9'') \times 0.42'' \times (1 - 0.80)$
$= 0.30\ in^2$



SECTION A-A

FLANGES (RM) = $5.44\ in^2$
WEB (RM) = $0.30\ in^2$

TOTAL (RM) SECTION $5.74\ in^2$

TOTAL (SL) = $19.83\ in^2$

TOTAL SECTION LOSS $= \frac{19.83}{25.57} =$
$= 77.55\%$

PIER #5, Col. #3
(LOOKING AHEAD)

Long Island Rail Road

BRIDGE INSPECTION REPORT

SKETCH

Bridge No.: 42-B-067

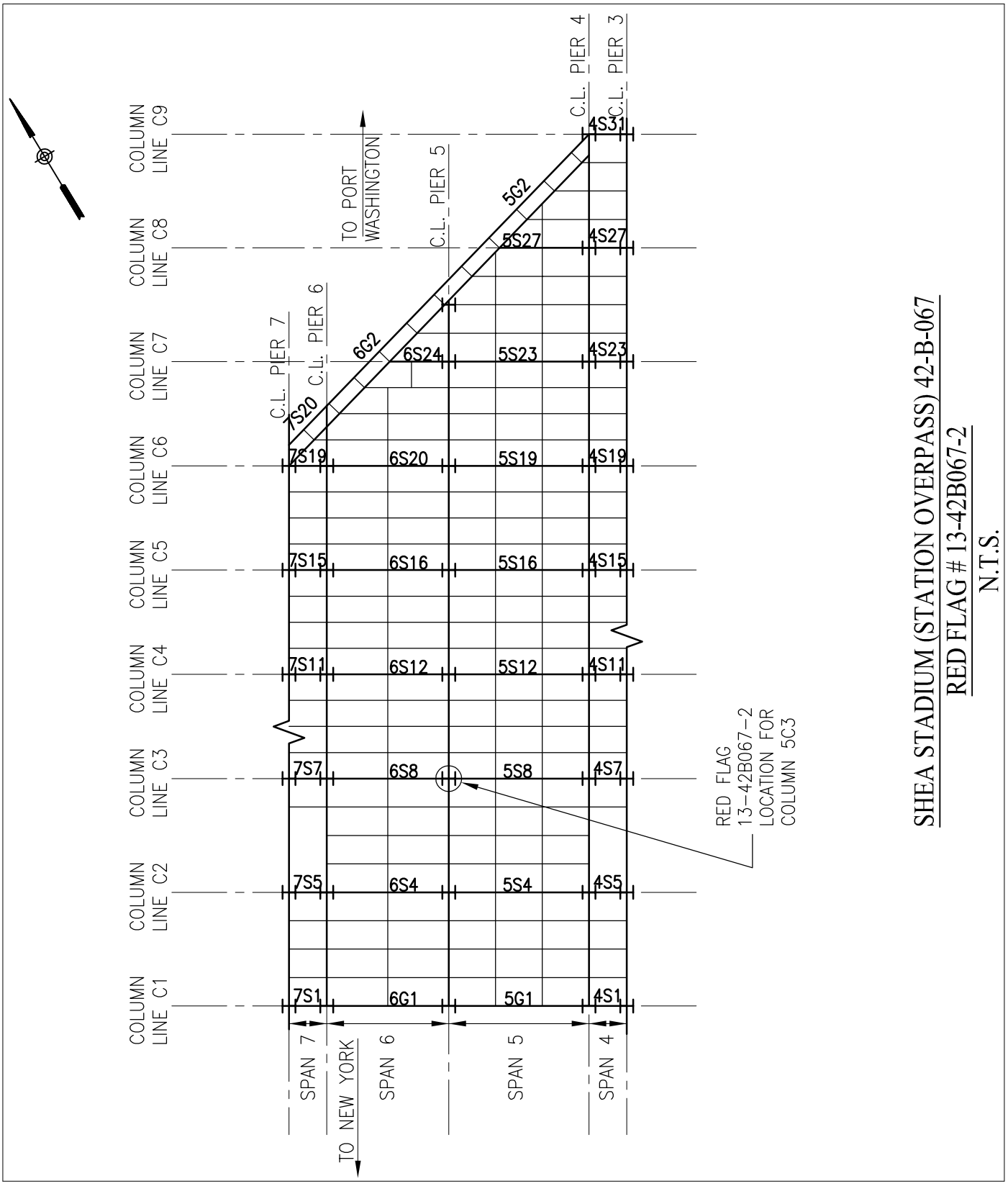
Bridge Name: SHEA STADIUM (STATION OVERPASS)

Sheet 1 of 1

Branch: PORT WASHINGTON

Inspector: M. GRAMY, PE

Date: 10/18/13



SHEA STADIUM (STATION OVERPASS) 42-B-067

RED FLAG # 13-42B067-2

N.T.S.

Long Island Rail Road

FLAGGED BRIDGE REPORT

INITIAL:

INSPECTOR: Mohammadreza Gramy

FLAG NUMBER: 13-42B067-3

X RED FLAG*

DATE OF INSPECTION: 10/17/2013

YELLOW FLAG*

PIA Status: No

SAFETY FLAG*

BRIDGE DESCRIPTION:

NAME: Shea Stadium (Station Overpass) BRIDGE #: 42-B-067 BIN: N/A

BRANCH: Pt. Washington LOCATION: Flushing COUNTY: Queens

DESCRIPTION OF FLAGGED CONDITION:

LOCATION: Pier 5, Column 5C4. (Column F24)

DESCRIPTION: Pier column 5C4 at the base exhibits severe corrosion and section loss with corrosion holes 4" wide x 2" high and 4" wide x 2.5" high on the web. The web exhibits up to 70% section loss surrounding thru holes. The column flanges adjacent to web holes exhibits up to 50% section loss. Total section loss at cross section where the holes are located is approximately 60%.

ACCESS: Walking on platform between tracks, extension ladder, foul time or track outage may be required.

Two (2) photos and two (2) sketches attached for clarification.

*The appropriate flag type in the upper left of this form shall be initialed by the individual who is the signatory

Long Island Rail Road

FLAGGED BRIDGE REPORT
PHOTO RECORD FORM

Bridge #: 42-B-067 **Bridge Name:** Shea Stadium (Station Overpass) **Sheet** 2 **of** 2

Inspector: Mohammadreza Gramy **Branch:** Pt. Washington **Date:** 10/17/2013

Photo No.: 1

Location: Pier 5, Column 5C4, looking ahead.

Description: General view of column. Sevre corrosion and section loss to web and flanges near the base.

Reference: 13-42B067-3 RED FLAG

Photo No.: 2

Location: Pier 5, Column 5C4 near the base, looking ahead.

Description: Web of column at base exhibits severe corrosion with holes up to 4" wide x 2" high and 4" wide x 2.5" high. Web exhibits up to 70% section loss surrounding thru holes. The column flanges adjacent to holes exhibits up to 50% section loss.

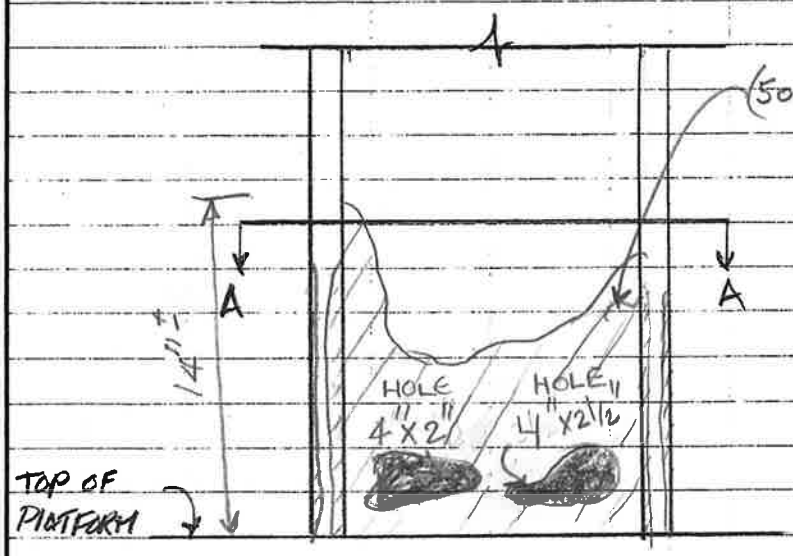
Reference: 13-42B067-3 RED FLAG

ASSET 42-B-067 **2013-2014 Comprehensive Inspection Program**

DATE 10/17/13 SHEET 1 OF 1

TEAM LEADER M. Gramy ASSISTANT TEAM LEADER & TRAINEE M. Cahill

FEATURE CARRIED: SHEA STADIUM (STATION OVERPASS)
FEATURE CROSSED: PT. WASHINGTON



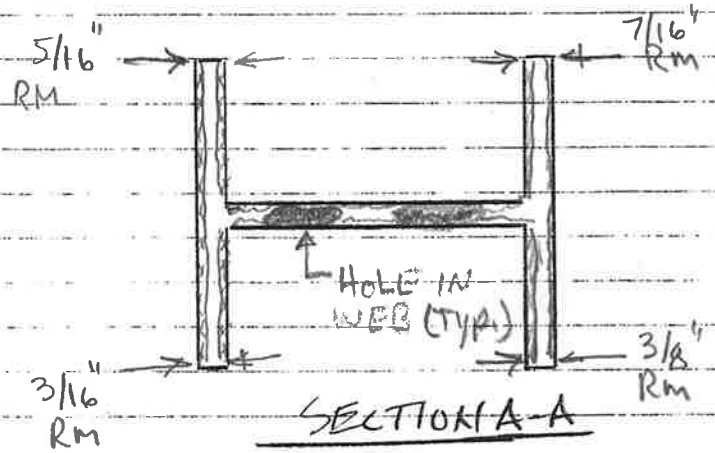
ELEVATION

ORIGINAL SIZE 14WF87

$A = 25.57 \text{ in}^2$
$d = 14''$
$t_w = 0.42''$
$b_f = 14.5''$
$t_f = 0.688''$

FLANGE(RM) $= \frac{5/16'' + 7/16'' + 3/16'' + 3/8''}{4} \times 14.5'' \times 2$
$= 9.51 \text{ in}^2$

WEB(RM) $= (14'' - 2 \times 0.688 - 4'' - 4'') \times 0.42 \times (1 - 0.70)$
$= 0.58 \text{ in}^2$



SECTION A-A

FLANGES (RM) $= 9.51 \text{ in}^2$
WEB (RM) $= 0.58 \text{ in}^2$

TOTAL SECT. (RM) 10.09 in^2

TOTAL (S.L) $= 15.47 \text{ in}^2$

TOTAL SECTION LOSS $= \frac{15.47}{25.57}$
$= 60.52\%$

PIER #5, COL. #4
(LOOKING AHEAD)

Long Island Rail Road

BRIDGE INSPECTION REPORT

SKETCH

Bridge No.: 42-B-067

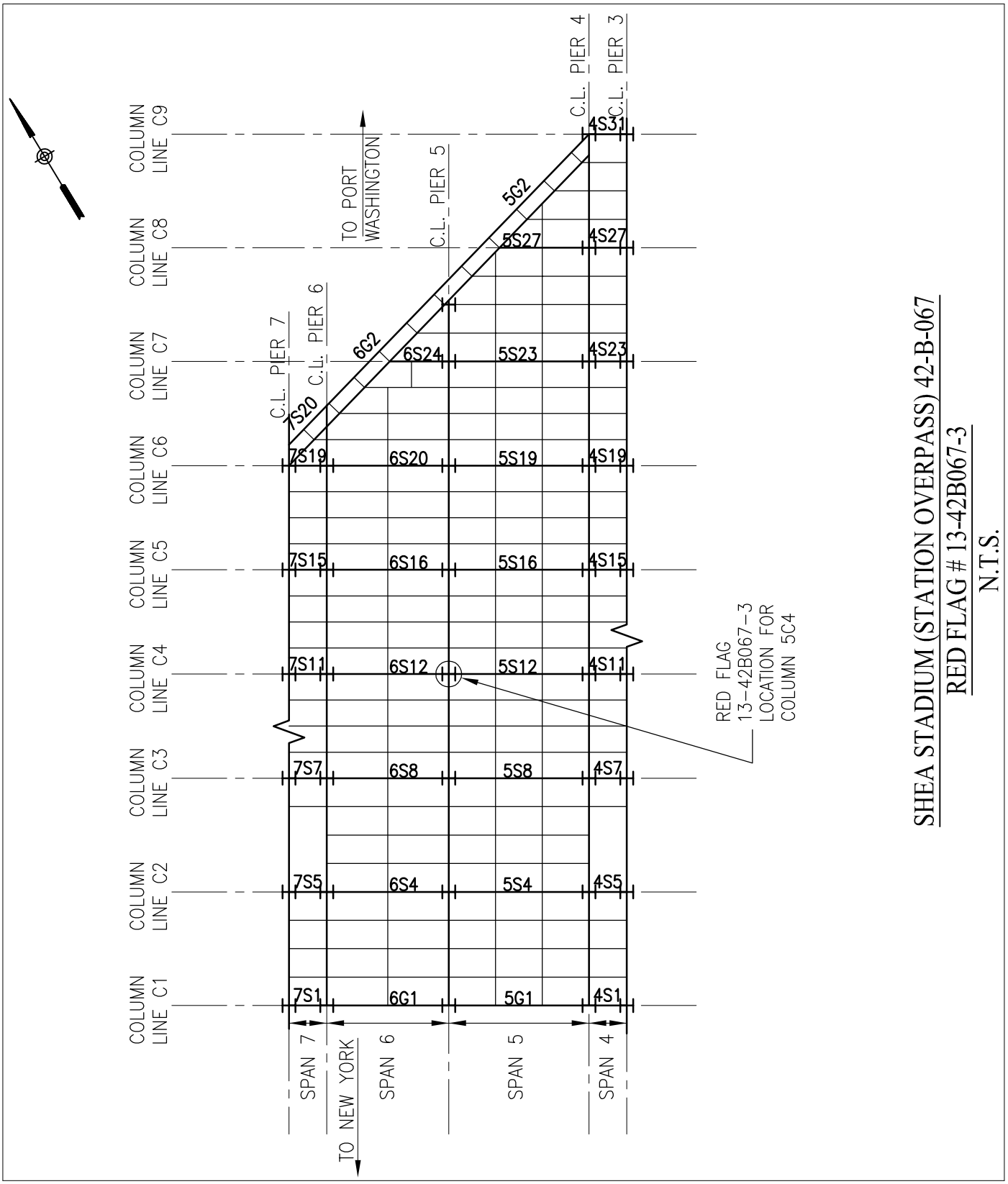
Bridge Name: SHEA STADIUM (STATION OVERPASS)

Sheet 1 of 1

Branch: PORT WASHINGTON

Inspector: M. GRAMY, PE

Date: 10/18/13



SHEA STADIUM (STATION OVERPASS) 42-B-067

RED FLAG # 13-42B067-3

N.T.S.

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*
_____ YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Jie Sun, P.E.*
FLAG NUMBER *P-001*
DATE OF INSPECTION *09 / 16 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *LIRR*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *26* COMPONENT: *N/A*

RATING: *1*

Location: Stringer along Column Line G at Span 26

Member: Span Item [28] Primary members (Stringer)

Description: The 10'-0" long stringer exhibits 100% loss of the web for most of its length. This beam is currently carrying little, if any, load. There is no structural distress observed in the deck or surrounding structure.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *4*

Flagged Bridge Report Completed By *Jie Sun , P.E.* D *10 / 1 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

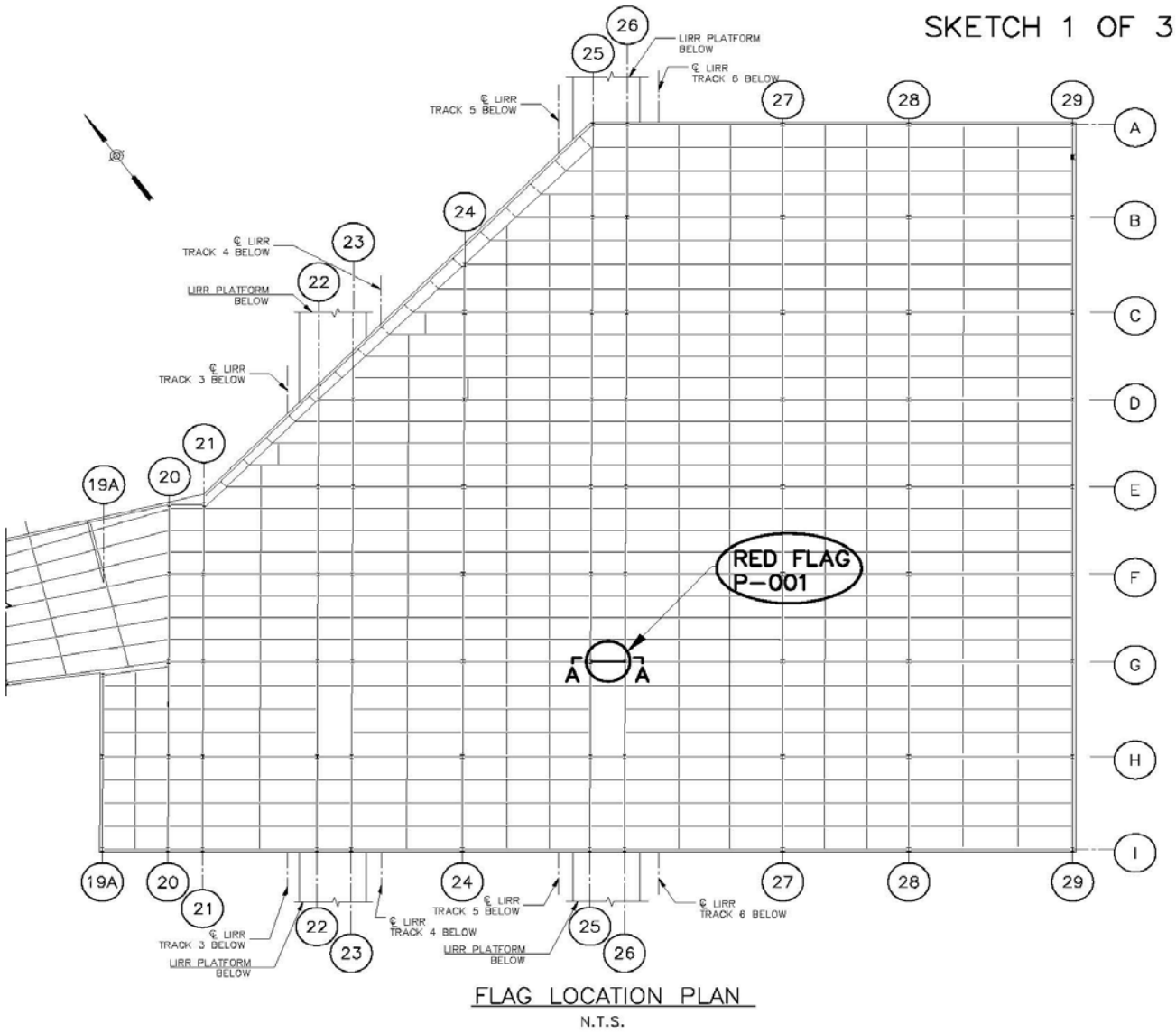
BD 186(4/95)

BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 2 **OF** 6

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR



FLAG LOCATION PLAN
N.T.S.

BD 186(4/95)

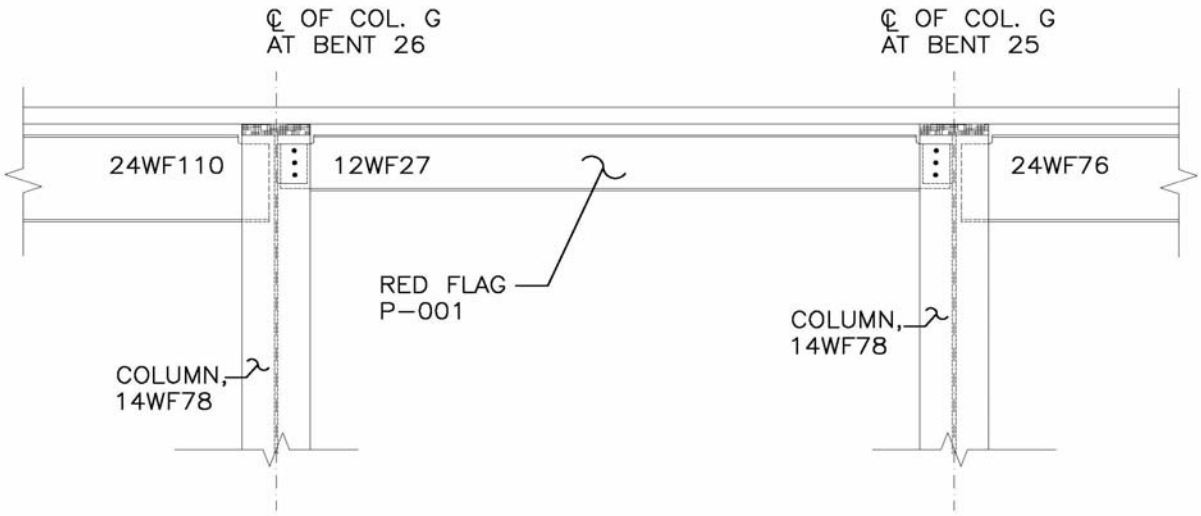
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 3 OF 6

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 2 OF 3



SECTION A-A, SPAN 26 STRINGER ALONG COL. LINE G, LOOKING WEST
N.T.S.

BD 186(4/95)

BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 4 **OF** 6

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 3 OF 3

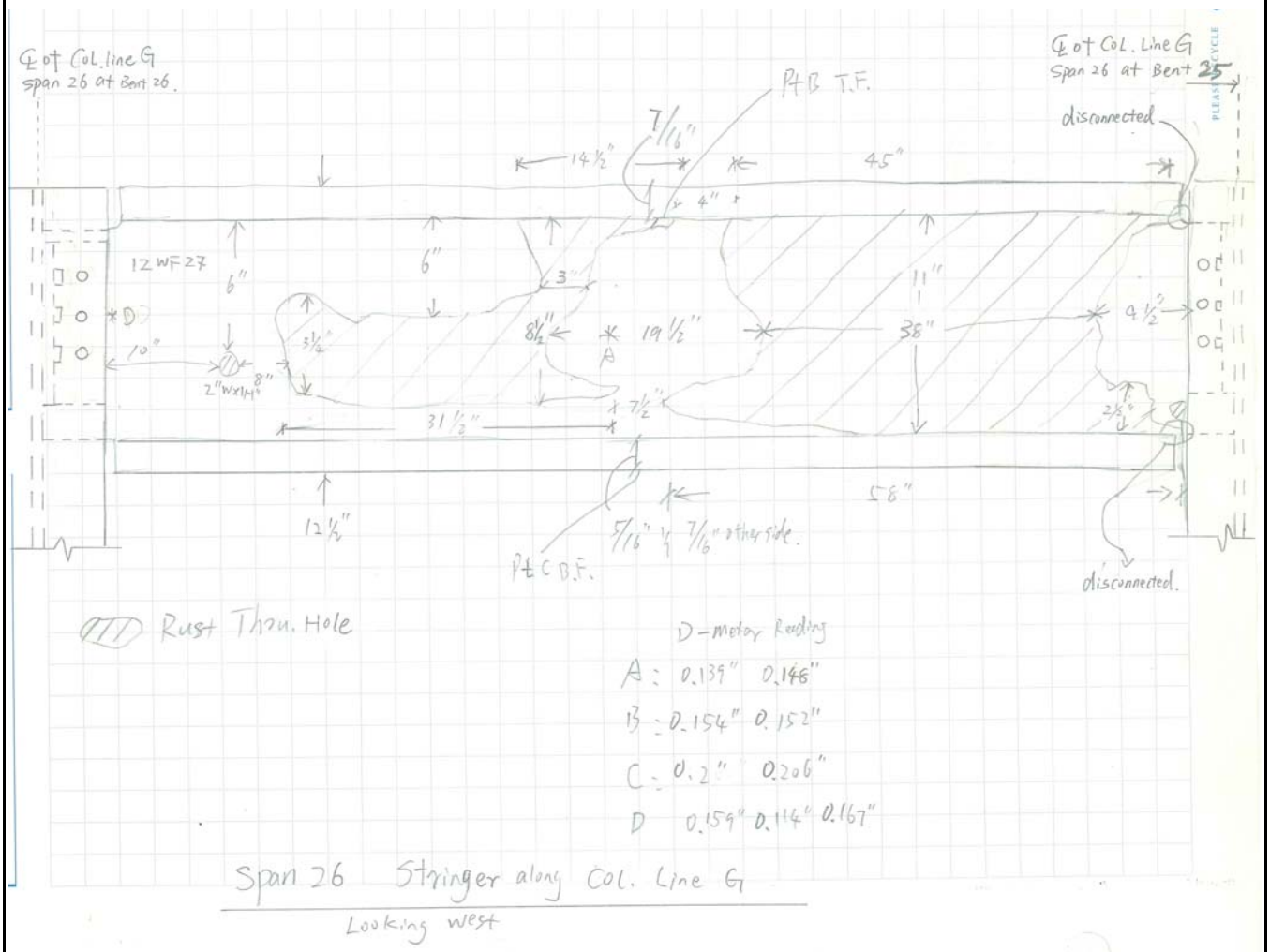
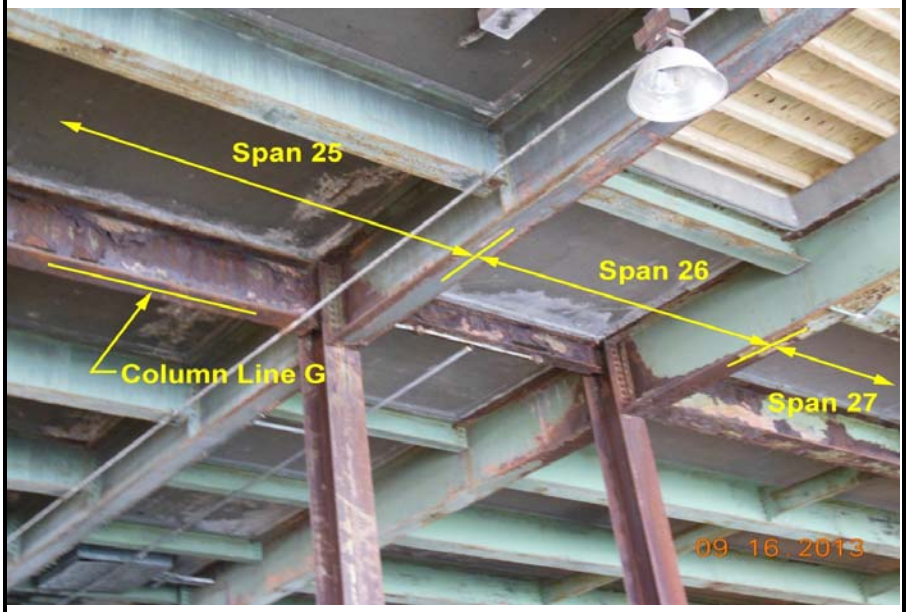


PHOTO NO. **P001-P1**

Location:
General view of stringers along Col. Line G in spans 25, 26, and 27
Looking Southeast

Description:
Advanced corrosion of stringer in Span 26 (shown before rust removal along web)

Reference:
RED FLAG P-001



ASST. TEAM LEADER: T.Ko

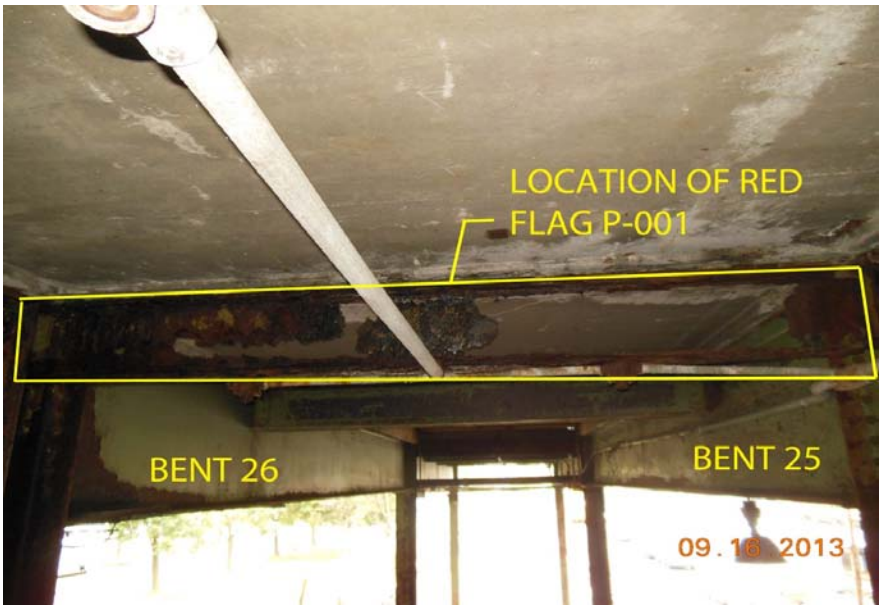
FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P001-P2**

Location:
Stringer along Col. Line G
Span 26
Looking West

Description:
Total 100% corrosion of stringer web (shown after rust removal)

Reference:
RED FLAG P-001



TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P001-P3**

Location:
Stringer along Col. Line G
Span 26
Looking Northwest

Description:
Total 100% corrosion of
stringer web

Reference:
RED FLAG P-001

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P001-04**

Location:
Stringer along Col. Line G
Span 26
Looking Southwest

Description:
Total 100% corrosion of
stringer web

Reference:
RED FLAG P-001

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*
____ YELLOW FLAG*
____ SAFETY FLAG*

INSPECTOR *Jie Sun, P.E.*
FLAG NUMBER *P-002*
DATE OF INSPECTION *09 / 16 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED ____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *LIRR*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO ____ YES ____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *25* COMPONENT: *N/A*

RATING: *1*

Location: Stringer along Column Line G at Span 25

Member: Span Item [28] Primary members (Stringer)

Description: The 37'-0" long stringer exhibits approximately 80% web section loss and 60% bottom flange section loss at mid-span and approximately 60% web section loss at the ends. This beam is currently carrying little, if any, load. There is no structural distress observed in the deck or surrounding structure.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *5*

Flagged Bridge Report Completed By *Jie Sun , P.E.* D *10 / 1 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To ____ of Regional Office on ____ at ____ O'clock

To ____ (Responsible Party) on ____ at ____ O'clock

By ____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BD 186(4/95)

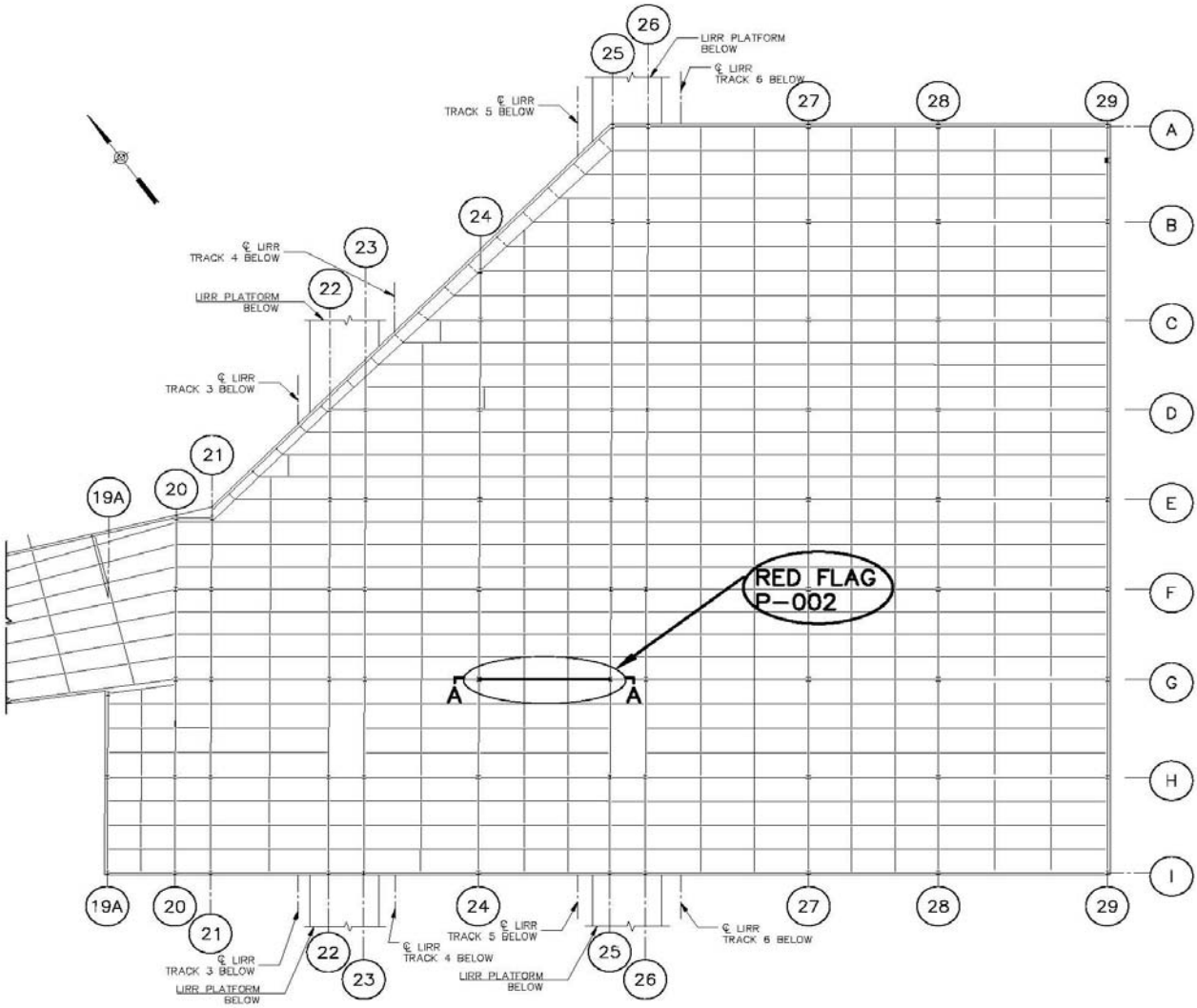
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 2 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 1 OF 4



FLAG LOCATION PLAN
N.T.S.

BD 186(4/95)

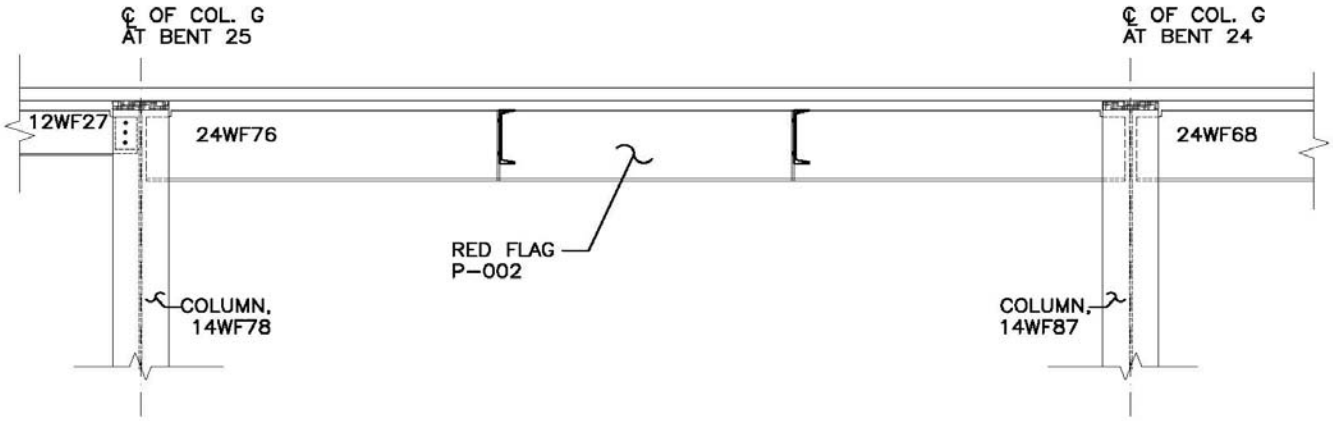
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 3 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 2 OF 3



SECTION A–A, SPAN 25 STRINGER ALONG COL. LINE G, LOOKING WEST
N.T.S.

BD 186(4/95)

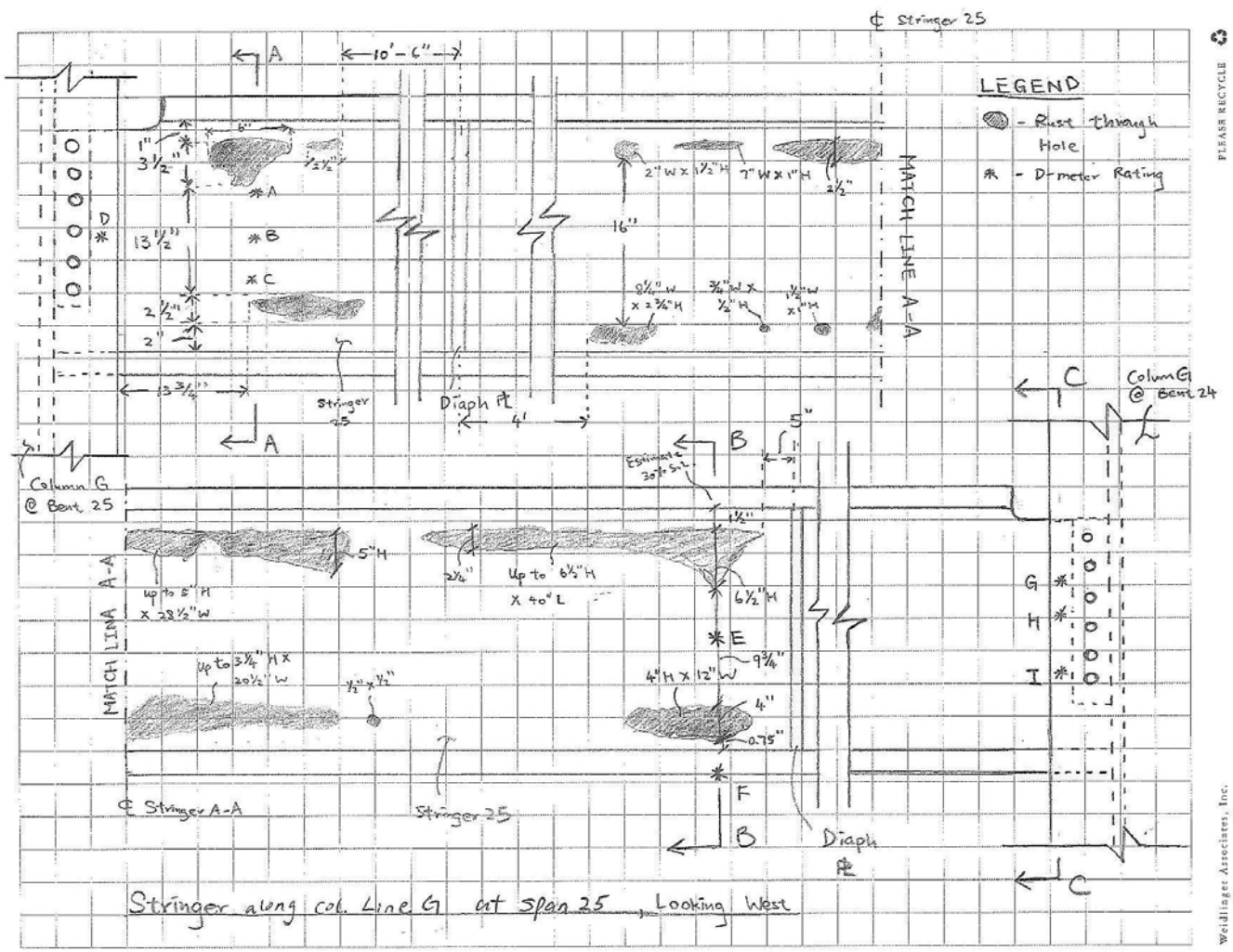
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 4 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 3 OF 4



BD 186(4/95)

BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 5 OF 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 4 OF 4

D-Meter Reading

Section 24WF76 :
As = 22.37 in² b = 8.985"
d = 23.91" tf = 0.682"
tw = 0.44" dw = 22.546"

	Location	Trial 1st	2nd	3rd	4th	Avg	S.L.
Section A-A	A	.224"	.258"				$1-\frac{.217}{.44}$
	B	.2"	.159"	.264"		.217"	= 51%
	C	.267"	.244"				
	D	.22"	.161"	.188"	.257"		
Section B-B	E - Web	.125"	.163"	.152"		.147"	$1-.147/.44 = 67\%$
	F - Bott. Fl.	.255"	.176"	Caliper 7/16"		.29"	$1-.29/.682 = 57\%$
Section C-C	G	.157"	.131"	.2"			$1-\frac{.1795}{.44}$
	H	.158"	.217"			.1795"	= 59%
	I	.214"					

* Average include all the values along the same section cut for the Web, assume the remaining web has the same avg. thickness.

- Web Section Loss @ A-A (South End) $= \frac{(.51)(1+13\frac{1}{2}+2)+(1)(3\frac{1}{2}+2\frac{1}{2})}{23.91} = \boxed{60\%}$

- Web Section Loss @ C-C (North End) $= \frac{(.59)(22.5)}{23.91} = 54.5$ say $\boxed{55\%}$

- Section B-B:

Web S.L. $= \frac{(.67)(1\frac{1}{2}+9\frac{3}{4}+\frac{3}{4})+(1)(6\frac{1}{2}+4)}{23.91} = 77.5\%$ say $\boxed{78\%}$

Top Flange S.L. = $\boxed{30\%}$ (Estimate)

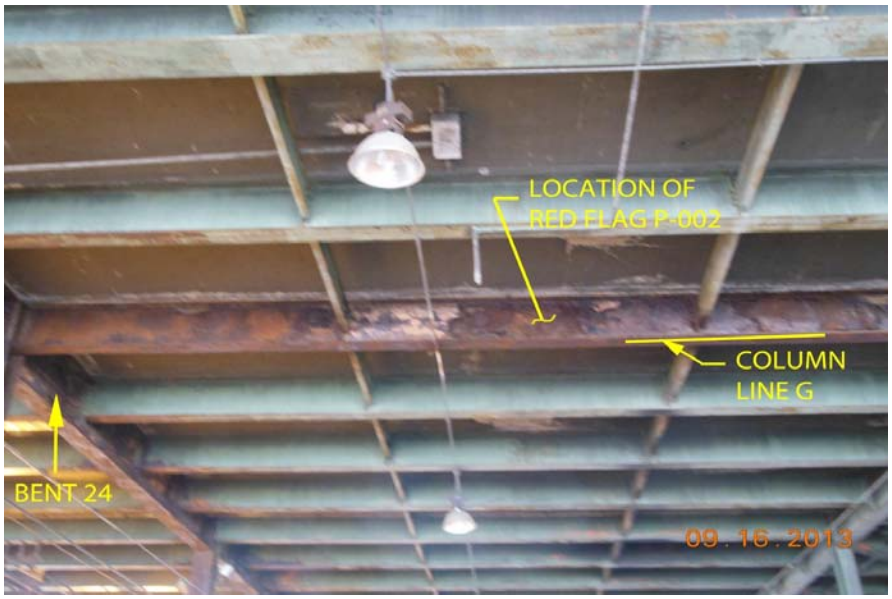
Bottom Flange S.L. = $\boxed{57\%}$ (By D-meter & Caliper)

PHOTO NO. **P002-P1**

Location:
Stringer along Col. Line G
Span 25
Looking East

Description:
Advanced corrosion of stringer web and flanges throughout span (shown before rust removal along web)

Reference:
Red Flag P-002



ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P002-P2**

Location:
Stringer along Col. Line G
Span 25 at midspan
Looking East

Description:
Advanced corrosion of stringer web and flanges at midspan (shown after rust removal). Estimated bottom flange section loss of 60%, and web section loss of 78%

Reference:
Red Flag P-002

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P002-P3**

Location:
Stringer along Col. Line G
Span 25 at midspan
Looking East

Description:
Close-up of corrosion on
stringer web and flanges
at midspan.

Reference:
Red Flag P-002

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P002-P4**

Location:
Stringer along Col. Line G
Span 25 at Bent 25
Looking West

Description:
Close-up view of web
at connection to Column
24 G. Estimated 60% overall
web section loss

Reference:
Red Flag P-002

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P002-P5**

Location:
Stringer along Col. Line G
Span 25 at Bent 24
Looking Northwest

Description:
Close-up view of web
at connection to Column
25-G. Estimated 55% overall
web section loss

Reference:
Red Flag P-002

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO.

Location:

Description:

Reference:

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*

YELLOW FLAG*

SAFETY FLAG*

INSPECTOR *Jie Sun, P.E.*

FLAG NUMBER *P-003*

DATE OF INSPECTION *09 / 12 / 2013*

SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED ____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *LIRR*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO ____ YES ____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *24* COMPONENT: *N/A*

RATING: *1*

Location: Stringer along Column Line G at Span 24

Member: Span Item [28] Primary members (Stringer)

Description: The 32'-2" long stringer exhibits approximately 65% web section loss at its end connections and severe delaminated corrosion at mid-span, which could not inspected hands-on due to the presence of LIRR power lines near mid-span. Based on the severely corroded condition of stringers along this column line G in spans 25 and 26, we expect similar advanced corrosion in the web and flanges at midspan. There is no structural distress observed in the deck or surrounding structure.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *6*

Flagged Bridge Report Completed By *Jie Sun , P.E.* D *10 / 1 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To ____ of Regional Office on ____ at ____ O'clock

To ____ (Responsible Party) on ____ at ____ O'clock

By ____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BD 186(4/95)

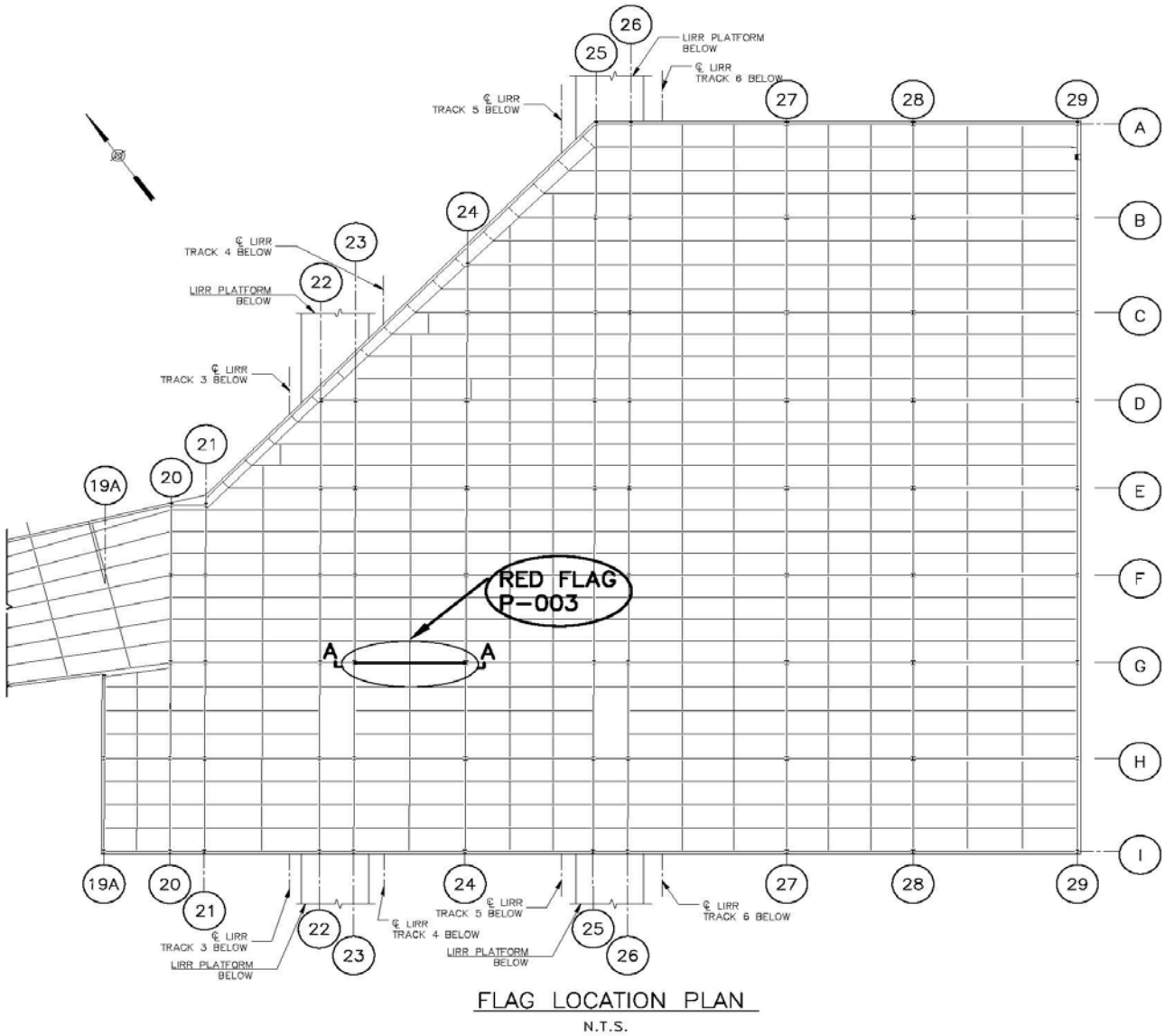
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 2 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 1 OF 4



FLAG LOCATION PLAN
N.T.S.

BD 186(4/95)

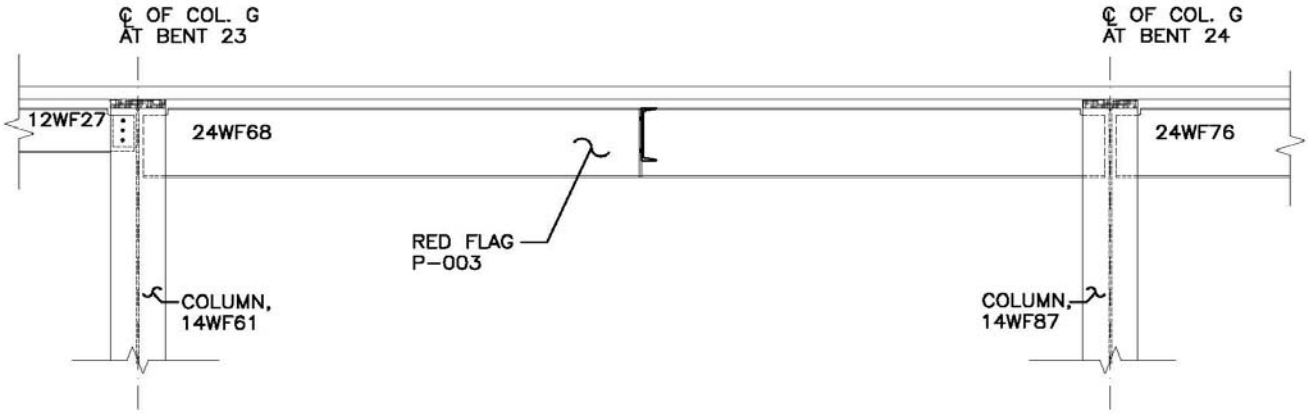
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 3 OF 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 2 OF 4



SECTION A–A, SPAN 24 STRINGER ALONG COL. LINE G, LOOKING EAST
N.T.S.

BD 186(4/95)

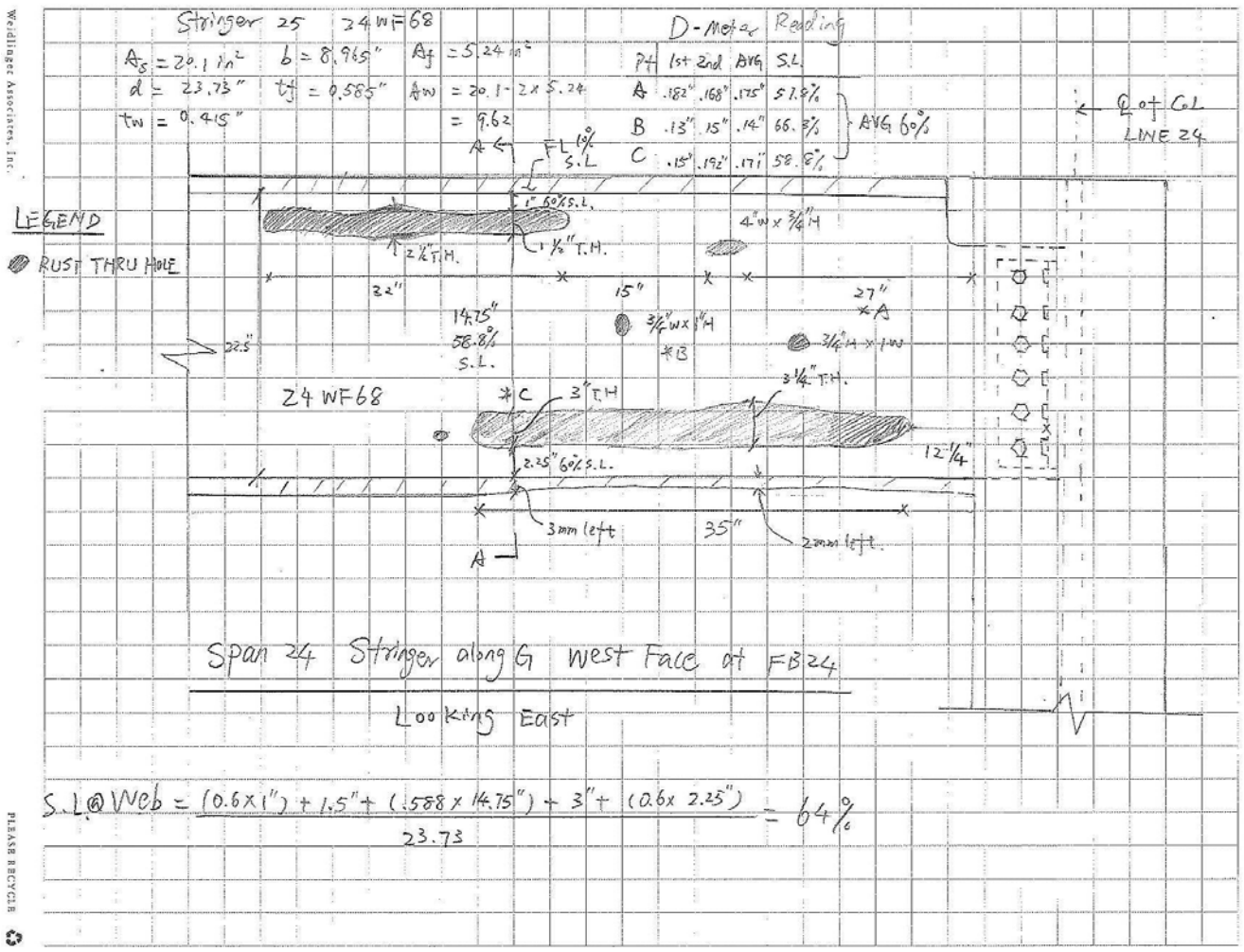
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 4 OF 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 3 OF 4



$$S.L.@Web = \frac{(0.6 \times 1") + 1.5" + (.588 \times 14.75") + 3" + (0.6 \times 2.25")}{23.73} = 64\%$$

BD 186(4/95)

BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 5 OF 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 4 OF 4

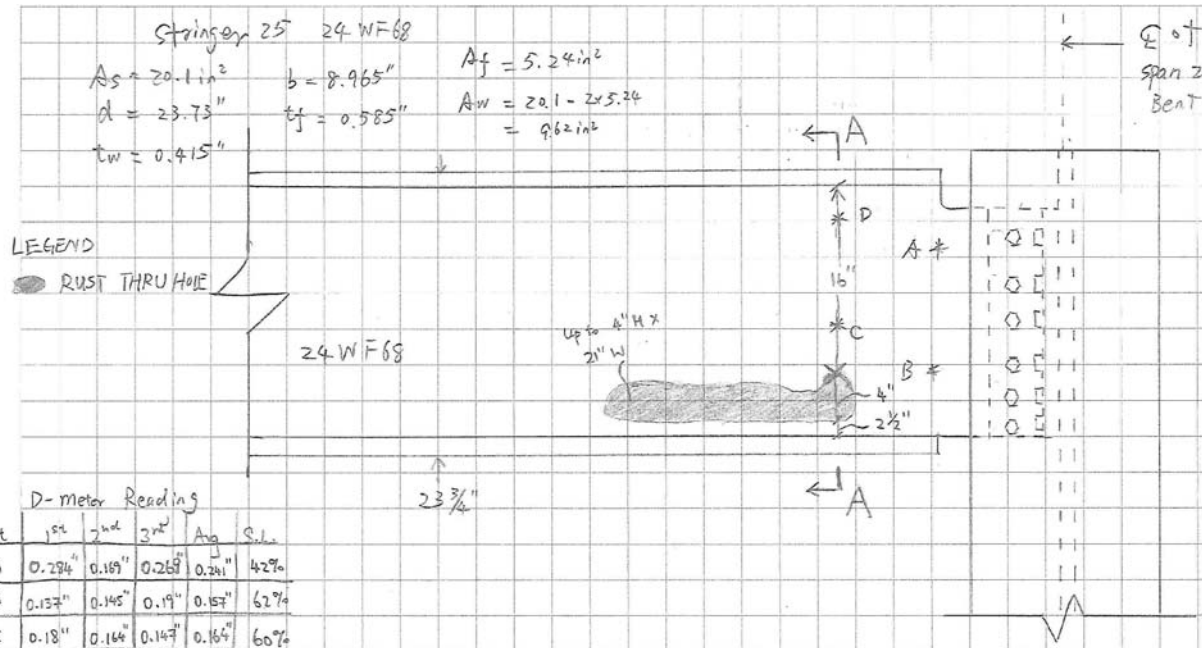
Stringer 25 24 WF68

$A_s = 20.1 in^2$ $b = 8.965''$ $A_f = 5.24 in^2$

$d = 23.73''$ $t_f = 0.585''$ $A_w = 20.1 - 2 \times 5.24 = 9.62 in^2$

$t_w = 0.415''$

℄ of Col. Line G span 24 at Bent 23



LEGEND
RUST THRU HOLE

D-meter Reading

Pt	1st	2nd	3rd	Avg	S.L.
A	0.284"	0.189"	0.269"	0.241"	42%
B	0.137"	0.145"	0.19"	0.157"	62%
C	0.18"	0.164"	0.147"	0.164"	60%
D	0.158"	0.146"	0.153"	0.152"	63%

Span 24 Stringer along Col. Line G East Face at Bent 23

Looking West

$$S.L. @ A-A = \frac{\frac{(0.63+0.6)}{2}(16+2\tfrac{1}{2}) + (1)(4)}{23.73''} = 64.8 \approx \boxed{65\%}$$

Weidlinger Associates, Inc.
PLEASE RECYCLE

PHOTO NO. **P003-P1**

Location:
General view of framing in Span 24 between Bent 23 and Bent 24
Looking East

Description:
Advanced corrosion of stringer throughout its length

Reference:
Red Flag P-003

ASST. TEAM LEADER: T.Ko
FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P003-P2**

Location:
Stringer along Col. Line G
Span 24 at Bent 24
Looking East

Description:
Advanced corrosion with approximate 64% web section loss at connection to Column 24-G

Reference:
Red Flag P-003

TEAM LEADER: J. Sun, P.E.
FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P003-P3**

Location:
Stringer along Col. Line G
Span 24 at Bent 24
Looking East

Description:
Close-up view of web
at connection to Column
24-G Estimated 64% overall
web section loss including
thru hole.

Reference:
Red Flag P-003

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P003-P4**

Location:
Stringer along Col. Line G
Span 24 near Bent 24
Looking Northeast

Description:
Close-up view of web
near connection to Column
24-G. Estimated 64% overall
web section loss.

Reference:
Red Flag P-003

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P003-P5**

Location:
Stringer along Col. Line G
Span 24 at Bent 23
Looking Northwest

Description:
Close-up view of web
at connection to Column
23-G Estimated 65% overall
web section loss including
thru hole.

Reference:
Red Flag P-003

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P003-P6**

Location:
Stringer along Col. Line G
Span 24 mid-span
Looking Northeast

Description:
General view of advanced web
corrosion near mid-span.
Could not access for further
inspection due to presence of
LIRR power lines
(shown in photo).

Reference:
Red Flag P-003

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*

YELLOW FLAG*

SAFETY FLAG*

INSPECTOR *Jie Sun, P.E.*

FLAG NUMBER *P-004*

DATE OF INSPECTION *09 / 13 / 2013*

SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED ____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *LIRR*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO ____ YES ____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *23* COMPONENT: *N/A*

RATING: *2*

Location: Column at Line G, Bent 23

Member: Span Item [38] Pier Columns

Description: Column exhibits severe corrosion with nearly 50% section loss at its top and approximately 60% section loss at its bottom. There is no indication of buckling or structural distress observed at the time of the inspection.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *6*

Flagged Bridge Report Completed By *Jie Sun , P.E.* D *10 / 1 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To ____ of Regional Office on ____ at ____ O'clock

To ____ (Responsible Party) on ____ at ____ O'clock

By ____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BD 186(4/95)

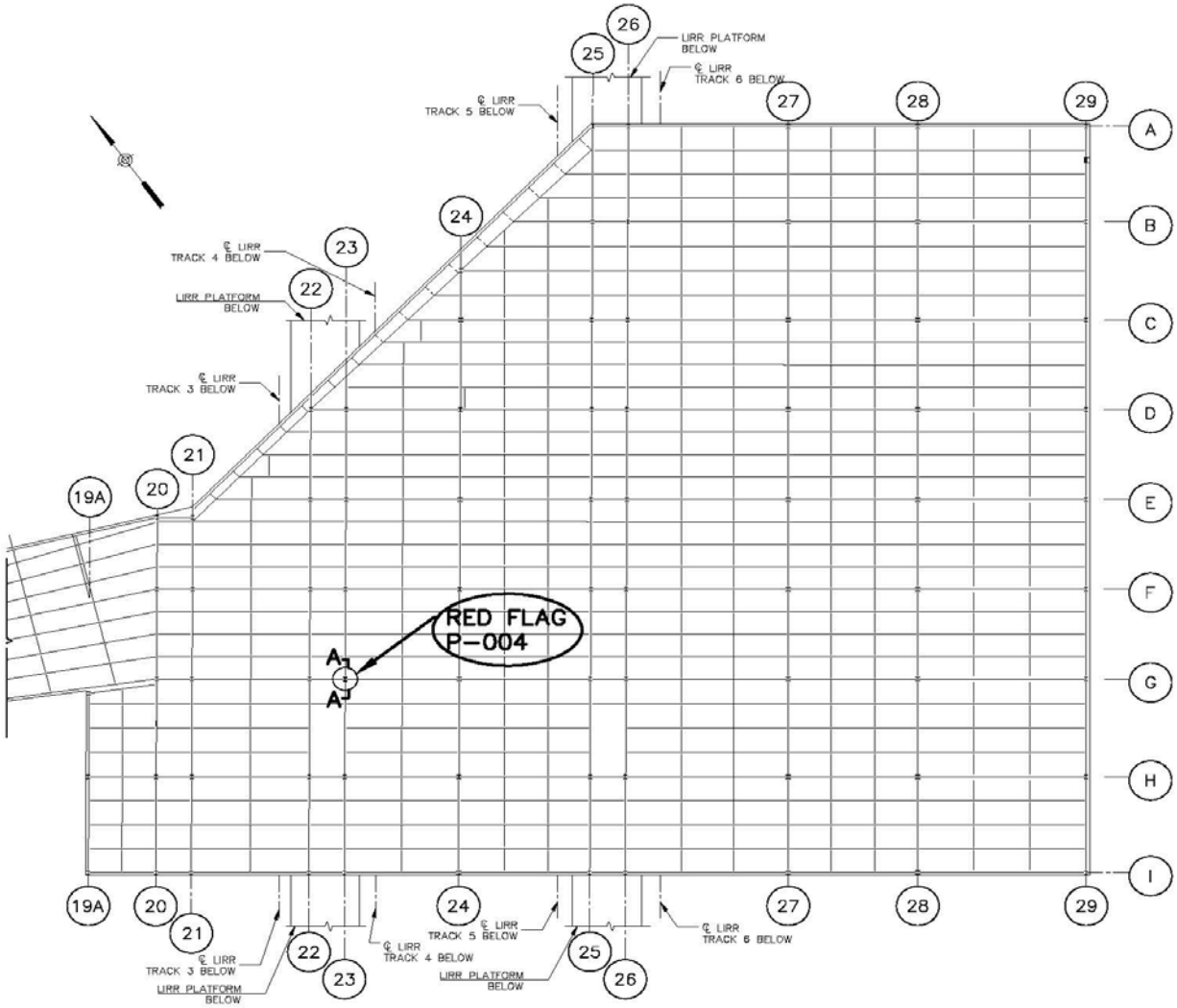
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 2 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 1 OF 4



FLAG LOCATION PLAN
N.T.S.

BD 186(4/95)

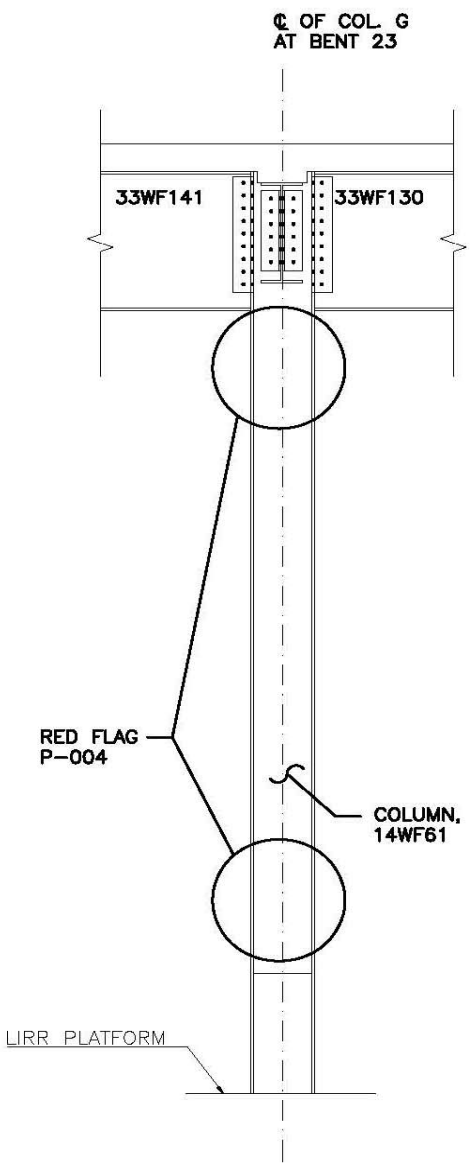
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 3 OF 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 2 OF 4



SECTION A–A COLUMN G. AT BENT 23 LOOKING NORTH
N.T.S.

BD 186(4/95)

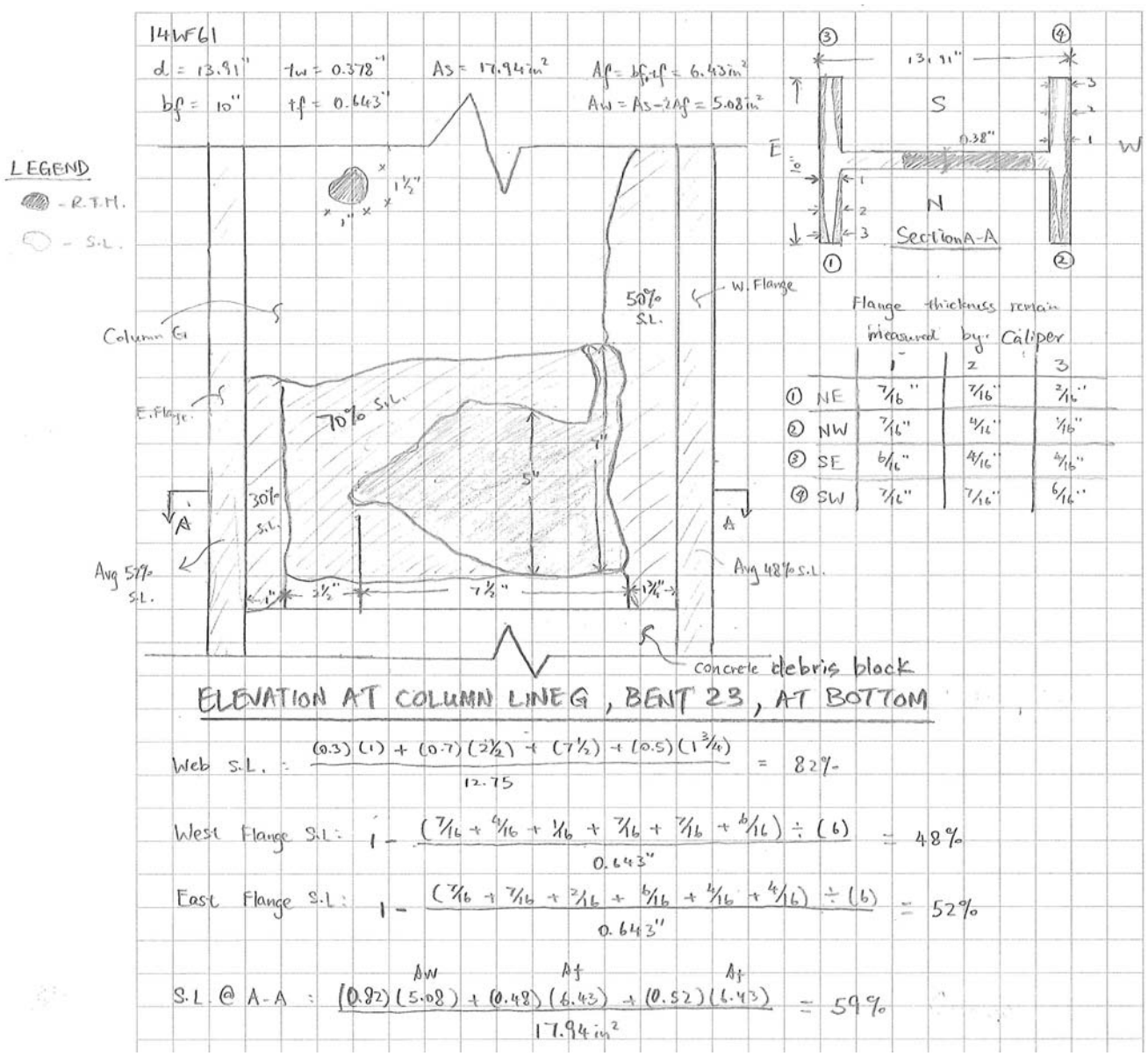
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 4 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 3 OF 4



BD 186(4/95)

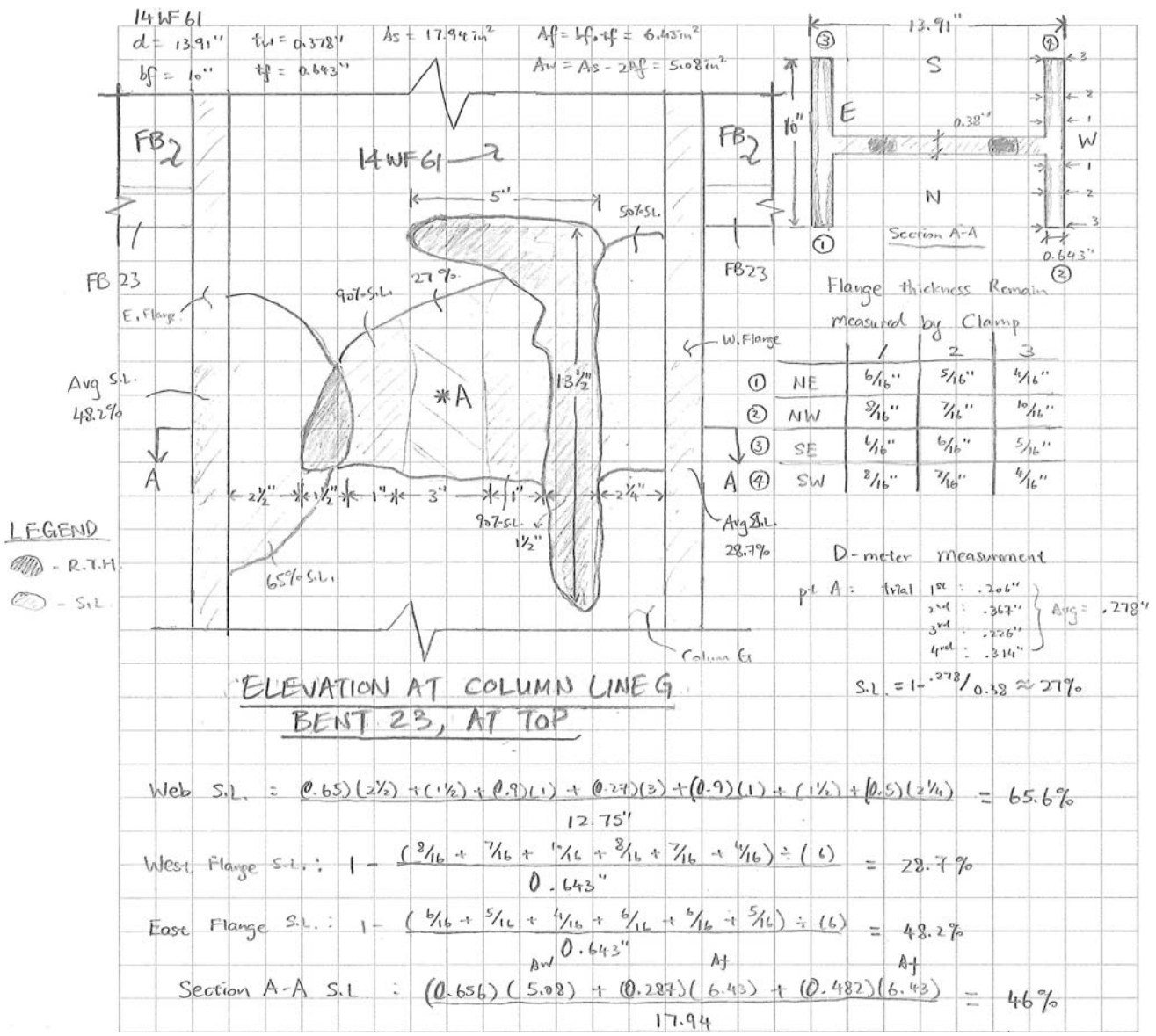
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 5 OF 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 01 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 4 OF 4



		1	2	3
①	NE	6/16"	5/16"	4/16"
②	NW	8/16"	7/16"	10/16"
③	SE	6/16"	6/16"	5/16"
④	SW	8/16"	7/16"	4/16"

Web S.L. = $\frac{(0.65)(2\frac{1}{2}) + (1\frac{1}{2}) + (0.9)(1) + (0.27)(3) + (0.9)(1) + (1\frac{1}{2}) + (0.5)(2\frac{1}{4})}{12.75"}$ = 65.6%

West Flange S.L.: $1 - \frac{(8/16 + 7/16 + 10/16 + 8/16 + 7/16 + 4/16) \div (4)}{0.643"}$ = 28.7%

East Flange S.L.: $1 - \frac{(6/16 + 5/16 + 4/16 + 6/16 + 6/16 + 5/16) \div (6)}{0.643"}$ = 48.2%

Section A-A S.L: $\frac{(0.656)(\overset{Aw}{5.08}) + (0.287)(\overset{Af}{6.43}) + (0.482)(\overset{Af}{6.43})}{17.94}$ = 46%

PHOTO NO. **P004-P1**

Location:
General view of Column 23-G
at Bent 23
(shown before rust removal)
Looking Southeast

Description:
Advanced corrosion near
top of column just below
stringer and floorbeam
connections

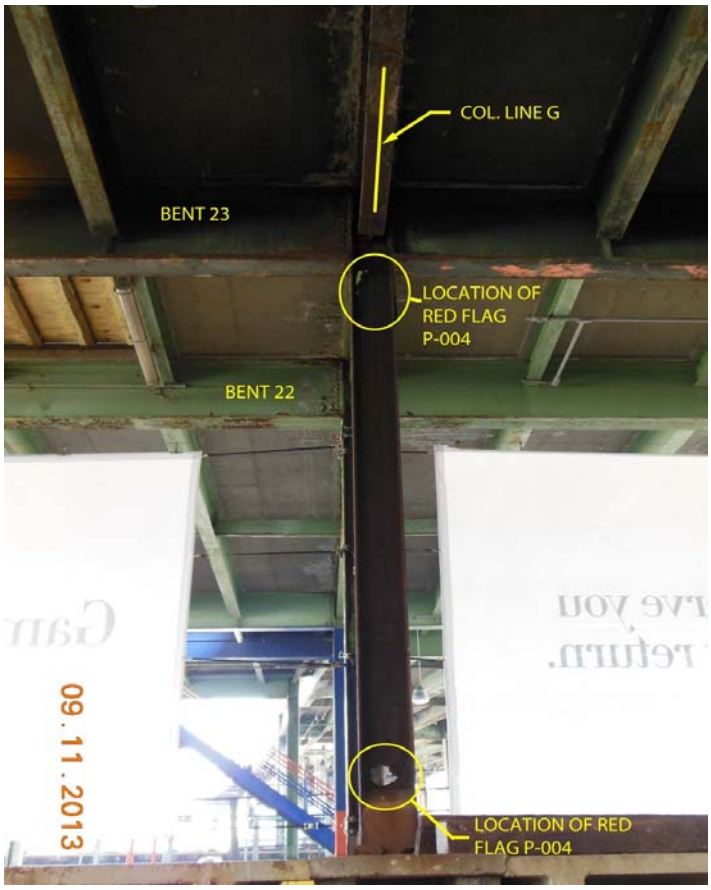
Reference:
Red Flag -P004

PHOTO NO. **P004-P2**

Location:
General view of Column
23-G at Bent 23 south face
(shown after rust removal)
Looking North

Description:
Advanced corrosion with thru
holes at top and bottom of
the column

Reference:
Red Flag -P004



ASST. TEAM LEADER: T.Ko
FEATURE UNDER: LIRR property and tracks

TEAM LEADER: J. Sun, P.E.
FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P004-P3**

Location:
Close-up view of Column 23-G at Bent 23
North face
(shown after rust removal)
Looking South

Description:
Advanced corrosion in web and flanges. Estimated 46% overall column section loss.

Reference:
Red Flag -P004

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P004-P4**

Location:
Close-up view of Column 23-G at Bent 23
South face
(shown after rust removal)
Looking North

Description:
Advanced corrosion in web and flanges. Estimated 46% overall column section loss.

Reference:
Red Flag -P004

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P005-P5**

Location:
Close-up view of Column 23-G at Bent 23 at bottom, North face Looking South

Description:
Advanced corrosion in web and flanges. Estimated 59% overall column section loss

Reference:
Red Flag P-004

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P005-P6**

Location:
Close-up view of Column 23-G at Bent 23 at bottom, South face Looking North

Description:
Advanced corrosion in web and flanges. Estimated 59% overall column section loss

Reference:
Red Flag P-004

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Jie Sun, P.E.*
FLAG NUMBER *P-005*
DATE OF INSPECTION *08 / 01 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *LIRR*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *28* COMPONENT: *N/A*

RATING: *3*

Location: Span 28 Stringer 18 near the south intermediate diaphragm

Member: Span Item [28] Primary members (Stringer)

Description: Severe impact damage was observed at Stringer 18 bottom flange, web, connecting diaphragms, and diaphragm connection plates. The length of the impact damage is 5'-4" long. Bottom flange is bent upward by 5", lower web bent westward by 6-1/4" from the center line, and west diaphragm is twisted. In addition, the diaphragm connection plate at east face of Stringer 18 exhibits a 7" long broken weld and a 3-1/2" separation from the stringer web due to the impact. No structural crack in the stringer was observed.

For information only: The minimum vertical clearance to the roadway below was measured to vary from 15'-9" to 16'-7". No structural distress of the deck or surrounding framing structure was observed.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *7*

Flagged Bridge Report Completed By *Jie Sun , P.E.* D *10 / 3 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BD 186(4/95)

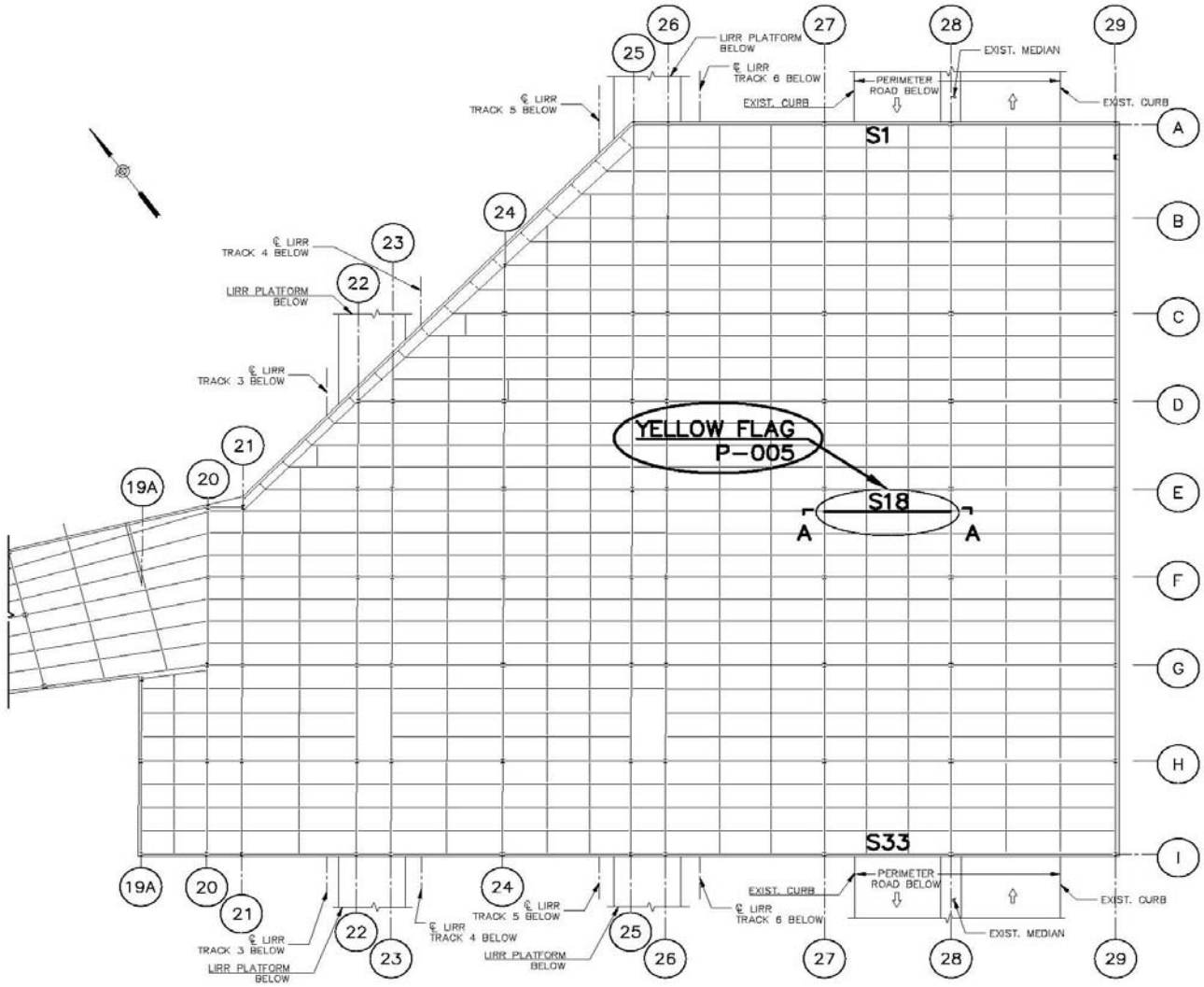
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 2 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 03 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 1 OF 3



FLAG LOCATION PLAN
N.T.S.

BD 186(4/95)

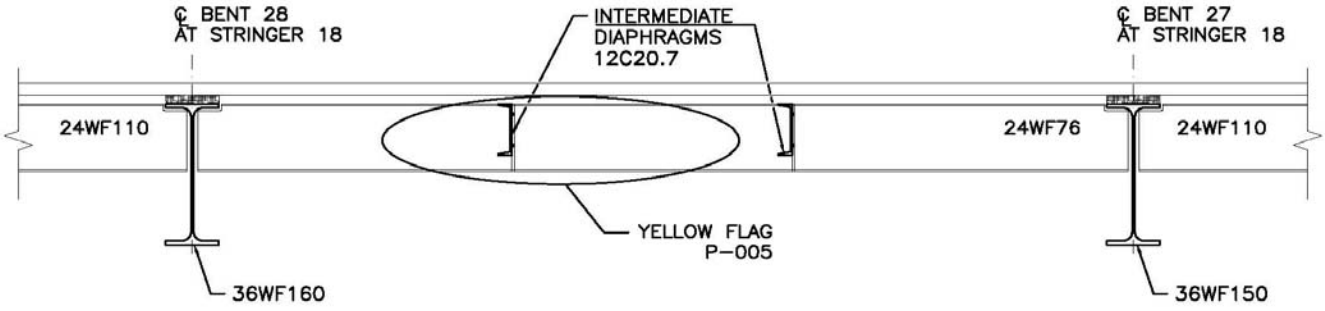
BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 3 OF 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 03 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 2 OF 3



SECTION A–A, SPAN 28 STRINGER 18, LOOKING WEST
N.T.S.

BD 186(4/95)

BIN _____

NYS DEPT. OF TRANSPORTATION
BRIDGE INSPECTION REPORT
SHEET 4 **OF** 8

TEAM LEADER: Jie Sun, P.E **ASST. TEAM LEADER:** Tung-for Ko **DATE** 10/ 03 / 13

Feature Carried: PASSERELLE PED.BRIDGE
Feature Crossed: LIRR

SKETCH 3 OF 3

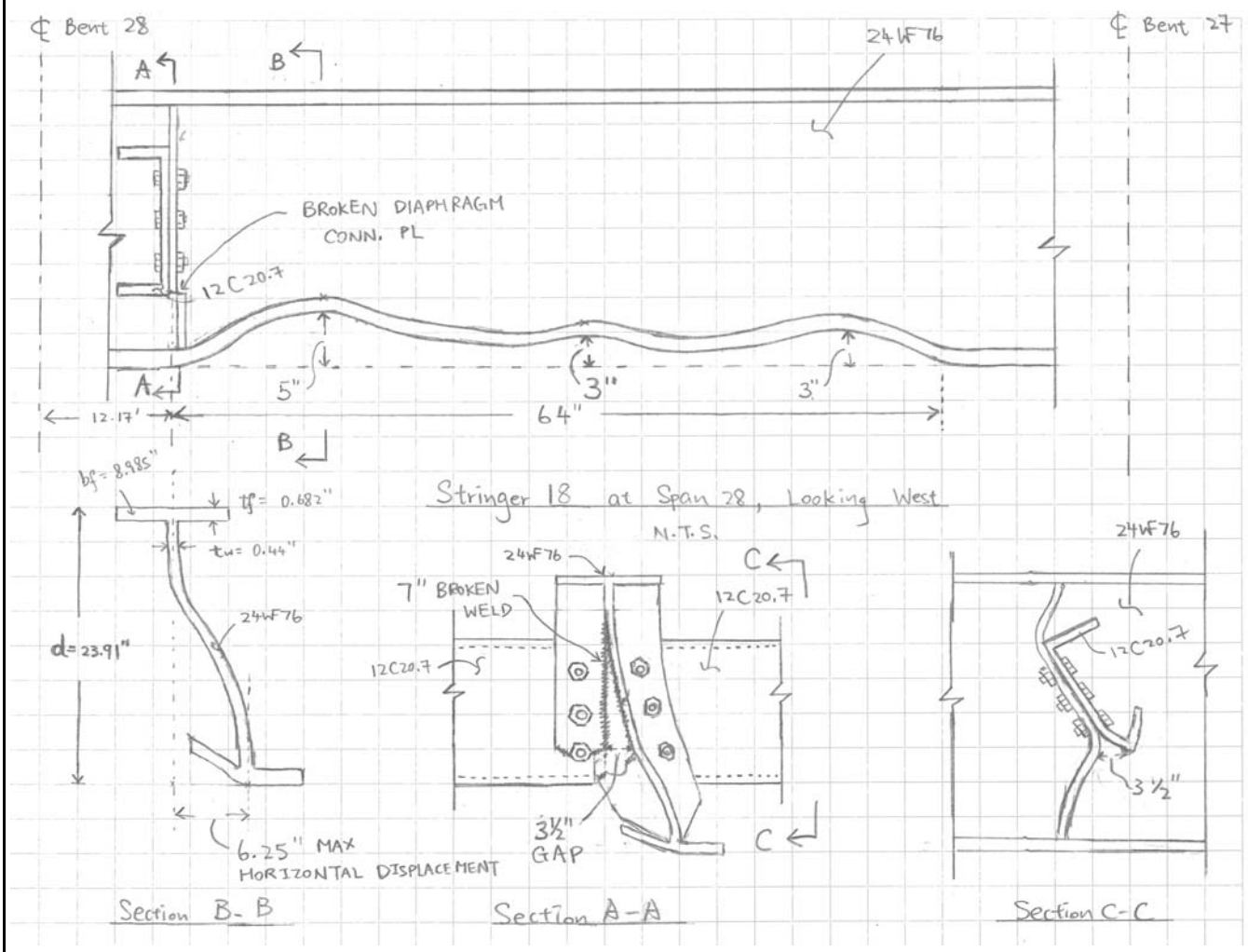
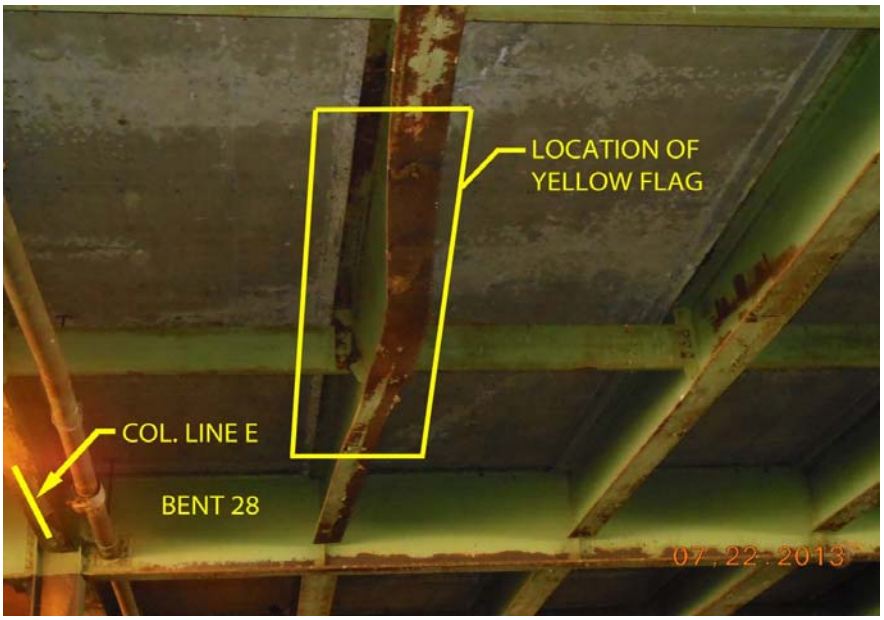


PHOTO NO. **P005-P1**

Location:
Stringer 18 at Span 28
Looking South & Up

Description:
General view of impact damage on the bottom flange of Stringer 18 and damaged diaphragm connection plate.

Reference:
Yellow Flag -P005



ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P005-P2**

Location:
Stringer 18 at Span 28
Looking Southwest & Up

Description:
General view of impact damage on the bottom flange of Stringer 18 and damaged diaphragm connection plate.

Reference:
Yellow Flag -P005

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P005-P3**

Location:
Stringer 18 east face
at the south diaphragm
connection
Looking North

Description:
The diaphragm connection
plate exhibits a broken weld
approximately 7" long
and up to 3.5" wide
separation from the web

Reference:
Yellow Flag -P005

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P005-P4**

Location:
Stringer 18 west face
at connection to the south
diaphragm
Looking North

Description:
Close-up view of the impact
damage at the diaphragm
and at the connection plate.

Reference:
Yellow Flag -P005

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P005-P5**

Location:
Stringer 18 east face
at Span 28, south Diaphragm
Looking Southwest

Description:
Stringer bottom flange
exhibits 64" long impact
damage. The impact caused
the bottom flange to bend
upward by 5"

Reference:
Yellow Flag -P005

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P005-P6**

Location:
Stringer 18 at Span 28
bottom flange
Looking South and Up

Description:
Stringer 18 bottom flange
exhibits impact damage.

Reference:
Yellow Flag P-005

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P005-P7**

Location:
Stringer 18 west face at
Span 28
Looking South

Description:
Stringer 18 web exhibits
impact damage.

Reference:
Yellow Flag P-005

ASST. TEAM LEADER: T.Ko

FEATURE UNDER: LIRR property and tracks

PHOTO NO.

Location:

Description:

Reference:

TEAM LEADER: J. Sun, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*

_____ YELLOW FLAG*

_____ SAFETY FLAG*

INSPECTOR *Charles Diamond*

FLAG NUMBER *P-006*

DATE OF INSPECTION *10 / 10 / 2013*

SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT Yard*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *18* COMPONENT: *N/A*

RATING: _____

Location: Span 18

Member: Stringer S3C

Description: Stringer S3C has completely sheared off web at the connection to floor beam 17B and the stringer is side swinging approximately 1/8".

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *2*

Flagged Bridge Report Completed By _____ D *10 / 18 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0005**

Location:
Stringer S3C
Span 18

Looking East

Description:
The stringer has completely sheared off the web connection to floor beam 17B and the stringer is side swinging approximately 1/8"

Reference:
RED FLAG P-006

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **DSCN0006**

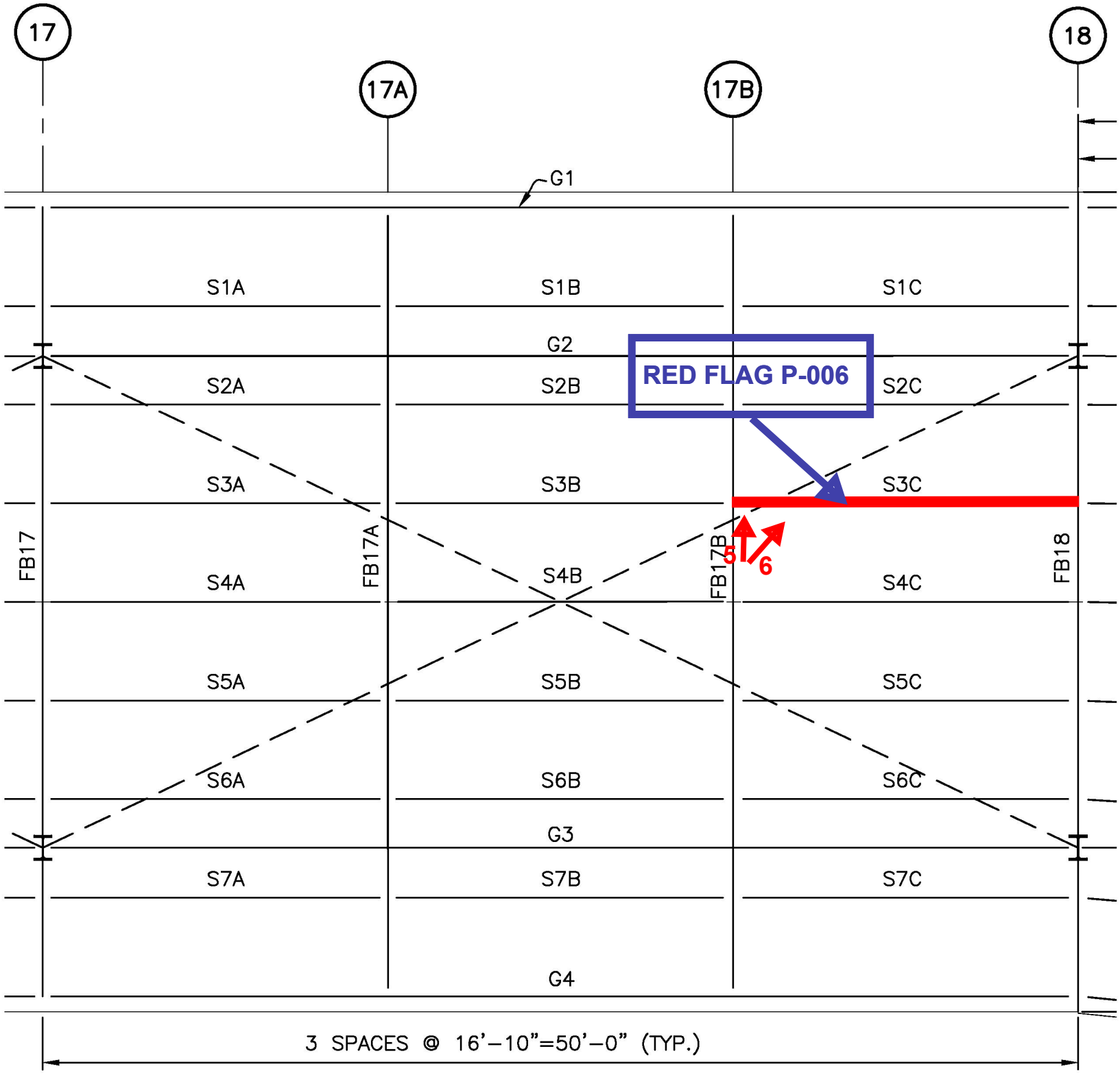
Location:
Stringer S3C
Span 18
Looking Southeast

Description:
The stringer has completely sheared off the web connection to floor beam 17B and the stringer is side swinging approximately 1/8"

Reference:
RED FLAG P-006

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge


17
17A
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G1
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S1B
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RED FLAG P-006
S3A
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FB17B
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S6B
S6C
G3
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S7B
S7C
G4
3 SPACES @ 16'-10"=50'-0" (TYP.)

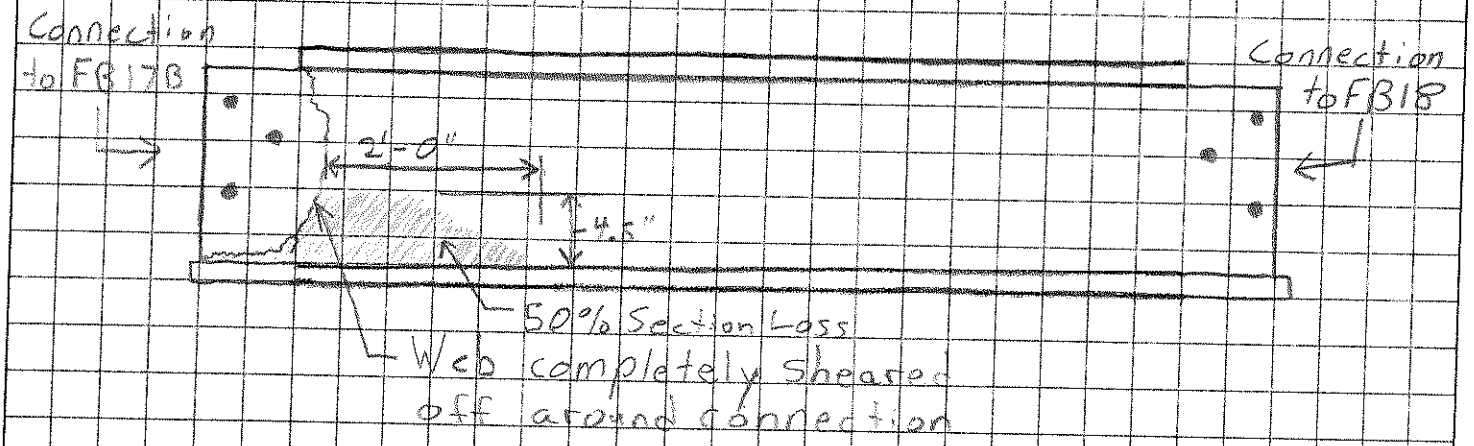
HAKS

Construction Management • Engineering • Architecture
Land Surveying • Material Testing

JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

RED FLAG P-006

Span 18 - Stringer S3C



NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*

_____ YELLOW FLAG*

_____ SAFETY FLAG*

INSPECTOR *Charles Diamond*

FLAG NUMBER *P-007*

DATE OF INSPECTION *10 / 10 / 2013*

SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT Yard*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *18* COMPONENT: *N/A*

RATING: _____

Location: Span 18

Member: Stringer S7C

Description: The redundant stringer S7C exhibits extreme corrosion with Up to 2" high perforation on its web along the bottom flange in various locations. Also, the web of stringer S7C at the connection to floor beam 17B is completely sheared off—the stringer is sagging approximately 1/8".

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By _____ D *10 / 18 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Signature of State Team Leader or Contract Engineer Representative

Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0002**

Location:
Stringer S7C
Span 18
Looking West

Description:
Extreme corrosion with up to 2" high perforation on the web The web is totally sheared off and the connection to floor beam 17B is completely sheared off. The stringer is sagging approximately 1/8".

Reference:
RED FLAG P-007

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **DSCN0001**

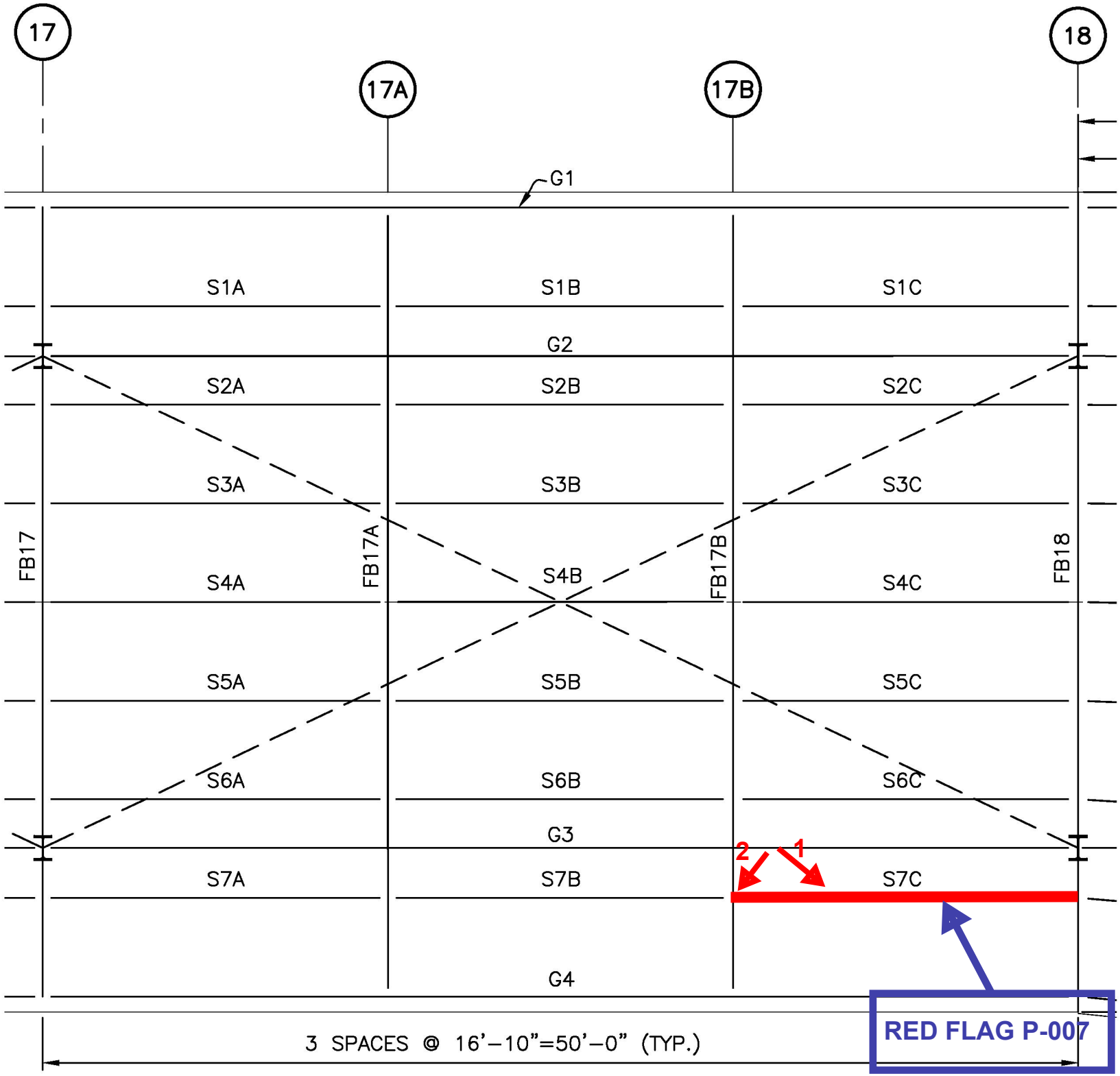
Location:
Stringer S7C
Span 18
Looking West

Description:
Extreme corrosion with up to 2" high perforation on the web The web is totally sheared off and the connection to floor beam 17B is completely sheared off. The stringer is sagging approximately 1/8".

Reference:
RED FLAG P-007

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge


17
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FB17
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FB17B
FB18
S4A
S4B
S4C
S5A
S5B
S5C
S6A
S6B
S6C
G3
2
1
S7A
S7B
S7C
G4
3 SPACES @ 16'-10"=50'-0" (TYP.)
RED FLAG P-007

HAKS

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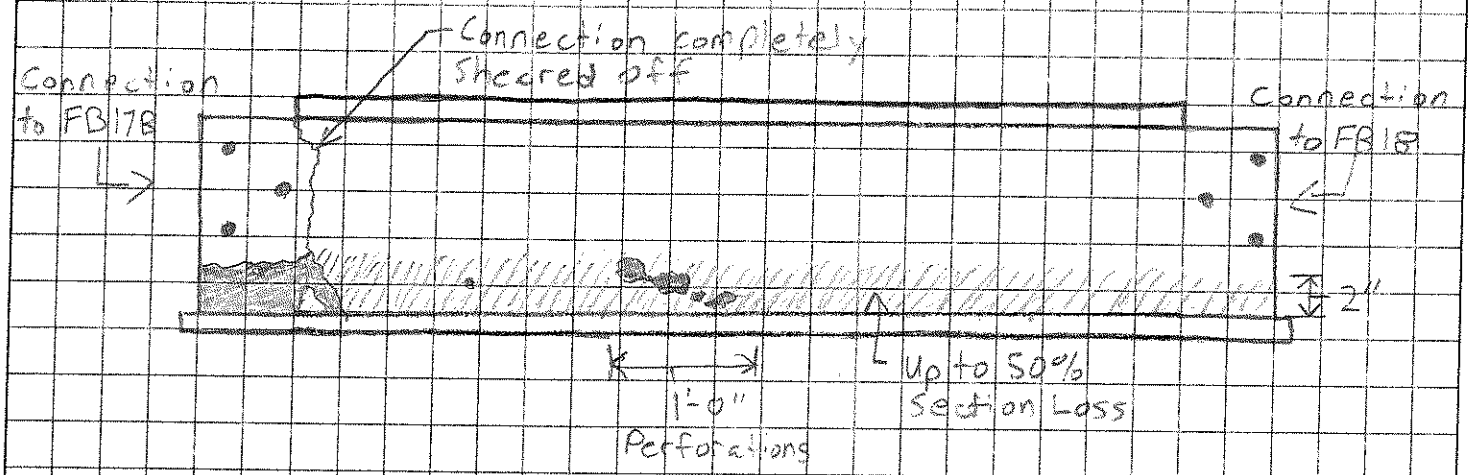
Land Surveying • Material Testing

JOB Paserelle Pedestrian Bridge
SHEET NO. 1 OF 1
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

RED FLAG P-007

Span 18 - Stringer S7C



NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*

_____ YELLOW FLAG*

_____ SAFETY FLAG*

INSPECTOR *Charles Diamond*

FLAG NUMBER *P-008*

DATE OF INSPECTION *10 / 10 / 2013*

SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT Yard*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *18* COMPONENT: *N/A*

RATING: _____

Location: Span 18

Member: Stringers S5C and S6C

Description: The redundant stringers S5C and S6C exhibit extreme corrosion with Up to 2" high perforation on its web along the bottom flange in various locations.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *2*

Flagged Bridge Report Completed By _____ D *10 / 18 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Signature of State Team Leader or Contract Engineer Representative _____ Date _____

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0003**

Location:
Stringer S6C
Span 18
Looking Northwest

Description:
Extreme corrosion with up to
2" high perforation on the web

Reference:
RED FLAG P-008

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. _____

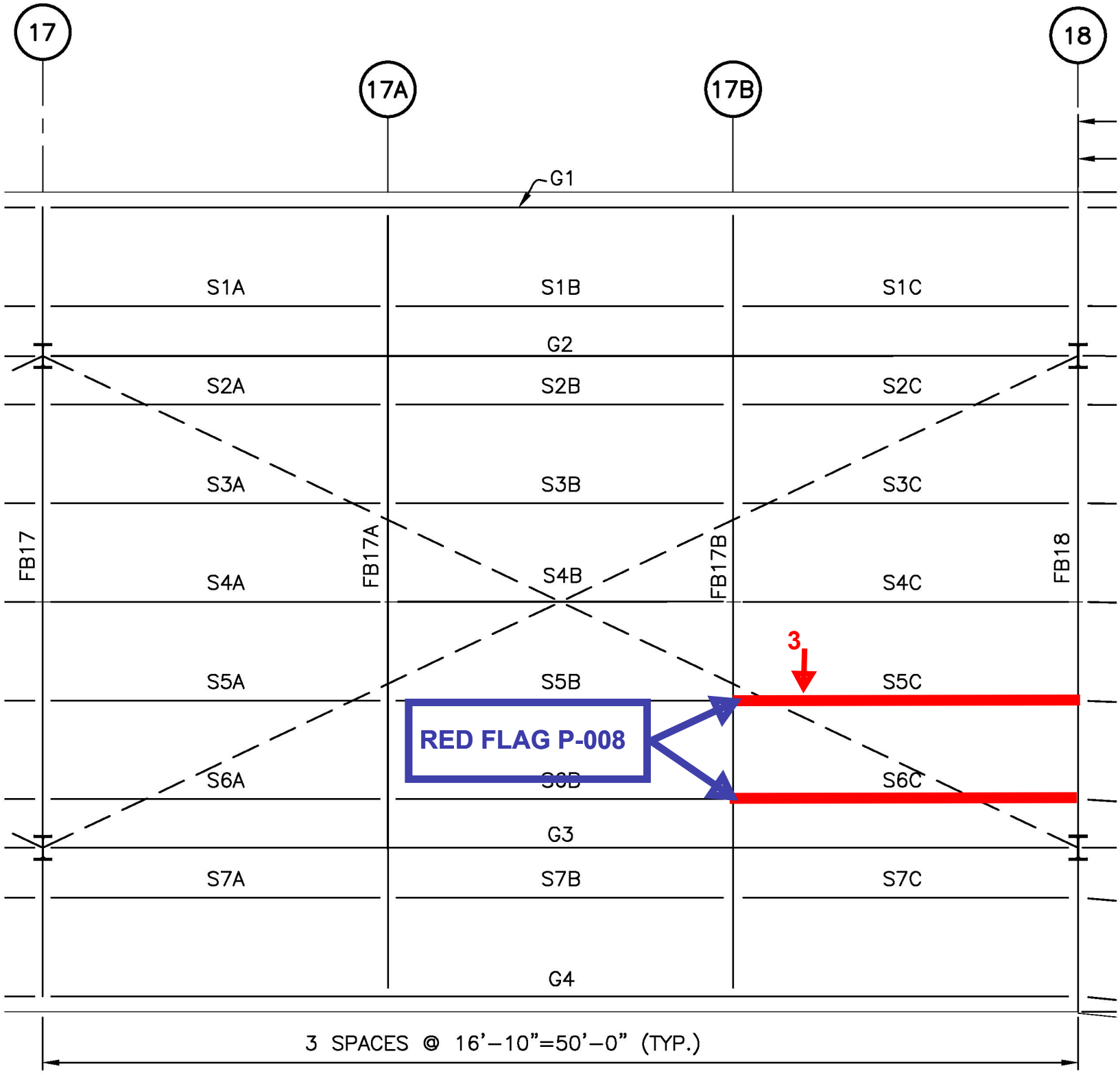
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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FB18
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S4C
3
S5A
S5B
S5C
RED FLAG P-008
S6A
S6B
S6C
G3
S7A
S7B
S7C
G4
3 SPACES @ 16'-10"=50'-0" (TYP.)

HAKS

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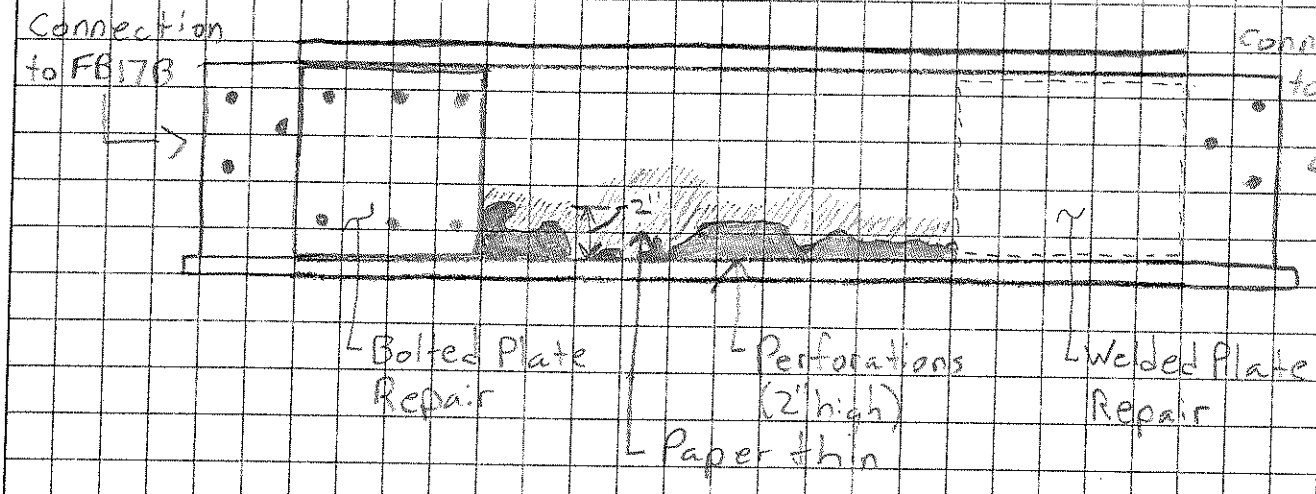
Land Surveying • Material Testing

JOB Paserelle Pedestrian Bridge
SHEET NO. 1 OF 2
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

RED FLAG P-008

Span 18 - Stringer S5C



HAKS

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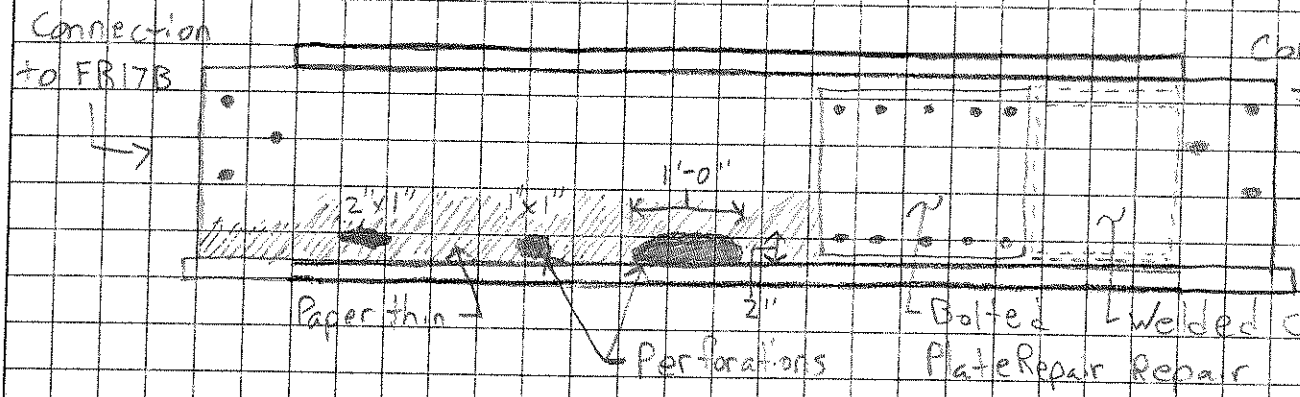
JOB Paserelle Pedestrian Bridge
SHEET NO. 2 OF 2
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

RED FLAG P-008

Span 18 - Stringer S6C



NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

X RED FLAG*

_____ YELLOW FLAG*

_____ SAFETY FLAG*

INSPECTOR *Charles Diamond*

FLAG NUMBER *P-009*

DATE OF INSPECTION *10 / 10 / 2013*

SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT Yard*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *18* COMPONENT: *N/A*

RATING: _____

Location: Span 18

Member: Stringer S7B

Description: Stringer S7B exhibits up to 1.5" high perforation on its web along the bottom flange for 90% of the span length. Other portions of the web are paper-thin. The top flange has a broken edge up to ¼". The bottom flange exhibits up to 15% section loss—it is shaky and is unable to function as designed.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *2*

Flagged Bridge Report Completed By _____ D *10 / 18 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0007**

Location:
Stringer S7B
Span 18
Looking Northeast

Description:
Stringer S7B exhibits up to 1.5" high perforation on its web along the bottom flange for 90% of the span length. Other portions of the web are paper-thin. The top flange has a broken edge up to ¼". The bottom flange exhibits up to 15% section loss—it is shaky and is unable to function as designed.

Reference:
RED FLAG P-009

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **DSCN0008**

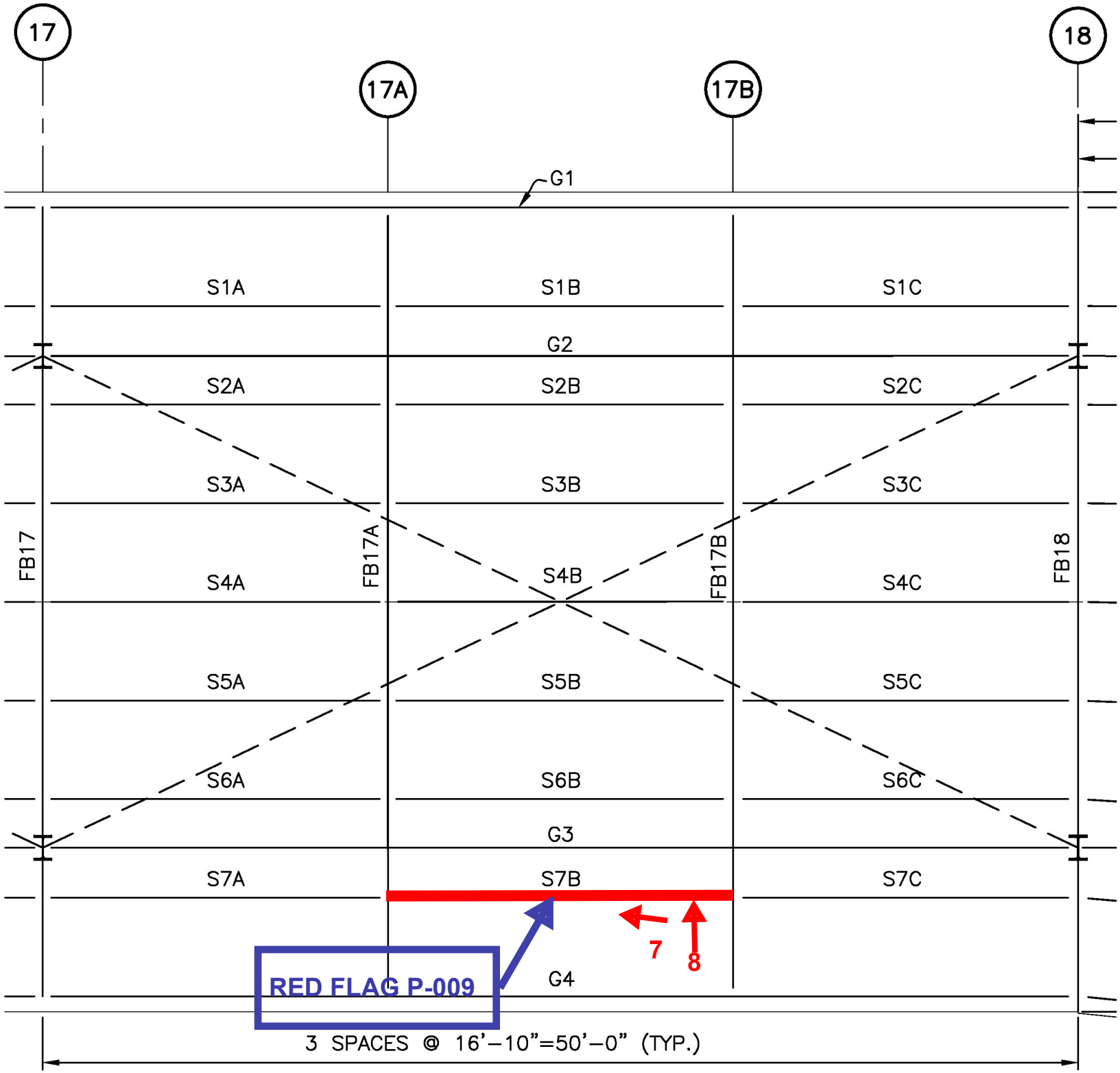
Location:
Stringer S7B
Span 18
Looking East

Description:
Stringer S7B exhibits up to 1.5" high perforation on its web along the bottom flange for 90% of the span length. Other portions of the web are paper-thin. The top flange has a broken edge up to ¼". The bottom flange exhibits up to 15% section loss—it is shaky and is unable to function as designed.

Reference:
RED FLAG P-009

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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FB17B
FB18
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S4C
S5A
S5B
S5C
S6A
S6B
S6C
G3
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S7B
S7C
7
8
RED FLAG P-009
G4
3 SPACES @ 16'-10"=50'-0" (TYP.)

HAKS

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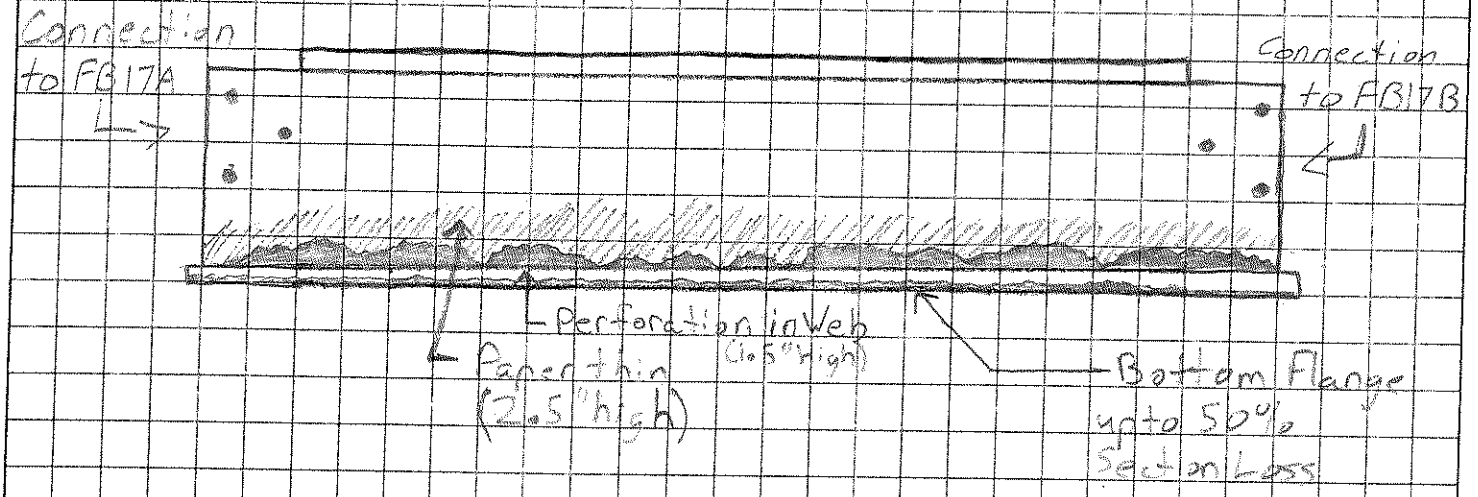
Land Surveying • Material Testing

JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

RED FLAG - P-009

Span 18 - Stringer S7B



NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*

X YELLOW FLAG*

_____ SAFETY FLAG*

INSPECTOR *Charles Diamond*

FLAG NUMBER *P-010*

DATE OF INSPECTION *10 / 10 / 2013*

SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *18* COMPONENT: *N/A*

RATING: _____

Location: Span 18

Member: Floor Beam 17B

Description: The non-redundant floor beam 17B exhibits perforation on the bottom 1/3 of its web along the web stiffener above its supporting member G3. The remaining portion of the web also exhibits extreme corrosion with up to 2" diameter holes and a few pinholes.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *4*

Flagged Bridge Report Completed By _____ D *10 / 18 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0009**

Location:
Floor Beam 17B
Span 18
Looking South

Description:
Perforation on bottom 1/3
of web

Reference:
YELLOW FLAG P-010

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **DSCN0010**

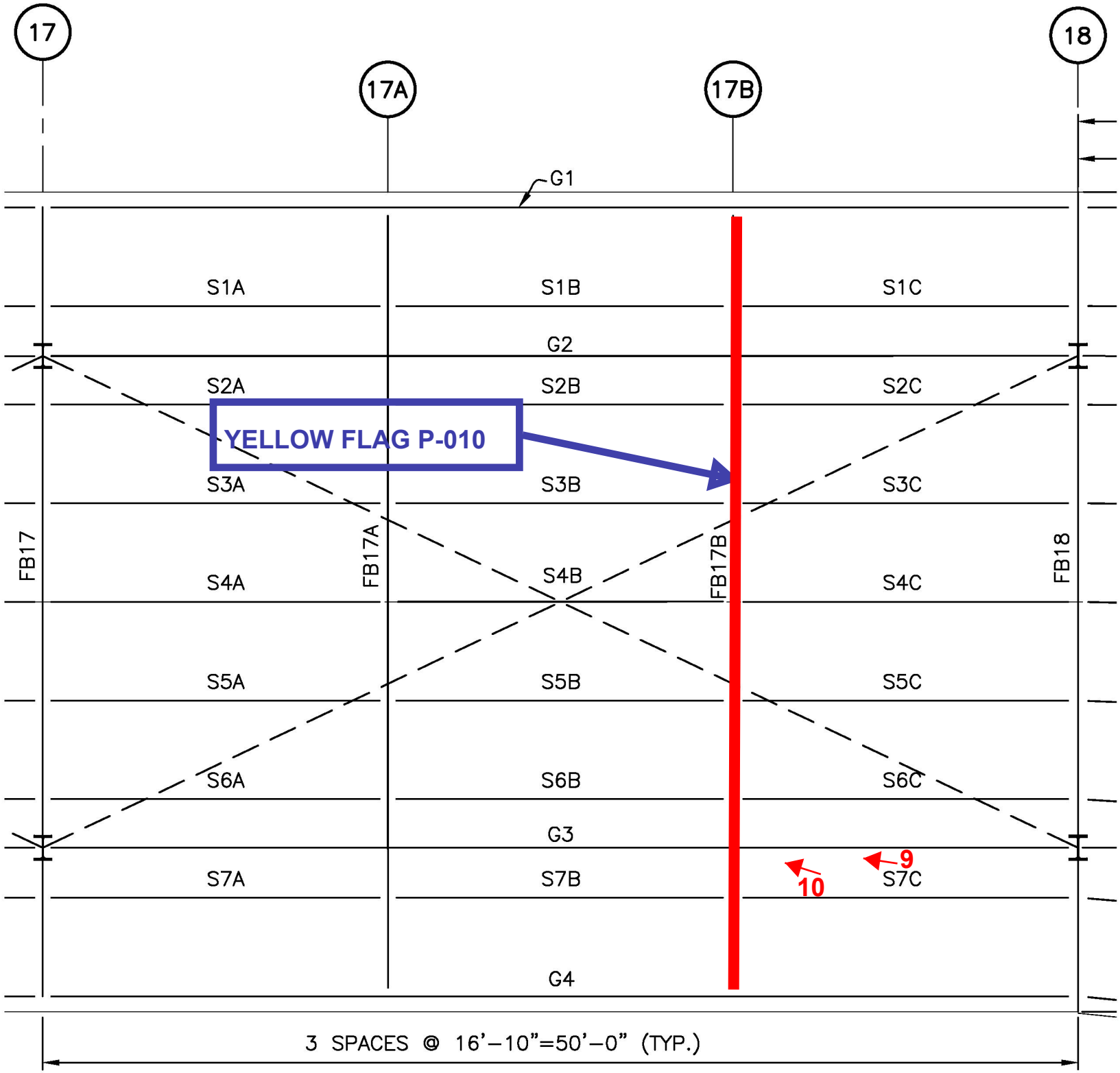
Location:
Floor Beam 17B
Span 18
Looking South

Description:
Perforation on bottom 1/3
of web

Reference:
YELLOW FLAG P-010

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S2C
YELLOW FLAG P-010
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S6A
S6B
S6C
G3
S7A
S7B
S7C
10
9
G4
3 SPACES @ 16'-10"=50'-0" (TYP.)

HAKS

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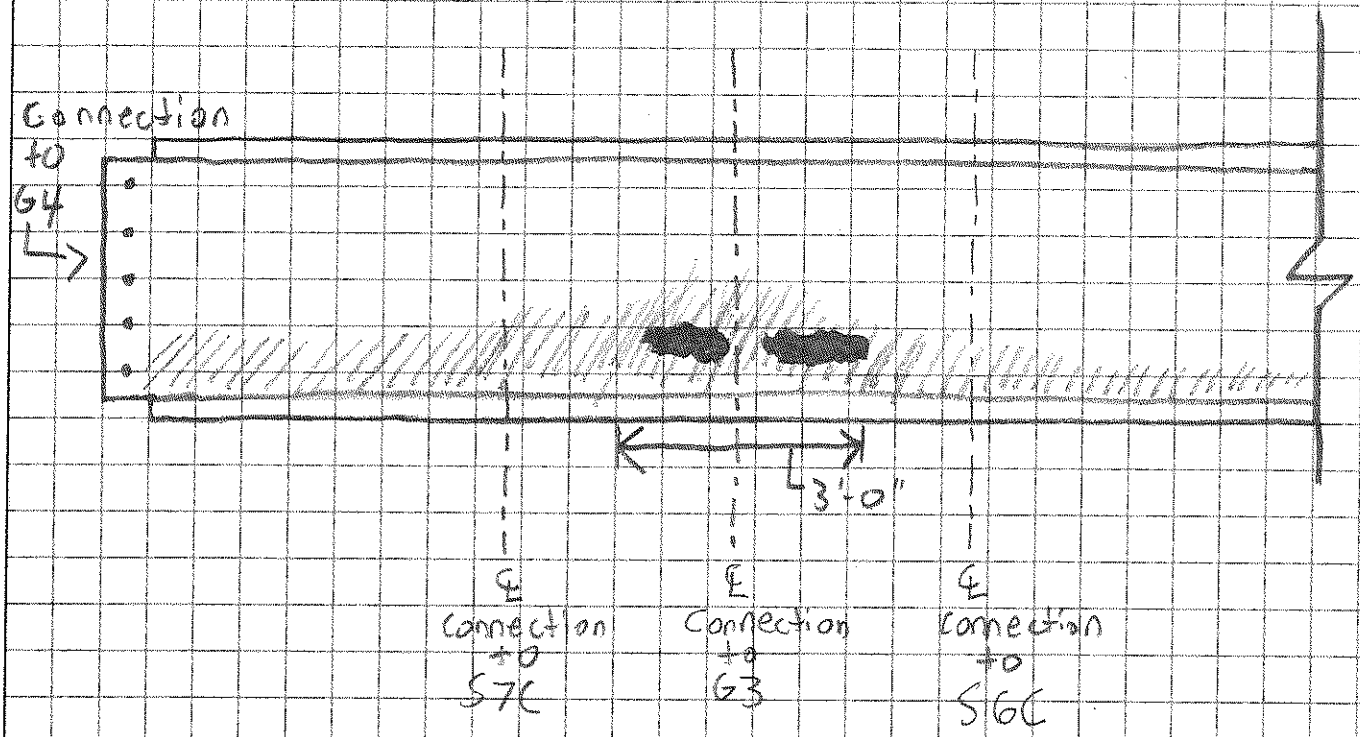
Land Surveying • Material Testing

JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-010

Span 18 - Floor beam FB17B



BD 242

Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
_____ YELLOW FLAG*
X SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-011*
DATE OF INSPECTION *10 / 23 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES X NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD X NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES X NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *12* COMPONENT: *N/A*

RATING: _____

Location: Span 12

Member: Timber Deck between stringers S4A and S5A

Description: The timber deck located between floor beams FB11 and FB11A, at 11 feet South of FB11 and between stringers S4A and S5A exhibits two soft boards. One board has a fine crack crossing the fibers at 1.5 feet from S4A. The board deflects about 1 inch when stepped on. There is a potential of a punch through. The adjacent floor board west of the board described above is soft as well and exhibits ½ inch deflection under loading.

PHOTOS ATTACHED? X YES ____ NO IF YES, NUMBER *3*

Flagged Bridge Report Completed By Mike Perng D *10 / 23 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Mike Perng PE#068842 10/23/13

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0179**

Location: Span 12
Timber deck at 11 feet south of Floor Beam FB11

Looking : South-West

Description:
General location view.

Reference:
Safety Flag P-011

ASST. TEAM LEADER:
FEATURE UNDER: NYCT Yard

PHOTO NO. **DSCN0180**

Location: Span 12
Timber deck at 11 feet south of Floor Beam FB11

Looking : South-West.

Description:
Floor board showing crack across fibers.

Reference:
Safety Flag P-011

TEAM LEADER: Mike Perng, P.E.
FEATURE CARRIED: Passerelle Pedestrian Bridge

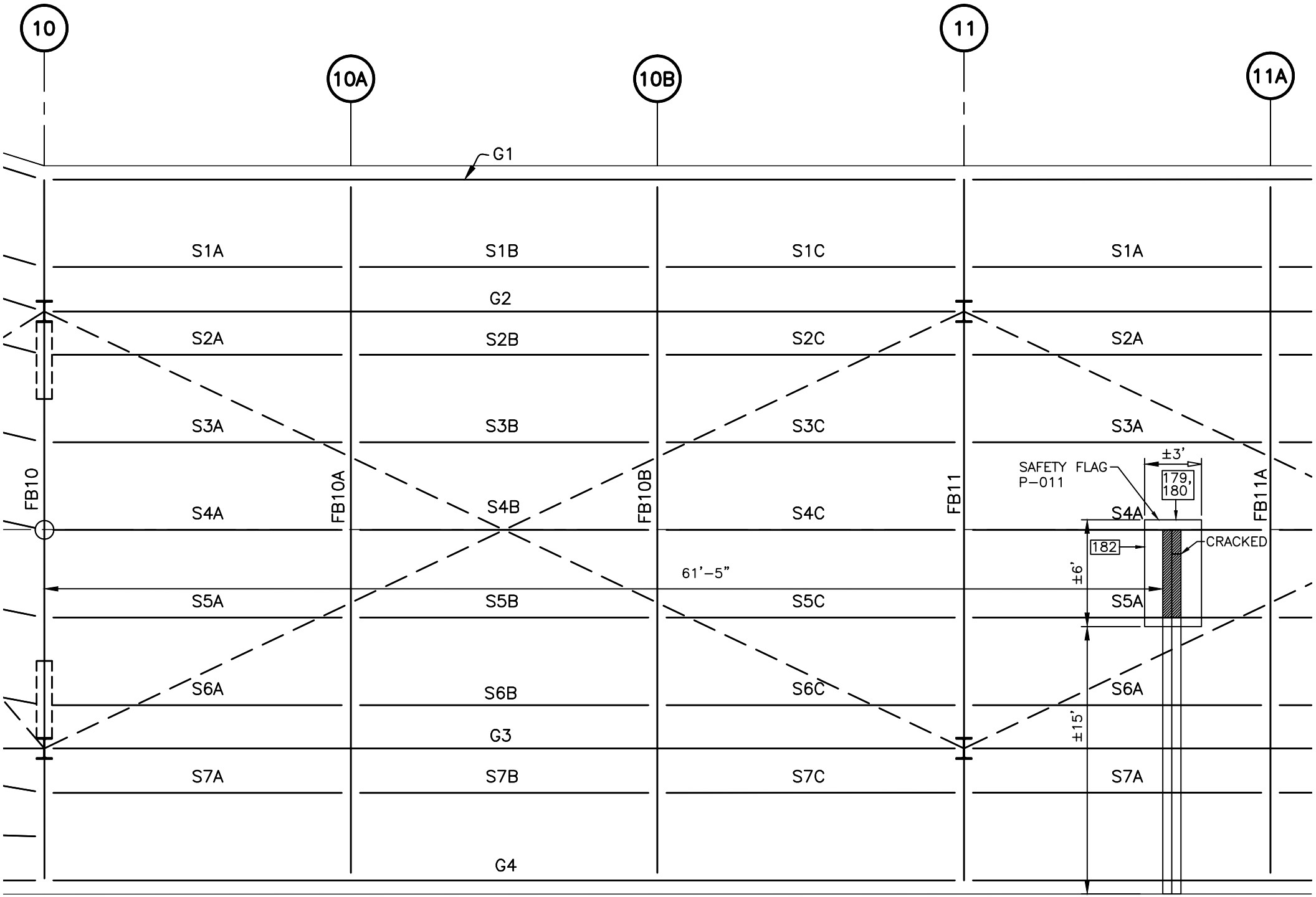
PHOTO NO. **DSCN0182**

Location: Span 12
Timber deck at 11 feet south of Floor Beam FB11

Looking : South-East.

Description:
Floor board showing 1" deflection under loading.

Reference:
Safety Flag P-011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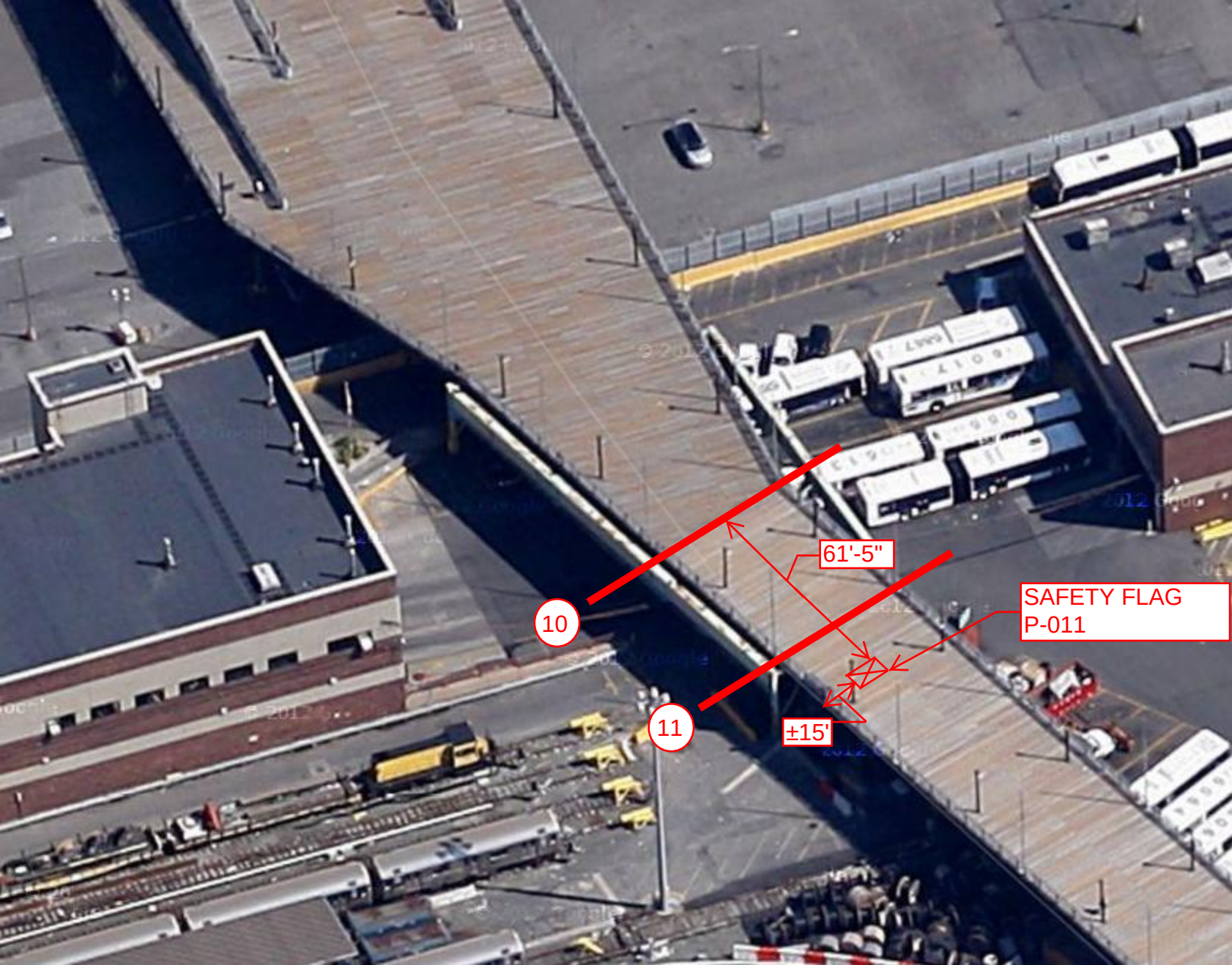
S6C
S6A
S7A
S7B
S7C
S7A
FB10
FB10A
FB10B
FB11
FB11A
61'–5"
±3'
SAFETY FLAG
P–011
179,
180
182
CRACKED
±6'
±15'


61'-5"
SAFETY FLAG
P-011
10
11
±15'

BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-012*
DATE OF INSPECTION *10 / 17 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *8* COMPONENT: *N/A*

RATING: _____

Location: Span 8

Member: Floor Beam 7A3 between Girder G3 & G4

Description: The non-redundant floor beam 7A3 exhibits perforation in the web near the connection to girder G3. The surrounding portion of the web near the perforations is paper thin exhibiting extreme corrosion. Total section loss for the member is 40%.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *2*

Flagged Bridge Report Completed By _____ D *10 / 24 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0058**

Location: Span 8
Floor Beam 7A3 between Girder G3 & G4

Looking :West

Description:
Perforation in web near connection to Girder G3.

Reference:
YELLOW FLAG P-012

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **DSCN0058**

Location: Span 8
Floor Beam 7A3 between Girder G3 & G4

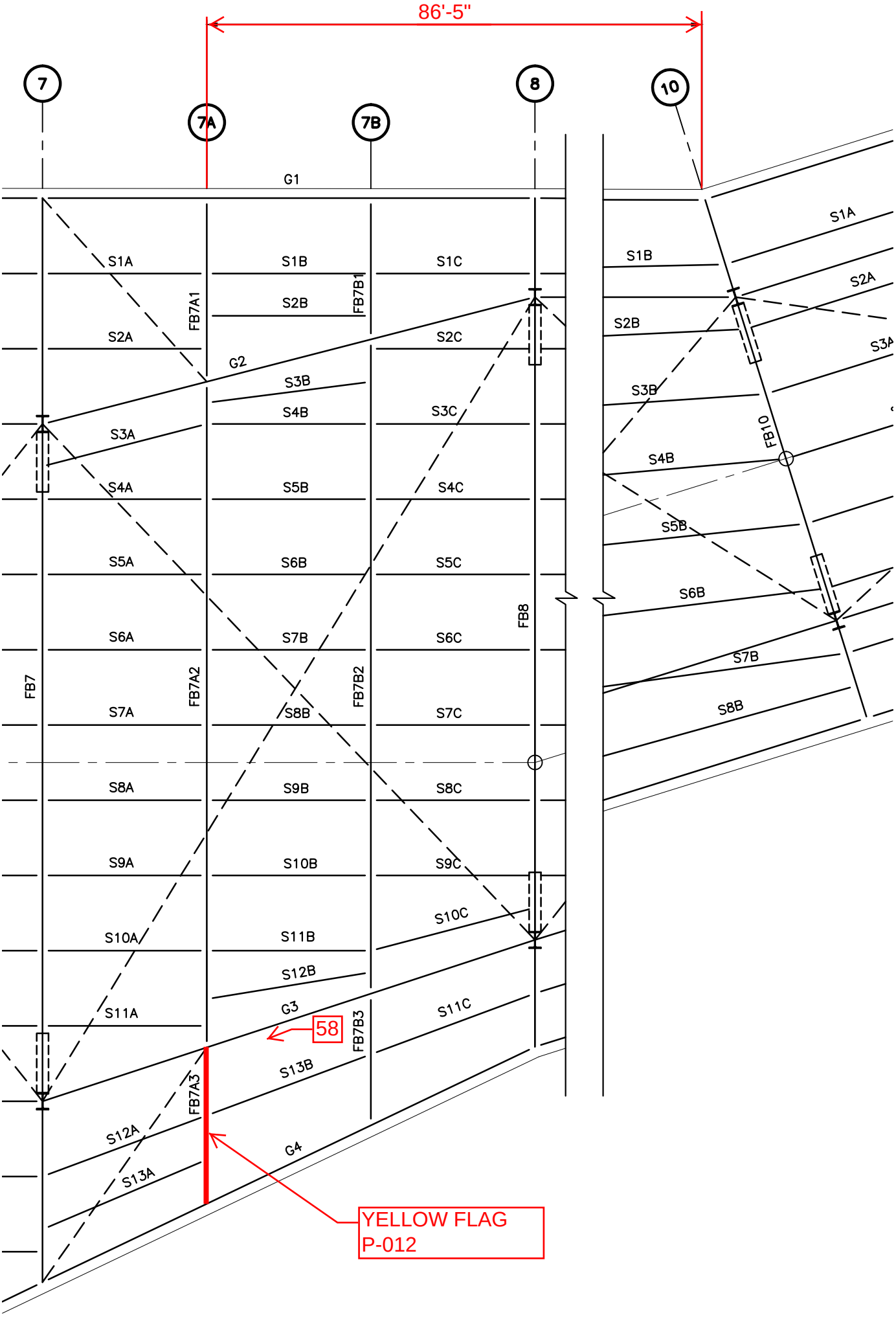
Looking: West
(Close Up)

Description:
Perforation & paper thin in web near connection to Girder G3.
Total web loss is 40%.

Reference:
YELLOW FLAG P-012

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S12B
S13A
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FB7A1
FB7A2
FB7A3
FB7B1
FB7B2
FB7B3
FB8
FB10
58
YELLOW FLAG
P-012

HAKS

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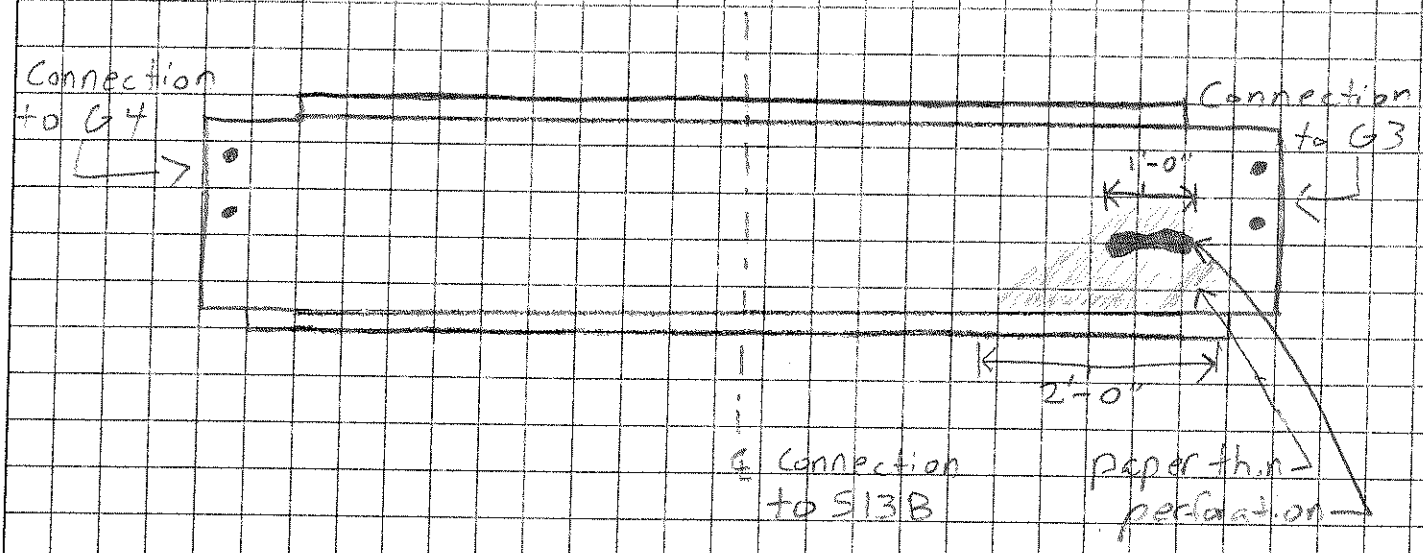
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-012

Span 8. Floor Beam FB-A3



Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR ***Aleksandr Novikov, P.E.***
FLAG NUMBER ***P-013***
DATE OF INSPECTION ***10 / 8 / 2013***
SUPERSEDES FLAG NO. ***NONE***

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN ***N/A*** REGION ***11*** COUNTY ***NEW YORK*** TOWN ***NEW YORK***

FEATURES: CARRIED ***PASSERELLE PED. BRIDGE*** CROSSED ***LIRR***

NUMBER OF SPANS BY TYPE ***29 Steel*** APPROXIMATE YEAR BUILT ***1964***

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: ***27*** COMPONENT: ***N/A***

RATING: ***2***

Location: Stringer along Column Line G at Span 27

Member: Span Item [28] Primary members (Stringer)

Description: The 46'-0" long stringer exhibits 60% loss of the web for most of the span and up to 80% loss at approximately one-third of the span. There are also some through holes located near a diaphragm connection at about one-third of the span. This beam is currently carrying little load. There is no structural distress observed in the deck or surrounding structure.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER ***4***

Flagged Bridge Report Completed By ***Aleksandr Novikov , P.E.*** D ***10 / 18 / 2013***

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** 1 **OF** 2

PHOTO NO. **P013-01**

Location:
Stringer along Col. Line G
Span 27
Looking Northwest

Description:
Total 50% corrosion of
stringer web at beginning
of span

Reference:
YELLOW FLAG P-013

ASST. TEAM LEADER: Tim Viani

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P013-02**

Location:
Stringer along Col. Line G
Span 27
Looking Northwest (Close-up)

Description:
Total 80% corrosion of
stringer web at one-third span

Reference:
YELLOW FLAG P-013

TEAM LEADER: A. Novikov, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

PHOTO NO. **P013-03**

Location:
Stringer along Col. Line G
Span 27
Looking North

Description:
Through holes in
stringer web at one-third
to midspan

Reference:
YELLOW FLAG P-013

ASST. TEAM LEADEf Tim Viani

FEATURE UNDER: LIRR property and tracks

PHOTO NO. **P013-04**

Location:
Stringer along Col. Line G
Span 27
Looking North (Close-up)

Description:
Through holes in
stringer web at one-third
of span

Reference:
YELLOW FLAG P-013

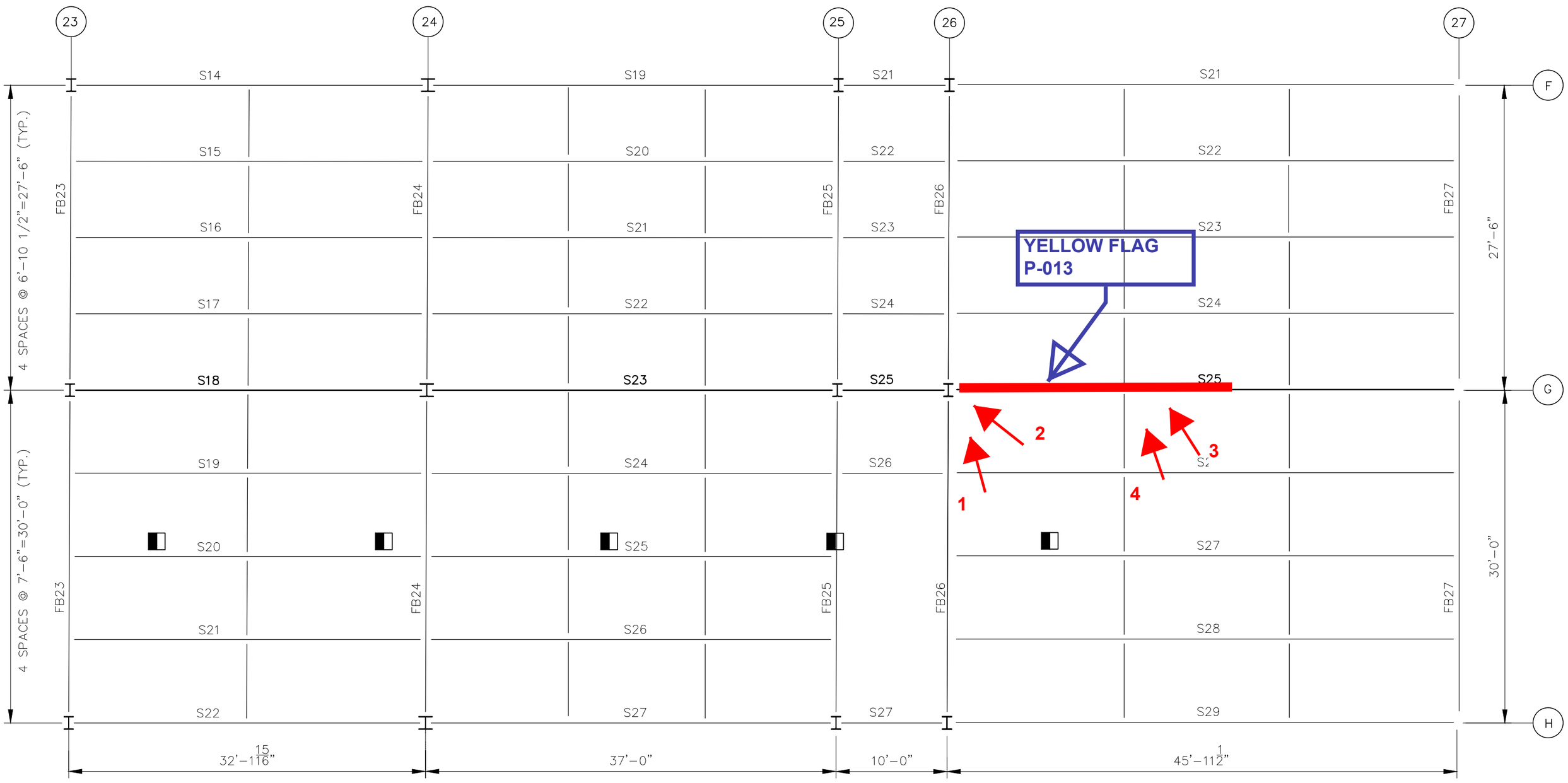
TEAM LEADER: A. Novikov, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge

LEGEND:

EXISTING MEMBER TO REMAIN — SXX

MEMBER TO BE REPAIRED/REPLACED — SXX



FLAG LOCATION PLAN

HAKS

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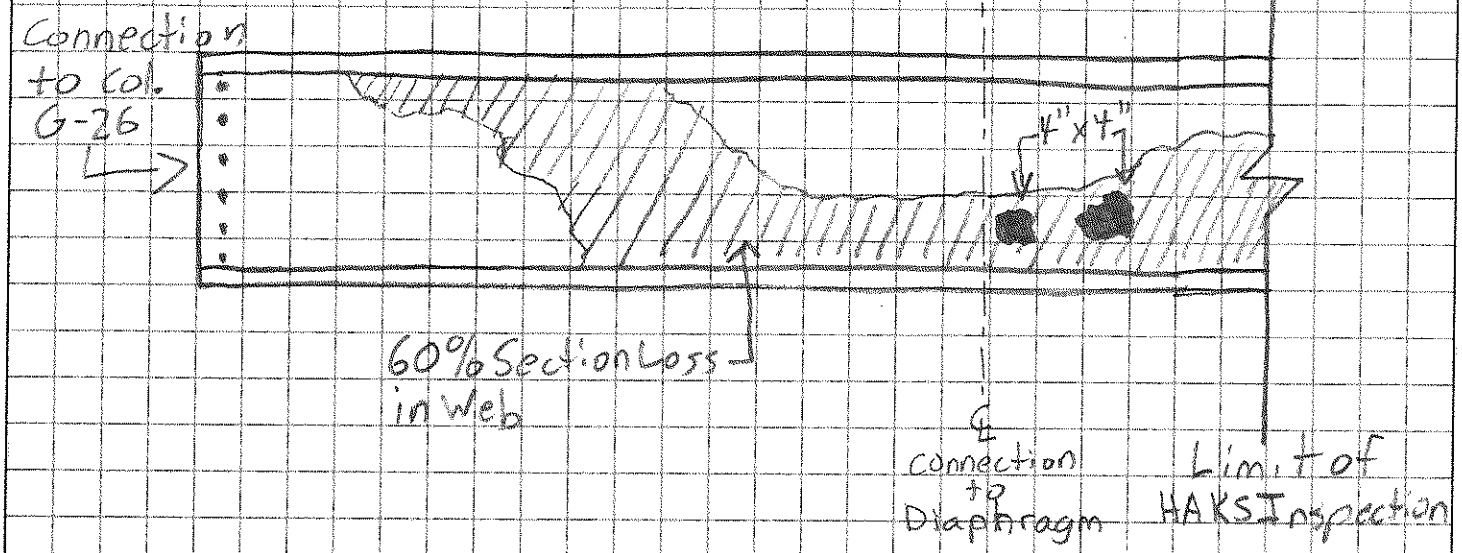
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-013

Span 27 - Stringer S25



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-014*
DATE OF INSPECTION *10 / 16 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *11* COMPONENT: *N/A*

RATING: _____

Location: Span 11

Member: Stringer S6B between Floorbeam FB10A & FB10B

Description: The redundant Stringer S6B exhibits extreme corrosion with large perforation and paper thin in the lower 4" of web along bottom flange for 12' long starting from 3' away from Floorbeam FB10A. The remaining web shows up to 10% section loss. Also, the flanges exhibit up to 15% section loss with broken 1/8" of top flange edge through out. The worst total section loss at 6' away from Floorbeam FB10A with 45% web loss and 20% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 24 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0049**

Location: Span 11
Stringer S6B between Floorbeam FB10A & FB10B.

Looking : Northeast.

Description:
Perforation and paper thin on stringer web with broken top flange edge.

Reference:
YELLOW FLAG P-014

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **N/A**

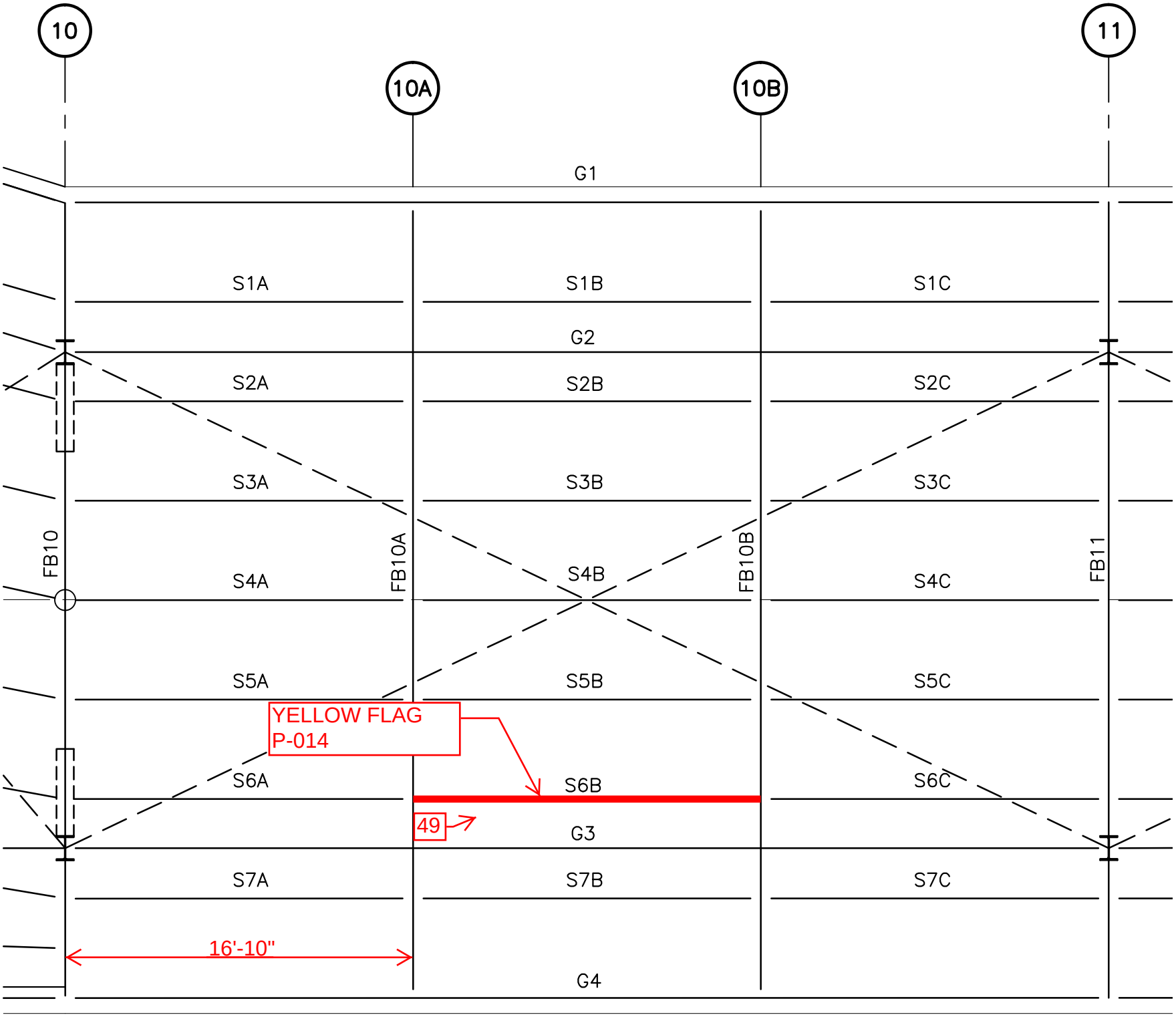
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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10A
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S1C
G2
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S3A
S3B
S3C
FB10
FB10A
FB10B
FB11
S4A
S4B
S4C
S5A
S5B
S5C
YELLOW FLAG
P-014
S6A
S6B
S6C
49
G3
S7A
S7B
S7C
16'-10"
G4

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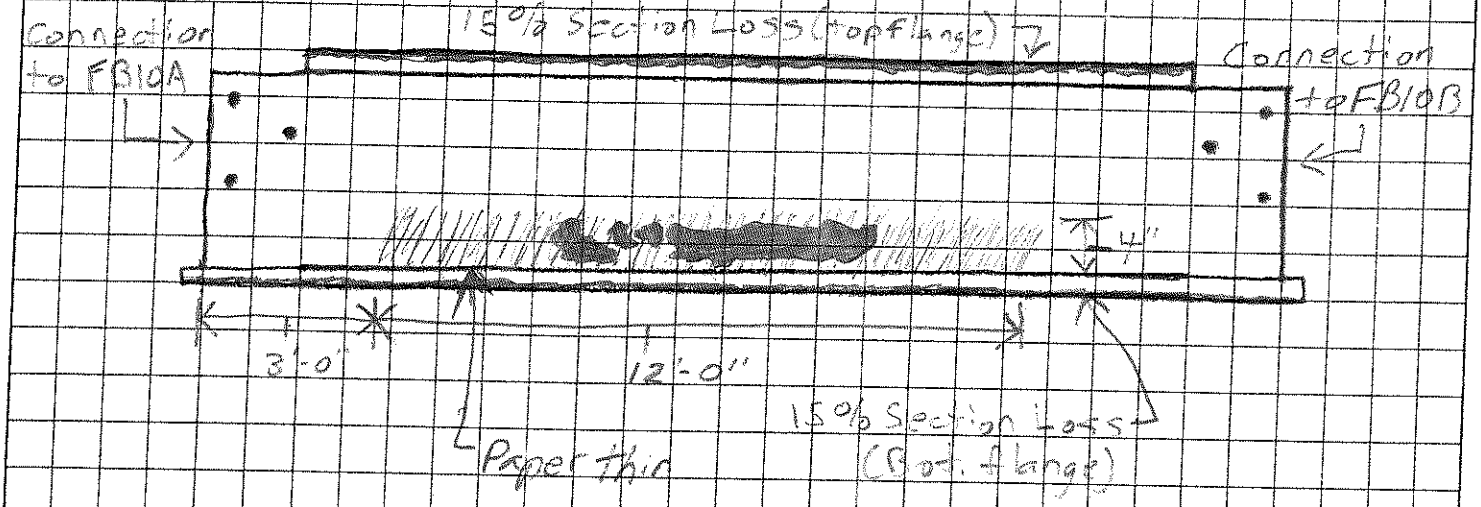
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG A-014

Span 11 - Stringer S6B



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-015*
DATE OF INSPECTION *10 / 16 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *12* COMPONENT: *N/A*

RATING: _____

Location: Span 12

Member: Stringer S6B between Floorbeam FB11A & FB11B

Description: The redundant Stringer S6B exhibits extreme corrosion with large perforation and paper thin in the lower 3" of web along bottom flange through out. Remaining of web shows up to 10% section loss. Also, the top flange exhibits up to 50% section loss with broken 1" edge through out. The bottom flange has less than 5% section loss. The worst total section loss is at 2' away from Floorbeam FB11B with 35% web loss and 30% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 24 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0045**

Location: Span 12
Stringer S6B between Floorbeam FB11A & FB11B.

Looking : Southwest.

Description:
Perforation and paper thin on stringer web with broken top flange edge.

Reference:
YELLOW FLAG P-015

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **N/A**

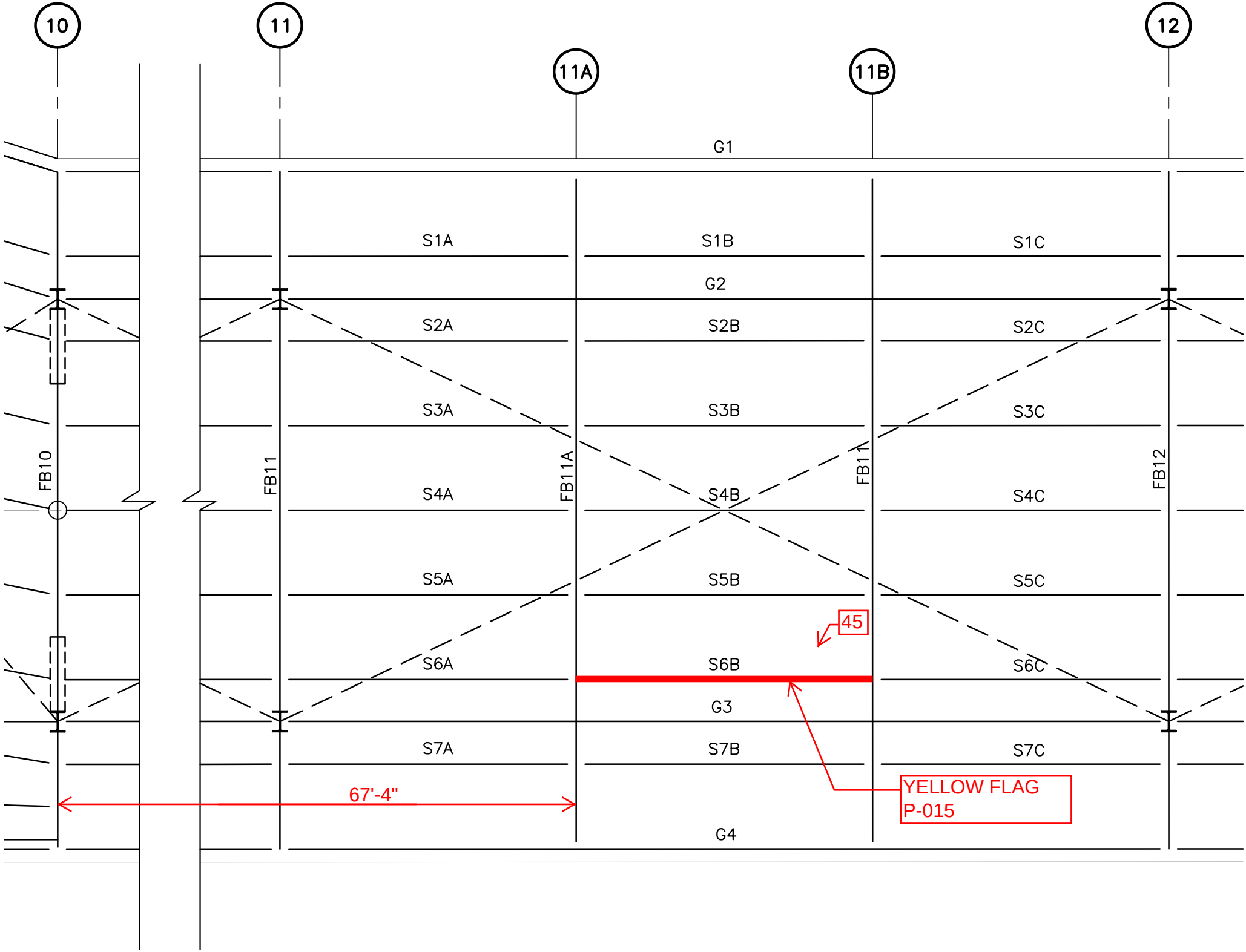
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S7B
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G4
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FB11
FB11A
FB11
FB12
45
67'-4"
YELLOW FLAG
P-015

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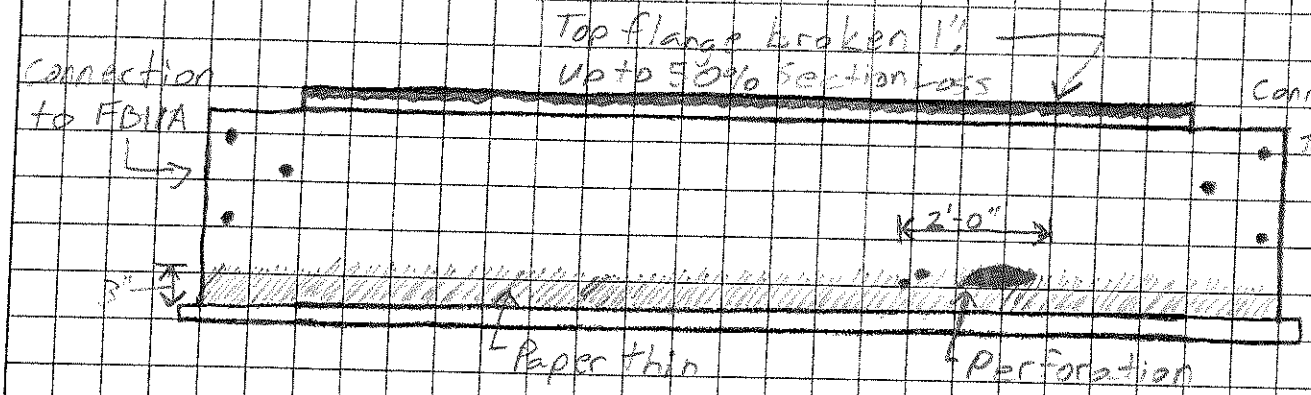
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-015

Span 12 - Stringer 5GB



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-016*
DATE OF INSPECTION *10 / 15 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *13* COMPONENT: *N/A*

RATING: _____

Location: Span 13

Member: Stringer S7C between Floorbeam FB12B & FB13

Description: The redundant Stringer S7C exhibits extreme corrosion with large perforation and paper thin in the lower 4" of web along bottom flange for 4' long starting from 5' away from Floorbeam FB13. Remaining web shows up to 20% section loss. Also, the top flange exhibits a range of 5% to 20% section loss with up to 1/8" broken edge through out. Bottom flange shows up to 5% section loss. The worst total section loss is at 8' away from Floorbeam FB13 with 50% web loss and 15% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 24 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0044**

Location: Span 13
Stringer S7C between Floorbeam FB12B & FB13.

Looking : South

Description:
Perforation and paper thin on stringer web with broken top flange edge.

Reference:
YELLOW FLAG P-016

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **N/A**

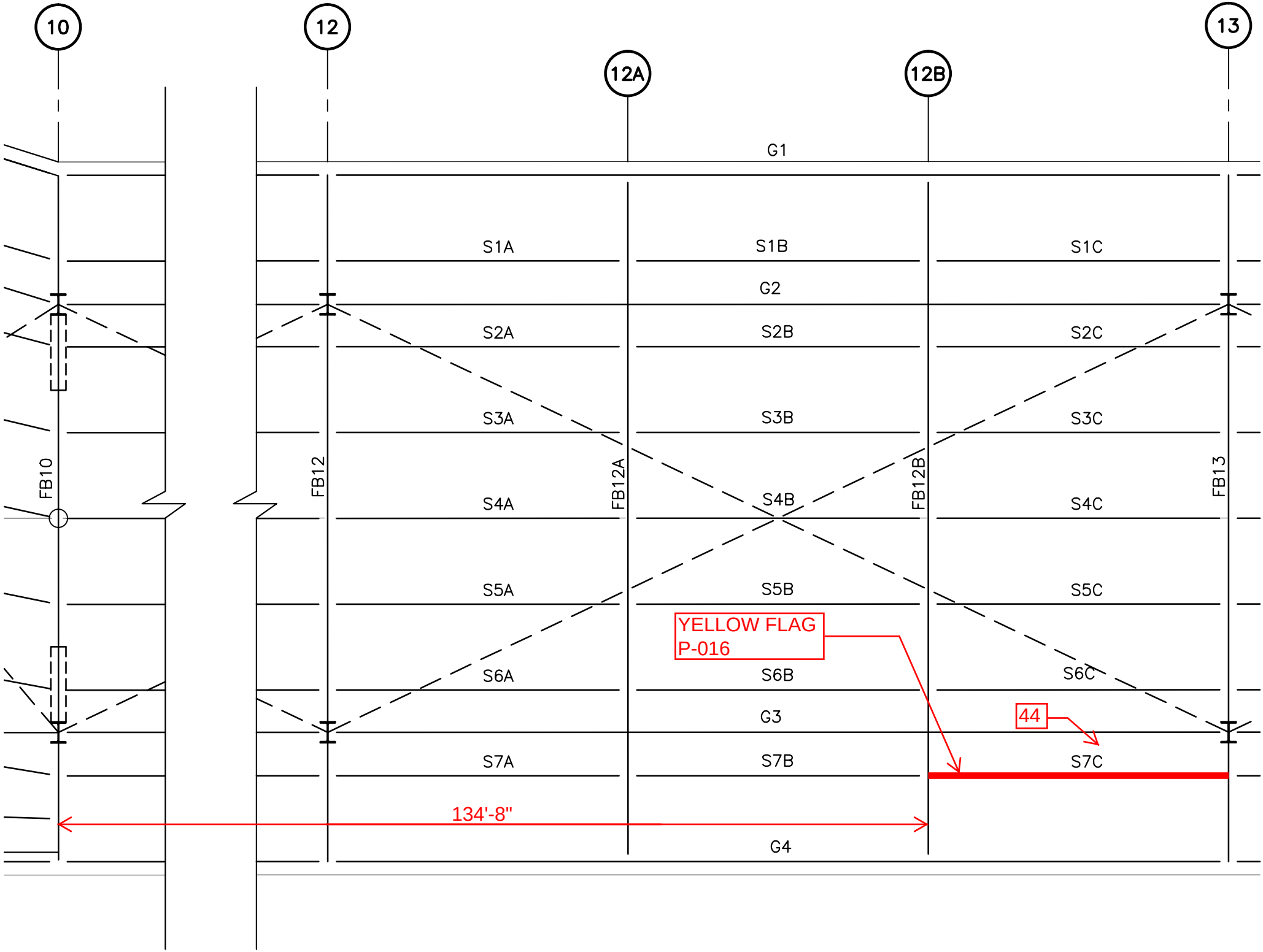
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S4A
S4B
S4C
S5A
S5B
S5C
S6A
S6B
S6C
G3
S7A
S7B
S7C
G4
FB10
FB12
FB12A
FB12B
FB13
YELLOW FLAG
P-016
44
134'-8"

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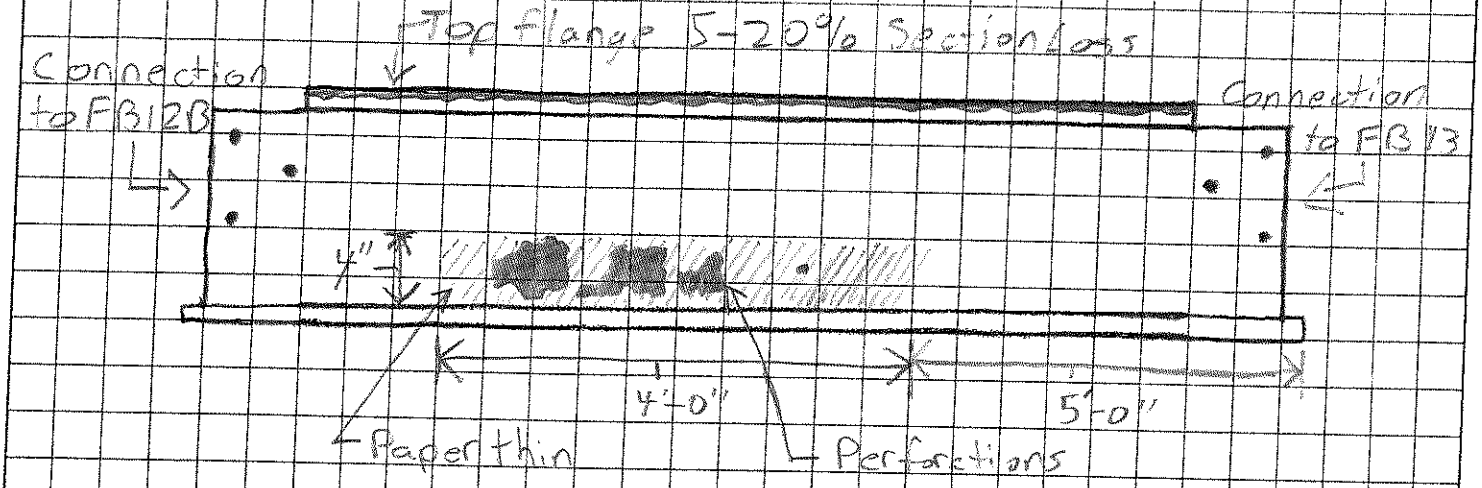
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-016

Span 13 - Stringer S7C



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-017*
DATE OF INSPECTION *10 / 14 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *15* COMPONENT: *N/A*

RATING: _____

Location: Span 15

Member: Stringer S7C between Floorbeam FB14B & FB15

Description: The redundant Stringer S7C exhibits extreme corrosion with large perforation and paper thin in the lower 2.5" of web along bottom flange for 12' long starting 3' away from Floorbeam FB15. Remaining web shows up to 20% section loss. Also, the top flange exhibits up to 20% section loss with up to 1/2" broken edge locally. Bottom flange shows up to 5% section loss. The worst total section loss is at 3' away from Floorbeam FB15 with 30% web loss and 20% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 24 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0036**

Location: Span 15
Stringer S7C between Floorbeam FB14B & FB15.

Looking : South-West

Description:
Perforation and paper thin on stringer web with broken top flange edge.

Reference:
YELLOW FLAG P-017

PHOTO NO. **N/A**

Location:

Description:

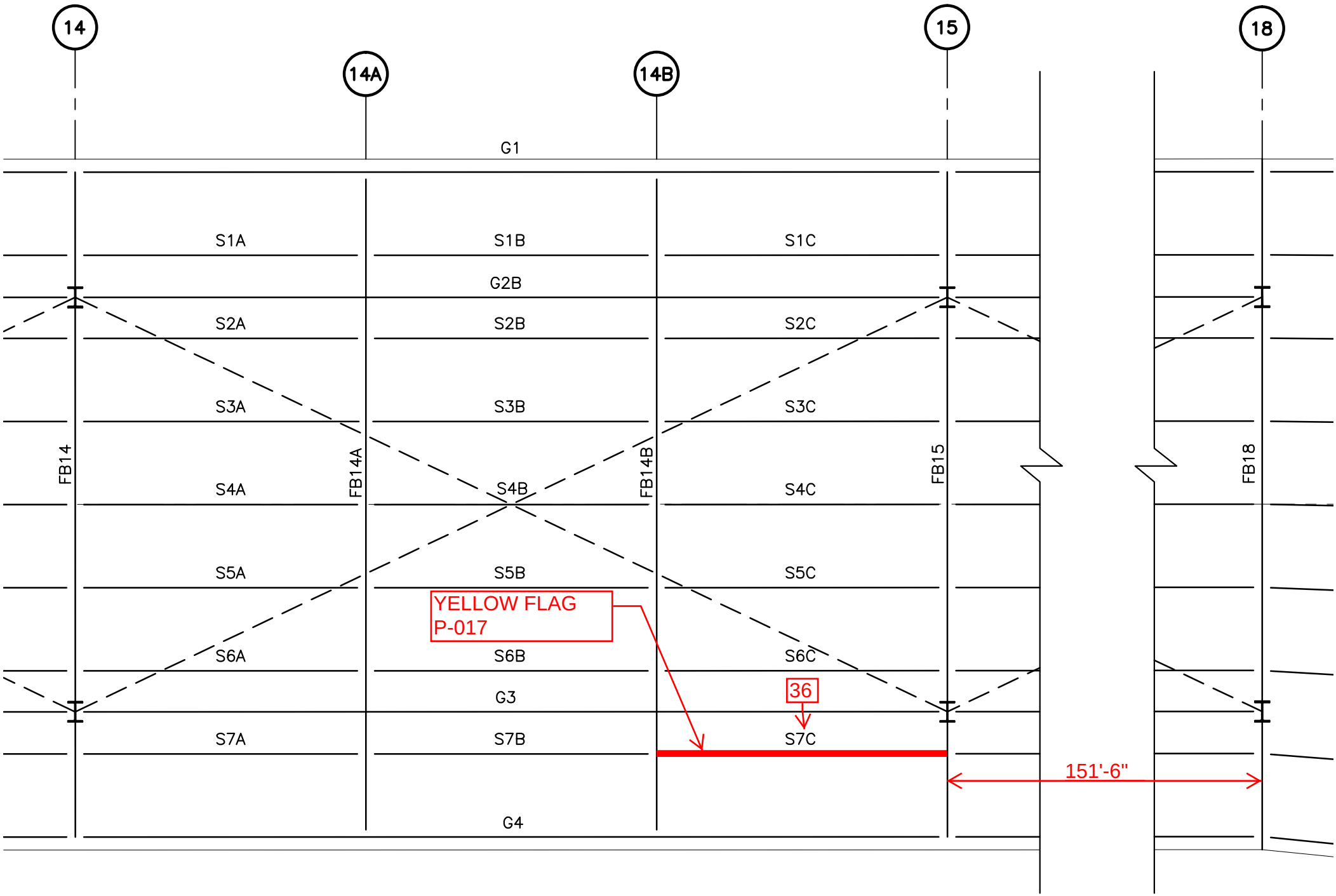
Reference:

ASST. TEAM LEADER: NYCT Yard

FEATURE UNDER:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge


14
14A
14B
15
18
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S1A
S1B
S1C
G2B
S2A
S2B
S2C
S3A
S3B
S3C
FB14
FB14A
FB14B
FB15
FB18
S4A
S4B
S4C
S5A
S5B
S5C
YELLOW FLAG
P-017
S6A
S6B
S6C
36
G3
S7A
S7B
S7C
151'-6"
G4

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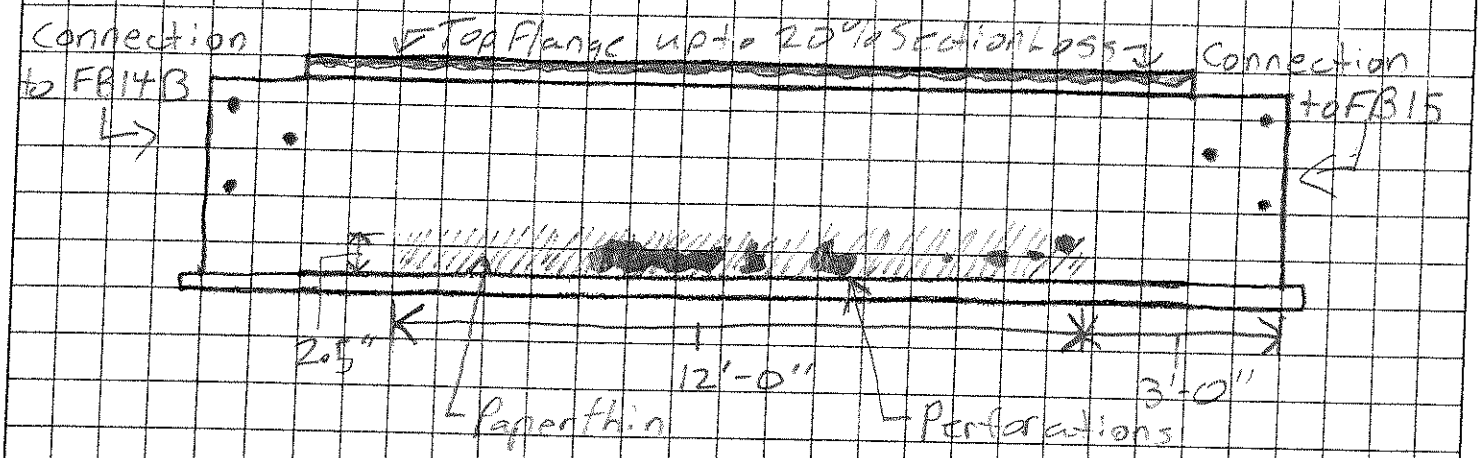
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-017

Span 15 - Stringer S7C



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-018*
DATE OF INSPECTION *10 / 18 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *16* COMPONENT: *N/A*

RATING: _____

Location: Span 16

Member: Stringer S5A between Floorbeams FB15 & FB15A

Description: The redundant Stringer S5A exhibits extreme corrosion with perforations in size of 2'L x 2"H & 6"H x 2"H, and, paper thin in the lower 2.5" of web along bottom flange for most of the member's length. Remaining web shows up to 5% section loss. Also, the flanges exhibit up to 15% section loss with up to 1/2" broken top flange at mid-span. The worst section loss is at mid-span with 30% web loss and 20% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 25 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0070**

Location: Span 16
Stringer S5A between Floorbeam FB15 &FB15A

Looking : Northwest

Description:
Perforation and paper thin on stringer web along bottom flange, and, broken edge on top flange.

Reference:
YELLOW FLAG P-018

ASST. TEAM LEADER:
FEATURE UNDER: NYCT Yard

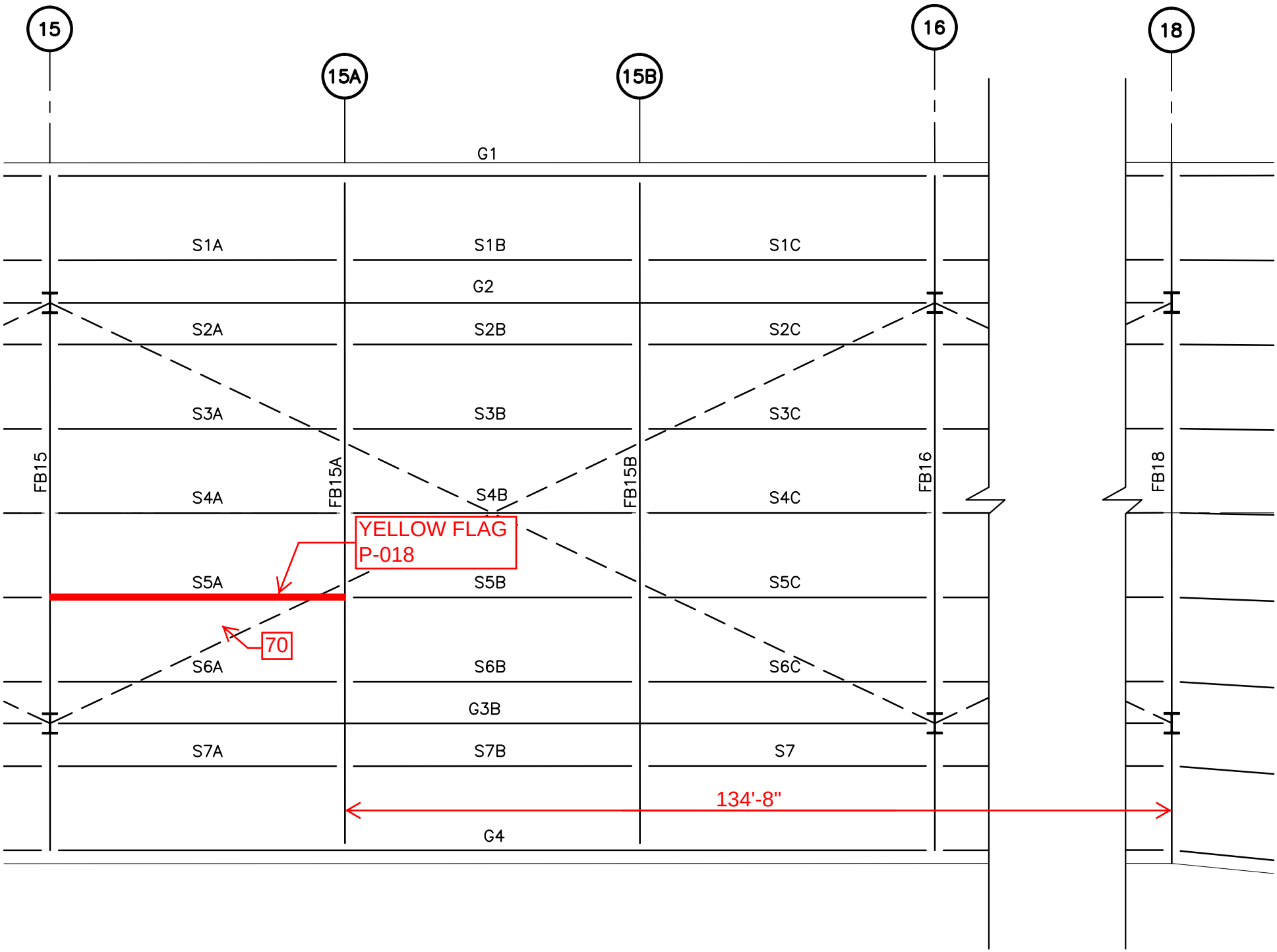
PHOTO NO. **N/A**

Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.
FEATURE CARRIED: Passerelle Pedestrian Bridge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FB15A
FB15B
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FB18
S4A
S4B
S4C
YELLOW FLAG
P-018
S5A
S5B
S5C
70
S6A
S6B
S6C
G3B
S7A
S7B
S7
134'-8"
G4

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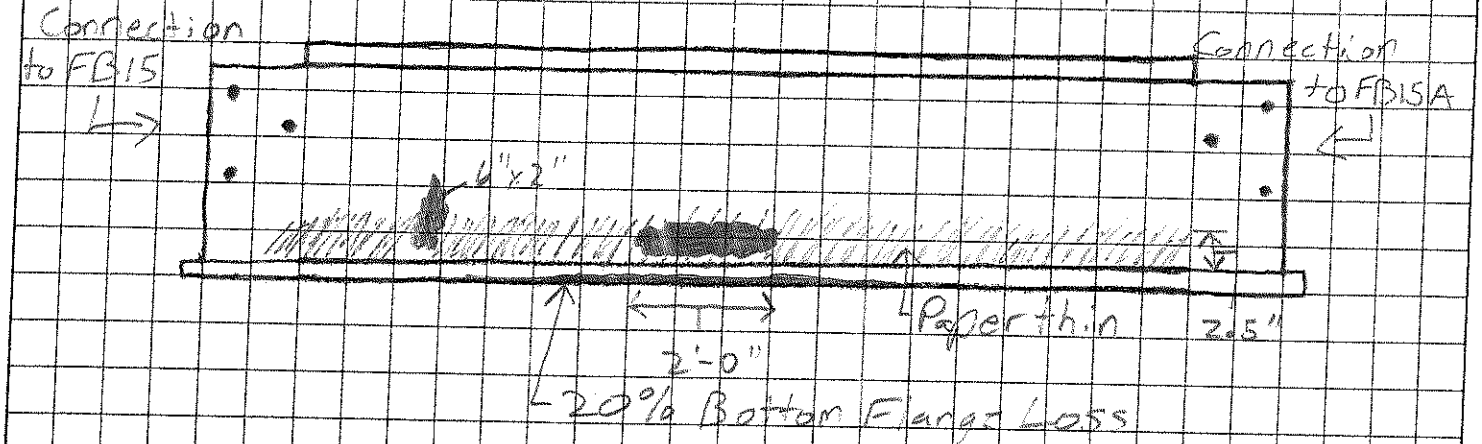
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-018

Span 16 - Stringer S5A



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-019*
DATE OF INSPECTION *10 / 14 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *16* COMPONENT: *N/A*

RATING: _____

Location: Span 16

Member: Stringer S6A between Floorbeams FB15 & FB15A

Description: The redundant Stringer S6A exhibits extreme corrosion with large perforation and paper thin in the lower 4" of web along bottom flange for 4' long near the center of the span. Another corroded hole of 1.5" diameter is noted on the web below the connection to FB15A. Also, both flanges exhibit 15% section loss through out. The worst cross section loss is at mid-span with 40% web loss and 15% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *2*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 25 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0035**

Location: Span 16
Stringer S6A between Floorbeam FB15 &FB15A

Looking : South

Description:
Perforation and paper thin on stringer web with moderate section loss on flanges near center of span.

Reference:
YELLOW FLAG P-019

PHOTO NO. **N/A**

Location:

Description:

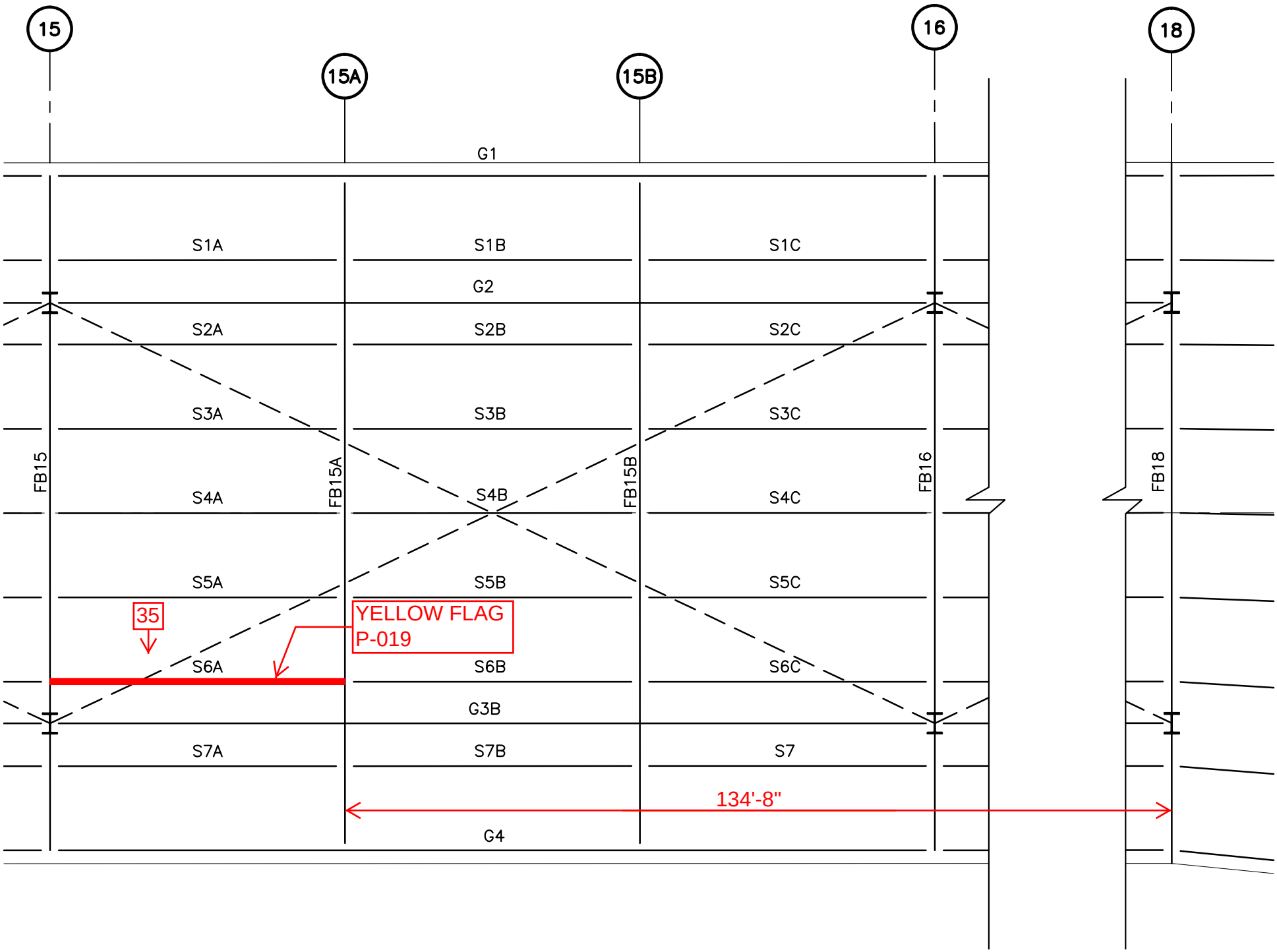
Reference:

ASST. TEAM LEADER: NYCT Yard

FEATURE UNDER:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S4C
S5A
S5B
S5C
S6A
S6B
S6C
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S7A
S7B
S7
G4
FB15
FB15A
FB15B
FB16
FB18
35
YELLOW FLAG
P-019
134'-8"

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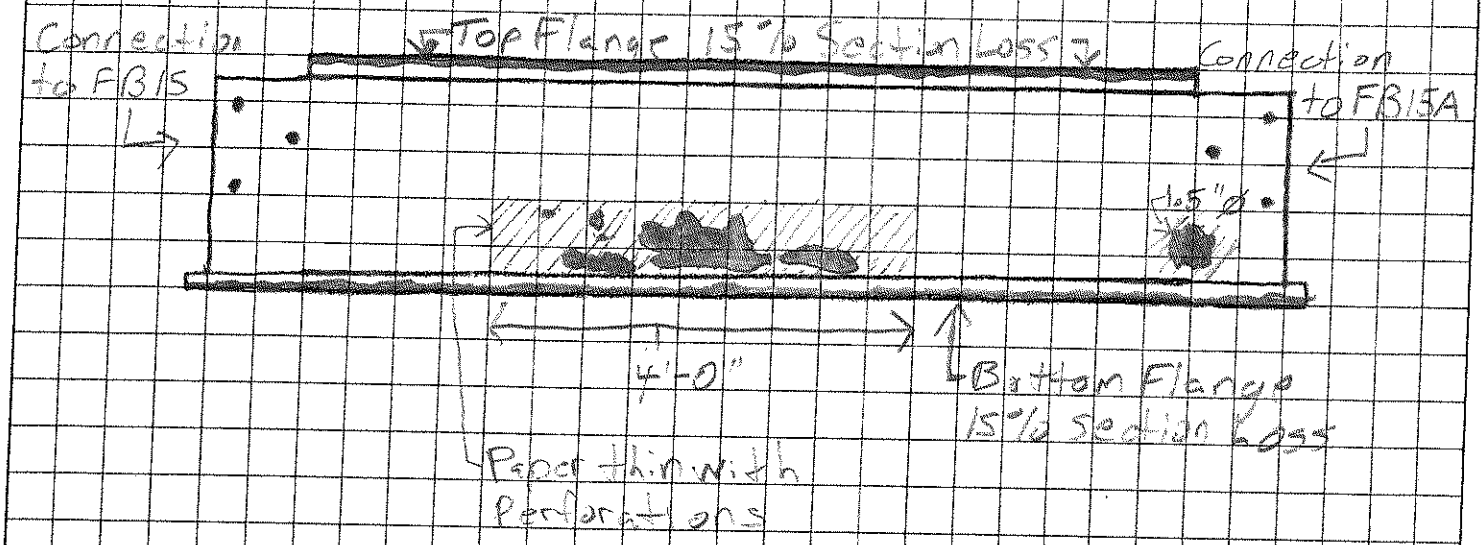
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-019

Span 16 - Stringer S6A



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-020*
DATE OF INSPECTION *10 / 14 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *16* COMPONENT: *N/A*

RATING: _____

Location: Span 16

Member: Stringers S5B and S6B between Floorbeams FB15A & FB15B.

Description: Both redundant Stringers S5B & S6B exhibits extreme corrosion with local perforation and paper thin in the lower 2" of web along bottom flange for upto 3' long starting 1.0' away from either ends. Remaining of web shows average of 5% section loss. Also, the top flanges exhibits average of 20% section loss. Bottom flange shows upto 5% section loss. The worst total section loss is at 1.5' away from Floorbeam FB15A with 25% web loss and 12% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 25 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0033**

Location: Span 16
Stringer S6B between Floorbeam FB15A & FB15B.
(Typ. For Stringer S5B between FB15A & FB15B)

Looking : West at connection to FB15A

Description:
Perforation and paper thin on stringer web.

Reference:
YELLOW FLAG P-020

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **N/A**

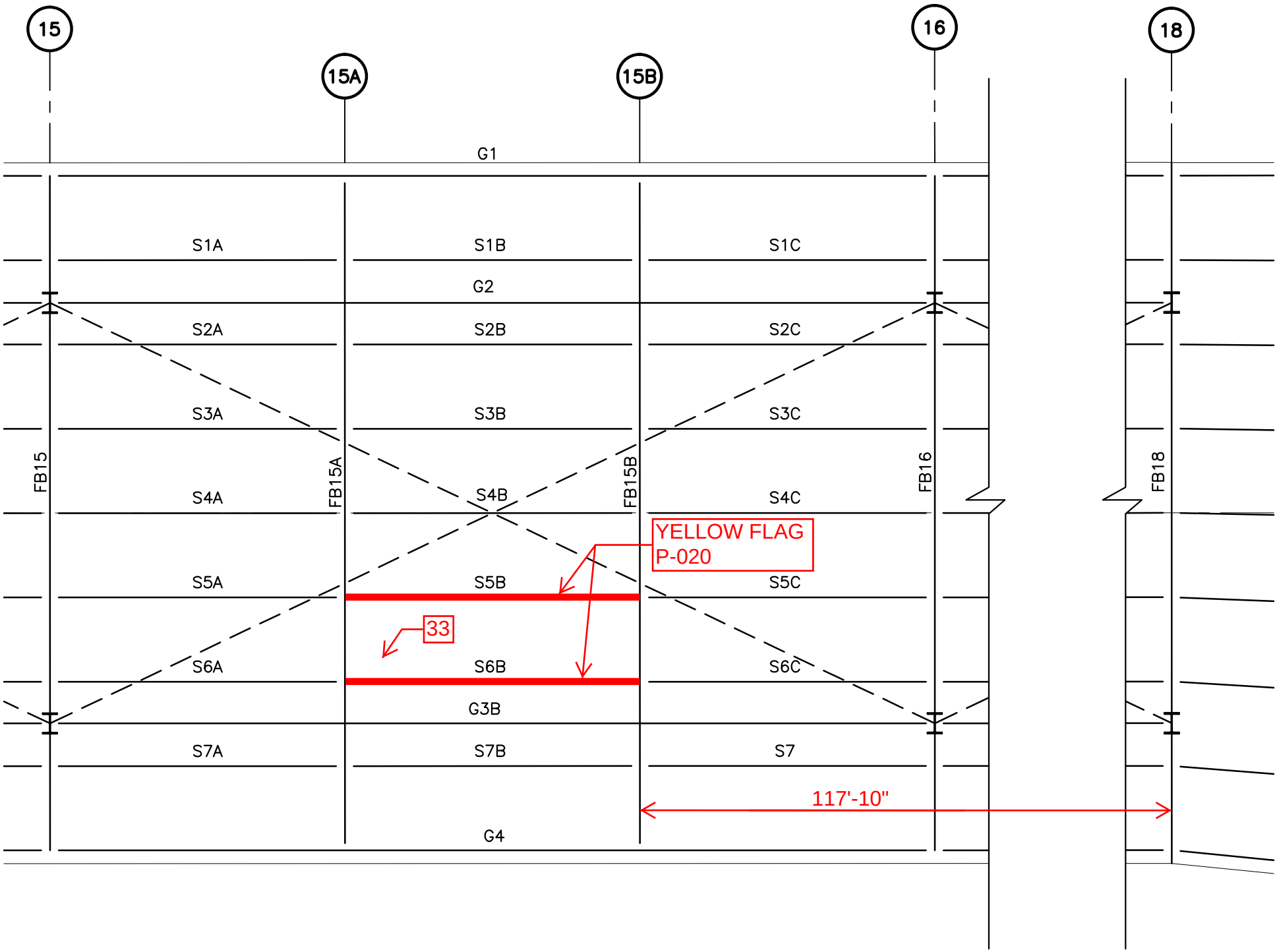
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S4A
S4B
S4C
YELLOW FLAG
P-020
S5A
S5B
S5C
33
S6A
S6B
S6C
G3B
S7A
S7B
S7
117'-10"
G4

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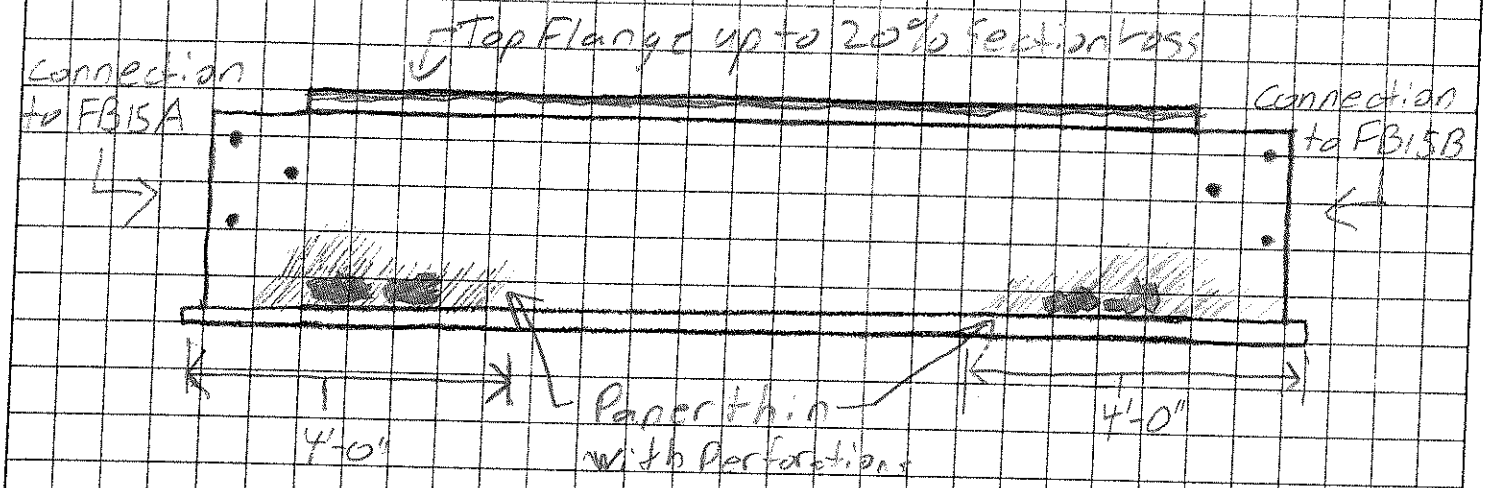
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-020

Span 16 - Stringer S6B (TYP. for S5B)



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-021*
DATE OF INSPECTION *10 / 18 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *17* COMPONENT: *N/A*

RATING: _____

Location: Span 17

Member: Stringers S6A and S7A between Floorbeams FB16 & FB16A.

Description: Both redundant adjacent Stringers S6A & S7A exhibit extreme corrosion near the connection to FB16A. The Stringer S6A exhibits perforation and paper thin in the lower 2" of web along bottom flange for 7' long. Remaining of web shows upto 10% section loss. Also, the top flange exhibits average of 10% section loss with broken 1" edge in 1' long . Bottom flange shows upto 5% section loss. The worst total section loss is at 2.5' away from Floorbeam FB16A with 25% web loss and 10% flange loss. The Stringer S7A also exhibits perforation and paper thin in the lower 1" of web along bottom flange for 4' long, and, the top flange exhibits broken 1/2" edge in 1.5' long. The worst total section is at 3' away from Floorbeam FB16A with 15% web loss and 10% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *2*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 25 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0068**

Location: Span 17
Stringer S6A between Floorbeam FB16 & FB16A.

Looking : South-West

Description:
Perforation and paper thin on stringer web with broken edge of top flange.

Reference:
YELLOW FLAG P-021

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **DSCN0067**

Location: Span 17
Stringer S7A between Floorbeam FB16 & FB16A.

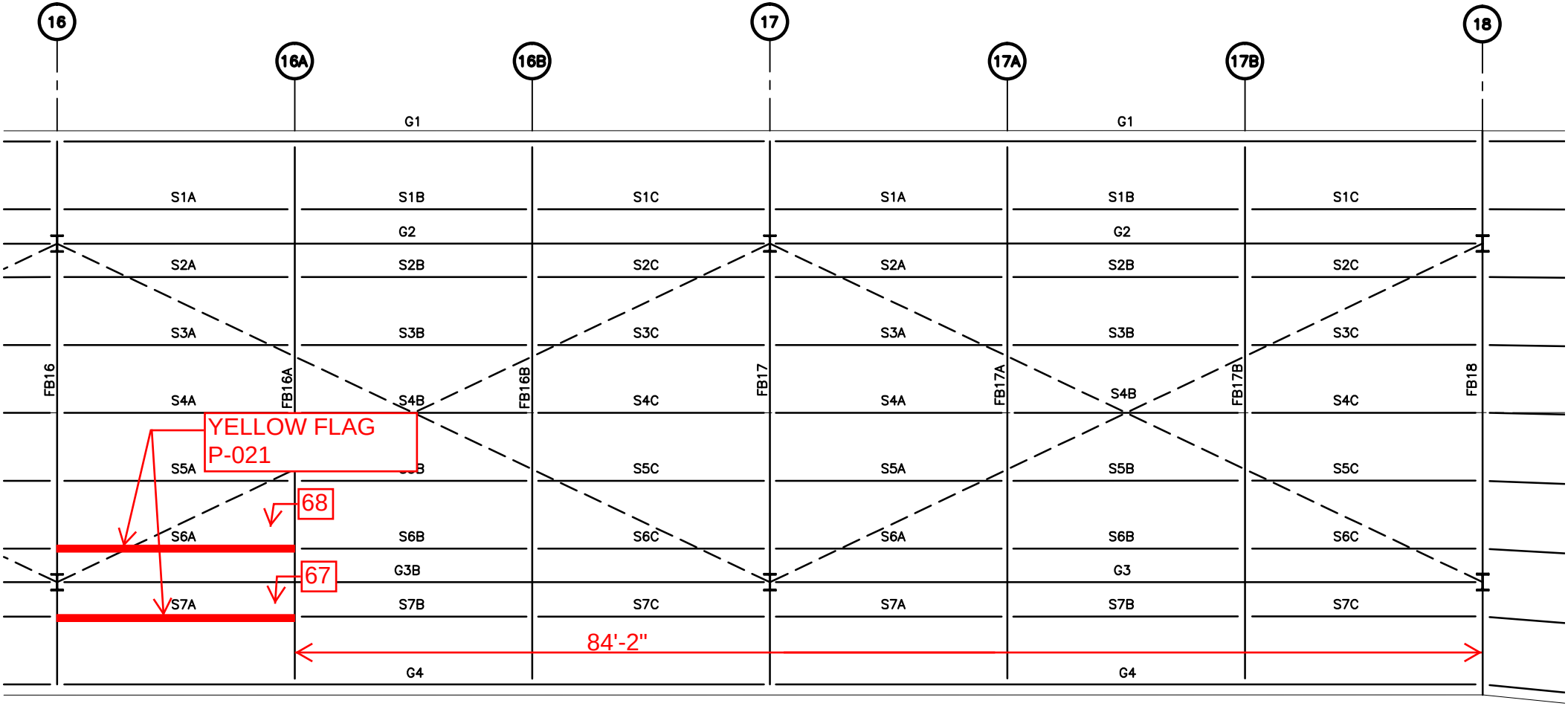
Looking : South-West

Description:
Perforation and paper thin on stringer web with broken edge of top flange.

Reference:
YELLOW FLAG P-021

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S4C
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S4C
YELLOW FLAG
P-021
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S5A
S5B
S5C
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S6A
S6B
S6C
S6A
S6B
S6C
67
G3B
G3
S7A
S7B
S7C
S7A
S7B
S7C
84'-2"
G4
G4

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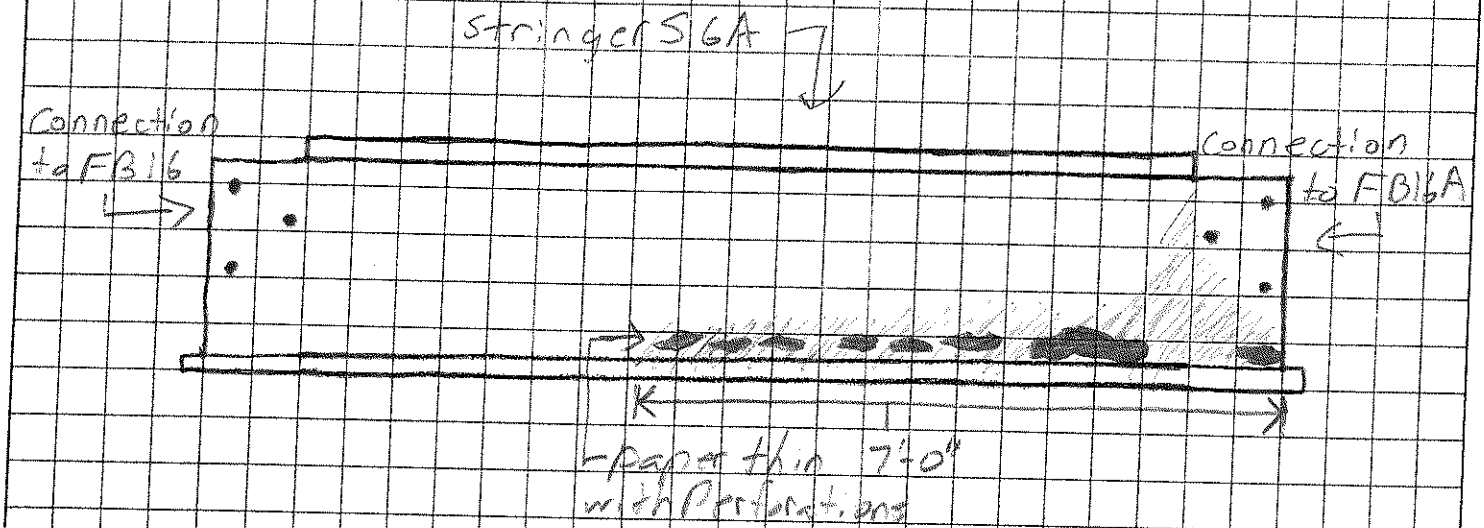
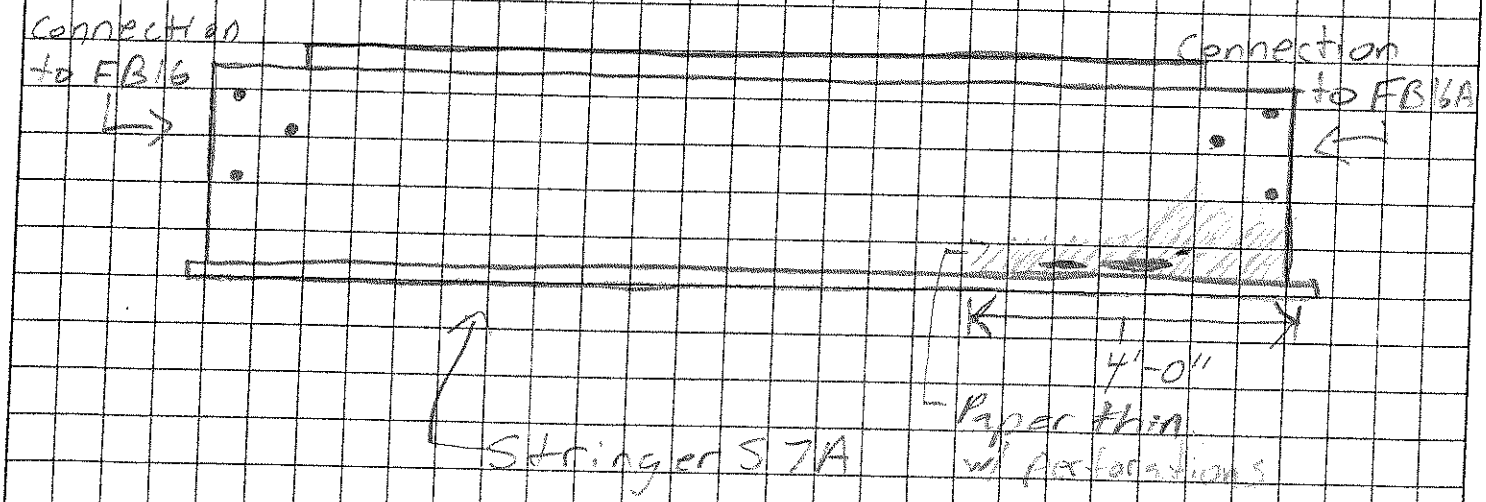
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-021

Span 17 - Stringers S6A + S7A





BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-022*
DATE OF INSPECTION *10 / 11 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *17* COMPONENT: *N/A*

RATING: _____

Location: Span 17

Member: Stringer S6B between Floorbeam FB16A & FB16B.

Description: The redundant Stringer S6B exhibits extreme corrosion with large perforation and paper thin on the lower web along bottom flange in size of 4"L x 3"H at quarter point from FB16A and 2'L x 4"H at quarter point from FB16B. Remaining of web shows upto 10% section loss. Also, the top flange exhibits upto 15% section loss with upto 1" broken edge randomly throughout. The worst cross section loss is at 4' away from Floorbeam FB16B with 45% web loss and 30% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 25 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader Date
or Contract Engineer Representative

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0027**

Location: Span 17
Stringer S6B between Floorbeam FB16A & FB16B.

Looking : West

Description:
Perforation and paper thin on stringer web with broken edge of top flange.

Reference:
YELLOW FLAG P-022

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **N/A**

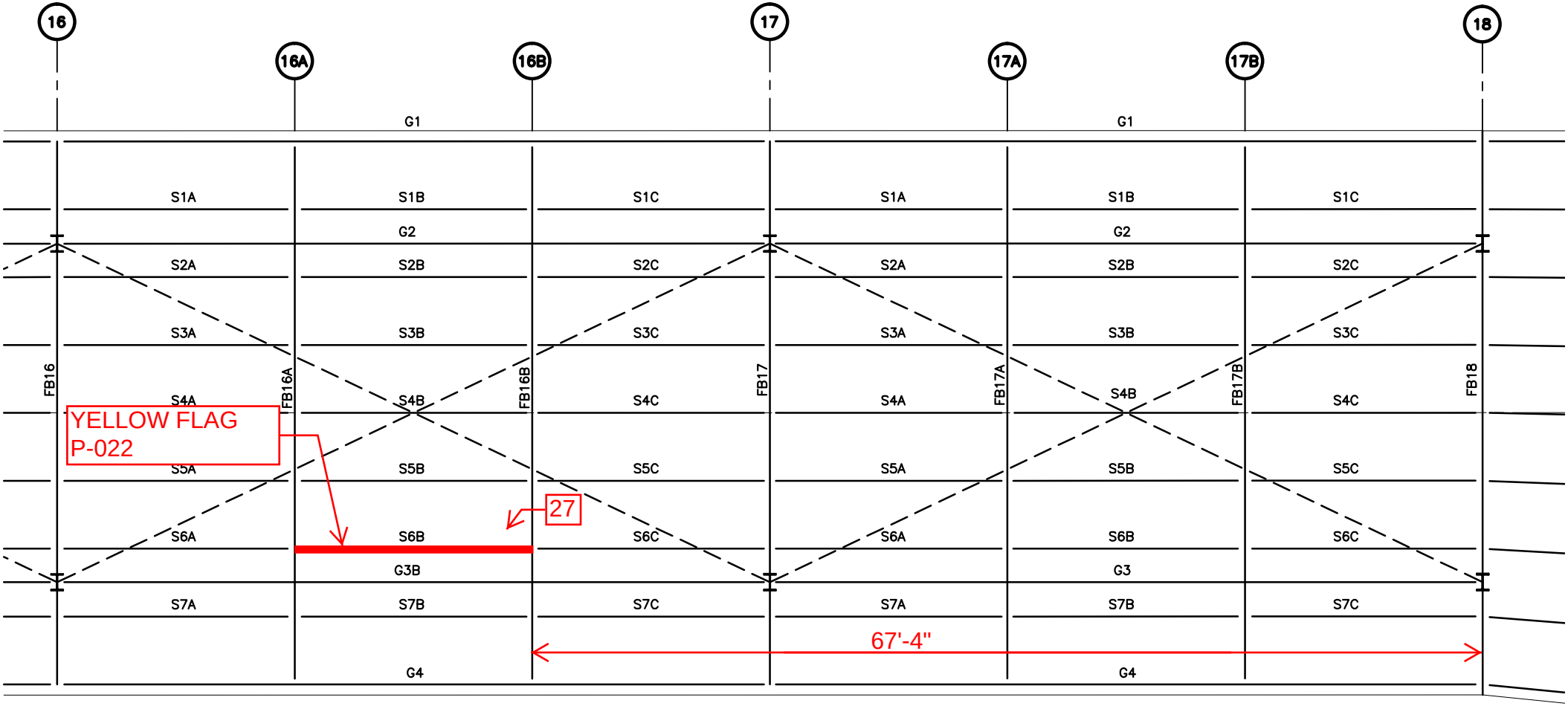
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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S6A
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S6B
S6C
G3B
G3
S7A
S7B
S7C
S7A
S7B
S7C
67'-4"
G4
G4

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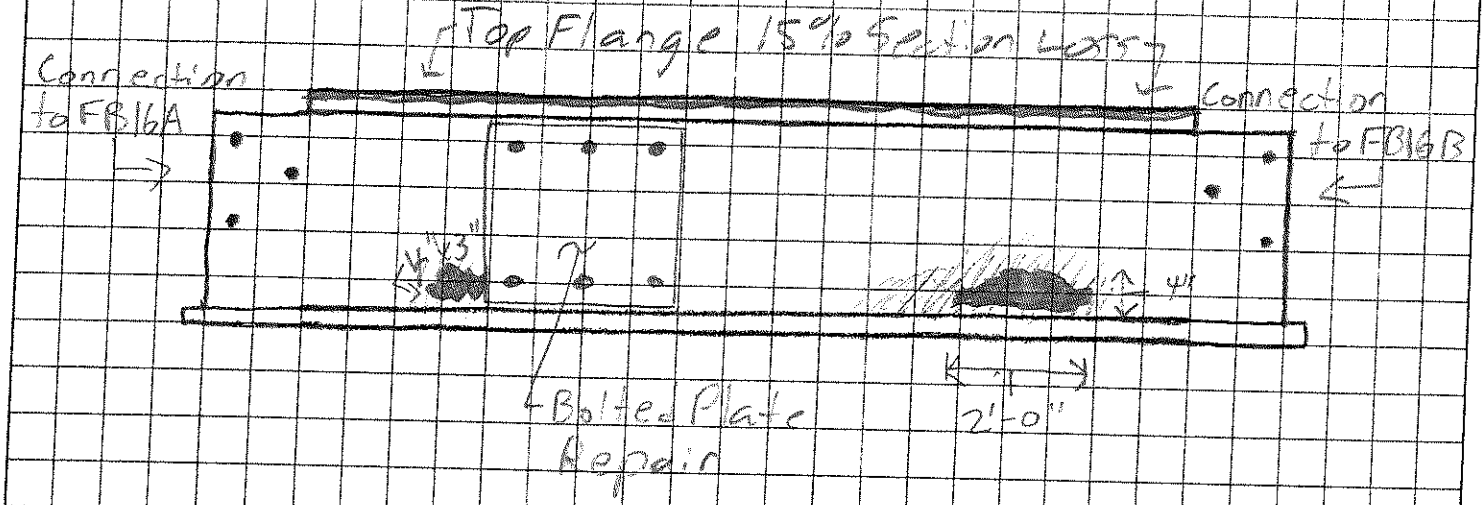
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

FLAG REPORT P-022

Span 17 - Stringer S6B



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-023*
DATE OF INSPECTION *10 / 18 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? _____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *17* COMPONENT: *N/A*

RATING: _____

Location: Span 17

Member: Stringer S7B between Floorbeam FB16A & FB16B.

Description: The redundant Stringer S7B exhibits extreme corrosion with large perforation and paper thin on the lower 3" web along bottom flange for 10' long starting from FB16A. Remaining of web shows upto 15% section loss. Also, the top flange exhibits upto 25% section loss with upto 1/2" broken edge throughout. The worst cross section loss is at connection to Floorbeam FB16A with 40% web loss and 20% flange loss.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 25 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0069**

Location: Span 17
Stringer S7B between Floorbeam FB16A & FB16B.

Looking : Northeast.

Description:
Perforation and paper thin on stringer web with broken edge of top flange.

Reference:
YELLOW FLAG P-023

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **N/A**

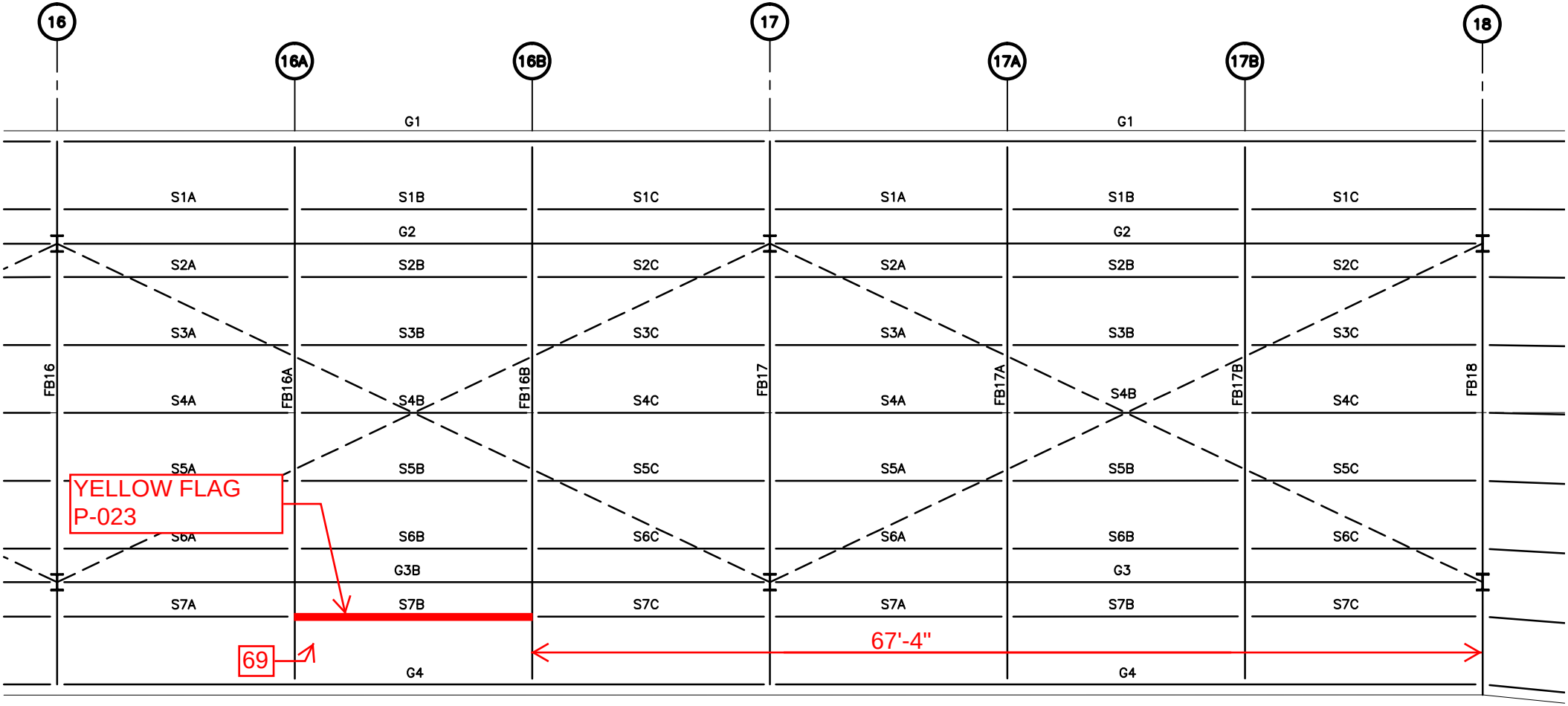
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge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YELLOW FLAG
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67'-4"
69
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G4

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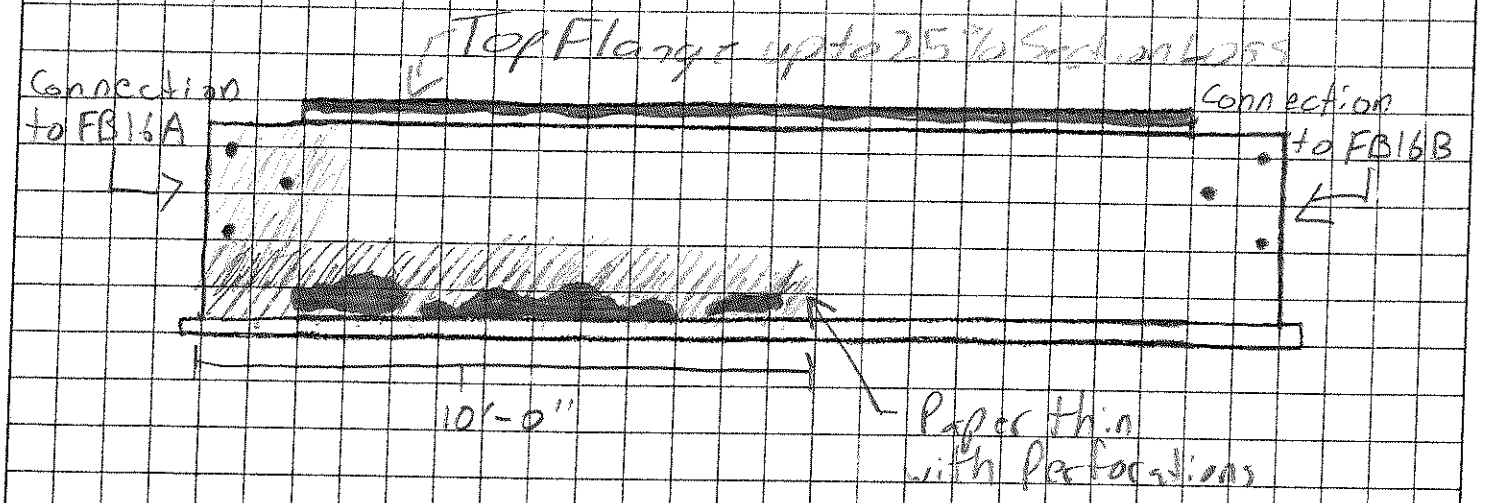
JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-023

Span 17 - Stringer S7B



BD 242 Sheet ____ of _____

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
FLAGGED BRIDGE REPORT

INITIAL:

_____ RED FLAG*
X YELLOW FLAG*
_____ SAFETY FLAG*

INSPECTOR *Mike Perng*
FLAG NUMBER *P-024*
DATE OF INSPECTION *10 / 11 / 2013*
SUPERSEDES FLAG NO. *NONE*

PROMPT INTERIM ACTION RECOMMENDED _____ YES *X* NO

BRIDGE DESCRIPTION:

BIN *N/A* REGION *11* COUNTY *NEW YORK* TOWN *NEW YORK*

FEATURES: CARRIED *PASSERELLE PED. BRIDGE* CROSSED *NYCT*

NUMBER OF SPANS BY TYPE *29 Steel* APPROXIMATE YEAR BUILT *1964*

POSTED FOR LOAD *X* NO _____ YES _____ TONS

IS BRIDGE WHOLLY OR PARTIALLY STATE OWNED? ____ YES *X* NO

DESCRIPTION OF FLAGGED CONDITION (Be specific as to exact nature and location of problem. include direction of orientation):

SPAN NO: *18* COMPONENT: *N/A*

RATING: _____

Location: Span 18

Member: Stringer S4A between Floorbeam FB17 & FB17A.

Description: The redundant Stringer S4A exhibits extreme corrosion with large perforation and paper thin on the lower 3" web along bottom flange throughout. Remaining of web shows 25% section loss. Also, the top flange exhibits upto 30% section loss with upto 1/4" broken edge throughout. Bottom flange shows 10% section loss as well. The cross section loss has 45% web loss and 25% flange loss through out.

PHOTOS ATTACHED? *X* YES ____ NO IF YES, NUMBER *1*

Flagged Bridge Report Completed By *Mike Perng* Date *10 / 25 / 2013*

VERBAL NOTIFICATION: (For Red Flags and Safety Flags with PIA only)

To _____ of Regional Office on _____ at _____ O'clock

To _____ (Responsible Party) on _____ at _____ O'clock

By _____

_____ _____
Signature of State Team Leader or Contract Engineer Representative — Date

* The appropriate caption in the upper left of this form shall be initialed by the individual who is the signatory.

BRIDGE INSPECTION REPORT **BIN:** _____ **SHEET** _____ **OF** _____

PHOTO NO. **DSCN0022**

Location: Span 18
Stringer S4A between Floorbeam FB17 & FB17A.

Looking : North

Description:
Perforation and paper thin on stringer web with corroded flanges and broken top flange edge.

Reference:
YELLOW FLAG P-024

ASST. TEAM LEADER:

FEATURE UNDER: NYCT Yard

PHOTO NO. **N/A**

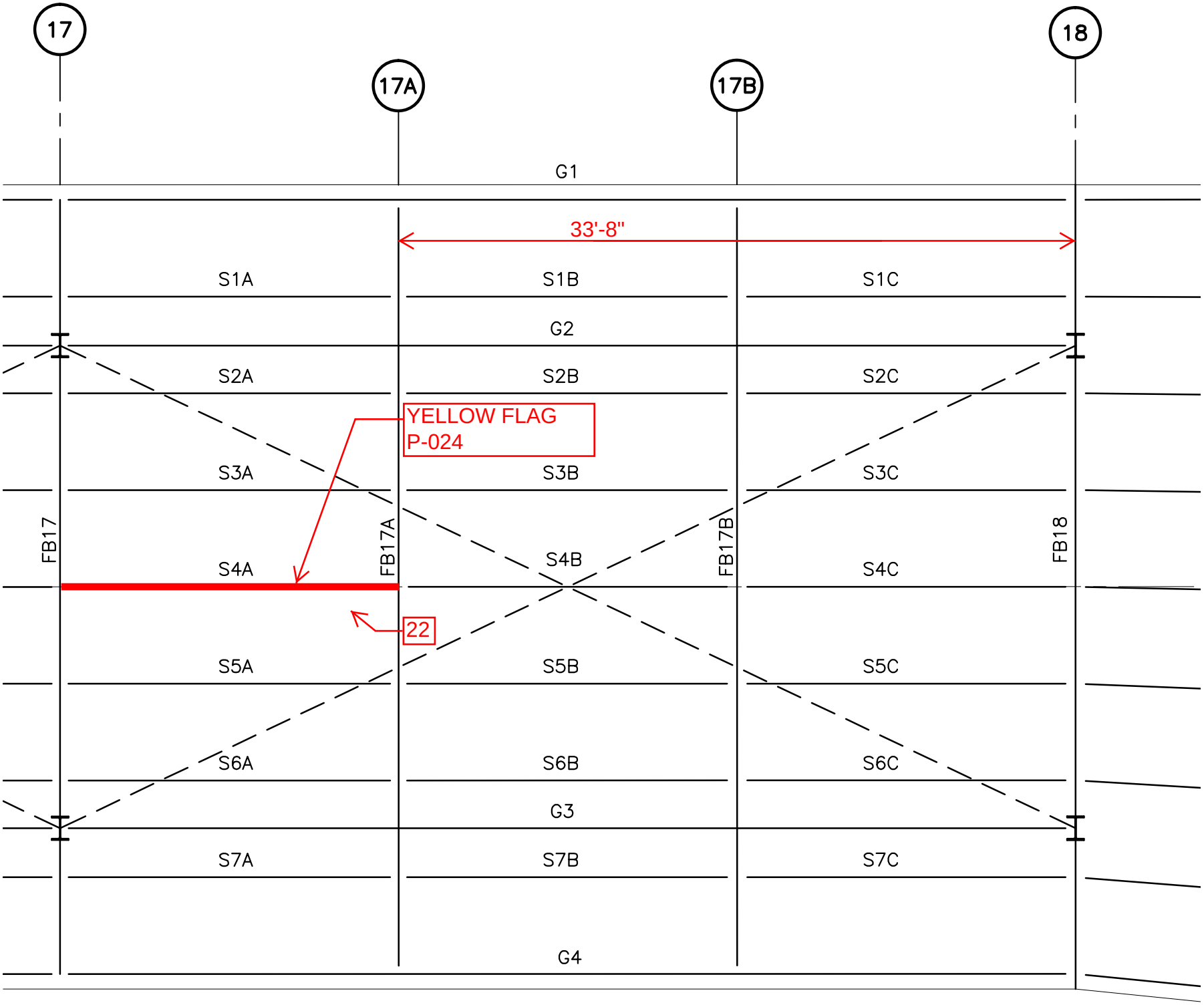
Location:

Description:

Reference:

TEAM LEADER: Mike Perng, P.E.

FEATURE CARRIED: Passerelle Pedestrian Bridge


17
17A
17B
18
G1
33'-8"
S1A
S1B
S1C
G2
S2A
S2B
S2C
YELLOW FLAG
P-024
S3A
S3B
S3C
FB17
FB17A
FB17B
FB18
S4A
S4B
S4C
22
S5A
S5B
S5C
S6A
S6B
S6C
G3
S7A
S7B
S7C
G4

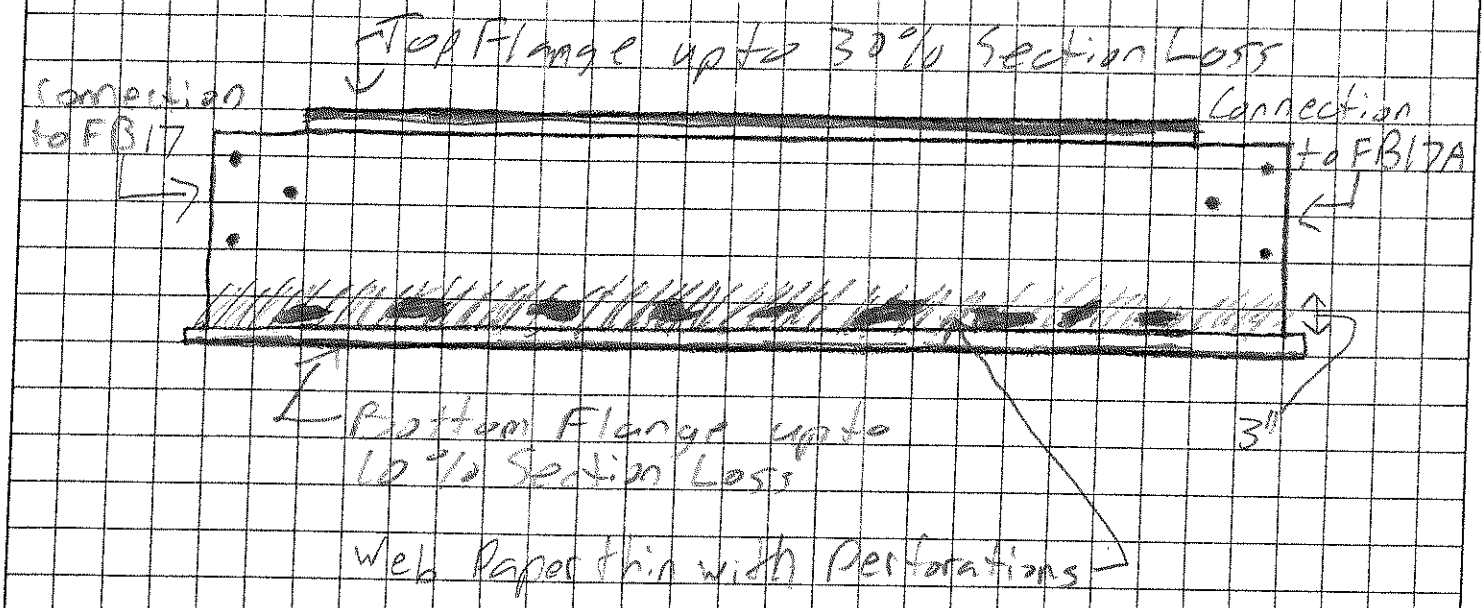
HAKS

Construction Management • Engineering • Architecture
Land Surveying • Material Testing

JOB Paserelle Pedestrian Bridge
SHEET NO. _____ OF _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____
SCALE N.T.S.

YELLOW FLAG P-024

Span 18 - Stringer S4A



Appendix C: Load Rating Report

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1.0 Introduction

Load ratings were initially rated by using the Allowable Stress Method (ASD). If the member did not meet the minimum required inventory rating for either pedestrian load or vehicle load H10, then it re-rated by using the Load Factor Method (LFD), in accordance with the guidelines of Appendix C of NYCDOT's Procedure for Bridge Reconstruction Project Report (BRPR). The as-built and as-inspected ratings were calculated according to the latest provisions of AASHTO Manual for Bridge Evaluation, 2nd Edition (2011) and AASHTO Standard Specification for Highway Bridge, 17th Edition (2002), and LRFD Guide Specifications for the Design of Pedestrian Bridges (2009).

The following loads were applied to the structure, including the canopies at the south end:

- Steel self-weight;
- Concrete 150 lb/ft^3 and wet timber plank 50 lb/ft^2;
- 10 psf superimposed dead load;
- 100 psf floor live load;
- Vehicle load H10; (Single Truck, 8 kips wheel load)
- Self-weight and snow load on the canopies; (Based on the latest NYC Building code) P_g = 25 lb/ft^2 at NYC region

The material properties were defined for the structure:

- Structural steel: F_y =33,000 psi (Year of construction between 1936 to 1963)
- Concrete: f'_c = 2,500 psi

Assumption made in the calculations:

- All superstructures were non-composite sections;

The superimposed dead load and pedestrian load are distributed based on the tributary area from the framing plan. The live load (H10 maintenance truck) was computed based on the moving load to produce maximum result typically at the mid-spans. Since most of the structural members were simple spans, the maximum moving load results were compared to the pedestrian load results to find which load governs.

Original Design Criteria / References

- Bridge built from plans dated in 1961;
- No reference to design code used;
- Live Load = 100 psf shown on plans;

1.1 Summary of Findings

As-Built load ratings findings (timber deck spans)
Generally, the stringers cannot carry vehicle wheel load of 8 kips. In comparison to the existing design, these timber deck spans were only analyzed for pedestrian live load of 100psf and found most of the stringers are okay. However, the floorbeams and girders are found that they do not rate for pedestrian live load of 100psf. The lowest load ratings for the floorbeam is 0.46 (ASD) and girder is 0.21 (ASD). Columns are okay for pedestrian live load of 100psf.

As-Built load ratings findings (concrete deck spans)
The stringers can carry vehicle wheel load of 8 kips. The stringers, floorbeam, column are found to be okay for pedestrian live load of 100psf, except for one location at the floorbeam (bent 24), structural component '24-6'. This floorbeam which supports the base of the existing canapy has a very low rating. No girders are found in the concrete deck spans.

As-Inspected load ratings findings (both timber and deck spans)
Only structural components with deteriorations found were re-computed for section properties based on the field measurements, then incorporated these numbers to calculate the As-Inspected load ratings.

- Incorporates inspected section losses of steel members;
- Spans 1-7 (timber deck): Very few members deteriorated and need repair. The lowest load ratings for the floorbeam is 0.46 (ASD) and girder is 0.21 (ASD);
- Spans 8-18 (timber deck): More advanced corrosion in stringers and floorbeams need repair (see note). Taking the average losses of 20% reduction to compute the as-inspected load ratings, the lowest load ratings for the floorbeam is 0.64 (ASD). Stringers and girders are okay;
- Spans 19-29 (concrete deck): Members under leaking expansion joints deteriorated, and need repair (see note).

Note: Refer to In-depth inspection section for locations.

1.2 Conclusion

The bridge appears that the structure was designed to "Minimum Design Loads in Buildings and Other Structures" from American Standard Building Code Requirements A58.1 (1945) but low in AASHTO bridge ratings especially from spans 1-18. The ratings are low for stringers, floorbeams, and girders except columns. It is not clear that whether the original design had used live load reduction to design the bridge based on the 1961 code. Corrosion is not severe for spans 1-18 and spans 19-29 areas. On the other hand, spans 8-18 have more advanced corrosion in stringers and floorbeams and these spans are typically carrying the NYTA cables. The load ratings in these area were estimated due to the fact that top flanges are not currently exposed with timber planks.

SPANS 1-7 AND 23-29
Load Rating Summary

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES, INC
Joint Venture

Summary for Load Rating (ASD and LFD methods) - Stringers

					As Built (ASD)				As Inspected (ASD)				As Inspected (LFD)			
	Location	Span	Stringer	Length (ft)	Section	Inventory LR		Operating LR		Inventory LR		Operating LR		Inventory LR		Operating LR	
						LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv
*	1	1	S1-S12	7.333	16WF36	See span 2				See span 2				See span 2			
*	2	2	S1-S12	21.000	16WF36	1.15	7.60	1.66	10.62	-	-	-	-	-	-	-	-
*	3	3-6	S1-S12	14.667	10WF21	0.54	4.14	0.82	5.81	0.53	-	0.81	-	0.51	-	0.85	-
	4	7	S13, S14	12.667	10WF21	0.70	4.20	1.02	5.87	0.67	-	0.98	-	0.61	-	1.01	-
	5	23	S13	10.000	12WF27	1.59	5.22	2.27	7.33	-	-	-	-	-	-	-	-
	6	24	S5-S19	32.161	24WF68	1.67	8.87	2.61	12.44								
*	7	24	S20	32.161	24WF110	0.86	6.53	2.40	10.62	0.86	-	2.40	-	1.95	-	3.26	-
*	8	25	S25	37.000	24WF145	0.75	9.19	2.32	13.48	0.75	-	2.32	-	1.91	-	3.20	-
	9	25	S26	37.000	24WF76	1.12	7.41	1.86	10.45	-	-	-	-	-	-	-	-
	10	26	S4	10.000	12WF27	1.58	5.20	2.26	7.30	-	-	-	-	-	-	-	-
	11	27	S4	45.958	24WF110	1.10	6.89	1.85	9.75	-	-	-	-	-	-	-	-
	12	27	S25	45.958	24WF110	1.19	7.22	1.97	10.21	1.10	-	1.85	-	-	-	-	-
*	13	27	S27	45.958	24WF160	1.19	7.43	2.32	11.20	-	-	-	-	-	-	-	-
	14	28	S4	36.500	24WF76	1.18	7.53	1.94	10.60	-	-	-	-	-	-	-	-
	15	28	S25	36.500	24WF76	1.27	7.89	2.06	11.10	1.25	-	2.04	-	-	-	-	-
	16	29	S13	47.500	24WF110	1.13	7.31	1.90	10.33	0.98	-	1.69	-	1.23	-	2.06	-
	17	29	S27	47.500	24WF110	0.97	6.63	1.67	9.40	0.97	-	1.67	-	1.24	-	2.07	-
	18	29	S26	47.500	24WF160 pl	1.03	5.88	2.85	9.53	-	-	-	-	-	-	-	-

Summary for Load Rating (ASD method and LFD methods) - Floorbeams

					As Built (ASD)				As Inspected (ASD)				As Inspected (LFD)			
	Location	Span	FB	Length (ft)	Section	Inventory LR		Operating LR		Inventory LR		Operating LR		Inventory LR		Operating LR	
						LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv
*	1	1	1-2	48.000	33WF130	0.57	4.92	1.00	6.94	0.57	-	1.00	-	0.77	-	1.29	-
	2	1	1-1,1-3	16.000	14WF34	0.84	3.87	1.34	5.47	0.84	-	1.34	-	0.97	-	1.63	-
	3	3	2A,B-2	48.000	30WF124	0.51	5.44	0.97	7.70	0.51	-	0.97	-	0.77	-	1.28	-
	4	3	3A,B-1	16.000	14WF30	0.64	3.56	1.08	5.06	0.64	-	1.08	-	0.79	-	1.32	-
	5	4	4A, 4B	48.000	30WF108	0.46	5.82	0.90	8.22	0.46	-	0.90	-	0.72	-	1.21	-
*	6	6	6-1	48.000	36WF300	2.88	11.37	4.25	15.87	-	-	-	-	-	-	-	-
*	7	6	6-2	31.000	24WF145	2.32	3.56	3.46	5.16	2.26	-	3.40	-	-	-	-	-
	8	7	6A-3	25.000	16WF50	0.56	4.39	1.01	6.24	0.53	-	0.98	-	0.75	-	1.25	-
*	9	7	7	74.667	36WF300	3.12	9.48	4.56	13.23	-	-	-	-	-	-	-	-
*	10	22	22-4	30.000	33WF141	0.43	8.10	1.63	11.86	0.43	-	1.63	-	1.46	-	2.44	-
*	11	24	24-6	30.000	33WF152	-0.42	4.13	0.25	6.32	-0.49	-	0.16	-	0.34	-	0.56	-
	12	24	24-5	27.500	33WF130	1.03	5.61	1.79	8.04	0.73	-	1.35	-	1.05	-	1.75	-
*	13	25	25-7	30.000	33WF141	0.01	7.12	1.10	10.54	0.01	-	1.10	-	1.10	-	1.83	-
*	14	26	26-7	30.000	33WF141	0.50	6.10	1.39	8.92	0.50	-	1.39	-	1.19	-	1.98	-
*	15	27	27-7	30.000	36WF170	0.73	4.72	1.41	6.84	0.67	-	1.32	-	0.97	-	1.62	-
	16	27	27-6	27.500	36WF150	0.80	4.31	1.44	6.24	0.74	-	1.35	-	1.07	-	1.78	-
*	17	28	28-8	30.000	36WF194	0.46	4.79	1.22	7.17	0.46	-	1.22	-	1.05	-	1.75	-
*	18	28	28-6	27.500	36WF160	0.88	4.47	1.56	6.46	0.88	-	1.56	-	1.21	-	2.02	-
*	19	29	29-8	30.000	30WF132	0.80	5.83	1.56	8.59	0.44	-	1.20	-	1.03	-	1.72	-
	20	29	29-6	27.500	30WF116	1.34	9.37	2.05	13.15	1.00	-	1.71	-	1.32	-	2.20	-

Summary for Load Rating (ASD method) - Girders

					As Built (ASD)				As Inspected (ASD)				As Inspected (LFD)			
	Location	Span	Girder	Length (ft)	Section	Inventory LR		Operating LR		Inventory LR		Operating LR		Inventory LR		Operating LR	
						LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv	LRm	LRv
	1	3	G1	44.000	24WF130	0.76	5.52	1.36	7.86	0.76	-	1.36	-	1.01	-	1.69	-
*	2	3	G2	44.000	36WF230	0.49	3.44	0.97	5.00	0.49	-	0.97	-	0.60	-	1.01	-
	3	4	G2	38.000	33WF130	0.36	4.37	0.79	6.27	0.36	-	0.79	-	0.49	-	0.82	-
	4	5	G1	38.000	33WF130	0.65	6.00	1.22	8.53	0.65	-	1.22	-	0.76	-	1.27	-
	5	6	G1	44.000	24WF76	0.67	8.78	1.21	12.29	0.67	-	1.21	-	0.75	-	1.26	-
	6	6	G2	44.000	33WF152	0.21	4.11	0.59	5.92	0.21	-	0.59	-	0.38	-	0.63	-
	7	7	G2	38.000	33WF130	0.36	4.37	0.79	6.27	0.36	-	0.79	-	0.49	-	0.82	-
	8	7	G4	42.120	24WF100	1.11	8.25	1.82	11.58	1.08	-	1.77	-	-	-	-	-
*	9	23-24	G2	59.920	33WF152	0.59	7.51	1.19	10.64	0.59	-	1.19	-	0.95	-	1.58	-

* *See hand calculations*

Group 1: ***FB 1-2****, FB 2-2, and FB 3-2 (****FB 1-2*** *governs);*

Group 2: ***FB 1-1****, FB 1-3, FB 2-1, FB 2-3, FB 3-1, and FB 3-3 (****FB 1-1*** *governs);*

Group 3: ***FB 2A-2, FB 2B-2,*** *FB 5A-2, and FB 5B-2 (****FB 2A,B-2*** *govern);*

Group 4: FB 2A-1, FB 2B-1, FB 5A-1, FB 5B-1,
FB 3A-1*, FB 3B-1, FB 6A-1, and FB 6B-1 (****FB 3A-1*** *governs);*

Group 5: FB 3A-2, FB 3B-2, ***FB 4A-2, FB 4B-2****, FB 6A-2, and FB 6B-2 (****FB 4A,B-2*** *govern);*

Group 6: FB 4-1, FB 5-1, and ***FB 6-1*** *(****FB 6-1*** *governs);*

Group 7: FB 23, ***FB 24*** *(****FB 24*** *governs);*

Summary for Load Rating (ASD method) - Columns

					As-built (ASD)								
	.ocation	Span	Column	L (ft)	Section	A (in2)	ry (in)	Pdl (kips)	Psdl (kips)	Pped (kips)	Pcan (kips)	Inventory LR LRm	Operating LR LRm
*	1	1	1K	15.250	14WF74	21.76	2.48	66.00	9.20	91.78	-	**2.29**	**3.06**
	2	1	1L	15.250	14WF84	24.71	3.02	96.00	13.80	137.67	-	**1.70**	**2.32**
*	3	2	2K	15.130	14WF95	27.94	3.71	127.30	21.00	210.57	-	**1.22**	**1.69**
	4	2	2L	15.130	14WF119	34.99	3.75	164.00	27.00	265.00	-	**1.20**	**1.67**
*	5	3	3K	16.380	14WF78	22.94	3.00	112.50	15.50	155.20	-	**1.19**	**1.69**
	6	3	3L	16.380	14WF111	32.65	3.73	127.60	17.90	180.00	-	**1.79**	**2.43**
*	7	4	4K	18.760	14WF87	25.56	3.70	95.40	14.10	142.00	-	**1.73**	**2.35**
	8	4	4L	18.760	14WF95	27.94	3.71	119.70	17.10	175.00	-	**1.44**	**1.99**
	9	5	5K,5L	-	14WF84	24.71	3.02	Not govern			-	See 4L	See 4L
	10	6	6K,6L	-	14WF78	22.94	3.00	Not govern			-	See 3K	See 3K
	11	7	7K,7L	-	14WF78	22.94	3.00	Not govern			-	See 3K	See 3K
*	12	22	22H	19.900	14WF61	17.94	2.45	53.68	5.72	57.24	40.57	**1.79**	**2.67**
*	13	23	23H	19.720	14WF61	17.94	2.45	51.64	5.57	55.74	28.53	**2.12**	**3.03**
*	14	24	24E	23.170	14WF87	25.56	3.70	85.91	9.51	95.10	0.00	**2.50**	**3.36**
*	15	24	24H	23.170	14WF87	25.56	3.70	94.44	10.37	103.74	54.79	**1.67**	**2.46**
*	16	25	25H	18.440	14WF78	22.94	3.00	59.09	6.30	63.00	47.21	**2.99**	**4.17**
*	17	26	26H	18.250	14WF78	22.94	3.00	73.34	7.64	76.44	26.34	**2.55**	**3.53**
*	18	27	27E	20.550	14WF87	25.56	3.70	105.69	11.34	113.38	0.00	**2.03**	**2.78**
*	19	27	27H	20.550	14WF95	27.94	3.71	115.04	12.37	123.69	6.31	**1.99**	**2.74**
*	20	28	28H	19.270	14WF95	27.94	3.71	117.72	12.60	126.00	30.59	**1.79**	**2.54**
*	21	29	29H	17.020	14WF61	17.94	2.45	69.56	7.13	64.13	42.06	**1.63**	**2.49**

ee hand calculations

Summary for Load Rating (ASD method)

	Location	Span	Stringer	Length (ft)	Spacing 1 (ft)	Spacing 2 (ft)	/ped (kips)	VH10 (kips)	Vcanopy (kips)	Inventory LR		Operating LR	
										LRm	LRv	LRm	LRv
*	1	1	S1-S12	7.333	5.333	5.333				See span 2		See span 2	
*	2	2	S1-S12	21.000	5.333	5.333	5.28	5.69	0.00	1.15	7.60	1.66	10.62
*	3	3-6	S1-S12	14.667	5.333	5.333	3.91	5.69	0.00	0.54	4.14	0.82	5.81
	4	7	S13, S14	12.667	5.333	5.333	3.38	5.69	0.00	0.70	4.20	1.02	5.87
	5	23	S13	10.000	6.875	7.500	3.59	5.45	0.00	1.59	5.22	2.27	7.33
	6	24	S5-S19	32.161	6.875	6.875	1.06	6.25	0.00	1.67	8.87	2.61	12.44
*	7	24	S20	32.161	7.500	7.500	2.06	6.82	44.97	0.86	6.53	2.40	10.62
*	8	25	S25	37.000	7.500	7.500	3.88	6.82	22.08	0.75	9.19	2.32	13.48
	9	25	S26	37.000	7.500	7.500	3.88	6.82	0.00	1.12	7.41	1.86	10.45
	10	26	S4	10.000	7.500	7.500	3.75	5.45	0.00	1.58	5.20	2.26	7.30
	11	27	S4	45.958	7.500	7.500	7.23	6.82	0.00	1.10	6.89	1.85	9.75
	12	27	S25	45.958	6.875	7.500	6.52	6.82	0.00	1.19	7.22	1.97	10.21
*	13	27	S27	45.958	7.500	7.500	7.23	6.82	32.27	1.19	7.43	2.32	11.20
	14	28	S4	36.500	7.500	7.500	3.69	6.82	0.00	1.18	7.53	1.94	10.60
	15	28	S25	36.500	6.875	7.500	3.12	6.82	0.00	1.27	7.89	2.06	11.10
	16	29	S13	47.500	6.875	6.875	6.33	6.25	0.00	1.13	7.31	1.90	10.33
	17	29	S27	47.500	7.500	7.500	7.81	6.82	0.00	0.97	6.63	1.67	9.40
	18	29	S26	47.500	7.500	7.500	7.81	6.82	52.63	1.03	5.88	2.85	9.53

* *See hand calculations*

Summary for Load Rating (ASD method)

	Location	Span	FB	Length (ft)	Spacing 1 (ft)	Spacing 2 (ft)	/ped (kips)	VH10 (kips)	Vcanopy (kips)	Inventory LR		Operating LR	
										LRm	LRv	LRm	LRv
*	1	1	1-2	48.000	5.333	5.333	8.00	10.00	0.00	0.57	4.92	1.00	6.94
	2	1	1-1,1-3	16.000	5.333	5.333	9.50	10.00	0.00	0.84	3.87	1.34	5.47
	3	3	2A,B-2	48.000	5.333	5.333	1.20	10.00	0.00	0.51	5.44	0.97	7.70
	4	3	3A,B-1	16.000	5.333	5.333	5.80	10.00	0.00	0.64	3.56	1.08	5.06
	5	4	4A, 4B	48.000	5.333	5.333	7.20	10.00	0.00	0.46	5.82	0.90	8.22
*	6	6	6-1	48.000	5.333	5.333	0.90	20.00	0.00	2.88	11.37	4.25	15.87
*	7	6	6-2	31.000	5.333	5.333	7.33	0.00	0.00	2.32	3.56	3.46	5.16
*	8	7	6A-3	25.000	5.333	5.333	3.32	10.00	0.00	0.56	4.39	1.01	6.24
*	9	7	7	74.667	5.333	5.333	6.96	10.00	0.00	3.12	9.48	4.56	13.23
*	10	22	22-4	30.000	7.500	7.500	1.47	10.00	24.88	0.43	8.11	1.63	11.87
*	11	24	24-6	30.000	7.500	7.500	8.90	10.00	33.46	-0.42	4.13	0.25	6.32
	12	24	24-5	27.500	6.875	6.875	5.67	10.00	0.00	0.88	4.93	1.55	7.08
*	13	25	25-7	30.000	7.500	7.500	3.56	10.00	28.96	0.01	7.12	1.10	10.54
*	14	26	26-7	30.000	7.500	7.500	8.67	10.00	16.14	0.50	6.10	1.39	8.92
*	15	27	27-7	30.000	7.500	7.500	6.38	10.00	3.86	0.73	4.72	1.41	6.84
	16	27	27-6	27.500	6.875	6.875	6.50	10.00	0.00	0.80	4.32	1.44	6.24
*	17	28	28-8	30.000	7.500	7.500	7.25	10.00	33.13	0.46	4.79	1.22	7.17
*	18	28	28-6	27.500	6.875	6.875	7.25	10.00	0.00	0.88	4.47	1.56	6.46
*	19	29	29-8	30.000	7.500	7.500	7.00	10.00	20.00	0.44	5.83	1.20	8.59
	20	29	29-6	27.500	7.500	7.500	7.00	10.00	0.00	1.00	9.38	1.71	13.16

* *See hand calculations*

Group 1: ***FB 1-2****, FB 2-2, and FB 3-2 (****FB 1-2*** *governs);*

Group 2: ***FB 1-1****, FB 1-3, FB 2-1, FB 2-3, FB 3-1, and FB 3-3 (****FB 1-1*** *gover*

Group 3: ***FB 2A-2, FB 2B-2,*** *FB 5A-2, and FB 5B-2 (****FB 2A,B-2*** *govern);*

Group 4: FB 2A-1, FB 2B-1, FB 5A-1, FB 5B-1,

FB 3A-1*, FB 3B-1, FB 6A-1, and FB 6B-1 (****FB 3A-1*** *governs);*

/15/13

Summary for Load Rating (ASD method) - Girders

								As-built (ASD)																	
Location	Span	Girder	Length (ft)	Spacing 1 (ft)	Spacing 2 (ft)	DF	Section	d (in)	tw (in)	Sx (in3)	Mdl (kip.ft)	Msdl (kip.ft)	Mped (kip.ft)	MH10 (kip.ft)	Mcanopy (kip.ft)	Vdl (kips)	Vsdl (kips)	Vped (kips)	VH10 (kips)	Vcanopy (kips)	Inventory LR		Operating LR	
																					LRm	LRv	LRm	LRv
1	3	G1	44.000	5.333	5.333	1.00	24WF130	24.25	0.565	330.70	233.23	33.00	300.67	96.00	0.00	18.75	2.58	23.43	5.00	0.00	0.76	5.52	1.36	7.86
* 2	3	G2	44.000	5.333	5.333	1.00	36WF230	35.88	0.765	835.50	698.31	94.60	945.85	96.00	0.00	52.63	7.03	70.36	5.00	0.00	0.49	3.44	0.97	5.00
3	4	G2	38.000	5.333	5.333	1.00	33WF130	33.10	0.580	404.80	370.44	51.93	513.00	81.00	0.00	29.89	4.10	40.50	5.00	0.00	0.36	4.37	0.79	6.27
4	5	G1	38.000	5.333	5.333	1.00	33WF130	33.10	0.580	404.80	317.24	38.00	385.07	81.00	0.00	25.69	3.00	30.40	5.00	0.00	0.65	6.00	1.22	8.53
5	6	G1	44.000	5.333	5.333	1.00	24WF76	23.91	0.440	175.40	127.11	18.33	176.00	96.00	0.00	9.11	1.25	12.00	5.00	0.00	0.67	8.78	1.21	12.29
6	6	G2	44.000	5.333	5.333	1.00	33WF152	33.50	0.635	486.40	516.02	68.93	689.33	96.00	0.00	36.06	4.70	47.00	5.00	0.00	0.21	4.11	0.59	5.92
7	7	G2	38.000	5.333	5.333	1.00	33WF130	33.10	0.580	404.80	370.44	51.93	513.00	81.00	0.00	29.89	4.10	40.50	5.00	0.00	0.36	4.37	0.79	6.27
8	7	G4	42.120	5.333	5.333	1.00	24WF100	24.00	0.468	248.90	141.22	21.06	189.54	91.30	0.00	10.61	1.50	13.50	5.00	0.00	1.11	8.25	1.82	11.58
* 9	23-24	G2	59.920	7.500	6.875	1.00	33WF152	33.50	0.635	486.40	426.66	43.74	440.89	-	0.00	26.85	2.74	27.20	-	0.00	0.59	7.51	1.19	10.64

* *See hand calculations*

Bent Elevations

Pier	Span (ft)	Top of Deck Elevation			1937 Footing			1961 Footing			
		Upper L EL	Mezz L EL	Lower L EL	Top Ped EL	Top Ftg EL	Ftg Thick (ft)	Type	Top Ped EL	Bott Ftg EL	Ftg Thick (ft)
1	7.33	40.70	25.45	18.10	12.37	10.33	2.33	E	12.50	6.00	2
2	21.00	40.58	25.45	19.78	12.37	10.50	2.33	E	12.50	6.00	2
3	44.00	38.41	28.25	23.30	11.87	Varies	2.33	E	12.00	6.00	Varies
4	38.00	35.71	30.63	26.34	11.87	10.00	2.33	E	12.00	6.00	2
5	38.00	33.00	33.00	29.38	10.87	9.00	2.33	E	11.00	6.00	2
6	44.00	33.27	-	32.90	10.87	8.83	2.33	E	11.00	6.00	2
7	38.00	33.50	-	-	13.87	8.33	2.33	-	-	-	-
8	34.91	33.26	-	-	13.87	10.33	2.33	-	-	-	-
9	36.35	33.00	-	-	13.87	10.33	2.33	-	-	-	-
10	38.00	33.00	-	-	13.87	10.33	2.33	-	-	-	-
11	50.50	33.00	-	-	13.87	10.33	2.33	-	-	-	-
12	50.50	33.00	-	-	13.87	10.33	2.33	-	-	-	-
13	50.50	33.00	-	-	13.87	9.33	2.33	-	-	-	-
14	50.50	33.00	-	-	13.87	11.83	2.33	-	-	-	-
15	50.50	33.00	-	-	13.87	11.83	2.33	-	-	-	-
16	50.50	33.00	-	-	13.87	10.83	2.33	-	-	-	-
17	50.50	33.00	-	-	10.37	8.83	2.33	-	-	-	-
18	50.50	33.00	-	-	~~8.37~~	~~5.33~~	~~2.33~~	S-CW	9.12	3.00	2.33
19	46.96	32.13	-	-	-	-	-	S-CW	9.12	Varies	Varies
20	55.27	31.12	-	-	-	-	-	C-L	10.42	Varies	2.5
21	10.00	30.93	-	-	-	-	-	C-L	10.42	Varies	2.5
22	33.16	30.32	-	-	-	-	-	C-L	10.42	5.00	2.5
23	10.00	30.14	-	-	-	-	-	C-L	10.42	5.00	2.5
24	33.16	29.54	-	-	-	-	-	S-CW	6.37	5.00	Varies
25	37.00	28.86	-	-	-	-	-	C-L	10.42	Varies	2.5
26	10.00	28.67	-	-	-	-	-	C-L	10.42	Varies	2.5
27	45.96	27.55	-	-	-	-	-	S	7.00	5.00	Varies
28	36.50	26.27	-	-	-	-	-	S	7.00	5.00	Varies
29	47.50	24.10	-	-	-	-	-	Varies	7.08	Varies	Varies

SPANS 1-7 AND 23-29
As Built Load Rating Calculations

Stringers

(ASD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES™ INC
Joint Venture

WEIDLINGER

PROJECT	Passerelle		
BY	Jason A.	DATE	9/20/13
CHECKED BY	TFKo	DATE	10/1/13

SUBJECT	Stringer 16WF36
	Spans 1&2 (S2-S15)
PAGE 1	OF 2

Stringer 16WF36 (Spans 1&2) (2-span)

$DF = \frac{5.33}{3.75} = 1.42$ Span lengths: 7'-4" cantilever, 21'-0" span 2

$w_{DL} = 0.05\,ksf \times 5.33' + 0.036\,klf = 0.302\,klf$ add 5% misc. 0.317

$w_{SDL} = 0.01$ " x " $= 0.053\,klf$

$w_{ped} = 0.1$ " x " $= 0.533\,klf$

Moment

$M_{DL} = \frac{0.317\,klf}{8(21^2)}(21+7.33)^2(21-7.33)^2 = 13.48\,kft$

$M_{SDL} = \frac{0.053\,klf}{"}$ " " " " $= 2.27\,kft$

$M_{ped} = \frac{0.533}{"}$ " " " " $= 22.658\,kft$

$M_{HLD} = \frac{16k(10\frac{1}{2}')(10\frac{1}{2}')}{21'}(1.42/2) = 59.64\,kft$ ← governs

Shear

$V_{DL} = \frac{0.317\,klf}{2(21)}(21^2+7.33^2) = 3.74\,k$

$V_{SDL} = \frac{0.053}{"}$ " " $= 0.63\,k$

$V_{ped} = \frac{0.533}{"}$ " " $= 6.28\,k$ ← govern

$V_{HLD} = 16k/2\,(1.42/2) = 5.68\,k$

PLEASE RECYCLE

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/20/13
CHECKED BY	TFKO	DATE	10/11/13

SUBJECT	Stringer 16WF36 Spans 1 & 2		
PAGE	2	OF	2

Load Rating 16WF36

Inventory $LR_M = \dfrac{18\,ksi \times 56.30\,in^3/12 - (13.48 + 2.27)}{59.64}$

$= 1.152$

$LR_V = \dfrac{11\,ksi \times 15.85'' \times 0.299'' - (3.74 + 0.12)}{6.28}$

$= 7.686$

Operating $LR_M = \dfrac{24.5\,ksi \times 56.30\,in^3/12 - (13.48 + 2.27)}{59.64}$

$= 1.663$

$LR_V = \dfrac{15\,ksi \times 15.85'' \times 0.299'' - (3.74 + 0.12)}{6.28}$

$= 10.705$

WEIDLINGER

PROJECT Passerelle	SUBJECT Stringer 10WF21
BY Jason A. DATE 9/20/13	Spans 3~6 (S1-S12)
CHECKED BY TF Ko DATE 10/11/13	PAGE 1 OF 2

Stringer 10WF21 (From Spans 3~7) (Typical)

$DF = \frac{5.33}{3.75} = 1.42$ Span length: 14.667'

add 5% misc

$W_{DL} = 0.05\ ksf \times 5.33' + 0.021\ klf = 0.288\ klf$ 0.302

$W_{SDL} = 0.01\ '' \times ''\ = 0.053\ klf$

$W_{ped} = 0.1\ '' \times ''\ = 0.533\ klf$

$M_{DL} = \frac{1}{8} W_{DL}\, \ell^2 = 8.12\ kft$

$M_{SDL} = '' W_{SDL} '' = 1.43\ kft$

$M_{ped} = '' W_{ped} '' = 14.33\ kft$

$M_{HLD} = 16^k \left(\frac{14.667'}{4}\right)(1.42/2) = 41.65\ kft$ ← governs

$V_{DL} = \frac{1}{2} W_{DL}\, \ell = 2.22^k$

$V_{SDL} = '' W_{SDL} '' = 0.39^k$

$V_{ped} = '' W_{ped} '' = 3.91^k$

$V_{HLD} = 16^k (1.42/2)/2 = 5.68^k$ ← governs

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/20/13
CHECKED BY	TFKO	DATE	10/11/13

SUBJECT	Stringer 10WF21
	Spans 3 & 6
PAGE	2 OF 2

Load Rating 10WF21

Inventory

$$LR_m = \frac{18\,ksi \times 21.50\,in^3/12 - (8.12 + 1.43)}{41.65}$$

$$= 0.54$$

$$LR_v = \frac{11\,ksi \times 9.9'' \times 0.24'' - (2.22^k + 0.39^k)}{5.68}$$

$$= 4.14$$

Operating

$$LR_m = \frac{24.5\,ksi \times 21.50/12 - (8.12 + 1.43)}{41.65}$$

$$= 0.82$$

$$LR_v = \frac{15\,ksi \times 9.9'' \times 0.24'' - (2.22^k + 0.39^k)}{5.68}$$

$$= 5.81$$

WEIDLINGER

PROJECT Passerelle	SUBJECT Stringer 24WF110
BY Jason A. DATE 9/20/13	Span 24 (S20)
CHECKED BY TFKo DATE 10/11/13	PAGE 1 OF 2

Stringer 24WF110 (Span 24)

$DF = \frac{7.5}{5.5} = 1.36$ Span length = 32.161'

$W_{DL} = \left(\frac{5.5}{12}\right)(7.5') \times 0.150\,pcf + 0.110\,klf = 0.626\,klf$

add 5% misc 0.657 klf

$W_{SDL} = 0.01\,ksf\,(7.5) = 0.075\,klf$

$W_{ped} = 0.1$ " " $= 0.75\,klf$

Moment

$M_{DL} = \frac{1}{8} W_{DL}\, l^2 = 84.93\,kft$

$M_{SDL} =$ " W_{SDL} " $= 9.70\,kft$

$M_{ped} =$ " W_{ped} " $= 96.97\,kft$ ← govern

$M_{HD} = \left(\frac{32.161}{2} - 2.8\right)(10^{k})\left(1.36/2\right) = 90.31\,kft$

$M_{canopy} = 40^{k}\left(\frac{7.83'}{2}\right) + 40\left(\frac{3.83'}{2}\right) = 233.20\,kft$ @ midspan

Shear

$V_{DL} = \frac{1}{2} W_{DL}\, l = 10.56^{k}$

$V_{SDL} =$ " W_{SDL} " $= 1.21^{k}$

$V_{ped} =$ " W_{LL} " $= 12.06^{k}$ ← control

$V_{HD} = 10^{k}\,(1.36/2) = 6.82^{k}$

$V_{canopy} = 40^{k}\,(7.83' + 28.331')/32.161' = 45^{k}$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/20/13
CHECKED BY	TFKo	DATE	10/11/13

SUBJECT	Stringer 24WF110
	span 24
	PAGE 2 OF 2

Load Rating 24WF110

Inventory

$$LR_m = \frac{18\,ksi \times 274.40\,in^3/12 - (84.93 + 9.70 + 233.2)}{96.97}$$

$$= 0.864$$

$$LR_v = \frac{11\,ksi \times 24.16'' \times 0.51'' - (10.56 + 1.21 + 45)}{12.06}$$

$$= 6.531$$

Operating

$$LR_m = \frac{24.5\,ksi \times 274.40/12 - (84.93 + 9.70 + 233.21)}{96.97}$$

$$= 2.397$$

$$LR_v = \frac{15\,ksi \times 24.16'' \times 0.51'' - (10.56 + 1.21 + 45)}{12.06}$$

$$= 10.62$$

WEIDLINGER

PROJECT Passerelle	SUBJECT Stringer 24WF145
BY Jason A. DATE 9/20/13	Span 25 (S25)
CHECKED BY TFK. DATE 10/11/2013	PAGE 1 OF 2

Stringer 24WF145 (Span 25)

$DF = \frac{7.5}{5.5} = 1.36$ Span length = 37'

$W_{DL} = \left(\frac{5.5}{12}\right)(7.5)(0.150) + 0.145 = 0.661$ klf add 5% misc. 0.694 klf

$W_{SDL} = 0.01 \times 7.5' = 0.075$ klf

$W_{ped} = 0.10$ " $= 0.75$ klf

Moment

$M_{DL} = \frac{1}{8} W_{DL} \ell^2 = 118.7$ kft

$M_{SDL} = \frac{1}{8} W_{SDL} \ell^2 = 12.83$ kft

$M_{ped} =$ " W_{ped} " $= 128.34$ kft ← governs

$M_{HIO} = (37'/2 - 2.8')(10')(1.36/2) = 106.76$ kft

$M_{canopy} = 40^k \frac{(16.578')(18.5')}{37'} = 331.56^{k'}$ @ midspan

Shear

$V_{DL} = \frac{1}{2} W_{DL} \ell = 12.83^k$

$V_{SDL} =$ " W_{SDL} " $= 1.39^k$

$V_{ped} =$ " W_{ped} " $= 13.88^k$ ← governs

$V_{HIO} = 10^k (1.36/2) = 6.80^k$

$V_{canopy} = 40^k \left(\frac{37' - 16.578'}{37}\right) = 22.08^k$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/20/13
CHECKED BY	TFKo	DATE	10/11/2013

SUBJECT	Stringer 24WF145 Span 25		
PAGE	2	OF	2

Load Rating 24WF145 (span 25)

Inventory $LR_m = \dfrac{18\ \text{ksi} \times 372.50\ \text{in}^3/12 - (118.7 + 12.83 + 331.56)}{128.34}$

$= 0.745$

$LR_v = \dfrac{11\ \text{ksi} \times 24.49'' \times 0.608'' - (12.83 + 1.39 + 22.08)}{13.875}$

$= 9.188$

Operating $LR_m = \dfrac{24.5 \times 372.5/12 - (118.7 + 12.83 + 331.56)}{128.34}$

$= 2.317$

$LR_v = \dfrac{15 \times 24.49 \times 0.608 - (12.83 + 1.39 + 22.08)}{13.875}$

$= 13.48$

Stringer 25 at Span 27, 24WF160 (S27)

$L := 45.9583\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$S1 := 7.5\ ft$

$S2 := 7.5\ ft$

$ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$Pcan := 40\ kip$

Loading on Stringers:

Stringer 24WF160:

$$S := \frac{S1 + S2}{2} = 7.5\ ft$$

$$DF := \frac{S}{5.5\ ft} = 1.364$$

$$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.16\ \frac{kip}{ft}\right) = 0.709\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$$

Moment & Shear:

Dead Load: $Mdl := \frac{wdl \cdot L^2}{8} = 187.298\ kip \cdot ft$

S-Dead Load: $Msdl := \frac{wsdl \cdot L^2}{8} = 19.802\ kip \cdot ft$

Pedestrain: $Mped := \frac{wped \cdot L^2}{8} = 198.016\ kip \cdot ft$

H-10: $Mh10 := \left(\frac{L}{2} - 2.8\ ft\right) \cdot (10\ kip) \cdot \left(\frac{DF}{2}\right) = 137.585\ kip \cdot ft$

Canopy: $Mcan := \frac{Pcan \cdot 8.88\ ft \cdot 22.98\ ft}{L} = 177.607\ kip \cdot ft$

Dead Load: $Vdl := \frac{wdl \cdot L}{2} = 16.302\ kip$

S-Dead Load: $Vsdl := \frac{wsdl \cdot L}{2} = 1.723\ kip$

Pedestrain: $Vped := \frac{wped \cdot L}{2} = 17.234\ kip$

H-10: $Vh10 := \frac{H10}{2} \cdot \frac{DF}{2} = 6.818\ kip$

Canopy: $Vcan := Pcan \cdot \left(\frac{37.0783}{45.9583}\right) = 32.271\ kip$

Load Rating

$Sx := 413.5\ in^3$ $Fy := 33\ ksi$ $d := 24.72\ in$ $tw := .656 \cdot in$

Inventory:

$$LRm := \frac{18\ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 1.19$$

$$LRv := \frac{11\ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 7.432$$

Operating:

$$LRm := \frac{24.5\ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 2.321$$

$$LRv := \frac{15\ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 11.196$$

SPANS 1-7 AND 23-29
As Built Load Rating Calculations

Floorbeams

(ASD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

WEIDLINGER

PROJECT	Paserelle Ped. Bridge		
BY	Jason A.	DATE	1/8/14
CHECKED BY	TFKo	DATE	8/22/14

SUBJECT	Span 1 FB 1~2		
	33WF130		
PAGE	1	OF	2

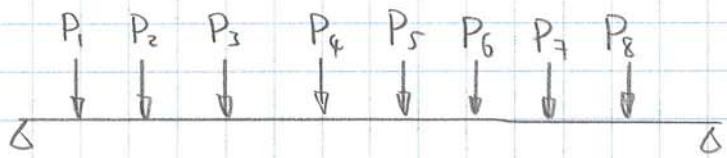
Beam loads:

Dead load $\left[(0.036 \times 21\tfrac{1}{2} + 0.021 \times 14.67\tfrac{1}{2}) + (21\tfrac{1}{2} + 14.67\tfrac{1}{2})(5.33')(0.05)\right]1.05$

$= 5.55$ k

SLL $(21\tfrac{1}{2} + 14.67\tfrac{1}{2})(5.33')(0.010\,\text{ksf}) = 0.95$ k

Ped. " " " $0.100\,\text{ksf} = 9.50$ k



Moment @ midspan $2\left[\frac{P_1(L/9)(8L/9)}{L}\left(\frac{0.5L}{8L/9}\right) + P_2\frac{(2L/9)(7L/9)}{L}\left(\frac{0.5L}{7L/9}\right) + P_3\frac{(3L/9)(6L/9)}{L}\left(\frac{0.5L}{6L/9}\right) + P_4\frac{(4L/9)(5L/9)}{L}\left(\frac{0.5L}{5L/9}\right)\right]$

$$= 2\left(\frac{1}{18}P_1L + \frac{2}{18}P_2L + \frac{3}{18}P_3L + \frac{4}{18}P_4L\right)$$

If $P_1 = P_2 = P_3 = P_4 = P_5 = P_6 = P_7 = P_8$

$$M_{max} = \frac{10}{9}PL$$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	1/8/14
CHECKED BY	TFKO	DATE	8/22/14

SUBJECT			
PAGE	2	OF	2

$M_{DL} = \frac{10}{9}(5.55^k)(48') = 296\ kft$

$M_{SDL} = \text{"}\ 0.95^k\ \text{"} = 50.67\ kft$

$M_{Ped} = \text{"}\ 9.5\ \text{"} = 506.7\ kft$ ← governs

$M_{HIO} = (48'/2 - 2.8')(10^k) = 212\ ^{k}ft$

$V_{DL} = 4 \times 5.55^k = 22.2\ ^k$

$V_{SDL} = \text{"}\ 0.95^k = 3.8\ ^k$

$V_{Ped} = \text{"}\ 9.5^k = 38^k$ ← governs

$M_{Hb} = 10^k$

Inv. Rating $LR_m = \frac{18 \times 404.8/12 - (296 + 50.67)}{506.7} = 0.51$

$LR_v = \frac{11 \times 33.10 \times 0.58 - (22.2 + 3.8)}{38} = 4.87$

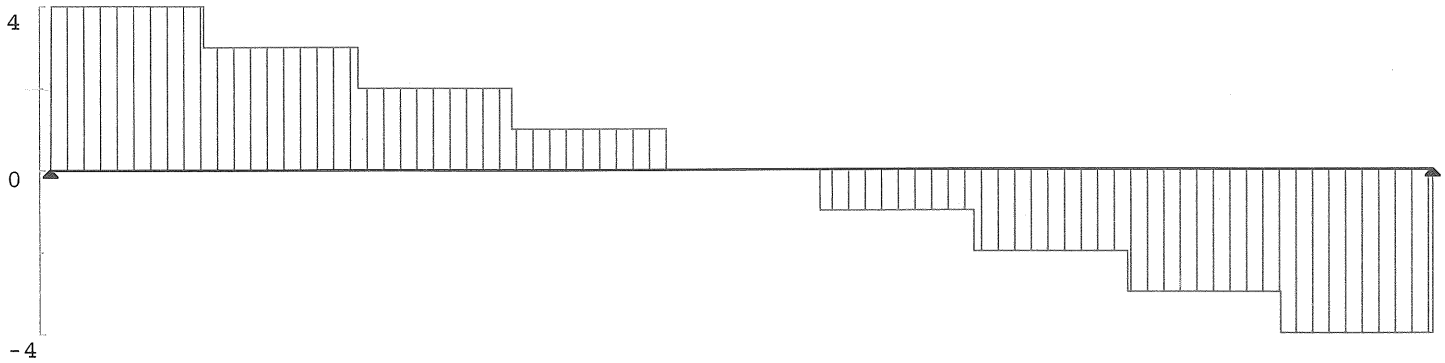
Operating Rating $LR_m = \frac{24.5 \times 404.8/12 - (296 + 50.67)}{506.7} = 0.95$

$LR_v = \frac{15 \times 33.10 \times 0.58 - (22.2 + 3.8)}{38} = 6.90$

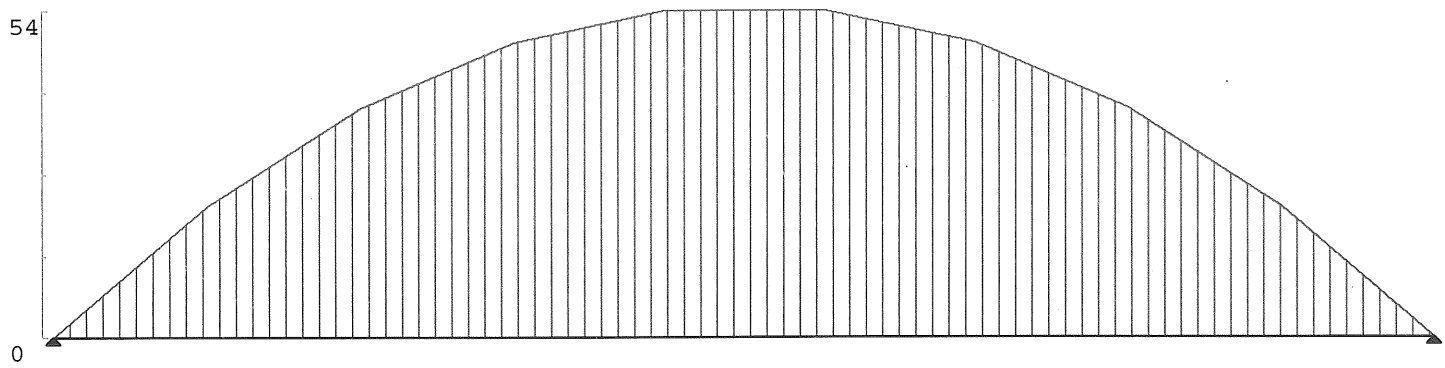
RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (48.00,0.00)

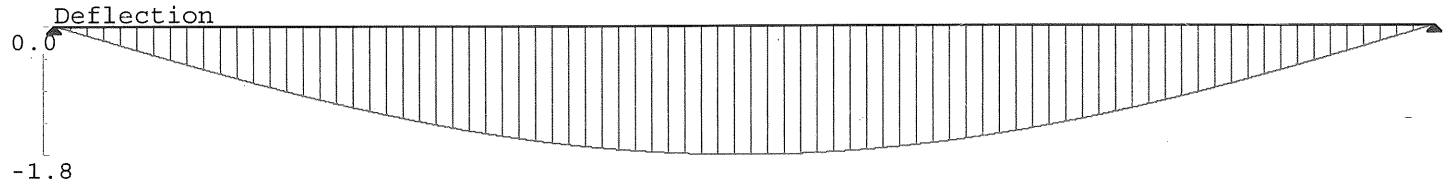
Shear



Moment



Deflection



Max DL Shear = 4.00 kips
Max Shear = 4.00 kips

Max Pos Moment = 53.33 kip-ft at 21.333 ft

$M_{max} = \frac{10}{9}(1^k)(48') = 53.33$ kft

WEIDLINGER

PROJECT	Passerelle		
BY	Jwon A.	DATE	9/20/13
CHECKED BY	TFKo	DATE	10/11/2013

SUBJECT	Floorbeam 36WF300
	Bent 6, FB6-1
PAGE	1 OF 10

Floorbeam 36WF300 (Bent 6, FB 6-1)

span 6 = 14.67'
span 7 = 12.67' } avg 13.67'

Stringer load 10WF21

DL = $[(5.33' \times 13.67')/2 \times 50\ psf + 0.021 \times \frac{14.67}{2}] \times 1.05$

= 2.06 k says 2k

SDL = 5.33' x 13.67/2 x 10 psf

= 0.4 k

Ped = " " " 100 psf

= 3.64 k says 3.7 k

H-10 see next page

Fascia load 24WF76 (span 6)

DL = (0.03 x 8' + 0.021 x 2 x 14.67'
+ 0.076 x 22'
+ 8' x 14.67' x 0.05) 1.05 ≈ 9 k

SDL = 8' x 14.67' x 0.01 = 1.2 k

Ped = " " 0.1 = 11.736 k

Fascia load 24WF76 (span 7)

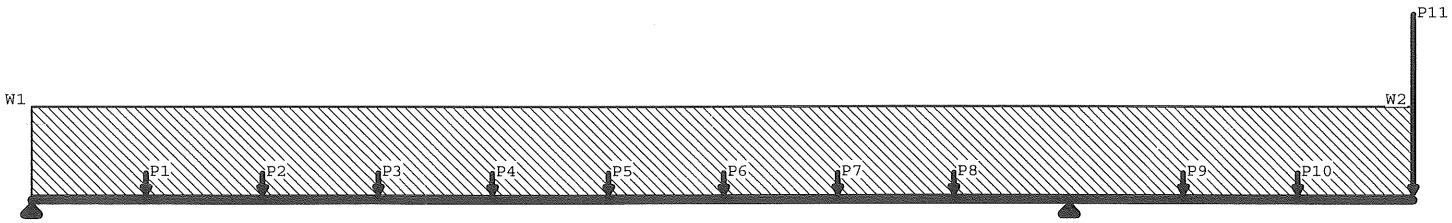
smaller, use same as span 6

Load Diagram

RAM SBeam v3.0.1
Licensed to:

10/07/13 13:38:52

Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)



Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	5.333	2.000	0.000	0.000	2.000
P2	10.667	2.000	0.000	0.000	2.000
P3	16.000	2.000	0.000	0.000	2.000
P4	21.333	2.000	0.000	0.000	2.000
P5	26.667	2.000	0.000	0.000	2.000
P6	32.000	2.000	0.000	0.000	2.000
P7	37.333	2.000	0.000	0.000	2.000
P8	42.667	2.000	0.000	0.000	2.000
P9	53.333	2.000	0.000	0.000	2.000
P10	58.667	2.000	0.000	0.000	2.000
P11	64.000	18.000	0.000	0.000	18.000
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.315	0.000	0.000	0.315
W2	64.000	0.315	0.000	0.000	0.315

33W300 selfweight

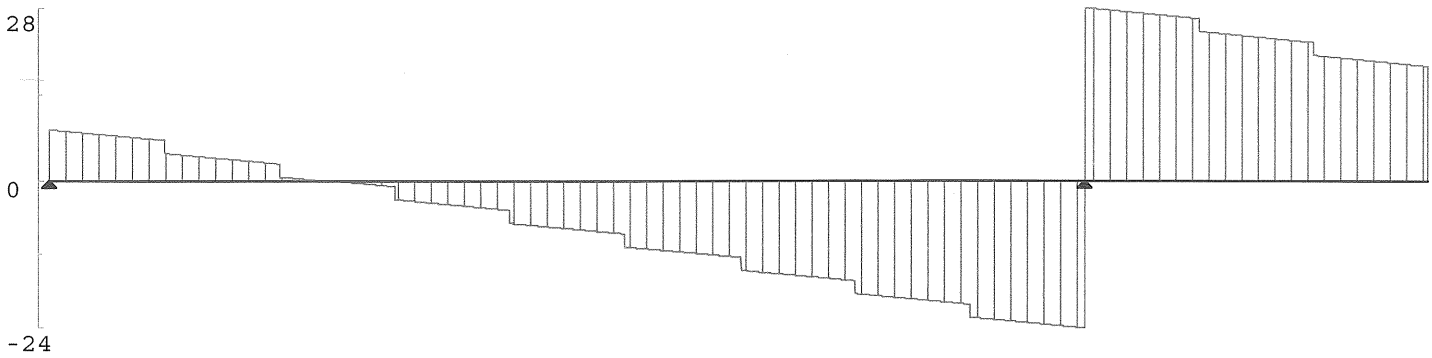
DL

3 of 10

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

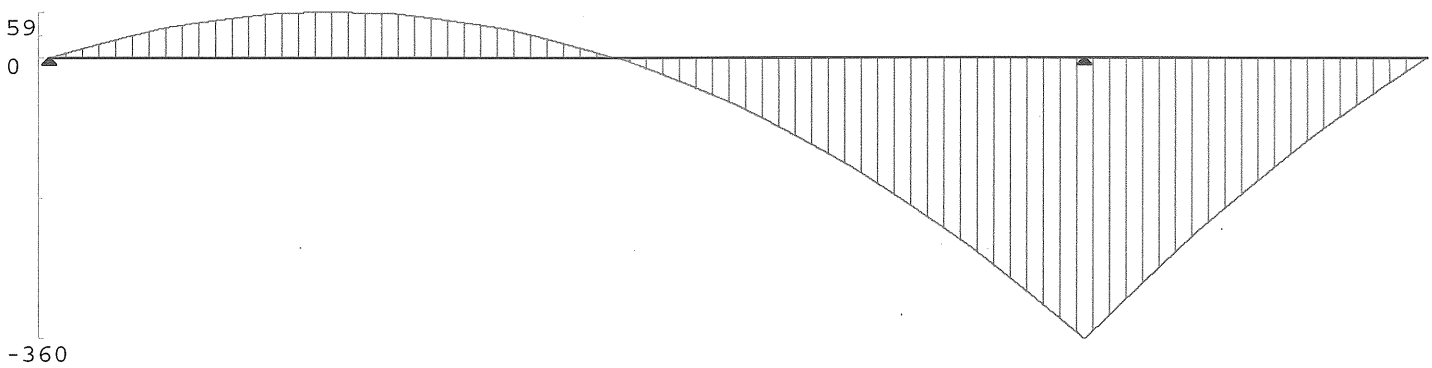
Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)

Shear

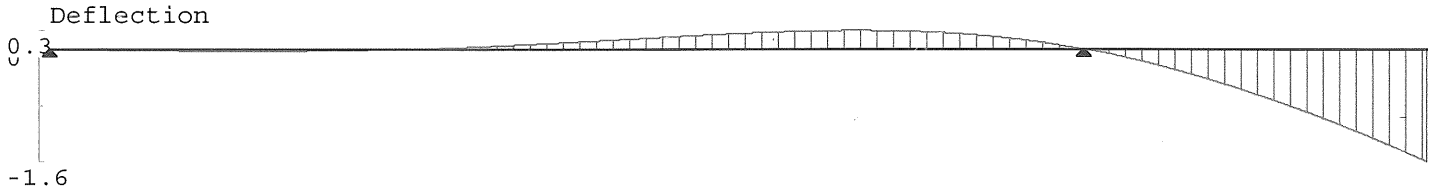


DL

Moment



Deflection



Max DL Shear = 27.04 kips
Max Shear = 27.04 kips

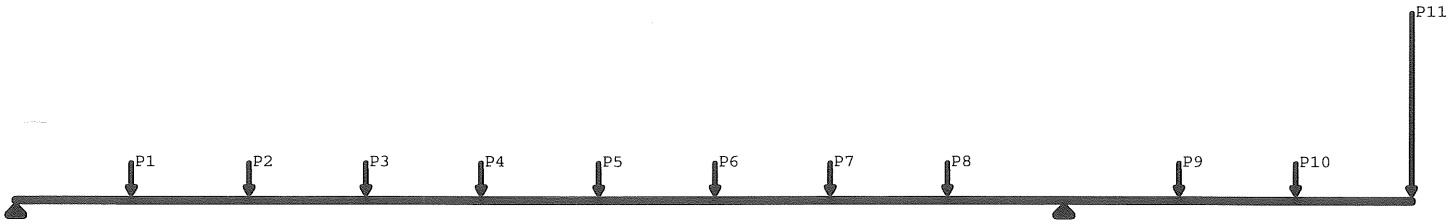
Max Pos Moment = 58.08 kip-ft at 12.876 ft
Max Neg Moment = -360.32 kip-ft at right support

Load Diagram

RAM SBeam v3.0.1
Licensed to:

10/07/13 13:41:41

Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)



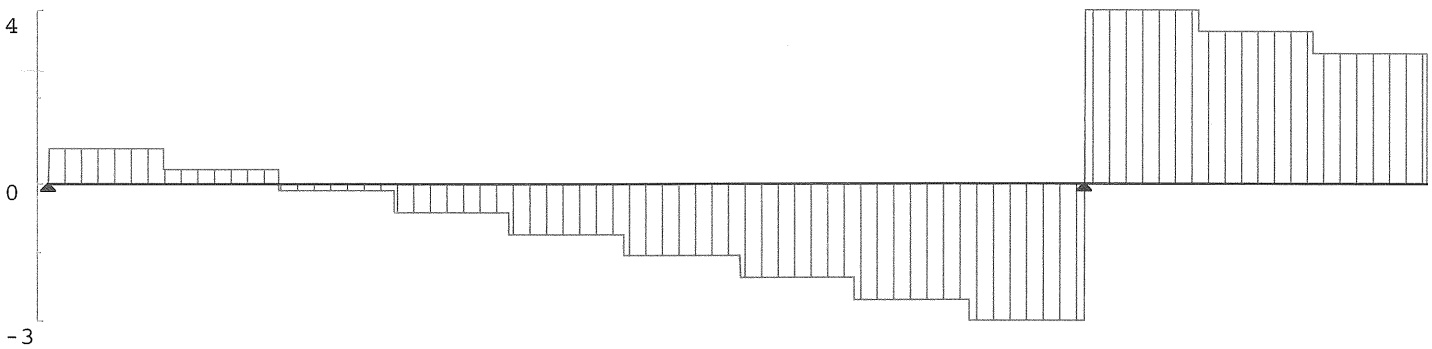
Load	Dist ft	DL kips	LL+ kips	LL- kips	Max Tot kips
P1	5.333	0.400	0.000	0.000	0.400
P2	10.667	0.400	0.000	0.000	0.400
P3	16.000	0.400	0.000	0.000	0.400
P4	21.333	0.400	0.000	0.000	0.400
P5	26.667	0.400	0.000	0.000	0.400
P6	32.000	0.400	0.000	0.000	0.400
P7	37.333	0.400	0.000	0.000	0.400
P8	42.667	0.400	0.000	0.000	0.400
P9	53.333	0.400	0.000	0.000	0.400
P10	58.667	0.400	0.000	0.000	0.400
P11	64.000	2.400	0.000	0.000	2.400

SDL

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

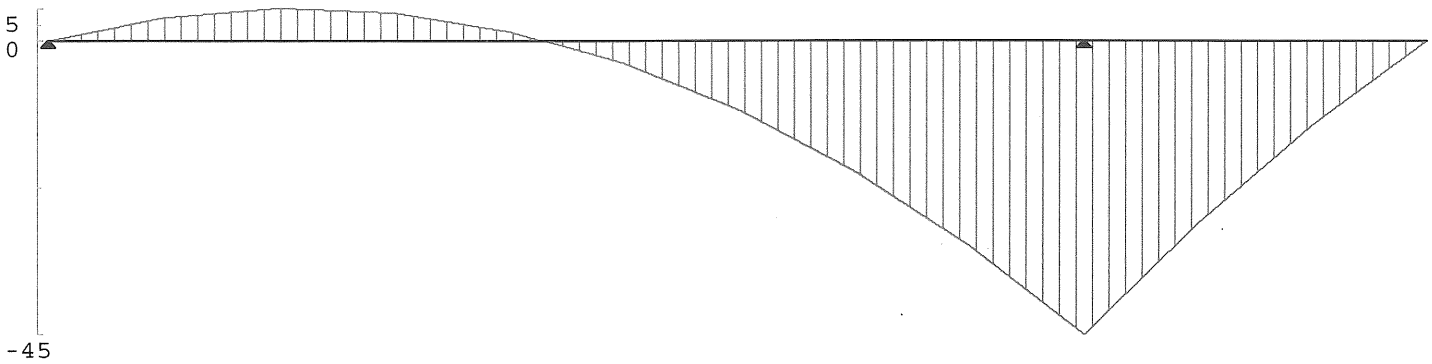
Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)

Shear

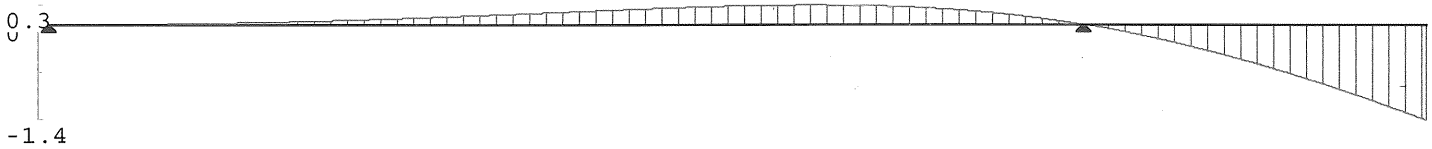


SDL

Moment



Deflection



Max DL Shear = 3.20 kips
Max Shear = 3.20 kips

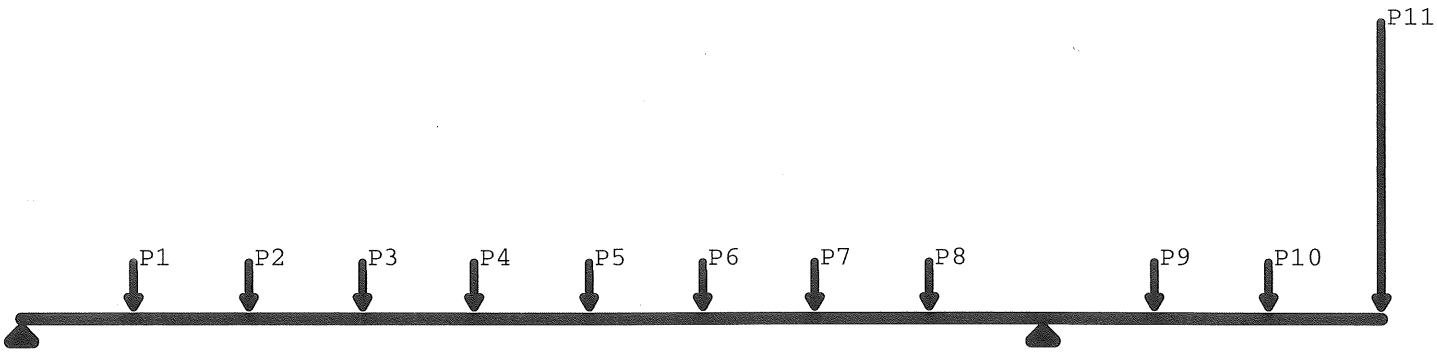
Max Pos Moment = 4.98 kip-ft at 10.667 ft
Max Neg Moment = -44.80 kip-ft at right support

Load Diagram

RAM SBeam v4.0
Licensed to: RAM International

11/12/13 11:00:31

Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)

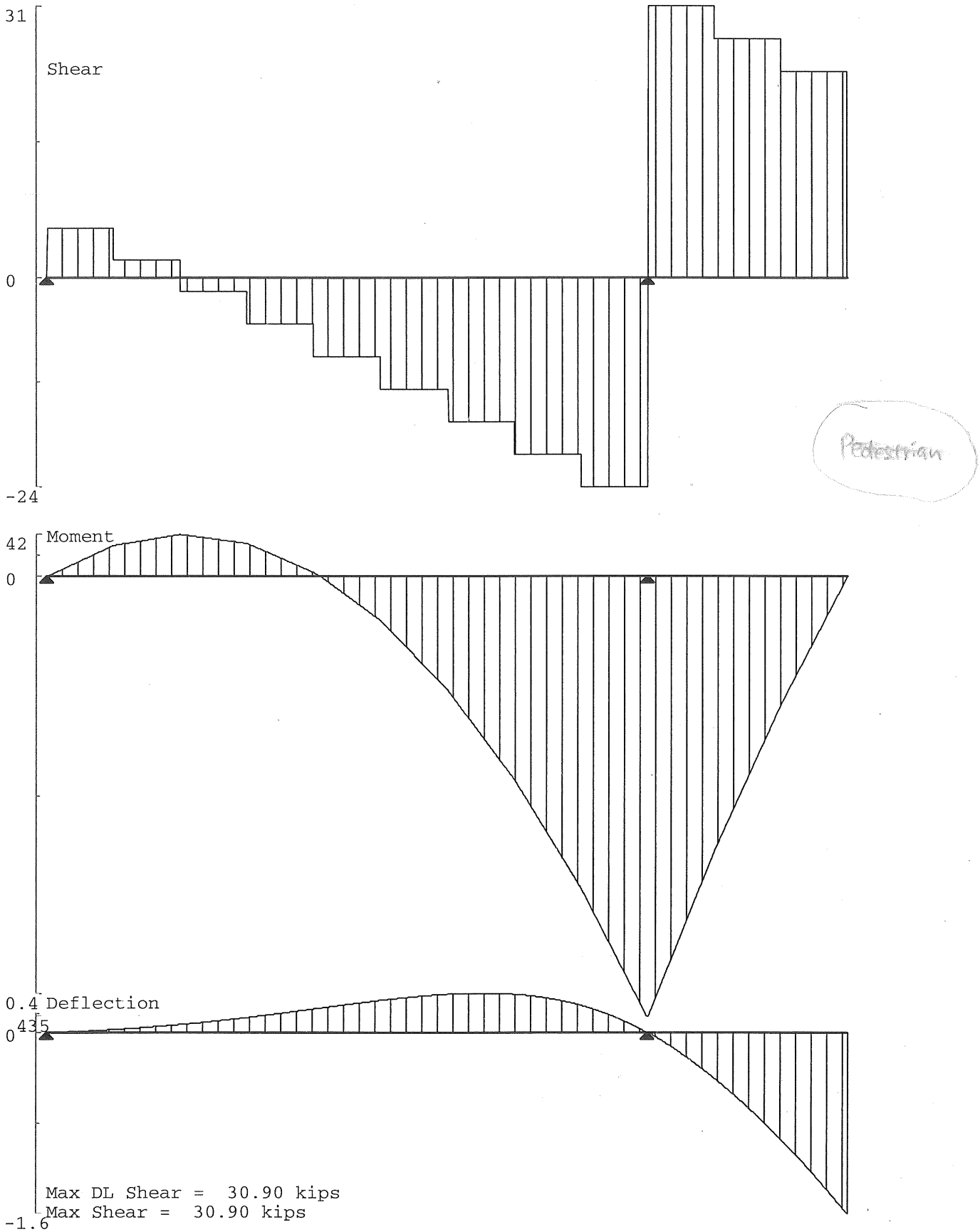


Load	Dist ft	DL kips	LL+ kips	LL- kips	Max Tot kips
P1	5.333	3.700	0.000	0.000	3.700
P2	10.667	3.700	0.000	0.000	3.700
P3	16.000	3.700	0.000	0.000	3.700
P4	21.333	3.700	0.000	0.000	3.700
P5	26.667	3.700	0.000	0.000	3.700
P6	32.000	3.700	0.000	0.000	3.700
P7	37.333	3.700	0.000	0.000	3.700
P8	42.667	3.700	0.000	0.000	3.700
P9	53.333	3.700	0.000	0.000	3.700
P10	58.667	3.700	0.000	0.000	3.700
P11	64.000	23.500	0.000	0.000	23.500

Pedestrian

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)



Max DL Shear = 30.90 kips
Max Shear = 30.90 kips

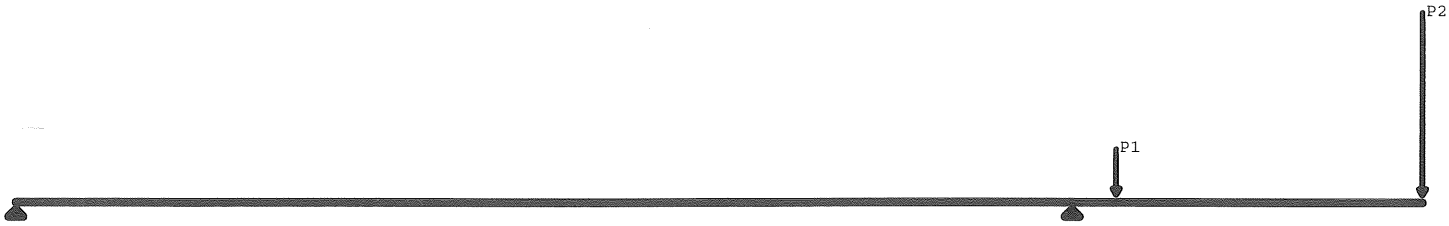
Max Pos Moment = 41.42 kip-ft at 10.667 ft
Max Neg Moment = -435.20 kip-ft at right support

Load Diagram

RAM SBeam v3.0.1
Licensed to:

10/07/13 13:49:23

Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)



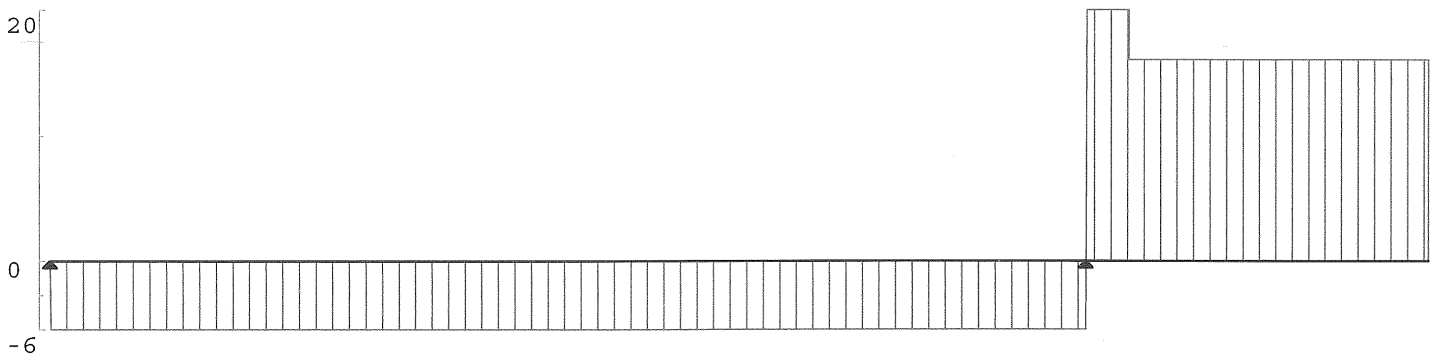
Load	Dist ft	DL kips	LL+ kips	LL- kips	Max Tot kips
P1	50.000	4.000	0.000	0.000	4.000
P2	64.000	16.000	0.000	0.000	16.000

H-10

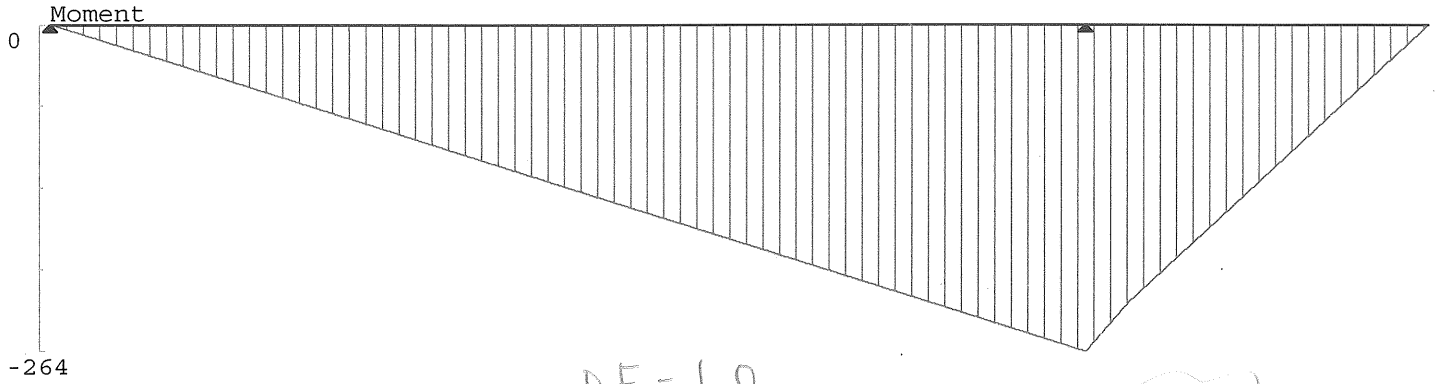
RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (64.00,0.00)

Shear



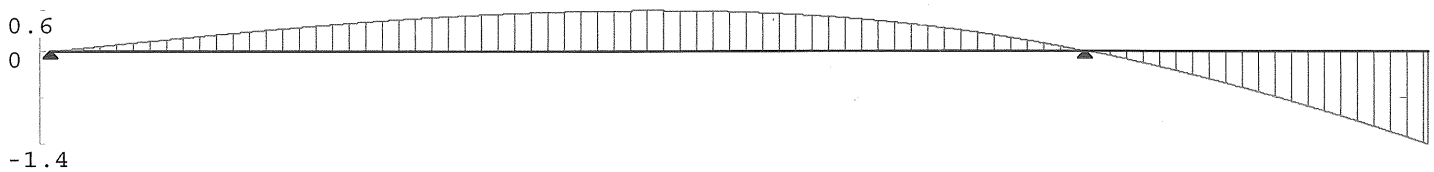
Moment



DF = 1.0

(H-10)

Deflection



pedestrian controls

Max DL Shear = 20.00 kips
Max Shear = 20.00 kips

Max Neg Moment = -264.00 kip-ft at right support

WEIDLINGER

PROJECT		SUBJECT	Floorbeam 36WF300
BY Jason A.	DATE 10/11/13		Bent 6, FB 6-1
CHECKED BY TFKO	DATE 10/11/13		PAGE 10 OF 10

Load rating 36WF300 (Bent 6, FB 6-1)

Inventory $LR_m = \dfrac{18 \times 1105.10/12 - (360.32 + 44.80)}{435.2}$

$= 2.88$

$LR_v = \dfrac{11 \times 36.72 \times 0.945 - (27.04 + 3.20)}{30.9}$

$= 11.37$

Operating $LR_m = \dfrac{24.5 \times 1105.10/12 - (360.32 + 44.80)}{435.2}$

$= 4.25$

$LR_v = \dfrac{15 \times 36.72 \times 0.945 - (27.04 + 3.20)}{30.9}$

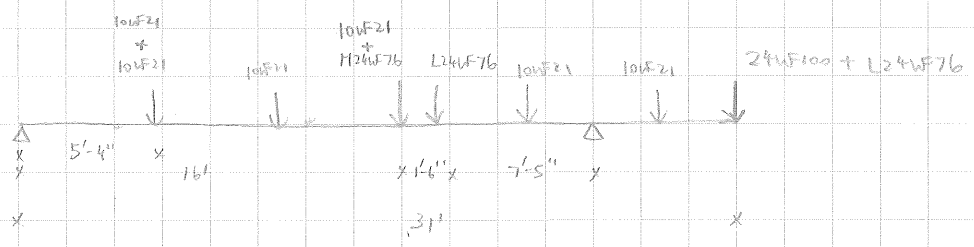
$= 15.86$

WEIDLINGER

PROJECT	Passerelle Load Rating	SUBJECT	FB 6 btw L & N @ Span 6
BY	T. F. Ko DATE 10/8/2013		24WF145 FB 6-2
CHECKED BY	Jason A DATE 10/10/13	PAGE	1 OF 9

→ Section : FB 24WF 145 (FB 6-2)

- assume there is no vehicle load, right at the ramp.



* assume

- Load from stringer 10WF21 :

DL: $\left[(5.333)(44/6)(50) + (21)(44/6)\right](1.05) = 2.215$ kips.

LL: $(5.333)(44/6)(100) = 3.91$ kips

SDL: $(5.333)(44/6)(10) = 0.39$ kips

- Load from M24 WF 76

Tri Area : $(1/2)(44/2)(7/9) = 137\ ft^2$ * assume the stringer 10WF21 has about 7/9 of area.

DL : $\left[\left[(16/7)(30) + (44/3)(76) + (50)(137.00) + (44/3)(21)(1.5)\right]1.05 = 9.69$ kips

SDL : $(137)(10) = 1.37$ kips

PL : $(137)(100) = 13.7$ kips

- Load from L24WF 76

Tri. Area : $(13.5/2)(44/2)(7/9) = 116\ ft^2$

DL : $\left[(13.5/2)(30) + (44/2)(76) + (116)(50) + (50 +)(13.5)(3/4) + (21)(44/3)(1.5)\right]1.05$

202.5 1672 5806.25 467

$= 8.76$ kips

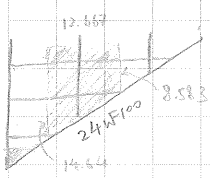
SDL = $(116)(10) = 1.16$ kips

PL = $(116)(100) = 11.6$ kips

WEIDLINGER

PROJECT		SUBJECT	
BY TF Ko	DATE 10/8/13	FB6-2	
CHECKED BY Jason A.	DATE 10/10/13	PAGE 2	OF 9

- Load from 24WF100



Tri Area: $\frac{(14.64 + 8.583)(12.667)}{2} + \frac{(6.33)(3)}{2} \approx 157\ ft^2$

993.5 675 210 7850

DL: $[(38/3)(21)(3.75) + (25/2)(50) + (42/2)(100) + (157)(50)](1.05) = 12.15$ kips

SDL: $(157)(10) = 1.57$ kips

PL: $(157)(100) = 15.7$ kips

- Load from Stringer 10WF21 (assume all about S = 5'-4" & 4'-6" & L = 44' or 38')

Span 7:

DL: $(44/6)[21 + (5.33)(50)](1.05) = 2.21$ kips

SDL: $(44/6)(5.33)(10) = 0.39$ kips

LL: $(44/6)(5.33)(100) = 3.9$ kips

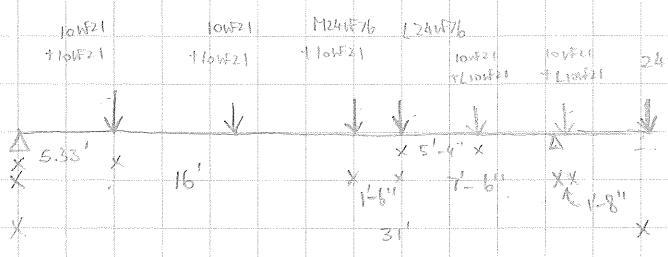
Span 8:

DL: $(38/6)[21 + (5.33)(50)]1.05 = 1.91$ kips

SDL: $(38/6)(5.33)(10) = 0.38$ kips

LL: $(38/6)(5.33)(100) = 3.8$ kips

- Load on floorbeam 24WF145



Lower Ramp: = 1.9 kips

DL: $(44/6)[21 + (4.5)(50)](1.05)$

SDL: $(44/6)(4.5)(10) = 0.33$ kips

LL: $(44/6)(4.5)(100) = 3.3$ kips

$w_{DL} = (.145)(1.05) = .15225$

kips							
DL:	4.12	4.12	11.6	8.76	3.81	3.81	20.91
SDL:	0.77	0.77	1.75	1.16	0.71	0.71	2.73
LL:	7.7	7.7	17.5	11.6	7.1	7.1	27.3

→ Input into S-Beam

WEIDLINGER

PROJECT		SUBJECT	
BY TFKo	DATE 10/8/13		FB6-2
CHECKED BY Jason A.	DATE 10/10/13	PAGE 3	OF 9

→ Shear & Moment Result from S-Beam:

max @ support

M_{DL} = 134.55 k-ft

M_{SDL} = 17.56 k-ft.

M_{LL} = 175.64 k-ft.

V_{DL} = 26.73 k

V_{SDL} = 3.73 k

V_{LL} = 37.33 k

→ Load Rating:

Inventory: $L_{RM} = \dfrac{18ksi \times 372.5 in^3/12 - (134.55 + 17.56)}{175.64} = 2.315$

$L_{RV} = \dfrac{11ksi \times 24.49'' \times .608'' - (26.73 + 3.73)}{37.33} = 3.572$

Operating: $L_{RM} = \dfrac{24.5ksi \times 372.5 in^3/12 - (134.55 + 17.56)}{175.64} = 3.464$

$L_{RV} = \dfrac{15ksi \times 24.49'' \times .608'' - (26.73 + 3.73)}{37.33} = 5.167$

Load Diagram

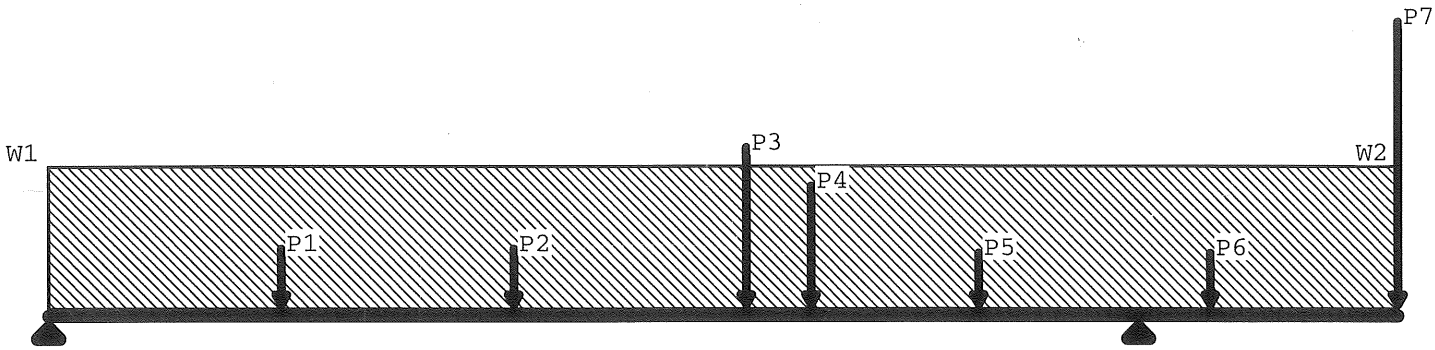
4 of 9

DL

RAM SBeam v4.0
Licensed to: RAM International

10/11/13 15:57:49

Span information (ft): I-End (0.00,0.00) J-End (31.00,0.00)

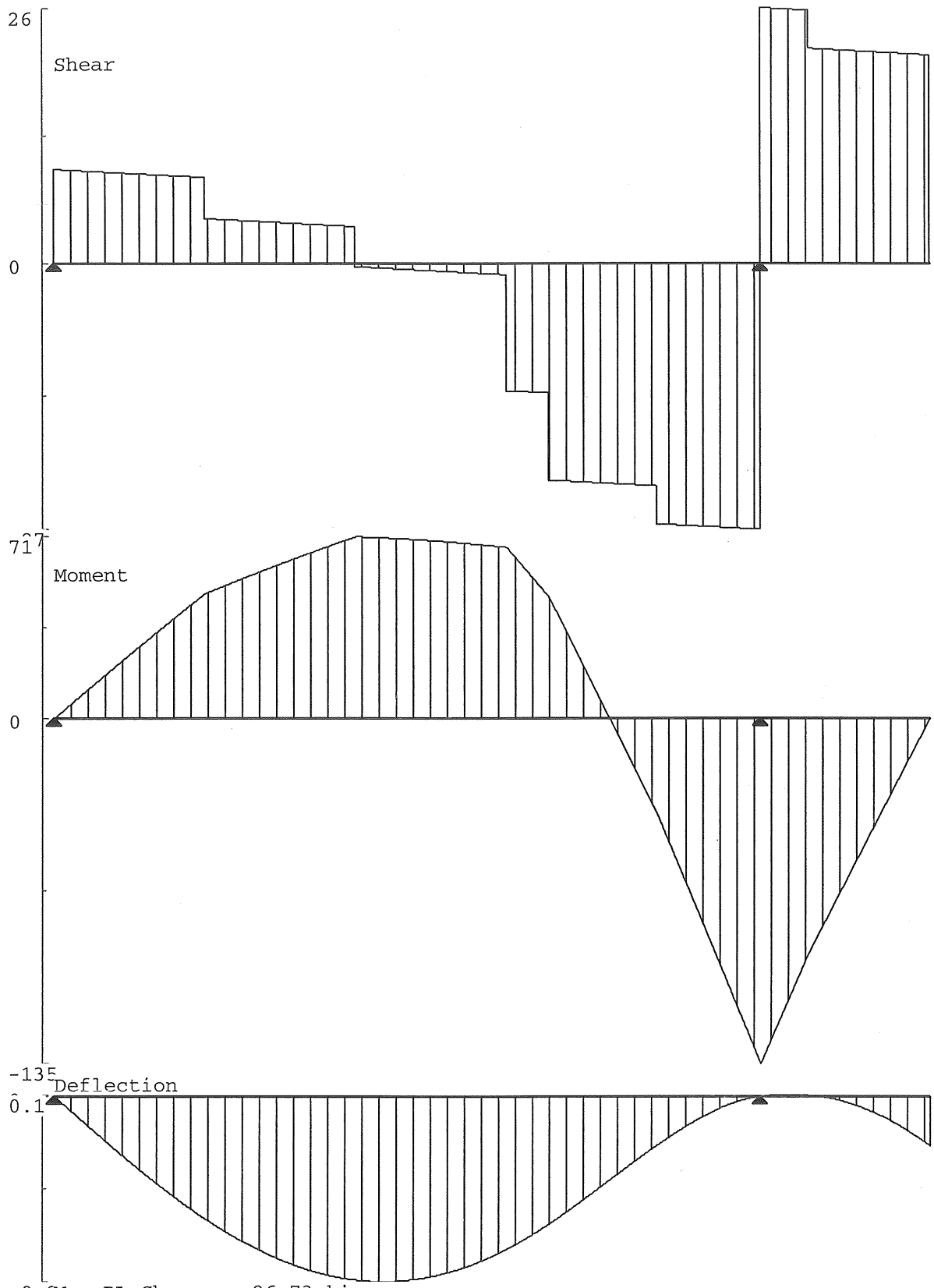


Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	5.330	4.120	0.000	0.000	4.120
P2	10.666	4.120	0.000	0.000	4.120
P3	16.000	11.600	0.000	0.000	11.600
P4	17.500	8.760	0.000	0.000	8.760
P5	21.333	3.810	0.000	0.000	3.810
P6	26.667	3.810	0.000	0.000	3.810
P7	31.000	20.910	0.000	0.000	20.910
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.152	0.000	0.000	0.152
W2	31.000	0.152	0.000	0.000	0.152

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

DL 5 of 9

Span information (ft): I-End (0.00,0.00) J-End (31.00,0.00)



Max DL Shear = 26.73 kips
Max Shear = 26.73 kips

Max Pos Moment = 70.53 kip-ft at 10.666 ft
Max Neg Moment = -134.55 kip-ft at right support

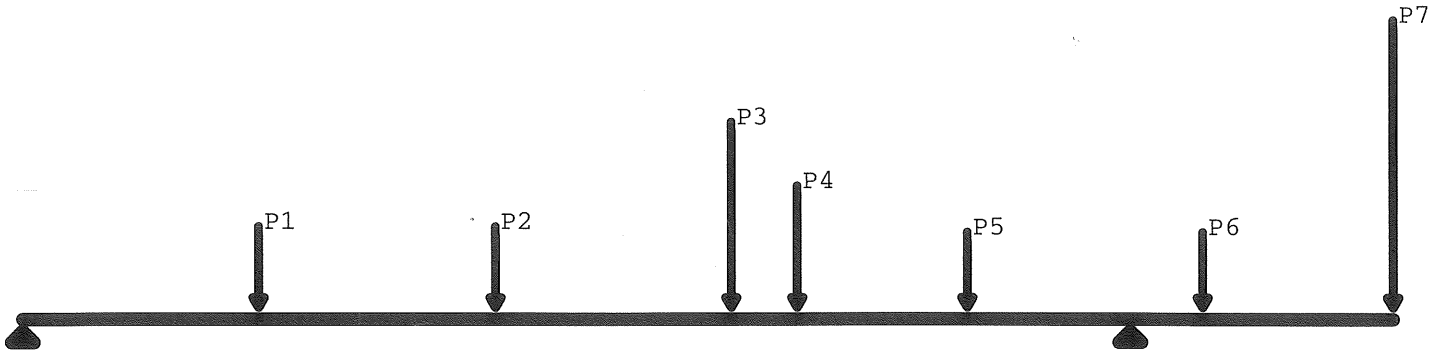
Load Diagram

SDL 6 of 9

RAM SBeam v4.0
Licensed to: RAM International

10/11/13 15:59:25

Span information (ft): I-End (0.00,0.00) J-End (31.00,0.00)

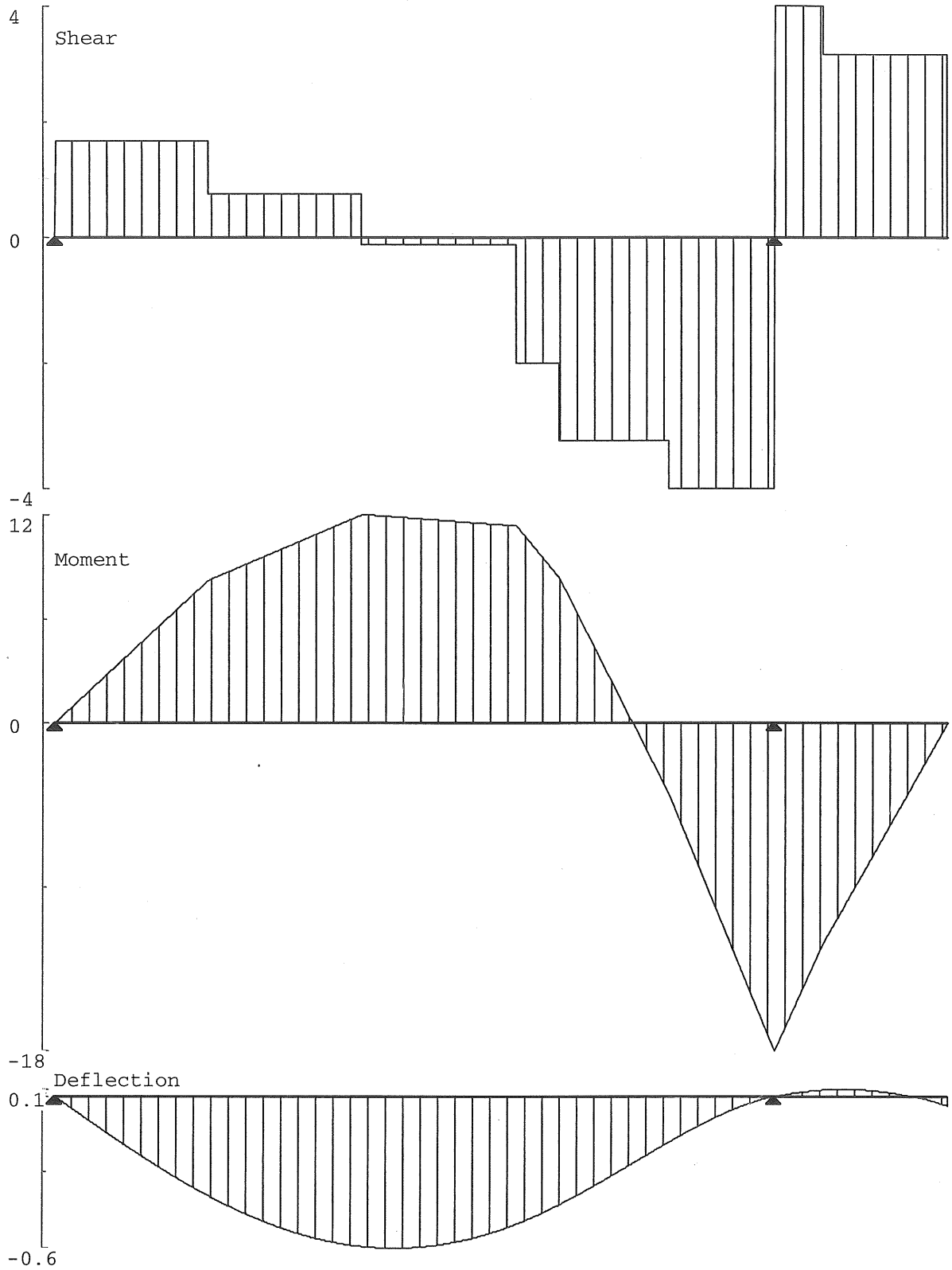


Load	Dist ft	~~DL~~ SDL kips	LL+ kips	LL- kips	Max Tot kips
P1	5.330	0.770	0.000	0.000	0.770
P2	10.666	0.770	0.000	0.000	0.770
P3	16.000	1.750	0.000	0.000	1.750
P4	17.500	1.160	0.000	0.000	1.160
P5	21.333	0.710	0.000	0.000	0.710
P6	26.667	0.710	0.000	0.000	0.710
P7	31.000	2.730	0.000	0.000	2.730

SDL 7 of 9

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (31.00,0.00)



Max DL Shear = 3.73 kips
Max Shear = 3.73 kips

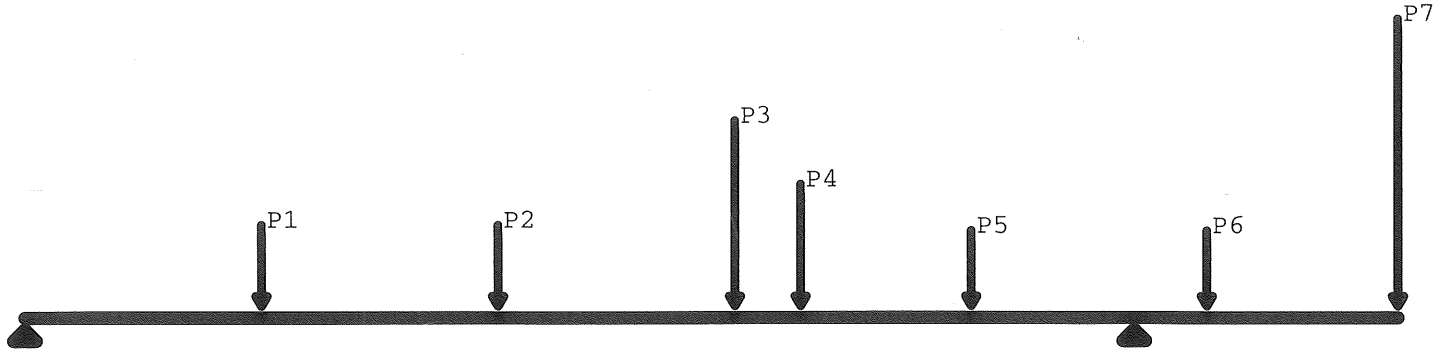
Max Pos Moment = 11.11 kip-ft at 10.666 ft
Max Neg Moment = -17.56 kip-ft at right support

Load Diagram

RAM SBeam v4.0
Licensed to: RAM International

11/12/13 11:24:34

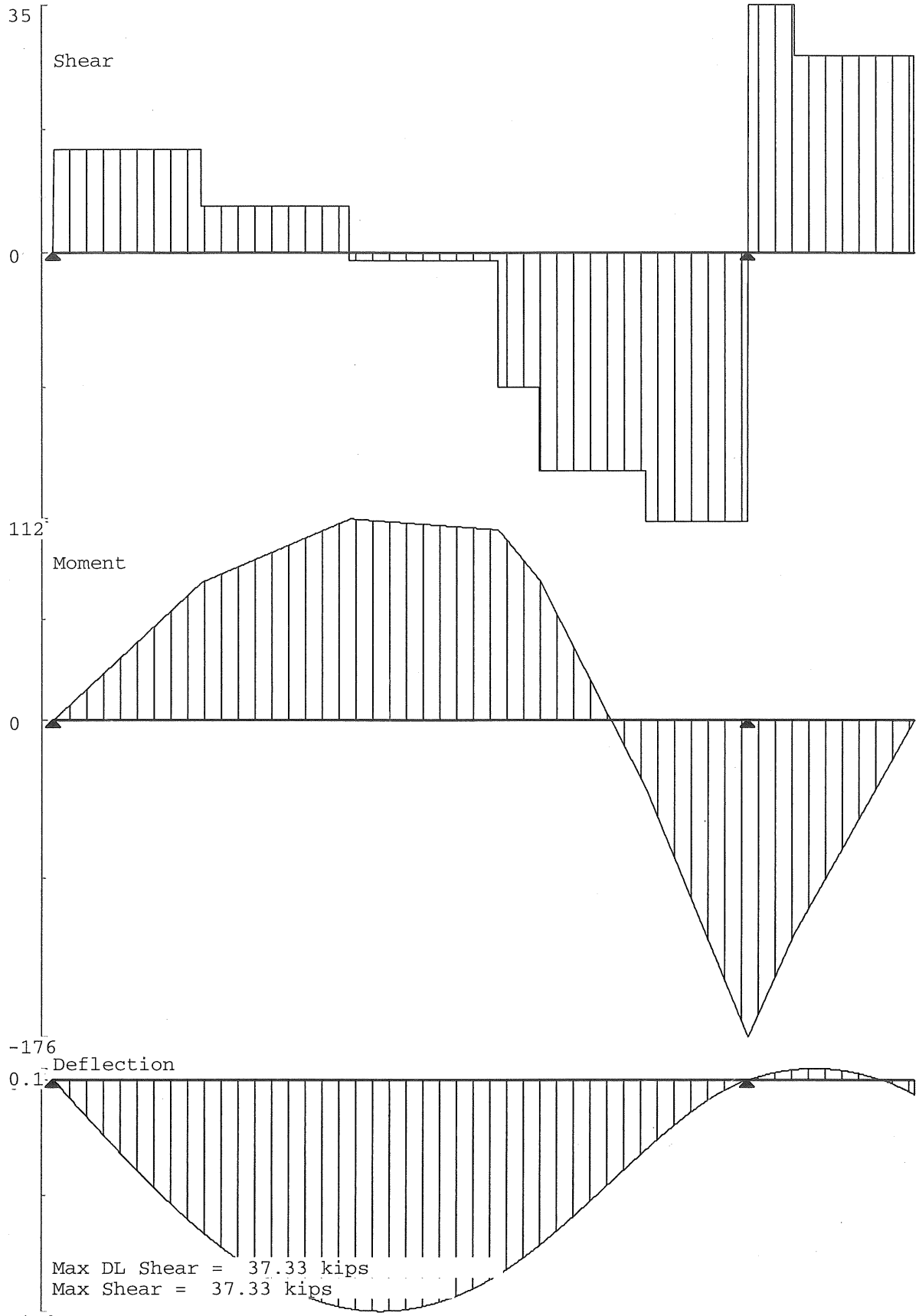
Span information (ft): I-End (0.00,0.00) J-End (31.00,0.00)



Load	Dist	~~DL~~ LL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	5.330	7.700	0.000	0.000	7.700
P2	10.666	7.700	0.000	0.000	7.700
P3	16.000	17.500	0.000	0.000	17.500
P4	17.500	11.600	0.000	0.000	11.600
P5	21.333	7.100	0.000	0.000	7.100
P6	26.667	7.100	0.000	0.000	7.100
P7	31.000	27.300	0.000	0.000	27.300

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (31.00,0.00)



Max DL Shear = 37.33 kips
Max Shear = 37.33 kips

Max Pos Moment = 111.11 kip-ft at 10.666 ft
Max Neg Moment = -175.64 kip-ft at right support

WEIDLINGER

PROJECT Passerelle Bridge	SUBJECT Floorbeam 6A-3Btw
BY TUNG FOR KO DATE 10/8/2013	Span 7 Column Lines L & M
CHECKED BY Jason A. DATE 10/10/13	PAGE 1 OF 7

Floorbeam 16 WF 50 (Bent 6, ~~FB~~ 6A-3) $d = 16.25''$ $t_w = .38''$

Span 6A = 12.67' FB length = 25' $t_f = .628''$ $b_f = 7.073''$

Span 6B = 12.67'

- Load from Stringers: L = 12.667' L = 7.55'

Stringer 10WF21

W_{DL} : $[(150/1000)(5.333) + .021]1.05 = .302$ k/ft

W_{SDL} : $(10/1000)(5.333) = .053$ k/ft

W_{LL} : $(100/1000)(5.333) = 0.53$ k/ft

W_{H-10} : = 20 k

$V_{DL} = \frac{W_{DL}\ell}{2} = \frac{(.302)(12.667)}{2} = 1.91$ k

$V_{SDL} = \frac{W_{SDL}\ell}{2} = \frac{(.053)(12.667)}{2} = 0.338$ k

$V_{LL} = \frac{W_{LL}\ell}{2} = \frac{(.53)(12.667)}{2} = 3.357$ K

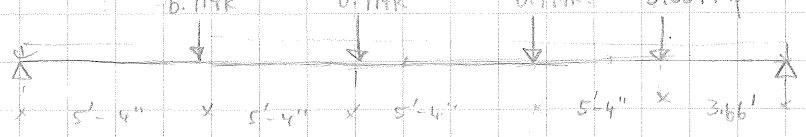
$V_{H-10} = 20/2 = 10$ k

$V_{DL} = \frac{W_{DL}\ell}{2} = \frac{(.302)(7.55)}{2} = 1.14$ k

$V_{SDL} = \frac{W_{SDL}\ell}{2} = \frac{(.053)(7.55)}{2} = 0.2$ k

$V_{LL} = \frac{W_{LL}\ell}{2} = \frac{(.53)(7.55)}{2} = 2$ kip

	10WF21 + 10WF21	" "	" "	" + 10WF21 (7.55')
DL	3.83k	3.83k	3.83k	3.05k
SDL	0.68k	0.68k	0.68k	0.54k
LL	6.714k	6.714k	6.714k	5.537 kip



$W_{DL} = .0525$ k/ft

* From S-Beam Results, moment @ mid-span:

M_{DL} = 57.53 k-ft (max 58.59 k-ft @ 10.67')

M_{SDL} = 9.49 k-ft (max 9.69 k-ft @ 10.67')

M_{LL} = 94.07 k-ft (max 96.03 k-ft @ 10.67')

M_{H-10} = (25/2 - 2.8') (10k) = 97 k-ft @ mid-span → govern

V_{DL} = 8.15 k

V_{SDL} = 1.33 k

V_{LL} = 13.32 k → govern

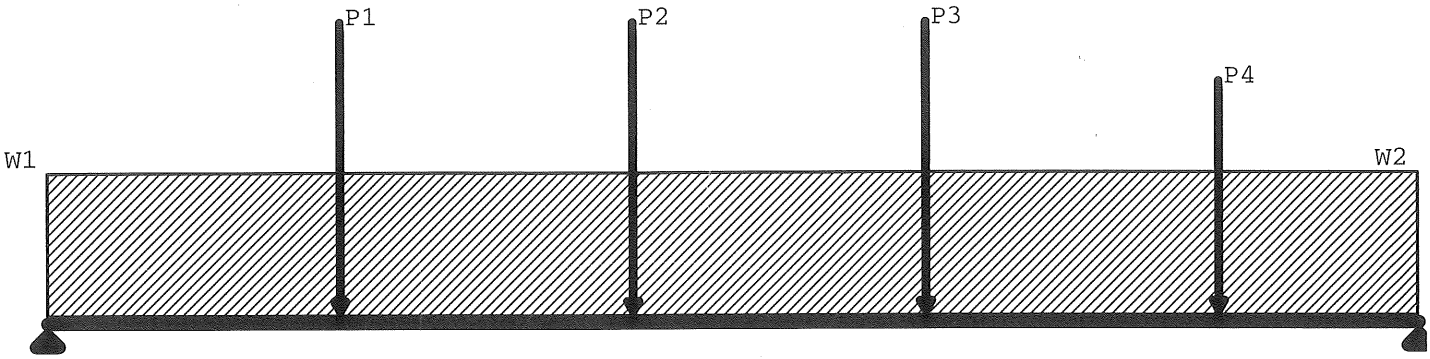
V_{H-10} = 10 k

Load Diagram

RAM SBeam v4.0
Licensed to: RAM International

10/08/13 14:28:1

Span information (ft): I-End (0.00,0.00) J-End (25.00,0.00)



Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	5.333	3.830	0.000	0.000	3.830
P2	10.666	3.830	0.000	0.000	3.830
P3	16.000	3.830	0.000	0.000	3.830
P4	21.333	3.050	0.000	0.000	3.050
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.052	0.000	0.000	0.052
W2	25.000	0.052	0.000	0.000	0.052

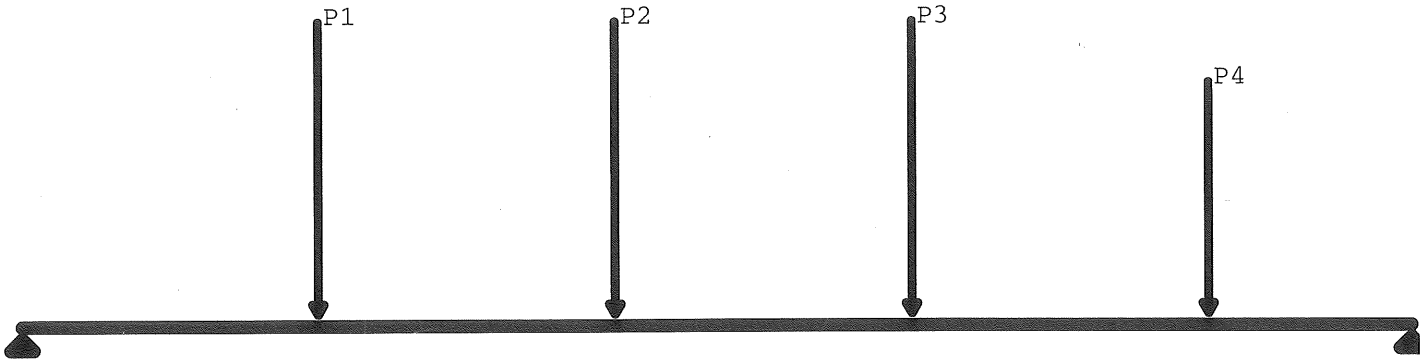
Load Diagram

SDL

RAM SBeam v4.0
Licensed to: RAM International

10/08/13 14:28:5

Span information (ft): I-End (0.00,0.00) J-End (25.00,0.00)



Load	Dist ft	SDL kips	LL+ kips	LL- kips	Max Tot kips
P1	5.333	0.680	0.000	0.000	0.680
P2	10.666	0.680	0.000	0.000	0.680
P3	16.000	0.680	0.000	0.000	0.680
P4	21.333	0.540	0.000	0.000	0.540

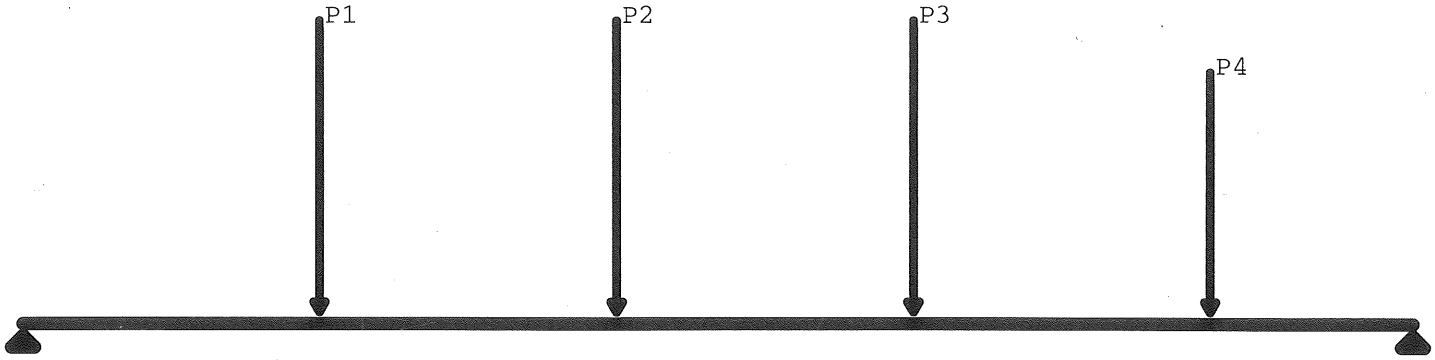
Load Diagram

RAM SBeam v4.0
Licensed to: RAM International

LL

11/12/13 11:35:41

Span information (ft): I-End (0.00,0.00) J-End (25.00,0.00)



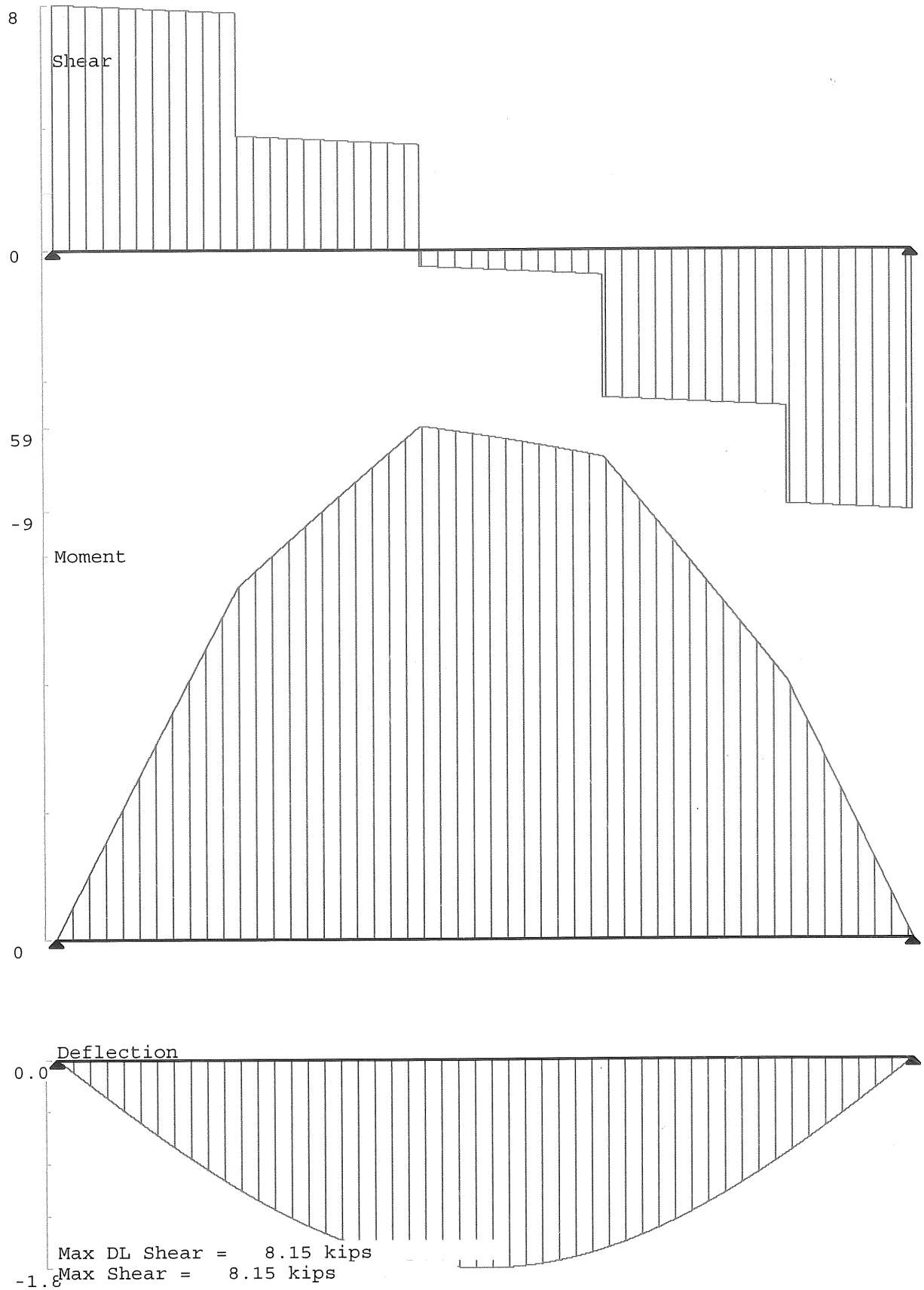
Load	Dist ft	~~DL~~ LL kips	LL+ kips	LL- kips	Max Tot kips
P1	5.333	6.714	0.000	0.000	6.714
P2	10.666	6.714	0.000	0.000	6.714
P3	16.000	6.714	0.000	0.000	6.714
P4	21.333	5.537	0.000	0.000	5.537

DL

5 of 7

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (25.00,0.00)



Max DL Shear = 8.15 kips

Max Shear = 8.15 kips

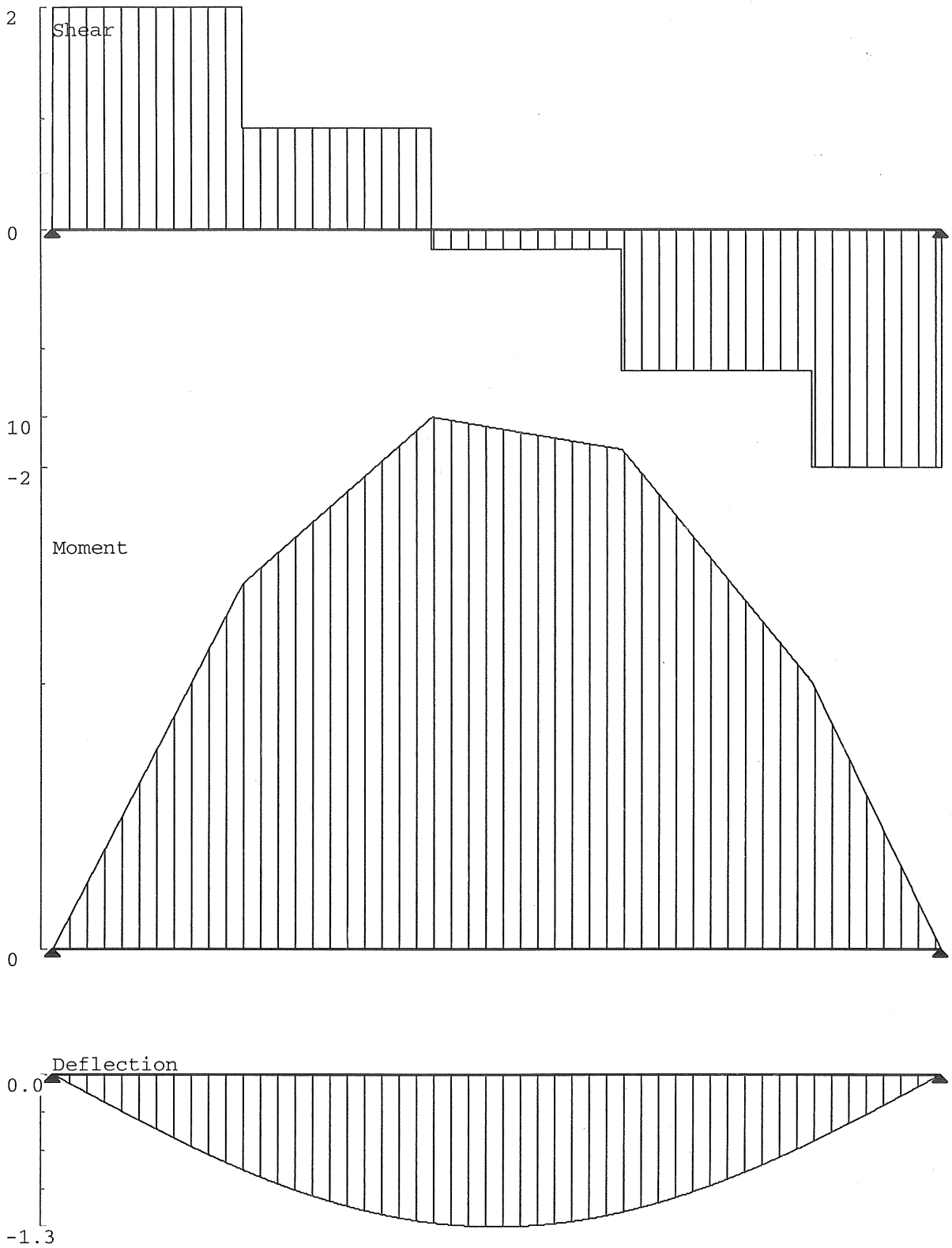
Max Pos Moment = 58.59 kip-ft at 10.666 ft

6 of 7

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (25.00,0.00)

SDL



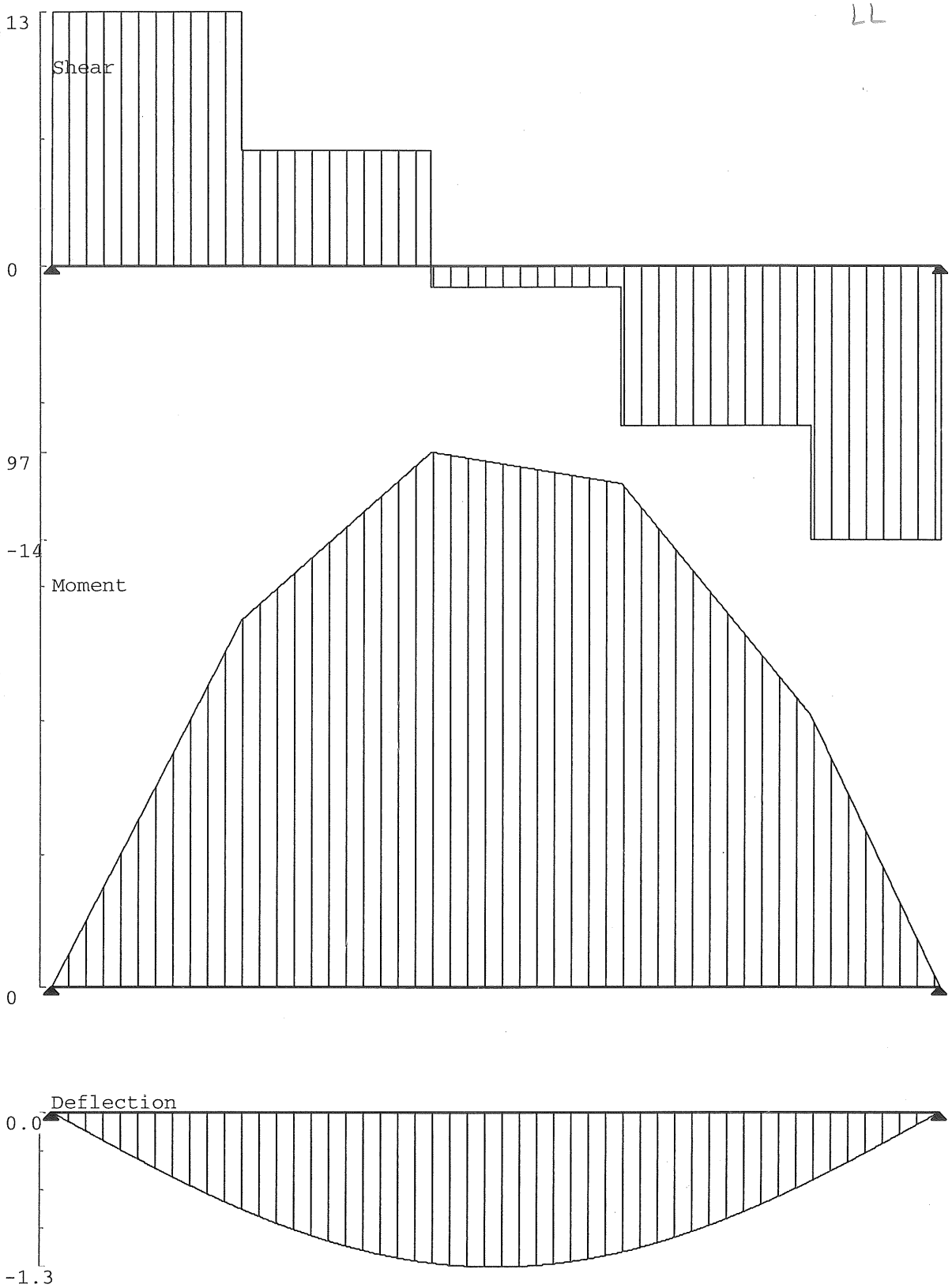
Max DL Shear = 1.33 kips
Max Shear = 1.33 kips

Max Pos Moment = 9.69 kip-ft at 10.666 ft

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (25.00,0.00)

LL



Max DL Shear = 13.32 kips
Max Shear = 13.32 kips

Max Pos Moment = 96.03 kip-ft at 10.666 ft

WEIDLINGER

PROJECT			
BY	Tunta for KO	DATE	9/27/2013
CHECKED BY	Jason A.	DATE	10/10/13

SUBJECT	Floorbeam 6A Btw (FBGA-3)
	Span 7 Column Lines L & M
PAGE	7 OF 7

Load Rating 16 W 50 (Bent 6A-3)

Inventory $L_{RM} = \frac{18\,ksi \times 80.7\,in^3/12 - (57.53 + 9.49)}{94.07}$

$= 0.574$

$L_{RV} = \frac{11\,ksi \times 16.25'' \times .38'' - (8.15 + 1.33)}{13.32}$

$= 4.388$

Operating $L_{RM} = \frac{24.5 \times 80.7\,in^3/12 - (57.53 + 9.49)}{94.07}$

$= 1.039$

$L_{RV} = \frac{15 \times 16.25'' \times .38 - (8.15 + 1.33)}{13.32}$

$= 6.242$

Floorbeam 7 at Span 7, 36WF300

$L := 76.83\ ft$ $\quad timber := 50\ \frac{lbf}{ft^2}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$S1 := 5.333\ ft$

$S2 := 5.333\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$DF := \frac{5.333}{3.75} = 1.422$

Loading on Stringers:

Stringer 10WF21 with spacing = 5'-4":

$S := \frac{S1}{2} + \frac{S2}{2} = 5.333\ ft$ $\quad Ls1 := 12.667\ ft$ $\quad Ls2 := 11.635\ ft$

$$wdl := 1.05 \cdot \left(timber \cdot S + 0.021\ \frac{kip}{ft}\right) = 0.302\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.053\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.533\ \frac{1}{ft} \cdot kip$$

Span 7 Stringers:

$$Vdl := wdl \cdot \frac{Ls1}{2} = 1.913\ kip$$

$$Vsdl := wsdl \cdot \frac{Ls1}{2} = 0.338\ kip$$

$$Vped := wped \cdot \frac{Ls1}{2} = 3.378\ kip$$

Span 8 Stringers:

$$Vdl := wdl \cdot \frac{Ls2}{2} = 1.757\ kip$$

$$Vsdl := wsdl \cdot \frac{Ls2}{2} = 0.31\ kip$$

$$Vped := wped \cdot \frac{Ls2}{2} = 3.102\ kip$$

Stringer 10WF21 with spacing = 2.67':

$S := 2.67\ ft$ $\quad Ls2 := 11.917\ ft$

$$wdl := 1.05 \cdot \left(timber \cdot S + 0.021\ \frac{kip}{ft}\right) = 0.162\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.027\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.267\ \frac{1}{ft} \cdot kip$$

Span 8 Stringers:

$$Vdl := wdl \cdot \frac{Ls2}{2} = 0.967\ kip$$

$$Vsdl := wsdl \cdot \frac{Ls2}{2} = 0.159\ kip$$

$$Vped := wped \cdot \frac{Ls2}{2} = 1.591\ kip$$

Stringer 10WF21 with spacing = 4.3':

$$S := 4.3\ ft \qquad Ls2 := 11.635\ ft$$

$$wdl := 1.05 \cdot \left(timber \cdot S + 0.021\ \frac{kip}{ft}\right) = 0.248\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.043\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.43\ \frac{1}{ft} \cdot kip$$

Span 8 Stringers:

$$Vdl := wdl \cdot \frac{Ls2}{2} = 1.442\ kip$$

$$Vsdl := wsdl \cdot \frac{Ls2}{2} = 0.25\ kip$$

$$Vped := wped \cdot \frac{Ls2}{2} = 2.502\ kip$$

<u>Load on Girders</u>

Load on Girder 24WF76 at Span 7 East Side:

$$At := \left(38\ ft \cdot 16\ \frac{ft}{2}\right)\left(\frac{7}{9}\right) = 236.444\ ft^2$$

$$Vdl1 := timber \cdot At = 11.822\ kip$$

$$Vdl2 := 2 \cdot \frac{.021 \cdot kip}{ft} \cdot 12.67 \cdot ft + 1 \cdot 0.03 \cdot \frac{kip}{ft} \cdot 16\ ft + .076 \cdot \frac{kip}{ft} \cdot 38\ ft = 3.9\ kip$$

$$Vdl := 0.5 \cdot 1.05 \cdot (Vdl1 + Vdl2) = 8.254\ kip$$

$$Vsdl := 0.5 \cdot At \cdot sdead = 1.182\ kip$$

$$Vped := 0.5 \cdot At \cdot ped = 11.822\ kip$$

Load on Girder 24WF76 at Span 8 East Side:
Assume the tributary area is half of the area of Span 8 btw column and East edge:

$$At := \left(0.5 \cdot \frac{(34.906 \cdot ft \cdot (16 \cdot ft + 7 \cdot ft))}{2}\right)\left(\frac{7}{9}\right) = 156.107\ ft^2$$

$$Vdl1 := timber \cdot At = 7.805\ kip$$

$$Vdl2 := 2 \cdot \frac{.021 \cdot kip}{ft} \cdot 12.67 \cdot ft + 0.03 \cdot \frac{kip}{ft} \cdot 10.67\ ft + .076 \cdot \frac{kip}{ft} \cdot 34.91 \cdot ft = 3.505\ kip$$

$$Vdl := 0.5 \cdot 1.05 \cdot (Vdl1 + Vdl2) = 5.938\ kip$$
$$Vsdl := 0.5 \cdot At \cdot sdead = 0.781\ kip$$
$$Vped := 0.5 \cdot At \cdot ped = 7.805\ kip$$

Load on Girder 24WF100 at Span 7 West Side:
Assume the tributary area is half of the area of Span 7 btw column and West edge:

$$At := \left((0.5) \cdot \left(\frac{(38 \cdot ft \cdot (31 \cdot ft + 12.833 \cdot ft))}{2}\right)\right) \cdot \left(\frac{7}{9}\right) = 323.877\ ft^2$$

$$Vdl1 := timber \cdot At = 16.194\ kip$$

$$Vdl2 := 3 \cdot \frac{.021 \cdot kip}{ft} \cdot 12.67 \cdot ft + 0.5 \cdot 0.05 \cdot \frac{kip}{ft} \cdot 24.94 \cdot ft + 0.5 \cdot 0.036\ \frac{kip}{ft} \cdot 18.888 \cdot ft$$

$$Vdl2 := Vdl2 + .1 \cdot \frac{kip}{ft} \cdot 42\ ft = 5.962\ kip$$

$$Vdl := 0.5 \cdot 1.05 \cdot (Vdl1 + Vdl2) = 11.632\ kip$$
$$Vsdl := 0.5 \cdot At \cdot sdead = 1.619\ kip$$
$$Vped := 0.5 \cdot At \cdot ped = 16.194\ kip$$

Load on Girder 24WF76 at Span 8 West Side:
Assume the tributary area is half of the area of Span 8 btw column and West edge:

$$At := \left(0.5 \cdot \frac{(34.91 \cdot ft \cdot (7.66 \cdot ft + 12.833 \cdot ft))}{2}\right)\left(\frac{7}{9}\right) = 139.108\ ft^2$$

$$Vdl1 := timber \cdot At = 6.955\ kip$$

$$Vdl2 := 1.5 \cdot \frac{.021 \cdot kip}{ft} \cdot 12.42 \cdot ft + 1 \cdot 0.021 \cdot \frac{kip}{ft} \cdot 10.25 \cdot ft + 0.076 \cdot \frac{kip}{ft} \cdot 38.69 \cdot ft$$

$$Vdl2 = 3.547\ kip$$
$$Vdl := 0.5 \cdot 1.05 \cdot (Vdl1 + Vdl2) = 5.514\ kip$$
$$Vsdl := 0.5 \cdot At \cdot sdead = 0.696\ kip$$
$$Vped := 0.5 \cdot At \cdot ped = 6.955\ kip$$

Uniform dead load on Floorbeam 36WF300:

Floorbeam: $.3 \cdot \frac{kip}{ft} \cdot 1.05 = 0.315 \frac{kip}{ft}$

Load Rating

$Sx := 1105.1\ in^3$ $Fy := 33\ ksi$ $d := 36.72\ in$ $tw := .945 \cdot in$

Inventory:

$$LRm := \frac{18\ ksi \cdot Sx - (324.33 \cdot kip \cdot ft + 41.3\ kip \cdot ft)}{414.52\ kip \cdot ft} = 3.117$$

$$LRv := \frac{11\ ksi \cdot d \cdot tw - (27.90\ kip + 3.49\ kip)}{36.96\ kip} = 9.478$$

Operating:

$$LRm := \frac{24.5\ ksi \cdot Sx - (324.33 \cdot kip \cdot ft + 41.3\ kip \cdot ft)}{414.52\ kip \cdot ft} = 4.561$$

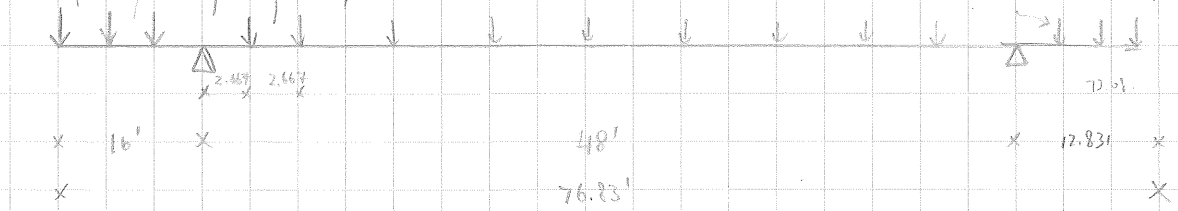
$$LRv := \frac{15\ ksi \cdot d \cdot tw - (27.90\ kip + 3.49\ kip)}{36.96\ kip} = 13.234$$

WEIDLINGER

PROJECT	SUBJECT
BY TUANG FOR KO DATE	
CHECKED BY Jason A DATE 10/11/13	PAGE 5 OF 11

36WF300

	24WF76 + 24WF76	10WF21 + 10WF21	10WF21	10WF21 + 16WF21								10WF21 + 10WF21	10WF21 + 10WF21	24WF100 + 24WF76
DL:	14.192	3.17	3.355	0.967	3.355	3.17						3.355	3.355	17.146
k SDL:	1.963	0.648	0.558	0.159	0.558	0.648						0.588	0.588	2.315
LL:	19.657	6.48	5.88	1.591	5.88	6.48						5.88	5.88	24

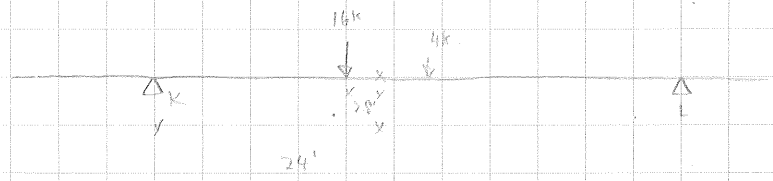


W_{DL} = 0.315 k/ft.

* assume the two 10WF21 + 10WF21 forces on the 12.83' cantilever part are the same

→ Moment & Shear from S-Beam:

@ K		@ L	
M_{DL}:	324.33 k-ft	M_{DL}:	289.86 k-ft
M_{SDL}:	41.3 k-ft	M_{SDL}:	37.62 k-ft
M_{pedl}:	414.57 k-ft	M_{pedl}:	400.27 k-ft
M_{HL-10}:	122 k-ft	M_{HL-10}:	122 k-ft
V_{DL}:	23.73 k	V_{DL}:	27.90 k
V_{SDL}:	2.81 k	V_{SDL}:	3.49 k
V_{pedl}:	36.96 k	V_{pedl}:	25.67 k
V_{HL-10}:	10k	V_{HL-10}:	10k



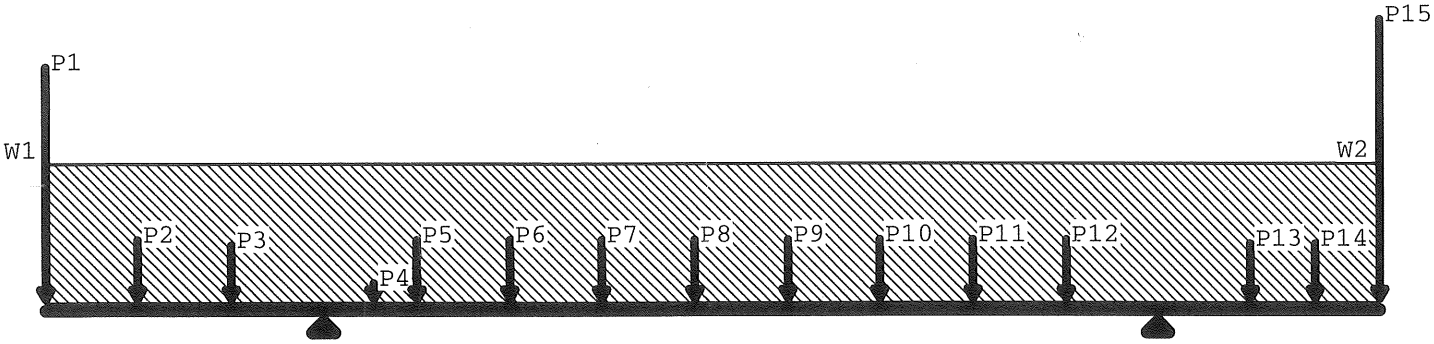
Load Diagram

DL

RAM SBeam v4.0
Licensed to: RAM International

10/11/13 11:24:11

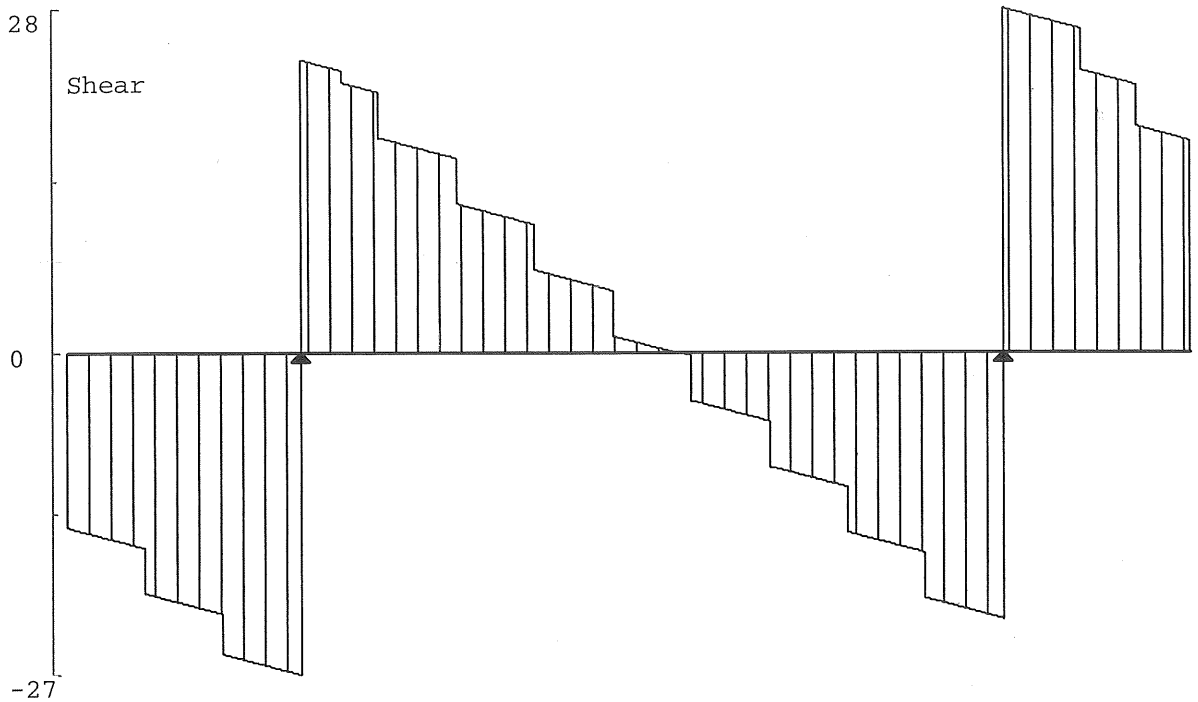
Span information (ft): I-End (0.00,0.00) J-End (76.83,0.00)

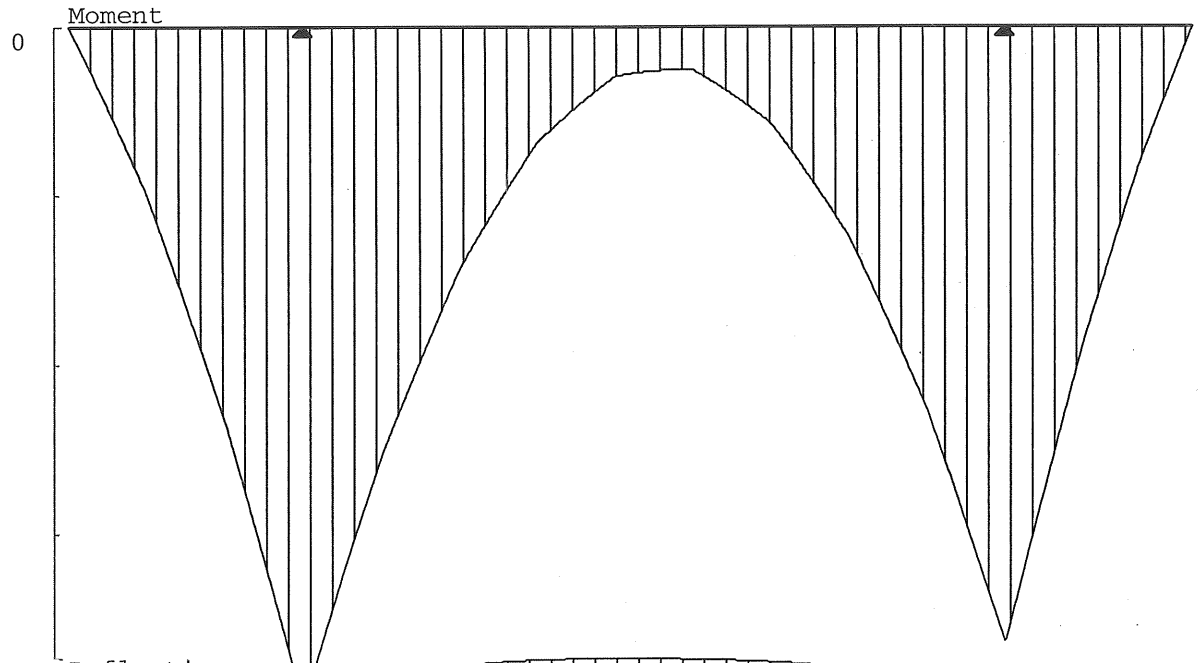


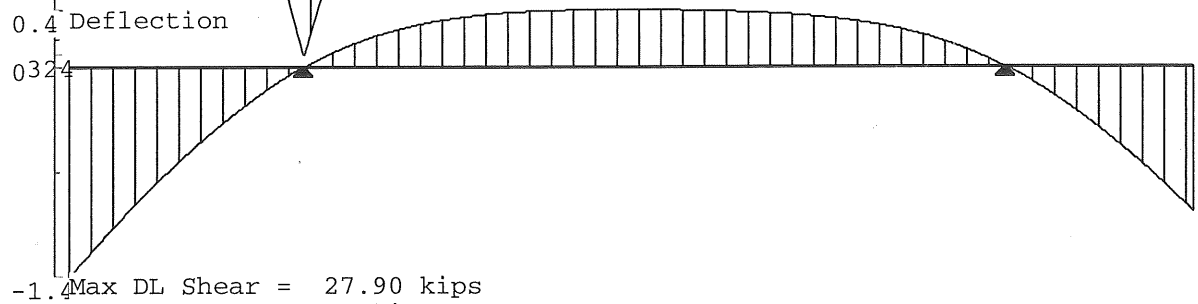
Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	0.000	14.192	0.000	0.000	14.192
P2	5.333	3.670	0.000	0.000	3.670
P3	10.666	3.355	0.000	0.000	3.263
P4	18.813	0.967	0.000	0.000	0.967
P5	21.333	3.355	0.000	0.000	3.670
P6	26.666	3.670	0.000	0.000	3.670
P7	32.000	3.670	0.000	0.000	3.670
P8	37.333	3.670	0.000	0.000	3.670
P9	42.666	3.670	0.000	0.000	3.670
P10	48.000	3.670	0.000	0.000	3.670
P11	53.333	3.670	0.000	0.000	3.670
P12	58.666	3.670	0.000	0.000	3.670
P13	69.333	3.355	0.000	0.000	3.355
P14	73.080	3.355	0.000	0.000	3.355
P15	76.830	17.146	0.000	0.000	17.146
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.315	0.000	0.000	0.315
W2	76.830	0.315	0.000	0.000	0.315

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams DL

Span information (ft): I-End (0.00,0.00) J-End (76.83,0.00)







Max DL Shear = 27.90 kips
Max Shear = 27.90 kips

Max Neg Moment = -324.33 kip-ft at left support

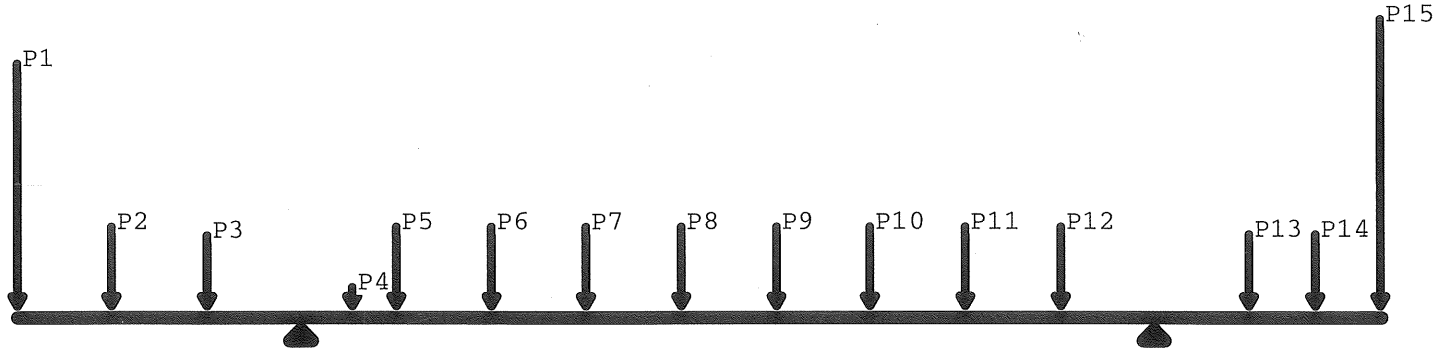
Load Diagram

RAM SBeam v4.0
Licensed to: RAM International

SDL

10/11/13 11:26:06

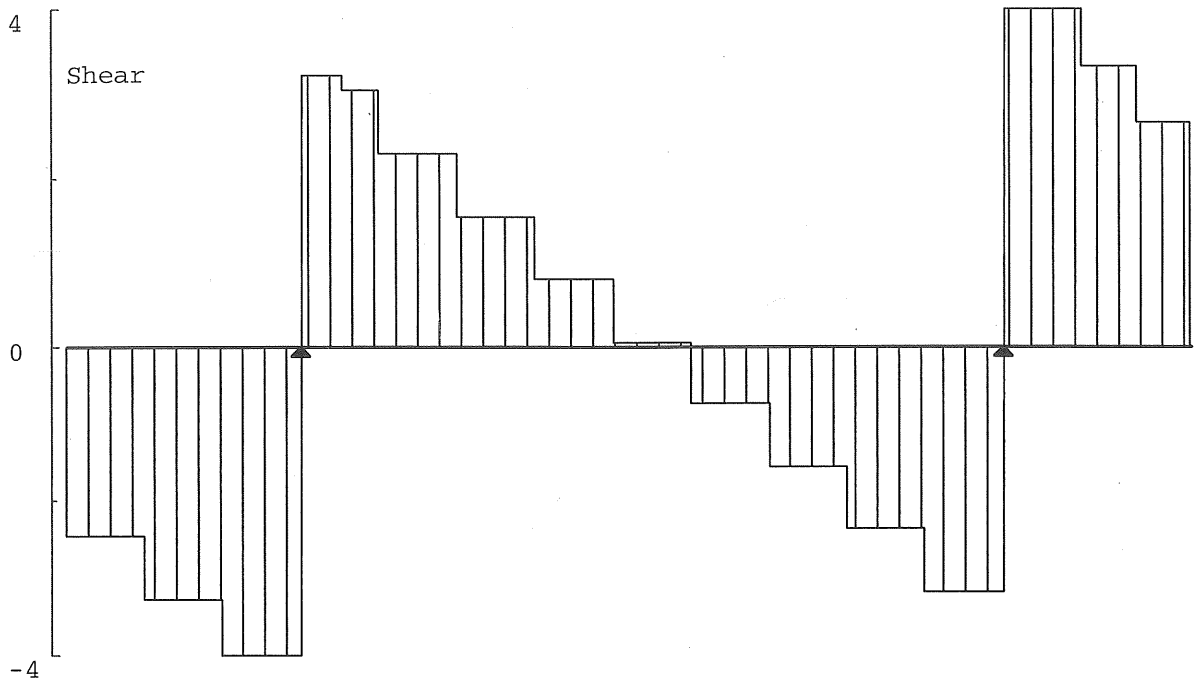
Span information (ft): I-End (0.00,0.00) J-End (76.83,0.00)

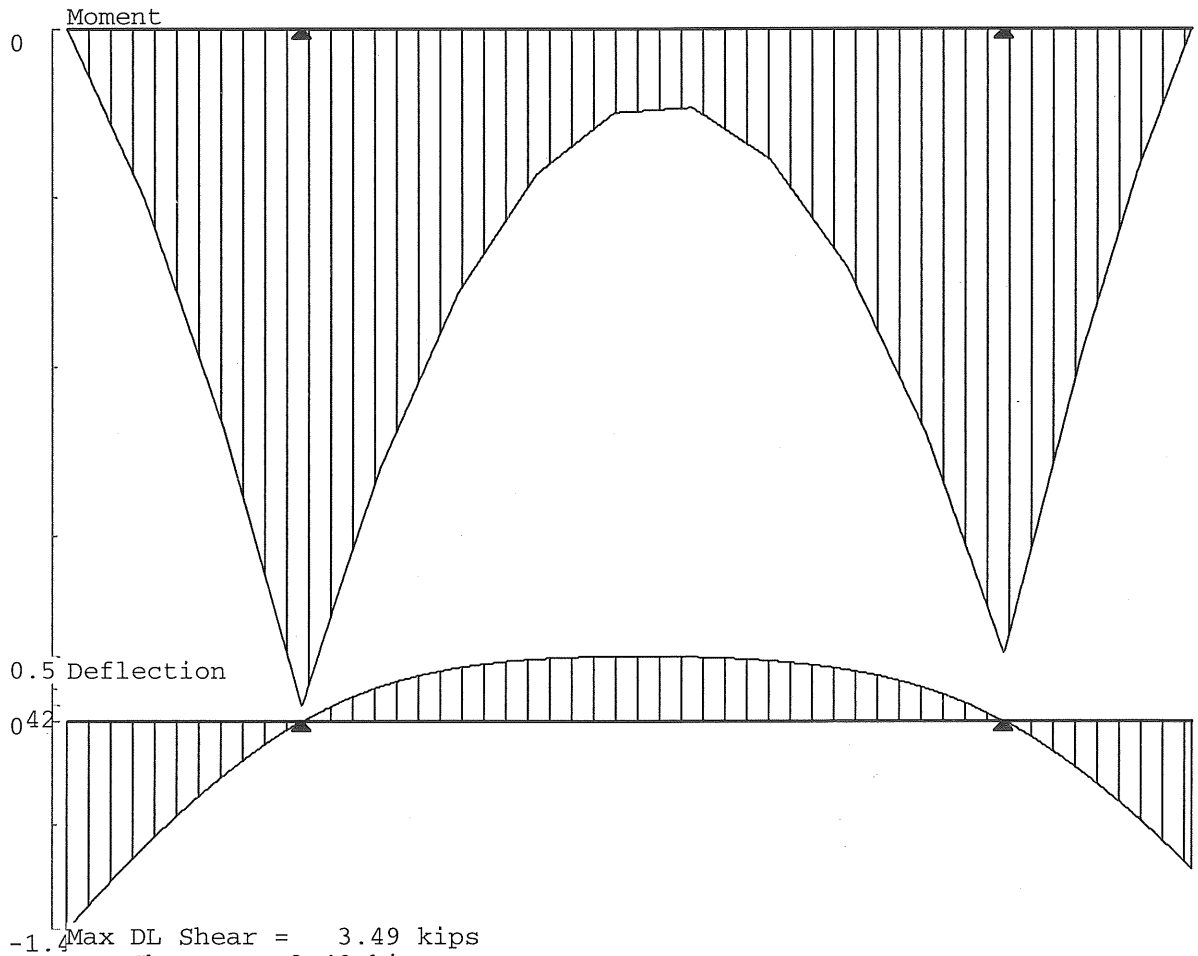


Load	Dist ft	~~DL~~ SDL kips	LL+ kips	LL- kips	Max Tot kips
P1	0.000	1.963	0.000	0.000	1.963
P2	5.333	0.648	0.000	0.000	0.648
P3	10.666	0.558	0.000	0.000	0.571
P4	18.813	0.159	0.000	0.000	0.159
P5	21.333	0.558	0.000	0.000	0.648
P6	26.666	0.648	0.000	0.000	0.648
P7	32.000	0.648	0.000	0.000	0.648
P8	37.333	0.648	0.000	0.000	0.648
P9	42.666	0.648	0.000	0.000	0.648
P10	48.000	0.648	0.000	0.000	0.648
P11	53.333	0.648	0.000	0.000	0.648
P12	58.666	0.648	0.000	0.000	0.648
P13	69.333	0.588	0.000	0.000	0.588
P14	73.080	0.588	0.000	0.000	0.588
P15	76.830	2.315	0.000	0.000	2.315

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams SDL 9 of 11

Span information (ft): I-End (0.00,0.00) J-End (76.83,0.00)





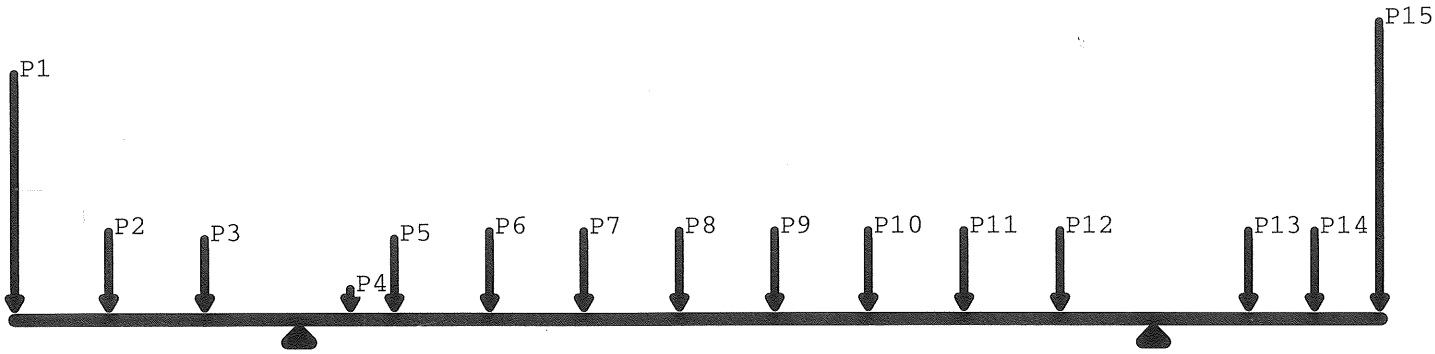
Max DL Shear = 3.49 kips
Max Shear = 3.49 kips

Max Neg Moment = -41.30 kip-ft at left support

Load Diagram

RAM SBeam v4.0
Licensed to: RAM International 11/12/13 13:19:51

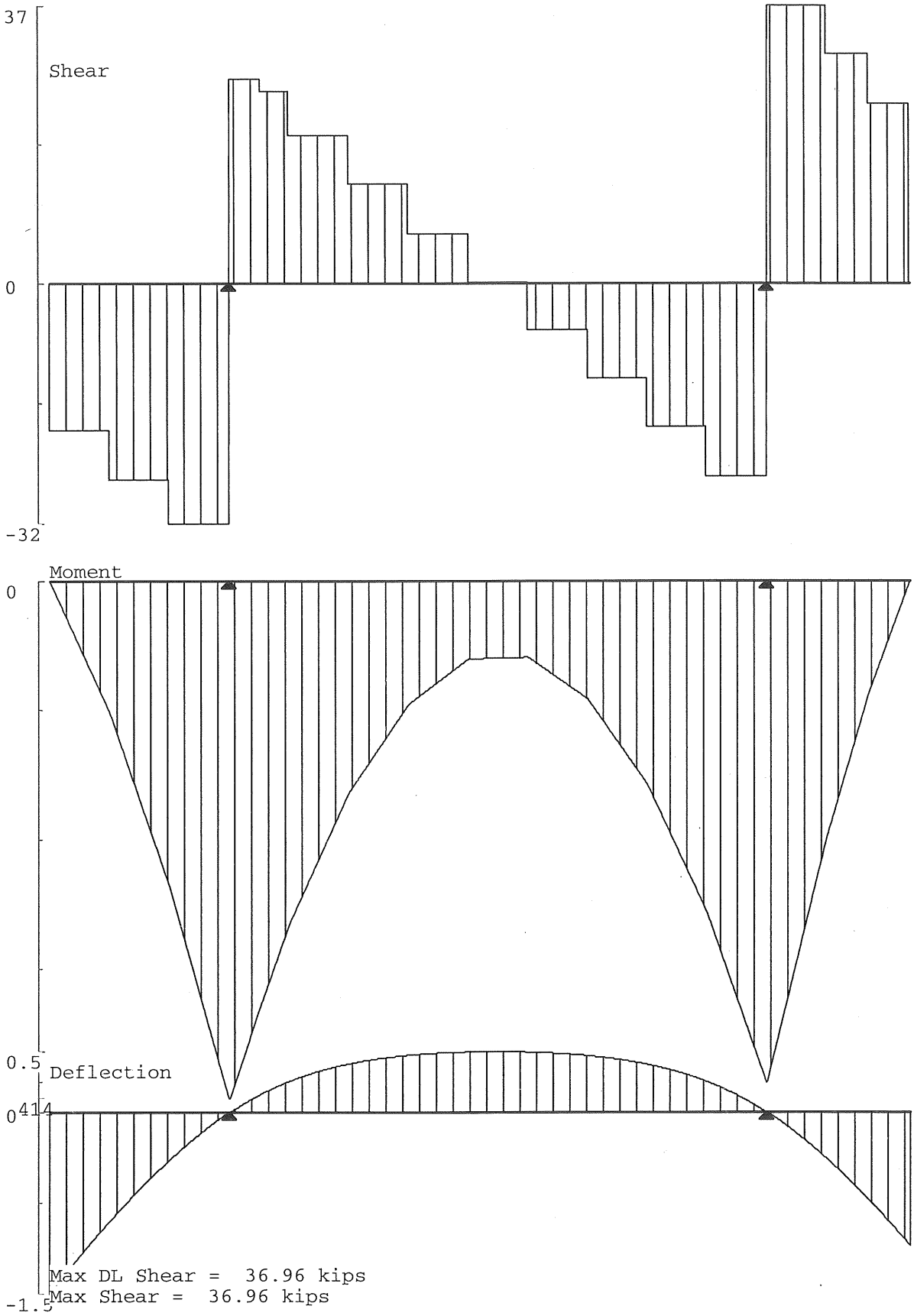
Span information (ft): I-End (0.00,0.00) J-End (76.83,0.00)



Load	Dist ft	~~DL~~ LL kips	LL+ kips	LL- kips	Max Tot kips
P1	0.000	19.627	0.000	0.000	19.627
P2	5.333	6.480	0.000	0.000	6.480
P3	10.666	5.880	0.000	0.000	5.880
P4	18.813	1.591	0.000	0.000	1.591
P5	21.333	5.880	0.000	0.000	5.880
P6	26.666	6.480	0.000	0.000	6.480
P7	32.000	6.480	0.000	0.000	6.480
P8	37.333	6.480	0.000	0.000	6.480
P9	42.666	6.480	0.000	0.000	6.480
P10	48.000	6.480	0.000	0.000	6.480
P11	53.333	6.480	0.000	0.000	6.480
P12	58.666	6.480	0.000	0.000	6.480
P13	69.333	6.480	0.000	0.000	6.480
P14	73.080	6.480	0.000	0.000	6.480
P15	76.830	24.000	0.000	0.000	24.000

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (76.83,0.00)



Max Neg Moment = -414.52 kip-ft at left support

Floorbeam 22 between G&Hat Span 22, 33WF141 FB 22-4

$L := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$S1 := 7.5\ ft$

$S2 := 7.5\ ft$

$ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

Loading on Stringers:

Stringer 24WF76:

$S := \frac{S1 + S2}{2} = 7.5\ ft$ $\quad Ls1 := 33.1615\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.076\ \frac{kip}{ft}\right) = 0.621\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 22 Stringers:

$Vdl76 := wdl \cdot \frac{Ls1}{2} = 10.3\ kip$

$Vsdl76 := wsdl \cdot \frac{Ls1}{2} = 1.244\ kip$

$Vped76 := wped \cdot \frac{Ls1}{2} = 12.436\ kip$

Stringer 24WF145:

$S := 7.5\ ft$ $\quad Ls1 := 33.1615\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.145\ \frac{kip}{ft}\right) = 0.694\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 22 Stringers:

$$Vdl145 := wdl \cdot \frac{Ls1}{2} = 11.501 \ kip$$

$$Vsdl145 := wsdl \cdot \frac{Ls1}{2} = 1.244 \ kip$$

$$Vped145 := wped \cdot \frac{Ls1}{2} = 12.436 \ kip$$

Canopy Shear force:

$$Pcan := 40 \ kip$$

$$Vcan145 := Pcan \cdot \left(\frac{10.477}{33.1615}\right) + Pcan \cdot \left(\frac{30.77}{33.1615}\right) = 49.753 \ kip$$

Stringer 12WF53:

$$S := 7.5 \ ft \qquad Ls1 := 10 \ ft$$

$$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5 \ in) + 0.053 \ \frac{kip}{ft}\right) = \qquad \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.075 \ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.75 \ \frac{1}{ft} \cdot kip$$

Span 23 Stringers:

$$Vdl53 := wdl \cdot \frac{Ls1}{2} = 2.985 \ kip$$

$$Vsdl53 := wsdl \cdot \frac{Ls1}{2} = 0.375 \ kip$$

$$Vped53 := wped \cdot \frac{Ls1}{2} = 3.75 \ kip$$

Stringer Load to Floorbeam:

At 7.5':

$Vdl7.5 := Vdl53 + Vdl76 = 13.285\ kip$

$Vsdl7.5 := Vsdl53 + Vsdl76 = 1.619\ kip$

$Vped7.5 := Vped53 + Vped76 = 16.186\ kip$

At 15':

$Vdl15 := Vdl145 = 11.501\ kip$

$Vsdl15 := Vsdl145 = 1.244\ kip$

$Vped15 := Vped145 = 12.436\ kip$

$Vcan15 := Vcan145 = 49.753\ kip$

At 22.5':

$Vdl22.5 := Vdl76 = 10.3\ kip$

$Vsdl22.5 := Vsdl76 = 1.244\ kip$

$Vped22.5 := Vped76 = 12.436\ kip$

Uniform dead load on Floorbeam:

Assume bracing self weight is uniform:

$$wdlfb := 1.05 \cdot \left(\frac{.141 \cdot kip}{ft}\right) + \frac{1.05 \cdot \frac{.0207 \cdot kip}{ft} \cdot 1 \cdot 7.5\ ft}{L} = 0.153\ \frac{kip}{ft}$$

Moment & Shear:

$$Mdl := \frac{wdlfb \cdot (L)^2}{8} + \left(\frac{Vdl7.5}{8} + \frac{Vdl15}{4} + \frac{Vdl22.5}{8}\right) \cdot L = 191.972\ ft \cdot kip$$

$$Msdl := \left(\frac{Vsdl7.5}{8} + \frac{Vsdl15}{4} + \frac{Vsdl22.5}{8}\right) \cdot L = 20.06\ ft \cdot kip$$

$$Mped := \left(\frac{Vped7.5}{8} + \frac{Vped15}{4} + \frac{Vped22.5}{8}\right) \cdot L = 200.596\ ft \cdot kip$$

$$Mcan := \frac{Vcan15 \cdot L}{4} = 373.147\ ft \cdot kip$$

$$Mh10 := \left(\frac{L}{2} - 2.8\ ft\right) \cdot 10\ kip = 122\ kip \cdot ft$$

$$Vdl := \frac{wdlfb \cdot L}{2} + Vdl7.5 \cdot \frac{3}{4} + Vdl15 \cdot \frac{1}{2} + Vdl22.5 \cdot \frac{1}{4} = 20.592\ kip$$

$$Vsdl := Vsdl7.5 \cdot \frac{3}{4} + Vsdl15 \cdot \frac{1}{2} + Vsdl22.5 \cdot \frac{1}{4} = 2.147\ kip$$

$$Vped := Vped7.5 \cdot \frac{3}{4} + Vped15 \cdot \frac{1}{2} + Vped22.5 \cdot \frac{1}{4} = 21.466\ kip$$

$$Vcan := \frac{Vcan15}{2} = 24.876\ kip$$

$$Vh10 := \frac{H10}{2} = 10\ kip$$

Load Rating

$Sx := 446.8\ in^3$ $Fy := 33\ ksi$ $d := 33.31\ in$ $tw := .605 \cdot in$

Inventory:

$$LRm := \frac{18\ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 0.424$$

$$LRv := \frac{11\ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 8.109$$

Operating:

$$LRm := \frac{24.5\ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 1.63$$

$$LRv := \frac{15\ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 11.864$$

Floorbeam 24 between G&Hat Span 24, 33WF152 FB 24-6

$L := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$S1 := 7.5\ ft$

$S2 := 7.5\ ft$

$ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

Loading on Stringers:

Stringer 24WF68 with spacing = 7.5':

$S := \frac{S1 + S2}{2} = 7.5\ ft$ $\quad Ls1 := 32.1615\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.068\ \frac{kip}{ft}\right) = 0.613\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 24 Stringers:

$Vdl68 := wdl \cdot \frac{Ls1}{2} = 9.854\ kip$

$Vsdl68 := wsdl \cdot \frac{Ls1}{2} = 1.206\ kip$

$Vped68 := wped \cdot \frac{Ls1}{2} = 12.061\ kip$

Stringer 24WF76 with spacing = 7.5':

$S := \frac{S1 + S2}{2} = 7.5\ ft$ $\quad Ls1 := 37\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.076\ \frac{kip}{ft}\right) = 0.621\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 25 Stringers:

$$Vdl76 := wdl \cdot \frac{Ls1}{2} = 11.492\ kip$$

$$Vsdl76 := wsdl \cdot \frac{Ls1}{2} = 1.388\ kip$$

$$Vped76 := wped \cdot \frac{Ls1}{2} = 13.875\ kip$$

Stringer 24WF110 with spacing = 7.5':

$$S := 7.5\ ft \qquad Ls1 := 32.1615\ ft$$

$$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.11\ \frac{kip}{ft}\right) = 0.657\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$$

Span 24 Stringers:

$$Vdl110 := wdl \cdot \frac{Ls1}{2} = 10.564\ kip$$

$$Vsdl110 := wsdl \cdot \frac{Ls1}{2} = 1.206\ kip$$

$$Vped110 := wped \cdot \frac{Ls1}{2} = 12.061\ kip$$

Canopy Shear force:

$$Pcan := 40\ kip$$

$$Vcan110 := Pcan \cdot \left(\frac{7.7876}{32.1615}\right) + Pcan \cdot \left(\frac{28.341}{32.1615}\right) = 44.934\ kip$$

Stringer 24WF145 with spacing = 7.5':

$$S := 7.5\ ft \qquad Ls1 := 37\ ft$$

$$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.145\ \frac{kip}{ft}\right) = 0.694\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$$

Span 24 Stringers:

$$Vdl145 := wdl \cdot \frac{Ls1}{2} = 12.833\ kip$$

$$Vsdl145 := wsdl \cdot \frac{Ls1}{2} = 1.388\ kip$$

$$Vped145 := wped \cdot \frac{Ls1}{2} = 13.875\ kip$$

Canopy Shear force:

$$Pcan := 40\ kip$$

$$Vcan145 := Pcan \cdot \left(1 - \frac{16.6578}{37}\right) = 21.992\ kip$$

Stringer Load to Floorbeam:

At 7.5':

$$Vdl7.5 := Vdl68 + Vdl76 = 21.347\ kip$$
$$Vsdl7.5 := Vsdl68 + Vsdl76 = 2.594\ kip$$
$$Vped7.5 := Vped68 + Vped76 = 25.936\ kip$$

At 15':

$$Vdl15 := Vdl110 + Vdl145 = 23.396\ kip$$
$$Vsdl15 := Vsdl110 + Vsdl145 = 2.594\ kip$$
$$Vped15 := Vped110 + Vped145 = 25.936\ kip$$
$$Vcan15 := Vcan110 + Vcan145 = 66.926\ kip$$

At 22.5':

$$Vdl22.5 := Vdl68 + Vdl76 = 21.347\ kip$$
$$Vsdl22.5 := Vsdl68 + Vsdl76 = 2.594\ kip$$
$$Vped22.5 := Vped68 + Vped76 = 25.936\ kip$$

Uniform dead load on Floorbeam:

Assume bracing self weight is uniform:

$$wdlfb := 1.05 \cdot \left(\frac{.152 \cdot kip}{ft}\right) + \frac{1.05 \cdot \frac{.0207 \cdot kip}{ft} \cdot 6 \cdot 7.5\ ft}{L} = 0.192\ \frac{kip}{ft}$$

Moment & Shear:

$$Mdl := \frac{wdlfb \cdot (L)^2}{8} + \left(\frac{Vdl7.5}{8} + \frac{Vdl15}{4} + \frac{Vdl22.5}{8}\right) \cdot L = 357.194\ ft \cdot kip$$

$$Msdl := \left(\frac{Vsdl7.5}{8} + \frac{Vsdl15}{4} + \frac{Vsdl22.5}{8}\right) \cdot L = 38.903\ ft \cdot kip$$

$$Mped := \left(\frac{Vped7.5}{8} + \frac{Vped15}{4} + \frac{Vped22.5}{8}\right) \cdot L = 389.03\ ft \cdot kip$$

$$Mcan := \frac{Vcan15 \cdot L}{4} = 501.942\ ft \cdot kip$$

$$Mh10 := \left(\frac{L}{2} - 2.8\ ft\right) \cdot 10\ kip = 122\ kip \cdot ft$$

$$Vdl := \frac{wdlfb \cdot L}{2} + Vdl7.5 \cdot \frac{1}{4} + Vdl15 \cdot \frac{1}{2} + Vdl22.5 \cdot \frac{3}{4} = 35.928\ kip$$

$$Vsdl := Vsdl7.5 \cdot \frac{1}{4} + Vsdl15 \cdot \frac{1}{2} + Vsdl22.5 \cdot \frac{3}{4} = 3.89\ kip$$

$$Vped := Vped7.5 \cdot \frac{1}{4} + Vped15 \cdot \frac{1}{2} + Vped22.5 \cdot \frac{3}{4} = 38.913\ kip$$

$$Vcan := \frac{Vcan15}{2} = 33.463\ kip$$

$$Vh10 := \frac{H10}{2} = 10\ kip$$

Load Rating

$Sx := 486.4\ in^3$ $Fy := 33\ ksi$ $d := 33.5\ in$ $tw := .635 \cdot in$

Inventory:

$$LRm := \frac{18\ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = -0.433$$

$$LRv := \frac{11\ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 4.131$$

Operating:

$$LRm := \frac{24.5\ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 0.244$$

$$LRv := \frac{15\ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 6.318$$

Floorbeam 25 between G & H at Span 25, 33WF141 FB25-7

$L := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$S1 := 7.5\ ft$

$S2 := 7.5\ ft$

$ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

Loading on Stringers:

Stringer 24WF76:

$S := \frac{S1 + S2}{2} = 7.5\ ft$ $\quad Ls1 := 37\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.076\ \frac{kip}{ft}\right) = 0.121\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 25 Stringers:

$Vdl76 := wdl \cdot \frac{Ls1}{2} = 11.492\ kip$

$Vsdl76 := wsdl \cdot \frac{Ls1}{2} = 1.388\ kip$

$Vped76 := wped \cdot \frac{Ls1}{2} = 13.875\ kip$

Stringer 24WF145:

$S := 7.5\ ft$ $\quad Ls1 := 37\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.145\ \frac{kip}{ft}\right) = \quad \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = \quad \frac{1}{ft} \cdot kip$

Span 25 Stringers:

$$Vdl145 := wdl \cdot \frac{Ls1}{2} = 12.833\ kip$$

$$Vsdl145 := wsdl \cdot \frac{Ls1}{2} = 1.388\ kip$$

$$Vped145 := wped \cdot \frac{Ls1}{2} = 13.875\ kip$$

Canopy Shear force:

$$Pcan := 40\ kip$$

$$Vcan145 := Pcan + Pcan \cdot \left(\frac{16.578}{37}\right) = 57.922\ kip$$

Stringer 12WF53:

$$S := 7.5\ ft \qquad Ls1 := 10\ ft$$

$$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.053\ \frac{kip}{ft}\right) = 0.597\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$$

Span 26 Stringers:

$$Vdl53 := wdl \cdot \frac{Ls1}{2} = 2.985\ kip$$

$$Vsdl53 := wsdl \cdot \frac{Ls1}{2} = 0.375\ kip$$

$$Vped53 := wped \cdot \frac{Ls1}{2} = 3.75\ kip$$

Stringer Load to Floorbeam:

At 7.5':

$Vdl7.5 := Vdl53 + Vdl76 = 14.78\ kip$

$Vsdl7.5 := Vsdl53 + Vsdl76 = 1.763\ kip$

$Vped7.5 := Vped53 + Vped76 = 17.625\ kip$

At 15':

$Vdl15 := Vdl145 = 12.833\ kip$

$Vsdl15 := Vsdl145 = 1.388\ kip$

$Vped15 := Vped145 = 13.75\ kip$

$Vcan15 := Vcan145 = 57.922\ kip$

At 22.5':

$Vdl22.5 := Vdl76 = 11.492\ kip$

$Vsdl22.5 := Vsdl76 = 1.388\ kip$

$Vped22.5 := Vped76 = 13.375\ kip$

Uniform dead load on Floorbeam:

Assume bracing self weight is uniform:

$$wdlfb := 1.05 \cdot \left(\frac{.141 \cdot kip}{ft}\right) + \frac{1.05 \cdot \frac{.0207 \cdot kip}{ft} \cdot 1.5 \cdot 7.5\ ft}{L} = 0.156\ \frac{kip}{ft}$$

Moment & Shear:

$$Mdl := \frac{wdlfb \cdot (L)^2}{8} + \left(\frac{Vdl7.5}{8} + \frac{Vdl15}{4} + \frac{Vdl22.5}{8}\right) \cdot L = 211.205\ ft \cdot kip$$

$$Msdl := \left(\frac{Vsdl7.5}{8} + \frac{Vsdl15}{4} + \frac{Vsdl22.5}{8}\right) \cdot L = 22.219\ ft \cdot kip$$

$$Mped := \left(\frac{Vped7.5}{8} + \frac{Vped15}{4} + \frac{Vped22.5}{8}\right) \cdot L = 222.188\ ft \cdot kip$$

$$Mcan := \frac{Vcan15 \cdot L}{4} = 434.416\ ft \cdot kip$$

$$Mh10 := \left(\frac{L}{2} - 2.8\ ft\right) \cdot 10\ kip = 122\ kip \cdot ft$$

$$Vdl := \frac{wdlfb \cdot L}{2} + Vdl7.5 \cdot \frac{3}{4} + \frac{Vdl15}{2} + Vdl22.5 \cdot \frac{1}{4} = 22.491 \ \ kip$$

$$Vsdl := Vsdl7.5 \cdot \frac{3}{4} + \frac{Vsdl15}{2} + Vsdl22.5 \cdot \frac{1}{4} = 2.363 \ \ kip$$

$$Vped := Vped7.5 \cdot \frac{3}{4} + \frac{Vped15}{2} + Vped22.5 \cdot \frac{1}{4} = 23.625 \ \ kip$$

$$Vcan := \frac{Vcan15}{2} = 28.961 \ \ kip$$

$$Vh10 := \frac{H10}{2} = 10 \ \ kip$$

Load Rating

$Sx := 446.8 \ in^3$ $\quad Fy := 33 \ ksi$ $\quad d := 33.31 \ in$ $\quad tw := .605 \cdot in$

Inventory:

$$LRm := \frac{18 \ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 0.011$$

$$LRv := \frac{11 \ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 7.105$$

Operating:

$$LRm := \frac{24.5 \ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 1.1$$

$$LRv := \frac{15 \ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 10.517$$

Floorbeam 26 between G & H at Span 26, 33WF141 FB26-7

$L := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$S1 := 7.5\ ft$

$S2 := 7.5\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

Loading on Stringers:

Stringer 12WF53:

$S := \frac{S1 + S2}{2} = 7.5\ ft$ $\quad Ls1 := 10\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.053\ \frac{kip}{ft}\right) = 0.597\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 26 Stringers:

$Vdl53 := wdl \cdot \frac{Ls1}{2} = 2.985\ kip$

$Vsdl53 := wsdl \cdot \frac{Ls1}{2} = 0.375\ kip$

$Vped53 := wped \cdot \frac{Ls1}{2} = 3.75\ kip$

Stringer 24WF160:

$S := 7.5\ ft$ $\quad Ls1 := 45.9583\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.16\ \frac{kip}{ft}\right) = 0.79\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 27 Stringers:

$$Vdl160 := wdl \cdot \frac{Ls1}{2} = 16.302\ kip$$

$$Vsdl160 := wsdl \cdot \frac{Ls1}{2} = 1.723\ kip$$

$$Vped160 := wped \cdot \frac{Ls1}{2} = 17.234\ kip$$

Canopy Shear force:

$$Pcan := 40\ kip$$

$$Vcan160 := Pcan \cdot \left(\frac{37.0783}{45.9583}\right) = 32.271\ kip$$

Stringer 24WF110:

$$S := 7.5\ ft \qquad Ls1 := 45.9683\ ft$$

$$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.11\ \frac{kip}{ft}\right) = 0.657\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$$

Span 27 Stringers:

$$Vdl110 := wdl \cdot \frac{Ls1}{2} = 15.098\ kip$$

$$Vsdl110 := wsdl \cdot \frac{Ls1}{2} = 1.724\ kip$$

$$Vped110 := wped \cdot \frac{Ls1}{2} = 17.238\ kip$$

Stringer Load to Floorbeam:

At 7.5':

$Vdl7.5 := Vdl53 + Vdl110 = 18.084\ kip$

$Vsdl7.5 := Vsdl53 + Vsdl110 = 2.099\ kip$

$Vped7.5 := Vped53 + Vped110 = 20.988\ kip$

At 15':

$Vdl15 := Vdl160 = 16.302\ kip$

$Vsdl15 := Vsdl160 = 1.723\ kip$

$Vped15 := Vped160 = 17.234\ kip$

$Vcan15 := Vcan160 = 32.271\ kip$

At 22.5':

$Vdl22.5 := Vdl110 = 15.098\ kip$

$Vsdl22.5 := Vsdl110 = 1.724\ kip$

$Vped22.5 := Vped110 = 17.238\ kip$

Uniform dead load on Floorbeam:

Assume bracing self weight is uniform:

$$wdlfb := 1.05 \cdot \left(\frac{.141 \cdot kip}{ft}\right) + \frac{1.05 \cdot \frac{.0207 \cdot kip}{ft} \cdot 1 \cdot 7.5\ ft}{L} = 0.153\ \frac{kip}{ft}$$

Moment & Shear:

$$Mdl := \frac{wdlfb \cdot (L)^2}{8} + \left(\frac{Vdl7.5}{8} + \frac{Vdl15}{4} + \frac{Vdl22.5}{8}\right) \cdot L = 263.392\ ft \cdot kip$$

$$Msdl := \left(\frac{Vsdl7.5}{8} + \frac{Vsdl15}{4} + \frac{Vsdl22.5}{8}\right) \cdot L = 27.261\ ft \cdot kip$$

$$Mped := \left(\frac{Vped7.5}{8} + \frac{Vped15}{4} + \frac{Vped22.5}{8}\right) \cdot L = 272.606\ ft \cdot kip$$

$$Mcan := \frac{Vcan15 \cdot L}{4} = 242.034\ ft \cdot kip$$

$$Mh10 := \left(\frac{L}{2} - 2.8\ ft\right) \cdot 10\ kip = 122\ kip \cdot ft$$

$$Vdl := \frac{wdlfb \cdot L}{2} + Vdl7.5 \cdot \frac{3}{4} + \frac{Vdl15}{2} + Vdl22.5 \cdot \frac{1}{4} = 27.79 \ \ kip$$

$$Vsdl := Vsdl7.5 \cdot \frac{3}{4} + \frac{Vsdl15}{2} + Vsdl22.5 \cdot \frac{1}{4} = 2.867 \ kip$$

$$Vped := Vped7.5 \cdot \frac{3}{4} + \frac{Vped15}{2} + Vped22.5 \cdot \frac{1}{4} = 28.668 \ \ kip$$

$$Vcan := \frac{Vcan15}{2} = 16.136 \ kip$$

$$Vh10 := \frac{H10}{2} = 10 \ kip$$

Load Rating

$Sx := 446.8 \ in^3$ $\quad Fy := 33 \ ksi$ $\quad d := 33.31 \ in$ $\quad tw := .605 \cdot in$

Inventory:

$$LRm := \frac{18 \ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 0.502$$

$$LRv := \frac{11 \ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 6.1$$

Operating:

$$LRm := \frac{24.5 \ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 1.39$$

$$LRv := \frac{15 \ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 8.912$$

Floorbeam 27 between G&Hat Span 27, 36WF170 FB 27-7

$L := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$S1 := 7.5\ ft$

$S2 := 7.5\ ft$

$ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

Loading on Stringers:

Stringer 24WF110:

$S := \frac{S1 + S2}{2} = 7.5\ ft$ $\quad Ls1 := 45.9583\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.11\ \frac{kip}{ft}\right) = 0.657\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 27 Stringers:

$Vdl110 := wdl \cdot \frac{Ls1}{2} = 15.095\ kip$

$Vsdl110 := wsdl \cdot \frac{Ls1}{2} = 1.723\ kip$

$Vped110 := wped \cdot \frac{Ls1}{2} = 17.234\ kip$

Stringer 24WF160:

$S := 7.5\ ft$ $\quad Ls1 := 45.9583\ ft$

$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.16\ \frac{kip}{ft}\right) = 0.709\ \frac{1}{ft} \cdot kip$

$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$

$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$

Span 27 Stringers:

$$Vdl160 := wdl \cdot \frac{Ls1}{2} = 16.302\ kip$$

$$Vsdl160 := wsdl \cdot \frac{Ls1}{2} = 1.723\ kip$$

$$Vped160 := wped \cdot \frac{Ls1}{2} = 17.234\ kip$$

Canopy Shear force:

$$Pcan := 40\ kip$$

$$Vcan160 := Pcan \cdot \left(1 - \frac{37.0783}{45.9583}\right) = 7.729\ kip$$

Stringer 24WF76:

$$S := 7.5\ ft \qquad Ls1 := 36.5\ ft$$

$$wdl := 1.05 \cdot \left(concrete \cdot S \cdot (5.5\ in) + 0.076\ \frac{kip}{ft}\right) = 0.621\ \frac{1}{ft} \cdot kip$$

$$wsdl := sdead \cdot S = 0.075\ \frac{1}{ft} \cdot kip$$

$$wped := ped \cdot S = 0.75\ \frac{1}{ft} \cdot kip$$

Span 28 Stringers:

$$Vdl76 := wdl \cdot \frac{Ls1}{2} = 11.337\ kip$$

$$Vsdl76 := wsdl \cdot \frac{Ls1}{2} = 1.369\ kip$$

$$Vped76 := wped \cdot \frac{Ls1}{2} = 13.688\ kip$$

Stringer Load to Floorbeam:

At 7.5':

$Vdl7.5 := Vdl76 + Vdl110 = 26.432\ kip$

$Vsdl7.5 := Vsdl76 + Vsdl110 = 3.092\ kip$

$Vped7.5 := Vped76 + Vped110 = 30.922\ kip$

At 15':

$Vdl15 := Vdl160 + Vdl76 = 27.639\ kip$

$Vsdl15 := Vsdl160 + Vsdl76 = 3.092\ kip$

$Vped15 := Vped160 + Vped76 = 30.922\ kip$

$Vcan15 := Vcan160 = 7.729\ kip$

At 22.5':

$Vdl22.5 := Vdl110 + Vdl76 = 26.432\ kip$

$Vsdl22.5 := Vsdl110 + Vsdl76 = 3.092\ kip$

$Vped22.5 := Vped110 + Vped76 = 30.922\ kip$

Uniform dead load on Floorbeam:

Assume bracing self weight is uniform:

$$wdlfb := 1.05 \cdot \left(\frac{.170 \cdot kip}{ft}\right) + \frac{1.05 \cdot \frac{.0207 \cdot kip}{ft} \cdot 6 \cdot 7.5\ ft}{L} = 0.211\ \frac{kip}{ft}$$

Moment & Shear:

$$Mdl := \frac{wdlfb \cdot (L)^2}{8} + \left(\frac{Vdl7.5}{8} + \frac{Vdl15}{4} + \frac{Vdl22.5}{8}\right) \cdot L = 429.279\ ft \cdot kip$$

$$Msdl := \left(\frac{Vsdl7.5}{8} + \frac{Vsdl15}{4} + \frac{Vsdl22.5}{8}\right) \cdot L = 46.383\ ft \cdot kip$$

$$Mped := \left(\frac{Vped7.5}{8} + \frac{Vped15}{4} + \frac{Vped22.5}{8}\right) \cdot L = 463.828\ ft \cdot kip$$

$$Mcan := \frac{Vcan15 \cdot L}{4} = 57.966\ ft \cdot kip$$

$$Mh10 := \left(\frac{L}{2} - 2.8\ ft\right) \cdot 10\ kip = 122\ kip \cdot ft$$

$$Vdl := \frac{wdlfb \cdot L}{2} + Vdl7.5 \cdot \frac{3}{4} + \frac{Vdl15}{2} + Vdl22.5 \cdot \frac{1}{4} = 43.418 \ \ kip$$

$$Vsdl := Vsdl7.5 \cdot \frac{3}{4} + \frac{Vsdl15}{2} + Vsdl22.5 \cdot \frac{1}{4} = 4.638 \ kip$$

$$Vped := Vped7.5 \cdot \frac{3}{4} + \frac{Vped15}{2} + Vped22.5 \cdot \frac{1}{4} = 46.383 \ \ kip$$

$$Vcan := \frac{Vcan15}{2} = 3.864 \ kip$$

$$Vh10 := \frac{H10}{2} = 10 \ kip$$

Load Rating

$Sx := 579.10 \ in^3$ $\quad Fy := 33 \ ksi$ $\quad d := 36.16 \ in$ $\quad tw := .680 \cdot in$

Inventory:

$$LRm := \frac{18 \ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 0.722$$

$$LRv := \frac{11 \ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 4.712$$

Operating:

$$LRm := \frac{24.5 \ ksi \cdot Sx - (Mdl + Msdl + Mcan)}{Mped} = 1.399$$

$$LRv := \frac{15 \ ksi \cdot d \cdot tw - (Vdl + Vsdl + Vcan)}{Vped} = 6.833$$

WEIDLINGER

PROJECT		SUBJECT	Floorbeam 36WF194
BY Jwon A.	DATE 9/20/13		Bent 28, FB 28-8
CHECKED BY TP Ko	DATE 10/11/2013		PAGE 1 OF 3

Floorbeam 36WF194 (Bent 28, FB 28-8)

Span 28 = 36'-6" FB length = 30'-0"

Span 29 = 47'-6"

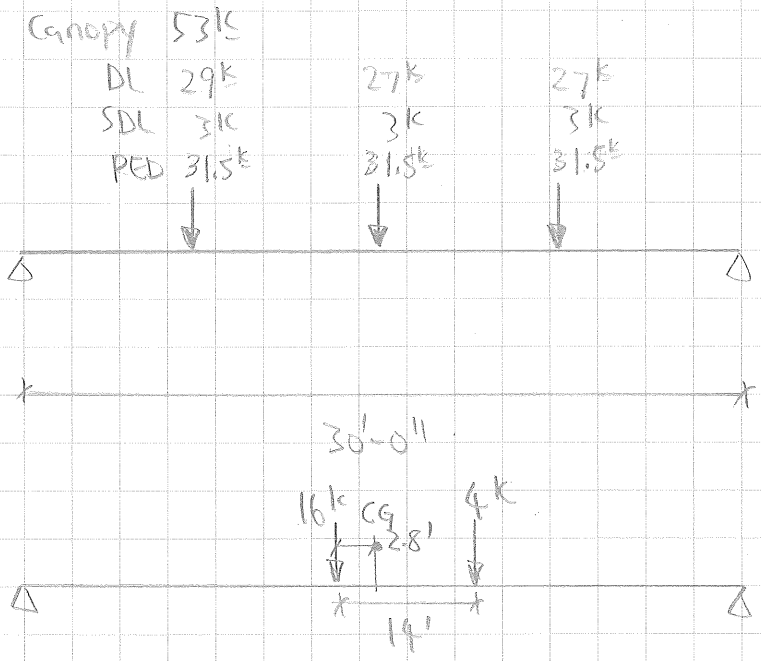
$$W_{DL} = \left(\frac{5.5}{12}\right)(7.5')(0.150\ pcf) + \begin{Bmatrix} 0.076\ klf \\ 0.116\ \text{"} \end{Bmatrix} = \begin{Bmatrix} 0.592\ klf \\ 0.626\ \text{"} \end{Bmatrix} \begin{matrix} \text{span 28} \\ \text{span 29} \end{matrix}$$

assume 5% misc. load

$$= \begin{Bmatrix} 0.622\ klf \\ 0.657\ klf \end{Bmatrix} \begin{matrix} \text{span 28} \\ \text{span 29} \end{matrix}$$

W_{SDL} = 0.01 ksf (7.5') = 0.075 klf

W_{ped} = 0.10 " " = 0.75 klf



$$P_{canopy} = 40^k\left(\frac{40}{47.5}\right) + 45^k\left(\frac{20}{47.5}\right) = 53^k$$

WEIDLINGER

PROJECT		SUBJECT	Floorbeam 36WF194
BY Jason A.	DATE 9/20/13		Bent 28, FB 28-8
CHECKED BY TFKo	DATE 10/11/13		PAGE 2 OF 3

Moment DF = 1.0 for F.B – AASHTO 3.23.3.1

$M_{DL} = \frac{1}{8}(0.194 \times 1.05)(30)^2 + \frac{1}{8}(29)(30) + \frac{1}{4}(27)(30) + \frac{1}{8}(27)(30)$

$= 435.42$ k·ft

$M_{SDL} = \frac{1}{8}(3)(30)(2) + \frac{1}{4}(3)(30) = 45$ kft

$M_{PED} = \frac{1}{8}(31.5)(30)(2) + \frac{1}{4}(31.5)(30) = 472.5$ kft ← governs

$M_{HI0} = (30' - 2.8')(10^k)(1) = 122$ kft

$M_{canopy} = 53^k/2\,(30'/4 + 7.5'/2) = 298.13$ kft @ midspan

Shear

$V_{DL} = \frac{1}{2}(0.194 \times 1.05)(30) + \frac{3}{4}(29^k) + \frac{1}{2}(27^k) + \frac{1}{4}(27^k) = 45.06^k$

$V_{SDL} = 4.5^k$

$V_{PED} = 47.25^k$

$V_{HI0} = 10^k$

$V_{canopy} = 53^k/2\,(3/4 + 1/2) = 33.13^k$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/20/13
CHECKED BY	TF Ko	DATE	10/11/13

SUBJECT	Floorbeam 36WF194
	Bent 28, FB 28-8
	PAGE 3 OF 3

Load Rating 36WF194 (Bent 28)

Inventory $LR_m = \dfrac{18\,ksi \times 663.6\,in^3/12 - (435.42 + 45 + 298.13)}{472.5}$

$= 0.459$

$LR_v = \dfrac{11\,ksi \times 36.48'' \times 0.77'' - (45.06 + 4.5 + 33.13)}{47.25}$

$= 4.789$

Operating $LR_m = \dfrac{24.5 \times 663.6/12 - (435.42 + 45 + 298.13)}{472.5}$

$= 1.22$

$LR_v = \dfrac{15 \times 36.48 \times 0.77 - (45.06 + 4.5 + 33.13)}{47.25}$

$= 7.167$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/20/13
CHECKED BY	TFKO	DATE	10/11/13

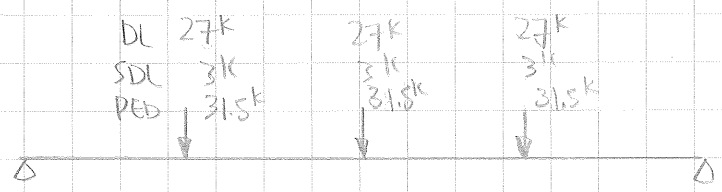
SUBJECT	Floorbeam 36WF160
	Bent 28, FB 28-6
PAGE	1 OF 2

Floorbeam 36WF160 (Bent 28, FB 28-6)

Span 28 = 36'-6"

Span 29 = 47'-6"

See loadings from FB28 36WF194 section (FB 28-8) except no canopy load



HIO truck does not govern

15.5925

Moment

$+\frac{1}{8}(0.160\times1.05)(27.5)^2$

$M_{DL} = \frac{1}{8}(27)(27.5)\times2 + \frac{1}{4}(27)(27.5) = 387$ kft

$M_{SDL} =$ " 3 " " " 3 " $= 41$ kft

$M_{PED} =$ " 31.5 " " " 31.5 " $= 433$ kft

Shear

$+\frac{1}{2}(0.160\times1.05)(27.5)$

$V_{DL} = \frac{3}{4}(27) + \frac{1}{2}(27) + \frac{1}{4}(27) = 42.81$ k

$V_{SDL} =$ " 3 " 3 " 3 $= 4.5$ k

$V_{PED} =$ " 31.5 " 31.5 " 31.5 $= 47.25$ k

WEIDLINGER

PROJECT		SUBJECT	Floorbeam 36WF160
BY Jason A.	DATE 9/20/13		Bent 28, FB 28-6
CHECKED BY TF Ko	DATE 10/11/13		PAGE 2 OF 2

Load Rating 36WF160 (Bent 28, FB 28-6)

Inventory

$$LR_m = \frac{18 \times 54'/12 - (387 + 41)}{433}$$

$$= 0.886$$

$$LR_v = \frac{11 \times 36'' \times 0.653'' - 42.81 - 4.5}{47.25}$$

$$= 4.471$$

Operating

$$LR_m = \frac{24.5 \times 54'/12 - (387 + 41)}{433}$$

$$= 1.562$$

$$LR_v = \frac{15 \times 36 \times 0.653 - 42.81 - 4.5}{47.25}$$

$$= 6.461$$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/20/13
CHECKED BY	TFK	DATE	10/11/13

SUBJECT	Floorbeam 30WF132
	Bent 29, FB 29-8
PAGE 1	OF 1

Floorbeam 30WF132 (Bent 29, FB 29-8) L = 30'

Span 29 = 47'-6"

See loading from FB28 36WF194 section

HIO truck does not govern

DL	15k	15k	15k
SDL	2k	2k	2k
PED	18k	18k	18k

39.089

See moment & shear calc. from Bent 28

$M_{DL} = 240.59k'$

$M_{SD} = 30k'$

$M_{PED} = 270k'$

$M_{canopy} = 32k/2\,(30'/4 + 7.5'/2) = 180k\text{ft}$

$V_{DL} = 24.58'$

$V_{SDL} = 3'$

$V_{PED} = 27'$

$V_{canopy} = 32k/2\,(3/4 + 1/2) = 20k$

For rating calculations, see summary table

SPANS 1-7 AND 23-29
As Built Load Rating Calculations

Columns

(ASD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

WEIDLINGER

PROJECT			
BY	JwonA.	DATE	9/19/13
CHECKED BY	TFKo	DATE	10/14/13

SUBJECT	column 14WF74
	Bent 1
PAGE	1 OF 2

Column 1K (14WF74)

⇒ Upper level col 1K

Deadload 0.036(6)(14.34') + 0.13(24') + 0.034(8') + 0.0207(32') = 7.2 k

Timber plank 0.050(32')(14.34') = 23 k

SDL 0.01 (" ") = 4.6 k

Pedestrian 0.10 " " = 45.89 k

⇒ Mezz level Col 1K

Same as upper level col 1K

DL = 6.5 k
Timber = 23 k
SDL = 4.6 k
PED = 45.89 k

⇒ Column selfweight 28.33' × 0.074 = 2.1 k

⇒ HID not govern by inspection

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/19/13
CHECKED BY	TFK	DATE	10/14/13

SUBJECT	Column 14WF74
	Bent 1
PAGE	2 OF 2

Total load

$P_{DL} = [(7.2 + 23) \times 2 + 2.1] \times 1.05 = 66^{K}$

$P_{SDL} = 4.6 \times 2 = 9.2^{K}$

$P_{PED} = 45.89 \times 2 = 91.78^{K}$

unbraced length above mezz. level El 40.70 - El 25.45

= 15.25' ← controls

below mezz level El 25.45 - El 12.37'

= 13.08'

Inventory $LR = \dfrac{[15.57 - 0.45(\frac{1 \times 15.25 \times 12}{2.48})^2/1000](21.76) - 66 - 9.2}{91.78}$

$= 2.295$

Operating $LR = \dfrac{[19.41 - 0.56(\frac{1 \times 15.25 \times 12}{2.48})^2/1000](21.76) - 66 - 9.2}{91.78}$

$= 3.06$

WEIDLINGER

PROJECT			
BY	Jalon A.	DATE	9/19/13
CHECKED BY	TPKo	DATE	10/16/13

SUBJECT	Column 14WF95
	Bent 2
PAGE 1	OF 3

Column 2K (14WF95)

⇒ upper level col. 2A

Dead load $0.030(8'\times 1\tfrac{1}{2}) + 0.084(24') + 0.021(6)(7.33') + 0.021(7.33) + 0.076(14.67')$ = 4.6 K

Timber Plank $0.050(22'\times 8' + 24'\times 7.33')$ = 17.6 K

SDL 0.010 (" " " ") = 3.5 K

Pedestrian 0.1 " " " " = 35.2 K

⇒ Mezz. level col 2K

Dead load $0.034(8') + 0.030(8') + 0.130(24) + 0.124(24)$
$0.036((6)(10\tfrac{1}{2}')) + 0.021(5)(1\tfrac{1}{2})(14.67')$
$0.230(22)$ = 16.3 K

Timber plank $0.050(32.5'\times 32')$ = 52 K

SDL 0.010 " " = 13 K

Pedestrian 0.1 " " = 130 K

⇒ Column selfweight $33'\times 0.095$ klf = 3 K

⇒ HLD not control by inspection

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/19/13
CHECKED BY	TFKo	DATE	10/14/13

SUBJECT	Column 14WF95
	Bent 2
PAGE 2	OF 3

⇒ Column 2K

Deadload $0.036(6)(10\frac{1}{2}') + 0.021(6)(7.33') + 0.034(8') + 0.130(24') = 6.6^{K}$

Timber Plank $0.050(17.83')(32') = 28.5^{K}$

SDL 0.01 " " $= 5.7^{K}$

Pedestrian 0.1 " " $= 57.1^{K}$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/19/13
CHECKED BY	TFKo	DATE	10/14/13

SUBJECT	Column 14WF95
	Bent 2
PAGE	3 OF 3

Total load

$P_{DL} = [(4.6+17.6)(2/3) + (6.6+28.5+16.3+52+3)](1.05)$

$= 127.3$ k

$P_{SDL} = 3.5 \times 2/3 + 5.7 + 13 = 21$ k

$P_{PED} = 35.2 \times 2/3 + 57.1 + 130 = 210.57$

unbraced length above mezz. level EL 40.58 - EL 25.45

$= 15.13'$ ← governs

below mezz level EL 25.45 - EL 12.37'

$= 13.08'$

$$\text{Inventory LR} = \frac{[15.57 - 0.45(\frac{1 \times 15.13 \times 12}{3.71})^2/1000](27.94\,in^2) - 127.3 - 21}{210.57}$$

$= 1.215$

$$\text{Operating LR} = \frac{[19.41 - 0.56(\frac{1 \times 15.13 \times 12}{3.71})^2/1000](27.94) - 127.3 - 21}{210.57}$$

$= 1.692$

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/19/13
CHECKED BY	TFK	DATE	10/16/13

SUBJECT	Column 14WF78
	Bent 3
PAGE	1 OF 2

Column 3K (14WF78)

⇒ upper level col 3K

Dead load $0.030(2\frac{1}{2})(8') + 0.076(33.67') + 0.021(14.67' + 19')$ = 3.9 K

Timber plank $0.050(33.67')(8')$ = 13.5 K

SDL 0.01 " " = 2.4 K

Pedestrian 0.1 " " = 24 K

⇒ Mezz level col 3K

Dead load $0.130(3)(24') + 0.030(3)(8') + 0.230(22') + 0.130(19') + 0.021(1.5)(5)(14.67' + 12.67')$ = 22 K

Timber plank $0.050(41')(32')$ = 65.6 K

SDL 0.01 " " = 13.1 K

Pedestrian 0.1 " " = 131.2 K

⇒ Column selfweight $26.54' \times 0.078$ = 2.1 K

⇒ HID not govern by inspection

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/19/13
CHECKED BY	TFKO	DATE	10/24/13

SUBJECT	Column 14WF78
	Bent 3
PAGE	2 OF 2

Total load

$P_{DL} = (3.9 + 13.5 + 22 + 63.6 + 2.1) \times 1.05 = 112.5^k$

$P_{SDL} = 2.4 + 13.1 = 15.5^k$

$P_{LL} = 24 + 131.2 = 155.2^k$

unbraced length above mezz. level EL 38.41 − EL 28.25

$= 10.16'$

below mezz. level EL 28.25 − EL 11.87'

$= 16.38'$ ← controls

$$\text{Inventory } LR = \frac{\left[(15.57 - 0.45\left(\frac{1 \times 16.38 \times 12}{3}\right)^2/1000\right](22.94) - 112.5 - 15.5}{155.2}$$

$= 1.188$

$$\text{Operating } LR = \frac{\left[(19.41 - 0.56\left(\frac{1 \times 16.38 \times 12}{3}\right)^2/1000\right](22.94) - 112.5 - 15.5}{155.2}$$

$= 1.683$

WEIDLINGER

PROJECT		SUBJECT	Column 14WF87
BY Jason A.	DATE 9/19/13		Bent 4
CHECKED BY TFKO	DATE 10/14/13		PAGE 1 OF 2

Column 4K (14WF87)

⇒ upper level col 4K

Dead load 0.030(3)(8) + 0.076(38') + 0.021(3)(12.67') = 4.40 k

Timber plank 0.050(38')(8') = 15.2 k

SDL 0.010 " " = 3 k

Pedestrian 0.1 " " = 30.44k

⇒ Mezz. level col 4K

similar to col 3K but apply 85%

DL = 22k × 0.85 = 18.7 k

Timber = 65.6 × 0.85 = 55.8 k

SDL = 13.1 " = 11.1 k

Ped = 131.2 " = 111.52 k

⇒ Colum selfweight 23.84' × 0.087 = 2.1 k

⇒ H10 not govern by inspection

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	9/19/13
CHECKED BY	Tarke	DATE	10/14/13

SUBJECT	Column 14WF78
	Bent 4
PAGE	2 OF 2

Total load

$P_{DL} = (4.40 + 18.7 + 16.5 + 49.2 + 2.1) \times 1.05 = 95.4^{k}$

$P_{SDL} = 3 + 11.1 = 14.1^{k}$

$P_{PED} = 30.44 + 111.52 = 142$

unbraced length above mezz level El. 35.71 - El 30.63

= 5.08'

below mezz level El. 30.63 - El 11.87'

= 18.76' ← controls

$$\text{Inventory LR} = \frac{\left[\left(15.57 - 0.45\left(\frac{1 \times 18.76 \times 12}{3.70}\right)^2/1000\right](25.56) - 95.4 - 14.1\right.}{142}$$

$= 1.728$

$$\text{Operating LR} = \frac{\left[\left(19.41 - 0.56\left(\frac{1 \times 18.76 \times 12}{3.70}\right)^2/1000\right](22.94) - 95.4 - 14.1\right.}{142}$$

$= 2.35$

Column H at Bent 22, 14WF61

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 33.1615\ ft$

$S2 := 10\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 19.9\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) - 5\ ft \cdot 15\ ft = 572.423\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .145\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 0.5 + .076 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 3 + .084 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 0.5$

$dl1 := dl1 + .027 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 2.5 = 6.016\ kip$

Floorbeam: $dl2 := .141 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .141 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 4.23\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 2 = 0.311\ kip$

Concrete: $dl4 := concrete \cdot 5.5\ in \cdot At = 39.354\ kip$

Column: $dl5 := .061 \cdot \frac{kip}{ft} \cdot H = 1.214\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 53.681\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 5.724\ kip$

Canopy: $DF_Can := \frac{15\ ft}{30\ ft} + \frac{1.9375\ ft}{7.3125\ ft} \cdot \frac{14.625\ ft}{29.25\ ft} + \frac{5.375\ ft}{7.3125\ ft} \cdot \frac{7.3125\ ft}{29.25\ ft} = 0.816$

$Pcan := DF_Can \cdot 40\ kip \cdot \left(\frac{30.7775}{33.1615} + \frac{10.427}{33.1615}\right) \frac{ft}{ft} = 40.568\ kip$

Live Load:

Pedestrain: $Pped := ped \cdot At = 57.242\ kip$

H-10: $Ph10 := 10\ kip$

PROJECT: PASSERELLE LOAD RATING
BY: TUNG FOR KO **DATE:** 10/08/2013
CHECKED BY: Jason A. **DATE:** 10/11/13

Load Rating

$As := 17.94\ in^2 \qquad ry := 2.45\ in \qquad K := 1 \qquad Fy := 33\ ksi \qquad H = 19.9\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 1.793$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.669$$

Column H at Bent 23, 14WF61

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 10\ ft$

$S2 := 32.1615\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 19.72\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) - 5\ ft \cdot 15\ ft = 557.423\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .11\ \frac{kip}{ft} \cdot \frac{S2}{2} \cdot .5 + .068 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 3 + .076 \cdot \frac{kip}{ft}\ \frac{S2}{2} \cdot .5$

$dl1 := dl1 + .027 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 2.5 = 5.113\ kip$

Floorbeam: $dl2 := .141 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .141 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 4.23\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 2 = 0.311\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 38.323\ kip$

Column: $dl5 := .061 \cdot \frac{kip}{ft} \cdot H = 1.203\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 51.639\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 5.574\ kip$

Canopy: $DF_Can := \frac{15\ ft}{30\ ft} + \frac{1.9375\ ft}{7.3125\ ft} \cdot \frac{14.625\ ft}{29.25\ ft} + \frac{5.375\ ft}{7.3125\ ft} \cdot \frac{7.3125\ ft}{29.25\ ft} = 0.816$

$Pcan := DF_Can \cdot 40\ kip \cdot \left(\frac{24.2855}{32.1615} + \frac{3.82}{32.1615}\right) \frac{ft}{ft} = 28.532\ kip$

Live Load:

Pedestrain: $Pped := ped \cdot At = 55.742\ kip$

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 17.94\ in^2$ $ry := 2.45\ in$ $K := 1$ $Fy := 33\ ksi$ $H = 19.72\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.122$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 3.027$$

Column E at Bent 24, 14WF87

$L1 := 27.5\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 27.5\ ft$

$S1 := 32.1615\ ft$

$S2 := 37\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 23.17\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) = 950.971\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .068\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 4 + .076 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 4 = 9.998\ kip$

Floorbeam: $dl2 := .130 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .130 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 3.575\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 3 + .0207 \cdot \frac{kip}{ft} \cdot \frac{L2}{4} \cdot 3 = 0.854\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 65.379\ kip$

Column: $dl5 := .087 \cdot \frac{kip}{ft} \cdot H = 2.016\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 85.913\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 9.51\ kip$

Canopy: $Pcan := 0$

Live Load:

Pedestrain: $Pped := ped \cdot At = 95.097\ kip$

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 25.56\ in^2 \qquad ry := 3.7\ in \qquad K := 1 \qquad Fy := 33\ ksi \qquad H = 23.17\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.498$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 3.364$$

Column H at Bent 24, 14WF87

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 32.1615\ ft$

$S2 := 37\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 23.17\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) = \left(1.037 \cdot 10^3\right)\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .11\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot .5 + .068 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 3 + .076 \cdot \frac{kip}{ft}\ \frac{S1}{2} \cdot .5 + .145 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot .5$

$dl1 := dl1 + .076 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 3 + .084 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot .5 = 11.112\ kip$

Floorbeam: $dl2 := .152 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .152 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 4.56\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 6 = 0.932\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 71.323\ kip$

Column: $dl5 := .087 \cdot \frac{kip}{ft} \cdot H = 2.016\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 94.439\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 10.374\ kip$

Canopy: $DF_Can := \frac{15\ ft}{30\ ft} + \frac{1.9375\ ft}{7.3125\ ft} \cdot \frac{14.625\ ft}{29.25\ ft} + \frac{5.375\ ft}{7.3125\ ft} \cdot \frac{7.3125\ ft}{29.25\ ft} = 0.816$

$Pcan := DF_Can \cdot 40\ kip \cdot \left(\left(\frac{7.8763}{32.1615}\right) + \left(\frac{28.3415}{32.1615}\right) + \left(\frac{20.422}{37}\right)\right) \frac{ft}{ft} = 54.788\ kip$

Live Load:

Pedestrain: $Pped := ped \cdot At = 103.742\ kip$

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 25.56\ in^2$ $ry := 3.7\ in$ $K := 1$ $Fy := 33\ ksi$ $H = 23.17\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 1.672$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.465$$

Column H at Bent 25, 14WF78

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 37\ ft$

$S2 := 10\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 18.44\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) - 15\ ft \cdot 5\ ft = 630\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .145\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot .5 + .076 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 3 + .084 \cdot \frac{kip}{ft}\ \frac{S1}{2} \cdot .5$

$dl1 := dl1 + .027 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 2.5 = 6.674\ kip$

Floorbeam: $dl2 := .141 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .141 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 4.23\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 4 = 0.621\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 43.313\ kip$

Column: $dl5 := .078 \cdot \frac{kip}{ft} \cdot H = 1.438\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 59.089\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 6.3\ kip$

Canopy: $DF_Can := \frac{15\ ft}{30\ ft} + \frac{1.9375\ ft}{7.3125\ ft} \cdot \frac{14.625\ ft}{29.25\ ft} + \frac{5.375\ ft}{7.3125\ ft} \cdot \frac{7.3125\ ft}{29.25\ ft} = 0.816$

$Pcan := 40\ kip \cdot 0.5 + DF_Can \cdot 40\ kip \cdot \left(\frac{16.495}{37}\right) + 40 \cdot kip \cdot \left(\frac{9.25}{29.25}\right) = 47.205\ kip$

Live Load:

Pedestrain: $Pped := ped \cdot At =$ 63 kip

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 22.94\ in^2 \qquad ry := 3\ in \qquad K := 1 \qquad Fy := 33\ ksi \qquad H = 18.44\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.991$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 4.171$$

Column H at Bent 26, 14WF78

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 10\ ft$

$S2 := 45.9583\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 18.25\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) - 15\ ft \cdot 5\ ft = 764.375\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .027\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 2.5 + .16 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 0.5 + .11 \cdot \frac{kip}{ft}\ \frac{S2}{2} \cdot 3.5 = 11.023\ kip$

Floorbeam: $dl2 := .141 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .141 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 4.23\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 4 = 0.621\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 52.551\ kip$

Column: $dl5 := .078 \cdot \frac{kip}{ft} \cdot H = 1.424\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 73.34\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 7.644\ kip$

Canopy: $DF_Can := \frac{15\ ft}{30\ ft} + \frac{1.9375\ ft}{7.3125\ ft} \cdot \frac{14.625\ ft}{29.25\ ft} + \frac{5.375\ ft}{7.3125\ ft} \cdot \frac{7.3125\ ft}{29.25\ ft} = 0.816$

$Pcan := DF_Can \cdot 40\ kip \cdot \left(\frac{37.078}{45.9583}\right) = 26.341\ kip$

Live Load:

Pedestrain: $Pped := ped \cdot At =$ 76.437 kip

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 22.94\ in^2$ $ry := 3\ in$ $K := 1$ $Fy := 33\ ksi$ $H = 18.25\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.549$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 3.526$$

Column E at Bent 27, 14WF87

$L1 := 27.5\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 27.5\ ft$

$S1 := 45.9583\ ft$

$S2 := 36.5\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 20.55\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) = \left(1.134 \cdot 10^3\right)\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .11\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 4 + .076 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 4 = 15.659\ kip$

Floorbeam: $dl2 := .150 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .150 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 4.125\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 4 + .0207 \cdot \frac{kip}{ft} \cdot \frac{L2}{4} \cdot 4 = 1.139\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 77.949\ kip$

Column: $dl5 := .087 \cdot \frac{kip}{ft} \cdot H = 1.788\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 105.692\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 11.338\ kip$

Canopy: $Pcan := 0$

Live Load:

Pedestrain: $Pped := ped \cdot At =$ 113.38 kip

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 25.56\ in^2$ $ry := 3.7\ in$ $K := 1$ $Fy := 33\ ksi$ $H = 20.55\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.027$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.783$$

Column H at Bent 27, 14WF95

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 45.9583\ ft$

$S2 := 36.5\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 20.55\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) = \left(1.237 \cdot 10^3\right)\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .16\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 0.5 + .11 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 3.5 + .076 \cdot \frac{kip}{ft}\ \frac{S2}{2} \cdot 4 = 16.233\ kip$

Floorbeam: $dl2 := .170 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .170 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 5.1\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 8 = 1.242\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 85.035\ kip$

Column: $dl5 := .095 \cdot \frac{kip}{ft} \cdot H = 1.952\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 115.041\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 12.369\ kip$

Canopy:

$$DF_Can := \frac{15\ ft}{30\ ft} + \frac{1.9375\ ft}{7.3125\ ft} \cdot \frac{14.625\ ft}{29.25\ ft} + \frac{5.375\ ft}{7.3125\ ft} \cdot \frac{7.3125\ ft}{29.25\ ft} = 0.816$$

$$Pcan := DF_Can \cdot 40\ kip \cdot \left(\frac{8.8802}{45.9583}\right) = 6.309\ kip$$

Live Load:

Pedestrain: $Pped := ped \cdot At = 123.687\ kip$

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 27.94\ in^2 \qquad ry := 3.71\ in \qquad K := 1 \qquad Fy := 33\ ksi \qquad H = 20.55\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 1.987$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.745$$

Column H at Bent 28, 14WF95

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 36.5\ ft$

$S2 := 47.5\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 19.27\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) = \left(1.26 \cdot 10^3\right)\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .076 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 4 + .16 \cdot \frac{kip}{ft}\ \frac{S2}{2} \cdot 0.5 + .11 \cdot \frac{kip}{ft} \cdot \frac{S2}{2} \cdot 3.5 = 16.592\ kip$

Floorbeam: $dl2 := .194 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .194 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 5.82\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 4 + .0207 \cdot \frac{kip}{ft} \cdot \frac{L2}{4} \cdot 4 = 1.242\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 86.625\ kip$

Column: $dl5 := .095 \cdot \frac{kip}{ft} \cdot H = 1.831\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 117.715\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 12.6\ kip$

Canopy: $DF_Can := \left(\frac{7.5}{30} + \frac{3.0833}{7.3125} \cdot \frac{14.625}{29.25} + \frac{4.229}{7.3125} \cdot \frac{7.3125}{29.25}\right) \frac{ft}{ft} = 0.605$

$Pcan := DF_Can \cdot 40\ kip \cdot \left(\frac{40}{47.5} + \frac{20}{47.5}\right) = 30.589\ kip$

Live Load:

Pedestrain: $Pped := ped \cdot At = 126\ kip$

H-10: $Ph10 := 10\ kip$

PROJECT: PASSERELLE LOAD RATING
BY: TUNG FOR KO **DATE:** 10/08/2013
CHECKED BY: JASON A. **DATE:** 10/11/13

Load Rating

$As := 27.94\ in^2 \qquad ry := 3.71\ in \qquad K := 1 \qquad Fy := 33\ ksi \qquad H = 19.27\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 1.788$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.545$$

Column H at Bent 29, 14WF61

$L1 := 30\ ft$ $\quad concrete := 150\ \frac{lbf}{ft^3}$ $\quad sdead := 10\ \frac{lbf}{ft^2}$

$L2 := 30\ ft$

$S1 := 47.5\ ft$

$S2 := 0\ ft$ $\quad ped := 100\ \frac{lbf}{ft^2}$ $\quad H10 := 20\ kip$

$H := 17.02\ ft$

Tributary Area: $At := \left(\frac{L1}{2} + \frac{L2}{2}\right) \cdot \left(\frac{S1}{2} + \frac{S2}{2}\right) = 712.5\ ft^2$

Loading to Column

Dead Load:

Self Weight:

Stringer: $dl1 := .16\ \frac{kip}{ft} \cdot \frac{S1}{2} \cdot .5 + .11 \cdot \frac{kip}{ft} \cdot \frac{S1}{2} \cdot 3.5 = 11.044\ kip$

Floorbeam: $dl2 := .172 \cdot \frac{kip}{ft} \cdot \frac{L1}{2} + .132 \cdot \frac{kip}{ft} \cdot \frac{L2}{2} = 4.56\ kip$

Bracing: $dl3 := .0207 \cdot \frac{kip}{ft} \cdot \frac{L1}{4} \cdot 4 = 0.621\ kip$

Concrete: $dl4 := concrete \cdot (5.5\ in) \cdot At = 48.984\ kip$

Column: $dl5 := .061 \cdot \frac{kip}{ft} \cdot H = 1.038\ kip$

$Pdl := 1.05 \cdot (dl1 + dl2 + dl3 + dl4 + dl5) = 69.56\ kip$

Superimpose-dead Load:

$Psdl := sdead \cdot At = 7.125\ kip$

Canopy: $DF_Can := \left(\frac{7.5}{30} + \frac{3.0833}{7.3125} \cdot \frac{14.625}{29.25} + \frac{4.229}{7.3125} \cdot \frac{7.3125}{29.25}\right) \frac{ft}{ft} = 0.605$

$Pcan := (DF_Can) \cdot 40\ kip \cdot \left(\frac{7.5}{47.5} + \frac{27.5}{47.5}\right) + 40\ kip \cdot \left(\frac{7.5}{30} + \frac{10.3958}{29.25}\right) \cdot \frac{ft}{ft}$

$Pcan = 42.06\ kip$

Live Load:

Pedestrain: $Pped := ped \cdot At =$ 64.125 kip

H-10: $Ph10 := 10\ kip$

Load Rating

$As := 17.94\ in^2 \qquad ry := 2.45\ in \qquad K := 1 \qquad Fy := 33\ ksi \qquad H = 17.02\ ft$

Inventory:

$$LR := \frac{\left(15570 \cdot psi - 0.45 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 1.629$$

Inventory:

$$LR := \frac{\left(19410 \cdot psi - 0.56 \cdot psi \cdot \left(\frac{K \cdot H}{ry}\right)^2\right) \cdot As - Pdl - Psdl - Pcan}{Pped} = 2.49$$

SPANS 1-7 AND 23-29
As Built Load Rating Calculations

Girders

(ASD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

WEIDLINGER

PROJECT			
BY Jason A.	DATE 9/19/13	SUBJECT Girder 36WF230 Span 3	
CHECKED BY TFK	DATE 10/14/2013	PAGE 1	OF 2

Girder 36WF230 (span 3) G2

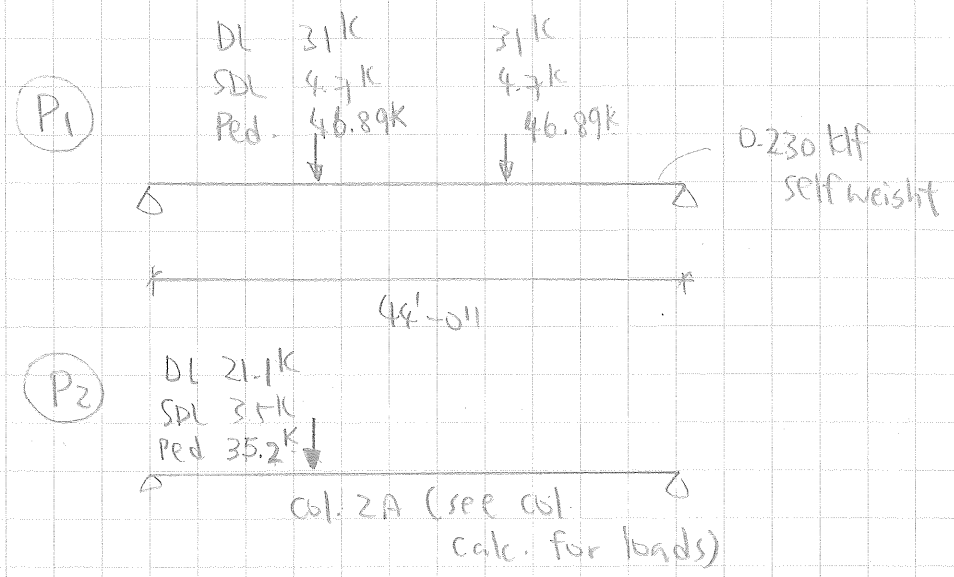
DF = 1.0 AASHTO 3.23.7.1 span length = 44'-0"

Deadload $0.124(24') + 0.03(8') + 0.021(10)(14.67') = 6.3^k$

Timber plank $0.050(14.67')(32') = 23.5^k$

SDL 0.010 " " $= 4.7^k$

Pedestrian 0.100 " " $= 46.89^k$



Moments: $M_{DL} = \left(\frac{1}{8}wl^2 + \frac{1}{3}P_1L + \frac{1}{6}P_2L\right)1.05 = 698.3$ kft

$M_{SDL} = \frac{1}{3}P_1L + \frac{1}{6}P_2L = 94.6$ kft

$M_{PED} =$ " " $= 945.85$ kft ← governs

$M_{HTD} = \left(\frac{44'}{2} - 2.8\right)(10^k) = 192.$ kft

WEIDLINGER

PROJECT		SUBJECT	Girder 36WF230
BY Jason A.	DATE 9/19/13		Span 3
CHECKED BY TFKo	DATE 10/14/2013		PAGE 2 OF 2

Shears: $V_{DL} = \left[\frac{1}{2} \times 0.23 \times 44 + 31^k + 21.1^k \left(\frac{2}{3}\right)\right] 1.0f = 52.6^k$

$V_{SDL} = (4.7^k + 3.5^k \times 2/3) = 7.0^k$

$V_{PED} = (46.88^k + 35.2 \times 2/3) = 70.36^k$ ← governs

$V_{HLO} = 10^k$

Inventory $LR_m = \dfrac{18\left(\frac{835.5}{12}\right) - 698.3 - 94.6}{945.9}$

$= 0.487$

$LR_V = \dfrac{11(0.765)(35.88) - 52.6 - 7.0}{70.4}$

$= 3.442$

Operating $LR_m = \dfrac{24.5\left(\frac{835.5}{12}\right) - 698.3 - 94.6}{945.9}$

$= 0.965$

$LR_V = \dfrac{15(0.765)(35.88) - 52.6 - 7.0}{70.4}$

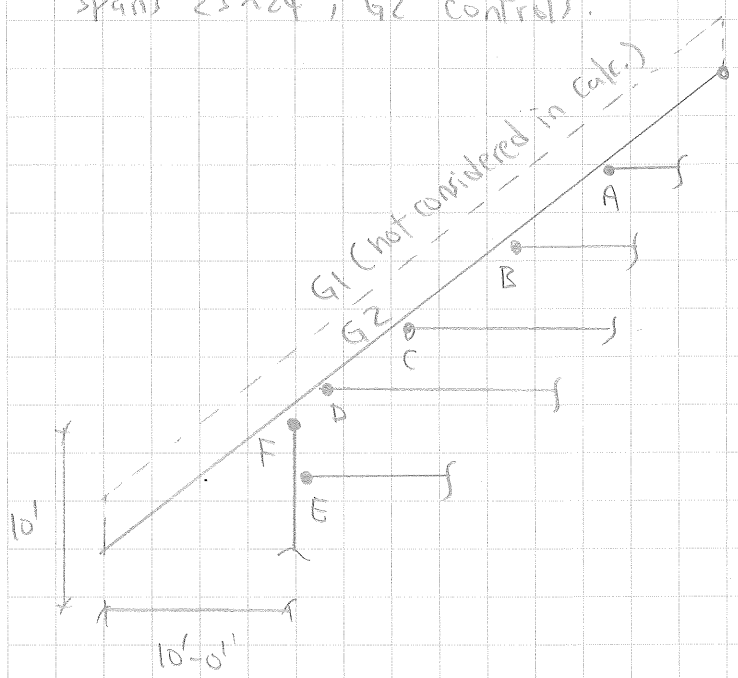
$= 5.00$

WEIDLINGER

PROJECT			
BY Jason A.	DATE 10/7/13	SUBJECT Girder 33W152 G2	
CHECKED BY TFK	DATE 10/24/2013	SPANS 23~24	
		PAGE 1	OF 9

Similar to span 22 and span 25, where G1 are not controlling over there

Spans 23~24, G2 controls.



Dist. weight for DL

$\left[\frac{5.5}{12}(6.875)(0.150) + 24\text{w}/68\right] \times 1.05$

$= 0.57$ klf

END REACTIONS (kips)

Pt	Sp.	D	SDL	PED	not governed H-10
A	7.42'	2.1	0.3	2.78	NA
B	14.86'	4.24	0.5	5.33	NA
C	21.73'	6.2	0.7	7.44	NA
D	28.50'	8.12	1.0	9.78	NA
E	32.59'	9.15	1.1	11	← 100% dist. to pt. 'F'
F	10'x10' x1/2	1.76	0.2	1.56	conservative

PLEASE RECYCLE

WEIDLINGER

PROJECT			
BY	Jason A.	DATE	10/7/13
CHECKED BY	TFK	DATE	10/14/13

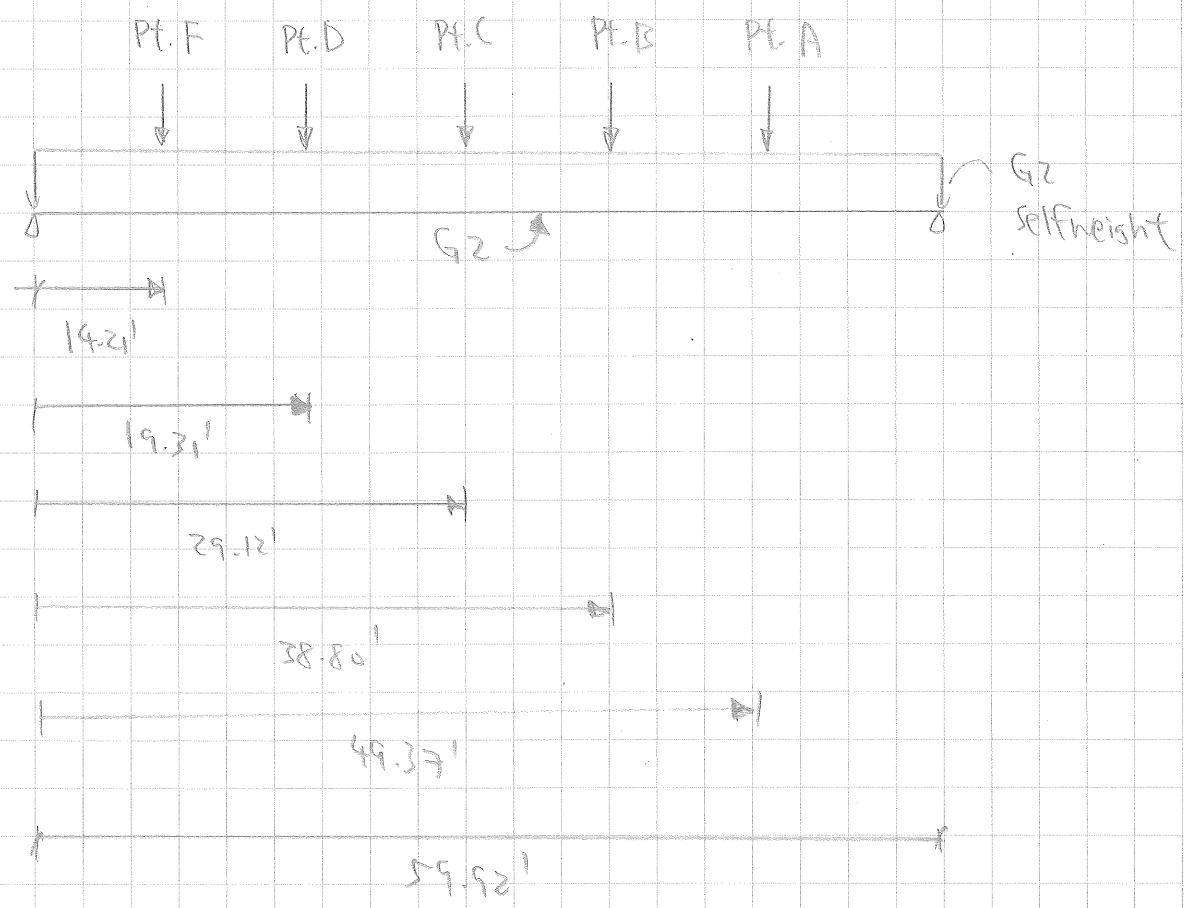
SUBJECT	Girder 33WF152 G2
	spans 23-24
PAGE 2	OF 9

G2 selfweight 0.152 klf

stab $3.2'/2\,(0.150)\left(\frac{5.5}{12}\right) \times 1.05 = 0.114$ klf

SDL $3.2'/2\,(0.01) = 0.016$ klf

Ped. $3.2'/2\,(0.10) = 0.160$ klf



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Load Diagram

RAM SBeam v4.0
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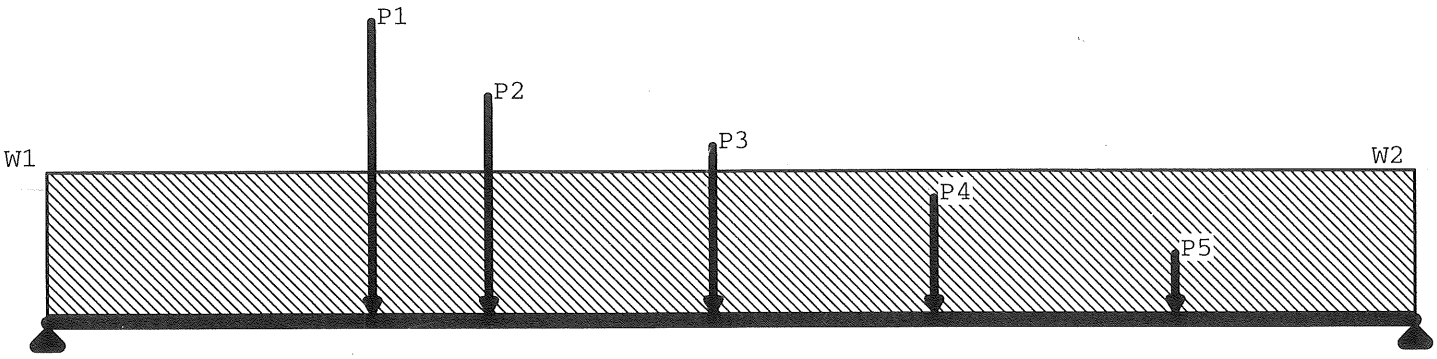
11/13/13 08:29:59

Span information (ft): I-End (0.00,0.00) J-End (59.92,0.00)

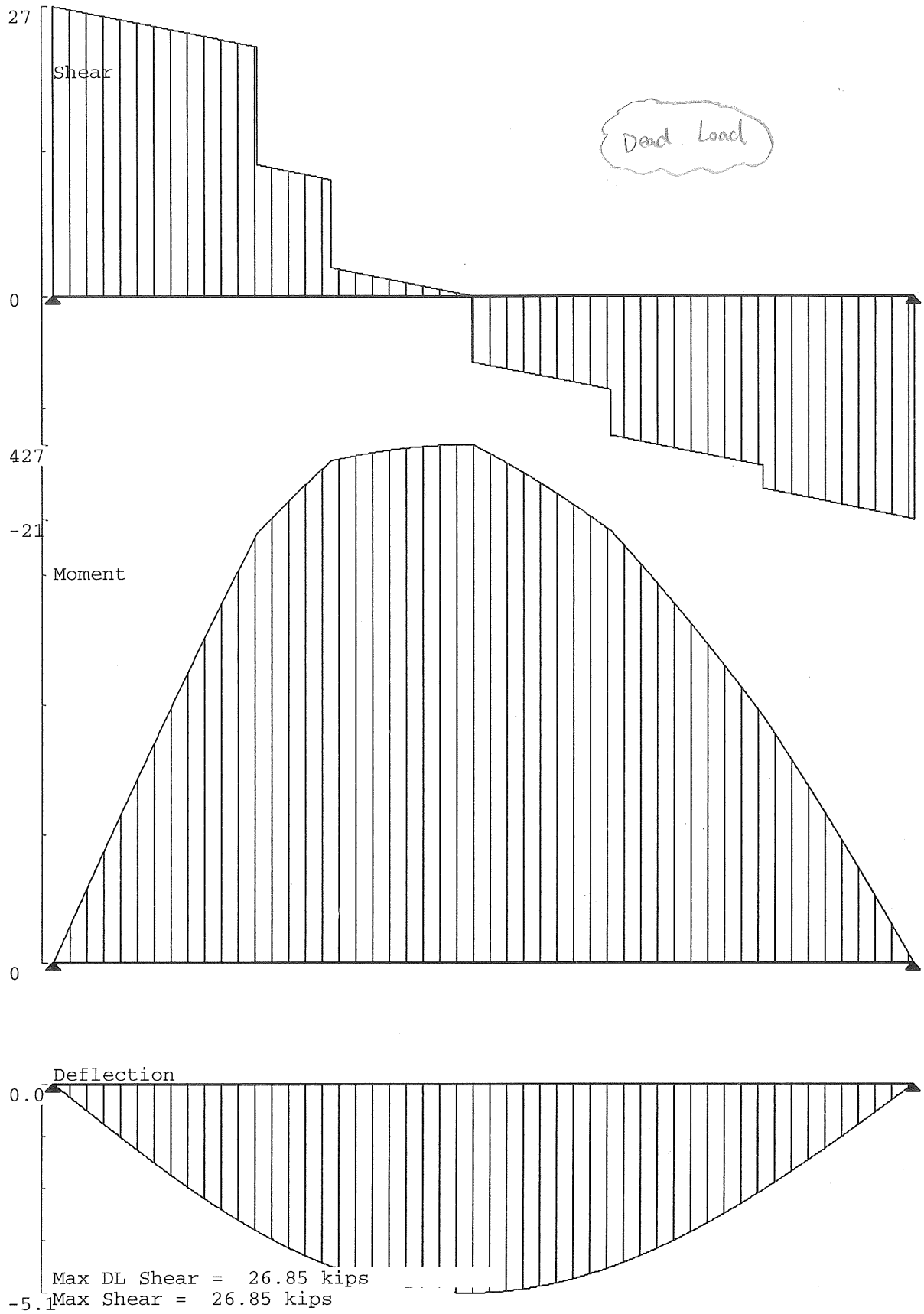


Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	14.210	10.910	0.000	0.000	10.910
P2	19.310	8.120	0.000	0.000	8.120
P3	29.120	6.200	0.000	0.000	6.200
P4	38.800	4.240	0.000	0.000	4.240
P5	49.370	2.100	0.000	0.000	2.100
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.266	0.000	0.000	0.266
W2	59.920	0.266	0.000	0.000	0.266

Dead Load

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (59.92,0.00)



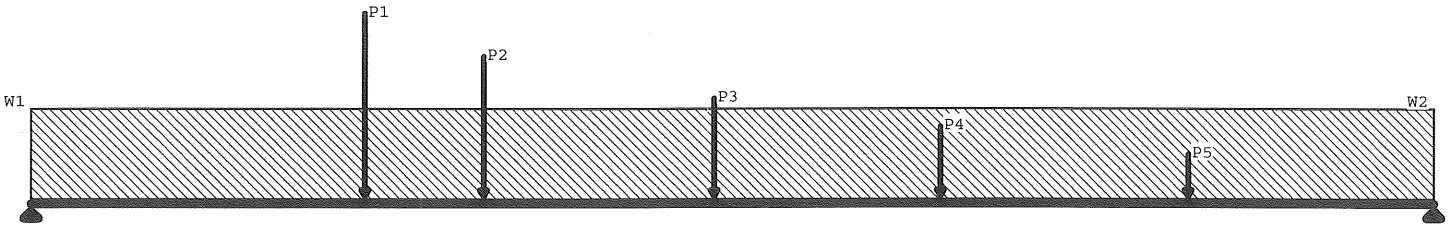
Max Pos Moment = 426.66 kip-ft at 29.120 ft

Load Diagram

RAM SBeam v3.0.1
Licensed to:

10/07/13 11:23:53

Span information (ft): I-End (0.00,0.00) J-End (59.92,0.00)



Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	14.210	1.300	0.000	0.000	1.300
P2	19.310	1.000	0.000	0.000	1.000
P3	29.120	0.700	0.000	0.000	0.700
P4	38.800	0.500	0.000	0.000	0.500
P5	49.370	0.300	0.000	0.000	0.300
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.016	0.000	0.000	0.016
W2	59.920	0.016	0.000	0.000	0.016

SDL

G2 SDL

SDL

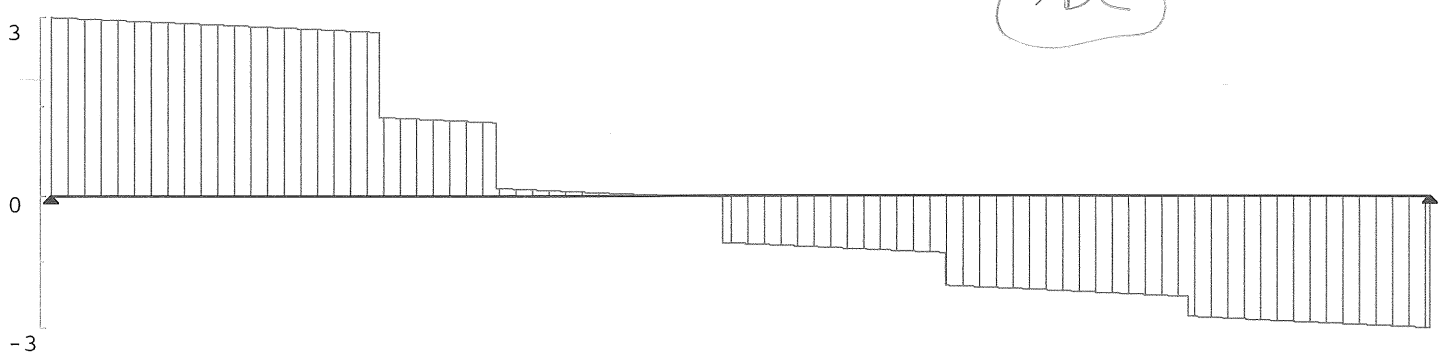
6 of 9

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

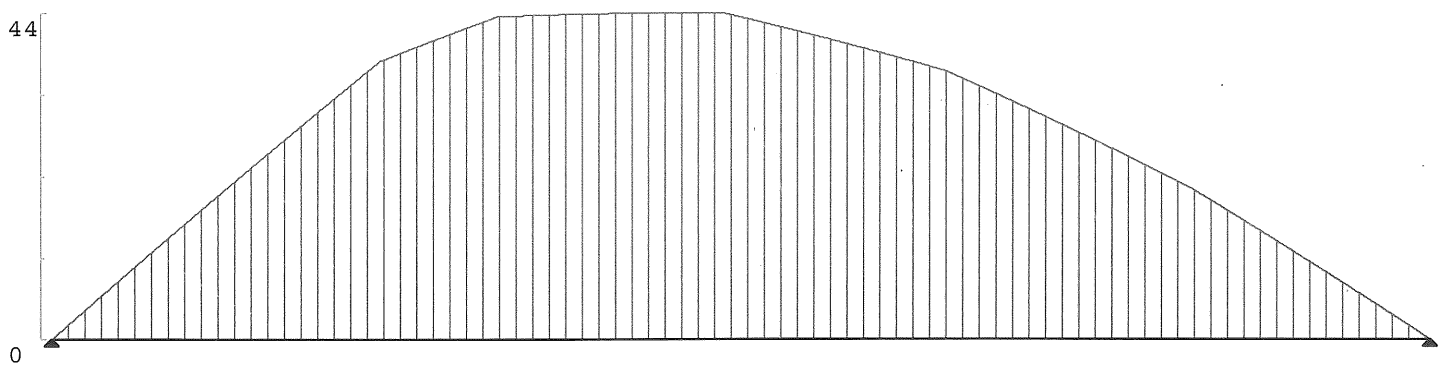
Span information (ft): I-End (0.00,0.00) J-End (59.92,0.00)

Shear

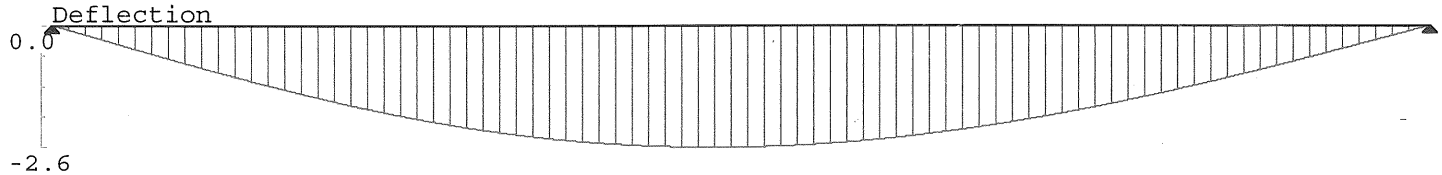
SDL



Moment



Deflection



Max DL Shear = 2.74 kips
Max Shear = 2.74 kips

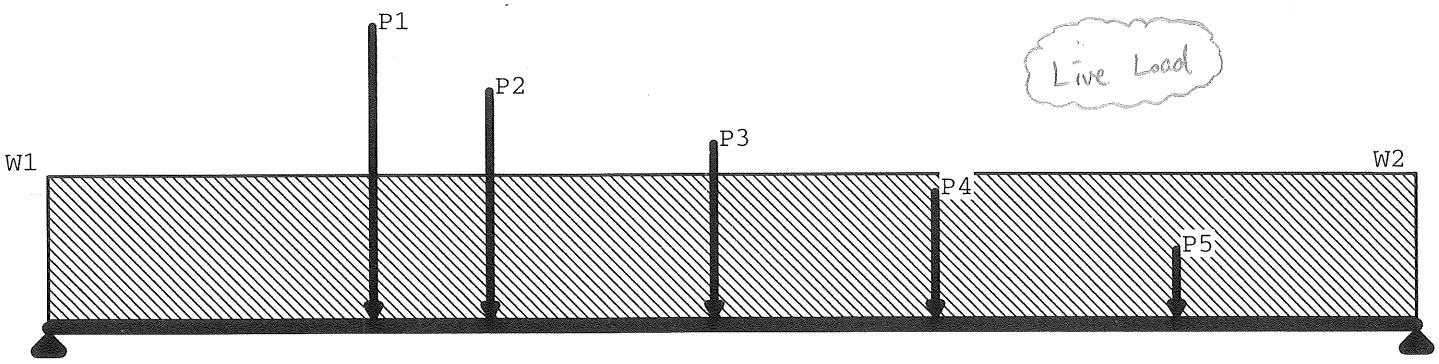
Max Pos Moment = 43.74 kip-ft at 29.120 ft

Load Diagram

RAM SBeam v4.0
Licensed to: RAM International

11/13/13 08:47:09

Span information (ft): I-End (0.00,0.00) J-End (59.92,0.00)



Load	Dist	~~DL~~ LL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	14.210	12.560	0.000	0.000	12.560
P2	19.310	9.780	0.000	0.000	9.780
P3	29.120	7.440	0.000	0.000	7.440
P4	38.800	5.330	0.000	0.000	5.330
P5	49.370	2.780	0.000	0.000	2.780
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.160	0.000	0.000	0.160
W2	59.920	0.160	0.000	0.000	0.160

RAM SBeam v3.0 Shear, Moment, and Deflection Diagrams

Span information (ft): I-End (0.00,0.00) J-End (59.92,0.00)

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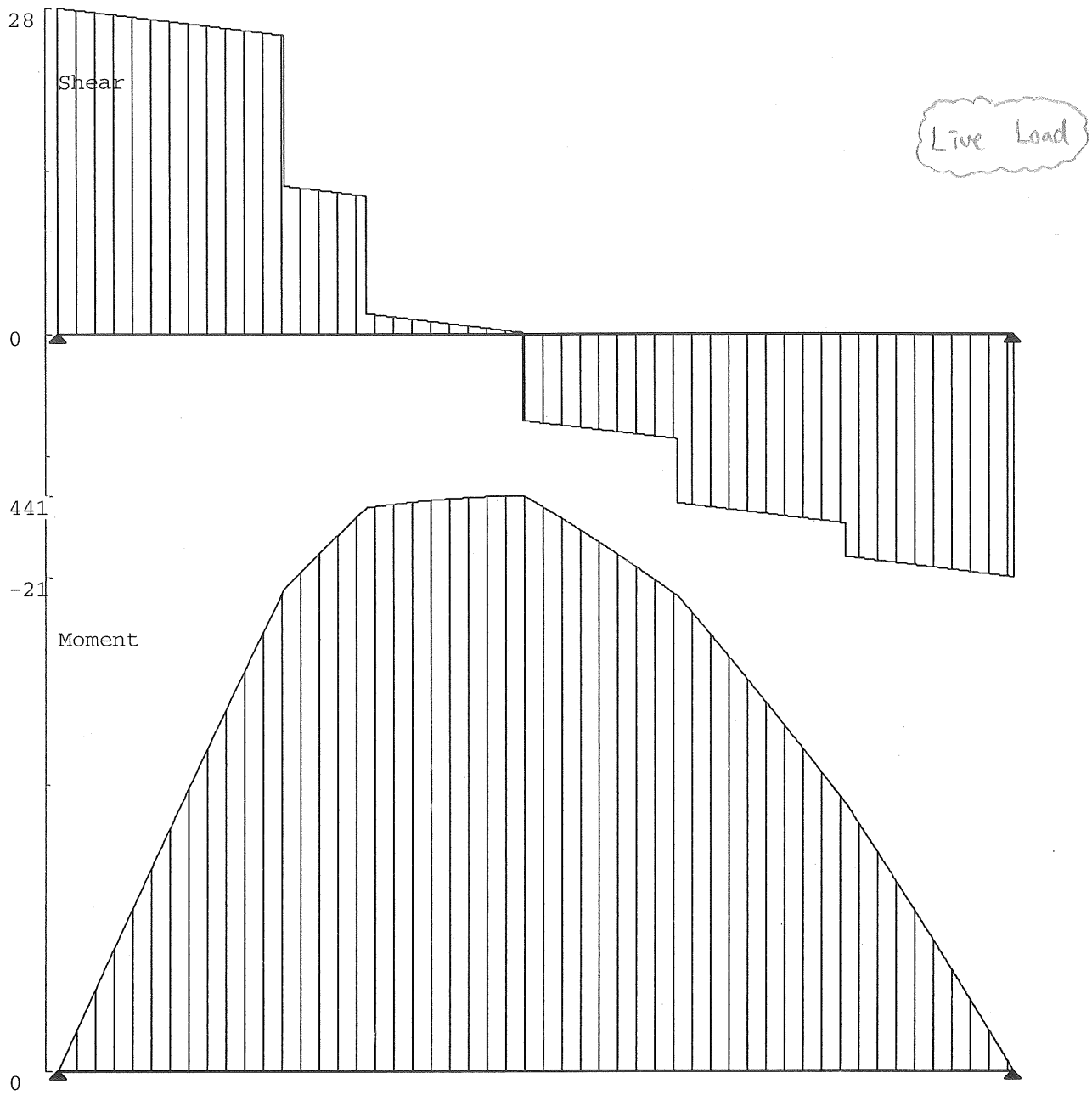

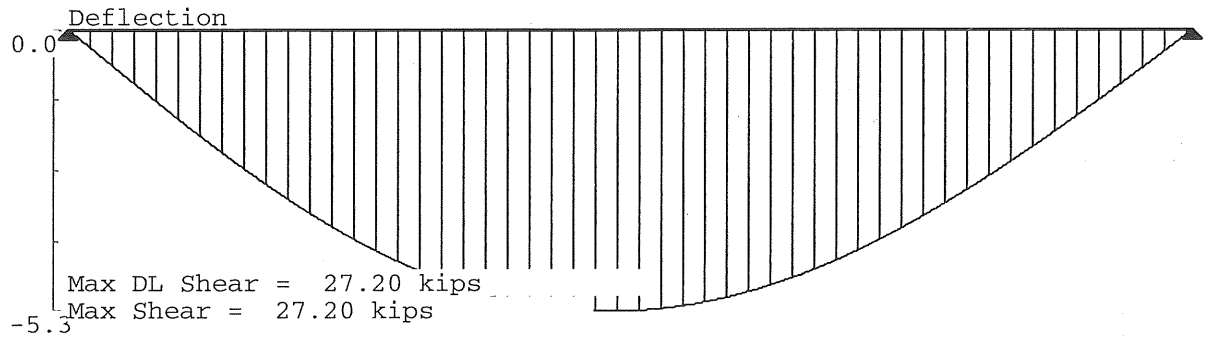

Max DL Shear = 27.20 kips
Max Shear = 27.20 kips

Max Pos Moment = 440.89 kip-ft at 29.120 ft

WEIDLINGER

PROJECT		SUBJECT	Girder 33WF152 G2
BY Jason A.	DATE 10/7/13		Spans 23-24
CHECKED BY TFKO	DATE 10/14/13		PAGE 9 OF 9

Load Rating 33WF152 G2 Spans 23~24

Inventory $LR_m = \dfrac{18.5 \times 486.4/12 - (426.66 + 43.74)}{440.89}$

$= 0.588$

$LR_v = \dfrac{11 \times 33.5'' \times 0.635'' - (26.85 + 2.74)}{27.2}$

$= 7.515$

Operating $LR_m = \dfrac{24.5 \times 486.4/12 - (426.66 + 43.74)}{396.83}$

$= 1.185$

$LR_v = \dfrac{15 \times 33.5 \times 0.635 - (26.85 + 2.74)}{27.2}$

$= 10.64$

SPANS 8-18 AND 19-23
Load Rating Summary

(ASD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

Made By: AR Date: 2-17-2014
Checked By: QH Date: 2-17-2014

Bridge: Paserelle Pedestrian Bridge Load rating - Flushing Meadow park
Rating Method: ASD

Description:
11- Span simply and continuous supported multi steel girder, stringer and floormeam bridge. Sections are W shape roll steel. The deck material is 3" thick timber plank deck.

Rating Approach:
Level 1 load rating, use 100 psf maximum live load,no truck H10 allowed. Load rating is base on timber plank deck, steel members as dead load (DL), and 20 lbs/ft^2 superimpose (SDL) for spans 8 to 18.

Geometry: All bridge information taken from plans.
Framing Plan, cross sections, connection details, lateral bracing and columns: DWG's Nos. 9 to 20 and DWG's 22 to 25.
Exiting (partial) framing plan, dwgs. Nos. 10,13 & 14 are included.

ANALYSIS

1. Stringers
Span 8 to Span 18 : Groups Nos. 1 & 2

2. Floorbeams:
Bent 8 to bent 18: Group Nos. 1 to 10

3. Girders:
Span 8 to 18 , Fascia and Interior girders:

4. Columns
Bent 8 Column W14x61
Bent 11 Column W14x68
Bent 18 Column W14x84
Bent 19 Column W14x87

LOADS

**** Dead Loads Summary Tables***
(Stringers)

STRINGERS

Group No.	Stringers (Dead Load)				
	Timber Deck+SDL (k/ft)	Stringer (k/ft)	Tot DL w (k/ft)	Moment (k-ft) M_{max}	Shear (k) V_{max}
1	0.087	0.021	0.108	1.938	0.646
2	0.080	0.025	0.105	3.725	0.885

$M_{max} = \frac{wl^2}{8}$ @ midspan $\qquad V_{max} = \frac{wl}{2}$ @ beam ends

*For Floorbeams(FBS) and Girders dead load calculation see the following sheets **' Dead Load Calculation'**

Stresses
Dead Load $f_{DL} = M^{DL}/S^{DL}$

Live Load $f_{LL} = M^{LL}/S^{LL}$

HAKS

RATINGS

$$R.F. = \frac{f_s - f_{DL} - f_{SDL}}{f_{LL}}$$

Allowable Stresses

			Inventory	Oprating
F_y =	33 ksi	f_s (ksi)	18.15	24.75
$f_s = 0.55F_y$ =	18.150 ksi	V_{all} =	11.000	ksi
$f_s = 0.75F_y$ =	24.750 ksi			

A. STRINGERS

Table 1 - Stringers Load Rating

Stringers	Stresses				Inventory	Operating	Inventory
Group No.	Dead (ksi) M_{DL}/S		Live (ksi) = M_{LL}/S		Rating Factor (R.F.) No section loss	Rating Factor (R.F.)	Rating Factor (R.F.) 20% SL
	No SL	20% SL	No SL	20% SL	$R.F.(f_s - f_{DL})/f_{LL}$	$R.F.(f_s - f_{DL})/f_{LL}$	
1 (Spans 8 & 9)	1.07	1.33	5.28	6.61	3.23	4.48	2.55
2 (Spans 10 to 18)	1.69	2.12	7.75	9.69	2.12	2.98	1.66

B. FLOORBEAMS

Table 2 - Floorbeams Load Rating

Span 11 to Span 18 Floorbeams	Stresses		Inventory	Operating	Inventory
Floorbeam No.	Dead (ksi) M_{DL}/S	Live (ksi) = M_{LL}/S	Rating Factor (R.F.) No SL	Rating Factor (R.F.)	Rating Factor (R.F.) 20% SL
	Dead Load	Pedestrian LL	$R.F.(f_s - f_{DL})/f_{LL}$	$R.F.(f_s - f_{DL})/f_{LL}$	$R.F.(f_s - f_{DL})/f_{LL}$
FB10 to FB17, FB10A to FB17A,FB10B to FB17B	1.94	7.75	2.09	2.94	
FB10 to FB17, FB10A to FB17A,FB10B to FB17B	2.42	9.69			1.62
FB18	4.58	5.85	2.32		
FB18	5.73	7.31			1.70

Table 3 - Floorbeams Load Rating

Span 8 to Span 18 Floorbeams	Stresses		Inventory	Operating	Inventory
Floorbeam No.	Dead (ksi)	Live (ksi) = M_{LL}/S	Rating Factor (R.F.) No SL	Rating Factor (R.F.)	Rating Factor (R.F.) 20% SL
	Dead Load DL	Pedestrian LL	$R.F.(f_s - f_{DL})/f_{LL}$	$R.F.(f_s - f_{DL})/f_{LL}$	$R.F.(f_s - f_{DL})/f_{LL}$
FB7	1.07	2.39	7.15	9.91	5.628
FB8	2.29	6.78	2.34	3.31	1.802
FB9	2.24	8.56	1.86	2.63	1.434
FB10	1.94	7.75	2.09	2.94	1.624
FB11 to FB17	1.94	7.28	2.23	3.14	1.729
FB7A-2, FB7B-2	4.31	14.51	0.95	1.41	0.704
FB8A-2	3.90	14.40	0.99	1.45	0.737
FB9A	7.64	10.75	0.98	1.59	0.64
FB18	4.58	5.85	2.32	3.45	1.70
FB8B-2	3.91	14.85	0.96	1.40	0.71

C. GIRDERS

Table 4 - Girders Load Rating

Span No.	Girder No.	Stresses				Inventory Rating Factor (R.F.)		Operating Rating Factor (R.F.)	
		Dead (ksi) Dead Load		Live (ksi) = M_{LL}/S Pedestrian LL		$R.F.(f_s - f_{DL})/f_{LL}$		$R.F.(f_s - f_{DL})/f_{LL}$	
		No SL	20% SL	No SL	20% SL	No SL	20% SL	No SL	20% SL
Span 8	G1, G4	3.01	3.77	6.91	8.64	2.19	1.66	3.14	
	G2, G3	5.76	7.21	7.32	9.15	1.69	1.20	2.59	
Span 9	G1, G4	3.01	3.77	7.27	9.09	2.08	1.58	2.99	
	G2, G3	6.18	7.73	9.28	11.60	1.29	0.90	2.00	
Span 10	G1, G4	3.01	3.77	3.88	3.43	3.91	4.20	5.61	
	G2	11.99	14.99	3.48	4.35	1.77	0.73	3.67	
	G3	9.68	12.10	7.74	9.68	1.09	0.63	1.95	
Spans 11 to 18	G1, G4	0.29	0.37	0.70	0.88	25.50	20.31	34.92	
	G2, G3	4.35	5.44	6.79	8.49	2.03	1.50	3.00	

D. COLUMN

Bent 8

Column size W14X61 r_y = 2.45 in l = 20.00 ft Assume A = 17.94 in^2

F_{all}=16980-0.53*(kl/r)= 11.89 ksi P_{all}= F_a* A(column) = 213.38 k

R.F. = (P_{all} - P_{DL})/P_{LL} = 1.39 No Section Loss (SL)

R.F. = (P_{all} - P_{DL})/P_{LL} = 1.04 20% Section Loss (SL)

Bent 11

Column size W14X68 r_y = 2.46 in l = 20.00 ft Assume A = 20.00 in^2

F_{all}=16980-0.53*(kl/r)= 11.94 ksi P_{all}= F_a* A(column) = 238.71 k

R.F. = (P_{all} - P_{DL})/P_{LL} = 1.98 No Section Loss (SL)

R.F. = (P_{all} - P_{DL})/P_{LL} = 1.49 20% Section Loss (SL)

Bent 18

Column size W14X84 r_y = 3.02 in l = 20.00 ft Assume A = 24.71 in^2

F_{all}=16980-0.53*(kl/r)= 13.63 ksi P_{all}= F_a* A(column) = 336.87 k

R.F. = (P_{all} - P_{DL})/P_{LL} = 2.65 No Section Loss (SL)

R.F. = (P_{all} - P_{DL})/P_{LL} = 1.94 20% Section Loss (SL)

Bent 19 - Column F-19

Column line 'F'

Column size W14*87 r_y = 3.70 in l = 20.00 ft Assume A = 25.56 in^2

F_{all}=16980-0.53*(kl/r)= 14.75 ksi P_{all}= F_a* A(column) = 377.01 k

R.F. = (P_{all} - P_{DL})/P_{LL} = 2.41 No Section Loss (SL)

R.F. = (P_{all} - P_{DL})/P_{LL} = 1.77 20% Section Loss (SL)

Table 5 - Columns Load Rating

Column Bent No.	Column Axial Load		Inventory	
	Dead Load	Pedestrian LL	Rating Factor (R.F.)	
	R_{DL} (k)	R_{PED} (K)	R.F. (P_{all}-R_{DL})/f_{LL} No SL	R.F. (P_{all}-R_{DL})/f_{LL} 20% SL
Bent 8	42.766	123.07	1.39	1.04
Bent 11	46.490	97.21	1.98	1.49
Bent 18	87.850	93.84	2.65	1.94
Bent 19	93.057	117.70	2.41	1.77

DEAD LOADS CALCULATIONS

1. Timber Plank Deck :

(Sapns 8 to 18)

Assume w_{SDL} = 0.020 k/ft^2 Including Misc.

	Timber Plank Deck DL (k/ft)	
Spans 8,9,10 to 18	Group 1 Spans 8,9 Stringer Nos. S1A to S13A, S1B to S13B, S1C to S11C	Group 2 Spans 10 to 18 Stringer Nos. S1A to S7A, S1B to S7B, S1C to S7C
thickness t_s (in)	3.00	
width - b (in)	64	57.75
Area = A = b x t_s (in^2)	192.00	173.25
Density - γ_{Timber} (k/ft^3)	0.050	
*Load DL+SDL (k/ft)	0.087	0.080

*$A*\gamma_{Timber} + w_{SDL}$ = 192*0.050/144+0.020 = 0.087 k/ft

Structure Steel Member:

2. Stringers:

	Stringer		
Span 8 to Span 10 Span 11 to Span 18	Group 1 Spans 8 & 9	Group 2 - Spans 10 to 18 S1A to S7A, S1B to S7B, S1C to S7C	
Stringer size	10 WF 21	10 WF 25	
Length L (ft)	12.00	16.83	Governs
load w_{st} (kip/ft)	0.021	0.025	
Total load (k/ft) w =DL+SDL+Str. wt.	0.108	0.105	
Moment (k-ft) M_{max} = w*L^2/8	1.938	3.725	
Shear (k) V_{max} = w*L/2	0.646	0.885	

w = DL+SDL+Str. wt. = 0.087+0.021 = 0.108 k/ft

M_{max} = w*L^2/8 = 0.108*12^2/8 = 1.938 k-ft

V_{max} = w*L/2 = 0.108*12/2 = 0.646 k

3. FLOORBEAMS

Uniform DL(Str. , Plank Deck, SDL) on FB
to find FB Moment & Reaction

	Floorbeams (Dead Loads)	
Bent 10 to Bent 17	Group 1 FB10 to FB17, FB10A to FB17A FB10B to FB17B	Group 10 FB18
Floorbeam size	21 WF 62	30WE 108
Length (ft)	38.50	38.50
Selfweight w1 (kip/ft)	0.062	0.108
Number of stringers & Fascia Girders	9	9
*Str. Uniform load + SDL w2 (k/ft)	0.177	0.670
**RC Slab or plank deck unif. load w3 (k/ft)	0.210	1.721
Total uniform load w=w1+w2+w3 (k/ft)	0.449	2.499
Moment (k-ft) M_{max}	20.528	114.251
Reaction (k) R_{max}	8.643	48.106

Staad output

* w2 = Number of stringer*stringer length*(str self wt. + SDL)/FB length = 9*16.83*(0.025+0.02)/38.5 = 0.177 k/ft

** w3 = Deck Plank thickness*wood Density*Stringer length 3/12*0.05*16.83 = 0.210 k/ft

	Floorbeams (Dead Loads)							
Bent 7 to Bent 9	Group 2 FB7	Group 3 FB7A1,FB7A3,FB7B1,FB7B3, FB8A1, FB8A3, FB8B1,FB8B3	Group 4 FB7A-2, FB7B-2	Group 5 FB8	Group 6 FB9	Group 7 FB8A-2	Group 8 FB9A	Group 9 FB8B-2
Floorbeam size	36 WF 300	10WF 21	27WF 102	33WF152	27WF102	24WF94	21WF73	24WF76
Length (ft)	77.00	13.00	47.00	60.00	49	42	42	38
Selfweight w1 (kip/ft)	0.300	0.021	0.102	0.152	0.102	0.094	0.073	0.076
Number of stringers	15	3	9	12	11	7	10	7
*Str. Uniform load + SDL w2 (k/ft)	0.096	0.114	0.094	0.098	0.129	0.082	0.193	0.091
**Timber plank deck uniform load w3 (k/ft)	0.150	0.150	0.150	0.150	0.175	0.150	0.188	0.150
Total uniform load w=w1+w2+w3 (k/ft)	0.546	0.285	0.346	0.400	0.406	0.326	0.453	0.317
Girder reaction (k)	-	-	-	-				
Moment (k-ft) M_{max} =w*L^2/8 , STAAD OUTPUT	99.236	6.011	95.598	92.987	49.722	71.88	95.90	57.15
Reaction (k) R_{max-FB} = w*L/2 STAAD OUTPUT	22.335	1.850	8.136	12.132	10.09	6.85	25.60	6.02

Staad output

* w2 = Number of stringer*stringer length*(str self wt. + SDL)/FB length = 15*12*(0.021+0.02)/77 = 0.096 k/ft

** w3 = Deck Plank thickness* Density*Stringer length = 3/12*0.050*12 = 0.150 k/ft

4. GIRDERS

(DL)

	GIRDER							
Spans 8,9,10 Spans 11 to 18	Fascia Girder Spans 8 to 10		Int. Girder Span 8	Int. Girder Span 9	Int. Girder Span 10	Int. Girder Span 10	Interior Girders Spans 11 to 18	Fascia Girders Spans 11 to 18
Girder No.	G1	G4	G2, G3	G2, G3	G2	G3	G2, G3	G1, G4
Girder size	24 WF 76	24 WF 76	24 WF120	24 WF100	24 WF84	24 WF130	36 WF160	24 WF 76
Length (ft) L	36.00	39.00	37.00	38.00	29.20	38.00	50.50	50.50
Self wt. unif. Load w(kip/ft)	0.076	0.076	0.120	0.100	0.084	0.130	0.160	0.076
*FBS end reaction on G1, G2 point load R_{max-FB} (k)	1.850	1.850	9.986	8.696	25.600	25.600	8.643	4.322
Railing Load (k/ft) W_{rail} on G1, G4	0.030	0.030						0.030
M_{max} = (W_{rail}+w)*L^2/8+ R_{max-FB} * L/3 (k-ft)	39.366	44.197	143.690	128.193	195.833	266.665	196.496	4.288
* , ** Girder end Reaction (k)	3.758	3.917	12.206	10.596	14.026	15.270	12.683	6.241

* G1, G4 FB7A1 end reaction+W_{rail} *L(G1)/2+w*L/2 = P1 = R_{max} = 3.758 K R_{max-FB} = 1.850 k

** G2, G3 (FB7A1+FB7A2)end reaction+w*L/2 = P2 = R_{max} = 12.206 K

Uniform DL on G1, G4 (spans 11 to 18) : w(DL) = W_{rail} +W_{DECK}+W_{self} = 0.03+3/12*7.25'/2*0.05+0.076 = 0.151 k/ft

(Three continuous spans) l = L/3 = 16.833 ft

M_{DL} G1, G4 (spans 11 to 18) : M(DL) = 0.1 * w(DL) * l^2 = 4.288 k-ft

5. COLUMNS

Dead Load (DL) carry by Column

Bent 8 (W14X61)

P_{DL} = G3 (spans 8&9) end reaction +G4(spans 8&9) end reaction+ FB8 end reaction = 11.468+10.596+2*3.917+12.132 = 42.766 k

Bent 11 (W14X68)

P_{DL}=G2(spans 11&12)end reaction+G1(spans 11&12)end reaction+FB11end reaction= 12.683*2+6.421*2+8.643 = 46.490 k

Bent 18 (W14X84)

$P_{DL} = R_{B18} + R_{B17B}$ +47/2*38.5*5.5/12*0.150/2 = 87.850 k

Bent 19 - Column F-19

W14X87

Dead Load R_{DL} = Conc. slab+stringer wt. +FBs wt. + Misc

Slab thickness	ts =	5.50	in	Conc spec. wt.	$\gamma_{CONC.}$ =	0.150	k/cf
Span 19 length	L1 =	47.00	ft	No. of stringer	n =	7	stringer
Span 20 length	L1 =	60.00	ft	FBS length	L_{FB} =~	44	ft
				Slab width	b =~	44	ft

P_{DL} = ts/12*(L1+L2)/2 * b/2 * $\gamma_{CONC.}$*1.15 Misc = 93.057 k

Table 1. Stringers Group - Section Properties

Group No.	Stringers						
	Section	Length (max) (ft)	Cantil. Length -ft	Spacing (ft)	Area (in^2)	As built Sec. Modulus S (in^3)	Sec. Modulus 20% Loss S_{insp} (in^3)
1	10 WF 21	12.00		5.33	6.28	21.8	17.44
2	10 WF 25	16.83		4.81	7.35	26.4	21.12

Table 2. Girders - Section Properties

Girder No.	Girders			
	Section	Area (in^2)	Section Modulus (in^3)	Section Modulus 20% loss (in^3)
G1,G4	24 WF 76	22.4	176.0	140.80
Span10 - G2	24WF 84	24.7	196.0	156.80
Span 9 - G2, G3	24WF 100	29.43	248.9	199.12
Span 8- G2, G3	24WF 120	35.3	299.1	239.28
Spans 11 to 18 - G2, G3	36 WF 160	47.0	542.0	433.60
Span10 - G3	24 WF 130	38.21	330.7	264.56
	14 WF 30	8.9	42.0	
	C12X25	7.4	18.2	
	33 WF 130	38.3	406.0	
	12 WF 27	7.97	34.1	
	24 WF 130	38.21	330.7	264.56

Table 3. Floorbeams - Section Properties

Floorbeam Group No.	Floorbeams					
	Section	Area (in^2)	Section Modulus (in^3)	Section Modulus -As Inspection (in^3)	Section Modulus 20% loss (in^3)	
1	21 WF 62	18.30	127.0	92.71	101.6	21 WF 62
2	36 WF 300	88.30	1110.0	810.30	888	36 WF 300
3	10WF 21	6.28	21.8	15.91	17.44	10WF 21
4	27 WF 102	30.01	266.3	194.40	213.04	27WF 102
5	33 WF 152	44.71	486.4	355.07	389.12	33WF152
6	27WF102	30.01	266.3	194.40	213.04	27WF102
7	24WF94	27.63	220.9	161.26	176.72	24WF94
8	21 WF 73	21.46	150.7	110.01	120.56	21WF73
9	24 WF 76	22.37	175.4	128.04	140.32	24WF76
10	30 WF 108	31.77	299.2	218.42	239.36	30WE 108

Table 4. Column - Section Properties

Column Location	Columns		
	Section	Area (in^2)	r_y (in)
Bent 8	14 WF 61	17.94	2.5
Bent 11	14 WF 68	20.00	2.5
Bent 18	14 WF 84	24.71	3.02
Bent 19	14 WF 87	25.56	3.7

Live Loads :

1. Pedestrian Load:

Pedestrian Live Load: w_{pdes} = 100 psf

2 . H10 Truck load: H10 = 20 k Not Used

1. STRINGERS:

Truck Load NOT USED

Group No.	Span No.	Stringer size	Wheel load P (K)	Distri. Factor DF	Stringer spacing s (ft)	Length L (ft)	M_{LL} = PL/4 *DF (k-ft)
1	Spans 8 & 9	10 WF 21	8	1.000	5.33	12.00	24.00
2	Spans 10 to 18	10 WF 25	8	1.000	0.00	16.83	33.67

Pedestrian Load

Group No.	Span No.	Stringer size		Distri. Factor DF	Stringer spacing s (ft)	Length L (ft)	$M_{LL} = w_{pedes}L^2/8$ (k-ft)
1	Spans 8 & 9	10 WF 21		1.000	5.33	12.00	**9.60**
2	Spans 10 to 18	10 WF 25		1.000	4.81	16.83	**17.05**

Stringer spacing s = 64 in spans 8 & 9
Stringer spacing s = 57.75 in spans 10 to 18

$M_{truck} = M_{LL}$ = 24.00 k-ft Spans 8 & 9
$M_{truck} = M_{LL}$ = 33.67 k-ft Spans 10 to 18

Impact I = 0

Note: Distribution factor DF = s/5.5 where 's' is stringer spacing.
DF = 1 Conservative

2. FLOORBEAMS:

Pedestrian Load :

Group No.	Span No.	FB No.	Floor - beams Size	Pedestrian Load wp (psf)	FB Spacing s (ft)	Length L (ft)	M_{LL} (k-ft) Staad output	R_{LL} (k) staad output
1	Span 11 to Span 18	FB10 to FB17, FB10A to FB17A, FB10B to FB17B	21 WF 62	100	16.83	38.50	77.00	32.40

Uniform pedestrian load Wped = wp*s = 1.683 k/ft

H10 Truck Load and Pedestrian Load:

Bent No.	FB No.	FB Section	M_{LL} (k-ft) one truck	R_{LL} (k)	M_{LL} (peds (k-ft)	R_{LL} Pd (k)
7	FB7	36 WF 300	168.00	19.00	221	49.80
8	FB8	33WF152	158	17.00	275	35.90
9	FB9	27WF102	114	17.40	190	38.53
	FB7A-2 FB7B-2	27WF102	164.00	15.00	322	27.43
	FB8B-2	24WF76	216.00	25.08	217	22.80
	FB8A-2	24WF94	144.00	14.90	265	25.20
	FB9A	21WF73	92.00	17.65	135	36.00
10	FB10	21WF62	73	18.20	82	38.53
11 to 17	FB11	21WF62	73	18.20	77	32.40
18	FB18	30WF108	73	18.20	146	61.41

3. GIRDERS:

wped = 0.100 ksf

Stringer Spacing s = 5.33 ft Spans 8,9 & :

Stringer Spacing s = 4.81 ft Span 10 to span 18 (bents 10 to 18)

Mped = wped * s*L^2/8 =	101.4	k-ft	Span 8	G1,G4
Mped = wped * 2*s*L^2/8 =	182.5	k-ft	Span 8	G2,G3
Mped = wped * s *L^2/8 =	106.667	k-ft	Span 9	G1,G4
Mped = wped *2* s *L^2/8 =	192.533	k-ft	Span 9	G2,G3
Mped = wped * s *L^2/8 =	56.843	k-ft	Span 10	G1,G4
Mped = wped *2* s *L^2/8 =	213.333	k-ft	Span 10	G2,G3
Mped = wped *2* s *L^2/8 =	306.827	k-ft	Spans 11 to 18	G2,G3

Span No.	Girder No.	Girder Section	Length L (ft)	M_{LL} (k-ft)	
8	G1, G4	24WF76	39	101.40	Ped Load
	G2,G3	24WF120	37	182.5	Ped Load
9	G1, G4	24WF76	40	106.67	Ped Load
	G2,G3	24WF100	38	192.533	Ped Load
10	G1, G4	24WF76	29.2	56.84	Ped Load
	G2	24WF84	29.2	56.84	Ped Load
	G3	24WF130	40	213.33	Ped Load
11 To 18	G2,G3	36WF160	50.5	306.83	Ped Load
11 To 18	G1,G4	24WF76	50.5	10.27	Ped Load

Uniform LL on G1, G4 (spans 11 to 18) : $w(LL) = W_{pdes} * 7.25/2 =$ 0.363 k/ft
(Three continuous spans) $l = L/3 =$ 16.833 ft
M_{LL} G1, G4 (spans 11 to 18) : $M(LL) = 0.1 * w(LL) * l^2 =$ 10.272 k-ft

Live Load (LL) carry by Column

Bent 8 (W14X61)

Area A = L * W
Length L = (span 8+ span 9)/2 =(35+36)/2= 35.5 ft
Width W= (48/2 + 16/3 * 2) = 34.67 ft

A = L*W = 1231 ft^2 $P_{LL} = A * w_{pedes} =$ 123.1 k

Bent 11 (W14X68)

Area A = L * W
Length L = (span 11+ span 12)/2 =(50.5*2)/2= 50.5 ft
Width W= (38.5/2) = 19.25 ft

A = L*W = 972 ft^2 $P_{LL} = A * w_{pedes} =$ 97.2 k

Bent 18 (W14X84)

Area A = L * W
Length L = (span18+span19)/2 =(47+50.5))/2= 48.75 ft
Width W= (38.5/2) = 19.25 ft

A = L*W = 938 ft^2 $P_{LL} = A * w_{pedes} =$ 93.8 k

Bent 19 - Column F-19

W14X87

Area A = L * W
Length L = (span19+span20)/2 =(47+60))/2 = 53.5 ft
Width W= slab width at bent 19/2 = 44/2 = 22.0 ft

A = L*W = 1177 ft^2 $P_{LL} = A * w_{pedes} =$ 117.7 k

Bridge: Paserelle Pedestrian Bridge Load rating

Rating Method: ASD

Description:

5 - Span simply and continuous supported multi steel girder, stringer and floormeam bridge. Sections are W shape roll steel. The deck material is 5.5" concrete deck slab thickness.

Rating Approach:

Level 1 load rating, use 100 psf maximum pedestrian live load or truck H10 as governs for spans 19 to 23.

Load rating is base on concrete slab deck, steel members as dead load and 20 lbs/ft^2 superimpose DL (SDL) for spans 19 to 23.

Geometry: All bridge information taken from plans.

Framing Plan, cross sections, connection details, lateral bracing and columns: Exist. dwg's Nos. 9 to 20 and DWG's 22 to 25.

Existing (partial)framing plan, dwgs nos. 10,13 & 14 are included.

ANALYSIS

1. Stringers

Span 19 ot Span 23 : Groups Nos. 3 to 9

2. Floorbeams:

Spans 19 to 23 (No group numbers)

3. Girders:

Span 22 , Interior girder G2

4. Columns

Bent 8 Column W14x61
Bent 11 Column W14x68
Bent 18 Column W14x84
Bent 19 Column W14x87

LOADS

Dead Loads Summary Tables

Main Level:

A. STRINGERS

Group No.	Stringers (Dead Load)				
	RC Slab Deck+SDL (k/ft)	Stringer (k/ft)	Tot DL w (k/ft)	Moment (k-ft) M_{max}	Shear (k) V_{max}
3	0.523	0.110	0.633	174.714	14.869
4	0.351	0.120	0.471	204.883	13.890
5	0.523	0.030	0.553	24.942	5.251
6	0.523	0.027	0.550	6.872	2.749
7	0.490	0.062	0.552	27.590	5.518
8	0.536	0.076	0.612	84.524	10.168
9	0.536	0.145	0.681	366.320	57.750

$M_{max} = \frac{wl^2}{8}$ @ midspan $\qquad V_{max} - \frac{wl}{2}$ @ beam ends

*For Floorbeams(FBS) and Girders dead load calculation see the following sheets **' Dead Load Calculation'**

Stresses

Dead Load $\qquad f_{DL} = \quad M^{DL}/S^{DL}$

Live Load $\qquad f_{LL} = \quad M^{LL}/S^{LL}$

RATINGS

$$R.F. = \frac{f_s - f_{DL} - f_{SDL}}{f_{LL}}$$

Allowable Stresses

A. STRINGERS

Table 1 - Stringers Load Rating

F_y = 33 ksi

	Inventory	
f_s (ksi)	18.15	24.75

Stringers	Stresses		Rating Factor (R.F.)
Group No.	Dead (ksi)	Live (ksi) = M_{LL}/S	$R.F.(f_s - f_{DL})/f_{LL}$
	Dead Load	H10 Truck or ped. Load	
3	7.652	4.555	2.305
4	8.220	5.376	1.847
5	7.126	14.805	0.745
6	2.418	9.597	1.639
7	7.743	4.580	2.272
8	5.763	7.067	1.753
9	11.801	3.339	1.901

B. FLOORBEAMS

Span 19 to Span 23 Floorbeams	Stresses		Rating Factor (R.F.)
Floorbeam No.	Dead (ksi)	Live (ksi) = M_{LL}/S	$R.F.(f_s - f_{DL})/f_{LL}$
	Dead Load	H10 Truck Load	
FB19-1	6.303	7.277	1.628
FB19-2	7.939	9.122	1.119
FB19-A1	1.407	2.679	6.251
FB19-A2	9.348	2.863	3.075
*FB20-3	11.551	4.369	1.510

*Dead(ksi)	fs(DL) = M/S = (131.248+300)*12/448 =	11.551	ksi	See DL and Canopy Moments(M_{max}, Mcan)
*Live (ksi)	fs(LL) = M/S = (163.13)*12/448 =	4.370	ksi	See LL Moments ($M_{PED.}$)

C. GIRDERS

Span No.	Girder No.	Stresses		Rating Factor (R.F.)
		Dead (ksi)	Live (ksi) = M_{LL}/S	$R.F.(f_s - f_{DL})/f_{LL}$
		Dead Load	H10 Truck Load	
*Span 22	G2	-	-	3.066

* See attached Calculation

D. COLUMN

Bent 19 - Column F-19

Column line 'F'

Column size W14*87 r_y = 3.70 in l = 20.00 ft Assume

F_{all}=16980-0.53*(kl/r)= 14.75 ksi P_{all}= F_a* A(column) = 377.01 k A = 25.56 in^2

R.F. = (P_{all} - P_{DL})/P_{LL} = 5.89 No Section Loss (SL)

Table 5 - Columns Load Rating

Column Bent No.	Column Axial Load		Inventory
	Dead Load	Pedestrian LL	Rating Factor (R.F.)
	R_{DL} (k)	R_{PED} (K)	R.F. (P_{all}-R_{DL})/f_{LL}
Bent 19	93.057	48.21	5.89

DEAD LOADS CALCULATIONS

1. Concrete Deck Slab:

Assume w_{SDL} = 0.020 k/ft^2 Including Misc.

	Reinf. Concrete slab DL (k/ft)							
		Group# 3 Span 19 S2 to S8	Group# 4 Span 20 S2 to S8	Group # 5 Span 20 S10 to S17	Group # 6 Spans 21, 23 S1 to S18	Group # 7 Span 22 S1 to S3	Group # 8 Span 22 S4 to S13 S15 to S17	Group # 9 Span 22 S14
thickness t_s (in)	5.50							
width - b (in)		87.75	57.75	87.75	87.75	82	90.00	90.00
Area = A = b x t_s (in^2)		482.63	317.63	482.63	482.63	451.00	495.00	495.00
Density - γ_{conc} (k/ft^3)	0.150							
*Load DL+SDL (k/ft)		0.523	0.351	0.523	0.523	0.490	0.536	0.536

Aγ_{Conc} + w_{SDL} = 352*0.150/144+0.020 = 0.523 k/ft

Structure Steel Member:

2. Stringers:

	Stringers						
Span 19 to Span 23	Group# 3 Span 19 S2 to S8	Group # 4 Span 20 S2 to S8	Group # 5 Span 20 S10 to S17	Group # 6 Spans 21, 23 S1 to S18	Group # 7 Span 22 S1 to S3	Group # 8 Span 22 S4 to S19	Group # 9 Span 22 S14
Stringer size	24WF110	24WF120	14 WF 30	12 WF 27	24 WF 62	24 WF 76	24 WF 145
Length L (ft)	47.00	59	19	10	20	33.25	33.25
load w_{st} (kip/ft)	0.110	0.120	0.030	0.027	0.062	0.076	0.145
Total load DL+SDL+Steel (k/ft)	0.633	0.471	0.553	0.550	0.552	0.612	0.681
Moment M_{max} (k-ft)	174.71	204.88	24.94	6.87	27.59	84.524	94.06

Stringer S14 - Span 22 $M_{DL+CANOPY}$ = 366.32 k-ft see staad output

V_{canopy} = 57.75 k

3. FLOORBEAMS

Uniform DL(Str. ,Conc. Deck, SDL) on FB
to find FB Moment & Reaction

	Reinf. Conc. Slab wt. (k/ft)				
Span 19 to Span 23	Spans 19, 20 Interior S2 to S8	Spans 20, 21 S10 to S17	Span 21 S2 to S9	Spans 22, 23 S1 to S14	Spans 22, 23 S15 to S18
thickness t_s (in)	5.50	5.50	5.50	5.50	5.50
width - b (in)	57.75	90.00	82.5	82.5	90.00
b x t_s (in^2)	317.63	495.00	453.75	453.75	495.00
Density - γ_{conc}(k/ft^3)	0.145	0.145	0.145	0.145	0.145
*Load (k/ft)	0.320	0.498	0.457	0.457	0.498

* b x ts * γ_{conc} = 317.63*0.145/144 = 0.320

	Floorbeams					
Bent 19 to Bent 23	FB19-1	FB19-A1	FB201, FB211	FB20-3, FB20-4, FB21-3, FB21-4	FB22-1	FB22-2,FB22-3, FB23-2, FB23-3,FB23-4
Floorbeam size	33 WF 141	33 WF 141	33WF 130	33WF 141	33WF 130	33WF 130
Length (ft)	20.25	22.50	22.00	30.00	27.50	27.50
Weight - w(kip/ft)	0.141	0.141	0.130	0.141	0.130	0.130

	Floorbeams					
Bent 19 to Bent 23	FB19-2	FB19-A2	FB20-2, FB21-2	FB22-4 , FB22-5 FB23-5, FB23-6	FB23-1	FB23-2
Floorbeam size	33 WF 141	33 WF 141	33WF 130	33 WF 141	33WF 130	33WF 130
Length (ft)	23.00	30.00	27.50	30.00	10.00	27.5
Weight - w(kip/ft)	0.141	0.141	0.130	0.141	0.130	0.130

	Floorbeams (Dead Loads)						
Bent 19 and Bent 20	FB19-1	FB19-2	FB19A-1	FB19A-2	FB20-1	FB20-2	FB20-3, FB20-4
Floorbeam size	33WF 141	33WF 141	33WF 141	33WF 141	33WF130	33WF130	33WF141
Length (ft)	20.25	23.00	22.50	30.00	22.00	27.50	30.00
Selfweight w1 (kip/ft)	0.141	0.141	0.141	0.141	0.130	0.130	0.141
Number of stringers n	3	3	3	3	3	3	3
*Str. Uniform load w2 (k/ft)	0.907	0.799	0.038	0.029	0.501	0.401	0.042
**Conc. Slab uniform load load w3 (k/ft)	3.542	3.542	0.651	0.651	2.313	2.313	0.984
Total uniform load w=w1+w2+w3 (k/ft)	4.591	4.482	0.830	0.821	2.944	2.844	1.167
Moment (k-ft) M_{max} = w*L^2/8	235.309	296.386	52.546	92.346	178.109	268.821	131.248
Shear (k) V_{max} =w*L/2	46.481	51.545	9.341	12.313	32.383	39.101	17.500

* n*Σ Str. Length*Str wt. (Span19,Span20)/2/FB length

w2 = 3*(47*0.110+59*0.120)/2/20.25 = 0.907 k/ft

** Slab thickness*Conc density*(Span 19 length+span 20 length)/2)+ SDL

W3 = (5.5/12*0.145)*(47+59)/2+0.02 = 3.542 k/ft

Canopy Column load Pcan = 40 k FB20-3 length L = 30.00 ft

FB20-3 Mcan = Pcan * L/4 = 300.00 k-ft

fs(DL) = M/S = (131.248+300)*12/448 = 11.551 ksi

FB19A-2 $M_{DL+CANOPY}$ = 349.000 k-ft see staad output

4. GIRDERS

See attached Dead Load Calculation - Girder 2, Span22

5. COLUMNS

Dead Load (DL) carry by Column

Bent 19 - Column F-19

W14X87

Dead Load R_{DL} = Conc. slab+stringer wt. +FBs wt. + Misc

Slab thickness ts = 5.50 in

Span 19 length L1 = 47.00 ft

Span 20 length L1 = 60.00 ft

Conc spec. wt. $\gamma_{CONC.}$ = 0.150 k/cf

No. of stringer n = 7 stringer

FBS length L_{FB} =~ 44 ft

Slab width b =~ 44 ft

P_{DL} = ts/12*(L1+L2)/2 * b/2 * $\gamma_{CONC.}$*1.15 Misc = 93.057 k

Table 1. Stringers Group - Section Properties

Group No.	Stringers				
	Section	Length (max) (ft)	Spacing (ft)	Area (in^2)	Sec.(in^3) Modulus
3	24WF 110	47.00	5.33	32.35	274
4	24WF 120	59.00	5.33	35.3	299.1
5	14 WF 30	19.00	5.33	8.85	42
6	12WF 27	10.00	5.33	7.97	34.1
7	24WF 62	20.00	6.88	18.2	131
8	24 WF 76	33.25	5.33	22.4	176
9	24 WF 145	33.25	7.50	42.62	372.5

Table 2. Girders - Section Properties

Girder No.	Girders		
	Section	Area (in^2)	Section Modulus (in^3)
1	24 WF 76	22.4	176.0
2	24WF 84	24.7	196.0
3	24WF 100	29.43	248.9
4	24WF 120	35.3	299.1
5	36 WF 160	47.0	542.0
6	33WF 130	38.26	404.8
7	14 WF 30	8.9	42.0
8	C12X25	7.4	18.2
9	33 WF 130	38.3	406.0
10	12 WF 27	7.97	34.1
11	24 WF 130	38.50	329.0

Table 3. Floorbeams - Section Properties

Floorbeam No.	Floorbeams		
	Section	Area (in^2)	Section Modulus (in^3)
1	21 WF 62	18.3	127.0
2	36 WF 300	88.3	1110.0
3	33 WF 152	44.70	487.0
4	27 WF 102	30.0	267.0
5	24 WF 76	22.4	176.0
6	21 WF 73	21.5	151.0
7	36 WF 160	47.0	542.0
8	33 WF 141	41.6	448.0
9	33 WF 130	38.3	406.0
10	30 WF 108	31.7	299.0
11	24 WF 94	27.7	222.0

Table 4. Column - Section Properties

Column Location	Columns		
	Section	Area (in^2)	r_y (in)
Bent 19	14 WF 87	25.56	3.7

Live Loads :

1. Pedestrian Load:

Pedestrian Live Load: wpd = 100.000 psf γconc = 150 pcf

2 . H10 Truck load:

H10 = 20.000 k

1. STRINGERS:

Group No.	Span No.	Stringer size	Wheel load P (K)	Distri. Factor DF	No Impact I	Length L (ft)	M_{LL} = PL/4 *DF (k-ft) Staad output
3	Span 19	24WF110	H10 Truck	1	0.00	47.00	104.00
4	Span 20	24WF120	H10 Truck	1	0.00	59.00	134.00
5	Span 20	14 WF 30	8	1.364	0.00	19.00	51.82
6	Spans 21, 23	12 WF 27	8	1.364	0.00	10.00	27.27
7	Span 22	24 WF 62	8	1.250	0.00	20.00	50.00
8	Span 22	24 WF 76	H10 Truck	1.364	0.00	33.25	93.46
* 9	Span 22	24 WF 145	H10 Truck	1.364	0.00	33.25	93.46

* Mped = 103.65 k-ft Governs see staad output

M_{LL} = 104 Ref. see ftaad output Group 3
M_{LL} = 134 k-ft Ref. see ftaad output Group 4
M_{LL} = 40 k-ft Ref. see ftaad output
M_{LL} = 68.54 k-ft Ref. see staad output Group 8
M_{LL} = 68.54 k-ft Ref. see staad output Group 9
Impact I = 50/(L+125) = 0

Note: Distribution factor DF = S/5.5 where 'S' is stringer spacing.

For S = 4'- 9 3/4"= ft DF=S/5.5= 0.875 Spans 11 to 20
S = 5'-4" = 4.8125 ft DF=S/5.5= 0.970 Spans 8,9 & 10
S = (27'-6")/4= 5.333 ft DF=S/5.5= 1.250 Span 22
S = 6'-10.5" = 6.875 ft DF=S/5.5= 1.364 Spans 20 to 23
S = 7'-6" = 7.5 ft

2. FLOORBEAMS:

H10 Truck Load:
Two trucks

Group No.	Span No.	FB No.	Floor - beams Size	Wheel load P (K)	Impact I	Length L (ft)	M(k-ft) Staad output	R_{LL} (k) staad output
	Span 19	FB191	33WF141	8.000	0.00	20.25	82.50	19.36
		FB192				23.00	104.00	20.87
	Span 20	FB19A1	33WF141	8.000	0.00	22.50	100.00	20.62
		FB19A2				30.00	160	23.466
	Span 20	FB203	33WF141	h10		30.00	*Mh10	**Vh10

H10 = 20 k **Vh10=H10/2 = 10
FB20-3 *Mh10 =(L/2-2.8)*H10/2 = 122 k-ft Do not Governs
FB19-2 one trucks ⟶ M_{LL} = 69.33 k-ft see staad output Do not Governs
FB19-1 one trucks ⟶ M_{LL} = 58.33 k-ft see staad output Do not Governs

Pedestrian Load:

Bent No.	Span No.	FB No.	Floor - beams Size	Pedestrian Load (wped psf)	FB Tributry width s (ft)	Length L (ft)	M_{Ped} (k-ft)	R_{ped} (k)
19	Span 19	FB19-2	33WF 141	100	51.50	23.00	340.54	59.23
19A	Span 20	FB19A-1	33WF 141	100	9.50	22.50	60.12	10.69
19A	Span 20	FB19A-2	33WF 141	100	9.50	30.00	106.88	14.25
*20	Span 20	*FB20-3	33WF141	100	14.50	30.00	163.13	21.75
19	Span 19	FB19-1	33WF 141	100	53.00	20.25	271.67	53.66

Governs

*M(LL)=100/1000*14.5*(30)^2/8= 163.13 k-ft

3. GIRDERS:

See attached Live Load Calculation - Girder 2, Span22

Live Load (LL) carry by Column

Bent 18 (W14X84)

Area A = L * W

Length L = (span18+span19)/2 =(47+50.5))/2= 48.75 ft

Width W= (38.5/2) = 19.25 ft

A = L*W = 938 ft^2

$P_{LL} = A^* w_{pedes}$ = 93.8 k

Bent 19 - Column F-19

W14X87

Area A = L * W

Length L = (span19+span20)/2 =(47+60))/2 = 21.915 ft

Width W= slab width at bent 19/2 = 44/2 = 22.0 ft

A = L*W = 482 ft^2

$P_{LL} = A^* w_{pedes}$ = 48.2 k

```
**************************************************
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*           STAAD.Pro V8i SELECTseries4          *
*           Version  20.07.09.21                 *
*           Proprietary Program of               *
*           Bentley Systems, Inc.                *
*           Date=    JAN 30, 2014                *
*           Time=    16:41:54                    *
*                                                *
*      USER ID: HAKS Engineering                 *
**************************************************
```

3" Timber deck w/100 PSF LL

```
     1. STAAD PLANE
INPUT FILE: one  Span -ends cantilever - FB7.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 29-JAN-14
     4. JOB NAME PASERELLER PEDESTRIAN BRIDGE
     5. JOB CLIENT DDC
     6. JOB NO 659
     7. JOB PART LOAD RATING
     8. JOB REF PLANK DECK
     9. ENGINEER NAME AR
    10. END JOB INFORMATION
    11. INPUT WIDTH 79
    12. UNIT FEET KIP
    13. JOINT COORDINATES
    14. 1 0 0 0; 2 13 0 0; 3 61 0 0; 4 77 0 0
    15. MEMBER INCIDENCES
    16. 1 1 2; 2 2 3; 3 3 4
    17. DEFINE MATERIAL START
    18. ISOTROPIC STEEL
    19. E 4.176E+006
    20. POISSON 0.3
    21. DENSITY 0.489024
    22. ALPHA 6E-006
    23. DAMP 0.03
    24. TYPE STEEL
    25. *FY=33 KSI
    26. STRENGTH FY 4752 FU 8352 RY 1.5 RT 1.2
    27. END DEFINE MATERIAL
    28. MEMBER PROPERTY AMERICAN
    29. 1 TO 3 TABLE ST W36X300
    30. SUPPORTS
    31. 2 3 PINNED
    32. CONSTANTS
    33. MATERIAL STEEL ALL
    34. DEFINE MOVING LOAD
    35. *TYPE 1 LOAD 8 8 8 8
    36. *REAR WHEEL LOAD IS 8 KIP
    37. TYPE 1 LOAD 8 8
    38. DIST 6
```

```
39. *DIST 6 4 6
40. *DISTANCE BETWEEN REAR WHEEL IS 6 FT
41. *SPACING BETWEEN TWO TRUCKS IS 4 FT
42. LOAD GENERATION 140
43. TYPE 1 0 0 0 XINC 0.5
44. LOAD 141 LOADTYPE DEAD  TITLE LOAD CASE 1-DL
45. MEMBER LOAD
46. 1 TO 3 UNI GY -0.546
47. LOAD 142 PEDESTRIAN LOAD - LL
48. MEMBER LOAD
49. *1/2(38'+35')/3 * 0.100 KSF  PED LOAD = 1.217 K/FT
50. 1 TO 3 UNI GY -1.217
51. *LOAD 142 LOADTYPE DEAD  TITLE LOAD CASE 2-DL2
52. *JOINT LOAD
53. *1 4 FY -15.531
54. *LOAD 143 LOADTYPE DEAD  TITLE LOAD CASE 3 - DL1+DL2
55. *REPEAT LOAD
56. *141 1 142 1
57. PERFORM ANALYSIS
```

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS	**4**	**NUMBER OF MEMBERS**	**3**
NUMBER OF PLATES	**0**	**NUMBER OF SOLIDS**	**0**
NUMBER OF SURFACES	**0**	**NUMBER OF SUPPORTS**	**2**

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

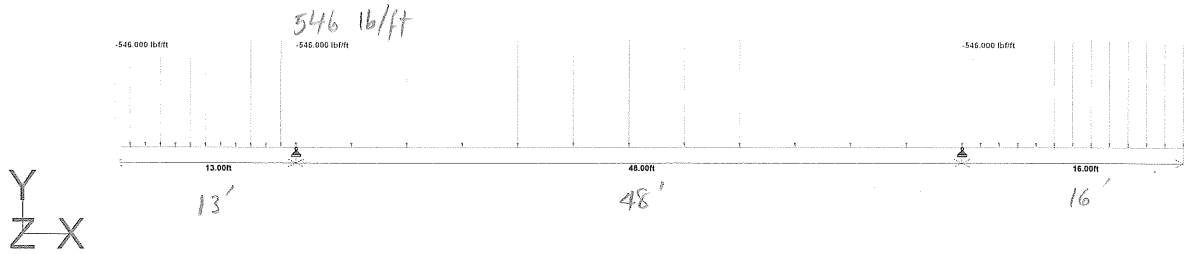
TOTAL PRIMARY LOAD CASES = 142, TOTAL DEGREES OF FREEDOM = 8

```
58. FINISH
```

FB 7

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 29-Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 30-Jan-2014 16:35	

FB 7



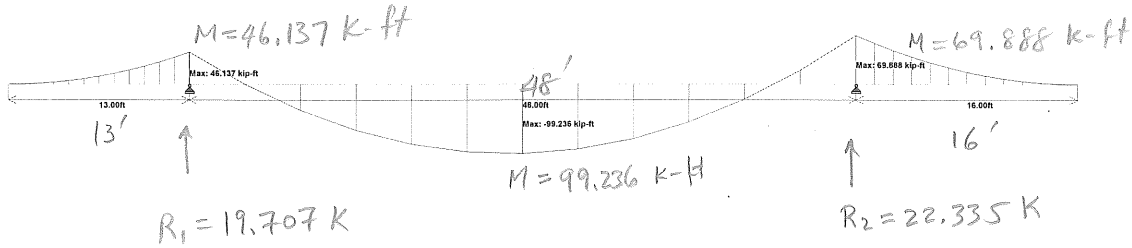
Load 141

Dead Load +SDL

DL+SDL

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 29-Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 30-Jan-2014 16:41	

FB7



Y
Z X

$f_{all} = 0.55(33) = 18.15$ ksi

$f_{DL} = \frac{M}{S} = \frac{99.236 \times 12}{1110} = 1.07$ ksi

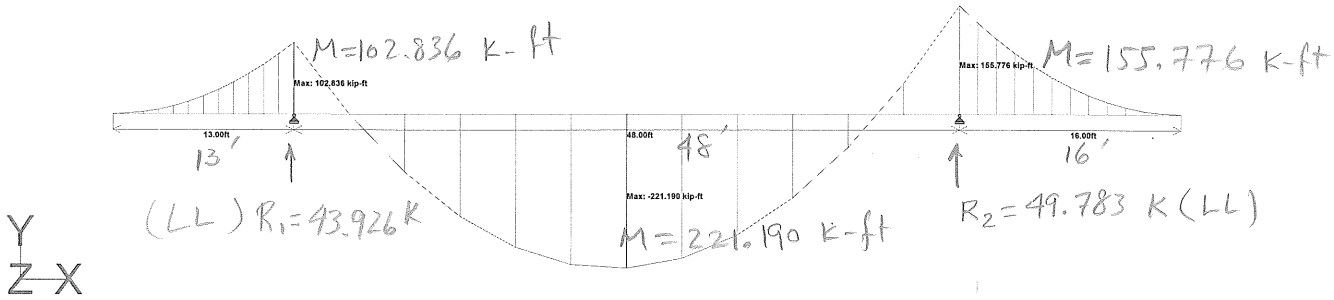
$f_{LL} = \frac{221.19 \times 12}{1110} = 2.39$ ksi

$R.F. = \frac{f_{all} - f_{DL}}{f_{LL}} = \frac{18.15 - 1.07}{2.39} = 7.15$

Load 141 : Bending Z
Moment - kip-ft
Dead Load +SDL

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 29-Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 30-Jan-2014 16:41	

FB 7



W_{LL} = 1.217 K/ft (LL)

Load 142 : Bending Z
Moment - kip-ft

Pedestrian L L

PEDESTRIAN LL

```
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*           STAAD.Pro V8i SELECTseries4          *
*           Version  20.07.09.21                 *
*           Proprietary Program of               *
*           Bentley Systems, Inc.                *
*           Date=    JAN 31, 2014                *
*           Time=    10: 6:40                    *
*                                                *
*      USER ID: HAKS Engineering                 *
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3" timber Deck w/100 psf LL

```
     1. STAAD PLANE
INPUT FILE: One Span end Cantilever - FB8-one truck-ped.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 31-JAN-14
     4. JOB NAME PASERELLER PEDESTRIAN BRIDGE
     5. JOB CLIENT DDC
     6. JOB NO 659
     7. JOB PART LOAD RATING
     8. *FB8 BENT 8
     9. JOB REF PLANK DECK
    10. ENGINEER NAME AR
    11. END JOB INFORMATION
    12. INPUT WIDTH 79
    13. UNIT FEET KIP
    14. JOINT COORDINATES
    15. 1 0 0 0; 2 7.5 0 0; 3 53 0 0; 4 60 0 0
    16. MEMBER INCIDENCES
    17. 1 1 2; 2 2 3; 3 3 4
    18. DEFINE MATERIAL START
    19. ISOTROPIC STEEL
    20. E 4.176E+006
    21. POISSON 0.3
    22. DENSITY 0.489024
    23. ALPHA 6E-006
    24. DAMP 0.03
    25. TYPE STEEL
    26. *FY=36 KSI
    27. STRENGTH FY 4752 FU 8352 RY 1.5 RT 1.2
    28. END DEFINE MATERIAL
    29. MEMBER PROPERTY AMERICAN
    30. 1 TO 3 TABLE ST W33X152
    31. SUPPORTS
    32. 2 3 PINNED
    33. CONSTANTS
    34. MATERIAL STEEL ALL
    35. DEFINE MOVING LOAD
    36. TYPE 1 LOAD 8 8
    37. DIST 6
    38. *TYPE 1 LOAD 8 8 8 8
```

STAAD PLANE -- PAGE NO. 2

```
39. *DIST 6 4 6
40. LOAD GENERATION 109
41. TYPE 1 0 0 0 XINC 0.5
42. LOAD 131 DEAD LOAD - DL1
43. MEMBER LOAD
44. 1 TO 3 UNI GY -0.400
45. LOAD 132 PEDESTRIAN LOAD - LL
46. MEMBER LOAD
47. *1/2(36'+35')/3 *0.100 KSF PED LOAD = 1.183 K/FT
48. 1 TO 3 UNI GY -1.183
49. PERFORM ANALYSIS
```

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS	**4**	**NUMBER OF MEMBERS**	**3**
NUMBER OF PLATES	**0**	**NUMBER OF SOLIDS**	**0**
NUMBER OF SURFACES	**0**	**NUMBER OF SUPPORTS**	**2**

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 111, TOTAL DEGREES OF FREEDOM = 8

```
50. FINISH
```

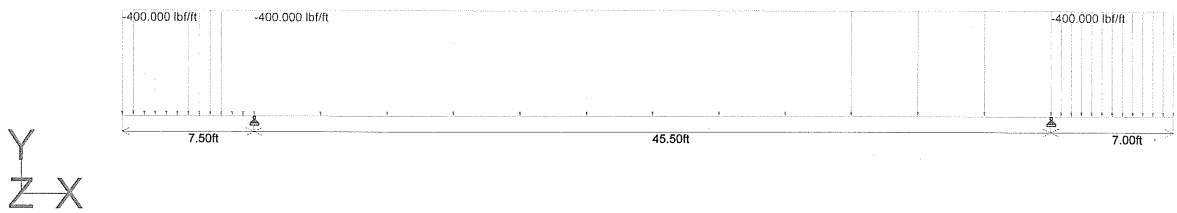
********** END OF THE STAAD.Pro RUN **********

**** DATE= JAN 31,2014 TIME= 10: 6:46 ****

FB-8

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 31-Jan-14	Chd
Client DDC	File One Span end Cantilever	Date/Time 31-Jan-2014 10:06	

FB8



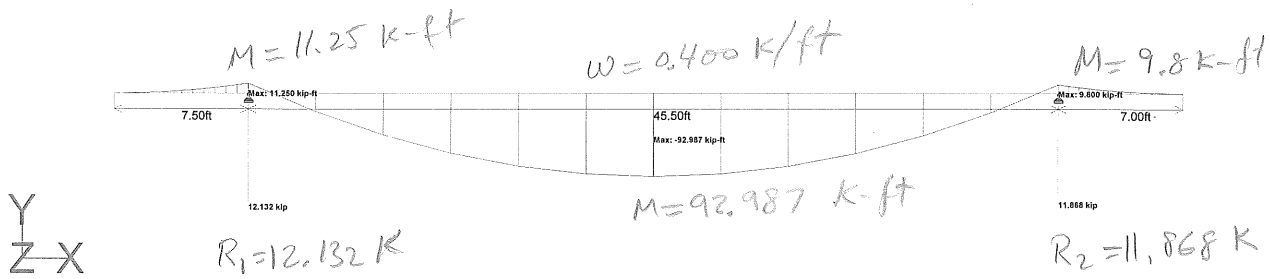
Load 131

FB8

FB8

Dead Load +SDL

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 31-Jan-14	Chd
Client DDC	File One Span end Cantilever	Date/Time 31-Jan-2014 10:06	



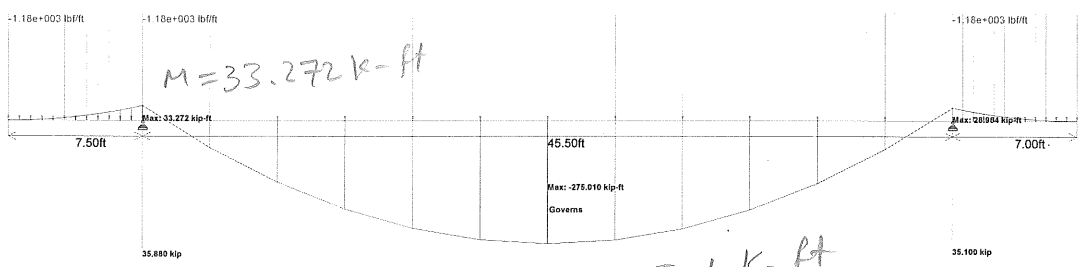
FB 8

Load 131 : Bending Z
Moment FB 8 ft

DL+SDL

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 31-Jan-14	Chd
Client DDC	File One Span end Cantilever	Date/Time 31-Jan-2014 10:06	

w=1.18 K/ft



K-ft

M= 28.984

Y
Z X

R_1=35.880

M=275.01 K-ft

R_2=35.100 K

FB 8 - LL

Load 132 : Bending Z
Moment FB-ft

Pedestrian LL

```
**************************************************
*                                                *
*           STAAD.Pro V8i SELECTseries4          *
*           Version  20.07.09.21                 *
*           Proprietary Program of               *
*           Bentley Systems, Inc.                *
*           Date=    JAN 31, 2014                *
*           Time=    15:42:12                    *
*                                                *
*      USER ID: HAKS Engineering                 *
**************************************************
```

3" Timber deck w/100 PSF LL

```
     1. STAAD PLANE
INPUT FILE: One Span end Cantilever - FB9.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 31-JAN-14
     4. JOB NAME PASERELLER PEDESTRIAN BRIDGE
     5. JOB CLIENT DDC
     6. JOB NO 659
     7. JOB PART LOAD RATING
     8. *FB8 BENT 8
     9. JOB REF PLANK DECK
    10. ENGINEER NAME AR
    11. END JOB INFORMATION
    12. INPUT WIDTH 79
    13. UNIT FEET KIP
    14. JOINT COORDINATES
    15. 1 0 0 0; 2 7 0 0; 3 41.5 0 0; 4 49 0 0
    16. MEMBER INCIDENCES
    17. 1 1 2; 2 2 3; 3 3 4
    18. DEFINE MATERIAL START
    19. ISOTROPIC STEEL
    20. E 4.176E+006
    21. POISSON 0.3
    22. DENSITY 0.489024
    23. ALPHA 6E-006
    24. DAMP 0.03
    25. TYPE STEEL
    26. *FY=36 KSI
    27. STRENGTH FY 4752 FU 8352 RY 1.5 RT 1.2
    28. END DEFINE MATERIAL
    29. MEMBER PROPERTY AMERICAN
    30. 1 TO 3 TABLE ST W27X102
    31. SUPPORTS
    32. 2 3 PINNED
    33. CONSTANTS
    34. MATERIAL STEEL ALL
    35. DEFINE MOVING LOAD
    36. *TYPE 1 LOAD 8 8 8 8
    37. TYPE 1 LOAD 8 8
    38. DIST 6
```

STAAD PLANE -- PAGE NO. 2

```
39. *DIST 6 4 6
40. LOAD GENERATION 87
41. TYPE 1 0 0 0 XINC 0.5
42. LOAD 131 DEAD LOAD - DL1
43. MEMBER LOAD
44. 1 TO 3 UNI GY -0.406
45. *1 2 3 UNI GY -1.058
46. LOAD 132 PEDESTRIAN LOAD - LL
47. *LOAD 132  LOADTYPE DEAD  TITLE LOAD CASE 1
48. MEMBER LOAD
49. *1/2(38'/2+36'/3)* 0.100 KSF PEDESTRIAN LOAD = 1.55 K/FT
50. 1 TO 3 UNI GY -1.55
51. *JOINT LOAD
52. *1  4 FY -15.531
53. *LOAD 133 - DL1+DL2
54. *REPEAT LOAD
55. *131 1 132 1
56. PERFORM ANALYSIS
```

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS	4	NUMBER OF MEMBERS	3
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	2

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 89, TOTAL DEGREES OF FREEDOM = 8

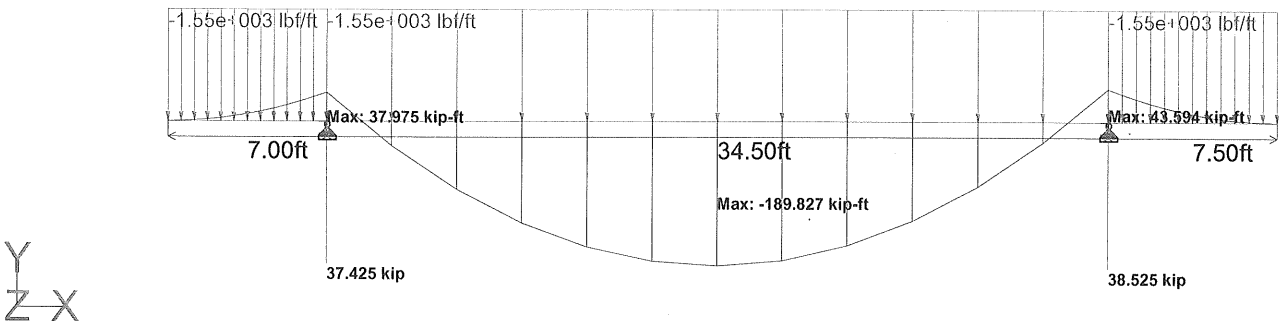
```
57. FINISH
```

********** END OF THE STAAD.Pro RUN **********

**** DATE= JAN 31,2014 TIME= 15:42:20 ****

FB-9

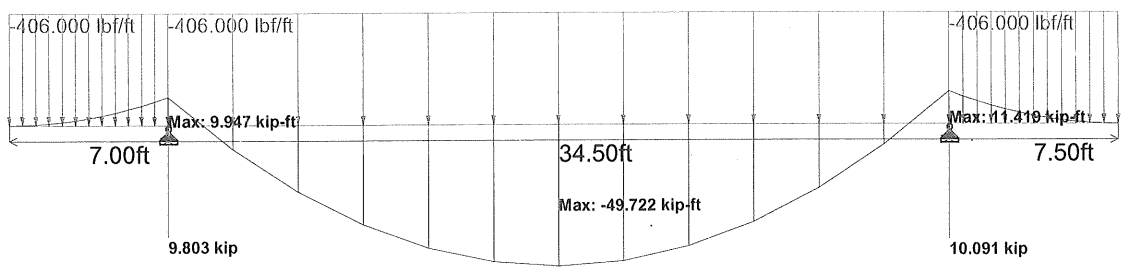
Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 31-Jan-14	Chd
Client DDC	File One Span end Cantilever	Date/Time 31-Jan-2014 15:42	



FB9
Pedestrian LL

Load 132 : **Bending Z**
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Plank Deck		
	By AR	Date 31-Jan-14	Chd
Client DDC	File One Span end Cantilever	Date/Time 31-Jan-2014 15:42	



FB9
DL+SDL

Load 131 : **Bending Z**
Moment - kip-ft

```
**************************************************
*                                                *
*           STAAD.Pro V8i SELECTseries4          *
*           Version  20.07.09.21                 *
*           Proprietary Program of               *
*           Bentley Systems, Inc.                *
*           Date=    JAN 29, 2014                *
*           Time=    13:29:44                    *
*                                                *
*      USER ID: HAKS Engineering                 *
**************************************************
```

3" Timber deck w/100 psf LL
FB11 To FB17 FB11B To FB17B
FB11A TO FB17A

```
     1. STAAD PLANE
INPUT FILE: one  Span -ends cantilever - FB11- Plank DL-Pedes LL.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 29- JAN-14
     4. JOB NAME PASERELLER PEDESTRIAN BRIDGE
     5. JOB CLIENT DDC
     6. JOB NO 659
     7. JOB PART LOAD RATING
     8. JOB REF TIMBER PLANK DECK
     9. ENGINEER NAME AR
    10. END JOB INFORMATION
    11. INPUT WIDTH 79
    12. UNIT FEET KIP
    13. JOINT COORDINATES
    14. 1 0 0 0; 2 7.25 0 0; 3 31.25 0 0; 4 38.5 0 0
    15. MEMBER INCIDENCES
    16. 1 1 2; 2 2 3; 3 3 4
    17. DEFINE MATERIAL START
    18. ISOTROPIC STEEL
    19. E 4.176E+006
    20. POISSON 0.3
    21. DENSITY 0.489024
    22. ALPHA 6E-006
    23. DAMP 0.03
    24. *TYPE STEEL
    25. *FY=33 KSI
    26. *STRENGTH FY 4752 FU 8352 RY 1.5 RT 1.2
    27. END DEFINE MATERIAL
    28. MEMBER PROPERTY AMERICAN
    29. 1 TO 3 TABLE ST W21X62
    30. SUPPORTS
    31. 2 3 PINNED
    32. CONSTANTS
    33. MATERIAL STEEL ALL
    34. DEFINE MOVING LOAD
    35. *REAR WHEEL LOAD IS 8 KIP
    36. TYPE 1 LOAD 8 8
    37. DIST 6
    38. *TYPE 1 LOAD 8 8 8 8
```

```
STAAD PLANE                                          -- PAGE NO.    2

 39. *DIST 6 4 6
 40. *DISTANCE BETWEEN REAR WHEEL IS 6 FT
 41. *SPACING BETWEEN TWO TRUCKS IS 4 FT
 42. LOAD GENERATION 67
 43. TYPE 1 0 0 0 XINC 0.5
 44. LOAD 101 DEAD LOAD CASE DL
**WARNING-A MOVING LOAD THAT WOULD HAVE BEEN APPLIED BEYOND THE X AND Z RANGES
         OF THE STRUCTURE HAS BEEN IGNORED. CASE=   67 WHEEL   2 OF   2
 45. MEMBER LOAD
 46. 1 TO 3 UNI GY -0.449
 47. *SEE DL CALC. W=W1+W2+W3=0.449 K/FT
 48. LOAD 102 PEDETRIAN LOAD - LL
 49. MEMBER LOAD
 50. *50.5/3 * 0.100 KSF PED = 1.683  K/FT
 51. 1 TO 3 UNI GY -1.683
 52. *JOINT LOAD
 53. *1 FY -57.6
 54. *4 FY -57.6
 55. *LOAD 3 DL1+DL2
 56. *REPEAT LOAD
 57. *1 1.0 2 1.0
 58. PERFORM ANALYSIS


        P R O B L E M   S T A T I S T I C S
        -----------------------------------

 NUMBER OF JOINTS          4  NUMBER OF MEMBERS       3
 NUMBER OF PLATES          0  NUMBER OF SOLIDS        0
 NUMBER OF SURFACES        0  NUMBER OF SUPPORTS      2


       SOLVER USED IS THE IN-CORE ADVANCED SOLVER

 TOTAL PRIMARY LOAD CASES =     69, TOTAL DEGREES OF FREEDOM =       8


 59. FINISH
```

FB 11 To FB 17

```
*********** END OF THE STAAD.Pro RUN ***********

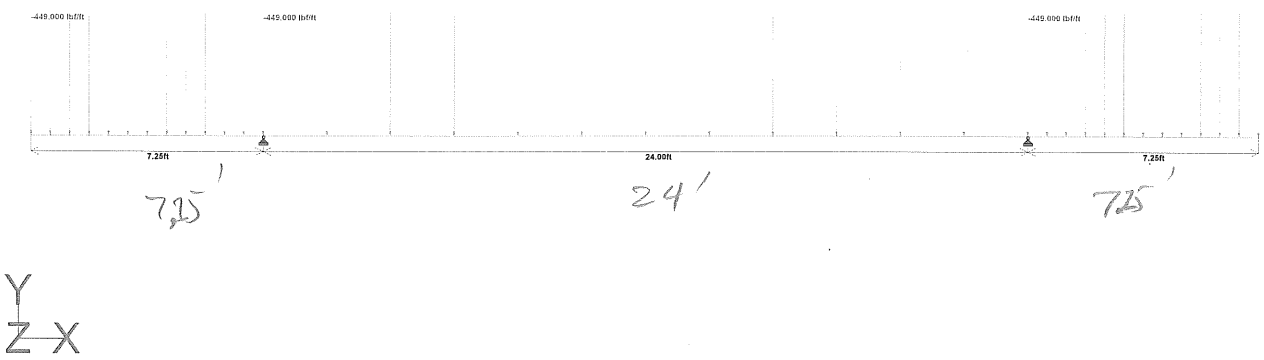
  **** DATE= JAN 29,2014   TIME= 13:29:49 ****

************************************************************
*        For questions on STAAD.Pro, please contact        *
*        Bentley Systems or Partner offices                *
*                                                          *
*            Telephone             Web / Email             *
*  USA       +1  (714) 974-2500                            *
*  UK        +44 (0) 808 101 9246                          *
*  SINGAPORE +65 6225-6158                                 *
*  FRANCE    +33 (0) 1 55238400                            *
*  GERMANY   +49 0931 40468                                *
*  INDIA     +91 (033) 4006-2021                           *
*  JAPAN     +81 (03)5952-6500    http://www.ctc-g.co.jp   *
*  CHINA     +86 21 6288 4040                              *
*  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                          *
*  Worldwide  http://selectservices.bentley.com/en-US/     *
*                                                          *
************************************************************
```

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Timber Plank Deck		
	By AR	Date 29- Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 29-Jan-2014 13:20	

FBs 10 TO FB 17

-449 lbs/ft DL +SDL

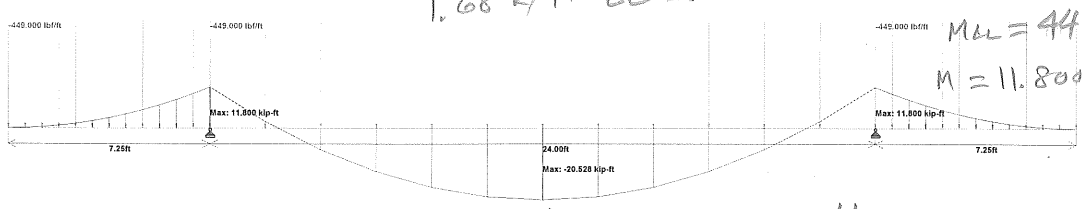


Load 101

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Timber Plank Deck		
	By AR	Date 29- Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 29-Jan-2014 13:29	

FBS 11 TO 17

1.68 k/ft LL Pedes. Load



$M_{LL} = 44$ k-ft

$M = 11.800$ k-ft

$M_{max\ DL+SDL} = -20.528$ k-ft

$M_{LL} \cong 77$ k-ft

Y
Z X

$R_{DL+SDL} = 8.643^k$

$R_{LL} = 32.4$ K
(Pedestrian)

$R = 8.643^K$

$R_{LL} = 32.4$ K

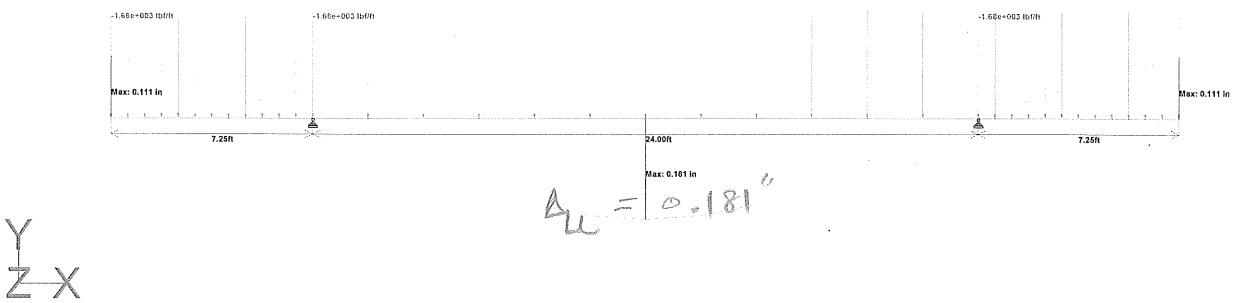
Moment.

Load 101 : Bending Z
Moment - kip-ft

Dead Load + SDL

DL + SDL

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Timber Plank Deck		
	By AR	Date 29- Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 29-Jan-2014 13:29	



$\Delta_u = 0.181''$

Load 102 : Displacement
Displacement - in

Pedestrian Load

Pedestrian Load displacement.

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.21                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    FEB  6, 2014                  *
*           Time=    11:45: 4                      *
*                                                  *
*      USER ID: HAKS Engineering                   *
****************************************************
```

3" Timber & 100 PSF LL

```
     1. STAAD PLANE
INPUT FILE: one  Span -ends cantilever - FB18- Plank-Conc- DL-Pedes LL.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 29- JAN-14
     4. JOB NAME PASERELLER PEDESTRIAN BRIDGE
     5. JOB CLIENT DDC
     6. JOB NO 659
     7. JOB PART LOAD RATING
     8. JOB REF TIMBER PLANK DECK
     9. ENGINEER NAME AR
    10. END JOB INFORMATION
    11. INPUT WIDTH 79
    12. UNIT FEET KIP
    13. JOINT COORDINATES
    14. 1 0 0 0; 2 7.25 0 0; 3 31.25 0 0; 4 38.5 0 0
    15. MEMBER INCIDENCES
    16. 1 1 2; 2 2 3; 3 3 4
    17. DEFINE MATERIAL START
    18. ISOTROPIC STEEL
    19. E 4.176E+006
    20. POISSON 0.3
    21. DENSITY 0.489024
    22. ALPHA 6E-006
    23. DAMP 0.03
    24. *TYPE STEEL
    25. *FY=33 KSI
    26. *STRENGTH FY 4752 FU 8352 RY 1.5 RT 1.2
    27. END DEFINE MATERIAL
    28. MEMBER PROPERTY AMERICAN
    29. 1 TO 3 TABLE ST W30X108
    30. SUPPORTS
    31. 2 3 PINNED
    32. CONSTANTS
    33. MATERIAL STEEL ALL
    34. DEFINE MOVING LOAD
    35. *REAR WHEEL LOAD IS 8 KIP
    36. TYPE 1 LOAD 8 8
    37. DIST 6
    38. *TYPE 1 LOAD 8 8 8 8
```

```
39. *DIST 6 4 6
40. *DISTANCE BETWEEN REAR WHEEL IS 6 FT
41. *SPACING BETWEEN TWO TRUCKS IS 4 FT
42. LOAD GENERATION 66
43. TYPE 1 0 0 0 XINC 0.5
44. LOAD 101 DEAD LOAD CASE DL
45. MEMBER LOAD
46. 1 TO 3 UNI GY -2.499
47. *SEE DL CALC. W=W1+W2+W3=2.499 K/FT
48. LOAD 102 PEDETRIAN LOAD - LL
49. MEMBER LOAD
50. *(50.5/3/2+47/2) * 0.100 KSF PED = 3.19  K/FT
51. 1 TO 3 UNI GY -3.19
52. *1 TO 3 UNI GY -1.683
53. *JOINT LOAD
54. *1 FY -57.6
55. *4 FY -57.6
56. *LOAD 3 DL1+DL2
57. *REPEAT LOAD
58. *1 1.0 2 1.0
59. PERFORM ANALYSIS
```

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS	**4**	**NUMBER OF MEMBERS**	**3**
NUMBER OF PLATES	**0**	**NUMBER OF SOLIDS**	**0**
NUMBER OF SURFACES	**0**	**NUMBER OF SUPPORTS**	**2**

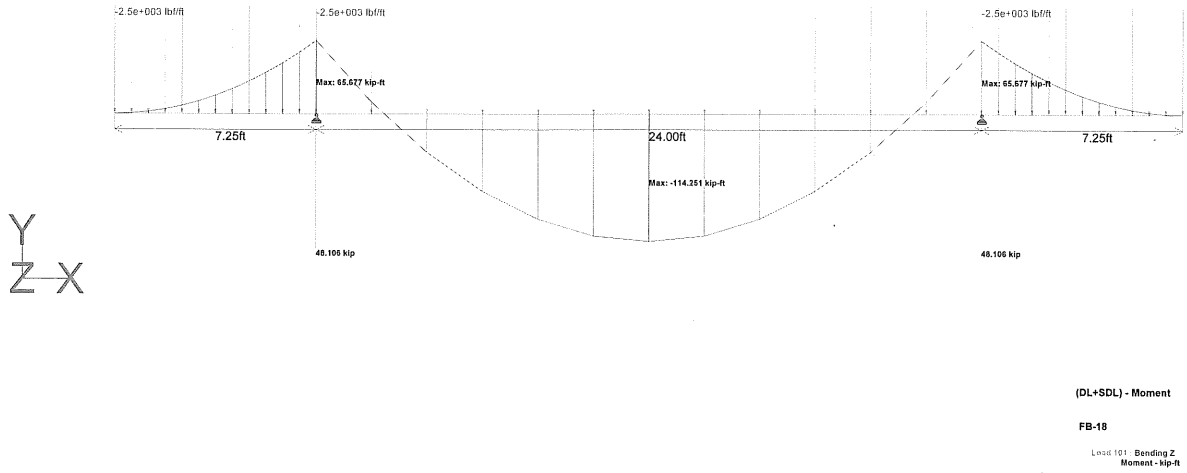
SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 68, TOTAL DEGREES OF FREEDOM = 8

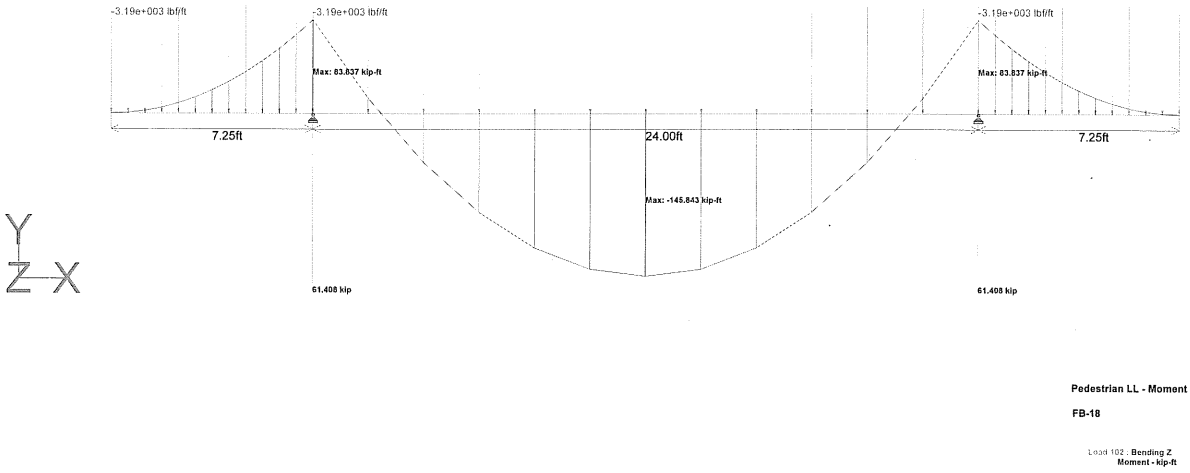
```
60. FINISH
```

FB18

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Timber Plank Deck		
	By AR	Date 29- Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 06-Feb-2014 11:44	



Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref Timber Plank Deck		
	By AR	Date 29- Jan-14	Chd
Client DDC	File one Span -ends cantileve	Date/Time 06-Feb-2014 11:44	



Pedestrian LL - Moment

FB-18

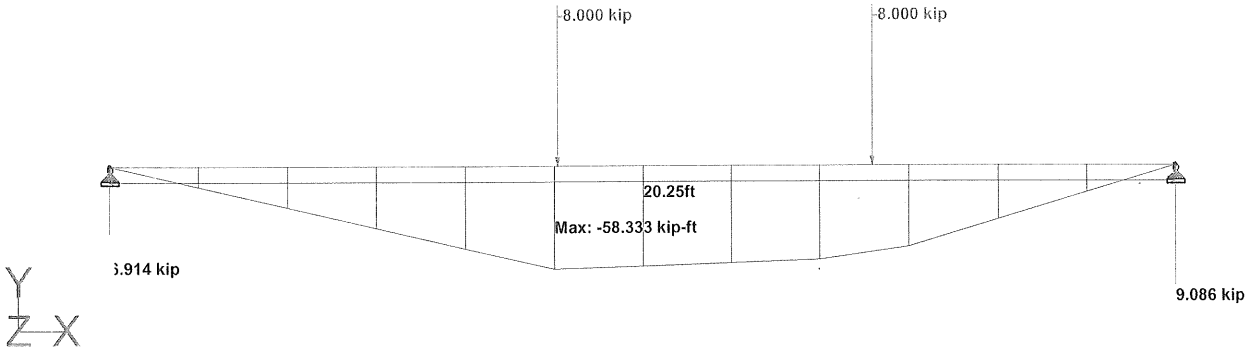
Load 102 , **Bending Z**
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 06-Sep-13	Chd
Client DDC	File Simple Span - FB19-1.st	Date/Time 08-Nov-2013 13:55	

FB 19-1

M_{LL}

one truck



Load 18 : Bending Z
Moment - kip-ft

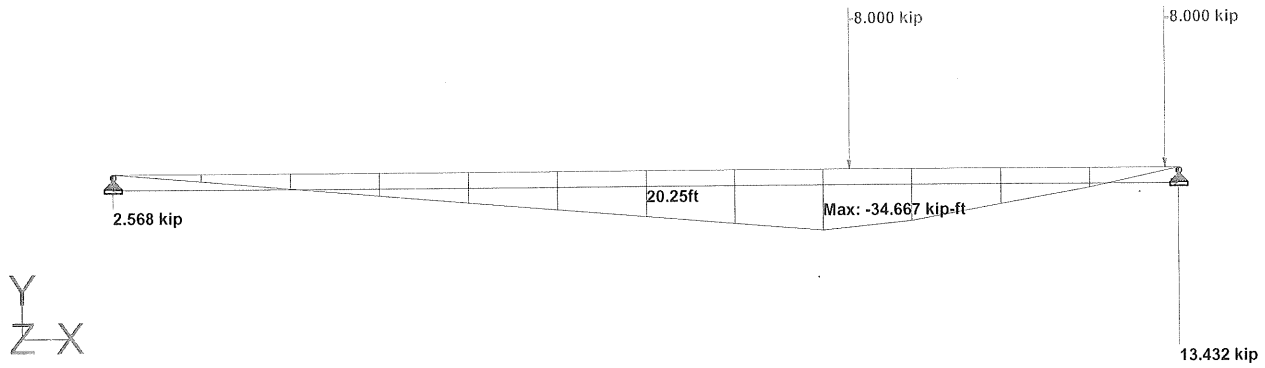
W 33X141

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 06-Sep-13	Chd
Client DDC	File Simple Span - FB19-1.st	Date/Time 08-Nov-2013 13:55	

FB19-1

R_{LL}

one truck.



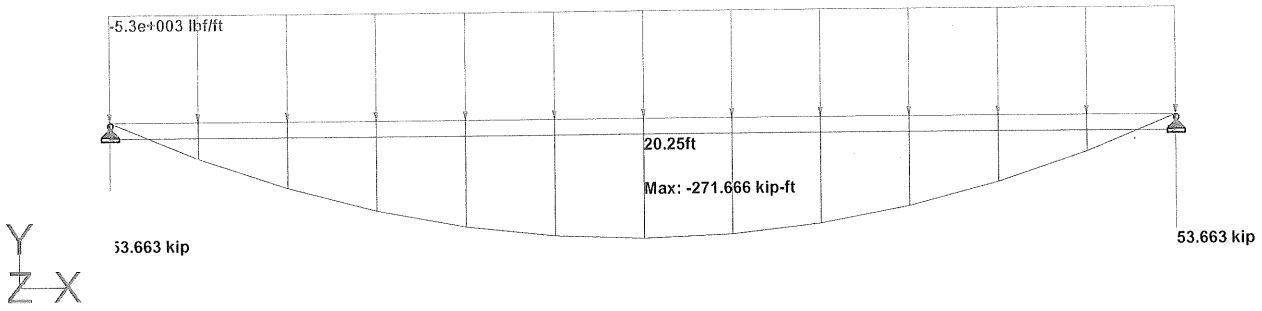
Load 29 : Bending Z
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 06-Sep-13	Chd
Client DDC	File Simple Span - FB19-1.st	Date/Time 08-Nov-2013 13:55	

FB 19 -1

Pedestrian Load

M_{ped}.



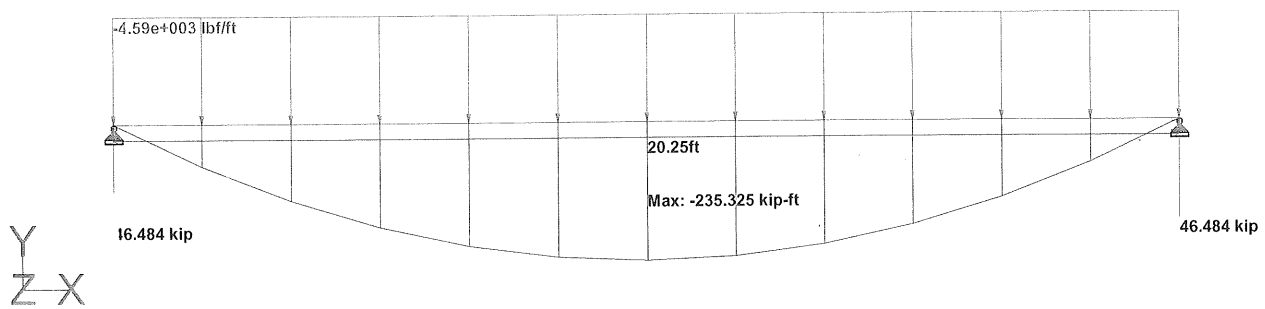
W33x141

Load 62 : Bending Z
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 06-Sep-13	Chd
Client DDC	File Simple Span - FB19-1.st	Date/Time 08-Nov-2013 13:55	

FB 19-1

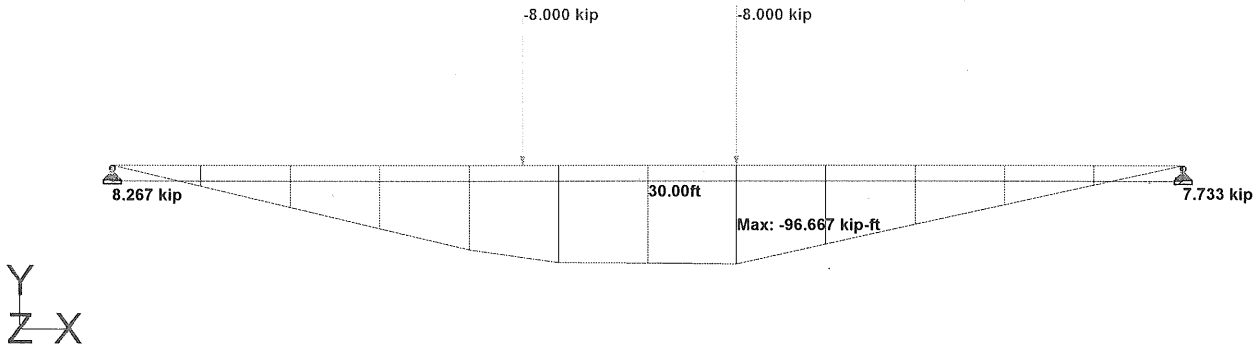
M_{DL+SDL}



Load 61 : **Bending Z**
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span - FB19A2-(	Date/Time 08-Nov-2013 11:39	

FB19A-2
one truck
M_{max}

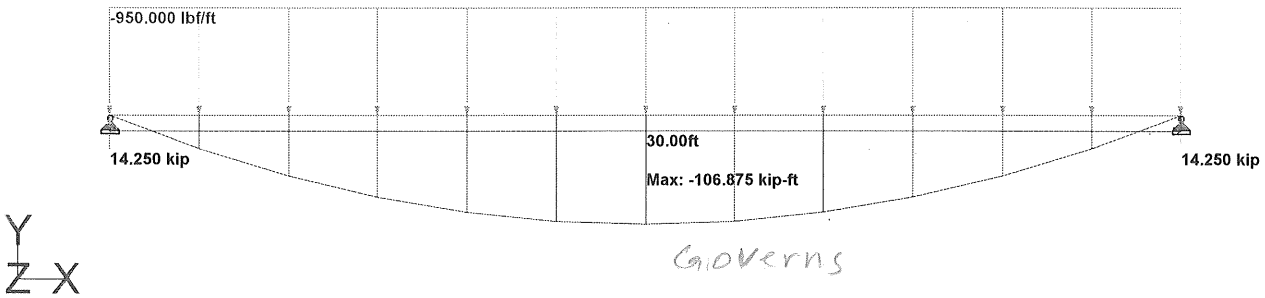


W 33x141

Load 24 : **Bending Z**
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span - FB19A2-(	Date/Time 08-Nov-2013 11:39	

FB19A-2
Pedestrian Load.



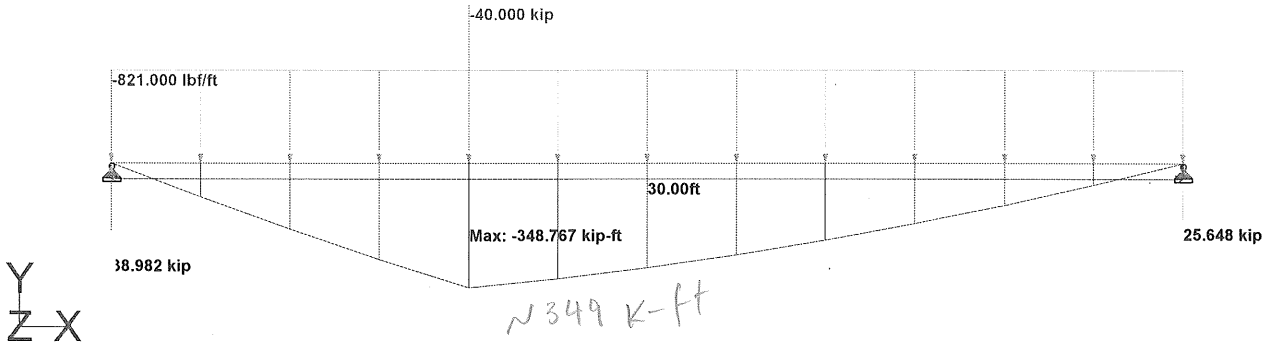
Governs

Load 93 : **Bending Z**
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span - FB19A2-(	Date/Time 08-Nov-2013 11:39	

FB 19A-2

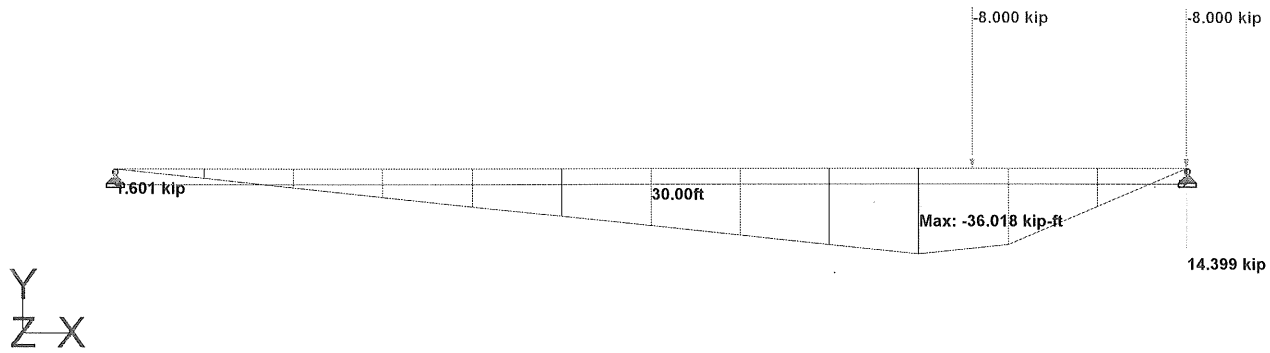
DL + CANOPY LOAD



Load 94 : **Bending Z**
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span - FB19A2-(	Date/Time 08-Nov-2013 11:39	

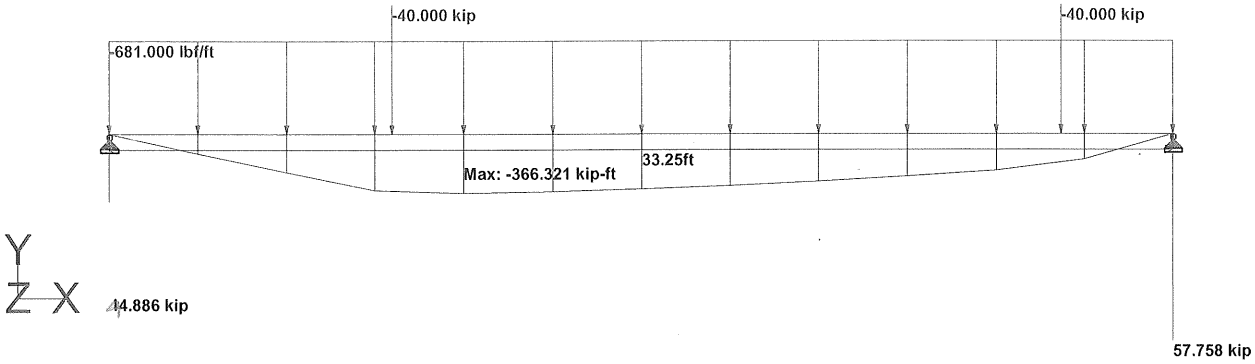
FB 19A-2
one truck
R_{max}



Load 49 : **Bending Z**
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span 22 - str24w	Date/Time 12-Nov-2013 08:57	

SPAN 22
stringer S14
CANOPY LOAD
$M_{DL+canopy}$

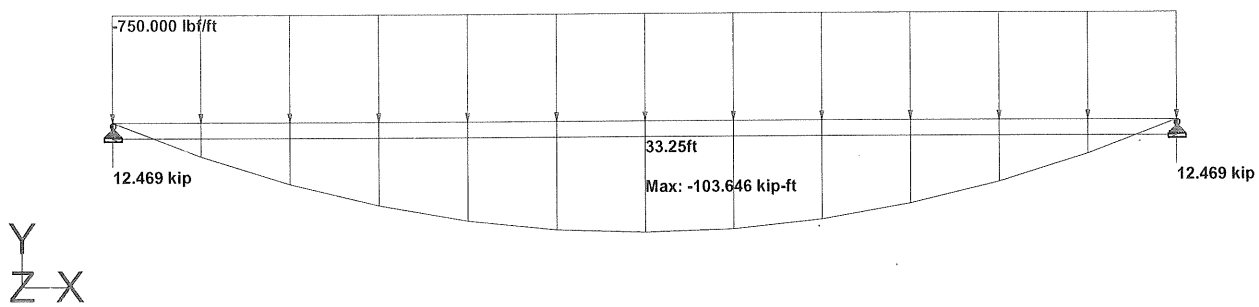


W24X145

Load 94 : Bending Z
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span 22 - str24w	Date/Time 12-Nov-2013 08:57	

SPAN 22
str. S-14
Ped. Load.
Mped.

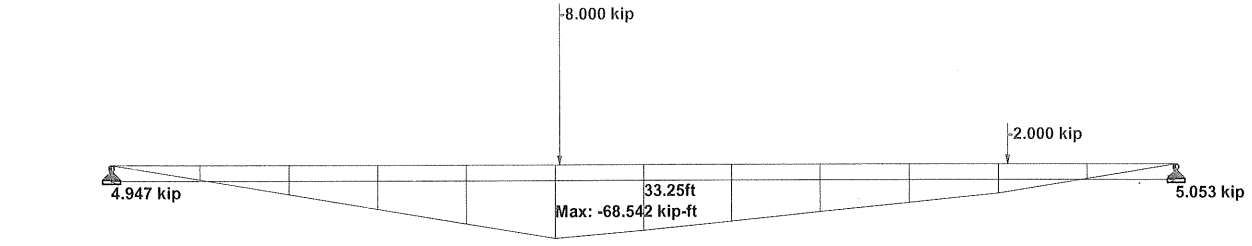


W24x145

Load 93 : Bending Z
Moment - kip-ft

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span 22 - str24w	Date/Time 12-Nov-2013 08:57	

span 22
stringer S14
truck Load.
M_{LL}



Y
Z X

$D.F. = \frac{S}{5.5} = \frac{7.5}{5.5} = 1.364$

$1.364 \times 68.542 = 93.46$ k-ft

Load 29 : Bending Z
Moment - kip-ft

Girder G2 - Span 22, 33WF152

L =	47.5	ft	γ_{conc} =	145	pcf	sdead =	10	psf
S1 =	6.875	ft						
S2 =	6.875	ft	ped =	100	psf	H10 =	20	kip
DF =	1	Slab thickness	ts =	5.5	in	Fy =	33	ksi

Loading on Stringers:

Stringer 24WF62 with spacing = 6.875 '

S = (S1+S2)/2 = 6.875 ft Ls1 = 7 ft

wdl=1.05*(γ_{conc} *S*ts + 0.062) = 0.545 k/ft

wsdl = sdead * S = 0.069 k/ft

wped =ped*S = 0.688 k/ft

Span 22 stringers:

Vdl62 = wdl * Ls1/2 = 1.907 k
Vsdl162= wsdl*Ls1/2 = 0.241 k
Vped62= wped*Ls1/2 = 2.406 k

Stringer 24WF62 with spacing = 6.875 '

S = (S1+S2)/2 = 6.875 ft Ls2 = 14 ft

wdl =1.05*(γ_{conc} *S*ts + 0.062)= 0.545 k/ft

wsdl = sdead * S = 0.069 k/ft

wped =ped*S = 0.688 k/ft

Span 22 stringers:

Vdl62 = wdl * Ls2/2 = 3.814 k
Vsdl62= wsdl*Ls2/2 = 0.481 k
Vped62 = wped*Ls2/2 = 4.813 k

Stringer 24WF62with spacing = 6.875'

S = (S1+S2)/2 = 6.875 ft Ls3 = 20 ft

wdl =1.05*(γ_{conc} *S*ts + 0.062)= 0.545 k/ft

wsdl = sdead * S = 0.069 k/ft

wped =ped*S = 0.688 k/ft

Span 20 stringers:

Vdl62 = wdl * Ls3/2 = 5.448 k
Vsdl62 = wsdl*Ls3/2 = 0.688 k
Vped62= wped*Ls3/2 = 6.875 k

Stringer 24WF76with spacing = 6.875'

S = (S1+S2)/2 = 6.875 ft Ls4= 27 ft

wdl =1.05*(γ_{conc} *S*ts + 0.076)= 0.560 k/ft

wsdl = sdead * S = 0.069 k/ft

wped =ped*S = 0.688 k/ft

Span 20 stringers:

Vdl76 = wdl * Ls4/2 = 7.554 k
Vsdl76 = wsdl*Ls4/2 = 0.928 k
Vped76= wped*Ls4/2 = 9.281 k

Stringer Load on Girder:

At 9.5':	Vdl9.5=Vdl62 =	1.907 k
	Vsdl9.5 =Vsdl62 =	0.241 k
	Vped9.5=Vped62=	2.406 k
At 19':	Vdl19 =Vdl62=	3.814 k
	Vsdl19=Vsdl62 =	0.481 k
	Vped19=Vped62=	4.813 k
At 28.5':	Vdl28.5 =Vdl62 =	5.448 k
	Vsdl28.5=Vsdl62 =	0.688 k
	Vpedl28.5 =Vped62 =	6.875 k

At 38': Vdl38 =Vdl76 = 7.554 k
Vsdl38=Vsdl76 = 0.928 k
Vped38 =Vped76 = 9.281 k

Uniform dead load on Girder:

Assume bracing self weight is uniform:

wdlgr=1.05*0.152+(1.05*0.207*3*6.875/L)= 0.169 k/ft

Moment & Shear:

Mdl = 188.37 k-ft see staad output

Msdl = 17.79 k-ft see staad output

Mped = 171 k-ft see staad output Governs

Mh10 = (L/2-2.8)*H10/4 = 104.75 k-ft

Vdl = 15.12 k
Vsdl = 1.38 k
Vped = 13.47 k Governs

Vh10 = 9.41 k

Load Rating

Sx = 487 in^3 Fy = 33 ksi d = 33.49 in
fs =0.33Fy = 18.15 ksi tw = 0.625 in
Say 18 ksi
Allowable shear capacity Fv = 11 k
Reduce load raing by 25% (1-0.25=0.75), because In calculation, there is not consider DL&LL from span between G1&G2.
Invetory:

LRm = (18*Sx-(Mdl+Msdl)/(Mped) = 3.066 X0.75 = 2.300

LRv = (11*d*tw-(Vdl+Vsdl/Vped = 15.868 X0.75 = 11.901

Operating:

LRm = (24.5*Sx/12-(Mdl+Msdl)/(Mped) = 4.609 X0.75 = 3.457

LRv = (15*d*tw-(Vdl+Vsdl)/Vped = 22.084 X0.75 = 16.563

```
****************************************************
*                                                  *
*           STAAD.Pro V8i SELECTseries4            *
*           Version  20.07.09.21                   *
*           Proprietary Program of                 *
*           Bentley Systems, Inc.                  *
*           Date=    FEB 27, 2014                  *
*           Time=    12: 8:53                      *
*                                                  *
*      USER ID: HAKS Engineering                   *
****************************************************
```

Girder G2 Span 22

```
     1. STAAD PLANE
INPUT FILE: Simple  Span 22 - G2- 33wf152.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 18-OCT-13
     4. JOB NAME PASERELLER PEDESTRIAN BRIDGE
     5. JOB CLIENT DDC
     6. JOB NO 659
     7. JOB PART LOAD RATING
     8. END JOB INFORMATION
     9. INPUT WIDTH 79
    10. *G2 SPAN 22
    11. UNIT FEET KIP
    12. JOINT COORDINATES
    13. 1 0 0 0; 2 47.5 0 0
    14. MEMBER INCIDENCES
    15. 1 1 2
    16. DEFINE MATERIAL START
    17. ISOTROPIC STEEL
    18. E 4.176E+006
    19. POISSON 0.3
    20. DENSITY 0.489024
    21. ALPHA 6E-006
    22. DAMP 0.03
    23. *FY=33 KSI
    24. END DEFINE MATERIAL
    25. MEMBER PROPERTY AMERICAN
    26. 1 TABLE ST W33X152
    27. SUPPORTS
    28. 1 2 PINNED
    29. CONSTANTS
    30. MATERIAL STEEL ALL
    31. *DEFINE MOVING LOAD
    32. *REAR WHEEL LOAD IS 8 KIP
    33. *TYPE 1 LOAD 8 2
    34. *DIST 14
    35. *TYPE 1 LOAD 8 8 8 8
    36. *DIST 6 4 6
    37. *DISTANCE BETWEEN REAR WHEEL IS 6 FT
    38. *LOAD GENERATION 90
```

```
39. *TYPE 1 0 0 0 XINC 0.5
40. *LOAD 200 DEAD LOAD CASE - DL1
41. LOAD 91 DEAD LOAD CASE 1 - DL1
42. MEMBER LOAD
43. *SELF WT
44. 1 UNI GY -0.169
45. LOAD 92 DEAD LOAD CASE 2 - DL2
46. MEMBER LOAD
47. 1 CON GY -1.907 9.5
48. 1 CON GY -3.814 19
49. 1 CON GY -5.448 27.5
50. 1 CON GY -7.554 38
51. LOAD 93 SUPERIMPOSE DEAD LOAD - SDL
52. MEMBER LOAD
53. 1 CON GY -0.241 9.5
54. 1 CON GY -0.481 19
55. 1 CON GY -0.688 27.5
56. 1 CON GY -0.928 38
57. LOAD 94 PEDESTRIAN LOAD - LL
58. MEMBER LOAD
59. 1 CON GY -0.688 9.5
60. 1 CON GY -4.813 19
61. 1 CON GY -6.875 27.5
62. 1 CON GY -9.281 38
63. LOAD 95 DL1+DL2
64. REPEAT LOAD
65. 91 1.0 92 1.0
66. PERFORM ANALYSIS
```

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS	**2**	**NUMBER OF MEMBERS**	**1**
NUMBER OF PLATES	**0**	**NUMBER OF SOLIDS**	**0**
NUMBER OF SURFACES	**0**	**NUMBER OF SUPPORTS**	**2**

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 5, TOTAL DEGREES OF FREEDOM = 2

```
67. FINISH
```

Girde G2 - Span 22

```
*********** END OF THE STAAD.Pro RUN ***********

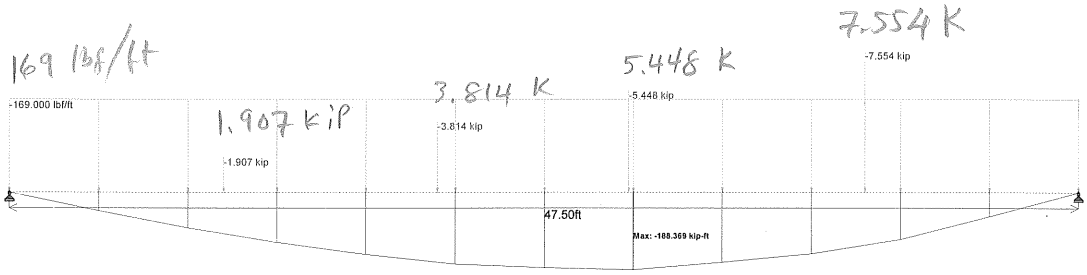
  **** DATE= FEB 27,2014   TIME= 12: 8:54 ****

**********************************************************
*        For questions on STAAD.Pro, please contact      *
*        Bentley Systems or Partner offices              *
*                                                        *
*            Telephone             Web / Email           *
*  USA       +1  (714) 974-2500                          *
*  UK        +44 (0) 808 101 9246                        *
*  SINGAPORE +65 6225-6158                               *
*  FRANCE    +33 (0) 1 55238400                          *
*  GERMANY   +49 0931 40468                              *
*  INDIA     +91 (033) 4006-2021                         *
*  JAPAN     +81 (03)5952-6500    http://www.ctc-g.co.jp *
*  CHINA     +86 21 6288 4040                            *
*  THAILAND  +66 (0)2645-1018/19 partha.p@reisoftwareth.com*
*                                                        *
*  Worldwide  http://selectservices.bentley.com/en-US/   *
*                                                        *
**********************************************************
```

Girder G2 - Span 22

Girder G-2 Span 22

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span 22 - G2- 33'	Date/Time 27-Feb-2014 11:08	



$M_{DL} = 188.369$ K-ft

Y
Z X

G1

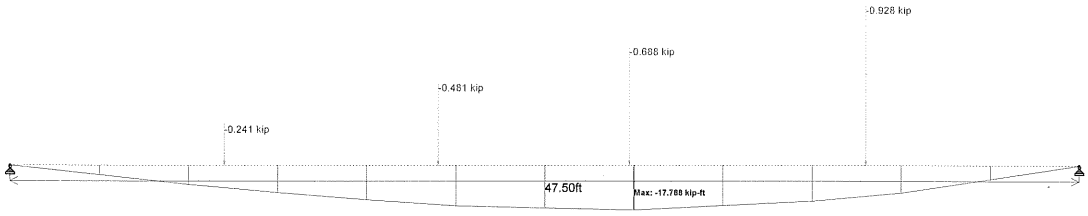
Load 95 : Bending Z
Moment - kip-ft

span 22

DL

Girder G2 - span 22

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span 22 - G2- 33'	Date/Time 27-Feb-2014 11:08	



$M_{SDL} = 17.788$ K-ft

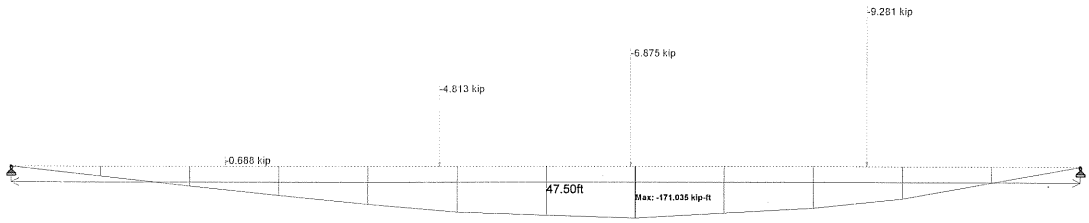
G1

Load 33 : Bending Z
Moment - kip-ft

Span 22

SDL

Software licensed to HAKS Engineering	Job No 659	Sheet No 1	Rev
	Part Load rating		
Job Title Pasereller Pedestrian Bridge	Ref		
	By	Date 18-Oct-13	Chd
Client DDC	File Simple Span 22 - G2- 33	Date/Time 27-Feb-2014 11:08	



$M_{Ped} = 171.035$ k·ft

Y
Z X

G1

Load 94 : Bending Z
Moment - kip-ft

Span 22

PEDES. LL

SPANS 1-7 AND 23-29
As Inspected Load Rating Summary

(ASD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

Summary for Load Rating (ASD method) - S

Location	Span	Stringer	Length (ft)	Spacing 1 (ft)	Spacing 2 (ft)	Ca/ped (kips)	VH10 (kips)	Vcanopy (kips)	Inventory LR		Operating LR	
									LRm	LRv	LRm	LRv
1	1	S1-S12	7.333	5.333	5.333				See span 2		See span 2	
2	2	S1-S12	21.000	5.333	5.333	6.28	5.69	0.00	1.15	7.60	1.66	10.62
3	3-6	S1-S12	14.667	5.333	5.333	3.91	5.69	0.00	0.53	4.14	0.81	5.81
4	7	S13A	12.667	5.333	5.333	3.38	5.69	0.00	-	4.17	-	5.82
	7	S14A	12.667	5.333	5.333	3.38	5.69	0.00	0.68	-	1.00	-
	7	S13B	12.667	5.333	5.333	3.38	5.69	0.00	0.67	-	0.98	-
5	23	S13	10.000	6.875	7.500	3.59	5.45	0.00	1.59	5.22	2.27	7.33
6	24	S5-S19	32.161	6.875	6.875	11.06	6.25	0.00	1.67	8.87	2.61	12.44
7	24	S20	32.161	7.500	7.500	12.06	6.82	44.97	0.86	6.53	2.40	10.62
8	25	S25	37.000	7.500	7.500	13.88	6.82	22.08	0.75	9.19	2.32	13.48
9	25	S26	37.000	7.500	7.500	13.88	6.82	0.00	1.12	7.41	1.86	10.45
10	26	S4	10.000	7.500	7.500	3.75	5.45	0.00	1.58	5.20	2.26	7.30
11	27	S4	45.958	7.500	7.500	17.23	6.82	0.00	1.10	6.89	1.85	9.75
12	27	S25	45.958	6.875	7.500	16.52	6.82	0.00	1.10	6.38	1.85	9.05
13	27	S27	45.958	7.500	7.500	17.23	6.82	32.27	1.19	7.43	2.32	11.20
14	28	S4	36.500	7.500	7.500	13.69	6.82	0.00	1.18	7.53	1.94	10.60
15	28	S25	36.500	6.875	7.500	13.12	6.82	0.00	1.25	6.96	2.04	9.83
16	29	S13	47.500	6.875	6.875	16.33	6.25	0.00	0.98	6.44	1.69	9.14
17	29	S27	47.500	7.500	7.500	17.81	6.82	0.00	0.97	6.63	1.67	9.40
18	29	S26	47.500	7.500	7.500	17.81	6.82	52.63	1.03	5.88	2.85	9.53

Summary for Load Rating (ASD method) - F

Location	Span	FB	Length (ft)	Spacing 1 (ft)	Spacing 2 (ft)	C/ped (kips)	VH10 (kips)	Vcanopy (kips)	Inventory LR		Operating LR	
									LRm	LRv	LRm	LRv
1	1	1-2	48.000	5.333	5.333	38.00	10.00	0.00	0.57	4.92	1.00	6.94
2	1	1-1,1-3	16.000	5.333	5.333	9.50	10.00	0.00	0.84	3.87	1.34	5.47
3	3	2A,B-2	48.000	5.333	5.333	31.20	10.00	0.00	0.51	5.44	0.97	7.70
4	3	3A,B-1	16.000	5.333	5.333	6.80	10.00	0.00	0.64	3.56	1.08	5.06
5	4	4A, 4B	48.000	5.333	5.333	27.20	10.00	0.00	0.46	5.82	0.90	8.22
6	6	6-1	48.000	5.333	5.333	30.90	20.00	0.00	2.88	11.37	4.25	15.87
7	6	6-2	31.000	5.333	5.333	37.33	0.00	0.00	2.26	-	3.40	-
8	7	6A-3	25.000	5.333	5.333	13.32	10.00	0.00	0.53	-	0.98	-
9	7	7	74.667	5.333	5.333	36.96	10.00	0.00	3.12	9.48	4.56	13.23
10	22	22-4	30.000	7.500	7.500	21.47	10.00	24.88	0.43	8.11	1.63	11.87
11	24	24-6	30.000	7.500	7.500	38.90	10.00	33.46	-0.49	-	0.16	-
12	24	24-5	27.500	6.875	6.875	35.67	10.00	0.00	0.73	-	1.35	-
13	25	25-7	30.000	7.500	7.500	23.56	10.00	28.96	0.01	7.12	1.10	10.54
14	26	26-7	30.000	7.500	7.500	28.67	10.00	16.14	0.50	6.10	1.39	8.91
15	27	27-7	30.000	7.500	7.500	46.38	10.00	3.86	0.67	-	1.32	-
16	27	27-6	27.500	6.875	6.875	46.50	10.00	0.00	0.74	-	1.35	-
17	28	28-8	30.000	7.500	7.500	47.25	10.00	33.13	0.46	4.79	1.22	7.17
18	28	28-6	27.500	6.875	6.875	47.25	10.00	0.00	0.88	4.47	1.56	6.46
19	29	29-8	30.000	7.500	7.500	27.00	10.00	20.00	0.44	5.83	1.20	8.59
20	29	29-6	27.500	7.500	7.500	27.00	10.00	0.00	1.00	9.38	1.71	13.16

Section with Significatn Section Loss

W/2-12-13

Summary for Load Rating (ASD method) - Girders

							As-built (ASD)																		
Location	Span	Girder	Length (ft)	Spacing 1 (ft)	Spacing 2 (ft)	DF	Section	d (in)	tw (in)	Sx (in3)	Mdl (kip.ft)	Msdl (kip.ft)	Mped (kip.ft)	MH10 (kip.ft)	Mcanopy (kip.ft)	Vdl (kips)	Vsdl (kips)	Vped (kips)	VH10 (kips)	Vcanopy (kips)	Inventory LR		Operating LR	
																					LRm	LRv	LRm	LRv
1	3	G1	44.000	5.333	5.333	1.00	24WF130	24.25	0.565	330.70	233.23	33.00	300.67	96.00	0.00	18.75	2.58	23.43	5.00	0.00	0.76	**5.52**	**1.36**	**7.86**
2	3	G2	44.000	5.333	5.333	1.00	36WF230	35.88	0.765	835.50	698.31	94.60	945.85	96.00	0.00	52.63	7.03	70.36	5.00	0.00	0.49	**3.44**	0.97	**5.00**
3	4	G2	38.000	5.333	5.333	1.00	33WF130	33.10	0.580	404.80	370.44	51.93	513.00	81.00	0.00	29.89	4.10	40.50	5.00	0.00	0.36	**4.37**	0.79	**6.27**
4	5	G1	38.000	5.333	5.333	1.00	33WF130	33.10	0.580	404.80	317.24	38.00	385.07	81.00	0.00	25.69	3.00	30.40	5.00	0.00	0.65	**6.00**	**1.22**	**8.53**
5	6	G1	44.000	5.333	5.333	1.00	24WF76	23.91	0.440	175.40	127.11	18.33	176.00	96.00	0.00	9.11	1.25	12.00	5.00	0.00	0.67	**8.78**	**1.21**	**12.29**
6	6	G2	44.000	5.333	5.333	1.00	33WF152	33.50	0.635	486.40	516.02	68.93	689.33	96.00	0.00	36.06	4.70	47.00	5.00	0.00	0.21	**4.11**	0.59	**5.92**
7	7	G2	38.000	5.333	5.333	1.00	33WF130	33.10	0.580	404.80	370.44	51.93	513.00	81.00	0.00	29.89	4.10	40.50	5.00	0.00	0.36	**4.37**	0.79	**6.27**
8	7	G4	42.120	5.333	5.333	1.00	24WF100	23.96	0.468	244.17	141.22	21.06	189.54	91.30	0.00	10.61	1.50	13.50	5.00	0.00	**1.08**	**8.24**	**1.77**	**11.56**
9	23-24	G2	59.920	7.500	6.875	1.00	33WF152	33.50	0.635	486.40	426.66	43.74	440.89	-	0.00	26.85	2.74	27.20	-	0.00	0.59	**7.51**	**1.19**	**10.64**

Summary for Load Rating (ASD method) - Columns

				As-built (ASD)								
cation	Span	Column	L (ft)	Section	A (in2)	ry (in)	Pdl (kips)	Psdl (kips)	Pped (kips)	Pcan (kips)	Inventory LR LRm	Operating LR LRm
1	1	1K	15.250	14WF74	21.76	2.48	66.00	9.20	91.78	-	**2.29**	**3.06**
2	1	1L	15.250	14WF84	24.71	3.02	96.00	13.80	137.67	-	**1.70**	**2.32**
3	2	2K	15.130	14WF95	27.94	3.71	127.30	21.00	210.57	-	**1.22**	**1.69**
4	2	2L	15.130	14WF119	34.99	3.75	164.00	27.00	265.00	-	**1.20**	**1.67**
5	3	3K	16.380	14WF78	22.94	3.00	112.50	15.50	155.20	-	**1.19**	**1.69**
6	3	3L	16.380	14WF111	32.65	3.73	127.60	17.90	180.00	-	**1.79**	**2.43**
7	4	4K	18.760	14WF87	25.56	3.70	95.40	14.10	142.00	-	**1.73**	**2.35**
8	4	4L	18.760	14WF95	27.94	3.71	119.70	17.10	175.00	-	**1.44**	**1.99**
9	5	5K,5L	-	14WF84	24.71	3.02	Not govern			-	See 4L	See 4L
10	6	6K,6L	-	14WF78	22.94	3.00	Not govern			-	See 3K	See 3K
11	7	7K,7L	-	14WF78	22.94	3.00	Not govern			-	See 3K	See 3K
12	22	22H	19.900	14WF61	17.94	2.45	53.68	5.72	57.24	40.57	**1.79**	**2.67**
13	23	23H	19.720	14WF61	17.94	2.45	51.64	5.57	55.74	28.53	**2.12**	**3.03**
14	24	24E	23.170	14WF87	25.56	3.70	85.91	9.51	95.10	0.00	**2.50**	**3.36**
15	24	24H	23.170	14WF87	25.56	3.70	94.44	10.37	103.74	54.79	**1.67**	**2.46**
16	25	25H	18.440	14WF78	22.94	3.00	59.09	6.30	63.00	47.21	**2.99**	**4.17**
17	26	26H	18.250	14WF78	22.94	3.00	73.34	7.64	76.44	26.34	**2.55**	**3.53**
18	27	27E	20.550	14WF87	24.02	3.72	105.69	11.34	113.38	0.00	**1.85**	**2.56**
19	27	27H	20.550	14WF95	27.94	3.71	115.04	12.37	123.69	6.31	**1.99**	**2.74**
20	28	28H	19.270	14WF95	27.94	3.71	117.72	12.60	126.00	30.59	**1.79**	**2.54**
21	29	29H	17.020	14WF61	17.94	2.45	69.56	7.13	64.13	42.06	**1.63**	**2.49**

Summary for Load Rating (ASD metho LFD (As Condition)

Location	Span	Stringer	Span Length (ft)	Section	Inventory LR		Operating LR	
					LRm	LRv	LRm	LRv
1	1	S1-S12		16WF36				
2	2	S1-S12		16WF36				
3	3-6	S1-12		10WF21				
4	7	S13A	12.67	10WF21	-	-	-	-
	7	S14A	12.67	10WF21	.62	-	1.04	-
	7	S13B	12.67	10WF21	.61	-	1.01	-
5	23	S13		12WF27				
6	24	S18		24WF68				
7	24	S20		24WF11(	95		3.26	
8	25	S25		24WF14!	91		3.20	
	25	S23		24WF76				
9	25	S26		24WF76				
	25	S25		12WF27				
10	26	S4		12WF27				
11	27	S4		24WF11(				
12	27	S25	45.96	24WF11(	-	-	-	-
13	27	S27		24WF16(				
14	28	S4		24WF76				
15	28	S25	36.50	24WF76	-	-	-	-
					-	-	-	-
16	29	S13	36.50	24WF11(	.23	-	2.06	-
17	29	S27		24WF11(	24		2.07	
18	29	S26		24WF160				

Red Flag

Summary for Load Rating (ASD metho LFD (As Condition)

Location	Span	FB	Span Length (ft)	Section	Inventory LR		Operating LR	
					LRm	LRv	LRm	LRv
1	1	1-2		33WF13(	0.77		1.29	
2	1	1-1,1-3		14WF34	0.97		1.63	
3	3	2A,B-2		30WF12	0.77		1.28	
4	3	3A,B-1		14WF30	0.79		1.32	
5	4	4A,4B		30WF10	0.72		1.21	
6	6	6-1		36WF30(				
7	6	6-2	31.00	24WF14!	-	-	-	-
8	7	6A-3	25.00	16WF50	.75	-	1.25	-
	7	6B	12.89	16WF36				
9	7	7		36WF30(				
10	22	22-4		33WF14	46		2.44	
11	24	24-6	30.00	33WF15	.34	-	0.56	-
12	24	24-5	27.50	33WF13(	.05	-	1.75	-
13	25	25-7		33WF14	10		1.83	

14	26	26-7		33WF141	19		1.98	
15	27	27-7	30.00	36WF170	**.97**	-	**1.62**	-
16	27	27-6	27.50	36WF150	**.07**	-	**1.78**	-
17	28	28-8		36WF194	05		1.75	
18	28	28-6		36WF160	21		2.02	
19	29	29-8		30WF132	03		1.72	
20	29	29-6		30WF116	32		2.20	

Summary for Load Rating (ASD metho

LFD (As Condition)

Location	Span	Girder	Span Length (ft)	Section	Inventory LR		Operating LR	
					Rm	LRv	LRm	LRv
1	3	G1		36WF130	01		1.69	
2	3	G2		36WF230	60		1.01	
3	4	G2		33WF130	49		0.82	
4	5	G1		33WF130	76		1.27	
5	6	G1		224WF76	75		1.26	
6	6	G2		33WF152	38		0.63	
7	7	G2		33WF130				
8	7	G4	42.11	24WF100	-	-	-	-
9	23-24	G2		33WF152	95		1.58	

Summary for Load Rating (ASD metho

Location	Span	Column	Unbraced L (ft)	Section
1	1	1K		14WF74
2	1	1L		14WF84
3	2	2K		14WF95
4	2	2L		14WF119
5	3	3K		14WF78
6	3	3L		14WF111
7	4	4K		14WF87
8	4	4L		14WF95
9	5	5K,5L		14WF84
10	6	6K,6L		14WF78
11	7	7K,7L		14WF78
12	22	22H		14WF61
13	23	23H		14WF61
	23	23E		14WF87
14	24	24E		14WF87
15	24	24H		14WF87
16	25	25H		14WF78
17	26	26H		14WF78
18	27	27E	20.55	14WF87
19	27	27H		14WF95
20	28	28H		14WF95
21	29	29H		14WF61

Red Flag

SPANS 1-7 AND 23-29
As Inspected Section Loss Calculations

(ASD Method)

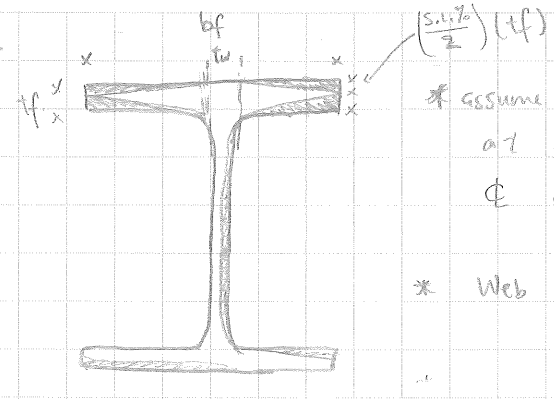
Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

WEIDLINGER

PROJECT	Passerelle Load Rating	SUBJECT	Section Loss
BY	TF Ko	DATE	10/14/13
CHECKED BY	Jason A.	DATE	10/15/13
		PAGE	1 OF 1

→



* assume the section loss of the flange is the worst at the edge & reduce linearly toward the ℄ of the I-Beam.

* Web S.L. should be similar to what observed.

→ The thickness loss of Flange of I-Beam:

Original Area of top flange: $(bf)(tf)$

Area of Flange Section loss: $(2)\left(\frac{S.L.\%}{2}\right)(tf)\left(\frac{bf}{2}\right) + (2)\left(\frac{S.L.\%}{2}\right)(tf)\left(\frac{bf - tw}{2}\right)$

$$= (S.L.\%)(tf)\left(\frac{bf}{2}\right) + (S.L.\%)(tf)\left(\frac{bf - tw}{2}\right)$$

$$= \frac{(S.L.\%)(tf)}{2}\left[2bf - tw\right]$$

* assume Area section loss % ≈ average thickness loss %.

Section loss of Area / thickness %

$$= \frac{\frac{(S.L.\%)(tf)}{2}\left[2bf - tw\right]}{(bf)(tf)}$$

→ Run this equation using different type of beam & S.L.

↳ Average value is about 50% of the observed section loss percentage.

⇒ So in as-condition calculation, we will use 50% of observed flange S.L. & 100% of web S.L.

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10/07/2013
CHECKED BY	Jason A.	DATE	10/15/13

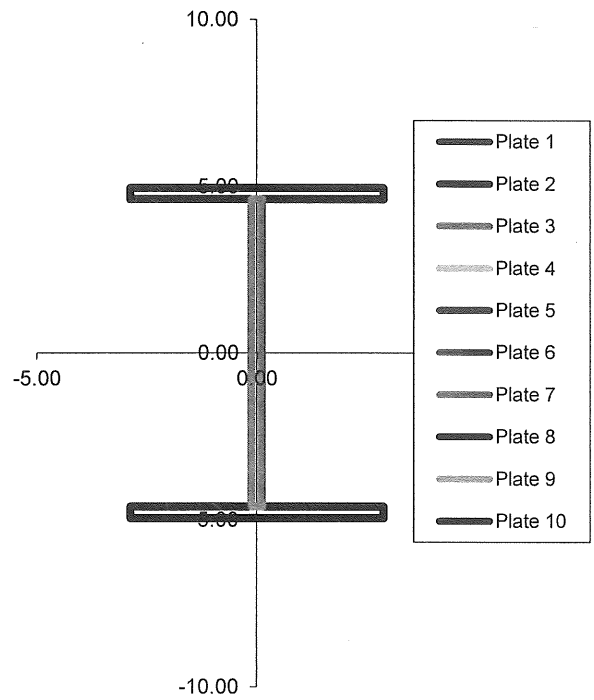
SUBJECT	Stringer 1-12 at Span 3-6				
10WF21					
		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	5.7500	0.3400		4.7800
2	5.7500	0.3400		-4.7800
3	0.2400	9.2200		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	6.12

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
105.05	21.22	21.22	4.14	23.79

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
10.78	3.75	3.75	1.33	5.75

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

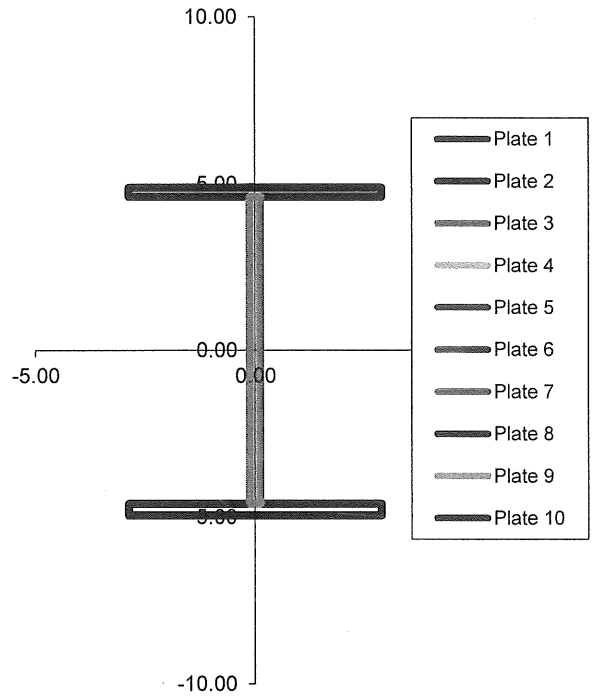
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	09-30-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Stringer 13A at Span 7				
10WF21	End section loss				
XXX		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	5.7500	0.2720 ~ 20% SL		4.7460
2	5.7500	0.3400		-4.7800
3	0.2400	9.2200		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.34	5.73

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
94.96	18.20	20.58	4.07	21.76

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
9.71	3.38	3.38	1.30	5.19

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

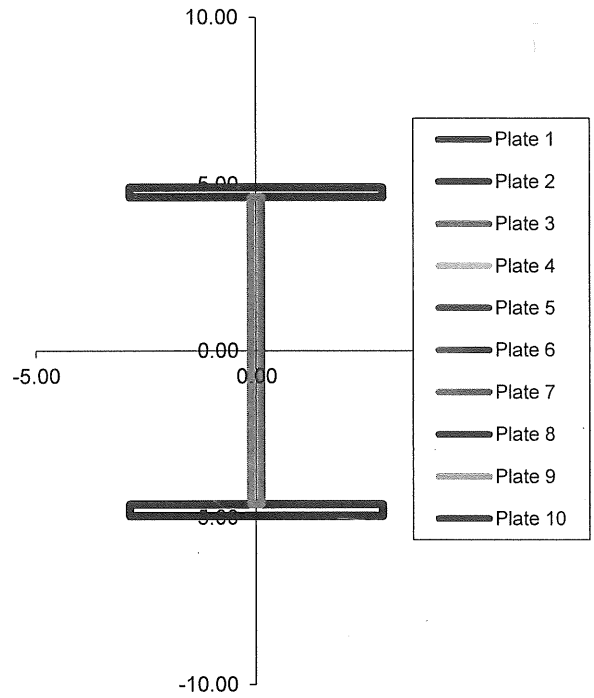
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Stringer 13B at Span 7				
10WF21	Midspan section loss				
XXX		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	5.7500	0.2890	15% SL	4.7545
2	5.7500	0.3400		-4.7800
3	0.2400	9.2200		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.25	5.83

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
97.58	18.96	20.75	4.09	22.29

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
9.98	3.47	3.47	1.31	5.33

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

S14A

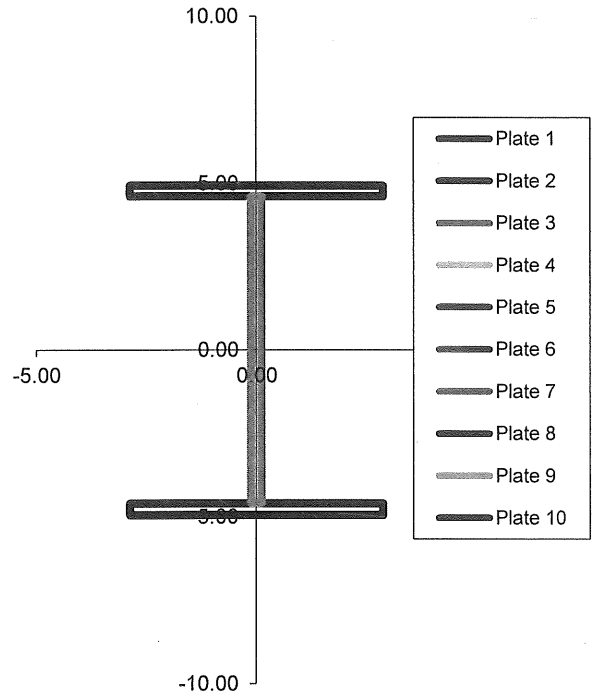
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	09-30-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Stringer 14A at Span 7				
10WF21	midspan section loss				
XXX		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	5.7500	0.3230		4.7715
2	5.7500	0.3400		-4.7800
3	0.2400	9.2200		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.08	6.03

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
102.62	20.47	21.07	4.13	23.30

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
10.51	3.66	3.66	1.32	5.61

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	10-11-2013
CHECKED BY	TFK	DATE	10/11/13

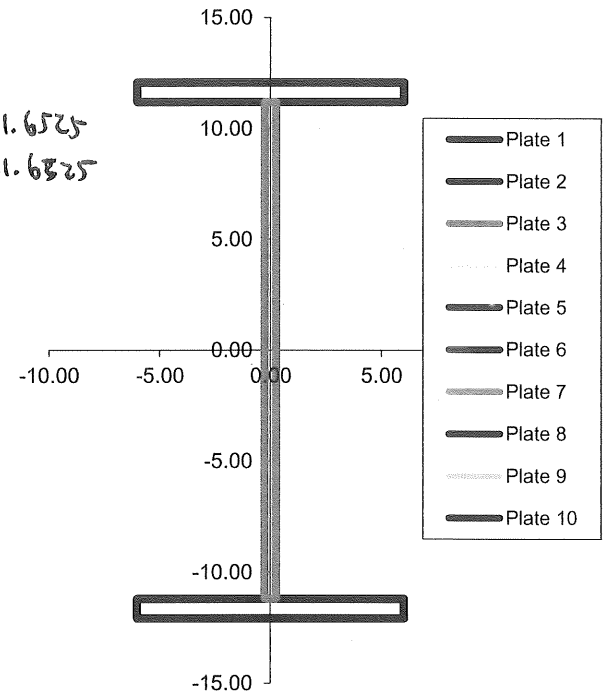
SUBJECT	Stringer S20 at span 24				
XXX					
XXX		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	12.0420	~~0.8800~~	0.855	~~11.6400~~ 11.6525
2	12.0420	~~0.8800~~	0.855	~~-11.6400~~ 11.6825
3	0.5100	22.4000		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	~~32.02~~ 32.04

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
~~3350.60~~	~~277.37~~ 271.37	~~277.37~~	~~10.14~~ 10.11	~~340.67~~ 304.21

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
~~256.36~~	~~42.58~~	~~42.58~~	~~2.80~~ 2.79	~~65.26~~ 63.45

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TFK	DATE	10/11/13

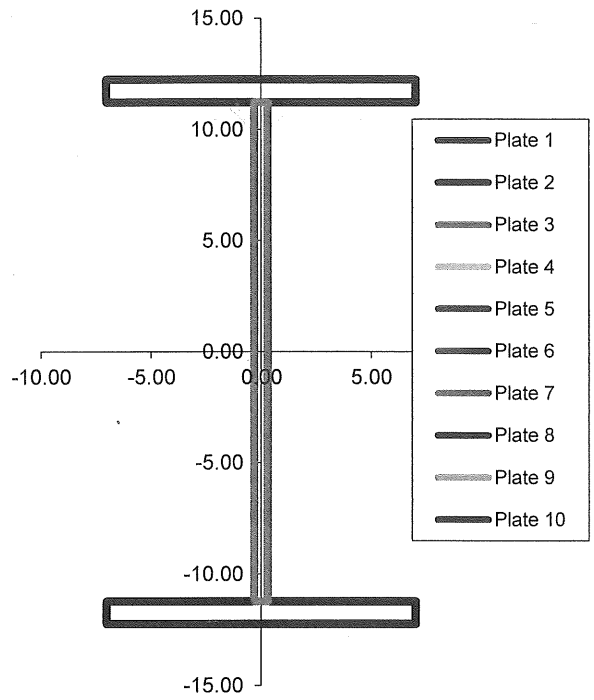
SUBJECT	Stringer S25 at span 25				
24WF145					
		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	14.0430	1.0200		11.7350
2	14.0430	1.0200		-11.7350
3	0.6080	22.4500		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	42.30

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
4520.85	369.20	369.20	10.34	412.79

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
471.21	67.11	67.11	3.34	102.65

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

S25 @ Span 27

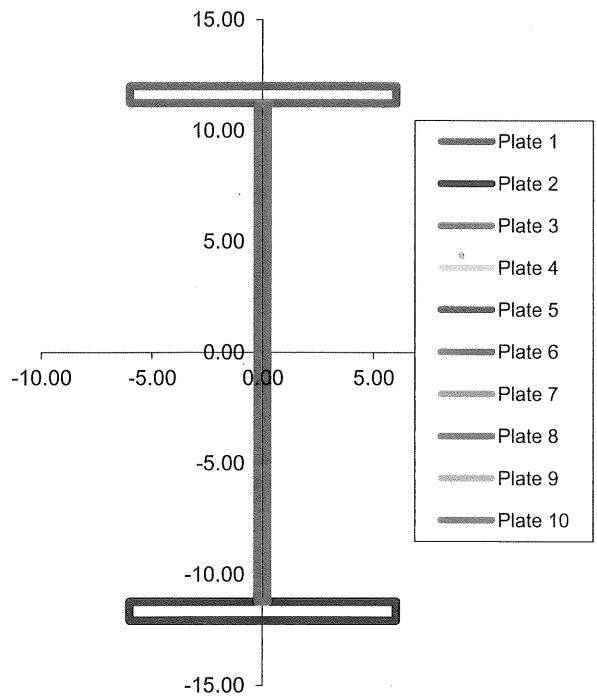
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Stringer 25 at Span 27				
24WF110					
XXX		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	12.0420	0.7695 ✓	10% S.L.	11.6098
2	12.0420	0.8550		-11.6525
3	0.4590 ✓	22.4500 ✓		0.0000
4				
5	10% S.L.			
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.41	29.87

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
3075.69	247.85	263.67	10.15	285.04

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
236.57	39.29	39.29	2.81	60.07

S25 @ span 28

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

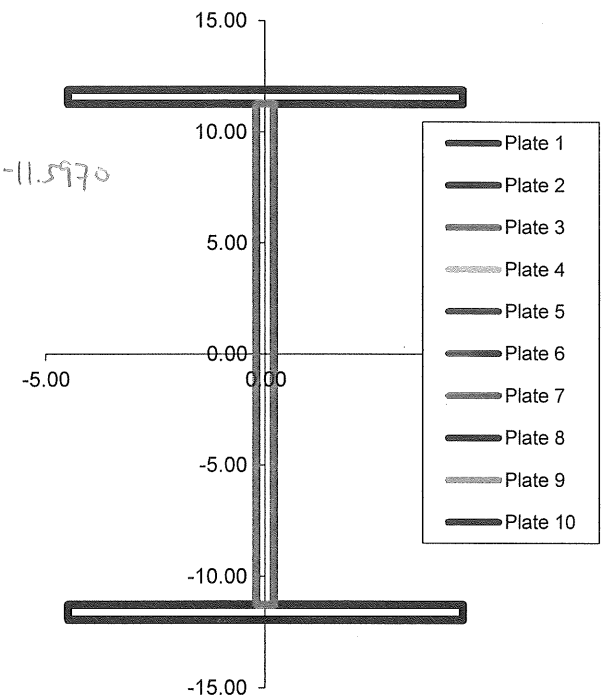
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Stringer 25 at Span 28 - Mid Span				
24WF76					
XXX		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	8.9850	0.6138	10%	11.5799
2	8.9850	~~0.6820~~	5%	~~11.6140~~ -11.5970
3	0.3960 ✓	22.5460 ✓		0.0000
4			0.6479	
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.36	~~20.57~~ 20.91

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
~~1942.09~~	~~158.64~~	~~167.42~~ 174.07	~~9.72~~	~~185.19~~ 186.81

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
~~78.44~~	~~17.46~~	~~17.46~~	~~1.95~~ 2.09	~~27.04~~

S25 @ Span 28

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

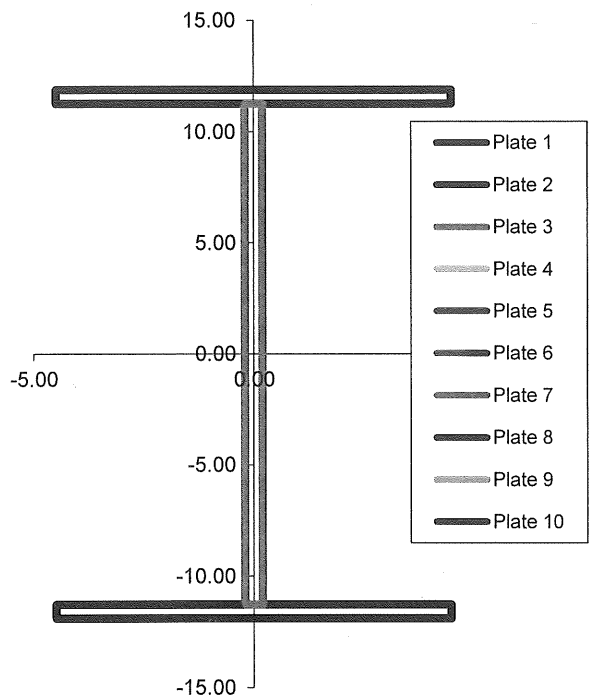
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Stringer 25 at Span 28 _End	End			
24WF76					
XXX		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	8.9850	0.6138	10% S.L	11.5799
2	8.9850	0.6138	10% S.L	-11.5799
3	0.3960	22.5460		0.0000
4				
5	10% S.L			
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	19.96

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
1857.60	156.27	156.27	9.65	178.05

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
74.32	16.54	16.54	1.93	25.66

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

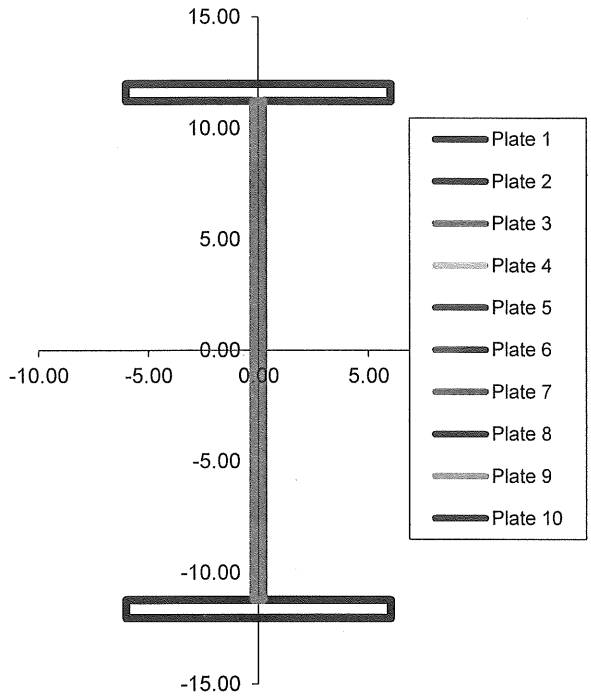
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Stringer 13 at Span 29				
24WF110					
XXX		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	12.0420	0.7695	10% S.L.	11.6098
2	12.0420	0.8123	5% S.L	-11.6311
3	0.4590	22.4500		0.0000
4				
5	100% S.L			
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.21	29.35

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
3004.68	246.18	254.06 ✓	10.12	279.09

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
230.35	38.26	38.26	2.80	58.52

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TFKD	DATE	10/11/13

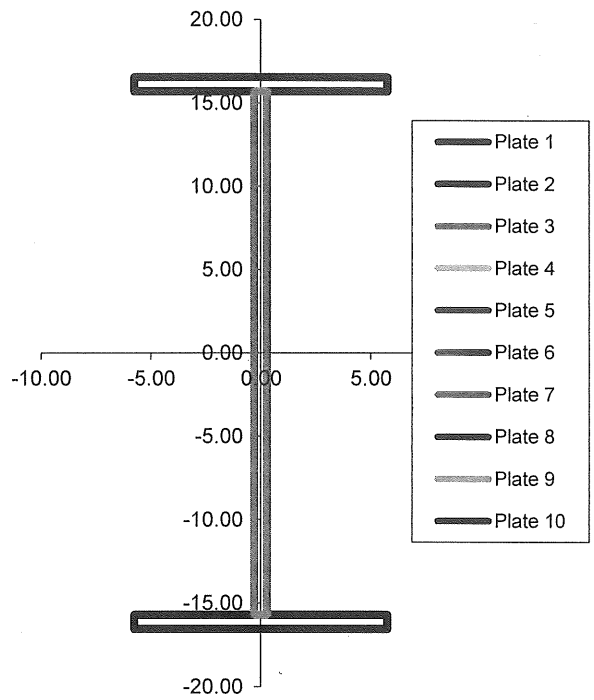
SUBJECT	Floorbeam	1 at span 1			
33WF130					
		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	11.5100	0.8550		16.1225
2	11.5100	0.8550		-16.1225
3	0.5800	31.3900		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	37.89

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
6612.20	399.53	399.53	13.21	460.20

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
217.80	37.85	37.85	2.40	59.28

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB 1-1, 1-3

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TFKo	DATE	10/11/13

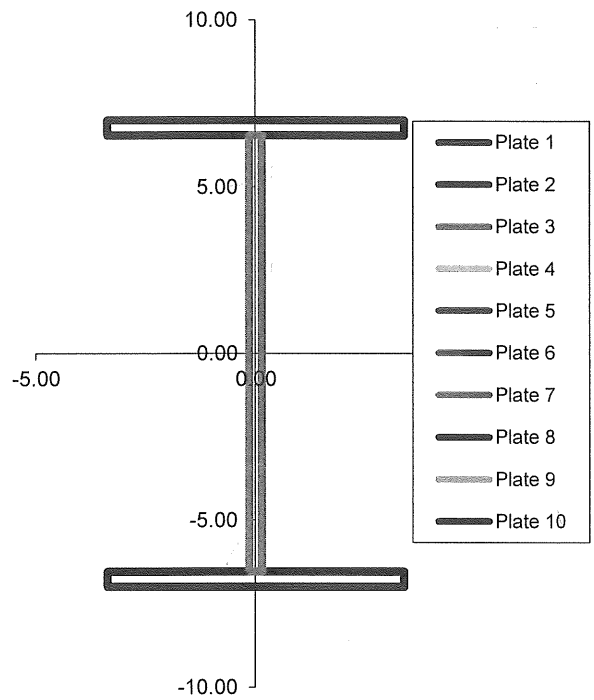
SUBJECT	Floorbeams ~~1E and 1W~~ at span 1			
14WF34				
	PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	6.7500	0.4530		6.7735
2	6.7500	0.4530		-6.7735
3	0.2870	13.0940		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	9.87

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
334.38	47.77	47.77	5.82	53.73

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
23.25	6.89	6.89	1.53	10.59

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

2A 2B -2

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TFKO	DATE	10/11/13

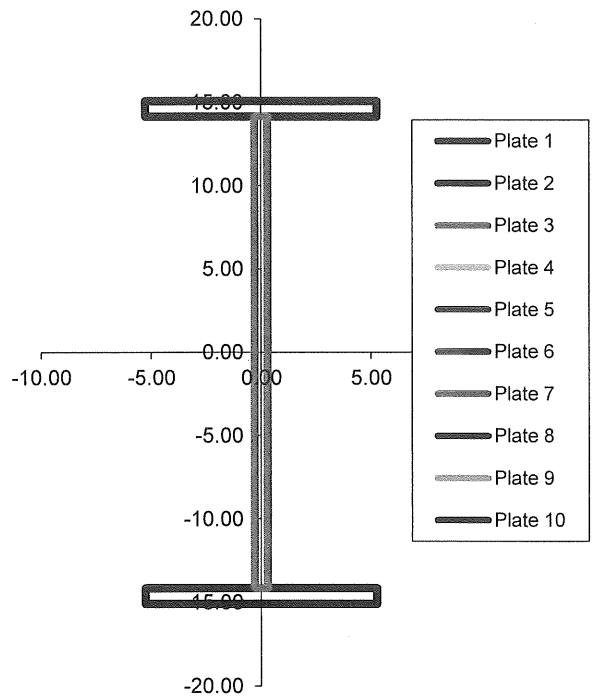
SUBJECT	Floorbeams ~~3A~~ and ~~3B~~ at span 3				
30WF124					
		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	10.5210	0.9300		14.6150
2	10.5210	0.9300		-14.6150
3	0.5850	28.3000		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	36.12

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
5286.25	350.55	350.55	12.10	403.13

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
180.98	34.40	34.40	2.24	53.89

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB 3A, B-1

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TF-KO	DATE	10/11/13

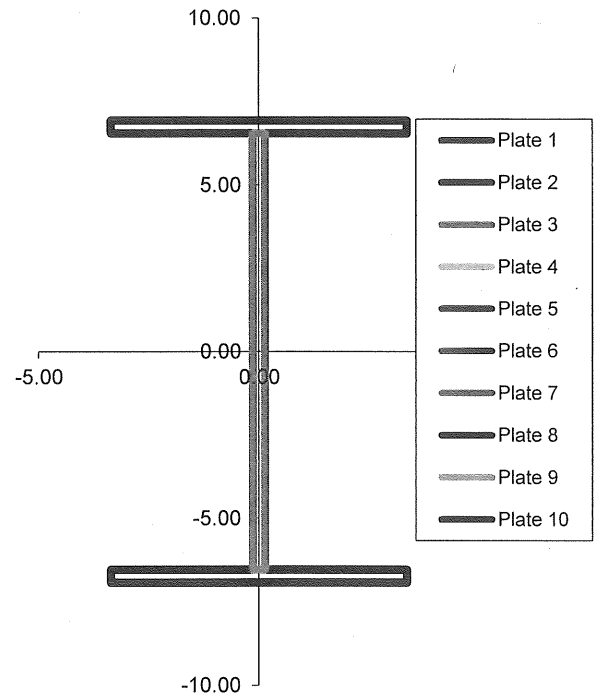
SUBJECT	Floorbeams ~~3E and 3W~~ at span 3				
14WF30					
		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	6.7330	0.3830		6.7385
2	6.7330	0.3830		-6.7385
3	0.2700	13.0940		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	8.69

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
284.76	41.09	41.09	5.72	46.33

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
19.51	5.79	5.79	1.50	8.92

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TF KO	DATE	10/11/13

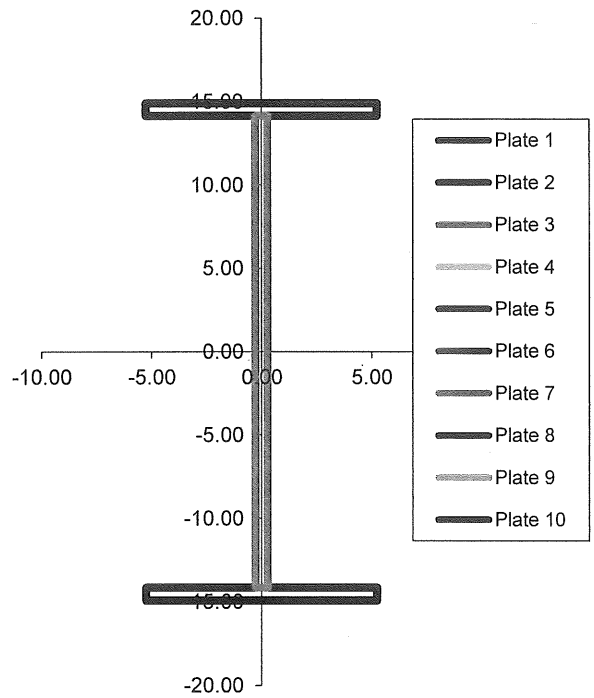
SUBJECT	Floorbeams 4A and 4B at span 4				
30WF108					
		PAGE	1	OF	1

Section Properties Calculation

no section loss

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	10.4840	0.7600		14.5300
2	10.4840	0.7600		-14.5300
3	0.5480	28.3000		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	31.44

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
4400.17	295.12	295.12	11.83	341.27

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
146.35	27.92	27.92	2.16	43.89

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB 6-2

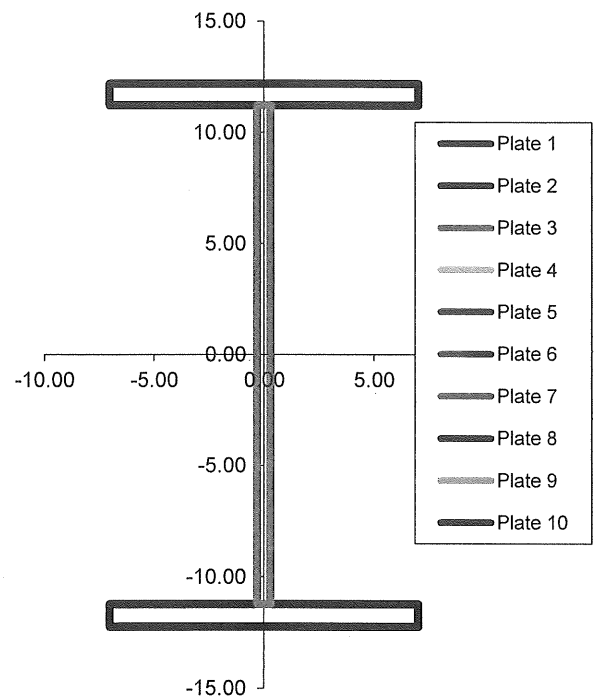
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Floorbeam ~~6W~~ at Span 6				
24WF145					
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	14.0430	0.9690	5% S.L	11.7095
2	14.0430	1.0200		-11.7350
3	0.6080	22.4500		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.21	41.58

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
4412.07	355.68	366.62	10.30	403.91

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
459.44	65.43	65.43	3.32	100.14

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB 6A-3

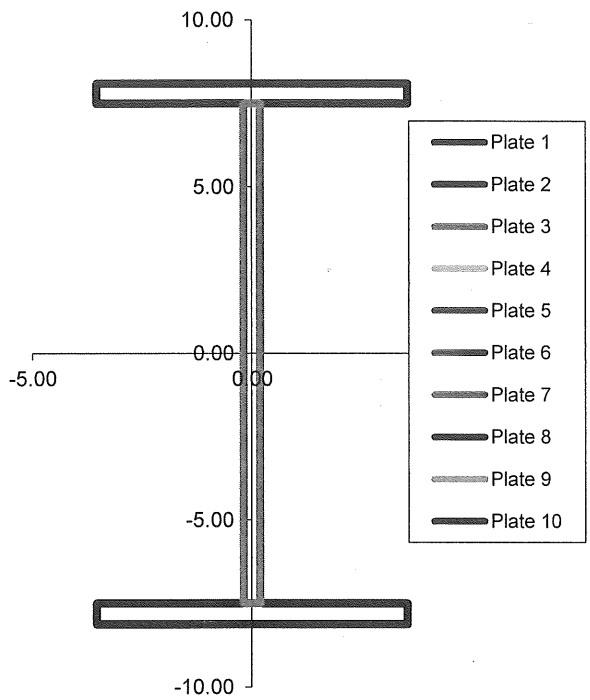
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Floorbeam ~~6A~~ btw Col. Line btw L & M at Span 7				
16WF50					
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	7.0730	0.5966	5% S.L	7.7953
2	7.0730	0.6280		-7.8110
3	0.3800 ✓	14.9940		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.13	14.36

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
634.22	77.16	79.28 ✓	6.65	88.93

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
36.18	10.23	10.23	1.59	15.86

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB24-6

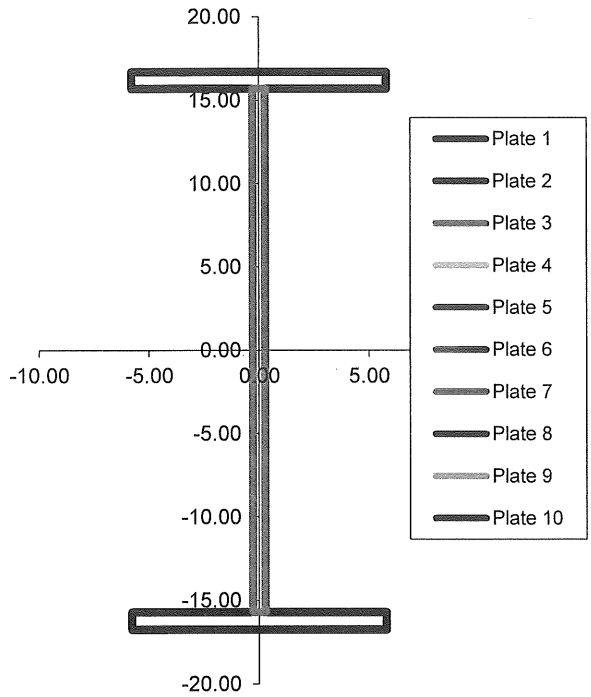
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Floorbeam 24 btw Col G & H at Span 24				
33WF152					
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	11.5650	1.0023	5% S.L	16.1961
2	11.5650	1.0550		-16.2225
3	0.5715	31.3900		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.24	41.73

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
7724.07	455.92	467.97	13.60	526.33

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
265.67	45.94	45.94	2.52	71.35

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB 24-5

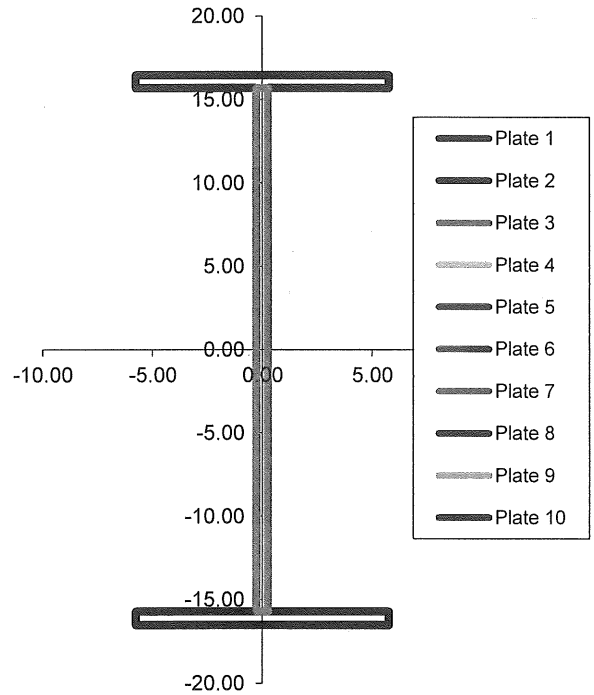
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Floorbeam 24 btw Col. F & G at Span 24				
33WF130	Midspan				
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	11.5100	0.7695	10% S.L.	16.0798
2	11.5100	0.8123	50% S.L	-16.1011
3	0.5220	31.3900		0.0000
4				
5	10% S.L			
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.23	34.59

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
6058.21	362.79	372.29	13.23	421.45

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
201.37	34.99	34.99	2.41	54.53

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

Floorbeam FB 25-7

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10/07/2013
CHECKED BY	Jason A.	DATE	10/10/13

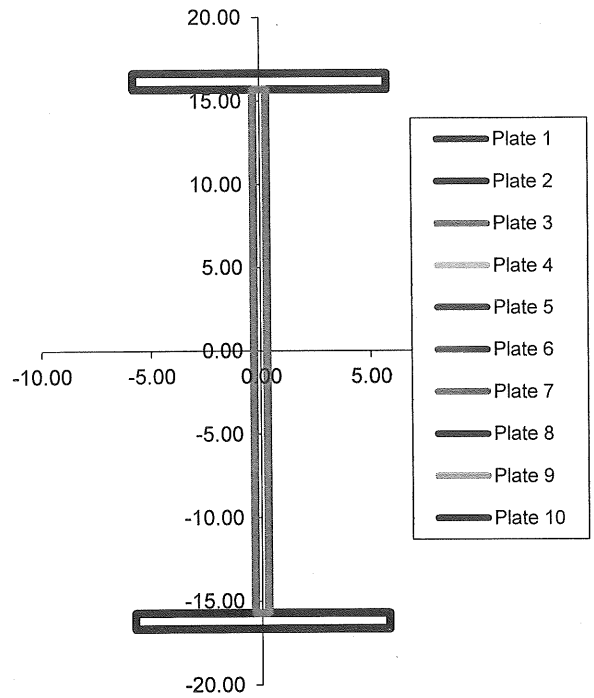
SUBJECT	Stringer 25 at Span ~~27~~ 26				
XXX					
XXX		PAGE	1	OF	1

Section Properties Calculation

no section loss

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	11.5350	0.9600		16.1750
2	11.5350	0.9600		-16.1750
3	0.6050	31.3900		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	41.14

33WF141

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
7355.45	441.64	441.64	13.37	507.26

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
246.15	42.68	42.68	2.45	66.74

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB 27-7

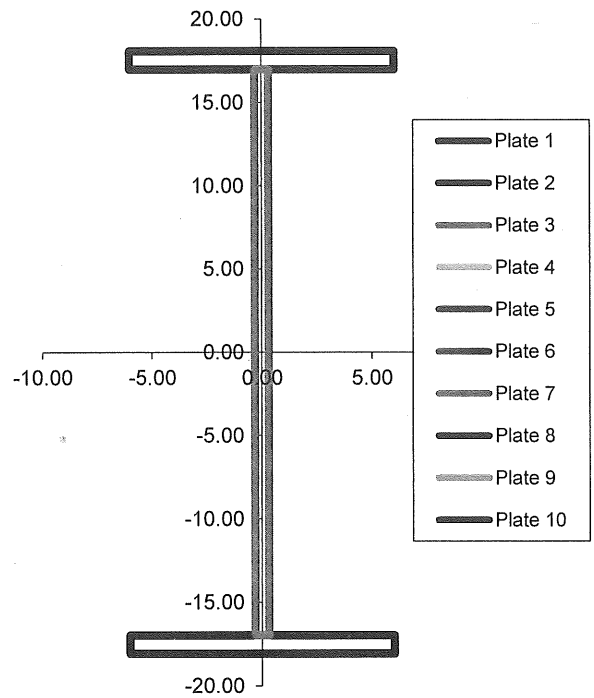
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Floorbeam 27 btw Col. G & H at Span 27				
36WF170					
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	12.0270	1.1000		17.5300
2	12.0270	1.1000		-17.5300
3	0.6120	33.9600		0.0000
4				
5	10% S.L.			
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	47.24

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
10131.10	560.35	560.35	14.64	640.29

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
319.59	53.15	53.15	2.60	82.74

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

FB 27-6

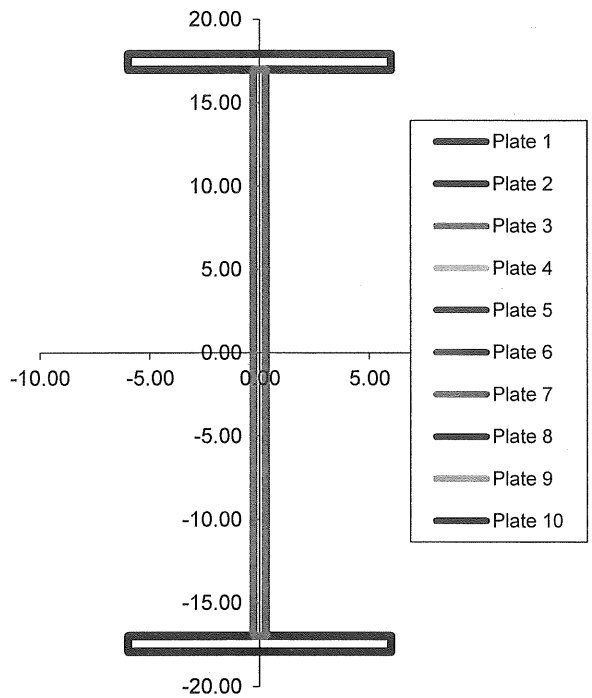
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Floorbeam 27 btw Col. F & G at Span 27				
36WF150					
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	11.9720	0.9400		17.4500
2	11.9720	0.9400		-17.4500
3	0.5625	33.9600		0.0000
4				
5	10% S.L			
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	41.61

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
8691.08	484.99	484.99	14.45	554.93

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
269.33	44.99	44.99	2.54	70.05

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

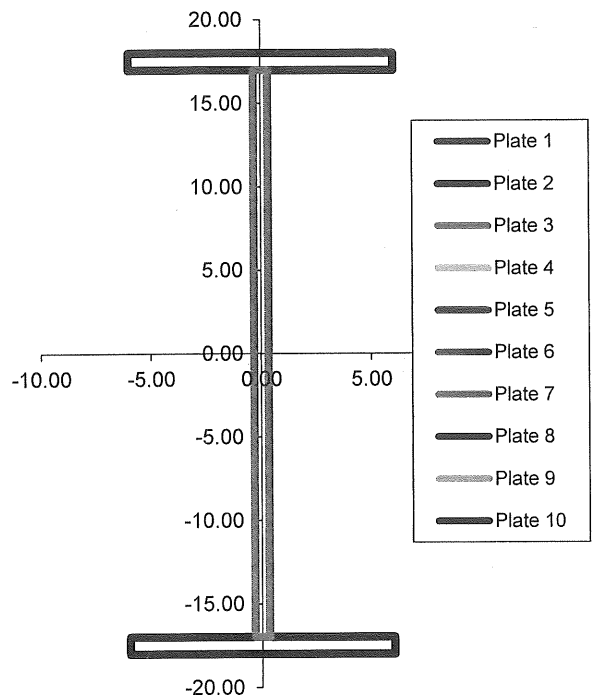
SUBJECT	Floorbeam 28-6 at Span 28				
36WF160					
		PAGE	1	OF	1

No section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	12.0000	1.0200		17.4900
2	12.0000	1.0200		-17.4900
3	0.6530	33.9600		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	46.66

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
9621.81	534.54	534.54	14.36	616.43

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
294.55	49.09	49.09	2.51	77.06

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	10-22-2013
CHECKED BY		DATE	

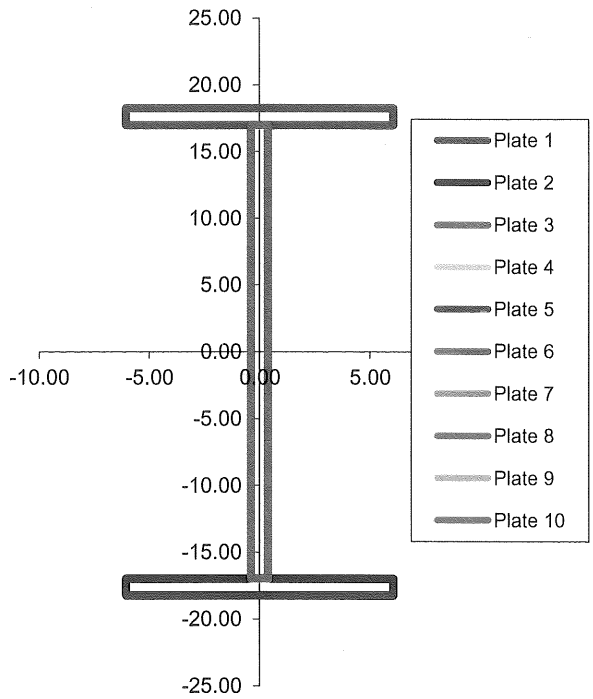
SUBJECT	FB 28-8
	36WF194
PAGE	1 OF 1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	12.1170	1.2600		17.6100
2	12.1170	1.2600		-17.6100
3	0.7700	33.9600		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	56.68

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
11986.38	657.15	657.15	14.54	759.73

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
374.89	61.88	61.88	2.57	97.53

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/18/2013
CHECKED BY	Jason A.	DATE	12/23/13

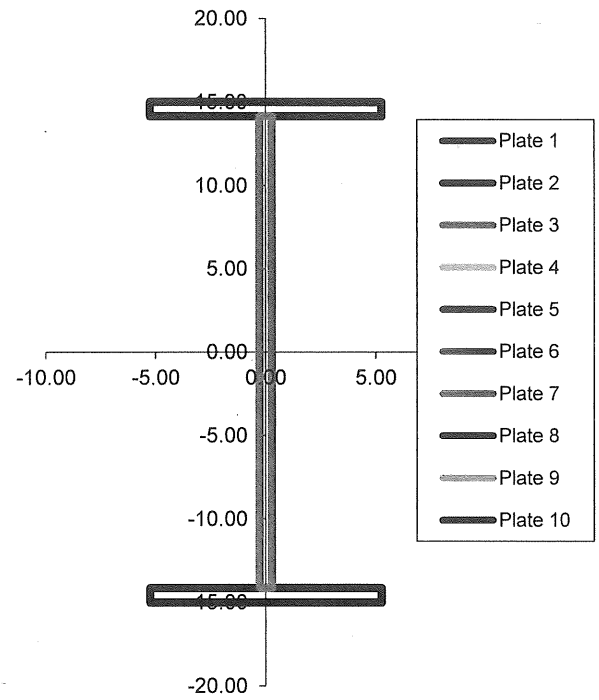
SUBJECT	Floorbeam 29-6 at Span 29				
30WF116					
		PAGE	1	OF	1

No section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	10.5000	0.8500		14.5750
2	10.5000	0.8500		-14.5750
3	0.5640	28.3000		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	33.81

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
4858.23	323.88	323.88	11.99	373.09

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
164.42	31.32	31.32	2.21	49.11

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	10-22-2013
CHECKED BY		DATE	

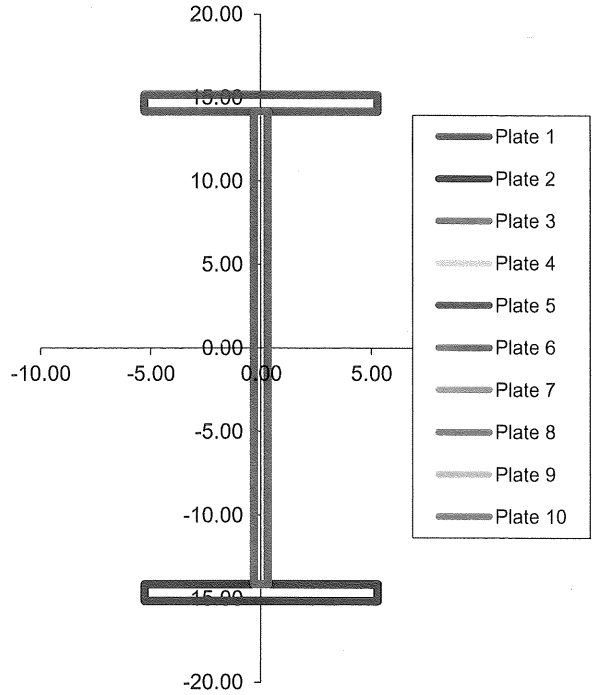
SUBJECT	FB 29-8				
	30WF132				
		PAGE	1	OF	1

Section Properties Calculation

no section loss

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	10.5510	1.0000		14.6500
2	10.5510	1.0000		-14.6500
3	0.6150	28.3000		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	38.51

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
5692.31	375.73	375.73	12.16	432.28

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
196.31	37.21	37.21	2.26	58.34

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PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	10-22-2013
CHECKED BY		DATE	

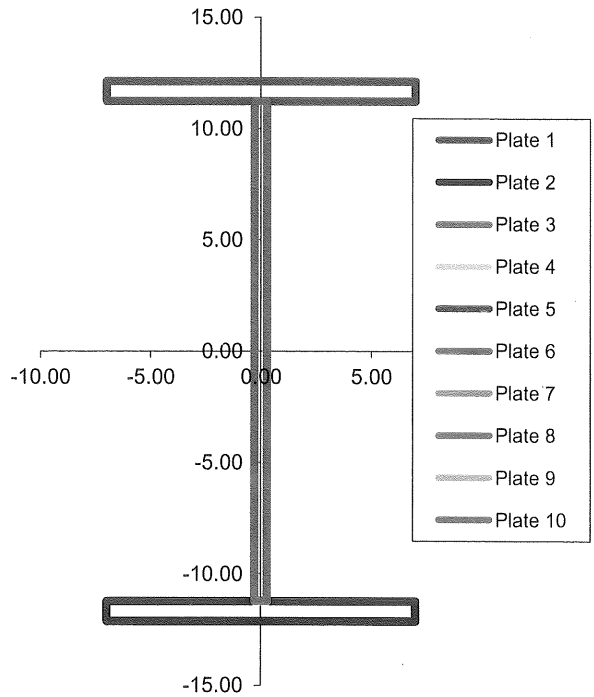
SUBJECT	Span 3, G1
	24WF130
PAGE	1 OF 1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	14.0000	0.9000		11.6750
2	14.0000	0.9000		-11.6750
3	0.5650	22.4500		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	37.88

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
3969.34	327.37	327.37	10.24	365.40

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
411.94	58.85	58.85	3.30	89.99

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TFKo	DATE	10/14/2013

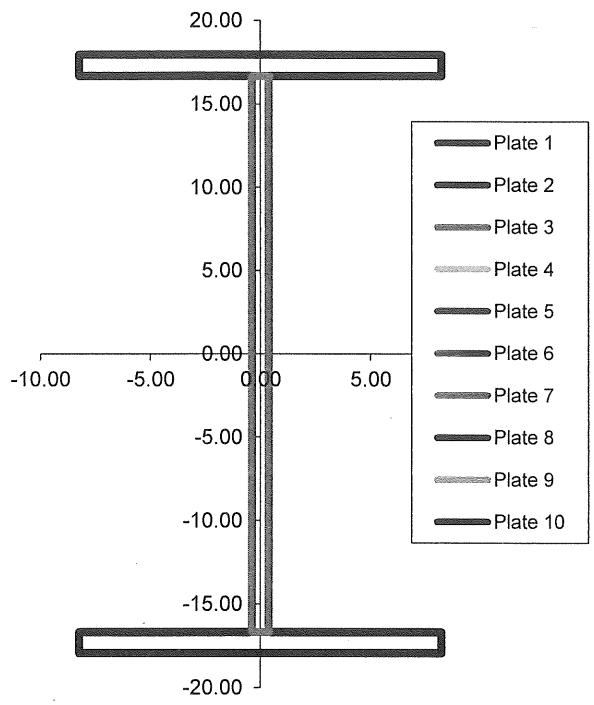
SUBJECT	Girder G2 at span 3				
36WF230					
		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	16.4750	1.2600		17.3100
2	16.4750	1.2600		-17.3100
3	0.7650	33.3600		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	67.04

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
14812.27	825.66	825.66	14.86	931.50

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
940.31	114.15	114.15	3.75	175.88

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TFW	DATE	10/14/13

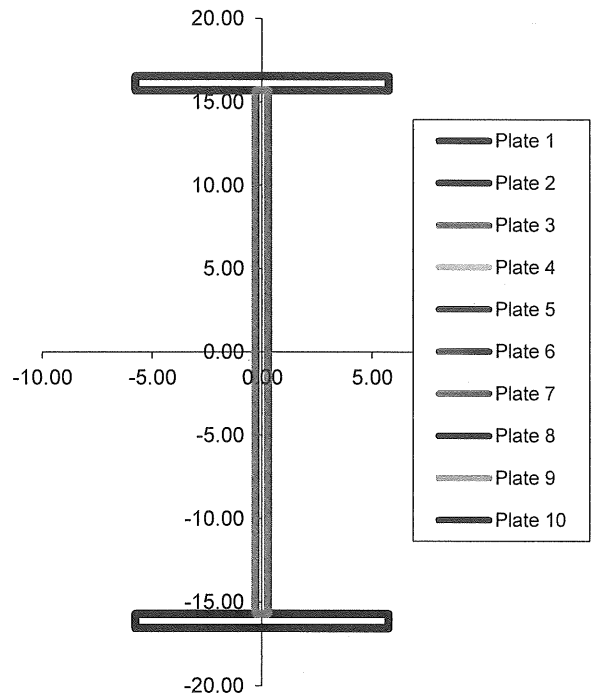
SUBJECT	Girders G1 and G2 at spans 4, 5 and 7				
33WF130					
		PAGE	1	OF	1

Section Properties Calculation

no section loss

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	11.5100	0.8550		16.1225
2	11.5100	0.8550		-16.1225
3	0.5800	31.3900		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	37.89

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
6612.20	399.53	399.53	13.21	460.20

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
217.80	37.85	37.85	2.40	59.28

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	10-22-2013
CHECKED BY		DATE	

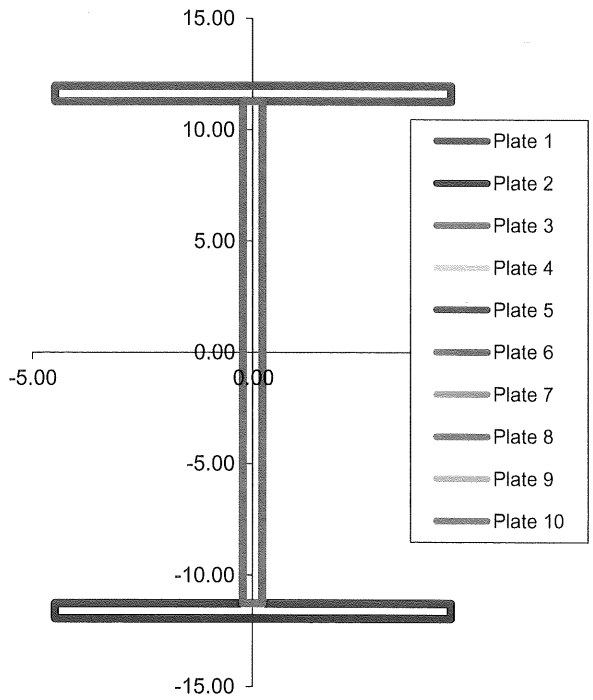
SUBJECT	Span 6, G1			
	24WF76			
		PAGE	1	OF 1

No section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	8.9850	0.6820		11.6140
2	8.9850	0.6820		-11.6140
3	0.4400	22.5460		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	22.18

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
2073.79	173.47	173.47	9.67	198.25

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
82.61	18.39	18.39	1.93	28.62

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Jason A.	DATE	09-23-2013
CHECKED BY	TF Ko	DATE	10/14/2013

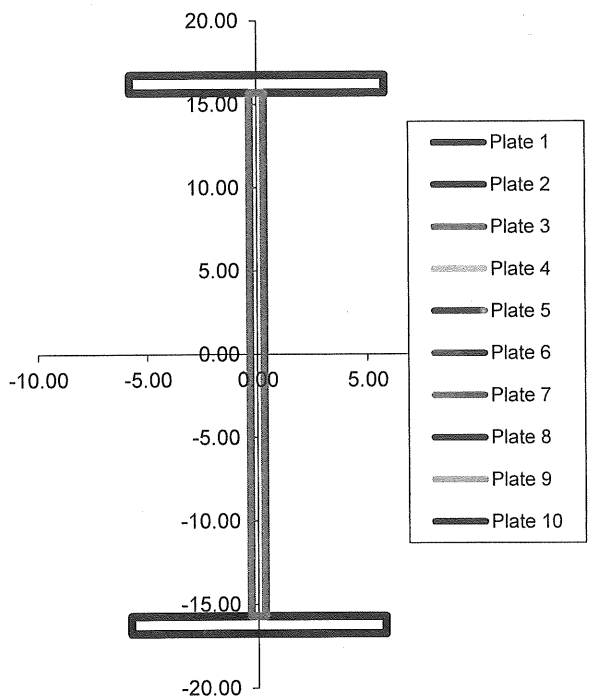
SUBJECT	Girder G2 at span 6				
33WF152					
		PAGE	1	OF	1

no section loss

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	11.5650	1.0550		16.2225
2	11.5650	1.0550		-16.2225
3	0.6350	31.3900		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	44.33

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
8060.86	481.25	481.25	13.48	552.29

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
272.65	47.15	47.15	2.48	73.72

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

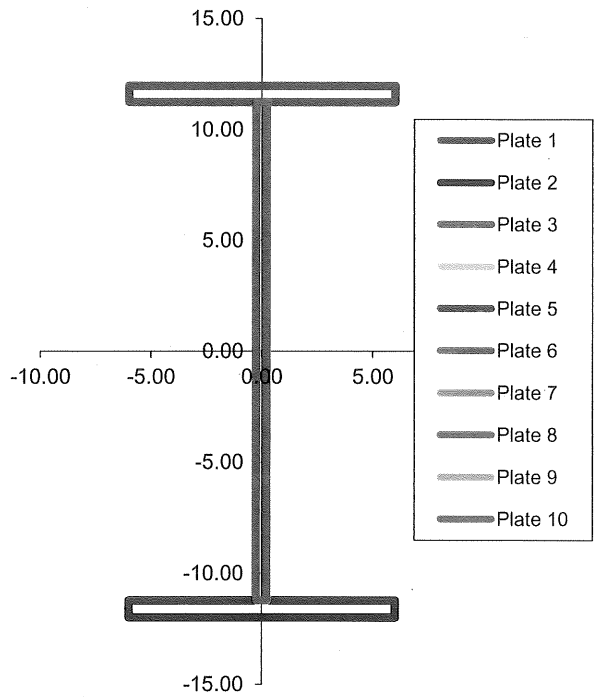
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11-1-2013
CHECKED BY	Jason A.	DATE	12/10/13

SUBJECT	Girder G4 at Span 7				
24WF100					
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	12.0000	0.7363	~ 5/16 S.L	11.5931
2	12.0000	0.7750		-11.6125
3	0.4680	22.4500		0.0000
4				
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	-0.19	28.64

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
2882.60	237.14	244.17	10.03	269.32

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
217.81	36.30	36.30	2.76	55.63

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

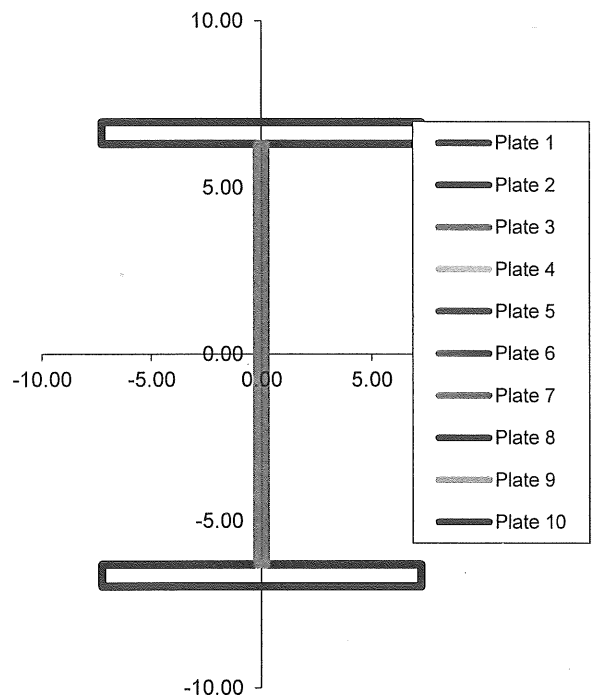
PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	10-14-2013
CHECKED BY	Jason A.	DATE	10/15/13

SUBJECT	Column 27E				
14WF87					
		PAGE	1	OF	1

Section Properties Calculation

PROPOSED PROPERTIES INPUT DATA:

Plate	Dimension		Center of Gravity	
	Width	Height	X_	Y_
	in	in	in	in
1	14.5000	0.6536	50% f.l.	6.6388
2	14.5000	0.6536	50% f.l.	-6.6388
3	0.4009	12.6240		0.0000
4	50% f.l.			
5				
6				
7				
8				
9				
10				



Center of Gravity		
X_	Y_	A
in	in	in^2
0.00	0.00	24.02

Elastic Properties Axis X-X				
I_x	$S_{x,top}$	$S_{x,bot}$	r_x	Z_x
in^4	in^3	in^3	in	in^3
903.28	129.68	129.68	6.13	141.81

Elastic Properties Axis Y-Y				
I_y	$S_{y,left}$	$S_{y,right}$	r_y	Z_y
in^4	in^3	in^3	in	in^3
332.16	45.82	45.82	3.72	69.22

SPANS 1-7 AND 23-29
As Inspected Load Rating Calculations

Stringers

(LFD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/13/13

SUBJECT	LFD Load Rating for flexural member				
		PAGE	1	OF	3

LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 10WF21, S1-12, Span 3-6 ✓

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	9.900	in	M_{DL} =	8.12	k.ft	V_{DL} =	2.22	kips
Flange width,	b =	5.750	in	M_{SDL} =	1.43	k-ft	V_{SDL} =	0.39	kips
Flange thickness,	t_f =	0.340	in	M_{PED} =	14.34	k.ft	V_{PED} =	3.91	kips
Web thickness,	t_w =	0.240	in	M_{H-10} =	41.72	k.ft	V_{H-10} =	5.69	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	1.250	in	Section modulus,			S_x=	21.22	in^3
Unbraced length,	L_b=	14.67	ft	Plastic modulus,			Z_x=	23.79	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 16.91 *(10-29)*

b/t limit = 75% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 38.42

D/t_w limit = 36% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 117.56

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 23.79 in^3

Moment capacity, $F_yZ = M_u$= 65.42 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 1.26

approximately 80% of maximum moment, M_1 = 82.36 k.ft

L_b/r_y = 140.80

$3.6 - 2.2\,(M_1/M_u) \times 10^6 / F_y$ = 25.17

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge	SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE 11/14/2013	
CHECKED BY	Jason A.	DATE 12/23/13	PAGE 2 OF 3

2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	16.91		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.24		
		D/150 =	0.06		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	176.00	in	
		$20{,}000{,}000A_f/F_yd$ =	119.68	in	*(10-101)*
		<	L_b		*n/a*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p$ =	65.37	in	*10.48.4.1*
If $L_b \le L_p$ then,		$F_yS_x = M_y = M_r$ =	n/a	k.ft	*(10-103d)*
		λ =	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
		≤	1.00		
		Use R_b =	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u$ =	58.36	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr}$ =	67.69	ksi	
		≤	F_y		
		Compression $F_{cr}S_xR_b = M_u$ =	58.36	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**58.36**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	38.42		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	42.35	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**42.35**	kips	*(10-113)*

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/23/13

SUBJECT	LFD Load Rating for flexural member				
		PAGE	3	OF	3

5. Calculate Load Rating Factor *AASHTO Appendix C*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

				Factor	Inventory	Operating	(C.2.3)
Moment Capacity, (as built)	M_u =	58.36	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	42.35	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.507	0.847
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.155	5.267

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.507	0.847

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	58.36	58.36
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	9.66	9.66

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.507	0.847
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.783	2.977
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

not used in the summary table

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CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/23/13

SUBJECT	LFD Load Rating for flexural member				
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 10WF21, S13A,B, Span 7

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	9.849	in	M_{DL} =	6.06	k.ft	V_{DL} =	1.91	kips
Flange width,	b =	5.750	in	M_{SDL} =	1.07	k-ft	V_{SDL} =	0.34	kips
Flange thickness,	t_f =	0.289	in	M_{PED} =	10.70	k.ft	V_{PED} =	3.38	kips
Web thickness,	t_w =	0.240	in	M_{H-10} =	36.03	k.ft	V_{H-10} =	5.69	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	1.210	in	Section modulus,			S_x=	20.58	in^3
Unbraced length,	L_b=	12.67	ft	Plastic modulus,			Z_x=	22.29	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 19.90 *(10-29)*

b/t limit = 88% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 38.63

D/t_w limit = 36% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 131.74

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 22.29 in^3

Moment capacity, $F_yZ = M_u$= 61.30 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 1.14

approximately 80% of maximum moment, M_1 = 69.96 k.ft

L_b/r_y = 125.62

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 33.00

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

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PROJECT	Passerelle Pedestrian Bridge	SUBJECT	LFD Load Rating for flexural member
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	19.90		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.24		
		D/150 =	0.06		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	152.00	in	
		$20{,}000{,}000A_f/F_yd$ =	102.26	in	*(10-101)*
		<	L_b		*n/a*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p$ =	63.28	in	*10.48.4.1*
If $L_b \le L_p$ then,		$F_yS_x = M_y = M_r$ =	n/a	k.ft	*(10-103d)*
		λ =	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
		≤	1.00		
		Use R_b =	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u$ =	56.60	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr}$ =	48.91	ksi	
		≤	F_y		
		Compression $F_{cr}S_xR_b = M_u$ =	56.60	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**56.60**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	38.63		*(10-116)*
		$6{,}000*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	42.59	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**42.59**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	56.60	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	42.59	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.605	1.010
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.212	5.362

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.605	1.010

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	56.60	56.60
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	10.40	10.40

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.605	1.010 ✓
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.833	3.059 ✓
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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PROJECT	Passerelle Pedestrian Bridge		
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SUBJECT	LFD Load Rating for flexural member				
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 10WF21, S14A, Span 7 ✓

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	9.880	in	M_{DL} =	6.06	k.ft	V_{DL} =	1.91	kips
Flange width,	b =	5.750	in	M_{SDL} =	1.07	k-ft	V_{SDL} =	0.34	kips
Flange thickness,	t_f =	0.323	in	M_{PED} =	10.70	k.ft	V_{PED} =	3.38	kips
Web thickness,	t_w =	0.240	in	$M_{H\text{-}10}$ =	36.03	k.ft	$V_{H\text{-}10}$ =	5.69	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	1.320	in	Section modulus,			S_x=	21.07	in^3
Unbraced length,	L_b=	12.67	ft	Plastic modulus,			Z_x=	23.30	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 17.80 *(10-29)*

b/t limit = 79% ; $4{,}110/\sqrt{F_y}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 38.48

D/t_w limit = 36% ; $19{,}230/\sqrt{F_y}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 121.79

$33{,}650/\sqrt{F_y}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 23.30 in^3

Moment capacity, $F_yZ = M_u$= 64.08 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 1.09

approximately 80% of maximum moment, M_1 = 69.96 k.ft

L_b/r_y = 115.15

$3.6 - 2.2\,(M_1/M_u) \times 10^6 / F_y$ = 36.30

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

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PROJECT	Passerelle Pedestrian Bridge		
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	17.80		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.24		
		D/150 =	0.06		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	152.00	in	
		$20{,}000{,}000A_f/F_yd$ =	113.93	in	*(10-101)*
		<	L_b		*n/a*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p$ =	69.03	in	*10.48.4.1*
If $L_b \le L_p$ then,	$F_yS_x = M_y = M_r$ =	n/a	k.ft	*(10-103d)*
	λ =	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
	≤	1.00		
	Use R_b =	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u$ =	57.94	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr}$ =	61.09	ksi	
	≤	F_y		
	Compression $F_{cr}S_xR_b = M_u$ =	57.94	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**57.94**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	38.48		*(10-116)*
		$6{,}000*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	42.42	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**42.42**	kips	*(10-113)*

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PROJECT	Passerelle Pedestrian Bridge		
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SUBJECT	LFD Load Rating for flexural member				
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	57.94	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	42.42	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.623	1.039
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.198	5.339

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.623	1.039

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	57.94	57.94
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	10.61	10.61

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.623	1.039
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.824	3.045
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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PROJECT	Passerelle Pedestrian Bridge		
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 24WF110, S20, Span 24

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	24.160	in	M_{DL} =	318.13	k.ft	V_{DL} =	55.53	kips
Flange width,	b =	12.042	in	M_{SDL} =	9.70	k-ft	V_{SDL} =	1.21	kips
Flange thickness,	t_f =	0.855	in	M_{PED} =	96.97	k.ft	V_{PED} =	12.06	kips
Web thickness,	t_w =	0.510	in	M_{H-10} =	90.55	k.ft	V_{H-10} =	6.82	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.790	in	Section modulus,			S_x=	271.37	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	304.21	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 14.08 *(10-29)*

b/t limit = 62% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 44.02

D/t_w limit = 42% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 109.93

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 304.21 in^3

Moment capacity, $F_yZ = M_u$= 836.58 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 509.28 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **836.58** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	14.08		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.51		
		D/150 =	0.15		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	0.00	in	
		$20{,}000{,}000A_f/F_yd$ =	258.28	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p$ =	145.91	in	*10.48.4.1*
If $L_b \le L_p$ then,		$F_yS_x = M_y = M_r$ =	746.27	k.ft	*(10-103d)*
		λ =	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
		≤	1.00		
		Use R_b =	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u$ =	746.27	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr}$ =	97.60	ksi	
		≤	F_y		
		Compression $F_{cr}S_xR_b = M_u$ =	746.27	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	44.02		*(10-116)*
		$6{,}000*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	219.14	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**219.14**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO Appendix C*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

				Factor	Inventory	Operating	(C.2.3)
Moment Capacity, (as built)	M_u =	836.58	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	219.14 /	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.950	3.256
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	5.555	9.273

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.950	3.256

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	836.58	836.58
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	124.80	124.80

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.950	3.256
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	2.206	3.682
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/23/13

SUBJECT	LFD Load Rating for flexural member				
		PAGE	1	OF	3

LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 24WF145, S25 at Span 25

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	24.490	in	M_{DL} =	118.70	k.ft	V_{DL} =	12.83	kips
Flange width,	b =	14.043	in	M_{SDL} =	344.39	k-ft	V_{SDL} =	23.47	kips
Flange thickness,	t_f =	1.020	in	M_{PED} =	128.34	k.ft	V_{PED} =	13.88	kips
Web thickness,	t_w =	0.608	in	M_{H-10} =	107.05	k.ft	V_{H-10} =	6.82	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	3.340	in	Section modulus,			S_x=	369.20	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	412.79	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 13.77 *(10-29)*

b/t limit = 61% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 36.92

D/t_w limit = 35% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 101.36

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 412.79 in^3

Moment capacity, $F_yZ = M_u$= 1135.17 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 704.41 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1135.17** k.ft

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
CHECKED BY	Joyce A.	DATE	12/23/13	PAGE 2	OF 3

2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	13.77		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.61		
		D/150 =	0.15		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	0.00	in	
		$20{,}000{,}000A_f/F_yd$ =	354.48	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p$ =	174.67	in	*10.48.4.1*
If $L_b \le L_p$ then,		$F_yS_x = M_y = M_r$ =	1015.30	k.ft	*(10-103d)*
		λ =	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
		≤	1.00		
		Use R_b =	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u$ =	1015.30	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr}$ =	102.14	ksi	
		≤	F_y		
		Compression $F_{cr}S_xR_b = M_u$ =	1015.30	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	36.92		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	261.25	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**261.25**	kips	*(10-113)*

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
CHECKED BY	Dagon A.	DATE	12/23/13	PAGE 3	OF 3

5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1135.17	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	261.25	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.914	3.196
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	7.107	11.863

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.914	3.196

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1135.17	1135.17
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	104.85	104.85

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.914	3.196
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	3.638	6.072
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/23/13

SUBJECT	LFD Load Rating for flexural member				
		PAGE	1	OF	3

LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 24WF110, S13, Span 29

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	24.032	in	M_{DL} =	172.54	k.ft	V_{DL} =	14.53	kips
Flange width,	b =	12.042	in	M_{SDL} =	19.39	k-ft	V_{SDL} =	1.63	kips
Flange thickness,	tf =	0.770	in	M_{PED} =	193.90	k.ft	V_{PED} =	16.33	kips
Web thickness,	t_w =	0.459	in	M_{H-10} =	130.94	k.ft	V_{H-10} =	6.25	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.800	in	Section modulus,			S_x=	254.06	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	279.09	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 15.65 *(10-29)*

b/t limit = 69% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 49.00

D/t_w limit = 46% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 122.24

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 279.09 in^3

Moment capacity, $F_yZ = M_u$= 767.50 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 536.22 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **767.50** k.ft

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
CHECKED BY	Jason A.	DATE	12/23/13	PAGE	2 OF 3

2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	15.65		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.46		
		$D/150 =$	0.15		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	233.69	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	146.43	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	698.67	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	698.67	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	79.05	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	698.67	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	49.00		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	197.61	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**197.61**	kips	*(10-113)*

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
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SUBJECT	LFD Load Rating for flexural member				
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	767.50	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	197.61	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.231	2.055
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	4.984	8.319

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.231	2.055

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	767.50	767.50
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	64.63	64.63

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.231	2.055
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	2.753	4.595
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/23/13

SUBJECT	LFD Load Rating for flexural member				
		PAGE	1	OF	3

LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 24WF110, S27, Span 29

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	24.160	in	M_{DL} =	185.27	k.ft	V_{DL} =	15.60	kips
Flange width,	b =	12.042	in	M_{SDL} =	21.12	k-ft	V_{SDL} =	1.78	kips
Flange thickness,	t_f =	0.855	in	M_{PED} =	211.52	k.ft	V_{PED} =	17.81	kips
Web thickness,	t_w =	0.510	in	M_{H-10} =	142.84	k.ft	V_{H-10} =	6.82	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.790	in	Section modulus,			S_x=	271.37	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	304.21	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 14.08 *(10-29)*

b/t limit = 62% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 44.02

D/t_w limit = 42% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, 4.68(b/t) + D/t_w = 109.93

33,650/√Fy = 185.24

≥ 4.68(b/t)+D/t_w *Compact*

If section is compact,

Plastic modulus, Z_x= 304.21 in^3

Moment capacity, $F_yZ = M_u$= 836.58 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 581.84 k.ft

L_b/r_y = 0.00

3.6 - 2.2 $(M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **836.58** k.ft

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	14.08		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.51		
		D/150 =	0.15		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	0.00	in	
		$20{,}000{,}000A_f/F_yd$ =	258.28	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p$ =	145.91	in	*10.48.4.1*
If $L_b \le L_p$ then,		$F_yS_x = M_y = M_r$ =	746.27	k.ft	*(10-103d)*
		λ =	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
		≤	1.00		
		Use R_b =	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u$ =	746.27	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr}$ =	97.60	ksi	
		≤	F_y		
		Compression $F_{cr}S_xR_b = M_u$ =	746.27	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	44.02		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	219.14	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**219.14**	kips	*(10-113)*

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PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member		
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	836.58	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	219.14	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.238	2.067
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	5.086	8.489

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.238	2.067

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	836.58	836.58
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	70.44	70.44

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.238	2.067
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	2.818	4.703
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

SPANS 1-7 AND 23-29
As Inspected Load Rating Calculations

Floorbeams

(LFD Method)

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/23/13

SUBJECT	LFD Load Rating for flexural member				
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF130, FB1-2, Span 1

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.100	in	M_{DL} =	279.31	k.ft	V_{DL} =	21.28	kips
Flange width,	b =	11.500	in	M_{SDL} =	40.00	k-ft	V_{SDL} =	3.00	kips
Flange thickness,	t_f =	0.855	in	M_{PED} =	506.67	k.ft	V_{PED} =	38.00	kips
Web thickness,	t_w =	0.580	in	M_{H-10} =	212.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.400	in	Section modulus,			S_x=	399.53	in^3
Unbraced length,	L_b=	5.33	ft	Plastic modulus,			Z_x=	460.20	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 13.45 *(10-29)*

b/t limit = 59% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 54.12

D/t_w limit = 51% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 117.07

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 460.20 in^3

Moment capacity, $F_yZ = M_u$= 1265.55 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.96

approximately 80% of maximum moment, M_1 = 1211.66 k.ft

L_b/r_y = 26.67

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 45.26

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1265.55** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	13.45		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.58		
		$D/150 =$	0.21		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	64.00	in	
		$20{,}000{,}000A_f/F_yd =$	180.03	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	125.51	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	1098.71	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	1098.71	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	107.01	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	1098.71	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	54.12		*(10-116)*
		$6{,}000*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	348.47	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**348.47**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1265.55	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	348.47	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.774	1.291
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.843	6.415

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.774	1.291

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1265.55	1265.55
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	95.35	95.35

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.774	1.291
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	2.153	3.593
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member		
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 14WF34, FB1-1,1-3, Span 1

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	14.000	in	M_{DL} =	25.14	k.ft	V_{DL} =	4.79	kips
Flange width,	b =	6.750	in	M_{SDL} =	4.00	k-ft	V_{SDL} =	0.75	kips
Flange thickness,	t_f =	0.453	in	M_{PED} =	50.67	k.ft	V_{PED} =	9.50	kips
Web thickness,	t_w =	0.287	in	M_{H-10} =	52.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	1.530	in	Section modulus,			S_x=	47.77	in^3
Unbraced length,	L_b=	5.33	ft	Plastic modulus,			Z_x=	53.73	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 14.90 *(10-29)*

b/t limit = 66% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 45.62

D/t_w limit = 43% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 115.36

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 53.73 in^3

Moment capacity, $F_yZ = M_u$= 147.76 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.82

approximately 80% of maximum moment, M_1 = 120.58 k.ft

L_b/r_y = 41.80

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 54.69

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **147.76** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	14.90		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.29		
		$D/150 =$	0.09		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	63.96	in	
		$20{,}000{,}000A_f/F_yd =$	132.37	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	80.01	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	131.37	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	131.37	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	87.20	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	131.37	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	45.62		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	71.93	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**71.93**	kips	*(10-113)*

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SUBJECT	LFD Load Rating for flexural member				
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	147.76	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	71.93	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.974	1.625
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	2.983	4.979

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.974	1.625

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	147.76	147.76
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	28.33	28.33

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.974	1.625
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	1.657	2.766
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

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PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member		
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 30WF124, FB 2A,2B-2, Span 3

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	30.160	in	M_{DL} =	277.50	k.ft	V_{DL} =	21.12	kips
Flange width,	b =	10.521	in	M_{SDL} =	42.67	k-ft	V_{SDL} =	3.20	kips
Flange thickness,	tf =	0.930	in	M_{PED} =	416.00	k.ft	V_{PED} =	31.20	kips
Web thickness,	t_w =	0.585	in	M_{H-10} =	212.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.240	in	Section modulus,			S_x=	350.55	in^3
Unbraced length,	L_b=	5.33	ft	Plastic modulus,			Z_x=	403.13	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 11.31 *(10-29)*

b/t limit = 50% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 48.38

D/t_w limit = 46% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 101.32

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 403.13 in^3

Moment capacity, $F_yZ = M_u$= 1108.61 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.95

approximately 80% of maximum moment, M_1 = 1055.15 k.ft

L_b/r_y = 28.57

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 45.64

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1108.61** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	11.31		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.59		
		$D/150 =$	0.19		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	64.00	in	
		$20{,}000{,}000A_f/F_yd =$	196.62	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	117.14	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	964.01	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	964.01	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	151.27	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	964.01	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	48.38		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	316.87	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**316.87**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1108.61	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	316.87	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.767	1.280
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	4.213	7.033

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.767	1.280

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1108.61	1108.61
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	83.54	83.54

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.767	1.280
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	2.341	3.908
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 14WF30, 3A,B-1, Span 3

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	13.860	in	M_{DL} =	25.01	k.ft	V_{DL} =	4.75	kips
Flange width,	b =	6.733	in	M_{SDL} =	4.27	k-ft	V_{SDL} =	0.80	kips
Flange thickness,	t_f =	0.383	in	M_{PED} =	38.93	k.ft	V_{PED} =	6.80	kips
Web thickness,	t_w =	0.270	in	M_{H-10} =	52.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	1.500	in	Section modulus,			S_x=	41.09	in^3
Unbraced length,	L_b=	5.33	ft	Plastic modulus,			Z_x=	46.33	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 17.58 *(10-29)*

b/t limit = 78% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 48.50

D/t_w limit = 46% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 130.77

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 46.33 in^3

Moment capacity, $F_yZ = M_u$= 127.41 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.95

approximately 80% of maximum moment, M_1 = 120.72 k.ft

L_b/r_y = 42.66

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 45.92

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **127.41** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	17.58		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.27		
		$D/150 =$	0.09		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	64.00	in	
		$20{,}000{,}000A_f/F_yd =$	112.76	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	78.44	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	113.00	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	113.00	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	62.64	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	113.00	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	48.50		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	67.67	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**67.67**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	127.41	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	67.67	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.792	1.322
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	2.786	4.650

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.792	1.322

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	127.41	127.41
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	24.40	24.40

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.792	1.322
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	1.538	2.568
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 30WF108, FB 4A,4B, Span 4

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	29.820	in	M_{DL} =	245.99	k.ft	V_{DL} =	18.72	kips
Flange width,	b =	10.484	in	M_{SDL} =	37.33	k-ft	V_{SDL} =	2.80	kips
Flange thickness,	tf =	0.760	in	M_{PED} =	362.67	k.ft	V_{PED} =	27.20	kips
Web thickness,	t_w =	0.548	in	M_{H-10} =	212.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.160	in	Section modulus,			S_x=	295.12	in^3
Unbraced length,	L_b=	5.33	ft	Plastic modulus,			Z_x=	341.27	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 13.79 *(10-29)*

b/t limit = 61% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 51.64

D/t_w limit = 49% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 116.20

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 341.27 in^3

Moment capacity, $F_yZ = M_u$= 938.49 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.98

approximately 80% of maximum moment, M_1 = 924.25 k.ft

L_b/r_y = 29.63

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 43.44

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **938.49** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	13.79		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.55		
		$D/150 =$	0.19		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	64.00	in	
		$20{,}000{,}000A_f/F_yd =$	161.94	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	112.96	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	811.58	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	811.58	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	101.74	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	811.58	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	51.64		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	296.83	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**296.83**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	938.49	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	296.83	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.724	1.209
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	4.555	7.603

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.724	1.209

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	938.49	938.49
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	70.74	70.74

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.724	1.209
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	2.543	4.246
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member		
BY	Tung For Ko	DATE	11/14/2013				
CHECKED BY	Jason A.	DATE	12/23/13		PAGE 1	OF	3

LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 16WF50, FB6A-3, Span 7

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	16.250	in	M_{DL} =	57.53	k.ft	V_{DL} =	8.13	kips
Flange width,	b =	7.073	in	M_{SDL} =	9.49	k-ft	V_{SDL} =	1.33	kips
Flange thickness,	tf =	0.597	in	M_{PED} =	97.07	k.ft	V_{PED} =	13.32	kips
Web thickness,	t_w =	0.380	in	M_{H-10} =	97.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	1.590	in	Section modulus,			S_x=	79.28	in^3
Unbraced length,	L_b=	5.33	ft	Plastic modulus,			Z_x=	88.93	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 11.85 *(10-29)*

b/t limit = 52% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 39.62

D/t_w limit = 37% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 95.07

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 88.93 in^3

Moment capacity, $F_yZ = M_u$= 244.56 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.97

approximately 80% of maximum moment, M_1 = 238.21 k.ft

L_b/r_y = 40.23

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 44.15

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **244.56** k.ft

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CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	11.85		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.38		
		$D/150 =$	0.10		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	63.96	in	
		$20{,}000{,}000A_f/F_yd =$	157.49	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	83.15	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	218.02	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	218.02	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	137.93	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	218.02	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	39.62		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	109.51	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**109.51**	kips	*(10-113)*

WEIDLINGER ASSOCIATES® INC
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PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	244.56	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	109.51	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.747	1.248
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.363	5.614

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.747	1.248

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	244.56	244.56
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	33.90	33.90

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.747	1.248
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.848	3.084
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member		
BY	Tung For Ko	DATE	11/14/2013				
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF141, FB22-4, Span 22

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.310	in	M_{DL} =	191.36	k.ft	V_{DL} =	20.51	kips
Flange width,	b =	11.535	in	M_{SDL} =	393.21	k-ft	V_{SDL} =	26.93	kips
Flange thickness,	t_f =	0.960	in	M_{PED} =	200.60	k.ft	V_{PED} =	21.47	kips
Web thickness,	t_w =	0.605	in	M_{H-10} =	122.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.350	in	Section modulus,			S_x=	446.80	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	507.26	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 12.02 *(10-29)*

b/t limit = 53% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 51.88

D/t_w limit = 49% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 108.12

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 507.26 in^3

Moment capacity, $F_yZ = M_u$= 1394.97 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 956.19 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1394.97** k.ft

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CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	12.02		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.61		
		D/150 =	0.21		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	0.00	in	
		$20{,}000{,}000A_f/F_yd$ =	201.48	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p$ =	122.90	in	*10.48.4.1*
If $L_b \le L_p$ then,		$F_yS_x = M_y = M_r$ =	1228.70	k.ft	*(10-103d)*
		λ =	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
		≤	1.00		
		Use R_b =	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u$ =	1228.70	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr}$ =	134.10	ksi	
		≤	F_y		
		Compression $F_{cr}S_xR_b = M_u$ =	1228.70	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	51.88		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	363.49	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**363.49**	kips	*(10-113)*

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PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member		
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1394.97	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	363.49	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.459	2.435
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	6.478	10.814

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.459	2.435

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1394.97	1394.97
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	129.64	129.64

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.459	2.435
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	3.357	5.604
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF152, FB24-6, Span 24

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.447	in	M_{DL} =	353.53	k.ft	V_{DL} =	35.44	kips
Flange width,	b =	11.565	in	M_{SDL} =	540.85	k-ft	V_{SDL} =	37.35	kips
Flange thickness,	t_f =	1.002	in	M_{PED} =	389.04	k.ft	V_{PED} =	38.90	kips
Web thickness,	t_w =	0.572	in	M_{H-10} =	122.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.390	in	Section modulus,			S_x=	467.97	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	526.33	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 11.54 *(10-29)*

b/t limit = 51% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 55.02

D/t_w limit = 52% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 109.02

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 526.33 in^3

Moment capacity, $F_yZ = M_u$= 1447.41 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 1605.52 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1447.41** k.ft

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CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
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SUBJECT	LFD Load Rating for flexural member			
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	11.54		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.57		
		D/150 =	0.21		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	0.00	in	
		$20{,}000{,}000A_f/F_yd$ =	210.04	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p$ =	124.99	in	*10.48.4.1*
If $L_b \le L_p$ then,		$F_yS_x = M_y = M_r$ =	1286.92	k.ft	*(10-103d)*
		λ =	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
		≤	1.00		
		Use R_b =	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u$ =	1286.92	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr}$ =	145.42	ksi	
		≤	F_y		
		Compression $F_{cr}S_xR_b = M_u$ =	1286.92	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	55.02		*(10-116)*
		$6{,}000*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	343.94	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**343.94**	kips	*(10-113)*

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SUBJECT	LFD Load Rating for flexural member				
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1447.41	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	343.94	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.337	0.563
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	2.953	4.930

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.337	0.563

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1447.41	1447.41
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	123.10	123.10

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.337	0.563
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.324	2.209
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF130,FB24-5, Span 24

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	32.972	in	M_{DL} =	284.97	k.ft	V_{DL} =	31.56	kips
Flange width,	b =	11.510	in	M_{SDL} =	34.38	k-ft	V_{SDL} =	3.75	kips
Flange thickness,	t_f =	0.770	in	M_{PED} =	326.98	k.ft	V_{PED} =	35.67	kips
Web thickness,	t_w =	0.522	in	M_{H-10} =	109.50	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.410	in	Section modulus,			S_x=	372.29	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	421.45	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 14.95 *(10-29)*

b/t limit = 66% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 60.21

D/t_w limit = 57% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 130.17

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 421.45 in^3

Moment capacity, $F_yZ = M_u$= 1158.99 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 899.76 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1158.99** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	14.95		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.52		
		D/150 =	0.21		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	0.00	in	
		$20{,}000{,}000A_f/F_yd$ =	162.91	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p$ =	126.03	in	*10.48.4.1*
If $L_b \le L_p$ then,	$F_yS_x = M_y = M_r$ =	1023.80	k.ft	*(10-103d)*
	λ =	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
	≤	1.00		
	Use R_b =	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u$ =	1023.80	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr}$ =	86.64	ksi	
	≤	F_y		
	Compression $F_{cr}S_xR_b = M_u$ =	1023.80	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	60.21		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	314.04	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**314.04**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1158.99	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	314.04	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.048	1.750
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.464	5.782

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.048	1.750

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1158.99	1158.99
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	127.05	127.05

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.048	1.750
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.841	3.073
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF141, FB25-7, Span 25

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.310	in	M_{DL} =	211.42	k.ft	V_{DL} =	22.60	kips
Flange width,	b =	11.535	in	M_{SDL} =	456.64	k-ft	V_{SDL} =	31.23	kips
Flange thickness,	t_f =	0.960	in	M_{PED} =	221.25	k.ft	V_{PED} =	36.96	kips
Web thickness,	t_w =	0.605	in	M_{H-10} =	122.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.350	in	Section modulus,			S_x=	446.80	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	507.26	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 12.02 *(10-29)*

b/t limit = 53% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 51.88

D/t_w limit = 49% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 108.12

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 507.26 in^3

Moment capacity, $F_yZ = M_u$= 1394.97 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 1078.87 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1394.97** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	12.02		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.61		
		$D/150 =$	0.21		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	201.48	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	122.90	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	1228.70	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	1228.70	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	134.10	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	1228.70	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	51.88		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	363.49	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**363.49**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1394.97	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	363.49	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.097	1.830
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.660	6.109

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.097	1.830

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1394.97	1394.97
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	157.93	157.93

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.097	1.830
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.847	3.083
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF141, FB26-7, Span 26

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.310	in	M_{DL} =	263.35	k.ft	V_{DL} =	27.71	kips
Flange width,	b =	11.535	in	M_{SDL} =	269.30	k-ft	V_{SDL} =	18.91	kips
Flange thickness,	tf =	0.960	in	M_{PED} =	272.60	k.ft	V_{PED} =	28.67	kips
Web thickness,	t_w =	0.605	in	M_{H-10} =	122.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.350	in	Section modulus,			S_x=	446.80	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	507.26	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 12.02 *(10-29)*

b/t limit = 53% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 51.88

D/t_w limit = 49% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 108.12

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 507.26 in^3

Moment capacity, $F_yZ = M_u$= 1394.97 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 1027.18 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1394.97** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	12.02		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.61		
		$D/150 =$	0.21		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	201.48	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	122.90	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	1228.70	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	1228.70	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	134.10	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	1228.70	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	51.88		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	363.49	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**363.49**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1394.97	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	363.49	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.188	1.982
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	4.868	8.126

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.188	1.982

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	1394.97	1394.97
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	134.49	134.49

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.188	1.982
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	2.531	4.225
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 36WF170, FB27-7, Span 27

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	36.160	in	M_{DL} =	425.58	k.ft	V_{DL} =	42.93	kips
Flange width,	b =	12.027	in	M_{SDL} =	176.63	k-ft	V_{SDL} =	13.32	kips
Flange thickness,	t_f =	1.100	in	M_{PED} =	463.83	k.ft	V_{PED} =	46.98	kips
Web thickness,	t_w =	0.612	in	M_{H-10} =	122.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.450	in	Section modulus,			S_x=	560.35	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	640.29	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 10.93 *(10-29)*

b/t limit = 48% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 55.49

D/t_w limit = 52% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 106.66

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 640.29 in^3

Moment capacity, $F_yZ = M_u$= 1760.80 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 1431.51 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1760.80** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	10.93		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.61		
		$D/150 =$	0.23		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	221.74	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	128.12	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	1540.96	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	1540.96	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	161.95	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	1540.96	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	$k =$ 5.0	$D/t_w =$	55.49		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	397.80	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**397.80**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1760.80	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	397.80	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.972	1.622
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.185	5.316

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.972	1.622

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1760.80	1760.80
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	172.18	172.18

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.972	1.622
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.624	2.711
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 36WF150, FB27-6, Span 27

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	35.840	in	M_{DL} =	372.39	k.ft	V_{DL} =	41.17	kips
Flange width,	b =	11.972	in	M_{SDL} =	41.25	k-ft	V_{SDL} =	4.50	kips
Flange thickness,	t_f =	0.940	in	M_{PED} =	426.25	k.ft	V_{PED} =	46.50	kips
Web thickness,	t_w =	0.563	in	M_{H-10} =	109.50	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.540	in	Section modulus,			S_x=	484.99	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	554.93	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 12.74 *(10-29)*

b/t limit = 56% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 60.32

D/t_w limit = 57% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 119.92

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 554.93 in^3

Moment capacity, $F_yZ = M_u$= 1526.06 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 1170.16 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1526.06** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	12.74		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.56		
		$D/150 =$	0.23		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	190.30	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	132.83	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	1333.72	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	1333.72	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	119.35	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	1333.72	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	60.32		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	365.95	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**365.95**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1526.06	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	365.95	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.069	1.784
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.038	5.072

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.069	1.784

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1526.06	1526.06
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	167.19	167.19

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.069	1.784
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.588	2.650
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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SUBJECT	LFD Load Rating for flexural member				
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 36WF160,FB28-6, Span 28

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	36.000	in	M_{DL} =	387.13	k.ft	V_{DL} =	42.81	kips
Flange width,	b =	12.000	in	M_{SDL} =	41.25	k-ft	V_{SDL} =	4.50	kips
Flange thickness,	t_f =	1.020	in	M_{PED} =	433.13	k.ft	V_{PED} =	47.25	kips
Web thickness,	t_w =	0.653	in	M_{H-10} =	109.50	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.510	in	Section modulus,			S_x=	534.54	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	616.43	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 11.76 *(10-29)*

b/t limit = 52% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 52.01

D/t_w limit – 49% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 107.06

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 616.43 in^3

Moment capacity, $F_yZ = M_u$= 1695.18 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 1197.43 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\,(M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1695.18** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	11.76		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.65		
		$D/150 =$	0.23		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	206.06	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	131.26	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	1469.99	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	1469.99	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	139.88	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	1469.99	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	52.01		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	424.45	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**424.45**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1695.18	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	424.45	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.211	2.022
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.540	5.909

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.211	2.022

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	1695.18	1695.18
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	185.68	185.68

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.211	2.022
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	1.884	3.145
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 36WF194, FB28-8, Span 28

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	36.480	in	M_{DL} =	435.42	k.ft	V_{DL} =	45.06	kips
Flange width,	b =	12.117	in	M_{SDL} =	343.13	k-ft	V_{SDL} =	37.63	kips
Flange thickness,	t_f =	1.260	in	M_{PED} =	472.50	k.ft	V_{PED} =	47.25	kips
Web thickness,	t_w =	0.770	in	M_{H-10} =	122.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.490	in	Section modulus,			S_x=	657.15	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	759.73	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 9.62 *(10-29)*

b/t limit = 43% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 44.10

D/t_w limit = 42% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 89.11

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 759.73 in^3

Moment capacity, $F_yZ = M_u$= 2089.26 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 1629.95 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **2089.26** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	9.62		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.77		
		$D/150 =$	0.23		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	253.65	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	130.22	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	1807.16	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	1807.16	k.ft	
Moment capaclty In compression,	$(4{,}400\ l/b)^2 = F_{cr} =$	209.34	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	1807.16	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	44.10		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	500.50	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**500.50**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	2089.26	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	500.50	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.051	1.754
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.833	6.398

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.051	1.754

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	2089.26	2089.26
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	215.21	215.21

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.051	1.754
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	1.880	3.139
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 30WF132, FB29-8, Span 29

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	30.300	in	M_{DL} =	240.59	k.ft	V_{DL} =	24.58	kips
Flange width,	b =	10.551	in	M_{SDL} =	210.00	k-ft	V_{SDL} =	23.00	kips
Flange thickness,	t_f =	1.000	in	M_{PED} =	270.00	k.ft	V_{PED} =	27.00	kips
Web thickness,	t_w =	0.615	in	M_{H-10} =	122.00	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.260	in	Section modulus,			S_x=	375.73	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	432.28	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 10.55 *(10-29)*

b/t limit = 47% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 46.02

D/t_w limit = 43% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 95.39

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 432.28 in^3

Moment capacity, $F_yZ = M_u$= 1188.77 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 937.33 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1188.77** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	10.55		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.62		
		$D/150 =$	0.19		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	211.04	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	118.19	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	1033.26	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	1033.26	k.ft	
Moment capacity in compression,	$(4{,}400\ l/b)^2 = F_{cr} =$	173.91	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	1033.26	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	46.02		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	333.12	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**333.12**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1188.77	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	333.12	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.029	1.718
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	4.630	7.728

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.029	1.718

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1188.77	1188.77
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	122.15	122.15

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.029	1.718
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	2.356	3.932
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
CHECKED BY	Jason A.	DATE	12/23/13

SUBJECT	LFD Load Rating for flexural member				
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 30WF116, FB29-6, Span 29

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	30.000	in	M_{DL} =	217.76	k.ft	V_{DL} =	24.17	kips
Flange width,	b =	10.525	in	M_{SDL} =	27.50	k-ft	V_{SDL} =	3.00	kips
Flange thickness,	t_f =	0.585	in	M_{PED} =	247.50	k.ft	V_{PED} =	27.00	kips
Web thickness,	t_w =	0.930	in	M_{H-10} =	109.50	k.ft	V_{H-10} =	10.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.160	in	Section modulus,			S_x=	327.90	in^3
Unbraced length,	L_b=	0.00	ft	Plastic modulus,			Z_x=	373.09	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 17.99 *(10-29)*

b/t limit = 80% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 31.00

D/t_w limit = 29% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 115.20

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 373.09 in^3

Moment capacity, $F_yZ = M_u$= 1026.00 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.00

approximately 80% of maximum moment, M_1 = 684.73 k.ft

L_b/r_y = 0.00

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 109.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1026.00** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	17.99		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.93		
		$D/150 =$	0.19		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	0.00	in	
		$20{,}000{,}000A_f/F_yd =$	124.39	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	112.96	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	901.73	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	901.73	k.ft	
Moment capacity in compression,	$(1{,}100\ t/b)^2 = F_{cr} =$	59.81	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	901.73	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	31.00		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	513.18	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**513.18**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1026.00	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	513.18	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.317	2.198
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	8.156	13.614

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.317	2.198

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1026.00	1026.00
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	112.47	112.47

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.317	2.198
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	4.652	7.766
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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PROJECT	Passerelle Pedestrian Bridge		
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SUBJECT	LFD Load Rating for flexural member				
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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 24WF130, G1, Span 3

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	24.250	in	M_{DL} =	233.23	k.ft	V_{DL} =	18.75	kips
Flange width,	b =	14.000	in	M_{SDL} =	33.00	k-ft	V_{SDL} =	2.58	kips
Flange thickness,	t_f =	0.900	in	M_{PED} =	300.67	k.ft	V_{PED} =	23.43	kips
Web thickness,	t_w =	0.565	in	M_{H-10} =	96.00	k.ft	V_{H-10} =	5.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	3.300	in	Section modulus,			S_x=	327.37	in^3
Unbraced length,	L_b=	14.67	ft	Plastic modulus,			Z_x=	365.40	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 15.56 *(10-29)*

b/t limit = 69% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 39.73

D/t_w limit = 38% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 112.53

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 365.40 in^3

Moment capacity, $F_yZ = M_u$= 1004.85 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.79

approximately 80% of maximum moment, M_1 = 798.84 k.ft

L_b/r_y = 53.33

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 56.09

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1004.85** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	15.56		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.57		
		D/150 =	0.15		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	176.00	in	
		$20{,}000{,}000A_f/F_yd$ =	314.90	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p$ =	172.58	in	*10.48.4.1*
If $L_b \le L_p$ then,	$F_yS_x = M_y = M_r$ =	n/a	k.ft	*(10-103d)*
	λ =	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
	≤	1.00		
	Use R_b =	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u$ =	900.27	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr}$ =	80.01	ksi	
	≤	F_y		
	Compression $F_{cr}S_xR_b = M_u$ =	900.27	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	39.73		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	242.78	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**242.78**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO Appendix C*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1004.85	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	242.78	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	1.010	1.685
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	4.230	7.060

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	1.010	1.685

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	1004.85	1004.85
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	79.06	79.06

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	1.010	1.685
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	2.320	3.872
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 36WF230, G2, Span 3

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	35.880	in	M_{DL} =	698.31	k.ft	V_{DL} =	52.63	kips
Flange width,	b =	16.475	in	M_{SDL} =	94.60	k-ft	V_{SDL} =	7.03	kips
Flange thickness,	tf =	1.260	in	M_{PED} =	945.85	k.ft	V_{PED} =	70.36	kips
Web thickness,	t_w =	0.765	in	M_{H-10} =	96.00	k.ft	V_{H-10} =	5.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	3.750	in	Section modulus,			S_x=	825.66	in^3
Unbraced length,	L_b=	14.67	ft	Plastic modulus,			Z_x=	931.50	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 13.08 *(10-29)*

b/t limit = 58% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 43.61

D/t_w limit = 41% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 104.80

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 931.50 in^3

Moment capacity, $F_yZ = M_u$= 2561.63 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.96

approximately 80% of maximum moment, M_1 = 2466.62 k.ft

L_b/r_y = 46.93

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 44.90

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	13.08		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.77		
		$D/150 =$	0.22		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	176.00	in	
		$20{,}000{,}000A_f/F_yd =$	350.64	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	196.11	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	2270.57	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	2270.57	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	113.24	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	2270.57	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**2270.57**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	43.61		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	488.46	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**488.46**	kips	*(10-113)*

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PROJECT	Passerelle Pedestrian Bridge		
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SUBJECT	LFD Load Rating for flexural member				
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	2270.57	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	488.46	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.604	1.008
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	2.691	4.492

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.604	1.008

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	2270.57	2270.57
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	169.78	169.78

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.604	1.008
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.412	2.356
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF130, G2, Span 4

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.100	in	M_{DL} =	370.44	k.ft	V_{DL} =	29.89	kips
Flange width,	b =	11.510	in	M_{SDL} =	51.93	k-ft	V_{SDL} =	4.10	kips
Flange thickness,	t_f =	0.855	in	M_{PED} =	513.00	k.ft	V_{PED} =	40.50	kips
Web thickness,	t_w =	0.580	in	M_{H-10} =	81.00	k.ft	V_{H-10} =	5.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.400	in	Section modulus,			S_x=	399.53	in^3
Unbraced length,	L_b=	12.67	ft	Plastic modulus,			Z_x=	460.20	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 13.46 *(10-29)*

b/t limit = 60% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 54.12

D/t_w limit = 51% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 117.12

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 460.20 in^3

Moment capacity, $F_yZ = M_u$= 1265.55 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 1.05

approximately 80% of maximum moment, M_1 = 1329.83 k.ft

L_b/r_y = 63.35

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 39.04

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	b/t =	13.46		*(10-100)*
		≤	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	t_w =	0.58		
		D/150 =	0.21		
		≤	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	L_b =	152.04	in	
		$20{,}000{,}000A_f/F_yd$ =	180.19	in	*(10-101)*
		≥	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p$ =	125.51	in	*10.48.4.1*
If $L_b \le L_p$ then,	$F_yS_x = M_y = M_r$ =	n/a	k.ft	*(10-103d)*
	λ =	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b$ =	n/a		*(10-103b)*
	≤	1.00		
	Use R_b =	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u$ =	1098.71	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr}$ =	106.83	ksi	
	≤	F_y		
	Compression $F_{cr}S_xR_b = M_u$ =	1098.71	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**1098.71**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	D/t_w=	54.12		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy}$=	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy}$=	92.32		
		C =	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p$ =	348.47	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u$=	**348.47**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1098.71	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	348.47	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.494	0.824
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.462	5.779

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.494	0.824

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M$ =	1098.71	1098.71
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V$ =	87.58	87.58

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.494	0.824
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	1.876	3.132
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V}$ =	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$ =	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$ =	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF130, G1, Span 5

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.100	in	M_{DL} =	317.24	k.ft	V_{DL} =	25.69	kips
Flange width,	b =	11.510	in	M_{SDL} =	38.00	k-ft	V_{SDL} =	3.00	kips
Flange thickness,	t_f =	0.855	in	M_{PED} =	385.07	k.ft	V_{PED} =	30.40	kips
Web thickness,	t_w =	0.580	in	M_{H-10} =	81.00	k.ft	V_{H-10} =	5.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.400	in	Section modulus,			S_x=	399.53	in^3
Unbraced length,	L_b=	12.67	ft	Plastic modulus,			Z_x=	460.20	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 13.46 *(10-29)*

b/t limit = 60% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 54.12

D/t_w limit = 51% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 117.12

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 460.20 in^3

Moment capacity, $F_yZ = M_u$= 1265.55 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.82

approximately 80% of maximum moment, M_1 = 1037.93 k.ft

L_b/r_y = 63.35

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 54.41

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	13.46		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.58		
		$D/150 =$	0.21		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	152.04	in	
		$20{,}000{,}000A_f/F_yd =$	180.19	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	125.51	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	n/a	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	1098.71	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	106.83	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	1098.71	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**1098.71**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	54.12		*(10-116)*
		$6{,}000*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	348.47	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**348.47**	kips	*(10-113)*

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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1098.71	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	348.47	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.762	1.272
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	4.717	7.874

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.762	1.272

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1098.71	1098.71
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	87.58	87.58

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.762	1.272
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	2.604	4.347
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

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LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF152, G2, Span 6

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.500	in	M_{DL} =	516.02	k.ft	V_{DL} =	36.06	kips
Flange width,	b =	11.565	in	M_{SDL} =	68.93	k-ft	V_{SDL} =	4.70	kips
Flange thickness,	tf =	1.055	in	M_{PED} =	689.33	k.ft	V_{PED} =	47.00	kips
Web thickness,	t_w =	0.635	in	M_{H-10} =	96.00	k.ft	V_{H-10} =	5.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.480	in	Section modulus,			S_x=	481.25	in^3
Unbraced length,	L_b=	14.67	ft	Plastic modulus,			Z_x=	552.29	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 10.96 *(10-29)*

b/t limit = 48% ; 4,110/√Fy = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 49.43

D/t_w limit = 47% ; 19,230/√Fy = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 100.74

33,650/√Fy = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 552.29 in^3

Moment capacity, $F_yZ = M_u$= 1518.80 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 1.19

approximately 80% of maximum moment, M_1 = 1805.02 k.ft

L_b/r_y = 70.98

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 29.86

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	10.96		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.64		
		$D/150 =$	0.21		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	176.04	in	
		$20{,}000{,}000A_f/F_yd =$	220.73	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	129.69	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	n/a	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	1323.44	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	161.11	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	1323.44	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**1323.44**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	49.43		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	381.51	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**381.51**	kips	*(10-113)*

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge	SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE 11/14/2013	
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1323.44	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	381.51	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.376	0.628
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	3.221	5.377

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.376	0.628

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1323.44	1323.44
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	91.37	91.37

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.376	0.628
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	1.725	2.879
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member		
BY	Tung For Ko	DATE	11/14/2013				
CHECKED BY	Jason A.	DATE	12/23/13		PAGE 1	OF	3

LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 24WF76, G1, Span 6

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	23.910	in	M_{DL} =	127.11	k.ft	V_{DL} =	9.11	kips
Flange width,	b =	8.985	in	M_{SDL} =	18.33	k-ft	V_{SDL} =	1.25	kips
Flange thickness,	t_f =	0.682	in	M_{PED} =	176.00	k.ft	V_{PED} =	12.00	kips
Web thickness,	t_w =	0.440	in	M_{H-10} =	96.00	k.ft	V_{H-10} =	5.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	1.930	in	Section modulus,			S_x=	173.47	in^3
Unbraced length,	L_b=	14.67	ft	Plastic modulus,			Z_x=	198.25	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1 (a) Compression flange, b/t = 13.17 *(10-29)*

b/t limit = 58% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 51.24

D/t_w limit = 48% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 112.90

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 198.25 in^3

Moment capacity, $F_yZ = M_u$= 545.19 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.84

approximately 80% of maximum moment, M_1 = 456.79 k.ft

L_b/r_y = 91.21

$3.6 - 2.2\ (M_1/M_u) \times 10^6 / F_y$ = 53.23

< L_b/r_y *n/a*

Moment capacity for compact section, $F_yZ = M_u$= **n/a** k.ft

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PROJECT	Passerelle Pedestrian Bridge	SUBJECT	LFD Load Rating for flexural member
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	13.17		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.44		
		$D/150 =$	0.15		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	176.04	in	
		$20{,}000{,}000A_f/F_yd =$	155.32	in	*(10-101)*
		$<$	L_b		*n/a*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,		$9{,}500\ r'/\sqrt{F_y} = L_p =$	100.93	in	*10.48.4.1*
If $L_b \leq L_p$ then,		$F_yS_x = M_y = M_r =$	n/a	k.ft	*(10-103d)*
		$\lambda =$	15400		
		$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
		$\leq$	1.00		
		Use $R_b =$	1.00		
Moment capacity in tension,		Tension $F_yS_x = M_u =$	477.04	k.ft	
Moment capacity in compression,		$(4{,}400\ t/b)^2 = F_{cr} =$	111.54	ksi	
		$\leq$	F_y		
		Compression $F_{cr}S_xR_b = M_u =$	477.04	k.ft	
Moment capacity for noncompact section,		Min { Tension or Compression of M_u } =	**477.04**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	51.24		*(10-116)*
		$6{,}000*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	189.87	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**189.87**	kips	*(10-113)*

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge		
BY	Tung For Ko	DATE	11/14/2013
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SUBJECT	LFD Load Rating for flexural member				
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5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	477.04	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	189.87	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.754	1.259
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	6.774	11.308

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.754	1.259

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	477.04	477.04
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	33.10	33.10

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.754	1.259
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	3.858	6.439
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
CHECKED BY	Jason A.	DATE	12/23/13	PAGE	1 OF 3

LFD Load Rating: Passerella Pedestrian Bridge for **NON-COMPOSITE UNSTIFFENED WEB** section only

Reference: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

THE MANUAL FOR BRIDGE EVALUATION (AASHTO), 2ND EDITION, 2011

PROPOSED PROPERTIES INPUT DATA: 33WF152, G2, Span 23-24

Rolled or fabricated flexural member:				Moments and shears:					
Total depth,	d =	33.500	in	M_{DL} =	426.66	k.ft	V_{DL} =	26.85	kips
Flange width,	b =	11.565	in	M_{SDL} =	43.74	k-ft	V_{SDL} =	2.74	kips
Flange thickness,	t_f =	1.055	in	M_{PED} =	440.89	k.ft	V_{PED} =	27.20	kips
Web thickness,	t_w =	0.635	in	M_{H-10} =	0.00	k.ft	V_{H-10} =	0.00	kips
Steel yield strength,	F_y =	33	ksi						
Radius of gyration,	r_y =	2.390	in	Section modulus,			S_x=	481.25	in^3
Unbraced length,	L_b=	5.33	ft	Plastic modulus,			Z_x=	552.29	in^3

1. Moment Capacity for Compact Section *AASHTO*

Section of rolled or fabricated flexural member meeting 10.48.1.1 shall be compact section, *10.48.1*

Section 10.48.1.1

(a) Compression flange, b/t = 10.96 *(10-29)*

b/t limit = 48% ; $4{,}110/\sqrt{Fy}$ = 22.62

≥ b/t *Compact*

(b) Web thickness, $D = d - 2t_f$ D/t_w = 49.43

D/t_w limit = 47% ; $19{,}230/\sqrt{Fy}$ = 105.86

≥ D/t_w *Compact*

(c) When b/t and D/tw exceed 75%, $4.68(b/t) + D/t_w$ = 100.74

$33{,}650/\sqrt{Fy}$ = 185.24

≥ $4.68(b/t)+D/t_w$ *Compact*

If section is compact,

Plastic modulus, Z_x= 552.29 in^3

Moment capacity, $F_yZ = M_u$= 1518.80 k.ft *(10-92)*

(d) Lateral bracing for compression flange, M_1/M_u = 0.83

approximately 80% of maximum moment, M_1 = 1254.60 k.ft

L_b/r_y = 26.78

$3.6 - 2.2\,(M_1/M_u) \times 10^6 / F_y$ = 54.02

≥ L_b/r_y *Braced*

Moment capacity for compact section, $F_yZ = M_u$= **1518.80** k.ft

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PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
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2. Moment Capacity for Braced Noncompact Section *10.48.2*

Section 10.48.2.1	(a) Compression flange,	$b/t =$	10.96		*(10-100)*
		$\leq$	24.00		*Noncompact*
	(b) Web thickness, *(For unstiffened web ONLY)*	$t_w =$	0.64		
		$D/150 =$	0.21		
		$\leq$	t_w		*Noncompact*
	(c) When b/t and D/tw exceed 75%,	$L_b =$	64.00	in	
		$20{,}000{,}000A_f/F_yd =$	220.73	in	*(10-101)*
		$\geq$	L_b		*Braced*

Section of rolled or fabricated flexural member meeting 10.48.2 shall be noncompact section,

$r' = r_y$ of compression flange,	$9{,}500\ r'/\sqrt{F_y} = L_p =$	124.99	in	*10.48.4.1*
If $L_b \leq L_p$ then,	$F_yS_x = M_y = M_r =$	1323.44	k.ft	*(10-103d)*
	$\lambda =$	15400		
	$1 - 0.002\ (D_ct_w/A_{fc})\ [D_c/t_w - \lambda/\sqrt{(M_r/S_x)}\ ] = R_b =$	n/a		*(10-103b)*
	$\leq$	1.00		
	Use $R_b =$	1.00		
Moment capacity in tension,	Tension $F_yS_x = M_u =$	1323.44	k.ft	
Moment capacity in compression,	$(4{,}400\ t/b)^2 = F_{cr} =$	161.11	ksi	
	$\leq$	F_y		
	Compression $F_{cr}S_xR_b = M_u =$	1323.44	k.ft	
Moment capacity for noncompact section,	Min { Tension or Compression of M_u } =	**n/a**	k.ft	

3. Moment Capacity for Partially Braced Section (not included in this calculation) *10.48.4*

4. Shear Capacity for Unstiffened Web, *AASHTO*

The shear capacity of webs of rolled or fabricated flexural member shall be, *10.48.8.1*

For unstiffened beams,	k = 5.0	$D/t_w =$	49.43		*(10-116)*
		$6{,}000^*\sqrt{k}/\sqrt{Fy} =$	73.85		*(10-117)*
		$7{,}500^*\sqrt{k}/\sqrt{Fy} =$	92.32		
		$C =$	1.00		
Plastic shear force,		$0.58F_yDt_w = V_p =$	381.51	kips	*(10-115)*
Shear capacity for unstiffened web,		$CV_p = V_u =$	**381.51**	kips	*(10-113)*

WEIDLINGER ASSOCIATES® INC
CONSULTING ENGINEERS

PROJECT	Passerelle Pedestrian Bridge			SUBJECT	LFD Load Rating for flexural member
BY	Tung For Ko	DATE	11/14/2013		
CHECKED BY	Jason A.	DATE	12/23/13	PAGE	3 OF 3

5. Calculate Load Rating Factor *AASHTO*

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear): *Appendix C*

				Factor	Inventory	Operating	*(C.2.3)*
Moment Capacity, (as built)	M_u =	1518.80	kip.ft	A_1	1.300	1.300	
Shear Capacity, (as built)	V_u =	381.51	kips	A_2	2.170	1.300	

Step 1: Determine the initial load rating factors		Inventory	Operating
Initial moment rating factor,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M$	0.948	1.583
Initial shear rating factor,	$[V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V$	5.812	9.701

Step 2: Determine controlling rating factors		Inventory	Operating
Minimum of RF_M or RF_V from Step 1	RF =	0.948	1.583

Step 3: Determine factored moment and shear using RF from Step 2		Inventory	Operating
Factored moment,	$A_1 (M_DL+M_{SDL}) + RF [A_2 \text{ Max } (M_{PED}, M_{H-10})] = M =$	1518.80	1518.80
Factored shear,	$A_1 (V_DL+V_{SDL}) + RF [A_2 \text{ Max } (V_{PED}, V_{H-10})] = V =$	94.44	94.44

Step 4: Determine final controlling rating factors		Inventory	Operating
Case A: If $V \le 0.6V_u$ and $M \le 0.75 M_u$	Step 1 RF_M =	-	-
	Step 1 RF_V =	-	-
Case B: If $V \le 0.6V_u$ and $M > 0.75 M_u$	Step 1 RF_M =	0.948	1.583
	$[0.6V_u - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	3.226	5.386
Case C: If $V > 0.6V_u$ and $M \le 0.75 M_u$			
	$[0.75M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	Step 1 RF_V =	-	-
Case D: Otherwise			
If $RF_{M_V} \ge [CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10}]$			
	$[2.2V_uM_u - A_1 (V_DL+V_{SDL})M_u - 1.6A_1(M_{DL}+M_{SDL})V_u] /$		
	$[A_2 \text{ Max } (V_{PED}, V_{H-10})M_u + 1.6 A_2 \text{ Max } (M_{PED},M_{H-10})V_u] = RF_{M-V} =$	-	-
Otherwise,	$[M_u - A_1 (M_DL+M_{SDL})] / [A_2 \text{ Max } (M_{PED}, M_{H-10})] = RF_M =$	-	-
	$[CV_p - A_1 (V_DL+V_{SDL})] / [A_2 \text{ Max } (V_{PED}, V_{H-10})] = RF_V =$	-	-

WIND LOAD
AICE 7-05

Passerelle Pedestrian Bridge
Contract No. HBPED700Q
Borough of Queen

HAKS | WEIDLINGER ASSOCIATES® INC
Joint Venture

PROJECT Passerelle Bridge	SUBJECT Interior Canopy Column reaction.
BY Jie Sun DATE 9/10/13	
CHECKED BY Jason A. DATE 9/10/13	PAGE 1 OF 4

ASCE 7-05 — Ref

For $P_g = 25\ lb/ft^2$ @ NYC Region — Fig 7-1

unbalanced Snow Loads for Mutiple folded plate Roof — 7.6.3

Snow density

$\gamma = 0.13 P_g + 14 \leq 30 pcf$ — 7-3

$= 0.13 \times 25 + 14$

$= 17.25\ pcf$

$C_e = 0.9$ Category B, fully exposed roof — Table 7-2

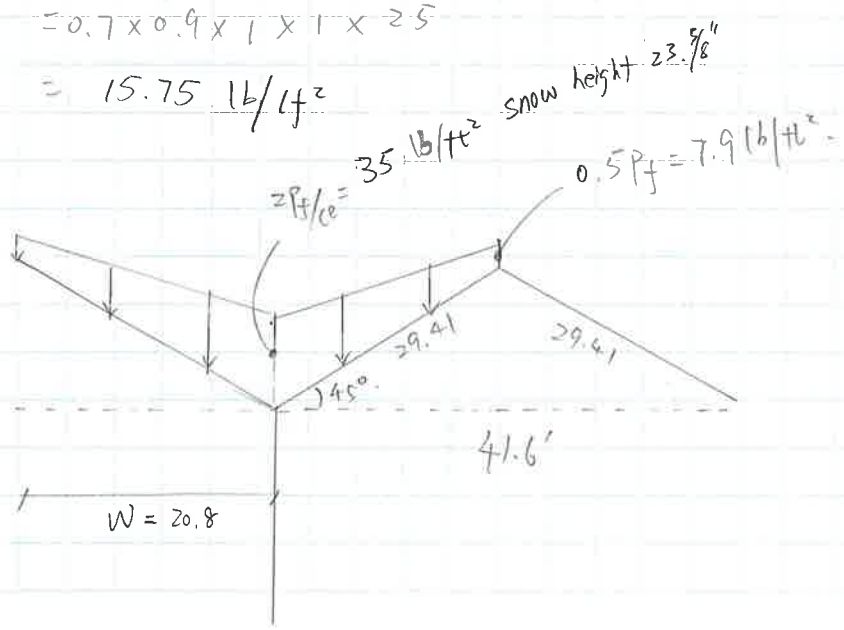
$C_t = 1.0$ — Table 7-3

$I_s = 1.0$ Risk II

$P_f = 0.7 C_e C_t I_s P_g$ — 7-1

$= 0.7 \times 0.9 \times 1 \times 1 \times 25$

$= 15.75\ lb/ft^2$



PROJECT Passerelle Bridge	SUBJECT Interior Canopy Column Reaction.
BY Jie Sun DATE 9/10/13	Canopy #2
CHECKED BY Jason A DATE 9/10/13	PAGE 2 OF 4

Averge the snow load.

$\frac{(35+7.9)}{2} = 21.45\ lb/ft^2$

Area of the roof that covered by snow

Area = (41.6') × 20' (2×W)

= 832 ft²

Tributary Area for Center Col.

= 41.6 / cos 45° × 20'

= 1176.63 ft²

Snow Load.

= 832 × 21.45 lb/ft²

= 17.85 K.

Dead Load (roof plates, wood soffit, metal deck, steel Frame)

① roof plates + wood Soffit assume 9 lb/ft²

② Metal deck (18 gage) 3 lb/ft² Table C3-1

③ steel Framing 30 lb/LF

Area of roof

① 9 × 1176.63 = 10.59 kips

② 3 × 1176.63 = 3.53 kips

③ (10 × 10' + 10 × 10') × 30 = 6 kips.

Total 20.12 kips

Steel Canopy Structure assume 60 lb/lf

(29.41 × 2 + 10') × 60 = 4.13 kips.

total Load at Center Col.

17.85 + 20.12 + 4.13

= 42.1 kips.

use 45K

PROJECT Passerelle Bridge	SUBJECT Exterior Canopy Column reaction.
BY Jie Sun DATE 9/10/13	Canopy #2
CHECKED BY Jason A. DATE 9/10/13	PAGE 3 OF 4

Load on the Canopy side column.

Following the same assumption for center column.

Avrage snow Load is 21.45 lb/ft²

Area of the roof that covered by snow by code.

Area = (41.6' × (10+8))
= 748.8 ft²

Tributary area
= 41.6 / cos 45° × 18
= 1058.96 ≈ 1059 ft²

Snow Load

= 748.8 × 21.45 (b/ft²
= 16.062 k

Dead Load (roof plates, wood soffit, metal deck, steel frame)

① 9 × 1059 = 9.53 kips
② 3 × 1059 = 3.18 kips
③ [(10×10') + (10×8)] × 30 = 5.4 kips

total = 18.11 kips

Steel Canopy structure, (col. & beams) assume 60 lb/LF

(29.41 × 2 + 10') × 60 = 4.13 kips.

total Load at Canopy side column

16.062 + 18.11 + 4.13

= 38.3 kips. use 40K

WEIDLINGER

PROJECT	Passarelle Load Rating	SUBJECT	Span 20 to 27 Canopy Column Reaction Canopy # 1
BY	T.F. Ko	DATE	9/30/2013
CHECKED BY	Jason A.	DATE	10/3/2013
		PAGE	4 OF 4

- Following the same assumption as Span 19 Canopy

Average snow load is 21.45 lb/ft²

- Area of the roof that covered by snow by code:

Area = (35.25)(20.38) = 718.4 ft²

Tributary Area:

= (35.25) / cos45° × 20.38

= 1016 ft²

Snow load : 718.4 × 21.45 = 15.4 kips

Dead Load :

① 9 × 1016 = 9.144 kips
② 3 × 1016 = 3.048 kips
③ [(4)×(20.38) + (4)(20.38)] 30 = 4.89 kips.

} 17.1 kips.

Steel Canopy Structure (Column & Beams), assume 60 lb/f

(35.25/2)(1/cos45) 10' × 60 = 3.6 kips

Total load at Canopy Column :

14.15 + 15.7 + 3.6 = 36.1 kips. ⇒ Use 40 kips

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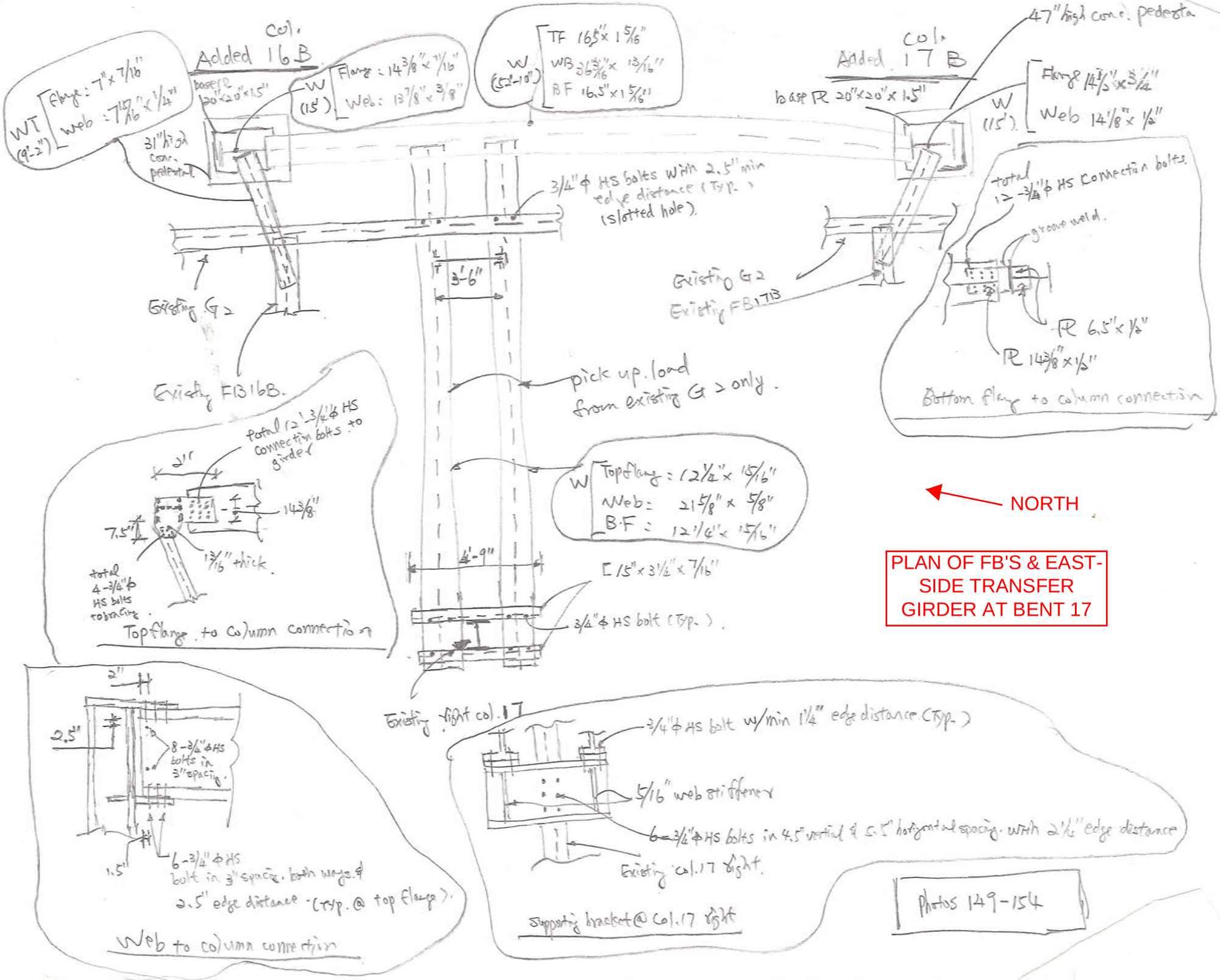
1.0 Field Measurements


Col. Added 16 B
WT (9'-2") Flange: 7" x 7/16" web: 7 15/16" x 1/4"
W (15') Flange: 14 3/8" x 7/16" Web: 13 7/8" x 3/8"
W (52'-10") TF 16.5" x 1 5/16" WB 36 3/16" x 13/16" BF 16.5" x 1 5/16"
base PL 20" x 20" x 1.5"
31" high conc. pedestal
3/4" ϕ HS bolts with 2.5" min edge distance (Typ.) (slotted hole).
Existing G2
Existing FB16B.
3'-6"
pick up load from existing G2 only.
W Top flange: 12 1/2" x 15/16" Web: 21 5/8" x 5/8" B.F: 12 1/2" x 15/16"
4'-9"
C 15" x 3 1/2" x 7/16"
3/4" ϕ HS bolt (Typ.).
Existing right col. 17
Col. Added 17 B
base PL 20" x 20" x 1.5"
47" high conc. pedestal
W (15') Flange 14 3/8" x 3/4" Web 14 1/8" x 1/2"
Existing G2
Existing FB17B
total 12 - 3/4" ϕ HS connection bolts
groove weld.
PL 6.5" x 1/2"
PL 14 3/8" x 1/2"
Bottom flange to column connection
total 12 - 3/4" ϕ HS connection bolts to girder
2"
14 3/8"
7.5"
1 3/16" thick.
total 4 - 3/4" ϕ HS bolts to bracing.
Top flange to column connection
2"
2.5"
8 - 3/4" ϕ HS bolts in 3" spacing.
1.5"
6 - 3/4" ϕ HS bolt in 3" spacing. both wings @ 2.5" edge distance (Typ. @ top flange).
Web to column connection
3/4" ϕ HS bolt w/min 1 1/4" edge distance (Typ.)
5/16" web stiffener
6 - 3/4" ϕ HS bolts in 4.5" vertical & 5.5" horizontal spacing with 2 1/2" edge distance
Existing col. 17 right.
Supporting bracket @ Col. 17 right
NORTH
PLAN OF FB'S & EAST-SIDE TRANSFER GIRDER AT BENT 17
Photos 149-154

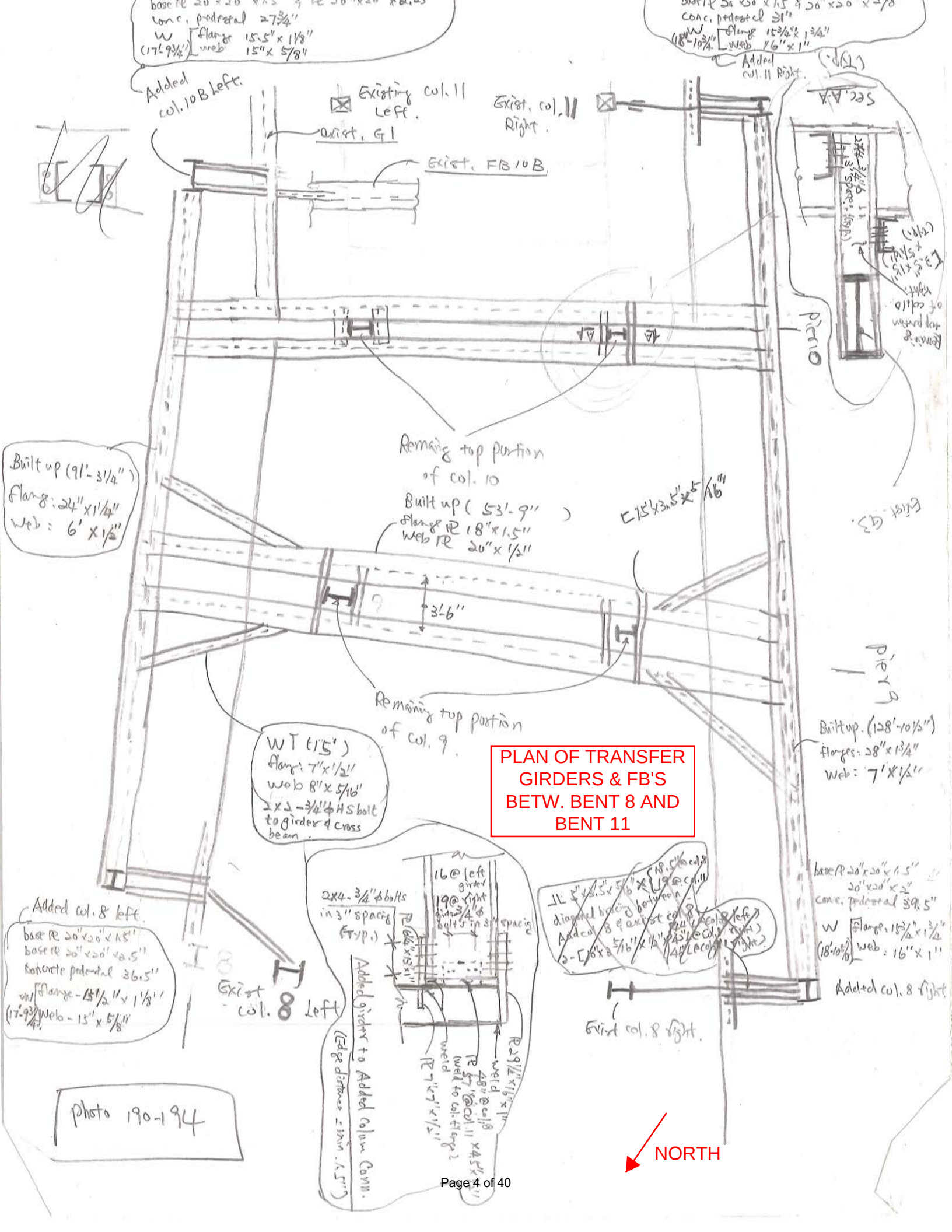
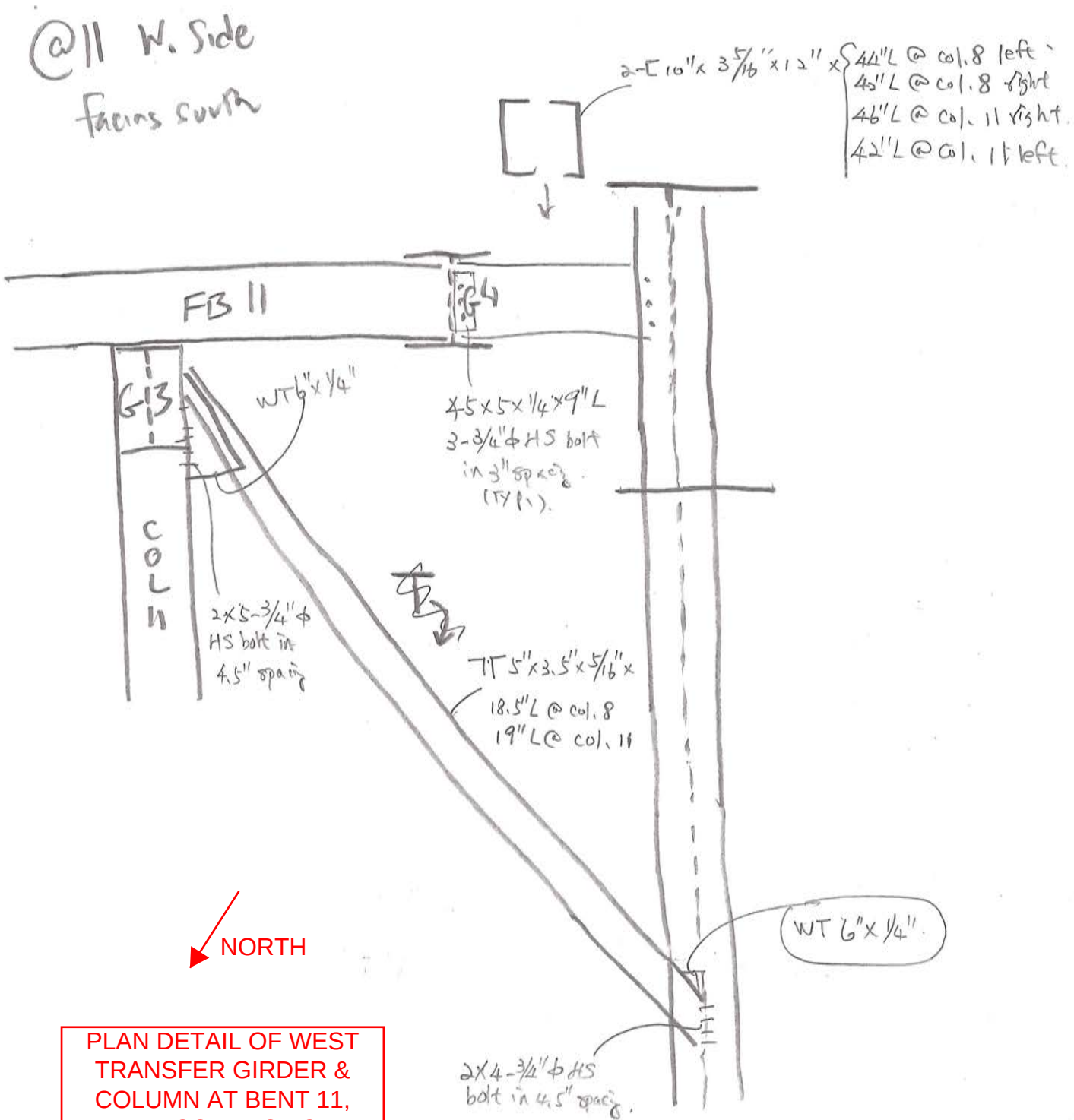
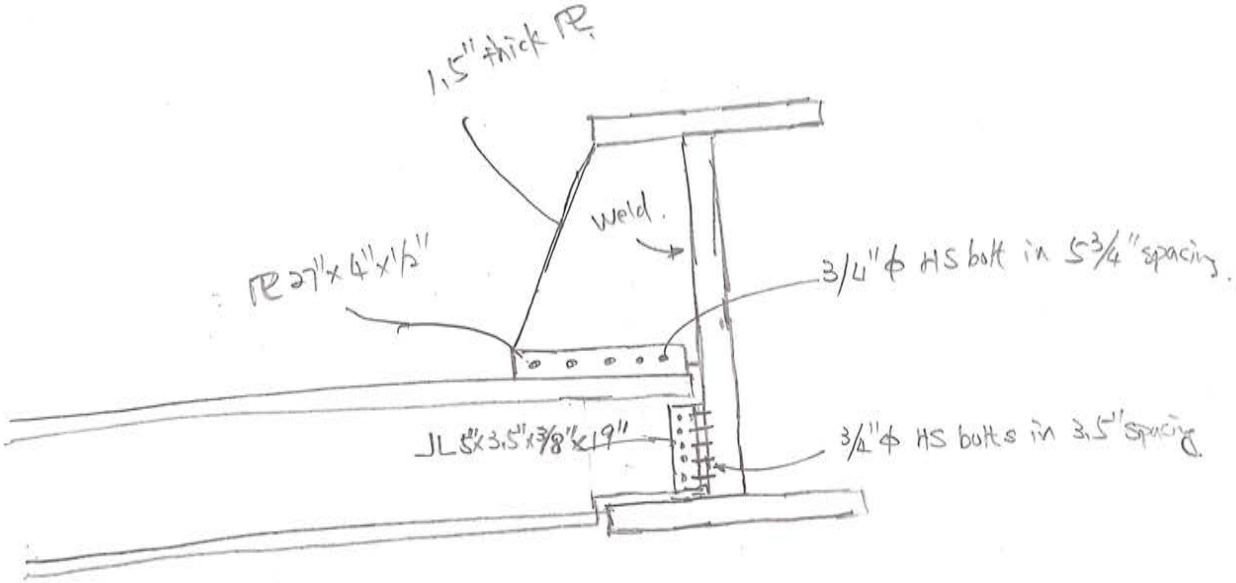


Photo 190-194



PLAN DETAIL OF WEST TRANSFER GIRDER & COLUMN AT BENT 11, FB11 CONNECTION



Added Cross beam @ piers 9 & 10 connecting to added girder

SECTION DETAIL OF TRANSVERSE FB (CROSS BEAM) CONNECTION TO TRANSFER GIRDER

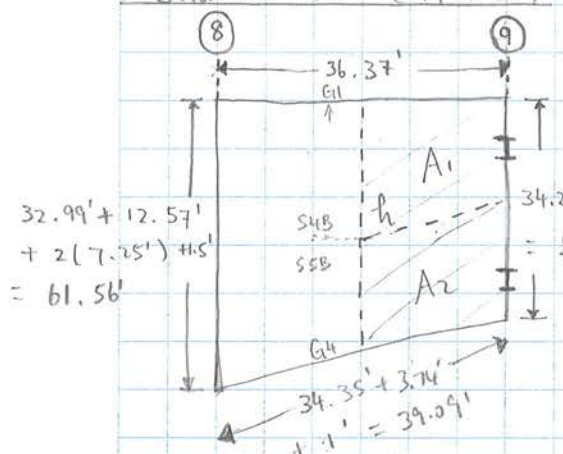
3.5 x 15 x 5/16"

2.0 Tributaries Load Calculations

WEIDLINGER

PROJECT	Passerelle Bridge	SUBJECT	Column 14WF61
BY	Tung For Ko DATE 9/2/2014		Bent 9, Column K
CHECKED BY	Jason A. DATE 9/3/14		PAGE 1 OF 2

Column 9K (14WF61)

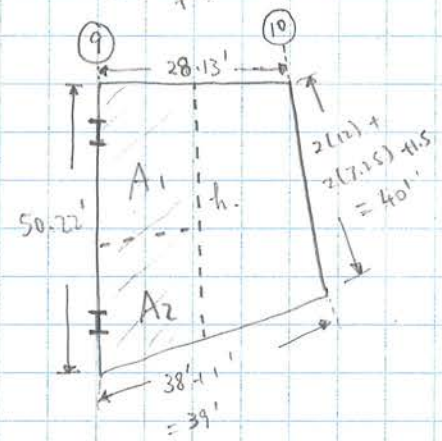


$h = \frac{1}{2}(48.72' + 60.06') = 55.89'$

$A = \frac{1}{2}(55.89 + 50.22)\left(\frac{36.37}{2}\right)$

$= 964.81\ ft^2$

$A_1 \approx 425.32\ ft^2$
$A_2 \approx 539.49\ ft^2$ } From Autocad



→ From Autocad, $A = 781.49\ ft^2$

$A_1 \approx 309.71\ ft^2$
$A_2 \approx 471.78\ ft^2$ } From Autocad.

Colum 9K Tributary Area $= 425.32 + 309.71$
$= 735.03\ ft^2$

Timber plank $= (0.0125)(735.03) = \underline{9.188\ kips}$

Beam Self Weight:

Stringer 10WF21 & 10WF25 $\left\{ \frac{36.37'}{2}\left(4(21)\right) + 14.83'(3(25)) + 0.5(16.21')(25) \right. = 2.84k$

Girder 24WF84, 24WF76 & 24WF100 $\left\{ \frac{36.37'}{2}(76 + 100) + 13.49'(76) + 14.4'(84) \right. = 5.44k$

Floorbeam 24WF76 & 27WF102 & 21WF73 $\left\{ \begin{matrix} 23.19'(76) \\ + \\ 21.26'(102) \end{matrix} + 0.5(20')(73) \right. = 4.66\ k$

Diaphragm ST6WF13.5 $\left\{ 25'(13.5) + 22.4'(13.5) \right. = 0.64\ k$

$\times 1.05$

$= \underline{14.26\ kips}$

WEIDLINGER

PROJECT			
BY	Tung For Ko	DATE	9/2/2014
CHECKED BY	Jason A.	DATE	9/3/14

SUBJECT	Column 14WF61
	Bent 9, Column K
	PAGE 2 OF 2

SDL: (0.01)(735.03) = 7.35 kips

Ped LL: (0.1)(735.03) = 73.5 kips

Column Self Weight: assume about 3' Remain (From Phot.)
(3)(61) = 0.183 kips

H-10 not governs.

DL = 23.45 K
SDL = 7.35 K
LL = 73.5 K

WEIDLINGER

PROJECT	Passerelle Bridge	SUBJECT	Column 14WF61
BY	Tung Fai Ko DATE 9/2/2014		Bent 9, Column L
CHECKED BY	JASON A. DATE 9/3/14		PAGE 1 OF 1

Column 9L (14WF61)

Column 9L Tributary Area: See the calculation in Colum 9K.

$$A = 539.49\ ft^2 + 471.78\ ft^2 = 1011.27\ ft^2$$

Timber Plank: $(0.0125)(1011.27) = \underline{12.64\ kips}$

Beam Self Weight:

Stringer 10WF21 & 10WF25: $\frac{36.37'}{2}[3(21)] + (6.09' + 12.69')(21) + 19.07'(21) + 17.26'[3.5(25)] + 19.52(25) = 3.94\ kips$

Girder 24WF100 & 24WF76 & 24WF130: $19.07'(100 + 76) + 19.11(130) + 20.13(76) = 7.37\ kips$

Floorbeam 24WF76 & 27WF102 & 21WF73: $29.35'(76) + 27.48'(102) + 23.11'(73)(0.5) = 5.88\ kips$

Diaphragm STL WF 13.5: $(13.5)(21.4') + (29.11')(13.5) = 0.68\ kips$

$\times 1.05 = \underline{18.76\ kips}$

SDL: $(0.01)(1011.27\ ft^2) = \underline{10.11\ kips}$

Ped LL: $(0.1)(1011.27\ ft^2) = \underline{101.13\ kips}$

H-10 not govern.

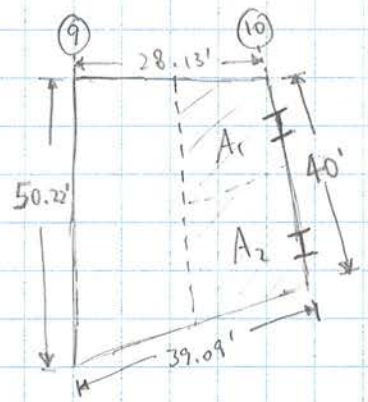
Column Self weight: $(3')(61) = \underline{0.183\ kips}$

DL: 31.4 k
SDL: 10.11 k
LL: 101.13 k

WEIDLINGER

PROJECT Passerelle Bridge	SUBJECT Column 14WF68
BY Tung Fer Ko DATE 9/2/2014	Bent 10, Column K
CHECKED BY Jason A. DATE 9/3/14	PAGE 1 OF 2

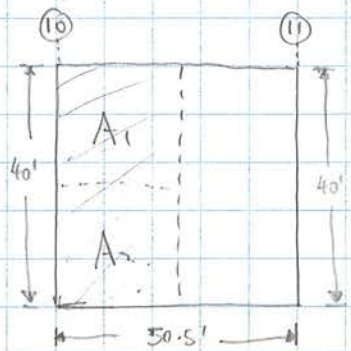
Column 10 K (14 WF 68)



From Autocad : $A = 700.20\ ft^2$

$A_1 = 299.24\ ft^2$

$A_2 = 400.96\ ft^2$



$A_1 = (40'/2)(50.5'/2) = 505\ ft^2$

$A_2 = A_1 = 505\ ft^2$

Column 10K Tributary Area :

$A = 299.24 + 505 = 804.24\ ft^2$

Timber Plank : $(0.0125)(804.24 ft^2) = \underline{10.05\ kips}$

Beam self Weight :

Stringer 10WF25 $\left\{ 15.43'\,[3.5(25)] + \frac{50.5'}{2}[3.5(25)] = 4.04\ kips \right.$

Girder 24WF76 & 24WF84 & 36WF160 $\left\{ 13.49'(76) + 14.7'(84) + \frac{50.5'}{2}(76) + 50.5'/2\,(160) = 8.22\ kips \right.$

Floorbeam 21WF73 & 21WF62 $\left\{ 0.5(20')(73) + (2)(38.5'/2)(62) = 3.12\ kips \right.$

Diaphragm 5T6 WF13.5 $\left\{ 15.82'(13.5) + 27.96'(13.5) = 0.59\ kips \right.$

$\times 1.05$

$= \underline{16.77\ kips}$

WEIDLINGER

PROJECT		SUBJECT	Column 14WF68
BY Tung Fan Ko	DATE 9/2/2014		Bent 10 , Column k
CHECKED BY JWONA.	DATE 9/3/14		PAGE 2 OF 2

SDL = (0.01) (804.24 ft²) = 8.04 kips

Ped LL = (0.1) (804.24 ft²) = 80.42 kips

H-10 not governs

Column Selfweight = (3') (68) = 0.204 kips

DL = 26.82 k
SDL = 8.04 k
LL = 80.42 k

WEIDLINGER

PROJECT	Passerelle Bridge		
BY	Tung Fai Ko	DATE	9/2/2014
CHECKED BY	Jason A.	DATE	9/3/14

SUBJECT	Column 14WF68		
Bent 10 , Column L			
	PAGE 1	OF	1

Column 10 L (14WF68)

Column 10 L Tributary Area:

$A = 400.96\ ft^2 + 505\ ft^2 = 905.96\ ft^2$

Timber Plank: $(0.0125)(905.96\ ft^2) = 11.32$ kips

Beam Self weight:

Stringer 10WF25: $\left\{\left[\frac{17.41' + 18.27'}{2}\right](3.5(25)) + \frac{50.5'}{2}[3.5(25)] = 3.77 \text{ kips}\right.$

Girder 24WF130 & 24WF76 & 36WF160: $\left\{(18.89')(130) + (20.13')(76) + \frac{50.5'}{2}(76) + \frac{50.5'}{2}(160) = 9.94 \text{ kips}\right.$

Floorbeam 21WF73 & 21WF62: $\left\{(23.11')(73)(0.5) + (2)(38.5'/2)(62) = 3.23 \text{ kips}\right.$

Diaphragm ST6WF13.5: $\left\{(20.43')(13.5) + (27.96')(13.5) = 0.65 \text{ kips}\right.$

$\times 1.05$

$= 18.47$ kips

SDL: $(0.01)(905.96\ ft^2) = 9.06$ kips

Ped LL: $(0.1)(905.96\ ft^2) = 90.6$ kips

H-10 not governs.

Column Self weight: $(3')(68) = 0.204$ kips

DL = 29.79 k
SDL = 9.06 k
LL = 90.6 k

WEIDLINGER

PROJECT		SUBJECT	Girder 36WF160
BY Tung Fou Ko	DATE 9/5/2014		@ Span 17, G2
CHECKED BY Jason A.	DATE 9/5/2014	PAGE 1	OF

Span 17 Girder 2, 36WF160

Load path : timber → Stringer → FB → Girder
↳ 2 intermediate FB is supported by the Girder
↳ 1 FB is transfer the load directly to the Column.

Each ½ FB Load :

Tributary Area : (16.83') (20') = 336.6 ft²
Dead Load : Stringer : (3.5) (16.83') (25) = 1.47 kips
FB : (19.25') (62) = 1.19 kips } ×1.05 = 2.793 kips.
Timber : (336.6 ft²) (0.0125) = 4.208 kips
SDL : (336.6 ft²) (0.01) = 3.37 kips.
LL : (336.6 ft²) (0.1) = 33.67 kips.

H-10 not governs
Diaphragm load transfer to Column directly : 0.7875k

FB Load : DL = 7.00 kip
SDL = 3.37 kip
LL = 33.67 kip

→ Apply FB Load @ 1/3 of the span of the Girder
→ Apply FB Load @ the exist. Column location with add. diaphragm load.

WEIDLINGER

PROJECT	Passerelle Bridge		
BY	Jason A.	DATE	9/2/14
CHECKED BY	TE	DATE	9/3/14.

SUBJECT	Transverse Girder		
span 16~17			
		PAGE	OF

Calculate loads @ Col 17 E

Deadload $[25k/_1 \times 16.83' \times 2 1/_2 + 160\ k/_1 \times 50.5' + 76\ k/_1 \times 50.5' + 62\ k/_1 \times 38.5'/_2 \times 3] \times 1.05$ = 21.7K

Add Diaphragm (55.9')(13.5) = 0.75K (13.216k)

Timber 0.0125 (20' x 50.5') = 12.65 K

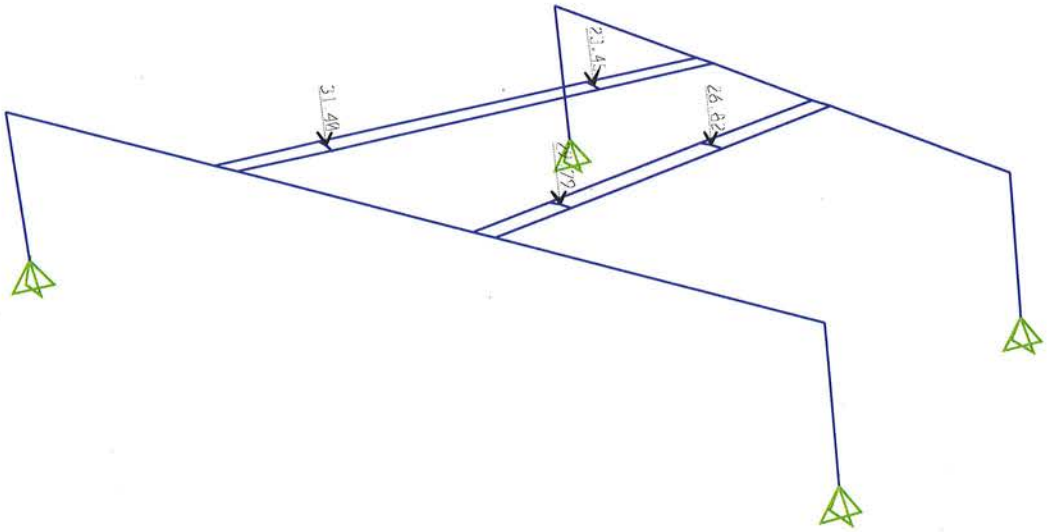
SDL 0.010 " " = 10.1 K

Ped. LL 0.100 " " = 101 K

H10 not governed

Use DL, SDL, LL for SAP 2000 model

3.0 Loads Inputs in SAP2000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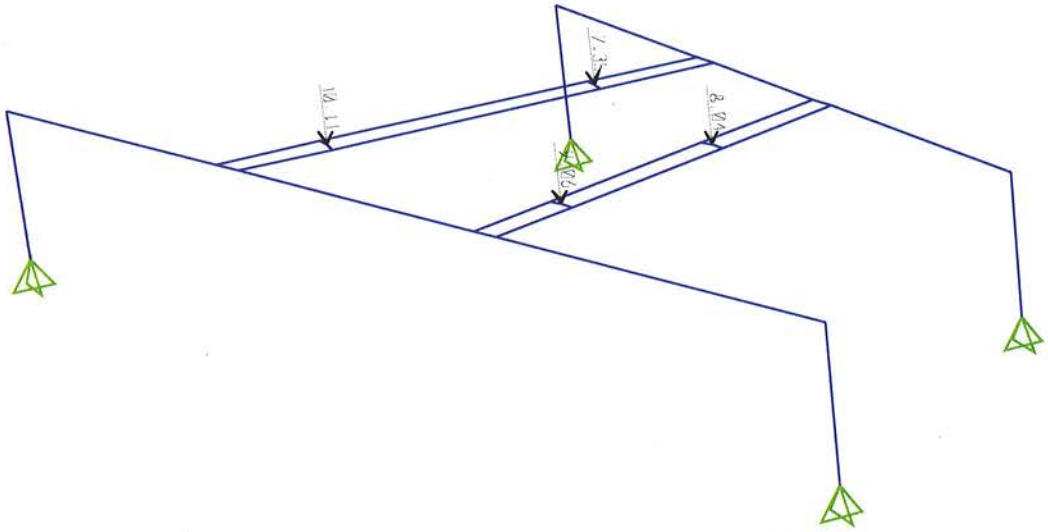
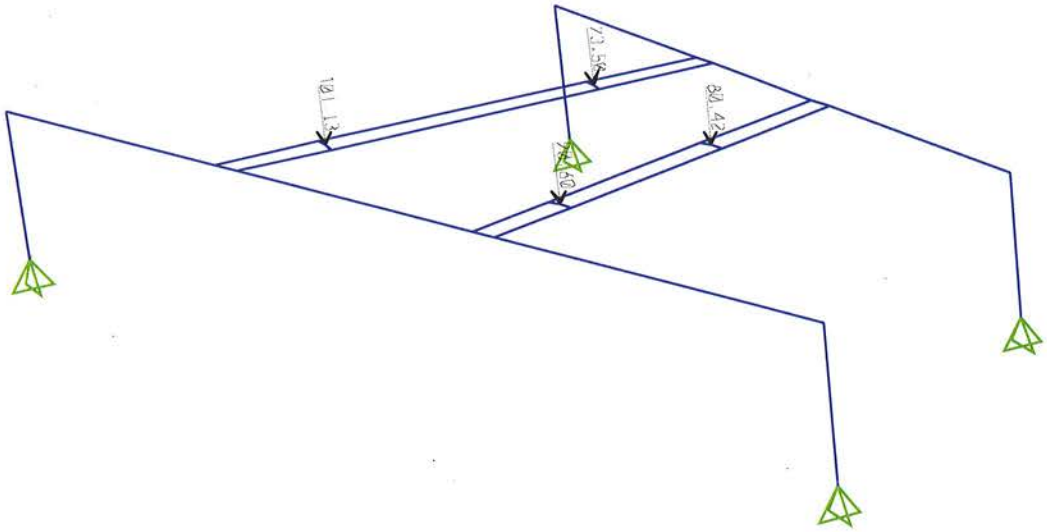
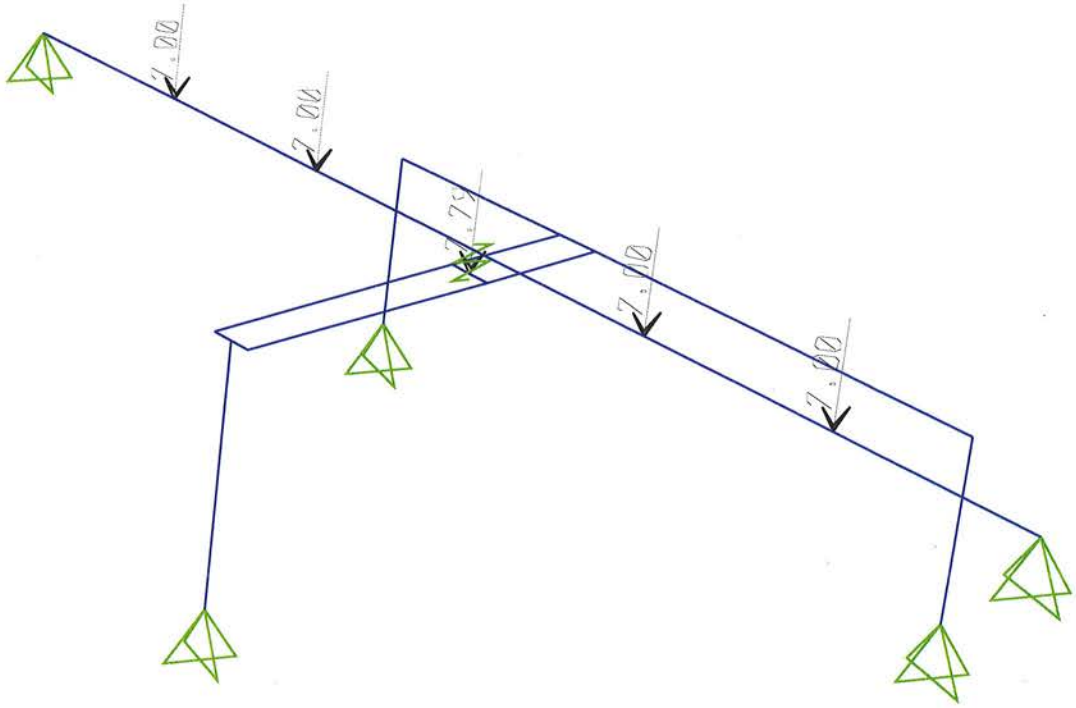
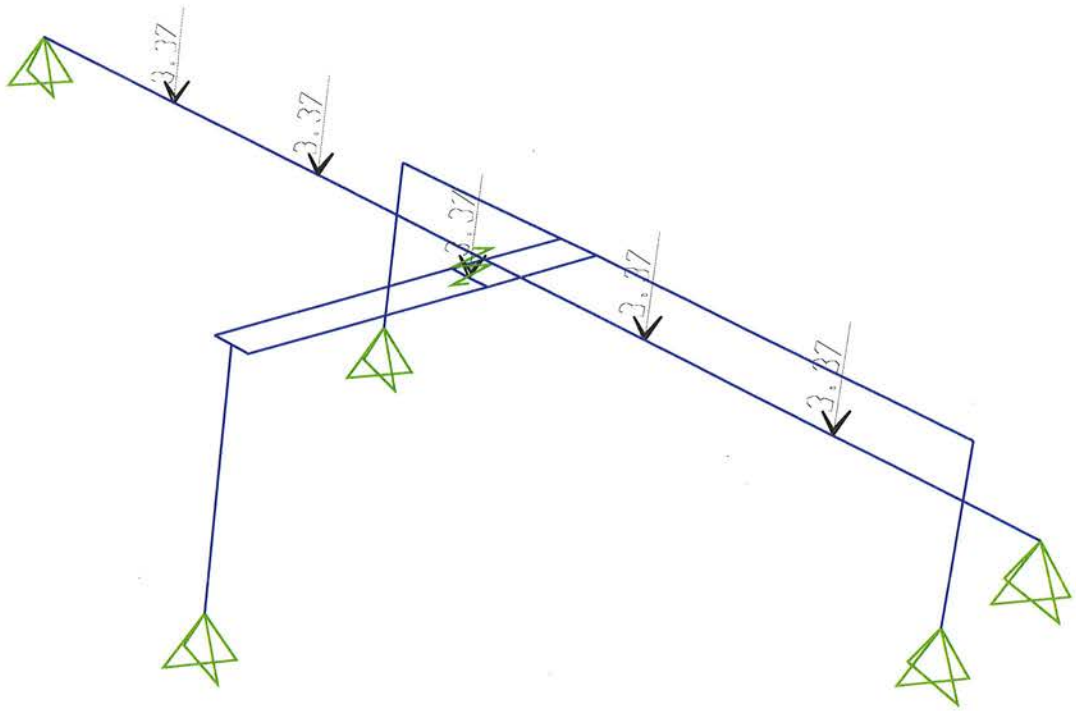
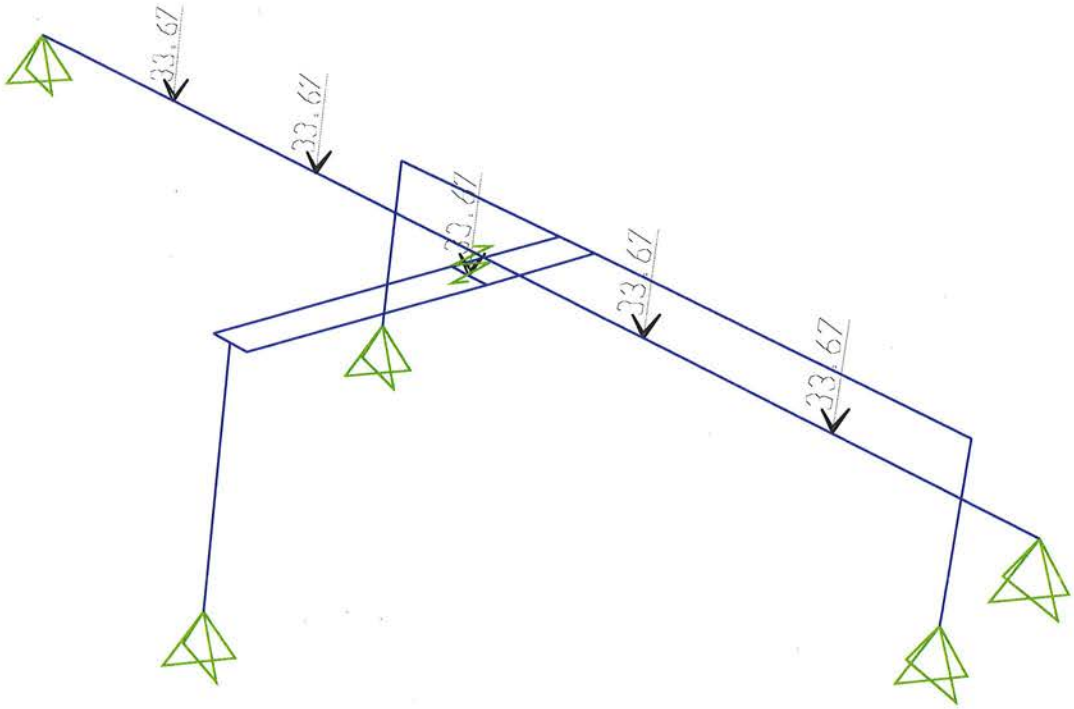
23.4
31.46


1.35
8.24
10.11


71.5K
84.42
10.13
74.10


2.00
2.00
2.75
2.00
2.00


3.37
3.37
3.37
3.37
3.37


33.67
33.67
33.67
33.67
33.67

4.0 SAP2000 Analysis Report

License #

SAP2000 Analysis Report

Prepared by

Weidlinger Associates, Inc.

Model Name: Tgirders_Tung.sdb

17 October 2014

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1. Model geometry

This section provides model geometry information, including items such as joint coordinates, joint restraints, and element connectivity.

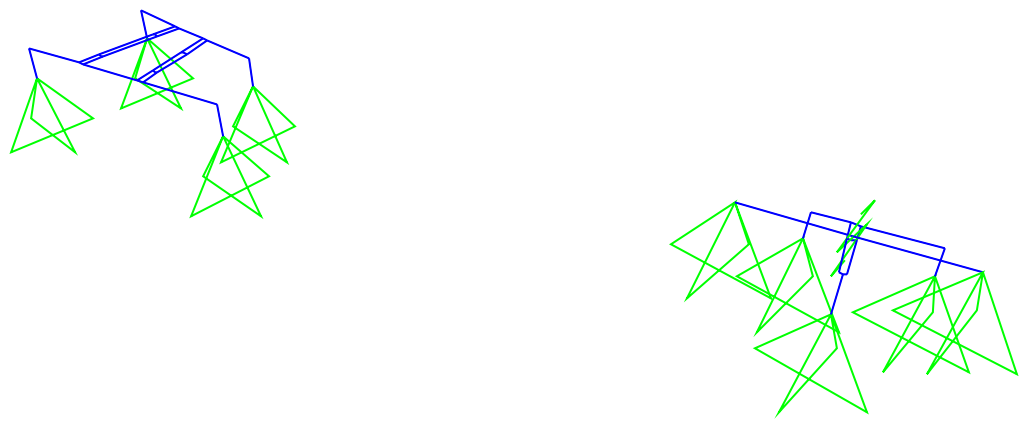


Figure 1: Finite element model

1.1. Joint coordinates

1.2. Joint restraints

1.3. Element connectivity

2. Material properties

This section provides material property information for materials used in the model.

Table 1: Material Properties 02 - Basic Mechanical Properties

Table 1: Material Properties 02 - Basic Mechanical Properties

Material	UnitWeight	UnitMass	E1	G12	U12	A1
	Lb/ft3	Lb-s2/ft4	Lb/ft2	Lb/ft2		1/F
33ksi	4.9000E+02	1.5230E+01	4176000000	1606153846	0.300000	6.5000E-06
4000Psi	1.5000E+02	4.6621E+00	519119496.	2162997900.0	0.200000	5.5000E-06

Table 1: Material Properties 02 - Basic Mechanical Properties

Material	UnitWeight	UnitMass	E1	G12	U12	A1
	Lb/ft3	Lb-s2/ft4	Lb/ft2	Lb/ft2		1/F
A992Fy50	4.9000E+02	1.5230E+01	4176000000	1606153846	0.300000	6.5000E-06

Table 2: Material Properties 03a - Steel Data

Table 2: Material Properties 03a - Steel Data

Material	Fy	Fu	FinalSlope
	Lb/ft2	Lb/ft2	
33ksi	4752000.00	4752000.00	-0.100000
A992Fy50	7200000.00	9360000.00	-0.100000

Table 3: Material Properties 03b - Concrete Data

Table 3: Material Properties 03b - Concrete Data

Material	Fc	FinalSlope
	Lb/ft2	
4000Psi	576000.00	-0.100000

3. Section properties

This section provides section property information for objects used in the model.

3.1. Frames

Table 4: Frame Section Properties 01 - General, Part 1 of 4

Table 4: Frame Section Properties 01 - General, Part 1 of 4

SectionName	Material	Shape	t3	t2	tf	tw	t2b	tfb
			ft	ft	ft	ft	ft	ft
C16B	A992Fy50	I/Wide Flange	1.27083	1.19792	0.05729	0.03125	1.19792	0.05729
C17B	A992Fy50	I/Wide Flange	1.30208	1.23958	0.06250	0.04167	1.17708	0.06250
C8-10B-K	A992Fy50	I/Wide Flange	1.43750	1.29167	0.09375	0.05208	1.29167	0.09375
C8-10B-L	A992Fy50	I/Wide Flange	1.62500	1.31250	0.14583	0.08333	1.31250	0.14583
FB08-10E	A992Fy50	I/Wide Flange	6.20833	2.00000	0.10417	0.04167	2.00000	0.10417
FB08-10W	A992Fy50	I/Wide Flange	7.29167	2.33333	0.14583	0.04167	2.33333	0.14583
FB09	A992Fy50	I/Wide Flange	1.91667	1.50000	0.12500	0.04167	1.50000	0.12500
FB10	A992Fy50	I/Wide Flange	1.91667	1.50000	0.12500	0.04167	1.50000	0.12500
FB16-17E	A992Fy50	I/Wide Flange	3.23438	1.37500	0.10938	0.06771	1.37500	0.10938
FB17	A992Fy50	I/Wide Flange	1.95833	1.02083	0.07813	0.05208	1.02083	0.07813
W14X68	A992Fy50	I/Wide Flange	1.16667	0.83333	0.06000	0.03458	0.83333	0.06000
W36X160	33ksi	I/Wide Flange	3.00000	1.00000	0.08500	0.05417	1.00000	0.08500

Table 4: Frame Section Properties 01 - General, Part 2 of 4

Table 4: Frame Section Properties 01 - General, Part 2 of 4

SectionName	Area	TorsConst	I33	I22	I23	AS2	AS3
	ft2	ft4	ft4	ft4	ft4	ft2	ft2
C16B	0.1734	0.000157	0.054599	0.016417	0.000000	0.0397	0.1144
C17B	0.2001	0.000218	0.063704	0.018422	0.000000	0.0543	0.1259
C8-10B-K	0.3073	0.000734	0.117982	0.033687	0.000000	0.0749	0.2018
C8-10B-L	0.4939	0.002771	0.226531	0.055019	0.000000	0.1354	0.3190
FB08-10E	0.6667	0.001602	4.631715	0.138925	0.000000	0.2587	0.3472
FB08-10W	0.9722	0.004803	9.879969	0.308813	0.000000	0.3038	0.5671
FB09	0.4444	0.001890	0.317507	0.070323	0.000000	0.0799	0.3125
FB10	0.4444	0.001890	0.317507	0.070323	0.000000	0.0799	0.3125
FB16-17E	0.5050	0.001447	0.889366	0.047467	0.000000	0.2190	0.2507
FB17	0.2534	0.000392	0.166452	0.013873	0.000000	0.1020	0.1329
W14X68	0.1389	0.000145	0.034819	0.005835	0.000000	0.0403	0.0833
W36X160	0.3264	0.000598	0.470679	0.014226	0.000000	0.1625	0.1417

Table 4: Frame Section Properties 01 - General, Part 3 of 4

Table 4: Frame Section Properties 01 - General, Part 3 of 4

SectionName	S33	S22	Z33	Z22	R33	R22
	ft3	ft3	ft3	ft3	ft	ft
C16B	0.085926	0.027410	0.093731	0.041389	0.56114	0.30770
C17B	0.096064	0.029722	0.107955	0.046169	0.56425	0.30343
C8-10B-K	0.164149	0.052160	0.183065	0.079054	0.61963	0.33110
C8-10B-L	0.278808	0.083838	0.320159	0.127925	0.67723	0.33375
FB08-10E	1.492096	0.138925	1.646701	0.210938	2.63582	0.45649
FB08-10W	2.709934	0.264697	2.941985	0.400029	3.18783	0.56359
FB09	0.331312	0.093763	0.364873	0.141348	0.84522	0.39778
FB10	0.331312	0.093763	0.364873	0.141348	0.84522	0.39778
FB16-17E	0.549946	0.069042	0.623905	0.106850	1.32712	0.30659
FB17	0.169993	0.027180	0.192237	0.041929	0.81054	0.23400
W14X68	0.059689	0.014005	0.066551	0.021354	0.50069	0.20497
W36X160	0.313786	0.028453	0.361111	0.044734	1.20087	0.20878

Table 4: Frame Section Properties 01 - General, Part 4 of 4

Table 4: Frame Section Properties 01 - General, Part 4 of 4

SectionName	AMod	A2Mod	A3Mod	JMod	I2Mod	I3Mod	MMod	WMod
C16B	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
C17B	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
C8-10B-K	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
C8-10B-L	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
FB08-10E	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
FB08-10W	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
FB09	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
FB10	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
FB16-17E	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
FB17	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
W14X68	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
W36X160	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000

3.2. Solids

Table 5: Solid Property Definitions

Table 5: Solid Property Definitions

SolidProp	Material	MatAngleA Degrees	MatAngleB Degrees	MatAngleC Degrees
SOLID1	4000Psi	0.000	0.000	0.000

4. Load patterns

This section provides loading information as applied to the model.

4.1. Definitions

Table 6: Load Pattern Definitions

Table 6: Load Pattern Definitions

LoadPat	DesignType	SelfWtMult	AutoLoad
DEAD	DEAD	1.000000	
DL	DEAD	0.000000	
SDL	DEAD	0.000000	
LL	DEAD	0.000000	

5. Load cases

This section provides load case information.

5.1. Definitions

Table 7: Load Case Definitions

Table 7: Load Case Definitions

Case	Type	InitialCond	ModalCase	BaseCase	DesActOpt	DesignAct
DEAD	LinStatic	Zero			Prog Det	Non-Composite
MODAL	LinModal	Zero			Prog Det	Other
DL	LinStatic	Zero			Prog Det	Non-Composite
SDL	LinStatic	Zero			Prog Det	Non-Composite
LL	LinStatic	Zero			Prog Det	Non-Composite

5.2. Static case load assignments

Table 8: Case - Static 1 - Load Assignments

Table 8: Case - Static 1 - Load Assignments

Case	LoadType	LoadName	LoadSF
DEAD	Load pattern	DEAD	1.050000
DL	Load pattern	DL	1.000000
SDL	Load pattern	SDL	1.000000
LL	Load pattern	LL	1.000000

5.3. Response spectrum case load assignments

Table 9: Function - Response Spectrum - User

Table 9: Function - Response Spectrum - User

Name	Period Sec	Accel	FuncDamp
UNIFRS	0.000000	1.000000	0.050000
UNIFRS	1.000000	1.000000	

6. Load combinations

This section provides load combination information.

Table 10: Combination Definitions

Table 10: Combination Definitions

ComboName	ComboType	CaseName	ScaleFactor
TOTAL DEAD LOAD	Linear Add	DEAD	1.000000
TOTAL DEAD LOAD		DL	1.000000
COMB1	Linear Add	DEAD	1.000000
COMB1		DL	1.000000
COMB1		LL	1.000000
COMB1		SDL	1.000000

7. Structure results

This section provides structure results, including items such as structural periods and base reactions.

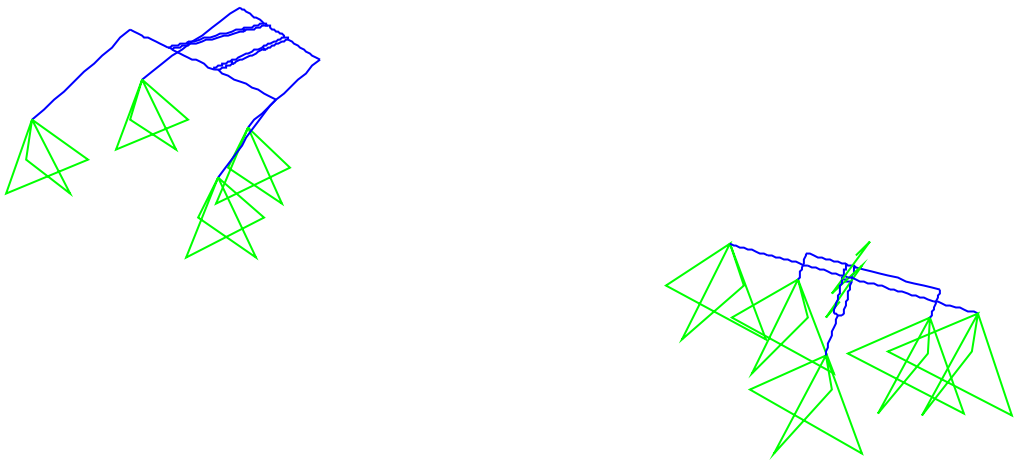


Figure 2: Deformed shape

7.1. Mass summary

7.2. Modal results

Table 11: Modal Participating Mass Ratios

Table 11: Modal Participating Mass Ratios

OutputCase	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
		Sec						
MODAL	1.000000	10.756396	0.74	4.240E-02	8.619E-11	0.74	4.240E-02	8.619E-11
MODAL	2.000000	2.808896	0.17	1.829E-02	6.425E-08	0.92	6.069E-02	6.433E-08
MODAL	3.000000	1.220469	1.446E-02	1.550E-04	1.715E-08	0.93	6.085E-02	8.148E-08
MODAL	4.000000	1.012459	9.583E-04	3.899E-04	5.794E-07	0.93	6.124E-02	6.608E-07
MODAL	5.000000	0.729965	4.235E-02	0.72	1.340E-06	0.97	0.78	2.001E-06
MODAL	6.000000	0.648844	4.586E-04	3.329E-02	1.183E-07	0.97	0.82	2.119E-06
MODAL	7.000000	0.466218	2.615E-05	3.176E-05	1.715E-08	0.97	0.82	2.137E-06
MODAL	8.000000	0.451951	1.571E-11	1.533E-10	8.239E-17	0.97	0.82	2.137E-06
MODAL	9.000000	0.421325	5.376E-05	9.558E-07	6.263E-07	0.97	0.82	2.763E-06
MODAL	10.000000	0.411561	2.641E-04	4.757E-04	4.985E-07	0.97	0.82	3.261E-06
MODAL	11.000000	0.366282	2.366E-04	1.064E-04	3.379E-07	0.98	0.82	3.599E-06
MODAL	12.000000	0.304256	4.031E-03	1.110E-03	2.915E-08	0.98	0.82	3.628E-06

7.3. Base reactions

Table 12: Base Reactions

Table 12: Base Reactions

OutputCase	StepType	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
		Lb	Lb	Lb	Lb-ft	Lb-ft	Lb-ft
DEAD		7.647E-10	-1.809E-09	213626.26	2029572644	-1117088742	-1.670E-05
DL		2.762E-09	-7.729E-09	147247.50	1397762916	-770651047	-6.654E-05
SDL		9.370E-10	-2.585E-09	51410.00	486509643.	-269594951	-2.238E-05
LL		9.378E-09	-2.581E-08	514000.00	4864191278	-2695410350	-2.237E-04

8. Joint results

This section provides joint results, including items such as displacements and reactions.

9. Frame results

This section provides frame force results.

10. Material take-off

This section provides a material take-off.

11. Design preferences

This section provides the design preferences for each type of design, which typically include material reduction factors, framing type, stress ratio limit, deflection limits, and other code specific items.

11.1. Steel design

Table 13: Preferences - Steel Design - AISC 360-10, Part 1 of 4

Table 13: Preferences - Steel Design - AISC 360-10, Part 1 of 4

THDesign	FrameType	PatLLF	SRatioLimit	MaxIter	SDC	SeisCode	SeisLoad	ImpFactor
Envelopes	SMF	0.750000	0.950000	1	D	Yes	Yes	1.000000

Table 13: Preferences - Steel Design - AISC 360-10, Part 2 of 4

Table 13: Preferences - Steel Design - AISC 360-10, Part 2 of 4

SystemRho	SystemSds	SystemR	SystemCd	Omega0	Provision	AMethod	SOMethod	SRMethod
1.000000	0.500000	8.000000	5.500000	3.000000	LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Table 13: Preferences - Steel Design - AISC 360-10, Part 3 of 4

Table 13: Preferences - Steel Design - AISC 360-10, Part 3 of 4

NLCoeff	PhiB	PhiC	PhiTY	PhiTF	PhiV	PhiVRolledI	PhiVT	PlugWeld
0.002000	0.900000	0.900000	0.900000	0.750000	0.900000	1.000000	0.900000	Yes

Table 13: Preferences - Steel Design - AISC 360-10, Part 4 of 4

Table 13: Preferences - Steel Design - AISC 360-10, Part 4 of 4

HSSWelding	HSSReduceT	CheckDefl	DLRat	SDLAndLLRat	LLRat	TotalRat	NetRat
ERW	No	No	120.000000	120.000000	360.000000	240.000000	240.000000

11.2. Concrete design

Table 14: Preferences - Concrete Design - ACI 318-11, Part 1 of 2

Table 14: Preferences - Concrete Design - ACI 318-11, Part 1 of 2

THDesign	NumCurves	NumPoints	MinEccen	PatLLF	UFLimit	SeisCat	PhiT	PhiCTied
Envelopes	24	11	Yes	0.750000	0.950000	D	0.900000	0.650000

Table 14: Preferences - Concrete Design - ACI 318-11, Part 2 of 2

Table 14: Preferences - Concrete Design - ACI 318-11, Part 2 of 2

PhiCSpiral	PhiV	PhiVSeismic	PhiVJoint
0.750000	0.750000	0.600000	0.850000

11.3. Aluminum design

Table 15: Preferences - Aluminum Design - AA-ASD 2000

Table 15: Preferences - Aluminum Design - AA-ASD 2000

FrameType	SRatioLimit	LatFact	UseLatFact
Moment Frame	1.000000	1.333333	No

11.4. Cold formed design

Table 16: Preferences - Cold Formed Design - AISI-ASD96

Table 16: Preferences - Cold Formed Design - AISI-ASD96

FrameType	SRatioLimit	OmegaBS	OmegaBUS	OmegaBLTB	OmegaVS	OmegaVNS	OmegaT	OmegaC
Braced Frame	1.000000	1.670000	1.670000	1.670000	1.670000	1.500000	1.670000	1.800000

12. Design overwrites

This section provides the design overwrites for each type of design, which are assigned to individual members of the structure.

12.1. Steel design

5.0 Calculations

Load Rating Summary for Transfer Girder

REFERENCE: AASHTO MANUAL FOR BRIDGE EVALUATION

E	29000	ksi	Inventory	Bending	0.55	fy	*TABLE 6B.5.2.1-1*
				Shear	11	ksi	
				Axial	2.12	S.F.	
			Operating	Bending	0.75	fy	*TABLE 6B.5.2.1-2*
				Shear	0.45	fy	
				Axial	1.7	S.F.	

	f_y (ksi)	Sx (in^3)	d (in)	tw (in)	Mdl (k-ft)	Msdl (k-ft)	Mped (k-ft)	Vdl (k)	Vsdl (k)	Vped (k)	Inventory		Operating	
											Bending	Shear	Bending	Shear
Bent 9 Floorbeam	50	572.51	23.00	0.50	300.23	61.87	618.86	22.69	4.88	48.81	1.53	2.03	2.31	4.74
Bent 10 Floorbeam	50	572.51	23.00	0.50	307.79	67.55	675.64	22.98	4.94	49.42	1.39	1.99	2.09	4.67
Plate Girder 8-10-E	50	2578.34	74.50	0.50	1617.43	247.48	2475.19	55.63	7.75	77.50	1.63	4.47	2.50	10.00
Plate Girder 8-10-W	50	4682.77	87.50	0.50	2998.01	433.78	4338.40	82.84	10.84	108.43	1.68	3.57	2.58	8.21
Bent 17 Floorbeam	50	293.75	23.50	0.63	136.33	39.18	391.45	12.64	3.42	34.15	1.27	4.26	1.90	9.21
Plate Girder 16B-17B-E	50	950.31	38.81	0.81	359.44	74.87	748.00	23.66	4.50	44.94	2.33	7.09	3.39	15.16
Exist. G2 @ Span 18	33	542.22	36.00	0.65	166.93	55.61	555.56	11.16	3.37	33.67	1.08	7.21	1.61	9.89

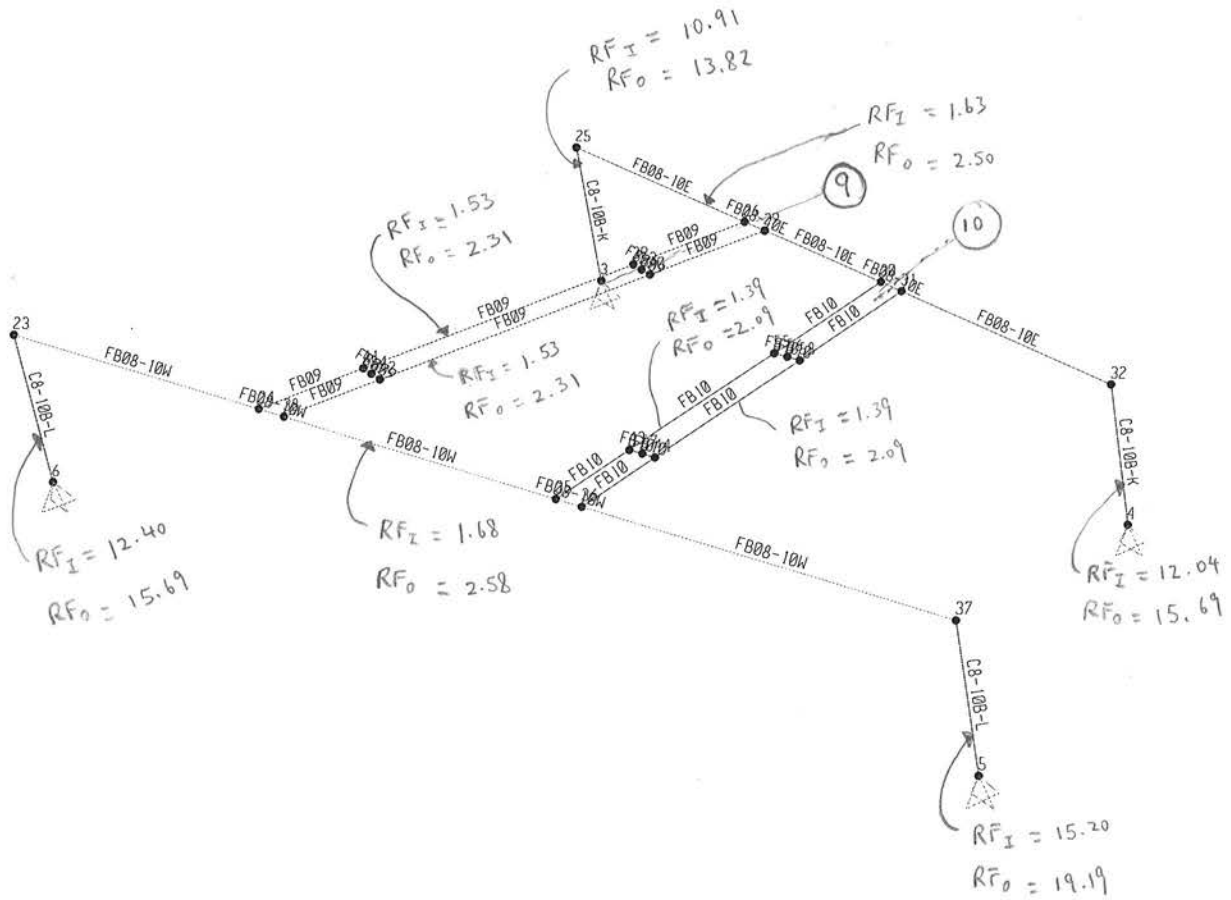
	f_y (ksi)	K	H (ft)	ry (in)	As (in^2)	Pdl (k)	Psdl (k)	Pped (k)	KL/r	Cc	Fa_Inven (ksi)	Fa_Oper (ksi)	Inventoy	Operating
Add. Column 8K	50	1.00	17.81	3.97	44.25	58.44	7.75	77.50	53.80	107.00	20.60	25.69	10.91	13.82
Add. Column 8L	50	1.00	18.89	4.01	71.13	87.64	10.84	108.43	56.59	107.00	20.29	25.30	12.40	15.69
Add. Column 10B-K	50	1.00	17.81	3.97	44.25	54.88	7.05	70.55	53.80	107.00	20.60	25.69	12.04	15.24
Add. Column 10B-L	50	1.00	18.90	4.01	71.13	78.78	8.92	89.16	56.62	107.00	20.28	25.29	15.20	19.19
Add. Column 16B-K	50	1.00	15.00	3.69	24.97	25.00	4.50	44.94	48.75	107.00	21.14	26.36	11.09	13.99
Add. Column 17B-K	50	1.00	15.00	3.64	28.81	17.20	2.34	23.33	49.44	107.00	21.07	26.27	25.18	31.61

6.0 Load Rating Summary

* Only show the load Rating Result for Bending for Beam / Girder.

R.F. I → Inventory

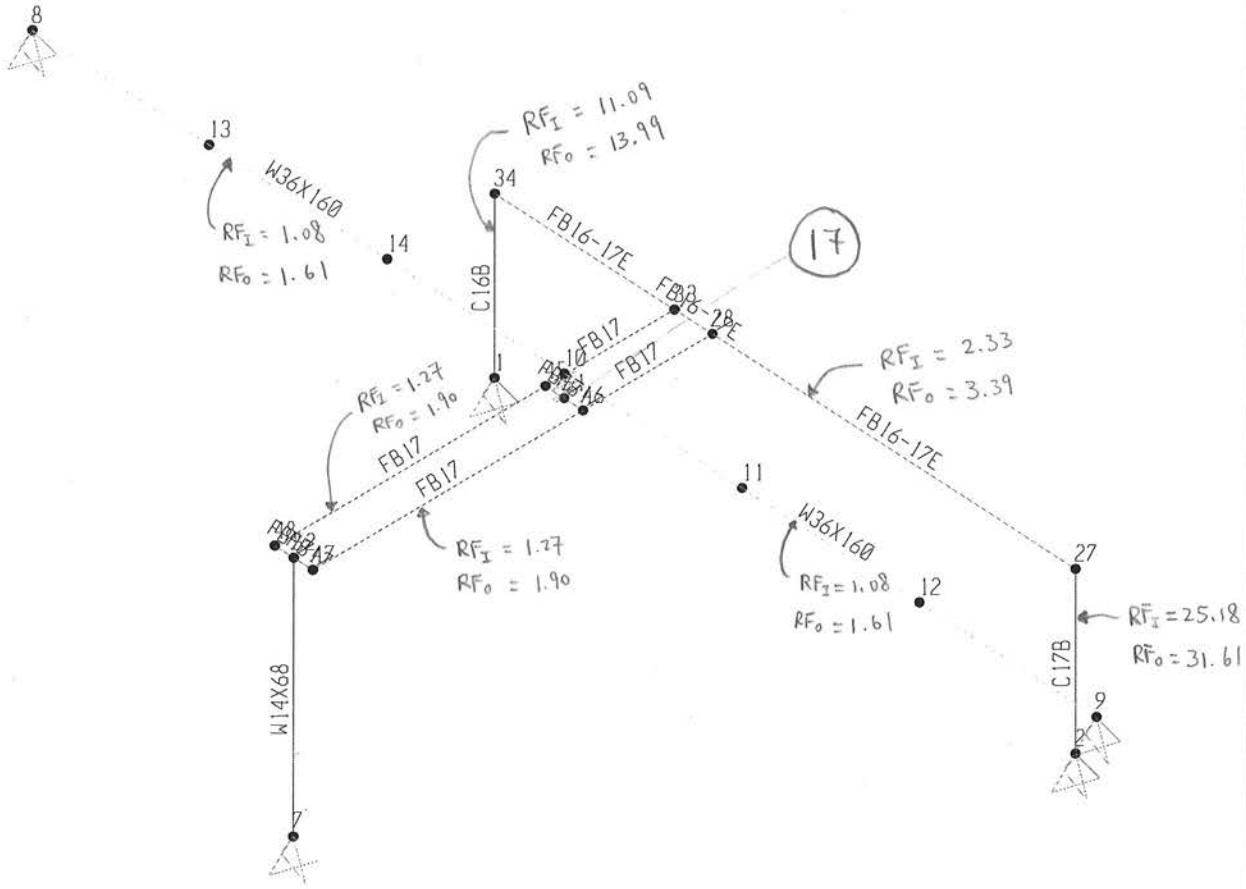
R.F. O → Operating.



* Only show the load Rating Result for Bending for Beam / Girder

RF_I → Inventory

RF_o → Operating



Appendix D: Traffic Study

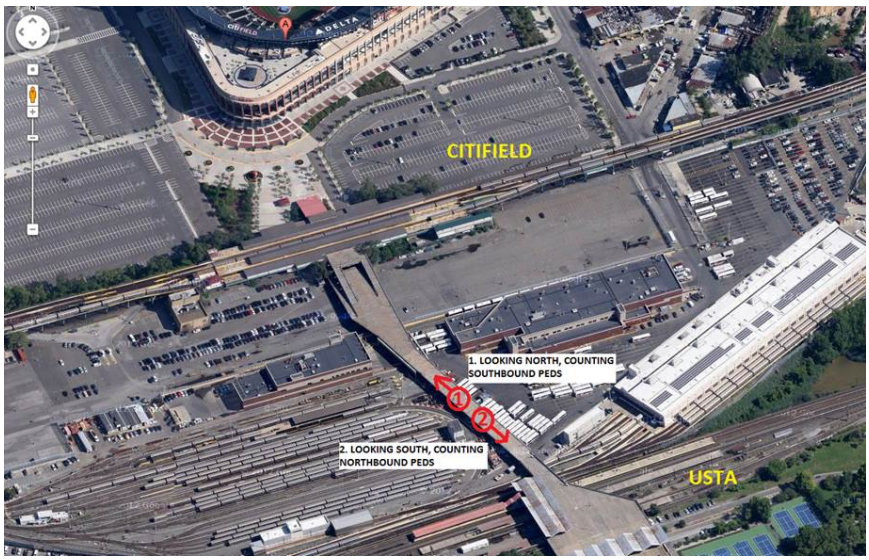
Passerelle Pedestrian Bridge - Pedestrian Traffic Study

Introduction and Project Description

The Passerelle Pedestrian Bridge is a multi-span bridge approximately 1,100 feet in length that connects a portion of Flushing Meadows Corona Park, including the USTA Billie Jean King National Tennis Center (NTC) and the adjacent LIRR Station (Mets – Willets Point) with the NYC Transit #7 subway line station (also referred to as Mets – Willets Point) and Citi Field. The bridge, constructed in 1939 and reconstructed in 1964, spans over the LIRR Port Washington Line tracks, the NYC Transit Corona Maintenance Yard, a Park road (owned by NYCDPR) and the Southfield/Commuter parking area.

The City of New York is investigating a rehabilitation of the bridge, and a pedestrian analysis is required. Vehicles (other than emergency and maintenance) are not permitted on the bridge; therefore a vehicular count program is not necessary. A pedestrian count program is required in order to determine the existing pedestrian volumes, and based on future NTC expansion, project this volume to an assumed rehabilitation year of the bridge. These volumes will be critical in the determination of the proposed width of the bridge, and the minimal recommended width required during the various stages of construction.

The majority of pedestrians who utilize the bridge are destined for/originate from the NYC Transit subway, located just north of the bridge. Pedestrian volumes are relatively low during the majority of the year, with users travelling between the subway station and the expansive Flushing Meadows Corona Park during the seasonable periods. Peak usage on the bridge drastically increases during the two week period (late August/early September) when the US Open occurs at the NTC. In addition, higher volumes occur on the bridge when a baseball game (or other event) is held at Citi Field, with pedestrians who arrive via the LIRR station, located just south of the bridge. Our pedestrian study will focus on data collected during the peak periods of the US Open event, and on a seasonable Sunday with a relatively large baseball event at Citi Field.



Data Collection Program

The 2013 dates for the US Open were between Monday, August 26th and Monday September 9th. A recently completed (May 2013) Final Environmental Impact Statement (FEIS) for the NTC stated that the greatest attendances occur during the first few days of the first week, when more matches are played simultaneously on the many courts around the NTC main stadiums. In addition, volumes

are greatest when a Mets home game (at nearby Citi Field) occurs on the same evening as the first week matches. Peak volumes are expected on the bridge from 4:00PM to 8:00PM, when users:

- leave the day session, where events are held at Arthur Ashe and Louis Armstrong Stadiums within the NTC and around the grounds (begins at 11:00AM, ends at varying times),
- arrive for the evening session (Arthur Ashe Stadium only) at the NTC (begins at 7:00PM, ends after 11:00PM),
- arrive for the evening Mets game at Citi Field (begins at 7:10PM).

This conflict occurred each of the first three evenings of the 2013 US Open (Monday – Wednesday, August 26th through 28th). In addition, Weidlinger obtained advance ticket sales data from the New York Mets, which showed virtually similar advance sales for each of the three days (20,670; 20,040 and 19,285 respectively). Pedestrian counts were obtained on the bridge on two of these evenings, Monday and Wednesday, based on the above advance sale information and weather forecasts.

In addition, pedestrian counts were obtained on Sunday, September 29th, the final baseball game of the year. Two special events were occurring for this game – both Fan Appreciation and Mike Piazza Mets Hall of Fame Induction ceremony. The advance ticket sale for this game was 41,891, which is a near sell out. Peak volumes were expected on the bridge from 10:00AM to 2:00PM, when users:

- arrive in the morning to go to Flushing Meadow Corona Park
- arrive for the afternoon Mets game at Citi Field (game began at 1:10PM)

The counts were performed by two Weidlinger personnel stationed south of the subway entrance, at the south end of the wooden boardwalk just north of the LIRR, with one responsible to count pedestrians walking south (towards the NTC and Flushing Meadow Corona Park), the other counting pedestrians walking north (from the NTC/Park). Counts were obtained in 15 minute intervals for the four hour periods.

A current level of service (LOS) was determined based on the pedestrian volumes and the width of the bridge. In addition to current LOS, an analysis was done for future LOS (an expansion of the NTC will occur as noted in the aforementioned FEIS, which will increase capacity and expected usage). It is necessary to find both current and future LOS to determine how pedestrian traffic would be affected when a portion of the bridge is closed during rehabilitation.

Pedestrian volumes obtained for August 26th and August 28th are depicted in Table 1. Volumes from September 29th are depicted in Table 2:

Table 1

	Counts	
Time Interval	**8.26.13**	**8.28.13**
4:00 PM - 4:15 PM	933	756
4:15 PM - 4:30 PM	731	948
4:30 PM - 4:45 PM	838	925
4:45 PM - 5:00 PM	914	974
5:00 PM - 5:15 PM	1,154	891
5:15 PM - 5:30 PM	1,510	924
5:30 PM - 5:45 PM	1,722	1,192
5:45 PM - 6:00 PM	2,044	1,464
6:00 PM - 6:15 PM	2,499	1,706
6:15 PM - 6:30 PM	2,283	2,309
6:30 PM - 6:45 PM	2,074	2,162
6:45 PM - 7:00 PM	2,533	2,553
7:00 PM - 7:15 PM	2,102	3,010
7:15 PM - 7:30 PM	1,670	2,006
7:30 PM - 7:45 PM	1,295	1,504
7:45 PM - 8:00 PM	1,177	1,114
Total	**25,479**	**24,438**

Table 2

	Counts
Time Interval	**9.29.13**
10:00 AM - 10:15 AM	76
10:15 AM - 10:30 AM	72
10:30 AM - 10:45 AM	104
10:45 AM - 11:00 AM	135
11:00 AM - 11:15 AM	215
11:15 AM - 11:30 AM	157
11:30 AM - 11:45 AM	652
11:45 AM - 12:00 PM	171
12:00 PM - 12:15 PM	1016
12:15 PM - 12:30 PM	379
12:30 PM - 12:45 PM	834
12:45 PM - 1:00 PM	110
1:00 PM - 1:15 PM	363
1:15 PM - 1:30 PM	142
1:30 PM - 1:45 PM	201
1:45 PM - 2:00 PM	94
Total	**4,721**

Pedestrian Capacity Analysis

Level of service is determined by measuring the flow rate of pedestrians. Pedestrian flow rate can be determined by the amount of pedestrians that pass through an effective width per minute (p/min/ft). The width of the bridge at its narrowest point is approximately thirty eight (38) feet railing-to- railing. It was observed during the pedestrian count program that an approximate six (6) foot - 6 (six) inch lane was cordoned off as a designated bike taxi lane, delineated by removable bollards. The full thirty eight (38) foot width will be used for analysis since this bike taxi lane can be removed.

The type of pedestrian flow that occurred during the count program is known as platooning. Platooning is when a large number of pedestrians enter a walkway simultaneously due to the arrival of a train or bus. In this case, it was the #7 train (arriving from either Manhattan or Flushing – the two terminal stations of the #7 line). These trains (both local and express) arrive approximately every two minutes. The level of service criteria for platooning based on pedestrian flow according to the NYC DCP Transportation Division (dated April 2006), are as follows:

Table 3

LOS	Space (ft^2/p)	Flow Rate (p/min/ft)
A	> 530	≤ 0.5
B	> 90-530	> 0.5-3
C	> 40-90	> 3-6
D	> 23-40	> 6-11
E	> 11-23	> 11-18
F	≤ 11	> 18

In general, any level of service above D is considered acceptable.

The following table shows flow rate and level of service for the 15 minute interval counts for the US Open dates:

Table 4

	August 26th, 2013			August 28th, 2013		
Time Interval	**Pedestrians**	**Flow Rate (p/min/ft)**	**LOS**	**Pedestrians**	**Flow Rate (p/min/ft)**	**LOS**
4:00 PM - 4:15 PM	933	1.64	B	756	1.33	B
4:15 PM - 4:30 PM	731	1.28	B	948	1.66	B
4:30 PM - 4:45 PM	838	1.47	B	925	1.62	B
4:45 PM - 5:00 PM	914	1.60	B	974	1.71	B
5:00 PM - 5:15 PM	1154	2.02	B	891	1.56	B
5:15 PM - 5:30 PM	1510	2.65	B	924	1.62	B
5:30 PM - 5:45 PM	1722	3.02	C	1192	2.09	B
5:45 PM - 6:00 PM	2044	3.59	C	1464	2.57	B
6:00 PM - 6:15 PM	2499	4.38	C	1706	2.99	B
6:15 PM - 6:30 PM	2283	4.01	C	2309	4.05	C
6:30 PM - 6:45 PM	2074	3.64	C	2162	3.79	C
6:45 PM - 7:00 PM	2533	4.44	C	2553	4.48	C
7:00 PM - 7:15 PM	2102	3.69	C	3010	5.28	C
7:15 PM - 7:30 PM	1670	2.93	B	2006	3.52	C
7:30 PM - 7:45 PM	1295	2.27	B	1504	2.64	B
7:45 PM - 8:00 PM	1177	2.06	B	1114	1.95	B

The following table shows flow rate and level of service for the 15 minute interval counts for the Mets game:

Table 5

	September 29th, 2013		
Time Interval	**Pedestrians**	**Flow Rate (p/min/ft)**	**LOS**
10:00 AM - 10:15 AM	76	0.13	A
10:15 AM - 10:30 AM	72	0.13	A
10:30 AM - 10:45 AM	104	0.18	A
10:45 AM - 11:00 AM	135	0.24	A
11:00 AM - 11:15 AM	215	0.38	A
11:15 AM - 11:30 AM	157	0.28	A
11:30 AM - 11:45 AM	652	1.14	B
11:45 AM - 12:00 PM	171	0.30	A
12:00 PM - 12:15 PM	1016	1.78	B
12:15 PM - 12:30 PM	379	0.66	B
12:30 PM - 12:45 PM	834	1.46	B
12:45 PM - 1:00 PM	110	0.19	A
1:00 PM - 1:15 PM	363	0.64	B
1:15 PM - 1:30 PM	142	0.25	A
1:30 PM - 1:45 PM	201	0.35	A
1:45 PM - 2:00 PM	94	0.16	A

As shown in the tables above, all fifteen (15) minute time intervals had level of service value of C or better, therefore making current conditions acceptable.

The peak period for US Open dates occurred on Wednesday, August 28th between 6:15PM and 7:15PM with a total of 10,034 visitors (2,309 + 2,162 + 2,553 + 3,010). The peak period for the Mets game occurred on Sunday September 29th between 11:45PM to 12:45PM with a total of 2,400 visitors (171 + 1016 + 379 + 834). There are spikes in demand during one (1) of the fifteen (15) minute time intervals during both of the peak hours; therefore a peak hour factor should be calculated to adjust both hourly volumes. Peak hour factor is found by dividing the hourly volume by four (4) times the peak fifteen (15) minute volume. The peak hour factors for these periods are:

$$PHF_1 = \frac{\mathrm{V}}{\mathrm{v15x4}} = \frac{10{,}034}{3{,}010\ x4} = 0.83 \text{ (US Open)}$$

$$PHF_2 = \frac{\mathrm{V}}{\mathrm{v15x4}} = \frac{2{,}400}{1{,}016\ x4} = 0.59 \text{ (Mets game)}$$

The adjusted peak hour volume is found by dividing the hourly volume by the peak hour factor. The adjusted peak hour volume is 12,040 (10,034/0.83) for August 28th and 4,064 (2,400/0.59) for September 29th.

Pedestrian volumes of 12,040 and 4,064 would produce flow rates of 5.28 (p/min/ft) and 1.78 (p/min/ft), which would result in an acceptable Levels of Service of C and B, respectively..

Future Conditions

As per the FEIS for the NTC, there is a proposed expansion (estimated construction of 2019) which will produce an estimated increase of 10,000 visitors to the US Open on a daily basis. In addition, the improvements include two new parking garages which will add approximately 389 parking spaces for visitors.

A key part of the future conditions analysis is determining how many of the projected 10,000 additional visitors will be using the Passerelle Bridge in order to get to the NTC. Table 4 below shows the anticipated travel demand assumptions and trip generation estimates for the 10,000 additional visitors (table included within the FEIS):

Table 6
Travel Demand Assumptions and Trip Generation Estimates

	Daily Trip Increment				Peak Period Departure Trip Increment		
Modal Split	**Percent**	**Person Trips**	**VOR (1)**	**Vehicle Trips (2)**	**Peak Period**	**Person Trips**	**Vehicle Trips (2)**
Auto	25.9%	2,590	2.01	1,288	35%	907	452
Taxi/Car Service	11.9%	1,190	1.67	713	35%	416	249
Charter Bus	4.2%	420	40.0	11	35%	147	4
MTA NYCT Bus	1.0%	100	--	--	35%	35	--
Subway	40.4%	4,040	--	--	35%	1,414	--
LIRR	13.0%	1,300	--	--	35%	455	--
Other	3.6%	360	--	--	35%	126	--
Total	**100.0%**	**10,000**	**--**	**2,012**	**--**	**3,500**	**705**

Notes: (1) Vehicle Occupancy Rate
(2) Projected total vehicle trip-ends will be 2,725 daily and 952 peak period with the additional taxi/car service round trips.

The percentage of visitors that use automobiles (25.9%) will be able to park in either existing or the new parking facilities adjacent to the NTC, and thus will not cross the Passerelle Bridge. Similarly, those arriving by taxi (11.9%) or charter buses (4.2%) will be dropped off on the west side of the NTC, thus also not using the Bridge. The LIRR Mets – Willett Point Station is immediately adjacent to the NTC, thus that percentage (13.0%) will not use the narrow thirty eight (38) foot wide portion of the Bridge being analyzed.

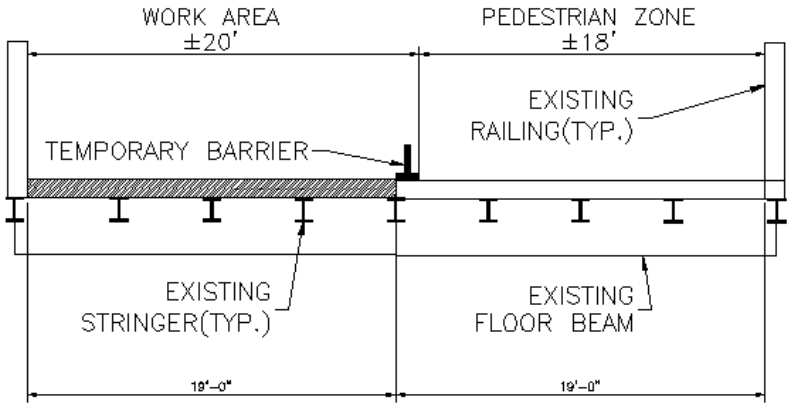
Visitors who use the #7 train (the largest percentage estimated at 40.9%) will have to cross the Bridge in order to attend the events at the NTC. In additional, those that arrive via a NYCT bus (Q-48 on Roosevelt Avenue) will also use the Bridge.

As per data shown in Table 6, an estimated 1,449 (1,414 + 35) additional visitors will be crossing the Bridge from the subway and bus respectively during the peak period. Note that the peak period referred to in this Table from the FEIS is the peak hour (6:00PM to 7:00PM).

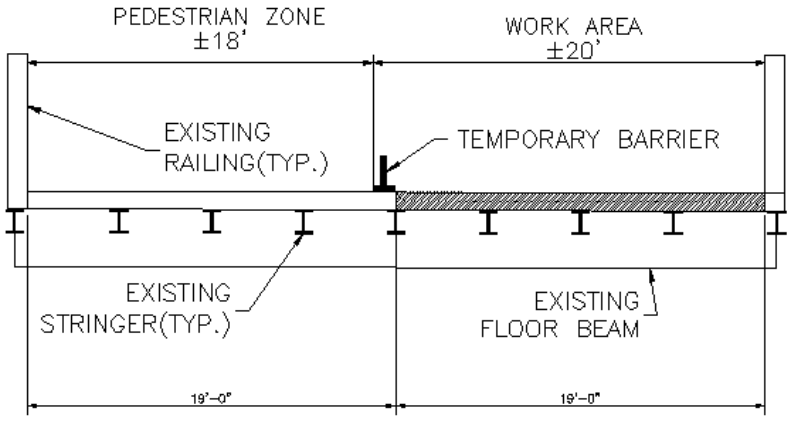
Adding the additional pedestrian volume from trip generations to the existing adjusted peak hour volume would produce a pedestrian volume of 13,489 (12,040 + 1,449). A pedestrian volume of 13,489 would produce a flow rate of 5.9(p/min/ft) and an acceptable Level of Service C.

Bridge Rehabilitation

The second analysis required will be an analysis of how the LOS for pedestrians is affected by the various stages of bridge rehabilitation. It is anticipated that the bridge will be rehabilitated one half at a time, thus two phases of work. As shown below, Phase 1 would have an approximate work zone of twenty (20) feet, thus leaving approximately eighteen (18) feet for pedestrian use. Phase 2 would flip construction to the other half, also resulting in a twenty (20) foot work zone and eighteen (18) foot pedestrian path.



PHASE 1



PHASE 2

Using the adjusted peak hour volumes of 12,040 (US Open) and 4,064 (Mets game) with an eighteen (18) foot wide pedestrian zone, the following flow rates and levels of service are produced:

Table 7

	Effective Width = 18'		
Event	**Adjusted Peak Hour Volumes**	**Flow Rate (p/min/ft)**	**LOS**
US Open	12,040	11.2	E
Mets Game	4,064	3.8	C

Conclusion

In order to avoid unacceptable Level of Service results (lower than D) on the Passerelle Bridge during the planned rehabilitation, reduction of the width of the bridge (to 18 feet) should be avoided during the US Open event. Unacceptable flow rates will occur during various times and dates during this two week period. Performing bridge rehabilitation that reduces the bridge to approximately 50% of its width during the other 50 weeks of the year should result in acceptable Level of Service values of C or better.

Appendix E: Concrete Coring Report

CONCRETE CORING PROGRAM

FOR

PASSERELLE PEDESTRIAN BRIDGE

CONTRACT NO. HBPED700Q

BOROUGH OF QUEENS

HAKS/WAI JV
40 WALL ST. 11th FLOOR
NEW YORK, NEW YORK 10005

DECEMBER 2013

Contract: HBPRED700Q

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APPENDIX A	SHEET NUMBER
Location Maps	1
Bridge Plan and Cross Sections	3

Contract: HBPRED700Q

INVITATION FOR PRICE QUOTE

Itemized proposal for performing concrete coring for the Passerelle Pedestrian Bridge.

Pursuant to the requirements of HAKS/WAI JV's contract with the New York City Department of Design and Construction (Contract No. HBPED700Q), price quotes, shall be submitted to HAKS/WAI JV at 40 Wall St. 11th Floor, New York, New York 10005, on or before 4:00 PM on September 20, 2013. Price estimates can be faxed for review to (212)747-1947, or emailed to qhashmi@haks.net.

Note: All price quotes are to be addressed to the attention of Quaiser Hashmi, P.E., Project Manager.

PRICE QUOTE INFORMATION

1. Form of Bid

All bids must be made only upon the blank form provided marked "Itemized Price Quote" for consideration. All blank spaces must be filled in (with ink), must give the price for each item and the aggregate amount for the work, and must be signed and acknowledged by the Bidder.

The prices are to include the furnishing of all materials, plant, equipment, tools and all other facilities, and the performance of all labor and services necessary or proper for the completion of the work.

2. Omissions and Discrepancies

Should a Bidder find discrepancies in, or omissions from the Drawings and/or Contract Documents, or should he be in doubt as to their meaning, he should notify the Engineer at once.

3. Acceptance of Proposals

The acceptance of the bids will be a notice in writing signed by HAKS/WAI JV, and shall be followed by a contract agreement for the outlined work.

4. Executing Contract Insurance

Within 2 days after entering into contract, the successful Contractor will be required to procure, maintain and submit the necessary documentation to substantiate the following:

The Contractor shall comply with the insurance requirements of New York City Department of Design and Construction Contract HBPED700Q. These requirements call for the naming of the City of New York, Long Island Rail Road, Metro Transit Authority, Citifield, New York City Department of Design and Construction, New York City Department of Transportation, New York City Department of Parks and Recreation, Sterling Mets, L.P., Sterling Mets Front Office, L.LC., Sterling Mets Operations, L.L.C., Queens Ballpark Company, L.L.C. and each of their respective owners, partners, members, and affiliates, whether direct or indirect, and all members, partners, shareholders, directors, officers, agents, representatives, employees, successors and assigns of any of the foregoing and the City of New York, the New York City Department of Parks and Recreation, the New York City Economic Development Corporation and the New York City Industrial Development Agency, and HAKS/WAI JV as additional insured. The successful bidder shall be required to file properly executed Vendex forms.

PROJECT DESCRIPTION

HAKS/WAI JV has been retained by NYCDDC to perform the Scope of Work services for the Passerelle Pedestrian Bridge, under NYCDDC Contract HBPED700Q. The Passerelle Pedestrian Bridge is a multi-span concrete steel frame structure. This test program includes concrete coring and laboratory testing of concrete cores.

Purpose of Test Program

The purpose of coring program is to determine the condition of deck and obtain test results for existing concrete properties.

The Contractor shall submit the bid by September 20, 2013.

Scope of Work

The scope of work consists of the following:

1. Six to eight (6-8) 4in dia. concrete cores, full depth in deck. Cores shall be taken perpendicular to the surface of slabs.

Notes:

A. All cores shall be taken at the locations identified in the field by HAKS/WAI JV, based on the visual inspection of existing deck conditions.

B. All field work shall be completed within **10 working days** from the date of notice to proceed. HAKS/WAI JV reserves the right to reduce or expand the scope of work in accordance with field conditions.

4. Perform Petrographic Analysis, Compressive Strength Test, Freeze-Thaw Test, and Chloride Content Test and submit test results.

5. Contractor shall provide all necessary maintenance and protection of traffic (MPT), material, equipment, scaffolding and personnel to perform the work to the satisfaction of HAKS/WAI JV and NYCDDC.

CONCRETE CORING PROGRAM

Under this program, the Contractor shall provide all labor, material, equipment, sources of power and water, insurance, and incidentals necessary to perform the following drilling operations under the full-time supervision of HAKS/WAI JV.

1. The core drilling operations shall be performed using 4" diameter, thin wall, diamond-bit core barrels. The drilling equipment shall be positioned and secured in place by the Contractor in a manner acceptable to HAKS/WAI JV and NYCDDC, and suitable for vertical core drilling operations. It is anticipated that a total of six to eight (6-8) cores shall be taken throughout the bridge deck.

2. After the removal of cores, all core holes shall be filled and patched with “Thorite”, manufactured by Thoro System Products, Inc. Miami, FL. or approved equal quick setting concrete. The procedure for filling core holes shall be as follows:

    a) Contractor shall thoroughly clean the drilled hole removing all foreign and loose material.

    b) Contractor shall coat the inside surface of the drilled hole with an approved bonding agent.

    c) Contractor shall then patch the hole in a watertight manner, with a quick-setting, non-shrink, non-metallic, non-staining grout, meeting ASTM-C827 specifications, approved by HAKS/WAI JV.

    d) Following the grouting, the contractor shall cover the surface of the grout-filled drilled hole with an approved cure/seal colorless coat conforming to ASTM C-309 specifications.

3. All cores recovered shall be photographed by the contractor using 35mm color film as soon as each core is dry enough to show pertinent details. Core photos shall be taken close-up, showing texture of core and shall indicate core number and size of core (by placing wooden ruler next to core). **Core logs and all photographs (two copies) with negatives shall be submitted to HAKS/WAI JV.** Cores shall be placed in suitable wooden storage boxes for delivery by Contractor to testing laboratory.

4. The testable cores (as determined by the Engineers) shall be tested by a qualified laboratory (contracted by the Contractor), approved by the NYCDDC. Three copies and an electronic PDF copy of the report shall be provided with original color photographs when appropriate.

The following tests shall be performed:

a) **Petrographic Analysis:** A visual petrographic analysis of each core sample (full length of core) shall be performed under this test.
b) **Compressive Strength Test:** Tests for compressive strength of selected samples shall be performed with conformance in ASTM C39.
c) **Freeze-Thaw Test:** Selected samples shall be tested for freeze-thaw analysis using NYSDDC method.
d) **Chloride Content Analysis:** Selected samples shall be tested for chloride content using the NYSDDC Materials Method. Depth of test sample from surface shall be determined by the Engineer.

5. Depending on the field conditions the Engineer may require laboratory tests. The Engineer shall determine in the field which of the coring will be performed.

6. No later than **two (2)** weeks following the completion of coring operations, including all core tests and analysis, a written test report shall be submitted to HAKS/WAI JV. It is realized that the Freeze-Thaw test may take more than two weeks. The original test report shall be stamped and signed by a licensed professional engineer for the testing laboratory.

7. Extreme caution shall be taken by contractor to prevent damages to existing structures. Any damage to existing structure due to contractor's operations shall be repaired by contractor to complete satisfaction of HAKS/WAI JV and NYCDDC at no charge to HAKS/WAI JV or NYCDDC.

8. It will be the contractor's responsibility to provide adequate protection to the general public at all times.

9. Contractor shall take adequate precautions to the satisfaction of The City of New York and utility companies to protect their facilities from being damaged during drilling or by any operations of the contractor.

10. Any damages to the property of the City of New York or private utility property caused by the contractor shall be repaired or replaced by the contractor at his own expense to the satisfaction of the City of New York, utility companies and HAKS/WAI JV.

11. Contractor shall comply with all requirements of Code 53, "Utility Mark Out", prior to starting any coring and excavation operation.

12. Maintenance and protection of traffic (including pedestrians) shall be performed by the contractor around work areas during the work day.

13. Contractor will be required to perform maintenance cleaning of the pavements within the contract limits when ordered by the engineer. Maintenance cleaning shall mean the removal of debris from any source, as ordered by the engineer. It will be the contractor's responsibility to provide adequate protection of the existing work and public safety at all times.

14. All material and equipment not in use, including employee's cars, shall be stored or parked to avoid interference with the normal flow of traffic.

15. The sole duty of the flagger shall be to direct traffic properly at all times. They shall not be used to more temporary signs or assist in other work and shall be positioned appropriately in advance of the work.

16. Contractor shall record all information, such as locations and sizes of all utilities, duct banks and footings, etc. encountered with respect to existing sidewalk surface, curb line, pavement surface and other permanent existing structures. The records shall include dimensions horizontally and elevations vertically.

17. The Contractor is not expected to get any permits from the stakeholders in the vicinity of the site.

Contract: HBPRED700Q

BID SHEET
ITEMIZED PRICE QUOTE

PASSERELLE PEDESTRIAN BRIDGE					
	Description	**Units**	**Quantity**	**Unit Price**	**Aggregate Amount**
1	Concrete Coring - all labor and materials (including mobilization, demobilization, at least two technicians for coring, pachometer and technician, all equipment and mater., sources of power & water, bucket truck as necessary, filling core holes with Quick Setting Concrete) 4 in diameter Cores in Slab	Each Core	8		
2	Petrographic Analysis (testing and reporting) all cores	Each Test	8		
5	Compressive Strength Test (testing and reporting)	Each Test	8		
6	Freeze-Thaw Test (testing and reporting)	Each Test	8		
7	Chloride Content Test (testing and reporting)	Each Test	8		
8	Mobilization, Maintenance and Protection of Traffic	Lump Sum	---	---	
				Total	

Note: Cost of insurance and all other incidentals are included in the bid price.

Total cost of work $_____

Company Name (print): _____

Name (print): _____

Signature: _____

Title (print): _____

ITEMIZED PROPOSAL

Name of Bidder _____
(Individual/Firm/Corporation, as case may be)

Residence of Bidder _____
Telephone Number_____
Federal Employer Identification Number (FEIN) _____

Place of Business of Bidder _____
Telephone Number _____
Fax Number _____

NAME OF PARTNERS RESIDENCE OF PARTNERS
(If Bidder is a FIRM, State here the Name and Residence of Each Member Thereof)

_____
_____
_____
_____

If Bidder is a CORPORATION, fill in the following blanks:

Organized under the laws of the State of _____

Name and Address of President _____

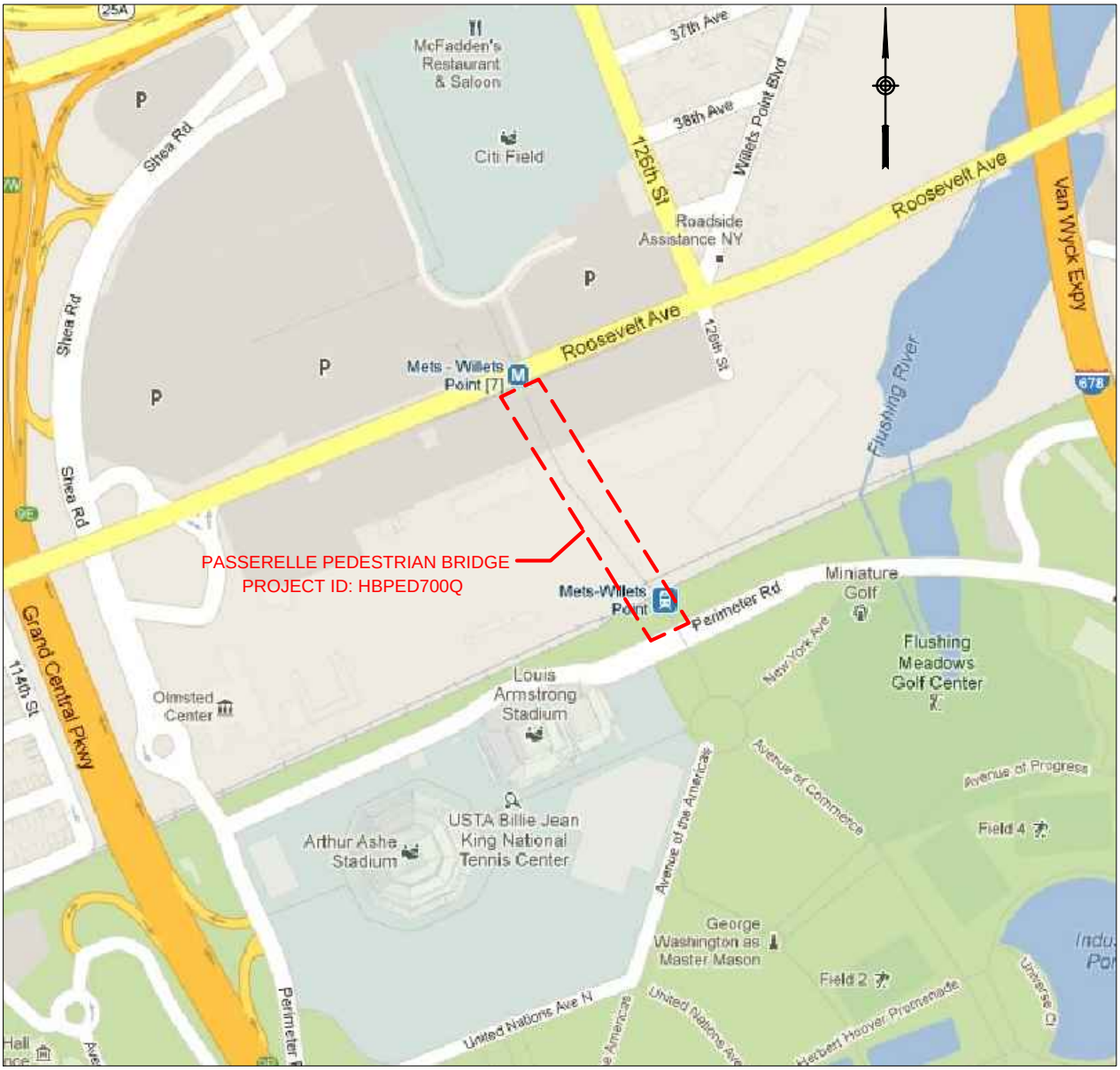
Name and Address of Vice President _____

Name and Address of Secretary _____

The undersigned also declares that he has carefully examined and fully understands information for Bidders, the Contract Agreement, Specifications, Plans and the Itemized Proposal, and he hereby proposes to furnish all the materials, adequate equipment, incidentals and sufficient labor, to progressively do all the work required to construct, finish and complete the work within the time

specified for the other incidental work included in this proposal, in accordance with the prices, at his own proper cost and expense and in a first class manner and in accordance with the Plans and Specifications, Notice to Bidders and Information to Bidders, all of which are a part of the Contract to be assumed, and in accordance with the plans, specifications, and detailed directions or instructions as may from time to time be given by the Engineer at the following prices, viz:

APPENDIX A



PASSERELLE PEDESTRIAN BRIDGE

SK-1

NOTE:
FOR BRIDGE LAYOUT, SEE SK-2.

AREA MAP

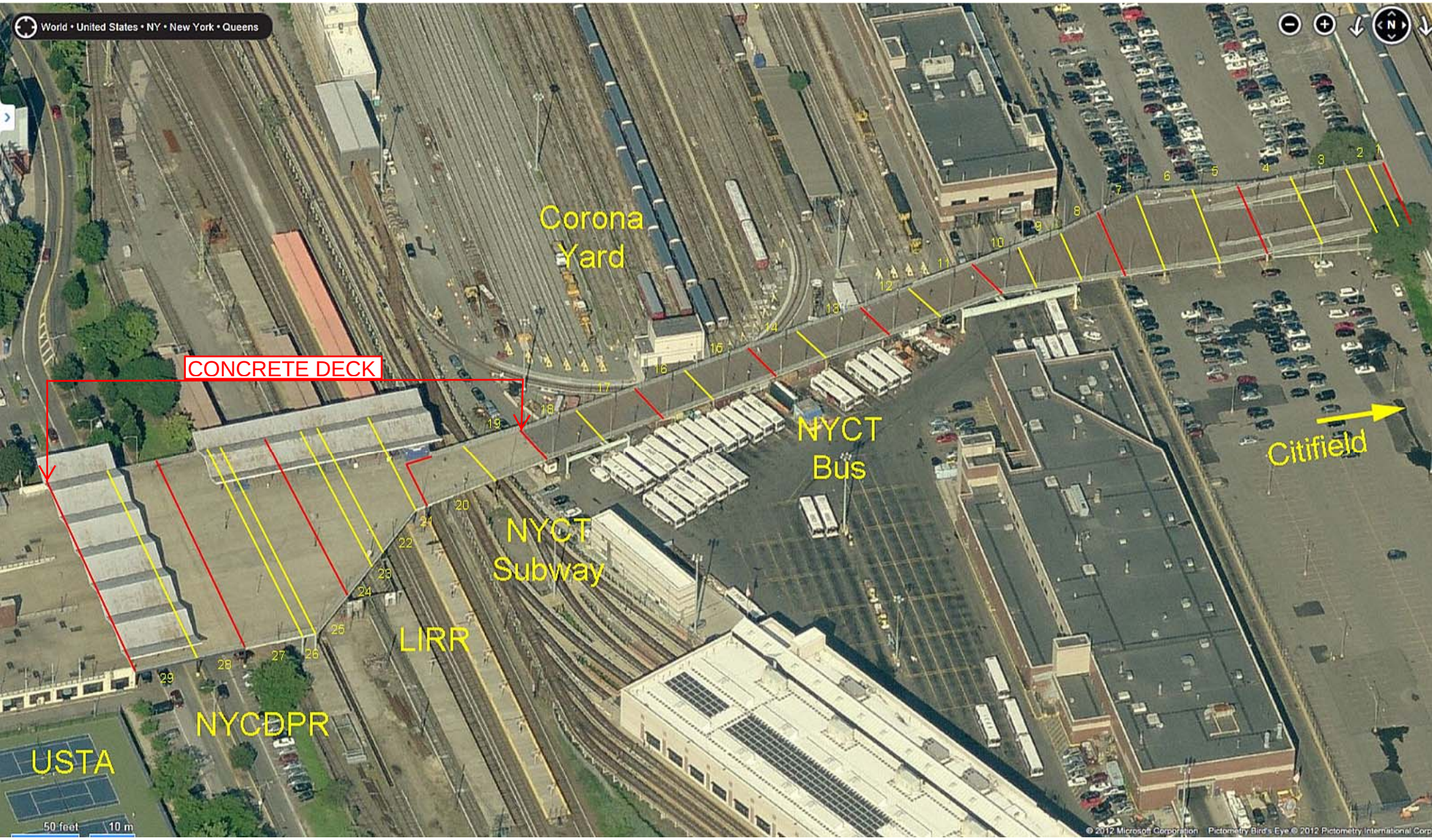
HAKS WEIDLINGER ASSOCIATES® INC
Joint Venture

40 WALL STREET, 11TH FLOOR
NEW YORK, N.Y. 10005

CITY OF NEW YORK
DEPARTMENT OF DESIGN AND CONSTRUCTION

PROJECT ID: HBPED700Q , PIN: 8502012HW0052P
PRE-SCOPING DESIGN SERVICES FOR
THE PASSERELLE PEDESTRIAN BRIDGE

BOROUGH OF QUEENS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NYCT Bus
Citifield
NYCT Subway
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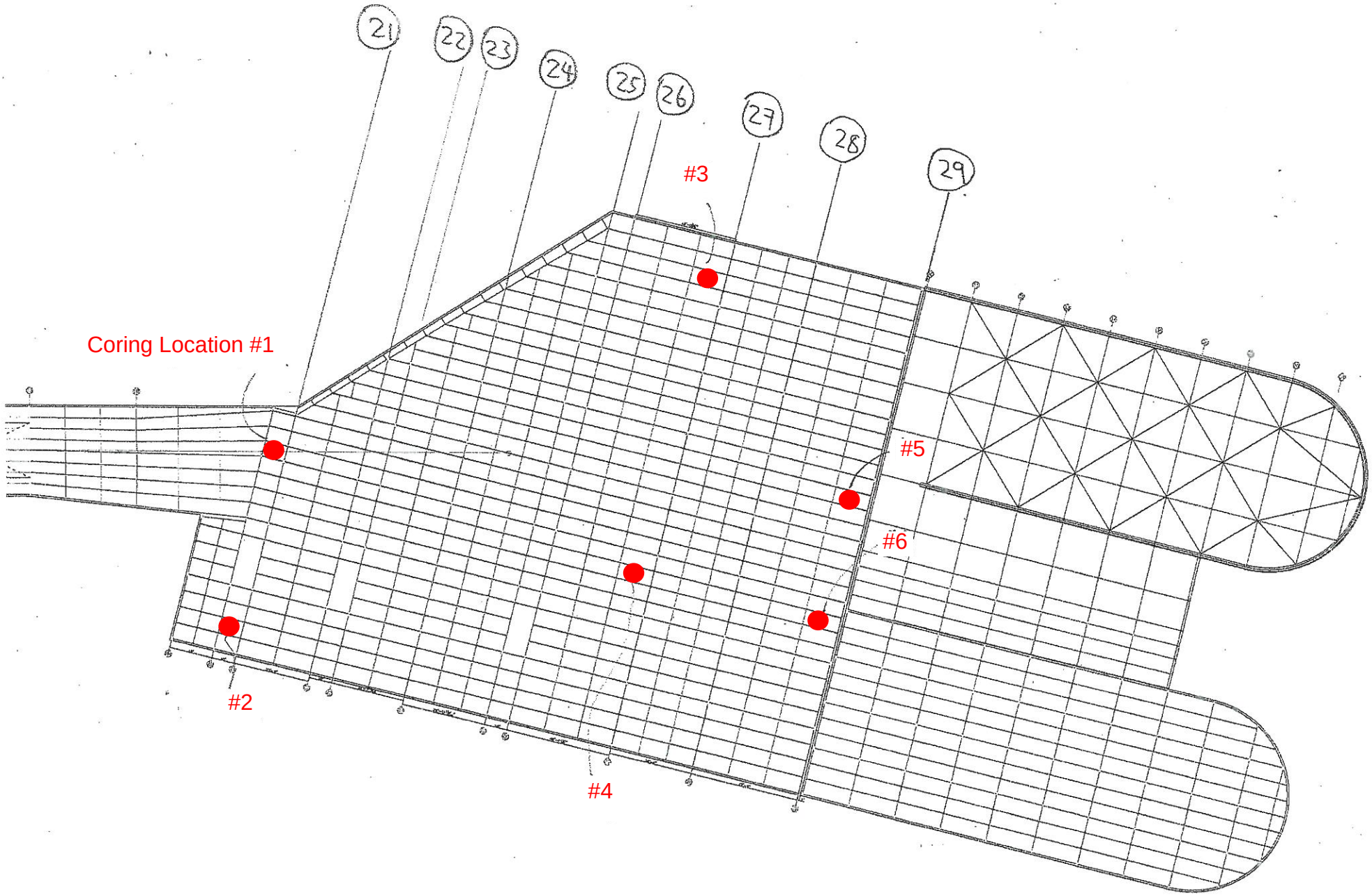
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Coring Location #1
#2
#3
#4
#5
#6

CRAIG
TESTING LABORATORIES, INC.
5439 Harding Highway • P.O. Box 427 • Mays Landing, NJ 08330-0427 • (609) 625-1700 • FAX (609) 625-1798

CLIENT: HAKS / WAI JV

PROJECT: Concrete Coring & Test Pit Program
Passerelle Pedestrian Bridge
Roosevelt Avenue and Perimeter Road
Queens, New York
Project ID#: HBPED700Q

SUBJECT: Concrete Core Drilling To Obtain
Test Cores for Laboratory Analysis

TEST REQUIRED: Compressive Strength Determination

DATE (S) DRILLED: April 15, 2014

DRILLED BY: CTL (M. Johnson)

DATE (S) TESTED: May 2, 2014

TESTED BY: CTL (J. Specht)

LAB NUMBER: 461000

LABORATORY ANALYSIS

Identification

Location:
Core #6 Bridge Deck
Core #8 Bridge Deck

Compressive Strength

CORE NUMBER	PREPARED LENGTH (INCHES)	DIAMETER (INCHES)	LENGTH DIAMETER RATIO	AREA (SQUARE INCHES)	CRUSHED AT (DAYS)	TOTAL LOAD (POUNDS)	CORRECTION FACTOR	COMPRESSIVE STRENGTH (PSI)
6	6.20	3.75	1.65	11.04	---	47,550	0.9720	4,185
8	5.70	3.75	1.52	11.04	---	37,850	0.9610	3,293

JS
Reported to:
Mr. Quaiser Hashmi, P.E. (Email)
Ms. Kristen Walker (Email)

Respectfully Submitted,
CRAIG TESTING LABORATORIES, INC.

Ian Craig

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AMERICAN ENGINEERING TESTING, INC.

CONSULTANTS
- ENVIRONMENTAL
- GEOTECHNICAL
- MATERIALS
- FORENSICS

REPORT OF CONCRETE ANALYSIS

PROJECT:
HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 461000

REPORTED TO:
CRAIG TESTING LABORATORIES, INC.
5439 HARDING WAY
PO BOX 427
MAYS LANDING, NJ

ATTN: JOHN SPECHT

APS JOB NO: 10-08215 **DATE:** JUNE 19, 2014

INTRODUCTION

This report presents the results of laboratory work performed by our firm on two concrete core samples submitted by Mr. John Specht of Craig Testing Laboratories, Inc. on May 16, 2014. We understand the concrete cores were obtained from a pedestrian bridge in the Flushing Meadows – Corona Park area of Queens, NY which is of historic significance. The bridge was reportedly constructed circa 1939 and was rehabilitated in the 1960's. The core samples represent concrete from the rehabilitation of the bridge in the 1960's. We also understand the bridge is of steel, wood and concrete construction and that the city of New York is conducting an 18 month long assessment of the structure to determine its condition. The scope of our work was limited to performing petrographic analysis on the samples to document the general overall condition of the concrete.

CONCLUSIONS

Based on our observations and testing, we believe:

1. The overall condition of the concrete was fair (Core #1) to fair to good (Core #7). The cement paste was moderately dense and hard with carbonation up to 8 mm. The crushed gabbroic coarse aggregate was hard, sound, and durable. The concrete was not purposefully air entrained and was placed with a moderate slump.

2. The concrete in both cores has poor durability. The concrete contained an air void system that is not consistent with current technology for resistance to freeze-thaw deterioration. We expect deterioration to occur if the concrete is exposed to freezing conditions when saturated.

SAMPLE IDENTIFICATION

Sample ID:	1	7
Sample Type:	Hardened Concrete Core	Hardened Concrete Core
Original Sample Dimensions:	95 mm (3-3/4") diameter x 95 mm (3-3/4") long	95 mm (3-3/4") diameter x 168 mm (6-5/8") long

TEST RESULTS

Our complete petrographic analysis documentation appears on the attached sheets entitled 24-LAB-001 "Petrographic Examination of Hardened Concrete, ASTM C856." A brief summary of the general physical characteristics of the concrete is as follows:

1. The coarse aggregate in both cores was comprised of 19 mm (3/4") maximum sized crushed gabbro/diabase that was well graded with good overall distribution.

2. Pozzolanic admixtures were not observed in either concrete sample.

3. The paste color in sample 1 was similar to very light gray and yellowish gray (Munsell® N8 and 5Y 8/1) and very pale orange (Munsell® 10YR 8/2) in the carbonated zone. The paste color in sample 7 was similar to light gray and yellowish gray (Munsell® N7 and 5Y 8/1) becoming medium light gray to medium gray (Munsell® N6 to N5) in the bottom 85 mm and very pale orange (Munsell® 10YR 8/2) in the carbonated zone.

4. The paste hardness in both samples was judged to be 'moderate' (Mohs ≈ 3) with the paste/aggregate bond considered fair.

5. The depth of carbonation in both samples was up to 8 mm following sub-vertical micro-cracking at the surface.

6. The w/cm was estimated to be between 0.38 and 0.50 with approximately 8 to 12% residual Portland cement clinker particles. No supplementary cementitious materials were observed in the concrete sample.

Air Content Testing

Sample ID	1	7
Total Air Content (%)	1.5	2.6
"Entrained" Air (%) voids < 1mm (0.040")	1.1	2.2
"Entrapped" Air (%) voids > 1mm (0.040")	0.4	0.4
Spacing Factor, in.	0.012	0.009

TEST PROCEDURES

Laboratory testing was performed on June 17, 2014 and subsequent dates. Our procedures were as follows:

Petrographic Analysis

A petrographic analysis was performed in accordance with AET Standard Operating Procedure 24- LAB-001, "Petrographic Examination of Hardened Concrete," ASTM C856-latest revision. The petrographic analysis consisted of reviewing the cement paste and aggregate qualities on a whole basis on saw cut and lapped, and fractured sections. Reflected light microscopy was performed under an Olympus SZX-12 binocular stereozoom microscope at magnifications up to 120x. The depth of carbonation was documented using a phenolphthalein pH indicator solution applied on freshly saw cut and lapped surfaces of the concrete sample. The paste-coarse aggregate bond quality was determined by fracturing a sound section of the concrete in the laboratory with a rock hammer.

The water/cementitious of the concrete was estimated by viewing a thin section of the concrete under a Nikon E600 polarizing light microscope at magnifications of up to 1000x. Thin section analysis was performed in accordance with Standard Operating Procedure 24-LAB-009, "Determining the Water/Cement of Portland Cement Concrete, AET Method." An additional, smaller, saw cut subdivision of the concrete sample is epoxy impregnated, highly polished, and then attached to a glass slide using an optically clear epoxy. Excess sample is saw cut from the glass and the thin slice remaining on the slide is lapped and polished until the concrete reaches 25 microns or less in thickness. Thin section analysis allows for the observation of portland cement morphology, including: phase identification, an estimate of the amount of residual material, and spatial relationships. Also, the presence and relative amounts of supplementary cementitious materials and pozzolans may be identified and estimated.

Air Content Testing

Air content testing was performed using Standard Operating Procedure 24-LAB-003, "Microscopical Determination of Air Void Content and Parameters of the Air Void System in Hardened Concrete, ASTM C457-latest revision." The linear traverse method was used. The concrete core was saw cut perpendicular with respect to the horizontal plane of the concrete as placed and then lapped prior to testing.

REMARKS

The test samples will be retained for a period of at least sixty days from the date of this report. Unless further instructions are received by that time, the samples may be discarded. Test results relate only to the items tested. No warranty, express or implied, is made.

Report Prepared By:
American Petrographic Services, Inc.

Scott F. Wolter, P.G.
President
MN License No. 30024
Phone: 651-659-1345
swolter@amengtest.com

24-LAB-001 Petrographic Examination of Hardened Concrete
ASTM C856

Project No.	10-08215	Date:	June 17, 2014
Sample ID:	1	Performed by:	B. Lemcke

I. General Observations

1. Sample Dimensions: Our analysis was performed on two lapped profiles of a 95 mm (3-3/4") x 95 mm (3-3/4") x 35 mm (1-3/8") thick section and a 76mm (3") x 52mm (2") thin section that were sawcut and prepared from the original 95 mm (3-3/4") diameter x 95 mm (3-3/4") long core.

2. Surface Conditions:
   Top: Fairly rough, mortar eroded and scaled surface with remnants of orange marker paint
   Bottom: Rough, irregular, fractured surface; with a rebar impression

3. Reinforcement: A rebar impression of undeterminable diameter was observed on the bottom surface of the concrete. No corrosion product was observed.

4. General Physical Conditions: The top surface was fairly rough, mortar eroded and scaled up to a maximum of 5 mm from the mortar eroded top surface. Many fine aggregates were exposed on the top surface. Several sub-angular, medium to coarse sand-sized grains of dark red-colored emery or corundum (aluminum oxide) were observed on the top surface and within the top 5 mm of the paste. The emery appeared to have been broadcast onto the top surface and worked into the surficial paste during finishing operations. Carbonation ranged from negligible up to 8 mm depth from the present top surface of the concrete proximate to sub-vertical microcracking. Several sub-vertical drying-shrinkage microcracks were observed proceeding from the top surface up to a maximum of 15 mm depth. A few fine sub-horizontal microcracks were observed within the top 1 mm of the present top surface proximate to scaled areas. Numerous microcracks were observed throughout the paste at various depths and orientations in a shrinkage-type pattern. The concrete contained a small amount of air entrainment. White, acicular ettringite was observed lining to partially filling several air voids below 17 mm depth from the present top surface. No evidence of active alkali-aggregate reactivity was observed.

II. Aggregate

1. Coarse: 19 mm (3/4") nominal sized crushed trap rock consisting of gabbro or diabase. The particles were mostly sub-angular with a few sub-rounded particles. A few flat and elongated particles were observed. The coarse aggregate appeared well graded and exhibited good overall distribution.

2. Fine: Natural quartz and lithic sand with some feldspar and mica. The grains were mostly sub-rounded with many smaller sub-angular particles. The fine aggregate appeared fairly graded and exhibited good overall uniform distribution.

III. Cementitious Properties

1.	Air Content:	1.5% total
2.	Depth of carbonation:	Ranged from negligible up to 8 mm depth from the top surface proximate to sub-vertical microcracking
3.	Pozzolan presence:	None observed
4.	Paste/aggregate bond:	Fair
5.	Paste color:	Similar to very light gray and yellowish gray (Munsell® N8 and 5Y 8/1) and very pale orange (Munsell® 10YR 8/2) in the carbonated zone
6.	Paste hardness:	Moderate (Mohs' ≈ 3)
7.	Microcracking:	Several sub-vertical drying-shrinkage microcracks were observed proceeding from the top surface up to a maximum of 15 mm depth. A few fine sub-horizontal microcracks were observed within the top 1 mm of the present top surface proximate to scaled areas. Numerous microcracks were observed throughout the paste at various depths and orientations in a shrinkage-type pattern.
8.	Secondary deposits:	White, acicular ettringite was observed lining to partially filling several air voids below 17 mm depth from the present top surface.
9.	w/cm:	Estimated at between 0.38 and 0.48 with approximately 8 to 10% residual portland cement clinker particles
10.	Cement hydration:	Alites: Fully Belites: Fully, few negligible

24-LAB-001 Petrographic Examination of Hardened Concrete
ASTM C856

Project No.	10-08215	Date:	June 18, 2014
Sample ID:	7	Performed by:	B. Lemcke

I. General Observations

1. Sample Dimensions: Our analysis was performed on the lapped profile of a 168 mm (6-5/8") x 94 mm (3-11/16") x 45 mm (1-3/4") thick section and a 76mm (3") x 52mm (2") thin section that were sawcut and prepared from the original 95 mm (3-3/4") diameter x 168 mm (6-5/8") long core.

2. Surface Conditions:
   Top: Fairly rough, mortar eroded surface with remnants of orange marker paint
   Bottom: Smooth, formed surface; placed on formwork with remnants of orange marker paint

3. Reinforcement: None observed.

4. General Physical Conditions: The top surface was fairly rough and mortar eroded. Many fine aggregates were exposed on the top surface. Several sub-angular, medium to coarse sand-sized grains of dark red-colored emery or corundum (aluminum oxide) were observed on the top surface and within the top 12 mm of the paste. A few isolated grains of emery were observed as deep as 45 mm from the top surface. The emery appeared to have been broadcast onto the top surface and worked into the surficial paste during finishing operations. Carbonation ranged from negligible up to 8 mm depth from the present top surface of the concrete proximate to sub-vertical microcracking. A few sub-vertical microcracks were observed proceeding from the top surface to 3 mm depth. The concrete contained a small amount of air entrainment. The distribution of the entrained air was very poor. Most of the entrained air was observed within the top 80 mm of the paste, below this depth entrained-sized air voids were scarce to not present at all. The concrete as a whole was not considered resistant to freezing and thawing under saturated conditions. The change in the volume of entrained air and slight difference in paste color at 85 mm depth from the top surface may represent a sub-horizontal cold joint. A few irregular-shaped air voids were observed throughout the paste. White, acicular ettringite was observed lining to partially filling many air voids below 35 mm depth from the present top surface. No evidence of active alkali-aggregate reactivity was observed.

II. Aggregate

1. Coarse: 19 mm (3/4") nominal sized crushed trap rock consisting of gabbro or diabase. The particles were mostly sub-angular to angular with a few sub-rounded particles. A few flat and elongated particles were observed. The coarse aggregate appeared well graded and exhibited good overall distribution.

2. Fine: Natural quartz and lithic sand with some feldspar and mica. The grains were mostly sub-rounded with many smaller sub-angular particles. The fine aggregate appeared fairly graded and exhibited good overall uniform distribution.

III. Cementitious Properties

1.	Air Content:	2.6% total
2.	Depth of carbonation:	Ranged from negligible up to 8 mm depth from the top surface proximate to sub-vertical microcracking
3.	Pozzolan presence:	None observed
4.	Paste/aggregate bond:	Fair
5.	Paste color:	Similar to light gray and yellowish gray (Munsell® N7 and 5Y 8/1) becoming medium light gray to medium gray (Munsell® N6 to N5) in the bottom 85 mm and very pale orange (Munsell® 10YR 8/2) in the carbonated zone
6.	Paste hardness:	Moderate (Mohs' ≈ 3)
7.	Microcracking:	A few sub-vertical microcracks were observed proceeding from the top surface to 3 mm depth.
8.	Secondary deposits:	White, acicular ettringite was observed lining to partially filling several air voids below 35 mm depth from the present top surface.
9.	w/cm:	Estimated at between 0.38 and 0.48 with approximately 8 to 10% residual portland cement clinker particles
10.	Cement hydration:	Alites: Fully Belites: Fully, few negligible

AMERICAN ENGINEERING TESTING, INC.

AIR VOID ANALYSIS

PROJECT:
HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 461000

REPORTED TO:
CRAIG TESTING LABORATORIES, INC.
5439 HARDING HIGHWAY
PO BOX 427
MAYS LANDING, NJ 08330-0427

ATTN: JOHN SPECHT

APS JOB NO: 10-08215

DATE: SEPTEMBER , 2013

Sample Number: 1

Conformance: The sample contains an air void system which is not consistent with current technology for freeze-thaw resistance.

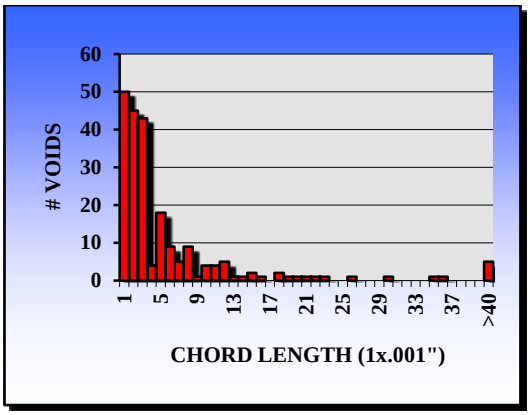
Sample Data

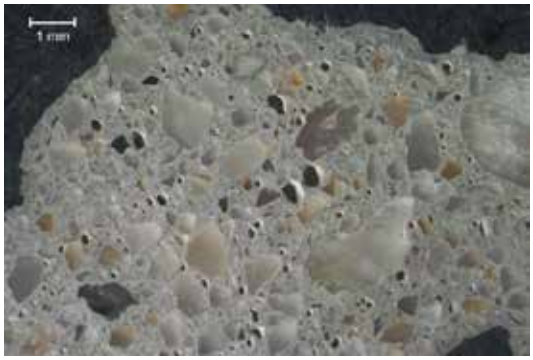
Description: Hardened Concrete Core

Dimensions: 96 mm (3-3/4") diameter by 96 mm (3-3/4") long

Test Data: By ASTM C457, Procedure A

Air Void Content %	1.5
Entrained, % < 0.040”(1mm)	1.1
Entrapped, %> 0.040”(1mm)	0.4
Air Voids/inch	2.4
Specific Surface, in^2/in^3	650
Spacing Factor, inches	0.012
Paste Content, % estimated	26
Magnification	50x
Traverse Length, inches	90
Test Date	6/16/2014
Technician	B. Lemcke





Magnification: 15x
Description: Hardened air void system.

550 Cleveland Avenue North | Saint Paul, MN 55114
Phone (651) 659-9001 | (800) 972-6364 | Fax (651) 659-1379 | www.amengtest.com | AA/EEO

AIR VOID ANALYSIS

PROJECT:
HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 461000

REPORTED TO:
CRAIG TESTING LABORATORIES, INC.
5439 HARDING HIGHWAY
PO BOX 427
MAYS LANDING, NJ 08330-0427

ATTN: JOHN SPECHT

APS JOB NO: 10-08215

DATE: SEPTEMBER , 2013

Sample Number: 7

Conformance: The sample contains an air void system which is not consistent with current technology for freeze-thaw resistance.

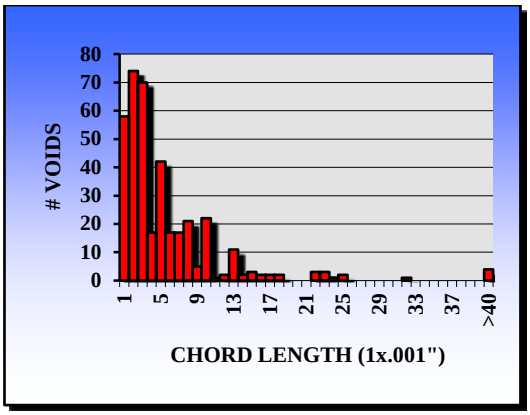
Sample Data

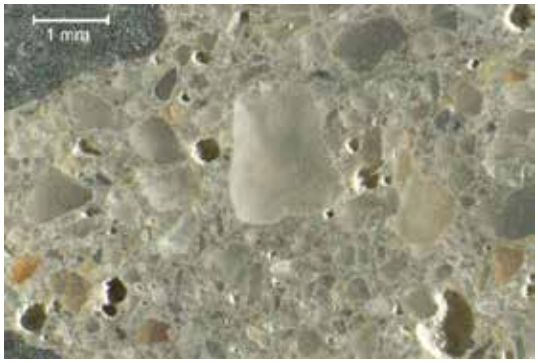
Description: Hardened Concrete Core

Dimensions: 96 mm (3-3/4") diameter by 96 mm (3-3/4") long

Test Data: By ASTM C457, Procedure A

Air Void Content %	2.6
Entrained, % < 0.040”(1mm)	2.2
Entrapped, %> 0.040”(1mm)	0.4
Air Voids/inch	4.2
Specific Surface, in^2/in^3	670
Spacing Factor, inches	0.009
Paste Content, % estimated	20
Magnification	50x
Traverse Length, inches	90
Test Date	6/18/2014
Technician	B. Lemcke





Magnification: 30x
Description: Hardened air void system.

550 Cleveland Avenue North | Saint Paul, MN 55114
Phone (651) 659-9001 | (800) 972-6364 | Fax (651) 659-1379 | www.amengtest.com | AA/EEO

AMERICAN ENGINEERING TESTING, INC.

CONSULTANTS
- ENVIRONMENTAL
- GEOTECHNICAL
- MATERIALS
- FORENSICS

REPORT OF CHEMICAL ANALYSIS

PROJECT:

HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY
JOB NUMBER: 461000

REPORTED TO:

CRAIG TESTING LABORATORIES, INC.
5439 HARDING HIGHWAY, PO BOX 427
MAYS LANDING, NJ 08330\

ATTN: JOHN SPECHT

APS PROJECT NO: 10-08215

DATE: JUNE 6, 2014

INTRODUCTION

This report presents the results of laboratory work performed by our firm on two concrete cores submitted to our laboratory by Mr. John Specht of Craig Testing Laboratories on May 15, 2014. The scope of our work was limited to documenting the chloride content profile in accordance with ASTM C1218, "Standard Test Method for Water-Soluble Chloride in Mortar and Concrete."

TEST RESULTS

Sample Identification	Water-Soluble Chloride by wt. % of sample	(ppm) mg/Kg
Core 3 (0-1”)	0.099	991
Core 3 (1-2”)	0.083	829
Core 5 (0-1”)	0.064	643
Core 5 (1-2”)	0.039	392

TEST PROCEDURES

Laboratory testing was performed on June 3, 2014 and subsequent dates. Testing was performed in accordance with ASTM C1218, "Standard Test Method for Water-Soluble Chloride in Mortar and Concrete." The concrete core was cut at the marked depths as measured from the top surface. The slices were crushed, oven dried, then ground to a powder passing a #20 US Standard Sieve.

REMARKS

The test samples will be retained for a period of at least thirty days from the date of this report. Unless further instructions are received by that time, the samples may be discarded. The test result relates only to the samples tested. No warranty, express or implied, is made.

American Engineering Testing, Inc.

Bill Rebel
Principal Chemist
Phone: 651-603-6633
brebel@amengtest.com

APS JOB NO: 10-08215 **DATE:** JUNE 19, 2014

PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

PHOTO: 1

SAMPLE ID: 1 **DESCRIPTION:** Profile view of the sample as received with the top surface to the left.

PHOTO: 2

SAMPLE ID: 1 **DESCRIPTION:** Top surface of the sample as received.

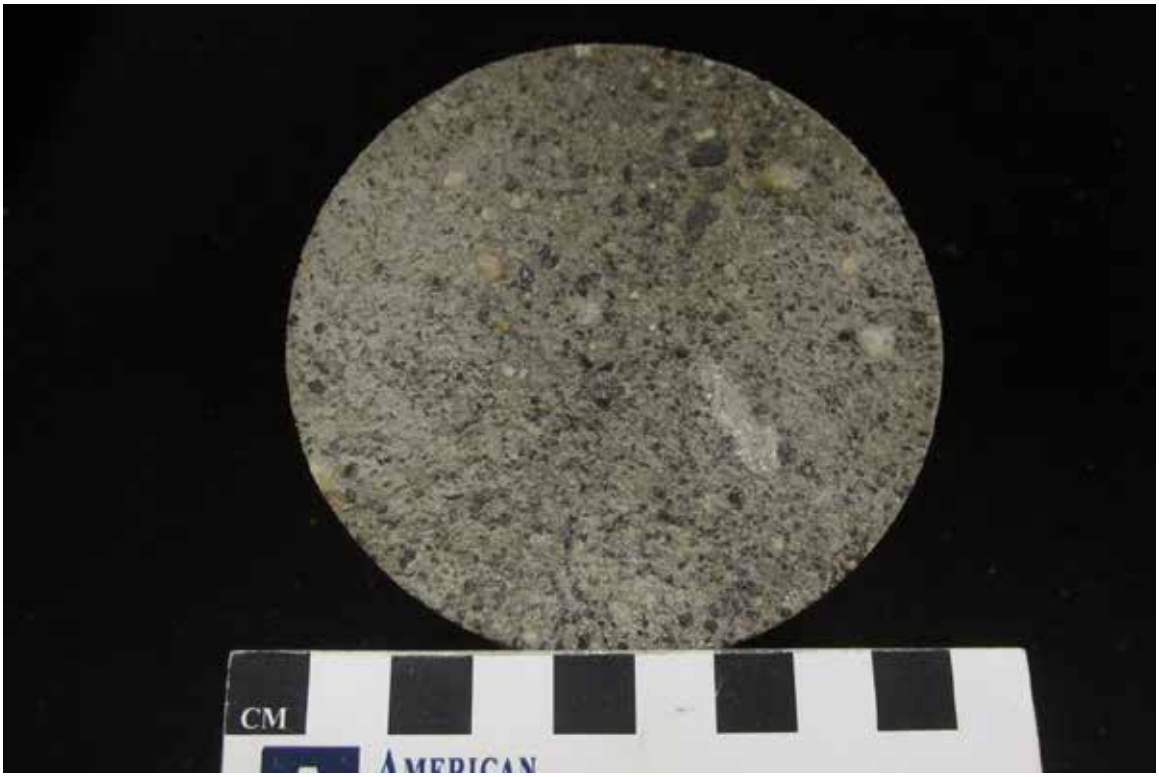
APS JOB NO: 10-08215 **DATE:** JUNE 19, 2014

PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

PHOTO: 3

SAMPLE ID: 7 **DESCRIPTION:** Profile view of the sample as received with the top surface to the left.

PHOTO: 4

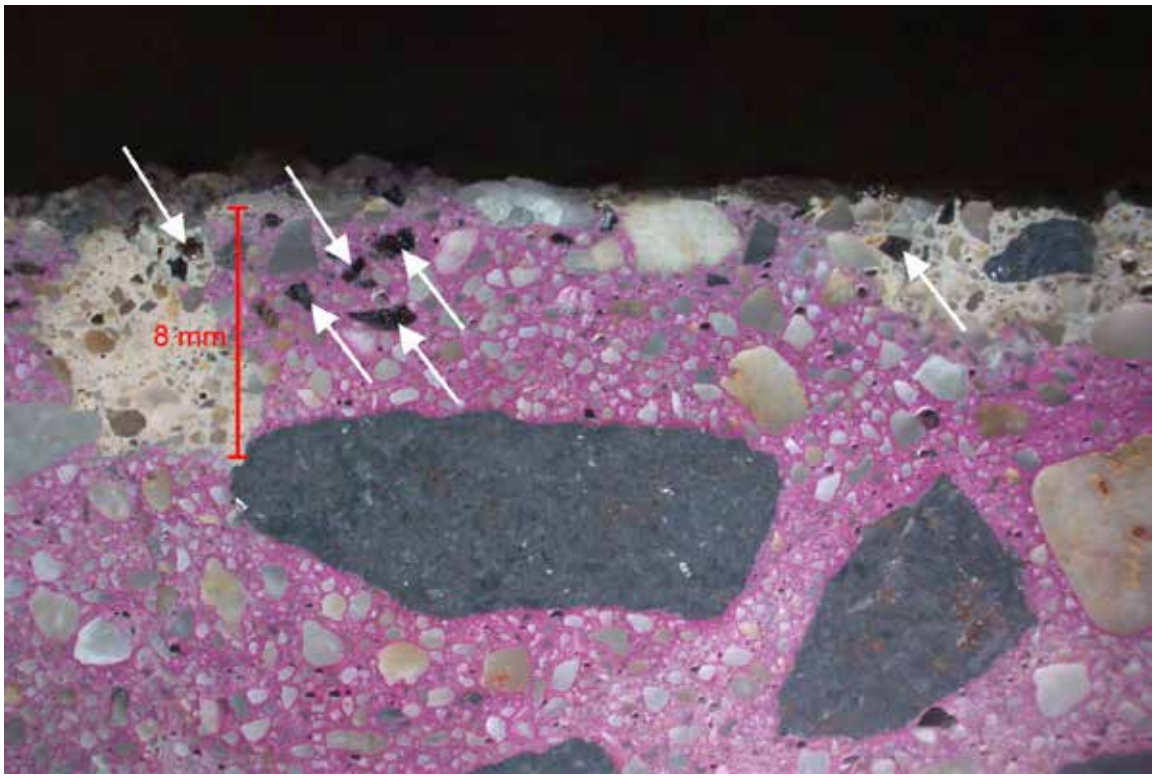


SAMPLE ID: 7 **DESCRIPTION:** Top surface of the sample as received.

APS JOB NO: 10-08215 **DATE:** JUNE 19, 2014

PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

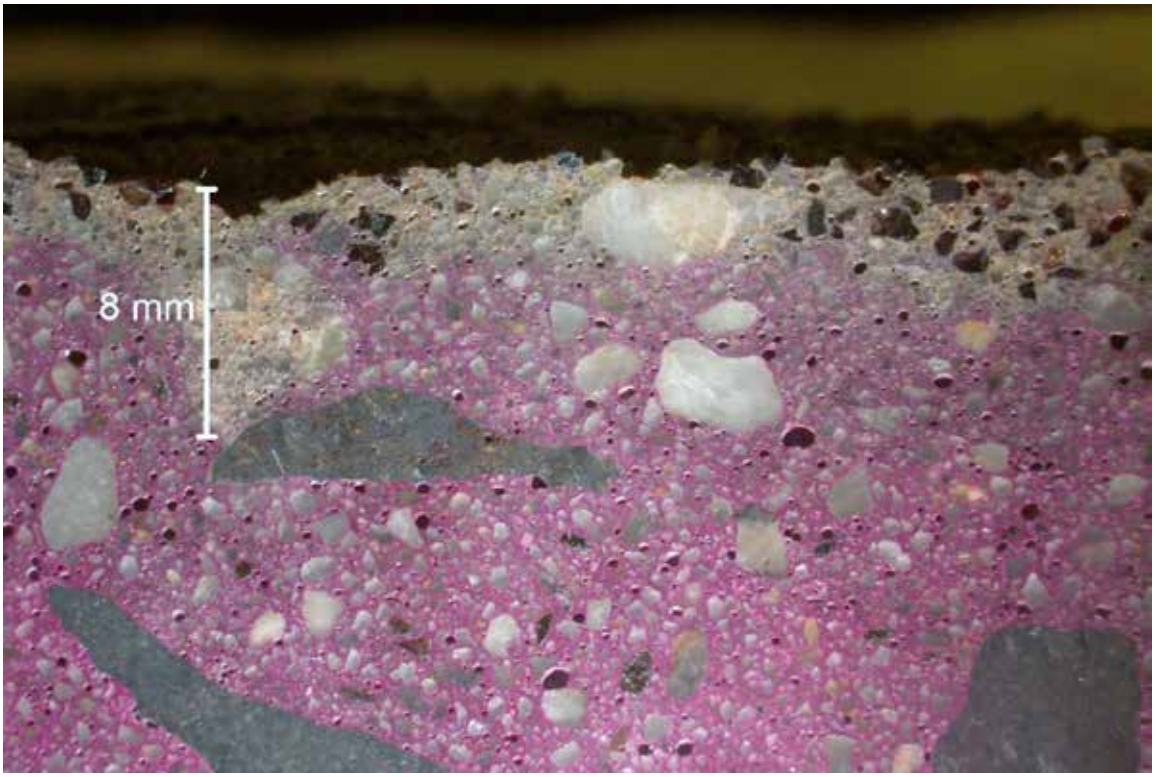
PHOTO: 5



SAMPLE ID: 1
MAG: 5x

DESCRIPTION: Carbonation (unstained) proceeds up to 8 mm depth from the present top surface proximate to sub-vertical microcracking. Dark red-colored grains of emery (corundum) were worked into the top surface (white arrows) during finishing of the concrete.

PHOTO: 6



SAMPLE ID: 7
MAG: 5x

DESCRIPTION: Carbonation (unstained) proceeds up to 8 mm depth from the present top surface proximate to sub-vertical microcracking. Note the dark red-colored grains of emery can be seen near the top surface of the concrete.

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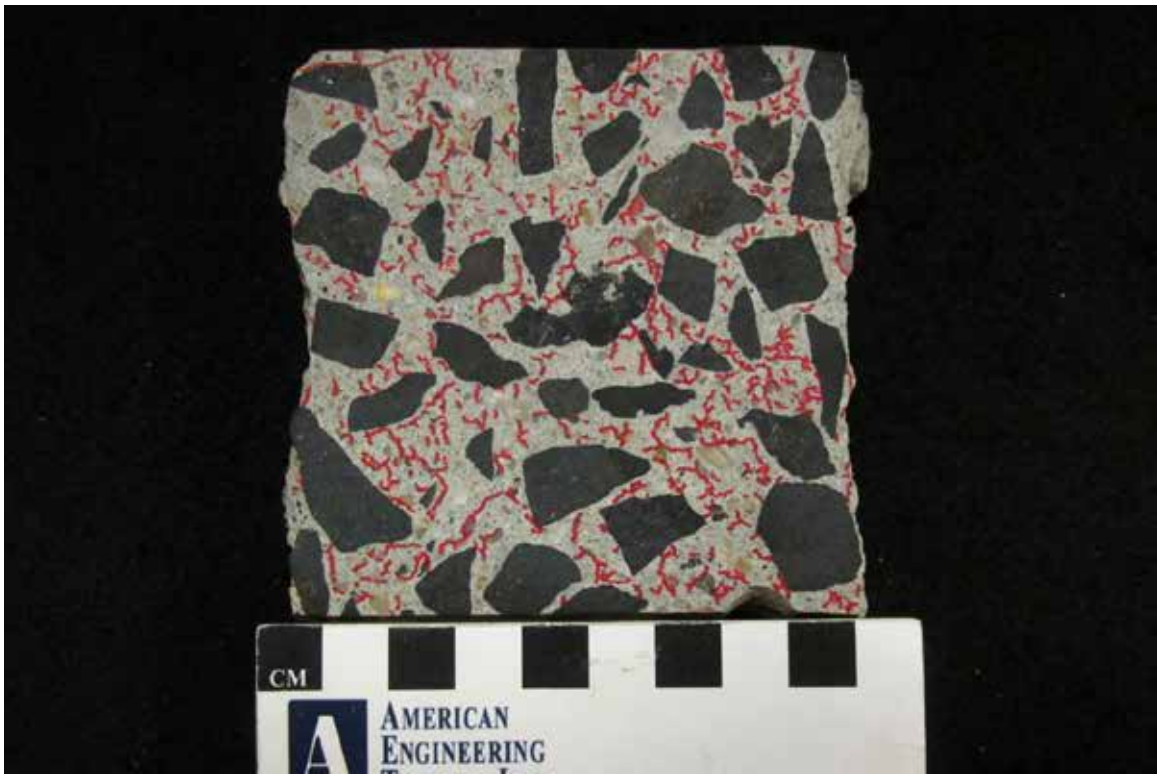
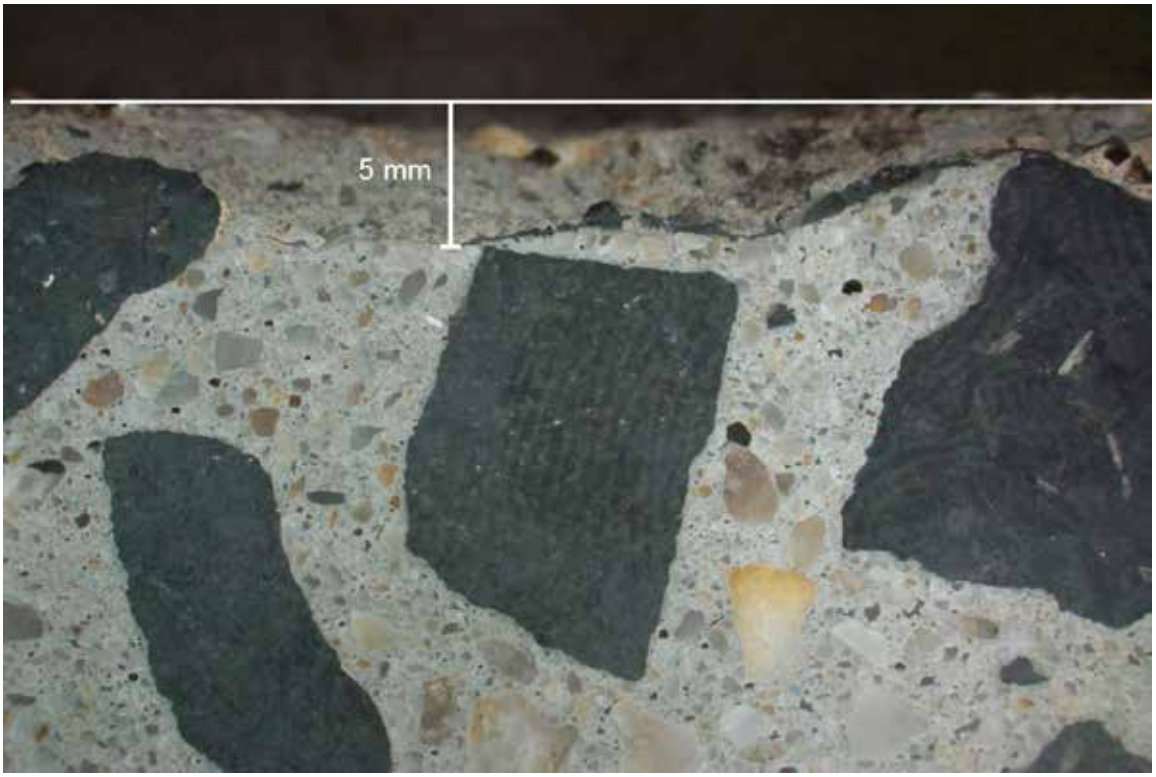
PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

PHOTO: 7



SAMPLE ID: 1 **DESCRIPTION:** Abundant shrinkage microcracks mapped in red ink on a lapped surface of the concrete. The top surface is to the left.

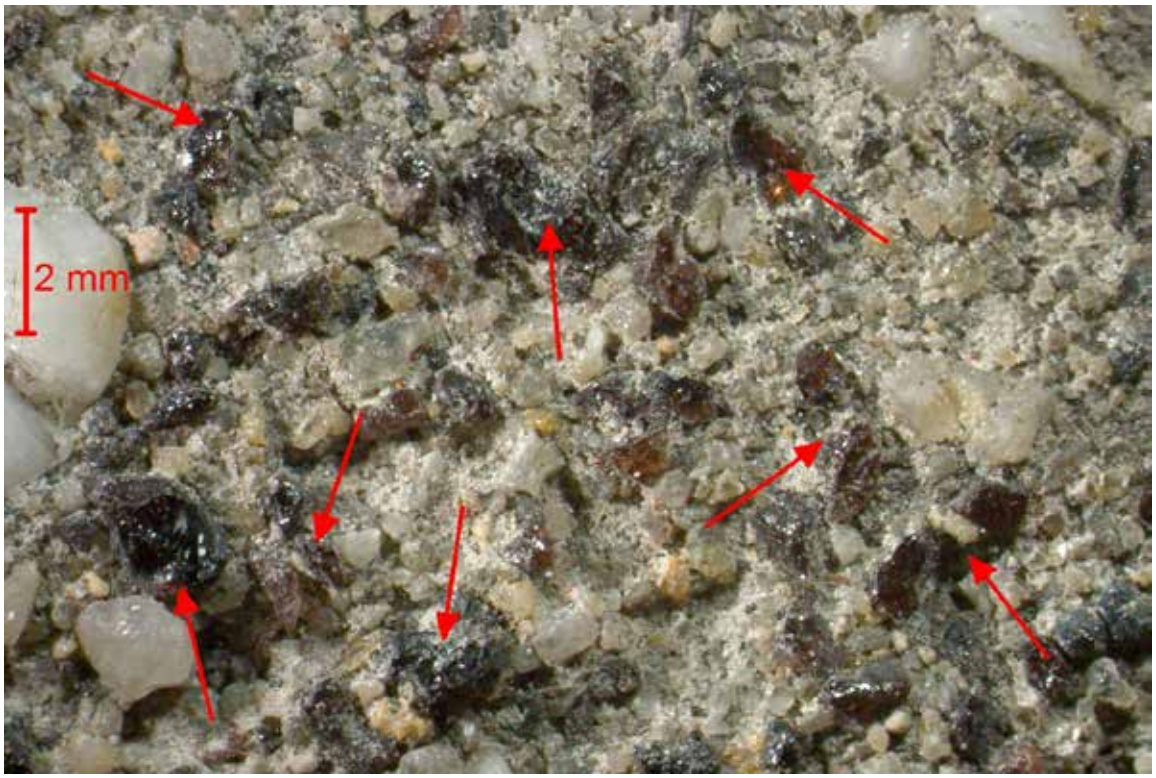
PHOTO: 8



SAMPLE ID: 1 **DESCRIPTION:** Scaling of the concrete was observed up to 5 mm depth from the present top surface of the concrete. This is possibly due to procurement of the sample.
MAG: 5x

APS JOB NO: 10-08215 **DATE:** JUNE 19, 2014
PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

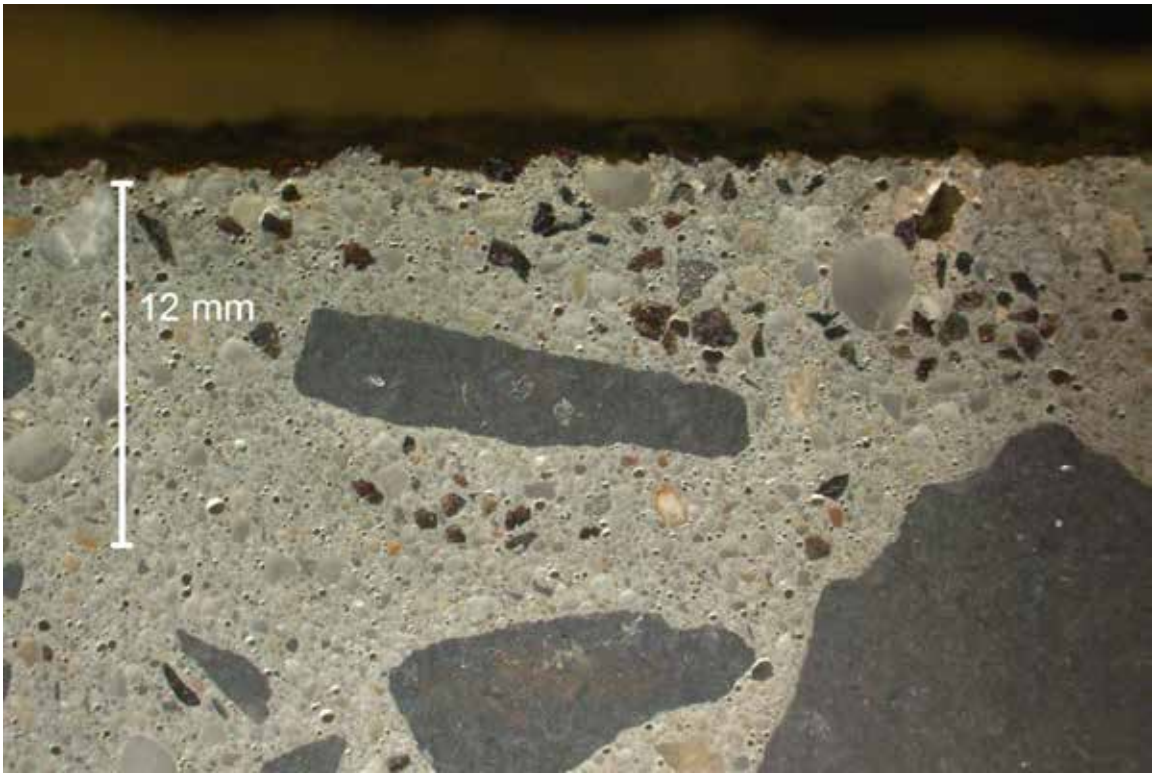
PHOTO: 9



SAMPLE ID: 7
MAG: 10x

DESCRIPTION: Dark red-colored grains of emery (red arrows) exposed on the top surface of the sample.

PHOTO: 10



SAMPLE ID: 7
MAG: 5x

DESCRIPTION: The grains of emery were generally observed down to 12 mm into the concrete from the top surface.

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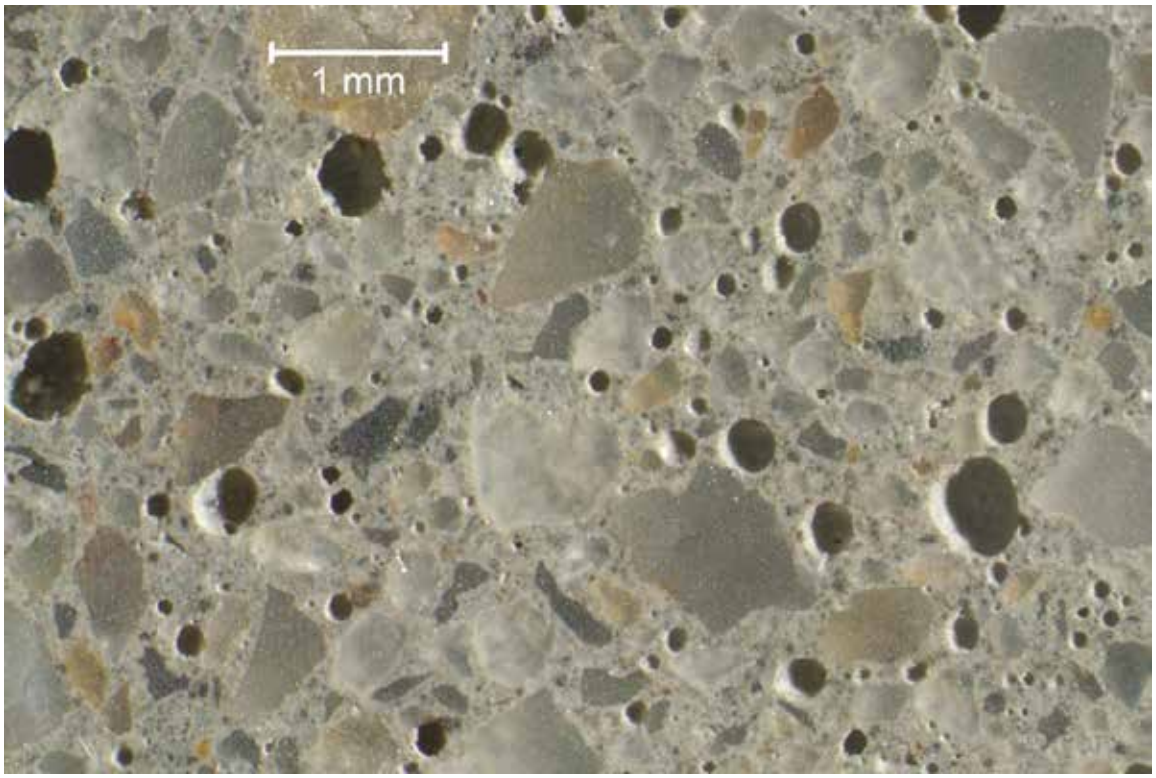
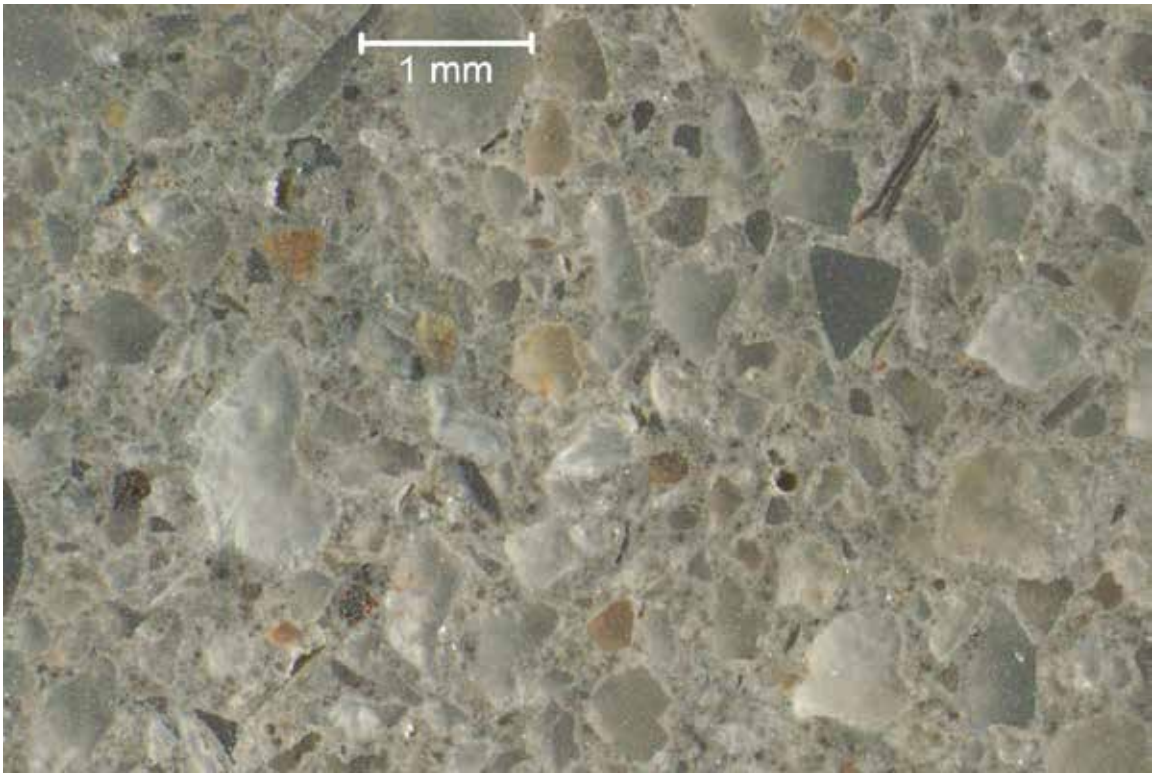
PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

PHOTO: 11



SAMPLE ID: 7
MAG: 30x
DESCRIPTION: Air entrainment in the top half of the sample. The concrete appears to be purposefully air entrained.

PHOTO: 12



SAMPLE ID: 7
MAG: 30x
DESCRIPTION: Lack of air entrainment in the bottom half of the core, also note the slightly darker paste color.

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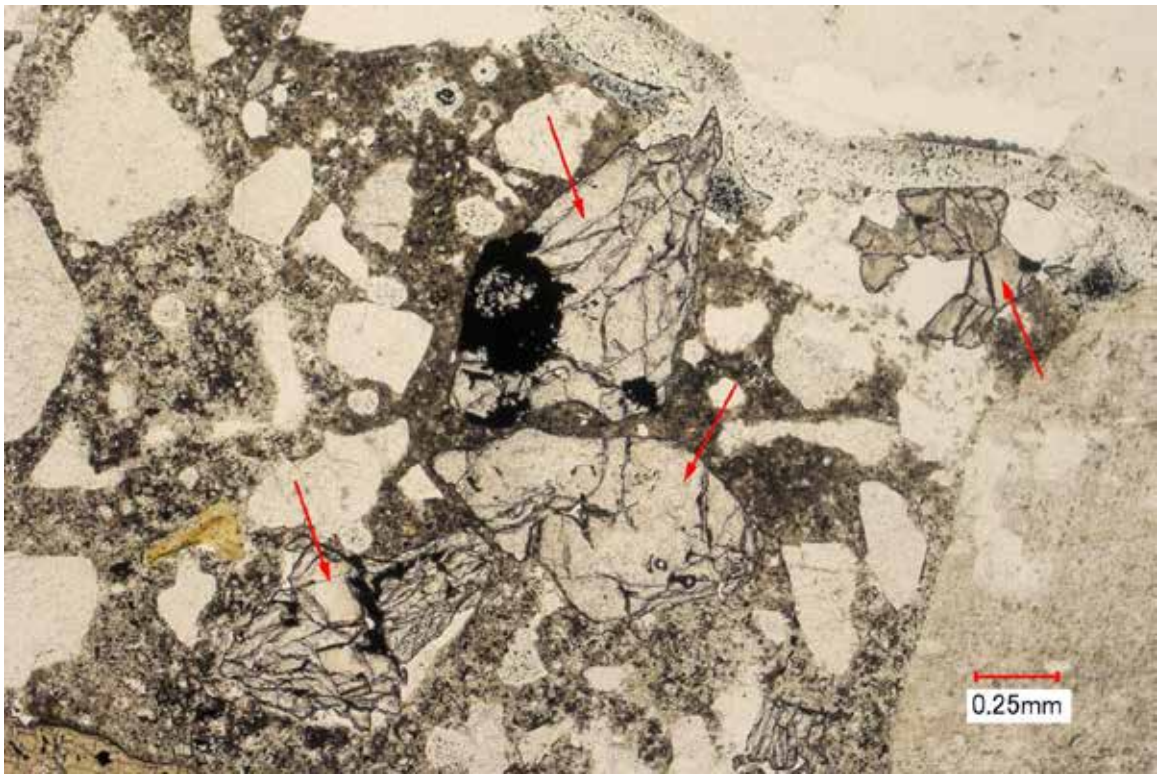
PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

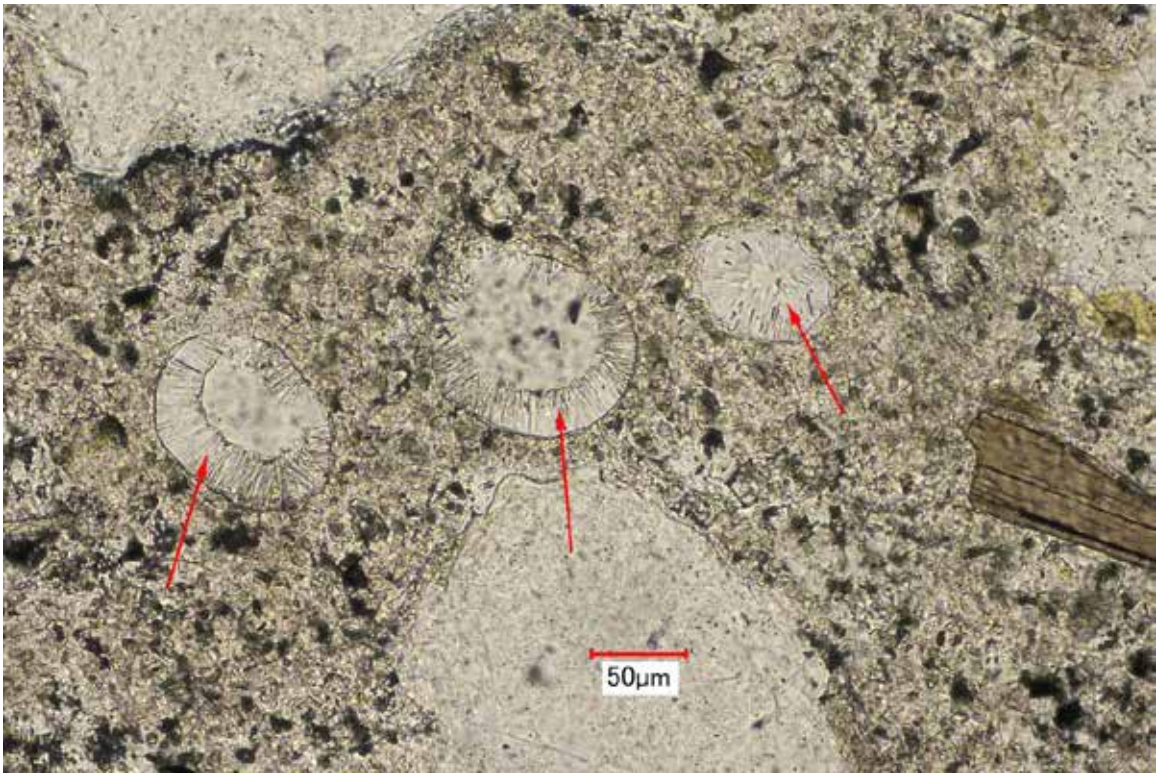
PHOTO: 13



SAMPLE ID: 1
MAG: 40x

DESCRIPTION: Sand-sized grains of angular emery (red arrows) proximate to the top surface in thin section of concrete under transmitted plane polarized light.

PHOTO: 14

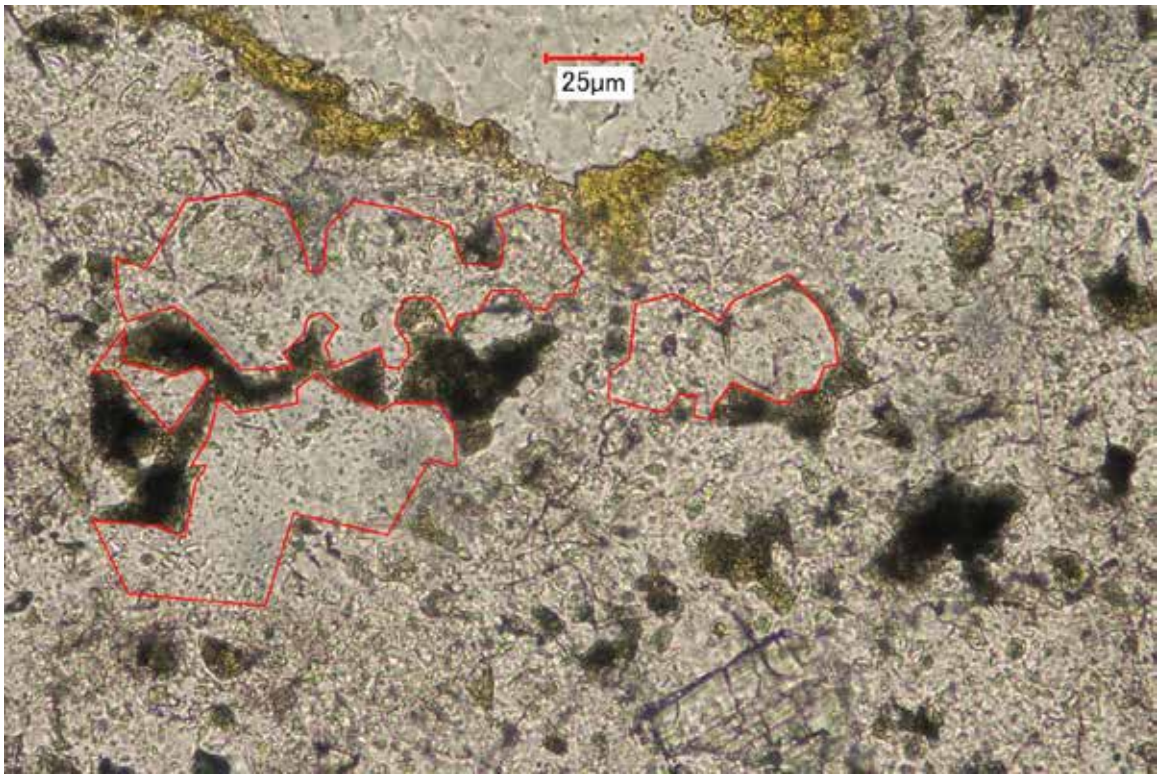


SAMPLE ID: 1
MAG: 200x

DESCRIPTION: Acicular ettringite partially filling entrained-sized air voids (red arrows) in thin section of concrete under transmitted plane polarized light.

APS JOB NO: 10-08215 **DATE:** JUNE 19, 2014
PROJECT: HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY JOB NUMBER: 46100

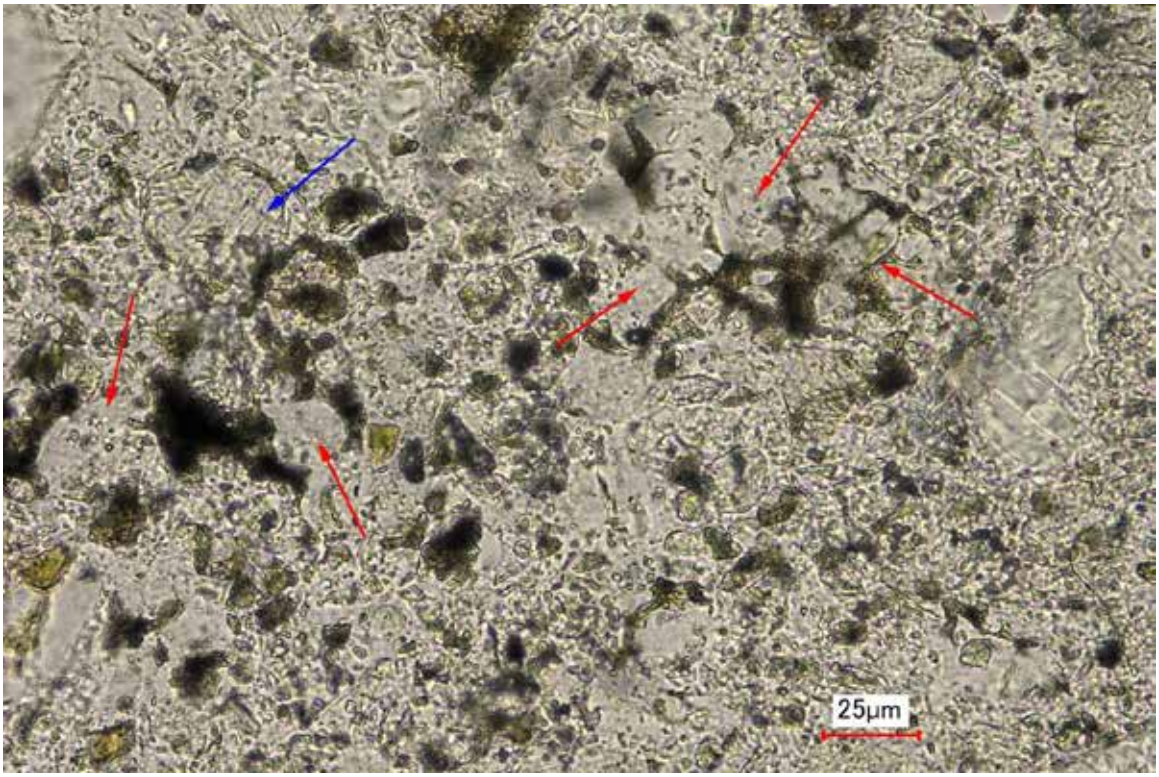
PHOTO: 15



SAMPLE ID: 1
MAG: 400x

DESCRIPTION: Fully hydrated portland cement clinker relicts (red outlines) in thin section of concrete under transmitted plane polarized light.

PHOTO: 16



SAMPLE ID: 7
MAG: 400x

DESCRIPTION: Fully hydrated alite portland cement clinker relicts (red arrows) and fully hydrated belite portland cement clinker relict (blue arrow) in thin section of concrete under transmitted plane polarized light.

AMERICAN ENGINEERING TESTING, INC.

CONSULTANTS
- ENVIRONMENTAL
- GEOTECHNICAL
- MATERIALS
- FORENSICS

REPORT OF FREEZING AND THAWING OF CONCRETE CORES

PROJECT:

HAKS-PASSERELLE PEDESTRIAN
QUEENS, NY

REPORTED TO:

CRAIG TESTING LABORATORIES, INC
5439 HARDING HIGHWAY
MAYS LANDING, NJ 08330-0427

ATTN: JOHN SPECHT

APS PROJECT NO: 10-08215.02

DATE: SEPTEMBER 23, 2014

INTRODUCTION

This report presents the test results of freezing and thawing of concrete cores. You requested and authorized the testing on May 15, 2014. The scope of our work was limited to conducting freeze-thaw testing on two concrete cores that were identified by you as #2 and #4.

TEST PROCEDURES

Testing of the specimens was conducted in accordance with ASTM C666/C 666M–03(2008), "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing, Procedure A – Freezing and Thawing in Water." Relative dynamic modulus readings could not be obtained on the samples provided. Mass measurements were used to evaluate the cores and visual observations. Test results are provided below after 319 cycles of freezing and thawing.

TEST RESULTS at 310 CYCLES

Sample Identification	Mass Change, %	Visual Observation
#2	-10.95	Significant Deterioration
#4	0.02	No Deterioration

REMARKS

We will retain the specimens until October 23, 2014 at which time they will be discarded unless we hear otherwise from you. We appreciate this opportunity to conduct specialized testing for you. Should you have any questions, please contact us.

Report Prepared By:
American Engineering Testing, Inc.

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PHOTOGRAPHS
APS PROJECT NO. 10-08215.02

Photo 1: Sample #2 after 319 freeze thaw cycles

Photo 2: Sample #4 after 319 Freeze thaw cycles

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· FORENSICS

REPORT OF CHEMICAL ANALYSIS

PROJECT:

HAKS – PASSERELLE PEDESTRIAN BRIDGE
QUEENS, NY
JOB NUMBER: 461000

REPORTED TO:

CRAIG TESTING LABORATORIES, INC.
5439 HARDING HIGHWAY, PO BOX 427
MAYS LANDING, NJ 08330\

ATTN: JOHN SPECHT

APS PROJECT NO: 10-08215

DATE: JUNE 6, 2014

INTRODUCTION

This report presents the results of laboratory work performed by our firm on two concrete cores submitted to our laboratory by Mr. John Specht of Craig Testing Laboratories on May 15, 2014. The scope of our work was limited to documenting the chloride content profile in accordance with ASTM C1218, "Standard Test Method for Water-Soluble Chloride in Mortar and Concrete."

TEST RESULTS

Sample Identification	Water-Soluble Chloride by wt. % of sample	(ppm) mg/Kg
Core 3 (0-1”)	0.099	991
Core 3 (1-2”)	0.083	829
Core 5 (0-1”)	0.064	643
Core 5 (1-2”)	0.039	392

TEST PROCEDURES

Laboratory testing was performed on June 3, 2014 and subsequent dates. Testing was performed in accordance with ASTM C1218, "Standard Test Method for Water-Soluble Chloride in Mortar and Concrete." The concrete core was cut at the marked depths as measured from the top surface. The slices were crushed, oven dried, then ground to a powder passing a #20 US Standard Sieve.

REMARKS

The test samples will be retained for a period of at least thirty days from the date of this report. Unless further instructions are received by that time, the samples may be discarded. The test result relates only to the samples tested. No warranty, express or implied, is made.

American Engineering Testing, Inc.

Bill Rebel
Principal Chemist
Phone: 651-603-6633
brebel@amengtest.com

Appendix F: Subsurface and Foundation Evaluation Report

Foundation Investigation

Pre-Scoping Design Services for the Passerelle Pedestrian Bridge
Queens, New York

Prepared for:

New York City Department of
Design and Construction

and

HAKS-Weidlinger JV

Prepared by:

Weidlinger Associates, Inc.
Consulting Engineers
40 Wall Street, 19th Floor
New York, New York 10005

July 22, 2014

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B. Logs of Nearby Test Borings
C. Logs of Test Pits Excavated for this Study
D. Photos of Test Pits and Existing Piles
E. Pile Foundation Summary Table

I Introduction

A. Background

This report summarizes our investigation of the capacity and condition of the existing foundation supporting the Passerelle Pedestrian Bridge and provides recommendations for reuse as part of the proposed bridge reconstruction. The location and orientation of the bridge are shown on the **Project Locus, Figure 1**. The overpass was originally constructed in about 1937 for the 1939 New York World's Fair. The original overpass was supported on 15 ton creosote treated timber piles. In about 1961, the bridge underwent a major reconstruction in preparation for the 1964 New York World's Fair. At this time, the eastern end of the bridge was entirely rebuilt to make a larger and wider bridge, with a ramped entrance that includes the Passerelle Building. New 20 ton capacity creosote treated timber piles were driven to support the new section of bridge. At the same time, the 1937 superstructure along the western half of the bridge was completely removed and replaced, with only the foundation pile caps from the 1937 construction left in place. Piles with batters of approximately 1 horizontal to 5 vertical exist at many of the pile caps to resist lateral loads.

Select pile groups at the western half of the bridge (bent lines 1 to 18) were load tested to confirm capacity. Some pile caps were reconstructed after cutting down damaged portions of the supporting piles. It appears that the damage was splintered tops that may have been the result of the original pile installation.

Currently, the bridge has a wooden deck over the western half while a concrete deck was used for the eastern portion of the bridge. We understand that the original 1937 wood deck was replaced at the time of the 1961 reconstruction. However, neither portion of the deck has been replaced since that 1964.

B. Proposed Rehabilitation

The existing bridge is to be rehabilitated to provide an additional 75 years of service life. Three alternatives are currently being considered. These include:

- Replacement of the bridge deck in-kind (timber and concrete decks) with repairs as needed to the superstructure,
- Replacement of the entire deck with a new concrete deck with repairs, as needed, to the superstructure,
- Replacement of the existing deck, superstructure, columns, and limited replacement of foundation pile caps.

Each of the options will reuse the existing timber piles as well as existing pile caps to the extent possible. New piles would be added to supplement or replace existing piles as required due to loads or condition of the existing piles.

C. Project Datum

Elevations in the report refer to the Queens Datum which was used on the drawings and plans of the original phases of construction. The Queens Datum is 1.625 ft. above NAVD88 so to convert to NAVD88, 1.625 ft. should be added to all elevations referenced herein.

D. Purpose and Scope

The purpose of this study is to assess the general conditions related to the existing bridge foundations and capacities and make recommendations for reuse of the existing foundations and the design of supplemental foundations, if required, for support of renovation or reconstruction of the Passerelle Pedestrian Bridge.

Specifically, our scope of work includes the following:

- Review available plans and reports related to subsurface conditions at and around the area of the bridge and installation of existing piles supporting the bridge.
- Visit the site to assess the general foundation performance of the existing bridge. Excavate two test pits to assess the existing condition of the piles supporting the bridge.
- Prepare a summary report providing a description of subsurface soil conditions based on available information, recommendations for existing pile capacity and pile condition for reuse of existing piles, and design criteria for installation of new piles to supplement or replace existing piles if required to support additional loads.

E. Limitations

The report was prepared in accordance with our specific scope of work and generally accepted geotechnical engineering practices. No warranty, either express or implied, is made. Conditions at locations other than those of specific test borings or test pits may vary from conditions described in this report. In addition, pile condition and groundwater levels may change with time and other factors including construction, tide, climate and usage.

II Subsurface and Foundation Conditions

A. Soil and Bedrock Condition

Ground surface is at about El. 10. Subsurface conditions along the alignment of the pedestrian bridge include approximately 20 ft. of fill including a significant amount of ash underlain by about 5 to 10 ft. of organic deposits. Approximately 50 to 60 ft. of varved silts underlie the organic deposits with the lower 5 to 10 ft. of the silt layer containing some organic material. The entire site is underlain by medium dense sand at a depth of about 80 ft. becoming more clayey with depth. Bedrock was not encountered in any of the test borings.

All piles bear in the sand layer at depths of about 90 to 100 ft. below existing grade. No drive records, records of load tests or details on the hammers used to drive the piles are available. The pile cut offs are at about El. 7 to 8 which is above the groundwater level. Timber piles, with or without creosote, are susceptible to rot where above the permanent groundwater level.

A profile along the alignment of the bridge from the original test borings drilled for the project is included in **Appendix A**.

B. Groundwater Level

Groundwater was encountered at depths of about 7 to 10 ft. below grade.

C. Pile Installation and Capacity

Piles currently supporting the western half of the bridge (Bents 1 to 18) which were installed in about 1937 have a design allowable capacity of 15 tons while the piles installed in 1961 supporting the remaining eastern end of the bridge have a design allowable capacity of 20 tons. Normally, the capacity is considered as an axial capacity and in some cases, a reduction is required in assessing the vertical capacity of battered piles. However, given the relatively slight batter, it is believed reasonable that the driven axial capacity be considered to be a vertical capacity.

Normally individual pile load tests to confirm capacity require a factor of safety of 2.0, while pile load tests on groups of piles is based on a factor of safety of 1.5. No documentation exists related to individual pile load tests conducted at the time of installation, although it is assumed that these were completed. However, group load tests conducted in about 1961 on the original 1937 piles confirmed a <u>vertical</u> capacity of 15 tons per pile.

Plans showing the pile locations, batter, and as-installed depths are included in **Appendix B**.

D. Test Pits Conducted For This Study

In order to assess the current condition of the timber piles, two test pits were excavated on April 16, 2014 by Craig Testing of Mays Landing, NJ. Specifically the test pits were excavated to expose representative piles from each phase of installation to assess potential for rotting of the timber piles. Each test pit was excavated using a rubber tracked mini-excavator to a depth below the existing groundwater level.

Test Pit No. 1 was excavated along the southern side of Pier 5-L which is supported on the 15 ton capacity 1937 phase piles. The area below the bridge at the western end is paved and currently serving as a parking area. The general site conditions at this end of the site indicate that the soil has subsided, exposing part or all of the pile caps and in some cases even the piles themselves. Specifically at Pier 5-L, the top of the pavement was about 12 inches below the top of the pile cap. The cap was found to be 28 inches in thickness. The two southern piles were exposed. No substantial gap was found below the pile cap which allowed observation of other piles in the group. The test pit was excavated to a depth of about 4 ft. below the top of the pavement, which was at the groundwater level. Hand shoveling was used to clearly expose each pile.

The two exposed piles were about 12 inches in diameter at the butt. Inspection found the pile to be in sound condition from the groundwater level to the underside of the pile cap. A core sample of the western most pile was take using a 1/8 inch increment bore tool to a depth of about inches. No rot or other discontinuities were identified in the core sample.

The test pit was backfilled with the excavated soil upon completion of inspection and photographing. Cold patch was used to replace the asphalt parking area.

Test Pit No. 2 was excavated along the western side of Pier 27-I which is supported on the 20 ton capacity 1961 phase piles. This pier is on the western edge of a roadway below the bridge and is paved, although not specifically a parking area. While the ground surface around each of the piers at the eastern side of the bridge are slightly raised above the surrounding area, indicated some general settlement, none of the pile caps are exposed. The test pit was initially excavated to the bottom of the pile cap, a depth of about 5 ft. below the top of pavement. A 6 inch deep void was observed below the cap such that all five piles supporting the cap could be observed. None of the piles at this level exhibited any signs of rot. However the test pit was too confined to use the incremental bore tool. The test pit was then continued to a depth of about 10 ft. Groundwater was encountered at a depth of about 9 ft. The pile in the southwestern corner of the cap was exposed the full to the full depth of the test pit using the backhoe bucket.

All of the exposed piles were about 12 inches in diameter at the butt. Inspection found sound condition of all exposed piles at the butt. In addition, the southwestern pile was found to be in sound condition from the groundwater level to the underside of the pile cap.

The test pit was backfilled with excavated soil upon completion of inspection and photographing. Cold patch was used to replace the asphalt ground cover.

Logs of the test pits are included in **Appendix C**, while photos of conditions encountered are included in **Appendix D**.

E. Assessment of Current Pile Condition and Capacity

Based on the observed conditions of representative piles and in lieu of further load tests that might confirm some additional capacity, it should be assumed that the allowable vertical compressive load is 15 tons for pile bents 1 to 18 and 20 tons for the remainder of the piles. Due to lack of a positive connection between the pile cap and the piles (which is typical of timber piles), no uplift capacity should be assumed for any of the piles.

F. Seismic Design Classification

Based on the available test borings, the site should be classified as a Site Class "E", Soft Soil Profile. Based on the blow counts reported for the silty soils which are within 60 ft. of the groundwater level, it is believed that the soils below the site could be susceptible to liquefaction. It is believed that the liquefaction could cause some loss of strength for foundations bearing in this material as well as some seismically induced settlement. However, it is not believed that catastrophic failure or collapse of the strata itself is likely, avoiding being classified as a Site Class "F".

III Foundation Recommendations for Rehabilitation

A. General Considerations

If the rehabilitated bridge is not required to meet current seismic design criteria, the existing piles installed as part of the 1937 and 1961 design efforts can be assumed to provide 15 tons and 20 tons of allowable capacity under non-seismic conditions as discussed above. Site settlements indicate that some down drag is likely also being carried by the piles. This down drag is believed to be due to consolidation of the near surface organic soils and limited to the downward force of soils above the organics which is relatively small. While there is some down drag on the piles, it is believed that those settlements would have existed at the time of the group pile load tests conducted in 1961, and thus have adequate factor of safety, even with some level of conventional down drag.

However, at the time of the original installation, down drag due to seismic events was not considered and the piles do not have adequate capacity to resist down drag loads, which would be present during a seismic event. Current pile design typically includes down drag caused by the densification of liquefaction susceptible soils and those above any liquefiable zone. At this site, this zone may extend as deep as 60 ft. below grade and result in down drag forces which are equal to or exceed the design capacity of the piles. If current seismic design criteria must be met, then new pile foundations with capacity to resist seismically induced down drag loads must be installed at all locations. Detailed recommendations for new piles which meet seismic requirements will be discussed in more detail below.

B. Pile Overstress Potential

Provided the bridge does not need to meet current seismic design criteria, it is anticipated that existing piles can withstand some limited amount of overstress. Normally we anticipate that driven piles have up to 10 percent excess capacity based on normal driving specifications and additional “freeze” gain over time. It is our recommendation that at locations where bridge renovations or modifications create overloads which are 10 percent or less of the allowable capacity based on the allowable design load, that it can be assumed that some overload is possible. However, if overstress is to be assumed, then at least two piles should be tested to loads of 220 percent of the previously assumed design load to confirm this overstress.

C. Supplemental Drilled-In Piles (If Required)

Provided the bridge does not need to meet current seismic design criteria and overstress on the existing piles is more than 10 percent, new piles will be required to support the additional loads. The new piles should be relatively compatible with the existing timber piles with respect to compressibility. It is our recommendation that 7 inch (nominal) diameter drilled in mini-piles be used to provide additional support. The piles would be founded in the same lower sand stratum as the existing timber piles with allowable loads that could range from 20 to 50 tons depending on

the additional capacity required. The new piles could also be battered and provide tension capacity, if required.

In accordance with the NYC building code, at least two load tests would be required to confirm the capacity of the new piles in each area of usage. Thus if used in both the eastern and western areas of the site, potentially 4 pile load tests would be required. The specific number of load tests will be determined once the number and locations of new piles required has been determined.

D. Evaluation of Adequacy of Existing Pile Foundations for Future Loading

The pile group capacities of all pile foundations are shown in **Table 1**. These available capacities are comparing to proposed approximate future loading conditions under the alternatives previously described in **Tables 2 and 3** which include the percentage of overstress (based on design loads which do not include current seismic loading factors). We assume that additional piles will be required where overstresses are greater than 110 percent which includes most of the pile foundations in the western half of the bridge (which were constructed circa 1937) and a few foundations in the eastern half of the bridge (which were constructed circa 1964) might require retrofitting.

E. Resistance of Lateral Loads using the Existing Piles

Where it is determined that the existing piles can be reused for support of the renovated bridge, lateral loads should be resisted by the battered component of the existing pile groups. Passive pressure is not generally strain compatible with the much stiffer resistance of battered piles and should not be included where piles are battered. However, should a group of piles consist of all vertical piles, passive pressure on the sides of the foundation can be assumed as a pressure based on an equivalent fluid unit weight of soil of 300 pcf.

F. New Piles with Seismic Capacity

If it is determined that the renovated bridge must meet current seismic criteria, then it is recommended that new piles be installed and existing piles abandoned. New Piles would include an allowance for potential down drag from liquefaction induced settlements. It is possible for a large variety of drilled-in or driven piles to be used at this site; however, in consideration for the adjacent facilities, it is currently recommended that for planning purposes drilled-in mini-piles be considered for this options. Again, with new construction a large range of pile capacities is possible depending on the loads required. Anticipating that the general layout of columns will remain the same with column loads ranging from 100 to 300 tons and anticipating that 3 to 4 piles are required under each foundation, it is recommended that 9-5/8 inch diameter piles with a capacity of 120 tons be used. We anticipate that a down drag allowance of 40 tons will be required to meet seismic design criteria resulting in an allowable, usable pile capacity of 80 tons.

The piles could be battered to resist lateral loads similar to the existing design. Up to 40 tons of uplift may be assumed for each pile. Alternatively, lateral resistance of vertical piles of up to 4 tons each may be assumed for piles spaced at least three feet on center.

G. Re-grading and Protection of Exposed Piles

Site settlements have occurred throughout the area. Specifically in the western portion of the site, pile caps have become exposed and in some cases the piles themselves are exposed. It is recommended, where possible, the site is regarded to cover existing piles and pile caps to provide protection against elements.

IV Recommendations

A. Additional Test Pits and Additional Load Tests

Only a limited investigation was possible as part of this study. In addition, many areas of piles were not accessible due to rail facilities or other restrictions to open excavations. Therefore, if it is elected to reuse the existing timber piles, it is recommended that additional test pits be incorporated as an early phase of the construction to confirm our assessment that the piles are in good condition at additional locations. Should any rotted piles be identified as part of this further conformation, piles should be cut off below the area of any rot and the pile caps extended to the new cut off depth. As mentioned in sections III.A and III.B above, additional load tests would be recommended, too.

B. Long-Term Monitoring Program

It is recommended that a vertical settlement reference points be installed on all piers supporting the bridge so that a program of long-term monitoring can be conducted to help identify potential rotted piles. It is recommended that the vertical settlement be monitored yearly. Any pier showing more than 1.0 inches of settlement should be more closely investigated. Further investigation should include an additional test pit to confirm the condition of the piles.

TABLES

TABLE 1

Pile Foundation Summary Tables
Passerelle Pedestrian Bridge

Pile capacities from past contract drawings

References:
1) 1937-1940 Original Foundation Construction Drawings ST-101 thru ST-105, ST-111, ST-111 shts 1 thru 5
2) 1961 Willets Point Pedestrian Overpass Foundation Drawings no. 18 to 21

West Half	Piles in Pile Group				Pile group capacity*				
Bent	J	K	L	N/M	J tons	K tons	L tons	N/M tons	Comments
1	4	7	8	4	60	105	120	60	
2	5	10	12	5	75	150	180	75	1K load-tested at 160T in 1961
3	5	9	12	6	75	135	180	90	3J load-tested at 110T, 3L load-tested at 270T in 1961
4	NA	11	10	4	NA	165	150	60	4N load-tested at 120T in 1961
5	NA	10	8	4	NA	150	120	60	5L load-tested at 160T in 1961
6	NA	9	7	4	NA	135	105	60	6K load-tested at 210T in 1961
7	NA	7	6	NA	NA	105	90	NA	7L load-tested at 135T in 1961
8	NA	6	6	NA	NA	90	90	NA	
9 to 17	NA	5	5	NA	NA	75	75	NA	11L, 14K, and 17L all load-tested at 110T
18	NA	7	7	NA	NA	105	105	NA	

*According to dwg. ST-101, design based on pile bearing capacity of --> 15 tons

East Half	Piles in Pile Group									Pile group capacity**								
Bent	A	B	C	D	E	F	G	H	I	A tons	B tons	C tons	D tons	E tons	F tons	G tons	H tons	I tons
19	NA	NA	NA	NA	5	8	5	NA	NA	NA	NA	NA	NA	100	160	100	NA	NA
20	NA	NA	NA	NA	14	15	15	13	10	NA	NA	NA	NA	280	300	300	260	200
20	NA	NA	NA	NA	14	15	15	13	10	NA	NA	NA	NA	280	300	300	260	200
22	NA	NA	NA	14	13	13	14	14	13	NA	NA	NA	280	260	260	280	280	260
23	NA	NA	NA							NA	NA	NA						
24	NA	8	8	8	8	8	8	8	5	NA	160	160	160	160	160	160	160	100
25 26	13	16	15	15	15	15	15	16	13	260	320	300	300	300	300	300	320	260
27	5	9	9	9	9	9	9	9	5	100	180	180	180	180	180	180	180	100
28	5	10	10	9	9	9	10	10	5	100	200	200	180	180	180	200	200	100
29	8	6	9	9	5	9	9	6	8	160	120	180	180	100	180	180	120	160

**According to dwg. No. 18, design based on pile bearing capacity of --> 20 tons

TABLE 2

Pile Foundation Summary Tables

Passerelle Pedestrian Bridge

Pile capacity vs. demand - Alternate 1: Timber/Concrete Deck Replaced In-Kind

West Half	Pile group capacity*				Future Column load/demand				Future Column load/demand - %			
Bent	J	K	L	N/M	J	K	L	N/M	J	K	L	N/M
	tons	tons	tons	tons	tons	tons	tons	tons				
1	60	105	120	60	32	**110**	**130**	36	53%	**105%**	**109%**	60%
2	75	150	180	75	56	**179**	**214**	63	75%	**120%**	**119%**	84%
3	75	135	180	90	60	**155**	**197**	73	80%	**115%**	**109%**	81%
4	NA	165	150	60	NA	165	**165**	**61**	NA	100%	**110%**	**102%**
5	NA	150	120	60	NA	**187**	**192**	**73**	NA	**125%**	**160%**	**121%**
6	NA	135	105	60	NA	**159**	**158**	**64**	NA	**118%**	**151%**	**107%**
7	NA	105	90	NA	NA	**143**	**141**	NA	NA	**136%**	**157%**	NA
8	NA	90	90	NA	NA	**104**	**104**	NA	NA	**116%**	**116%**	NA
9 to 17	NA	75	75	NA	NA	**97**	**97**	NA	NA	**129%**	**129%**	NA
18	NA	105	105	NA	NA	**120**	**120**	NA	NA	**114%**	**114%**	NA

*According to dwg. ST-101, design based on pile bearing capacity of --> 15 tons

Assumptions

Column loads are DL + pedestrian LL of 100 psf

Assume LIRR platform at Bents 22/23 and 25/26 have LL of 100 psf

Red values denotes Future Load exceeds Capacity

Yellow highlighted columns exceed 110% of capacity --> Retrofit

Assume new concrete deck is normal weight

East Half	Pile group capacity**									Future Column load/demand								
Bent	A	B	C	D	E	F	G	H	I	A	B	C	D	E	F	G	H	I
	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons
19	NA	NA	NA	NA	100	160	100	NA	NA	NA	NA	NA	NA	89	157	75	NA	NA
20	NA	NA	NA	NA	280	300	300	260	200	NA	NA	NA	NA	97	138	113	106	86
21	NA	NA	NA	NA	280	300	300	260	200	NA	NA	NA	NA	101	113	124	123	94
22	NA	NA	NA	280	260	260	280	280	260	NA	NA	NA	184	219	219	243	255	187
23	NA	NA	NA							NA	NA	NA						
24	NA	160	160	160	160	160	160	160	100	NA	74	106	129	129	134	156	**172**	**102**
25	260	320	300	300	300	300	300	320	260	208	266	231	219	259	269	252	262	243
26																		
27	100	180	180	180	180	180	180	180	100	72	140	135	130	130	130	137	143	70
28	100	200	200	180	180	180	200	200	100	92	174	173	151	144	151	173	174	92
29	160	120	180	180	100	180	180	120	160	76	105	114	111	93	111	114	104	77

**According to dwg. No. 18, design based on pile bearing capacity of --> 20 tons

	Future Column load/demand - %								
Bent	A	B	C	D	E	F	G	H	I
19	NA	NA	NA	NA	89%	98%	75%	NA	NA
20	NA	NA	NA	NA	35%	46%	38%	41%	43%
21	NA	NA	NA	NA	36%	38%	41%	47%	47%
22	NA	NA	NA	66%	84%	84%	87%	91%	72%
23	NA	NA	NA						
24	NA	46%	66%	80%	80%	83%	98%	**107%**	**102%**
25	80%	83%	77%	73%	86%	90%	84%	82%	93%
26									
27	72%	78%	75%	72%	72%	72%	76%	79%	70%
28	92%	87%	86%	84%	80%	84%	86%	87%	92%
29	48%	87%	63%	62%	93%	62%	63%	87%	48%

TABLE 3

Pile Foundation Summary Tables

Passerelle Pedestrian Bridge

Pile capacity vs. demand - Alternate 2 or 3: New Deck throughout with overlay or pavers

West Half	Pile group capacity*				Future Column load/demand				Future Column load/demand - %			
Bent	J	K	L	N/M	J	K	L	N/M	J	K	L	N/M
	tons	tons	tons	tons	tons	tons	tons	tons				
1	60	105	120	60	43	154	180	49	71%	146%	150%	82%
2	75	150	180	75	76	253	300	86	101%	169%	166%	115%
3	75	135	180	90	85	221	279	102	113%	164%	155%	114%
4	NA	165	150	60	NA	234	237	89	NA	142%	158%	148%
5	NA	150	120	60	NA	260	268	102	NA	173%	224%	170%
6	NA	135	105	60	NA	220	220	89	NA	163%	210%	149%
7	NA	105	90	NA	NA	198	196	NA	NA	188%	217%	NA
8	NA	90	90	NA	NA	144	144	NA	NA	161%	160%	NA
9 to 17	NA	75	75	NA	NA	133	133	NA	NA	178%	178%	NA
18	NA	105	105	NA	NA	149	149	NA	NA	142%	142%	NA

*According to dwg. ST-101, design based on pile bearing capacity of --> 15 tons

Assumptions

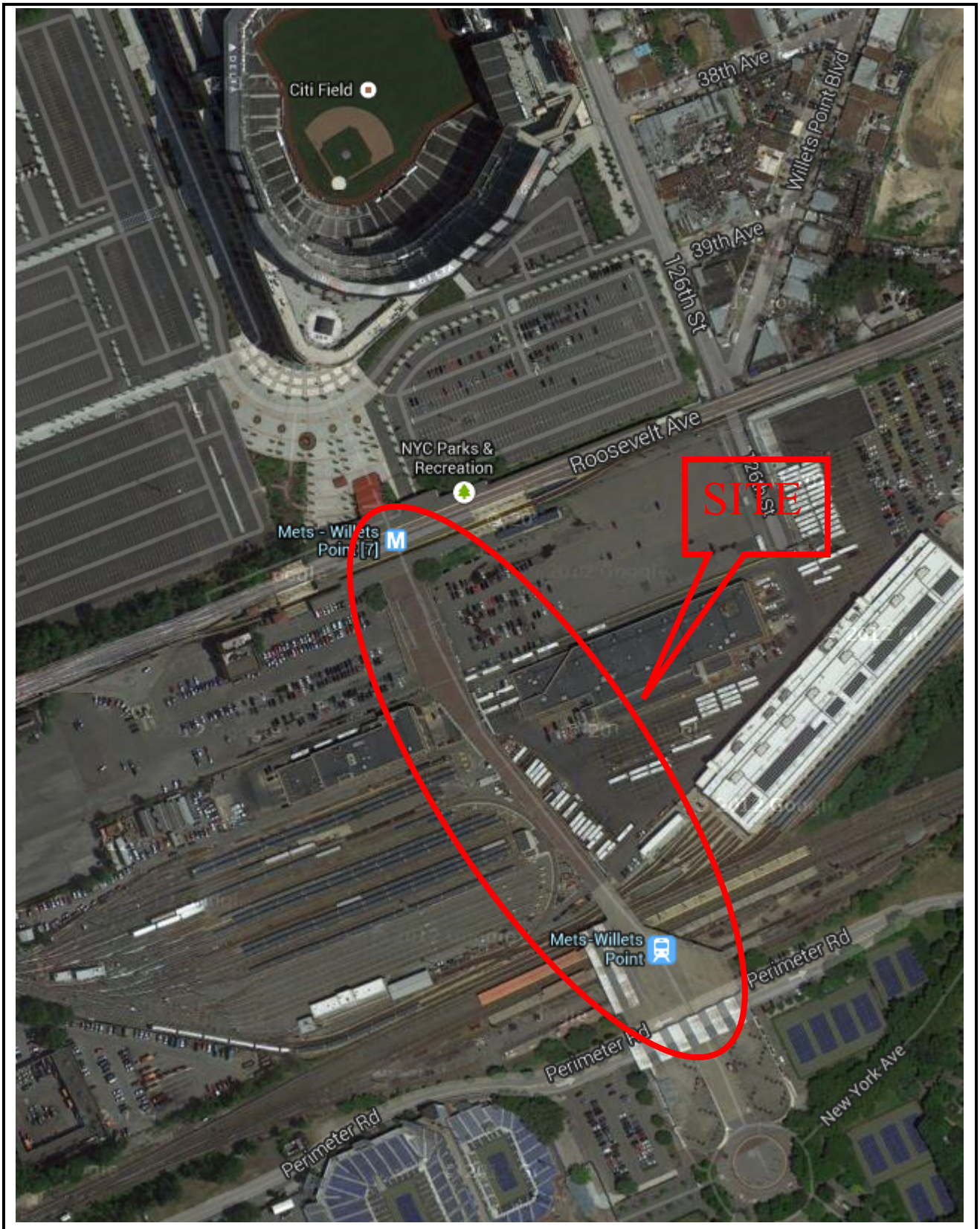
Assume 7" new deck + 3" overlay/pavers
Column loads are DL + pedestrian LL of 100 psf
Assume LIRR platform at Bents 22/23 and 25/26 have LL of 100 psf
Red values denotes Future Load exceeds Capacity
Yellow highlighted columns exceed 110% of capacity --> Retrofit
Assume new concrete deck is normal weight

East Half	Pile group capacity**									Future Column load/demand								
Bent	A	B	C	D	E	F	G	H	I	A	B	C	D	E	F	G	H	I
	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons	tons
19	NA	NA	NA	NA	100	160	100	NA	NA	NA	NA	NA	NA	112	198	94	NA	NA
20	NA	NA	NA	NA	280	300	300	260	200	NA	NA	NA	NA	113	165	129	118	94
21	NA	NA	NA	NA	280	300	300	260	200	NA	NA	NA	NA	118	132	144	142	105
22	NA	NA	NA	280	260	260	280	280	260	NA	NA	NA	214	255	255	280	292	208
23	NA	NA	NA							NA	NA	NA						
24	NA	160	160	160	160	160	160	160	100	NA	93	130	159	159	164	188	205	121
25	260	320	300	300	300	300	300	320	260	236	308	265	256	307	316	294	303	276
26																		
27	100	180	180	180	180	180	180	180	100	92	180	174	167	167	167	176	183	84
28	100	200	200	180	180	180	200	200	100	114	215	213	189	182	189	213	215	114
29	160	120	180	180	100	180	180	120	160	90	129	137	134	115	134	137	129	91

**According to dwg. No. 18, design based on pile bearing capacity of --> 20 tons

	Future Column load/demand - %								
Bent	A	B	C	D	E	F	G	H	I
19	NA	NA	NA	NA	112%	124%	94%	NA	NA
20	NA	NA	NA	NA	40%	55%	43%	45%	47%
21	NA	NA	NA	NA	42%	44%	48%	55%	53%
22	NA	NA	NA	76%	98%	98%	100%	104%	80%
23	NA	NA	NA						
24	NA	58%	81%	99%	99%	102%	118%	128%	121%
25	91%	96%	88%	85%	102%	105%	98%	95%	106%
26									
27	92%	100%	96%	93%	93%	93%	98%	102%	84%
28	114%	108%	106%	105%	101%	105%	106%	108%	114%
29	56%	108%	76%	74%	115%	74%	76%	107%	57%

FIGURES



WEIDLINGER ASSOCIATES INC
CONSULTING ENGINEERS

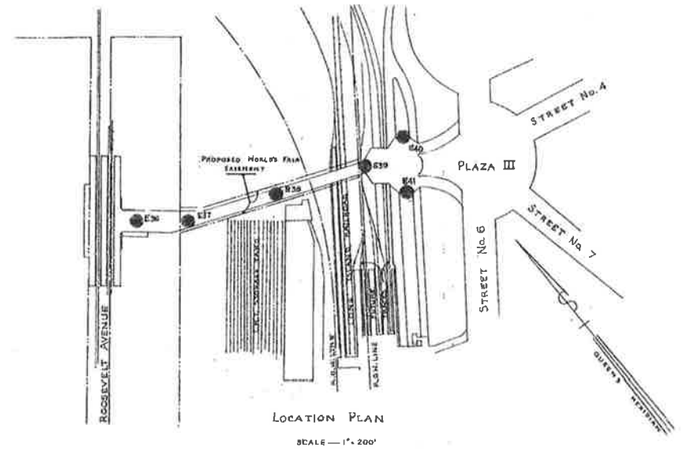
27 Wormwood Street, Suite 200
Boston, MA 02210

http://www.wai.com

FIGURE 1
Project Locus
Passerelle Pedestrian Bridge
Queens, New York

APPENDIX A - Logs of Test Borings along Bridge Alignment

REVISIONS			PRINTS ISSUED			
No.	Date	Description	To	Date	Copies	Pcks.



LOCATION PLAN

SCALE — 1" = 200'

BORING N° E36

2½" Casing

300# Casing Hammer 24" Fall

300# Spoon Hammer 24" Fall

BORING N° E37

2½" Casing

300# Casing Hammer 24" Fall

300# Spoon Hammer 24" Fall

BORING N° E38

4" & 2½" Casing

300# Casing Hammer 24" Fall

300# Spoon Hammer 24" Fall

BORING N° E39

2½" Casing

300# Casing Hammer 24" Fall

300# Spoon Hammer 24" Fall

BORING N° E40

2½" Casing

300# Casing Hammer 24" Fall

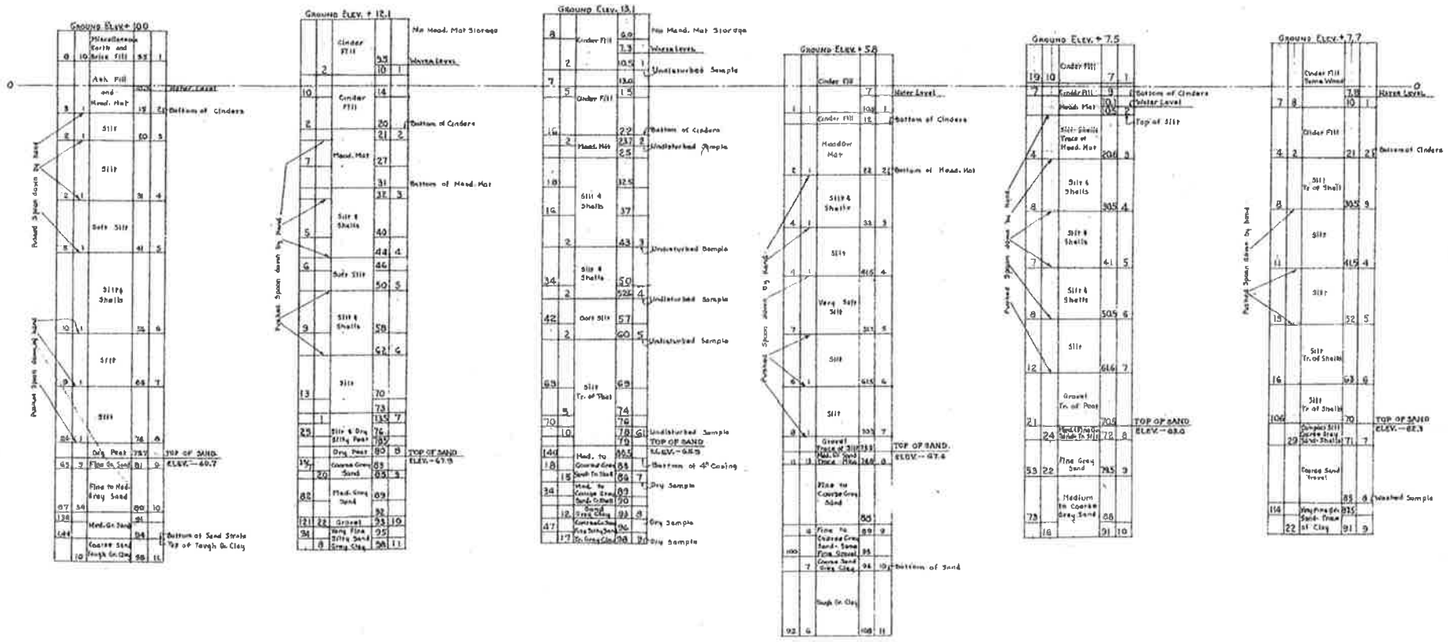
300# Spoon Hammer 24" Fall

BORING N° E41

2½" Casing

300# Casing Hammer 24" Fall

300# Spoon Hammer 24" Fall



NOTES

Figures in Left Hand Column indicate the number of blows on Casing per 12" Penetration.
Figures in next Column indicate the number of blows on Spoon per 12" Penetration.
Figures in Right Hand Column indicate Soil Sample Number.
Figures in next Column indicate depth below Surface of Ground.
Water Levels shown, unless marked "Sand", are taken before porous strata was pierced- after Sand was reached the Water level fluctuates, probably due to tidal action.

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

EXPLORATORY BORINGS FOR RAMP
PLAZA III TO I.R.T. & B.M.T. SUBWAY STATION

Sprague & Hinrichs, Inc. — CONTRACT # C-56

Designed by: FIELD INSPECTION SECTION

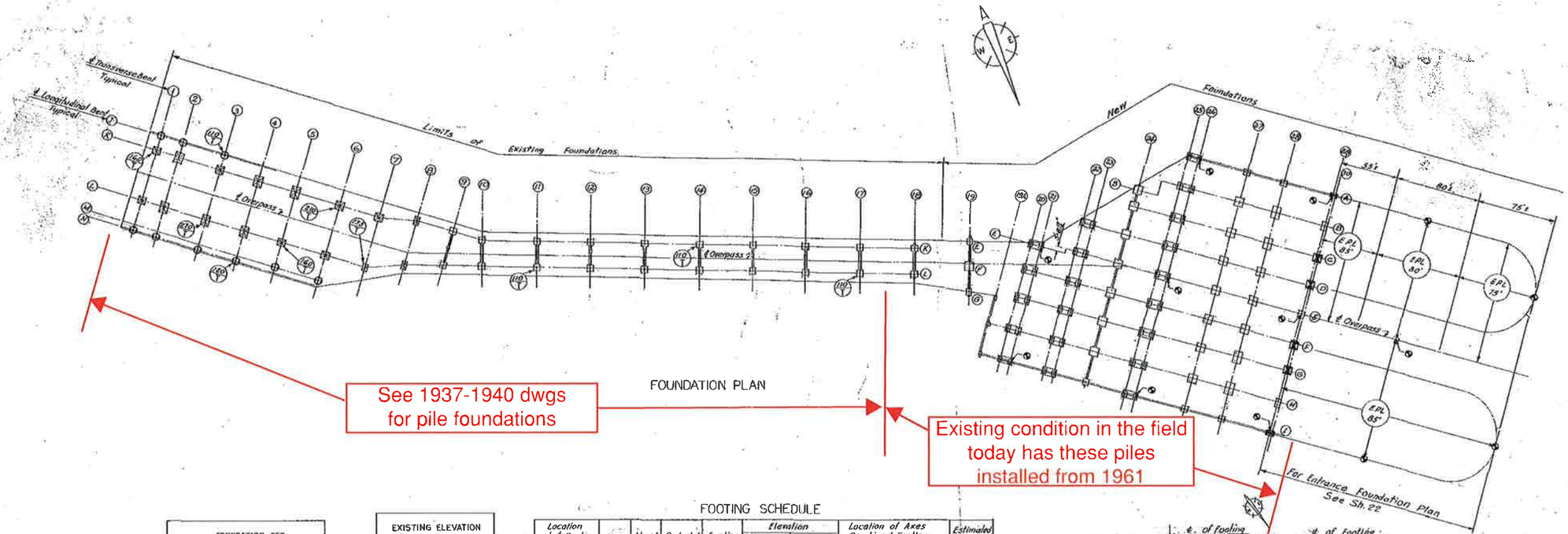
Drawn by: D.P.M.
Checked by: P.B.R.
Cross checked by:
Submitted by: R.J. CARELLO
Scale: VERTICAL SCALE: 1" = 10'
Date: JULY 3, 1937

Approved for Construction
Chief Engineer

Location: X-M

Drawing No.: B-145
Sheet ___ of ___

APPENDIX B - Plans of Existing Piles



FOUNDATION PLAN

See 1937-1940 dwgs for pile foundations

Existing condition in the field today has these piles installed from 1961

FOUNDATION KEY	
	EXISTING PILE GROUP TO REMAIN NEW FOOTING TYPE E
	EXISTING FOUNDATION-REPLACE SPALLED CONCRETE TIE A.O.B.E
90 T	GROUP PILE TEST-NO. INDICATES TEST LOADING IN TONS
	EXISTING FOUNDATION TO REMAIN
	NEW FOUNDATION TYPE C-L COMBINED WITH L.I.R.R.
	NEW FOUNDATION TYPE C-E COMBINED WITH ENTRANCE
	NEW FOUNDATION TYPE G SINGLE
	NEW FOUNDATION TYPE S-CW SINGLE & CRASH WALL

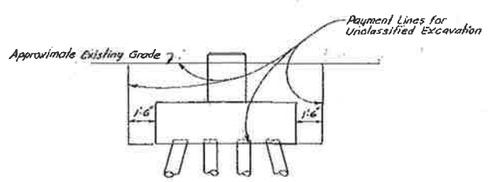
EXISTING ELEVATION	
Transverse Bent	Bottom of Col. Base ft
1, 2	12.50
3, 4	12.00
5, 6, 7	11.00
8 to 16	14.00
17	10.50

Note: All Existing Elevations Shown above to be verified by Contractor.

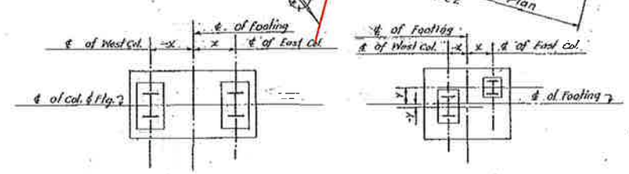
FOOTING SCHEDULE

Location at ℄ Bents Transv.	Longi.	Type	No. of Piles	Pedestal Height	Footing Thickness	Elevation Bottom of Col. Base ft	Bottom of Footing	Location of Axes Combined Footing Left Col. -x	y	Right Col. x	y	Estimated Pile Length (EPL)
1	J&N	E	4*	4'-6"	2'-0"	12.50	6.00	-	-	-	-	
2	J&N		5*	4'-6"	2'-0"	12.50		-	-	-	-	
3	J&N		5*	2'-6" 4'-0"	2'-0" 2'-6"	12.00		-	-	-	-	Existing Piles
4	N		4*	4'-0"	2'-0"	12.00		-	-	-	-	
5	M		4*	3'-0"	2'-0"	11.00		-	-	-	-	
6	M	E	4*	3'-0"	2'-0"	11.00	6.00	-	-	-	-	
18	K&L	S-CW	7**	3'-9½"	2'-6"	9.12	3.00	-	-	-	-	
19	E&G	S-CW	3	2'-1½"	2'-0"	9.12	5.00	-	-	-	-	
	F	S-CW	8	2'-1½"	2'-6"	9.12	4.50	-	-	-	-	
19A	G&J	S-CW	2	2'-0"	2'-0"	6.37	2.37	-	-	-	-	
	H	S-CW	3	2'-0"	2'-0"	6.37	2.37	-	-	-	-	
20 & 21	E	C-L	14	5'-0"	2'-6"	10.42	2.92	5'-0"	-	5'-0"	-	
	F&G		15					5'-4"	-	4'-8"	-	
	H		13					5'-6"	-	4'-6"	-	100'
	I		10					5'-3"	-	4'-9"	-	
	O		14					5'-0"	-	5'-0"	-	
22 & 23	E		13						-		-	
	F		13						-		-	
	G		14						-		-	
	H		14						-		-	
	I	C-L	13	5'-0"		10.42	2.92	5'-0"	-	5'-0"	-	
24	B&H	S-CW	8	2'-0"	2'-6"	6.37	1.87	-	-	-	-	
	I	S-CW	5	2'-0"	2'-0"	6.37	2.37	-	-	-	-	
25 & 26	A	C-L	13	5'-0"	2'-6"	10.42	2.92	4'-0"	-	5'-4"	-	
	B		16					5'-4"	-	4'-8"	-	
	C		15						-		-	
	D		15						-		-	
	E		15						-		-	95'
	F		15						-		-	
	G		15						-		-	
	H		16					5'-4"	-	4'-8"	-	
	I	C-L	13	5'-0"	2'-6"	10.42	2.92	5'-0"	-	5'-0"	-	
27	A	S	5	2'-0"	2'-0"	7.00	3.00	-	-	-	-	
	B&H		9	1'-6"	2'-6"			-	-	-	-	
	I		5	2'-0"	2'-0"			-	-	-	-	
28	A		8	2'-0"	2'-0"			-	-	-	-	
	B		10	1'-0"	3'-0"			-	-	-	-	
	C		10	1'-0"	3'-0"			-	-	-	-	60'
	D		9	1'-6"	2'-6"			-	-	-	-	
	E		9	1'-6"	2'-6"			-	-	-	-	
	F		9	1'-6"	2'-6"			-	-	-	-	
	G		10	1'-0"	3'-0"			-	-	-	-	
	H		10	1'-0"	3'-0"			-	-	-	-	
	I	S	5	2'-0"	2'-0"	7.00	3.00	-	-	-	-	
29	A	C-E	8	1'-0"	3'-0"	7.08	3.08	11½"	-	11"	8"	
	B	S	6	1'-6"	2'-6"			-	-	-	-	
	C	C-E	9	1'-0"	3'-0"			10½"	17¾"	12½"	-22"	
	D	C-E	9	1'-0"	3'-0"			10½"	-18¼"	12½"	21¾"	
	E	S	5	2'-0"	2'-0"			-	-	-	-	85'
	F	C-E	9	1'-0"	3'-0"			10½"	18¼"	12½"	-21¾"	
	G	C-E	9	1'-0"	3'-0"			10½"	-17¾"	12½"	22"	
	H	S	6	1'-6"	2'-6"			-	-	-	-	
	I	C-E	8	1'-0"	3'-0"	7.08	3.08	11½"	-	11"	8"	

* Indicates number of existing piles - to remain.
** Indicates 2 New piles added and 5 existing piles to remain.



PAYMENT LINES FOR EXCAVATION
Not to Scale



TYPE C-L TYPE C-B

LOCATION OF COMBINED FOOTING
Not to Scale

Notes: 20 Tons

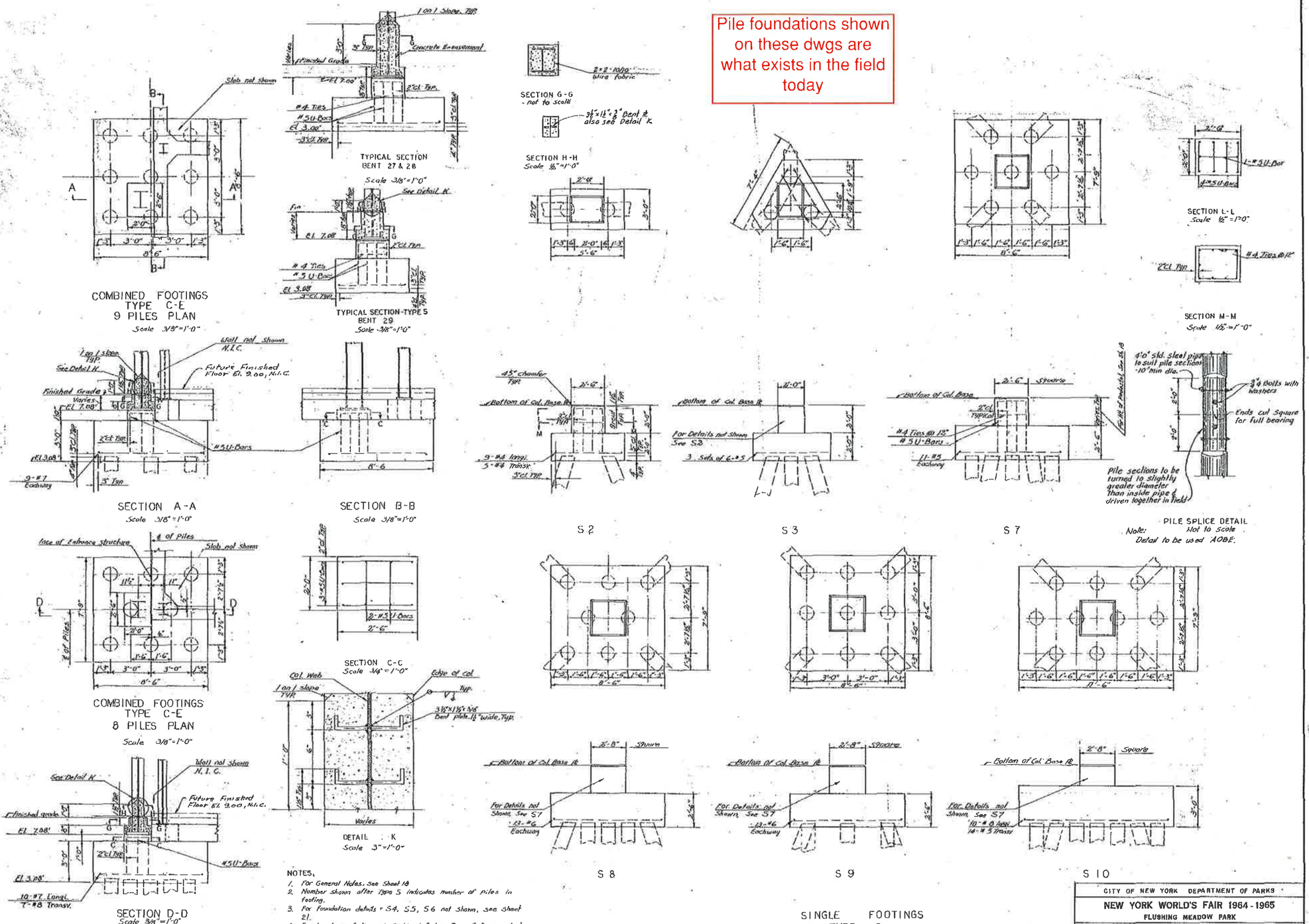
1. All Concrete for Foundations shall be Class B-32 Concrete.
2. Concrete placed under certain Existing Pile Footings due to ground settlement shall be Class D-18 Concrete. For detail, see Detail 'X' Sh. 20.
3. Designed Bearing Capacity of Creosoted Timber Pile to be 20 Tons.
4. Pile to be battered one horizontal to five vertical wherever indicated.
5. For Details of Foundation Type E, see Sh. 20.
6. For Details of Foundations at Bents 18, 19, and 19A, see Sh. 20 & 21
7. For Foundation Details - Type C-B and Type S, see Sh. 19.
8. For Foundation Details - Type C-L, Type S and Type S-CW, see Sh. 21.
9. Column to be placed at center of single footing and their Axes coincide.
10. Actual pile length to be determined by test piles driven adjacent to boring location shown as ⊗
11. For Locations of Columns and Bents, see Framing Plans, Shs. 10, 12, 13, & 14.
12. Where Interference with Existing Piles occurs, they Will be Pulled as Ordered by The Engineer and Paid for Under "Remove Existing Timber Piles."
13. For Pile Splice Detail, See Sh. 19.
14. Length of Borings shall be a minimum 10' below the Estimated Pile Length and 20' into sand.
15. Test Piles shall be driven 10' below Estimated Pile Length and shall be jetted, if ordered by the Engineer, to pierce any intermediate hard layer.

Rev. 3-8-62

CITY OF NEW YORK DEPARTMENT OF PARKS			
NEW YORK WORLD'S FAIR 1964-1965			
FLUSHING MEADOW PARK			
PEDESTRIAN OVERPASS WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK FOUNDATION PLAN AND DETAILS			
DRAWING NO. 18	CONTRACT NO. 9-39-861	DATE 11-10-61 SCALE 1"=40'	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK

In Charge of
Made by
Traced by
Checked by

Pile foundations shown on these dwgs are what exists in the field today



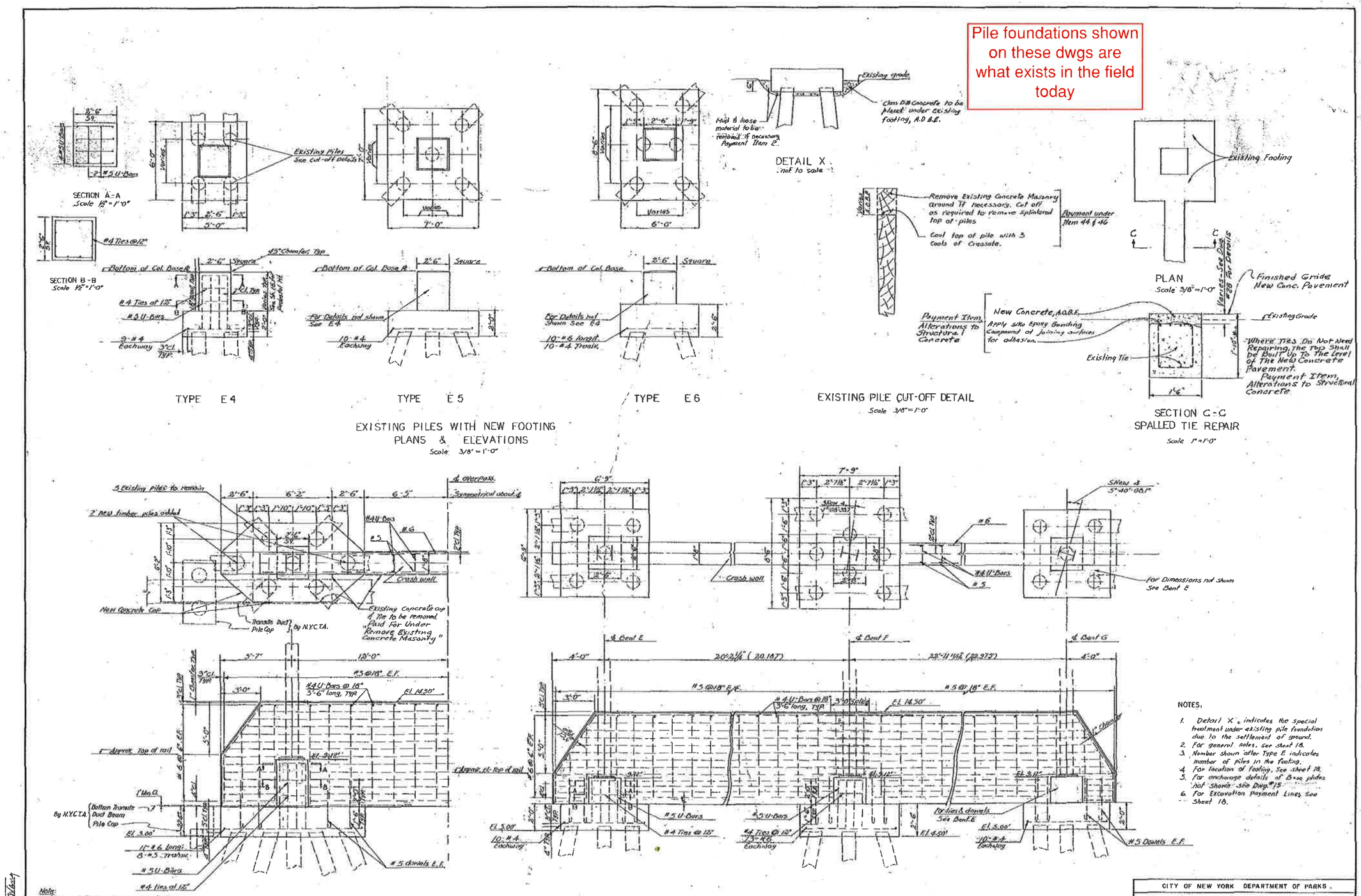
NOTES.

1. For General Notes, See Sheet 18
2. Number shown after Type S indicates number of piles in footing.
3. For foundation details - S4, S5, S6 not shown, see sheet 21.
4. For Location of Axes of Combined Footing Type C-E, see sheet 18
5. For anchorage details of Base Plates not shown, see sheet 15.
6. For Reinforcement in beams not shown, see sheet 23.
7. For Excavation Payment Lines, See Sheet 18.

Rev 3-8-62

CITY OF NEW YORK DEPARTMENT OF PARKS			
NEW YORK WORLD'S FAIR 1964-1965 FLUSHING MEADOW PARK			
PEDESTRIAN OVERPASS WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK FOUNDATION DETAILS + TYPE C-B AND TYPE S			
DRAWING NO. 15	CONTRACT NO. 9-99-861	DATE 11-10-61 SCALE As shown	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK

Pile foundations shown on these dwgs are what exists in the field today



NOTES:

1. Detail "X" indicates the special treatment under existing pile foundation due to the settlement of ground.
2. For general notes, see Sheet 18.
3. Number shown after Type E indicates number of piles in the footing.
4. For location of footing, See sheet 18.
5. For anchorage details of Base plates not shown see Dwg. #15.
6. For Excavation Payment Lines, See Sheet 18.

CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

PEDESTRIAN OVERPASS

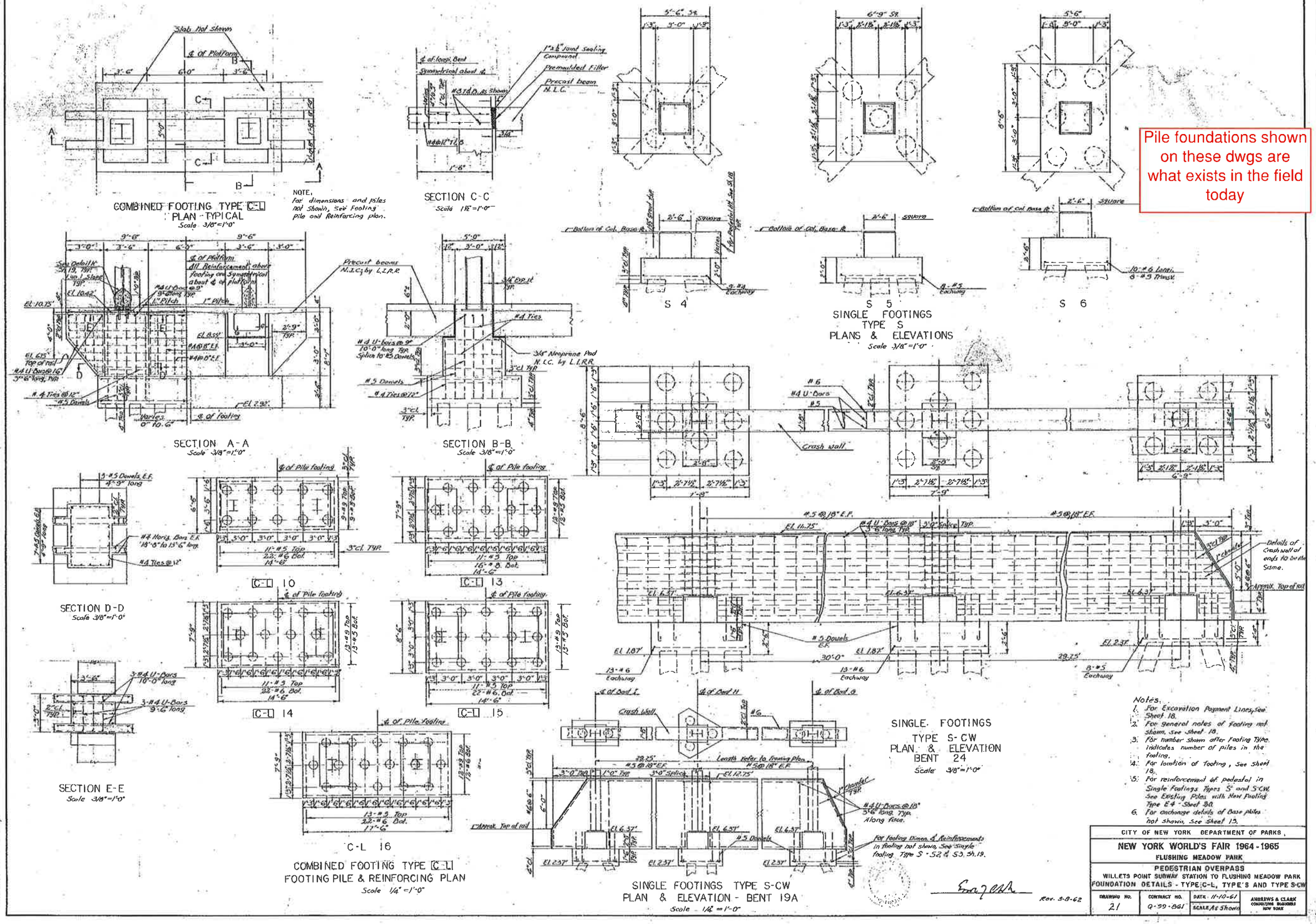
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK

FOUNDATION DETAILS - TYPE E, BENT 18 & BENT 19

DRAWING NO.	CONTRACT NO.	DATE	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
20	9-59-861	11-10-61	
		SCALE As Shown	

In Charge of: E.C. Zack
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]

Pile foundations shown on these dwgs are what exists in the field today



Notes.

1. For Excavation Payment Lines, See Sheet 18.
2. For general notes of footing not shown, see Sheet 18.
3. For number shown after footing Type, indicates number of piles in the footing.
4. For location of footing, See Sheet 18.
5. For reinforcement of pedestal in Single Footings Types S and S-CW See Existing Piles with New Footing Type E4 - Sheet 30.
6. For exchange details of Base plates not shown, See Sheet 15.

CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

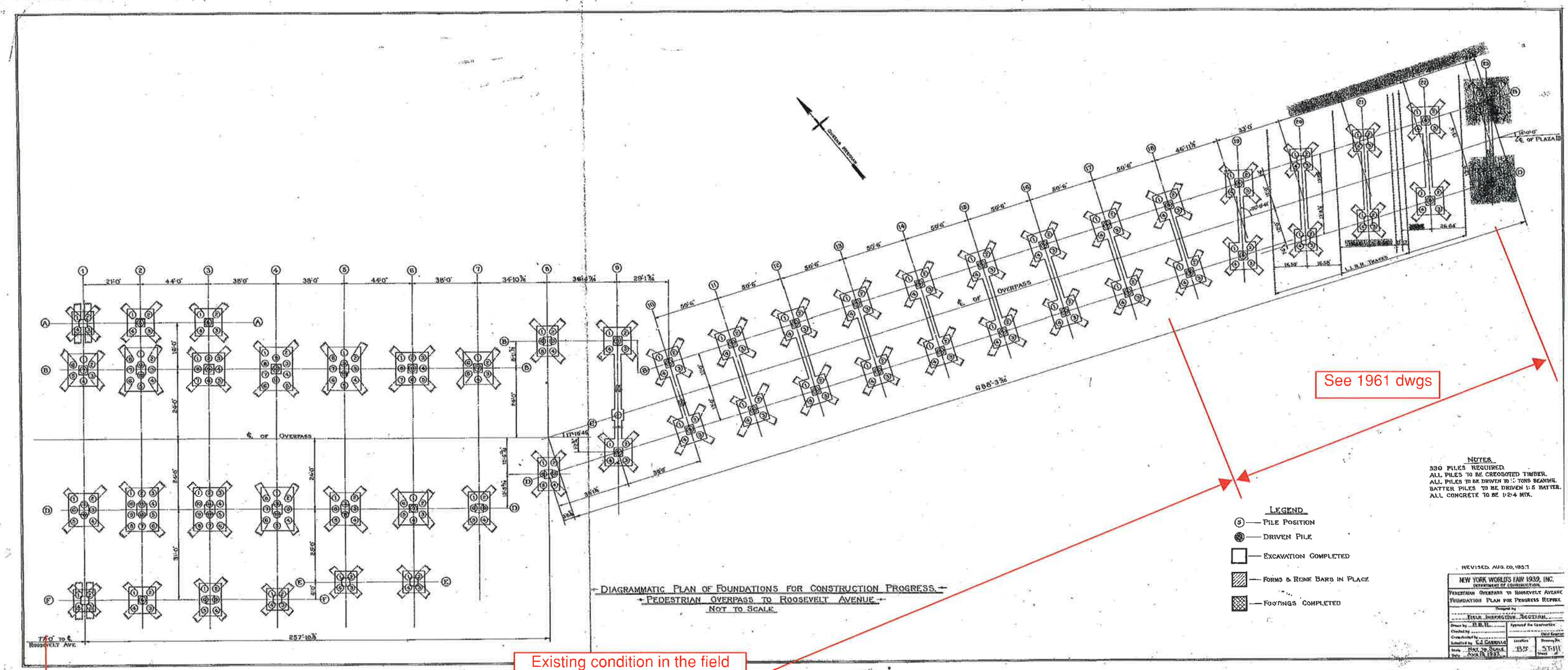
PEDESTRIAN OVERPASS

WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK

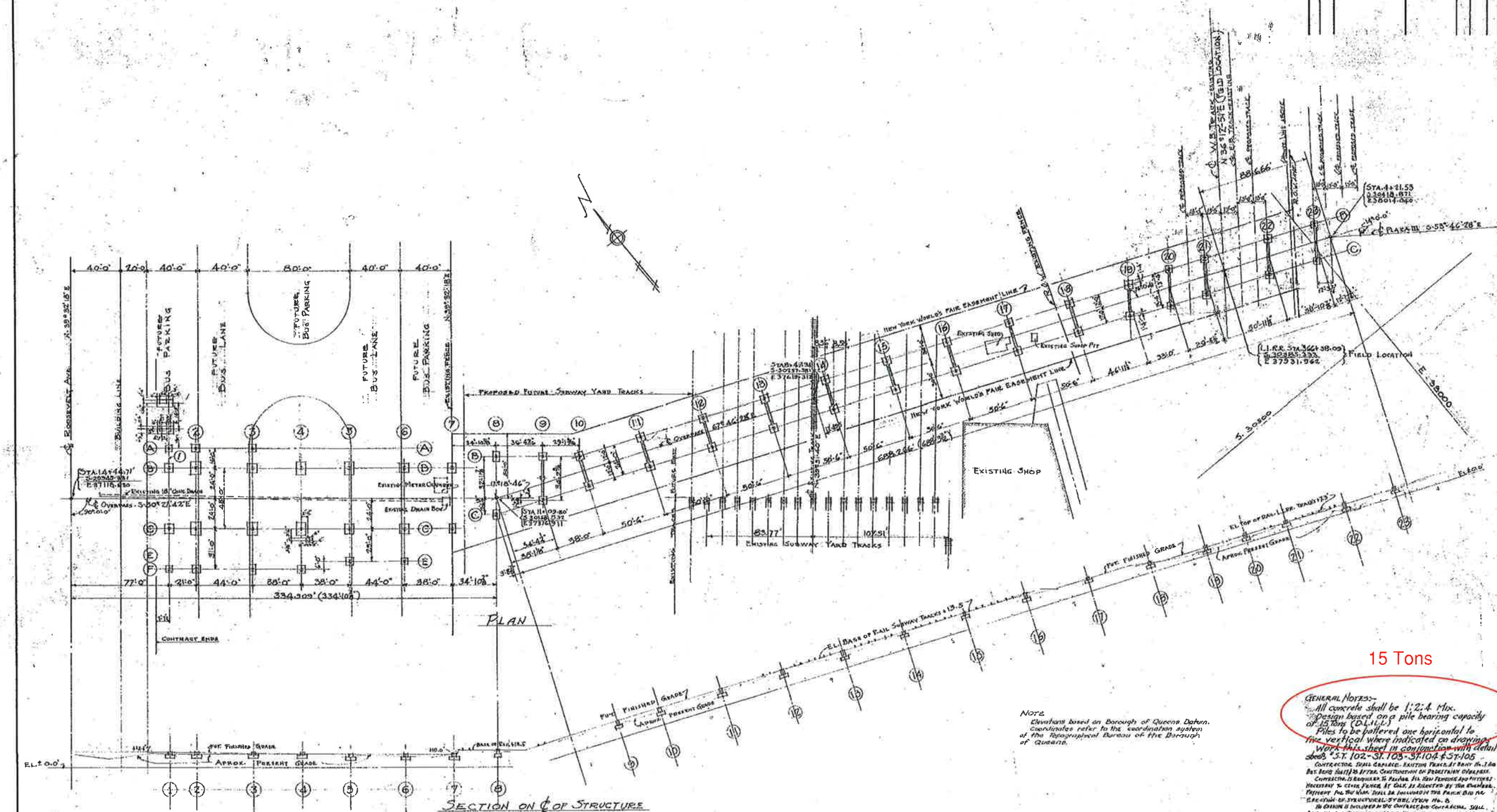
FOUNDATION DETAILS - TYPE C-L, TYPE'S AND TYPE S-CW

DRAWING NO.	CONTRACT NO.	DATE	SCALE	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
21	Q-99-661	11-10-61	As Shown	

In Charge of: [illegible]
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]


See 1961 dwgs
Existing condition in the field today has these piles installed from 1937-1940
DIAGRAMMATIC PLAN OF FOUNDATIONS FOR CONSTRUCTION PROGRESS
PEDESTRIAN OVERPASS TO ROOSEVELT AVENUE
NOT TO SCALE
℄ OF OVERPASS
℄ OF PLAZA
L.I.R.R. TRACKS
ROOSEVELT AVE
LEGEND
PILE POSITION
DRIVEN PILE
EXCAVATION COMPLETED
FORMS & REINF BARS IN PLACE
FOOTINGS COMPLETED
NOTES.
330 PILES REQUIRED.
ALL PILES TO BE CREOSOTED TIMBER.
ALL PILES TO BE DRIVEN TO 15 TONS BEARING.
BATTER PILES TO BE DRIVEN 1:5 BATTER.
ALL CONCRETE TO BE 1:2:4 MIX.
REVISED AUG. 20, 1937
NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION
PEDESTRIAN OVERPASS TO ROOSEVELT AVENUE
FOUNDATION PLAN FOR PROGRESS REPORT
FIELD INSPECTION SECTION
B.B.H.
E.J. CARDILLO
NOT TO SCALE
AUG. 6, 1937
B5
ST-111

REVISIONS			PRINTS ISSUED			
No.	Date	Description	To	Date	Copies	Pulls
1	4/15	LOCATION OF DIVES 21, 22, 23 CHANGED & LOCATION OF TRACKS CHANGED ETC.				



15 Tons

GENERAL NOTES:-
All concrete shall be 1:2:4 Mix.
Design based on a pile bearing capacity of 15 Tons (D.L.H.L.)
Piles to be jettered one horizontal to five vertical where indicated on drawings.
Work this sheet in conjunction with detail sheets S.T. 102-S.T. 103-S.T. 104 & S.T. 105

CONTRACTOR SHALL GUARANTEE EXISTING FENCE AT RIGHT HAND BUT FOOD SUITABLE DURING CONSTRUCTION OF PEDESTRIAN OVERPASS. CONTRACTOR IS REQUIRED TO PROVIDE ALL NEW FENCING AND FITTINGS NECESSARY TO COVER FENCE AT COST AS DIRECTED BY THE OWNER. PAYMENT FOR THE WORK SHALL BE INCLUDED IN THE PRICE BID FOR PEDESTRIAN OVERPASS STRUCTURE, STEEL, ITEM No. 8.
NO DEDUCTION IS INCLUDED IN THE CONTRACT FOR CONTRACTOR SHALL BACKFILL THE SPACE EXCAVATED HEREIN, AS DIRECTED BY THE ENGINEER.

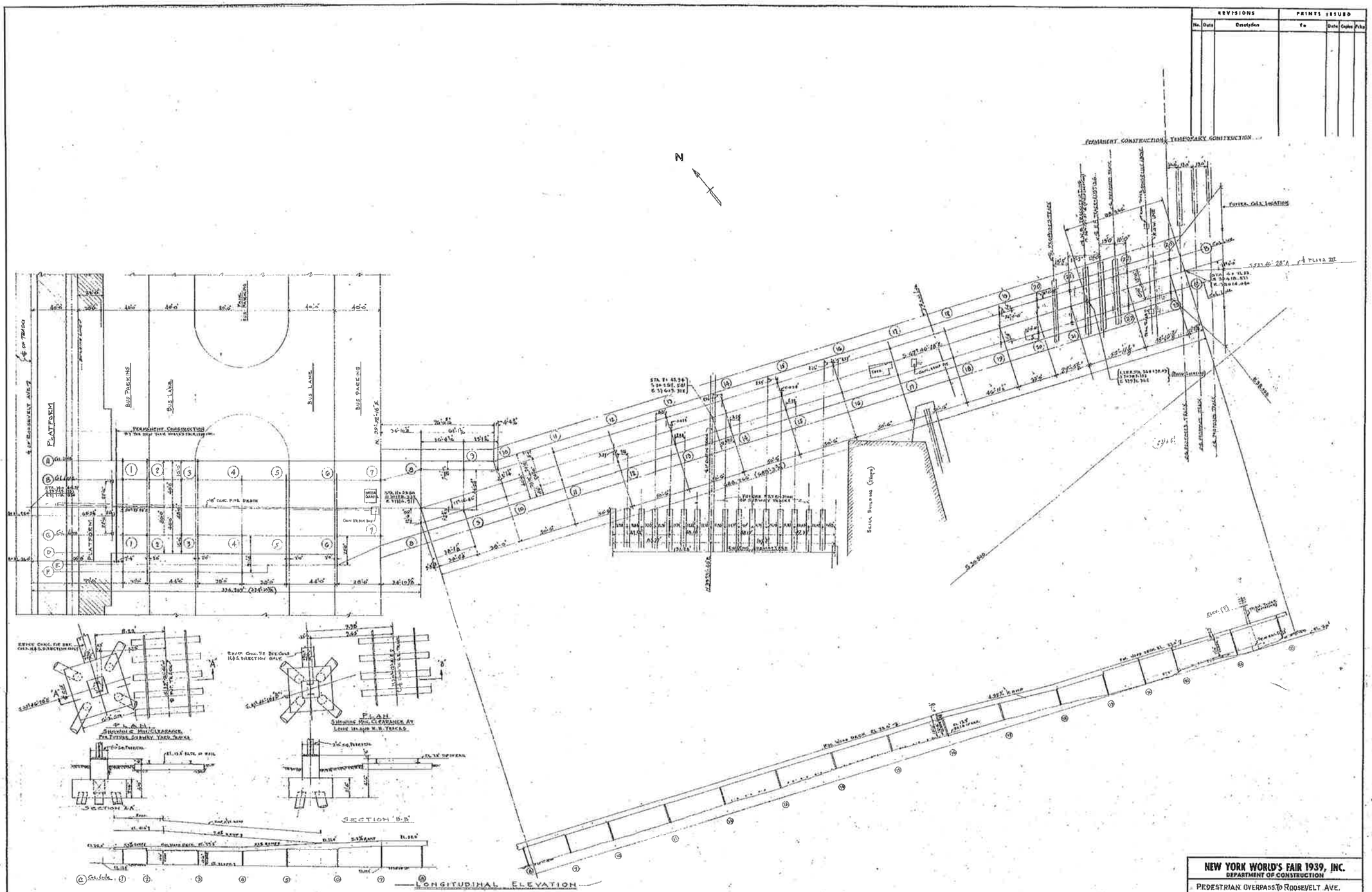
NOTE
Elevations based on Borough of Queens Datum. Coordinates refer to the coordination system of the Topographical Bureau of the Borough of Queens.

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

PEDESTRIAN OVERPASS TO ROOSEVELT AVE.
GENERAL FOUNDATION PLAN
Designed by DEPARTMENT OF CONSTRUCTION

Drawn by A. J. Dillon	Approved for Construction	
Checked by E.A.C.	Chief Engineer	
Cross checked by	Location	Drawing No.
Submitted by H. A. Sub.	B5	ST.101
Scale 1" = 30'		Sheet
Date July 8, 1939		

REVISIONS			PRINTS ISSUED			
No.	Date	Description	To	Date	Copies	Pickup



6-22-37
6-25-37

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

PEDESTRIAN OVERPASS TO ROOSEVELT AVE.
COLUMN CENTER PLAN

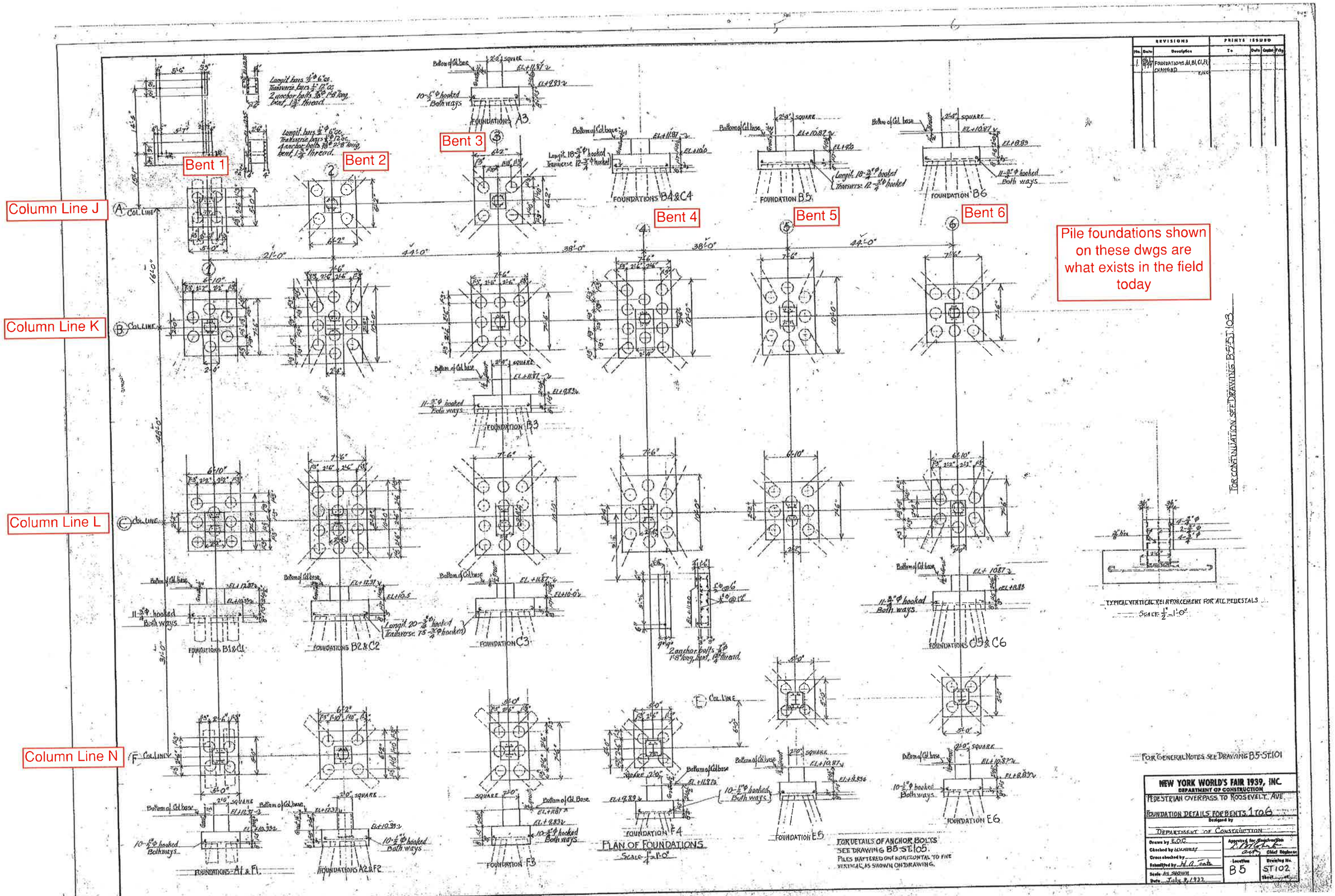
Designed by DEPARTMENT OF CONSTRUCTION

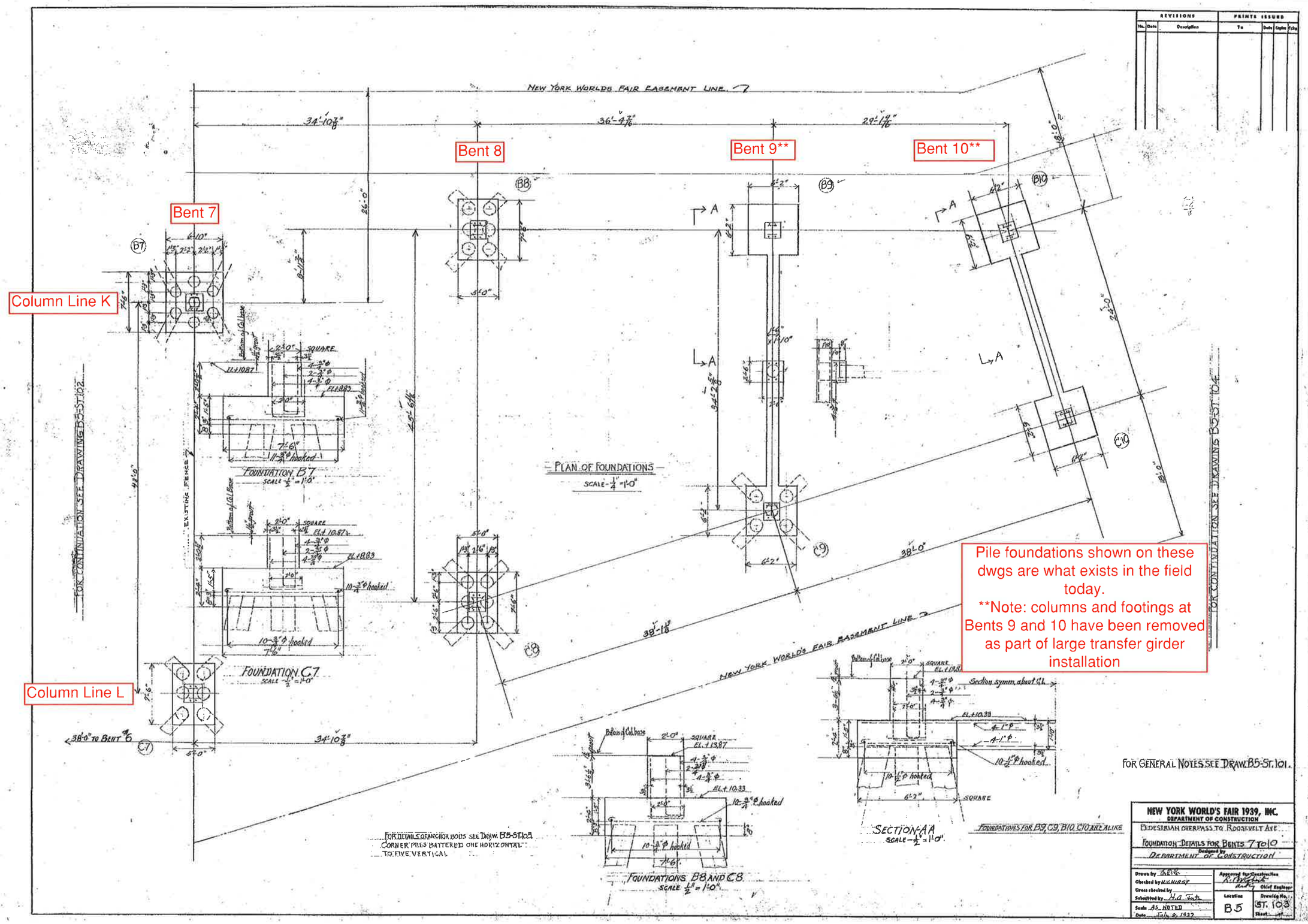
Drawn by H.W.S.
Checked by T.O. Moeschler
Cross checked by
Submitted by
Scale 1" = 30'-0"
Date

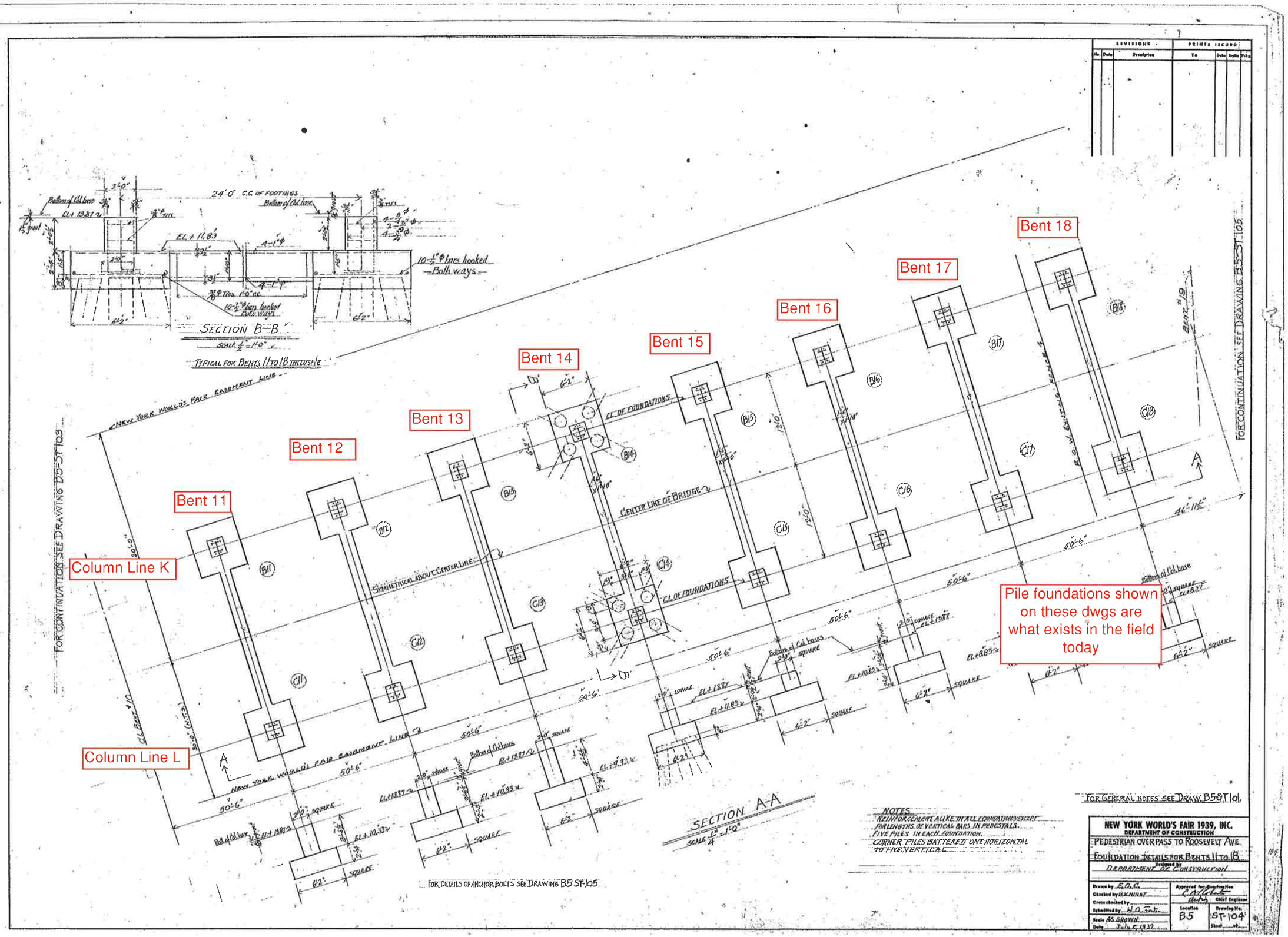
Approved for Construction
Chief Engineer

Location B5

Drawing No. ST1001
Sheet ___ of ___


Bent 1
Bent 2
Bent 3
Bent 4
Bent 5
Bent 6
Column Line J
Column Line K
Column Line L
Column Line N
Pile foundations shown on these dwgs are what exists in the field today
FOUNDATIONS A3
FOUNDATIONS B4 & C4
FOUNDATION B5
FOUNDATION B6
FOUNDATION B3
FOUNDATIONS B1 & C1
FOUNDATIONS B2 & C2
FOUNDATION C3
FOUNDATIONS C5 & C6
FOUNDATIONS A1 & F1
FOUNDATIONS A2 & F2
FOUNDATION F3
FOUNDATION F4
FOUNDATION E5
FOUNDATION E6
PLAN OF FOUNDATIONS
SCALE: 1/4" = 1'-0"
TYPICAL VERTICAL REINFORCEMENT FOR ALL PEDESTALS
SCALE: 1/2" = 1'-0"
FOR CONTINUATION SEE DRAWING B5-ST.103
FOR GENERAL NOTES SEE DRAWING B5-ST.101
FOR DETAILS OF ANCHOR BOLTS SEE DRAWING B5-ST.105.
PILES BATTERED ONE HORIZONTAL TO FIVE VERTICAL, AS SHOWN ON DRAWING.
REVISIONS
PRINTS ISSUED
NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION
PEDESTRIAN OVERPASS TO ROOSEVELT AVE.
FOUNDATION DETAILS FOR BENTS 1 TO 6
Designed by
DEPARTMENT OF CONSTRUCTION
Drawn by E.O.C.
Checked by
Cross checked by
Submitted by H. A. Tech
Scale AS SHOWN
Date July 8, 1937
Approved for Construction
Chief Engineer
Location B5
Drawing No. ST102
Sheet


Bent 7
Bent 8
Bent 9**
Bent 10**
Column Line K
Column Line L
Pile foundations shown on these dwgs are what exists in the field today.
**Note: columns and footings at Bents 9 and 10 have been removed as part of large transfer girder installation
NEW YORK WORLDS FAIR EASEMENT LINE
NEW YORK WORLD'S FAIR EASEMENT LINE
PLAN OF FOUNDATIONS
SCALE - 1/4" = 1'-0"
FOUNDATION B7
SCALE 1/2" = 1'-0"
FOUNDATION C7
SCALE 1/2" = 1'-0"
FOUNDATIONS B8 AND C8
SCALE 1/2" = 1'-0"
SECTION A-A
SCALE - 1/2" = 1'-0"
FOUNDATIONS FOR B9, C9, B10, C10 ARE ALIKE
FOR CONTINUATION SEE DRAWING B5-ST.102
FOR CONTINUATION SEE DRAWING B5-ST.104
EXISTING FENCE
38'-0" TO BENT 6
FOR DETAILS OF ANCHOR BOLTS SEE DRAW. B5-ST.105
CORNER PILES BATTERED ONE HORIZONTAL TO FIVE VERTICAL
FOR GENERAL NOTES SEE DRAW. B5-ST.101.
NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION
PEDESTRIAN OVERPASS TO ROOSEVELT AVE.
FOUNDATION DETAILS FOR BENTS 7 TO 10
DEPARTMENT OF CONSTRUCTION
Drawn by
Checked by M.K.HURST
Cross checked by
Submitted by H.A. Fuhr
Scale AS NOTED
Date July 8, 1937
Approved for Construction
Chief Engineer
Location B5
Drawing No. ST. 103
Sheet


Bent 11
Bent 12
Bent 13
Bent 14
Bent 15
Bent 16
Bent 17
Bent 18
Column Line K
Column Line L
Pile foundations shown on these dwgs are what exists in the field today
24'-0" C.C. OF FOOTINGS
SECTION B-B
TYPICAL FOR BENTS 11 TO 18 INCLUSIVE
SECTION A-A
CENTER LINE OF BRIDGE
SYMMETRICAL ABOUT CENTER LINE
CL OF FOUNDATIONS
NEW YORK WORLD'S FAIR EASEMENT LINE
FOR CONTINUATION SEE DRAWING B5-ST-103
FOR CONTINUATION SEE DRAWING B5-ST-105
FOR DETAILS OF ANCHOR BOLTS SEE DRAWING B5 ST-105
FOR GENERAL NOTES SEE DRAW. B5-ST-101
NOTES
REINFORCEMENT ALIKE IN ALL FOUNDATIONS EXCEPT FOR LENGTHS OF VERTICAL BARS IN PEDESTALS.
FIVE PILES IN EACH FOUNDATION.
CORNER PILES BATTERED ONE HORIZONTAL TO FIVE VERTICAL.
REVISIONS
PRINTS ISSUED
NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION
PEDESTRIAN OVERPASS TO ROOSEVELT AVE.
FOUNDATION DETAILS FOR BENTS 11 TO 18
DEPARTMENT OF CONSTRUCTION
Drawn by E.O.C.
Checked by W.K.HURST
Cross checked by
Submitted by H.O.S.Jr.
Scale AS SHOWN
Date July 6, 1937
Approved for Construction
Chief Engineer
Location B5
Drawing No. ST-104

Pile foundations shown on these dwgs no longer exist in the field today. See 1961 drawings for pile foundation dwgs.

REVISIONS.			PRINTS ISSUED			
No.	Date	Description	To	Date	Copies	Prints
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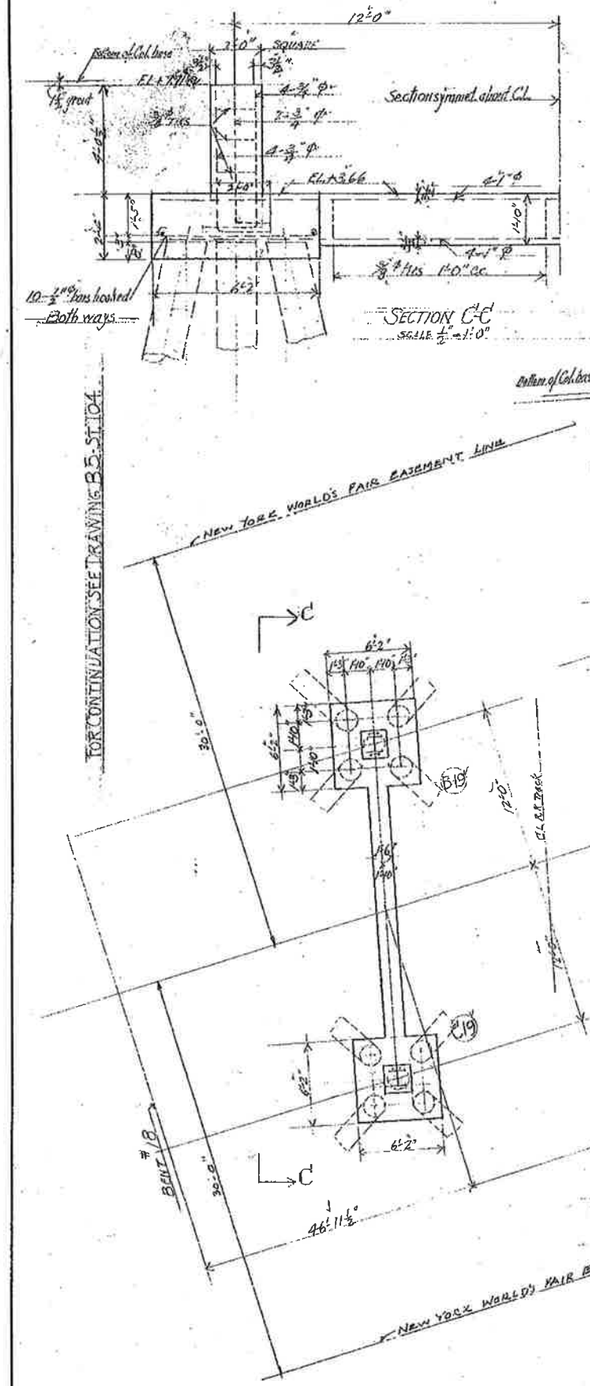
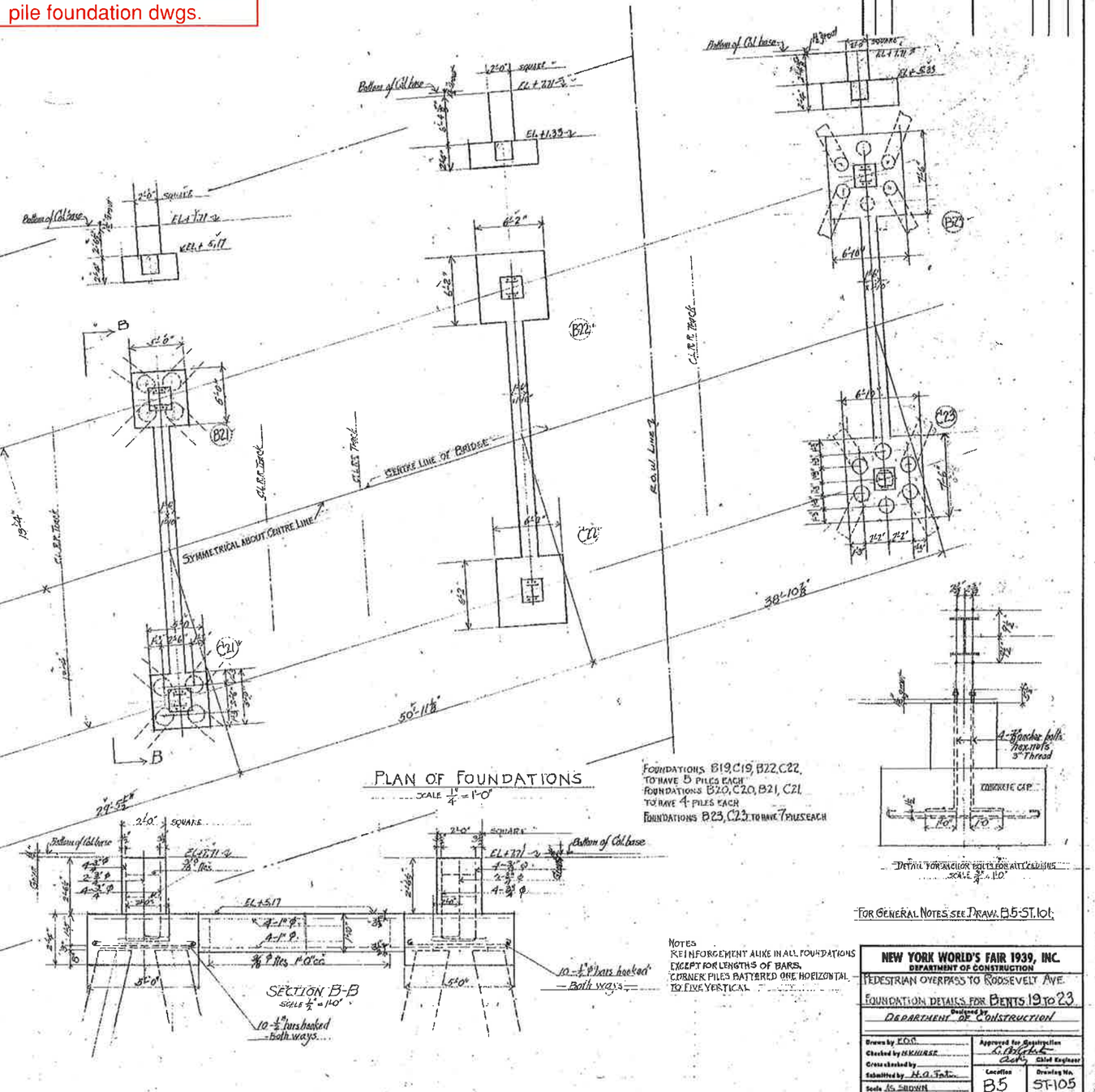




FOR GENERAL NOTES SEE DRAW. B5-ST.101.

NOTES
REINFORCEMENT ALIKE IN ALL FOUNDATIONS EXCEPT FOR LENGTHS OF BARS.
CORNER PILES BATTERED ONE HORIZONTAL TO FIVE VERTICAL

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

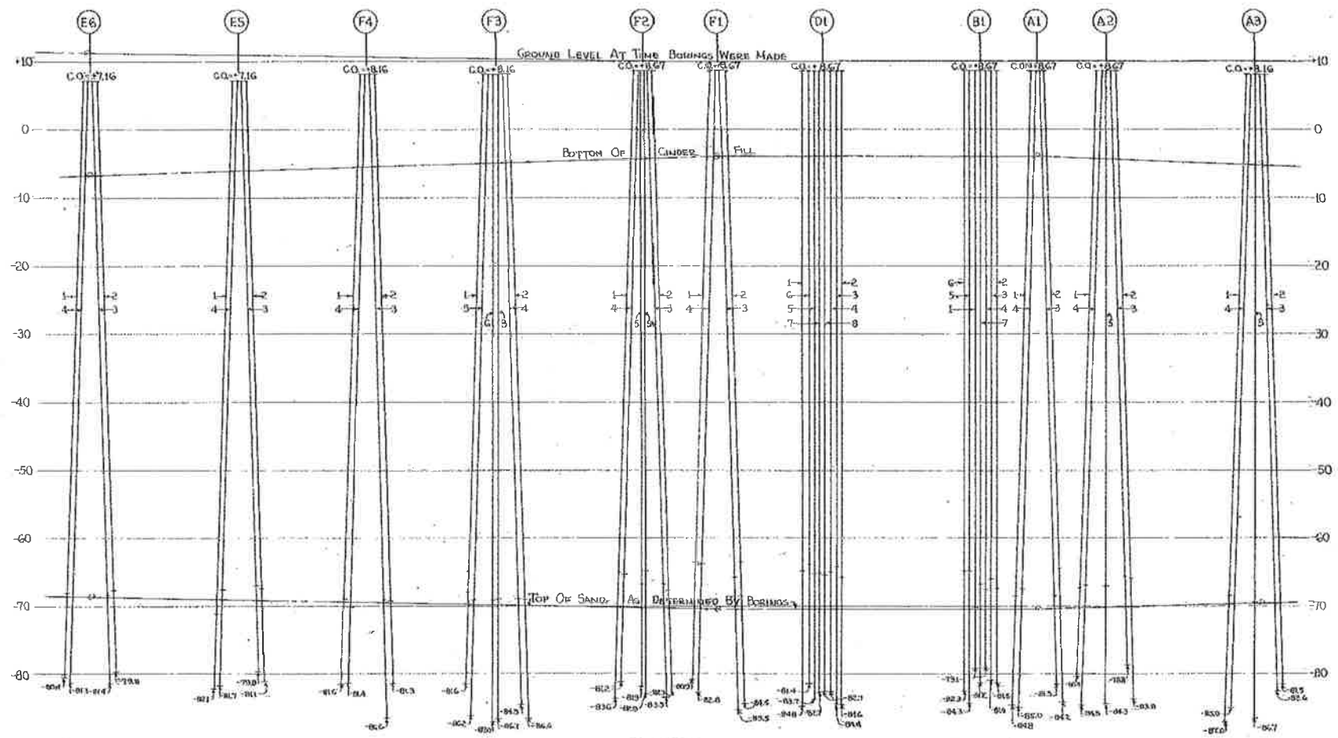
PEDESTRIAN OVERPASS TO ROOSEVELT AVE.

FOUNDATION DETAILS FOR BENTS 19 TO 23

Designed by DEPARTMENT OF CONSTRUCTION

Drawn by E.O.G.	Approved for Construction	
Checked by H.K.HURSE		Chief Engineer
Cross checked by	Location	Drawing No.
Submitted by H.A. Fate	B5	ST-105
Scale As Shown		Sheet ___ of ___
Date July 8, 1937		

REVISIONS			PRINTS ISSUED			
No.	Date	Description	To	Date	Copies	Purp.



Pile No.

Pile

Tip

Notes

Represents Top Of Firm Material As Determined When Piles Were Driven.

For Location Plan See Drawing No. B5. ST-111.

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

Pedestrian Overpass To Roosevelt Avenue
Pile Record Section

Designed by
Field Inspection Section

Drawn by D.R.H.
Checked by
Cross checked by
Submitted by E.J. Carlisle
Scale Vert. Scale 1/8"=1'0
Date November 24, 1937

Approved for Construction
Chief Engineer

Location B-5

Drawing No. ST-111
Sheet 1 of 5

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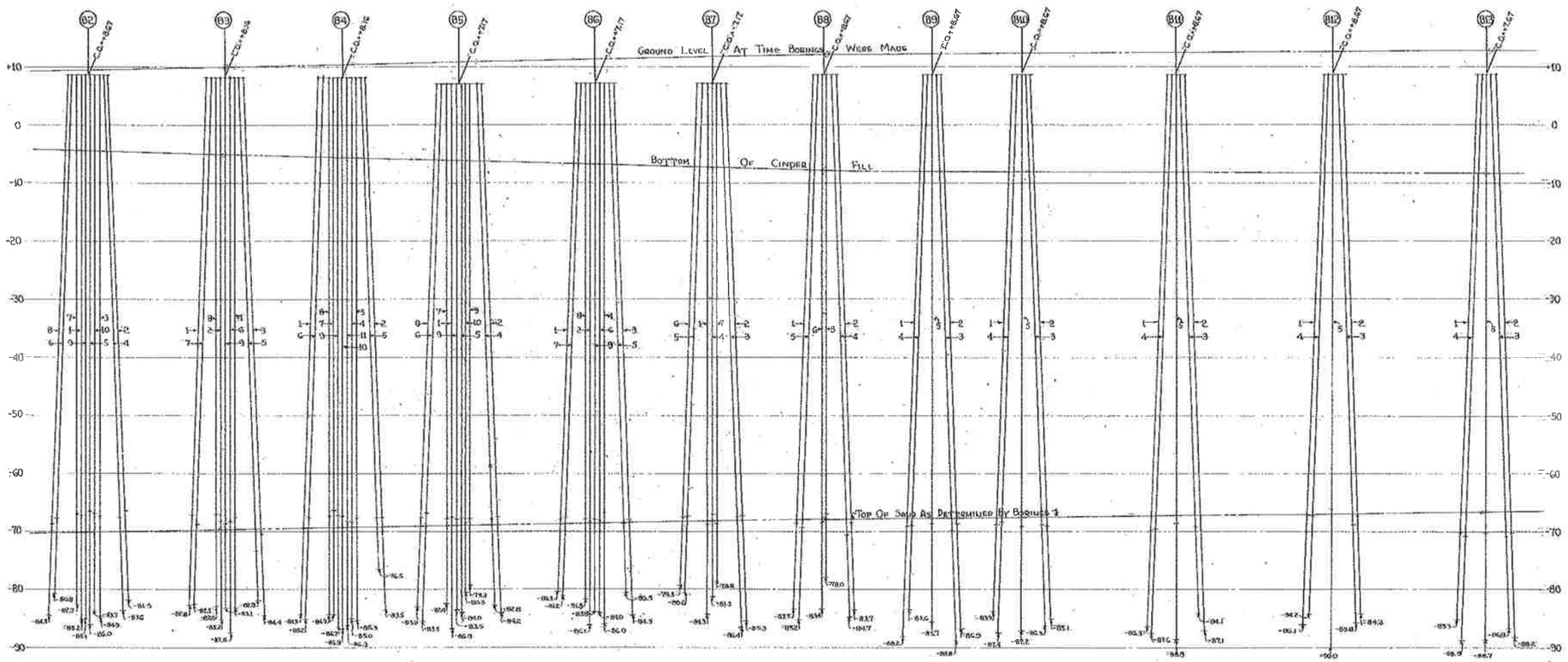

PILE NO.

PILE

TIP

NOTES

REPRESENTS TOP OF RIGH MATERIAL AS DETERMINED WHEN PILES WERE DRIVEN.

FOR LOCATION PLAN SEE DRAWING NO. B5. ST 111.

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

PEDESTRIAN OVERPASS TO ROOSEVELT AVENUE
PILE RECORD SECTION

Designed by

FIELD INSPECTION SECTION

Drawn by D.B.M.
Checked by
Cross checked by
Submitted by E.J. CARRILLO
Scale VERT. SCALE: 1/16"=1'-0"
Date NOVEMBER 24, 1937

Approved for Construction

Chief Engineer

Location B-5

Drawing No. ST-111

Sheet 2 of 5

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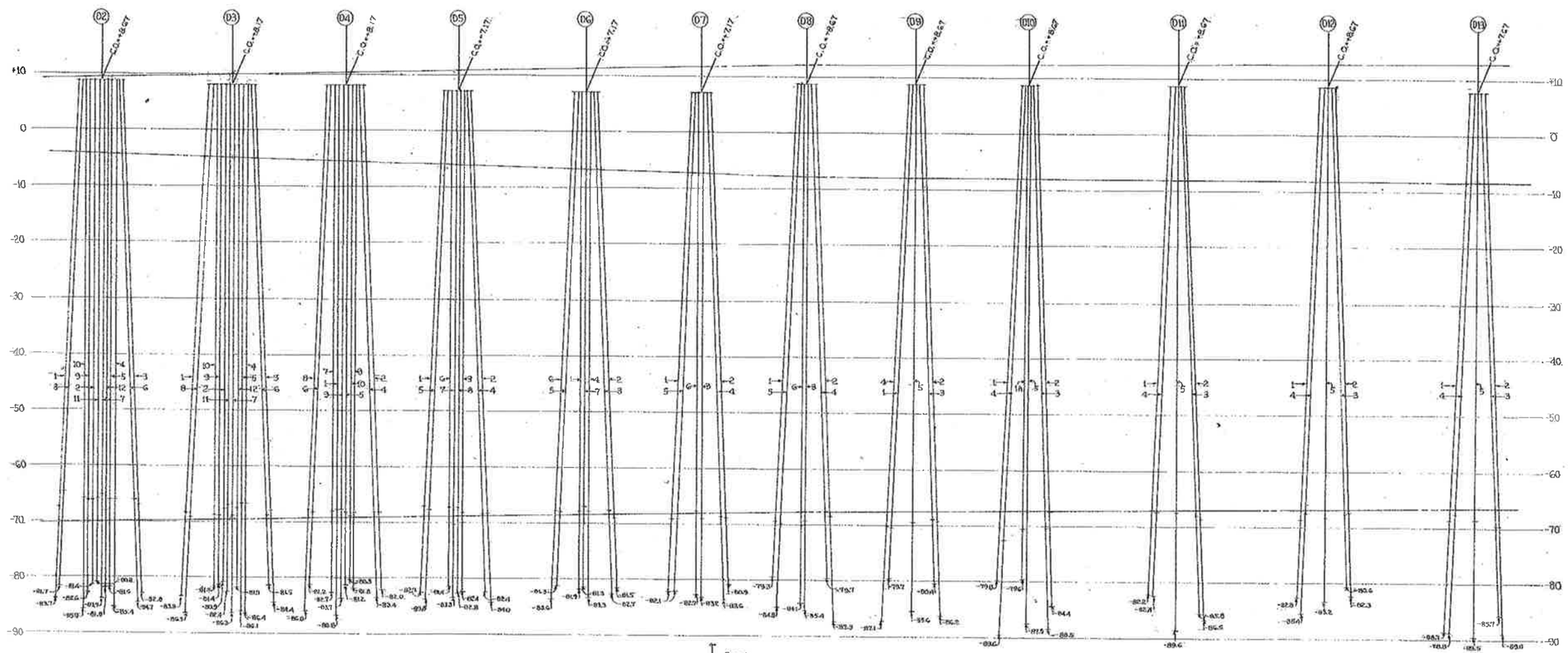

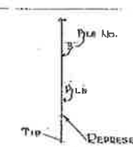

NOTES

Represents Top Of Firm Material As Determined When Piles Were Driven.

For Location Plan See Drawing No. B5.ST 111.

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

Pedestrian Overpass To Roosevelt Avenue
Pile Record Section

Designed by
Field Inspection Section

Drawn by O.R.M.
Checked by
Cross checked by
Submitted by E.J. Carrillo
Scale Vert. Scale 1/8"=1'-0"
Date November 30, 1939

Approved for Construction
Chief Engineer

Location B-5
Drawing No. ST-111
Sheet 3 of 5

REVISIONS			PRINTS ISSUED			
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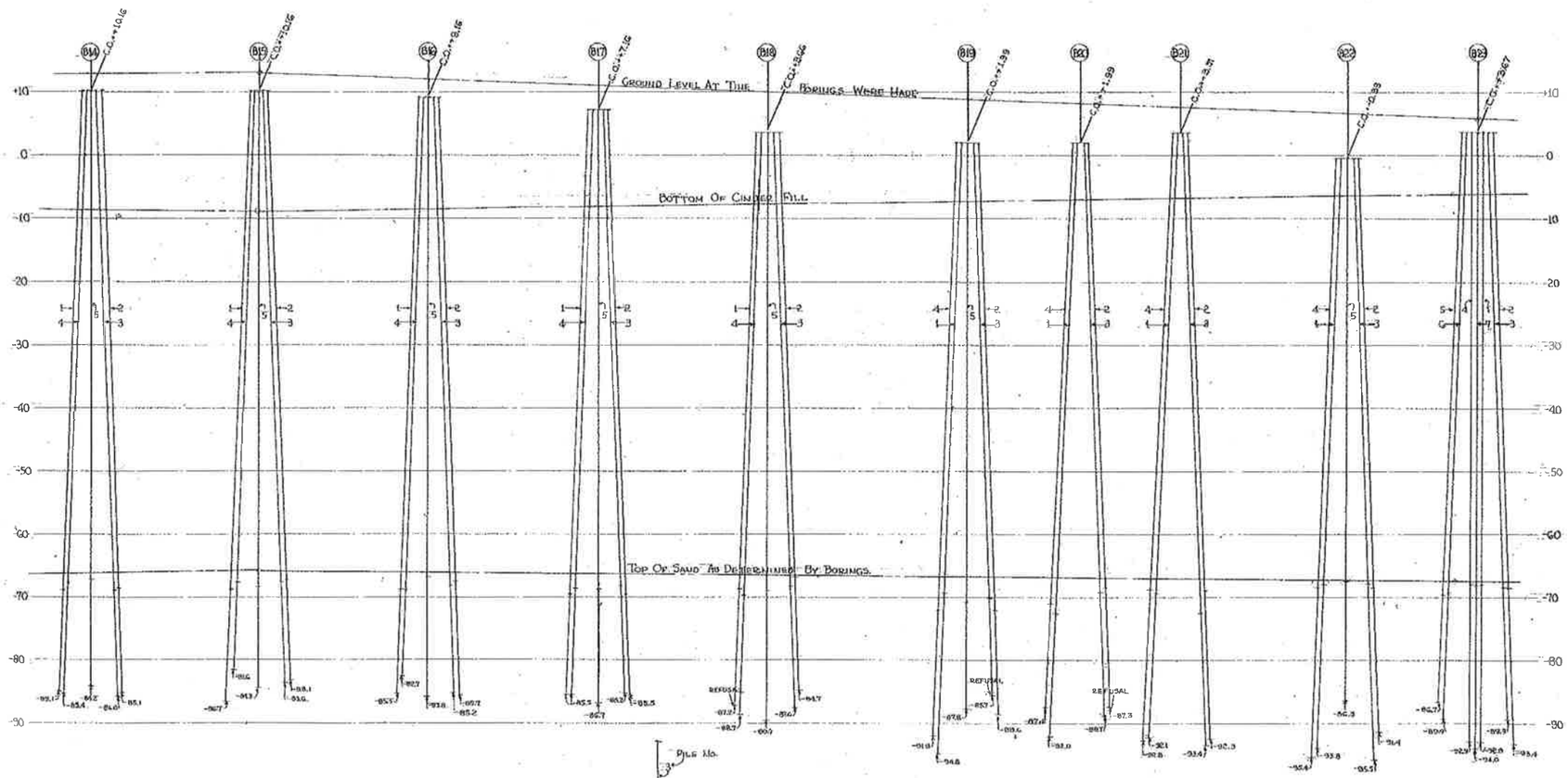


Notes

Represents Top Of Firm Material As Determined When Piles Were Driven.

For Location Plan See Drawing No. B5. ST 111.

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

Pedestrian Overpass To Roosevelt Avenue
Pile Record Section

Designed by
Field Inspection Section

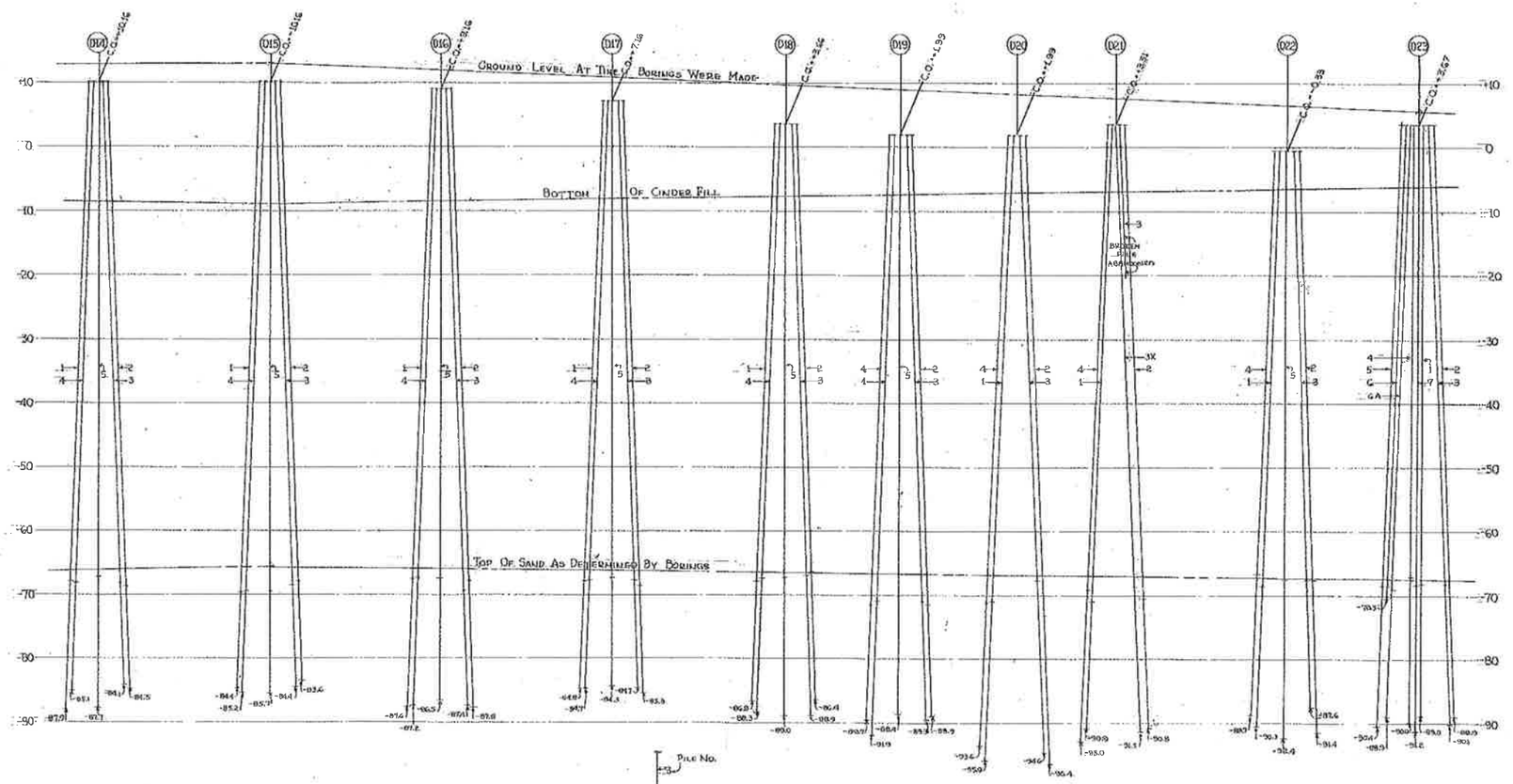
Drawn by D.R.M.
Checked by
Cross checked by
Submitted by E.J. Carollo
Scale Vert. 1/8"=1'-0" Hor. 1/4"=1'-0"
Date November 24, 1937

Approved for Construction
Chief Engineer

Location B-5

Drawing No. ST-111
Sheet 4 of 5

REVISIONS			PRINTS ISSUED			
No.	Date	Description	To	Date	Copies	Pulg



Notes

Tip represents Top Of Firm Material As Determined When Piles Were Driven.

For Location Plan See Drawing No. B5. ST 111.

NEW YORK WORLD'S FAIR 1939, INC.
DEPARTMENT OF CONSTRUCTION

Pedestrian Overpass To Roosevelt Avenue
Pile Record Section

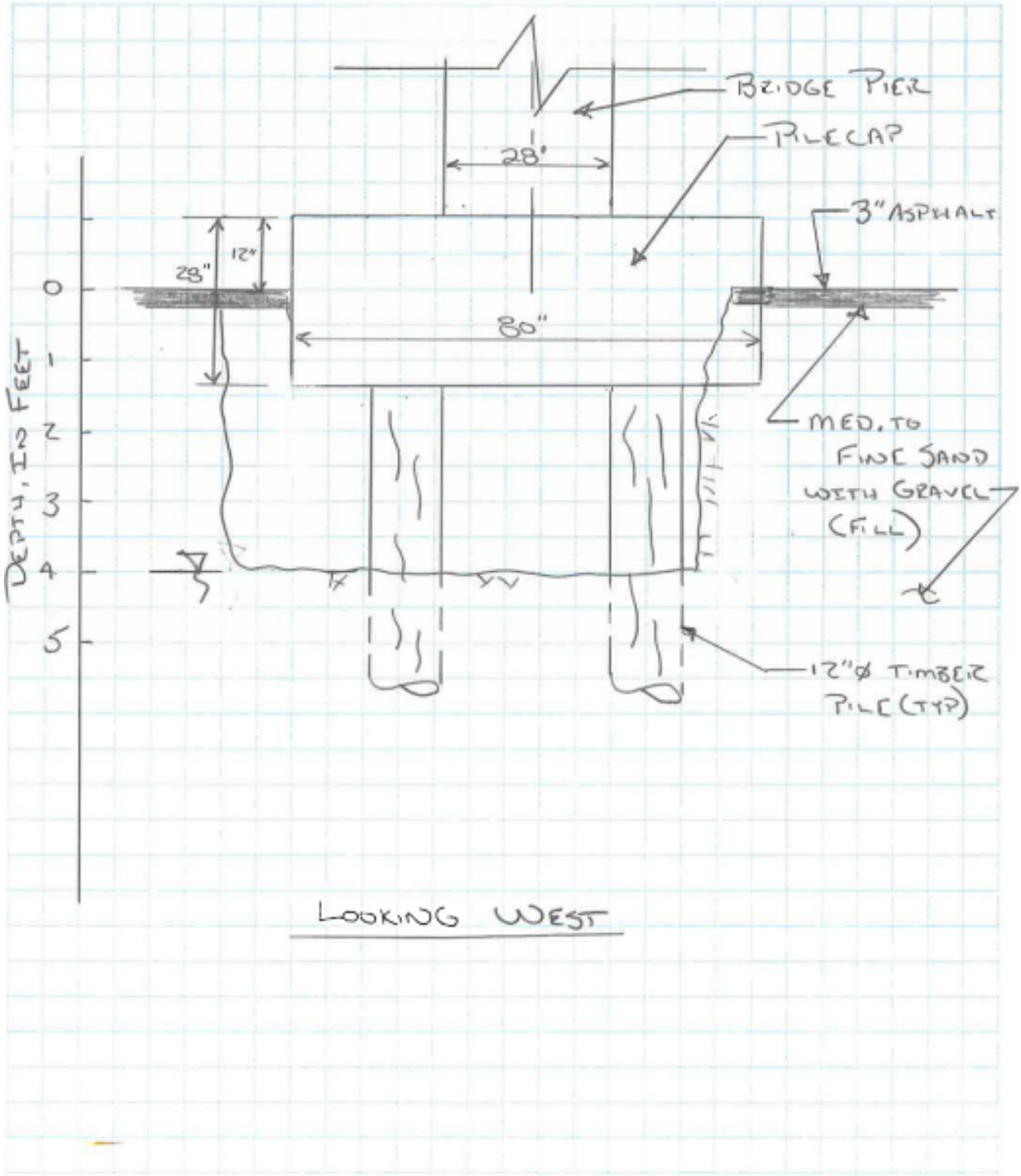
Designed by: Field Inspection Section

Drawn by: B. & M.
Checked by:
Cross checked by:
Submitted by: E. J. Carrillo
Scale: Vert. Scale, 1" = 10'
Date: November 24, 1937

Approved for Construction
Chief Engineer

Location: B-5
Drawing No.: ST-111
Sheet 5 of 5.

APPENDIX C - Logs of Test Pits Excavated for this Study

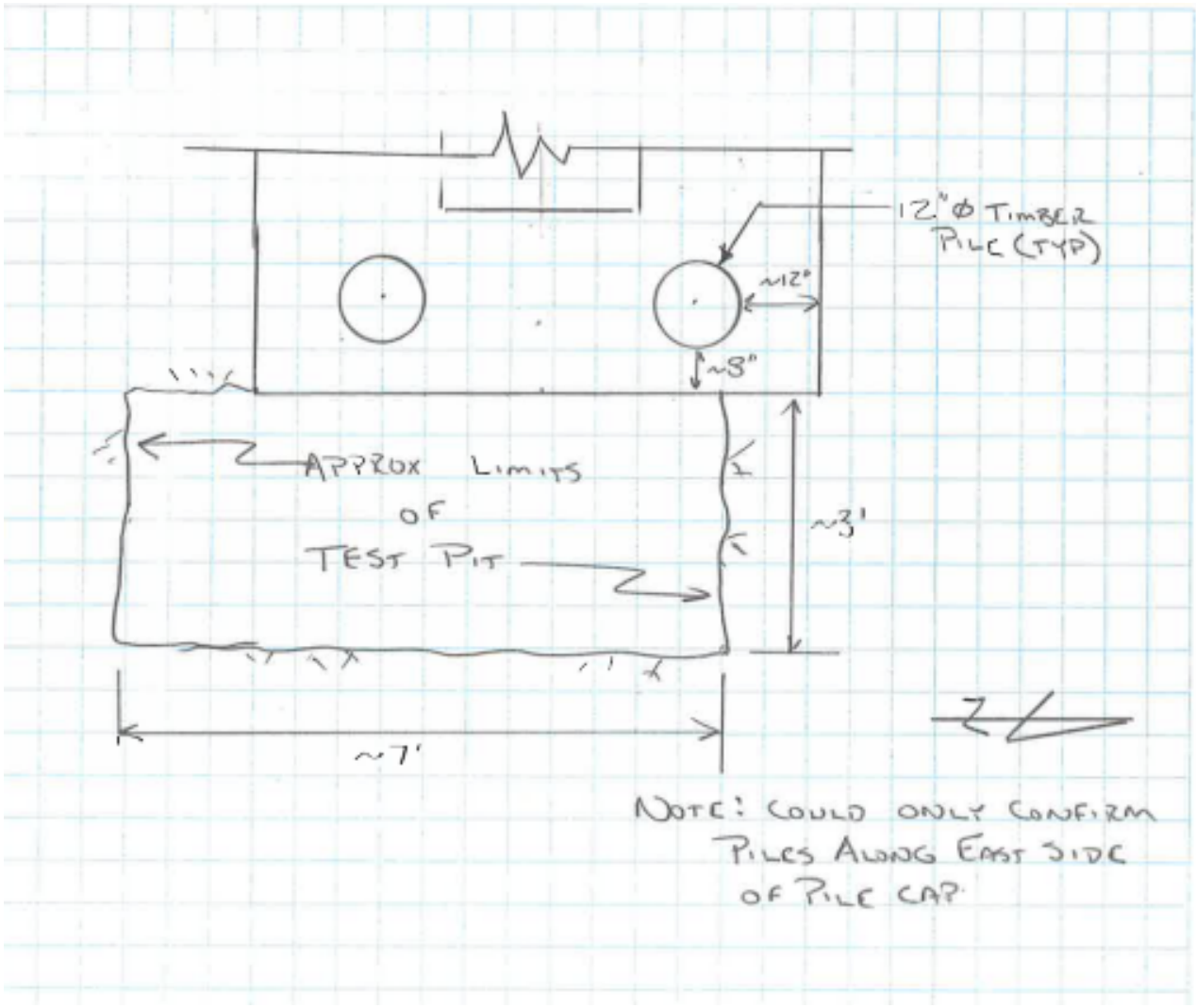


WEIDLINGER ASSOCIATES INC
CONSULTING ENGINEERS

27 Wormwood Street, Suite 200
Boston, MA 02210

http://www.wai.com

FIGURE C-1
Test Pit No. 1 Log
Passerelle Pedestrian Bridge
Queens, New York

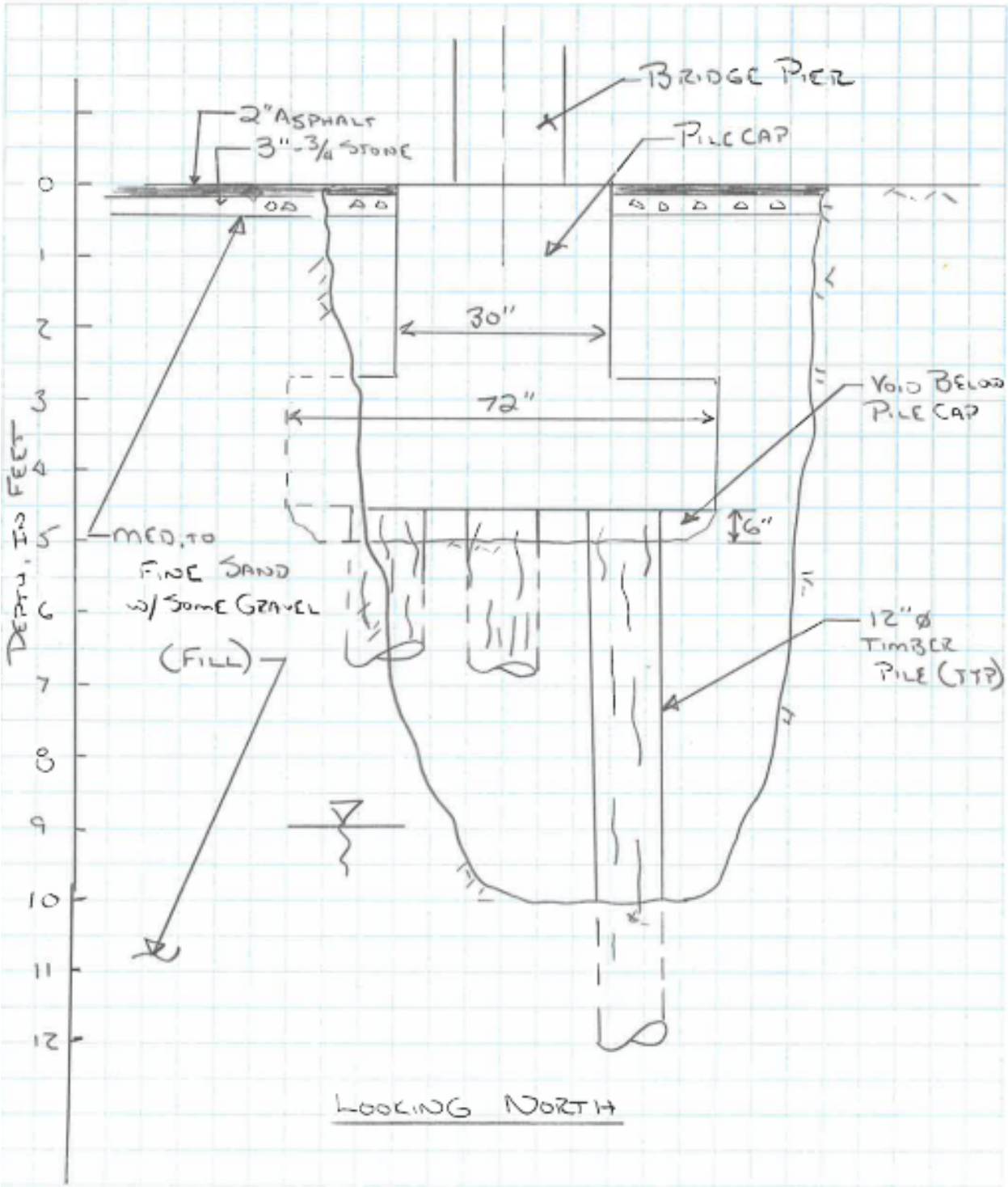


WEIDLINGER ASSOCIATES INC
CONSULTING ENGINEERS

27 Wormwood Street, Suite 200
Boston, MA 02210

http://www.wai.com

FIGURE C-2
Test Pit No. 1 Plan
Passerelle Pedestrian Bridge
Queens, New York



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Boston, MA 02210

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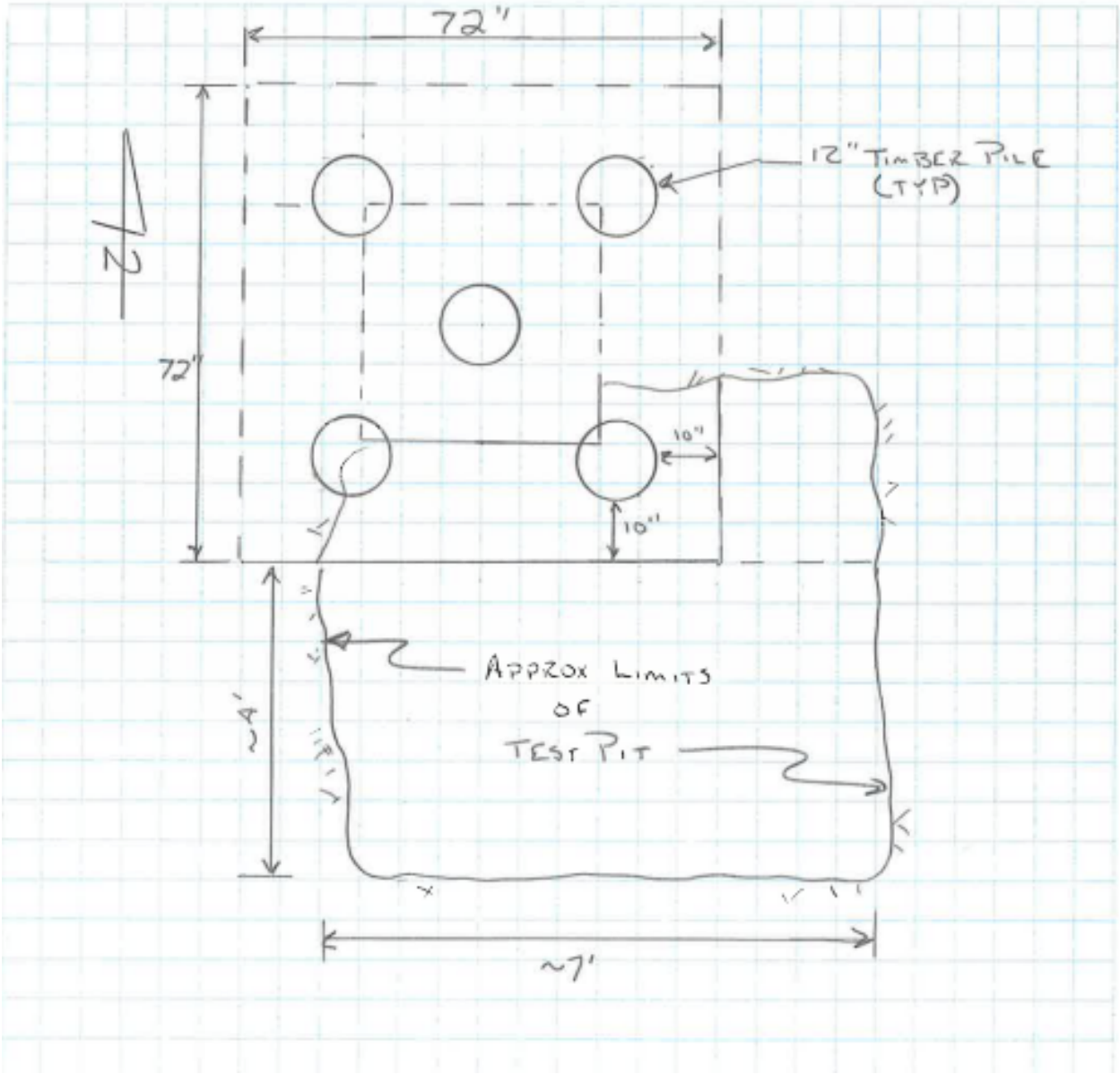
FIGURE C-3
Test Pit No. 2 Log
Passerelle Pedestrian Bridge
Queens, New York



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Boston, MA 02210
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FIGURE C-4
Test Pit No. 2 Plan
Passerelle Pedestrian Bridge
Queens, New York

APPENDIX D - Photos of Test Pits and Existing Piles

Test Pit No 1

Photo 1: Test Pit Excavation

Photo 2: Exposing Two Corner Piles Adjacent to Pit

Photo 3: Core Sample of Pile

Photo 4: Corner Pile With Groundwater

Test Pit No. 2

Photo 1: Test Pit Excavation

Photo 2: Gap Below Pile Cap Exposing Corner and Center Piles

Photo 3: Gap Below Pile Cap Exposing Two Corner Piles

Photo 4: Final Test Pit Excavation Prior to Backfilling

Appendix G: Phase 1 Corridor Assessment Report

- FINAL -

Phase I Corridor Assessment Report

For

Passerelle Pedestrian Bridge

Area between Roosevelt Avenue and Perimeter Road

Queens, New York

DDC PROJECT NO. HBPED700Q

WORK ORDER NO. 9182-Weston-R-8654

CONTRACT REGISTRATION NO. 20131407562

Prepared for:

Bureau of Environmental and Geotechnical Services
30-30 Thomson Avenue, Fifth Floor
Long Island City, New York 11101

Prepared by:

Weston Solutions of New York, Inc.
205 Campus Drive
Edison, New Jersey 08837

Weston Project No. 14513.009.182.0001

September 19, 2013

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EXECUTIVE SUMMARY

Weston Solutions of New York, Inc. (Weston®) has prepared this Phase I Corridor Assessment Report (CAR) for the New York City Department of Design and Construction (DDC) to reasonably determine the potential for environmental concern and possible contamination posed by properties within or adjacent to the Corridor. Construction activities will be performed along the Corridor. The Corridor consists of:

- The area between Roosevelt Avenue and Perimeter Road near the Willets Point section of Queens, New York (also known as the Passerelle Pedestrian Bridge).

The length of the Corridor is approximately 0.21 miles (1,120 feet).

The Corridor Assessment process involved conducting a site reconnaissance to document current property usage and conditions, a review of historical Sanborn Fire Insurance Maps to document past property use, and a review of a regulatory agency database report to identify Corridor properties and adjoining sites of potential environmental concern.

Based on the Risk Criteria protocol established by the DDC, Weston identified five (5) sites categorized as initially having a "High" risk in the database report. There were no sites categorized as initially having a "Moderate" risk with respect to potential impact on the project.

Based on modifying information, Weston recommends that one (1) site remain as a "High" risk site, three (3) of the "High" risk sites be re-classified as "Low" risk sites, and one (1) of the "High" risk sites be re-classified as a "Moderate" risk site. The sites were re-classified because they were found to be outside of the Corridor limits or were identified on one (1) or more of the regulatory agency databases, but modifying information or professional judgment indicated a lower risk to the environment. Modifying information included sites with spills that had been closed by the New York State Department of Environmental Conservation (NYSDEC), sites identified on one (1) or more databases with no evidence or records of spills or other concerns, or older sites that were redeveloped or located such that they no longer posed significant risks. Other modifying information included if the site was upgradient or downgradient or sites that were determined to be located far enough away from the site such that they did not pose significant risks.

During the review of the Sanborn maps, two (2) sites were identified as "High" risk sites that were not identified within the Environmental Data Resources, Inc. (EDR) of Milford, Connecticut:

- A subway car repair facility/subway tracks; and
- A former auto repair shop, which is now a portion of the Citi Field parking lot.

During the site reconnaissance, two (2) train stations were identified as "High" risk sites:

- At the northern end of the Corridor is the Metropolitian Transportation Authority (MTA) Mets-Willets Point train station; and
- At the southern end of the Corridor is the Long Island Railroad (LIRR) Mets-Willets Point train station.

Therefore, the final evaluation has identified five (5) "High" risk sites and one (1) "Moderate" risk site with respect to potential impact on the Project Corridor.

The final "High" risk sites are listed below:

HIGH RISK SITES

1. Konica Photo Imaging at Citi Field, 123-01 Roosevelt Avenue, (Map ID A1);
2. Subway Car Repair Facility/Subway Tracks, Block 2018 Lot 1000, (No Map ID);
3. Former Skylights Auto Sales Facility/Auto Repair Facility, 12050 Unknown Road, (No Map ID);
4. LIRR Mets-Willets Point train station, Perimeter Road, (No Map ID); and,
5. MTA Mets-Willets Point train station (7 Train Line), Roosevelt Avenue, (No Map ID).

The final "Moderate" risk site is listed below:

MODERATE RISK SITE

1. Flushing Meadows Park, 123-30 Roosevelt Avenue, (Map ID A4).

Following the standard DDC investigation report, a Phase II Subsurface Corridor Investigation (SCI) is proposed. The Phase II SCI will consist of two (2) soil borings per "High" risk site and one (1) soil boring per "Moderate" risk site to determine if the Corridor has been impacted. However, since many of the "High" and "Moderate" risk sites are in close proximity to each other, the number of soil borings may be reduced by placing borings in representative locations along the Corridor. A total of eleven (11) soil borings are recommended in Table 2 based on DDC protocols. However, it should be noted that the MTA 7 Train line and LIRR Mets-Willets Point train stations as well as the subway car repair facility/subway tracks are present at grade level, for which access to the proposed soil boring locations may not be gained due to the close proximity of numerous train tracks.

During the Phase II SCI site assessment activities, photoionization detector (PID) screening will be instituted and soil logging will be conducted by a qualified geologist. The on-site field geologist will make a sampling determination based on the results of field screening including PID readings, visual and odor observations, and at the discretion of the DDC.

1.0 INTRODUCTION

1.1 Background Information

Weston Solutions of New York, Inc. (Weston®) has prepared this Phase I Corridor Assessment Report (CAR) for the New York City Department of Design and Construction (DDC) to reasonably determine the potential for environmental concern and possible contamination posed by properties within or adjacent to the Corridor. Construction activities will be performed along the Corridor. The Corridor consists of:

- The area between Roosevelt Avenue and Perimeter Road near the Willets Point section of Queens, New York (also known as the Passerelle Pedestrian Bridge).

The Corridor location is identified on the Topographic Corridor Location Map included on Figure 1 and the Corridor Area Map included on Figure 2.

1.2 Scope of Work

At the request of the DDC, the scope of the assessment was limited to:

1. Conducting a site visit and reconnaissance of the Corridor and adjacent properties;

2. Providing photographic documentation of properties within, and adjacent to, the Corridor that are categorized as initial “High” or “Moderate” risk sites, or otherwise considered a potential environmental concern;

3. Conducting a review of Sanborn Fire Insurance Maps to document historical property use;

4. Conducting a review of government regulatory agency databases for properties along the Corridors and adjoining sites that are listed; and

5. Submitting to the DDC a written Phase I CAR summarizing the sites or issues identified during the assessment that are considered to be of potential environmental concern to the project, provide recommendations for additional investigation and possible means of corrections, and write a conclusion based on observations.

2.0 CORRIDOR INFORMATION

2.1 Corridor Location, Description and Use

The project Corridor is located in Queens, New York. The length of the Corridor is approximately 0.21 miles (1,120 feet) in length along the Passerelle Pedestrian Bridge between Roosevelt Avenue and Perimeter Road.

Property usage in the Corridor consists of a pedestrian bridge connecting the Metropolitian Transportation Authority (MTA) and Long Island Railroad (LIRR) Mets-Willets Point train stations and is located above a subway car repair facility/train tracks which is a "High" risk site and Flushing Meadows Corona Park which is a "Moderate" risk site along with Citi Field.

Natural gas lines are expected to be present surrounding the project Corridor, as well as the potential for the presence of petroleum storage tanks. A review of the Environmental Data Resources, Inc. (EDR) of Milford, Connecticut database identified several properties with in-service aboveground and underground storage tanks. In addition to natural gas, other buried utilities such as telecommunication, electric, water, and sewer lines are expected to be present along the project Corridor.

2.2 Description of Surrounding Properties

The Corridor is primarily surrounded by park property, stadium property, and train station and subway car repair facility property. The Flushing Meadows Corona Park is considered a "Moderate" risk site. Konica Photo Imaging is located inside Citi Field at 12301 Roosevelt Avenue and is considered a "High" risk site. The subway car repair facility/subway tracks below the Corridor are considered "High" risk sites. The Corridor area is bound to the north by MTA Mets-Willets Points train station and Roosevelt Avenue, to the east by the subway car repair facility/subway tracks and Flushing Creek and Van Wyck Expressway, to the south by LIRR Mets-Willets Point train station and Perimeter Road, and to the west by the subway car repair facility/subway tracks and Shea Road.

2.3 Site and Regional Topographic Setting

Weston reviewed the *United States Geologic Survey (USGS) 7.5-minute Topographic Quadrangle Maps* for Flushing (1995) to determine topography throughout the Corridor. The Corridor is located at elevations ranging from approximately 20 feet to 50 feet above mean sea level (msl). Surface runoff is expected to follow an eastern slope flowing towards the Flushing Creek.

2.4 Site and Regional Geology and Hydrogeology

Based on the New York State Department of Environmental Conservation (NYSDEC), Water Power and Control Commission report titled *Ground Water in Bronx, New York, and Richmond Counties, with Summary Data on Kings and Queens Counties, New York City, New York*, the

Corridor's geology is expected to consist of Pleistocene to recent age glacial and glacialfluvial sediments (outwash) and fluvial deposits that are composed of sands and gravels, derived from melt-water of the retreating glaciers. The upper portions of the glacial and fluvial deposits have been disturbed by a long history of development activities thus resulting in a layer of fill material which is classified as Urban Land. Urban Land refers to soils that have been altered by urban development such as buildings and streets, where at least 85 percent of the surface is covered with asphalt, concrete, or other impervious building material. Typically, these soils have been mixed with other materials such as brick and concrete. Below the soil cover is bedrock that is at an approximate depth of between 300 to 350 feet below ground surface (ftbgs) and consists of a crystalline metamorphic rock unit composed of schist and gneiss. This bedrock is mapped as the Hartland Formation.

The nearest body of water is the Flushing Creek located to the east of the Corridor. Flushing Creek opens up into Flushing River then into the East River and Flushing Bay. Based on a review of topographic mapping, groundwater is anticipated to be present at depths of approximately 20 to 35 ftbgs. Groundwater flow direction is anticipated to be east towards Flushing Creek. Groundwater flow direction may vary due to seasonal fluctuations in precipitation, local usage demands, geology, underground structures, or dewatering operations. Flood zones are geographic areas that the Federal Emergency Management Agency (FEMA) has defined according to varying levels of flood risk. These zones are depicted on a community's Flood Insurance Rate Map (FIRM). Each zone reflects the severity or type of flooding in the area. For this site, the information is provided on Flood Plain Panels 3604970113F and 3604970114F from the FEMA/DFIRM flood data. According to the FEMA Flood Plain Panels, the Corridor is located within a 100-year flood zone and no wetlands were identified within the Corridor boundaries.

3.0 CORRIDOR EVALUATION

Based on a review of the EDR database report and historical Sanborn Maps, as well as the performance of a Corridor reconnaissance, the Corridor and surrounding properties were evaluated to determine land use and associated potential environmental risks for the proposed construction project. The sites identified as a result of this evaluation are placed in a Risk Category of "High", "Moderate", or "Low" in accordance with Risk Criteria that have been established by DDC to allow for consistent evaluation of the potential risk posed by sites. The Risk Criteria established by DDC are identified in Table 1.

3.1 Site Visit and Reconnaissance

On August 16, 2013, an initial site reconnaissance was conducted by Dyna Krumich-Ogonowski, Senior Project Scientist of Weston, utilizing the available project materials. On August 20, 2013, a follow-up site visit was conducted to gather additional information and photographs. The available materials included an environmental database report by EDR, which was used to identify and evaluate individual properties within and adjacent to the Corridor.

The site reconnaissance consisted of a walk-through survey of the Corridor to identify sites of potential environmental concern, based on their current use and/or conditions, within and adjacent to the Corridor. The entire Corridor was visually inspected during the walkthrough survey and the sites categorized as initial "High" or initial "Moderate" risk sites were photographed. The visual inspection included the Corridor, adjacent properties, and surrounding properties within approximately 1/8 of a mile of the centerline along the Corridor. The sites identified during the reconnaissance were correlated with street address, block and lot information, regulatory agency database listings, and Sanborn Map review. The Corridor consists primarily of a pedestrian bridge above a subway car repair facility which connects the MTA and LIRR Mets-Willets Point train stations to Flushing Meadows Park. The MTA and LIRR Mets-Willets Point train stations are considered "High" risk sites based on Table 1. The subway car repair facility/subway tracks was not listed in the database but was identified during the Sanborn map review. Repair facilities are similar to auto repair shops which are listed as "High" risk sites on Table 1; therefore, the subway car repair facility/subway tracks is included as a "High" risk site.

Summaries of the site reconnaissance are presented in Table 2. Site photographs are presented in Appendix A.

3.2 Historical Sanborn Map Review

EDR provided historical Sanborn Fire Insurance Maps of the Corridor and the surrounding areas. The purpose of the Sanborn Map review was to identify sites of potential environmental concern within and adjacent to the Corridor based on their historical use. Weston reviewed copies of Sanborn Fire Insurance Maps for parcels within and adjacent to the Corridor for the years 1902, 1914, 1931, 1950, 1986, 1995, and 2007. This information was utilized during the site visit and

reconnaissance to correlate historical properties of environmental concern with their current street address, block, and lot information. All final "High" and "Moderate" risk sites were identified on the Historical Sanborn Maps and confirmed during the site reconnaissance. Summaries of the historical map review are presented in Table 2. Copies of the Sanborn Fire Insurance Maps are provided as Appendix B.

3.3 Regulatory Agency Database Report Review

A regulatory agency database report was obtained from EDR to review available regulatory agency environmental databases to identify sites that are known to be contaminated or have potential environmental concerns within a 1/8-mile radius of the Corridor. Databases searched by EDR include, but are not limited to: National Priority List (NPL); Comprehensive Environmental Response, Compensation, and Liability Information System (CERCLIS); CERCLIS No Further Remedial Action Planned (NFRAP); Resource Conservation and Recovery Act (RCRA) Transportation, Storage, and Disposal facilities (TSD), RCRA Corrective Action Report (CORRACTS); RCRA Generators (GEN); Emergency Response Notification System (ERNS); New York State Inactive Hazardous Waste Sites (State Sites); New York State 1990 Spills (Spills-1990), New York State Solid Waste Landfills (SWL), New York State Regulated Underground Storage Tanks / Aboveground Storage Tanks (REG UST/AST), and New York State Leaking Underground Storage Tanks (LUST). All "High" and "Moderate" risk sites identified in the regulatory database report were located during the site reconnaissance. The Environmental Regulatory Database Report is provided as Appendix C. The results of the assessment are presented in Table 2, Summary Assessment Results (High and Moderate Risk Sites). Figure 3 depicts the 1/8-mile radius surrounding the Passerelle Pedestrian Bridge (Corridor) in the database.

4.0 FINDINGS AND RECOMMENDATIONS

Weston conducted a site visit and reconnaissance of the project Corridor to document current usage and conditions, reviewed Sanborn Fire Insurance Maps to document historical usage, and reviewed regulatory agency databases to identify sites with reported environmental conditions that could impact the scope during construction of the project.

Based on the Risk Criteria protocol established by the DDC, Weston identified five (5) sites categorized as initially having a "High" risk in the database report. There were no sites categorized as initially having a "Moderate" risk with respect to potential impact on the project.

Based on modifying information, Weston recommends that one (1) site remain as a "High" risk site, three (3) of the "High" risk sites be re-classified as "Low" risk sites, and one (1) of the "High" risk sites be re-classified as a "Moderate" risk site. The sites were re-classified because they were found to be outside of the Corridor limits or were identified on one (1) or more of the regulatory agency databases, but modifying information or professional judgment indicated a lower risk to the environment. Modifying information included sites with spills that had been closed by the NYSDEC, sites identified on one (1) or more databases with no evidence or records of spills or other concerns, or older sites that were redeveloped or located such that they no longer posed significant risks. Other modifying information included sites that were determined to be located far enough away from the site such that they did not pose significant risks.

During the review of the Sanborn maps, two (2) sites were identified as "High" risk sites that were not identified within the EDR:

- A subway car repair facility below the pedestrian bridge along the Corridor; and
- A former auto repair shop, which is now a portion of the Citi Field parking lot located north of the Corridor.

During the site reconnaissance, two (2) train stations were identified as "High" risk sites:

At the northern end of the Corridor is the MTA Mets-Willets Point train station; and
At the southern end of the Corridor is the LIRR Mets-Willets Point train station.

Therefore, the final evaluation has identified five (5) "High" risk sites and one (1) "Moderate" risk site with respect to potential impact on the Project Corridor.

The final "High" risk sites are listed below:

<u>HIGH RISK SITES</u>

1. Konica Photo Imaging at Citi Field, 123-01 Roosevelt Avenue, (Map ID A1);
2. Subway Car Repair Facility/Subway Tracks, Block 2018 Lot 1000, (No Map ID);

3. Former Skylights Auto Sales Facility/Auto Repair Facility, 12050 Unknown Road, (No Map ID);
4. LIRR Mets-Willets Point train station, Perimeter Road, (No Map ID); and,
5. MTA Mets-Willets Point train station, Roosevelt Avenue, (No Map ID).

The final "Moderate" risk site is listed below:

MODERATE RISK SITE

1. Flushing Meadows Park, 123-30 Roosevelt Avenue, (Map ID A4).

Following the standard DDC investigation report, a Phase II Subsurface Corridor Investigation (SCI) is proposed. The SCI will consist of two (2) soil borings per "High" risk site and one (1) soil boring per "Moderate" risk site to determine if the Corridor has been impacted. However, since the "High" and "Moderate" risk sites are predominately in close proximity to each other, the number of soil borings may be reduced by placing borings in representative locations along the Corridor. A total of eleven (11) soil borings are recommended in Table 2 based on DDC protocols. However, it should be noted that the MTA 7 Train line and LIRR Mets-Willets Point train stations as well as the subway car repair facility/subway tracks are present at grade level, for which access to the proposed soil boring locations may not be gained due to the close proximity of numerous train tracks.

During the Phase II SCI site assessment activities, photoionization detector (PID) screening will be instituted and soil logging will be conducted by a qualified geologist. The on-site field geologist will make a sampling determination based on the results of field screening including PID readings, visual and odor observations, and at the discretion of the DDC.

Report Prepared By:

Dyna Krumich-Ogonowski
Senior Project Scientist

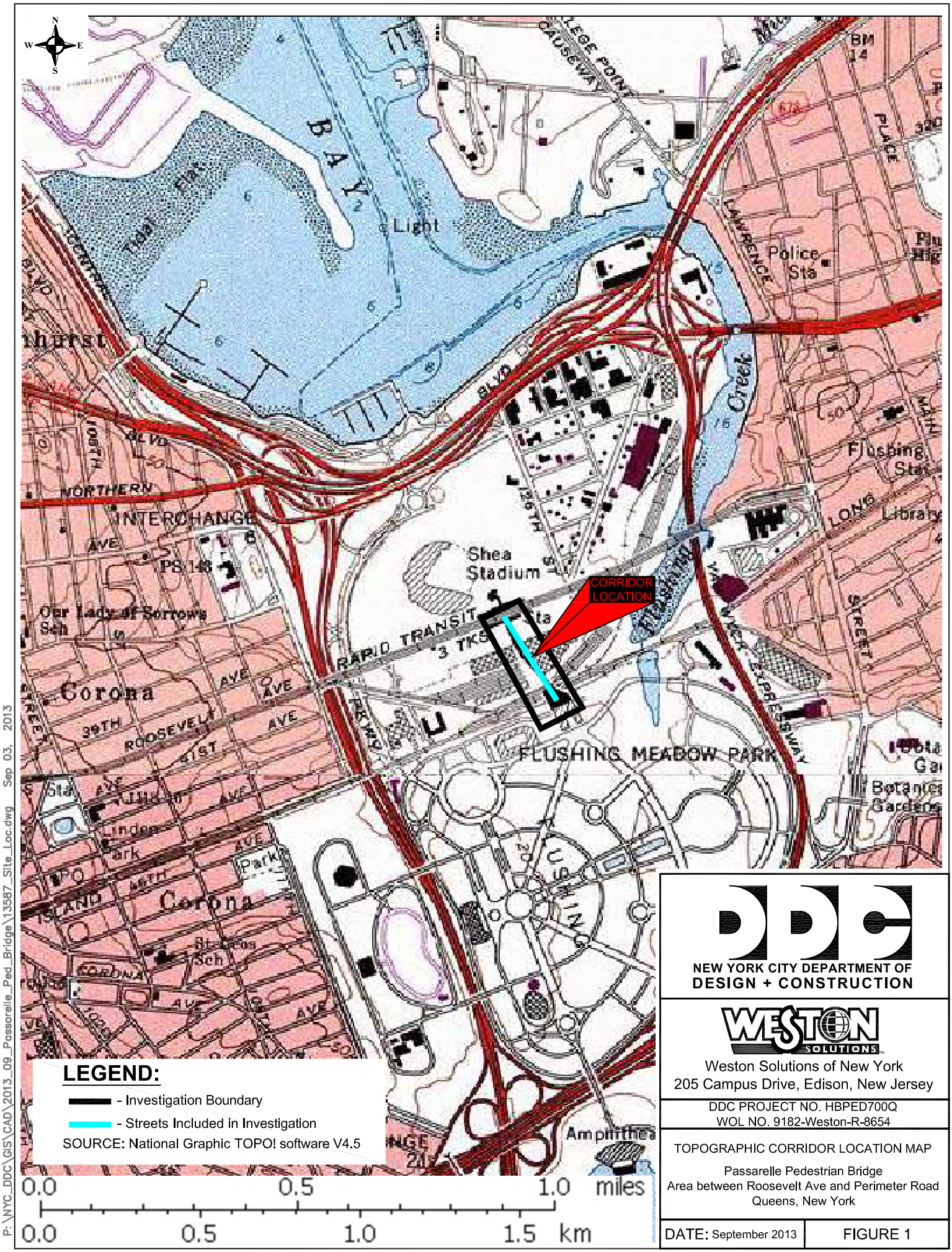
Report Reviewed By:

Michelle Afflitto
Lead Project Manager

Ryan Brown
Project Manager

FIGURES

FIGURE 1 – TOPOGRAHIC CORRIDOR LOCATION MAP
FIGURE 2 – CORRIDOR AREA MAP
FIGURE 3 – DATABASE RADIUS MAP SEARCH



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205 Campus Drive, Edison, New Jersey

DDC PROJECT NO. HBPED700Q
WOL NO. 9182-Weston-R-8654

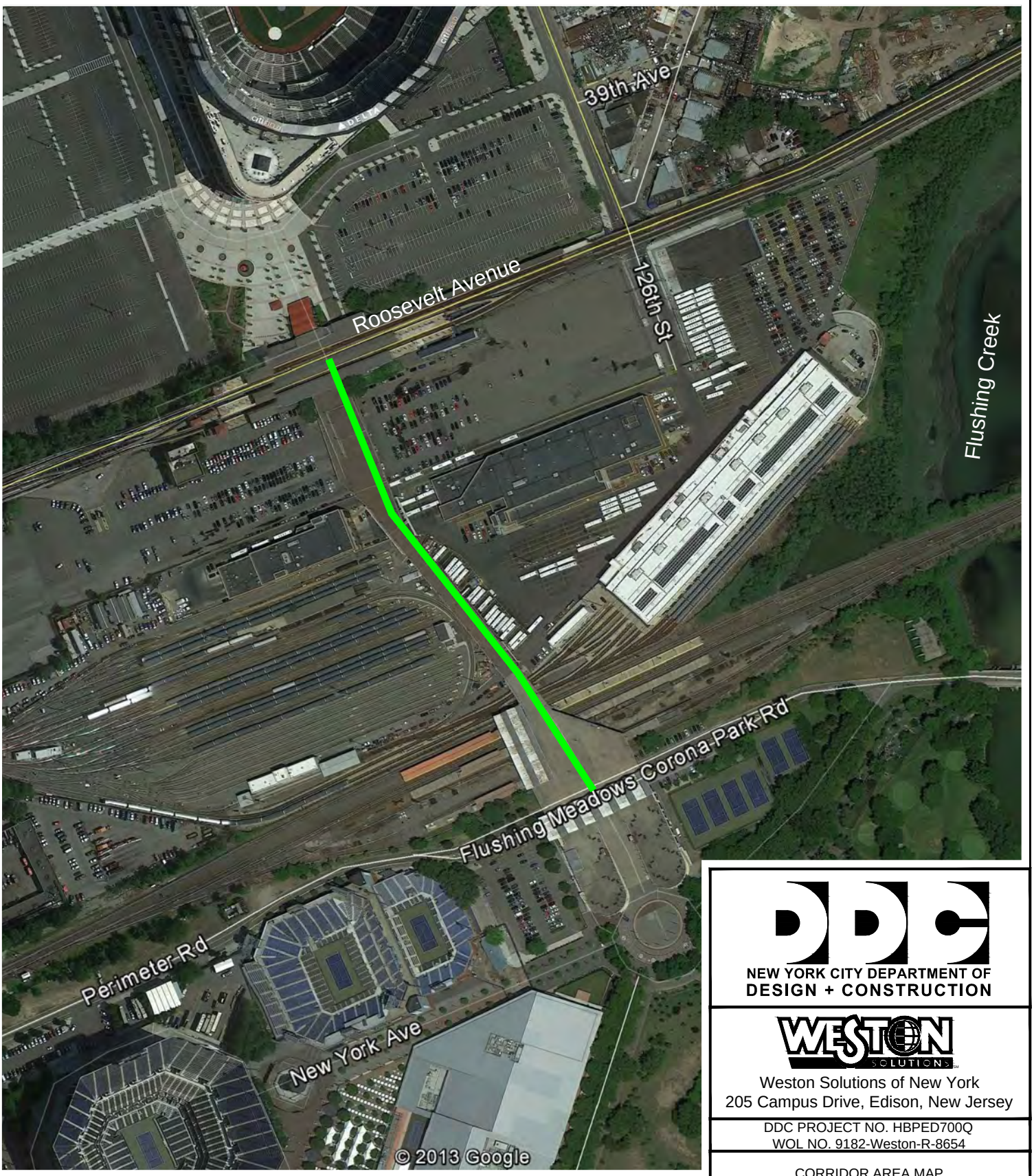
TOPOGRAPHIC CORRIDOR LOCATION MAP

Passarelle Pedestrian Bridge
Area between Roosevelt Ave and Perimeter Road
Queens, New York

DATE: September 2013

FIGURE 1

P:\NYC_DDC\GIS\CAD\2013_09_Passarelle_Ped_Bridge\13587_Site_Loc.dwg Sep 03, 2013



LEGEND:

EXTENT OF CORRIDOR

SOURCE: Google Earth Pro software - N.T.S.

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WOL NO. 9182-Weston-R-8654

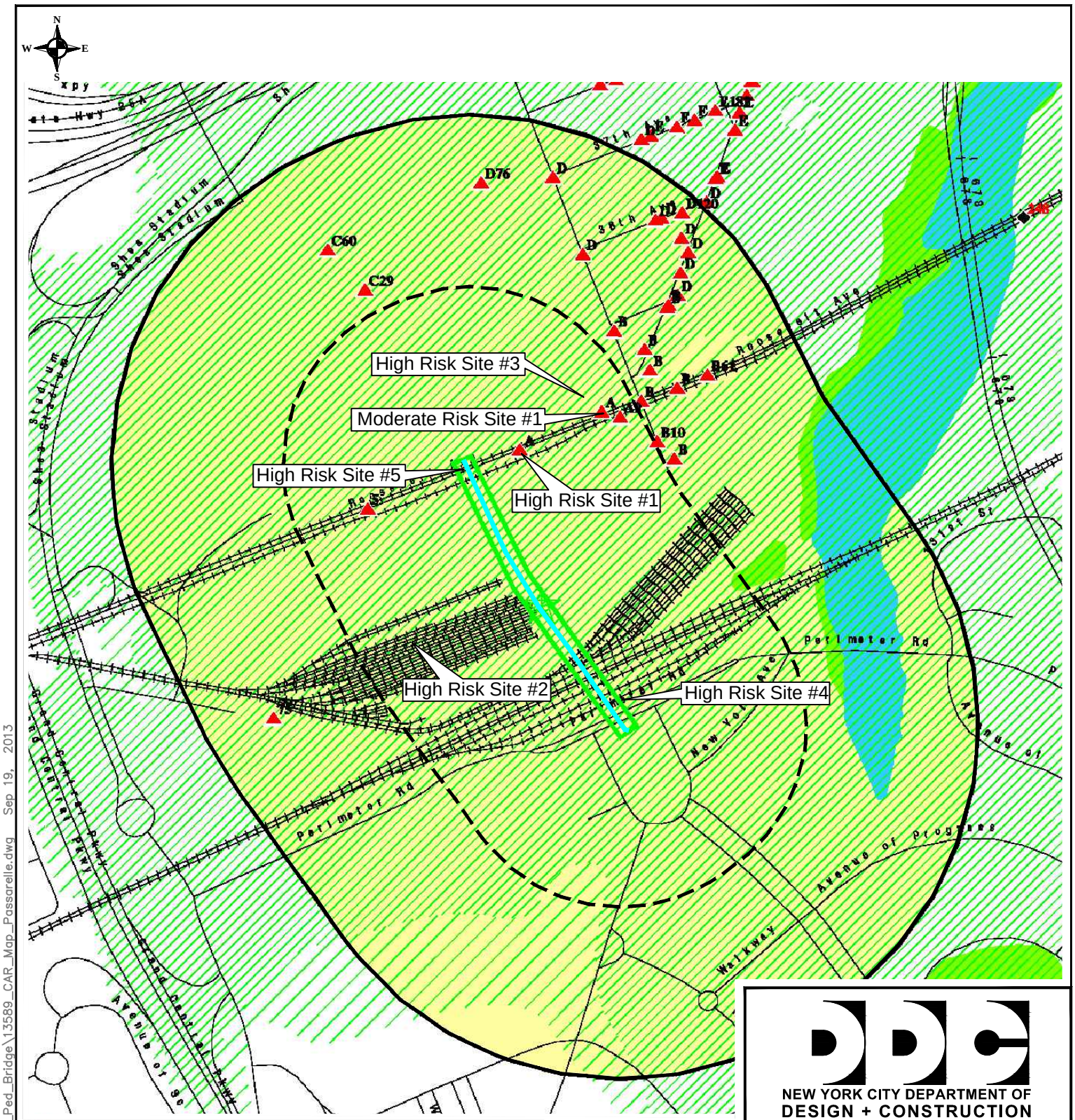
CORRIDOR AREA MAP

Passarelle Pedestrian Bridge
Area between Roosevelt Ave and Perimeter Road
Queens, New York

DATE: September 2013

FIGURE 2

P:\NYC_DDC\GIS\CAD\2013_09_Passarelle_Ped_Bridge\13588_Corr_Map.dwg Sep 04, 2013



Target Property

Sites at elevations higher than or equal to the target property

Sites at elevations lower than the target property

Manufactured Gas Plants

Sensitive Receptors

National Priority List Sites

Dept. Defense Sites

1/8 Mile Search Boundary

1/4 Mile Search Boundary

Investigation Boundary

Streets Included in Investigation

Indian Reservations BIA

Oil & Gas pipelines from USGS

100-year flood zone

500-year flood zone

National Wetland Inventory

State Wetlands

0 1/16 1/8 1/4 Mi

SOURCE: EDR Map. Inquiry #3687420.2s

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205 Campus Drive, Edison, New Jersey

DDC PROJECT NO. HBPED700Q
WOL NO. 9182-Weston-R-8654

DATABASE RADIUS MAP SEARCH

Passarelle Pedestrian Bridge
Area between Roosevelt Ave and Perimeter Road
Queens, New York

DATE: September 2013

FIGURE 3

P:\NYC_DDC\GIS\CAD\2013_09_Passarelle_Ped_Bridge\13589_CAR_Map_Passarelle.dwg Sep 19, 2013

TABLES

TABLE 1 – RISK CRITERIA

TABLE 2- SUMMARY ASSESSMENT RESULTS (HIGH AND MODERATE RISK SITES)

TABLE 1 - RISK CRITERIA

Risk Category	Land Use
HIGH	a. Sites identified on the regulatory agency database as NYSDEC Spill (NY Spills) sites b. Sites identified on the regulatory agency database as Leaking Storage Tank Incident Report (LTANKS) facilities c. Sites identified on the regulatory agency database as NYSDEC Inactive Hazardous Waste Disposal (IHWDS) sites d. Sites identified on the regulatory agency database as Emergency Response Notification System (ERNS) sites e. Sites observed during the Site Reconnaissance or identified on the regulatory agency database as RCRA Generators of hazardous waste (i.e., dry cleaners, service stations, etc.) f. Electric power/natural gas generation and transmission facilities (i.e., generators, substations, transformers, etc.) g. Transportation facilities - airports, heliports, bus depots, railroad track rights-of-way, harbors and marinas h. Motor vehicle dealerships with service stations, general automotive repair shops and service stations i. Industrial/Manufacturing facilities (i.e., paper mills, pulp mills, meat packing plants, textiles, wood finishing/preserving, fertilizers, cement, steel works, furnaces, foundries, motor vehicle parts and accessories) j. Petroleum wholesalers, retailers, storage facilities (i.e., gasoline filling stations, bulk terminals, refineries, etc.) k. High Intensity Agriculture (i.e., nurseries, farms, feed lots, orchards, etc.) l. Waste treatment and disposal facilities (i.e., landfills) m. Printers, photo shops n. Golf courses
MODERATE	o. Sites identified on the regulatory agency database as Underground Storage Tank (UST) facilities p. Hospitals, clinics, doctor's offices q. Construction activities (i.e., Highway and street construction, wrecking and demolition work) r. Commercial office buildings s. Warehouses t. Vacant land, previously developed
LOW	u. Retail buildings, restaurants, etc. v. Private residences, apartment buildings w. Schools x. Vacant land, no prior development y. Small Quantity (<30 gallons) Spills of Class I Materials Closed by NYSDEC

TABLE 2
SUMMARY ASSESSMENT RESULTS
(HIGH AND MODERATE RISK SITES)

Site ID	Map ID / Page No./ Photo Log No.	Site Address / Block-Lot[1]	Site Reconnaissance Information	Historical Use (Sanborn Map Review)		Regulatory Agency Database Review[2]	Potential Environmental Concern	Initial Risk Category (High or Moderate)	Modifying Information	Final Risk Category (High or Moderate)	Recommendations
HIGH RISK SITES											
1	A1/Page 8/Photo 1	Konica Photo Imaging within CitiField Stadium 123-01 Roosevelt Ave Block 1787 Lot 20	Off the corridor/Upgradient	1902 Sanborn Map:	N/A	RCRA NonGen/NLR - EPA ID No. NYR000114470	Metals	High	None	High	Two (2) soil borings are recommended within the Corridor in the vicinity of 123-01 Roosevelt Avenue to assess the impact from the "High" risk site.
				1914 Sanborn Map:	N/A						
				1931 Sanborn Map:	N/A						
				1950 Sanborn Map:	N/A						
				1986 Sanborn Map:	N/A						
				1995 Sanborn Map:	N/A						
				2007 Sanborn Map:	N/A						
2	Photo 2	Subway Car Repair Facility /Subway Tracks Block 2018 Lot 1000	On the corridor	1902 Sanborn Map:	Vacant	Not Available	Petroleum	High	Area identified during site walk and on Sanborns	High	Two (2) soil borings are recommended within the Corridor in the vicinity of the Subway Car Repair Facility to assess the impact from the "High" risk site.
				1914 Sanborn Map:	Vacant						
				1931 Sanborn Map:	Car Barn & Repair Shop						
				1950 Sanborn Map:	Subway Car Repairs						
				1986 Sanborn Map:	Subway Car Repairs						
				1995 Sanborn Map:	Subway Car Repairs						
				2007 Sanborn Map:	Subway Car Repairs						
3	Photo 3	Skylights - Auto Sales Facility/Auto Repair Facility with Gasoline USTs 12050 Unknown Road Block 2018 Lot 1500	Off the corridor/Upgradient	1902 Sanborn Map:	Vacant	Not Available	Petroleum	High	Property identified on 1931 Sanborn	High	Two (2) soil borings are recommended within the Corridor in the vicinity of the former Skylights - Auto Sales Facility to assess the impact from the "High" risk site.
				1914 Sanborn Map:	Vacant						
				1931 Sanborn Map:	Auto Facility						
				1950 Sanborn Map:	Parking						
				1986 Sanborn Map:	Parking						
				1995 Sanborn Map:	Parking						
				2007 Sanborn Map:	Parking						
4	Photo 4	LIRR Mets-Willets Point Train Station/ Perimeter Road/Block 2018 Lot 1000	On the Corridor	1902 Sanborn Map:	Vacant	Not Available	Petroleum	High	Area identified during site walk and on Sanborns	High	Two (2) soil borings are recommended within the Corridor in the vicinity of the train station to assess the impact from the "High" risk site.
				1914 Sanborn Map:	Vacant						
				1931 Sanborn Map:	Car Barn & Repair Shop						
				1950 Sanborn Map:	Subway Car Repairs						
				1986 Sanborn Map:	Subway Car Repairs						
				1995 Sanborn Map:	Subway Car Repairs						
				2007 Sanborn Map:	Subway Car Repairs						
5	Photo 5	MTA Mets-Willets Point Train Station/ Roosevelt Avenue/Block 2018 Lot 1000	On the Corridor	1902 Sanborn Map:	Vacant	Not Available	Petroleum	High	Area identified during site walk and on Sanborns	High	Two (2) soil borings are recommended within the Corridor in the vicinity of the train station to assess the impact from the "High" risk site.
				1914 Sanborn Map:	Vacant						
				1931 Sanborn Map:	Car Barn & Repair Shop						
				1950 Sanborn Map:	Subway Car Repairs						
				1986 Sanborn Map:	Subway Car Repairs						
				1995 Sanborn Map:	Subway Car Repairs						
				2007 Sanborn Map:	Subway Car Repairs						
MODERATE RISK SITE											
1	A4/Page 15/Photo 6	Flushing Meadows Park 123-30 Roosevelt Ave Block 2018 Lot 1	Off the corridor/Downgradient	1902 Sanborn Map:	Vacant	NY LTANKS - Spill No. 0807272 (tank test failure with no discernable spill. CLOSED 10/12/2010)	Petroleum	High	Spill is closed	Moderate	One (1) soil boring is recommended within the Corridor in the vicinity of 123-30 Roosevelt Avenue to assess the impact from the "Moderate" risk site.
				1914 Sanborn Map:	Vacant						
				1931 Sanborn Map:	Building						
				1950 Sanborn Map:	Commercial						
				1986 Sanborn Map:	Public Park						
				1995 Sanborn Map:	Public Park						
				2007 Sanborn Map:	Public Park						

[1] Block and Lot information obtained from NYC Department of Buildings, where available.

[2] The regulatory database search information for this table was obtained from Environmental Data Resources, Inc.

APPENDIX A

SITE RECONNAISSANCE PHOTOGRAPHS

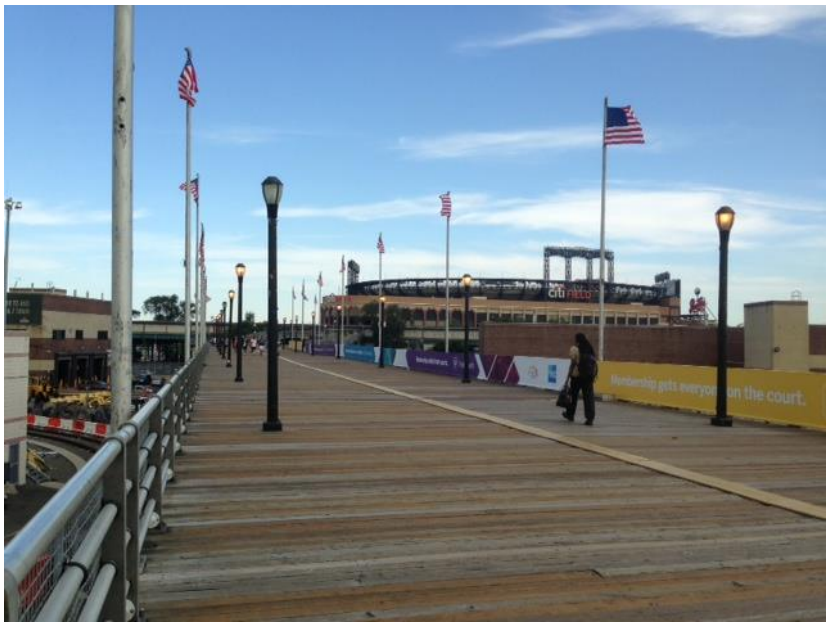


Photo 1 – Konica Photo Imaging within Citi Field, 123-01 Roosevelt Avenue, (Map ID A1), Off the Corridor (HIGH RISK SITE #1)

Photo 2 – Subway Car Repair Facility/Subway Tracks, Block 2018 Lot1000, (No Map ID), On the Corridor (HIGH RISK SITE #2)

Photo 3 – Former Skylights Auto Sales Facility/Auto Repair Facility, 12050 Unknown Road, (No Map ID), Off the Corridor (HIGH RISK SITE #3)

Photo 4 – LIRR Mets-Willets Point Train Station, Roosevelt Avenue, (No Map ID), On the Corridor (HIGH RISK SITE #4)

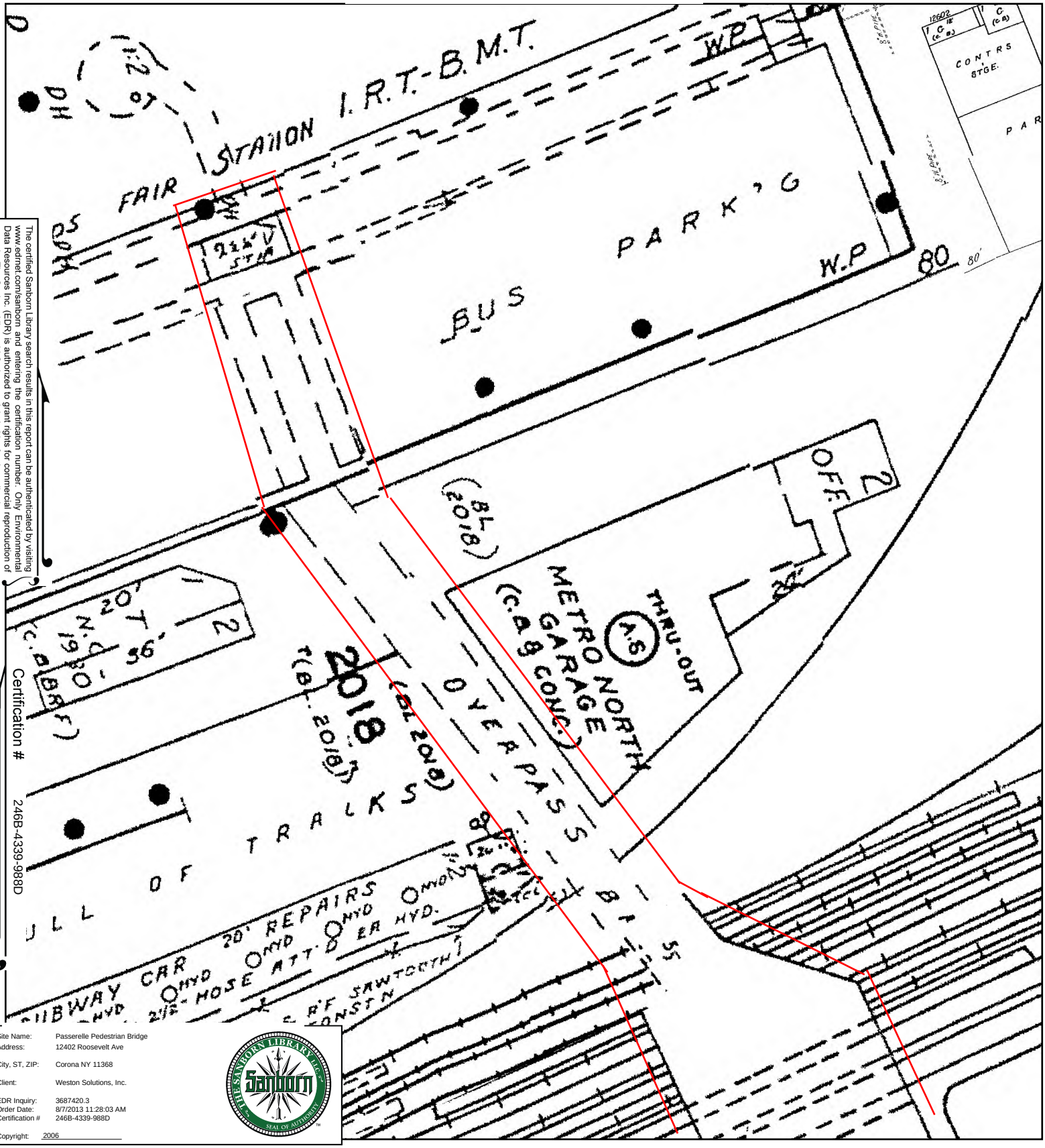
Photo 5 – MTA Mets-Willets Point Train Station, Perimeter Road, (No Map ID), On the Corridor (HIGH RISK SITE #5)

Photo 6 – Flushing Meadows Park, 123-30 Roosevelt Avenue, (Map ID A4), On the Corridor (MODERATE RISK SITE #1)

APPENDIX B

SANBORN FIRE INSURANCE MAPS

2006 Certified Sanborn Map



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Certification # 246B-4339-988D

Site Name:	Passerelle Pedestrian Bridge
Address:	12402 Roosevelt Ave
City, ST, ZIP:	Corona NY 11368
Client:	Weston Solutions, Inc.
EDR Inquiry:	3687420.3
Order Date:	8/7/2013 11:28:03 AM
Certification #	246B-4339-988D
Copyright:	2006

This Certified Sanborn Map combines the following sheets.
Outlined areas indicate map sheets within the collection.

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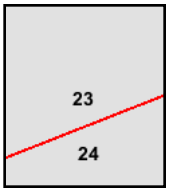

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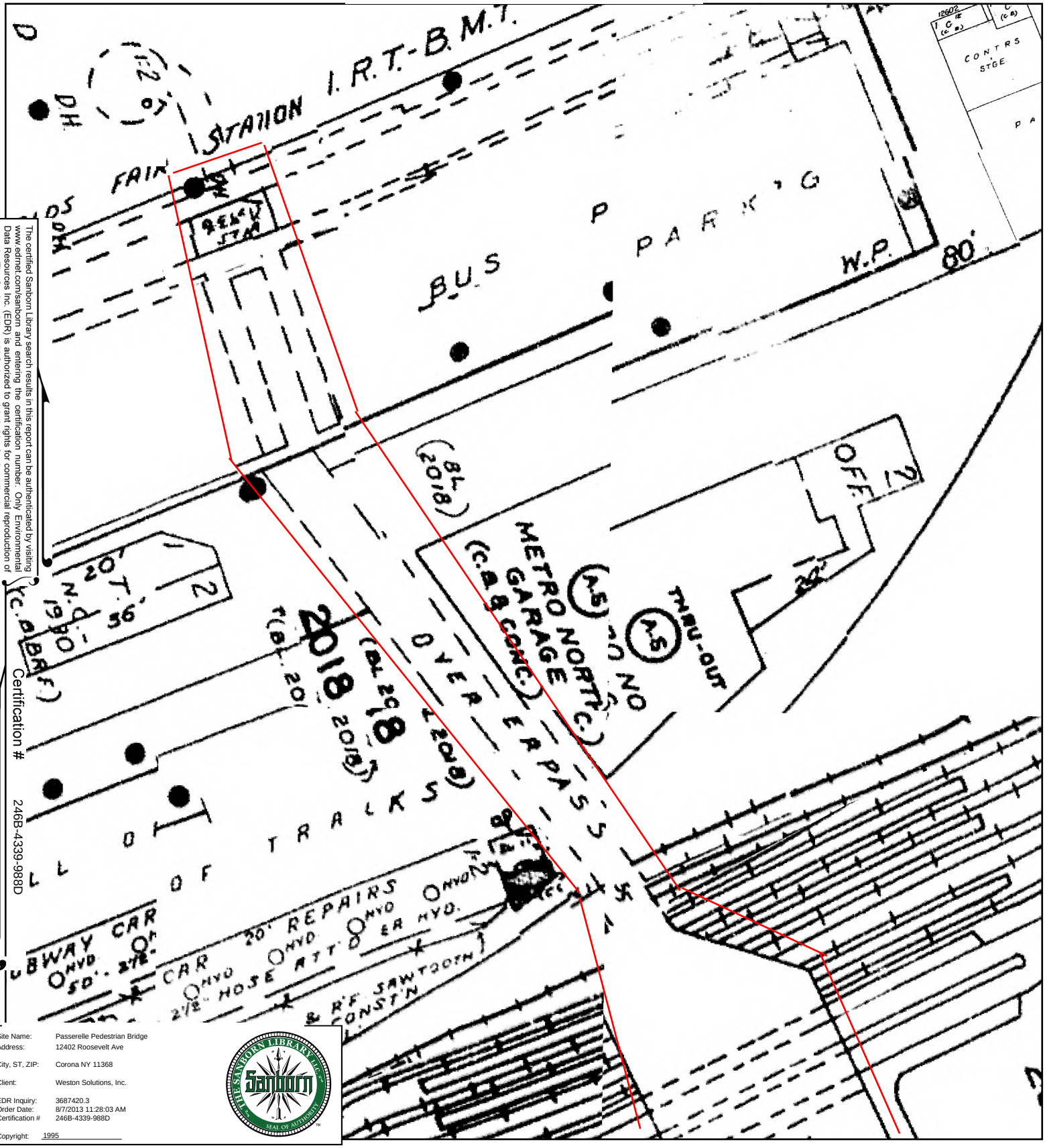
1995 Certified Sanborn Map



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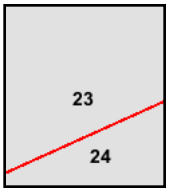

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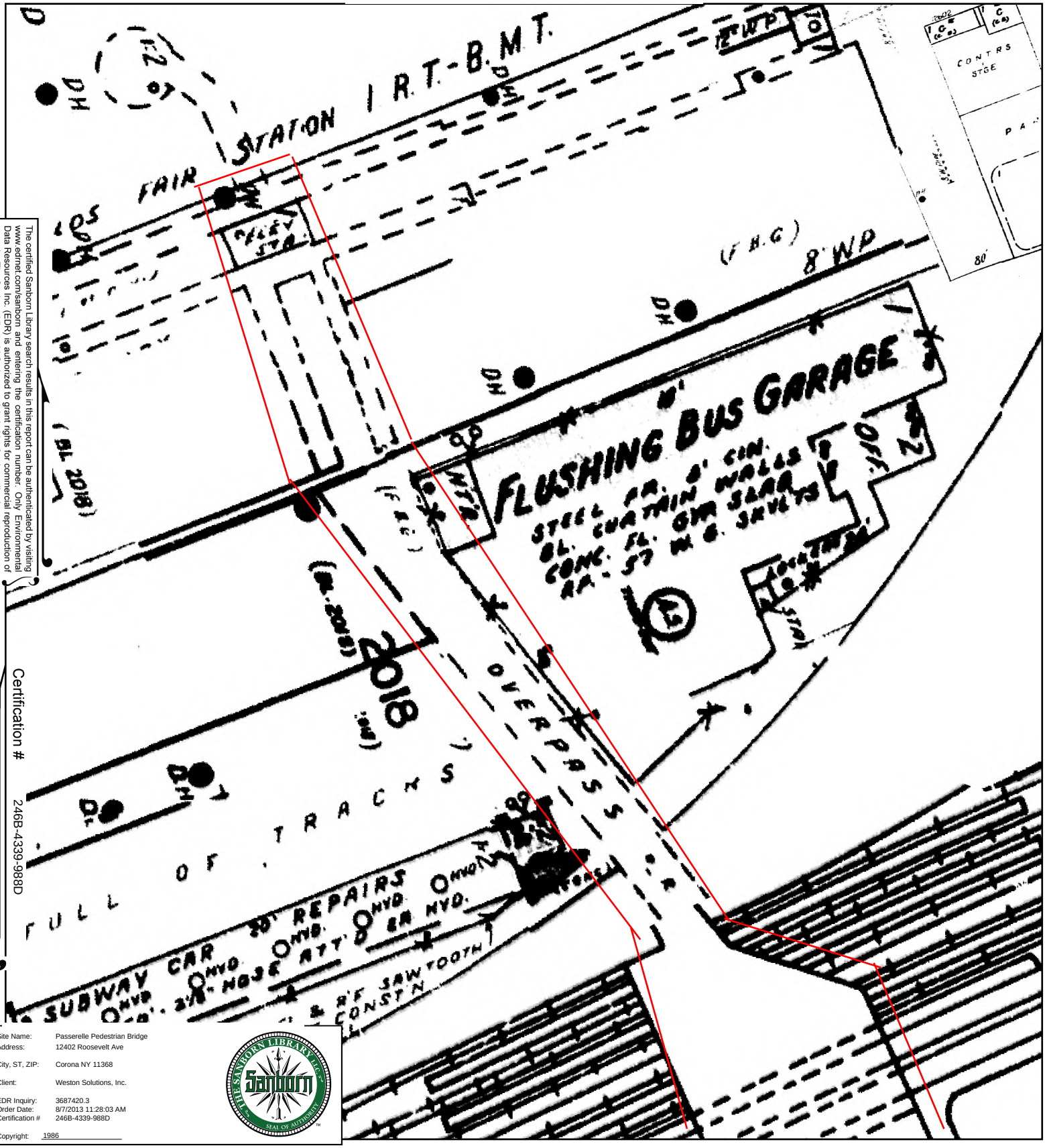
N

1986 Certified Sanborn Map



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Certification # 246B-4339-988D

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Address:	12402 Roosevelt Ave
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Client:	Weston Solutions, Inc.
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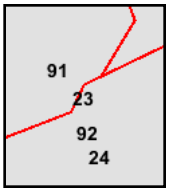

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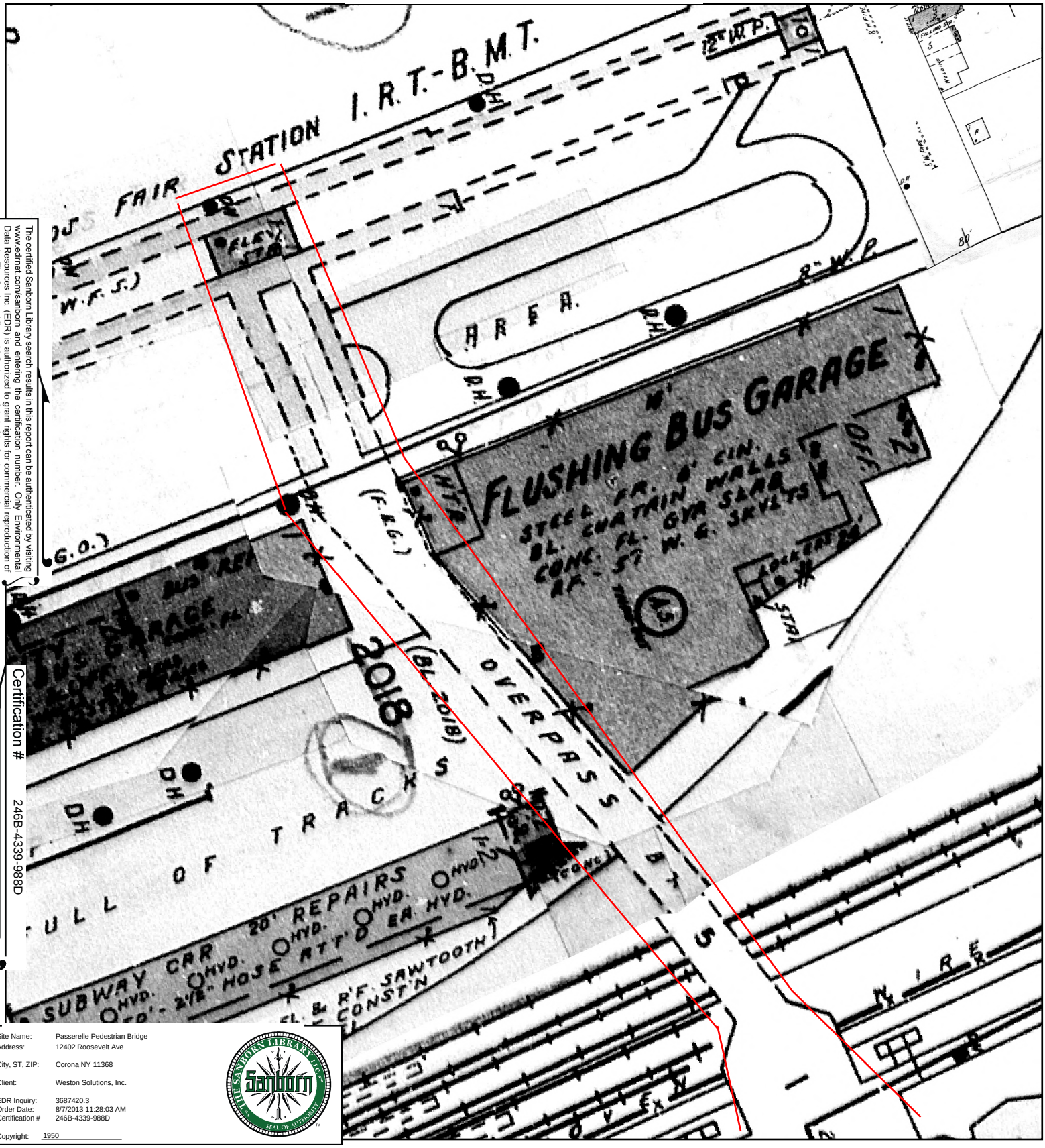
N

1950 Certified Sanborn Map



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Client:	Weston Solutions, Inc.
EDR Inquiry:	3687420.3
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Copyright:	1950

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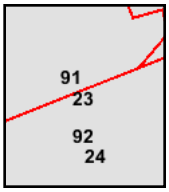

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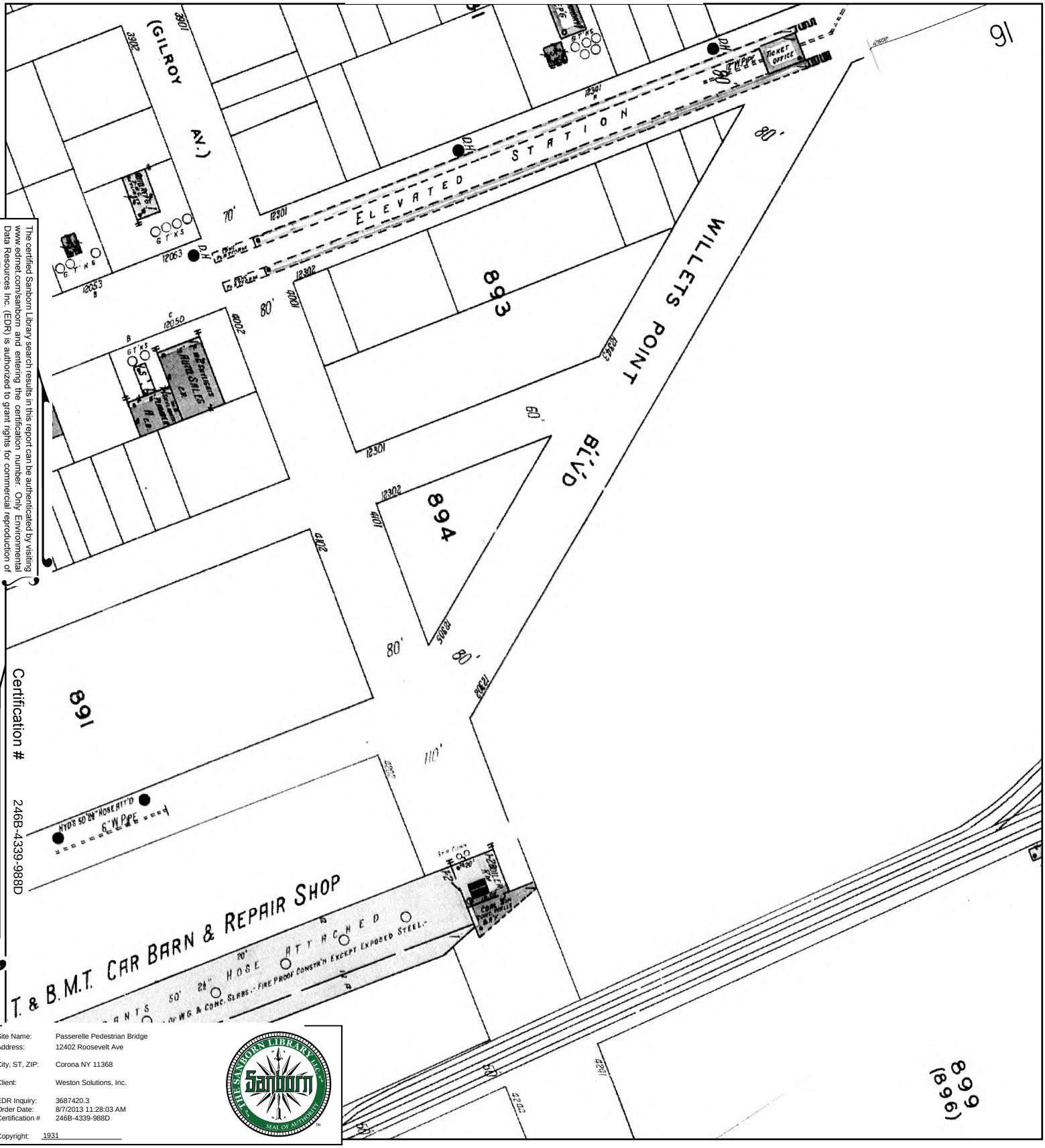
1931 Certified Sanborn Map



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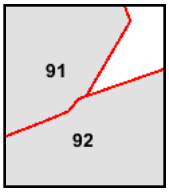

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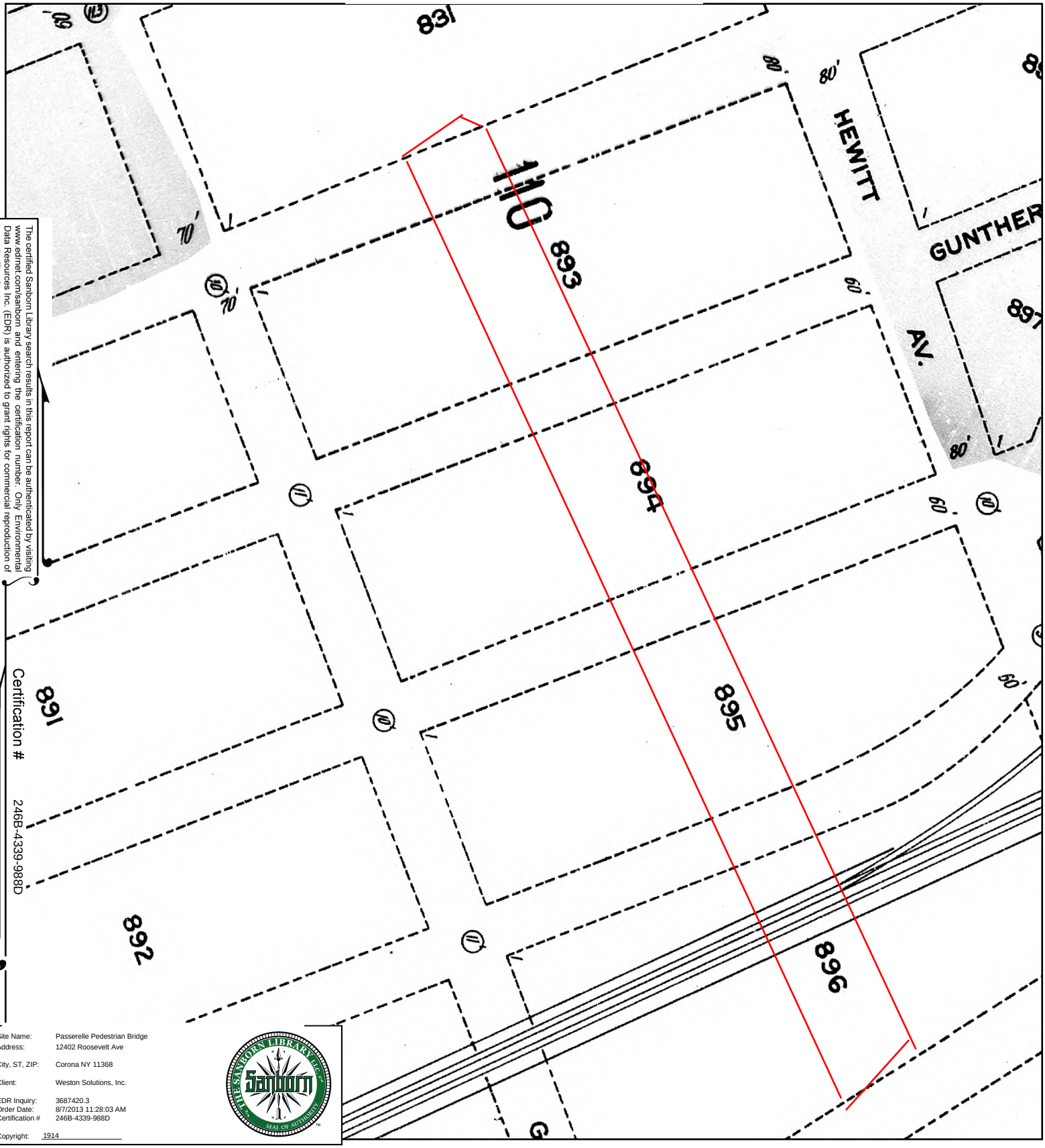
1914 Certified Sanborn Map



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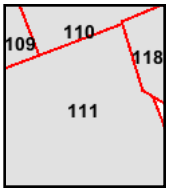

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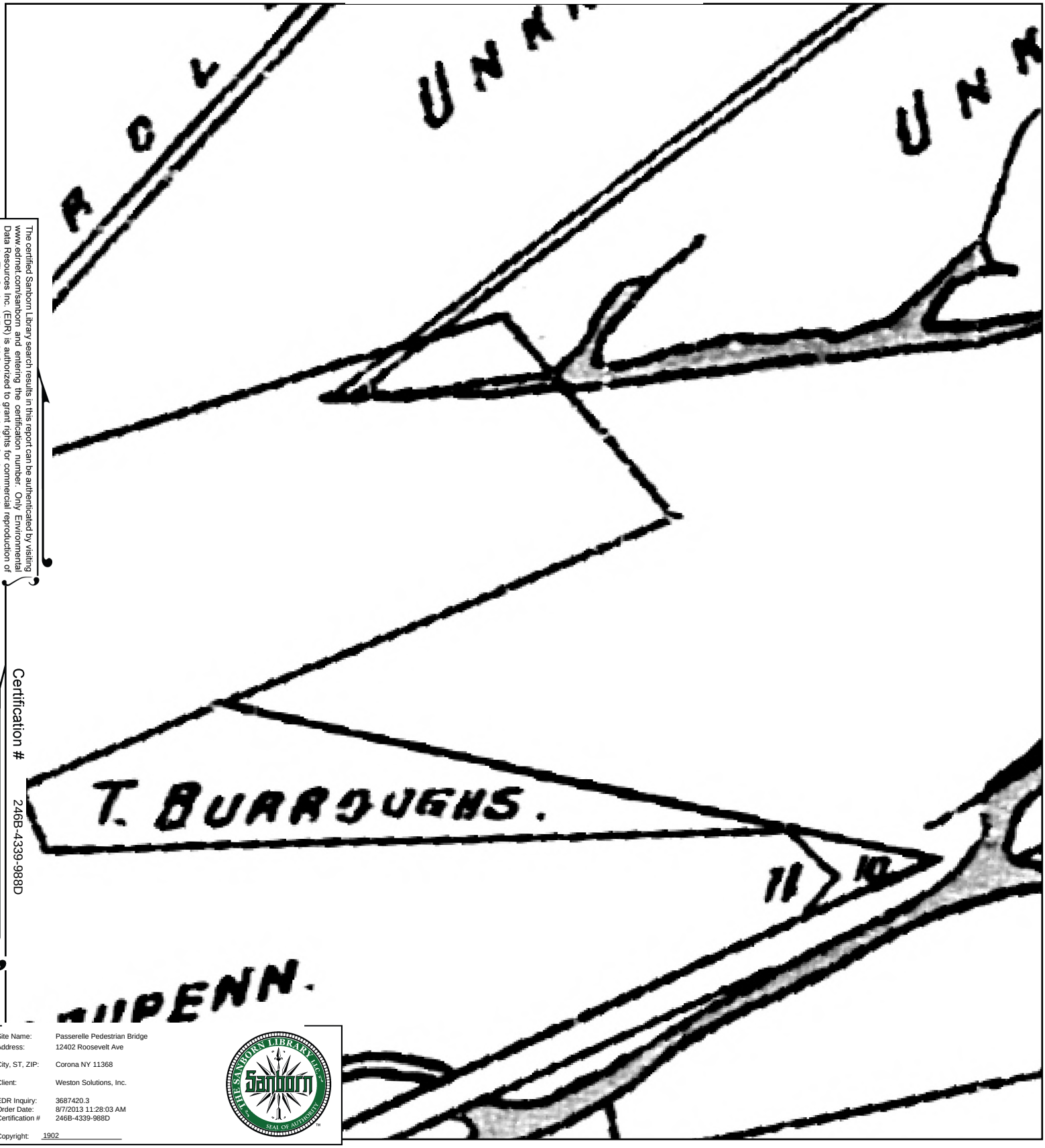
1902 Certified Sanborn Map



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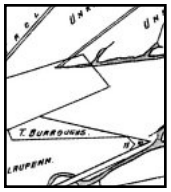

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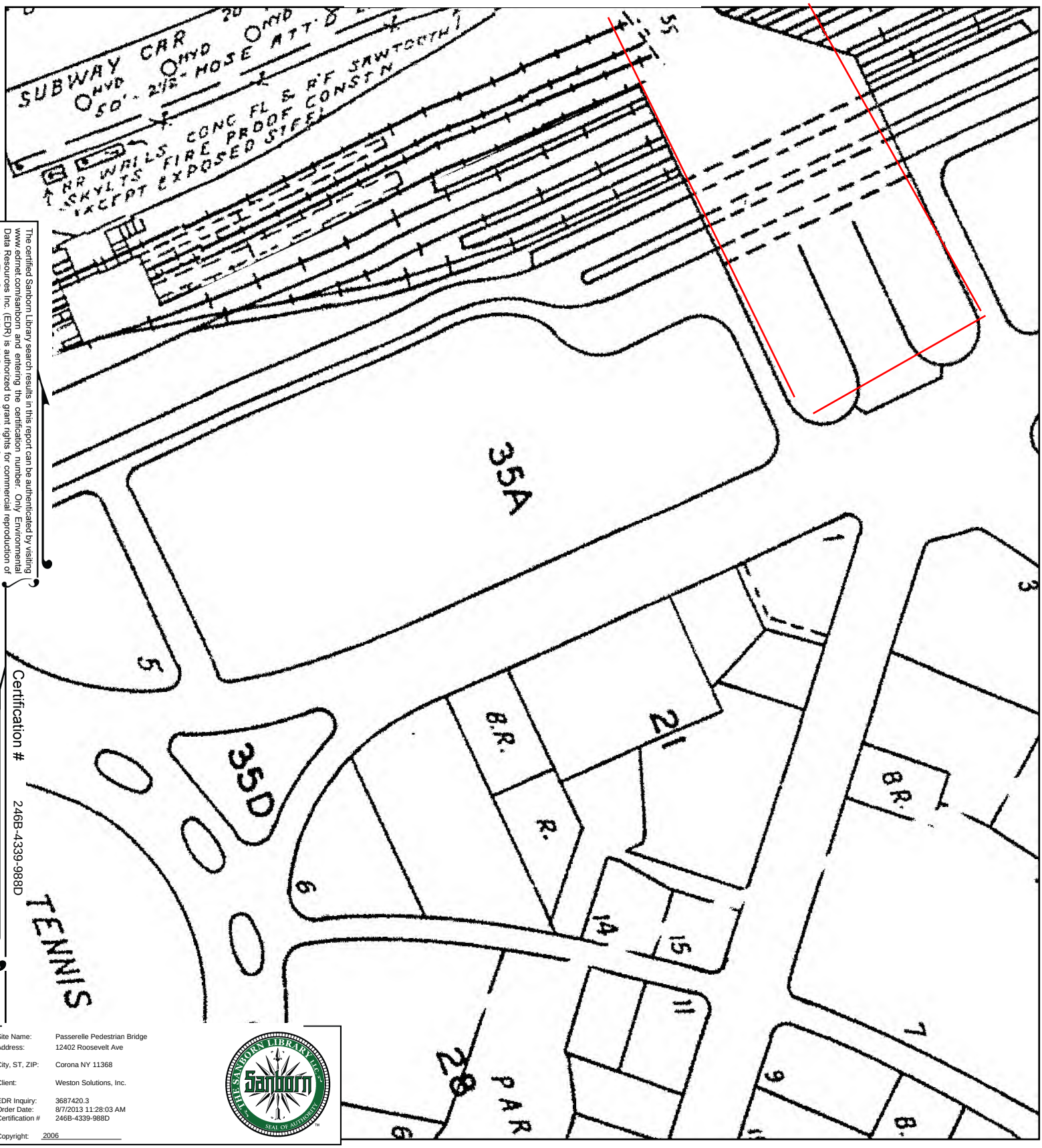
2006 Certified Sanborn Map



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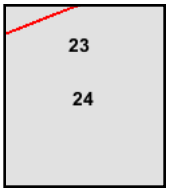

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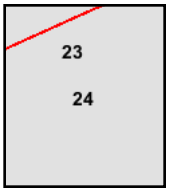

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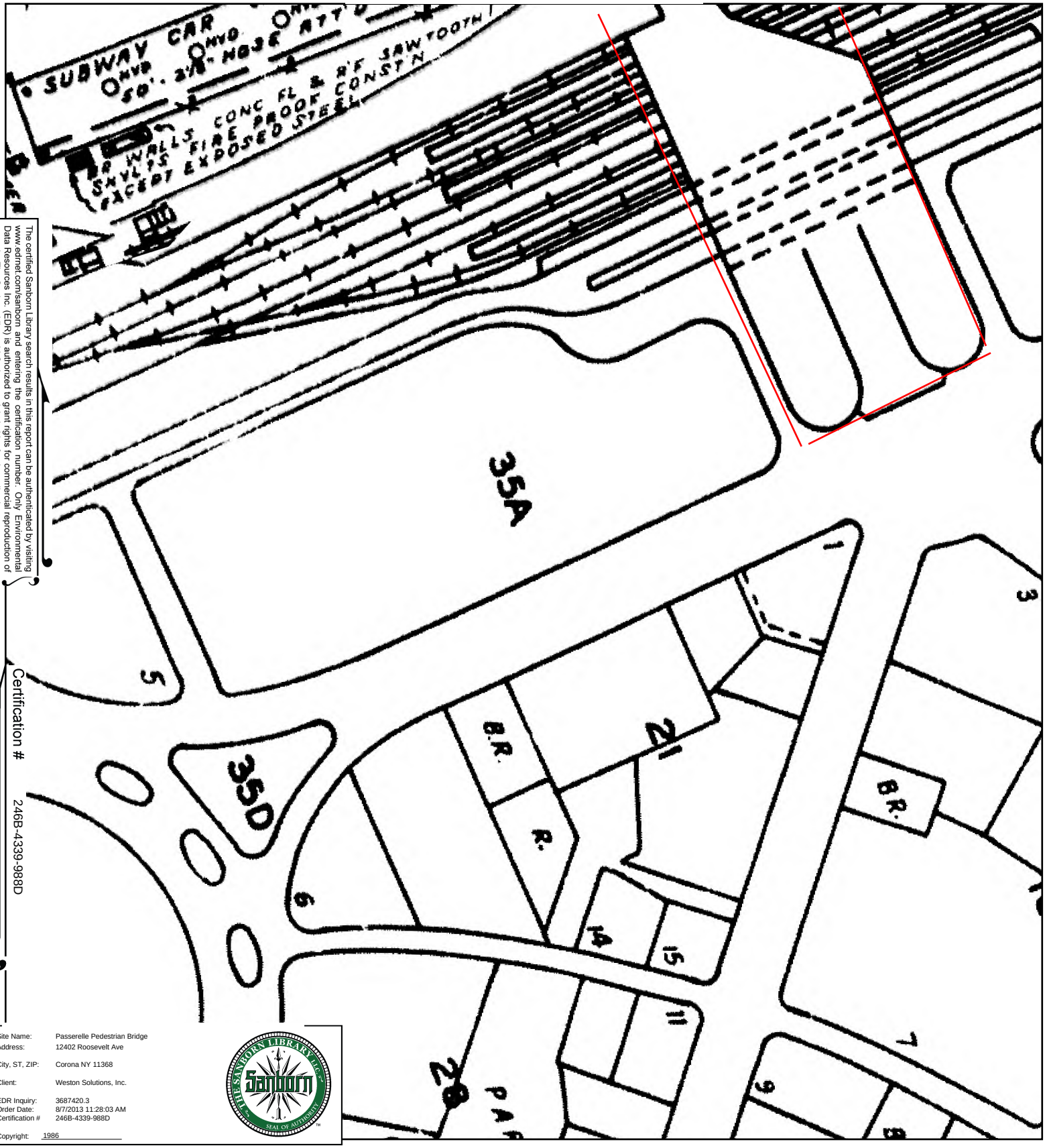
1986 Certified Sanborn Map



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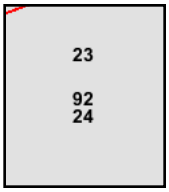

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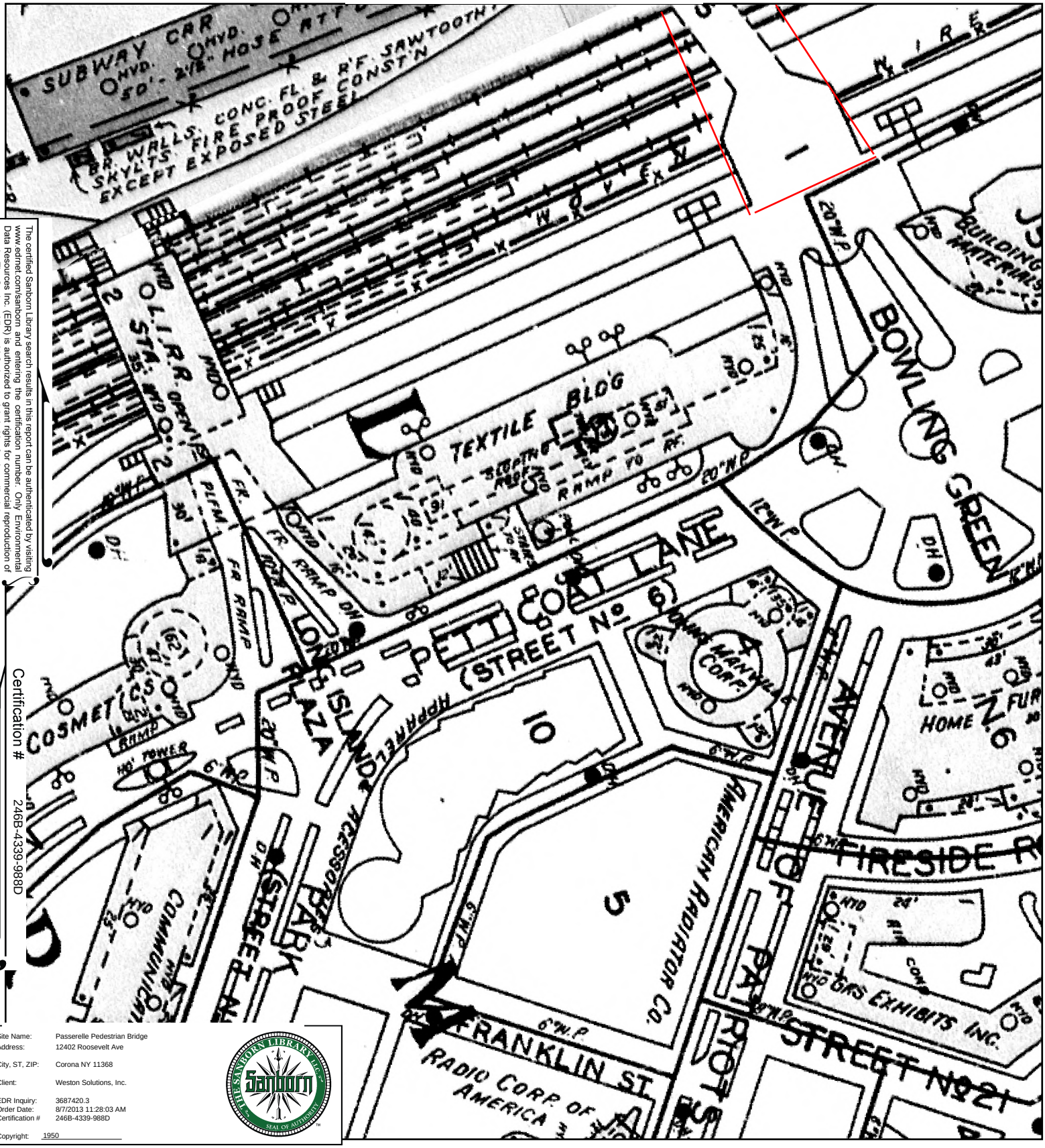
1950 Certified Sanborn Map



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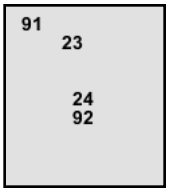

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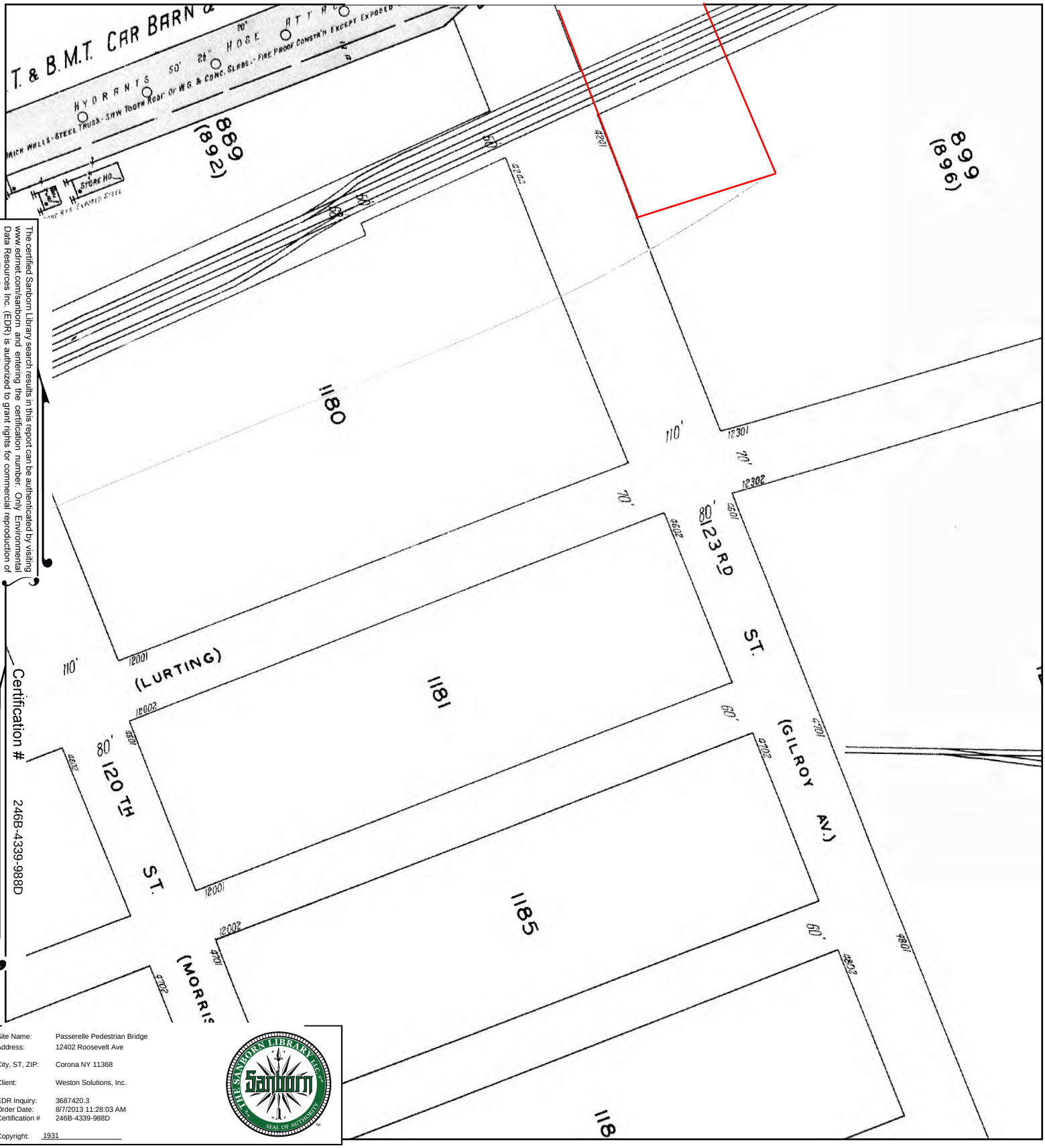

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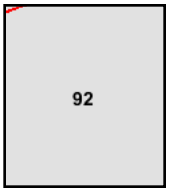

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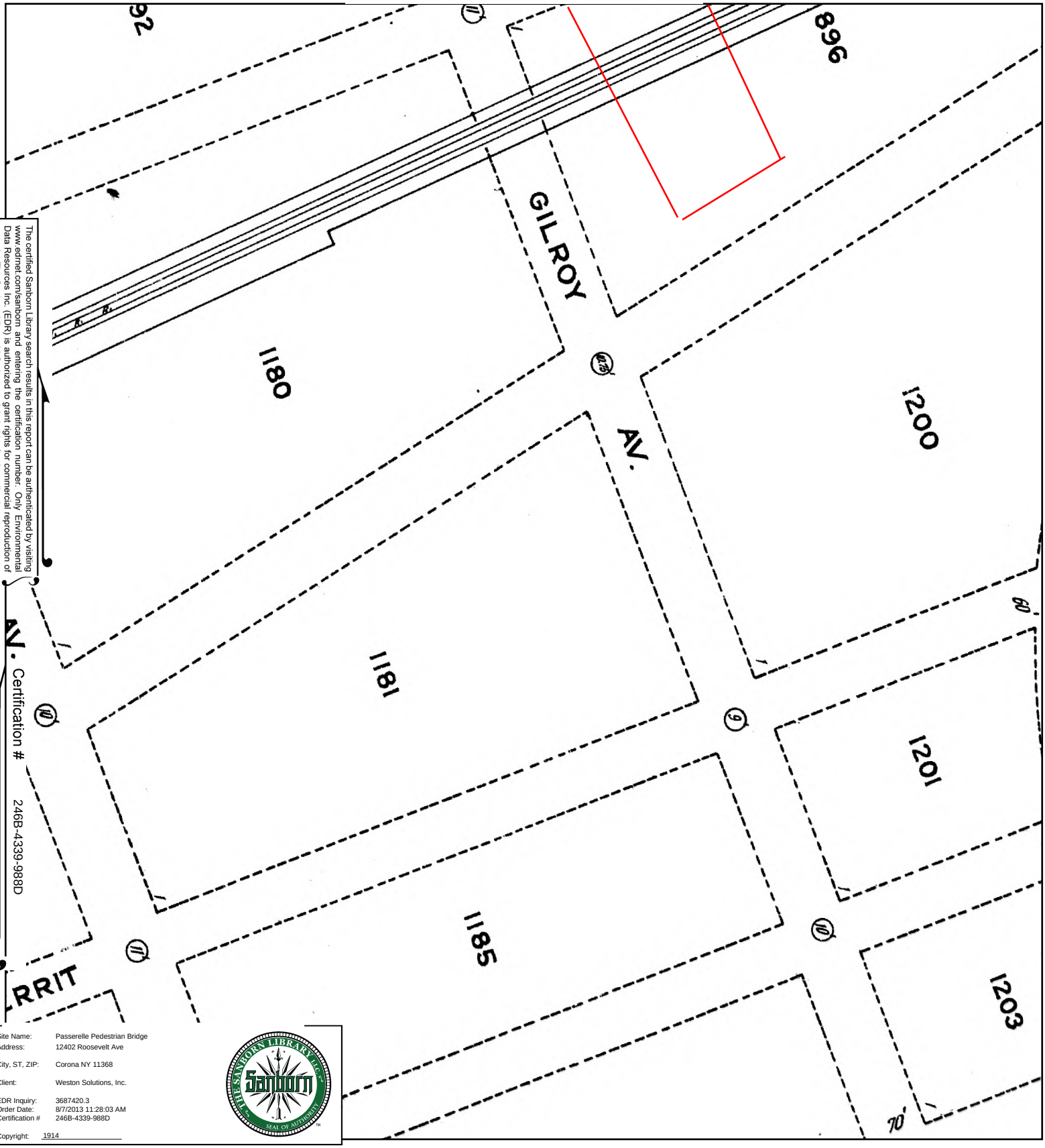

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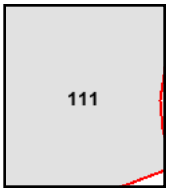

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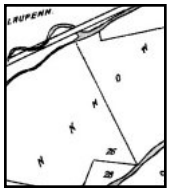

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APPENDIX C

REGULATORY AGENCY DATABASE REPORT (INCLUDED ON CD)

APPENDIX D

QUALIFICATIONS OF ENVIRONMENTAL PROFESSIONALS

MICHELA S. AFFLITTO

Qualifications Summary

- Over 12 years of experience in preliminary investigation/site inspection (PA/SI), remedial investigation (RI), and remedial action (RA) preparation, management, and implementation.
- Primary author on several preliminary assessments, site investigations, Remedial Investigation Work Plans and reports, Health and Safety Plans, Remedial Action Work Plans and reports, Soil Reuse Plans, Quality Assurance Project Plans, and Environmental Cost Estimate Reports.
- Management of municipality brownfields redevelopment program including assistance with Hazardous Discharge Site Remediation Fund (HDSRF) and U.S. environmental Protection agency (EPA) grant applications; coordination with New Jersey Economic Development Authority (NJEDA) and New Jersey Department of Environmental Protection (NJDEP); and development of site-specific scope of work.
- Municipality environmental management activities including coordination with federal/state/local agencies; underground storage tank (UST) system removal, upgrade, and installation; immediate remedial action coordination; and oversight of field activities.
- Municipality environmental compliance management program implementation, including more than 60 public buildings.

Education

B.S., Environmental Engineering—Polytechnic University (1997)

Credentials

40-Hour Hazardous Waste Site Training Course, OSHA 29 CFR 1910.120(e)(3), IMS Safety Services, Inc. (1998)
8-Hour Hazardous Waste Refresher Course, OSHA 29 CFR 1910.120(e)(8), WESTON (2009)
8-Hour Hazardous Waste Site Supervisor Training Course, OSHA 29 CFR 1910.120(e)(4), Woodward-Clyde (1998)
24-Hour Asbestos Abatement Inspector, EPA AHERA/ NYSDOH, New York Environmental Training Institute (1998)
4-Hour Asbestos Abatement Inspector Refresher Course, EPA AHERA/NYSDOH, NAETI (2003)

Employment History

2004-Present	WESTON
1997-2004	URS Corporation
1993-1997	Maher Terminals

Key Projects

Industrial Site Recovery Act (ISRA) Investigation/Remedial Investigation (RI)/Remediation, Newark, NJ, Confidential Client, Project Engineer/Project Manager. The site has been accepted into the New Jersey Department of Environmental Protection (NJDEP) Brownfield Development Area (BDA) Program. The acceptance into this program required the ISRA-related activities to move on an accelerated schedule to allow for the redevelopment of the property, and consideration of development issues during remedy selection and continued operations. Work at this former specialty chemical manufacturing plant consisted of the investigation and remediation of contaminated soil and groundwater. Has coordinated and managed several soil and groundwater investigations at the site, which have consisted of the remedial investigation of 33 areas of concern (AOCs), including 4 separate aquifer units, and a sediment and surface-water investigation of an adjacent water body. AOC-specific Remedial Action Work Plans (RAWPs) have been prepared, approved by the NJDEP, and implemented, which included soil excavation and off-site

Key Projects (Continued)

disposal of dioxin-contaminated and volatile organic compound (VOC)-contaminated soil. Also prepared the site-wide RAWP, which includes recommendations for remediation of soil, groundwater, and vapor intrusion submitted to NJDEP.

Preliminary Assessment (PA)/Site Investigation (SI)/Remedial Investigation (RI)/Remediation, Former Humble Oil, Newark, NJ, City of Newark, Project Manager. Managed the completion of a PA and SI for submittal to NJDEP. The PA identified five AOCs. The SI activities were conducted to investigate these AOCs and indicated that lead was present in soil in one AOC at levels above New Jersey Department of Environmental Protection (NJDEP) remedial standards. Based on the results of the SI, an RI was conducted, which included the installation of soil borings to delineate the vertical and horizontal extent of lead-contaminated soil encountered during the SI. In order to facilitate the unrestricted re-use of the site, following the RI activities active remediation of the lead-contaminated soil was completed through soil excavation. Post-excavation soil samples were collected following the soil excavation activities to confirm lead-contaminated soil had been removed. A remedial investigation/remedial action (RI/RA) report was prepared and submitted to NJDEP.

Underground Storage Tank (UST) Closure/Remedial Investigation (RI)/Remediation, First Street Park, Newark, NJ, City of Newark, Project Manager. Managed the closure of five USTs including two 3,000-gallon gasoline USTs, one 4,000-gallon gasoline UST, and two 1,000-gallon gasoline USTs. All the USTs had evidence of leaking and required remediation. As a result, soil remediation was conducted, which entailed the excavation and off-site disposal of over 300 tons of contaminated soil. The closure activities performed were reported to New Jersey Department of Environmental Protection (NJDEP) in a UST Closure Report and complied with the NJDEP UST requirements in the New Jersey Administrative Code (N.J.A.C. 7:14B).

Remedial Investigation (RI), Former Duralac Facility, Newark, NJ, City of Newark, Project Manager. Managed RI activities including the preparation of a New Jersey Department of Environmental Protection (NJDEP)-approved Work Plan, supervision of field work, which included advancement of 17 soil borings, and installation and sampling of 5 monitoring wells. The results were summarized in an RI Report, which included findings, recommendations, and conclusions for a remedial action.

Remedial Investigation (RI), Former Synfax Facility, Newark, NJ, City of Newark, Project Manager. Managed RI activities including the preparation of a New Jersey Department of Environmental Protection N(JDEP-)approved Work Plan, supervision of field work, which included advancement of five soil borings, and the installation and sampling of five monitoring wells. The results were summarized in an RI Report, which included findings, recommendations, and conclusions for a remedial action.

U.S. Environmental Protection Agency (EPA) Grant Applications, Newark, NJ, City of Newark, Project Manager. Managed the preparation of EPA assessment and clean-up grant applications on behalf of the City of Newark, with a total award of $1 million to date.

Environmental Support, Various Locations, City of Newark, NJ, Project Manager. Provided environmental support and consulting services to various city departments. Managed over 15 brownfield sites funded through Hazardous Discharge Site Remediation Fund (HDSRF). Coordinated with New Jersey Department of Environmental Protection (NJDEP) and New Jersey

Key Projects (Continued)

Economic Development Authority (NJEDA) to obtain additional funding from HDSRF, close-out project (completing Schedule B), and assisted city in providing project status to NJEDA. Provided assistance to perform the day-to-day activities and tasks typically conducted by the Project Manager of the Environmental Division, including management of over 25 sites where different stages of environmental investigations were being conducted. Project also included preparation of an environmental status report each month to provide updates to the Department of Engineering. The reports included narrative descriptions for the active projects, including an update on activities performed and necessary actions.

Underground Storage Tank (UST) Investigations, Newark, NJ, City of Newark, Project Manager. Managed the oversight of five UST removals at various locations in the city. Prepared technical memo to the city with recommendations for remedial actions.

Brownfields Redevelopment Program, City of Newark, NJ, Assistant Project Manager. Managed environmental assessments (EAs) of 11 brownfields properties. This work included assistance in obtaining state grants for the city, coordination with several city departments to facilitate the progress of the projects, preliminary assessments (Pas) and site investigations (SIs) of each property, and the management of remedial investigations (RIs) and remedial action selections.

Groundwater Remediation, City Hall Parking Lot, City of Newark, NJ, Project Manager. Provided assistance to comply with the requirements and immediate measures stipulated by New Jersey Department of Environmental Protection (NJDEP) for the groundwater remediation as a result of a former leaking gasoline underground storage tank (UST). This included the preparation of a draft Classified Exception Area (CEA) for groundwater contamination. Work included the review of historical reports and data to be incorporated into the CEA for submittal to NJDEP prior to the installation of a groundwater treatment system. Coordinated activities for the preparation of a groundwater remediation specification, groundwater sampling, indoor air sampling, and design and installation of an indoor air remediation system. Project also included the preparation of quarterly Remedial Action Progress Reports (RAPRs) to provide an update to NJDEP. The RAPRs included details of all activities completed in the past period, proposed activities for the upcoming period, and anticipated challenges.

Soil Remediation, Engine Company 28 Site, City of Newark, NJ, Project Manager. Coordinated activities for the remediation of copper-contaminated soil. Project involved the removal of up to 25 cubic yards of contaminated soil. Project also entailed preparation of a Remedial Action Report and Baseline Ecological Evaluation in accordance with New Jersey Department of Environmental Protection (NJDEP) requirements. No Further Action (NFA) from NJDEP was successfully granted.

Groundwater Sampling, Newark, NJ, Confidential Client, Project Engineer. Coordinated field activities for the groundwater sampling of monitoring wells. Utilized low-flow sampling procedures in accordance with U.S. Environmental Protection Agency (EPA) protocols. Also supervised activities of subcontractors.

Phase I Environmental Site Assessment (ESA) Reports, New Jersey and Maryland, Confidential Client, Project Engineer. Prepared Phase I ESA Reports for three manufacturing plants in accordance with American Society for Testing and Materials (ASTM) standards.

Key Projects (Continued)

Project components entailed compilation, review, and interpretation of regulatory database files, aerial photographs, fire insurance maps, and municipal office information. Project also consisted of preparing Phase II scope of work, which included conclusions and recommendations.

Underground Storage Tank (UST) Remediation, Various Locations, City of Newark, NJ, Project Manager. Managed the preparation and implementation of the RI at 16 active city facilities. Remedial investigation (RI) activities included supervision of field work, which included advancement of soil borings, advancement of temporary wells, collection of soil and water samples, and installation and sampling of monitoring wells. It also included the preparation of the RI Work Plan and report, and the Remedial Action Selection Report and Work Plan. Responsibilities also included coordination with facility operators and correspondence with New Jersey Department of Environmental Protection (NJDEP).

Removal of 14 Underground Storage tanks (USTs) at 8 Facilities, City of Newark, NJ, Assistant Project Manager. Assisted city in obtaining funding through loans financed through New Jersey Economic Development Authority (NJEDA) to conduct removal and remediation activities. Performed oversight of removal activities at active city facilities, which included the preparation of bid documents, coordination of the public bid with the city, field oversight of removal contractor, approval of work prior to contractor payment release, and review of closure reports prior to submittal to New Jersey Department of Environmental Protection (NJDEP).

Environmental Consulting Services, New York, NY, New York City School Construction Authority (NYCSCA), Assistant Project Manager. Assisted NYCSCA in reviewing and presenting comments on Phase I Environmental Site Assessments (ESAs), Phase II expanded site investigations (ESIs), and remediation specifications.

Phase I Environmental Site Assessments (ESAs), Brooklyn, Queens, and the Bronx, NY, NYCSCA, Project Engineer. Directed Phase I site assessments following New York State Department of Environmental Conservation (NYSDEC) protocols and American Society for Testing and Materials (ASTM) standards in Brooklyn, Queens, and the Bronx, NY. Project components entailed compilation, review, and interpretation of regulatory database files, aerial photographs, fire insurance maps, and municipal office information. Project also consisted of preparing Phase II scope of work, which included conclusions and recommendations.

Asbestos Inspections at Newark Airport, Newark, NJ, Continental Airlines, Project Engineer. While employed by URS, conducted asbestos inspections at various locations throughout the airport following EPA protocol.

Environmental Compliance Management Program, City of Newark, NJ, Staff Engineer. Served as Team Leader for the environmental compliance program for over 60 public buildings in the City of Newark. Responsibilities included environmental compliance and asbestos surveys for each building and compiling and reviewing all audit data. Prepared a final report, which included a database review for each site and findings, recommendations, and conclusions.

Boylan Street Recreation Center RI, City of Newark, NJ, Project Engineer/Assistant Project Manager. RI activities included preparation of an NJDEP-approved Work Plan, supervision of field work, which included advancement of eight soil borings, and installation and sampling of four monitoring wells. Prepared an RI Report, which included findings, recommendations, and

Key Projects (Continued)

conclusions for a remedial action. Project evolved into performing additional confirmation sampling, which included soil vapor sampling, additional monitoring well installation, and preparation of a RAWP for soil and groundwater.

Qualifications Summary

- Over 9 years of professional experience.
- Soil and sediment sampling; porewater sampling; groundwater sampling; vapor intrusion sampling and surface-water sampling.
- Preparation of Stormwater Pollution Prevention Plans (SWPPPs), Remedial Investigation Workplan (RIWP), Remedial Investigation Reports (RIR), Remedial Action Workplan (RAWP), Phase I and Phase II Assessment reports, Receptor Evaluations, Ecological Evaluations.
- Data analysis management and cross-section diagrams from analytical data and lithology.
- Creation and delivery of educational and training presentations.
- Wetland delineation certified.
- Stream assessments and rapid bioassessment sampling.
- Phytoplankton and macroinvertebrate identification and enumeration.

DYNA L. KRUMICH-OGONOWSKI

Education

B.S., Marine Science - Richard Stockton College (2002)

Training

40-Hour Hazardous Materials Incident Response Operations, OSHA 29 CFR 1910.120(e)(3)(i), EPA (2007)
8-Hour Hazardous Waste Refresher Course, OSHA 29 CFR 1910.120(e)(8), WESTON (2008), WESTON (2013)
30-Hour Construction Safety and Health Training Course, WESTON (2010)
Wetland Delineation Certification, Rutgers University (2008)
Stream Assessment Training, Stroud Water Research Center (2006)
Advanced Chironomidae Identification Workshop, New Jersey Water Monitoring Coordinating Council (2006)
CPR and First Aid Certified (2013)
Underground Utilities Competent Person, WESTON (2009)
Fall Protection Competent Person Initial, WESTON (2010)
Excavation and Trenching Competent Person, WESTON (2010)
Boating Safety, U.S. Coast Guard Auxiliary (2009)
2013 GHS Hazard Communication Training, WESTON (2013)

Employment History

2007 to Present WESTON [8-07 to Present]
2005 to 2007 Monmouth County Health Dept [3-05 to 8-07]
2004 to 2005 The Goddard School [7-04 to 3-05]

Key Projects

Remedial Investigation, Spill Response, Multiple Locations, Confidential Client, Task Manager. Prepared scope of work (SOW), letter reports and remedial investigation reports as well as provided oversight on remedial investigation activities such as soil sampling, sub-slab soil gas port installation, vapor intrusion sampling, post-excavation sampling, and receptor evaluation. Managed vapor intrusion investigation and spill response. Coordinated contractors for spill response, sample analyses, drilling and waste disposal. Coordinated team members for investigation activities. Coordinated with client and provided daily updates on project status. Provided remedial investigation and remedial action design for the remediation of a former waste oil underground storage tank in compliance with the NJ Technical Requirements for Site Remediation. Prepared proposal

Key Projects (Continued)

cost estimate and procurement documents. Responsible for ensuring contractor adherence to the bid specifications, ensuring site environmental and regulatory compliance, and coordinating with the property owners. Also acted as on-site representative for client during spill response activities, reviewed all invoices and submittals prior to final acceptance/approval. [8-12 to Present; WESTON]

Remedial Investigation (RI) and Ecological Risk Assessment, Edison, NJ, U.S. Army Corps of Engineers (USACE), Task Manager. Assisted in directing the focused RI of the former Raritan Arsenal, a 3,200-acre site. Work was conducted under the CERCLA and NJDEP technical requirements for site remediation. Lead all tasks for the report for the Phase II remedial investigation/feasibility study (RI/FS) for the site. As Task Manager for the RI, lead investigations including delineation of 8 groundwater plumes, supplemental soil RIs, execution of remedial action work plans (RAWPs) for contaminated soil removal activities, investigation of vapor intrusion impacts to site buildings from contaminated groundwater. Responsible for preparation for field activities such as equipment, supplies, forms, and labels, and organization of team members and subcontractors. Responsible for sample management. Field activities include installation of monitoring wells; sampling of sediment, groundwater, soil, surface water, indoor air, soil gas and waste classification sampling. Supervised drillers performing installation of monitoring wells and abandonment of wells. Responsible for recording the lithology of the soils. Prepared Request for Proposals and purchase requisitions for several subcontractors. Recorded all site activities. Conducted GPS data collection for the sample locations. Held daily health and safety briefings for the teams. Held biweekly status conference calls with the client and prepared monthly Progress reports for the client. Assisted with negotiations of new contracts. Prepared Sitewide Supplemental RIR, Annual Indoor Air Quality reports, Feasibility Study, Bi-annual CEA reports, Groundwater Compliance Monitoring reports. [7-09 to Present; WESTON]

Evaluation of Mercury-Contaminated Sediment, Surface- and Groundwater, Field Lead, Atlantic City Naval Air Station (NAS) Formerly Used Defense Site (FUDS), Atlantic City, NJ, U.S. Army Corps of Engineers (USACE), New York and New England Districts. Responsible for preparation for field activities such as equipment, supplies, forms, and labels, and organization of team members and subcontractors. Responsible for sample management, which included a trace metals sampling method called “clean and ultraclean” (U.S. Environmental Protection Agency [EPA] 1666). Field activities include installation of shallow temporary wells; sampling of sediment, groundwater, porewater, and surface water; and waste classification sampling. Supervised drillers performing installation of 24-foot temporary wells. Assisted in the lithology of the soils. Created cross-section diagrams based on analytical results and lithology for groundwater phases. Prepared purchase requisitions for waste class sampling and disposal. Recorded all site activities. Conducted GPS data collection for the sample locations. Held daily health and safety briefings for the teams. Prepared a Sitewide Supplemental RIR. [5-08 to Present; WESTON]

Supplemental Groundwater Investigation, Field Lead, Wall, NJ, Confidential Client. Responsible for preparation for field activities such as equipment, supplies, forms, and labels, and organization of subcontractors. Supervised well installation using mud rotary equipment. Described the lithology of the soil. Collected and managed soil and sediment samples. Supervised Membrane Interface Probe (MIP) sampling. Prepared purchase requisitions for

Key Projects (Continued)

subcontractors. Held daily health and safety briefings for the team. Conducted global positioning system (GPS) data collection for the test hole locations. [9-08 to 12-11; WESTON]

Post-Closure Monitoring and Maintenance, Oceanside Landfill, Field Lead, Hempstead, NY, Town of Hempstead Department of Sanitation. Conducted semiannual surface-water and groundwater sample monitoring events. Responsible for preparation of the field effort as well as all sampling, sample management, and data recording. Held daily health and safety briefings for the team. [4-08 to 10-11; WESTON]

Active Remediation of Xylene-Contaminated Soils, Project Scientist, Newark, NJ, Confidential Client. Responsible for air monitoring for the project. Calibrated the equipment. Collected samples using SKC pumps for volatile organic compounds (VOCs) and metals. Recorded, monitored, and downloaded the data from the DataRams. Oversight oflaboratory technicians for groundwater sampling. Prepared the Baseline Ecological Evaluation. [10-08 to 11-10; WESTON]

Preparation of Stormwater Pollution Prevention Plan (SWPPP), Project Scientist, Edison, NJ, Port Authority of New York and New Jersey. Organized interview and questionnaire information into tables to assess the potential nonpoint source pollutants occurring due to site-related activities. Described site-specific activities and the potential damage that could occur due to the stormwater runoff. Recommended best management practices (BMPs) and implementation program. [12-08 to 2-09; WESTON]

Fish Passage Feasibility Study (FS), Field Lead, Rahway River Water Supply Dam, Rahway, NJ, New England Interstate Water Pollution Control Commission (NEIWPCC). Conducted oversight of subcontractors performing subsurface utility locating by air/vacuum test hole service. Held daily health and safety briefings for the team. Conducted GPS data collection for the test hole locations. Assisted with a fisheries survey using the electroshock fishing method, then identified, weighed, and released fish. Recorded site activities. [5-08 to 12-08; WESTON]

Groundwater Monitoring, Light Nonaqueous Phase Liquid (LNAPL) Remediation Pilot Study, Fords, NJ, Hatco, Assistant Project Scientist. Responsible for preparation for the field effort as well as sample management and data recording. Prepared the purchase requisition for the subcontracted laboratory. [7-08 to 11-08; WESTON]

Gibbsboro Remedial Investigation (RI), Gibbsboro, NJ, Confidential Client, Assistant Project Scientist. Performed sample collection and sample management for soil and sediment samples. Collected porewater samples. Assisted with recording field notes on a tablet PC using FieldFastSM. Used the X-ray fluorescence (XRF) unit to assist with pre-analysis efforts of sample and required depths. Conducted GPS data collection for the sample locations. [9-07 to 3-08; WESTON]

Cooperative Coastal Monitoring Program (CCMP), Freehold, NJ, New Jersey Department of Environmental Protection (NJDEP), Monmouth County Health Department (MCHD), Laboratory Technician. Performed microbiological testing on samples from 61 sites on a weekly basis during May through August for Enterococcus bacteria. Analyzed samples after subsequent incubation. Recorded data into MCHD inventory systems and NJDEP Earth911 website. Recreational bathing standards, in regard to public health, were upheld during the CCMP sampling season. [5-05 to 8-07; Monmouth County Health Department]

Key Projects (Continued)

Ambient Surface-Water Monitoring Program, Freehold, NJ, MCHD, Laboratory Technician. Assisted in restructuring the program. Coordinated meetings for the MCHD laboratory and water team. Assisted in the analysis selection for each surface-water classification. Performed the methods required for the analysis of total suspended solids (TSS), fecal coliforms, ammonia, and total phosphorus. Managed the data in the ambient monitoring database. [9-06 to 5-07; Monmouth County Health Department]

Pilot Project for the Comparison of Quantitative Polymerase Chain Reaction (qPCR) and EPA Method 1600, Freehold, NJ, MCHD, Laboratory Technician. Assisted in comparing an Enterococcus water quality test for recreational bathing using qPCR side-by-side test to evaluate the correlation between the qPCR method and the membrane filtration method (EPA Method 1600). Performed the filtration method on the site-specific samples followed by freezing the qPCR samples, which were then transported to the EPA laboratory for the Cepheid Smart Cycler procedure. Organized the intake of the samples, the chain-of-custodies for the project, and storage of the samples. [10-06 to 5-07; Monmouth County Health Department]

Publications

Costa, R. E., Krumich, D., Brown R. H. 2009. "AN ITERATIVE SHALLOW HYDROGEOLOGIC INVESTIGATION OF ELEVATED MERCURY CONCENTRATIONS, ATLANTIC COUNTY, NEW JERSEY". New Jersey Coastal Plain Stratigraphy and Coastal Process. 26th Annual Field Conference, Geological Association of New Jersey. September 2009.

Krumich, D. 2006. “Commonly Encountered Summer Nuisance Conditions – Macroalgae, Phytoplankton, Red Tide and Foam/Spindrift.” Presented at Public Recreational Bathing Seminar for Environmental Health Specialists, Monmouth County Health Department, Freehold, NJ, March 2006.

Krumich, D., and E. Cosgrove. 2005. “The Monmouth County Health Department’s Environmental Laboratory and its Involvement with Public Health and Awareness Concerning the Environment as well as an Outline of Algae, Macroinvertebrates, and the Cooperative Coastal Monitoring Program,” Presented to four environmental science classes and six biology classes (110 students) at Freehold Township High School, Freehold, NJ. October 2005.

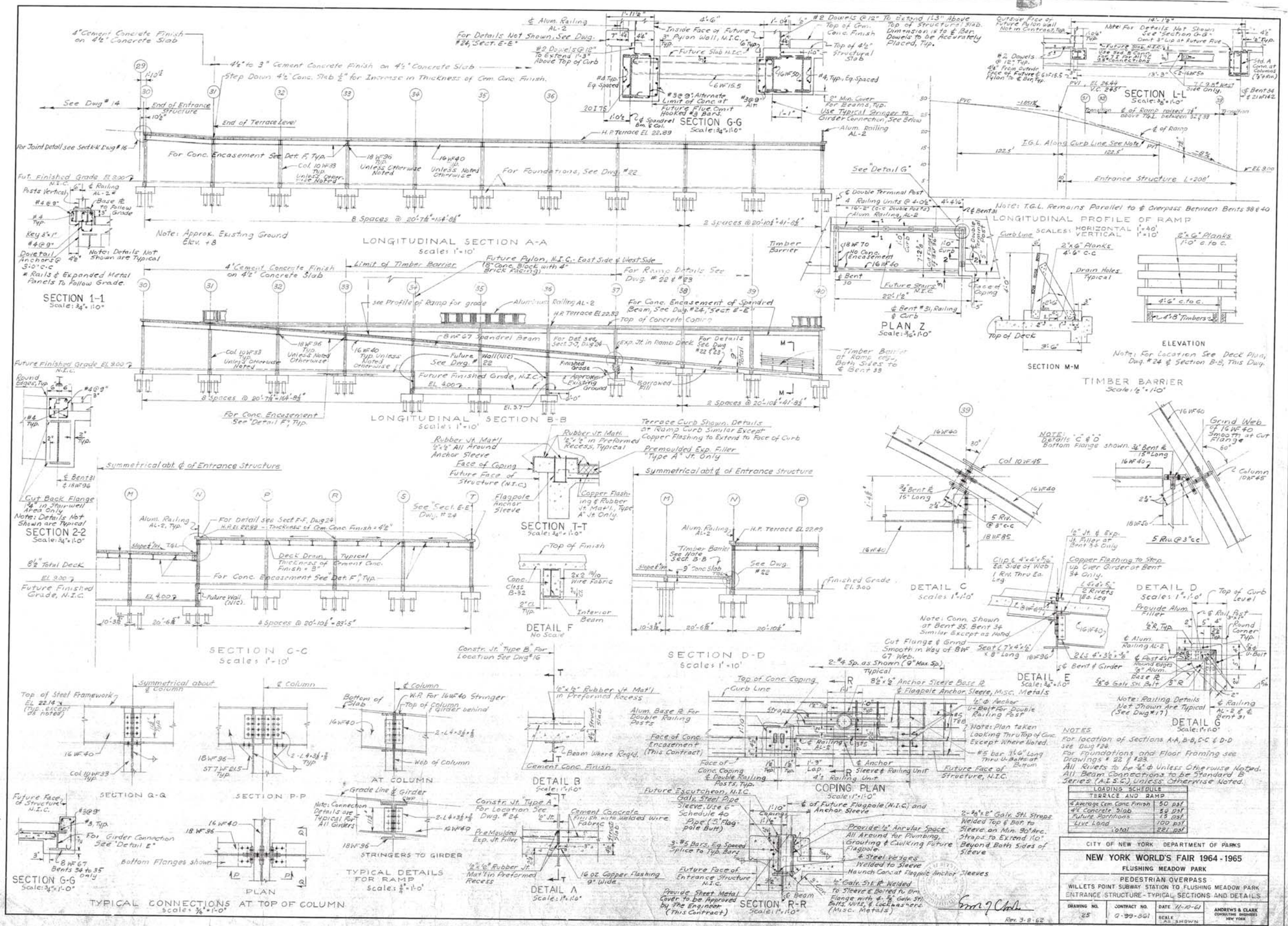
APPENDIX E

STATEMENT OF LIMITATIONS

STATEMENT OF LIMITATIONS

This Phase I Corridor Assessment Report (CAR) was limited to the review of commercial regulatory database report and Sanborn Fire Insurance Maps provided by the client, and an on-site inspection. The site inspection was limited to observation of exterior surficial conditions only from public right of way only. Such an inspection cannot be expected to reveal all hazardous materials or situations that might be present on-site; some hazardous materials or conditions may exist and not be detected because they are beyond the scope of this study. The investigation was conducted in a manner consistent with that level of care and skill exercised by environmental professionals currently practicing under similar conditions and was based on information made available to the representatives of Weston Solutions of New York, Inc. All documents prepared or furnished by Weston Solutions of New York, Inc. pursuant to this project are to be used in the context of the scope of services contracted. This document is not intended or represented to be suitable for reuse by the client or others on modifications of the project scope. This Phase I CAR has been prepared for the use of the client and agents thereof.

Appendix H: Record Drawings



NOTES

For location of Sections A-A, B-B, C-C & D-D see Dwg #28.
For Foundations and Floor Framing see Drawings # 22 & #23.
All Rivets to be 3/4" ϕ Unless Otherwise Noted.
All Beam Connections to be Standard B Series (A.I.S.C.) Unless Otherwise Noted.

LOADING SCHEDULE TERRACE AND RAMP	
¢ Average Conc. Conc. Finish	150 psf
4½" Concrete Slab	56 psf
Future Partitions	15 psf
Live Load	100 psf
Total	321 psf

CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964 - 1965

FLUSHING MEADOW PARK

PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
ENTRANCE STRUCTURE - TYPICAL SECTIONS AND DETAILS

DRAWING NO.	CONTRACT NO.	DATE	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
25	0-99-861	11-10-61	
		SCALE AS SHOWN	

Rev. 3-8-62

In Charge of: E. J. Clark
Made by: Richard Pedone
Traced by:
Checked by: Chas. Nitz

NOTE:
All existing cables are to be kept in continuous operation during reconstruction and permanently fastened to the new structure.

Ramp Down to Main Level
℄ Existing 27 KV Line (Exercise Care in This Area)
Exist. 120-Pair Paper Insulated Lead covered #19 AWG Telephone Cable on Messenger Wire under superstructures.
1 Story Cinder Block Garage
Temporary Tel. & Tel. & Sig. Lines in wood trunking along W. Bd. Track
Existing T&T. Poles
Existing 100 PR P.I.L.C. #19 AWG Tel. Cable on Messenger Wire to M3A Satellite in Bus Garage
℄ Proposed N.Y. City Transit Authority Storage Tracks
W.P.1 S 30369.579 E 37944.083
S 30418.737 E 38011.834
℄ Pole Line
℄ N.B. Track (Existing) Brg. N 36°-11'-19.9" E
℄ E.B. Track Relocated (Not in Contract)
℄ High Tension Line (Existing)
℄ L.I.R.R. R.O.W. Line
L.I.R.R. Platform
L.I.R.R. Platform (Not in Contract)
Future Chain Link Fence N.I.C.
℄ Bus Road "A" N 35°-57'-48.1" E
Mall
℄ Bus Road "A" S 35°-57'-48.1" W
S 30067.000 E 38359.000
S 30493.342 E 38114.659 W.P.3
Future Stairway Not in Contract
Limit of Terrace Level, El. 22.89
Future Slab, N.I.C. (Typical)
℄ Existing Overpass
Exist. Tel. Ten. Box on Mezzanine
Upper Level
Main Level
℄ Existing Overpass
℄ Existing Overpass Brg. S 67°-46'-36.6" E
East Face of Structure
West Face of Structure
40'-0"
℄ Roosevelt Ave.
Existing I.R.T. Station Columns
Ramp Up From Lower Level
Begin Timber Deck
Willets Point I.R.T. Subway Station
Demolish Existing Circuit Breaker House. Payment Item, "Demolish Existing Superstructure."
Brick Building
1 Story Cinder Block Building
Exist. 20 PR P.I.L.C. #19 AWG Tel. Cable on Messenger Wire to Ten. Box in Inspector's Shed.
Wire Mesh Protective Fence & Alum. Railing AL-1
W.P.2 S 30339.492 E 37817.816
Future Roof Not in Contract
℄ Bent 19 A
A.E.S. Single 27 KV Submarine Cables
Future Roof, Not in Contract
Face of Entrance Structure
℄ Entrance Structure Brg. S 54°-02'-11.9" E
Future Pylon, N.I.C.
Curb Line, Typ.
Future Stairway, N.I.C.
Face of Conc. Coping, Typ.
Dimensions Face to Face of Structure
Limit of Work for This Contract
FAIR ENTRANCE
Terrace Level
Ramp
Demolish Existing Superstructure and Replace as Shown in The Plans. See Sections A-A & B-B
End Timber Deck & Structure on Existing Pile Foundations
Begin Concrete Deck & Structure on New Pile Foundations
Building Line M-H
Building Line R-32
℄ New L.I.R.R. Tracks (Not in Contract) Brg. N 36°-11'-19.9" E
High Tension Tower to be Relocated (See Specifications)
Note: ---- Denotes Timber Barrier

PLAN
SCALE: 1"=40'

Electric Service Pedestal N.I.C.
New 6' High Chain Link Fence
Drainage N.I.C.
New Asphalt Conc. Pavement on Existing Conc. Pavement
Face of Structure Above
Limit of Area to be Graded
Face of Structure Above

GRADING BELOW OVERPASS
SCALE: 1"=40'

Level El. 33.00
P.V.I. El. 33.00
1.85%
Begin Wire Mesh Protective Fence East Face & West Face
Top of Alum. Railing AL-1 & Wire Mesh Protective Fence
Future Roof, Not in Contract
P.V.I. El. = 26.44
V.C. = 2.45'
Future Pylon, N.I.C.
Terrace Level El. 22.89 (H.P.)
P.V.I. El. 9.00
Flagpole, N.I.C., Typ.
Top of Alum. Railing AL-2
Future Stairway Not in Contract
Existing Partitions to be Removed by N.Y.C.T.A., N.I.C.
Top of Aluminum Railing AL-1, Typical
Alum. Chain Link Fence, 2" Mesh See Dwg. #10
Min. V.C. = 19'-8"
Min. V.C. = 17'-0"
Min. V.C. = 12'-0"
℄ Track
Limit of Wire Mesh East Face & West Face
Future Parapet Not In Contract Timber Barrier This Contract
Entrance Structure For Extent of Work to be Included in This Contract See Drawing Nos. 22 to 25
Note: Lampposts Not Shown

ELEVATION
SCALE: 1"=40'

15'-0" 17'-6" 46'-6" 17'-6"
℄ Existing Overpass
Face of Structure
Upper Level Ramp
Upper Level
Aluminum Railing, AL-1, Typ.
Face of Structure
Timber Deck - 3"x6" Planks
Lower Level Ramp
Main Level
Existing Ducts to Remain in Service & Relocate on new Structure, Item 19
New Pile Cap Existing Piles to Remain
15'-0" 16'-0" 24'-0" 24'-0" 16'-0"
℄ Column, Typ.
New Pile Cap Existing Piles to Remain
Existing Pile Foundation To Remain
Existing Steel Superstructure Above Top of Column Base Pl's to be Removed
See Dwg. #16, Typ.
N L K J

SECTION A-A
SCALE: 1"=10'

40'-0"
℄ Existing Overpass
1/16" Typ. 1/16" Typ.
12'-0" 12'-0"
℄ Column, Typ.
Existing Pile Foundation To Remain Except at Bent #18
Existing Steel Superstructure Above Top of Column Base Pl's to be Removed.
L SECTION B-B K

SCALE: 1"=10'

S 30187.954 E 38243.861
S 30418.737 E 38011.834
#8
S 30785.737 E 39111.307
1058.474' S 55°-25'-41.2" E
#9
℄ Overpass
Survey Base Line, Typical
℄ Entrance Structure S 54°-02'-11.9" E
1172.172' S 11°-57'-39.9" E
S 31276.380 E 38472.457
#24
1473.206' S 37°-28'-07.6" W
S 31368.518 E 37349.368
#7
S 30444.788 E 38047.650

LOCATION PLAN
SCALE: 1"=400'

Future Roof, Not in Contract
Alum. Railing, AL-1 & Wire Mesh Protective Fence
Alum. Railing AL-1 Around Stairwell, Typ.
Top of L.I.R.R. Platform
Top of Rail
Stairs, Platforms and Tracks To be Built by L.I.R.R. (N.I.C.)

SECTION C-C
SCALE: 1"=10'

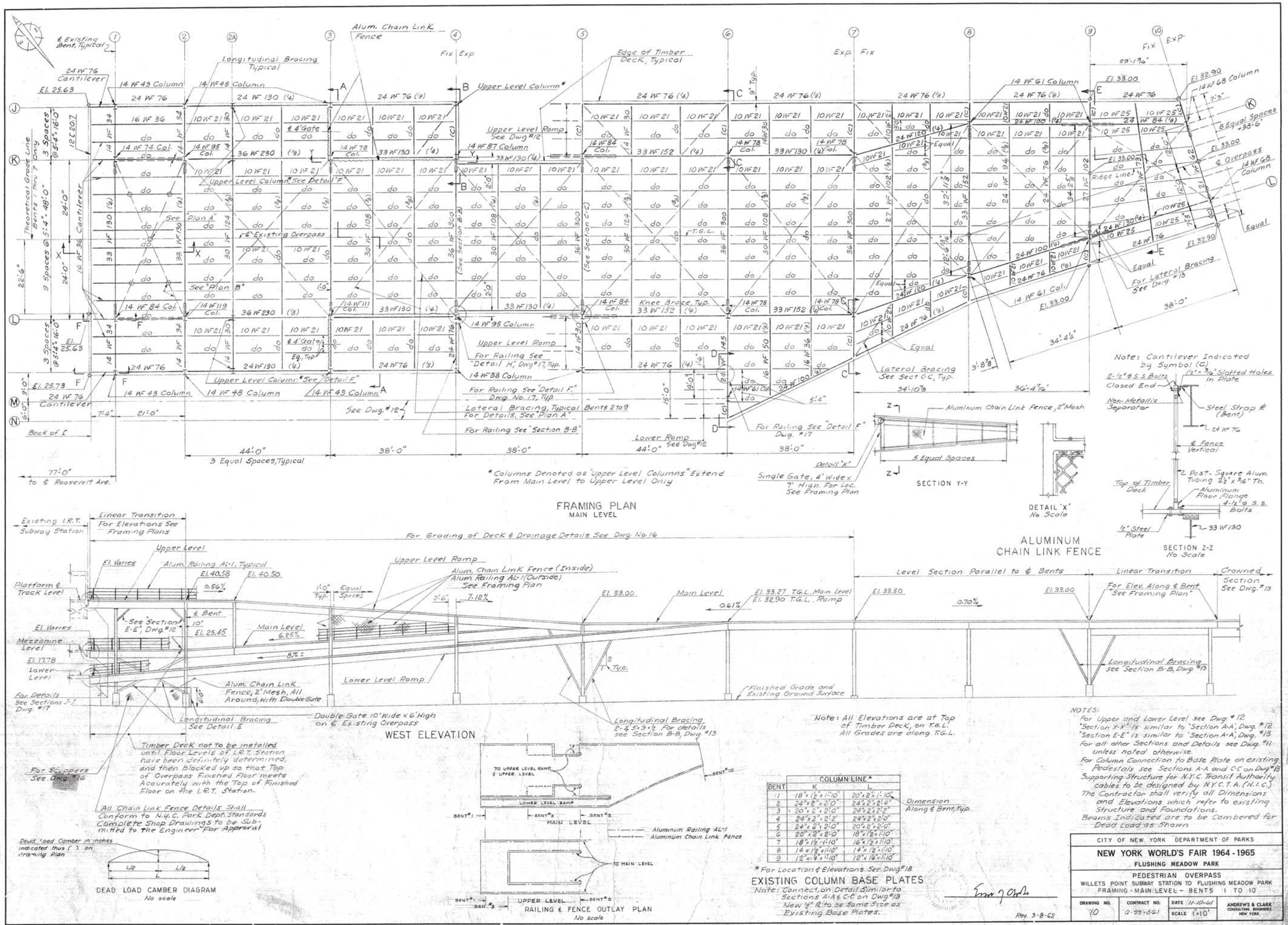
℄ Entrance Structure
Reinforced Concrete Slab, Typ. 5½" Thickness

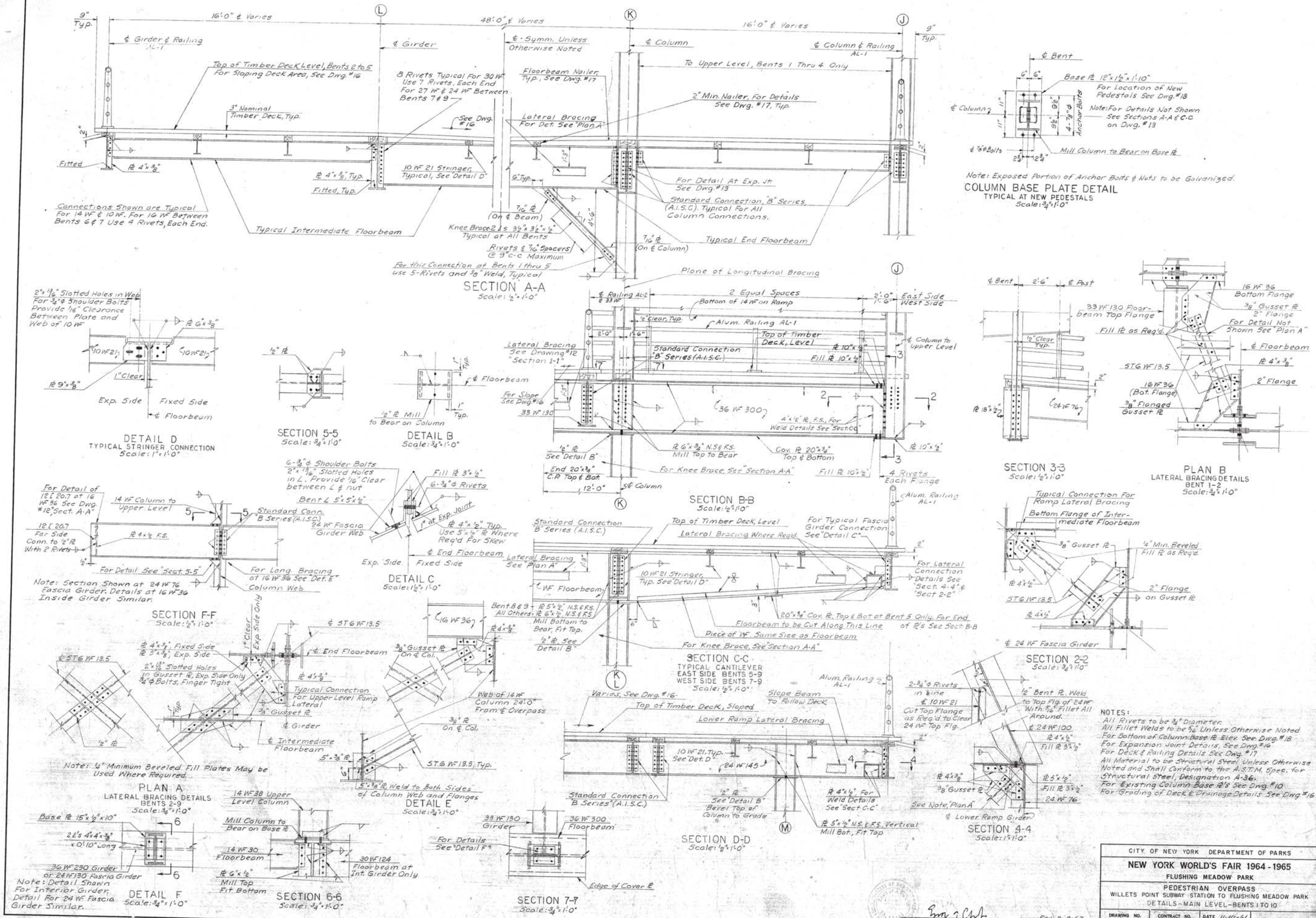
NOTES:
All Coordinates Refer to The World's Fair Coordinate System.
All Elevations Refer to The Borough of Queens Datum Which is 2.725' Above The U.S. Coast and Geodetic Survey Datum at Mean Sea Level at Sandy Hook.
Contractor to Verify All Dimensions and Elevations which Refer to Existing Structure and Foundations.
Live Load = 100 psf
For Railing and Deck Details See Dwg. #17
For Location & Type of Foundations See Dwg. #18
Centerlines of Bents 1 thru 18 are Located on The Bent Centerlines of the Existing Overpass.
For Lighting See Dwg. #26
For Fire Protection System See Dwg. #27
Profile Shown is Approx. For Exact Profile See Dwg. Nos. 10, 14 & 25

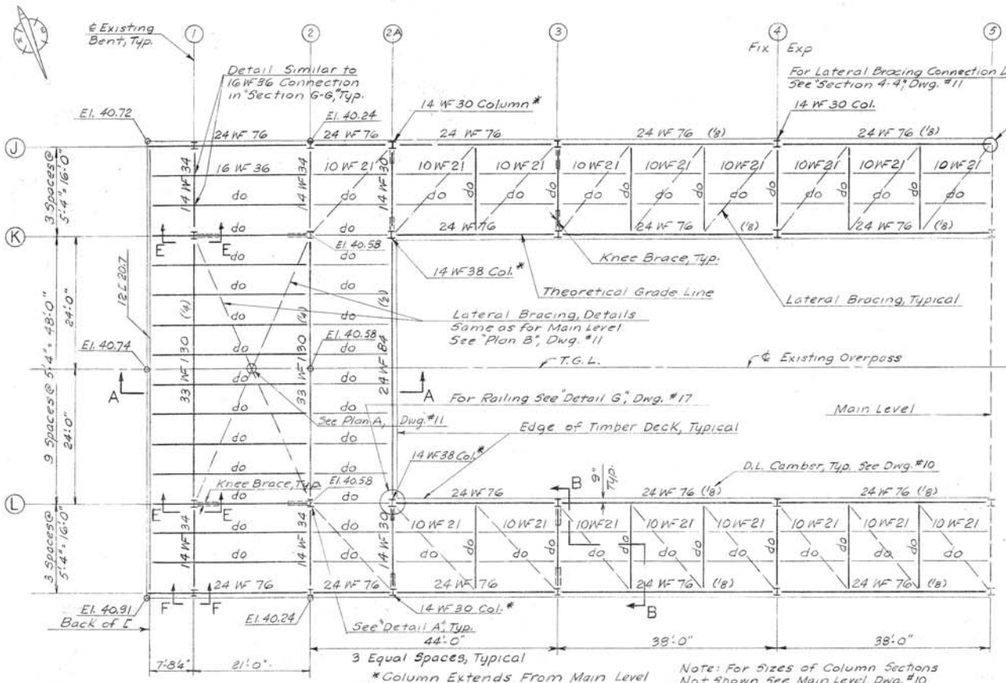
CITY OF NEW YORK DEPARTMENT OF PARKS			
NEW YORK WORLD'S FAIR 1964-1965			
FLUSHING MEADOW PARK			
PEDESTRIAN OVERPASS WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK PLAN, ELEVATION AND SECTIONS			
DRAWING NO. 9	CONTRACT NO. Q-28-3G1	DATE 11-20-61 SCALE As Shown	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK

Rev. 3-15-62

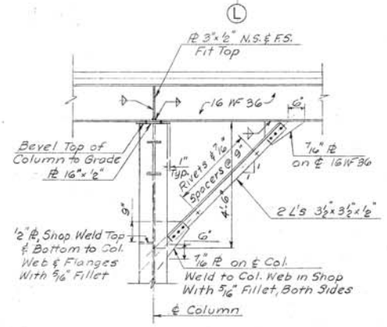
In Charge of C.
Made by
Traced by
Checked by


FRAMING PLAN
MAIN LEVEL
WEST ELEVATION
RAILING & FENCE OUTLAY PLAN
No scale
DEAD LOAD CAMBER DIAGRAM
No scale
SECTION Y-Y
DETAIL 'X'
No Scale
ALUMINUM
CHAIN LINK FENCE
SECTION Z-Z
No Scale
EXISTING COLUMN BASE PLATES
Note: All Elevations are at Top of Timber Deck, on T.G.L. All Grades are along T.G.L.
NOTES:
For Upper and Lower Level see Dwg. #12
"Section X-X" is similar to "Section A-A", Dwg. #12
"Section E-E" is similar to "Section A-A", Dwg. #13
For all other Sections and Details see Dwg. #11 unless noted otherwise.
For Column Connection to Base Plate on existing Pedestals see Sections A-A and C-C on Dwg. #13
Supporting Structure for N.Y.C. Transit Authority cables to be designed by N.Y.C.T.A. (N.I.C.)
The Contractor shall verify all Dimensions and Elevations which refer to existing Structure and Foundations.
Beams Indicated are to be Cambered for Dead Load as Shown
CITY OF NEW YORK DEPARTMENT OF PARKS
NEW YORK WORLD'S FAIR 1964-1965
FLUSHING MEADOW PARK
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
FRAMING - MAIN LEVEL - BENTS 1 TO 10
DRAWING NO. 10
CONTRACT NO. Q-29-861
DATE 11-10-61
SCALE 1"=10'
ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
Rev. 3-8-62
In Charge of: L.J. Bach
Made by: K.S. Udelman
Traced by: M. Meador
Checked by: M. Meador

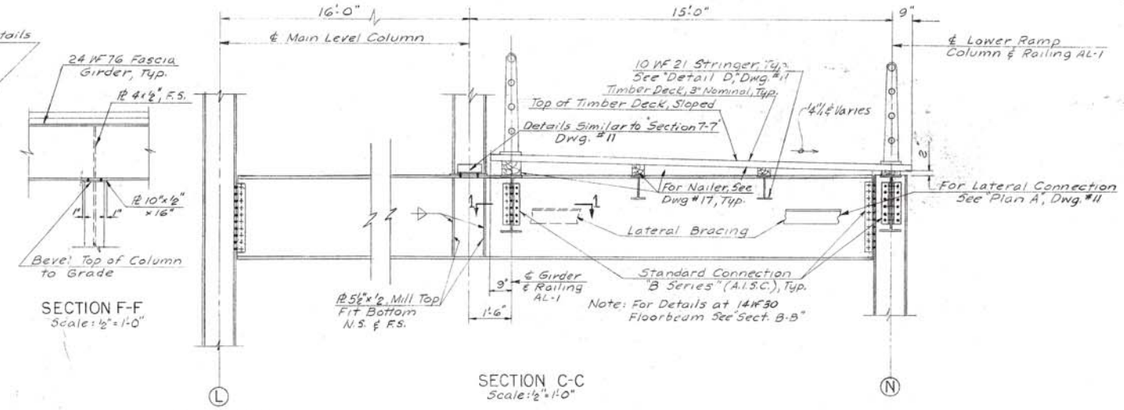

SECTION A-A
Scale: 1/2"=1'-0"
DETAIL D
TYPICAL STRINGER CONNECTION
Scale: 1"=1'-0"
SECTION 5-5
Scale: 3/4"=1'-0"
DETAIL B
Scale: 3/4"=1'-0"
SECTION B-B
Scale: 1/2"=1'-0"
COLUMN BASE PLATE DETAIL
TYPICAL AT NEW PEDESTALS
Scale: 3/4"=1'-0"
Note: Exposed Portion of Anchor Bolts & Nuts to be Galvanized.
SECTION 3-3
Scale: 1/2"=1'-0"
PLAN B
LATERAL BRACING DETAILS
BENT 1-2
Scale: 3/4"=1'-0"
DETAIL C
Scale: 1 1/2"=1'-0"
SECTION F-F
Scale: 1/2"=1'-0"
SECTION C-C
TYPICAL CANTILEVER
EAST SIDE BENTS 5-9
WEST SIDE BENTS 7-9
Scale: 1/2"=1'-0"
SECTION 2-2
Scale: 3/4"=1'-0"
PLAN A
LATERAL BRACING DETAILS
BENTS 2-9
Scale: 3/4"=1'-0"
DETAIL E
Scale: 3/4"=1'-0"
SECTION D-D
Scale: 1/2"=1'-0"
SECTION 4-4
Scale: 1 1/2"=1'-0"
DETAIL F
Scale: 3/4"=1'-0"
SECTION 6-6
Scale: 3/4"=1'-0"
SECTION 7-7
Scale: 3/4"=1'-0"
NOTES:
All Rivets to be 3/4" Diameter.
All Fillet Welds to be 5/16" Unless Otherwise Noted
For Bottom of Column Base PL Elev. See Dwg. #18
For Expansion Joint Details, See Dwg. #19
For Deck & Railing Details See Dwg. #17
All Material to be Structural Steel Unless Otherwise Noted and Shall Conform to the A.S.T.M. Spec. for Structural Steel, Designation A-36.
For Existing Column Base PL's See Dwg. #10
For Grading of Deck & Drainage Details See Dwg. #16
CITY OF NEW YORK DEPARTMENT OF PARKS
NEW YORK WORLD'S FAIR 1964-1965
FLUSHING MEADOW PARK
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
DETAILS - MAIN LEVEL - BENTS 1 TO 10
DRAWING NO. 11
CONTRACT NO. 0-99-861
DATE 11-10-61
SCALE AS SHOWN
ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
Rev. 8-8-62



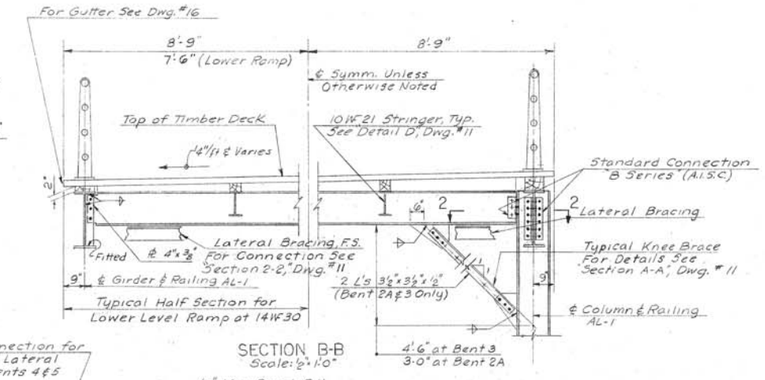
FRAMING PLAN
UPPER LEVEL & RAMPS
Scale: 1"=10'



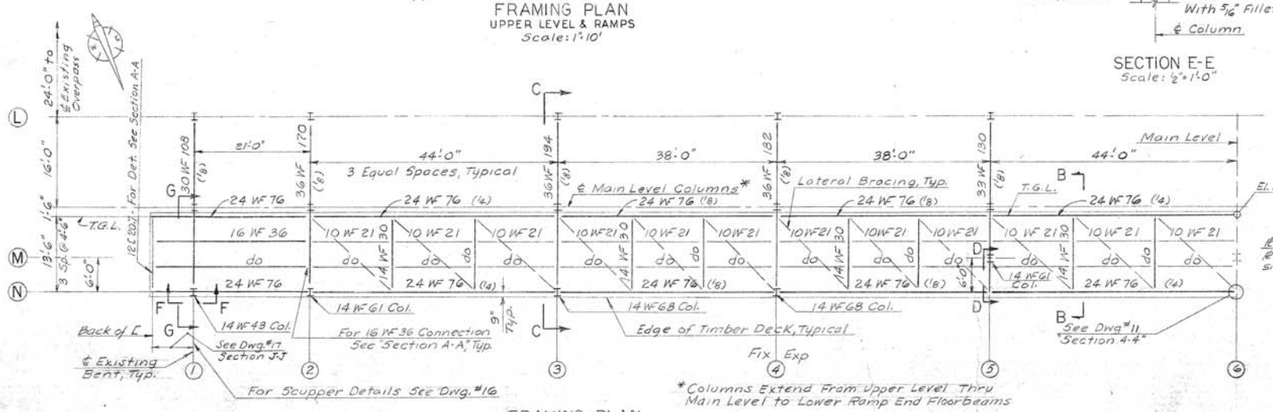
SECTION E-E
Scale: ½"=1'-0"



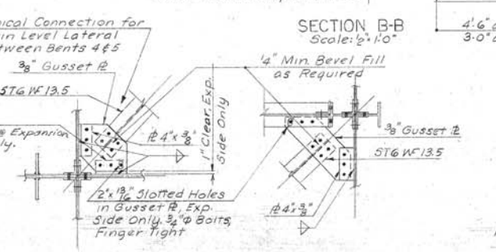
SECTION C-C
Scale: ½"=1'-0"



SECTION B-B
Scale: ½"=1'-0"

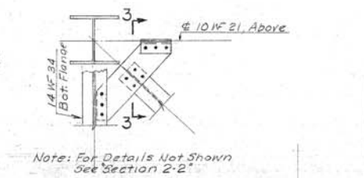


FRAMING PLAN
LOWER LEVEL RAMP
Scale: 1"=10'

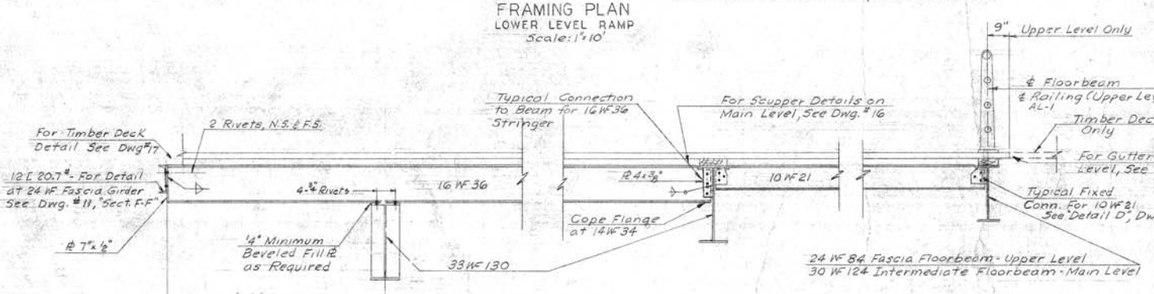


SECTION 1-1
Scale: ¾"=1'-0"

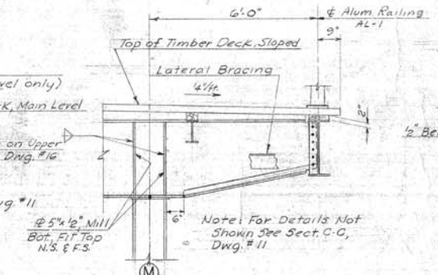
SECTION 2-2
Scale: ¾"=1'-0"



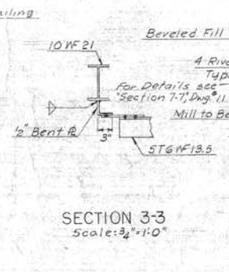
DETAIL A
Scale: ¾"=1'-0"



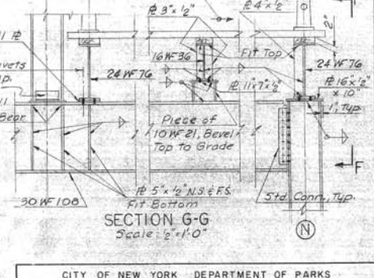
SECTION A-A
Scale: ½"=1'-0"



SECTION D-D
Scale: ½"=1'-0"



SECTION 3-3
Scale: ¾"=1'-0"



SECTION G-G
Scale: ½"=1'-0"

NOTES:
For Notes See Drawing No. 11

Rev. 9-6-67

CITY OF NEW YORK DEPARTMENT OF PARKS

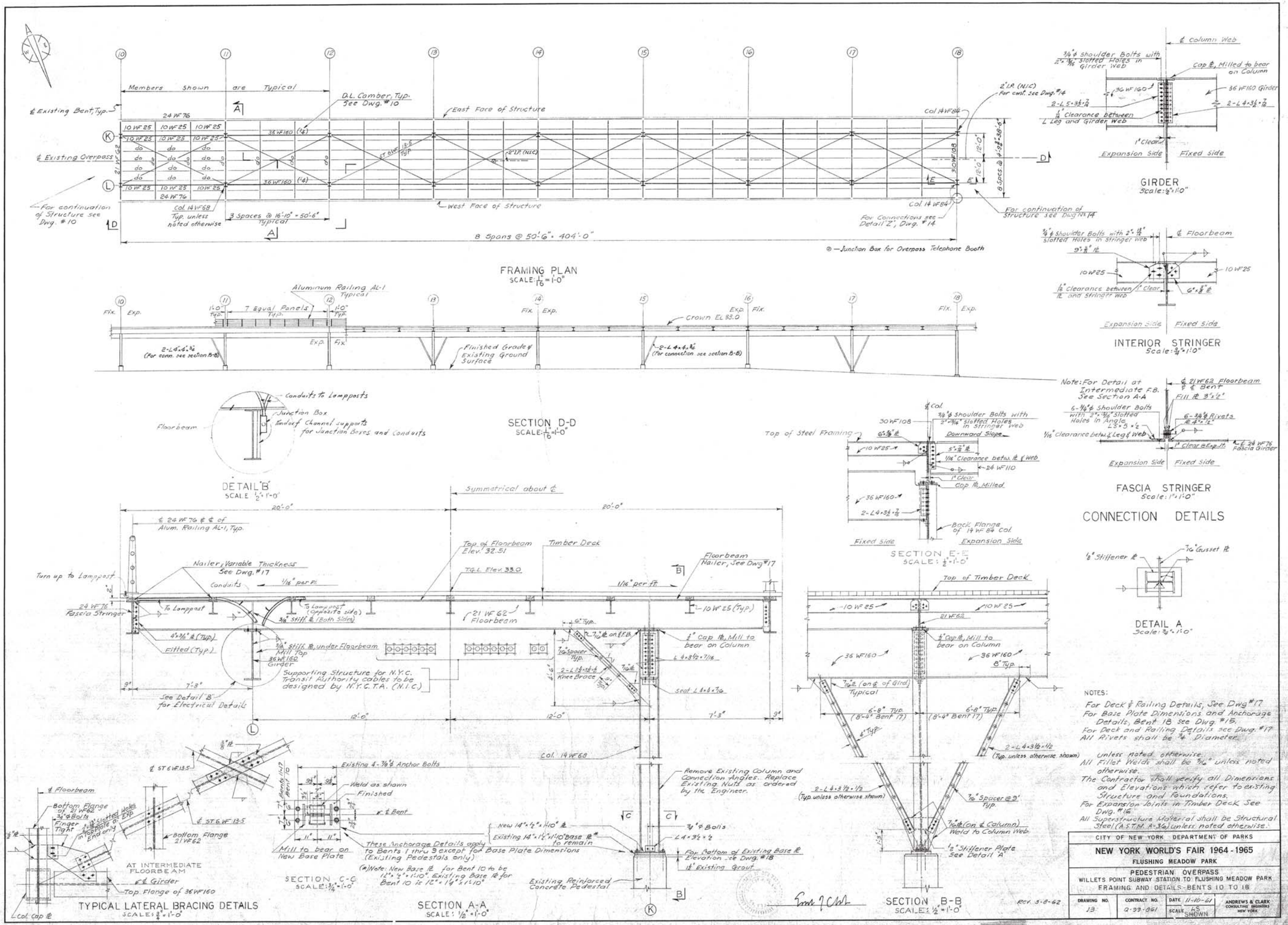
NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
FRAMING AND DETAILS - RAMPS - BENTS 1 TO 6

DRAWING NO.	CONTRACT NO.	DATE 11-10-61	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
12	2-99-861	SCALE AS SHOWN	

In Charge of: [illegible]
Made by: R. S. Martelli
Traced by:
Checked by: W. Blasco


FRAMING PLAN
SCALE: 1/16" = 1'-0"
Members shown are Typical
D.L. Camber, Typ. See Dwg. #10
East Face of Structure
West Face of Structure
℄ Existing Bent, Typ.
℄ Existing Overpass
For continuation of Structure see Dwg. #10
Col. 14 WF 68 Typ. unless noted otherwise
3 Spaces @ 16'-10" = 50'-6" Typical
8 Spans @ 50'-6" = 404'-0"
Col. 14 WF 84
For Connections see Detail 2, Dwg. #14
For continuation of Structure see Dwg. #14
Junction Box for Overpass Telephone Booth
GIRDER
Scale: 1/2" = 1'-0"
INTERIOR STRINGER
Scale: 3/4" = 1'-0"
FASCIA STRINGER
Scale: 1" = 1'-0"
CONNECTION DETAILS
DETAIL 'B'
SCALE 1/2" = 1'-0"
SECTION D-D
SCALE: 1/16" = 1'-0"
Aluminum Railing AL-1 Typical
7 Equal Panels Typ.
Crown EL 33.0
Finished Grade ≠ Existing Ground Surface
Note: For Detail at Intermediate F.B. See Section A-A
SECTION E-E
SCALE: 1/2" = 1'-0"
Top of Steel Framing
Downward Slope
Fixed Side
Expansion Side
Symmetrical about ℄
Top of Floorbeam Elev. 32.51
Timber Deck
T.G.L. Elev. 33.0
Floorbeam Nailer, See Dwg. #17
Nailer, Variable Thickness See Dwg. #17
Turn up to Lamppost
Supporting Structure for N.Y.C. Transit Authority cables to be designed by N.Y.C.T.A. (N.I.C.)
See Detail 'B' for Electrical Details
Col. 14 WF 68
Remove Existing Column and Connection Angles. Replace Existing Nuts as ordered by the Engineer.
Existing Reinforced Concrete Pedestal
Existing 4-7/8" ϕ Anchor Bolts
These Anchorage Details apply to Bents 1 thru 9 except for Base Plate Dimensions (Existing Pedestals only)
(*) Note: New Base ℞ for Bent 10 to be 12" × 1/2" × 1'-10". Existing Base ℞ for Bent 10 is 12" × 1/4" × 1'-10"
SECTION C-C
SCALE: 3/4" = 1'-0"
SECTION A-A
SCALE: 1/2" = 1'-0"
SECTION B-B
SCALE: 1/2" = 1'-0"
Top of Timber Deck
TYPICAL LATERAL BRACING DETAILS
SCALE: 3/4" = 1'-0"
AT INTERMEDIATE FLOORBEAM
Top Flange of 36 WF 160
Bottom Flange 21 WF 62
Rev. 3-8-62
NOTES:
For Deck & Railing Details, See Dwg. #17
For Base Plate Dimensions and Anchorage Details, Bent 18 see Dwg. #18
For Deck and Railing Details see Dwg. #17
All Rivets shall be 3/4" Diameter unless noted otherwise.
All Fillet Welds shall be 5/16" unless noted otherwise.
The Contractor shall verify all Dimensions and Elevations which refer to existing Structure and Foundations.
For Expansion Joints in Timber Deck See Dwg. #16
All Superstructure Material shall be Structural Steel (A.S.T.M. A-36) unless noted otherwise.
CITY OF NEW YORK DEPARTMENT OF PARKS
NEW YORK WORLD'S FAIR 1964-1965
FLUSHING MEADOW PARK
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
FRAMING AND DETAILS - BENTS 10 TO 18
DRAWING NO. 13
CONTRACT NO. 2-99-061
DATE 11-10-61
SCALE AS SHOWN
ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
In Charge of
Made by
Traced by
Checked by

ANCHORAGE DETAILS - PLANS & SECTIONS FOR FUTURE ROOF COLUMNS
Scale: 3/4"=1'-0"

DETAIL OF TELEPHONE CONDUITS
Scale 1"=1'-0"

SECTION 4-4
Scale 1"=1'-0"

SECTION 5-5

FRAMING PLAN

Bent No.	Top Slab El. at ℄ Bent Along Theo. G.L.
18	33.00
19	32.13
19A	31.47
20	31.18
21	30.93
22	30.32
23	30.14
24	29.54
25	28.86
26	28.67
27	27.55
28	26.27
29	24.10

TYPICAL RAILPOST SPACING at STAIRWELL
Scale 3/4"=1'-0"

DECK PLAN
Scale 1"=50'

PROFILE
Scale 1"=50' Horiz.
1"=40' Vert.

CORNER REINFORCEMENT DETAILS
Scale 1/2"=1'-0"

SECTION 1-1
Scale 1/2"=1'-0"

ELEVATION 2-2
Scale 1/4"=1'-0"

SOUTH ELEVATION
Note: Railing not shown

TYPICAL SCORING PLAN
Not to Scale

PLAN

STAIRWELL DETAILS

SECTION 3-3

Rev. 3-8-62
Rev. 8-22-62

Notes:
All fillet welds 5/16 unless otherwise noted
For railing details see Dwg. #17
Minimum splice length for reinforcement is 20 diameters
All concrete to be Class B-32
Concrete walking surfaces to be scored with 1/4" scores
Layout to be submitted to the engineer for approval
Beams to be cambered as shown in framing & in table.
■ indicates location of future roof column.
All Material to be Structural Steel unless noted.
For Drainage and Joint Details see Dwg. #16
For Entrance Structure see Dwgs. #24 & #25.
( ) Denotes Dead Load Camber.
[ ] Denotes Vertical Curve Camber.
All Rubber Jt. Mat'l to be colored to match concrete deck.
For Scoring on Deck see Dwg #16
(B) conduits leading down to LIRR platform telephone booth.
(O) conduits leading to Junction Box for Overpass Telephone booth.

CITY OF NEW YORK DEPARTMENT OF PARKS

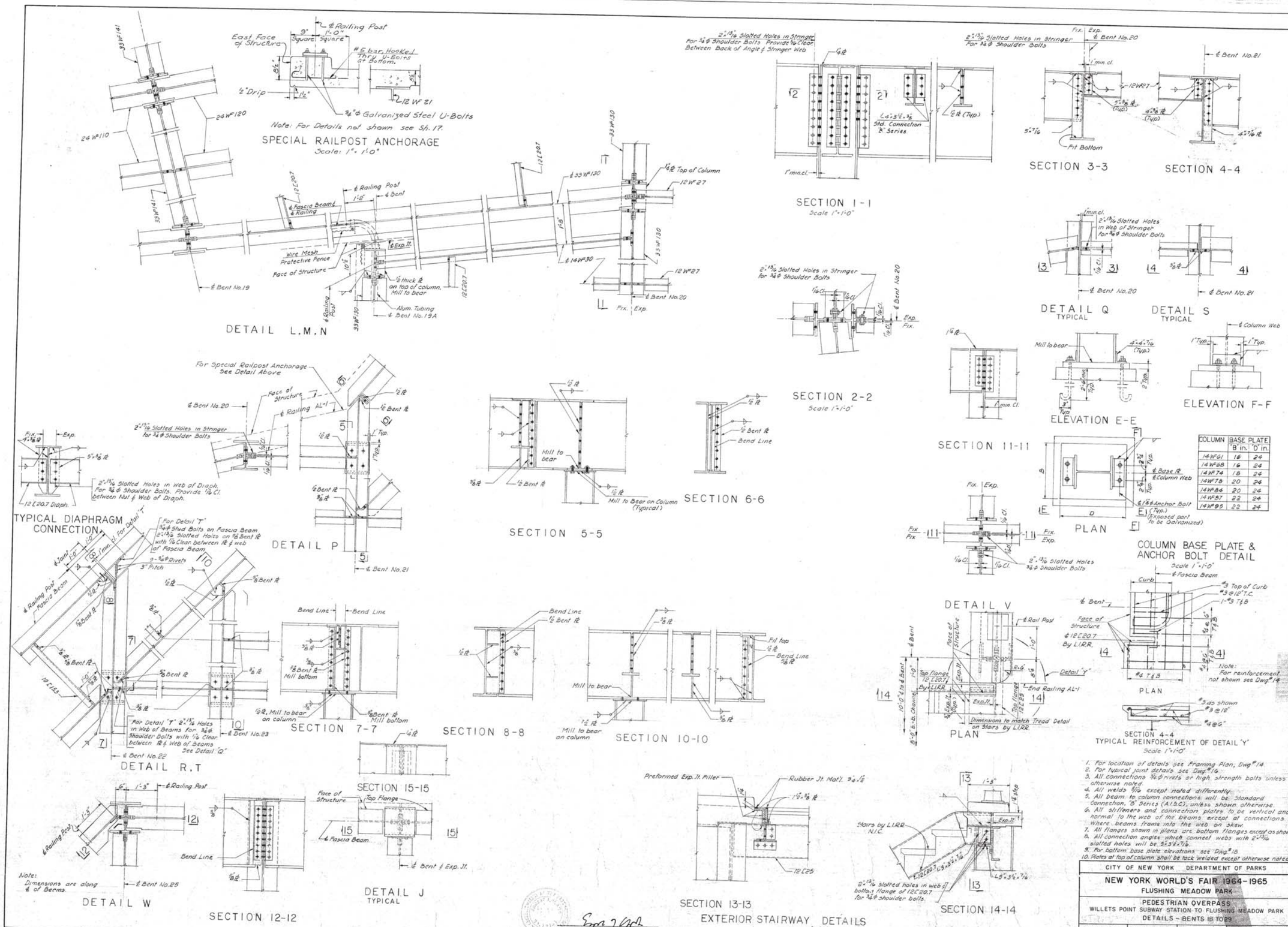
NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
FRAMING - BENTS 18 TO 29

DRAWING NO.	CONTRACT NO.	DATE 11-10-61	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
14	Q-99-061	SCALE 1/8"=1'-0" & 1/4" As shown	

In Charge of: [illegible]
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]



COLUMN	BASE PLATE 'B' in.	BASE PLATE 'D' in.
14WF61	16	24
14WF68	16	24
14WF74	18	24
14WF78	20	24
14WF84	20	24
14WF87	22	24
14WF95	22	24

1. For location of details see Framing Plan, Dwg.# 14
2. For typical joint details see Dwg.# 16
3. All connections ⅞"ϕ rivets or high strength bolts unless otherwise noted.
4. All welds 5/16 except noted differently.
5. All beam to column connections will be Standard Connection, "B" Series (AISC); unless shown otherwise.
6. All stiffeners and connection plates to be vertical and normal to the web of the beams except at connections where beams frame into the web on skew.
7. All flanges shown in plans are bottom flanges except as shown.
8. All connection angles which connect webs with 2"-13/16 slotted holes will be 3"x3"x7/16.
9. For bottom base plate elevations see Dwg.# 18
10. Plates at top of column shall be tack welded except otherwise noted.

CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964-1965
FLUSHING MEADOW PARK

PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
DETAILS - BENTS 18 TO 29

DRAWING NO.	CONTRACT NO.	DATE	SCALE
15	0-99-061	11-10-61	1"=1'-0" except as shown

ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK

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Made by: [illegible]
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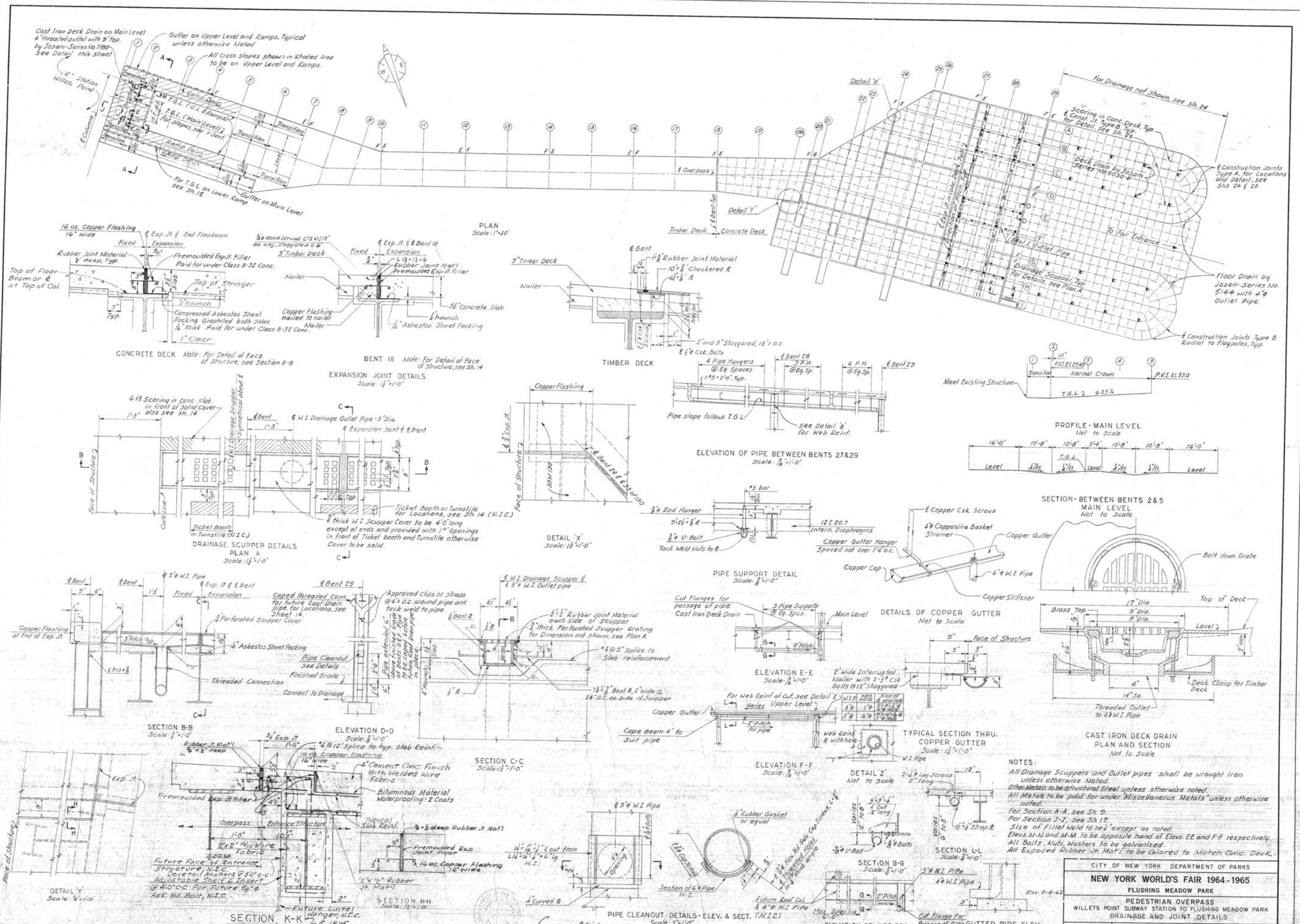
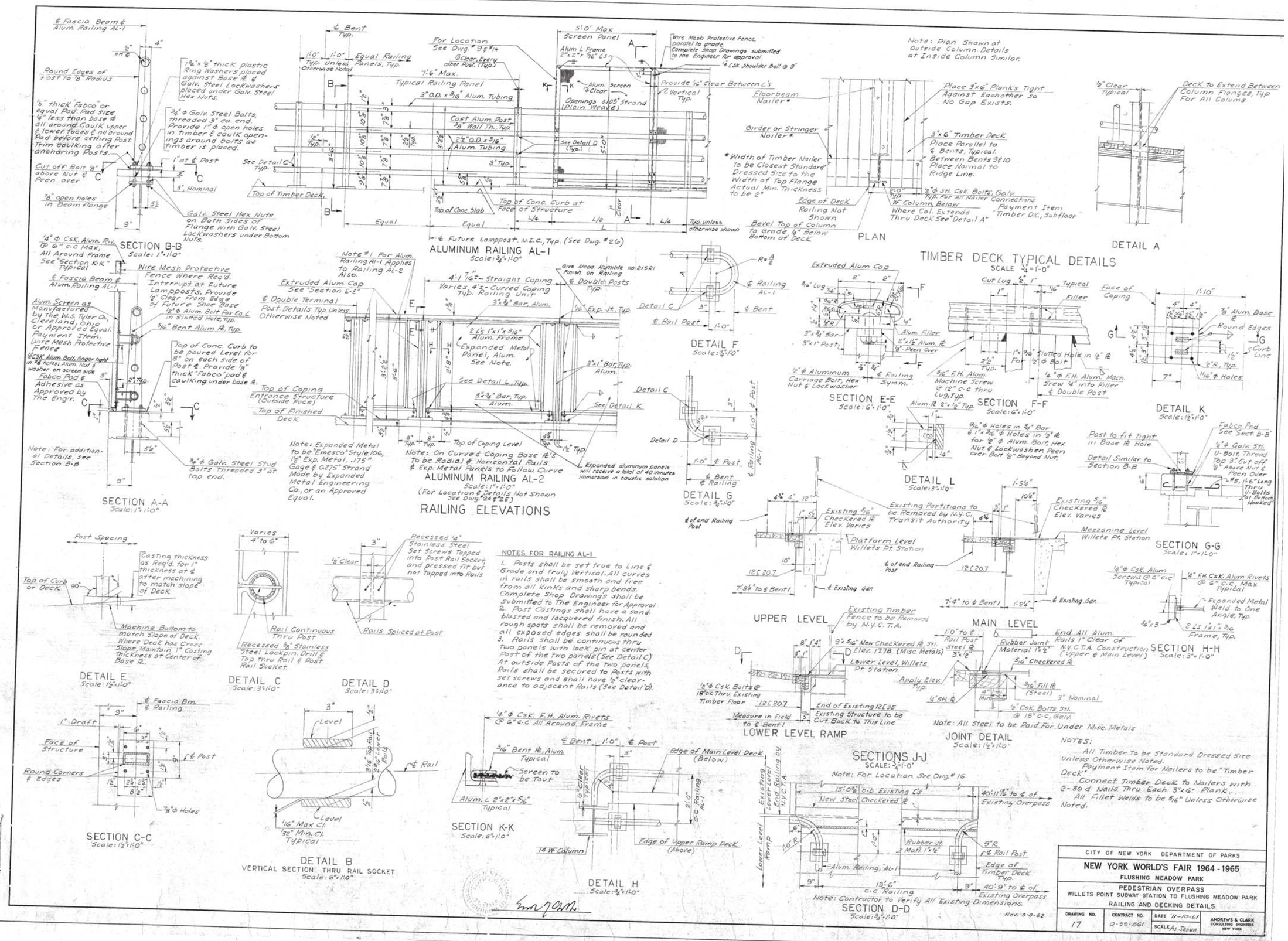
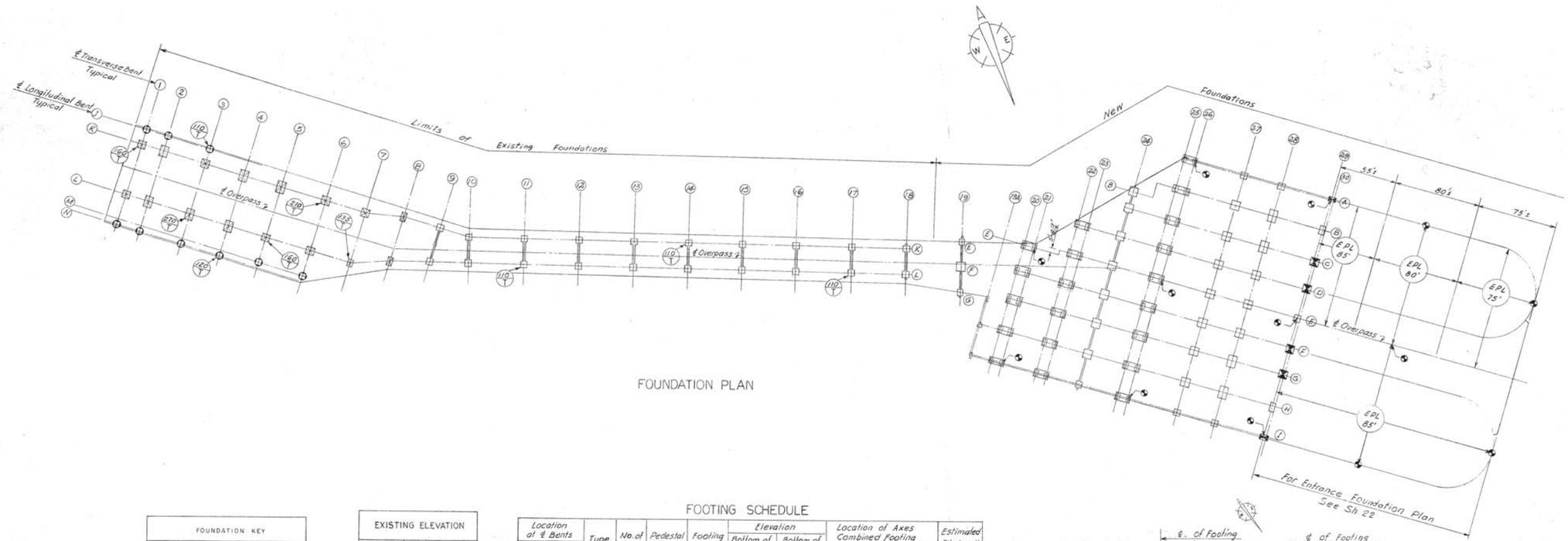

Cast Iron Deck Drain on Main Level 4" threaded outlet with 9" top, by Josam-Series No. 77180-See Detail this sheet
Gutter on Upper Level and Ramps, Typical unless otherwise Noted
All Cross slopes shown in Shaded Area to be on Upper Level and Ramps.
"R" Station Willets Point
Transition
Ramp Down
T.G.L. (U.L. & Ramps)
T.G.L. (Main Level) For slopes see Y-Sect.
Level
For T.G.L. on Lower Ramp see Sh. 12
Gutter on Main Level
℄ Columns
℄ Overpass
Detail 'X'
Detail 'Y'
Timber Deck
Concrete Deck
For Drainage not shown, see Sh. 24
Scoring in Conc. Deck, Typ. & Const. Jt. Type B, Typ. for Detail, See Sh. 26
Deck Drain by Josam Series No. 25050-6
℄ Construction Joints Type A, for Locations and Detail, see Shs. 24 & 25.
Exp. Jt. in Conc. Deck, Typ. Reactions & Pitch, Typ.
To Fair Entrance
W.I. Outlet Pipe
Drainage Scupper, Typ. for Details, see Plan A
Floor Drain by Josam-Series No. 5144 with 4" ϕ Outlet Pipe.
℄ Construction Joints Type B Radial to Flagpoles, Typ.
PLAN
Scale: 1"=40'
16 oz. Copper Flashing 16" Wide
℄ Exp. Jt. & End Floorbeam
Fixed
Expansion
Rubber Joint Material 3/8" deep, Typ.
Premoulded Exp. Jt. Filler Paid for under Class B-32 Conc.
Top of Floor Beam or ⅊ at Top of Col.
Top of Stringer
5" Slab
℄ haunch
3" Typ.
Compressed Asbestos Sheet Packing Graphited both sides 1/16" thick Paid for under Class B-32 Conc.
1" Clear
CONCRETE DECK Note: For Detail at Face of Structure, see Section B-B
3/8" ϕ Wood Screws C'ts 12" ea. way, Staggered @ 6"
3" Timber Deck
Nailer
℄ Exp. Jt. & ℄ Bent 18
L 1½ × 1½ × 6
Rubber Joint Mat'l
Premoulded Exp. Jt. Filler
Copper Flashing nailed to nailer
5½" Concrete Slab
℄ haunch
1/16" Asbestos Sheet Packing
BENT 18 Note: For Detail at Face of Structure, see Sh. 14
EXPANSION JOINT DETAILS
Scale: 1½"=1'-0"
℄ Bent
1" ϕ Rubber Joint Material
10" × ⅜" Checkered ⅊
6" × ⅜" ⅊
2" and 3" Staggered, 18" c. o. c.
⅜" ϕ Csk. Bolts
TIMBER DECK
4 Pipe Hangers @ Eq. Spaces
1" × 5" × 2'-0", Typ.
℄ Bent 28
3 P.H. @ Eq. Sp.
4 P.H. @ Eq. Sp.
℄ Bent 29
Pipe slope follows T.G.L.
See Detail 'Z' for Web Reinf.
ELEVATION OF PIPE BETWEEN BENTS 27 & 29
Scale: ⅜"=1'-0"
Meet Existing Structure
P.V.I. El. 25.49
Normal Crown
P.V.I. El. 33.0
T.G.L. @ 0.25%
PROFILE - MAIN LEVEL
Not to Scale
16'-0" Level
10'-8" ½"/ft.
10'-8" ½"/ft.
T.G.L.
5'-4"
10'-8" ½"/ft.
10'-8" ½"/ft.
16'-0" Level
6 ft. Scoring in conc. slab in front of Solid cover also see Sh. 14
1'-3"
℄ Bent
℄ W.I. Drainage Outlet Pipe - 5" Dia.
℄ Expansion Joint & ℄ Bent
W.I. Drainage Scupper, Symmetrical about ℄
Face of Structure
Curb Line
Ticket Booth or Turnstile (N.I.C.)
Ticket Booth or Turnstile for Locations, see Sh. 14 (N.I.C.)
⅜" thick W.I. Scupper Cover to be 4'-0" long except at ends and provided with 1" openings in front of Ticket Booth and Turnstile otherwise Cover to be solid.
DRAINAGE SCUPPER DETAILS
PLAN A
Scale: 1½"=1'-0"
Copper Flashing
℄ Exp. Jt.
Face of Structure
℄ Bent 28 & 29 & 33 W 130
DETAIL 'X'
Scale: 1½"=1'-0"
*5 bar
¾" ϕ Rod Hanger
9" × 2½" × ⅜" ⅊
⅜" ϕ U-Bolt
Tack Weld Nuts to ⅊
12 [20.7
Interm. Diaphragms
PIPE SUPPORT DETAIL
Scale: ⅜"=1'-0"
℄ Copper Csk. Screws
4" ϕ Copper Wire Basket Strainer
Copper Gutter
Copper Gutter Hanger Spaced not over 2'-6" o.c.
Copper Cap
4" ϕ W.I. Pipe
Copper Stiffener
DETAILS OF COPPER GUTTER
Not to Scale
SECTION - BETWEEN BENTS 2 & 5
MAIN LEVEL
Not to Scale
Bolt down Grate
Brass Top
17" Dia.
9" Dia.
8" Dia.
Top of Deck
Level
2" Min.
4"
14" Sq.
Deck Clamp for Timber Deck
Threaded Outlet to 4" ϕ W.I. Pipe
CAST IRON DECK DRAIN
PLAN AND SECTION
Not to Scale
℄ Bent
℄ 5" ϕ W.I. Pipe
℄ Exp. Jt. & ℄ Bent
Fixed
Expansion
Copper Flashing at End of Exp. Jt.
3" Pitch, Typ.
¾" Perforated Scupper Cover
16" Asbestos Sheet Packing
L 3 × 3 × ⅜
Threaded Connection
SECTION B-B
Scale: ⅜"=1'-0"
℄ Bent 29
Capped threaded Conn. for future Roof-Drain pipe for Locations, see Sheet 14
Approved clips or straps @ 4'-0" o.c. around pipe and tack weld to pipe
Pipe extended 6" above finished deck surface to be capped & active until future Roof Drain pipe is installed.
Pipe Cleanout See Details
Finished Grade
Connect to Drainage
ELEVATION D-D
Scale: ⅜"=1'-0"
℄ W.I. Drainage Scupper & ℄ 5" ϕ W.I. Outlet pipe
⅜" × ⅜" Rubber Joint Material each side of Scupper
¾" thick Perforated Scupper Grating for Dimension not shown, see Plan A
*4 @ 9" Splice to Slab reinforcement
1½" × ⅜" Bent ⅊, 2" wide @ 24" O.C. ea. side of Scupper
SECTION C-C
Scale: ½"=1'-0"
Cut Flanges for passage of pipe
Cast Iron Deck Drain
3 Pipe Supports @ Eq. Spcs.
Main Level
2" Pitch
ELEVATION E-E
Scale: ⅜"=1'-0"
9"
Face of Structure
5"
2" Wide Interrupted Nailer with 2-½" ϕ Csk Bolts @ 12" Staggered
For Web Reinf at Cut, see Detail 'Z'
Varies
Upper Level
Copper Gutter
2" Pitch for pipe
Cope Beam 4" to Suit pipe
ELEVATION F-F
Scale: ⅜"=1'-0"
Web Reinf. ⅊ with hole
W.I. Pipe
TYPICAL SECTION THRU. COPPER GUTTER
Scale: 1½"=1'-0"
DETAIL 'Z'
Not to Scale
2-½" ϕ Lag Screws 2" long
½" × ⅜" Strap ⅊
SECTION L-L
Scale: ⅜"=1'-0"
NOTES:
All Drainage Scuppers and Outlet pipes shall be wrought iron unless otherwise Noted.
Other Metals to be Structural Steel unless otherwise noted.
All Metals to be paid for under "Miscellaneous Metals" unless otherwise noted.
For Section A-A, see Sh. 9
For Section J-J, see Sh. 17
Size of Fillet Weld to be ¼" except as noted.
Elevs. M-N and M-M. To be opposite hand of Elevs. E-E and F-F respectively.
All Bolts, Nuts, Washers to be galvanized.
All Exposed Rubber Jt. Mat'l to be Colored to Match Conc. Deck.
Rubber Jt. Mat'l ¾" × 2" deep
Exp. Jt.
Premoulded Exp. Jt. Filler
16 oz. Copper Flashing 16" Wide
4" Cement Conc. Finish With Welded Wire Fabric
Bituminous Material Waterproofing - 2 Coats
Overpass
Entrance Structure
1'-0"
10½"
Typical Slab Reinf.
¾" × ½" deep Rubber Jt. Mat'l
Premoulded Exp. Joint Filler
16 oz. Copper Flashing 16" wide
2" × 2" #10 Wire Fabric
Future Face of Entrance Structure N.I.C.
Dovetail Anchors @ 30" c. c.
Adjustable Shelf ∠ Insert @ 40" c. c. For Future ⅝" ϕ Csk. Hd. Bolt, N.I.C.
2" × ½" Rubber Jt. Mat'l.
Future Lintel Hanger, N.I.C.
℄ 18 WF
SECTION K-K
Scale: 1½"=1'-0"
DETAIL 'Y'
Scale: ¼"=1'-0"
SECTION H-H
Scale: 1½"=1'-0"
℄ 5" ϕ W.I. Pipe
¼" × ¼" × 6" Lg. W.I.
¼" Curved ⅊
Section of 6" ϕ Pipe W.I.
¼" Rubber Gasket or equal
¼" ϕ Non-Mtl. Gaskt. Cap Screws
¼" ϕ Threaded Holes in ¼" thick angle
PIPE CLEANOUT DETAILS - ELEV. & SECT. (N.I.C.)
Scale: 3"=1'-0"
Varies 6"-10"
⅜" ϕ U-Bolt
⅜" ϕ Bolts
SECTION G-G
Scale: ⅜"=1'-0"
Future Roof Col. & 4" ϕ W.I. Pipe
Max. Spacing 6'-0"
5" ϕ W.I. Pipe
4" ϕ W.I. Pipe
2" Pitch
Cut Flange for Passage of Pipe
ELEVATION OF PIPE FOR FUTURE "HOOF DRAINS ALONG BENT 29
Scale: ¾"=1'-0"
GUTTER PIPE ELEV. ABOVE PEDESTAL
Scale: ⅜"=1'-0"
Rev. 3-8-62
In Charge of: Made by: Traced by: Checked by:
CITY OF NEW YORK DEPARTMENT OF PARKS
NEW YORK WORLD'S FAIR 1964-1965
FLUSHING MEADOW PARK
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
DRAINAGE AND JOINT DETAILS
DRAWING NO. 16
CONTRACT NO. 9-99-861
DATE 11-16-61
SCALE As Shown
ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK


SECTION B-B
Scale: 1"=1'-0"
SECTION A-A
Scale: 1"=1'-0"
ALUMINUM RAILING AL-1
Scale: 3/4"=1'-0"
ALUMINUM RAILING AL-2
Scale: 1"=1'-0"
(For Location & Details Not Shown See Dwg. #24 & #25)
RAILING ELEVATIONS
PLAN
DETAIL A
TIMBER DECK TYPICAL DETAILS
SCALE 3/4"=1'-0"
DETAIL F
Scale: 3/4"=1'-0"
SECTION E-E
Scale: 6"=1'-0"
SECTION F-F
Scale: 6"=1'-0"
DETAIL K
Scale: 1 1/2"=1'-0"
DETAIL L
Scale: 3"=1'-0"
DETAIL G
Scale: 3/4"=1'-0"
SECTION G-G
Scale: 1"=1'-0"
SECTION H-H
Scale: 3"=1'-0"
DETAIL E
Scale: 1 1/2"=1'-0"
DETAIL C
Scale: 3"=1'-0"
DETAIL D
Scale: 3"=1'-0"
SECTION C-C
Scale: 1 1/2"=1'-0"
DETAIL B
VERTICAL SECTION THRU RAIL SOCKET
Scale: 6"=1'-0"
SECTION K-K
Scale: 6"=1'-0"
DETAIL H
Scale: 3/4"=1'-0"
UPPER LEVEL
LOWER LEVEL RAMP
MAIN LEVEL
JOINT DETAIL
Scale: 1 1/2"=1'-0"
SECTIONS J-J
SCALE: 3/4"=1'-0"
SECTION D-D
Scale: 3/4"=1'-0"
NOTES FOR RAILING AL-1
1. Posts shall be set true to Line & Grade and truly Vertical. All curves in rails shall be smooth and free from all kinks and sharp bends. Complete Shop Drawings shall be submitted to The Engineer for Approval.
2. Post Castings shall have a sand-blasted and lacquered finish. All rough spots shall be removed and all exposed edges shall be rounded.
3. Rails shall be continuous thru two panels with lock pin at center Post of the two panels (See Detail C). At outside Posts of the two panels, Rails shall be secured to Posts with set screws and shall have 1/8" clearance to adjacent Rails (See Detail D).
NOTES:
All Timber to be Standard Dressed Size Unless Otherwise Noted.
Payment Item for Nailers to be "Timber Deck".
Connect Timber Deck to Nailers with 2-30 d Nails Thru Each 3"x6" Plank.
All Fillet Welds to be 5/16" Unless Otherwise Noted.
CITY OF NEW YORK DEPARTMENT OF PARKS
NEW YORK WORLD'S FAIR 1964-1965
FLUSHING MEADOW PARK
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
RAILING AND DECKING DETAILS
DRAWING NO. 17
CONTRACT NO. Q-22-861
DATE 11-10-61
SCALE As Shown
ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
Rev. 3-8-62



FOUNDATION PLAN

FOUNDATION KEY	
	EXISTING PILE GROUP TO REMAIN NEW FOOTING TYPE E
	EXISTING FOUNDATION-REPLACE SPALLED CONCRETE TIE A.O.B.E.
90 T	GROUP PILE TEST-NO. INDICATES TEST LOADING IN TONS
	EXISTING FOUNDATION TO REMAIN
	NEW FOUNDATION TYPE C-L COMBINED WITH L.I.R.R.
	NEW FOUNDATION TYPE C-E COMBINED WITH ENTRANCE
	NEW FOUNDATION TYPE S SINGLE
	NEW FOUNDATION TYPE S-CW SINGLE & CRASH WALL

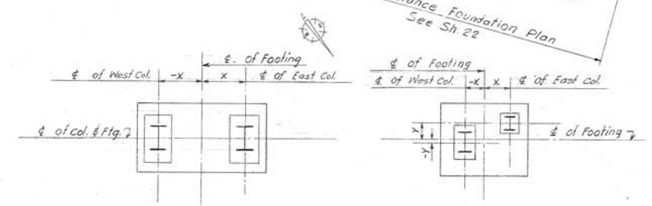
EXISTING ELEVATION	
Transverse Bent	Bottom of Col. Base ℄
1, 2	12.50
3, 4	12.00
5, 6, 7	11.00
8 to 16	14.00
17	10.50

Note: All Existing Elevations Shown above to be verified by Contractor.

FOOTING SCHEDULE

Location at ℄ Bents Transv.	Longi.	Type	No. of Piles	Pedestal Height	Footing Thickness	Elevation Bottom of Col. Base ℄	Elevation Bottom of Footing	Location of Axes Combined Footing Left Col. -x	Left Col. y	Right Col. x	Right Col. y	Estimated Pile Length (EPL)
1	J&N	E	4*	4'-6"	2'-0"	12.50	6.00	-	-	-	-	
2	J&N	↑	6*	4'-6"	2'-0"	12.50	↑	-	-	-	-	
3	J–N		2*–6*	4'-0"–3'-6"	2'-0"–2'-6"	12.00		-	-	-	-	Existing Piles
4	N		4*	4'-0"	2'-0"	12.00		-	-	-	-	
5	M	↓	4*	3'-0"	2'-0"	11.00	↓	-	-	-	-	
6	M	E	4*	3'-0"	2'-0"	11.00	6.00	-	-	-	-	
18	K&L	S-CW	7**	5'-9½"	2'-4"	9.12	3.00	-	-	-	-	
19	E&G	S-CW	5	2'-1½"	2'-0"	9.12	5.00	-	-	-	-	
	F	S-CW	8	2'-1½"	2'-6"	9.12	4.50	-	-	-	-	
19A	G&I	S-CW	2	2'-0"	2'-0"	6.37	2.37	-	-	-	-	
	H	S-CW	3	2'-0"	2'-0"	6.37	2.37	-	-	-	-	
20	E	C-L	14	5'-0"	2'-6"	10.42	2.92	5'-0"	-	5'-0"	-	
&	F&G	↑	15	↑	↑	↑	↑	5'-4"	-	4'-8"	-	
21	H		13					5'-6"	-	4'-6"	-	100'
	I		10					5'-3"	-	4'-9"	-	
	D		14					5'-0"	-	5'-0"	-	
22	E		13					↑	-	↑	-	
&	F		13						-		-	
23	G		14						-		-	
	H	↓	14	↓	↓	↓	↓		-		-	
	I	C-L	13	5'-0"	↑	10.42	2.92	5'-0"	-	5'-0"	-	
24	B to H	S-CW	8	2'-0"	2'-6"	6.37	1.87	-	-	-	-	
	I	S-CW	5	2'-0"	2'-0"	6.37	2.37	-	-	-	-	
	A	C-L	13	5'-0"	2'-6"	10.42	2.92	4'-8"	-	5'-4"	-	
	B	↑	16	↑	↑	↑	↑	5'-4"	-	4'-8"	-	
25	C		15					↑	-	↑	-	
&	D		15						-		-	
26	E		15						-		-	95'
	F		15						-		-	
	G		15						-		-	
	H	↓	16	↓	↓	↓	↓	5'-4"	-	4'-8"	-	
	I	C-L	13	5'-0"	2'-6"	10.42	2.92	5'-0"	-	5'-0"	-	
	A	S	5	2'-0"	2'-0"	7.00	3.00	-	-	-	-	
27	B to H	↑	9	1'-6"	2'-6"	↑	↑	-	-	-	-	
	I		5	2'-0"	2'-0"			-	-	-	-	
	A		5	2'-0"	2'-0"			-	-	-	-	
	B		10	1'-0"	3'-0"			-	-	-	-	
	C		10	1'-0"	3'-0"			-	-	-	-	
28	D		9	1'-6"	2'-6"			-	-	-	-	90'
	E		9	1'-6"	2'-6"			-	-	-	-	
	F		9	1'-6"	2'-6"			-	-	-	-	
	G		10	1'-0"	3'-0"			-	-	-	-	
	H	↓	10	1'-0"	3'-0"	↓	↓	-	-	-	-	
	I	S	5	2'-0"	2'-0"	7.00	3.00	-	-	-	-	
	A	C-E	8	1'-0"	3'-0"	7.08	3.08	11½"	-	11"	-½"	
	B	S	6	1'-6"	2'-6"	↑	↑	-	-	-	-	
	C	C-E	9	1'-0"	3'-0"			10⅛"	17¾"	12⅝"	-22"	
	D	C-E	9	1'-0"	3'-0"			10¼"	-18¼	12¼"	21⅞"	
	E	S	5	2'-0"	2'-0"			-	-	-	-	85'
	F	C-E	6	1'-0"	3'-0"			10¼"	18¼	12¼"	-21⅞"	
	G	C-E	9	1'-0"	3'-0"			10⅛"	-17¾"	12⅝"	22"	
	H	S	6	1'-6"	2'-6"	↓	↓	-	-	-	-	
	I	C-E	8	1'-0"	3'-0"	7.08	3.08	11½"	-	11"	½"	

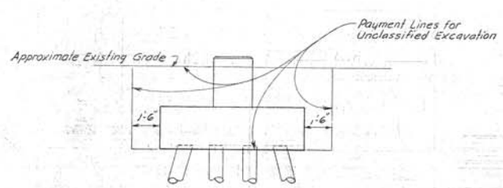
* Indicates number of existing piles to remain.
** Indicates 2 New piles added and 5 existing piles to remain.



TYPE C-L

TYPE C-B

LOCATION OF COMBINED FOOTING
Not to Scale



PAYMENT LINES FOR EXCAVATION
Not to Scale

Notes:

1. All Concrete for Foundations shall be Class B-32 Concrete.
2. Concrete placed under certain Existing Pile Footings due to ground settlement shall be Class D-18 Concrete. For detail, see Detail 'X' Sh. 20.
3. Designed Bearing Capacity of Creosoted Timber Pile to be 20 Tons.
4. Pile to be battered one horizontal to five vertical wherever indicated.
5. For Details of Foundation Type E, see Sh. 20.
6. For Details of foundations at Bents 18, 19, and 19A, see Sh. 20, & 21.
7. For Foundation Details - Type C-B and Type S, see Sh. 19.
8. For Foundation Details - Type C-L, Type S, and Type S-CW, see Sh. 21.
9. Column to be placed at center of single footing and their Axes coincide.
10. Actual pile length to be determined by test piles driven adjacent to boring location shown as ●.
11. For locations of Columns and Bents, see Framing Plans, Shs. 10, 12, 13, & 14.
12. Where Interference with Existing Piles occurs, They Will be Pulled as Ordered by The Engineer and Paid for Under 'Remove Existing Timber Piles.'
13. For Pile Splice Detail, See Sh. 19.
14. Length of Borings shall be a minimum 10' below the Estimated Pile Length and 20' into sand.
15. Test Piles shall be driven 10' below Estimated Pile Length and shall be jetted, if ordered by the Engineer, to pierce any intermediate hard layer.

In Charge of: [illegible]
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]

Rev. 3-8-62

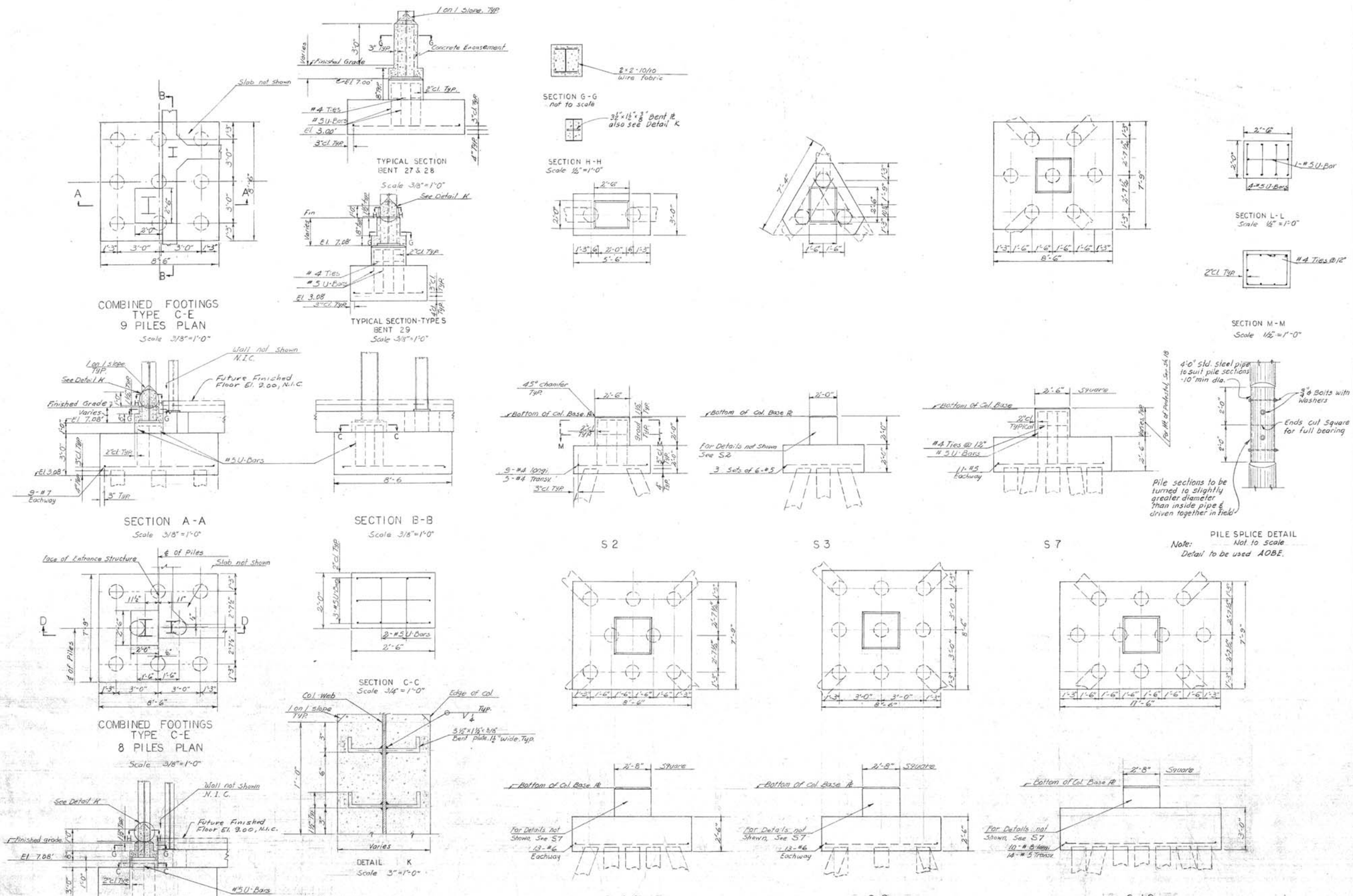
CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
FOUNDATION PLAN AND DETAILS

DRAWING NO.	CONTRACT NO.	DATE	SCALE	
18	9-99-861	11-10-61	1" = 40'	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK



NOTES:

1. For General Notes, See Sheet 18
2. Number shown after Type S indicates number of piles in footing.
3. For foundation details - S4, S5, S6 not shown, see sheet 21.
4. For Location of Axes of Combined footing Type C-E, see sheet 18
5. For anchorage details of Base Plates not shown, see sheet 15.
6. For Reinforcement in beams not shown, See Sheet 23.
7. For Excavation Payment Lines, See Sheet 18.

SINGLE FOOTINGS TYPE S PLANS & ELEVATIONS

Scale 3/8"=1'-0"

Rev. 3-8-62

CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

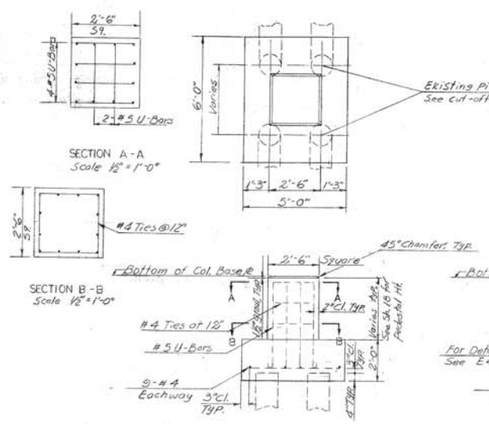
PEDESTRIAN OVERPASS

WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK

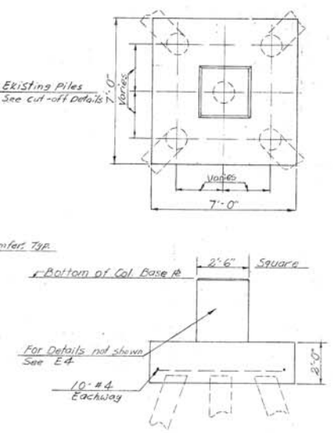
FOUNDATION DETAILS - TYPE C-B AND TYPE S

DRAWING NO.	CONTRACT NO.	DATE	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
19	Q-99-861	11-10-61	
		SCALE As shown	

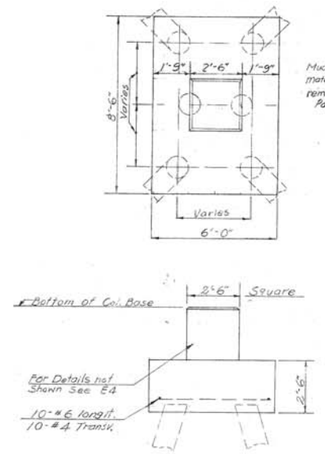
In Charge of: [illegible]
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]



TYPE E4

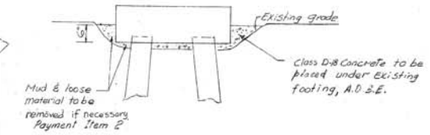


TYPE E5

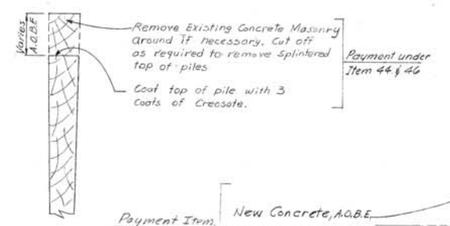


TYPE E6

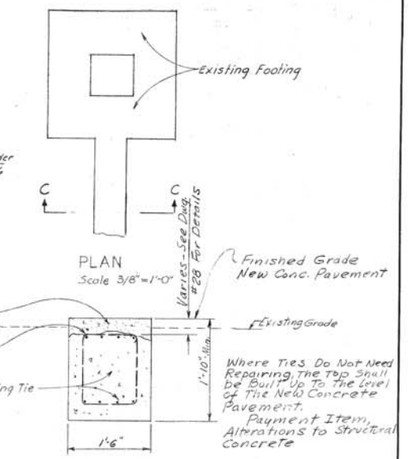
EXISTING PILES WITH NEW FOOTING
PLANS & ELEVATIONS
Scale 3/8" = 1'-0"



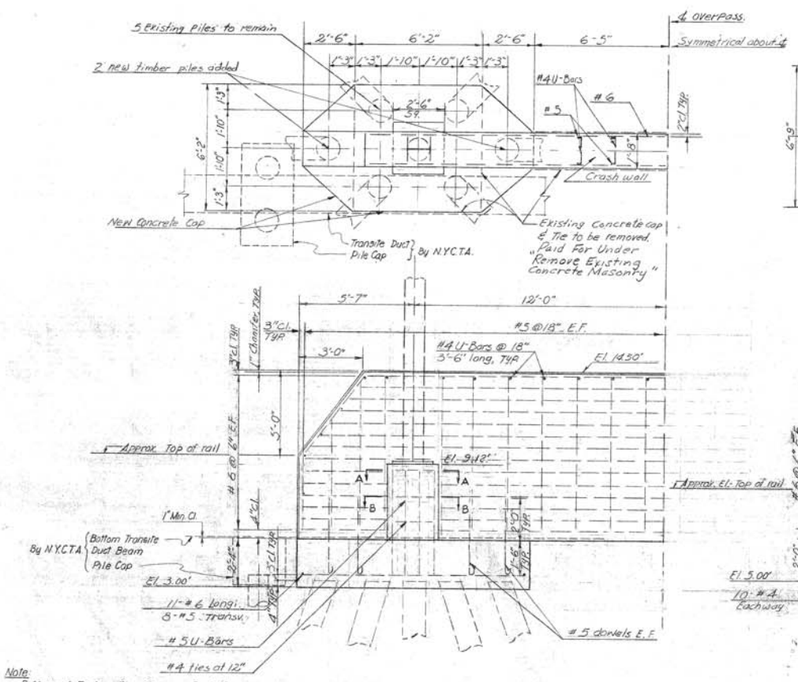
DETAIL X
not to scale



EXISTING PILE CUT-OFF DETAIL
Scale 3/8" = 1'-0"

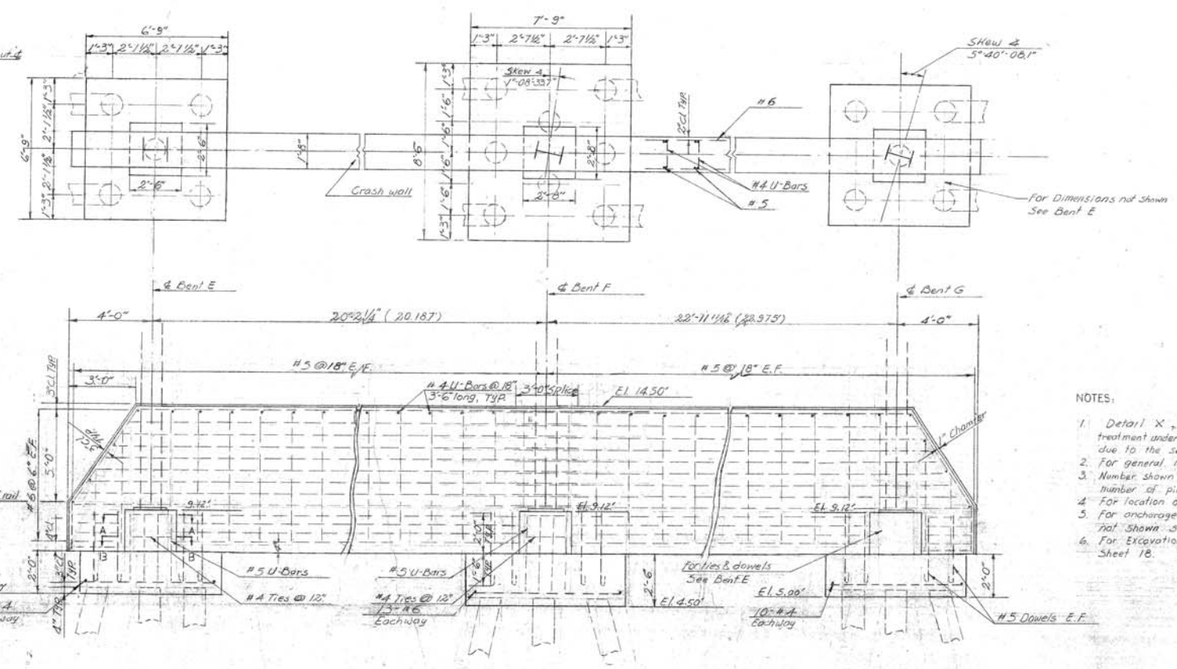


SECTION C-C
SPALLED TIE REPAIR
Scale 1" = 1'-0"



Note:
Bottom of Footing Elevations in Bent 18 shall be checked with Bottom of Transite Duct Beam and may be lowered if found necessary.
In removing Existing Concrete Cap and driving Timber Piles, care must be taken not to damage Transite Ducts & Footings built by N.Y.C.T.A

BENT 18 FOUNDATION
HALF PLAN & ELEVATION
Scale 3/8" = 1'-0"



BENT 19 FOUNDATION
PLAN & ELEVATION
Scale 3/8" = 1'-0"

NOTES:

1. Detail X, indicates the special treatment under existing pile foundation due to the settlement of ground.
2. For general notes, see sheet 18.
3. Number shown after Type E indicates number of piles in the footing.
4. For location of footing, See sheet 18.
5. For anchorage details of Base plates not shown see Dwg. #15
6. For Excavation Payment Lines See Sheet 18.

In Charge of: [illegible]
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]

Rev. 3-8-62

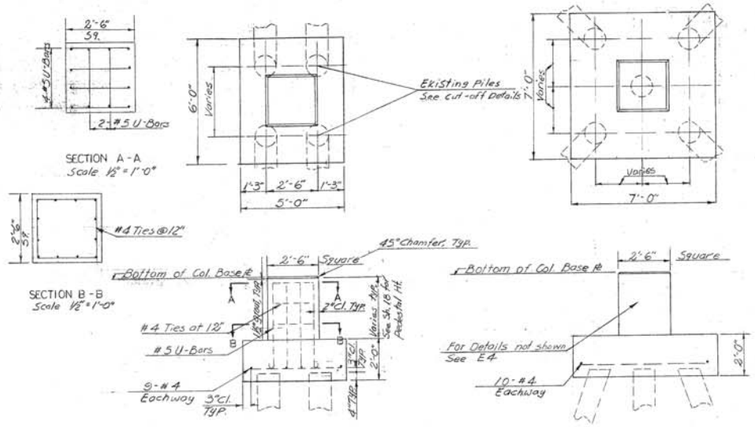
CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

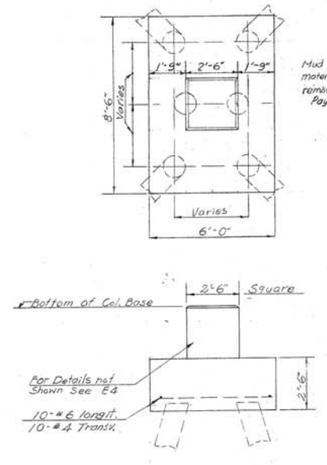
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
FOUNDATION DETAILS - TYPE E, BENT 18 & BENT 19

DRAWING NO.	CONTRACT NO.	DATE 11-10-61	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
20	Q-99-861	SCALE As Shown	



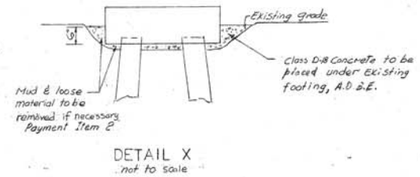
TYPE E4

TYPE E5

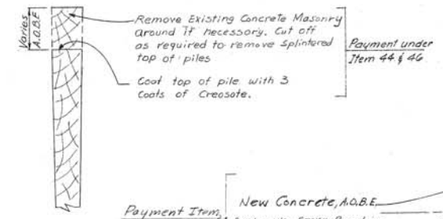


TYPE E6

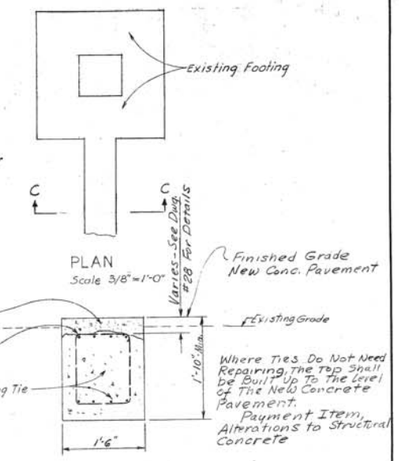
EXISTING PILES WITH NEW FOOTING
PLANS & ELEVATIONS
Scale 3/8" = 1'-0"



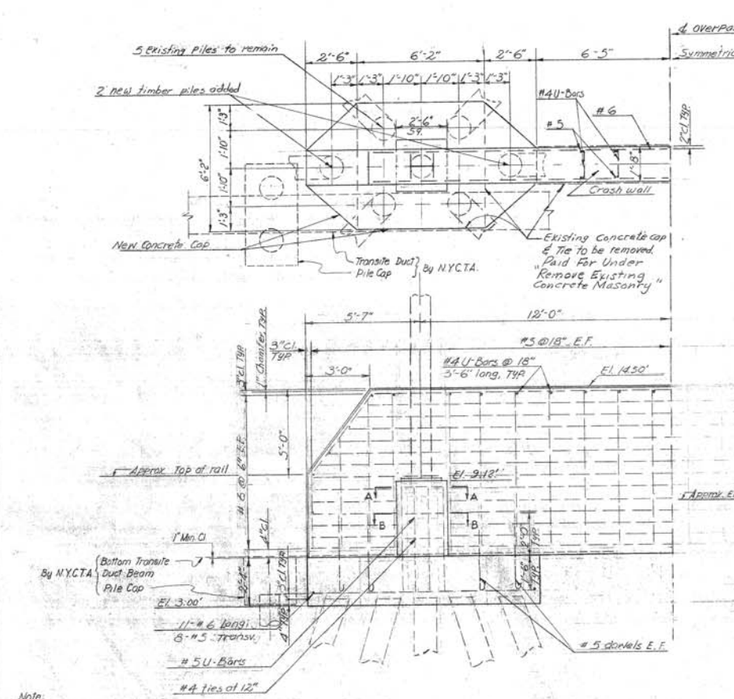
DETAIL X
not to scale



EXISTING PILE CUT-OFF DETAIL
Scale 3/8" = 1'-0"

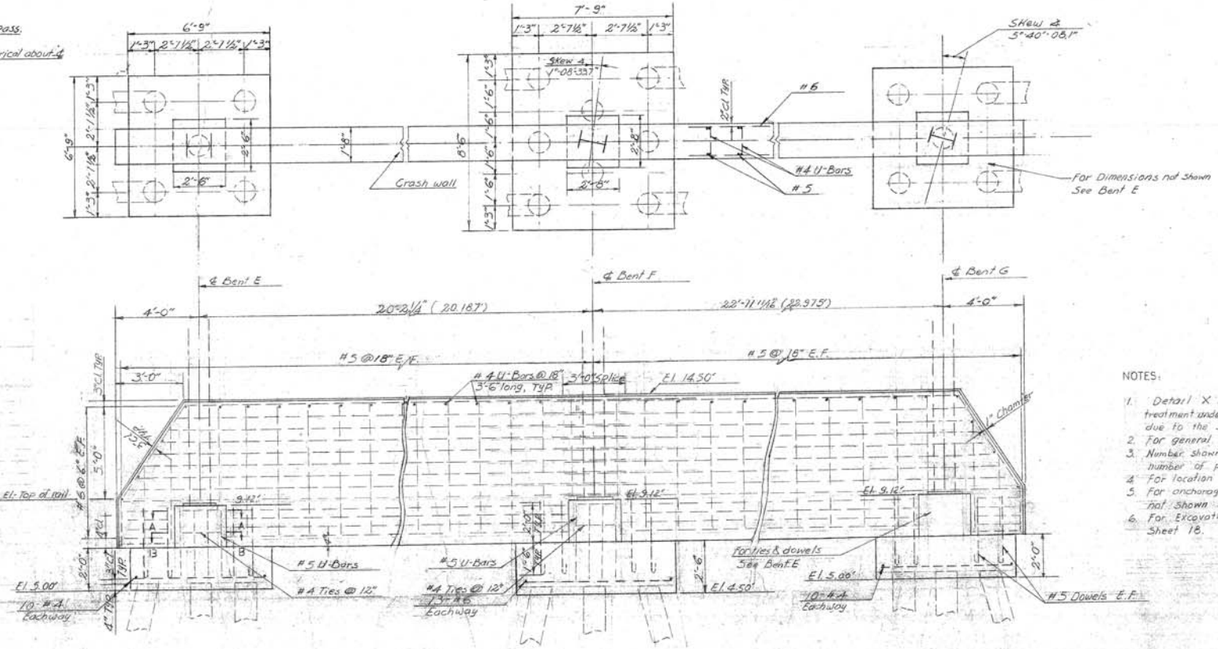


SECTION C-C
SPALLED TIE REPAIR
Scale 1" = 1'-0"



Note:
Bottom of Footing Elevations in Bent 18 shall be checked with Bottom of Transite Duct Beam and may be lowered if found necessary.
In removing Existing Concrete Cap and driving Timber Piles, care must be taken not to damage Transite Ducts & Footings built by N.Y.C.T.A.

BENT 18 FOUNDATION
HALF PLAN & ELEVATION
Scale 3/8" = 1'-0"



BENT 19 FOUNDATION
PLAN & ELEVATION
Scale 3/8" = 1'-0"

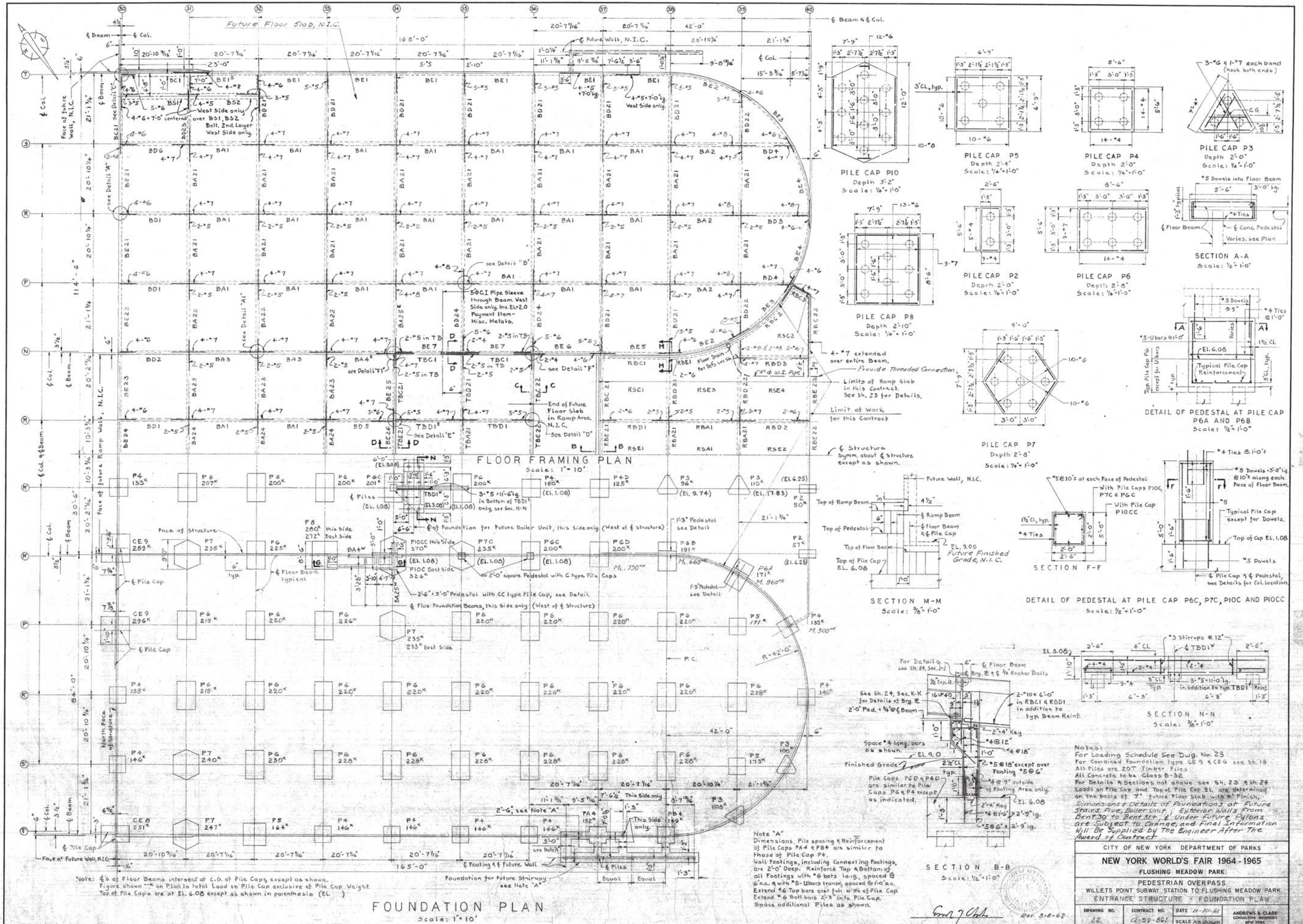
NOTES:

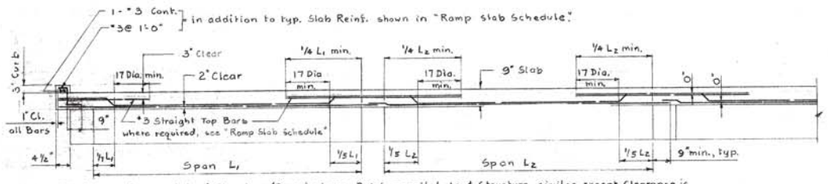
1. Detail "X" indicates the special treatment under existing pile foundation due to the settlement of ground.
2. For general notes, see sheet 18.
3. Number shown after Type E indicates number of piles in the footing.
4. For location of footing, See sheet 18.
5. For anchorage details of Base plates not shown see Dwg. #15
6. For Excavation Payment Lines See Sheet 18.

In Charge of: [illegible]
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]

Rev. 8-8-62

CITY OF NEW YORK DEPARTMENT OF PARKS			
NEW YORK WORLD'S FAIR 1964-1965			
FLUSHING MEADOW PARK			
PEDESTRIAN OVERPASS WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK FOUNDATION DETAILS - TYPE E, BENT 18 & BENT 19			
DRAWING NO. 20	CONTRACT NO. Q-29-861	DATE 11-10-61 SCALE AS Shown	ANDREWS & CLARK CONSULTING ENGINEERS N.Y. NEW YORK

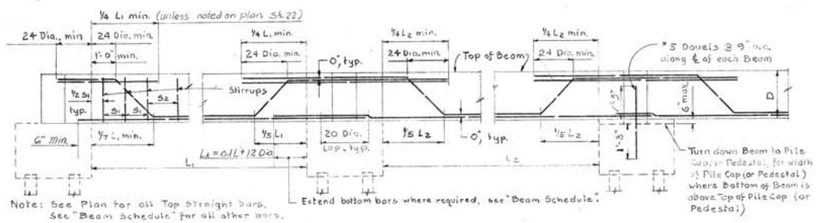

FLOOR FRAMING PLAN
Scale: 1" = 10'
FOUNDATION PLAN
Scale: 1" = 10'
PILE CAP P10
Depth 3'-2"
Scale: 1/4" = 1'-0"
PILE CAP P5
Depth 2'-4"
Scale: 1/4" = 1'-0"
PILE CAP P4
Depth 2'-0"
Scale: 1/4" = 1'-0"
PILE CAP P3
Depth 2'-0"
Scale: 1/4" = 1'-0"
PILE CAP P8
Depth 2'-10"
Scale: 1/4" = 1'-0"
PILE CAP P2
Depth 2'-0"
Scale: 1/4" = 1'-0"
PILE CAP P6
Depth: 2'-8"
Scale: 1/4" = 1'-0"
SECTION A-A
Scale: 1/2" = 1'-0"
PILE CAP P7
Depth 2'-8"
Scale: 1/4" = 1'-0"
DETAIL OF PEDESTAL AT PILE CAP P6A AND P6B
Scale: 1/2" = 1'-0"
SECTION M-M
Scale: 3/8" = 1'-0"
SECTION F-F
DETAIL OF PEDESTAL AT PILE CAP P6C, P7C, P10C AND P10CC
Scale: 1/2" = 1'-0"
SECTION N-N
Scale: 3/8" = 1'-0"
SECTION B-B
Scale: 1/2" = 1'-0"
Future Floor Slab, N.I.C.
Future Walk, N.I.C.
Face of future Walk, N.I.C.
Face of future Ramp Walk, N.I.C.
Face of Future Wall, N.I.C.
Face of Structure
North Face of Structure
End of Future Floor Slab in Ramp Area, N.I.C.
Limit of Work for this Contract
Limits of Ramp Slab in this Contract. See Sh. 23 for Details.
Provide Threaded Connection
Foundation for Future Stairway see Note "A"
Note: ℄'s of Floor Beams intersect at C.O. of Pile Caps, except as shown.
Figure shown "K" on Plan is total Load on Pile Cap exclusive of Pile Cap Weight
Top of Pile Caps are at El. 6.08 except as shown in parenthesis (El.)
Note "A"
Dimensions, Pile spacing & Reinforcement of Pile Caps P4A & P8A are similar to those of Pile Cap P4
Wall footings, including Connecting Footings, are 2'-0" Deep. Reinforce Top & Bottom of all Footings with #6 bars long, spaced @ 6" o.c. & with #5 - Ubars transv. spaced @ 1'-0" o.c.
Extend #6 Top bars over full width of Pile Cap
Extend #6 Bottom bars 2'-3" into Pile Cap.
Space additional Piles as shown.
Notes:
For Loading Schedule See Dwg. No. 23
For Combined Foundation Type CE9 & CE6 see Sh. 18
All Piles are 20T Timber Piles
All Concrete to be Class B-3Z
For Details & Sections not shown see Sh. 23 & Sh. 24
Loads on Pile Cap and Top of Pile Cap El. are determined on the basis of 7" future Floor Slab with 4" Finish.
Dimensions & Details of Foundations at Future Stairs, Flue, Boiler Unit, Exterior Walls from Bent 30 to Bent 31+, & Under Future Pylons are Subject to Change, and Final Information Will Be Supplied by the Engineer After the Award of Contract
CITY OF NEW YORK DEPARTMENT OF PARKS
NEW YORK WORLD'S FAIR 1964 - 1965
FLUSHING MEADOW PARK
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
ENTRANCE STRUCTURE - FOUNDATION PLAN
DRAWING NO. 22
CONTRACT NO. G-59-861
DATE 11-10-61
SCALE As shown
ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
Rev. 3-8-62



Note: Reinforcement normal to ℄ Structure (Ramp) shown. Reinf. parallel to ℄ Structure similar except Clearance is 1½" at Bottom and 3½" at Top.

TYPICAL RAMP SLAB DETAILS

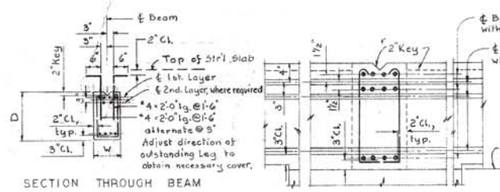
Not to Scale



Note: See Plan for all Top Straight Bars. See "Beam Schedule" for all other bars.

TYPICAL BEAM DETAILS

Not to Scale



SECTION THROUGH BEAM

not to Scale

TYPICAL DETAIL AT BEAM INTERSECTION

Scale: 1" = 1'-0"

X N indicates Reinforcement Normal to ℄ Structure (Ramp)
P indicates Reinforcement Parallel to ℄ Structure (Ramp)

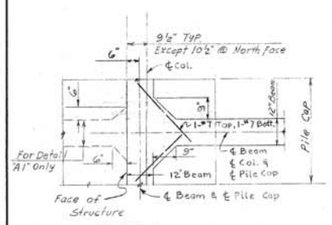
RAMP SLAB SCHEDULE

Slab Mark	Direction X	Bar Size and Spacing: Truss	Straight, Bott.	Straight, Top	Remarks
RSA1	N	#4@16"	#4@16"	#3@12" over RBA1	
	P	#4@16"	#4@16"	#3@12" over co. RBA21	
RSC1	N	#4@12"	#4@18"		
	P	#4@12"	#4@18"		
RSC2	N	#4@12"	#4@18"	#3@12" over RBD3	
	P	#4@12"	#4@18"		
RSE1	N	#4@12"	#4@15"	#3@12" over RBD1	
	P	#4@16"	#4@18"		
RSE2	N	#4@12"	#4@15"	#3@12" over RBD2	
	P	#4@16"	#4@18"		
RSE3	N	#4@16"	#4@18"		
	P	#4@12"	#4@15"	#3@12" over RBD21	
RSE4	N	#4@12"	#4@15"		
	P	#4@16"	#4@18"	#3@9" over RBD22	

** Stirrups to Extend Full length of Beam @ 12" Max. Spacing where not indicated in Table.

BEAM SCHEDULE

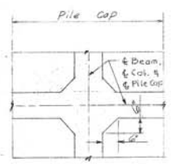
Beam Mark	Size W	Size D	Straight No.	Straight Size	Truss No.	Truss Size	Stirrups** No.	Stirrups Size	Stirrups Spacing each End	Remarks
BA1	12"	24"	2	#8	2	#7	22	#3	4@6, 3@7½, 4@9	Extend By La into BD1, BD5 & BD6
BA2	12"	24"	2	#8	2	#7	22	#3	4@6, 3@7½, 4@9	Extend By La into BD3, BA4
BA3	12"	24"	1, 1	#8sg	1, 1	#8sg	30	#3	6@4, 3@6, 6@9	Extend By La into BD2
BC1	24"	24"	4	#8	4	#8	18	#3	3@8, 6@9½	
BD1	12"	24"	2	#9	2	#9	28	#3	6@4½, 3@7, 5@9	
BD2	12"	24"	2	#8	2	#9	40	#3	6@3, 6@4, 4@5½, 4@8	
BD3	12"	24"	2	#9	2	#9	28	#3	6@4½, 3@7, 5@9	
BD4	12"	24"	2	#7	2	#8	24	#3	6@4½, 3@7, 3@9	
BD5	18"	16"	2	#8	2	#7	16	#3	8@9	West Side only Hook #8 bars into BC21 & extend by La over BD23
BD5	12"	24"	2	#9	2	#9	28	#3	6@4½, 3@7, 5@9	
BE1	12"	24"	2	#4	2	#9	16	#3	8@9	
BE2	12"	24"	2	#8	2	#8	34	#5	6@3, 4@6, 7@9	
BE3	12"	23	2	#8	2	#8	34	#5	6@3, 4@6, 7@9	
BE4	12"	23"	2	#8	2	#8	34	#5	6@3, 4@6, 7@9	
BE5	12"	24"	2	#8	2	#8	30	#3	6@3½, 4@6, 5@9	
BE2	18"	16"	2	#8	2	#7	16	#3	8@9	West Side only Hook #8 bars into BD21 & extend by La over BD23
BA21	12"	23"	2	#6	2	#8	22	#3	4@6, 3@7½, 4@9	Extend By La into Beams BA25, BD24
BA22	12"	23"	2	#6	2	#8	22	#3	4@6, 3@7½, 4@9	
BA23	12"	23"	2	#6	2	#8	22	#3	4@6, 3@7½, 4@9	
BC21	12"	23"	2	#7	2	#9	18	#3	3@8, 6@9½	
—	—	—	—	—	—	—	—	—	—	
BD21	12"	23"	2	9	2	#8	28	#3	6@4½, 3@7, 5@9	
BD22	12"	23"	2	#8	2	#8	14	#3	4@6, 3@9	
BE21	12"	23"	2	#4	2	#9	16	#3	8@9	
BE22	12"	23"	2	#4	2	#9	16	#3	8@9	
BE23	12"	23"	2	#4	2	#9	16	#3	8@9	
BE24	12"	23"	2	#4	2	#9	16	#3	8@9	
RSA1	12"	24"	2	#6	2	#9	22	#3	4@6, 3@7½, 4@9	
RBA21	12"	23"	2	#4	2	#9	22	#3	4@6, 3@7½, 4@9	
RBC1	12"	24"	2	#4	2	#8	—	—		
RBC2	12"	24"	2	#4	2	#8	28	#3	14@9	
RBC3	12"	23"	—	—	2	#7	—	—		
RBC21	12"	23"	2	#5	2	#8	—	—		
RBC22	12"	23	—	—	2	#5	—	—		
RBD1	12"	24"	2	#7	2	#9	16	#3	8@9½	
RBD2	12"	24"	2	#7	2	#9	16	#3	8@9½	
RBD3	12"	24"	2	#7	2	#9	16	#3	8@9½	Extend all bars into RBD22 & hook
RBD21	12"	23"	2	#7	2	#9	16	#3	8@9½	
RBD22	12"	23"	2	#9	2	#9	24	#3	18@3, 4@5, 2@10	Near End from Beam RBD3
							9	#3	4@5, 5@10	For End from Beam RBD3
RBE1	12"	24"	2	#4	2	#8	28	#3	14@9.	
RBE21	12"	23"	2	#4	2	#8	—	—		
RBE22	12"	23"	2	#4	2	#8	—	—		
RBE23	12"	23"	2	#4	2	#8	—	—		
BA4	12"	24"	2	#9	2	#9	40	#3	6@3, 6@4, 4@5½, 4@8	
BD6	12"	24"	2	#9	2	#9	28	#3	6@4½, 3@7, 5@9	
BE6	12"	24"	2	#9	2	#7	—	—		
BE7	12"	24"	2	#5	2	#9	16	#3	8@9	Extend By La into BA4
BA24	12"	23"	2	#6	2	#8	16	#3	8@9.	
BA25	12"	23"	2	#8	2	#8	28	#3	6@4½, 3@7, 5@9	
BD25	12"	23"	2	#9	2	#8	28	#3	6@4½, 3@7, 5@9	
BD24	12"	23"	2	#9	2	#9	28	#3	6@4½, 3@7, 5@9	
BE25	12"	23"	2	#5	2	#8	16	#3	8@9.	Extend By La into BA25
BE26	12"	23"	2	#5	2	#8	16	#3	8@9.	
TBD1	12"	24"	2	#5	2	#9	18	#3	3@8, 6@9½	
TBD21	12"	24"	2	#9	2	#9	28	#3	6@4½, 3@7, 5@9	Extend By La into Beam TBD1
TBC21	12"	23"	2	#6	2	#8	22	#3	4@6, 3@7½, 4@9	
TBC21	12"	23"	2	#5	2	#9	18	#3	3@8, 6@9	
TBD21	12"	23"	2	#9	2	#8	28	#3	6@4½, 3@7, 5@9	
TBE21	12"	23"	2	#4	2	#8	16	#3	8@9.	
TBE22	12"	23"	2	#4	2	#9	16	#3	8@9.	
TBC22	12"	23"	2	#7	2	#9	22	#3	5@6, 6@9	



DETAIL A

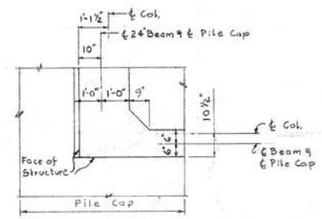
Note: Detail A1 similar except as noted.

Scale: ½" = 1'-0"



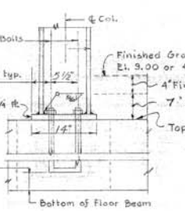
DETAIL B

Scale ½" = 1'-0"



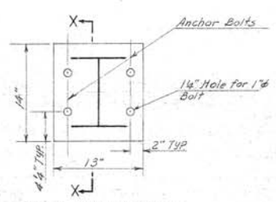
DETAIL C

Scale ½" = 1'-0"



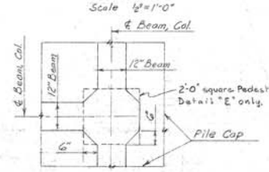
SECTION X-X

Scale 1" = 1'-0"



TYPICAL COLUMN ANCHORAGE DETAIL

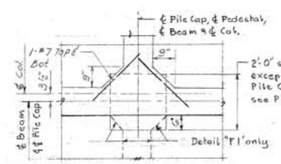
Scale 1½" = 1'-0"



DETAIL D

Note: Detail "E" similar, except as noted.

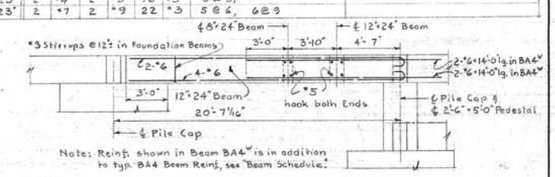
Scale: ½" = 1'-0"



DETAIL F

Note: "Detail F1" similar, except as noted

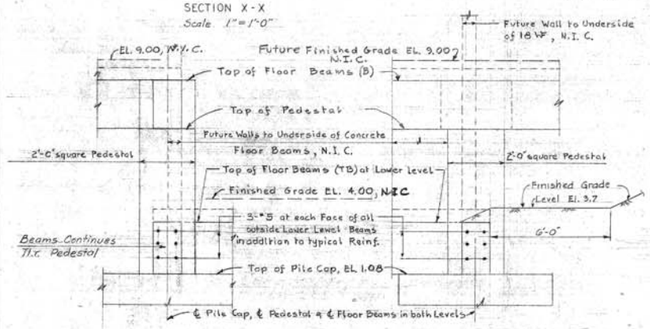
Scale: ½" = 1'-0"



Note: Reinf. shown in Beam BA4' is in addition to typ. BA4 Beam Reinf., see "Beam Schedule."

SECTION G-G

Scale: ½" = 1'-0"

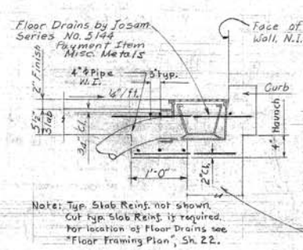


SECTION D-D

Scale ½" = 1'-0"

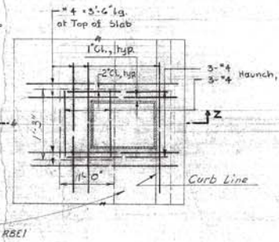
SECTION C-C

Scale ½" = 1'-0"



Note: Typ. Slab Reinf. not shown. Cut typ. Slab Reinf. if required for location of Floor Drains see "Floor Framing Plan", Sh. 22.

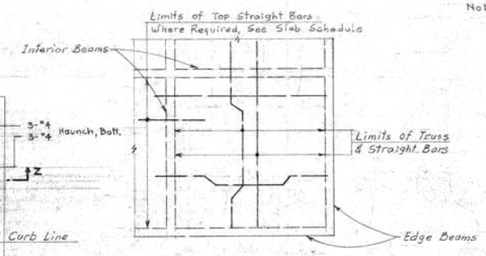
SECTION Z-Z



PLAN

FLOOR DRAINS INSTALLATION DETAILS

Not to Scale



TYPICAL RAMP SLAB REINFORCING DETAILS

Not to Scale

LOADING SCHEDULE	
FUTURE FLOOR SLAB	
4" Cinder Conc. Fill & Finish	50 psf
7" Concrete Slab	88 psf
Partitions	20 psf
Live Load	100 psf
Total	258 psf

Rev 3-8-62

CITY OF NEW YORK DEPARTMENT OF PARKS

NEW YORK WORLD'S FAIR 1964-1965

FLUSHING MEADOW PARK

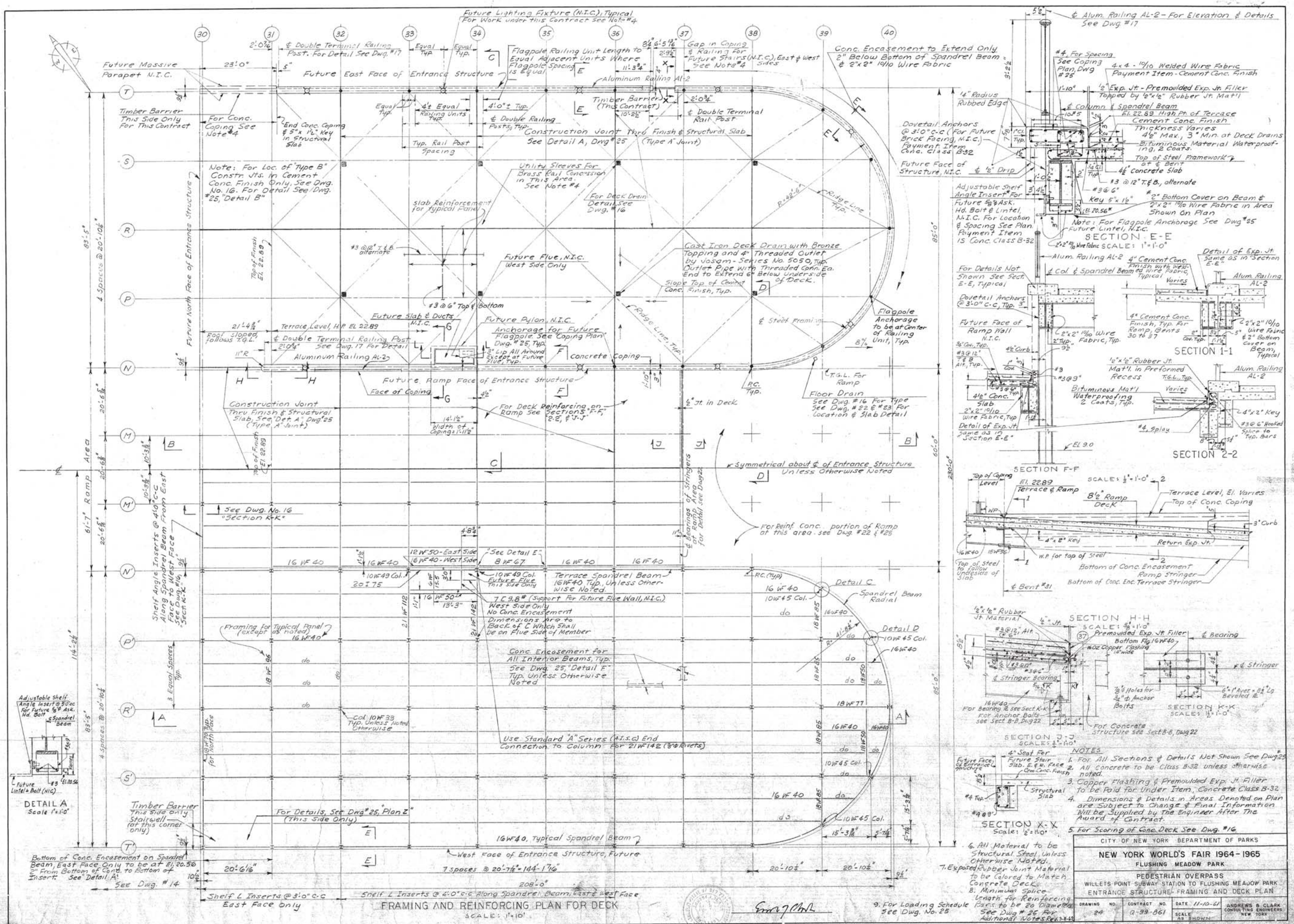
PEDESTRIAN OVERPASS

WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK

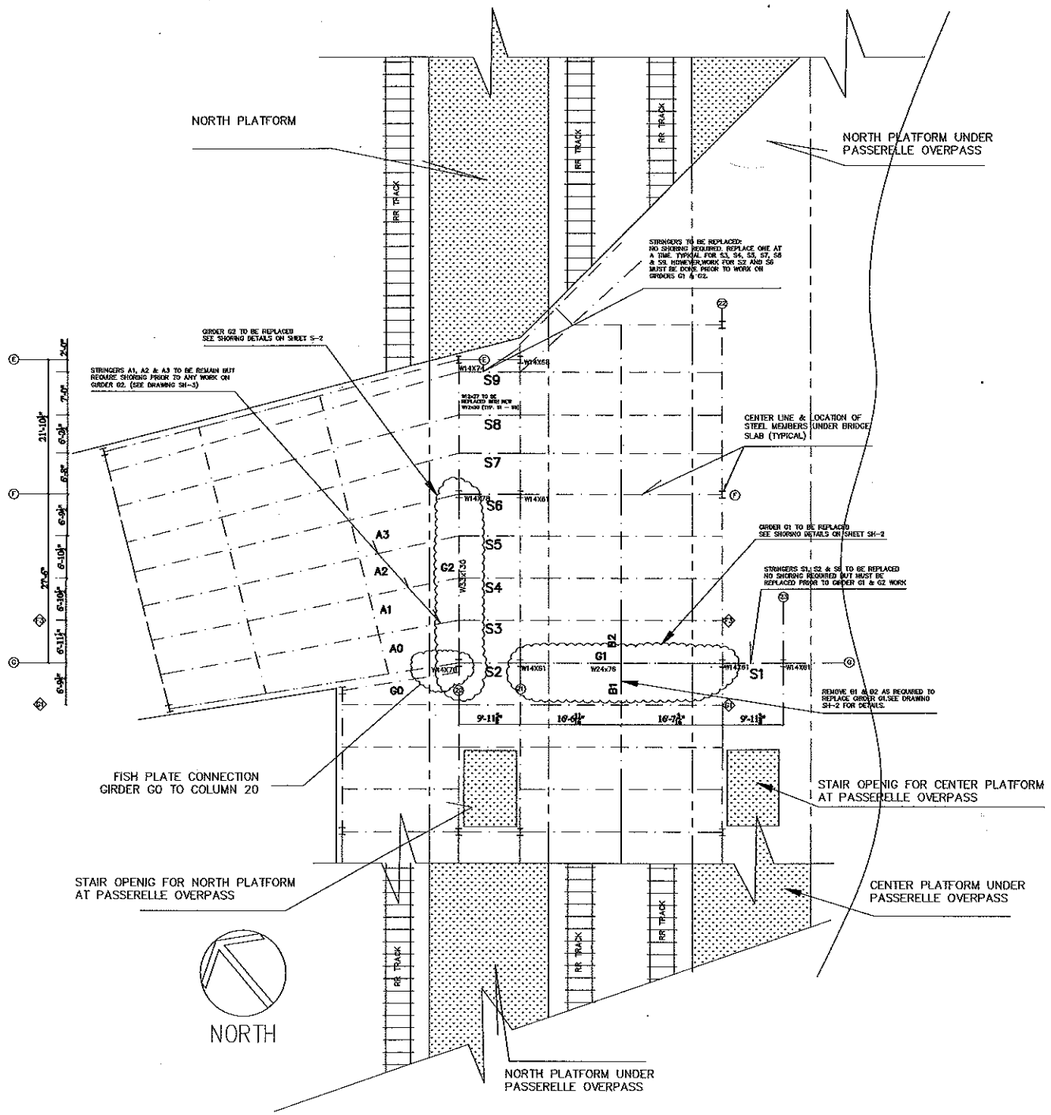
ENTRANCE STRUCTURE - BEAM DETAILS AND SCHEDULES

DRAWING NO.	CONTRACT NO.	DATE	ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK
23	Q 99-861	11-10-61 SCALE As shown	

In Charge of: [illegible]
Made by: [illegible]
Traced by: [illegible]
Checked by: [illegible]


Future Lighting Fixture (N.I.C.), Typical For Work under this Contract See Note #4
Future Massive Parapet N.I.C.
Future East Face of Entrance Structure
Aluminum Railing AL-2
Flagpole Railing Unit Length to Equal Adjacent Units Where Flagpole Spacing is Equal
Gap in Coping & Railing For Future Stairs (N.I.C.), East & West Sides
Conc. Encasement to Extend Only 2" Below Bottom of Spandrel Beam & 2"x2" 10/10 Wire Fabric
Timber Barrier (This Contract)
Timber Barrier This Side Only For This Contract
For Conc. Coping See Note #4
End Conc. Coping & 5"x 1½" Key in Structural Slab
Construction Joint Thru Finish & Structural Slab See Detail A, Dwg #25 (Type A Joint)
Note: For Loc. of Type B Constr. Jts. in Cement Conc. Finish Only, See Dwg. No. 16. For Detail See Dwg. #25, "Detail B"
Utility Sleeves For Brass Rail Concession in This Area. See Note #4
For Deck Drain Detail, See Dwg. #16
Slab Reinforcement for Typical Panel
Future Flue, N.I.C. West Side Only
Cast Iron Deck Drain with Bronze Topping and 4" Threaded Outlet by Josam - Series No. 5050, Typ. Outlet Pipe with Threaded Conn. Ea. End to Extend 6" Below Underside of Deck.
Slope Top of Cement Conc. Finish, Typ.
Future Slab & Ducts N.I.C.
Future Pylon, N.I.C.
Anchorage for Future Flagpole See Coping Plan Dwg. #25, Typ.
Terrace Level, H.P. EL 22.89
Roof sloped, follows T.G.L.
Aluminum Railing AL-2
Concrete Coping
Future Ramp Face of Entrance Structure
Face of Coping
Construction Joint Thru Finish & Structural Slab, See Det. A, Dwg #25 (Type A Joint)
For Deck Reinforcing on Ramp See Sections F-F, 2-2, & J-J
Flagpole Anchorage to be at Center of Railing Unit, Typ.
Floor Drain See Dwg. #16 For Type See Dwg. #22 & #23 For Location & Slab Detail
Symmetrical about ℄ of Entrance Structure Unless Otherwise Noted
For Reinf. Conc. portion of Ramp of this area, see Dwg. #22 & #25
See Dwg. No. 16 "Section K-K"
Shelf Angle Inserts @ 4'0" c-c Along Spandrel Beam From East Face to West Face "Sect. K-K"
Terrace Spandrel Beam 16WF40 Typ. Unless Otherwise Noted.
7 C 9.8# (Support For Future Flue Wall, N.I.C.) West Side Only No Conc. Encasement Dimensions Are to Back of C Which Shall be on Flue Side of Member
Conc. Encasement For All Interior Beams, Typ. See Dwg. 25, "Detail F" Typ. Unless Otherwise Noted
Framing for Typical Panel (except as noted)
Use Standard "A" Series (A.I.S.C.) End Connection to Column For 21WF142 (8 Bolts)
Adjustable Shelf Angle Insert @ 3'0" c-c For Future 5/8" Ask. Hd. Bolt & Spandrel Beam
Future Lintel + Bolt (N.I.C.)
DETAIL A
Scale 1"=1'-0"
Timber Barrier This Side Only Stairwell (for this corner only)
For Details, See Dwg. #25, "Plan Z" (This Side Only)
16WF40, Typical Spandrel Beam
West Face of Entrance Structure, Future
7 Spaces @ 20'-7 7/16" = 144'-1 7/16"
208'-0"
Shelf L Inserts @ 3'-0" c-c East Face Only
Shelf L Inserts @ 4'-0" c-c Along Spandrel Beam, East & West Face
Bottom of Conc. Encasement on Spandrel Beam, East Face Only to be at El. 20.56 5" from Bottom of Conc. to Bottom of Insert. See Detail A
See Dwg. #14
FRAMING AND REINFORCING PLAN FOR DECK
SCALE: 1"=10'
℄ Alum. Railing AL-2 - For Elevation & Details See Dwg. #17
4 x 4 - 10/10 Welded Wire Fabric Payment Item - Cement Conc. Finish
½" Exp. Jt. - Premoulded Exp. Jt. Filler Topped by ⅛"x⅛" Rubber Jt. Mat'l
℄ Column & Spandrel Beam
El. 22.89 High Pt. of Terrace Cement Conc. Finish Thickness Varies 4⅛" Max., 3" Min. at Deck Drains
Bituminous Material Waterproofing, 2 Coats.
Top of Steel Framework at ℄ Bent
4½" Concrete Slab
#3 @ 12" T. & B., alternate
Key 5"x 1½"
2" Bottom Cover on Beam & 2"x2" 10/10 Wire Fabric in Area Shown on Plan
Note: For Flagpole Anchorage See Dwg. #25
Future Face of Structure, N.I.C.
Dovetail Anchors @ 3'-0" c-c (For Future Brick Facing, N.I.C.) Payment Item Conc. Class B-32
Adjustable Shelf Angle Insert For Future 5/8" Ask. Hd. Bolt & Lintel, N.I.C. For Location & Spacing See Plan Payment Item IS Conc. Class B-32
SECTION E-E
SCALE: 1"=1'-0"
Alum. Railing AL-2
For Details Not Shown See Sect. E-E, Typical
Dovetail Anchors @ 3'-0" c-c, Typ.
Future Face of Ramp Wall N.I.C.
Detail of Exp. Jt. Same as in Section E-E
4" Cement Conc. Finish with Wire Fabric, Typical
SECTION 1-1
SECTION 2-2
Bituminous Mat'l Waterproofing 2 Coats, Typ.
Detail of Exp. Jt. Same as in Section E-E
SECTION F-F
SCALE: ½"=1'-0"
Terrace Level, El. Varies
Top of Conc. Coping
Return Exp. Jt.
Bottom of Conc. Encasement
Ramp Stringer
Bottom of Conc. Enc. Terrace Stringer
SECTION H-H
SCALE: ½"=1'-0"
Premoulded Exp. Jt. Filler
16 oz Copper Flashing 18" wide
℄ Bearing
℄ Stringer
SECTION K-K
SCALE: ½"=1'-0"
SECTION J-J
SCALE: ½"=1'-0"
SECTION X-X
Scale: ½"=1'-0"
Detail C
Spandrel Beam Radial
Detail D
Conc. Encasement For All Interior Beams, Typ. See Dwg. 25, "Detail F" Typ. Unless Otherwise Noted
Col. 10WF33 Typ. Unless Noted Otherwise
Use Standard "A" Series (A.I.S.C.) End Connection to Column For 21WF142 (8 Bolts)
For Details, See Dwg. #25, "Plan Z" (This Side Only)
NOTES
1. For All Sections & Details Not Shown See Dwg. #25
2. All Concrete to be Class B-32 unless otherwise noted.
3. Copper Flashing & Premoulded Exp. Jt. Filler to be Paid for Under Item, Concrete Class B-32
4. Dimensions & Details in Areas Denoted on Plan are Subject to Change & Final Information Will be Supplied by the Engineer After The Award of Contract.
5. For Scoring of Conc. Deck See Dwg. #16
6. All Material to be Structural Steel Unless Otherwise Noted.
7. Exposed Rubber Joint Material to be Colored to Match Concrete Deck.
8. Minimum Splice Length for Reinforcing Bars to be 20 Diameters See Dwg. #25 For Additional Notes
9. For Loading Schedule See Dwg. No. 25
CITY OF NEW YORK DEPARTMENT OF PARKS
NEW YORK WORLD'S FAIR 1964-1965
FLUSHING MEADOW PARK
PEDESTRIAN OVERPASS
WILLETS POINT SUBWAY STATION TO FLUSHING MEADOW PARK
ENTRANCE STRUCTURE - FRAMING AND DECK PLAN
DRAWING NO. 24
CONTRACT NO. Q-99-861
DATE 11-10-61
SCALE AS SHOWN
ANDREWS & CLARK CONSULTING ENGINEERS NEW YORK

REVISIONS			
REV	DRAWN BY	DESCRIPTION	DATE
A	J.SEEMAN	RELEASE	12/29/08



LEGEND:

(#) COLUMN LINE DESIGNATIONS AS PER ORIGINAL WORLD'S FAIR STEEL DRAWINGS

◇# PCS SUBCOLUMN LINE DESIGNATION

SHORING PLAN & LOCATION

TEMPORARY SHORING FOR THE EMERGENCY REPLACEMENT OF STEEL MEMBERS G1, G2, AND S1 THROUGH S9 @ PASSERELLE OVERPASS, QUEENS, NY

GENERAL NOTES:
- SEE STEEL SHOP DRAWING FOR NEW MEMBER & CONNECTION DETAILS
- ALL STEEL WORK & CONNECTIONS TO FOLLOW APPLICABLE CODES. ALL WORK TO BE PAINTED AS PER NYCDPR CONTRACT SPECIFICATIONS
- FABRICATE REPLACEMENT STEEL MEMBERS AND CONNECTIONS ACCORDING TO VERIFIED FIELD DIMENSIONS AND APPROVED SHOPDRAWINGS.

CONCRETE REMOVAL NOTES:
- NO REMOVALS OF CONCRETE SLAB ARE ANTICIPATED.
- NYC DPR ENGINEER TO INSPECT AND DETERMINE IF ANY SLABS REQUIRE REPLACEMENT. ANY CONCRETE REPLACEMENT MUST BE DONE AFTER ALL STEEL WORK IS COMPLETED.

SHORING NOTES & SEQUENCE OF WORK
(WORK MUST BE PERFORMED IN THE SEQUENCE SHOWN BELOW):

1. STRINGERS S1 AND S2 MUST BE REPLACED PRIOR TO ANY OTHER WORK. NO SHORING REQUIRED.
2. STRINGERS B1 & B2 NEED TO BE REMOVED FOR REPLACEMENT OF GIRDER G1. REINSTALL STRINGERS AFTER G1 IS REPLACED. (SEE SHORING PLAN ON DWG SH-2)
3. GIRDER G1 REQUIRES SHORING PRIOR TO REPLACEMENT (SEE SHORING PLAN ON DWG SH-2)
4. STRINGERS S7, S8 &S9 ARE TO BE REPLACED PRIOR TO GIRDER G2 WORK. NO SHORING IS REQUIRED FOR THIS WORK.
5. STRINGERS S3, S4 & S5 NEED TO BE REMOVED PRIOR TO REPLACEMENT OF GIRDER G2. SHORING IS REQUIRED (SEE DWG SH-3)
6. STRINGERS A1, A2 & A3 REQUIRE SHORING PRIOR TO REPLACEMENT OF GIRDER G2. (SEE DWG SH-3)
7. GIRDER G2 REQUIRES SHORING PRIOR TO REPLACEMENT (SEE NOTES 5 & 6, AND DWG SH-3)
8. BEAMS OR MEMBERS NOT LABELED DO NOT AFFECT WORK, DO NOT NEED SHORING, AND ARE TO REMAIN IN EXISTING CONDITIONS.
9. ALL WORK TO COMPLY WITH SPECIFICATIONS & APPLICABLE CODES

AS BUILT

"AS BUILT"

Padilla Construction Services
299 Main Street, Westbury, NY, 11590
Emergency Construction of Expansion Joints, Structural Steel, and Concrete at the Passarelle Overpass Located in Flushing Meadows-Corona Park, Borough of Queens
Contract: Q099-708M
Date: 12/29/08
I certify that this "As Built" drawing is complete and accurate

Jefferson Sa

CONSTRUCTED FOR
THE CITY OF NEW YORK
DEPARTMENT OF PARKS & RECREATION

DRAWING TITLE: SHORING PLAN

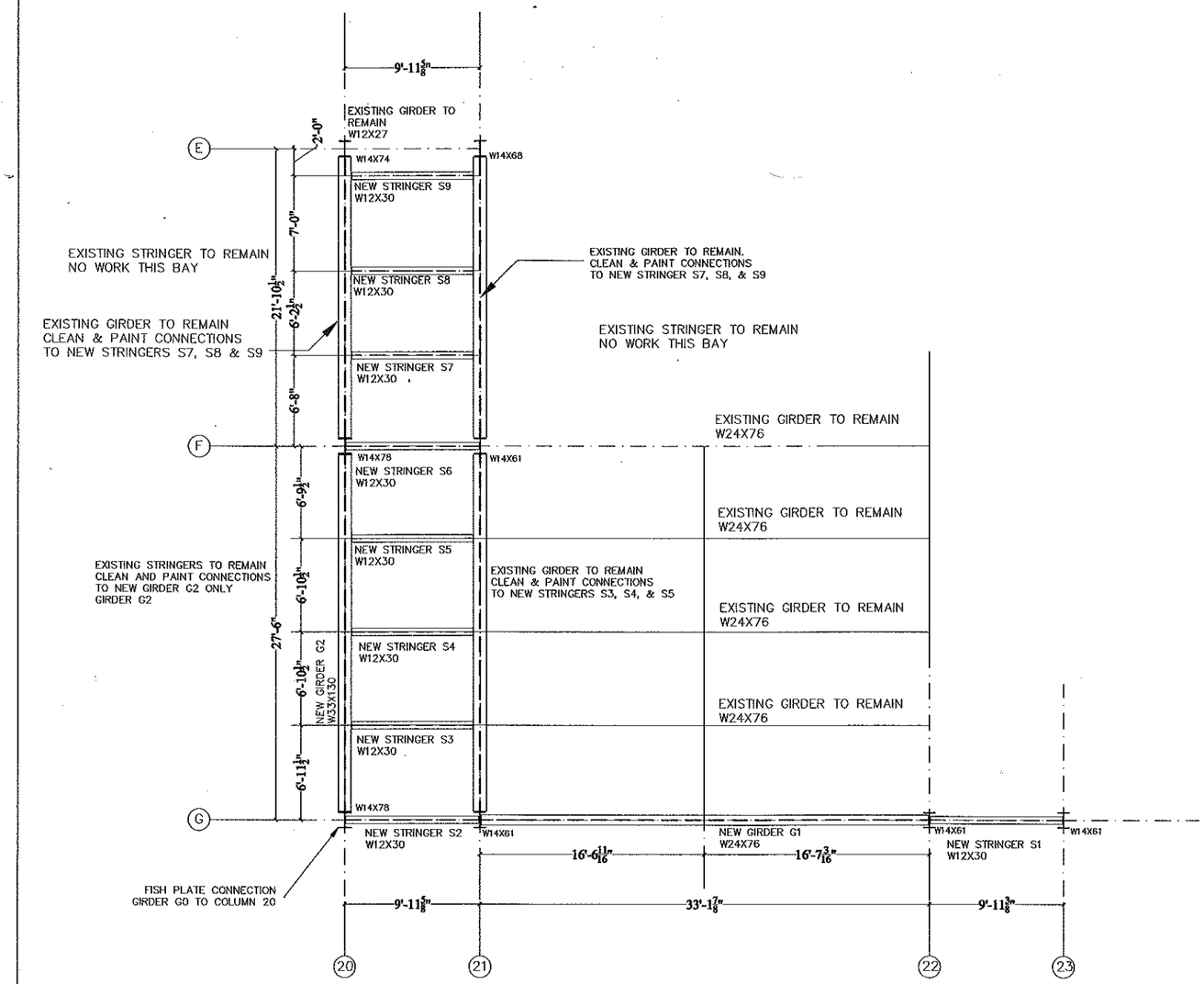
CONSTRUCTED BY: **PADILLA CONSTRUCTION SERVICES, INC**
299 MAIN STREET, WESTBURY, 11590
(516) 338-8848

AS BUILT

PROJECT: THE EMERGENCY RECONSTRUCTION OF EXPANSION JOINTS, STRUCTURAL STEEL, AND CONCRETE AT THE PASSARELLE OVERPASS LOCATED IN FLUSHING MEADOWS-CORONA PARK, BOROUGH OF QUEENS

DRAWN BY	J. SEEMAN	SIZE	CONTRACT NO.	REV
CHECKED BY	J. SA	D	Q099-708M	A
		SCALE NTS	DRAWING DATE 12/29/08	SHEET 1 OF 7

REVISIONS			
REV	DRAWN BY	DESCRIPTION	DATE
A	J.SEEMAN	RELEASE	12/29/08



STRUCTURAL NOTES:

ALL NEW GIRDERS, STRINGERS, CONNECTIONS, PLATES, BOLTS, WELD SIZES ARE IDENTICAL TO THE ORIGINAL DESIGN. HENCE, THE STRUCTURAL MEMBERS REPLACED ARE RATED AND HAVE THE SAME CAPACITY AS THE ORIGINAL MEMBERS AND DESIGN.

STRUCTURAL AND STEEL NOTES:

1– TO MAINTAIN STRENGTH OF THE EXISTING CONNECTIONS, THE SAME SIZE, QUANTITY AND LOCATIONS OF EXISTING FASTENERS AND WELDS.
2– ALL BOLTS TO BE TORQUE CONTROL HIGH STRENGHT A325 BOLTS
3– ALL WELDS ARE 5/16" FILLET WELDS, E70XX ELECTRODES, U.O.N.
4– ALL STEEL TO BE A36 GRADE AND AS PER AISC AND NYC BUILDING CODE REQTS.
5– FABRICATION AND ERECTION TO BE AS PER CURRENT AISC CODES AND REQTS.
6– SHORING IS REQUIRED FOR REMOVAL & REPLACEMENT OF MEMBERS G1 AND G2. FOLLOW SEQUENCE AND DETAILS SHOWN ON SHORING & SUPPORT PLANS. FOR OTHER MEMBERS FOLLOW ERECTION SEQUENCE SHOWN BELOW.
7– NEW STEEL TO BE ERECTED ½" LOWER. TOP OF FLANGE TO BE UNDERSIDE OF DECK TO BE STEEL WELDED AND GROUTED TO OBTAIN FULL BEARING SUPPORT OF THE EXISTING DECK.

CONSTRUCTION & ERECTION SEQUENCE:

1– Remove and replace stringers S1 & S2
2– Install top side shoring support
3– Core slab & install under slab support for G1 replacement
4– Remove and replace girder G1
5– Remove and replace stringer S6
6– Shore stringers east of G2 and replace girder G2
(NOTE: TEMPORARILY RELOCATE 2 ELECTRIC CONDUITS & RESTORE AFTER)
7– Remove and replace stringers S3, S4 and S5
8– Remove and replace stringers S7, S8 and S9
9– Clean and paint connections to remain and girder G3
10–Rehab expansion joints above slab

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Contract: Q099-708M
Date: 12/29/08
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Jefferson Sa

CONSTRUCTED FOR
THE CITY OF NEW YORK
DEPARTMENT OF PARKS & RECREATION

DRAWING TITLE: ERECTION & LAYOUT PLAN

AS BUILT

CONSTRUCTED BY
PADILLA CONSTRUCTION SERVICES, INC
299 MAIN STREET, WESTBURY, 11590
(516) 338-6848

PROJECT: THE EMERGENCY RECONSTRUCTION OF EXPANSION JOINTS, STRUCTURAL STEEL, AND CONCRETE AT THE PASSARELLE OVERPASS LOCATED IN FLUSHING MEADOWS–CORONA PARK, BOROUGH OF QUEENS

DRAWN BY	J. SEEMAN	SIZE	D	CONTRACT NO.	Q099–708M	REV	A
CHECKED BY	J. SA	SCALE	NTS	DRAWING DATE	12/29/08	SHEET	2 OF 7

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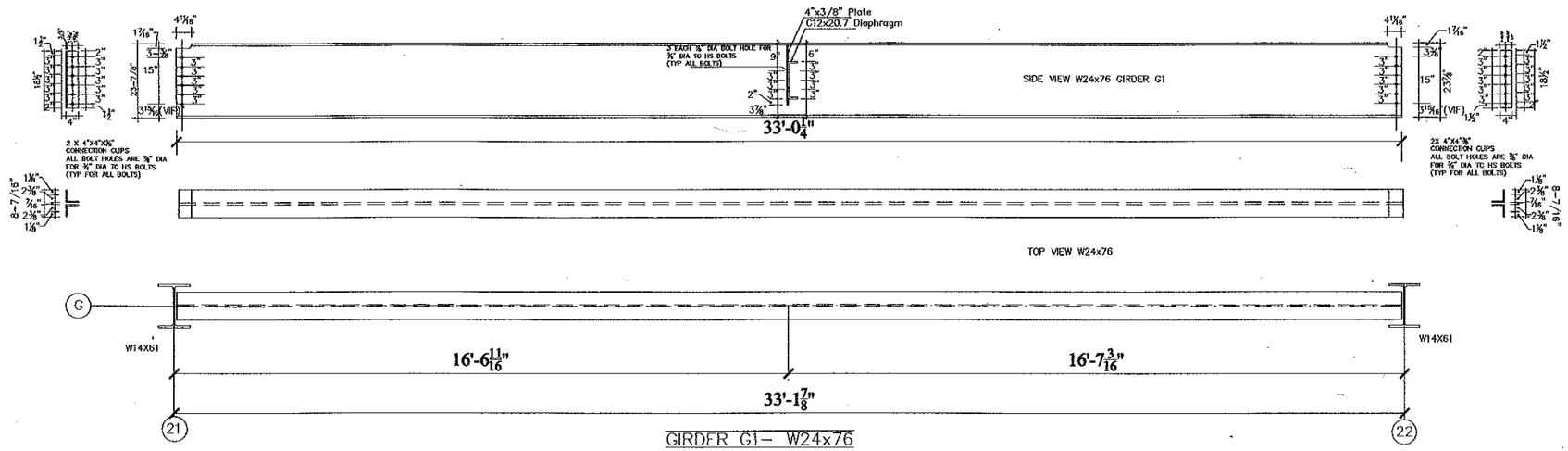
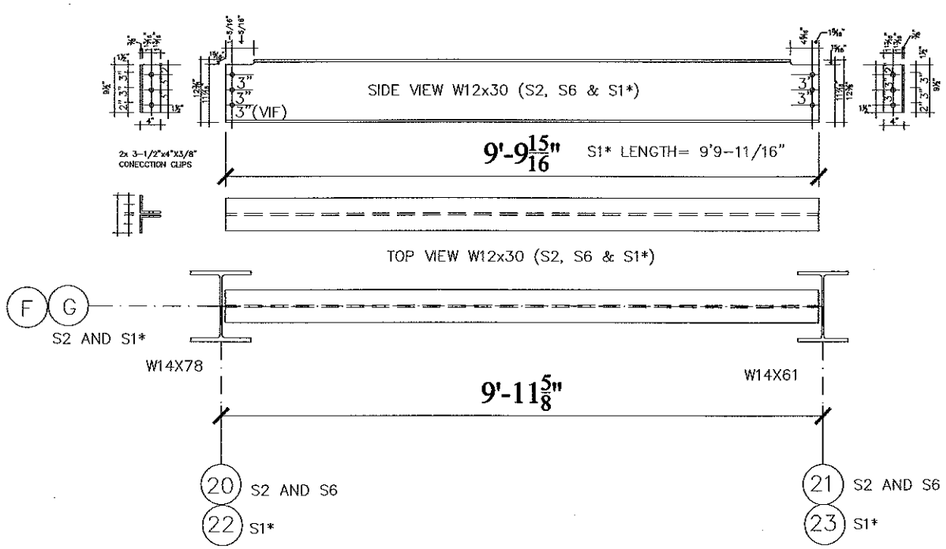
A	J.SEEMAN	RELEASE	12/29/08



GIRDER G1– W24x76



STRINGER S2 AND S6 AND S1*– W12x30

MATERIALS TABLE FOR G1, S1, S2 AND S6							
BEAM NO.	DESCRIPTION	DESIGNATION	QUANTITY	LENTGHT	#/LF	WEIGHT/UNIT	TOTAL WEIGHT
S1	STRINGER S1	W12x30	1	9' 9-11/16"	30	294.21	294.21
S2 &S6	STRINGERS S2 & S6	W12x30	2	9' 9-15/16"	30	294.64	589.68
ANGLE CLIPS	S1, S2 & S6 CONNECTIONS	4"x3-1/2"X3/8" ANGLES	12	9' 0-1/2"	9.1	82.278	987.35
G1	GIRDER G1	W24x76	1	33' 0-1/4"	76	2509.60	2509.60
ANGLE CLIPS	G1 CONNECTIONS	4"x3-1/2"X3/8" ANGLES	4	16-1/2"	9.1	14.03	56.12
CONNECTION BOLTS	2"x3/4" DIA	HS TC A325 BOLTS	30	2"	N/A		

STRUCTURAL NOTES:

ALL NEW GIRDERS, STRINGERS, CONNECTIONS, PLATES, BOLTS, WELD SIZES ARE IDENTICAL TO THE ORIGINAL DESIGN. HENCE, THE STRUCTURAL MEMBERS REPLACED ARE RATED AND HAVE THE SAME CAPACITY AS THE ORIGINAL MEMBERS AND DESIGN.

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DRAWING TITLE: G1 S1 S2 S6 DETAIL

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CONTRACT NO.: Q099–708M
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SCALE: NTS
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SHEET: 3 OF 7

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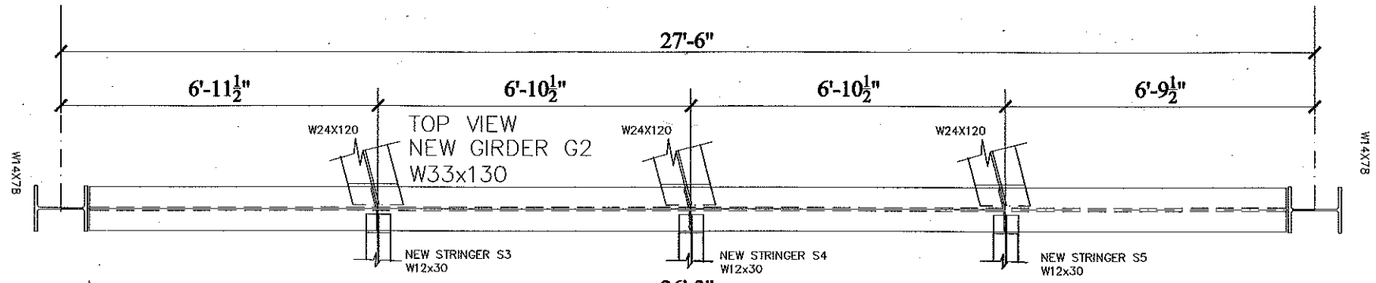
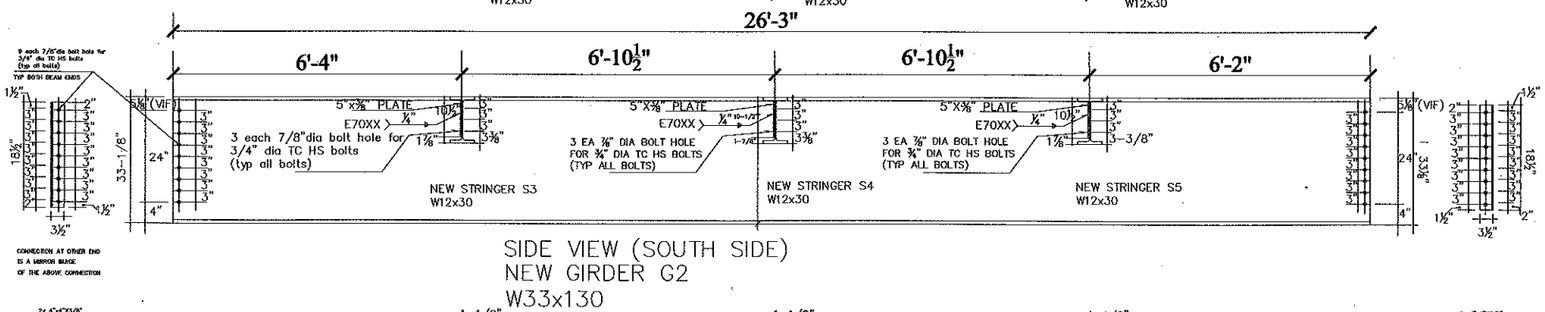
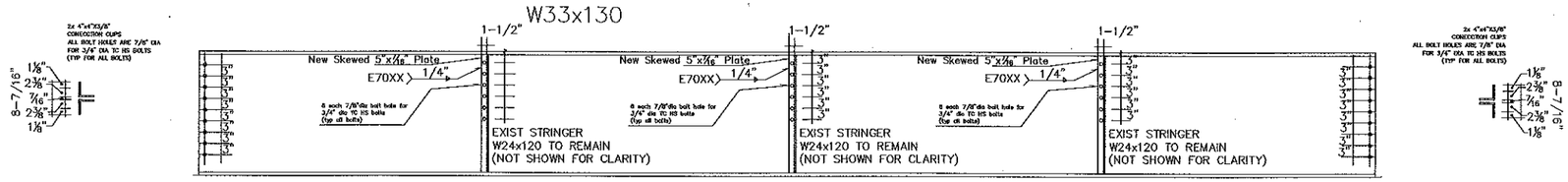




SIDE VIEW (SOUTH SIDE)
NEW GIRDER G2
W33x130



SIDE VIEW (NORTH SIDE)

GIRDER G2– W33x130

MATERIALS TABLE FOR G2							
BEAM NO.	DESCRIPTION	DESIGNATION	QUANTITY	LENTGTH	#/LF	WEIGHT/UNIT	TOTAL WEIGHT
G2	GIRDER G2	W33x130	1	28"-3"	130	3412.50	3412.50
ANGLE CLIPS	G2 CONNECT TO COLUMNS	4"x3-1/2"x3/8" ANGLES	4	18-1/2"	9.1	14.03	56.12
ANGLE CLIPS	G2 CONNECT TO STRINGERS W12x30	5"x3/8" PLATE	3	9-3/4"	6.385	5.19	15.56
ANGLE CLIPS	G2 CONNECT TO EXIST W24x76	5"x7/16" PLATE	3	31-3/8"	7.445	19.48	58.40

STRUCTURAL NOTES:

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DRAWING TITLE: G2 DETAIL

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DRAWN BY	J. SEEMAN	SIZE	D	CONTRACT NO.	Q099–708M	REV	A
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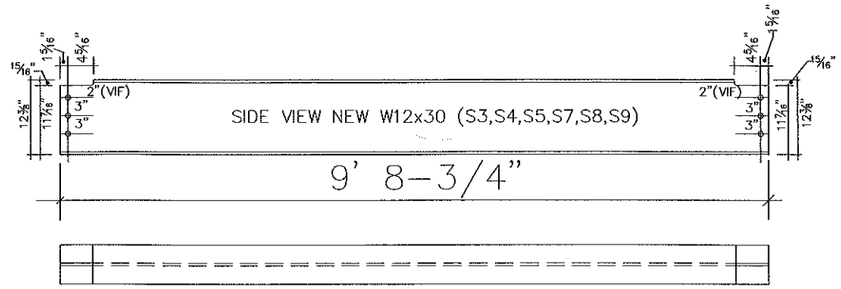
A	J.SEEMAN	RELEASE	12/29/08

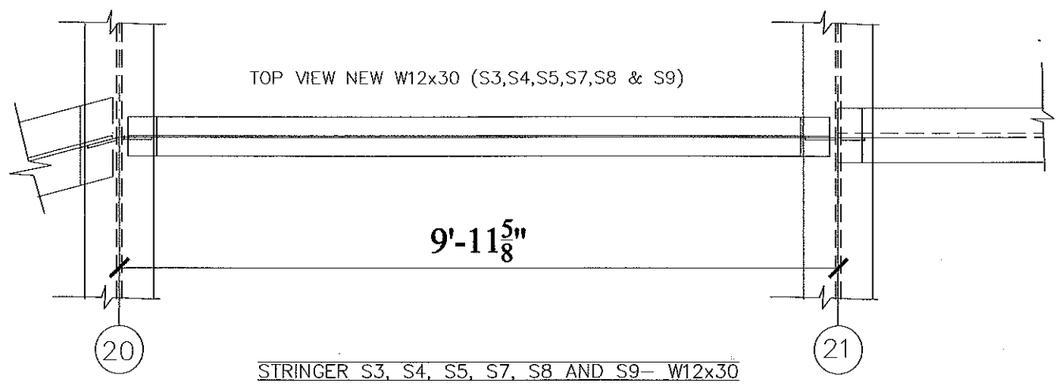
CONNECT TO NEW GIRDER G2
(G2 5"x3/8" PLATE)



CONNECT TO EXISTING 4"x3/8" PLATE
ON EXISTING W24x76

TOP VIEW NEW W12x30 (S3,S4,S5,S7,S8 & S9)

EXISTING W24x120 TO REMAIN
CLEAN CONNECTION ONLY



EXISTING W24x76 TO REMAIN
NO WORK ON THIS SIDE/BAY

STRINGER S3, S4, S5, S7, S8 AND S9– W12x30

MATERIALS TABLE FOR S3, S4, S5, S7, S8 & S9

BEAM NO.	DESCRIPTION	DESIGNATION	QUANTITY	LENTGHT	#/LF	WEIGHT/UNIT	TOTAL WEIGHT
S3,S4,S5,S7,S8 & S9	STRINGERS	W12x30	6	9' 8-3/4"	30	291.87	1,751.25
ANGLE CLIPS	BENT 20 CONNECTIONS	SEE GIRDER G2	0		N/A		
CONNECTION BOLTS	2"x3/4" DIA	HS TC A325 BOLTS	36	2"	N/A		

STRUCTURAL NOTES:

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Contract: Q099-708M

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DRAWING TITLE: S3 S4 S5 S7 S8 S9 DETAIL

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DRAWN BY: J. SEEMAN
CHECKED BY: J. SA
SIZE: D
CONTRACT NO.: Q099–708M
REV: A
SCALE: NTS
DRAWING DATE: 12/29/08
SHEET: 5 OF 7

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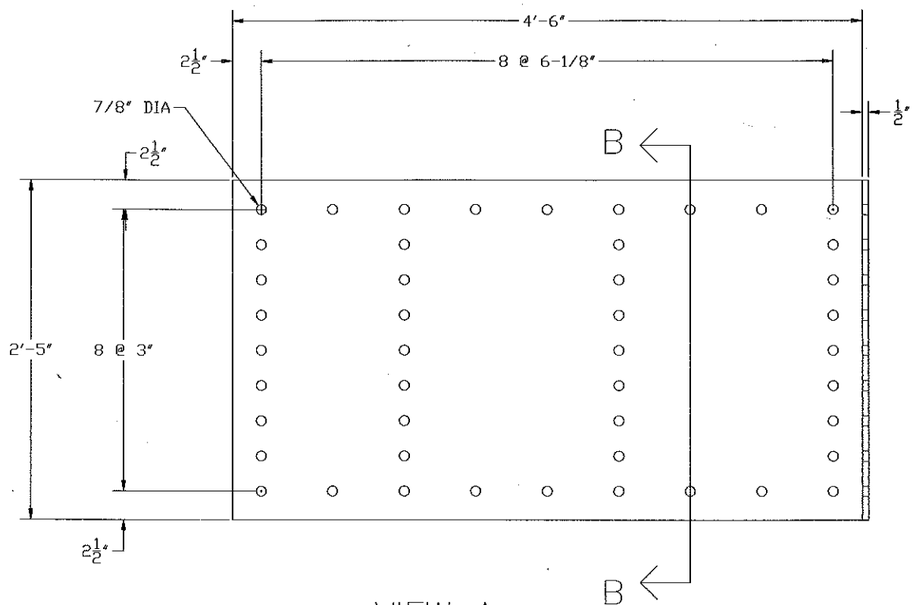
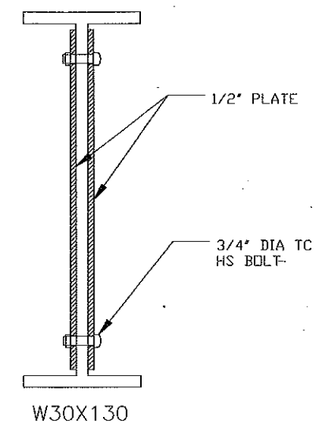
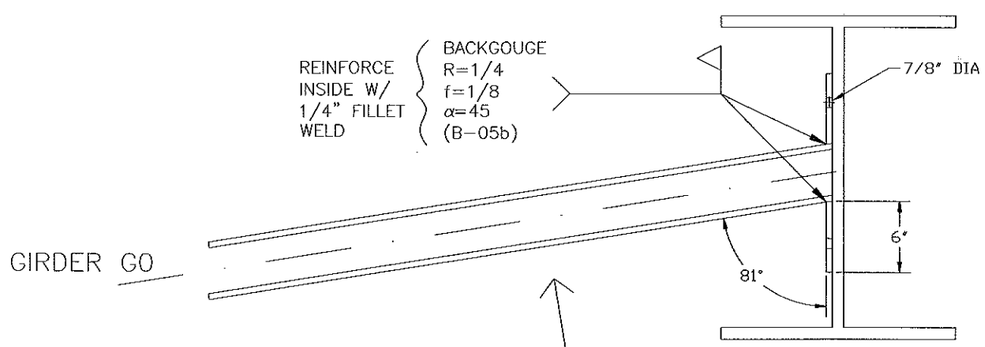
REVISIONS			
REV	DRAWN BY	DESCRIPTION	DATE
A	J.SEEMAN	RELEASE	12/29/08



VIEW A
NTS



SECTION B-B



TOP VIEW
NTS

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DRAWING TITLE: FISH PLATE CONNECTION DETAIL

AS BUILT

DRAWN BY: J. SEEMAN
CHECKED BY: J. SA

CONSTRUCTED BY
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(516) 338-8848

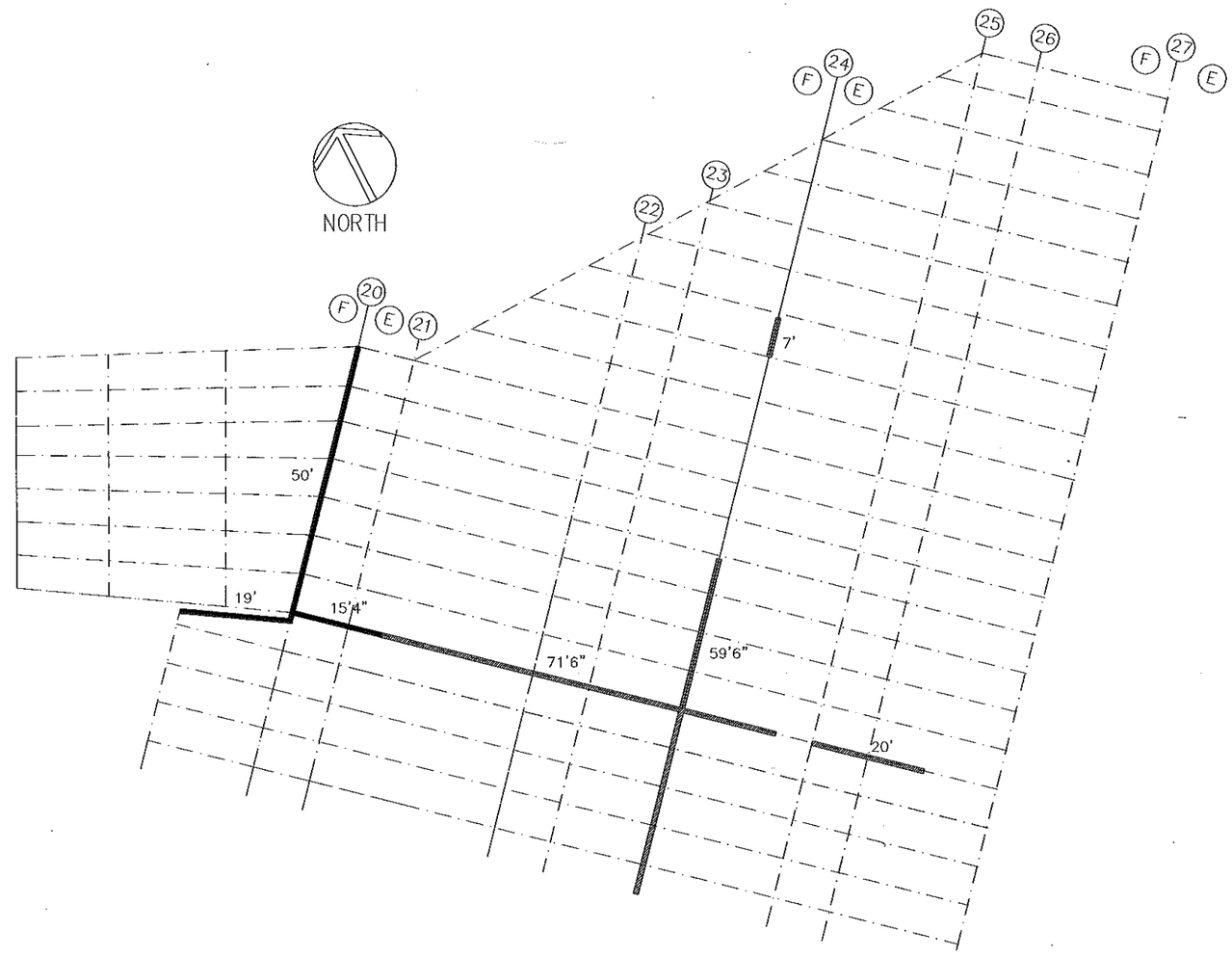
PROJECT
THE EMERGENCY RECONSTRUCTION OF EXPANSION JOINTS, STRUCTURAL STEEL, AND CONCRETE AT THE PASSARELLE OVERPASS LOCATED IN FLUSHING MEADOWS-CORONA PARK, BOROUGH OF QUEENS

SIZE	CONTRACT NO.	REV
D	Q099-708M	A

SCALE: NTS | DRAWING DATE: 12/29/08 | SHEET: **6 OF 7**

AS BUILT

REVISIONS			
REV	DRAWN BY	DESCRIPTION	DATE
A	J.SEEMAN	RELEASE	12/29/08



TYPE A (84.33 LF)

TYPE B (158 LF)

AS BUILT

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DRAWING TITLE: EXPANSION JOINT LAYOUT PLAN

AS BUILT

CONSTRUCTED BY: **PADILLA CONSTRUCTION SERVICES, INC**
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DRAWN BY	J. SEEMAN	SIZE	D	CONTRACT NO.	Q099-708M	REV	A
CHECKED BY	J. SA	SCALE	NTS	DRAWING DATE	12/29/08	SHEET	7 OF 7