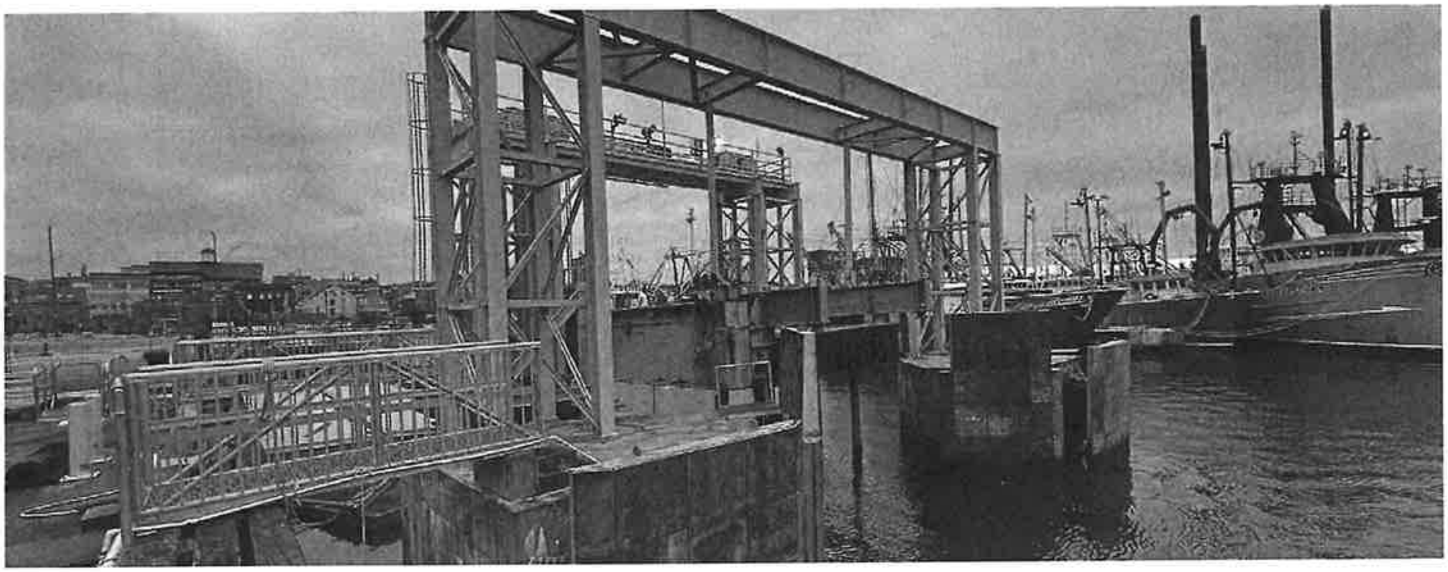

# New Bedford State Pier
# Transfer Bridge
# New Bedford, Massachusetts

**Collins Project No. 16077.00**

# Existing Conditions Report
# November 2024


**SUBMITTED BY:**

COLLINS ENGINEERS INC.

1485 South County Trail, Suite 103
East Greenwich, Rhode Island 02818

**SUBMITTED TO:**

MassDevelopment

93 State Pier
New Bedford, MA 02740

## *REVISION RECORD*

| Rev # | Date Issued | Description of Revisions |
|---|---|---|
| 0 | 9/20/2024 | Draft Existing Conditions Report |
| 1 | 11/14/2024 | Winch Inspection Report |
| 2 | 11/27/2024 | Final Existing Conditions Report |
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## *TABLE OF CONTENTS*

# 1.0 INTRODUCTION

## 1.1 Purpose and Scope

The purpose of this Existing Conditions Report is to provide an overall assessment of the condition of the steel-framed Transfer Bridge (Roll-on Roll-off ramp) and appurtenances at the New Bedford State Pier and provide recommended actions to address deficiencies. Collins completed a Routine Level inspection of the above-water components on August 8, 2024, which included the steel-framed Roll-on Roll-off ramp, steel tower structure, fender system components, concrete foundations/pier caps, and steel piles. The fender panels along the north side of the State Pier were not included in the scope of the inspection; however, any gross defects observed during the inspection were noted. The mechanical and electrical components including the winch and pulley systems were inspected by American Crane and Hoist Corporation on October 23, 2024.

The following report includes a description of the structural elements inspected, the inspection methodology, a summary of inspection findings and condition ratings, remaining service life estimation, and repair recommendations. Inspection photographs are included in Appendix A of this report, and drawings to supplement the information contained in this report are provided in Appendix B.

## 1.2 Facility Description

The Transfer Bridge (the Ramp) at the New Bedford State Pier is located on the Acushnet River and is comprised of a steel frame Roll-on Roll-off ramp, a hoist tower structure with a catwalk, two (2) berthing dolphins and fender systems which support a steel guide tower, a pile-supported concrete abutment, and an offset pile supported dolphin. A winch and cable system allows the Bridge to raise and lower to meet incoming ferries. The Bridge is primarily used by pedestrians to board and debark incoming and outgoing water ferry services; however, the Bridge was originally designed to accommodate up to HS-20 vehicle loads.

Repair drawings from Webster Engineering Co., Inc. dated 1999 were made available to Collins prior to the inspection and depict modifications and repairs to the pier and Roll-on Roll-off ramp. The ramp was originally constructed circa 1990 for the Fore River Shipyard in Quincy, MA, and was relocated and modified for New Bedford State Pier in or around 2000.

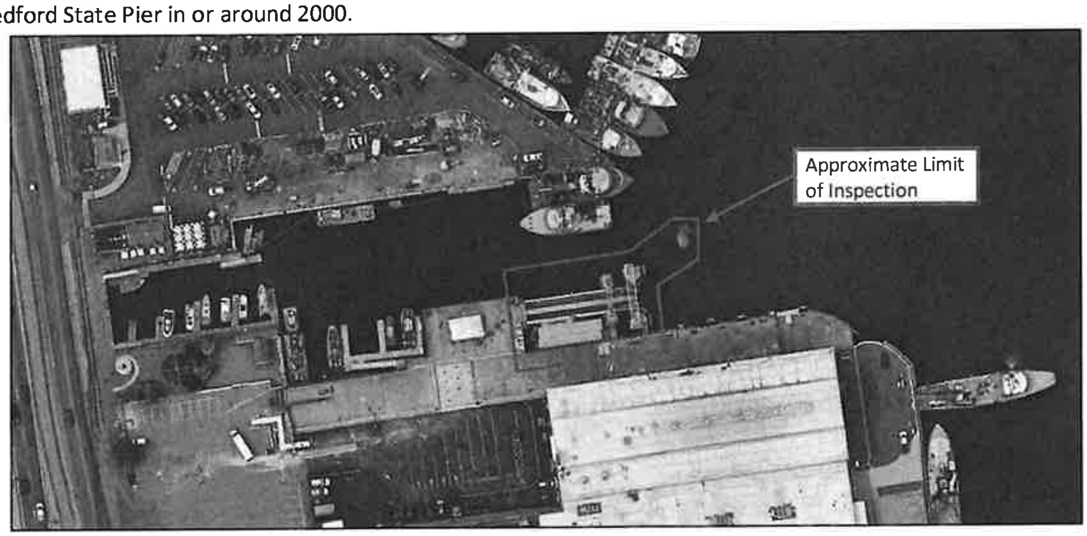



**Figure 1: Aerial (Reference: Google Earth)**

The steel framed Roll-on Roll-off ramp is approximately 112' long x 20' wide and has two (2) welded steel plate girders constructed with a 6' tall x ½" thick web and 24" wide x 1-3/4" thick flanges as the ultimate superstructure supports. The girders support eight (8) W33x130 floorbeams and one (1) W30x132 floorbeam at the west end. The floorbeam bottom flanges are coped at the connection area to the girder. Within the floorbeam spans are five (5) W21x93 joists which have a coped top flange at the connection point with the floorbeam. The joists support a steel grating which consists of 6" x 3/8" bearing bars spaced at 3" on center with 1-3/4" x 1/4" cross bars spaced at 3" on center with a walking plate 5' wide x 3/8" thick that runs the length of the center of the deck grating. There are 12" high x 12" wide timber curbs along each side of the ramp decking. The ramp is supported with hinged bearings on top of a pile supported pier abutment at the west end and is suspended by double 1-3/4" diameter cables at the east end at the hoist towers on either side of the ramp, which are founded on the concrete dolphins with 16" diameter concrete-filled steel pipe piles. Per available plans, the piles are driven and socketed a minimum of 25' into bedrock. The hoist towers consist of W12x50 columns with lateral and diagonal stiffeners, with steel cables used to raise and lower the ramp with a counterweight and electric winch. There is a catwalk 44' long x 7' wide that connects the tops of the hoist towers, and approximately 47'-5" from the west end of the ramp there is a manual hoist winch assembly.

There are three (3) pile supported dolphin fender systems in the vicinity of the ramp. Two (2) are directly east to the hoist towers, designated as the north and south dolphin fenders, and one (1) is approximately 50' northeast into the river, designated as the northeast dolphin fender. The north and south dolphins adjacent to the ramp are connected by a steel support tower which is used to align the ramp during raising and lowering operations. The dolphins consist of a mass concrete cap supported by concrete-filled steel pipe piles, and rubber leg fenders and steel fender frames with UHMW facing plates. The north and south fender systems adjacent to the ramp have an inner and outer fender panel, and the northeast fender system in the river has a single fender panel on the south face.

The inspection of the existing pier and wharf was not within the inspection scope, nor was the fender system located east of the Ramp along State Pier. There are several barges stored beneath the Ramp, and although they do not affect the structural capacity, it should be noted that if broken loose may impact ferry operations.

### 1.3 Inspection Methodology

Collins completed a visual and tactile inspection of the waterfront infrastructure within the scope of the work located at the New Bedford State Pier in accordance with the ASCE Waterfront Facilities Inspection and Assessment Manual (MOP 130) to develop a general condition evaluation of structural members and locate areas of significant deterioration.

The Level I inspection was performed on the entirety of the structure and consisted of a visual inspection to document general conditions, record abnormalities, and locate observable defects, and a tactile inspection was completed on representative structural members with observed deterioration to document other deterioration that may not be visibly present.

The above and below water structural components are typically evaluated and assigned condition ratings using the American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice No. 130 (MOP 130): Waterfront Facilities Inspection and Assessment Manual. The terminology used to describe the observed conditions, the rating criteria, and component grades are developed from the MOP 130 (2015 edition) and are described in Section 1.4 RATING CRITERIA.

### 1.4 Rating Criteria

The general condition assessment ratings for the inspected structural elements and component groups are based on a six-point assessment scale developed by ASCE. The six condition ratings are as follows:

| | |
|---|---|
| Good (6) | No visible damage or only minor damage is noted. Structural elements may show very minor deterioration, but no overstressing is observed. No repairs are required. |
| Satisfactory (5) | Limited minor to moderate defects or deterioration are observed, but no overstressing is observed. No repairs are required. |
| Fair (4) | All primary structural elements are sound, but minor to moderate defects or deterioration are observed. Localized areas of moderate to advanced deterioration may be present but do not significantly reduce the load-bearing capacity of the structure. Repairs are recommended, but the priority of the recommended repairs is low. |
| Poor (3) | Advanced deterioration or overstressing is observed on widespread portions of the structure but does not significantly reduce the load-bearing capacity of the structure. Repairs may be carried out with moderate urgency. |
| Serious (2) | Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of primary structural components. Local failures are possible and loading restrictions may be necessary. Repairs may need to be carried out on a high-priority basis with urgency. |
| Critical (1) | Very advanced deterioration, overstressing, or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and load restrictions should be implemented as necessary. Repairs may need to be carried out on a very priority basis with strong urgency. |

For developing condition state ratings for steel members, **Figure 2** below from MOP 130 was used.

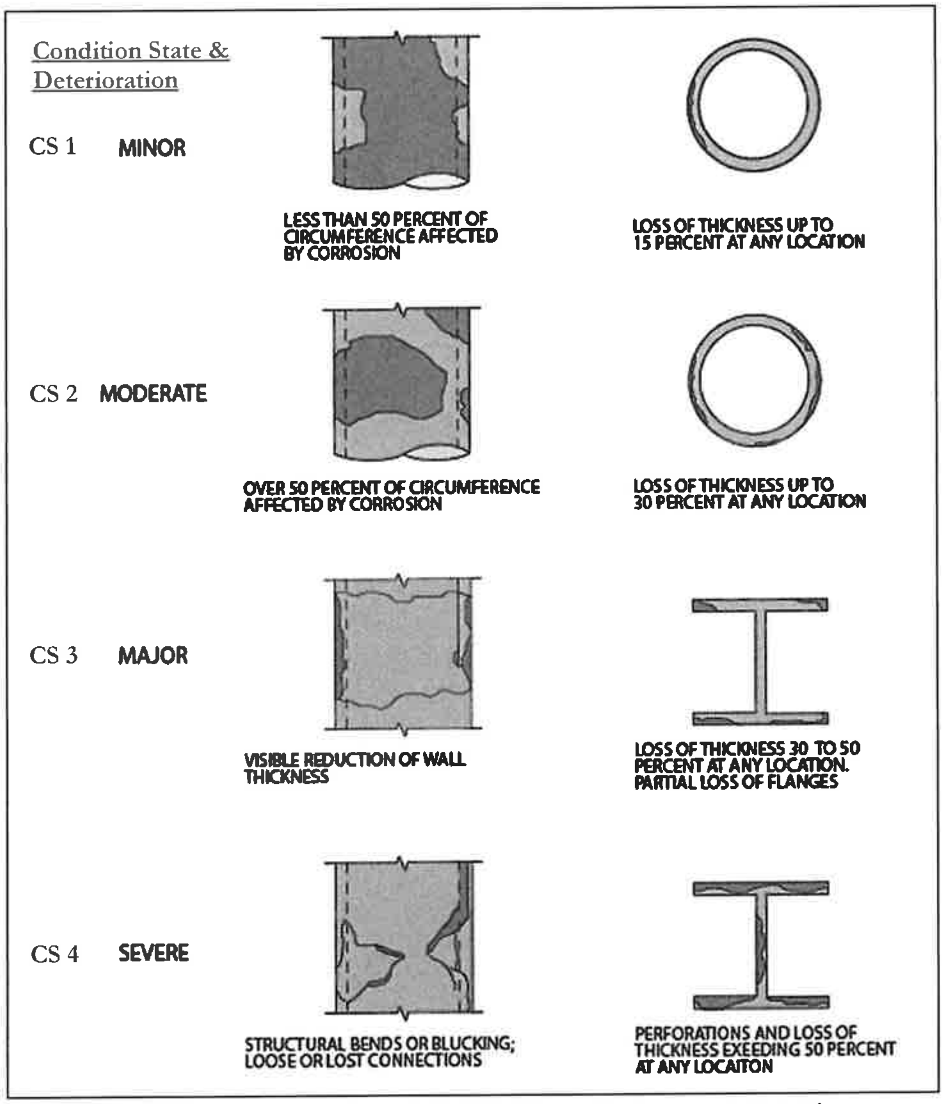



**Figure 2: Condition Ratings for Steel Elements (Reference: MOP 130)**

For developing condition state ratings for timber elements, **Figure 3** below from MOP 130 was used.

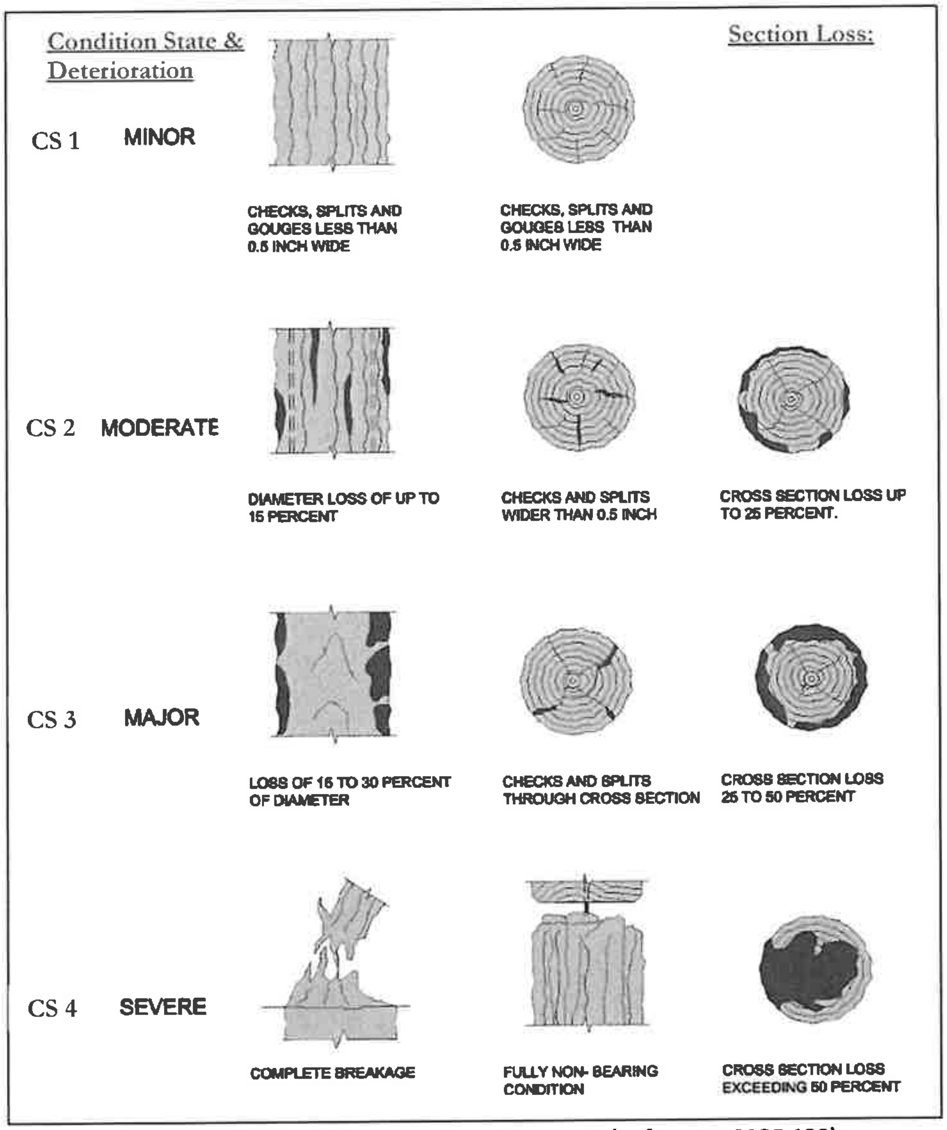
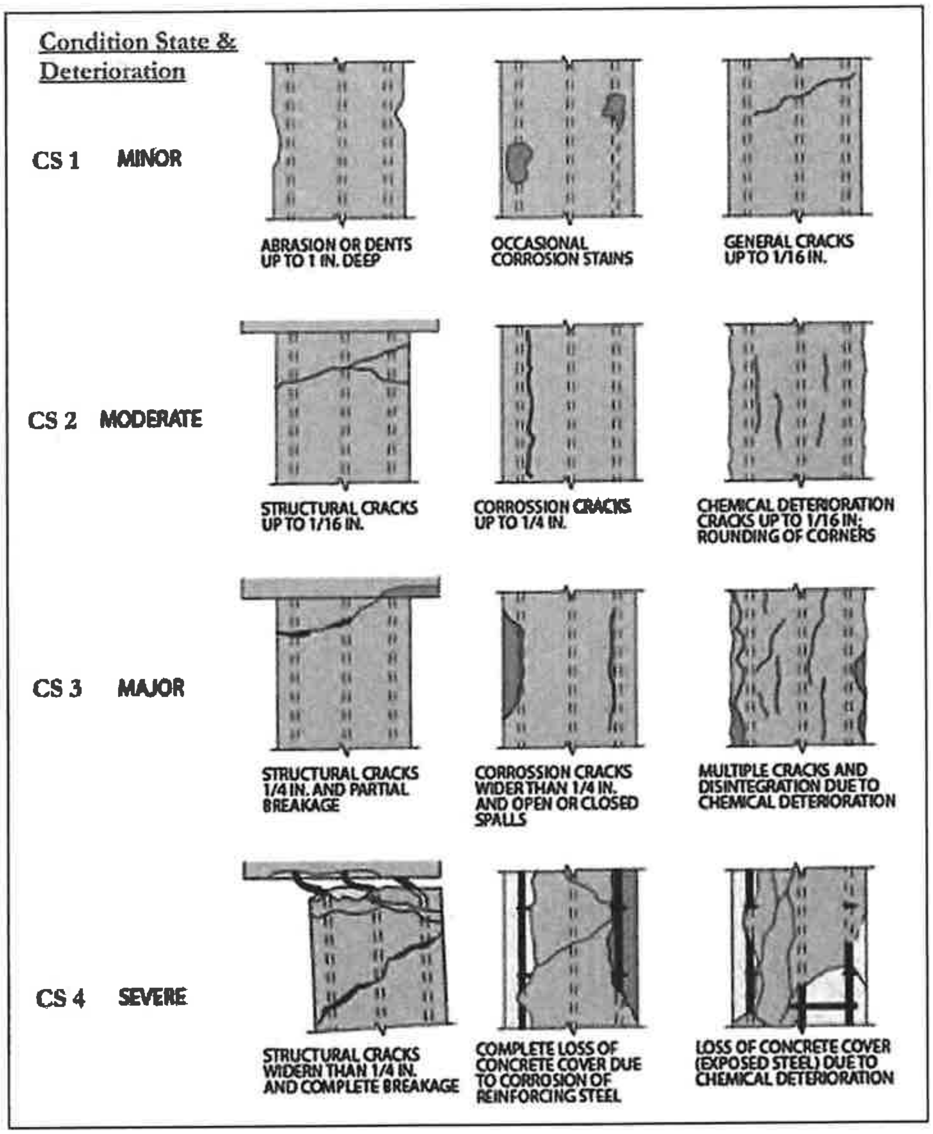



**Figure 3: Condition Ratings for Timber Elements (Reference: MOP 130)**

For developing condition state ratings for concrete elements, **Figure 4** below from MOP 130 was used.



## 2.0 INSPECTION FINDINGS

Overall, the Ramp infrastructure appears to be in **Fair to Satisfactory** condition overall. The following provides details of the observed deficiencies for each element inspected, an estimate of remaining service life, and repair recommendations. The figures in Appendix B provide additional information and locations of observed deficiencies.

### 2.1 Observations

<u>Transfer Bridge Structure</u>

The transfer bridge structure consisting of the welded plate girders, floorbeams, joists, deck grating, and timber curbs was observed to be in **Fair** condition overall. There is coating loss with bubbling on the joists and floor beams that covers approximately 25% of the surface area. In the areas that have coating loss, there is light to moderate corrosion and rust with negligible section loss, most notably at the east end of the pier where ferries berth. The connection bolts at the floorbeam-girder and joist-floorbeam connections have moderate corrosion with isolated areas of laminated rust. The hinged bearings were observed to have no significant defects. On the topside, the walking plate and the deck grating were observed to have no significant defects. The steel transition plate that connects the ramp and the adjacent platform is unstable and slightly bent, presenting a potential tripping hazard. Additionally, the south marker light at the ramp entrance is missing. The timber curbs along the deck edges have moderate severe rot and section loss most notably near the entrance; however, the timber curbs are in place to prevent vehicles from coming in contact with the girders and do not affect the load capacity of the structure.

<u>Concrete Pier Connection and Cable Foundation</u>

The concrete pier connection at the west of the ramp and concrete hoist tower foundations that support the east side of the ramp were observed to be in **Fair** Condition. The hoist tower foundations were observed to have isolated spalls up to 8" diameter x 1" deep with no other significant defects observed. The piles supporting the hoist towers exhibited approximately 50% loss of coating within the tidal and splash zone. Within the areas of coating loss, pitting up to 1/8" deep was typically observed with isolated areas of laminated rust up to 1/4" deep. Minor marine growth was observed within the tidal zone.

The reinforced concrete "abutment" had isolated hairline cracks with efflorescence; the concrete was sounded and no significant hollow areas were found. An isolated spall up to 18" diameter x 2" deep was observed on the northeast corner of the abutment. Minor marine and algae growth was observed on the abutment and supporting piles. The steel pipe support piles exhibited approximately 50% loss of coating within the tidal and splash zone exhibited areas of laminated rust up to 1/8" deep covering approximately 10% of the surface area.

<u>Concrete Dolphins and Fender Systems</u>

The dolphins and fender systems were observed to be in overall **Fair** condition. The mass concrete caps on all dolphins had hairline map cracking with efflorescence throughout the surface with no hollow areas observed. There is a spall on the southeast corner of the northeast dolphin fender 16" high x 12" wide x 3" deep and scaling/spalling on the northeast corner full height x 8" wide x 3" deep. The steel fender panels exhibited coating loss on approximately 10% of the total surface area, concentrated within the splash zone. Within the areas of coating loss, areas of laminated rust up to 1/8" deep was observed. The chains and hardware were observed to have minimal rust in the splash zone. The UHMW facing plates were gouged with moderate to heavy scraping, most notably on the inner fenders adjacent to the ramp. Several studs that connect the UHMW facing plates to the steel panel were missing, and a missing facing plate was noted on the northeast dolphin fender within the tidal zone.

Cable and Support Towers
The steel cable and support towers to the north and south of the ramp were observed to be in **Satisfactory** condition. There were no significant deficiencies on the steel superstructure that makes up the four (4) towers. The catwalk connecting the hoist towers was observed to have chipped and fading paint throughout, but no significant corrosion or deterioration was noted.

Winch and Pulley Systems
The winch and pulley systems were inspected by American Crane and Hoist Corporation (ACHC) on October 23, 2024, and were observed to be in overall **Fair** condition. Although the ramp is raised and lowered on a daily basis depending on tides and ferry operations, ACHC noted that it appeared as though the system had not been used in several years, likely due to minimal observed wear. The most significant deficiencies include rotting on the guards, decreased diameter of the wire rope by approximately 0.21" with loose strands throughout, and deterioration of the brake pads. Although it does not appear to affect the safety or operations of the transfer bridge, there was a loose bolt noted on the electric winch for the counterweight, and the steel angles on the wire rope counterweights are bent on both sides.

The Winch Inspection Report is in Appendix D.

## 2.2 Structural Capacity

As part of the evaluation of the structure, Collins completed a load rating of the ramp superstructure which is included in Appendix C. Reduction factors have been applied to the calculations based on the deficiencies and deterioration described in this report. **Based upon the structural configuration and the conditions observed at the time of the inspection, the ramp can safely accommodate pedestrian loading and has been rated to accommodate HS-20.**

## 2.3 Remaining Useful Life

Waterfront structures typically have a design life of 25 to 50-years, but the probable service life expectancy estimates are highly subjective based on the material, existing conditions, environmental exposure, wear and tear, and the type of construction.

The original steel ramp superstructure is approximately 35 years old and was rehabbed approximately 25 years ago when it was moved to its current location. The steel pipe piles were installed at that time, and are approximately 25 years old. The superstructure is located above the splash zone, minimizing the exposure to the harsh marine environment. The steel support piles show signs of moderate section loss which has been minimized by the original pile coating. Although the coating has reduced the overall section loss and extended the service life, it is mostly worn away within the splash zone and corrosion rates will most likely accelerate if not repaired. **Based upon the conditions observed at the time of the inspection, the estimated remaining useful service life of approximately 25-30 years, which could be extended with proper maintenance and repair.**

## 3.0 RECOMMENDATIONS

The following recommendations outlined below should be implemented to increase the remaining useful service life of the Roll-on Roll-off ramp at the New Bedford State Pier. See Table 1 below for a breakdown of the recommendations and rough order of magnitude cost estimates.

Ramp Structure

Recommended repairs to the ramp structure include recoating of all structural elements, including girders, floorbeams, joists, and grating, and replacement of the timber curbs. Although no significant section loss was observed during the inspection, the observed rust and corrosion will continue to advance without remediation. The application of an epoxy or marine-grade coating will mitigate future section loss from occurring. The timber curbs are more important to the structure if the ramp is used for vehicle access, so replacement of the timber curbs, although not structurally significant, will greatly improve the aesthetics of the ramp. The transition plate at the west end of the ramp should be securely connected or replaced with a pivoting ramp to mitigate and/or remove the potential tripping hazard. Although the pivot bearings were found to have no significant deficiencies, regular inspection and application of grease will help them continued to operate problem-free.

Piles

Recommended repairs to the steel pipe support piles include cleaning and recoating the upper 12' of the hoist tower piles and upper 7' of the abutment piles. The application of a coal tar epoxy or similarly effective coating to the piles will help to mitigate further advancement of the observed corrosion, ensuring the concrete within the steel pipe is not exposed to seawater.

Fender Systems

It is recommended that the missing UHMW facing plate be replaced at northeast dolphin fender to avoid vessel impact damage to the steel fender and damage to incoming vessels. The lower chain hardware for all fender panels should be monitored as it is within the splash zone, and the strength of the chains will reduce as corrosion continues to worsen.

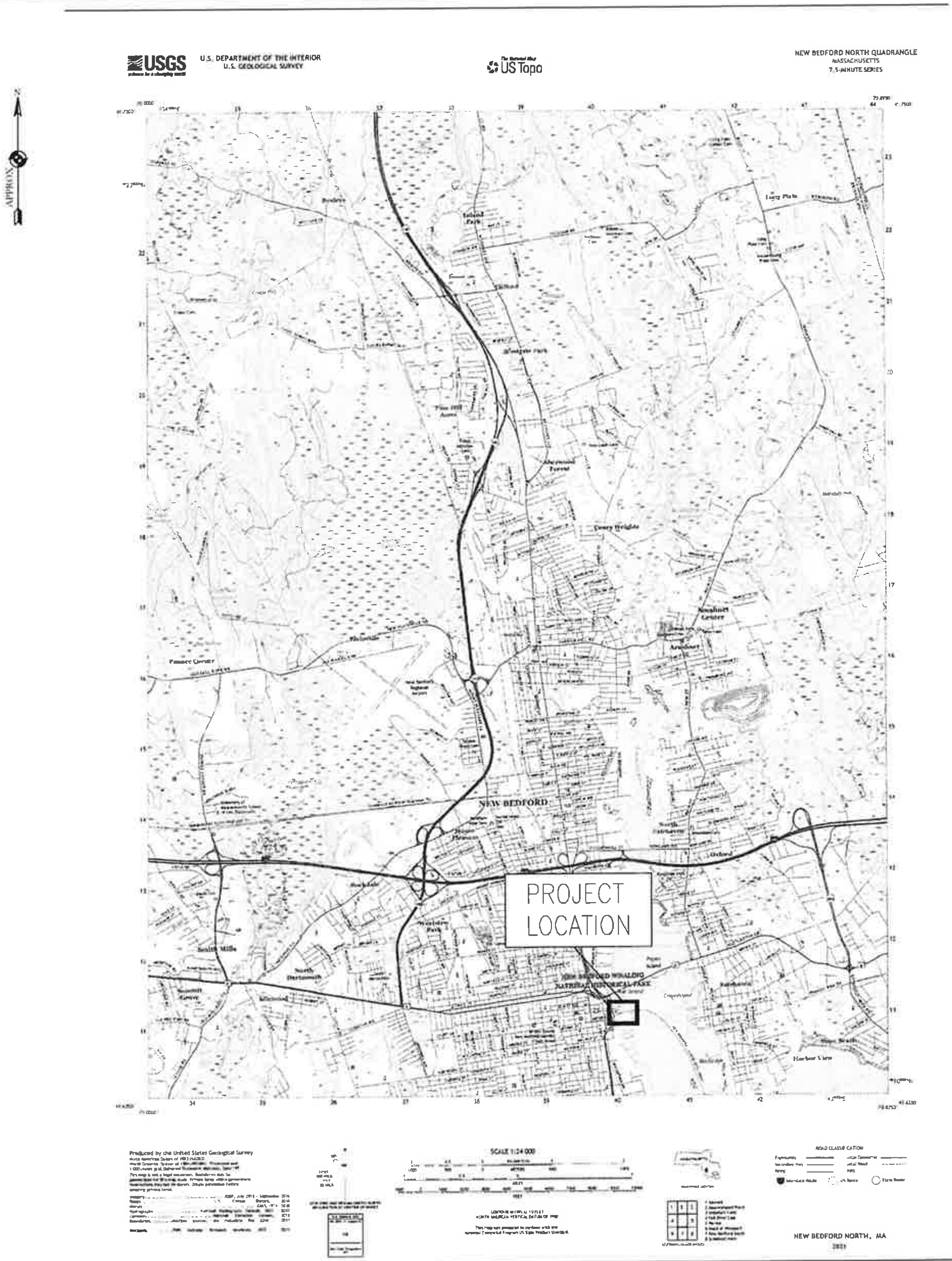
Winch and Pulley Systems

It is recommended that the wire rope, guards, and brake pads be replaced on the winch and pulley systems. When replacing the wire rope, the winch should be cleaned, painted and lubricated. Additionally, the hand wheel and ratchet should be cleaned and lubricated, and the gearcase should be lubricated. The loose bolt on the transfer bridge counterweight should be tightened and the bent angles should be replaced.

Routine Inspections

For future budgeting purposes, it is recommended that routine inspections are completed every 4 years in accordance with industry guidance presented in MOP 130. The inspection frequency is a function of the condition rating of the structure (Fair) and type of environment (aggressive).

**Report Figures**

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| checked by: | |
| approved by: | |
| QA/QC by: | |
| project no.: | |
| drawing no.: | MAP FIGURES |
| date: | SEPTEMBER 2024 |

FIGURE 1

1 OF 3

NEW BEDFORD RORO RAMP

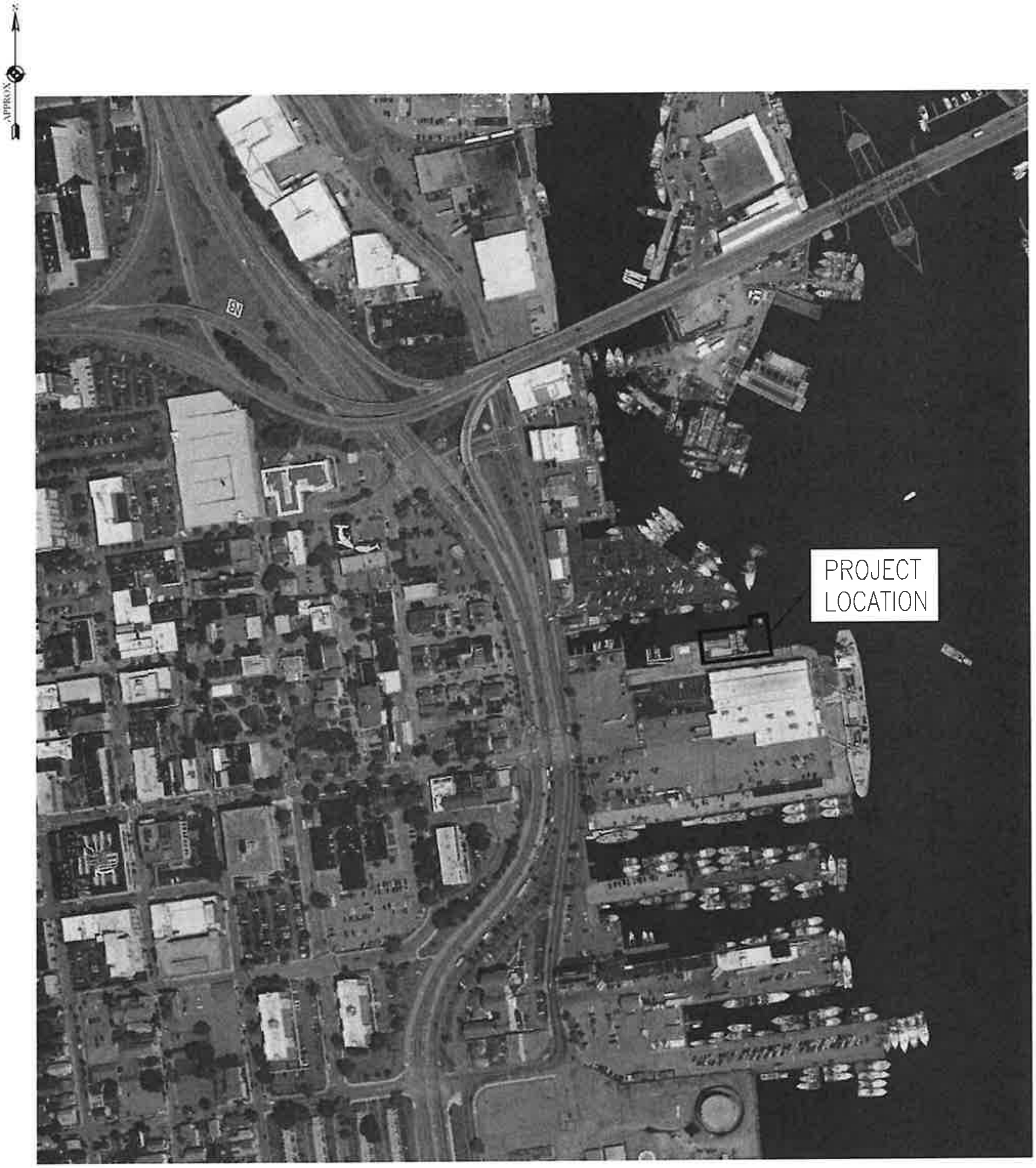
USGS LOCATION PLAN

NEW BEDFORD STATE PIER INSPECTION

NEW BEDFORD, MA

2024

COLLINS ENGINEERS INC

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| project no.: | |
| drawing no.: | MAP FIGURES |
| date: | SEPTEMBER 2024 |

FIGURE 2

2 OF 3

NEW BEDFORD RORO RAMP

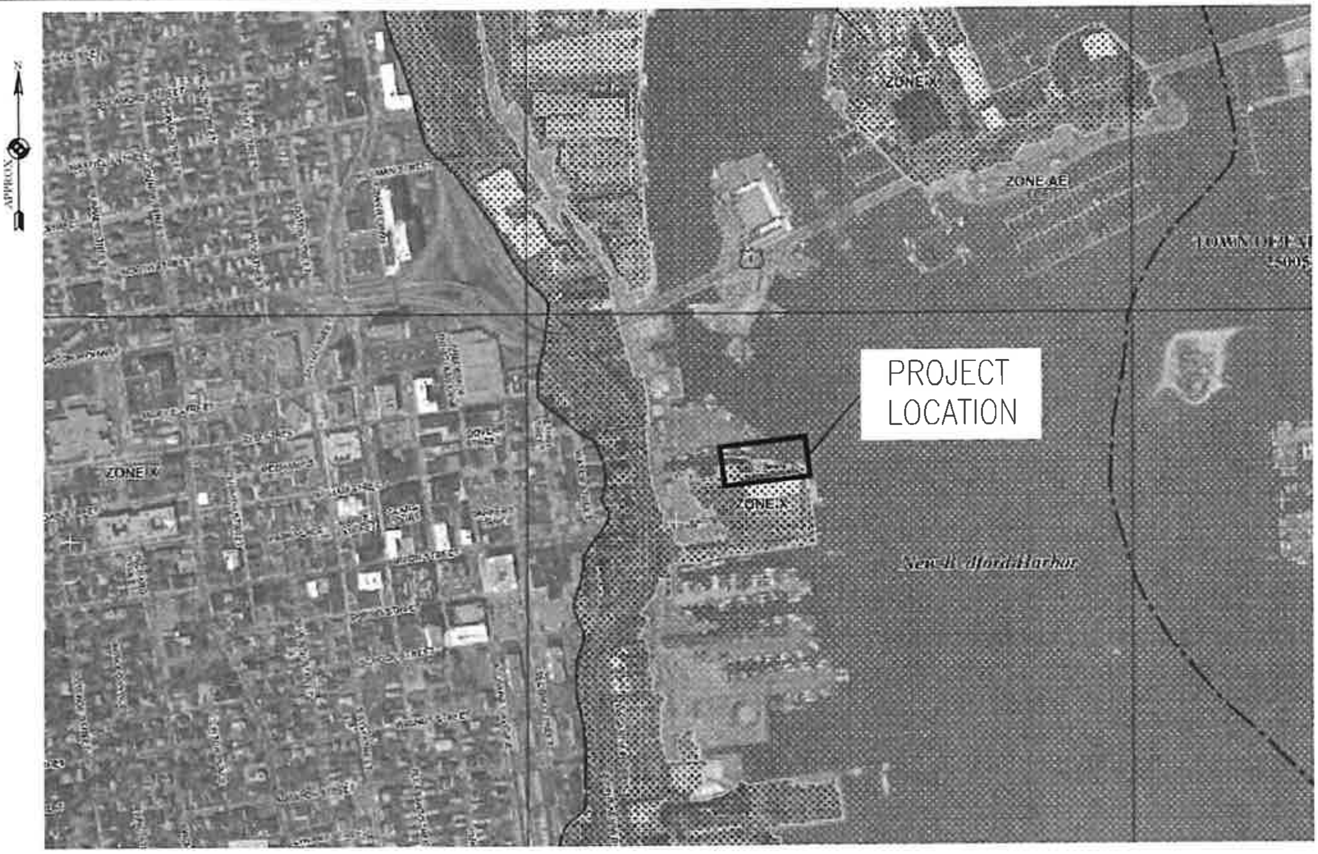
AERIAL LOCATION PLAN

NEW BEDFORD STATE PIER RAMP INSPECTION

NEW BEDFORD, MA

2024

COLLINS ENGINEERS INC.




NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0393G

# FIRM

FLOOD INSURANCE RATE MAP

**BRISTOL COUNTY, MASSACHUSETTS**
**(ALL JURISDICTIONS)**

PANEL 393 OF 550
(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

| COMMUNITY | NUMBER | PANEL | SUFFIX |
|---|---|---|---|
| [illegible], TOWN OF | [illegible] | 0393 | G |
| NEW BEDFORD, CITY OF | [illegible] | 0393 | G |

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

**MAP NUMBER**
**25005C0393G**

**MAP REVISED**
**JULY 16, 2014**

Federal Emergency Management Agency

**MAP SCALE 1" = 500'**

250 0 500 1000 FEET

150 0 150 300 METERS

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at [illegible] or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, NNGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at (301) 713-3242, or visit its website at [illegible].

**Base Map** information shown on this FIRM was derived from digital orthophotography. Base map files were provided in digital form by Massachusetts Geographic Information System (MassGIS). Ortho imagery was produced at a scale of 1:5,000. Aerial photography is dated April 2005.

**The profile baselines** depicted on this map represent the hydraulic modeling baselines that match the flood profiles in the FIS report. As a result of improved topographic data, the **profile baseline**, in some cases, may deviate significantly from the channel centerline or appear outside the SFHA.

This map reflects more detailed and up-to-date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables for multiple streams in the Flood Insurance Study Report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on this map.

**Corporate limits** shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

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| approved by: | |
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| project no.: | |
| drawing no.: | MAP FIGURES |
| date: | SEPTEMBER 2024 |

FIGURE 3
3 OF 3

NEW BEDFORD RORO RAMP
FEMA FLOOD MAP
NEW BEDFORD STATE PIER RAMP INSPECTION

NEW BEDFORD, MA | 2024

COLLINS ENGINEERS INC

## Appendix A – Photographs

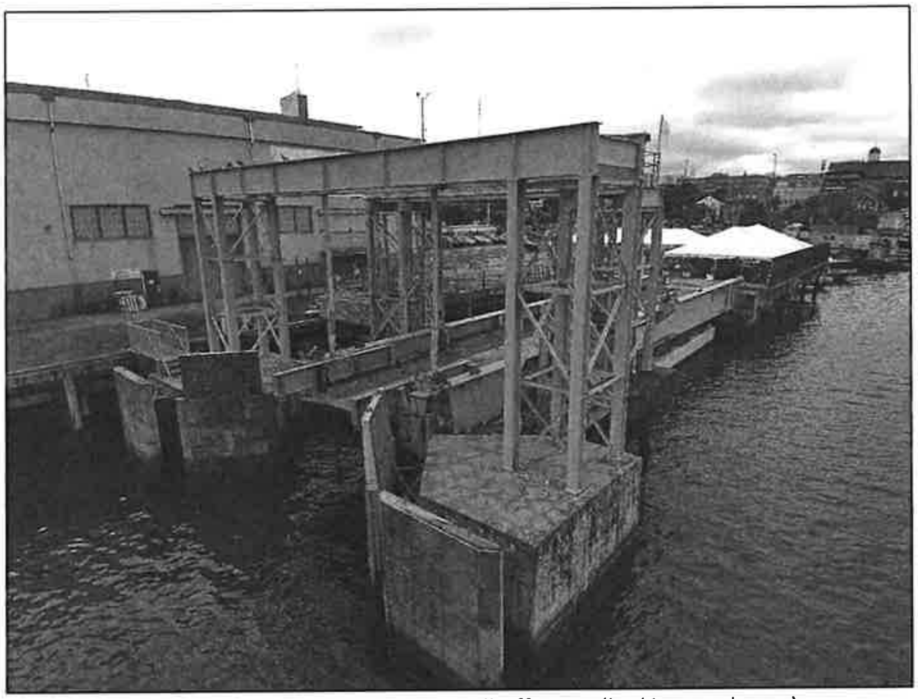


Photo 1 – Overall View of Roll-on Roll-off Ramp (looking southwest)

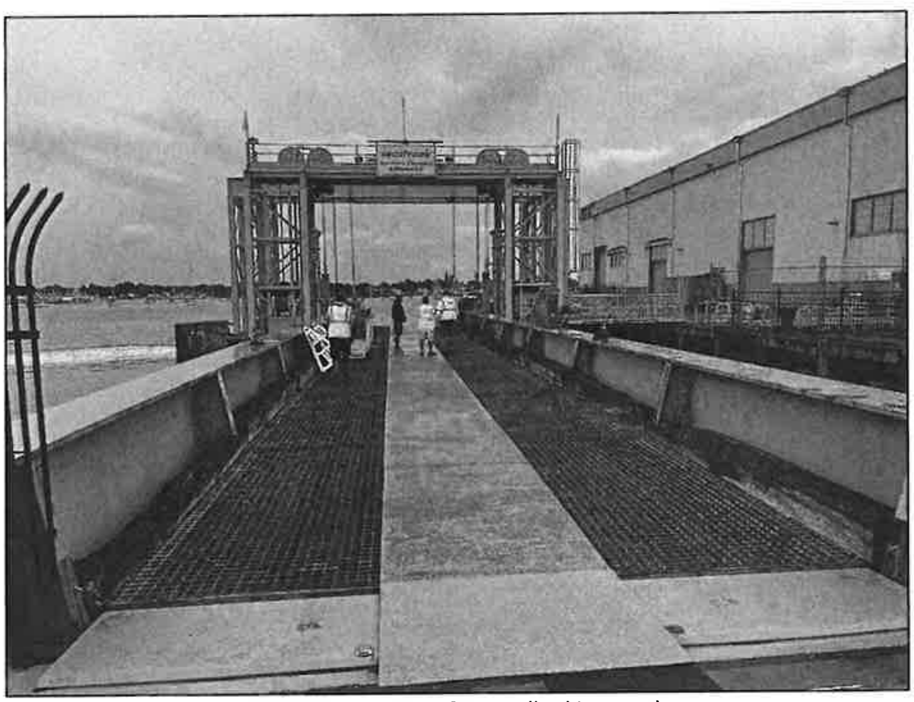


Photo 2 –Topside of Ramp (looking east)


Photo 3 – Typical Dolphin Fender Condition (north dolphin shown)

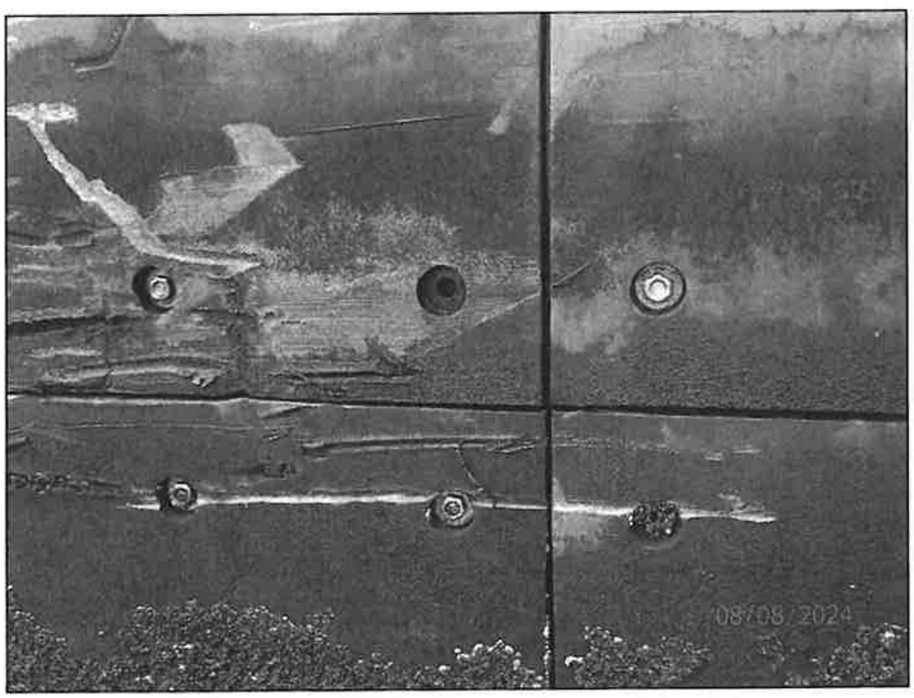



Photo 4 – View of fender plate scraping and missing stud

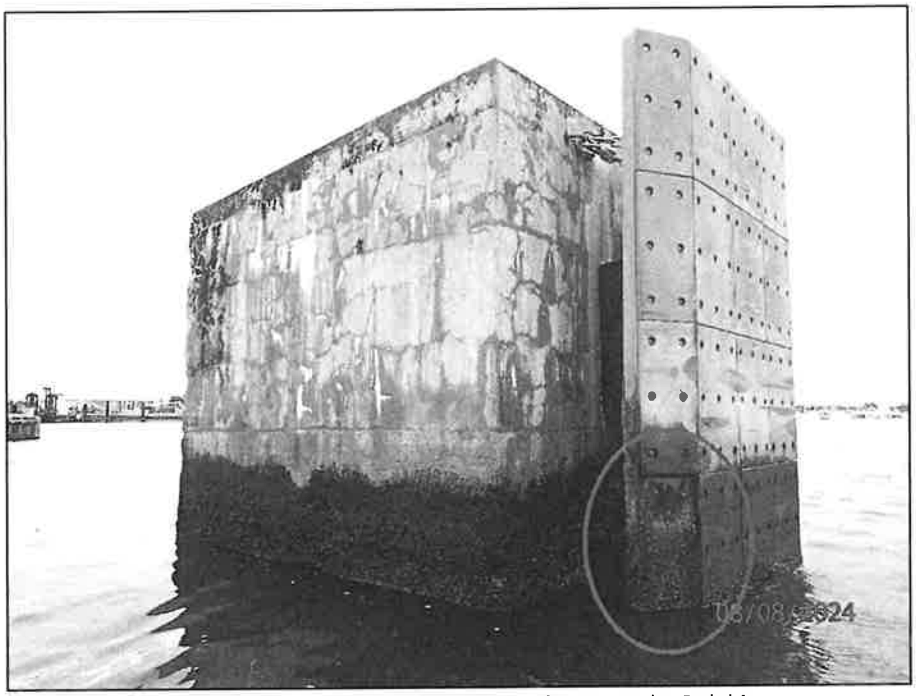



Photo 5 – Missing facing plate on Northeast Fender Dolphin

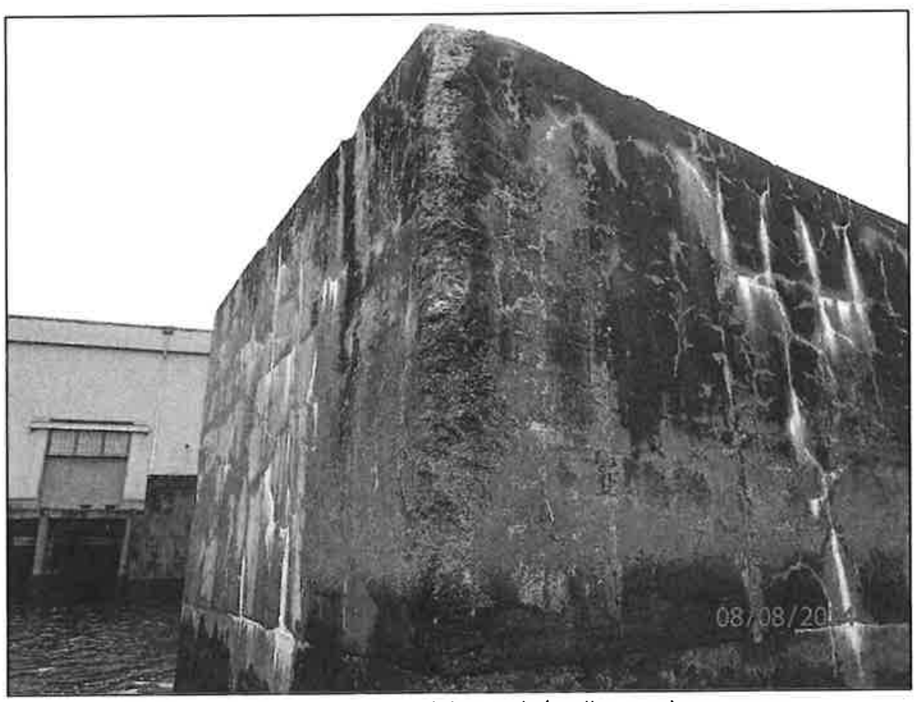



Photo 6 –Northeast Fender Dolphin scale/spall at northeast corner

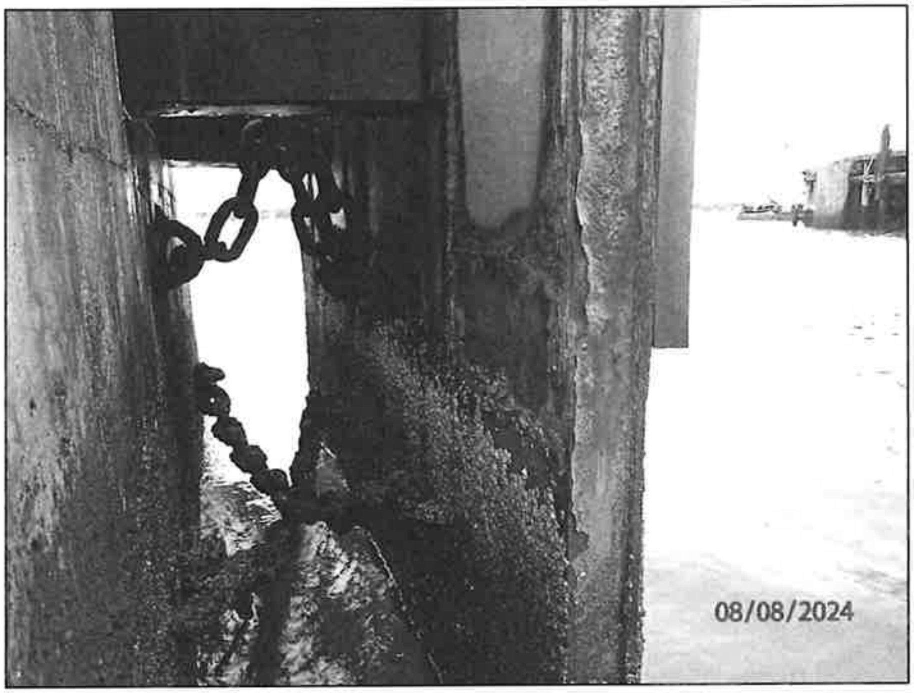



Photo 7 – Typical Fender Steel Panel and Chain Hardware Condition

Photo 8 –Underside of Ramp Superstructure (looking west)

Photo 9 – Typical Missing/Peeling Paint and Corrosion of Ramp

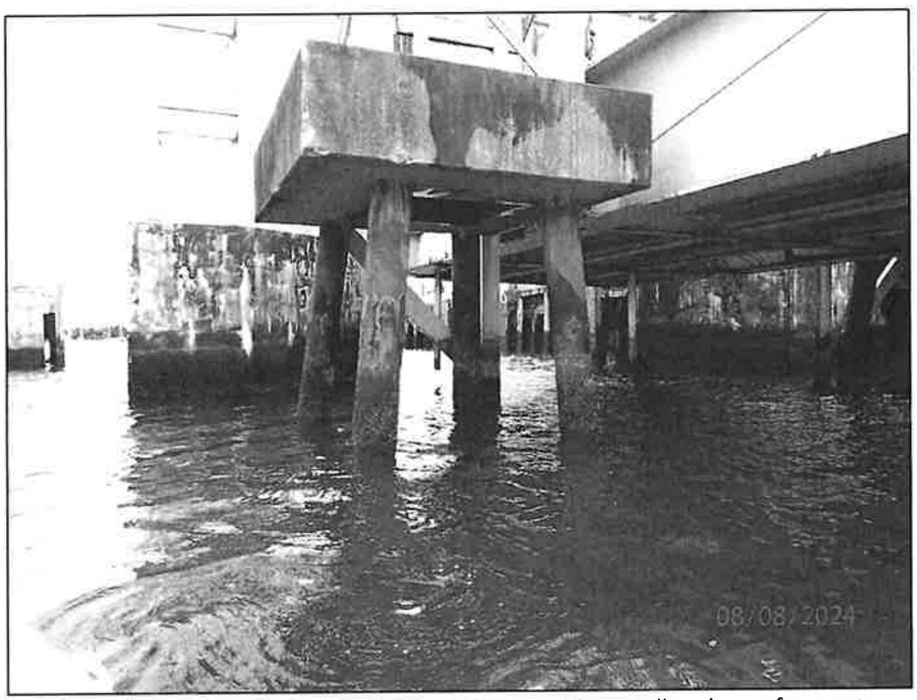


Photo 10 – Hoist Tower Dolphin looking east, note minor spalls at base of concrete

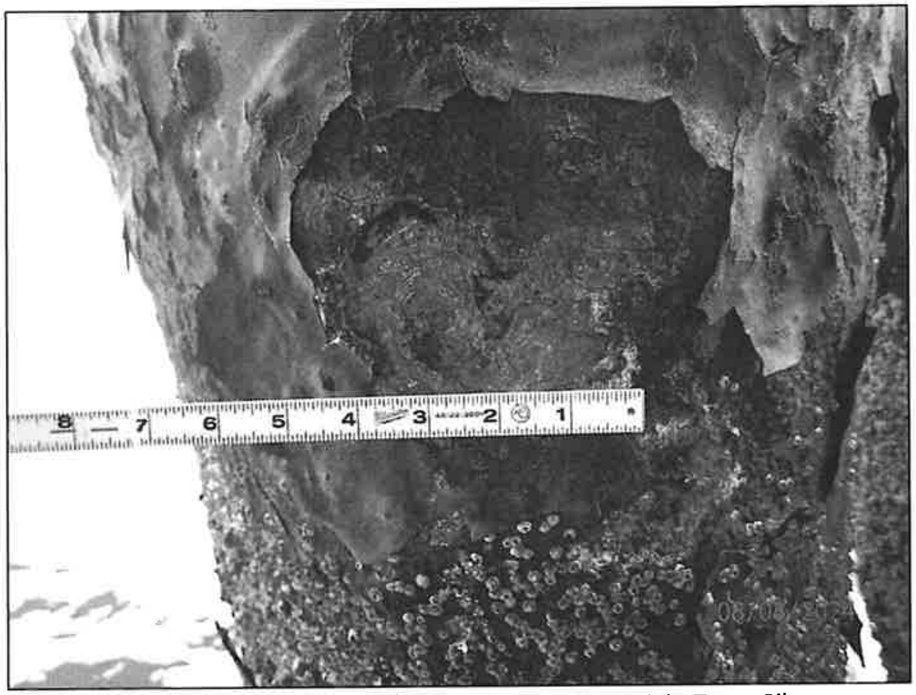


Photo 11 – View of up to 1/4" deep Section Loss at Hoist Tower Pile

Photo 12 – Abutment Elevation (looking southwest)

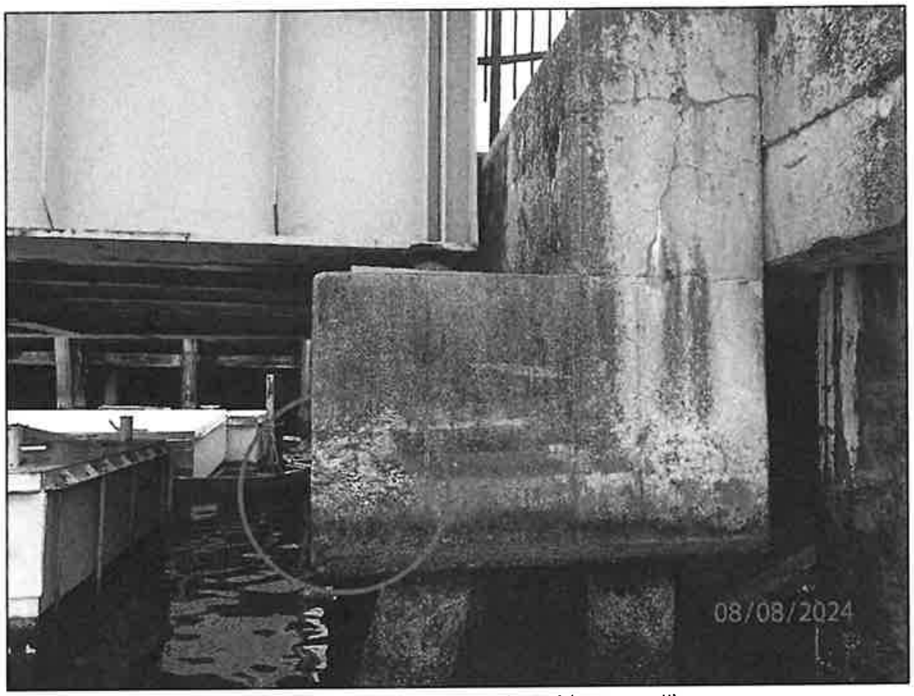



Photo 13 – Abutment North End (note spall)

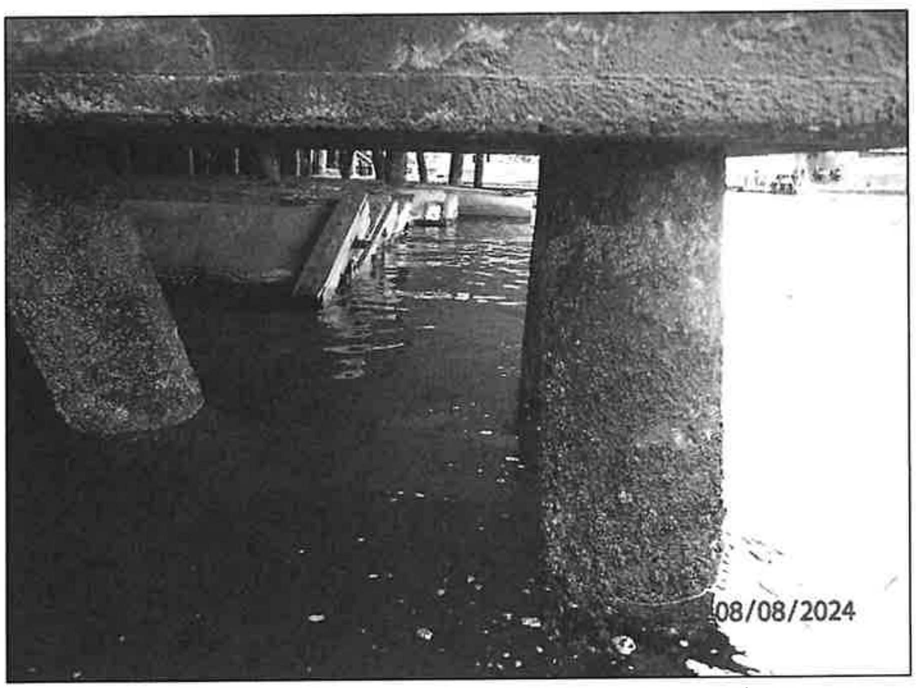



Photo 14 – Typical Abutment Pile Condition (looking west)

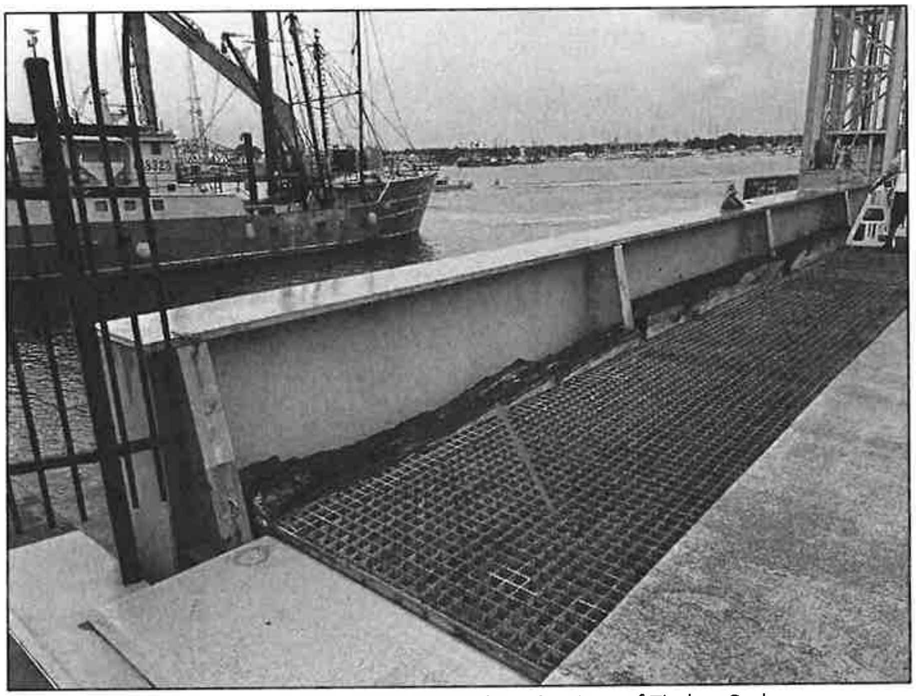


Photo 15 – View of Severe Rot and Section Loss of Timber Curb

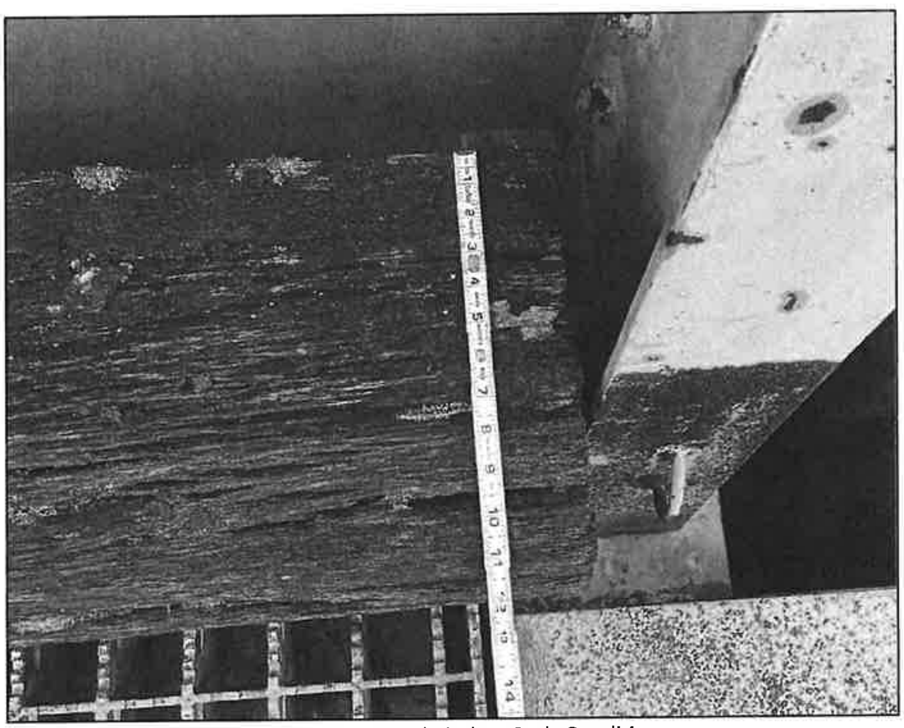


Photo 16 – Typical Timber Curb Condition

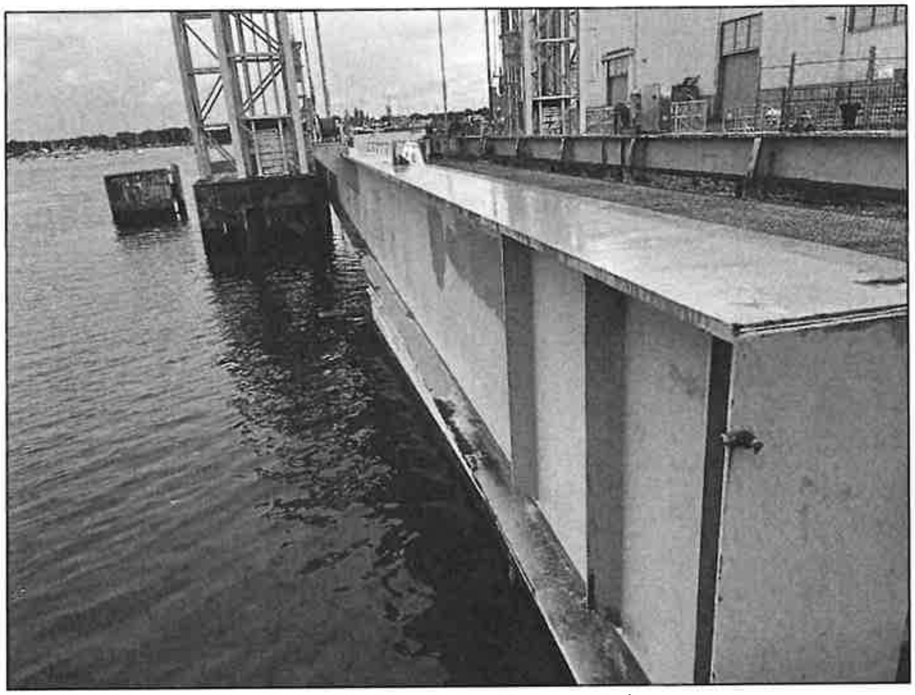


Photo 17 – View of North Ramp Girder

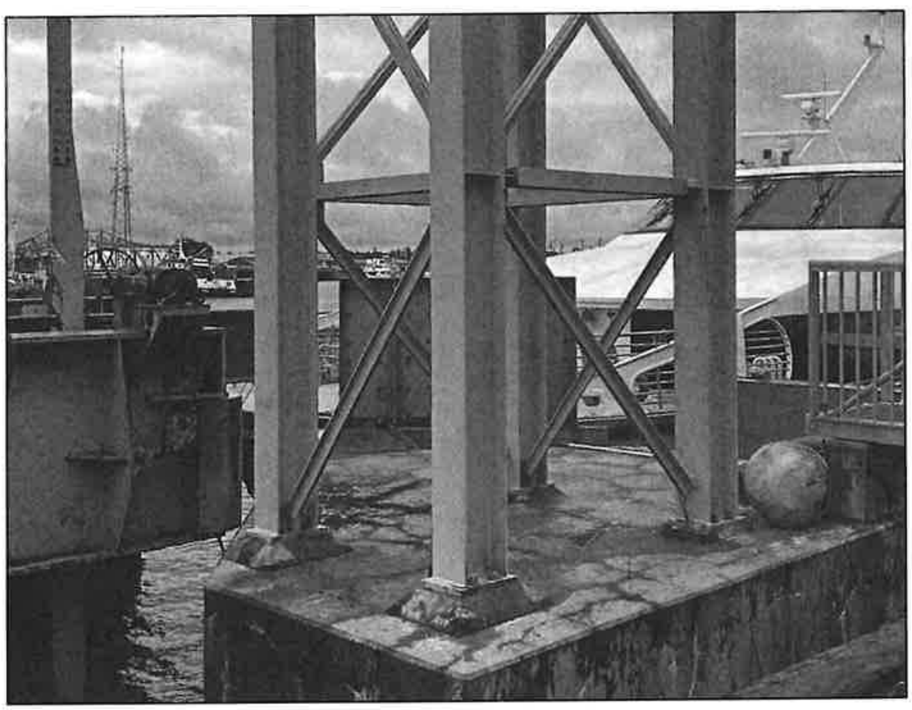


Photo 18 – View of Southeast Hoist Tower Base

Photo 19 – View of Southwest Hoist Tower Base


Photo 20 – Cable reel mounted to the north girder

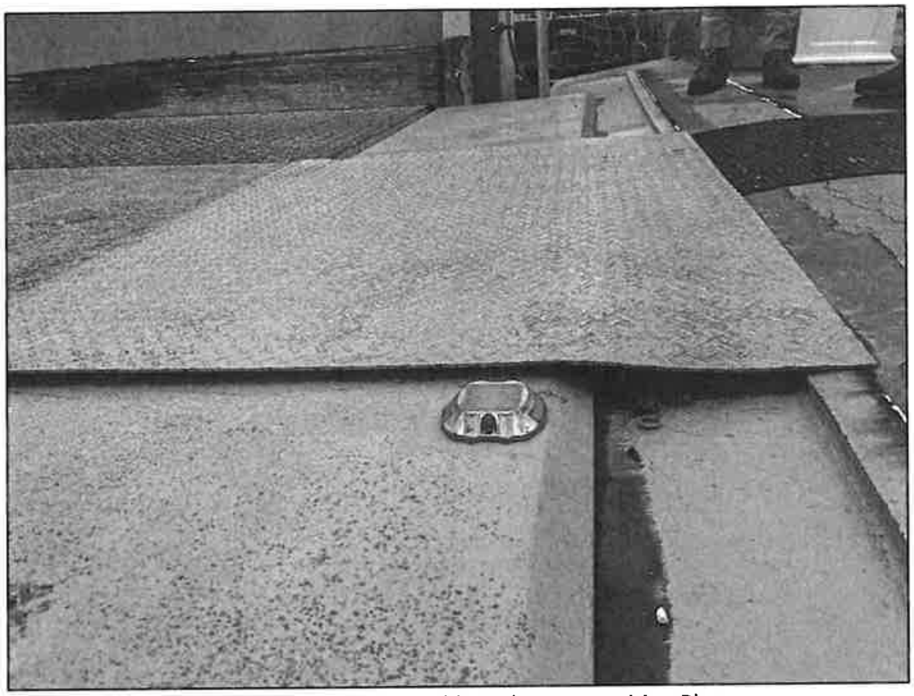
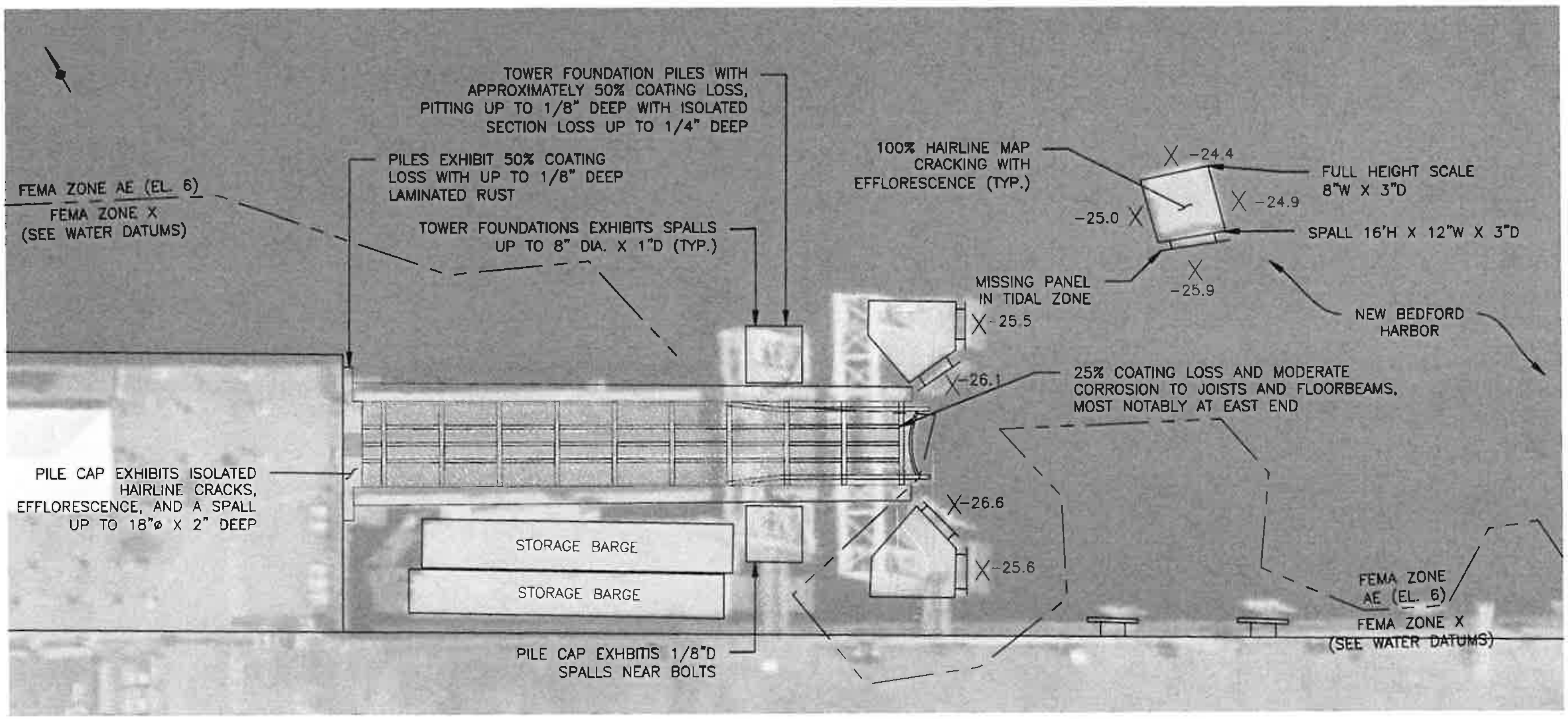


Photo 21 – View of Unstable and Bent Transition Plate

## Appendix B – Report Drawings




SUBSTRUCTURE EXISTING CONDITIONS

NOTES:

1. BASE PLAN AND EXISTING CONDITIONS REFERENCE SITE VISIT COMPLETED BY COLLINS ENGINEERS ON AUGUST 08, 2024.
2. ELEVATIONS REFERENCE MLW UNLESS OTHERWISE NOTED.

WATER DATUMS:

| | | |
|---|---|---|
| FEMA FLOOD (ZONE AE) | 6.00' | — — — — |
| FEMA FLOOD (ZONE X)[1] | | |
| MEAN HIGH WATER (MHW) | 3.60' | |
| MEAN LOW WATER (MLW) | 0.00' | |

REFERENCES:
NOAA: NEW BEDFORD HARBOR, MA STATION #8447636

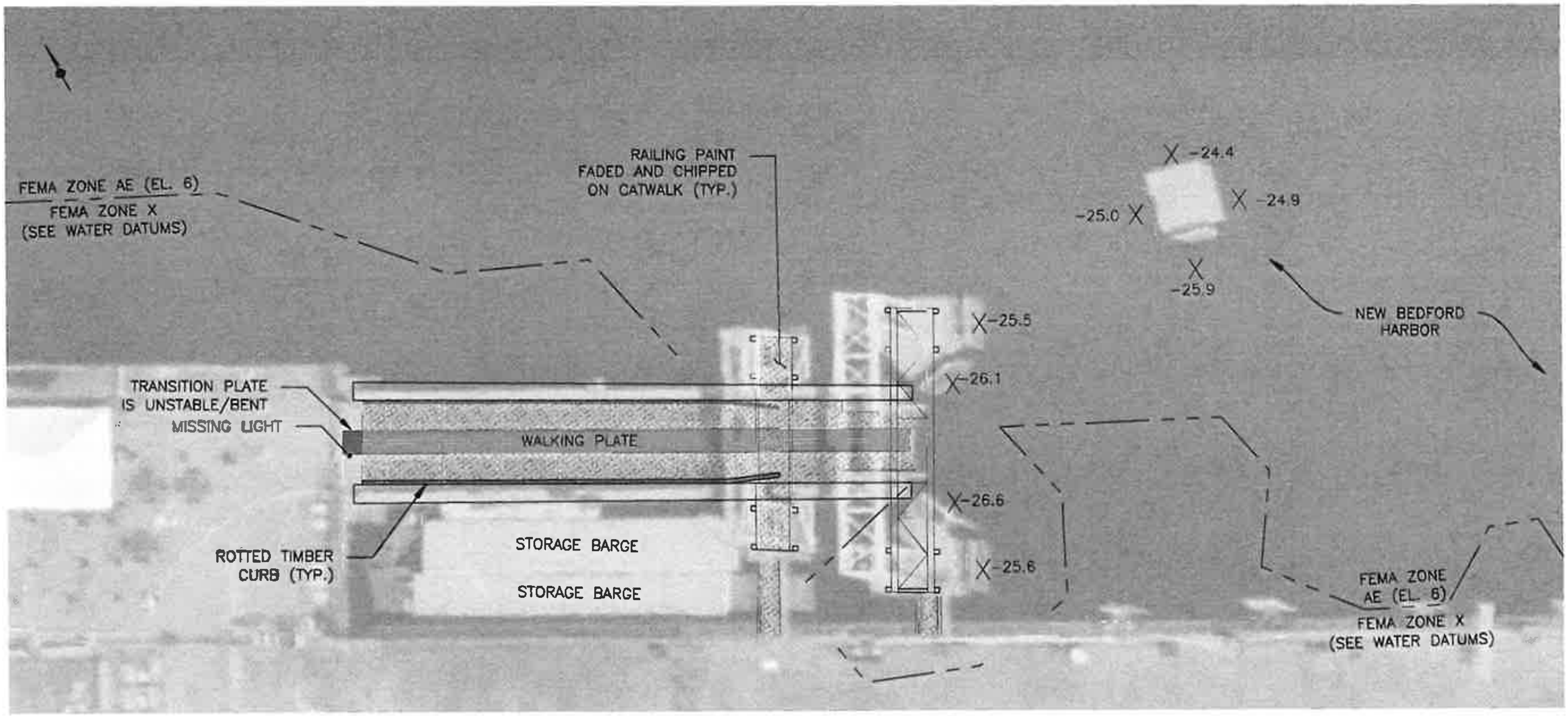
FEMA: FIRM, BRISTOL, COUNTY, MA, ALL JURISDICTIONS PANEL 393 OF 550, MAP #25005C0393G, REVISED JULY 16, 2014

[1] AREA WITH REDUCED FLOOD RISK DUE TO LEVEE

TYPICAL DEFICIENCIES:

1. PILES EXHIBIT PITTING UP TO ⅛" DEEP, MINOR TO MODERATE CORROSION, AND APPROX. 50% COATING LOSS WITH MARINE GROWTH IN THE LOWER SPLASH ZONE.
2. FENDER PANELS EXHIBIT MISSING BOLTS, LAMINATED RUST ON HARDWARE, AND MODERATE TO HEAVY ABRASION DAMAGE (TYP.)
3. CONNECTION BOLTS EXHIBIT MODERATE CORROSION (TYP.)

| GRAPHIC SCALE | DATE | | | |
|---|---|---|---|---|
| 0 20' 40' OR AS INDICATED | SEPTEMBER 2024 | COLLINS ENGINEERS, INC. 1485 S COUNTY TRAIL, STE 103 EAST GREENWICH, RI 02818 COLLINS ENGINEERS | NEW BEDFORD RORO RAMP EXISTING CONDITIONS TOWN OF NEW BEDFORD | FIG. NO. 1 |




## SUPERSTRUCTURE EXISTING CONDITIONS

NOTES:

1. BASE PLAN AND EXISTING CONDITIONS REFERENCE SITE VISIT COMPLETED BY COLLINS ENGINEERS ON AUGUST 08, 2024.
2. ELEVATIONS REFERENCE MLW UNLESS OTHERWISE NOTED.

WATER DATUMS:

| | |
|---|---|
| FEMA FLOOD (ZONE AE) | 6.00' |
| FEMA FLOOD (ZONE X)[1] | |
| MEAN HIGH WATER (MHW) | 3.60' |
| MEAN LOW WATER (MLW) | 0.00' |

REFERENCES:
NOAA: NEW BEDFORD HARBOR, MA STATION #8447636

FEMA: FIRM, BRISTOL, COUNTY, MA, ALL JURISDICTIONS PANEL 393 OF 550, MAP #25005C0393G, REVISED JULY 16, 2014

[1] AREA WITH REDUCED FLOOD RISK DUE TO LEVEE

TYPICAL DEFICIENCIES:

1. DECK EXHIBITS RUST AND CHIPPED PAINT THROUGHOUT
2. TIMBER CURB IN WORST CONDITION NEAR ITS WESTERN EXTENT

| GRAPHIC SCALE | DATE | | |
|---|---|---|---|
| 0 20' 40' OR AS INDICATED | SEPTEMBER 2024 | COLLINS ENGINEERS, INC. 1485 S COUNTY TRAIL, STE 103 EAST GREENWICH, RI 02818 | NEW BEDFORD RORO RAMP EXISTING CONDITIONS TOWN OF NEW BEDFORD FIG. NO. 2 |

**Appendix C – Load Rating Calculation**

COLLINS ENGINEERS INC.

| | |
|---|---|
| Sheet No. | 1 of 16 |
| Project No. | 15-16077.00 |
| Phase No. | |
| Revision No. | |
| Date | 9/20/2024 |

# Design Calculations

**Project Title:** New Bedford State Pier Inspection

**Client:** Mass Development

**Purpose of Calculations:**
The purpose of these calculations is to determine if the existing RoRo ramp is capable of supporting pedestrian and vehicle loadings in New Bedford, Massachusetts. The analysis includes ratings of all elements providing structural support.

**Conclusions:**

1. Load Rating Analysis
   a. Based on the existing condition of the RoRo ramp at the time of inspection (8/8/2024), it may be rated for the State Legal Load of HS-20 and pedestrian access.
   b. The limiting structural element is the grating

**Originated By:** Andrew VanRoy **Date: 9/19/2024**

**Printed Name:** Andrew VanRoy, E.I.T.

**Checked By:** Chris Sylvia **Date: 9/20/2024**

**Printed Name:** Christopher Sylvia, E.I.T.

**Reviewed By:** Ryan McCoy **Date: 9/20/2024**

**Printed Name: Ryan McCoy, P.E.**

*The following calculations were prepared under my direct supervision.*

COMMONWEALTH OF MASSACHUSETTS
RYAN M. McCOY
CIVIL
No. 49958
REGISTERED
PROFESSIONAL ENGINEER

**NOTE: These calculations were prepared for the specific loadings and conditions associated with this specific site. Any reuse of these calculations or structure(s), without the express written consent of Collins Engineers, Inc., will be at the sole risk of the user and without liability or legal exposure to Collins Engineers, Inc.**

# Design Calculations

**Design Codes and References Used in Calculations:**

1. Plans titled "Harbor Development Commission New Bedford State Pier Ferry Terminal, Contract No. 2: Transfer Bridge Modifications and Relocation" Dated December 1999 by Webster Engineering Co., Inc.
2. American Institute of Steel Construction, Steel Construction Manual, 14th Edition
3. Indiana Grating Inc. Structural Specification for 38-W-4 Heavy Duty Welded Steel Grating
4. ACI 318-19 Building Code Requirements for Structural Concrete
5. AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002
6. AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, December 2009

**Assumptions:**

1. The allowable span length of the existing grating is approximately 67% of that for 38-W-4 Heavy Duty Welded Steel Grating with bearing bar size 6"x1/2".
2. Weight of concrete is 150 pcf ($\gamma_C$=150 pcf).
3. Weight of steel is 0.284 lb/in$^3$ ($\gamma_S$=0.284 lb/in$^3$).
4. There have been no significant deviations in the existing structure from the historic plans.
5. The conditions of the existing site remain unchanged since the date of inspection.

## Structural Analysis

Summary:

*This analysis includes a structural assessment of the New Bedford State Pier Roll-on Roll-off Ramp. An inspection conducted by Collins Engineers on August 8, 2024, indicated defects that presently effect the structural capacity of the structure, as defined below.*

## Section A: Inputs and Loadings

Geometry Input:

| | |
|---|---|
| $L_J := 13.75\ ft$ | Length of Joist per span |
| $W_J := 4\ ft$ | Joist spacing |
| $L_{Girder} := 112\ ft$ | Girder Length |
| $L_{JT} := 13.75\ ft$ | Length of T-Joist per span |
| $L_B := 20\ ft$ | Length of Beam |
| $A_{Grating} := L_J \cdot W_J = 55\ ft^2$ | Max. Unsupported Grating Area |
| $L_{Pile} := 40\ ft$ | Pile Length (PLACEHOLDER) |
| $d_{Piles} := 16\ in$ | Pile Diameter |
| $\gamma_C := 150\ pcf$ | Unit weight of concrete (assumed) |
| $\gamma_S := 0.284\ \frac{lbf}{in^3}$ | Unit weight of steel (assumed) |

Structural Strengths

| | |
|---|---|
| $f_y := 36\ ksi$ | Yield strength of steel per as-built drawings |
| $E := 29000\ ksi$ | Steel Modulus of Elasticity |
| $F_{LL} := 1.6$ | Live load factor, *UFC 4-152-01 Table 3-7* |
| $F_{DL} := 1.2$ | Dead load factor, *UFC 4-152-01 Table 3-7* |

Loadings

| | |
|---|---|
| $P_{LL\_Pedestrian} := 90\ psf$ | Pedestrian Live Load, *AASHTO LRFD Guide Specs for the Design of Pedestrian Bridges, Section 3.1* |
| $P_{LL\_Pedestrian\_LF} := P_{LL\_Pedestrian} \cdot W_J = 360\ \frac{lbf}{ft}$ | Pedestrian Live Load per foot of joist |
| $P_{LL\_HS20} := 16\ kip$ | HS20 Live Load for (1) wheel, *AASHTO Standard Specs for Highway Bridges, 17th Edition* |
| $\phi_b := 0.9$ | Moment Capacity reduction factor |

$\phi_v := 0.75$ — Shear Capacity reduction factor

$x_{HS20} := 6\ ft$ — HS20 wheel spacing

## Section B: Load Ratings

### Grating

$W_{Grating} := 41.21\ psf$ — Grating weight for 6"x3/8" bearing bar grating, *Indiana Gratings Inc.*

$P_{DL\_Grating} := F_{DL} \cdot W_{Grating} = 49.452\ psf$ — Grating self weight

$D_{Allow\_HS20} := 96\ in$ — Max distance between spans that grating can support HS20 loading, *Indiana Gratings Inc.*

$D_{Allow\_HS20\_Reduced} := D_{Allow\_HS20} \cdot 0.67 = 5.36\ ft$ — Reduction of allowable span based on bearing bar spacing

$FS_{Grating} := \frac{D_{Allow\_HS20\_Reduced}}{W_J} = 1.34$ — Factor of Safety

$Check := \text{if}\left(FS_{Grating} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

*The existing grating is capable of supporting HS20 and pedestrian loadings*

### Joists

Known Values

$W_{Joist} := 93\ plf$ — Weight of W21X93 Joist, *AISC Table 3.2*

$Z_{x\_Joist} := 221\ in^3$ — Plastic section modulus about the x-axis, *AISC Table 3.2*

$r_{y\_Joist} := 1.84\ in$ — Radius of Gyration about the y-axis, *AISC Table 1.1*

$J_{Joist} := 6.03\ in^4$ — Torsional Constant, *AISC Table 1.1*

$c := 1$ — Constant for doubly symmetric shapes, *AISC Equation F2-8a*

$h_{o\_Joist} := 20.7\ in$ — Distance between flange centroids, *AISC Table 1.1*

$S_{x\_Joist} := 192\ in^3$ — Elastic modulus about the x-axis, *AISC Table 1.1*

$r_{ts\_Joist} := 2.24\ in$ — Effective radius of gyration, *AISC Table 1.1*

$C_{B\_Joist} := 1.32$ — Bending modification factor, *AISC Table 3-1*

$b_{f\_Joist} := 8.42\ in$ — Flange Width, *AISC Table 1-1*

$t_{f\_Joist} := 0.930\ in$ — Flange Thickness, *AISC Table 1-1*

$h_{w\_Joist} := 18.375\ in$ — Clear distance between flanges, *AISC Chapter 1.1*

$t_{w\_Joist} := 0.580\ in$ — Thickness of the web, *AISC Chapter 1.1*

**Determine Section Classification**

$\lambda_{pf\_Joist} := 0.38 \cdot \sqrt[2]{\dfrac{E}{f_y}} = 10.785$ — Flange Lower limit of Limiting Width-to-Thickness Ratio for Flange, *AISC Table B4.1(b)*

$Ratio_{Flange\_Joist} := \dfrac{\dfrac{b_{f\_Joist}}{2}}{t_{f\_Joist}} = 4.527$ — Flange Width-to-Thickness Ratio

$\boxed{Check} := \text{if}\left(Ratio_{Flange\_Joist} < \lambda_{pf\_Joist}, \text{“Compact in Flange”}, \text{“Check Upper Limit”}\right) = \text{“Compact in Flange”}$

$\lambda_{pw\_Joist} := 3.76 \cdot \sqrt[2]{\dfrac{E}{f_y}} = 106.717$ — Web Lower limit of Limiting Width-to-Thickness Ratio for Web, *AISC Table B4.1(b)*

$Ratio_{Web\_Joist} := \dfrac{h_{w\_Joist}}{t_{w\_Joist}} = 31.681$ — Web Width-to-Thickness Ratio

$\boxed{Check} := \text{if}\left(Ratio_{Web\_Joist} < \lambda_{pw\_Joist}, \text{“Compact in Web”}, \text{“Check Upper Limit”}\right) = \text{“Compact in Web”}$

**Determine Allowable Moment**

$L_{p\_Joist} := 1.76 \cdot r_{y\_Joist} \cdot \sqrt[2]{\dfrac{E}{f_y}} = 7.659\ ft$ — Limiting laterally unbraced length for yielding, *AISC Equation F2-5*

$L_{r\_Joist} := 1.95 \cdot r_{ts\_Joist} \cdot \dfrac{E}{0.7\ f_y} \cdot \sqrt[2]{\dfrac{J_{Joist} \cdot c}{S_{x\_Joist} \cdot h_{o\_Joist}} + \sqrt[2]{\left(\dfrac{\left(J_{Joist} \cdot c\right)}{S_{x\_Joist} \cdot h_{o\_Joist}}\right)^2 + 6.76 \cdot \left(\dfrac{\left(0.7 \cdot f_y\right)}{E}\right)^2}} = 27.272\ ft$ — Limiting laterally unbraced length for LTB, *AISC Equation F2-6*

$\boxed{Check} := \text{if}\left(L_J \le L_{p\_Joist}, \text{“LTB does not apply”}, \text{“LTB applies”}\right) = \text{“LTB applies”}$

$\boxed{Check} := \text{if}\left(L_{p\_Joist} < L_J < L_{r\_Joist}, \text{“Use Equation F2–2”}, \text{“Use Equation F2–3”}\right) = \text{“Use Equation F2–2”}$

$M_{p\_Joist} := f_y \cdot Z_{x\_Joist} = 663\ kip \cdot ft$ — Plastic moment, *AISC Chapter F2.1*

$M_{n\_Yield\_Joist} := M_{p\_Joist} = 663\ kip \cdot ft$ — Nominal flexural strength in yielding

$M_{n\_LTB\_Joist} := C_{B\_Joist} \cdot \left(M_{p\_Joist} - \left(M_{p\_Joist} - 0.7 \cdot f_y \cdot S_{x\_Joist}\right)\left(\dfrac{L_J - L_{p\_Joist}}{L_{r\_Joist} - L_{p\_Joist}}\right)\right) = 768.662\ kip \cdot ft$

$\boxed{Check} := \text{if}\left(M_{n\_Yield\_Joist} < M_{n\_LTB\_Joist}, \text{“Yielding Controls”}, \text{“LTB Controls”}\right) = \text{“Yielding Controls”}$

$\phi_b M_{n\_Joist} := \phi_b \cdot M_{n\_Yield\_Joist} = 596.7\ kip \cdot ft$ — Reduced Nominal Flexural Strength

Determine Allowable Shear

$A_{W\_Joist} := 11.45\ in^2$ — Area of the web, *AISC Table 1.1*

h/tw:=32.3 — Compact Section Criteria, *AISC Table 1.1*

*Joist meets compact section criteria of* $h/t_w \le 2.24 \cdot \sqrt{\frac{E}{f_y}}$ — Constants, *AISC G2.1(a)*

$\therefore\ \phi_{v\_Joist} := 1.00$

$k_v := 5$ — Constant, *AISC Chapter G.2*

*Beam meets compact section criteria of* $h/t_w \le 1.10 \cdot \sqrt{k_v \cdot \frac{E}{f_y}}$ — *AISC G2.1(b)*

$\therefore\ C_{v\_Joist} := 1.0$

$V_{n\_Joist} := 0.6 \cdot f_y \cdot A_{W\_Joist} \cdot C_{v\_Joist} = 247.32\ kip$ — Nominal Shear Strength, *AISC Equation G2-1*

$\phi_v V_{n\_Joist} := \phi_{v\_Joist} \cdot V_{n\_Joist} = 247.32\ kip$ — Reduced Nominal Shear Strength

Determine Maximum Moment and Shear Loadings

$P_{DL\_Joist} := P_{DL\_Grating} \cdot A_{Grating} + F_{DL} \cdot (W_{Joist} \cdot L_J) = 4.254\ kip$ — Dead Load as a point force

*Assume pedestrian and HS-20 loadings occur at the center of the joist for maximum moment*

**Scenario 1 - Pedestrian Loading**

$P_{LL\_Joist\_P} := F_{LL} \cdot (P_{LL\_Pedestrian\_LF} \cdot L_J) = 7.92\ kip$ — Pedestrian Live Load as a point force

$P_{Load\_Joist\_P} := P_{DL\_Joist} + P_{LL\_Joist\_P} = 12.174\ kip$ — Pedestrian Total Loading

$M_{Max\_Joist\_P} := \frac{P_{Load\_Joist\_P} \cdot L_J}{4} = 41.849\ kip \cdot ft$ — Maximum moment, *AISC Table 3-23.7*

$FS_{M\_Joist\_P} := \frac{\phi_b M_{n\_Joist}}{M_{Max\_Joist\_P}} = 14.258$ — Factor of Safety in Moment

$Check := \text{if}\left(FS_{M\_Joist\_P} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$V_{Max\_P} := \frac{P_{Load\_Joist\_P}}{2} = 6.087\ kip$ — Maximum Shear Force, *AISC Table 3-23.7*

$FS_{V\_Joist\_P} := \frac{\phi_v V_{n\_Joist}}{V_{Max\_P}} = 40.63$ — Factor of Safety in Shear

$Check := \text{if}\left(FS_{V\_Joist\_P} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

**Scenario 2 - HS-20 Loading**

$P_{LL\_Joist\_HS20} := F_{LL} \cdot (P_{LL\_HS20}) = 25.6\ kip$ — HS20 Live Load

$P_{Load\_Joist\_HS20} := P_{DL\_Joist} + P_{LL\_Joist\_HS20} = 29.854\ kip$ — HS20 Total Load

$M_{Max\_Joist\_HS20} := \frac{P_{Load\_Joist\_HS20} \cdot L_J}{4} = 102.624\ kip \cdot ft$ — Maximum moment, *AISC Table 3-23.7*

$FS_{M\_Joist\_HS20} := \frac{\phi_b M_{n\_Joist}}{M_{Max\_Joist\_HS20}} = 5.814$ — Factor of Safety in Moment

$Check := \text{if}\left(FS_{M\_Joist\_P} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$V_{Max\_HS20} := \frac{P_{Load\_Joist\_HS20}}{2} = 14.927\ kip$ — Maximum Shear Force, *AISC Table 3-23.7*

$FS_{V\_Joist\_HS20} := \frac{\phi_v V_{n\_Joist}}{V_{Max\_HS20}} = 16.568$ — Factor of Safety in Shear

$Check := \text{if}\left(FS_{V\_Joist\_HS20} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

**Beams**

Known Values

$W_{Beam} := 130\ \frac{lbf}{ft}$ — Weight of W33x130 Beam

$L_{B\_Unbraced} := 4\ ft$ — Unbraced length of beam

$Z_{x\_Beam} := 467\ in^3$ — Plastic Section Modulus about the x-axis, *AISC Table 1.1*

$r_{y\_Beam} := 2.39\ in$ — Radius of Gyration about the y-axis, *AISC Table 1.1*

$J_{Beam} := 7.37\ in^4$ — Torsional Constant, *AISC Table 1.1*

$c_{Beam} := 1$ — Constant for doubly symmetric shapes, *AISC Equation F2-8a*

$h_{o\_Beam} := 32.2\ in$ — Elastic Modulus about the x-axis, *AISC Table 1.1*

$b_{f\_Beam} := 11.5\ in$ — Flange Width, *AISC Table 1.1*

$t_{f\_Beam} := 0.855\ in$ — Flange Thickness, *AISC Table 1.1*

$h_{w\_Beam} := 28.875\ in$ — Distance between Flanges, *AISC Table 1.1*

$t_{w\_Beam} := 0.580\ in$ — Web thickness, *AISC Table 1.1*

Determine Section Classification

$S_{x\_Beam} := 406\ in^3$ — Elastic modulus about the x-axis, *AISC Table 1.1*

$r_{ts\_Beam} := 2.94\ in$ — Effective Radius of Gyration, *AISC Table 1.1*

$\lambda_{pf\_Beam} := 0.38 \cdot \sqrt[2]{\frac{E}{f_y}} = 10.785$ — Flange Lower limit of Limiting Width-to-Thickness Ratio for Flange, *AISC Table B4.1(b)*

$Ratio_{Flange\_Beam} := \frac{\frac{b_{f\_Beam}}{2}}{t_{f\_Beam}} = 6.725$ — Flange Width-to-Thickness Ratio

$\boxed{Check} :=$ **if** $(Ratio_{Flange\_Beam} < \lambda_{pf\_Beam}$, "Compact in Flange", "Check Upper Limit") = "Compact in Flange"

$\lambda_{pw\_Beam} := 3.76 \cdot \sqrt[2]{\frac{E}{f_y}} = 106.717$ — Web Lower limit of Limiting Width-to-Thickness Ratio for Web, *AISC Table B4.1(b)*

$Ratio_{Web\_Beam} := \frac{h_{w\_Beam}}{t_{w\_Beam}} = 49.784$ — Web Width-to-Thickness Ratio

$\boxed{Check} :=$ **if** $(Ratio_{Web\_Beam} < \lambda_{pw\_Beam}$, "Compact in Web", "Exceeds Upper Limit") = "Compact in Web"

## Determine Allowable Moment

$L_{p\_Beam} := 1.76 \cdot r_{y\_Beam} \cdot \sqrt[2]{\frac{E}{f_y}} = 9.949\ ft$ — Limiting laterally unbraced length for yielding, *AISC Equation F2-5*

$\boxed{Check} :=$ **if** $(L_{B\_Unbraced} \le L_{p\_Beam}$, "LTB does not apply", "LTB applies") = "LTB does not apply"

$M_{p\_Beam} := f_y \cdot Z_{x\_Beam} = (1.401 \cdot 10^3)\ kip \cdot ft$ — Plastic moment, *AISC Chapter F2.1*

$M_{n\_Yield\_Beam} := M_{p\_Beam} = (1.401 \cdot 10^3)\ kip \cdot ft$ — Nominal flexural strength

$\phi_b M_{n\_Beam} := \phi_b \cdot M_{n\_Yield\_Beam} = (1.261 \cdot 10^3)\ kip \cdot ft$ — Reduced nominal flexural strength

## Determine Allowable Shear

$A_{W\_Beam} := 18.21\ in^2$ — Area of the web, *AISC Table 1.1*

$h/t_w := 51.7$ — Compact Section Criteria, *AISC Table 1.1*

*Beam meets compact section criteria of* $h/t_w \le 2.24 \cdot \sqrt[2]{\frac{E}{f_y}}$ — *AISC G2.1(a)*

$\therefore\ \phi_{v\_Beam} := 1.00$

$\boxed{k_v} := 5$ — Constant, *AISC Chapter G.2*

*Beam meets compact section criteria of* $h/t_w \le 1.10 \cdot \sqrt[2]{k_v \cdot \frac{E}{f_y}}$ — Constant, *AISC G2.1(b)*

$\therefore\ C_{v\_Beam} := 1.0$

$V_{n\_Beam} := 0.6 \cdot f_y \cdot A_{W\_Beam} \cdot C_{v\_Beam} = 393.336\ kip$ — Nominal Shear Strength, *AISC Equation G2-1*

$\phi_v V_{n\_Beam} := \phi_{v\_Beam} \cdot V_{n\_Beam} = 393.336\ kip$ — Reduced Nominal Shear Strength

## Determine Maximum Moment and Shear Loadings

$P_{DL\_Beam\_Self} := F_{DL} \cdot W_{Beam} = 156\ \frac{lbf}{ft}$ — Self weight dead load acting on the beam as a distributed force

$P_{DL\_Beam\_Self\_Point} := P_{DL\_Beam\_Self} \cdot L_B = 3.12\ kip$ — Self weight dead load acting as a point force

*Assume the one wheel from the HS20 load is located at center of floorbeam and other wheel is offset 6'*

$P_{LL\_HS20\_Beam} := F_{LL} \cdot P_{LL\_HS20} = 25.6\ kip$ Live load

$$R_1 := \frac{\left(\begin{array}{l}(P_{DL\_Joist} \cdot 2\ ft) + (P_{DL\_Joist} \cdot 6\ ft) + (P_{DL\_Joist} \cdot 10\ ft) + (P_{DL\_Joist} \cdot 14\ ft) \\ + (P_{DL\_Joist} \cdot 18\ ft) + (P_{LL\_HS20\_Beam} \cdot 10\ ft) + (P_{LL\_HS20\_Beam} \cdot 16\ ft)\end{array}\right)}{20\ ft} = 43.916\ kip$$

$R_2 := (P_{DL\_Joist} \cdot 5) + (2 \cdot P_{LL\_HS20\_Beam}) - R_1 = 28.556\ kip$ Reaction Forces

*Shear = 0 at center of beam, therefore this is max moment*

$$M_{Max\_Beam} := \left(P_{DL\_Beam\_Self} \cdot \frac{L_B}{2} \cdot \frac{L_B}{4}\right) + \left(P_{LL\_HS20\_Beam} \cdot \frac{L_B}{2}\right) + (P_{DL\_Joist} \cdot 2\ ft) + (P_{DL\_Joist} \cdot 6\ ft) + \left(P_{DL\_Joist} \cdot \frac{L_B}{2}\right) = 340.378\ ft \cdot kip$$

Maximum moment acting on beam

$V_{Max\_Beam} := R_1 = 43.916\ kip$ Maximum shear force acting on beam

$FS_{Beam\_M} := \frac{\phi_b M_{n\_Beam}}{M_{Max\_Beam}} = 3.704$ Factor of safety in moment

$FS_{Beam\_V} := \frac{\phi_v V_{n\_Beam}}{V_{Max\_Beam}} = 8.957$ Factor of safety in shear

$Check := \text{if}\,(FS_{Beam\_M} > 1.2, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$Check := \text{if}\,(FS_{Beam\_V} > 1.2, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

*The beams are capable of supporting vehicles and pedestrian loads*

## Girders

### Known Values

$b_{f\_Girder} := 2\ ft$ Width of the flange

$t_{f\_Girder} := 1.75\ in$ Thickness of the flange

$b_{w\_Girder} := 6\ ft$ Width of the web

$t_{w\_Girder} := 0.5\ in$ Thickness of the web

$h_{Girder} := t_{f\_Girder} + b_{w\_Girder} = 6.146\ ft$ Total length of beam $I_{y\_Gir}$

$L_{b\_Unbraced\_Girder} := 13.75\ ft$ Unbraced girder length

### Calculated Characteristics

$$I_{y\_Girder} := \frac{b_{f\_Girder} \cdot (b_{w\_Girder} + 2 \cdot t_{f\_Girder})^3}{12} - \frac{(b_{f\_Girder} - t_{w\_Girder}) \cdot ((b_{w\_Girder} + 2 \cdot t_{f\_Girder}) - (2 \cdot t_{w\_Girder}))^3}{12} = (5.098 \cdot 10^4)\ in^4$$

Moment of inertia about the weak axis

$$A_{Girder} := 2 \cdot (t_{f\_Girder} \cdot b_{f\_Girder}) + (b_{w\_Girder} \cdot t_{w\_Girder}) = 0.833\ ft^2$$

Cross-sectional area

$$r_{y\_Girder} := \sqrt[2]{\frac{I_{y\_Girder}}{A_{Girder}}} = 20.611\ in$$

Radius of gyration about the weak axis

$$S_{x\_Girder} := \frac{I_{y\_Girder}}{\frac{1}{2} \cdot h_{Girder}} = 5.985\ gal$$

Elastic section modulus about the weak axis

$$A_f := t_{f\_Girder} \cdot b_{f\_Girder} = 42\ in^2$$

Flange Area

$$A_w := t_{w\_Girder} \cdot b_{w\_Girder} = 36\ in^2$$

Web Area

$$y_f := b_{w\_Girder} + \frac{t_{f\_Girder}}{2} = 72.875\ in$$

Flange Centroid

$$y_w := \frac{b_{w\_Girder}}{2} = 36\ in$$

Web Centroid

$$y_{NA} := \frac{A_f \cdot y_f + A_w \cdot y_w}{A_f + A_w} = 55.856\ in$$

Neutral Axis location

$$A_{w1} := (b_{w\_Girder} - y_{NA}) \cdot t_{w\_Girder} = 8.072\ in^2$$

Upper web area

$$d_{w1} := \frac{b_{w\_Girder} - y_{NA}}{2} = 8.072\ in$$

Distance from neutral axis to uppwer web centroid

$$A_{w2} := y_{NA} \cdot t_{w\_Girder} = 27.928\ in^2$$

Lower Web Area

$$d_{w2} := \frac{y_{NA}}{2} = 27.928\ in$$

Distance from neutral axis to lower web centroid

$$Z_{f\_Girder} := A_f \cdot (y_f - y_{NA}) = 3.094\ gal$$

Flange plastic section modulus

$$Z_{w1} := A_{w1} \cdot d_{w1} = 0.282\ gal$$

Upper web plastic section modulus

$$Z_{w2} := A_{w2} \cdot d_{w2} = 3.376\ gal$$

Lower web plastic section modulus

$$Z_x := Z_{f\_Girder} + Z_{w1} + Z_{w2} = 6.753\ gal$$

Total plastic section modulus

## Determine Section Classification

$$\lambda_{pf\_Girder} := 0.38 \cdot \sqrt[2]{\frac{E}{f_y}} = 10.785$$

Flange Lower limit for Width-to-Thickness Ratio

$$Ratio_{Flange\_Girder} := \frac{\frac{b_{f\_Beam}}{2}}{t_{f\_Girder}} = 3.286$$

Flange Width-to-Thickness ratio

$Check := \text{if}\left(Ratio_{Flange\_Girder} < \lambda_{pf\_Girder}, \text{"Compact in Flange"}, \text{"Check Upper Limit"}\right) = \text{"Compact in Flange"}$

$\lambda_{pw\_Girder} := 3.76 \cdot \sqrt{\frac{E}{f_y}} = 106.717$ Web Lower limit of Limiting Width-to-Thickness Ratio, *Table B4-1b*

$Ratio_{Web\_Girder} := \frac{b_{w\_Girder}}{t_{w\_Girder}} = 144$ Web Width-to-Thickness Ratio

$Check := \text{if}\left(Ratio_{Web\_Girder} < \lambda_{pw\_Girder}, \text{"Compact in Web"}, \text{"Exceeds Lower Limit"}\right) = \text{"Exceeds Lower Limit"}$

$\lambda_{rw\_Girder} := 5.70 \cdot \sqrt{\frac{E}{f_y}} = 161.779$ Web Upper limit of Limiting Width-to-Thickness Ratio, *Table B4-1b*

$Check := \text{if}\left(Ratio_{Web\_Girder} < \lambda_{rw\_Girder}, \text{"Noncompact in Web"}, \text{"Slender in Web"}\right) = \text{"Noncompact in Web"}$

$L_{p\_Girder} := 1.76 \cdot r_{y\_Girder} \cdot \sqrt{\frac{E}{f_y}} = 85.8\ ft$ Lower limit of limiting length

$Check := \text{if}\left(L_{b\_Unbraced\_Girder} < L_{p\_Girder}, \text{"LTB does not apply"}, \text{"LTB is applicable"}\right) = \text{"LTB does not apply"}$

Determine Allowable Moment

$I_{yc} := \frac{t_{f\_Girder} \cdot b_{f\_Girder}^{3}}{12} = \left(2.016 \cdot 10^{3}\right)\ in^{4}$ Moment of intertia of the compression flange about the y-axis

$Check := \text{if}\left(\frac{I_{yc}}{I_{y\_Girder}} < 0.23, \text{"Use Eqn F4.2(c)(6)(ii)"}, \text{"Use Eqn F4.2(c)(6)(i)"}\right) = \text{"Use Eqn F4.2(c)(6)(ii)"}$

$R_{pc} := 1.0$ Web plastification factor, *AISC F4.2(c)(6)(ii)*

$I_{xc} := \frac{b_{f\_Girder} \cdot t_{f\_Girder}^{3}}{12} + A_f \cdot y_f^{2} = \left(2.231 \cdot 10^{5}\right)\ in^{4}$ Moment of intertia of the compression flange about the x-axis

$S_{xc} := \frac{I_{xc}}{\left(y_f - y_{NA}\right)} = 56.738\ gal$ Elastic section modulus referred to compression flange,

$M_{yc} := f_y \cdot S_{xc} = \left(3.932 \cdot 10^{4}\right)\ ft \cdot kip$ Yield moment in the compression flange

$M_{n\_Girder} := M_{yc} \cdot R_{pc} = \left(3.932 \cdot 10^{4}\right)\ ft \cdot kip$ Nominal flexural moment

$\phi_b M_{n\_Girder} := \phi_b \cdot M_{n\_Girder} = \left(3.539 \cdot 10^{4}\right)\ ft \cdot kip$ Reduced nominal flexural moment

Determine Allowable Shear

$a := 13\ ft$ Distance between transverse stiffeners

$k_{v\_Girder} := 5 + \frac{5}{\left(\frac{a}{b_{w\_Girder}}\right)^{2}} = 6.065$ Constant, *AISC Equation G2-5*

$Check := \text{if}\left(\frac{b_{w\_Girder}}{t_{w\_Girder}} < 1.10 \cdot \sqrt{k_v \cdot \frac{E}{f_y}}, \text{"Use Eqn G2-3"}, \text{"Use Eqn G2-4"}\right) = \text{"Use Eqn G2-4"}$

$$C_{v\_Girder} := \frac{1.10 \cdot \sqrt[2]{k_v \cdot \frac{E}{f_y}}}{\frac{b_{w\_Girder}}{t_{w\_Girder}}} = 0.485$$

Constant, *AISC Equation G2-4*

$$V_n := 0.6 \cdot f_y \cdot A_w \cdot C_{v\_Girder} = 376.981 \ \boldsymbol{kip}$$

Nominal shear strength, *AISC Equation G2-1*

$$\phi_v V_{n\_Girder} := \phi_v \cdot V_n = 282.736 \ \boldsymbol{kip}$$

Reduced nominal shear strength

Determine Maxmium Moment and Shear Loadings

$$P_{DL\_Girder\_Self} := F_{DL} \cdot \left(A_{Girder} \cdot \gamma_S\right) = 490.752 \ \frac{\boldsymbol{lbf}}{\boldsymbol{ft}}$$

Dead load of the girder's weight acting as a distributed load

$$P_{DL\_Beams} := \frac{\left(P_{DL\_Beam\_Self\_Point} + P_{DL\_Joist}\right)}{2} = 3.687 \ \boldsymbol{kip}$$

Dead loads from the above members (Halved for one girder)

$$P_{LL\_HS20\_Girder} := F_{LL} \cdot P_{LL\_HS20} \cdot 3 = 76.8 \ \boldsymbol{kip}$$

Assume 3 wheel loads per girder

*Assume the HS20 load is located in the middle of hoists and abutment for maximum moment*

$$R_{Girder\_West} := \frac{-\left(\begin{array}{l} 23.25 \cdot \frac{P_{DL\_Beams}}{2} + P_{DL\_Beams} \cdot 12.5 - P_{DL\_Beams} \cdot 0.25 \\ -14 \cdot P_{DL\_Beams} - 27.75 \cdot P_{DL\_Beams} - 27 \cdot \left(P_{DL\_Girder\_Self} \cdot L_{Girder}\right) \\ -34.25 \cdot P_{LL\_HS20\_Girder} - 41.15 \cdot P_{DL\_Beams} - 48.5 \cdot P_{LL\_HS20\_Girder} \\ -55.25 \cdot P_{DL\_Beams} - 69 \cdot P_{DL\_Beams} \\ -82.75 \cdot P_{DL\_Beams} \end{array}\right)}{83} = 106.266 \ \boldsymbol{kip}$$

Vertical reaction force at abutment by taking moment about hoists

$$R_{Girder\_East} := 2 \cdot \left(\frac{P_{DL\_Beams}}{2}\right) + 7 \cdot \left(P_{DL\_Beams}\right) + \left(P_{DL\_Girder\_Self} \cdot L_{Girder}\right) + \left(2 \cdot P_{LL\_HS20\_Girder}\right) - R_{Girder\_West} = 131.795 \ \boldsymbol{kip}$$

Vertical reaction force at hoists

$$M_{Max\_Girder} := -\left(7 \ \boldsymbol{ft} \cdot P_{LL\_HS20}\right) - \left(157.25 \ \boldsymbol{ft} \cdot P_{DL\_Beams}\right) - \left(35.25 \ \boldsymbol{ft} \cdot P_{DL\_Girder\_Self} \cdot L_{Girder}\right) + \left(42 \ \boldsymbol{ft} \cdot R_{Girder\_East}\right) = \left(2.906 \cdot 10^3\right) \ \boldsymbol{ft} \cdot \boldsymbol{kip}$$

Maximum moment acting on girder

$$V_{Max\_Girder} := R_{Girder\_East} = 131.795 \ \boldsymbol{kip}$$

Maximum shear force acting on girder

$$FS_{Girder\_M} := \frac{\phi_b M_{n\_Girder}}{M_{Max\_Girder}} = 12.177$$

Factor of safety in moment

$$Check := \text{if}\left(FS_{Girder\_M} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$FS_{Girder\_V} := \frac{\phi_v V_{n\_Girder}}{V_{Max\_Girder}} = 2.145$$

Factor of safety in shear

$$Check := \text{if}\left(FS_{Girder\_V} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

*The girders can support both pedestrian and vehicle loads*

**Piles**

*Piles consists of 16" diameter, concrete filled pipe piles with a capacity of 60 tons per as-built drawings*

Determine Allowable Axial Compression

$\phi_c := 0.9$ — Assume piles have 90% of their capacity remaining

$P_n := 120\ kip$ — Design capacity of piles per as-built drawings

$\phi_c P_n := \phi_c \cdot P_n = 108\ kip$ — Allowable compression strength per pile

$P_{n\_Piles\_Total\_Abutment} := 7 \cdot \phi_c P_n = 756\ kip$ — *Total resistance from (7) piles at abutment*

$P_{n\_Piles\_Total\_Hoist} := 4 \cdot \phi_c P_n = 432\ kip$ — *Total resistance from (4) piles at hoist*

Determine Axial Loadings

Abutment Weight

$L_{Upper\_Segment} := 29\ ft$

$w_{Upper\_Segment} := 3\ ft$

$H_{Upper\_Segment} := 3.917\ ft$

$V_{Upper\_Segment} := L_{Upper\_Segment} \cdot w_{Upper\_Segment} \cdot H_{Upper\_Segment} = \left(2.549 \cdot 10^{3}\right)\ gal$

$L_{Lower\_Segment} := 29\ ft$

$w_{Lower\_Segment} := 6.5\ ft$

$H_{Lower\_Segment} := 8.208\ ft$

$V_{Lower\_Segment} := L_{Lower\_Segment} \cdot w_{Lower\_Segment} \cdot H_{Lower\_Segment} = \left(1.157 \cdot 10^{4}\right)\ gal$

$V_{Abutment\_Total} := V_{Upper\_Segment} + V_{Lower\_Segment} = \left(1.412 \cdot 10^{4}\right)\ gal$

$W_{Abutment\_Total} := V_{Abutment\_Total} \cdot \gamma_C = 283.198\ kip$

Weight of the abutment, *as-built drawings*

Ramp Weight

$A_{grating} := (112 - 4.75)\ ft \cdot 26\ ft = \left(2.789 \cdot 10^{3}\right)\ ft^2$ — Total area of the grating

$W_{Grating\_Total} := W_{Grating} \cdot A_{grating} = 114.914\ kip$ — Weight of the grating, *as-built drawings*

$L_{Joists} := 5 \cdot (112 - 5.5)\ ft = 532.5\ ft$ — Total length of 5 joists

$W_{Joists\_Total} := W_{Joist} \cdot L_{Joists} = 49.523\ kip$ — *Weight of the Joists*

$L_{Beam} := 20\ ft$ — Length of beams

$W_{Beam\_West} := 132\ \dfrac{lbf}{ft}$ — *W30x132 western beam*

$W_{Beam\_Total} := W_{Beam\_West} \cdot L_{Beam} + 8 \cdot \left(W_{Beam} \cdot L_{Beam}\right) = 23.44\ kip$ — *Weight of all beams (9 total on ramp)*

$L_{TeeBeam\_East} := 2 \cdot (11\ ft + 13.75\ ft \cdot 3) = 104.5\ ft$ — *Total Length of Tee Beams on eastern side of ramp*

$W_{TeeBeam\_East} := 32.5\ \frac{lbf}{ft}$ — *WT6x32.5 western Tee Beams*

$W_{TeeBeam\_East\_Total} := L_{TeeBeam\_East} \cdot W_{TeeBeam\_East} = 3.396\ kip$ — *Total Weight of Tee Beams*

$W_{Girder\_Total} := 2 \cdot (L_{Girder} \cdot A_{Girder} \cdot \gamma_S) = 91.607\ kip$ — Total Weight of Girders

$$W_{Ramp\_Total} := W_{Grating\_Total} + W_{Joists\_Total} + W_{Beam\_Total} + W_{TeeBeam\_East\_Total} + W_{Girder\_Total} = 282.88\ kip$$

Total Weight of Ramp

## Weight of Hoist Towers

$L_{cap} := 12.25\ ft$

$W_{cap} := 11.42\ ft$

$H_{cap} := 3\ ft$

$V_{cap} := L_{cap} \cdot W_{cap} \cdot H_{cap} = (3.139 \cdot 10^3)\ gal$

$W_{Hoist\_cap} := V_{cap} \cdot \gamma_C = 62.953\ kip$

Dimensions and Weights of the Hoist Cap, *as-built drawings*

$W_{pile\_hoist} := 50\ \frac{lbf}{ft}$ — W12x50 Steel Piles

$W_{angles\_hoist} := 5.90\ \frac{lbf}{ft}$ — L3x2x3/8 Angles

$W_{beam\_hoist} := 12\ \frac{lbf}{ft}$ — W6x12 Horizontal Beams

$W_{bridge\_bar} := 44\ \frac{lbf}{ft}$ — W21x44 Horizontal Beams

$W_{bridge\_side} := 135\ \frac{lbf}{ft}$ — W36x135 Horizontal Beams

$W_{counter\_weight} := 57\ in \cdot 57\ in \cdot 48\ in \cdot \gamma_S = 44.29\ kip$ — 48" Tall counterweight

$W_{pile\_hoist\_total} := W_{pile\_hoist} \cdot 22.13\ ft \cdot 4 = 4.426\ kip$ — Total weight for (4) piles 22.13' long

$W_{angles\_hoist\_total} := W_{angles\_hoist} \cdot 10\ ft \cdot 24 = 1.416\ kip$ — *Total weight for (24) angles 10' long*

$W_{beam\_hoist\_total} := W_{beam\_hoist} \cdot 7\ ft \cdot 12 = 1.008\ kip$ — *Total Weight for (12) horizontal beams 7' long*

$W_{bridge\_bar\_total} := W_{bridge\_bar} \cdot 7\ ft = 0.308\ kip$ — *Total Weight for (1) horizontal bridge bar*

$W_{bridge\_side\_total} := W_{bridge\_side} \cdot \frac{27.5}{2}\ ft \cdot 2 = 3.713\ kip$ — *Total Weight for half the length of (2) bridge side beam, 27.5' long*

$$W_{Hoist\_Total} := W_{Hoist\_cap} + W_{counter\_weight} + W_{pile\_hoist\_total} + W_{angles\_hoist\_total} + W_{beam\_hoist\_total} + W_{bridge\_bar\_total} + W_{bridge\_side\_total} = 118.114\ kip$$

Total Weight of Hoist mechanism and cap

**Determine Axial Load on Hoist Piles**

$$P_{DL\_Hoist\_Piles} := F_{DL} \cdot \left( \frac{W_{Ramp\_Total}}{4} + W_{Hoist\_Total} \right) = 226.6\ \mathbf{kip}$$

Dead load acting on the hoist piles

<u>Pedestrian Loading Scenario</u>

$$P_{LL\_Pedestrian\_Hoist} := F_{LL} \cdot \left( P_{LL\_Pedestrian} \cdot \frac{\left( \left( L_{Girder} - 4.75\ \mathbf{ft} \right) \cdot L_{Beam} \right)}{4} \right) = 77.22\ \mathbf{kip}$$

Pedestrian loading on (1/4) of the ramp

$$P_{Hoist\_Pedestrian\_Total} := P_{DL\_Hoist\_Piles} + P_{LL\_Pedestrian\_Hoist} = 303.82\ \mathbf{kip}$$

Total Loading

$$FS_{Hoist\_Piles\_Pedestrian} := \frac{P_{n\_Piles\_Total\_Hoist}}{P_{Hoist\_Pedestrian\_Total}} = 1.422$$

*Factor of Safety*

$Check := \text{if}\left(FS_{Hoist\_Piles\_Pedestrian} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

<u>HS-20 Loading Scenario</u>

$$P_{LL\_HS20\_Hoist} := F_{LL} \cdot \left( 2 \cdot P_{LL\_HS20} \right) = 51.2\ \mathbf{kip}$$

HS-20 Load from (2) wheels

$$P_{Total\_Hoist\_HS20} := P_{DL\_Hoist\_Piles} + P_{LL\_HS20\_Hoist} = 277.8\ \mathbf{kip}$$

Total load on the hoist piles

$$FS_{Hoist\_Piles\_HS20} := \frac{P_{n\_Piles\_Total\_Hoist}}{P_{Total\_Hoist\_HS20}} = 1.555$$

*Factor of Safety*

$Check := \text{if}\left(FS_{Hoist\_Piles\_HS20} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

**Determine Axial Load on Abutment Piles**

$$P_{DL\_Abutment\_Piles} := F_{DL} \cdot \left( \frac{W_{Ramp\_Total}}{2} \right) = 169.728\ \mathbf{kip}$$

Dead load acting on the abutment piles

<u>Pedestrian Loading Scenario</u>

$$P_{LL\_Pedestrian\_HS20} := F_{LL} \cdot \left( P_{LL\_Pedestrian} \cdot \frac{\left( L_{Girder} - 4.75\ \mathbf{ft} \right) \cdot L_{Beam}}{2} \right) = 154.44\ \mathbf{kip}$$

Pedestrian loading on (1/2) of the ramp

$$P_{Abutment\_Pedestrian\_Total} := P_{DL\_Abutment\_Piles} + P_{LL\_Pedestrian\_HS20} = 324.168\ \mathbf{kip}$$

Total Loading

$$FS_{Abutment\_Piles\_Pedestrian} := \frac{P_{n\_Piles\_Total\_Abutment}}{P_{Abutment\_Pedestrian\_Total}} = 2.332$$

*Factor of Safety*

$Check := \text{if}\left(FS_{Hoist\_Piles\_Pedestrian} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

<u>HS-20 Loading Scenario</u>

$$P_{LL\_HS20\_Abutment} := F_{LL} \cdot \left( 4 \cdot P_{LL\_HS20} \right) = 102.4\ \mathbf{kip}$$

HS-20 Load from (4) wheels

$$P_{Total\_Abutment\_HS20} := P_{DL\_Abutment\_Piles} + P_{LL\_HS20\_Abutment} = 272.128\ \mathbf{kip}$$

Total load on the hoist piles

$$FS_{Abutment\_Piles\_HS20} := \frac{P_{n\_Piles\_Total\_Abutment}}{P_{Total\_Abutment\_HS20}} = 2.778$$

*Factor of Safety*

$$Check := \text{if}\left(FS_{Hour\_Piles\_HS20} > 1.2, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

*The existing piles are **capable** of **supporting** HS-20 and p**edestrian** loads*

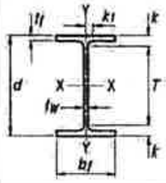
| Bar Size, Inches | Maximum Safe Concentrated Load*, Lbs. - Clear Span 1'-0" | 1'-6" | 2'-0" | 2'-6" | 3'-0" | 3'-6" | 4'-0" | 4'-6" | 5'-0" | 5'-6" | 6'-0" | 7'-0" | 8'-0" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 × 1/4 | 1407 | 938 | 703 | 563 | 469 | 402 | | | | | | | |
| 1 × 3/8 | 2107 | 1404 | 1053 | 843 | 702 | 602 | | | | | | | |
| 1 1/4 × 1/4 | 2193 | 1462 | 1097 | 877 | 731 | 627 | 548 | | | | | | |
| 1 1/4 × 3/8 | 3287 | 2191 | 1643 | 1315 | 1096 | 939 | 822 | | | | | | |
| 1 1/2 × 1/4 | 3160 | 2107 | 1580 | 1264 | 1053 | 903 | 790 | 702 | | | | | |
| 1 1/2 × 5/16 | 3947 | 2631 | 1973 | 1579 | 1316 | 1128 | 987 | 877 | | | | | |
| 1 1/2 × 3/8 | 4740 | 3160 | 2370 | 1896 | 1580 | 1354 | 1185 | 1053 | | | | | |
| 1 3/4 × 1/4 | 4300 | 2867 | 2150 | 1720 | 1433 | 1229 | 1075 | 956 | 860 | | | | |
| 1 3/4 × 3/8 | 6447 | 4298 | 3223 | 2579 | 2149 | 1842 | 1612 | 1433 | 1289 | | | | |
| 2 × 1/4 | 5613 | 3742 | 2807 | 2245 | 1871 | 1604 | 1403 | 1247 | 1123 | | | | |
| 2 × 5/16 | 7020 | 4680 | 3510 | 2808 | 2340 | 2006 | 1755 | 1560 | 1404 | | | | |
| 2 × 3/8 | 8420 | 5613 | 4210 | 3368 | 2807 | 2406 | 2105 | 1871 | 1684 | | | | |
| 2 1/4 × 1/4 | 7107 | 4738 | 3553 | 2843 | 2368 | 2030 | 1777 | 1579 | 1421 | 1292 | | | |
| 2 1/4 × 3/8 | 10660 | 7107 | 5330 | 4264 | 3553 | 3046 | 2665 | 2369 | 2132 | 1938 | | | |
| 2 1/2 × 1/4 | 8773 | 5849 | 4387 | 3509 | 2924 | 2507 | 2193 | 1950 | 1755 | 1595 | 1462 | | |
| 2 1/2 × 5/16 | 10967 | 7311 | 5483 | 4387 | 3656 | 3133 | 2742 | 2437 | 2193 | 1994 | 1828 | | |
| 2 1/2 × 3/8 | 13160 | 8773 | 6580 | 5264 | 4387 | 3760 | 3290 | 2924 | 2632 | 2393 | 2193 | | |
| 3 × 1/4 | 12633 | 8422 | 6317 | 5053 | 4211 | 3610 | 3158 | 2807 | 2527 | 2297 | 2106 | | |
| 3 × 5/16 | 15793 | 10529 | 7897 | 6317 | 5264 | 4512 | 3948 | 3510 | 3159 | 2872 | 2632 | | |
| 3 × 3/8 | 18947 | 12631 | 9473 | 7579 | 6316 | 5413 | 4737 | 4210 | 3789 | 3445 | 3158 | | |
| 3 × 1/2 | 25267 | 16844 | 12633 | 10107 | 8422 | 7219 | 6317 | 5615 | 5053 | 4594 | 4211 | | |
| 3 1/2 × 1/4 | 17193 | 11462 | 8597 | 6877 | 5731 | 4912 | 4298 | 3821 | 3439 | 3126 | 2866 | 2456 | |
| 3 1/2 × 3/8 | 25793 | 17196 | 12897 | 10317 | 8598 | 7370 | 6448 | 5732 | 5159 | 4690 | 4299 | 3685 | |
| 3 1/2 × 1/2 | 34387 | 22924 | 17193 | 13755 | 11462 | 9825 | 8597 | 7641 | 6877 | 6252 | 5731 | 4912 | |
| 4 × 1/4 | 22460 | 14973 | 11230 | 8984 | 7487 | 6417 | 5615 | 4991 | 4492 | 4084 | 3743 | 3209 | |
| 4 × 5/16 | 28073 | 18716 | 14037 | 11229 | 9358 | 8021 | 7018 | 6239 | 5615 | 5104 | 4679 | 4010 | |
| 4 × 3/8 | 33687 | 22458 | 16843 | 13475 | 11229 | 9625 | 8422 | 7486 | 6737 | 6125 | 5614 | 4812 | |
| 4 × 1/2 | 44913 | 29942 | 22457 | 17965 | 14971 | 12832 | 11228 | 9981 | 8983 | 8166 | 7486 | 6416 | |
| 4 1/2 × 1/4 | 28420 | 18947 | 14210 | 11368 | 9473 | 8120 | 7105 | 6316 | 5684 | 5167 | 4737 | 4060 | 3553 |
| 4 1/2 × 3/8 | 42633 | 28422 | 21317 | 17053 | 14211 | 12181 | 10658 | 9474 | 8527 | 7752 | 7106 | 6090 | 5329 |
| 4 1/2 × 1/2 | 56847 | 37898 | 28423 | 22739 | 18949 | 16242 | 14212 | 12633 | 11369 | 10336 | 9474 | 8121 | 7106 |
| 5 × 1/4 | 35093 | 23396 | 17547 | 14037 | 11698 | 10027 | 8773 | 7799 | 7019 | 6381 | 5849 | 5013 | 4387 |
| 5 × 5/16 | 43860 | 29240 | 21930 | 17544 | 14620 | 12531 | 10965 | 9747 | 8772 | 7975 | 7310 | 6266 | 5483 |
| 5 × 3/8 | 52633 | 35089 | 26317 | 21053 | 17544 | 15038 | 13158 | 11696 | 10527 | 9570 | 8772 | 7519 | 6579 |
| 5 × 1/2 | | 46787 | 35090 | 28072 | 23393 | 20051 | 17545 | 15596 | 14036 | 12760 | 11697 | 10026 | 8773 |
| 5 1/2 × 1/4 | | 28307 | 21230 | 16984 | 14153 | 12131 | 10615 | 9436 | 8492 | 7720 | 7077 | 6066 | 5308 |
| 5 1/2 × 3/8 | | 42458 | 31843 | 25475 | 21229 | 18196 | 15922 | 14153 | 12737 | 11579 | 10614 | 9098 | 7961 |
| 5 1/2 × 1/2 | | 56613 | 42460 | 33968 | 28307 | 24263 | 21230 | 18871 | 16984 | 15440 | 14153 | 12131 | 10615 |
| 6 × 1/4 | | 33689 | 25267 | 20213 | 16844 | 14438 | 12633 | 11230 | 10107 | 9188 | 8422 | 7219 | 6317 |
| 6 × 5/16 | | 42107 | 31580 | 25264 | 21053 | 18046 | 15790 | 14036 | 12632 | 11484 | 10527 | 9023 | 7895 |
| 6 × 3/8 | | 50529 | 37897 | 30317 | 25264 | 21655 | 18948 | 16843 | 15159 | 13781 | 12632 | 10828 | 9474 |
| 6 × 1/2 | | | 50530 | 40424 | 33687 | 28874 | 25265 | 22458 | 20212 | 18375 | 16843 | 14437 | 12633 |

| BB Size | CB Ctrs | % Open Area* Bearing Bar Thickness 1/4" | 5/16" | 3/8" | 1/2" |
|---|---|---|---|---|---|
| Thru 2 1/2" | 4" cc | 82% | 80% | 77% | – |
| | 2" cc | 76% | 73% | 71% | – |
| 3" to 6" | 4" cc | 84% | 82% | 79% | 74% |
| | 2" cc | 80% | 78% | 75% | 71% |

Loads are theoretical, and are based on a unit stress of 20,000 psi.

*Based on 5.053 bars/ft of grating width. Bearing bars 2 3/8" c.c.

Note: When serrated grating is specified, the depth of grating required for a specified load will be 1/4" greater than that shown in these tables.




## Table 1-1 (continued)
# W-Shapes
### Dimensions

| Shape | Area, $A$ | Depth, $d$ | | Web Thickness, $t_w$ | | Web $\frac{t_w}{2}$ | Flange Width, $b_f$ | | Flange Thickness, $t_f$ | | Distance $k$ $k_{des}$ | Distance $k$ $k_{det}$ | Distance $k_1$ | Distance $T$ | Workable Gage |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | in.$^2$ | in. | | in. | | in. | in. | | in. | | in. | in. | in. | in. | in. |
| W21×93 | 27.3 | 21.6 | 21 5/8 | 0.580 | 9/16 | 5/16 | 8.42 | 8 3/8 | 0.930 | 15/16 | 1.43 | 1 5/8 | 15/16 | 18 3/8 | 5 1/2 |
| ×83[c] | 24.4 | 21.4 | 21 3/8 | 0.515 | 1/2 | 1/4 | 8.36 | 8 3/8 | 0.835 | 13/16 | 1.34 | 1 1/2 | 7/8 | | |
| ×73[c] | 21.5 | 21.2 | 21 1/4 | 0.455 | 7/16 | 1/4 | 8.30 | 8 1/4 | 0.740 | 3/4 | 1.24 | 1 7/16 | 7/8 | | |
| ×68[c] | 20.0 | 21.1 | 21 1/8 | 0.430 | 7/16 | 1/4 | 8.27 | 8 1/4 | 0.685 | 11/16 | 1.19 | 1 3/8 | 7/8 | | |
| ×62[c] | 18.3 | 21.0 | 21 | 0.400 | 3/8 | 3/16 | 8.24 | 8 1/4 | 0.615 | 5/8 | 1.12 | 1 5/16 | 13/16 | | |
| ×55[c] | 16.2 | 20.8 | 20 3/4 | 0.375 | 3/8 | 3/16 | 8.22 | 8 1/4 | 0.522 | 1/2 | 1.02 | 1 3/16 | 13/16 | | |
| ×48[c,f] | 14.1 | 20.6 | 20 5/8 | 0.350 | 3/8 | 3/16 | 8.14 | 8 1/8 | 0.430 | 7/16 | 0.930 | 1 1/8 | 13/16 | | |
| W21×57[c] | 16.7 | 21.1 | 21 | 0.405 | 3/8 | 3/16 | 6.56 | 6 1/2 | 0.650 | 5/8 | 1.15 | 1 5/16 | 13/16 | 18 3/8 | 3 1/2 |
| ×50[c] | 14.7 | 20.8 | 20 7/8 | 0.380 | 3/8 | 3/16 | 6.53 | 6 1/2 | 0.535 | 9/16 | 1.04 | 1 1/4 | 13/16 | | |
| ×44[c] | 13.0 | 20.7 | 20 5/8 | 0.350 | 3/8 | 3/16 | 6.50 | 6 1/2 | 0.450 | 7/16 | 0.950 | 1 1/8 | 13/16 | | |
| W18×311[h] | 91.6 | 22.3 | 22 3/8 | 1.52 | 1 1/2 | 3/4 | 12.0 | 12 | 2.74 | 2 3/4 | 3.24 | 3 7/16 | 1 3/8 | 15 1/2 | 5 1/2 |
| ×283[h] | 83.3 | 21.9 | 21 7/8 | 1.40 | 1 3/8 | 11/16 | 11.9 | 11 7/8 | 2.50 | 2 1/2 | 3.00 | 3 3/16 | 1 5/16 | | |
| ×258[h] | 76.0 | 21.5 | 21 1/2 | 1.28 | 1 1/4 | 5/8 | 11.8 | 11 3/4 | 2.30 | 2 5/16 | 2.70 | 3 | 1 1/4 | | |
| ×234[h] | 68.6 | 21.1 | 21 | 1.16 | 1 3/16 | 5/8 | 11.7 | 11 5/8 | 2.11 | 2 1/8 | 2.51 | 2 3/4 | 1 3/16 | | |
| ×211 | 62.3 | 20.7 | 20 5/8 | 1.06 | 1 1/16 | 9/16 | 11.6 | 11 1/2 | 1.91 | 1 15/16 | 2.31 | 2 9/16 | 1 3/16 | | |
| ×192 | 56.2 | 20.4 | 20 3/8 | 0.960 | 15/16 | 1/2 | 11.5 | 11 1/2 | 1.75 | 1 3/4 | 2.15 | 2 7/16 | 1 1/8 | | |
| ×175 | 51.4 | 20.0 | 20 | 0.890 | 7/8 | 7/16 | 11.4 | 11 3/8 | 1.59 | 1 9/16 | 1.99 | 2 7/16 | 1 1/4 | 15 1/8 | |
| ×158 | 46.3 | 19.7 | 19 3/4 | 0.810 | 13/16 | 7/16 | 11.3 | 11 1/4 | 1.44 | 1 7/16 | 1.84 | 2 3/8 | 1 1/4 | | |
| ×143 | 42.0 | 19.5 | 19 1/2 | 0.730 | 3/4 | 3/8 | 11.2 | 11 1/4 | 1.32 | 1 5/16 | 1.72 | 2 3/16 | 1 3/16 | | |
| ×130 | 38.3 | 19.3 | 19 1/4 | 0.670 | 11/16 | 3/8 | 11.2 | 11 1/8 | 1.20 | 1 3/16 | 1.60 | 2 1/16 | 1 3/16 | | |
| ×119 | 35.1 | 19.0 | 19 | 0.655 | 5/8 | 5/16 | 11.3 | 11 1/4 | 1.06 | 1 1/16 | 1.46 | 1 15/16 | 1 3/16 | | |
| ×106 | 31.1 | 18.7 | 18 3/4 | 0.590 | 9/16 | 5/16 | 11.2 | 11 1/4 | 0.940 | 15/16 | 1.34 | 1 13/16 | 1 1/8 | | |
| ×97 | 28.5 | 18.6 | 18 5/8 | 0.535 | 9/16 | 5/16 | 11.1 | 11 1/8 | 0.870 | 7/8 | 1.27 | 1 3/4 | 1 1/8 | | |
| ×86 | 25.3 | 18.4 | 18 3/8 | 0.480 | 1/2 | 1/4 | 11.1 | 11 1/8 | 0.770 | 3/4 | 1.17 | 1 5/8 | 1 1/16 | | |
| ×76[c] | 22.3 | 18.2 | 18 1/4 | 0.425 | 7/16 | 1/4 | 11.0 | 11 | 0.680 | 11/16 | 1.08 | 1 9/16 | 1 1/16 | | |
| W18×71 | 20.9 | 18.5 | 18 1/2 | 0.495 | 1/2 | 1/4 | 7.64 | 7 5/8 | 0.810 | 13/16 | 1.21 | 1 1/2 | 7/8 | 15 1/2 | 3 1/2[g] |
| ×65 | 19.1 | 18.4 | 18 3/8 | 0.450 | 7/16 | 1/4 | 7.59 | 7 5/8 | 0.750 | 3/4 | 1.15 | 1 7/16 | 7/8 | | |
| ×60[c] | 17.6 | 18.2 | 18 1/4 | 0.415 | 7/16 | 1/4 | 7.56 | 7 1/2 | 0.695 | 11/16 | 1.10 | 1 3/8 | 13/16 | | |
| ×55[c] | 16.2 | 18.1 | 18 1/8 | 0.390 | 3/8 | 3/16 | 7.53 | 7 1/2 | 0.630 | 5/8 | 1.03 | 1 5/16 | 13/16 | | |
| ×50[c] | 14.7 | 18.0 | 18 | 0.355 | 3/8 | 3/16 | 7.50 | 7 1/2 | 0.570 | 9/16 | 0.972 | 1 1/4 | 13/16 | | |
| W18×46[c] | 13.5 | 18.1 | 18 | 0.360 | 3/8 | 3/16 | 6.06 | 6 | 0.605 | 5/8 | 1.01 | 1 1/4 | 13/16 | 15 1/2 | 3 1/2[g] |
| ×40[c] | 11.8 | 17.9 | 17 7/8 | 0.315 | 5/16 | 3/16 | 6.02 | 6 | 0.525 | 1/2 | 0.927 | 1 3/16 | 13/16 | | |
| ×35[c] | 10.3 | 17.7 | 17 3/4 | 0.300 | 5/16 | 3/16 | 6.00 | 6 | 0.425 | 7/16 | 0.827 | 1 1/8 | 3/4 | | |

[c] Shape is slender for compression with $F_y$ = 50 ksi.
[f] Shape exceeds compact limit for flexure with $F_y$ = 50 ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

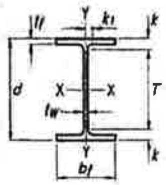




## Table 1-1 (continued)
# W-Shapes
### Properties

W21-W18

| Nom-inal Wt. | Compact Section Criteria $\frac{b_f}{2t_f}$ | $\frac{h}{t_w}$ | Axis X-X $I$ | $S$ | $r$ | $Z$ | Axis Y-Y $I$ | $S$ | $r$ | $Z$ | $r_{ts}$ | $h_o$ | Torsional Properties $J$ | $C_w$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| lb/ft | | | in.$^4$ | in.$^3$ | in. | in.$^3$ | in.$^4$ | in.$^3$ | in. | in.$^3$ | in. | in. | in.$^4$ | in.$^6$ |
| 93 | 4.53 | 32.3 | 2070 | 192 | 8.70 | 221 | 92.9 | 22.1 | 1.84 | 34.7 | 2.24 | 20.7 | 6.03 | 9940 |
| 83 | 5.00 | 36.4 | 1830 | 171 | 8.67 | 196 | 81.4 | 19.5 | 1.83 | 30.5 | 2.21 | 20.6 | 4.34 | 8630 |
| 73 | 5.60 | 41.2 | 1600 | 151 | 8.64 | 172 | 70.6 | 17.0 | 1.81 | 26.6 | 2.19 | 20.5 | 3.02 | 7410 |
| 68 | 6.04 | 43.6 | 1480 | 140 | 8.60 | 160 | 64.7 | 15.7 | 1.80 | 24.4 | 2.17 | 20.4 | 2.45 | 6760 |
| 62 | 6.70 | 46.9 | 1330 | 127 | 8.54 | 144 | 57.5 | 14.0 | 1.77 | 21.7 | 2.15 | 20.4 | 1.83 | 5960 |
| 55 | 7.87 | 50.0 | 1140 | 110 | 8.40 | 126 | 48.4 | 11.8 | 1.73 | 18.4 | 2.11 | 20.3 | 1.24 | 4980 |
| 48 | 9.47 | 53.6 | 959 | 93.0 | 8.24 | 107 | 38.7 | 9.52 | 1.66 | 14.9 | 2.05 | 20.2 | 0.803 | 3950 |
| 57 | 5.04 | 46.3 | 1170 | 111 | 8.36 | 129 | 30.6 | 9.35 | 1.35 | 14.8 | 1.68 | 20.5 | 1.77 | 3190 |
| 50 | 6.10 | 49.4 | 984 | 94.5 | 8.18 | 110 | 24.9 | 7.64 | 1.30 | 12.2 | 1.64 | 20.3 | 1.14 | 2570 |
| 44 | 7.22 | 53.6 | 843 | 81.6 | 8.06 | 95.4 | 20.7 | 6.37 | 1.26 | 10.2 | 1.60 | 20.3 | 0.770 | 2110 |
| 311 | 2.19 | 10.4 | 6970 | 624 | 8.72 | 754 | 795 | 132 | 2.95 | 207 | 3.53 | 19.6 | 176 | 76200 |
| 283 | 2.38 | 11.3 | 6170 | 565 | 8.61 | 676 | 704 | 118 | 2.91 | 185 | 3.47 | 19.4 | 134 | 65900 |
| 258 | 2.56 | 12.5 | 5510 | 514 | 8.53 | 611 | 628 | 107 | 2.88 | 166 | 3.42 | 19.2 | 103 | 57600 |
| 234 | 2.76 | 13.8 | 4900 | 466 | 8.44 | 549 | 558 | 95.8 | 2.85 | 149 | 3.37 | 19.0 | 78.7 | 50100 |
| 211 | 3.02 | 15.1 | 4330 | 419 | 8.35 | 490 | 493 | 85.3 | 2.82 | 132 | 3.32 | 18.8 | 58.6 | 43400 |
| 192 | 3.27 | 16.7 | 3870 | 380 | 8.28 | 442 | 440 | 76.8 | 2.79 | 119 | 3.28 | 18.7 | 44.7 | 38000 |
| 175 | 3.58 | 18.0 | 3450 | 344 | 8.20 | 398 | 391 | 68.8 | 2.76 | 106 | 3.24 | 18.4 | 33.8 | 33300 |
| 158 | 3.92 | 19.8 | 3060 | 310 | 8.12 | 356 | 347 | 61.4 | 2.74 | 94.8 | 3.20 | 18.3 | 25.2 | 29000 |
| 143 | 4.25 | 22.0 | 2750 | 282 | 8.09 | 322 | 311 | 55.5 | 2.72 | 85.4 | 3.17 | 18.2 | 19.2 | 25700 |
| 130 | 4.65 | 23.9 | 2460 | 256 | 8.03 | 290 | 278 | 49.9 | 2.70 | 76.7 | 3.13 | 18.1 | 14.5 | 22700 |
| 119 | 5.31 | 24.5 | 2190 | 231 | 7.90 | 262 | 253 | 44.9 | 2.69 | 69.1 | 3.13 | 17.9 | 10.6 | 20300 |
| 106 | 5.96 | 27.2 | 1910 | 204 | 7.84 | 230 | 220 | 39.4 | 2.66 | 60.5 | 3.10 | 17.8 | 7.48 | 17400 |
| 97 | 6.41 | 30.0 | 1750 | 188 | 7.82 | 211 | 201 | 36.1 | 2.65 | 55.3 | 3.08 | 17.7 | 5.86 | 15800 |
| 86 | 7.20 | 33.4 | 1530 | 166 | 7.77 | 186 | 175 | 31.6 | 2.63 | 48.4 | 3.05 | 17.6 | 4.10 | 13600 |
| 76 | 8.11 | 37.8 | 1330 | 146 | 7.73 | 163 | 152 | 27.6 | 2.61 | 42.2 | 3.02 | 17.5 | 2.83 | 11700 |
| 71 | 4.71 | 32.4 | 1170 | 127 | 7.50 | 146 | 60.3 | 15.8 | 1.70 | 24.7 | 2.05 | 17.7 | 3.49 | 4700 |
| 65 | 5.06 | 35.7 | 1070 | 117 | 7.49 | 133 | 54.8 | 14.4 | 1.69 | 22.5 | 2.03 | 17.7 | 2.73 | 4240 |
| 60 | 5.44 | 38.7 | 984 | 108 | 7.47 | 123 | 50.1 | 13.3 | 1.68 | 20.6 | 2.02 | 17.5 | 2.17 | 3850 |
| 55 | 5.98 | 41.1 | 890 | 98.3 | 7.41 | 112 | 44.9 | 11.9 | 1.67 | 18.5 | 2.00 | 17.5 | 1.66 | 3430 |
| 50 | 6.57 | 45.2 | 800 | 88.9 | 7.38 | 101 | 40.1 | 10.7 | 1.65 | 16.6 | 1.98 | 17.4 | 1.24 | 3040 |
| 46 | 5.01 | 44.6 | 712 | 78.8 | 7.25 | 90.7 | 22.5 | 7.43 | 1.29 | 11.7 | 1.58 | 17.5 | 1.22 | 1720 |
| 40 | 5.73 | 50.9 | 612 | 68.4 | 7.21 | 78.4 | 19.1 | 6.35 | 1.27 | 10.0 | 1.56 | 17.4 | 0.810 | 1440 |
| 35 | 7.06 | 53.5 | 510 | 57.6 | 7.04 | 66.5 | 15.3 | 5.12 | 1.22 | 8.06 | 1.51 | 17.3 | 0.506 | 1140 |




# Table 1-1 (continued)
## W-Shapes
### Dimensions

| Shape | Area, $A$ | Depth, $d$ | | Web Thickness, $t_w$ | | Web $\frac{t_w}{2}$ | Flange Width, $b_f$ | | Flange Thickness, $t_f$ | | Distance $k$ $k_{des}$ | Distance $k$ $k_{det}$ | Distance $k_1$ | Distance $T$ | Workable Gage |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | in.$^2$ | in. | | in. | | in. | in. | | in. | | in. | in. | in. | in. | in. |
| W36×652$^{h}$ | 192 | 41.1 | 41 | 1.97 | 2 | 1 | 17.6 | 17 5/8 | 3.54 | 3 9/16 | 4.49 | 4 13/16 | 2 3/16 | 31 3/8 | 7 1/2 |
| ×529$^{h}$ | 156 | 39.8 | 39 3/4 | 1.61 | 1 5/8 | 13/16 | 17.2 | 17 1/4 | 2.91 | 2 15/16 | 3.86 | 4 3/16 | 2 | | |
| ×487$^{h}$ | 143 | 39.3 | 39 3/8 | 1.50 | 1 1/2 | 3/4 | 17.1 | 17 1/8 | 2.68 | 2 11/16 | 3.53 | 4 | 1 7/8 | | |
| ×441$^{h}$ | 130 | 38.9 | 38 7/8 | 1.36 | 1 3/8 | 11/16 | 17.0 | 17 | 2.44 | 2 7/16 | 3.39 | 3 3/4 | 1 7/8 | | |
| ×395$^{h}$ | 116 | 38.4 | 38 3/8 | 1.22 | 1 1/4 | 5/8 | 16.8 | 16 7/8 | 2.20 | 2 3/16 | 3.15 | 3 7/16 | 1 13/16 | | |
| ×361$^{h}$ | 106 | 38.0 | 38 | 1.12 | 1 1/8 | 9/16 | 16.7 | 16 3/4 | 2.01 | 2 | 2.96 | 3 5/16 | 1 3/4 | | |
| ×330 | 96.9 | 37.7 | 37 5/8 | 1.02 | 1 | 1/2 | 16.6 | 16 5/8 | 1.85 | 1 7/8 | 2.80 | 3 1/8 | 1 3/4 | | |
| ×302 | 89.0 | 37.3 | 37 3/8 | 0.945 | 15/16 | 1/2 | 16.7 | 16 5/8 | 1.68 | 1 11/16 | 2.63 | 3 | 1 11/16 | | |
| ×282$^{c}$ | 82.9 | 37.1 | 37 1/8 | 0.885 | 7/8 | 7/16 | 16.6 | 16 5/8 | 1.57 | 1 9/16 | 2.52 | 2 7/8 | 1 5/8 | | |
| ×262$^{c}$ | 77.2 | 36.9 | 36 7/8 | 0.840 | 13/16 | 7/16 | 16.6 | 16 1/2 | 1.44 | 1 7/16 | 2.39 | 2 3/4 | 1 5/8 | | |
| ×247$^{c}$ | 72.5 | 36.7 | 36 5/8 | 0.800 | 13/16 | 7/16 | 16.5 | 16 1/2 | 1.35 | 1 3/8 | 2.30 | 2 5/8 | 1 5/8 | | |
| ×231$^{c}$ | 68.2 | 36.5 | 36 1/2 | 0.760 | 3/4 | 3/8 | 16.5 | 16 1/2 | 1.26 | 1 1/4 | 2.21 | 2 9/16 | 1 9/16 | | |
| W36×256 | 75.3 | 37.4 | 37 3/8 | 0.960 | 15/16 | 1/2 | 12.2 | 12 1/4 | 1.73 | 1 3/4 | 2.48 | 2 5/8 | 1 5/16 | 32 1/8 | 5 1/2 |
| ×232$^{c}$ | 68.0 | 37.1 | 37 1/8 | 0.870 | 7/8 | 7/16 | 12.1 | 12 1/8 | 1.57 | 1 9/16 | 2.32 | 2 7/16 | 1 1/4 | | |
| ×210$^{c}$ | 61.9 | 36.7 | 36 3/4 | 0.830 | 13/16 | 7/16 | 12.2 | 12 1/8 | 1.36 | 1 3/8 | 2.11 | 2 5/16 | 1 1/4 | | |
| ×194$^{c}$ | 57.0 | 36.5 | 36 1/2 | 0.765 | 3/4 | 3/8 | 12.1 | 12 1/8 | 1.26 | 1 1/4 | 2.01 | 2 3/16 | 1 3/16 | | |
| ×182$^{c}$ | 53.6 | 36.3 | 36 3/8 | 0.725 | 3/4 | 3/8 | 12.1 | 12 1/8 | 1.18 | 1 3/16 | 1.93 | 2 1/8 | 1 3/16 | | |
| ×170$^{c}$ | 50.0 | 36.2 | 36 1/8 | 0.680 | 11/16 | 3/8 | 12.0 | 12 | 1.10 | 1 1/8 | 1.85 | 2 | 1 3/16 | | |
| ×160$^{c}$ | 47.0 | 36.0 | 36 | 0.650 | 5/8 | 5/16 | 12.0 | 12 | 1.02 | 1 | 1.77 | 1 15/16 | 1 1/8 | | |
| ×150$^{c}$ | 44.3 | 35.9 | 35 7/8 | 0.625 | 5/8 | 5/16 | 12.0 | 12 | 0.940 | 15/16 | 1.69 | 1 7/8 | 1 1/8 | | |
| ×135$^{c,v}$ | 39.9 | 35.6 | 35 1/2 | 0.600 | 5/8 | 5/16 | 12.0 | 12 | 0.790 | 13/16 | 1.54 | 1 11/16 | 1 1/8 | | |
| W33×387$^{h}$ | 114 | 36.0 | 36 | 1.26 | 1 1/4 | 5/8 | 16.2 | 16 1/4 | 2.28 | 2 1/4 | 3.07 | 3 3/16 | 1 7/16 | 29 5/8 | 5 1/2 |
| ×354$^{h}$ | 104 | 35.6 | 35 1/2 | 1.16 | 1 3/16 | 5/8 | 16.1 | 16 1/8 | 2.09 | 2 1/16 | 2.88 | 2 15/16 | 1 3/8 | | |
| ×318 | 93.7 | 35.2 | 35 1/8 | 1.04 | 1 1/16 | 9/16 | 16.0 | 16 | 1.89 | 1 7/8 | 2.68 | 2 3/4 | 1 5/16 | | |
| ×291 | 85.6 | 34.8 | 34 7/8 | 0.960 | 15/16 | 1/2 | 15.9 | 15 7/8 | 1.73 | 1 3/4 | 2.52 | 2 5/8 | 1 5/16 | | |
| ×263 | 77.4 | 34.5 | 34 1/2 | 0.870 | 7/8 | 7/16 | 15.8 | 15 3/4 | 1.57 | 1 9/16 | 2.36 | 2 7/16 | 1 1/4 | | |
| ×241$^{c}$ | 71.1 | 34.2 | 34 1/8 | 0.830 | 13/16 | 7/16 | 15.9 | 15 7/8 | 1.40 | 1 3/8 | 2.19 | 2 1/4 | 1 1/4 | | |
| ×221$^{c}$ | 65.3 | 33.9 | 33 7/8 | 0.775 | 3/4 | 3/8 | 15.8 | 15 3/4 | 1.28 | 1 1/4 | 2.06 | 2 1/8 | 1 3/16 | | |
| ×201$^{c}$ | 59.1 | 33.7 | 33 5/8 | 0.715 | 11/16 | 3/8 | 15.7 | 15 3/4 | 1.15 | 1 1/8 | 1.94 | 2 | 1 3/16 | | |
| W33×169$^{c}$ | 49.5 | 33.8 | 33 7/8 | 0.670 | 11/16 | 3/8 | 11.5 | 11 1/2 | 1.22 | 1 1/4 | 1.92 | 2 1/8 | 1 3/16 | 29 5/8 | 5 1/2 |
| ×152$^{c}$ | 44.9 | 33.5 | 33 1/2 | 0.635 | 5/8 | 5/16 | 11.6 | 11 5/8 | 1.06 | 1 1/16 | 1.76 | 1 15/16 | 1 1/8 | | |
| ×141$^{c}$ | 41.5 | 33.3 | 33 1/4 | 0.605 | 5/8 | 5/16 | 11.5 | 11 1/2 | 0.960 | 15/16 | 1.66 | 1 13/16 | 1 1/8 | | |
| ×130$^{c}$ | 38.3 | 33.1 | 33 1/8 | 0.580 | 9/16 | 5/16 | 11.5 | 11 1/2 | 0.855 | 7/8 | 1.56 | 1 3/4 | 1 1/8 | | |
| ×118$^{c,v}$ | 34.7 | 32.9 | 32 7/8 | 0.550 | 9/16 | 5/16 | 11.5 | 11 1/2 | 0.740 | 3/4 | 1.44 | 1 5/8 | 1 1/8 | | |

[c] Shape is slender for compression with $F_y$ = 50 ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the $h/t_w$ limit for shear in AISC *Specification* Section G2.1(a) with $F_y$ = 50 ksi.



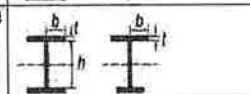
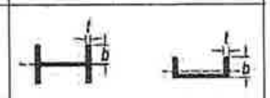
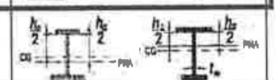
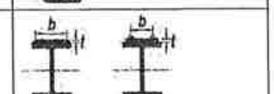
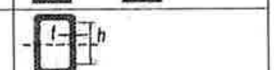
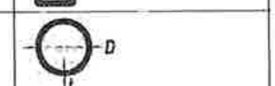


# Table 1-1 (continued)
## W-Shapes
### Properties

W36-W33

| Nominal Wt. | Compact Section Criteria $\frac{b_f}{2t_f}$ | Compact Section Criteria $\frac{h}{t_w}$ | Axis X-X $I$ | Axis X-X $S$ | Axis X-X $r$ | Axis X-X $Z$ | Axis Y-Y $I$ | Axis Y-Y $S$ | Axis Y-Y $r$ | Axis Y-Y $Z$ | $r_{ts}$ | $h_o$ | Torsional Properties $J$ | Torsional Properties $C_w$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| lb/ft | | | in.$^4$ | in.$^3$ | in. | in.$^3$ | in.$^4$ | in.$^3$ | in. | in.$^3$ | in. | in. | in.$^4$ | in.$^6$ |
| 652 | 2.48 | 16.3 | 50600 | 2460 | 16.2 | 2910 | 3230 | 367 | 4.10 | 581 | 4.96 | 37.6 | 593 | 1130000 |
| 529 | 2.96 | 19.9 | 39600 | 1990 | 16.0 | 2330 | 2490 | 289 | 4.00 | 454 | 4.80 | 36.9 | 327 | 846000 |
| 487 | 3.19 | 21.4 | 36000 | 1830 | 15.8 | 2130 | 2250 | 263 | 3.96 | 412 | 4.74 | 36.6 | 258 | 754000 |
| 441 | 3.48 | 23.6 | 32100 | 1650 | 15.7 | 1910 | 1990 | 235 | 3.92 | 368 | 4.69 | 36.5 | 194 | 681000 |
| 395 | 3.83 | 26.3 | 28500 | 1490 | 15.7 | 1710 | 1750 | 208 | 3.88 | 325 | 4.61 | 36.2 | 142 | 575000 |
| 361 | 4.16 | 28.6 | 25700 | 1350 | 15.6 | 1550 | 1570 | 188 | 3.85 | 293 | 4.58 | 36.0 | 109 | 509000 |
| 330 | 4.49 | 31.4 | 23300 | 1240 | 15.5 | 1410 | 1420 | 171 | 3.83 | 265 | 4.53 | 35.9 | 84.3 | 456000 |
| 302 | 4.96 | 33.9 | 21100 | 1130 | 15.4 | 1280 | 1300 | 156 | 3.82 | 241 | 4.53 | 35.6 | 64.3 | 412000 |
| 282 | 5.29 | 36.2 | 19600 | 1050 | 15.4 | 1190 | 1200 | 144 | 3.80 | 223 | 4.50 | 35.5 | 52.7 | 378000 |
| 262 | 5.75 | 38.2 | 17900 | 972 | 15.3 | 1100 | 1090 | 132 | 3.76 | 204 | 4.46 | 35.5 | 41.6 | 342000 |
| 247 | 6.11 | 40.1 | 16700 | 913 | 15.2 | 1030 | 1010 | 123 | 3.74 | 190 | 4.42 | 35.4 | 34.7 | 316000 |
| 231 | 6.54 | 42.2 | 15600 | 854 | 15.1 | 963 | 940 | 114 | 3.71 | 176 | 4.40 | 35.2 | 28.7 | 292000 |
| 256 | 3.53 | 33.8 | 16800 | 895 | 14.9 | 1040 | 528 | 86.5 | 2.65 | 137 | 3.24 | 35.7 | 52.9 | 168000 |
| 232 | 3.86 | 37.3 | 15000 | 809 | 14.8 | 936 | 468 | 77.2 | 2.62 | 122 | 3.21 | 35.5 | 39.6 | 148000 |
| 210 | 4.48 | 39.1 | 13200 | 719 | 14.6 | 833 | 411 | 67.5 | 2.58 | 107 | 3.18 | 35.3 | 28.0 | 128000 |
| 194 | 4.81 | 42.4 | 12100 | 664 | 14.6 | 767 | 375 | 61.9 | 2.56 | 97.7 | 3.15 | 35.2 | 22.2 | 116000 |
| 182 | 5.12 | 44.8 | 11300 | 623 | 14.5 | 718 | 347 | 57.6 | 2.55 | 90.7 | 3.13 | 35.1 | 18.5 | 107000 |
| 170 | 5.47 | 47.7 | 10500 | 581 | 14.5 | 668 | 320 | 53.2 | 2.53 | 83.8 | 3.11 | 35.1 | 15.1 | 98500 |
| 160 | 5.88 | 49.9 | 9760 | 542 | 14.4 | 624 | 295 | 49.1 | 2.50 | 77.3 | 3.09 | 35.0 | 12.4 | 90200 |
| 150 | 6.37 | 51.9 | 9040 | 504 | 14.3 | 581 | 270 | 45.1 | 2.47 | 70.9 | 3.06 | 35.0 | 10.1 | 82200 |
| 135 | 7.56 | 54.1 | 7800 | 439 | 14.0 | 509 | 225 | 37.7 | 2.38 | 59.7 | 2.99 | 34.8 | 7.00 | 68100 |
| 387 | 3.55 | 23.7 | 24300 | 1350 | 14.6 | 1560 | 1620 | 200 | 3.77 | 312 | 4.49 | 33.7 | 148 | 459000 |
| 354 | 3.85 | 25.7 | 22000 | 1240 | 14.5 | 1420 | 1460 | 181 | 3.74 | 282 | 4.44 | 33.5 | 115 | 408000 |
| 318 | 4.23 | 28.7 | 19500 | 1110 | 14.5 | 1270 | 1290 | 161 | 3.71 | 250 | 4.40 | 33.3 | 84.4 | 357000 |
| 291 | 4.60 | 31.0 | 17700 | 1020 | 14.4 | 1160 | 1160 | 146 | 3.68 | 226 | 4.34 | 33.1 | 65.1 | 319000 |
| 263 | 5.03 | 34.3 | 15900 | 919 | 14.3 | 1040 | 1040 | 131 | 3.66 | 202 | 4.31 | 32.9 | 48.7 | 281000 |
| 241 | 5.66 | 35.9 | 14200 | 831 | 14.1 | 940 | 933 | 118 | 3.62 | 182 | 4.29 | 32.8 | 36.2 | 251000 |
| 221 | 6.20 | 38.5 | 12900 | 759 | 14.1 | 857 | 840 | 106 | 3.59 | 164 | 4.25 | 32.6 | 27.8 | 224000 |
| 201 | 6.85 | 41.7 | 11600 | 686 | 14.0 | 773 | 749 | 95.2 | 3.56 | 147 | 4.21 | 32.6 | 20.8 | 198000 |
| 169 | 4.71 | 44.7 | 9290 | 549 | 13.7 | 629 | 310 | 53.9 | 2.50 | 84.4 | 3.03 | 32.6 | 17.7 | 82400 |
| 152 | 5.48 | 47.2 | 8160 | 487 | 13.5 | 559 | 273 | 47.2 | 2.47 | 73.9 | 3.01 | 32.4 | 12.4 | 71700 |
| 141 | 6.01 | 49.6 | 7450 | 448 | 13.4 | 514 | 246 | 42.7 | 2.43 | 66.9 | 2.98 | 32.3 | 9.70 | 64400 |
| 130 | 6.73 | 51.7 | 6710 | 406 | 13.2 | 467 | 218 | 37.9 | 2.39 | 59.5 | 2.94 | 32.2 | 7.37 | 56600 |
| 118 | 7.76 | 54.5 | 5900 | 359 | 13.0 | 415 | 187 | 32.6 | 2.32 | 51.3 | 2.89 | 32.2 | 5.30 | 48300 |

# TABLE B4.1b
## Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

| | Case | Description of Element | Width-to-Thickness Ratio | Limiting Width-to-Thickness Ratio: $\lambda_p$ (compact/ noncompact) | Limiting Width-to-Thickness Ratio: $\lambda_r$ (noncompact/ slender) | Examples |
|---|---|---|---|---|---|---|
| Unstiffened Elements | 10 | Flanges of rolled I-shaped sections, channels, and tees | $b/t$ | $0.38\sqrt{\frac{E}{F_y}}$ | $1.0\sqrt{\frac{E}{F_y}}$ |  |
| Unstiffened Elements | 11 | Flanges of doubly and singly symmetric I-shaped built-up sections | $b/t$ | $0.38\sqrt{\frac{E}{F_y}}$ | [a] [b] $0.95\sqrt{\frac{k_c E}{F_L}}$ |  |
| Unstiffened Elements | 12 | Legs of single angles | $b/t$ | $0.54\sqrt{\frac{E}{F_y}}$ | $0.91\sqrt{\frac{E}{F_y}}$ |  |
| Unstiffened Elements | 13 | Flanges of all I-shaped sections and channels in flexure about the weak axis | $b/t$ | $0.38\sqrt{\frac{E}{F_y}}$ | $1.0\sqrt{\frac{E}{F_y}}$ |  |
| Unstiffened Elements | 14 | Stems of tees | $d/t$ | $0.84\sqrt{\frac{E}{F_y}}$ | $1.03\sqrt{\frac{E}{F_y}}$ |  |
| Stiffened Elements | 15 | Webs of doubly-symmetric I-shaped sections and channels | $h/t_w$ | $3.76\sqrt{\frac{E}{F_y}}$ | $5.70\sqrt{\frac{E}{F_y}}$ |  |
| Stiffened Elements | 16 | Webs of singly-symmetric I-shaped sections | $h_c/t_w$ | [c] $\frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$ | $5.70\sqrt{\frac{E}{F_y}}$ |  |
| Stiffened Elements | 17 | Flanges of rectangular HSS and boxes of uniform thickness | $b/t$ | $1.12\sqrt{\frac{E}{F_y}}$ | $1.40\sqrt{\frac{E}{F_y}}$ |  |
| Stiffened Elements | 18 | Flange cover plates and diaphragm plates between lines of fasteners or welds | $b/t$ | $1.12\sqrt{\frac{E}{F_y}}$ | $1.40\sqrt{\frac{E}{F_y}}$ |  |
| Stiffened Elements | 19 | Webs of rectangular HSS and boxes | $h/t$ | $2.42\sqrt{\frac{E}{F_y}}$ | $5.70\sqrt{\frac{E}{F_y}}$ |  |
| Stiffened Elements | 20 | Round HSS | $D/t$ | $0.07\frac{E}{F_y}$ | $0.31\frac{E}{F_y}$ |  |

[a] $k_c = 4/\sqrt{h/t_w}$ but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.
[b] $F_L = 0.7F_y$ for major axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \ge 0.7$;
$F_L = F_y S_{xt}/S_{xc} \ge 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$.
[c] $M_y$ is the moment at yielding of the extreme fiber. $M_p$ = plastic bending moment, kip-in. (N-mm)
$E$ = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
$F_y$ = specified minimum yield stress, ksi (MPa)

## F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section B4.1 for flexure.

> **User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5 and M4×6 have compact flanges for $F_y$ = 50 ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \le 65$ ksi (450 MPa).

The *nominal flexural strength*, $M_n$, shall be the lower value obtained according to the *limit states* of *yielding (plastic moment)* and *lateral-torsional buckling*.

### 1. Yielding

$$M_n = M_p = F_y Z_x \tag{F2-1}$$

where

$F_y$ = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
$Z_x$ = plastic section modulus about the *x*-axis, in.³ (mm³)

### 2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the *limit state* of *lateral-torsional buckling* does not apply.
(b) When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \tag{F2-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr}S_x \le M_p \tag{F2-3}$$

where

$L_b$ = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in. (mm)

$$F_{cr} = \frac{C_b\pi^2E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{F2-4}$$

and where

$E$ = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
$J$ = torsional constant, in.⁴ (mm⁴)
$S_x$ = elastic section modulus taken about the *x*-axis, in.³ (mm³)
$h_o$ = distance between the flange centroids, in. (mm)





> **User Note:** The square root term in Equation F2-4 may be conservatively taken equal to 1.0.

> **User Note:** Equations F2-3 and F2-4 provide identical solutions to the following expression for lateral-torsional buckling of doubly symmetric sections that has been presented in past editions of the AISC LRFD Specification:
>
> $$M_{cr} = C_b\frac{\pi}{L_b}\sqrt{EI_yGJ + \left(\frac{\pi E}{L_b}\right)^2 I_yC_w}$$
>
> The advantage of Equations F2-3 and F2-4 is that the form is very similar to the expression for lateral-torsional buckling of singly symmetric sections given in Equations F4-4 and F4-5.

The limiting lengths $L_p$ and $L_r$ are determined as follows:

$$L_p = 1.76r_y\sqrt{\frac{E}{F_y}} \tag{F2-5}$$

$$L_r = 1.95r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_xh_o} + \sqrt{\left(\frac{Jc}{S_xh_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \tag{F2-6}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_yC_w}}{S_x} \tag{F2-7}$$

and the coefficient $c$ is determined as follows:

(a) For doubly symmetric I-shapes: $c = 1$ (F2-8a)

(b) For channels: $c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$ (F2-8b)

> **User Note:** For doubly symmetric I-shapes with rectangular flanges, $C_w = \frac{I_yh_o^2}{4}$ and thus Equation F2-7 becomes
>
> $$r_{ts}^2 = \frac{I_yh_o}{2S_x}$$
>
> $r_{ts}$ may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-sixth of the web:
>
> $$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{1}{6}\frac{ht_w}{b_ft_f}\right)}}$$

## F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis with noncompact webs and singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section B4.1 for flexure.





**User Note: I-shaped members for which this section is applicable may be designed conservatively using Section F5.**

The nominal flexural strength, $M_n$, shall be the lowest value obtained according to the *limit states* of compression flange *yielding*, *lateral-torsional buckling*, compression flange *local buckling*, and tension flange yielding.

### 1. Compression Flange Yielding

$$M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \qquad \text{(F4-1)}$$

where

$M_{yc}$ = *yield moment* in the compression flange, kip-in. (N-mm)

### 2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the *limit state* of *lateral-torsional buckling* does not apply.

(b) When $L_p < L_b \le L_r$

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_LS_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc} \qquad \text{(F4-2)}$$

(c) When $L_b > L_r$

$$M_n = F_{cr}S_{xc} \le R_{pc}M_{yc} \qquad \text{(F4-3)}$$

where

$$M_{yc} = F_yS_{xc} \qquad \text{(F4-4)}$$

$$F_{cr} = \frac{C_b\pi^2E}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h_o}\left(\frac{L_b}{r_t}\right)^2} \qquad \text{(F4-5)}$$

For $\frac{I_{yc}}{I_y} \le 0.23$, $J$ shall be taken as zero

where

$I_{yc}$ = moment of inertia of the compression flange about the $y$-axis, in.$^4$ (mm$^4$)

The *stress*, $F_L$, is determined as follows:

(i) When $\frac{S_{xt}}{S_{xc}} \ge 0.7$

$$F_L = 0.7F_y \qquad \text{(F4-6a)}$$

(ii) When $\frac{S_{xt}}{S_{xc}} < 0.7$

$$F_L = F_y\frac{S_{xt}}{S_{xc}} \ge 0.5F_y \qquad \text{(F4-6b)}$$





The limiting laterally *unbraced length* for the limit state of *yielding*, $L_p$, is determined as:

$$L_p = 1.1r_t\sqrt{\frac{E}{F_y}} \qquad \text{(F4-7)}$$

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, $L_r$, is determined as:

$$L_r = 1.95r_t\frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_o} + \sqrt{\left(\frac{J}{S_{xc}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} \qquad \text{(F4-8)}$$

The web *plastification* factor, $R_{pc}$, shall be determined as follows:

(i) When $I_{yc}/I_y > 0.23$

(a) When $\frac{h_c}{t_w} \le \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \qquad \text{(F4-9a)}$$

(b) When $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yc}} \qquad \text{(F4-9b)}$$

(ii) When $I_{yc}/I_y \le 0.23$

$$R_{pc} = 1.0 \qquad \text{(F4-10)}$$

where

$M_p$ $= F_yZ_x \le 1.6F_yS_{xc}$

$S_{xc}$, $S_{xt}$ = elastic section modulus referred to compression and tension flanges, respectively, in.$^3$ (mm$^3$)

$\lambda$ $= \frac{h_c}{t_w}$

$\lambda_{pw}$ = $\lambda_p$, the limiting slenderness for a compact web, Table B4.1b

$\lambda_{rw}$ = $\lambda_r$, the limiting slenderness for a noncompact web, Table B4.1b

$h_c$ = twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of *fasteners* at the compression flange or the inside faces of the compression flange when welds are used, for *built-up sections*, in. (mm)

The effective radius of gyration for lateral-torsional buckling, $r_t$, is determined as follows:

(i) For I-shapes with a rectangular compression flange

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h_o}{d}+\frac{1}{6}a_w\frac{h^2}{h_o d}\right)}} \tag{F4-11}$$

where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \tag{F4-12}$$

$b_{fc}$ = width of compression flange, in. (mm)
$t_{fc}$ = compression flange thickness, in. (mm)

(ii) For I-shapes with a channel cap or a *cover plate* attached to the compression flange

$r_t$ = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, in. (mm)
$a_w$ = the ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components

**User Note:** For I-shapes with a rectangular compression flange, $r_t$ may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-third of the compression portion of the web; in other words

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{6}a_w\right)}}$$

## 3. Compression Flange Local Buckling

(a) For sections with compact flanges, the *limit state* of *local buckling* does not apply.
(b) For sections with noncompact flanges

$$M_n = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right) \tag{F4-13}$$

(c) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_{xc}}{\lambda^2} \tag{F4-14}$$

## G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

### 1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, $V_n$, of unstiffened or stiffened webs according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6 F_y A_w C_v \tag{G2-1}$$





(a) For webs of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$:

$$\phi_v = 1.00 \text{ (LRFD)} \qquad \Omega_v = 1.50 \text{ (ASD)}$$

and

$$C_v = 1.0 \tag{G2-2}$$

User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for $F_y$ = 50 ksi (345 MPa).

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, $C_v$, is determined as follows:

(i) When $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \tag{G2-3}$$

(ii) When $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{G2-4}$$

(iii) When $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_y} \tag{G2-5}$$

where

$A_w$ = area of web, the overall depth times the web thickness, $dt_w$, in.$^2$ (mm$^2$)
$h$ = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)
= for built-up welded sections, the clear distance between flanges, in. (mm)
= for built-up bolted sections, the distance between *fastener* lines, in. (mm)
= for tees, the overall depth, in. (mm)
$t_w$ = thickness of web, in. (mm)

The web plate *shear buckling* coefficient, $k_v$, is determined as follows:

(i) For webs without *transverse stiffeners* and with $h/t_w < 260$:

$$k_v = 5$$

except for the stem of tee shapes where $k_v = 1.2$.





(ii) For webs with transverse stiffeners:

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{G2-6}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)}\right]^2$$

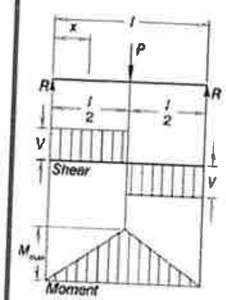
where

$a$ = clear distance between transverse stiffeners, in. (mm)

User Note: For all ASTM A6 W, S, M and HP shapes except M12.5×12.4, M12.5×11.6, M12×11.8, M12×10.8, M12×10, M10×8 and M10×7.5, when $F_y$ = 50 ksi (345 MPa), $C_v$ = 1.0.

## Table 3-23 (continued)
# Shears, Moments and Deflections

### 7. SIMPLE BEAM — CONCENTRATED LOAD AT CENTER




Total Equiv. Uniform Load ........ $= 2P$

$R = V$ ........ $= \frac{P}{2}$

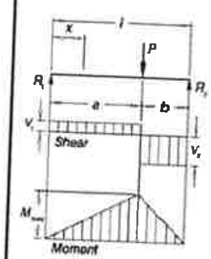
$M_{max}$ (at point of load) ........ $= \frac{Pl}{4}$

$M_x$ (when $x < \frac{l}{2}$) ........ $= \frac{Px}{2}$

$\Delta_{max}$ (at point of load) ........ $= \frac{Pl^3}{48EI}$

$\Delta_x$ (when $x < \frac{l}{2}$) ........ $= \frac{Px}{48EI}(3l^2 - 4x^2)$

### 8. SIMPLE BEAM — CONCENTRATED LOAD AT ANY POINT




Total Equiv. Uniform Load ........ $= \frac{8Pab}{l^2}$

$R_1 = V_1$ ( $= V_{max}$ when $a < b$) ........ $= \frac{Pb}{l}$

$R_2 = V_2$ ( $= V_{max}$ when $a > b$) ........ $= \frac{Pa}{l}$

$M_{max}$ (at point of load) ........ $= \frac{Pab}{l}$

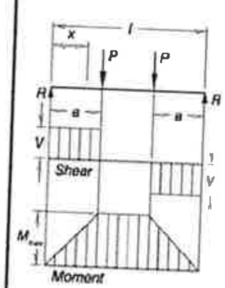
$M_x$ (when $x < a$) ........ $= \frac{Pbx}{l}$

$\Delta_{max}$ $\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right)$ ........ $= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIl}$

$\Delta_a$ (at point of load) ........ $= \frac{Pa^2b^2}{3EIl}$

$\Delta_x$ (when $x < a$) ........ $= \frac{Pbx}{6EIl}(l^2 - b^2 - x^2)$

### 9. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED




Total Equiv. Uniform Load ........ $= \frac{8Pa}{l}$

$R = V$ ........ $= P$

$M_{max}$ (between loads) ........ $= Pa$

$M_x$ (when $x < a$) ........ $= Px$

$\Delta_{max}$ (at center) ........ $= \frac{Pa}{24EI}(3l^2 - 4a^2)$

$\Delta_{max}$ (when $a = \frac{l}{3}$) ........ $= \frac{23Pl^3}{648EI}$

$\Delta_x$ (when $x < a$) ........ $= \frac{Px}{6EI}(3la - 3a^2 - x^2)$

$\Delta_x$ (when $a < x < (l - a)$) ........ $= \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$

**Appendix D – Winch Inspection Report**

AMERICAN
CRANE & HOIST

# American Crane and Hoist Corporation

1234 Washington Street, Boston, MA 02118; Phone: (617) 482 8383, Fax : 8384

November 8, 2024

Job #43059

Collins Engineers Inc.
1485 South Country Trail
Suite 103
East Greenwich, RI 02818

RE: Marine Winch, 49 State Pier
New Bedford

Attn: Christopher Sylvia

Dear Chris,

Enclosed are the OSHA Regulation 1910.179 Inspection Reports dated ***October 23, 2024***
for your ☐ Cranes ☐ Monorails ☐ Hoists ☒ Winches.

These reports reflect the conditions at the time of inspection. **CONDITIONS CAN CHANGE DRASTICALLY WITH A SINGLE MISUSE!**
Therefore, your operators should perform a pre-operation inspection before using this equipment.

**Be advised that this Inspection made no evaluation of the Design, Engineering and Installation of your System.**

Please take notice of items emphasized with the following color codes.

| | | |
|---|---|---|
| **Red** | **DANGER** | Indicates an imminently hazardous situation which, if not corrected, will result in death or serious injury. This signal word is limited to the most extreme situations. |
| **Orange** | **WARNING** | Indicates a potentially hazardous situation which, if not corrected, could result in death or serious injury. |
| **Yellow** | **CAUTION** | Indicates a potentially hazardous situation which, if not corrected, may result in minor or moderate injury. |
| **Blue** | **NOTICE** | Indicates a potential situation which, if not corrected, could or will result in equipment damage or failure. |

Please contact the undersigned with any question or concerns you may have regarding these reports.

Truly yours,

Arthur Leon, P.E.

AMERICAN CRANE & HOIST

# American Crane and Hoist Corporation

1234 Washington Street, Boston, MA 02118; Tel (617) 482-8383; Fax (617) 482-8384

## WINCH INSPECTION REPORT

Customer Name: **Collins Engineers Inc** Our Job No: **43059**

Address: **49 State Pier, New Bedford, MA 02740**

Inspected by: **Robert McConaghy &Tom Grochowski** Date Inspection: **10/23/24** Serial #: **E0440589**

Mfg.: **Beebe Bros** Capacity: **10,000 Lbs** Model: **10000B40**

Other I D: Hp: Voltage: **480V** Phase:

Type: Location: **Main Winch** Winch #:

**LEGEND:** *G*=Good; *F*=Fair; *CL*= Clean and/or Lubricate: *R*=Repair or Replacement Required

| Item | Description | G | F | CL | R | Work done / Comments / Remarks |
|---|---|---|---|---|---|---|
| Mechanical Portion | Frame | X | | | | |
| | Bolts | X | | | | |
| | Gear Box | X | | | | |
| | Guards | | X | | | |
| | Drum | X | | | | |
| | Drum Clamps | X | | | | |
| | Drum Bearing | X | | | | |
| | Rope | | | | X | Replace (Reduced Diameter .729, Loose strands) |
| | Rope Guide, Pressure Roller | — | | | | |
| | Rope Clamps | X | | | | |
| | Sheaves and Sheave Assemblies | X | | | | |
| | Safety Latch | — | | | | |
| | Hook / Hook Block | — | | | | |
| | Live End fittings | — | | | | |
| | Free Wheeling Clutch | — | | | | |
| Power & Controls | Air / Electrical Motor | X | | | | |
| | Electrical control | X | | | | |
| | Push Button | | X | | | |
| | Control Valve | — | | | | |
| | Limit Switch | — | | | | |
| | Slack Line Limit Switch | — | | | | |
| | Safety Torque Limiter | — | | | | |
| | Electrical Motor Brake | X | | | | |
| | Mechanical Load Brake | — | | | | |
| | Accessible Power Disconnecting Means | X | | | | |
| | | | | | | |
| Miscellaneous | Directional Indication / Label | | X | | | |
| | Warning Sign /Label | — | | | | |
| | Capacity Label | X | | | | |
| | Lubrication | | X | | | |
| | Auxiliary equipment | | X | | | |
| | | | | | | |
| | | | | | | |

**Comments/Remarks:**

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## EMERENCY WINCH

| Component | G | F | R | C | L | Comments/Remarks |
|---|---|---|---|---|---|---|
| Hand Wheel or Crank | X | | | X | X | |
| Guards | | | X | | | Rotted out |
| Wire Rope | | | X | | | Replace (Reduced Diameter .729, Loose strands) |
| Load Hook | — | | | | | |
| Hook Bolt | — | | | | | |
| | | | | | | |
| Gearcase | X | | | | X | |
| Brakes | | | X | | | Replace Brake Pad |
| Ratchet | | | | X | X | |
| ANSI Warning Labels | — | | | | | |
| Capacity Label | — | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |

**Further Comments/Remarks:**

**Winch appears not to have been used for years**

**When replacing wire rope, winch should be cleaned, painted, and lubricated.**

# Transfer Bridge Auxiliary Equipment

A) Transfer Bridge Counterweights

- Electric Winch side eye bolt loose
- 1¾” wire rope <u>Fair condition</u>

B) Wire Rope Counterweights

- ⅞” Wire Rope <u>Fair condition</u>
- Top angle iron bent both sides
- Sockets, Sheave Blocks, Shackles, hardware, etc <u>Good condition</u>

C) Transfer Bridge Safety Locks

- Manual Locks <u>OK</u>