Mary Hopkins Goodrich Bridge
Structural Assessment Report

Stockbridge, Massachusetts

REHABILITATION RESTORATION CONSTRUCTION

Prepared for:

Town of Stockbridge &
Foresight Land Services, Inc.
1496 West Housatonic St.
Pittsfield MA 01201

Prepared by:

JDB Consulting Engineers, Inc.
835 Samoset Rd.
Eastham, MA 02642

May 4, 2023

Contents

Appendix:

II DESCRIPTION OF BRIDGE

This office has completed our structural field investigation on March 10, 2023 of the existing pedestrian bridge along the referenced site above as requested by the town of Stockbridge and your office.

This bridge is a single span structure carrying a 3 ft.-11 in. wide public access trail lane over the Housatonic River in the town of Stockbridge, Massachusetts. The year this bridge was constructed is 1936 (The Stockbridge Story 1789-1989 written and published by People of Stockbridge, MA 1989).

The superstructure consists of a 11½" x 1½" timber walkway deck planking spanning between a continuous 3½"x 3½" x 5/16" structural steel angle each side of the walkway floor. These two floor angles are supported and carried on two C4x5.4 structural steel floor beams spaced approximately 3 ft - 1 ½ in. on center. The structural steel floor beams, two walkway angles and other steel members make-up two laterally braced structural steel truss system which consists of the main 95 ft. – 10 in. bridge span crossing the Housatonic River below.

Each single truss (along the overall double structural steel truss system) is connected and supported along one upper 1" diameter suspension cable directly supports a series of ¾" diameter steel suspender rods spaced longitudinally approximately 6 ft.-3 in. along the entire 95 ft.-10 in. bridge and truss span. The suspender rods are vertically connected to the two C4x5.4 lower steel floor beams which are also connected to the bottom of each truss.

The two main 1" diameter suspension cables are seated on two internal structural steel W-sections of which has been inferred based on examination and evaluation of other past suspension bridges of similar design. The Internal steel pylons are founded on a base footing of concrete and stone of which act as main substructural support for the entire superstructure.

The suspension cable backstay is anchored below grade to a concrete dead man. The approximate size of this concrete dead man has not been determined.

No plans of the original superstructure were made available to JDB Consulting Engineers, Inc. for use in determining the live load capacity of this bridge.

III PAST INFORMATION USED THROUGHOUT THIS INVESTIGATIVE EVALUATION

The following documents were used and provided by this office and others to complete this evaluation during the course of this investigation:

- Bridge Betterment Plans and Specifications for Mary Hopkins Goodrich Memorial Bridge by JDB Consulting Engineers (Dated: May 1992 – 72 Sheets)

- Bridge Betterment Computations for Mary Hopkins Goodrich Memorial Bridge by JDB Consulting Engineers (Dated: December 1991 – 8 Sheets)

- Field Notes for Mary Hopkins Goodrich Memorial Bridge by JDB Consulting Engineers (Dated: December 1991 – 12 Sheets)

- NAVFAC Design Manual Structural Engineering Department of the Navy (Dated: December 1970 – Fig. 2-2)

VI PEDESTRIAN LOAD RATING, CRITERIA AND RESULTS

The inventory capacity of this bridge was rated in accordance with the provisions and latest interim revisions of the 1994 edition of the "Manual for Condition Evaluation of Bridges," the 1983 edition of the "Manual for Maintenance Inspection of Bridges," and the 1997 edition of the "Guide Specifications for Design of Pedestrian of Bridges," published by the American Association of State Highway and Transportation Officials (AASHTO).

The inventory bridge rating is the load capacity of which can safely be utilized on an existing structure for an indefinite period of time.

The minimum pedestrian live load as recommended per AASHTO "Guide Specification for Design of Pedestrian Bridge" for the two main suspension cables is 65 psf while the minimum live load as recommended for all secondary members is 85 psf.

The live loadings used were applied to produce the maximum stress.

All data (member sizes, etc.) required to rate this bridge, for which no plans were available, were obtained by JDB Consulting Engineers, Inc. personnel during their site visits.

The consideration of the four underground backstay anchors leading to and connected to the weighted underground dead men along the end of each suspension cable are critical for rating purposes, these elements should be revealed and inspected and rated to complete the present load rating for this bridge when executing rehabilitation betterments along the superstructure.

The critical components of this bridge were the suspension cables seated on the stone end pylons of which have an inventory rating of 16 psf.

This bridge should be posted for 16 psf (*40 pedestrian users) load designation.

*ASCE 7-98 Minimum Design Loads for Building and Other Structures inferred recommended use: 150 lbs. per person occupying 5 sq. ft. surface area.

Refer to the summary table found in the appendix of this report for a breakdown of the final bridge rating for all specific structural members and elements found throughout this bridge crossing.

V PREVENTIVE LONG-TERM MAINTENANCE / REHABILITATION COSTS AND RECOMMENDATIONS

Rehabilitation of the existing bridge superstructure is recommended to upgrade the condition of this bridge crossing for the next 40 years.

The rehabilitation repairs outlined herein will allow this bridge structure to be upgraded to meet the present pedestrian load capacity required as per AASHTO bridge standards which is: 85 psf (217 pedestrian users).

The associated description and estimated construction cost structurally upgrade and maintain this bridge crossing is as follows:

Recommended bridge rehabilitation repairs mainly will require replacement of the two suspension cables and repoint the mortared joints along the two stone masonry pylons by temporarily removing the main bridge truss by crane along this site.

Also, all new recently repointed and old mortared joints and the stone masonry surfaces would be cleaned and sealed with a surface protector along all the mortared joints to minimize long-term deterioration and weatherization. The main bridge truss would be cleaned and repainted before being reinstalled over the river crossing by crane.

The estimated construction cost for bridge rehabilitation for this bridge crossing including final engineering design is: $94,780.00

Refer to the last figure found in the appendix of this report for a breakdown of the final bridge betterment cost for this bridge.

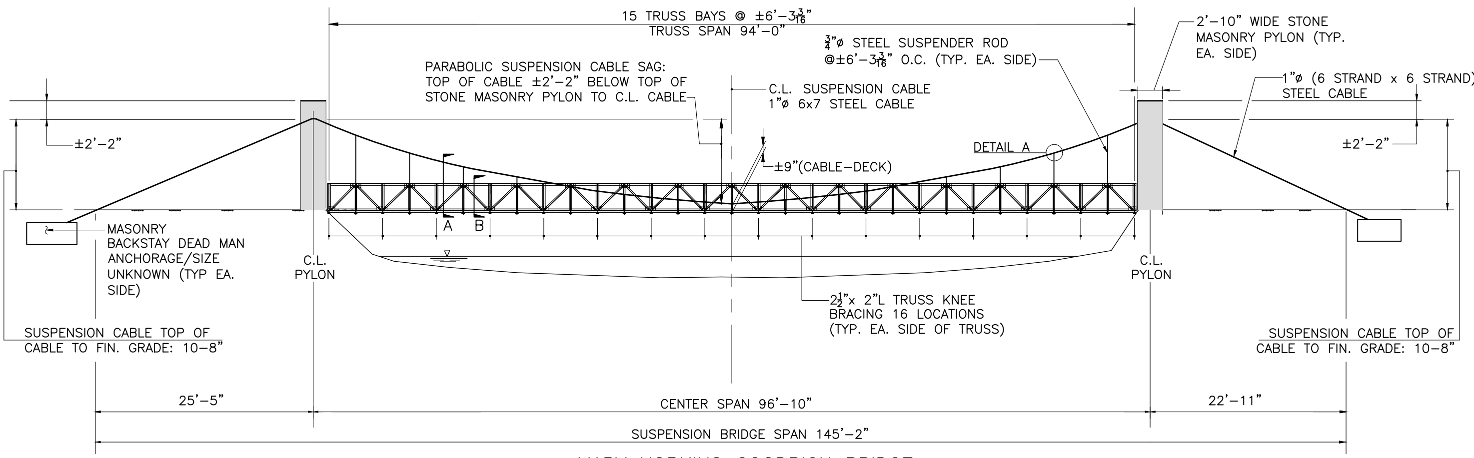
VI LIMITATIONS OF INVESTIGATION

The evaluation contained herein was based on observed measurements and conditions found when several field reconnaissance and a visual inspection visits were completed by the engineer and in part on existing engineering data, plans and reports completed by the engineer and by others.

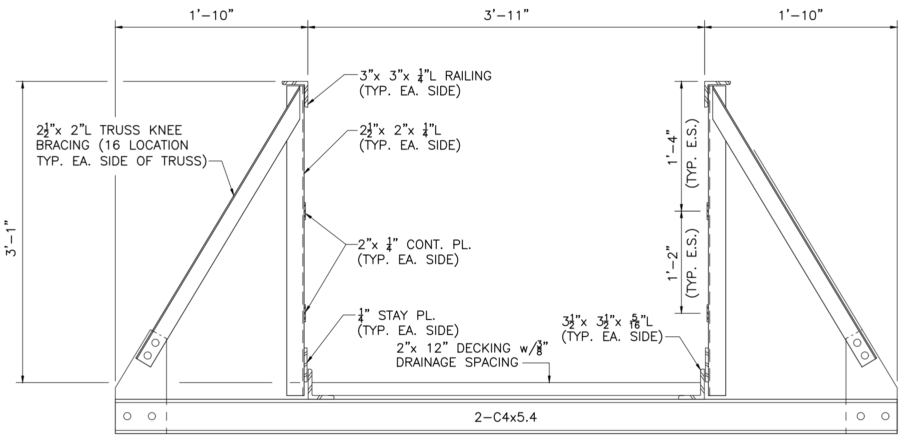
If additional information is brought to the engineer’s attention in the future the analyses, results and recommendations presented herein may be altered as determined by the engineer.

APPENDIX A: Figures

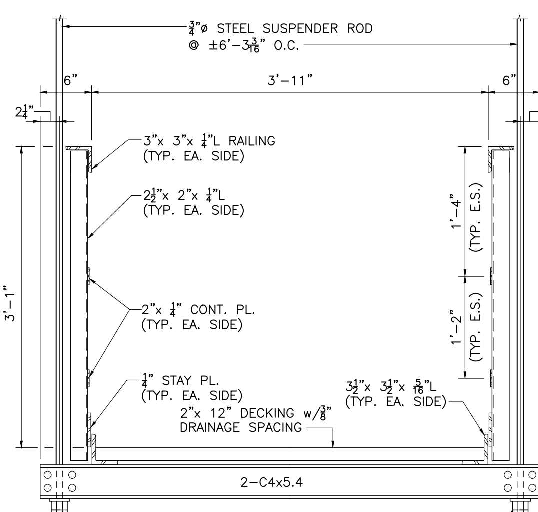
APPENDIX B: Structural Computations



MARY HOPKINS GOODRICH BRIDGE
SOUTH ELEVATION
NOT TO SCALE



SECTION A: LATERAL KNEE BRACE
NOT TO SCALE



SECTION B: STEEL CABLE STAY
NOT TO SCALE

S1

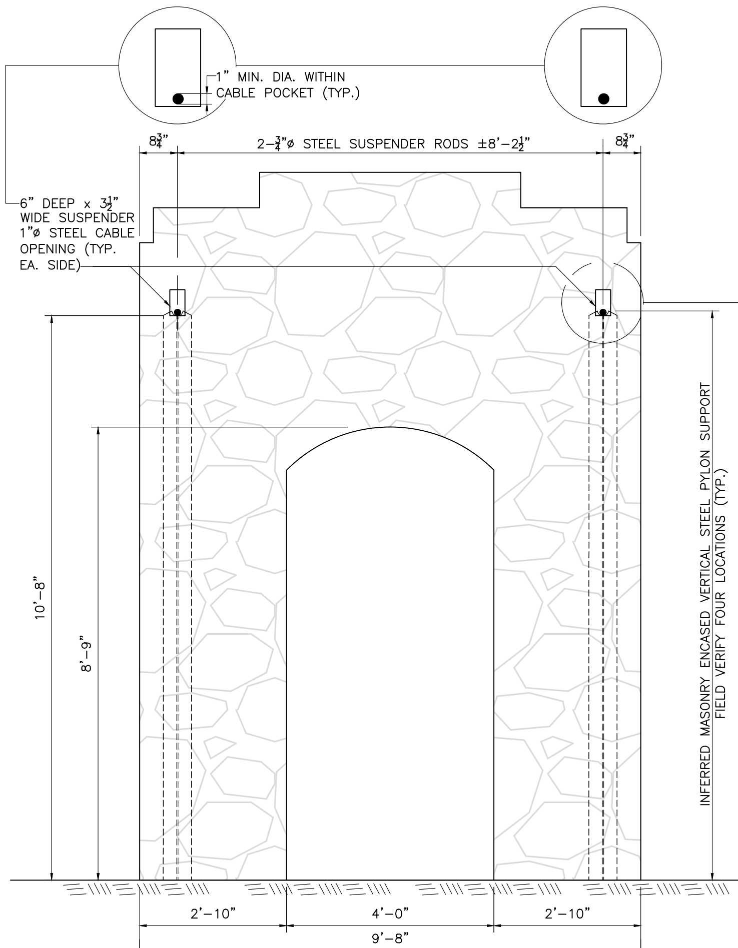
AS BUILT
PLANS & DETAILS

SCALE: AS NOTED | DATE: 04.05.23 | DRAWN BY: JDB | CHECKED BY: DL | PROJ. NO: 02303

Mary Hopkins Goodrich
Pedestrain Bridge
Park St. Stockbridge, MA

JDB Consulting Engineers Inc.
835 Samoset Rd., Eastham, MA 02642

NO.	DATE	BY	REVISION DESCRIPTION USE ONLY PRINTS OF LASTEST DATE



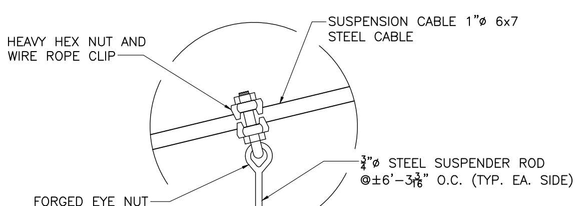
EAST & WEST STONE MASONRY PYLON ELEVATION

NOT TO SCALE

INTERIOR PHOTOGRAPH

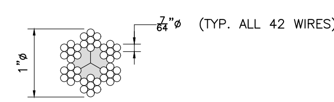
TYPICAL SUSPENDER CABLE & SEAT

NOT TO SCALE



DETAIL A: SUSPENDER ROD – STEEL CABLE STAY CONNECTION

NOT TO SCALE



1936 SUSPENSION 6x7 CABLES

NOT TO SCALE

NOTES:

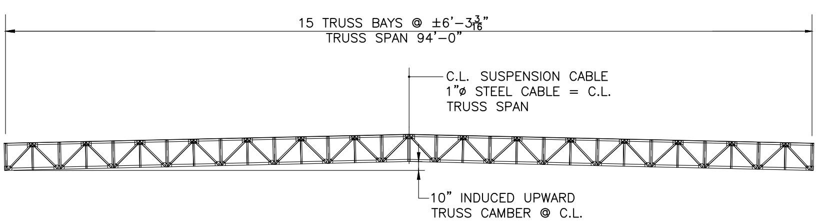
1.0 CORROSION RATE: 0.0016 IN. PER 5 YEARS, NAVFAC DM–2 STRUCTURAL ENGINEERING DESIGN MANUAL (FIGURE 2–2 TIME CORROSION CURVES NON–MARINE ENVIRONMENT)

2.0 AREA OF 42 WIRES=0.394 SQ. IN.

3.0 AREA OF 42 WIRES AFTER 87 YEARS USE DUE TO CORROSION=0.15 SQ. IN. / 21 WIRES SUBJECTED TO CORROSION: 0.25 SQ. IN.

4.0 BREAKING STRENGTH 1"ø 6x7 WIRE ROPE 1936 = $(\frac{28\ ksi}{36\ ksi})$ x 39.7 TONS = 28.66 TONS

5.0 BREAKING STRENGTH 1"ø 6x7 WIRE ROPE 1936 HALF THE WIRES ARE CORRODED = $(\frac{28}{36})$ x 39.7 TONS x 0.25/0.394 = 18.2 TONS



TRUSS ELEVATION

NOT TO SCALE

S2

AS BUILT
PLANS & DETAILS

SCALE: AS NOTED | DATE: 04.05.23 | DRAWN BY: JDB | CHECKED BY: DL | PROJ. NO. 02303

Mary Hopkins Goodrich
Pedestrain Bridge
Park St. Stockbridge, MA

JDB Consulting Engineers Inc.
835 Samoset Rd., Eastham, MA 02642

NO.	DATE	BY	REVISION DESCRIPTION USE ONLY PRINTS OF LASTEST DATE

BREAKDOWN OF BRIDGE RATING

TOWN / CITY: Stockbridge — BRIDGE NO.: Mary Hopkins Goodrich

CARRIES: Pedestrians — OVER: Housatonic River

STRUCTURE NO: N/A — BIN NO: N/A

BRIDGE COMPONENT	INVENTORY RATING LOAD CAPACITY (PSF)
Timber Decking	
Steel Floor Beams	380.00
Longitudinal Truss Beams	472.00
Steel Suspension Hangers	148.00
Steel Suspension Cable	16.00
Steel Suspension Cable 1936	46.00
* Stone/Steel End Pylons	85.00
** Uplift Resistance Dead Man	
** Lateral Resistance of Dead	
Man	
* Based on field inspection	
** To be determined	

comments:
Minimum pedestrian live load as recommended per AASHTO “Guide Specification for Design of Pedestrian Bridge” for the main suspension cables is 65 psf while the minimum live load as recommended for all secondary members is 85 psf.

MARY HOPKINS GOODRICH BRIDGE REHABILITATION COSTS

PROJECT: Mary Hopkins Goodrich Bridge
LOCATION: Park Street, Stockbridge, MA

5/4/23

				ESTIMATE	
ESTIMATE OF QUANTITIES AND COST - BRIDGE REHABILITATION BETTERMENTS					
REF. No.	DESCRIPTION	QTY.	UNIT	UNIT PRICE	AMOUNT
1	CLEANING EXISTING STONE MASONRY	2	PYLON	1650	$ 3,300.00
2	REPOINT & SEAL EXISTING STONE MASONRY JOINTS	80	SF	45	$ 3,600.00
3	TEMPORARILY BRACE EXISTING TRUSS SUSPENDER RODS & REMOVE TRUSS				$ -
4	WITH CONSTRUCTION CRANE AND RESET IN PLACE	3	DAY	7500	$ 22,500.00
5	REPLACE TWO EXISTING SUSPENSION CABLES WITH COATED STEEL CABLES	380	FT	66	$ 25,080.00
6	TRAFFIC CONTROLS FOR CONSTRUCTION OPERATIONS	1	LS	2500	$ 2,500.00
7	REPLACE EXISTING TIMBER BRIDGE DECKING	600	BF	5	$ 3,000.00
8	CLEAN & PAINT EXISTING BRIDGE TRUSS AFTER REMOVED FROM BRIDGE SEATS	2	DAY	1500	$ 3,000.00
9	EXCAVATION AND BACKFILLING TWO DEAD MAN AREAS	3	CY	600	$ 1,800.00
10	ENGINEERING & CONSTRUCTION INSPECTION	1	LS	18000	$18,000
	PROJECT COST:				**$ 82,780.00**
	15% CONTINGENCY COST				**$ 12,000.00**
	FINAL PROJECT COST:				**$ 94,780.00**

APPENDIX B: Structural Computations

F1..3 Strong axis bending of I-shaped members and channels:

Design and deflection of compact I-beam (simply supported single span) braced or unbraced with a uniformly distributed load and/or concentrated load at midspan and/or concentrated load at one end.

Project: Mary Goodrich Hopkins Bridge
Date: 04.06.23
Type: Transverse Span=4′-0″

INPUT DATA:

Span - L (ft.);
Unbraced beam length - L_u (ft.);
Uniformly distributed applied load - W (k/ft.);
Steel yield stress - F_y (ksi);

In[69]:= $L = 4$

Out[69]=
4

In[70]:= $L_u = 4$

Out[70]=
4

In[71]:= $W = \left(\frac{6.25\text{ ft}}{2}\right)\left(85\frac{\text{lbs}}{\text{ft}^2} + 4\frac{\text{lbs}}{\text{ft}^2}\right)\left(\frac{\text{kip}}{1000\text{ lbs}}\right)\left(\frac{\text{ft}}{\text{kip}}\right)$

Out[71]=
0.278125

In[72]:= $F_y = 26$

Out[72]=
26

Beam properties (inches):

Beam section: C4x5.4

In[73]:= $d = 4$

Out[73]=
4

In[74]:= $t_w = 0.184$

Out[74]=
0.184

In[75]:= $b_f = 1.584$

Out[75]=
1.584

In[76]:= $t_f = 0.296$

Out[76]=

0.296

In[77]:= $k = 0.6875$

Out[77]=

0.6875

In[78]:= $I_x = 3.85$

Out[78]=

3.85

SOLUTION:

Computed beam properties:

In[79]:= $s_x = \frac{2 I_x}{d}$

Out[79]=

1.925

Allowable stresses and deflections:

Bending:

In[80]:= $F_{b1} = 0.55 F_y$

Out[80]=

14.3

In[81]:= $F_{b2} = \frac{12000}{\frac{12 L_u d}{A_f}}$

Out[81]=

29.304

In[82]:= $F_b = \text{If}[F_{b1} < F_{b2}, F_{b1}, F_{b2}]$

Out[82]=

14.3

Applied stresses and deflections:

Bending, shear and deflection:

In[83]:= $f_b = N\left[\frac{W L^2 12}{8 s_x}\right]$

Out[83]=

3.46753

SUMMARY:

Bending stress and shear stresses (upper case allowable, lower case applied) - (ksi)

In[84]:= F_b

Out[84]=

```
14.3
```

In[85]:= f_b

Out[85]=

```
3.46753
```

Rating - (psf)

In[86]:= $R_b = N\left[85\,\frac{F_b - f_b\left(\frac{4}{85+4}\right)}{f_b\left(\frac{85-4}{85+4}\right)}\right]$

Out[86]=

380.961

F1..3 Strong axis bending of I-shaped members and channels:

Design and deflection of compact I-beam (simply supported single span) braced or unbraced with a uniformly distributed load and/or concentrated load at mid span and/or concentrated load at one end.

Project: Mary Goodrich Hopkins Bridge

Date: 04.06.23

Type: Longitudinal Span=6′-3-3/16″

- INPUT DATA:

Span - L (ft.);
Uniformly distributed applied load - W (k/ft.);
Steel yield stress - F_y (ksi);

In[1]:= $L = 3.2$

Out[1]= 3.2

In[2]:= $W = (2.0\ \text{ft})\left(85\ \frac{\text{lbs}}{\text{ft}^2} + 4\ \frac{\text{lbs}}{\text{ft}^2}\right)\left(\frac{\text{kip}}{1000\ \text{lbs}}\right)\left(\frac{\text{ft}}{\text{kip}}\right)$

Out[2]= 0.178

In[3]:= $F_y = 26$

Out[3]= 26

Beam properties (inches):

Beam section: $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{5}{16}$" L

In[4]:= $d = 2.51$

Out[4]= 2.51

In[5]:= $I_x = 2.45$

Out[5]= 2.45

- SOLUTION:

Computed beam properties:

In[6]:= $s_x = \frac{I_x}{d}$

Out[6]= 0.976096

Allowable stresses:
Bending:

In[7]:= $F_b = 0.55\, F_y$

Out[7]= 14.3

Applied stresses :
Bending, shear:

In[8]:= $f_b = N[\frac{W\, L^2\, 12}{8\, s_x}]$

Out[8]= 2.80104

SUMMARY:

Bending stress and shear stresses (upper case allowable, lower case applied) - (ksi)

In[9]:= F_b

Out[9]= 14.3

In[10]:=
f_b

Out[10]=
2.80104

Rating - (psf)

In[12]:=
$$R_b = N[85\, \frac{F_b\ -\ f_b\, (\frac{4}{85+4})}{f_b\, (\frac{85-4}{85+4})}]$$

Out[12]=
472.608

Parabolic cable supported suspension pedestrian bridge AASHTO 85 psf Live Load-see note for AASHTO live load truck loading:

Project: Mary Goodrich Hopkins Bridge

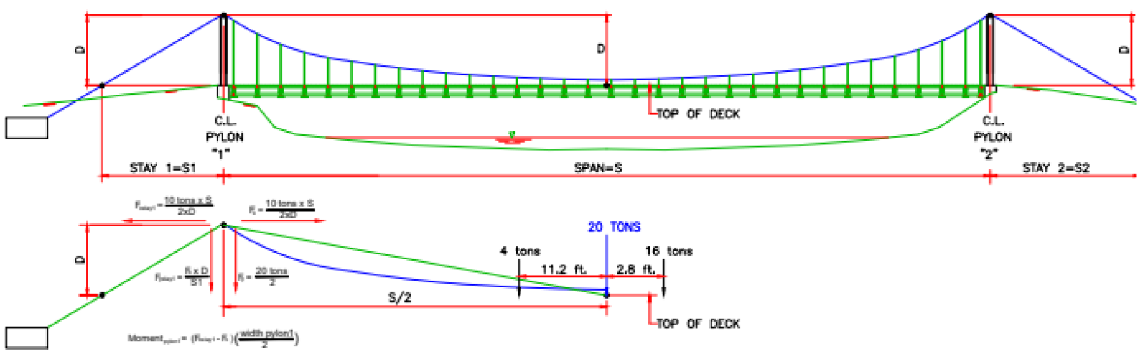
Date: 04.06.23

Type: Span=96′-10″

Note: Main pylon reaction due to truck loading: for a non orthotropic stiffen longitudinally lower deck - determine the maximum vertical compressive load superimposed on the support pylon being evaluated applying the total truck wheel load as a concentrated single wheel load at the centerline of the span-see illustration below.

```
In[1]:=Show[ Import["suspensionbridgepylon.pdf"][[1]], AspectRatio → Automatic, ImageSize → 8 * 70]
```

Out[1]=



AASHTO H-20 LOADS FOR TWO PYLONS ALONG SUSPENSION BRIDGE EACH END OF MAIN SPAN (S)

Structural Equations for Parabolic and Catenary Cables:

```
In[2]:= Show[ Import["StructuralSteelDesinersHandbookMerrittSection15.pdf"][[1]],
   AspectRatio → Automatic, ImageSize → 8*70]
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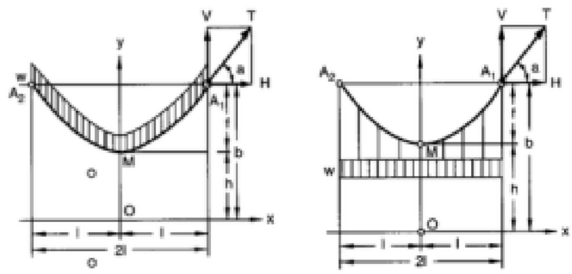
Out[2]=



	Catenary	Parabola
Cable ordinate y†	$y = (e^{x/h} + e^{-x/h})h/2$ $= h \cosh x/h$	$y = h + x^2/2h$
Coordinate b of attachment points A_1 and A_2	$b = f + h$ $= h \cosh l/h$	$b = f + h$ $= h + l^2/2h$ $= (l^2 + 4f^2)/2f$
Sag-span ratio a	$a = f/2l$	$a = f/2l$
Slope y' of cable	$y' = \sinh x/h$	$y' = x/h$
Ordinate h of cable low point	$h = H/w$	$h = H/w$ $= l^2/2f$ $= l/4a$ $= f/8a^2$
Sag f	$f = h(\cosh l/h - 1)$	$f = l^2/2h$
Half length s of cable	$s = \sqrt{b^2 - h^2}$ $= \sqrt{f^2 + 2hf}$ $= \sqrt{2fb - f^2}$	$s = \frac{l}{2h}\sqrt{h^2 + l^2} + \frac{h}{2} \times [\log_e (l + \sqrt{h^2 + l^2}) - \log_e h]$ $= \frac{l}{2h}\sqrt{h^2 + l^2} + \frac{h}{2}\sinh^{-1}\frac{l}{h}$ $\approx l\left(1 + \frac{8}{3}a^2 - \frac{32}{5}\alpha^4 + \frac{256}{7}a^6 - \cdots\right)$
Angle α of cable at A_1 and A_2	$\tan\alpha = \sin l/h$ $= \frac{1}{h}\sqrt{f^2 + 2fh}$ $= \frac{1}{b-f}\sqrt{2fb - f^2}$ $= s/h$ $= \frac{1}{h}\sqrt{b^2 - h^2}$ $\cos\alpha = h/b$ $= h/(h + f)$ $= (b - f)/b$ $\sin\alpha = \frac{1}{b}\sqrt{2fb - f^2}$ $= s/b$ $= \frac{1}{b}\sqrt{b^2 - h^2}$ $= \frac{1}{h+f}\sqrt{f^2 + 2fh}$	$\tan\alpha = l/h$ $= \sqrt{2f/h}$ $= \sqrt{2f/(b - f)}$ $= 2f/l$ $= 4a$ $= 1/\sqrt{1 + 16a^2}$ $\cos\alpha = h/\sqrt{h^2 + l^2}$ $= h/\sqrt{h^2 + 2fh}$ $= (b - f)/\sqrt{b^2 - f^2}$ $= l/\sqrt{l^2 + 4f^2}$ $\sin\alpha = \sqrt{2f/(b + f)}$ $= l/\sqrt{h^2 + l^2}$ $= \sqrt{2f/(h + 2f)}$ $= 2f/\sqrt{l^2 + 4f^2}$ $= 4a/\sqrt{1 + 16a^2}$
Vertical component V of cable tension	$V = w\sqrt{b^2 - h^2}$ $= w\sqrt{f^2 + 2fh}$ $= w\sqrt{2fb - f^2}$ $= ws$	$V = w\sqrt{2fh}$ $= wl$ $= 4wah$
Horizontal component H of cable tension	$H = wh$ $= w(b - f)$	$H = wh$ $= wl^2/2f$ $= wl/4a$ $= wf/8a^2$
Cable tension T	$T = wb$	$T = w\sqrt{h^2 + l^2}$ $= w\sqrt{2fh + h^2}$ $= wh\sqrt{1 + 16a^2}$

*Adapted from H. Odenhausen, "Statical Principles of the Application of Steel Wire Ropes in Structural Engineering," *Acier-Stahl-*

Steel, no. 2, pp. 51–65, 1965.
† Since

$$\cosh \frac{x}{h} = 1 + \frac{(x/h)^2}{2!} + \frac{(x/h)^4}{4!} + \frac{(x/h)^6}{6!} + \cdots$$

the parabolic profile (obtained by dropping the third and subsequent terms) is an approximation for the catenary. The accuracy of this approximation improves as sag f becomes smaller.

▪ INPUT DATA:

Diameter of steel suspender rods - D_{sr} (in);
Cable (wire rope 6x7 / 1" Dia. rope breaking strength 28.66 tons as built 1939) nominal strength uncoated with factor of safety of 3 of one cable - f_{cable} (kips);

Area of cable with all 42 - 7/64" diameter wires along all six bundled wire rope strands are 100% effective and corrosion free A_c (sq. in.)

Area of cable with half - 7/64" diameter wires along all six bundled wire rope strands are corroded NAVFAC DM-2 Naval Design Manual Figure 2-2 (Structural Engineering Sheet 2-2-8 Time Corrosion Curves for Steel) 87 years of steel corrosion loss along each wire - i.e. 21 wires have a wire diameter after corrosion of: $\frac{7/64\text{ in}}{2}$ - 0.0016 in x $\frac{87\text{ years}}{5\text{ years}}$ = 0.0268 in. A_{cc} (sq. in.)

REVISED CABLE BREAKING STRNGHT BASED ON NO COROSION: Cable (wire rope 6x7 / 1" Dia. rope breaking strength 28.66 tons as built 1939) nominal strength uncoated with a factor of safety of 2.5 - f_{Rcable} (kips);

REVISED CABLE BREAKING STRNGHT BASED ON COROSION: Cable (wire rope half the wire are corroded with a diamenter of: 0.268 in.) nominal strength uncoated with a factor of safety of 2.5 - f_{Rcable} (kips);

Yield stress of steel suspender rods factor of safety of 3 - F_{crod} (ksi);
Main span - S (ft.);
Centerline to centerline cables - C_b (ft.);
Spacing hangers - S_h (ft.);
Cable sag - f_{sag} (ft.);
Ordinate of low cable - h (ft.);
Height of pylon from top of deck to cable - D_p (ft.);
Uniformly distributed live load - secondary members - L_u (psf);
Uniformly distributed live load - main members (cables, pylons & dead man) - L_{main} (psf);

```
In[3]:=Dsr = 0.75
Out[3]=0.75
```

In[4]:= $f_{cable} = N[(\frac{28.66\text{ ton}}{2.5})(\frac{2\text{ kips}}{\text{ton}})(\frac{1}{\text{kips}})]$

```
Out[4]=22.928
```

In[5]:= $A_c = N[42(\pi(\frac{\frac{7}{64}}{2})^2)]$

Out[5]= 0.394617

In[6]:= $A_{cc} = 0.25$

Out[6]= 0.25

In[7]:= $f_{Rcable} = N[(\frac{28.66\,ton}{2.5})(\frac{2\,kips}{ton})(\frac{1}{kips})(\frac{A_{cc}}{A_c})]$

Out[7]= 14.5255

In[8]:= $F_{crod} = \frac{26}{3}$

Out[8]= $\frac{26}{3}$

In[9]:= $S = 96.833$

Out[9]= 96.833

In[10]:=
$C_b = 4.54$

Out[10]=
4.54

In[11]:=
$S_h = 6.26$

Out[11]=
6.26

In[12]:=
$f_{sag} = 10.25$

Out[12]=
10.25

In[13]:=
$D_p = 11$

Out[13]=
11

In[14]:=
$L_u = 85$

Out[14]=
85

In[15]:=

$L_{main} = 65$

Out[15]=

65

PARABOLIC CABLE PROPERIES:

Cable sag f_{sag} (ft)

In[16]:=

f_{sag}

Out[16]=

10.25

Ordinate dimension from cable low point h_o (ft)

In[17]:=

$$h_o = \frac{\left(\frac{S}{2}\right)^2}{2\ f_{sag}}$$

Out[17]=

114.349

Length of entire cable L_{cable} (ft.)

In[18]:=

$$l_c = \frac{S}{2}$$

Out[18]=

48.4165

In[19]:=

$$L_{cabled} = \frac{l_c}{h_o} \sqrt{h_o^2 + l_c^2} + h_o\,(\mathrm{Log}[l_c + \sqrt{h_o^2 + l_c^2}\,] - \mathrm{Log}[h_o])$$

Out[19]=

99.6531

■ SOLUTION:

DEAD LOAD: Hangers & Parabolic cables

Weight of entire 3.15 ft. long lateral truss (two trusses) knee bays including timber decking & steel connector plates @ 6.265625 ft. O.C W_{lk} (lbs.)

In[20]:=

$$W_{lk} = \left(2\left(3.62\,\frac{\text{lbs}}{\text{ft}}\right)3.1\,\text{ft} + 2\left(3.62\,\frac{\text{lbs}}{\text{ft}}\right)3.1\,\text{ft} + 2\left(3.7\,\frac{\text{lbs}}{\text{ft}}\right)3.62\,\text{ft} + 4\left(1.7\,\frac{\text{lbs}}{\text{ft}}\right)3.62\,\text{ft} + 2\left(7.2\,\frac{\text{lbs}}{\text{ft}}\right)3.62\,\text{ft} + 2\left(5.4\,\frac{\text{lbs}}{\text{ft}}\right)8\,\text{ft} + \left(20\,\frac{\text{lbs}}{\text{ft}^3}\right)\left(\frac{1.5\,\text{in}}{12\,\frac{\text{in}}{\text{ft}}}\right)1\,\text{ft}\times 3.15\,\text{ft}\right)1.2\,\frac{1}{\text{lbs}}$$

Out[20]=
291.234

Weight of entire 3.15 ft. long lateral truss (two trusses) cable stay (not including two suspender hanger rods) timber decking & steel connector plates @ 6.265625 ft. O.C W_{ls} (lbs.)

In[21]:=

$$W_{ls} = \left(2\left(3.7\,\frac{\text{lbs}}{\text{ft}}\right)3.15\,\text{ft} + 2\left(3.62\,\frac{\text{lbs}}{\text{ft}}\right)3.1\,\text{ft} + 4\left(1.7\,\frac{\text{lbs}}{\text{ft}}\right)3.62\,\text{ft} + 2\left(7.2\,\frac{\text{lbs}}{\text{ft}}\right)3.15\,\text{ft} + 2\left(5.4\,\frac{\text{lbs}}{\text{ft}}\right)4.1\,\text{ft} + \left(20\,\frac{\text{lbs}}{\text{ft}^3}\right)\left(\frac{1.5\,\text{in}}{12\,\frac{\text{in}}{\text{ft}}}\right)1\,\text{ft}\times 3.15\,\text{ft}\right)1.2\,\frac{1}{\text{lbs}}$$

Out[21]=
201.462

TENSILE RATING: Hangers

Applied dead load tension T_{dlh} (kips)

In[22]:=

$$W_f = (W_{lk} + W_{ls})$$

Out[22]=
492.696

In[23]:=

$$T_{dlh} = \frac{(W_f)}{1000}$$

Out[23]=
0.492696

Applied live load tension T_{llh} (kips)

In[24]:=

$$T_{llh} = L_{main}\, C_b\, S_h\, \frac{1}{2\times 1000}$$

Out[24]=
0.923663

Tensile capacity of rod T_{caph} (kips)

```
In[25]:=
```

$$T_{caph} = \pi\,(\frac{3}{8})^2\,0.55\,F_{crod}$$

```
Out[25]=
        2.10585
```

Hanger rating R_{frod} (psf)

```
In[26]:=
```

$$R_{frod} = \frac{L_u\,(T_{caph} - T_{dlh})}{T_{llh}}$$

```
Out[26]=
        148.45
```

TENSILE RATING: One Cable - Originally As Built

PARABOLIC CABLE PROPERIES:

Ordinate dimension from cable low point h_o (ft)

```
In[27]:=
```

$$h_o = \frac{(\frac{S}{2})^2}{2\,f_{sag}}$$

```
Out[27]=
        114.349
```

Applied dead load tension in cable T_{dlca} (kips)

```
In[28]:=
```

$$l_c = \frac{S}{2}\,f_{sag}$$

```
Out[28]=
        496.269
```

Entire dead load per cable D_{tdl} $(\frac{\text{kips.}}{\text{ft}})$

```
In[29]:=
```

$$D_{tdl} = (\frac{(W_{lk} + W_{ls})\,\text{lbs}}{2 \times 3.1\,\text{ft} \times 1000\,\frac{\text{lbs}}{\text{kips}}})\,\frac{\text{ft}}{\text{kips}}$$

```
Out[29]=
        0.0794671
```

Applied dead load tension T_{dlca} (kips)

In[30]:=

$$T_{dlca} = D_{tdl}\sqrt{h_o^2 + \left(\frac{S}{2}\right)^2}$$

Out[30]=

9.86797

Applied live load tension T_{llca} (kips)

In[31]:=

$$T_{llca} = \frac{C_b}{2}\frac{L_{main}}{1000}\sqrt{h_o^2 + \left(\frac{S}{2}\right)^2}$$

Out[31]=

18.3223

Tensile capacity of cable T_{capca} (kips)

In[32]:=

$$T_{capca} = f_{cable}$$

Out[32]=

22.928

Cable rating R_{ca} (psf)

In[33]:=

$$R_{ca} = \frac{L_{main}\left(T_{capca} - T_{dlca}\right)}{T_{llca}}$$

Out[33]=

46.3316

In[34]:=

TENSILE RATING: One Cable - half the wires are corroded

PARABOLIC CABLE PROPERIES:

Ordinate dimension from cable low point h_o (ft)

In[35]:=

$$h_o = \frac{\left(\frac{S}{2}\right)^2}{2\,f_{sag}}$$

Out[35]=

114.349

Applied dead load tension in cable T_{dlca} (kips)

In[36]:=

$$l_c = \frac{S}{2} f_{sag}$$

Out[36]=

496.269

Entire dead load per cable D_{tdl} $(\frac{kips.}{ft})$

In[37]:=

$$D_{tdl} = \left(\frac{(W_{lk} + W_{ls})\ lbs}{2 \times 3.1\ ft \times 1000\ \frac{lbs}{kips}}\right) \frac{ft}{kips}$$

Out[37]=

0.0794671

Applied dead load tension T_{dlca} (kips)

In[38]:=

$$T_{dlca} = D_{tdl} \sqrt{h_o^2 + \left(\frac{S}{2}\right)^2}$$

Out[38]=

9.86797

Applied live load tension T_{llca} (kips)

In[39]:=

$$T_{llca} = \frac{C_b}{2} \frac{L_{main}}{1000} \sqrt{h_o^2 + \left(\frac{S}{2}\right)^2}$$

Out[39]=

18.3223

Tensile capacity of cable T_{capca} (kips)

In[40]:=

$$T_{capca} = f_{Rcable}$$

Out[40]=

14.5255

Cable rating R_{ca} (psf)

In[41]:=

$$R_{ca} = \frac{L_{main}\,(T_{capca} - T_{dlca})}{T_{llca}}$$

Out[41]=

16.523