Report on Techniques for Bridge Strengthening

Design Example – Stringer Retrofit - Composite Action and Continuity

Changes

June 2018

U.S. Department of Transportation
Federal Highway Administration

FHWA-HIF-18-044

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Foreword

This design example is targeted at bridge owners and bridge engineers who have been tasked with strengthening an existing bridge. It is intended to be an aid in designing appropriate bridge strengthening retrofits. Each example, in the set of examples, covers a different situation for which strengthening is commonly needed.

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<tr><td colspan="4">16. Abstract
This example involves the replacement of stringers during re-decking on an existing truss/floorbeam/stringer bridge. The existing stringers are non-composite rolled W24x76 beams, that were designed for HS-20 live loads. The design criteria is to provide new stringers to obtain a HS-25 live load rating factor equal to or greater than 1.0, while minimizing the weight of the new stringers. The flexural live load ratings of the new stringers were significantly increased by both making the stringers composite with the new deck and changing the continuity of the stringer spans. This example only involves a study of the flexural resistance of a typical interior span for the new and existing stringers using the Strength-I Limit State. This example is based on AASHTO LRFD Bridge Design Specifications, 7th Edition.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Design Procedure

The following American Association of State Highway and Transportation Officials (AASHTO) documents were used for this example.

Publication Title	Publication Year	Publication Number	Available for Download
AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014	2014	---	No
Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, 2003	2003	—	Yes

Summary of Design/Analysis Procedure:

First, the bridge data, material properties and section properties must be defined. It is also necessary to identify the standard or specification that will be used for the analysis/design along with the required design live loading and applicable load combinations and design factors.

The solution of the example will follow the following general steps:

Step 1. Calculate the new and existing dead load and live load moments.

Step 2. Calculate the non-composite and composite section properties for the new and existing stringers.

Step 3. Calculate nominal flexural resistance of the new and existing stringers.

Step 4. Calculate new and existing live load rating factors.

A summary will be given at the end of the example, comparing the changes between the new and existing stringer's flexural resistances and the dead load and live load moments.

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

A_{c3n} = area of transformed slab for long term composite section (in.^{2})
A_{cn} = area of transformed slab for short term composite section (in.^{2})
A_s = area of deck slab longitudinal reinforcing steel within effective width (in.^{2})
A_{str} = gross area of rolled beam stringer(in.^{2})
b_{3n} = transformed width of deck slab for long term composite section (in.)
b_{eff} = slab effective flange width for composite stringers (in.)
b_f = width of the flange of a rolled shape (in.)
b_n = transformed width of deck slab for short term composite section (in.)
C_b = moment gradient modifier
d = depth of a rolled shape (in.)
d_{As} = distance between centers of gravity of the stringer and the slab steel (in.)
DF_{M1} = moment live load distribution for single lane
DF_{M2} = moment live load distribution for two lanes
D_p = distance from top of deck to the N.A. of the composite section at the plastic moment (in.)
d_s = distance between centers of gravity of the stringer and the slab in compression (in.)
D_t = total depth of composite section (in.)
E_c = modulus of elasticity of concrete (ksi)
eg = distance between centers of gravity of the beam and the deck (in.)
E_s = modulus of elasticity of steel (ksi)
f'_c = compressice strength of concrete deck slab (ksi)
F_{nc} = nominal resistance of a flange (ksi)
$F_{nc(FLB)}$ = nominal compression flange local buckling flexural resistance (ksi)
$F_{nc(LTB)}$ = nominal compression flange lateral torsional buckling flexural resistance (ksi)
F_{ue} = specified minimum tensile strength of existing steel (ksi)
F_{un} = specified minimum tensile strength of new steel (ksi)
F_{ye} = specified minimum yield strength of existing steel (ksi)
F_{yn} = specified minimum yield strength of new steel (ksi)
F_{yr} = compression flange stress at onset of nominal yielding (ksi)
I_{c3n} = moment of inertia of long term composite section (in.^{4})
I_{cn} = moment of inertia of short term composite section (in.^{4})
I_{cnf} = moment of inertia of negative flexure composite section (in.^{4})
IM = live load impact factor
I_{os3n} = moment of inertia of transformed slab for long term composite section (in.^{4})
I_{osn} = moment of inertia of transformed slab for short term composite section (in.^{4})
I_x = moment of inertia of rolled beam about major principal axis (in.^{4})
K_g = longitudinal stiffness parameter (in.^{4})

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

L = span length of stringer (ft)
L_b = unbraced length of stringer (in.)
L_p = limiting unbraced length to achieve nominal flexural resistance of M_p (in.)
L_r = limiting unbraced length to achieve onset of nominal yielding in flange (in.)
M_{AD} = additional moment on short term composite section to cause nominal yielding (k-in.)
M_{CR} = elastic lateral torsional buckling moment(k-in.)
M_{DC1} = moment due to non-composite dead load (k-in.)
M_{DC2} = moment due to composite dead load (k-in.)
M_{DW} = moment due to wearing surface (k-in.)
M_{LL} = moment due to live load (k-in.)
M_{LL+I} = moment due to live load plus impact (k-in.)
M_n = nominal moment resistance (k-in.)
M_p = plastic moment (k-in.)
M_r = factored flexural resistance (k-in.)
M_u = moment due to factored loads (k-in.)
M_{UL} = moment from 1 kip/ft uniform load (k-in.)
M_y = yield moment (k-in.)
n = modular ratio, E_s/E_c
n_{cs} = number of stringer s in the cross section which share a DC2 or DW uniform dead load
R_b = web load shedding factor
RF = live load rating factor
R_h = hybrid factor
r_T = radius of gyration of compression flange plus 1/3 of the compression web area (in.)
S = stringer spacing in within cross section (in.)
$S_{c3n\ bf}$ = section modulus for bottom flange on long term composite section (in.^{3})
$S_{c3n\ s}$ = section modulus for slab on long term composite section (in.^{3})
$S_{cn\ bf}$ = section modulus for bottom flange on short term composite section (in.^{3})
$S_{cn\ s}$ = section modulus for slab flange on short term composite section (in.^{3})
$S_{c\ nfbf}$ = section modulus for bottom flange on negative flexure composite section (in.^{3})
S_x = section modulus of existing member about the major principal axis (in.^{3})
t_f = thickness of the flange of a rolled shape (in.)
t_{hnch} = thickness of haunch (in.)
t_{sc} = thickness of slab in compression for composite section (in.)
t_{slab} = thickness of deck slab (in.)
t_w = thickness of the web of a rolled shape (in.)
uw_c = uniform density weight of concrete (lb./ft^3)
uw_{DC1} = uniform weight of non-composite dead load (lb./ft)
uw_{DC2} = uniform weight of composite dead load (lb./ft)
uw_{DW} = uniform density weight of wearing surface (lb./ft)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

uw_{mb} = uniform weight of median barrier (lb./ft)
uw_p = uniform weight of parapet (lb./ft)
uw_s = uniform density weight of steel (lb./ft^3)
uw_{ws} = uniform density weight of wearing surface dead load (lb./ft^3)
$W_{c\text{-}c}$ = the curb-to-curd width for wearing surface dead load (in.)
wt_{hnch} = uniform weight of haunch per stringer (lb./ft)
wt_m = uniform weight of miscellaneous dead loads per stringer (lb./ft)
wt_{mb} = uniform weight of median barrier per stringer (lb./ft)
wt_p = uniform weight of parapet per stringer (lb./ft)
wt_{slab} = uniform weight of slab per stringer (lb./ft)
wt_{str} = uniform weight of stringer (lb./ft)
wt_{ws} = uniform weight of wearing surface per stringer (lb./ft)
y_{bs} = distance to bottom of slab in compression to c.g. of stringer (in.)
y_{sc} = distance between c.g of slab in compression to c.g. of stringer (in.)
y'_{c3n} = distance between c.g of stringer and c.g of long term composite section (in.)
y'_{cn} = distance between c.g of stringer and c.g of short term composite section (in.)
y'_{cnf} = distance between c.g of stringer and c.g of negative flexure composite section (in.)
$+M_c$ = factored flexural resistance for composite section in positive flexure (k-in.)
$+M_{nc}$ = factored flexural resistance for non-composite section in positive flexure (k-in.)
$-M_c$ = factored flexural resistance for composite section in negative flexure (k-in.)
$-M_{nc}$ = factored flexural resistance for non-composite section in negative flexure (k-in.)
ϕM_n = factored flexural resistance (k-in.)
ϕ_c = resistance factor for compression
ϕ_f = resistance factor for flexure
ϕ_u = resistance factor for fracture on net section of tension member
ϕ_y = resistance factor for yielding on gross section of tension member
γ_{DC} = load factor for dead load, non-composite and composite
γ_{DW} = load factor for future wearing surface
γ_{LL} = load factor for live load and live load impact
η_D = load modifier for ductility
η_i = load modifier relating to ductility, redundancy and operational classification
η_I = load modifier for operational classification
η_R = load modifier for redundancy
λ_f = slenderness ratio for the compression flange
λ_{pf} = limiting slenderness ratio for a compact flange
λ_{rf} = limiting slenderness ratio for a noncompact flange

Worked Design Example

Introduction:

This example involves the replacement of stringers during re-decking on an existing truss/floorbeam/stringer bridge. The existing stringers are non-composite rolled W24x76 beams, that were designed for HS-20 live loads. The design criteria is to provide new stringers to obtain a HS-25 live load rating factor equal to or greater than 1.0, while minimizing the weight of the new stringers. The flexural live load ratings of the new stringers were significantly increased by both making the stringers composite with the new deck and changing the continuity of the stringer spans. This example only involves a study of the flexural resistance of a typical interior span for the new and existing stringers using the Strength-I Limit State.

This example will be based on AASHTO LRFD Bridge Design Specifications, 7th Edition.

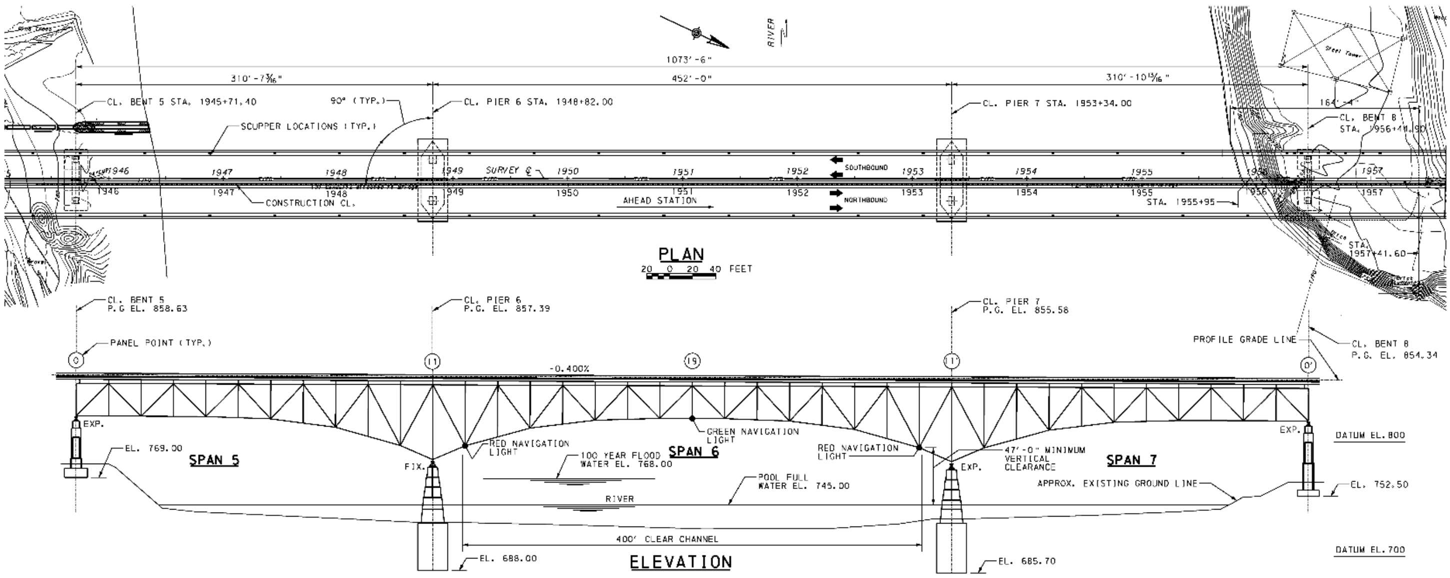
Bridge Data:

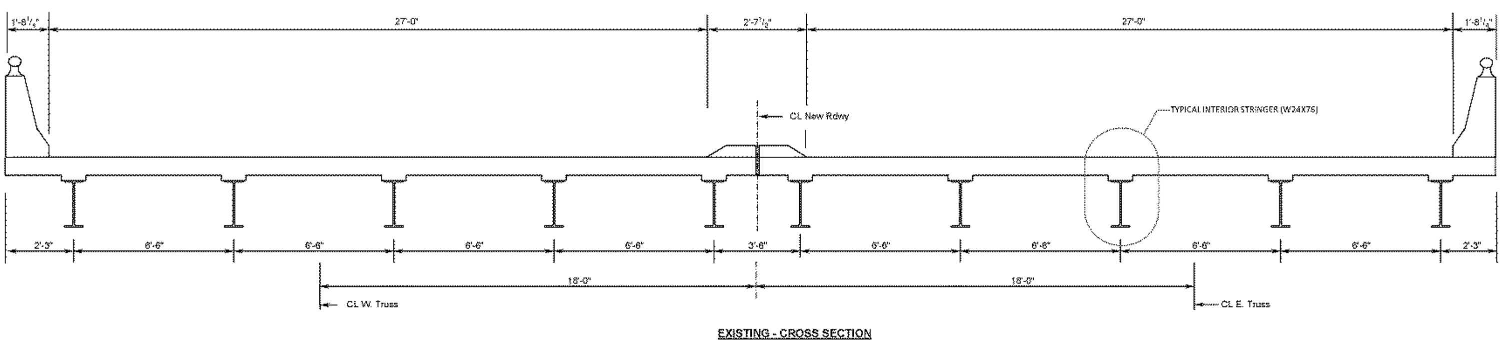
Bridge Type:	3 – Span Continuous Deck Truss.
Span length:	1073.5 ft between centerline of bearings
Year Built:	1961
Location:	State of Pennsylvania
Stringers:	Non-Composite Rolled W24x76
Barrier Type:	F-Shape (560 lb./ft.)
Out-to -Out of Bridge:	60’-0”
Curb-to -Curb Width:	27’-0” Each Direction
Slab Thickness:	8.0 in.
Overlay Thickness:	2.5 in.
Haunch Height:	1.5 in.
Stringer Spacing:	6’-6”
Panel Length:	28’-3”
Unbraced Length:	14’-$1^1/_2$” (Diaphragms at Mid-span)

Material Properties:

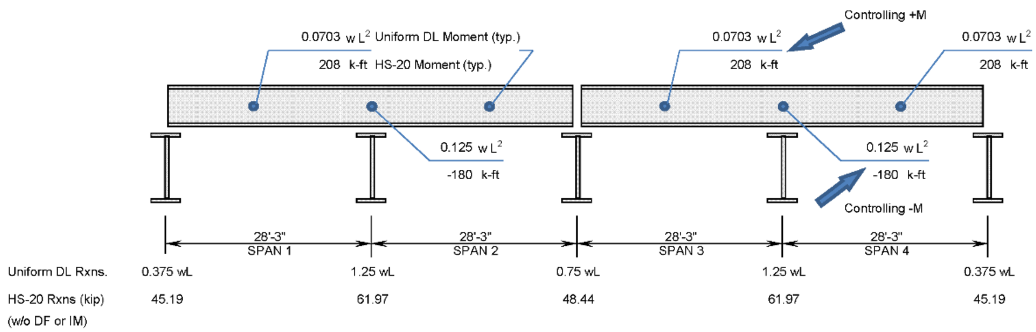
Steel Modulus of Elasticity:	E_s = 29,000 ksi
Concrete Modulus of Elasticity:	E_c = 3,640 ksi
Existing Steel Yield Strength:	F_{ye} = 36 ksi (ASTM A36)
Existing Steel Tensile Strength:	F_{ue} = 58 ksi
New Steel Yield Strength:	F_{yn} = 36 ksi (ASTM A36)
New Steel Tensile Strength:	F_{un} = 58 ksi
Concrete Compressive Strength:	f_c‘ = 3.5 ksi
Unit Weight of Steel:	uw_s = 490 lb./ft^3
Unit Weight of Concrete:	uw_c = 150 lb./ft^3
Unit Weight of Overlay:	uw_{ws} = 145 lb./ft^3

DESIGN EXAMPLE



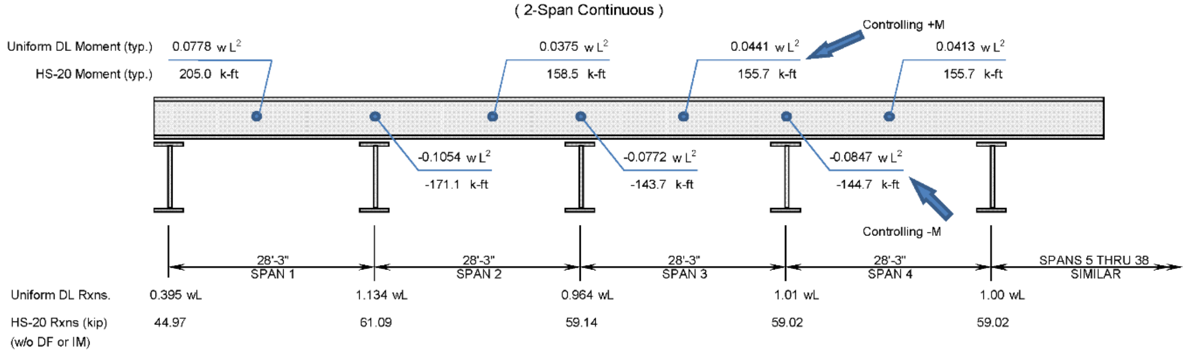


DESIGN EXAMPLE



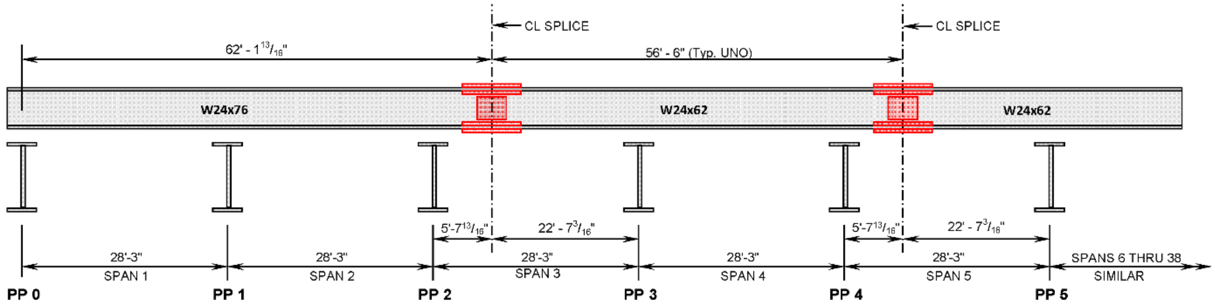
EXISTING - STRINGER SPANS

(2-Span Continuous)



NEW - STRINGER SPANS

(Multiple-Span Continuous)



NEW - STRINGER SPANS

(End Span W24x76 and Interior Spans W24x62)

LRFD Factors:

For this example use: $\eta_D = 1.0$ $\eta_R = 1.0$ $\eta_I = 1.0$ therefore: $\eta_i = 1.0$

Resistance Factors	
Type of Resistance	**Factor, f**
Flexure	$\phi_f = 1.00$
Axial Compression	$\phi_c = 0.95$
Tension, fracture in An	$\phi_u = 0.80$
Tension, yielding in Ag	$\phi_y = 0.95$

Load Combinations and Load Factors				
Limit State	**Load Factors**			
	DC	**DW**	**LL**	**IM**
Strength I	1.25	1.50	1.75	1.33
Service II	1.00	1.00	1.30	1.33

Dead Load and Live Load Moments:

Calculate the deal and live load moments for both the new and existing conditions. Assume the deck is replaced in-kind, so there is no significant change in uniform dead loads between the new and existing conditions. The live loads will be increased to HS-25 for the new condition, as compared to HS-20 for the existing condition. The moments will also vary, in some cases, significantly between the new and existing conditions due to a change in the continuity of the stringer arrangements.

Dead Loads:

Beam self weight

wt_{str} = 76 lb./ft. W24x76

Deck Slab

t_{slab} = 8 in. deck slab thickness (without overlay)

S = 6.5 ft. girder spacing (tributary width for slab weight on girder)

$wt_{slab} = t_{slab}\, S\, uw_c$ (1 ft./12 in.) = (8 in.)(6.5 ft.)(150 lb./ft.^{3}) (1 ft./12 in.) = 650 lb./ft.

Deck Haunch

t_{hnch} = 1.5 in. typical haunch height (top of top flange to bottom of deck slab)

w_{hnch} = 12 in. typical width of haunch (1.5 in beyond top flange on each side)

$wt_{hnch} = t_{hnch}\, w_{hnch}\, uw_c$ (1 ft.^{2}/144 in.^{2}) = (8 in.)(12 in.)(150 lb./ft.^{3} (1 ft.^{2}/144 in.^{2}) = 18.8 lb./ft.

Dead Loads (continued):

Concrete Parapet

uw_P = 560 lb./ft. weight of one f-shape parapet per linear foot

n_{cs} = 5 girders (number of girders to share the weight of one parapet)

wt_p = $1(uw_p)/\ n_P$ = 1(560 lb./ft.) / (5 girders) = 112 lb./ft.

Median Barrier

uw_{mb} = 107.5 lb./ft. weight of one f-shape parapet per linear foot

n_{cs} = 5 girders (number of girders to share the weight of one median barrier)

wt_{mb} = $1(uw_{mb})/\ n_P$ = 1(107.5 lb./ft.) / (5 girders) = 21.5 lb./ft.

Miscellaneous

wt_m = 20 lb./ft. assumed weight for miscellaneous items: stiffeners, cross frame, etc..

Deck Overlay (Wearing Surface)

w_{ws} = 30 psf (2.5 in. overlay thickness)

$W_{c\text{-}c}$ = 27.0 ft. Curb-to-curb width

n_{cs} = 5 girders (number of girders to share the weight of the overlay)

wt_{ws} = $W_{c\text{-}to\text{-}c}\ w_{ws}$ (1 ft./12 in.) / n_{cs} = (30 psf)(27.0 ft.)/ (5)

wt_{ws} = 162.0 lb./ft.

Summary

$uw_{DC1} = wt_{str} + wt_{slab} + wt_{hnch} + wt_m$

uw_{DC1} = (76.0 + 650 + 20) lb./ft. = 746 lb./ft

$uw_{DC2} = wt_p + wt_{bm}$

uw_{DC2} = (112+21.5) lb./ft. = 133.5 lb./ft (for existing, assumed non-composite beam, add to uw_{DC1})

$uw_{DW} = wt_{ws}$ = 162.0 lb./ft. (for existing, assumed non-composite beam, add to uw_{DC1})

Assume deck is replaced in-kind. The only change in dead loads between new and existing will be the self weight of the stringers. This should be a small enough difference to neglect.

Live Loads:

Non-Composite Section Properties (Obtained from AISC published properties for W24x76):

A_{str} = 22.4 in.^{2} I_x = 2,100 in.^{4} d = 23.92 in. S_x = 176 in.^{3}

Live Load Distribution Factors:

from above:

t_{slab} = 8 in L = 28.25 ft. E_s = 29,000 ksi E_c = 3,640 ksi I_x = 2,100 in.^{4}

S = 6.5 ft. A_{grd} = 22.4 in.^{2}

also calculate:

n = E_s / E_c = 29,000 ksi / 3,640 ksi = 7.97, use 8.0

eg = d/2 + t_{hnch} + t_{slab}/2 = 23.92 in./2 + 1.5 in. + 8 in./2 = 17.46 in.

K_g = n (I_x + A_{str} eg^2) = 8 (2,100 in.^{4} + 22.4 in.^{2} (17.46 in.)^2) = 71,429 in.^{4}

$$\left(\frac{K_g}{12\,L\,t_{slab}^{\ 3}}\right)=\left(\frac{71{,}429\text{ in.}^4}{12\,(28.25\,\text{ft.})(8\,\text{in.})^3}\right)=0.412$$

Interior Beam – Moment Distribution Factor (use for New and Existing): LRFD Table 4.6.2.2.2b-1

One Lane

$$DF_{M1}=0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12\,L\,t_{slab}^{\ 3}}\right)^{0.1}=0.06+\left(\frac{6.5\,\text{ft.}}{14}\right)^{0.4}\left(\frac{6.5\,\text{ft.}}{28.25\,\text{ft.}}\right)^{0.3}(0.412)^{0.1}=0.493$$

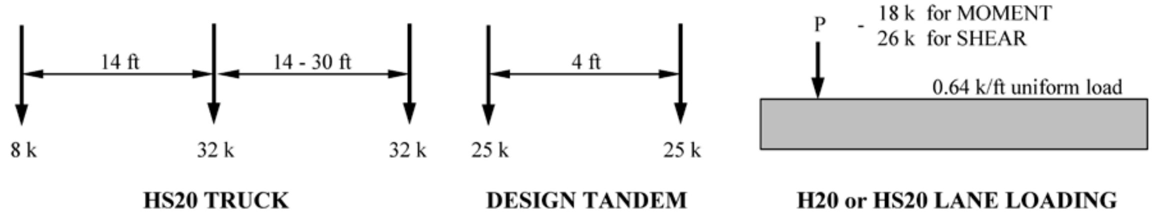
Two Lanes

$$DF_{M2}=0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12\,L\,t_{slab}^{\ 3}}\right)^{0.1}=0.075+\left(\frac{6.5\,\text{ft.}}{9.5}\right)^{0.6}\left(\frac{6.5\,\text{ft.}}{28.25\,\text{ft.}}\right)^{0.2}(0.412)^{0.1}=0.618$$

Dynamic Load Allowance, IM (impact factor):

IM = 33% LRFD Table 3.6.2.1-1

Design Vehicular Live Load: LRFD Table 3.6.1.2



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Live Loads (continued):

The moments values for M_{UL} and M_{LL} were determined using, CONTINOUS BEAM ANALYSIS (CBA) software, for PennDOT. The unit dead loads were then factored using calculated dead load uniform weights for DC1, DC2 and DW. The basic HS-20 live loads were factored using calculated distribution factors and impact factors.

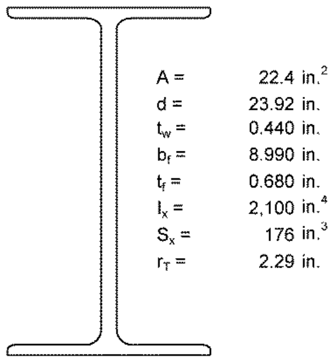
DESIGN MOMENTS						
Location	M_{UL} k-ft	M_{DC1} k-ft	M_{DC2} k-ft	M_{DW} k-ft	M_{LL} k-ft	M_{LL+I} k-ft
Existing (2-Span Continuous)						
Span 1	56.0	59.3	-	-	208.4	171.3
Support 1	-99.5	-105.5	-	-	-180.0	-148.0
New (Multispan Continuous)						
Span 1	62.1	47.5	8.3	10.1	205.0	168.6
Support 1	-84.1	-64.3	-11.2	-13.6	-171.1	-140.7
Span 2	26.9	20.6	3.6	4.4	158.5	130.3
Support 2	-61.6	-47.1	-8.2	-10.0	-143.7	-118.1
Span 3	33.4	25.6	4.5	5.4	155.8	128.1
Support 3	66.3	50.7	8.9	10.7	-144.7	-119.0

M_{UL} = Moment on simple span with 1 k./ft. uniform unit load.
M_{DC1} = Non-composite dead load moment, M_{UL} DC1 (1 kip/1,000 lb.)
M_{DC2} = Non-composite dead load moment, M_{UL} DC2 (1 kip/1,000 lb.)
M_{DW} = Dead load moment from wearing surface, M_{UL} DW (1 kip/1,000 lb.)
M_{LL} = Live load moment for HS-20 Truck
M_{LL+I} = Live load moment plus impact, (M_{LL} DF_M) (1+IM)

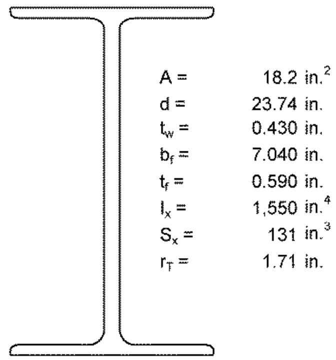
The live load moments for both the new and existing conditions shown above, are basic HS-20 moments for comparison. The new condition live load moments can be increased to HS-25 by simply factoring up the HS-20 moments by a factor of 1.25.

Note that the Span 1 and Support 1 +M and –M values only varied slightly between the 2-span continuous and the multi-span continuous stringer arrangements. However, for the typical interior span and interior support on the multi-span arrangement, starting at Span 3 and Support 3, both the +M and –M values were reduced significantly as compare to the 2-span continuous arrangement. This was used to reduce the stringer weights for the interior spans, while only using a similar sized stringer as the existing stringer, at the end spans. The other technique used to reduce the stringer weight, was to make the new stringers composite with the deck.

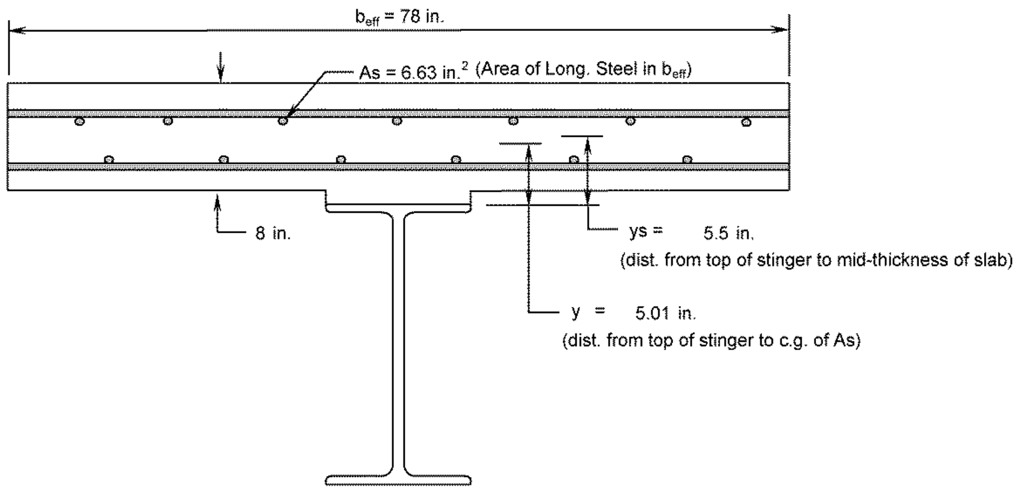
Determine the Non-composite and Composite Section Properties:



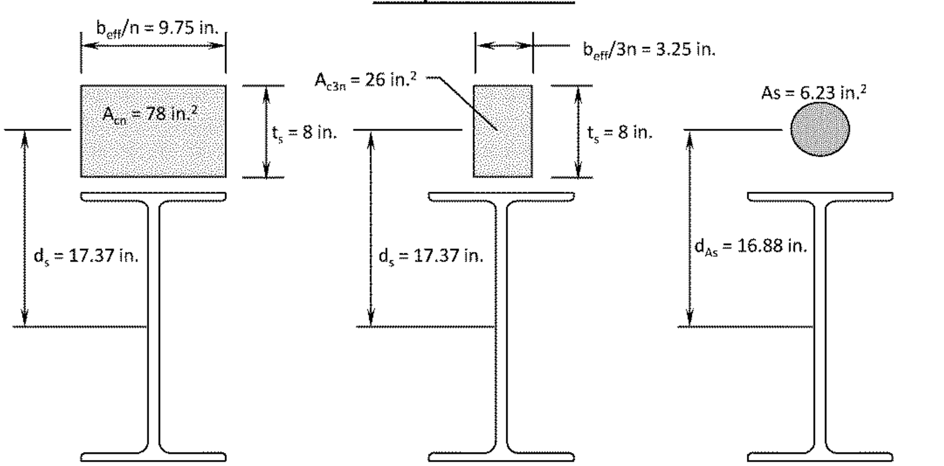
Non-Composite W24x76



Non-Composite W24x62



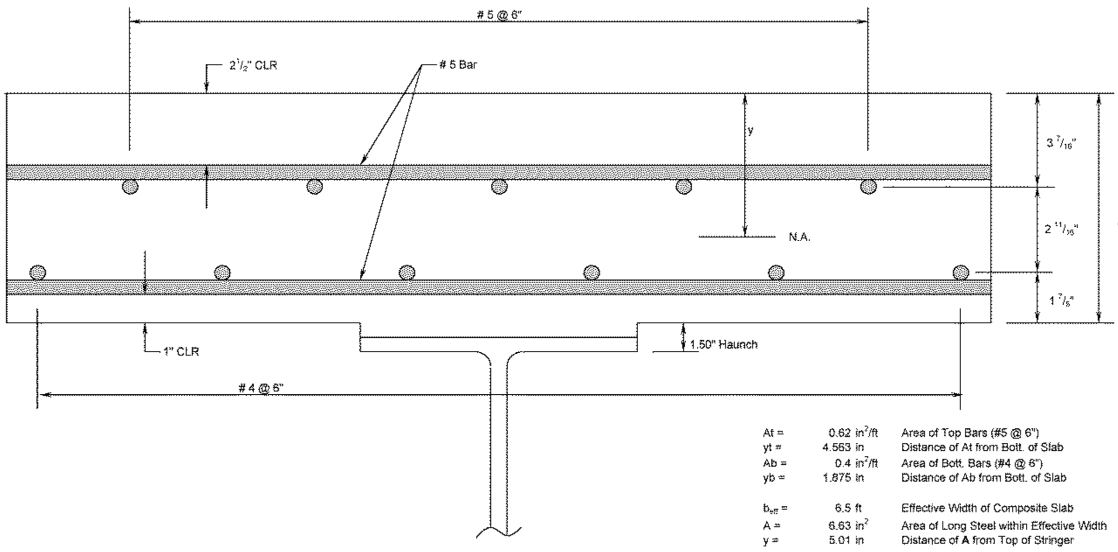
Composite W24x62



Composite W24x62 - n
(Short Term Composite for LL+I)

Composite W24x62 - 3n
(Long Term Composite for DC2 & DW)

Composite W24x62 -M
(Negative Flexure Composite)



Interior Stringer at Negative Moment Region

Determine the Non-composite and Composite Section Properties:

The non-composite section properties was taken from AISC published values for the W-shapes.

Composite Section Properties (New W24x62):

The effective width of the slab for the composite section of an interior stringer is taken as the average of the stinger spacing, on either side of the stringer.

S	= 6.5 ft	Typical interior stringer spacing	
b_{eff}	= 6.5 ft or 78 in.	Effective width of slab	LRFD 4.6.2.6
n	= 8	Modular ratio	
ts	= 8 in.	Thickness of slab	
b_n	= 9.75 in.	Transformed slab width for short term composite section (n)	
b_{3n}	= 3.25 in.	Transformed slab width for short term composite section (3n)	
A_s	= 6.63 in.^{2}	Area of longitudinal steel in effective width of slab	
$A_{c\ n}$	= 78 in.^{2}	Transformed slab area for short term composite section (n)	
$A_{c\ 3n}$	= 26 in.^{2}	Transformed slab area for short term composite section (n)	
d	= 23.74 in.	Depth of W24x62	
A_{str}	= 6.63 in.^{2}	Area of W24x62	
d_s	= 17.37 in.	Distance between centroids of the W24x62 and the slab	
d_{As}	= 16.88 in.	Distance between centroids of the W24x62 and A_s	

Determine the Non-composite and Composite Section Properties (continued):

Short Term (n)Composite Section Properties (New W24x62):

$y'_{cn} = (A_{str}\, y_{str} + A_{cn}\, d_s) / (A_{str} + A_{cn}) = ((18.2 \text{ in.}^2)(0 \text{ in.}) + (78 \text{ in.}^2)(17.37 \text{ in.})) / (18.2 \text{ in.}^2 + 78 \text{ in.}^2)$
$y'_{cn} = 14.08$ in. distance from centroids of the W24x62 and the composite section.

$y'_{cn} = 14.08 \text{ in.} > d/2 = 23.74/2 = 11.87$ in., which put the c.g. of the composite section into the slab. The slab concrete below the c.g. must not be used, since this is the tension zone. Therefore adjust the A_{cn} and d_s values and recalculate y'_{cn}, this may take several iterations.

Assume only the top 7.26 in. of the slab are in compression.

$A_{cn} = 7.26 \text{ in.} (9.75 \text{ in.}) = 70.77 \text{ in.}^2$
$d_s = d/2 + t_{hnch} + t_s - t_{sc}/2 = 11.87 \text{ in.} + 1.5 \text{ in.} + 8 \text{ in.} - (7.26 \text{ in.})/2 = 17.74 \text{ in.}$

$y'_{cn} = ((18.2 \text{ in.}^2)(0 \text{ in.}) + (70.77 \text{ in.}^2)(17.74 \text{ in.})) / (18.2 \text{ in.}^2 + 70.77 \text{ in.}^2) = 14.11 \text{ in.}$

Check the location of the bottom of the assumed slab in compression.

$y_{bs} = d/2 + t_{hnch} + t_s - t_{sc} = 11.87 \text{ in.} + 1.5 \text{ in.} + 8 \text{ in.} - 7.26 \text{ in.} = 14.11 \text{ in} = y'_{cn}$ OK

Determine the short term composite section moment of inertia, $I_{c\,n}$, section modulus for the top of slab, $S_{cn\,s}$ and the bottom of the bottom flange, $S_{cn\,bf}$.

$d_s = t_{sc}/2 = 7.26 \text{ in.} / 2 = 3.63 \text{ in.}$
$I_{osn} = (t_{sc}^3\, b_n) / 12 = ((7.26 \text{ in.})^3 (9.75 \text{ in})) / 12 = 310.91 \text{ in.}^4$

$I_{cn} = \Sigma\, Io + \Sigma\, Ad^2 = I_{osn} + A_{cn}\, d_s^2 + I_{str} + A_{str}\, y'^2_{cn}$

$I_{cn} = 310.91 \text{ in.}^4 + (70.77 \text{ in.}^2)(3.63 \text{ in.})^2 + 1{,}550 \text{ in.}^4 + (18.2 \text{ in.}^2)(14.11 \text{ in.})^2 = 6{,}418 \text{ in.}^4$

$S_{cn\,s} = I_{cn} / t_{sc} = 6{,}418 \text{ in.}^4 / 7.26 \text{ in.} = 884 \text{ in.}^3$

$S_{cn\,bf} = I_{cn} / (y'_{cn} + d/2) = 6{,}418 \text{ in.}^4 / (14.11 \text{ in.} + 11.87 \text{ in.}) = 247 \text{ in.}^3$

Determine the Non-composite and Composite Section Properties (continued):

Long Term (3n)Composite Section Properties (New W24x62):

y'_{c3n} = $(A_{str}\, y_{str}+A_{c3n}\, d_s) / (A_{str}+A_{c3n})$ = ((18.2 in.^{2})(0 in.)+(26 in.^{2})(17.37 in.)) / (18.2 in.^{2}+26 in.^{2})
y'_{c3n} = 10.22 in. distance from centroid of the W24x62 to the composite section

y'_{c3n} = 10.22 in. < d/2 = 23.74/2 = 11.87 in., which puts the c.g. of the composite section into the rolled steel shape. Therefore the assumption of using the full slab thickness is OK.

Determine the long term composite section moment of inertia, $I_{c\,3n}$, section modulus for the top of slab, $S_{c3n\,s}$ and the bottom of the bottom flange, $S_{c3n\,bf}$.

$I_{os3n} = (t_s^3\, b_{3n}) / 12$ = ((8 in.)^3(3.25 in))/ 12 = 138.67 in.^{4}

d_s = $d/2 + t_{hnch} + t_s/2 - y'_{c3n}$ = 11.87 in. + 1.5 in. + 4 in. – 10.22 in. = 7.15 in.

$I_{c3n} = \Sigma\, Io + \Sigma\, Ad^2 = I_{os3n} + A_{cn}\, d_s^2 + I_{str} + A_{str}\, y'^2_{c3n}$

I_{c3n} = 138.67 in.^{4} + (26 in.^{2})(7.15 in.)^2 + 1,550 in.^{4} + (18.2 in.^{2})(10.22 in.)^2 = 4,920 in.^{4}

$S_{c3n\,s} = I_{c3n} / (d_s + t_s / 2)$= 4,920 in.^{4} / (7.15 in.+ (8 in.)/2) = 441 in.^{3}

$S_{c3n\,bf} = I_{c3n} / (y'_{c3n} + d/2)$= 4,920 in.^{4} / (10.22 in. + 11.87 in.) = 223 in.^{3}

Negative Flexure Composite Section Properties (New W24x62):

y'_{cnf} = $(A_{str}\, y_{str}+A_s\, d_{As}) / (A_{str}+A_s)$ = ((18.2 in.^{2})(0 in.)+(6.63 in.^{2})(16.88 in.)) /(18.2 in.^{2}+6.63 in.^{2})
y'_{cnf} = 4.51 in.

Determine the negative flexure composite section moment of inertia, $I_{c\,nf}$, and the section modulus for the bottom of the bottom flange, $S_{cnf\,bf}$.

$I_{cnf} = \Sigma\, Io + \Sigma\, Ad^2 = A_s\,(d_{As} - y'_{cnf})^2 + I_{str} + A_{str}\, y'^2_{cnf}$

I_{cnf} = (6.63 in.^{2})(16.88 - 4.51 in.)^2 + 1,550 in.^{4} + (18.2 in.^{2})(4.51 in.)^2 = 2,935 in.^{4}

$S_{c\,nfbf} = I_{cnf} / (d_s / 2 + y'_{cnf})$= 2,935 in.^{4} / (11.87 in.+ 4.51)= 179 in.^{3}

Determine Existing and New Factored Flexural Resistances:

The top flange of the existing stringers are fully embedded into the concrete deck slab. Per AASHTO Manual for Condition Evaluation of Bridges, Section 6.6.9.3 and C6.6.9.3, the top flange may be assumed to be adequately braced by the concrete deck. The flexural resistance calculations for positive flexure will be based on a continually braced compression flange. For negative flexure, the bottom flange is braced by diaphragms at the floorbeams and midspan, so the unbraced length is 28.25 ft. / 2 = 14.125 ft.

Existing W24x76 Non-composite Flexural Resistance:

Compression Flange Flexural Resistance – Flange Local Buckling:

$$\lambda_f = \frac{b_{ft}}{2t_{ft}} = \frac{8.99\,\text{in.}}{2(0.68\,\text{in.})} = 6.61$$ LRFD Eqn. 6.10.8.2.2-3

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 10.785$$ LRFD Eqn. 6.10.8.2.2-4

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 15.894$$ LRFD Eqn. 6.10.8.2.2-5

if $\lambda_f \le \lambda_{pf}$, $6.61 \le 10.784$ yes, then $F_{nc(FLB)} = F_y = 36\,\text{ksi}$ LRFD Eqn. 6.10.8.2.2-1

Compression Flange Flexural Resistance – Lateral Torsional Buckling:

$r_T = 2.29\,\text{in.}$ $F_{yr} = 0.7F_y = 25.2\,\text{ksi}$ $R_h = 1.0$ $R_b = 1.0$ and $C_b = 1.0$

$$L_p = 1.0\,r_T\sqrt{\frac{E}{F_y}} = 2.29\,\text{in.}\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 65.0\,\text{in.}$$ LRFD Eqn. 6.10.8.2.3-1

$$L_r = \pi r_T\sqrt{\frac{E}{F_{yr}}} = \pi\,(2.29\,\text{in.})\sqrt{\frac{29{,}000\,\text{ksi}}{25.2\,\text{ksi}}} = 245\,\text{in.}$$ LRFD Eqn. 6.10.8.2.3-5

if $L_b \le L_p$, $169.5\,\text{in.} \le 65\,\text{in.}$, then $F_{nc(LTB)} = F_y = 36\,\text{ksi}$ LRFD Eqn. 6.10.8.2.3-1 and 2

else

if $L_p < L_b \le L_r$, $65\,\text{in.} < 169.5\,\text{in.} \le 245\,\text{in.}$, then

$$F_{nc(LTB)} = C_b\left[1-\left(1-\frac{F_{yr}}{R_hF_y}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]R_bR_hF_y =$$

$$F_{nc(LTB)} = 1.0\left[1-\left(1-\frac{25.2\,\text{ksi}}{36\,\text{ksi}}\right)\left(\frac{169.5\,\text{in.}-65\,\text{in.}}{245\,\text{in.}-65\,\text{in.}}\right)\right]36\,\text{ksi} = 29.73\,\text{ksi}$$

New W24x62 Non-composite Flexural Resistance:

Compression Flange Flexural Resistance – Flange Local Buckling:

$$\lambda_f = \frac{b_{ft}}{2\,t_{ft}} = \frac{7.04\,\text{in.}}{2(0.59\,\text{in.})} = 5.97$$ LRFD Eqn. 6.10.8.2.2-3

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 10.785$$ LRFD Eqn. 6.10.8.2.2-4

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 15.894$$ LRFD Eqn. 6.10.8.2.2-5

if $\lambda_f \le \lambda_{pf}$, $5.97 \le 10.784$ yes, then $F_{nc(FLB)} = F_y = 36\,\text{ksi}$ LRFD Eqn. 6.10.8.2.2-1

Compression Flange Flexural Resistance – Lateral Torsional Buckling:

$r_T = 1.71\text{in.}$ $F_{yr} = 0.7F_y = 25.2\text{ ksi}$ $R_h = 1.0$ $R_b = 1.0$ and $C_b = 1.0$

$$L_p = 1.0\,r_T\sqrt{\frac{E}{F_y}} = 1.71\text{in.}\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 48.5\,\text{in.}$$ LRFD Eqn. 6.10.8.2.3-1

$$L_r = \pi\,r_T\sqrt{\frac{E}{F_{yr}}} = \pi\,(1.71\text{in.})\sqrt{\frac{29{,}000\,\text{ksi}}{25.2\,\text{ksi}}} = 182\,\text{in.}$$ LRFD Eqn. 6.10.8.2.3-5

if $L_b \le L_p$, $169.5\text{in.} \le 48.5\text{in.}$, then $F_{nc(LTB)} = F_y = 36\,\text{ksi}$ LRFD Eqn. 6.10.8.2.3-1 and 2

else

if $L_p < L_b \le L_r$, $48.5\text{ in.} < 169.5\text{in.} \le 182\text{ in.}$, then

$$F_{nc(LTB)} = C_b\left[1 - \left(1 - \frac{F_{yr}}{R_h F_y}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]R_b R_h F_y =$$

$$F_{nc(LTB)} = 1.0\left[1 - \left(1 - \frac{25.2\text{ ksi}}{36\text{ ksi}}\right)\left(\frac{169.5\text{ in.} - 48.5\text{ in.}}{182\text{ in.} - 48.5\text{ in.}}\right)\right]36\text{ ksi} = 26.2\text{ ksi}$$

Existing W24x76 Stringer Non-composite Flexural Resistance:

Compression Flange Flexural Resistance:

F_{nc}= minimum of $F_{nc(FLB)}$ and $F_{nc(LTB)}$ = 29.73 ksi

$\phi M_n = \phi_b F_{nc} S_x =$ (1.0)(29.73 ksi)(176 in.^{3}) (1 ft. / 12 in.) = 436 k-ft

Tension Flange Flexural Resistance:

F_{nc}= F_y = 36 ksi

$\phi M_n = \phi_b F_{nc} S_x =$ (1.0)(36 ksi)(176 in.^{3}) (1 ft. / 12 in.) = 528 k-ft

New W24x62 Stringer Non-composite Flexural Resistance:

Compression Flange Flexural Resistance:

F_{nc}= minimum of $F_{nc(FLB)}$ and $F_{nc(LTB)}$ = 26.2 ksi

$\phi M_n = \phi_b F_{nc} S_x =$ (1.0)(26.2 ksi)(131 in.^{3}) (1 ft. / 12 in.) = 286 k-ft

Tension Flange Flexural Resistance:

F_{nc}= F_y = 36 ksi

$\phi M_n = \phi_b F_{nc} S_x =$ (1.0)(36 ksi)(131 in.^{3}) (1 ft. / 12 in.) = 393 k-ft

New W24x62 Stringer Composite Negative Flexural Resistance:

Compression Flange Flexural Resistance:

F_{nc}= minimum of $F_{nc(FLB)}$ and $F_{nc(LTB)}$ = 26.2 ksi, same as non-composite section

S_{cnf} = 179 in.^{3}, composite negative flexure section modulus for bottom flange

$\phi M_n = \phi_b F_{nc} S_{cnf} =$ (1.0)(26.2 ksi)(179 in.^{3}) (1 ft. / 12 in.) = 391 k-ft

New W24x62 Stringer Composite Positive Flexural Resistance:

Recall:

S_x = 131 in.^{3}		Non-composite Section
$S_{cn\ s}$ = 884 in.^{3}	$S_{cn\ bf}$ = 247 in.^{3}	Short Term (n) Composite Section
$S_{c3n\ s}$ = 441 in.^{3}	$S_{c3n\ bf}$ = 223 in.^{3}	Long Term (n) Composite Section

New W24x62 Stringer Composite Positive Flexural Resistance (continued):

The positive flexural resistance of a composite section is dependent on the factored moments applied to the section. Refer to LRFD Appendix D6.2.2 for determining the yield moment .

Use the Span 3 moments for these calculations:

Unfactored Moments:

M_{DC1} = 25.6 k-ft M_{DC2} = 4.5 k-ft M_{DW}= 5.4 k-ft M_{LL+I} = 128.1 k-ft

γ_{DC} = 1.25 γ_{Dw} = 1.5 γ_{L} = 1.75

$\gamma_{HS\text{-}25}$ = 1.25

Factored Moments:

M_{DC1} = 32 k-ft M_{DC2} = 5.6 k-ft M_{DW}= 8.1 k-ft M_{LL+I} = 280.2 k-ft

Determine the yield moment, My:

$$F_y = \frac{M_{DC1}}{S_x} + \frac{M_{DC2} + M_{DW}}{S_{c3n}} + \frac{M_{AD}}{S_{cn}} \qquad \text{LRFD Eqn. D6.2.2} - 1$$

$$M_{AD} = \left(F_y - \frac{M_{DC1}}{S_x} + \frac{M_{DC2} + M_{DW}}{S_{c3n}} \right) S_{cn}$$

$$M_{AD} = \left(36\text{ ksi} - \frac{304\text{ k}\cdot\text{in.}}{131\text{ in.}^3} + \frac{164.4\text{ k}\cdot\text{in.}}{223\text{ in.}^3} \right) \cdot 247\text{ in.}^3\ (1\text{ ft}/12\text{ in.}) = 678\text{ k}\cdot\text{ft}$$

$$My = M_{DC1} + M_{DC2} + M_{DW} + M_{AD} = 32\text{ k}\cdot\text{ft} + 5.6\text{ k}\cdot\text{ft} + 8.1\text{ k}\cdot\text{ft} + 678\text{ k}\cdot\text{ft} = 724\text{ k}\cdot\text{ft}$$

Determine the composite plastic moment, Mp:

$A_{str}\ F_y$ = 18.2 in.^{2}(36 ksi) = 655.2 kip, tension force

set compression force equal to the tension force and solve for the thickness of the slab in compression, t_{sc}.

$t_{sc}\ b_{eff}\ f'_c$ = 2.4 in. (78 in.)(3.5 ksi) = 655.2 kip compression force

New W24x62 Stringer Composite Positive Flexural Resistance (continued):

$d_s = t_{sc} /2 = 2.4$ in. / 2 = 1.2 in.
$d_{str} = d/2 + t_{hnch} + t_s – t_{sc}$ = 11.87 in. + 1.5 in. + 8 in. – 2.4 in =18.97 in.

$M_p = b_{eff}\, t_{sc}\, f'_c\, d_s + A_{str}\, F_y\, d_{str}$ = 78 in.(2.4 in.)(3.5 ksi)(1.2 in.) +18.2 in.^{2} (36 ksi)(18.97 in.)
M_p = 13,215 k-In. or 1,101 k-ft

Using LRFD 6.10.7.1.2 – Nominal Flexural Resistance (Composite Section in Positive Flexure)
$D_p = t_{sc}$ = 2.4 in
$D_t = d + t_{hnch} + t_s$ = 23.74 in. + 1.5 in. + 8 in. =33.24 in.

$$\text{if } D_p = 2.4\text{ in.} \le 0.1\,D_t = 3.32\text{ in., yes then}$$

$$M_n = M_p = 1{,}101\text{ k}\cdot\text{ft}$$

and

$$M_n \le 1.3\,M_y = 1.3\,(724\text{ k}\cdot\text{ft}) = 941\text{ k}\cdot\text{ft}$$

$$\phi_f M_n = 1.0(941\text{ k}\cdot\text{ft}) = 941\text{ k}\cdot\text{ft}$$

Summarize the Factored Flexural Resistance for the new and existing stringers.

FACTORED FLEXURAL RESISTANCES				
Stringer	Positive Flexure		Negative Flexure	
	$+M_{NC}$ k-ft	$+M_C$ k-ft	$-M_{NC}$ k-ft	$-M_C$ k-ft
Existing W24x76	528	-	436	-
New W24x62	393	941	286	391
Difference (k-ft)	548 k-ft	413 k-ft	105 k-ft	-45 k-ft
Difference (%)	+ 139 %	+ 78 %	37%	-10%

$+M_{NC}$ = Non-composite factored, positive flexural resistance
$+M_C$ = Composite factored, positive flexural resistance (based on Span 3, factored moments at STR-I)
$-M_{NC}$ = Non-composite factored, negative flexural resistance
$-M_C$ = Composite factored, negative flexural resistance

Note: Positive moment is flexure causing tension in the bottom flange.

Determine Flexural Live Load Ratings:

The general load rating equation is as follows (simplified LRFR Eqn. 6-1):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + I)}$$ where:

γ_{DC}, γ_{DW} and γ_{LL} are the LRFD load factors
DC, DW and LL+I are the force effects
C is the member factored capacity

Flexural Ratings for Existing Non-Composite W24x76 Stringer:

recall:

γ_{DC} = 1.25	γ_{Dw} = 1.5	γ_L = 1.75
$+M_{DC1}$ = 59.3 k-ft	$+M_{DW}$ = 0 k-ft	$+M_{LL+I}$ = 171.3 k-ft (HS-20)
$-M_{DC1}$ = -105.5 k-ft	$-M_{DW}$ = 0 k-ft	$-M_{LL+I}$ = -148.0 k-ft (HS-20)
$+\phi M_n$ = 528 k-ft	$-\phi M_n$ = 436 k-ft	

Positive Flexure:

$$RF = \frac{\phi M_n - \gamma_{DC} M_{DC} - \gamma_{DW} M_{DW}}{\gamma_L M_{LL+I}} = \frac{528\,k \cdot ft - (1.25)(59.3\,k \cdot ft) - (1.5)(0\,k \cdot ft)}{(1.75)(171.3\,k \cdot ft)} = 1.51$$

Negative Flexure:

$$RF = \frac{\phi M_n - \gamma_{DC} M_{DC} - \gamma_{DW} M_{DW}}{\gamma_L M_{LL+I}} = \frac{436\,k \cdot ft - (1.25)(105.5\,k \cdot ft) - (1.5)(0\,k \cdot ft)}{(1.75)(148\,k \cdot ft)} = 1.17 \text{ Controls}$$

Check the controlling rating for HS-25.

Since HS-25 live load is simply 1.25 (HS-20 live load), the HS-25 rating factor can be determined by diving the HS-20 rating factor by 1.25

RF = 1.17 / 1.25 = 0.94 < 1.0.

Therefore, the existing stringers are not sufficient for HS-25 live loads.

Flexural Ratings for New Composite W24x62 Stringer at Typical Interior Span:

recall:

γ_{DC} = 1.25	γ_{Dw} = 1.5	γ_L = 1.75	
$+M_{DC1}$ = 25.6 k-ft	$+M_{DC2}$ = 4.5 k-ft	$+M_{DW}$ = 5.4 k-ft	$+M_{LL+I}$ = 128.1 k-ft (HS-25)
$-M_{DC1}$ = -50.7 k-ft	$-M_{DC2}$ = 8.9 k-ft	$-M_{DW}$ = 10.7 k-ft	$-M_{LL+I}$ = -119 k-ft (HS-25)
$+\phi M_n$ =941 k-ft	$-\phi M_n$ = 391 k-ft		

Positive Flexure:

$$RF = \frac{\phi M_n - \gamma_{DC}(M_{DC} + M_{DC2}) - \gamma_{DW} M_{DW}}{\gamma_L M_{LL+I}}$$

$$= \frac{941 k \cdot ft - (1.25)(25.6 k \cdot ft + 4.5 k \cdot ft) - (1.5)(5.4 k \cdot ft)}{(1.75)(128.1 k \cdot ft)} = 3.99$$

Negative Flexure:

$$RF = \frac{\phi M_n - \gamma_{DC}(M_{DC} + M_{DC2}) - \gamma_{DW} M_{DW}}{\gamma_L M_{LL+I}}$$

$$= \frac{391 k \cdot ft - (1.25)(50.7 k \cdot ft + 8.9 k \cdot ft) - (1.5)(10.7 k \cdot ft)}{(1.75)(119 k \cdot ft)} = 1.44$$

Therefore, the new W24x64 composite stringers are sufficient for HS-25 live loading on the typical interior spans.

Summary

Using the methods of making the new stringers composite with the deck and changing the continuity of the new stringers, it was possible to use a lighter stringer sections, even with a 25% increase in the live loading (HS-20 to HS-25).

The following is a brief discussion for each of the methods used:

Non-composite to Composite stringers:

For positive flexure, there is a significant increase in the flexural resistance for a composite section compared with a non-composite section. For this example, there was a 139% increase in the flexural resistance for a composite W24x62 in positive flexure compared to a non-composite W24x62 in positive flexure.

For negative flexure, there is a less significant increase in the flexural resistance for a composite section compared with a non-composite section. For a composite section in negative flexure, the longitudinal deck reinforcing steel within the effective width can be considered in the flexural resistance. For this example, there was a 37% increase in the flexural resistance for a composite W24x62 in negative flexure compared to a non-composite W24x62 in negative flexure.

Change in Stringer Continuity from 2-Span Continuous to Multi-span Continuous:

For this example, there was a significant reduction in both the positive and negative moments in the multi-span arrangement compared to the 2-span arrangement. The moment reductions begin in Span 2 and Support 3 and continue for the typical interior spans. Span 3 and Support 4 has slightly higher moments than the typical interior span, so these were the moments used in the live load ratings. There were only small differences in the Span 1 and Support 2 moments between the 2-span and multi-span arrangements. This resulted in no reduction in the stringer size for the end 2 spans. The positive flexure composite resistance was increased significantly but the negative composite flexural resistance was not increased enough to compensated for the increased HS-25 live loading in the new condition.

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